

AN INVESTIGATION OF OUT-OF-PLANE LOADED SPRAYED GFRP STRENGTHENED MASONRY WALLS

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SUMMARY

Forty-eight concrete masonry walls sections were reinforced by spraying with Glass Fibre Reinforced Polymer (GFRP) then subjected to out-of-plane loads. Tests were performed in four different configurations: one-way simply supported walls of 800x800 spanning vertically or horizontally and cantilevered 800x1400 walls also spanning either vertically or horizontally. The sprayed GFRP layers were nominally 3 or 5-mm thick. The effect of using fasteners to anchor the glass-fibre layers to the wall was also investigated. The results show the lateral resistance of a wall can be greatly improved. Information on the behaviour of masonry walls with this type of reinforcement is provided.

INTRODUCTION

The use of fibre-reinforced polymer (FRP) has increased during the last few decades and many new applications have been studied. In the case of masonry structures, FRPs can be used effectively either to strengthen new construction or to rehabilitate old buildings. These applications have created several new avenues of research.

Although correct safety procedures must be followed, spraying glass fibre reinforced polymer (SGFRP) on to a masonry surface is a simple operation. The lateral resistance of structural masonry can be greatly improved through the application of this material (Haddad et al. 2007). Structural situations where this procedure could be advantageous include the strengthening of basement walls, earth, water or grain retaining walls, shear walls, and the retrofit and rehabilitation of masonry. In cases where the tension face also needs to resist

water penetration, the glass fibre composite may also act as an impermeable barrier.

Hollow concrete block masonry wallettes were constructed from 190-mm concrete units with type S mortar and tested in different configurations. The intention was to test small sections of SGFRP masonry where the internal forces are easily defined in order to investigate the shear and flexural strength. Some of the test configurations, like the cantilever walls, are believed to have been tried for the first time.

Experimental studies on the effects of different types of surface bonded GFRP (sheets or rebars, mounted in a limited surface area and not sprayed) on masonry are reported in Albert et al. (2001), Al-Salloum and Almusallam (2005), Turco et al. (2005), Triantafillou (1998), Hamoush et al. (2002), Corradi et al. (2002) and Galati et al. (2005). These studies report increases of 4.5 to 26 times in flexural strength and up to 2.5 times in shear strength, compared to unreinforced walls. When flexural failure is reported, failure is usually due to debonding of the FRP or rupture of the fibres. The use of SGFRP for concrete repair is reported in Boyd (2000) while its use in masonry strengthening is reported in Shaheen and Shrive (2005), Haddad et al. (2007) and Parsekian et al. (2007). The types of failure mode reported include FRP debonding, sliding-shear, flexure-shear or flexure. In the case of GFRP-masonry tests, flexure-shear is the most common cause of failure, being the flexural failure investigated most frequently analytically.

ANALYTIC ESTIMATIONS

The flexural strength of an unreinforced masonry section can be calculated directly from its flexural tensile strength - equal to 0.40 MPa or 0.80 MPa for the tension being normal or parallel to the bed joint as in CSA S304.1-04. The same properties are specified in ACI 530-05/ASCE 5-05/ TMS 402-05 as 0.431 and 0.862 MPa respectively, which are similar to the CSA values. Considering the Canadian standard and taking into account the bedded area of a 32-mm face-shell 190x790mm masonry section, as in the tests, gives moment strengths of 1.36 kNm and 2.73 kNm. The MSJC specified moment strengths are 7.75% higher.

The flexural strength of an FRP-strengthened section can be assessed theoretically through equilibrium of the reinforced section, taking into account the stress-strain relationship of each material and the composite section strain and stress distributions. A refined approach to this problem was assessed in Parsekian et al. (2007). For the specific case of hollow-block masonry strengthened with FRPs, Albert et al. (2001) proposed a simplified formula, assuming a triangular compressive stress distribution along the masonry face-shell thickness. The results compare well with those of the refined approach. From this last reference the resistant moment of a section is the lesser of:

$$a) M_{\max} = f_{u,FRP} \cdot A_{FRP} \cdot \left(d - \frac{a}{3}\right) \text{ in the case of FRP tension rupture, or}$$

$$b) M_{\max} = 0.5 \cdot f'_m \cdot b \cdot a \cdot \left(d - \frac{a}{3}\right) \text{ in the case of masonry crushing,}$$

with $f_{u,FRP}$ = FRP tension strength; f'_m = masonry compressive strength.

