CYCLIC IN-PLANE SHEAR BEHAVIOUR OF NSM FRP STRENGTHENED URM WALL PANELS

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ABSTRACT

An experimental study was conducted to assess the effect on strength, displacement capacity and ductility of strengthening unreinforced masonry (URM) shear panels with near surface mounted (NSM) fibre reinforced polymer (FRP) strips. A total of twenty three (23) wall panels (5 URM and 18 reinforced) were subjected to vertical pre-compression combined with increasing reversing cycles of in-plane lateral displacement under fixed-fixed boundary conditions. Two wall aspect ratios were tested: aspect ratio 1.0 (1200mm high x 1200mm long x 110mm thick) and aspect ratio 0.5 (1040mm high x 1910mm long x 110mm thick). For aspect ratio 1.0, eight panels were tested (2 URM and 2 specimens for each of three different NSM FRP reinforcement schemes). For aspect ratio 0.5, fifteen panels were tested (3 URM and 2 specimens for each of six different NSM FRP reinforcement schemes). The experimental program was designed to produce diagonal cracking in the URM specimens and hence investigate the effectiveness of the various reinforcement schemes in controlling this failure mode. This was achieved for the aspect ratio 1.0 panels for which the study revealed that the FRP strengthening was effective in improving the ultimate load resisted by the wall panels (increases of up to 9%), the displacement capacity (133%) and ductility (108%) compared to the URM response. For the aspect ratio 0.5 panels, base sliding failures dominated the experimental program, making it difficult to fully assess the effectiveness of the various reinforcing schemes.

KEYWORDS: masonry, strengthen/retrofit, cyclic, shear, nsm, frp

INTRODUCTION

Past seismic events have highlighted the vulnerability of unreinforced masonry (URM) construction to damage under earthquake induced loading [1]. This vulnerability arises from the high mass, low tensile strength and limited ductility of URM. During seismic events URM walls subjected to in-plane shear loading, with or without vertical pre-compression, may fail by sliding along a mortar bed joint, diagonal cracking through the wall, by rocking, or by a combination of modes [2]. The failure mode(s) observed depend on the wall aspect (height/length) ratio, the level of vertical pre-compression, the boundary conditions imposed on the wall and the masonry material properties. Sliding and rocking failure modes may allow considerable displacement capacity and energy dissipation during earthquake shaking and provided they do not result in collapse of the wall, they may be tolerable to some extent. Diagonal cracking on the other hand usually is associated with very limited displacement capacity and post peak strength and therefore is undesirable. To address this vulnerability, existing URM shear walls may be retrofitted / strengthened in order to improve their strength and/or ductility during seismic loading. In particular, such strengthening usually aims to restrain the development of failure modes which involve diagonal cracking and instead induce sliding and/or rocking type failure modes.

Petersen [3] investigated the use of fibre reinforced polymer (FRP) strips imbedded into thin slots cut into the face of URM (a technique referred to as near surface mounting or NSM). Using a series of diagonal tension (in-plane shear) tests conducted in accordance with [4] combined with finite element modelling, [3] found that the technique was effective in improving the strength and ductility of URM shear panels which failed by bed joint sliding or diagonal cracking. Petersen [3] investigated various NSM FRP reinforcement schemes and found that vertical reinforcement was more effective than horizontal reinforcement in restraining bed joint sliding and diagonal cracking. Konthesingha et al. [5] extended the work of [3] by studying the performance of damaged URM panels which were retrofitted using NSM FRP reinforcement and then tested under vertical pre-compression combined with increasing reversing cycles of in-plane lateral displacement. The wall panels were supported under cantilever style boundary conditions (free to rotate (in-plane) at the top) and prevented from sliding at the base. The study showed that the FRP retrofitting was effective in restoring the load resisting capacities and improving the displacement capacities and energy dissipation of the pre-damaged walls compared with the URM responses. The retrofitting scheme which used a combination of horizontal and vertical reinforcement showed better performance than schemes which used only horizontal reinforcement in terms of ultimate load capacity, displacement capacity and energy dissipation.

