

## **HORIZONTAL BENDING OF FRP RETROFITTED MASONRY SMALL WALL SPECIMENS**

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### **SUMMARY**

This paper presents the results of a series of four tests on masonry small wall specimens retrofitted with horizontally oriented near surface mounted (NSM) carbon fibre-reinforced polymer (CFRP) strips, subjected to horizontal bending. The specimens were tested in a vertical orientation with variation in superimposed vertical compressive loading. The application of compressive load improved the confinement efficiency, and thus increased the ultimate strength of the specimens.

### **BACKGROUND**

Unreinforced masonry (hereafter termed ‘masonry’) structures are susceptible to failure when subjected to out-of-plane loads induced by wind and earthquakes. As a result, there is an increasing need for retrofit of masonry construction to strengthen and rehabilitate seismically vulnerable structures. The use of FRP materials for strengthening reinforced concrete members is well established (e.g. Teng et al. 2002; Oehlers & Seracino 2004), and recently, its use has been extended for retrofit of masonry (e.g. Kuzik et al. 2003; Galati et al. 2006). The application of vertical FRP strips to strengthen masonry walls has been shown to improve the vertical bending capacity and thus the ultimate wall capacity (e.g. Albert et al. 2001; Willis et al. 2007b).

Pull tests have been conducted (Petersen et al. 2007) with applied compressive stress across the bed joints, simulating the effect of overburden on horizontally oriented NSM FRP strips. Pull test results provide a lower bound estimate of the intermediate crack (IC) debonding (i.e. debonding that emanates from a tension crack in the masonry at some intermediate point along the FRP strip) resistance of a one-way vertically spanning beam due to the effects of curvature and multiple cracks (Willis et al. 2007a). The orthogonal arrangement of the bed and perpend joints influences the bending response about different axes. As a result, the failure mechanism for a horizontally spanning masonry section retrofitted with horizontal

FRP strips will be different to that of a vertically spanning section retrofitted with vertical strips. For the vertical strip case, the major plane of weakness is created by the horizontal bed joints. Typically, for an unretrofitted wall, first cracking occurs in a single bed joint at approximately wall mid-height and thus the contribution of the vertical bending capacity to overall wall capacity is considered to be negligible for design purposes. The use of vertical FRP strips greatly increases the contribution of this component to maximum strength. However, for the horizontal strip case, cracking can occur in flexure through the perpendicular joints or brick units, or in torsion through the bed joints, or a combination of all three mechanisms. The interaction of the FRP retrofit scheme with this more complex masonry failure mechanism is the focus of this research.

The objective of the current study was to investigate the horizontal bending behaviour of masonry sections strengthened with horizontal FRP strips. To achieve this objective, an experimental study including a series of four tests on small wall specimens strengthened with NSM CFRP strips was conducted. Test variables included the level of applied vertical compressive stress and the cross-sectional dimensions of the FRP strip. A fifth specimen with no FRP retrofit and no compressive stress was also tested as a control. Increasing the compressive stress was expected to improve the confinement efficiency of the NSM strip and thus increase the maximum moment capacity of the section.

## EXPERIMENTAL STUDY

### Material Properties

The brick units used in the experimental study were typical 3-core clay brick units, with nominal dimensions of  $230 \times 110 \times 76$  mm. The mortar consisted of Portland cement, hydrated lime and sand, combined in a cement:lime:sand ratio of 1:1:6. A 10 mm thick mortar joint was used for all joints. Tests were conducted to determine the material properties of the masonry. The bond wrench and compressive strength tests were conducted in accordance with AS 3700-2001 (Standards Australia 2001), while the lateral modulus of rupture of the brick unit test was conducted in accordance with AS/NZS 4456.15:2003 (Standards Australia 2003). The mean values of the flexural tensile strength of the masonry,  $f_{mt}$ , compressive strength of the masonry,  $f_m$ , modulus of elasticity of the masonry,  $E_m$ , and lateral modulus of rupture of the brick unit,  $f_{ut}$ , were 0.19 MPa, 15.0 MPa, 8993 MPa and 3.57 MPa with coefficients of variation (COV) of 0.57, 0.11, 0.14, and 0.07, respectively. Full details of the masonry material tests are presented elsewhere (Callaghan et al. 2006).