A_{FRP} = FRP area; d = section depth; a = face-shell thickness; b = section width.

Taking $f_{u,SGFRP} = 83$ MPa (Table 1), $f'_m = 24.4$ MPa, $a = 32$ mm and $b = 790$ mm, as in the tests, results in moments of resistance of 26 kNm and 43 kNm for the masonry section tested, with 3 and 5 mm nominal SGFRP layer thicknesses respectively (characteristic thicknesses of

2.2 and 3.2mm were considered as discussed in the next section). These values are for both vertical and horizontal bending directions.

According to the CSA S304.1-04, when there is no pre-compression, failure of unreinforced masonry at the mortar bed-joints in shear is limited to $0.16 \left(2 - \frac{M}{V \cdot d} \right) \sqrt{f'_m} A_e$, with $0.25 < M/Vd < 1$. Considering the simply supported tests, the M/Vd ratio at the first bed joint was 0.82, except when the supports were moved over the fibre (EX case). In this case, the ratio was 0.53. With the section tested and $f'_m = 24.4$ MPa, the maximum shear forces are limited to 47 kN and 58 kN, for M/Vd equal to 0.82 and 0.53 respectively. In the cantilever tests the M/Vd ratio was limited to 1.0 and the shear force limited to 40.0 kN.

The ACI 530-05/ASCE 5-05/ TMS 402-05 limits the nominal shear to $0.83 \left(4.0 - 1.75 \frac{M}{V \cdot d} \right) \sqrt{f'_m} A_e$, which gives shear forces of 53 kN, 64 kN and 47 kN for the two simply supported cases with M/Vd equal to 0.82 and 0.53 and for the cantilever tests respectively, some 10 to 17% greater than values calculated from the CSA standard.

EXPERIMENTAL INVESTIGATION

The dimensional properties, absorption and compressive strength of the hollow concrete blocks were characterized. Three 5-cm cube mortar samples were moulded at random times on the five days of wall construction and tested at 28 days of age (except for samples of days 1 and 2 which were tested at 40 days and the results corrected to 28 days). Five face-shell-bedded, two-block-high prisms were constructed and tested in compression with face-shell capping. The results of these tests together with the properties of the glass fibre and resin used in the composite are presented in Table 1. The properties of the SGFRP composite can be evaluated theoretically from its components following the procedures recommended in Boyd (2000). These calculations take into account the elastic modulus, strength, proportion of each material (polyester matrix or fibreglass), as well as the efficiency of the randomly sprayed fibres. Since the concepts and formulas involved are extensive, they are not repeated here. The SGFRP tensile strength was estimated to be 96.4 MPa and the elastic modulus 9.08 GPa. These results are close to values reported in Boyd (2000) for a SGFRP with similar characteristics, but differ significantly from values reported in Haddad et al. (2007), where the proportions of the materials were different. Further tension tests on SGFRP coupons are still in progress.

The SGFRP consisted of chopped glass fibre randomly bonded together inside a polyester matrix. The curing agent of the resin was a Methyl Ethyl Ketone Peroxide (MEKP), the commercial product Cadoc D-50. These materials were mixed together and sprayed onto the wall in an air jet from a spray gun attached to a 120-psi air pressure line. During application, acetone was periodically used to clean the finishing tools and spray gun. The volume of each material in the final composite was: fibreglass – 19%, polyester resin – 79%, MEKP – 2%. The glassfibre was chopped inside the gun to a length of 20-mm. Within the gel time after spraying, the fibres were rolled into the surface. After curing, the edges of the GFRP were trimmed with a knife.

Even though a thickness gauge was used to guide the thickness of the sprayed layer, this dimension is highly dependent on the experience of the contractor. While an experienced contractor was hired, some variation in the thickness of the GFRP sheet was expected.