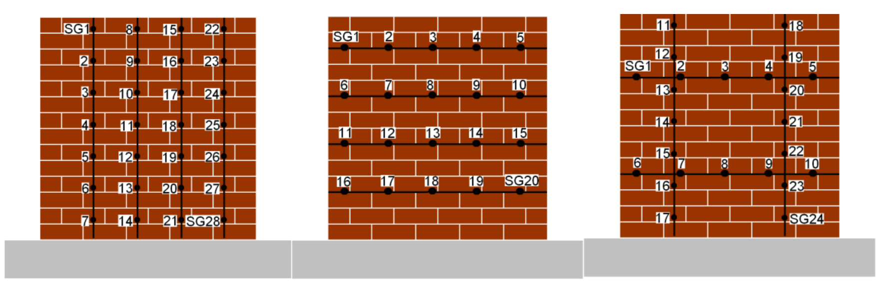
The aim of the current study was to build on the results of [5] by conducting a similar experimental study but rather than using cantilever style support conditions the wall panels were supported in a purpose designed test frame which imposes fixed-fixed boundary conditions during cyclic shearing. The paper reports the results of tests on twenty three (23) clay brick URM shear panels strengthened using a range of NSM FRP reinforcement schemes. The paper reports the failure modes and load versus displacement responses for the panels and discusses the effectiveness of the strengthening schemes in terms of strength, displacement capacity and ductility. Further details can be found in [6].

EXPERIMENTAL PROGRAM

Twenty three (23) unreinforced masonry wall panels were constructed by the same mason using the same brick type and the same mortar mix proportions. Two wall aspect ratios (height/length) were considered: 8 panels had aspect ratio 1.0 (AR1) (1200mm high x 1200mm long x 110mm thick) and 15 panels had aspect ratio 0.5 (AR05) (1040mm high x 1910mm long x 110mm thick). The wall panels were constructed from a single leaf of solid (no holes or indentations) extruded clay bricks (230mm long x 110mm wide x 76mm high) and were founded on reinforced concrete footing beams (200mm x 200mm cross section x 1600mm or 2400mm long for AR1 and AR05 respectively). The wall panels were built in running bond with both the bed and head mortar joints 10mm thick and fully filled. The average flexural tensile strength of the brick units was 2.28MPa, with a coefficient of variation (COV) of 27% (determined in accordance with [7]).

The mortar mix proportions were 1:1:6 (cement : lime : sand by volume) plus an air entraining admixture to the manufacturer's recommended dosage. The latter was used to improve the workability of the mortar. The wall panels were constructed using 11 separate mortar batches, with each panel typically containing mortar from 3 to 4 different batches (exact details can be found in [6]). For each batch of mortar, two x 6 unit high masonry prisms (10 joints) were constructed and tested by bond wrench [8] to determine the flexural tensile bond strength for the masonry. This test was chosen as a measure of control between batches because it is simple to perform and can be (loosely) correlated with shear bond strength, which was expected to control the performance of the specimens in the current study. All but one of the 23 wall panel specimens and sets of bond wrench prisms were air cured in the laboratory for between 21 and 47 weeks prior to testing. The mean flexural tensile bond strengths determined for the masonry used in each of the 22 wall specimens ranged from 0.94MPa to 1.36MPa, resulting in a mean masonry flexural tensile bond strength for all 22 walls of 1.13MPa [6]. The exception was Specimen UAR05-3 (AR05) which was constructed from a single mortar batch at a later date specifically to assess the behaviour of a specimen with lower brick to mortar bond strength. For this specimen the wall panel and bond wrench specimens were tested at an age of four weeks and the mean flexural tensile bond strength was 0.39MPa. The masonry compressive strength as determined in accordance with [8] was 21.26MPa (COV of 8%).