The mean value of  $f_{mt}$  was relatively low as typical mean and COV values obtained from laboratory prepared specimens are expected to be in the order of 0.60 MPa and 0.30 (Willis 2004). This discrepancy may be attributed to workmanship and temperature effects. Typically, for masonry retrofitted with vertical FRP strips, this would have negligible effect on experimental behaviour, because the mortar joints do not have a significant impact on the bond behaviour of FRP retrofitted masonry as crack propagation occurs primarily through the brick units (Yang et al. 2006; Willis et al. 2007b). However, for horizontal FRP strips, the failure mode will differ, hence the masonry behaviour is likely to be of greater importance.

Pultruded carbon FRP (CFRP) strips were used to retrofit all test specimens. The key material properties of the FRP provided by the manufacturer were the modulus of elasticity,  $E = 165$  GPa, and the rupture strain,  $\epsilon_{rup} = 14000 \mu\epsilon$ .

## Horizontal Bending Test Set-up

The horizontal bending test design developed in related research (Willis 2004; Willis et al. 2004) for unretrofitted masonry specimens was used for this study. Each specimen was 3.5 units long and six courses high, with overall dimensions of 830 mm × 520 mm. The even number of courses ensures that the specimen is symmetric in its load resistance (Lawrence 1983). The tension face of each specimen was painted white for crack observation. Full details of the test set-up are presented elsewhere (Willis 2004; Willis et al. 2004).

The FRP retrofitting scheme used a single NSM (i.e. the FRP strip is oriented within a cut in the brickwork so that its longer cross-sectional dimension is parallel to the bed joints) CFRP strip located at the tension side of each specimen that extended past the supports. The FRP strip was embedded within the brick units as pull tests (Yang et al. 2006) have indicated that strip embedment through mortar reduces bond strength. For a vertical FRP strip passing through perpend joints, the confinement area consists of mortar for approximately half the strip length, this corresponded to a reduction in strength of the order of 10% (Yang et al. 2006). Clearly, embedding a horizontal FRP strip in a bed joint would further decrease the strength. Also, preparation of the NSM joint by saw cutting along the length of a bed joint could potentially compromise the strength of the specimen prior to retrofitting. Due to the even number of courses, the FRP strip was embedded in the third course from the base of the specimen. This non-symmetric strip placement is not deemed to be significant in terms of load resistance and it was considered more important to keep the specimen geometry consistent with previous research (Willis 2004; Willis et al. 2004).

The experimental set-up is shown in Figure 1a. A four-point loading arrangement was used, creating a region of constant maximum bending moment and zero shear. Static loading was applied under displacement control at a slow and repeatable rate using a simple hydraulic jack to obtain post-ultimate data, with readings taken at 0.5 kN intervals. Out-of-plane displacements were measured using Linear Variable Differential Transformer (LVDT) displacement transducers located at the mid-span and each support of the specimen (Figure 1a) to determine the displacement relative to any support movement. Strain gauges were located along the FRP strip. A total of five strain gauges were used for Test 1 however, strain gauge 1 was broken prior to testing. All other specimens had only four strain gauges, with no strain gauge at the mid-span of the FRP strip (Figure 1a).

For the level of compressive stress, previous related research on the horizontal bending response of unretrofitted masonry sections (Willis 2004; Willis et al. 2004) adopted a range between 0 and 0.25 MPa. Work being conducted in parallel with the current study (Petersen et al. 2007) has investigated the effect of applied stress on pull tests. These tests used a maximum value of 1 MPa, which resulted in an increase in the debonding resistance of NSM specimens by approximately 50%. For the current experimental study, a maximum value of 0.75 MPa was chosen due to the limitation of the springs available. Three levels of compressive stress (i.e.  $\sigma_v = 0, 0.375$  and 0.75 MPa) were tested to investigate the trend in behaviour. Compressive stress was exerted by four springs located above the solid concrete blocks at the top of each specimen, where applicable (Figure 1b). The frictional resistance at the horizontal edges of the specimen was minimised using grease and Teflon. This was measured and subtracted from the applied load.