Table 1: Material properties

	Property	Result	Test Method
Concrete blocks – standard unit	Width / length / height / web thickness (nominal = 190 / 390 / 190 / 32)	191.11 ± 0.76 / 390.92 ± 0.53 / 190.07 ± 0.42 / 32.87 ± 0.35	ASTM C140-03 CSA A165.1-04)
	Compressive strength average	31.3 ± 1.5 MPa	
	Characteristic compressive strength (f'_m)	29.0 MPa	
	Absorption	12.8 ± 0.2 %	
	Density	$1,754 \pm 20$ kg/m ³	
	Average net area	$40,167 \pm 454$ mm ²	
Mortar	Compressive strength	11.9 ± 2.1 MPa	ASTM C109-05
Prism	Compressive strength	27.1 ± 1.7 MPa	CSA A369.1-M90
	Characteristic compressive strength	24.4 MPa (CSA S304.04)	
GFRP	Tensile strength average	101 ± 12 MPa	ASTM D638-03
	Characteristic tensile strength	83 MPa	
	Elastic Modulus	9.5 ± 1.6 GPa	

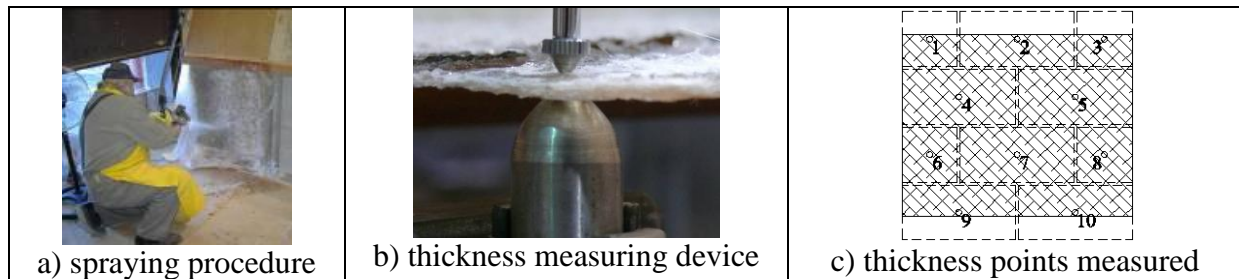


Figure 1: GFRP spraying and thickness measuring

After the structural tests, a special device was assembled where a displacement transducer was mounted over a steel ball-point (Figure 1-b) in order to assess the variability in the thickness of the layer. Placing a sheet between the two points gave the thickness at that location. Both parts were clamped to cantilevered steel sections so that the sheet could be moved along and the thickness measured at any point. After testing each wall, the GFRP sheet was removed. The thickness was then measured at ten different points (Figure 1-c). The average thickness of each layer was calculated. For the 3-mm specified layer, the thickness was 3.7 ± 0.9 mm with a 95% characteristic value of 2.2 mm (149 points measured). For the nominal 5-mm layer, these values were 5.1 ± 1.1 mm, and 3.2 mm (130 points measured). While the average value can be considered as a good result in both cases, one can observe that there is high variability. The average value could be used for cost-estimation and the characteristic value for design.

Specimens

48 walls were subjected to out-of-plane load in four different set-ups: simply-supported or cantilevered, horizontal or vertical span. The simply-supported walls were two-blocks long by four courses high (790x790mm), while the cantilevered walls were two-blocks long by seven courses high (790x1390mm). All walls were built by an experienced mason on small wood pallets with no bonding agent between the pallet and the blocks, and subsequently sprayed with GFRP to one of the nominal thicknesses. In the simply-supported case, the edges of the GFRP were trimmed after curing on lines 80 mm from the edges of the masonry where the

supports were to be placed during testing. In some specimens, the SGFRP sheet was anchored at its ends with #14x1 ¼” screw fasteners and 5/16” washers. The fasteners were anchored to the blocks with a high-performance non-shrink grout (Sika Grout 212 HP). In other tests the supports were moved over the SGFRP layer. The test programme is detailed in Table 2.