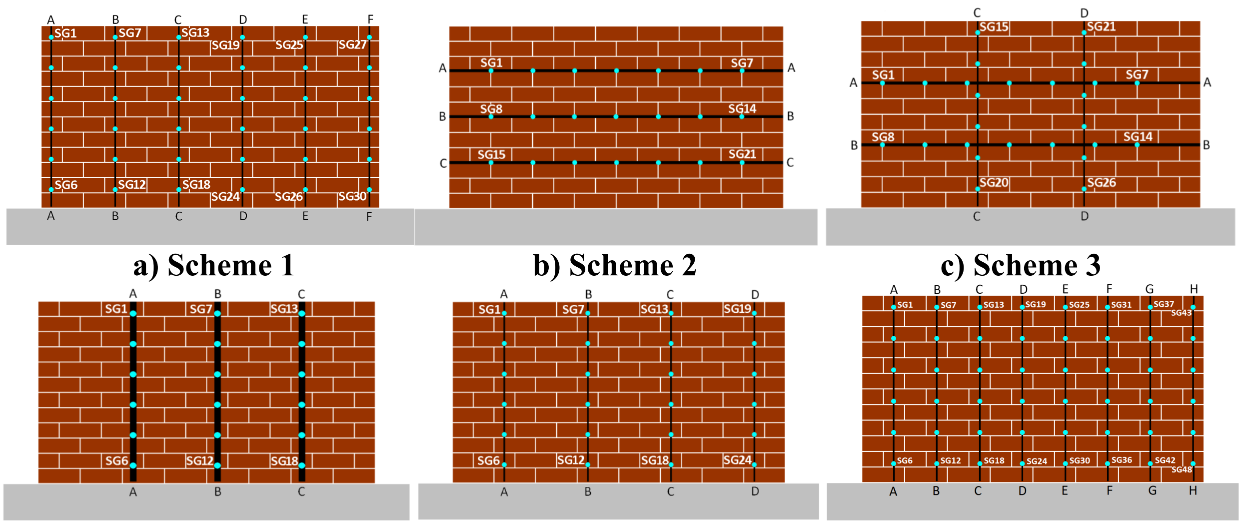
Two of the AR1 and three of the AR05 wall panels remained unreinforced as control specimens. The remaining six AR1 walls were strengthened using the three NSM FRP reinforcing schemes shown in Figure 1 (two repeats per scheme). The remaining twelve AR05 walls were strengthened using the six reinforcing schemes shown in Figure 2 (two repeats per scheme). For AR1, each scheme used the same quantity of FRP material. For AR05, Schemes 1 to 4 used the same quantity of FRP, with Schemes 5 and 6 using lower and higher reinforcement ratios respectively, than the other schemes. In all schemes the reinforcement was applied only on one side of the wall because in practice it is usually not possible to access both sides of an existing wall. The reinforcement schemes were designed to resist diagonal cracking and bed joint sliding failures within the masonry and as such the vertically aligned strips were not anchored into the reinforced concrete footing beams.



a) Scheme 1 **b) Scheme 2** **c) Scheme 3**

Figure 1: Reinforcement Schemes for AR1 Showing Strain Gauge (SG) Locations

In Figures 1 and 2 (except Scheme 4 for AR05), each black line represents a single NSM unidirectional pultruded carbon FRP (CFRP) strip. Each strip was 15mm wide x 1.4mm thick. For Scheme 4, each black line represents two such strips glued back to back (15mm x 2.8mm) to investigate the effect of increased reinforcement spacing compared to Scheme 1. To install the reinforcement, slots were saw cut into the wall surface and the CFRP strips were glued into the slots using a two-part epoxy adhesive (15mm side of the strip was orientated normal to the face of the wall). For schemes with only vertical reinforcement, the slots were cut 15mm deep into the wall x 8mm wide, located in the brick units, midway between mortar head joints. The horizontal slots were 15mm deep x 10mm wide (bed joint thickness) located in the mortar bed joints. For the grid arrangements in Scheme 3, 30mm deep x 8mm vertical slots and 15mm deep x 10mm horizontal slots were cut into the same side of the walls. The vertical strips were installed first followed by the horizontal strips. The elastic modulus and the rupture strain of the CFRP were 207050MPa [6] and 12000 με (manufacturer's data) respectively. The flexural strength of the epoxy adhesive is greater than 30MPa according to the manufacturer's data. For 11 of the wall panels (shown with an asterisk in Tables 1 and 2) the FRP strips were fitted with strain gauges to record the axial strain distributions along the lengths of the strips (Figures 1 and 2).



a) Scheme 1 **b) Scheme 2** **c) Scheme 3**

d) Scheme 4 **e) Scheme 5** **f) Scheme 6**

Figure 2: Reinforcement Schemes for AR05 Showing Strain Gauge (SG) Locations

Scheme 2 in each aspect ratio was selected for investigation because FRP strips inserted into the mortar bed joints can be completely concealed which is advantageous from an aesthetic viewpoint. Scheme 1 was selected because [3] found that vertical reinforcement provided greater increases in strength and ductility than horizontal reinforcement for walls subjected to in-plane shear loading. Variations of Scheme 1 were selected to assess the influence of changes in reinforcement spacing (Scheme 4) and reinforcement ratio (Schemes 5 and 6). Scheme 3 was selected for investigation because [5] showed that a combination of vertical and horizontal strips performed better than alternative schemes for retrofitting damaged walls under cyclic in-plane shear loading.

