SHEAR RESPONSE OF MASONRY WALLS WITH EXTERNAL CFRP REINFORCEMENT

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ABSTRACT :

In this study was characterized the shear behavior of full-scale masonry walls, externally retrofitted with carbon fiber reinforced polymers (*CFRP*), subjected to cyclic in-plane shear loads. Two configurations of reinforcement were studied: horizontally and diagonally oriented fibers (0° and 45º approximately with respect to the masonry courses, respectively). Twenty four clay-brick walls were tested: eight walls had horizontal steel shear reinforcement (*RM*), while 16 had no shear reinforcement (*URM*). Two walls had eccentric CFRP reinforcement (fibers bonded on only one side of the wall), and also two walls with special anchorage details of the CFRP reinforcement were tested. The effects of configuration and amount of CFRP reinforcement on the shear response were compared in terms of cracking, maximum strength, failure mechanism and displacement capacity.

The maximum strength of the URM walls increased in 50 to 80%, while for similar CFRP reinforcement ratios the maximum strength of the RM walls increased only 2 to 34%. All the walls showed large increase of deformation capacity, between 40 and 180%. The CFRP reinforcement produces a distributed pattern of cracks, with thinner cracks than the walls with no CFRP. Using more than one strip of CFRP reinforcement produces a less brittle failure, with residual strength, than using a single strip of CFRP reinforcement. The anchorage details were very effective in improving deformation capacity and failure mode of the walls. Finally, an expression to estimate the maximum strength is proposed.

KEYWORDS: Carbon fiber, Retrofit, Masonry walls, Shear test.

1. INTRODUCTION

Masonry has been typically used in low income and historic buildings, which have suffered large damage due to in-plane shear actions during resent earthquakes, in Chile (Figure 1), USA and Mexico (Klingner 2004), Iran (Fallahi et al. 2003), among others, showing the need for the structural retrofit of this type of buildings.

Figure 1 Damages due to in-plane shear action in Chilean masonry buildings, Tocopilla earthquake (M_w=7.7), 2007.

A reinforcing technique consisting of FRP externally bonded to the walls is currently under study. This technique has as advantages low weight-strength ratio, short installation periods, and very low intervention on the structure.

The use of externally bonded FRP as in-plane shear reinforcement for masonry walls has been under study since 1995. Several experimental programs have been carried out: a network of diagonal FRP strips and horizontal FRP reinforcement (discrete strips or full covering of the wall surfaces), subjected to monotonic, cyclic and/or dynamic shear loading (Schwegler 1995, Stratford et al. 2004, ElGawady et al. 2005, Alcaíno and Santa María, 2008), obtaining important increases in the shear strength of the walls.

In this paper are reported the results of in-plane cyclic shear tests of twenty four full-scale masonry walls with nominal dimensions 2400x1975x14mm, designed fail in shear before bending failure occurred. Three walls had no external reinforcement when tested (control walls); four walls were pre-damaged by loading them up to the maximum strength of the corresponding control wall, then they were repaired with CRFP fabric strips, and later tested to failure (repaired walls); and 17 walls were reinforced with CFRP fabric strips (retrofitted walls) and then tested to failure. Three RM walls were retrofitted with diagonal CFRP strips in both sides of the walls (one was a repaired wall); two RM walls (one a repaired wall) had horizontal CFRP strips; two RM walls had eccentric CFRP reinforcement (the strips were bonded on one side of the wall). Seven URM walls were retrofitted with diagonal CFRP strips (one was a repaired wall and one had special anchorage detailing of the CFRP reinforcement); seven URM walls had horizontal CFRP strips (one was a repaired wall and one had special anchorage detailing of the CFRP reinforcement). Two types of special anchorage details were tested: in the wall with horizontal strip the reinforcement consisted of two CFRP strip hairpins overlapped 300mm at midlength of the wall; in the wall diagonally retrofitted hairpins with legs 600mm long were bonded to both ends of each diagonal strip.

The effects of the configuration and amount of CFRP reinforcement on the shear response were compared in terms of cracking, maximum strength, failure mechanism and displacement capacity: the URM walls had an increase in maximum strength of 50 to 80%, while for similar CFRP reinforcement ratios the RM walls had 2 to 34% increase. All the walls showed large increase of deformation capacity, from 40 to 180%. The CFRP reinforcement produces a more distributed pattern of cracks, with thinner cracks than the walls without CFRP. Using more than one strip of CFRP reinforcement produces a less brittle failure, with residual strength, than using a single strip of CFRP reinforcement. The anchorage details were very effective in improving deformation capacity and failure mode of the walls**.** Finally, expressions to estimate the cracking and the maximum strength are proposed.