Table 2: Test type and number of specimens

Legend	Description			n
URM-H	790 x 790 mm wall, simply supported, 700-mm span (except EX where span = 580mm), two line loads each 100-mm from mid-span	Horizontal span	Unreinforced	4
SH-H-3			3-mm FRP, no anchorage	4
SH-H-5			5-mm FRP, no anchorage	4
SH-EX-H-3			3-mm FRP, no anchorage, fibre extend over support	3
SH-EX-H-5			5-mm FRP, no anchorage, fibre extend over support	3
URM-V		Vertical span	Unreinforced	4
SH-V-3			3-mm FRP, no anchorage	4
SH-V-5			5-mm FRP, no anchorage	4
SH-AN-V-3			3-mm FRP, no anchorage, fibre extend over support	3
SH-AN-V-5			5-mm FRP, no anchorage, fibre extend over support	3
FL-H-3	1390 x 790 mm wall, cantilevered, 1200-mm span, line load 100-mm from free edge	Horizontal span	3-mm FRP, no anchorage	3
FL-H-5			5-mm FRP, no anchorage	3
FL-V-3		Vertical span	3-mm FRP, no anchorage	3
FL-V-5			5-mm FRP, no anchorage	3

Test set-up #1 and #2 – simply support wall at vertical span and horizontal span

An overview of the test is given in Figure 2-a. A steel load frame was assembled as in Figure 2-e. Since the FRP was 80-mm from edges of the specimens, the supports bore on the blocks only, except for the “EX” case, where the supports bore on the fibre layer. Bearing on the blocks is more likely to occur in a real application as the repair will most likely be performed after the wall is built. A set of steel-plates and rollers was placed under each wall to allow the bottom of the wall to rotate freely and to slide (Figure 2-d). Each bearing point consisted of a combination of roller, steel plate and 12-mm fibreboard (Figure 2-c). Ten linear displacement transducers (LSC) were used to evaluate wall rotation. Four LSC were placed in front of each load line, four in front of each support line, and two at the top of the wall 160-mm apart. Additionally, a unidirectional 6.35-mm length strain gauge was placed at the centre of the GFRP, 30 mm below the mid-span mortar joint. Displacement was applied by a MTS actuator of 160kN-capacity in displacement control at a rate of 0.8mm/min. The horizontal span test arrangement was similar to the vertical one but with the load lines and supports rotated by 90° (Figure 2-b). In order to prevent the actuator and the wall from translating horizontally due to any possible eccentricity, both were braced laterally.

Test set-up #3 and #4 – cantilevered wall at vertical span and horizontal span

In the vertical cantilever tests, the bottom course of each 790x1390mm wall was compressed against beam sections with a hydraulic jack. A force of 80-kN was applied, resulting in approximately 0.5 MPa of compression stress to clamp the bottom part of the wall (Figure 2-f and g). A pair of LSC was placed at each of three levels (195 mm, 700 mm and 1,195mm) from the bottom of the wall. Four strain-gauges, two on the compression side and two on the tension side, were mounted at the mid-height of the penultimate bottom course, each 200mm from the wall centreline. A line load was applied at the mid-height of the top course, resulting

in a 1,200-mm span. Displacement was applied at this height at a rate of 3.0mm/min. In the horizontal span case, the steel frame was rearranged and two box-sections were used to clamp the left side of the wall (Figure 2-h and i). The core closest to the left side of each wall was grouted as the clamp was centred on this core. A line-load was applied at the other end, centred on the last core, providing a cantilever length of 1,200 mm. The weight of the wall was supported by a cart with wheels allowing free translation in the load direction, except at the clamp. Each specimen was instrumented and loaded similarly as in the vertical tests.

RESULTS AND DISCUSSION

The results are summarized in Table 3. The results are organized with respect to the measured SGFRP thickness rather than the nominal value. The displacements were filtered from the fibre crushing in the shear tests and rigid body rotation in the cantilever tests. Rigid body rotation was calculated by comparing the readings of different points to those of an elastic-line cantilevered-beam. The “Expected failure loads” for those simply supported cases expected to fail in shear were calculated following CSA standards.

Of the four unreinforced walls tested in the vertical span, two broke at the central mortar joint and two at the 3rd course mortar joint. While the moment is constant and greater at the central joint, the specimen is very sensitive to any eccentricity and set-in movements of the test because of the very small failure load. Further, masonry is notoriously variable and the resistance of the 3rd course joint may have been less in proportion to the applied moment compared to the centre joint in those specimens. The other four unreinforced walls, tested in horizontal bending, failed with a crack at the central head joint which then stepped along the nearest bed and head joints. The failure loads were only one third of that expected for the vertical span and about 60% of that expected for the horizontal span.