The test setup and instrumentation (displacement potentiometers – POTS) are shown in Figure 3. The apparatus was used to impose zero in-plane rotation (fixed-fixed) boundary conditions to the upper and lower edges of the specimens while they were subjected to vertical pre-compression combined with in-plane lateral displacement. The concrete footing beam (8) was prevented from rotating and sliding. The spreader beam (4) was glued (7) to the wall (6) using two part epoxy adhesive in order to transfer the shear load to the wall and prevent the spreader beam from sliding on the wall. The vertical pre-compression load was applied to the wall panel using the hydraulic jack (2) and was kept constant during the test. The wall specimens were then subjected to monotonic or cyclic (refer Tables 1 and 2) in-plane shear displacements (displacement controlled) using the hydraulic actuator (5). The pin jointed links (3) between the reaction frame (1) and spreader beam ensured that the beam was free to move horizontally and vertically during testing but was unable to rotate [6].

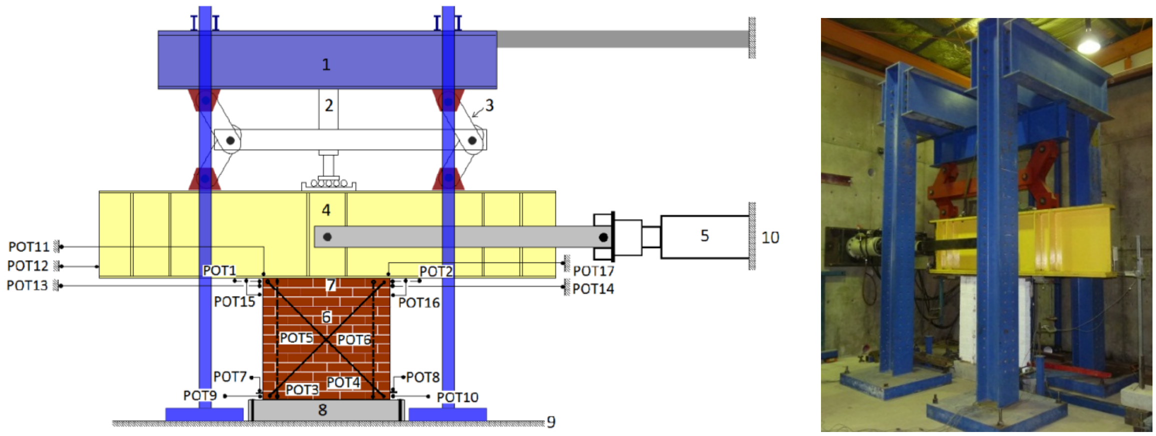


Figure 3: Test setup and instrumentation

For AR1, all walls were tested under 2.6MPa vertical pre-compression stress. For AR05, eight walls were tested under 1.8MPa and seven under 2.1MPa. Pre-compression levels were selected using finite element (FE) modelling in an attempt to induce diagonal cracking through the walls for the URM specimens (rather than base sliding or rocking). This failure mode was targeted so that the effectiveness of subsequent FRP reinforcement could be meaningfully assessed. Upon constructing the specimens, it was discovered that the masonry bond strength achieved was significantly higher than assumed in the experimental design phase. This increased the in-plane shear strengths of the walls so that for a given level of vertical pre-compression the walls were more likely to fail by rocking or base sliding than by diagonal cracking. To alleviate this problem the pre-compression levels determined from the FE simulations were increased to the above levels (2.6, 1.8 and 2.1 MPa respectively). As discussed in the next section, this adjustment was successful for the AR1 walls, but only partially successful for the AR05 walls due to limitations associated with the available testing equipment.

For each aspect ratio, one URM specimen and one strengthened specimen were tested under monotonic in-plane lateral displacement while the remaining specimens were subjected to increasing reversing cycles of in-plane lateral displacement. The monotonic displacement was applied at a rate of 1mm/minute (measured at the actuator) until failure of the specimen occurred. In this context failure was defined by a post peak reduction of the in-plane shear load of more than 20% of the maximum load resisted. The cyclic displacement history was developed with reference to [9] as shown in Figure 4a. This required estimates for the actuator displacement when the wall first cracks (d_{cr}) and the maximum actuator displacement (d_{max}) (for a post peak load drop of 20%), hence the need to conduct monotonic tests prior to the cyclic tests. In the current study, the URM monotonic tests were used to determine values of d_{cr} and d_{max} for walls of each aspect ratio. Further details can be found in [6]. The monotonic tests of the strengthened specimens were conducted to investigate the effectiveness of the strengthening under monotonic loading for comparison with the URM results.