2. MATERIALS

The walls were fabricated using hollow clay bricks (140x290x112mm), with approximately 13-mm-thick mortar joints. The average compressive strength and modulus of elasticity of the masonry were 11.3MPa and 6618MPa, respectively. The shear strength and shear modulus of the masonry measured in diagonal compression tests were 0.81MPa and 2571MPa, respectively. The mortar tensile-flexural strength was 5.0MPa and its compressive strength was 23.4MPa. The CFRP reinforcement consisted of 0.13mm thick carbon fiber fabric, bonded to the masonry walls on site using the procedure indicated by the producer. The nominal modulus of elasticity of the fabric as informed by the producer was 230kN/mm^2, while the measured modulus of elasticity was 250kN/mm^2. The nominal maximum strength was 4.3kN/mm^2.

3. TEST SPECIMENS

The specimens were identified as follows: first was indicated the retrofitted configuration (*H*= horizontal strips; *D*= Diagonal strips; *HR*= Repair with horizontal strips; *DR*= Repair with diagonal strips; *HE*= Eccentric horizontal strips; *DE*= Eccentric diagonal strips; *HA*= Horizontal strips with special anchorage detail or *DA*= Diagonal strips with special anchorage details); then was indicated the wall type (*URM* or *RM*); followed by the number of strips on each side of the walls and the width in mm of each CFRP strip; finally, the number of the specimen.

Previous bond tests showed that to achieve the maximum strength it was necessary to bond the CFRP to the substrate of the clay bricks. The surface of the walls was polished with a common sander machine, and then the mortar joints were filled with mortar to level the wall surface, obtaining a maximum bond strength of 0.24kN per millimeter of width of the CFRP strip (Alcaíno and Santa María, 2008). The adherence test, surface preparation and CFRP bonding procedures are shown in Figure 2.

The CFRP reinforcement ratio ρ was calculated as indicated in Eqn.3.1, where b_f and t_f are the total width and thickness of the fabric that crosses a potential diagonal crack; α is the angle between the fabric and the masonry courses; h and b are the height and the thickness of the wall. Notice that the reinforcement ratio does not represent the amount of CFRP used, and therefore it does not represent the real cost associated with the reinforcement scheme. It can be seen in Table 1 that some walls with the same amount of CFRP reinforcement have different reinforcement ratios.

$$\rho = b_f \cdot t_f \cdot \cos(\alpha) / b \cdot h \tag{3.1}$$

Table 1. CFRP Reinforcement of the walls tested.

Specimen ID	#	Reinforcement type	FRP reinforcement	FRP amount (m^2)	ρ (‰)
URM (Control)	2	-	-	-	-
D-URM-1x300	2	Diagonal	1 x 30cm	3.37	0.20
D-URM-1x200	2	Diagonal	1 x 20cm	2.25	0.13
D-URM-3x100	1	Diagonal	3 x 10cm	2.25	0.20
DR-URM-1x200	1	Diagonal	1 x 20cm	2.25	0.13
DA-URM-1x200	1	Diagonal	1 x 20cm	3.40	0.13
H-URM-3x150	2	Horizontal	3 x 15cm	1.78	0.42
H-URM-3x100	2	Horizontal	3 x 10cm	1.19	0.28
H-URM-1x300	1	Horizontal	1 x 30cm	1.19	0.28
HR-URM3x100	1	Horizontal	3 x 10cm	1.19	0.28
HA-URM-3x100	1	Horizontal	3 x 10cm	1.45	0.28
RM (Control)	1	-	-	-	-
D-RM-1x100	1	Diagonal	1 x 10cm	1.12	0.07
DE-RM-1x200	1	Diagonal	1 x 20cm	2.25	0.13
D-RM-1x200	1	Diagonal	1 x 20cm	2.25	0.13
DR-RM-1x200	1	Diagonal	1 x 20cm	2.25	0.13
H-RM-3x100	1	Horizontal	3 x 10cm	1.19	0.28
HE-RM-3x200	1	Horizontal	3 x 20cm	1.19	0.28
HR-RM-3x100	1	Horizontal	3 x 10cm	1.19	0.28

Figure 2 Bond test, surface preparation and CFRP bonding.

4. CYCLIC SHEAR TEST PROCEDURE

The in-plane cyclic load and a simultaneous constant vertical load were applied by means of hydraulic rams attached to the reaction frame (Figure 3). The horizontal displacement-controlled loading consisted of two cycles at each displacement level, starting at 0.2mm and increasing up to 24mm, if failure did not occur before (see the loading program in Figure 3). The nominal vertical load was 98kN, approximately equivalent to the load of a first floor wall in a three story building with concrete slabs and a light roof. The walls were fixed to the floor and free to rotate at the top with the load applied 1700mm from the top of the bottom transfer beam (aspect ratio

M/Vd≈0.86). The horizontal displacement was measured at the top transfer beam with a horizontal transducer. More details can be found in Alcaíno (2007).