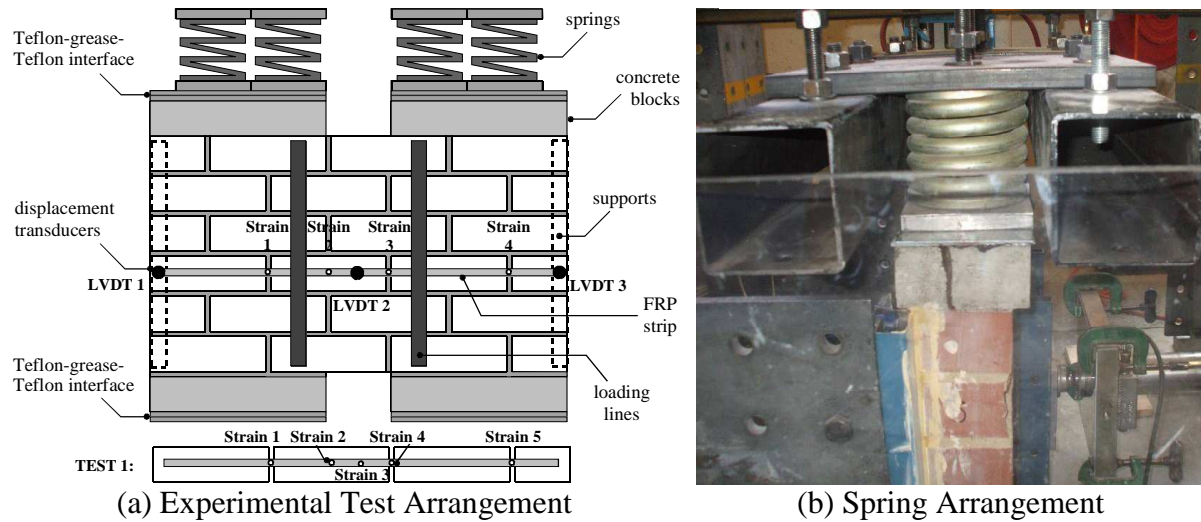


Figure 1. Horizontal Bending Test Set-up

## EXPERIMENTAL RESULTS

Four tests were conducted with the level of compressive stress and FRP strip dimensions varied. A fifth specimen with no FRP retrofit and no compressive stress was also tested as a control. The test schedule and experimental results are summarised in Table 1. Figure 2 shows the load-displacement behaviour, and the experimental response for each test is described in detail in this section.

Table 1. Summary of Horizontal Bending Test Results

Test	Strip thickness, $t_p$ (mm)	Strip width, $b_p$ (mm)	Compressive stress, $\sigma_v$ (MPa)	Failure mode	Maximum load, $P_{max}$ (kN)	Displacement at $P_{max}$ , $\Delta_{max}$ (mm)	Maximum strain, $\epsilon_{max}$ ( $\mu\epsilon$ )
1	2.4	20	0	premature masonry failure	14.8	0.63	2420
2	1.2	15	0	DI debonding	12.8	3.75	4704
3	1.2	15	0.75	FRP buckling	18.4	4.28	7008
4	1.2	15	0.375	FRP buckling	20.4	8.08	7707
5	-	-	0	masonry failure	9.4	0.23	-