The cyclic shear displacement was applied in reversing cycles of increasing amplitude at a constant frequency of 0.004Hz as recommended by [9]. Each complete cycle was repeated three times at the same amplitude in the form of a sinusoidal wave. The testing was terminated when one side (direction of displacement) of the wall failed even if the other side had not reached its maximum load. As for the monotonic tests, failure was defined by a post peak reduction of the in-plane shear load of more than 20% of the maximum load resisted for the direction being considered. During the tests, the vertical pre-compression load, the lateral in-plane (shear) load and all displacements were continuously logged and the nature and extent of the cracking was continuously observed.

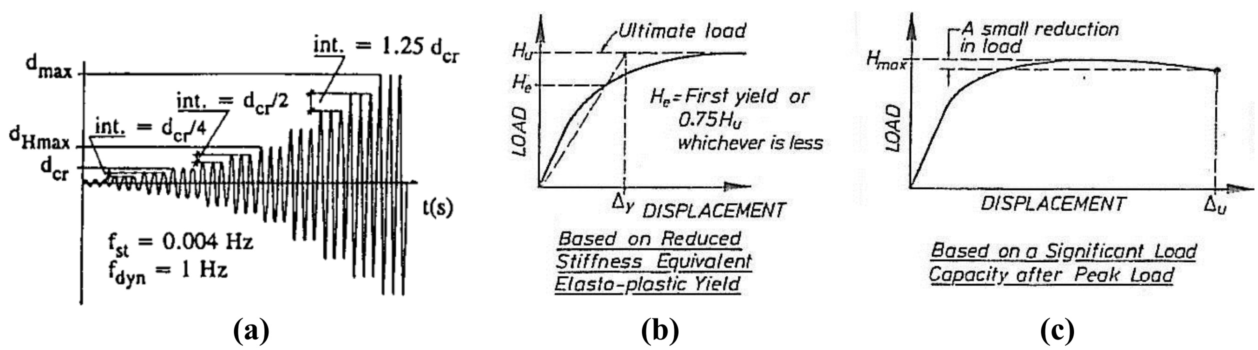


Figure 4: (a) Cyclic Displacement Time History [9], (b) Definition for Yield Displacement [10] and (c) Definition for Maximum Available (Ultimate) Displacement [10]

Available ductility factors were determined for the AR1 walls from the envelopes of lateral in-plane load versus displacement of the spreader beam (POT12 in Figure 3) using Equation 1 [10]. Note that the beam displacement was equal in each case to the displacement along the top edge of the wall panel by virtue of the beam being epoxy bonded to the masonry.

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{1}$$

where μ is the available ductility factor, Δ_y is the beam displacement at yield determined using the approach illustrated in Figure 4b and Δ_u is the maximum available (ultimate) beam displacement determined using the approach illustrated in Figure 4(c). For the determination of Δ_u in Figure 4c, [10] suggested a 20% reduction in load and this was adopted for the current study. In the case of cyclic loading, the values of Δ_y and Δ_u were determined for the displacement direction in which failure was first observed. Ductility factors were not calculated for the AR05 walls. As discussed in the next section, most of the AR05 walls failed via sliding at the base of the wall. Therefore, the maximum displacements for those walls were the displacements at the point of test termination (prior to 20% drop in load) and did not represent, in a consistent way, the displacement capacities of the specimens.

RESULTS AND DISCUSSION

The experimental results are summarised in Tables 1 and 2. The specimen ID indicates whether the specimen was unreinforced (U) or strengthened (S), the aspect ratio (AR1 or AR05), the reinforcement scheme (if reinforced) (S1 to S6) and the replicate number. The asterisk indicates specimens for which strain gauges were fitted to the FRP strips. Values bolded and underlined are those for the failure direction. The values in brackets represent % increases compared to the URM specimen tested under the same conditions. For cases for which neither direction is bolded, the test was terminated due to excessive sliding prior to "failure" as defined by 20% drop in maximum load, in which case the % increases in maximum load are based on the lower of the maximum loads in the two directions. Figure 5 shows envelopes of the recorded plots of in-plane shear load versus beam displacement (POT12 in Figure 3) for the cyclically displaced specimens.