All simply-supported SGFRP-walls tested over a vertical span (SH-V) failed through shear at either the symmetrical 1st or 3rd mortar joint with a diagonal crack in the block towards the centre of the wall (Figure 3-a). The failure was explosive with no prior damage being observed. When the GFRP was not anchored, the sheet debonded after the block broke. The walls with end anchorage failed similarly to the ones without anchorage. When the GFRP did not debond, cracking occurred along a diagonal line through a block, going from the support line to the line of load application (Figure 3-b). The GFRP sheet did not appear to be damaged after failure in all tests and could be removed in one piece. The failure load was close to that expected for unreinforced masonry. There is a small tendency of a small increase in strength with increasing the SGFRP thickness. End anchorage of the GFRP did not alter the results.

In the SH-H tests the webs of the blocks broke within a diagonal line from the support to the load point (Figure 3-c) at about half the expected load. The same happened when the fibre extended over supports (Figure 3-d). The GFRP layer appeared not to be damaged. In the vertical cantilever test the walls failed at the first and second courses from the bottom where the moment values were highest. It was possible to observe diagonal cracks within the clamped course. Vertical tension cracks splitting the face-shells of the blocks could be seen at the 2nd course. In some walls, SGFRP tearing was observed with no debonding. This only occurred after the peak load was reached and with large crack opening. Typical cracking of one specimen with SGFRP tearing is shown in Figure 4-a. The failure loads were close to those expected for 3-mm SGFRP thick layers, but lower than expected for 5-mm. However, it

should be noted that the tensile strength of the SGFRP used in the calculations was assessed from 2.5-mm thick coupon tests. It may be possible that the 5-mm strength is not the same.