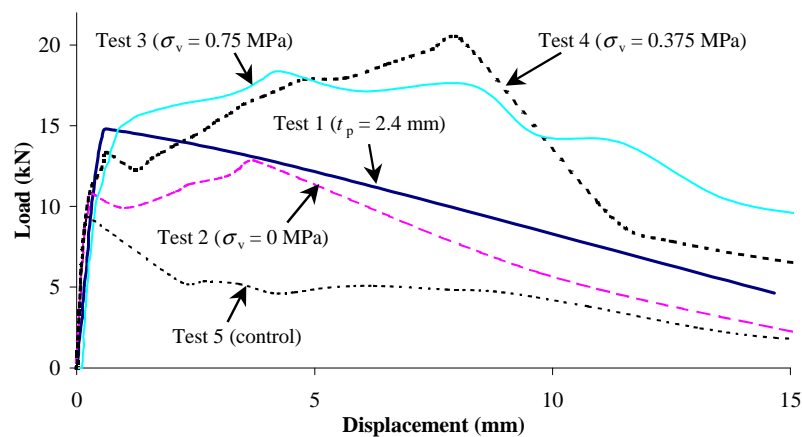


Figure 2. Load-displacement Behaviour

### Test 5 (control test with no FRP retrofit and no compressive stress)

A control test was conducted to determine the strength of an unretrofitted masonry specimen. First cracking initiated at the top of the specimen as this edge was unrestrained. The specimen failed in stepped cracking with five bed joint cracks and six perpend joint cracks in the failure mechanism (Figure 3). The maximum load capacity was 9.4 kN with a corresponding displacement of 0.23 mm (Figure 2). The typical load-displacement behaviour reported previously (Willis 2004; Willis et al. 2004) was observed. This consisted of linear response up to perpend joint cracking, then softening as bed joint cracking occurred until the full crack pattern formed at maximum load.



Figure 3. Final Crack Pattern for Test 5

### Test 1 (2.4 mm × 20 mm NSM CFRP strip with no compressive stress)

First visible cracking initiated at the top mid-span of the specimen as this edge was free from restraint. Line cracking propagated downwards and stepped cracking propagated towards the vertical edge of the specimen. Additional line cracking occurred at the bottom of the specimen (Figure 4a). Crack propagation was interrupted at the FRP retrofitted course of brickwork due to its increased strength and stiffness relative to the masonry. Hence, the crack propagated along the bed joint above the FRP strip towards the vertical edge of the specimen. The masonry failed before the FRP strip could significantly contribute to strength. For this test, the ball-and-socket joint between the load cell and the loading lines was located above the FRP strip. As a result, there was a non-symmetric application of the load as the loading line pivoted about the ball-and-socket joint toward the top half of the specimen. This caused the cracking pattern to follow the path of least resistance through the unretrofitted masonry section at the top of the specimen (Figure 4a). This premature failure mode was further encouraged by the partial restraint at the base of the specimen due to frictional resistance. The load-displacement behaviour (Figure 2) indicates approximately linear response up to the maximum load of 14.8 kN with a corresponding displacement of 0.63 mm.

Figure 4b shows the load-strain behaviour of the FRP strip. The FRP strip initially had very small contribution to specimen strength as the FRP strain was small. At maximum load, the maximum FRP strain was only  $434 \mu\epsilon$  at Strain 4, which is approximately 2.5% of the IC debonding strain, confirming that premature masonry failure occurred due to the loading arrangement, resulting in inefficient use of the FRP material. After maximum load was reached, the FRP strain began to increase significantly as the loading was only resisted by the FRP retrofitted course at this stage. The maximum FRP strain was approximately  $2400 \mu\epsilon$ .