Table 1: Experimental Results for Aspect Ratio 1.0 (AR1) Specimens

Specimen ID	Type of loading	Maximum Shear load (kN)		Maximum beam displacement (mm)		Ductility factor μ	Mode of failure
		East	West	East	West		
UAR1-1	monotonic	253.2	NA	10.4	NA	2.76	Bed joint sliding one course from top
UAR1-2	cyclic	**<u>257.2</u>**	-249.9	**<u>6.9</u>**	-5.9	1.59	Diagonal cracking
SAR1S1-1*	monotonic	267.8 (+6%)	NA	20.8 (+100%)	NA		Sliding at base
SAR1S1-2*	cyclic	**<u>274.6</u>** (+7%)	-238.9	**<u>12.1</u>** (+75%)	-8.1	2.52 (+58%)	Diagonal cracking
SAR1S2-1*	cyclic	**<u>254.7</u>** (-1%)	-242.5	**<u>14.9</u>** (+116)	-13.9	2.24 (+41%)	Diagonal cracking then bed joint sliding two courses from bottom
SAR1S2-2	cyclic	**<u>259.1</u>** (+1%)	-239.7	**<u>16.1</u>** (+133%)	-14.4	3.31 (+108%)	Bed joint sliding one course from top
SAR1S3-1*	cyclic	260.8 (+1%)	-265.6	23.6	-15.4	5.65	Sliding damage prior to testing
SAR1S3-2	cyclic	**<u>280.3</u>** (+9%)	-262.5	**<u>15.6</u>** (+126%)	-12.3	3.28 (+106%)	Diagonal cracking, crushing

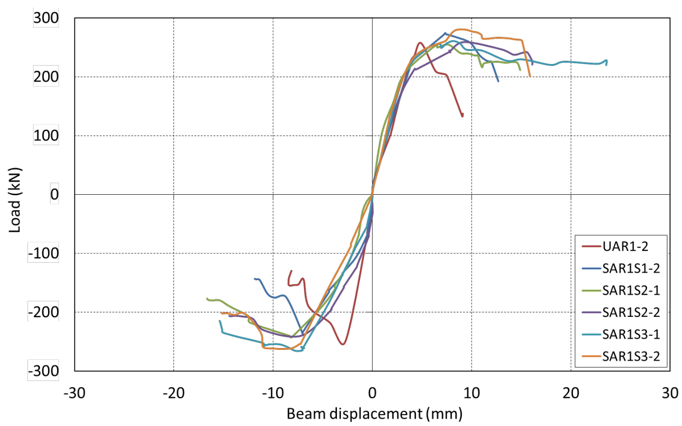
For the AR1 specimens, the effectiveness of the NSM FRP reinforcing under cyclic loading was clear. The URM specimen (UAR1-2) failed by diagonal cracking (Figure 6a). Negligible sliding was observed at the base of the wall. This mode of failure was non-ductile with limited displacement capacity (Figure 5a) and a calculated ductility factor of 1.59 (Table 1). Specimen SAR1S1-2 also failed by diagonal cracking (Figure 6b). However, the presence of the reinforcement (4 vertical strips) had the effect of restraining the diagonal cracks and allowing the load to continue to increase post cracking. The maximum load, displacement capacity and ductility were all increased compared to the URM specimen (Figure 5a and Table 1). Strain gauges on the FRP strips located close to the diagonal cracks recorded strains up to 2000με illustrating the role that the reinforcement played in restraining the diagonal cracking. No debonding or rupture of the FRP strips was observed.

Specimens SAR1S2-1 and SAR1S2-2 (Scheme 2) both failed by bed joint sliding. In the case of SAR1S2-1, diagonal cracks developed through the wall but these were restrained by the presence of the reinforcement (4 horizontal strips) before failure occurred along an unstrengthened bed joint two courses above the base of the wall (Figure 6c). For SAR1S2-2, diagonal cracking was not observed. The specimen failed by sliding along the bed joint one course below the top of the wall. The sliding failures observed for these two specimens resulted in ductile behaviour with large displacement capacities and calculated ductility factors (Table 1 and Figure 5a). The FRP strips did not debond or rupture during the tests, despite recording strains up to 9824με (SG15 in SAR1S2-1, Figure 6c). It should be noted that the performance of Scheme 2 (4 horizontal FRP strips) was comparable to Schemes 1 and 3 (which included vertical FRP strips) when using the current fixed-fixed boundary conditions. This was a vast improvement compared to the tests performed by [3] using the much less confined boundary conditions associated with the diagonal tension test [4]. As for the current study, [3] found that horizontal reinforcement resulted in bed joint sliding failures but in the latter tests these led to sudden and complete loss of load resisting capacity of the reinforced panels. The more realistic fixed-fixed boundary conditions used in the current study show that horizontal reinforcement, which is preferred from an aesthetic viewpoint, induces a ductile sliding failure mode which can provide significant improvements compared to the URM response, particularly in terms of displacement capacity and ductility.