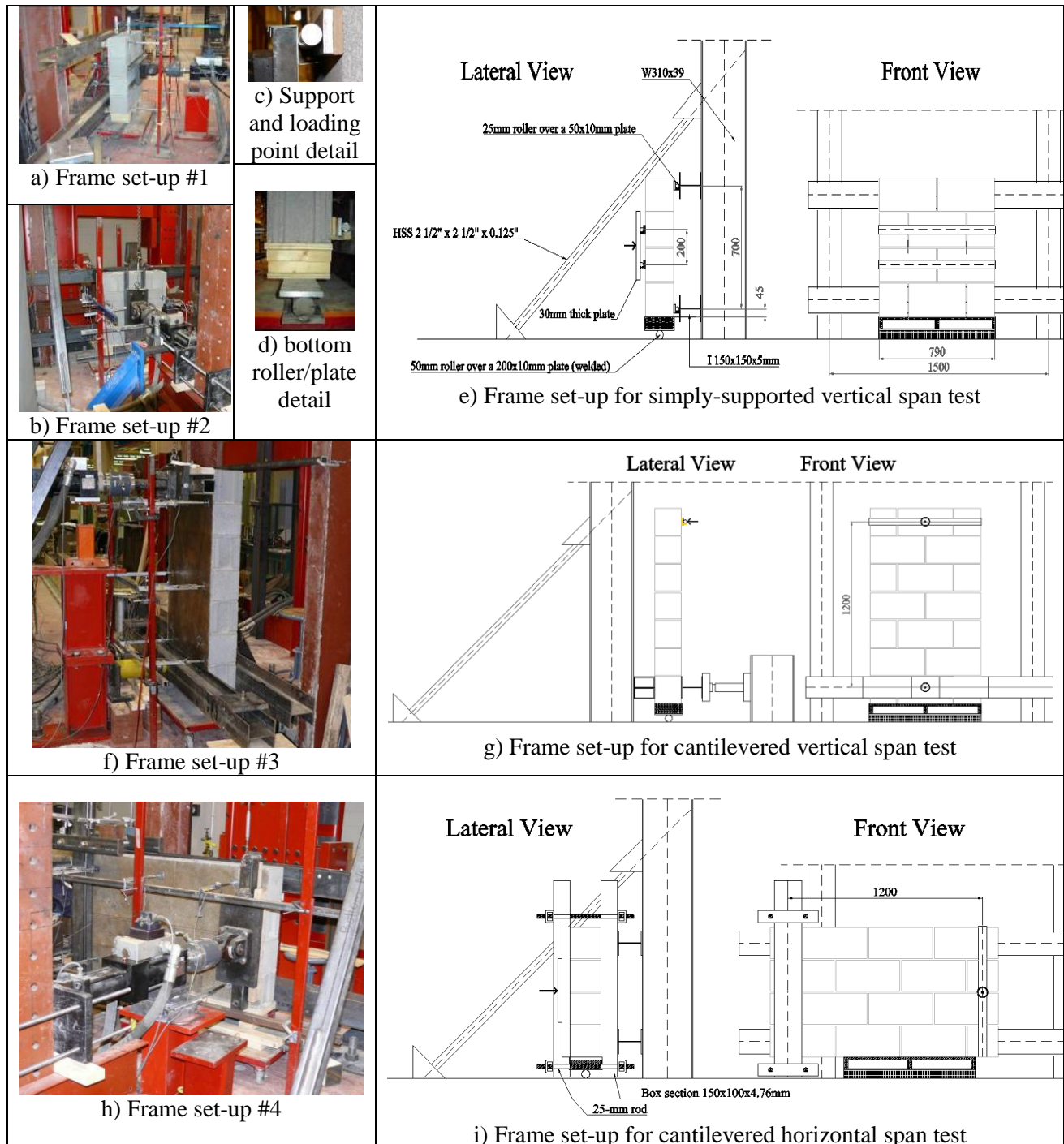


Figure 2: Test configurations

The horizontal-span cantilevered walls failed at approximately one third of the expected flexural failure load through breaking of the web/face-shell interfaces. A small crack at a web/face-shell interface would be observed on the tension side of the wall, close to the clamping. This crack would grow and spread to other interfaces until all were broken (Figure 4-b).