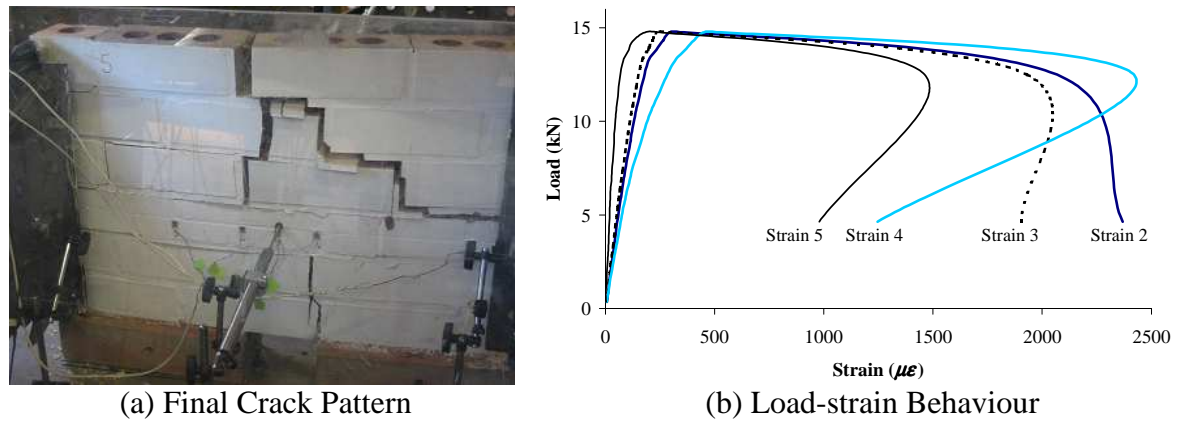


Figure 4. Experimental Response for Test 1

#### Test 2 (1.2 mm × 15 mm NSM CFRP strip with no compressive stress)

Observations from Test 1 resulted in modifications being made to the test arrangement for Test 2. The point of application of load was centralised with respect to the location of the FRP strip, and the cross-sectional dimensions of the FRP strip were reduced to decrease the rigidity of the specimen. First visible cracking initiated through perpend joints at the top mid-span of the specimen as it was free from restraint (Figure 5a). The cracking propagated downwards until it reached the FRP retrofitted course of brickwork, at an applied load of 10.9 kN with a corresponding displacement of 0.31 mm. The FRP strip then provided a greater contribution to specimen strength until it buckled at an applied load of 12.8 kN with a corresponding displacement of 3.75 mm. The brick near the left loading line was pushed off, causing detachment of the brick where the FRP strip was buckled. Failure was by cleavage (or displacement induced (DI)) debonding. This debonding mechanism has been identified in the literature (e.g. Dai et al. 2007; Willis et al. 2007b) but is yet to be quantified, hence it was unable to be designed against. The load-displacement behaviour (Figure 2) shows that the maximum load and corresponding displacement were approximately 37% and 1630% greater than the control test (Test 5) respectively. This indicates that FRP retrofit can increase the strength and ductility of masonry sections subjected to horizontal bending. However, the increase in load was not significant due to the DI form of premature failure.

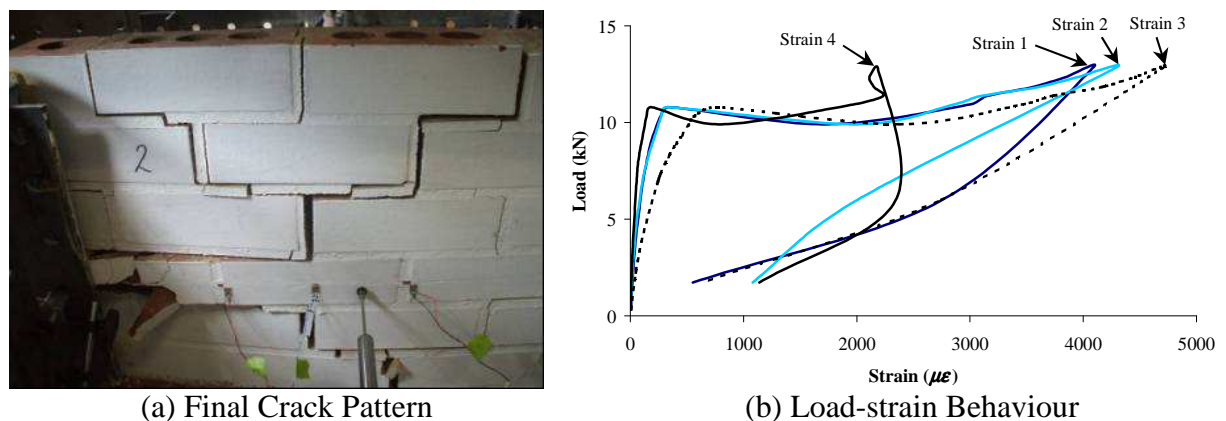


Figure 5. Experimental Response for Test 2



The load-strain behaviour is shown in Figure 5b. The maximum FRP strain was  $4700 \mu\epsilon$  at Strain 3. The level of strain in the FRP strip was low, indicating premature failure, i.e. the FRP strip did not significantly contribute to strength. Note that the pull tests (Petersen et al. 2007) exert a state of axial tension, whereas the small wall specimen tests are predominantly in flexure, hence bending is likely to enhance the tendency for DI debonding.