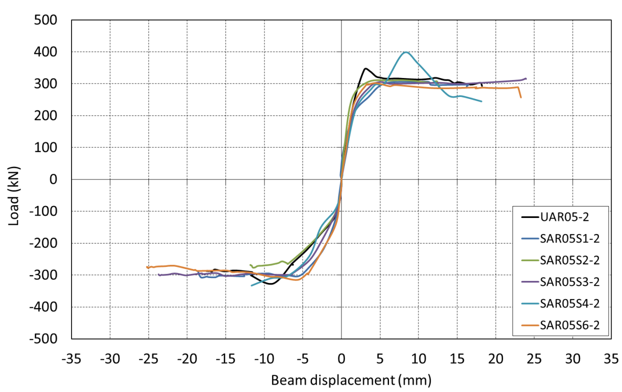
Table 2: Experimental Results for Aspect Ratio 0.5 (AR05) Specimens

Specimen ID	Type of loading	Maximum Shear load (kN)		Maximum beam displacement (mm)		Mode of failure
		East	West	East	West	
Pre-compression 1.8MPa						
UAR05-1	monotonic	**450.0**	NA	**12.1**	NA	Sliding at base then diagonal cracking
UAR05-2	cyclic	344.7	-327.2	18.2	-17.5	Rocking, sliding at base, crushing in bottom two courses
SAR05S1-2	cyclic	306.2	**-307.4** (-6%)	16.5	**-18.6**	Rocking, sliding at base, then diagonal cracking
SAR05S2-2	cyclic	311.4	-276.8 (-15%)	12.6	-12.0	Sliding damage prior to testing
SAR05S3-2	cyclic	316.7	-302.8 (-7%)	24.0	-23.9	Rocking plus sliding at base
SAR05S4-2	cyclic	**397.0** (+21%)	-333.0	**10.1**	-11.8	Sliding at base then diagonal cracking
SAR05S5-1*	monotonic	333.1 (-26%)	NA	14.9	NA	Sliding at base
SAR05S6-2	cyclic	302.1 (-8%)	-315.2	23.4	-25.2	Rocking, sliding at base, then diagonal cracking
Pre-compression 2.1MPa						
UAR05-3	cyclic	**292.4**	-321.8	**11.7**	-7.3	Rocking then diagonal cracking
SAR05S1-1*	cyclic	386.2	-357.6	16.2	-17.2	Rocking plus sliding at base
SAR05S2-1*	cyclic	352.2	-341.5	16.6	-18.2	Sliding damage prior to testing
SAR05S3-1*	cyclic	373.0	-365.4	17.1	-17.1	Rocking plus sliding at base
SAR05S4-1*	cyclic	354.5	-365.3	31.0	-24.0	Rocking plus sliding at base
SAR05S5-2*	cyclic	385.9	-373.1	16.0	-17.9	Rocking plus sliding at base
SAR05S6-1*	cyclic	**456.2**	-382.4	**11.7**	-11.5	Rocking, sliding, diagonal cracking

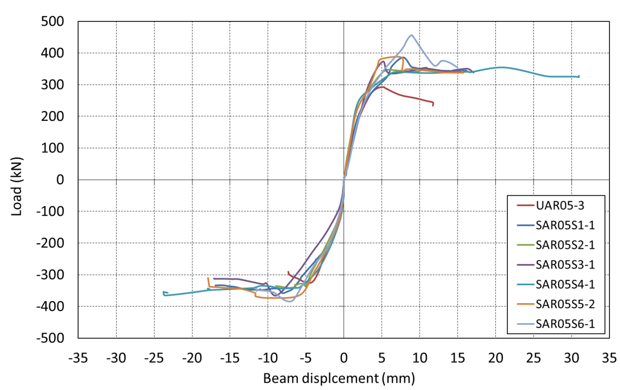
Specimen SAR1S3-1 was damaged due to an accidental, sudden large displacement of the horizontal actuator prior to testing. This overload caused failure of the bed joint one course below the top of the wall, damaged the vertical FRP strips where they crossed this joint and caused sliding failure along the base of the wall between the wall and footing beam. Without repair the wall was tested. Due to the pre-existing damage the wall displaced by sliding at the base. It was able to resist a maximum load and sustain cyclic displacements comparable to the other specimens (Table 1). Specimen SAR1S3-2 failed primarily by diagonal cracking with some crushing type damage to the bottom of the wall (Figure 6d). The wall initially started sliding at the base and then started to crack diagonally through the wall. The Scheme 3 reinforcement was effective in increasing the maximum load, displacement capacity and ductility compared to the observed URM response (Table 1) and it could be argued that this specimen performed the best overall when considering all three of these performance measures simultaneously. No debonding or rupture of the FRP strips was observed.