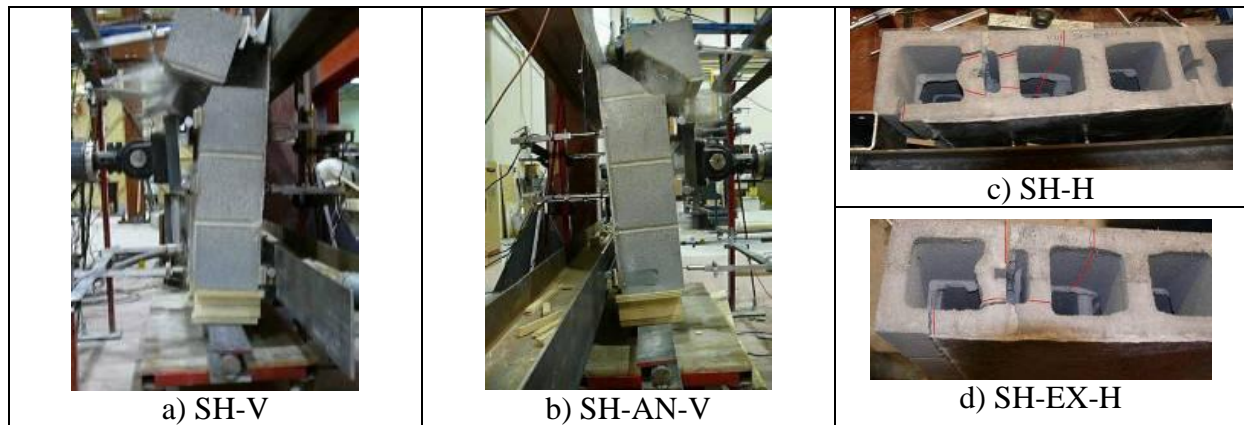


Figure 3: SH tests: failure modes

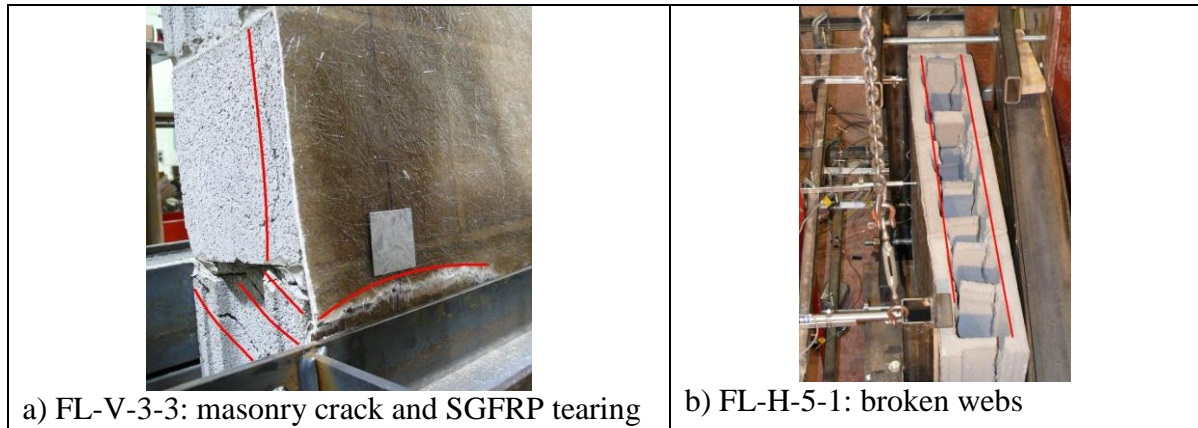


Figure 4: FL tests: failure modes

Table 3: Test Results Compared to Actual SGFRP Thickness

Test Type	Actual Thickness (mm)	Failure Load (kN)				Average / Expected (%)	Displacement (mm)		
		Average	n	SD	Expected		60% Load	100% Load	δ 100% / δ 60%
URM-V	-	3	3	0	11	29%			
SH-V	$2 \geq th < 4$	106	3	5	94	113%	0.21	0.56	2.7
	$4 \leq th < 6$	113	5	15	94	120%	0.20	0.55	2.7
SH-AN-V	$2 \geq th < 4$	98	2	0	94	104%	0.21	0.51	2.4
	$4 \leq th < 6$	112	5	12	94	119%	0.24	0.62	2.6
URM-H	-	12	4	2	22	56%	0.09	0.21	2.3
SH-H	$2 \geq th < 4$	50	1		94	53%	0.23	0.43	1.9
	$4 \leq th < 6$	45	4	2	94	48%	0.16	0.35	2.2
SH-EX-H	$2 \geq th < 4$	63	3	0	116	54%	0.18	0.49	2.7
	$4 \leq th < 6$	67	3	1	116	58%	0.17	0.39	2.3
FL-V	$2 \geq th < 4$	20	3	1	26	77%	8.07	16.41	2.0
	$4 \leq th < 6$	20	2	0	43	47%	7.03	15.99	2.3
FL-H	$2 \geq th < 4$	11	2	1	26	41%	3.60	10.57	2.9
	$4 \leq th < 6$	14	3	3	43	33%	3.42	8.39	2.5

CONCLUSIONS

From the analysis of the experiments to this point, it can be observed that:

- Flexural failure of unreinforced walls was smaller than predicted from the flexural tensile strength specified in the CSA and MSJC codes;
- A large increase in lateral load capacity occurred with the SGFRP strengthened walls;
- The thickness of the sprayed GFRP layer varied considerably, even with an experienced contractor using a thickness control gauge. In the work reported here, characteristic (95% confidence) values of 2.2 and 3.2 mm were measured for specified thicknesses of 3.0 and 5.0 mm. The characteristic values should be considered in design.
- There was little increase in shear strength with increasing SGFRP thickness in vertically-spanning masonry. The adoption of the unreinforced masonry limit in this case seems appropriate.
- With horizontal spanning, the shear failure occurred earlier, with the webs of the blocks breaking at approximately half of the strength expected for unreinforced masonry. Care should be taken regarding this type of failure.
- The flexural strength of sprayed masonry was smaller than expected, with the block web/face-shell interface being critical in some cases.

The tests will be analyzed further and complementary tests performed to help understand the observed behaviours.

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