#### Test 3 (1.2 mm $\times$ 15 mm NSM CFRP strip with $\sigma_v = 0.75$ MPa)

First visible cracking occurred at mid-span at an applied load of 9.4 kN. Unlike Test 2, cracks occurred at the second course of brickwork because the top of the specimen was partially restrained due to the application of compressive stress. Predominantly line failure occurred as the increase in bed joint torsional resistance due to the large compressive stress prevented stepped failure. The maximum load was 18.4 kN due to buckling of the FRP strip (Figure 6a). The FRP strip was able to buckle about its weak axis with a relatively large radius of curvature without rupture. The load-displacement behaviour (Figure 2) shows that the load capacity of Test 3 was 43% higher than Test 2 ( $\sigma_v = 0$  MPa) as the increase in compressive stress improved confinement efficiency and increased the resistance of the mortar joints, resulting in the prevention of premature DI debonding and change of failure mode. In addition, ductility increased with larger residual displacement.

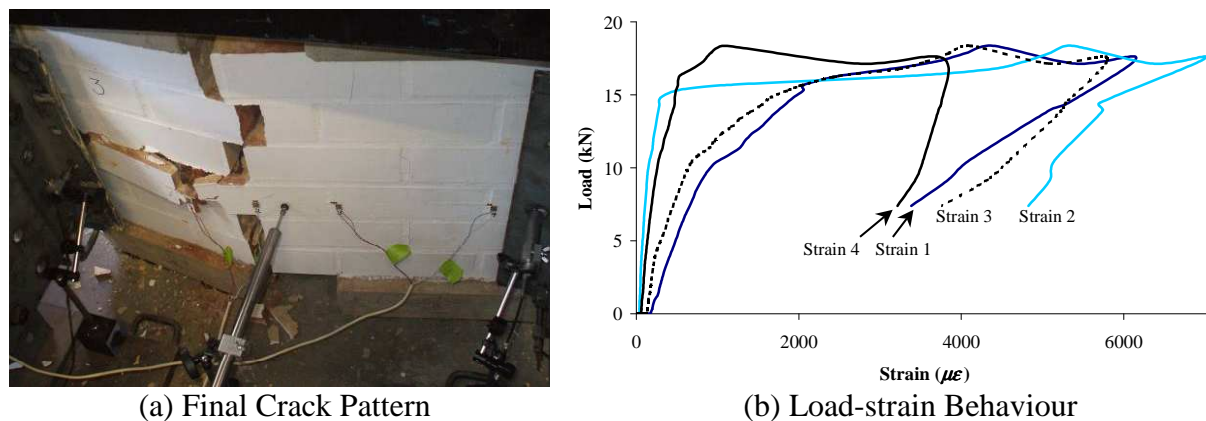


Figure 6. Experimental Response for Test 3

Failure of the FRP strip occurred as it buckled at the location of the perpend joint near Strain 1. Analysis of the load-strain behaviour indicated that all of the strain gauges were well below the FRP rupture strain. Figure 6b shows that the increase in FRP strain in the elastic region was very small hence little contribution was made to specimen strength. Once masonry cracking occurred, the FRP strip started to have significant increase of strain as the majority of the load was sustained by the FRP strip. Figure 6b shows that with constant load, the FRP strain continued to increase until major cracks formed and failure occurred. Although Figure 6b suggests that failure could have occurred near Strain 2 as it has the highest strain, the FRP strip actually buckled in the perpend joint at Strain 1. Comparison of the strain profiles for Tests 2 and 3 shows that there are two zones for the FRP strip, i.e. the elastic and plateau zones. For the elastic zone, the contribution of strength by the FRP strip is very low and the load resistance is mainly by the masonry, whereas for the plateau zone, the contribution to strength by the FRP strip is increased significantly due to crack development in the masonry. The cracking strains in the masonry are so small that it is reasonable to assume that the FRP strip does not carry significant load until cracking occurs.

#### Test 4 (1.2 mm × 15 mm NSM CFRP strip with $\sigma_v = 0.375$ MPa)

First visible cracking occurred in a perpend joint at mid-span at a load of approximately 16 kN. As the load exceeded 17.5 kN, line cracking continued to propagate downwards but was interrupted at the FRP retrofitted course of brickwork. No cracking was visible at the bed joint at specimen mid-height until the sudden slip of mortar above the FRP retrofitted course occurred (Figure 7a). The mortar failed in shear, resulting in a large displacement, and the FRP strip buckled (Figure 7a), in a similar manner to Test 3. The load-displacement behaviour (Figure 2) indicates that masonry failure occurred at 13.3 kN, following by softening, with the load resisted by the FRP strip. The load capacity was 20.4 kN at a displacement of 8.1 mm (Figure 2). The load-strain behaviour (Figure 7b) indicates that the maximum strain was approximately 7700  $\mu\epsilon$  at Strain 2, which is of similar magnitude to the maximum strain in Test 3 (7000  $\mu\epsilon$ ) due to the similar failure mode.