(a) AR1: Pre-compression 2.6MPa



(b) AR05: Pre-compression 1.8MPa



(c) AR05: Pre-compression 2.1MPa

Figure 5: Envelopes of Lateral In-plane (Shear) Load versus Beam Displacement (POT12) for Cyclically Displaced Specimens

Unfortunately, the results for the AR05 walls are much less conclusive. Almost all of the AR05 walls failed, at least in part, by sliding at the base (Table 2). The maximum loads observed were therefore quite consistent across all specimens, simply being a function of the frictional capacity of the sliding plane between the base of the wall and the footing beam. Even for cases in which other failure modes were observed the behaviour was dominated by sliding such that in most cases the post peak load did not drop by 20% before the test was terminated. Therefore, it was not possible to determine consistent measures of displacement capacity and ductility by which the relative performance of specimens could be compared. The recorded strains in the FRP strips were also very low indicating that the reinforcement played little role in the behaviour. The experimental program was designed to result in predominantly diagonal cracking failures. However, the higher than expected masonry bond strength resulted in walls with high resistance to in-plane shear cracking so that the walls preferentially failed by sliding at the base. Therefore, it is not possible to draw meaningful conclusions regarding the effectiveness of the FRP strengthening schemes applied to the AR05 walls under cyclic in-plane shear loading.

(a) UAR1-2 **(b) SAR1S1-2** **(c) SAR1S2-1** **(d) SAR1S3-2**

Figure 6: Failure Modes for Aspect Ratio 1 (AR1) Specimens

The URM wall tested under a pre-compression level of 2.1MPa (Specimen UAR05-3) did fail by diagonal cracking without sliding at the base but this was due to its lower masonry bond strength and so it is not possible to directly compare the performance of this specimen with the strengthened walls. However, this test did validate the original design of the testing program, showing that diagonal cracking failures would have occurred had this lower strength masonry been achieved (as planned) in the other specimens.

The unexpectedly high bond strength observed in all but one of the 23 specimens required the pre-compression levels to be increased above those determined using finite element modelling during the experimental design phase of the project. In the case of the AR1 walls the available testing apparatus could accommodate the increase required to achieve diagonal cracking failures. Unfortunately, the same was not true for the AR05 walls, with even the 2.1MPa tests resulting is predominantly sliding based failures.

Using material strength properties lower than those observed experimentally, [6] conducted finite element analyses to study numerically the effectiveness of the six schemes proposed here for the AR05 walls. Details of this study are beyond the scope of the current paper but can be found in [6].

CONCLUSION

An experimental study was conducted to assess the effect on strength, displacement capacity and ductility of reinforcing unreinforced masonry (URM) shear panels with near surface mounted (NSM) fibre reinforced polymer (FRP) strips. Twenty three wall panels (considering two different wall aspect ratios and six different NSM FRP reinforcing schemes) were subjected to vertical pre-compression combined with monotonic or increasing reversing cycles of in-plane lateral displacement under zero rotation (fixed-fixed) support conditions.

Wall specimens of aspect ratio 1.0 failed predominantly via diagonal cracking. The presence of NSM FRP reinforcement had the effect of restraining the diagonal cracking, allowing the walls to resist, on average, slightly greater load (up to 9% increase) and in all cases, considerably improving the displacement capacity (up to 133%) and ductility (up to 108%) compared to the URM response. Scheme 3 (combination of vertical and horizontal FRP strips) performed the best overall in terms of increases across all three performance measures. No debonding or rupture of the FRP strips was observed in any of the tests. For the aspect ratio 0.5 panels, base sliding failures dominated the experimental program, making it difficult to fully assess the effectiveness of the various reinforcing schemes.

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