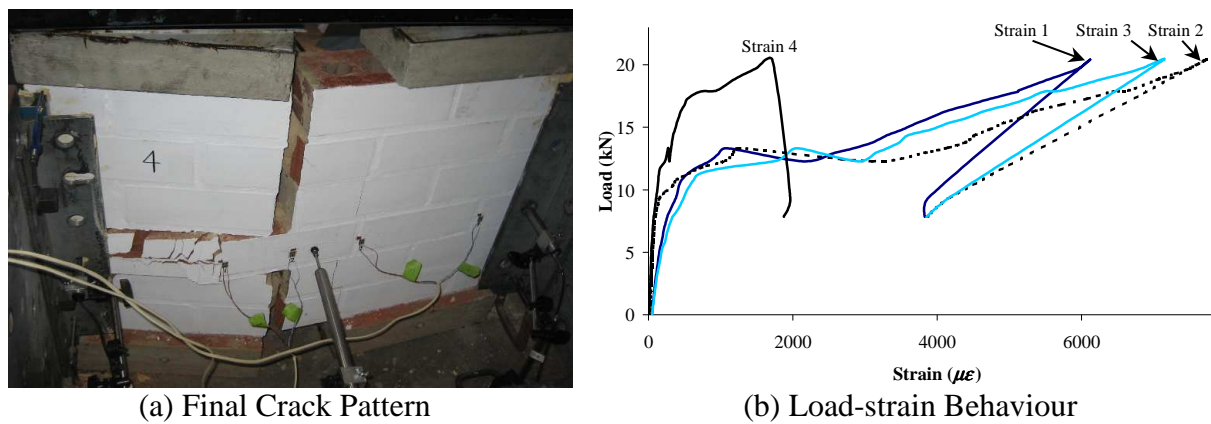


Figure 7. Experimental Response for Test 4

#### Comparison of Results

The test variables included the level of compressive stress and FRP strip dimensions. No comparison can be made between tests with variation in FRP strip dimensions as Test 1 failed prematurely by masonry failure due to the test arrangement. For the tests with variation in compressive stress, Tests 2, 4 and 3 were compared with  $\sigma_v = 0, 0.375$  and  $0.75$  MPa, respectively. Test 4 ( $\sigma_v = 0.375$  MPa) showed an increase in horizontal bending capacity of 44% compared to Test 2 ( $\sigma_v = 0$  MPa). The relationship between compressive stress and horizontal bending capacity is not linear due to a change in response from DI debonding to FRP buckling. An increase in compressive stress improves the confinement efficiency of the NSM strip, causing the failure mode to change and become more desirable. The maximum capacities for Tests 3 and 4 were approximately equal due to the similar failure mode. Also, the displacement at maximum load increased with compressive stress (Table 1).

#### CONCLUSIONS AND RECOMMENDATIONS

This paper presents a series of pilot tests investigating the horizontal bending capacity of masonry sections retrofitted with horizontal NSM CFRP strips subjected to variation in compressive stress. The following conclusions and recommendations are made:



(1) The application of compressive stress incurs the following: (1) improves confinement efficiency of the FRP strip; (2) delays the onset and propagation of masonry cracking; (3) increases bed joint torsional resistance; (4) increases friction along the NSM joint; (5) prevents premature failure mode, and, (6) for IC debonding to occur, must overcome FRP-to-masonry interface bond, which is enhanced by compressive stress.

(2) The current loading arrangement applies a large change in curvature to the FRP strip which contributed to the DI debonding failure observed in Test 2. For future research, it is recommended that the length of the specimens and the loading arrangement be increased to further investigate the potential for DI debonding to occur. Also, although NSM strips are more efficient than externally bonded strips, they are more susceptible to DI debonding due to their orientation.

(3) The FRP strip greatly increased the strength and stiffness of a single course of brickwork. This is important for design to determine the strip spacing to prevent local masonry failure between strips. Also, if the FRP strip causes a course of brickwork to be too stiff, then shear failure may occur along a bed joint resulting in inefficient use of the FRP strip.

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## REFERENCES

Albert, M.L., Elwi, A.E., Cheng, J.J.R., "Strengthening of Unreinforced Masonry Walls using FRPs", *Journal of Composites for Construction*, Vol. 5, No. 2, 2001, pp. 76-84.

Callaghan, P., Hydbom, N., McKay, D., "FRP Retrofit of Unreinforced Brick Masonry", Honours Research Project Report, School of Civil and Environmental Engineering, The University of Adelaide, Australia, 2006.

Dai, J., Ueda, T., Sato, Y., "Bonding Characteristics of Fiber Reinforced Polymer Sheet-Concrete Interfaces under Dowel Load", *Journal of Composites for Construction*, Vol. 11, No. 2, 2007, pp. 138-148.

Galati, N., Tumialan, G., Nanni, A., "Strengthening with FRP Bars of URM Walls Subject to Out-of-plane Loads", *Construction and Building Materials*, Vol. 20, No. 1-2, 2006, pp. 101-110.

Kuzik, M.D., Elwi, A.E., Cheng, J.J.R., "Cyclic Flexure Tests of Masonry Walls Reinforced with Glass Fiber Reinforced Polymer Sheets", *Journal of Composites for Construction*, Vol. 7, No. 1, 2003, pp. 20-30.

Lawrence, S.J., “Behaviour of Brick Masonry Walls under Lateral Loading”, Ph.D. Thesis, The University of New South Wales, Australia, 1983.

Oehlers, D.J., Seracino, R., “Design of FRP and Steel Plated RC Structures: Retrofitting of Beams and Slabs for Strength, Stiffness and Ductility”, Elsevier, Kidlington, Oxford, UK, 2004.

Petersen, R.B., Masia, M.J., Seracino, R., “Influence of Plate Orientation and Amount of Precompression on the Bond Strength Between NSM CFRP Strips and Masonry”, *Proceedings of the 10<sup>th</sup> North American Masonry Conference*, St. Louis, USA, 2007.

Standards Australia, “AS 3700-2001: Masonry Structures”, Standards Australia, Sydney, 2001.

Standards Australia, “AS/NZS 4456.15:2003: Masonry Units and Segmental Pavers - Methods of Test; Method 15: Determining Lateral Modulus of Rupture”, Standards Australia, Sydney, 2003.

Teng, J.G., Chen, J.F., Smith, S.T., Lam, L., “*FRP Strengthened RC Structures*”, Wiley, London, England, 2002.

Willis, C.R., “Design of Unreinforced Masonry Walls for Out-of-plane Loading”, Ph.D. Thesis, School of Civil and Environmental Engineering, The University of Adelaide, Australia, 2004.

Willis, C.R., Griffith, M.C., Lawrence, S.J., “Horizontal Bending of Unreinforced Clay Brick Masonry”, *Masonry International*, Vol. 17, No. 3, 2004, pp. 109-121.

Willis, C.R., Wu, C., Seracino, R., Griffith, M.C., “Displacement Induced and Intermediate Crack Debonding in FRP Strengthened One-way Masonry Beams”, *Proceedings of the First Asia-Pacific Conference on FRP in Structures, APFIS 2007*, Hong Kong, submitted for review, 2007a.

Willis, C.R., Yang, Q., Seracino, R., Griffith, M.C., “Damaged Masonry Walls in Two-way Bending Retrofitted with Vertical FRP Strips”, *Special Issue on FRP Composites for Construction and Building Materials*, submitted for review, 2007b.

Yang, Q., Willis, C.R., Seracino, R., Xia, S.H., Griffith, M.C., “Bond Tests on FRP Retrofitted URM Prisms”, *Proceedings of the 19<sup>th</sup> Australasian Conference on the Mechanics of Structures and Materials, ACMSM 19*, Christchurch, New Zealand, 2006, pp. 133-139.