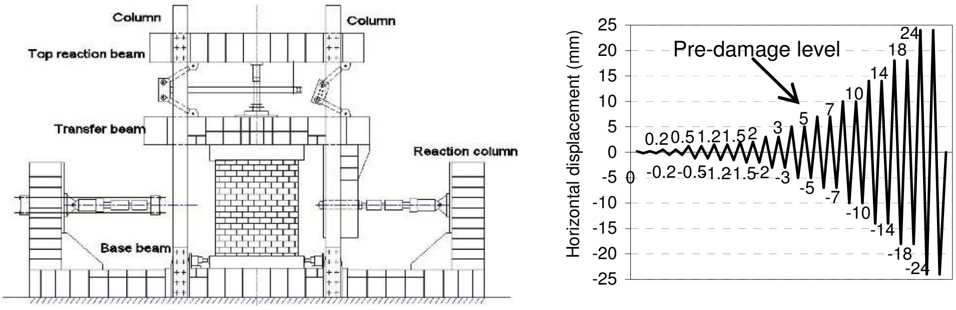

Figure 3 Reaction frame and horizontal displacement program for the cyclic shear tests.

5. DISCUSSION OF RESULTS

In Table 2 are shown the cracking pattern and main failure modes of each type of wall. Also are shown the horizontal displacement and load (average between positive and negative directions and between the walls 1 and 2 in the corresponding cases) at which the first major crack occurred (Δ_{cr}, V_{cr}); and the average of the maximum horizontal load with its corresponding displacement (Δ_{Vmax}, V_{max}). Also are reported the ratios of retrofitted or repaired to non retrofitted wall displacements and strengths.

In Figure 4 are shown typical hysteretic response curves of the different retrofitted configurations. It can be seen that the behavior of the walls was approximately linear until the first major crack occurred. After cracking the walls had non-linear behavior: the shape of the loops of the retrofitted walls is similar to the non-retrofitted walls, showing narrow loops and a degradation of the stiffness as the deformation increase. After the walls reached the maximum shear strength, the load capacity began to decrease until failure occurred. It was observed that there was less decrease of load capacity as the CFRP strips were more distributed. Also, the decrease of load capacity after peak strength was reached was smaller in the horizontally retrofitted walls. The anchorage details were very effective in improving the deformation capacity of the walls and changed the failure mode to a less brittle mode.

The eccentric horizontal CFRP reinforcement did not produce an increase in the cracking shear or maximum strength of the wall, while the eccentric diagonal reinforcement had a moderate increase of the maximum strength. Both configurations showed significant increase in the deformation capacity. Finally, the pre-damaged masonry walls repaired with CFRP strips showed, as expected, an initial stiffness lower than the retrofitted walls, but the maximum strength and post peak behavior was similar to the behavior observed in the retrofitted walls.

5.1 Cracking pattern

The first major crack in the URM and RM retrofitted walls occurred at displacement levels between 2.5 and 3.0mm. This is an increase of 100 to 150% of the cracking displacement of the URM control walls, while the displacement of the RM walls did not change with respect to the RM control wall. The cracking strength of the URM retrofitted walls increased up to 60%, while no difference between control and retrofitted RM walls was observed.

In Figure 5 is shown the cracking pattern at 7mm of displacement, which in most cases is very close to the maximum strength level. The URM walls had one large crack in each main diagonal. The walls with CFRP strips presented several cracks, increasing the number of cracks and decreasing their width as the number of strips increased.

5.2 Failure modes

The failure modes are listed in Table 2 and shown in Figure 6. The walls with no CFRP strips had a brittle failure, with two main wide diagonal cracks.

The walls diagonally retrofitted with one strip per diagonal had a brittle failure with a large sudden loss of strength. The most common failure mechanism in this walls was as follows: the bricks at the ends of the walls were damaged due to high compressive stresses produced by in-plane bending of the walls; failure occurred when peeling stresses at the end of a strip broke the previously damaged bricks, the fabric delaminated along 50% of its length on one face of the wall and then the strength dropped sharply. Failure occurred between 10 and 14mm of horizontal displacement, generally during the first 14mm displacement cycle. Failure of the walls with three diagonal strips per direction and with anchorage details occurred at a larger horizontal displacement, close to 18mm, but with a similar maximum strength. The repaired walls (DR-URM-1x200 and DR-RM-1x200) reached a maximum strength similar to that of the corresponding retrofitted walls (D-URM-1x200 and D-RM-1x200).

Table 2. Cracking pattern, failure modes and average results: first major crack and maximum strength

Specimen ID	Cracking pattern	Failure mode	First major crack				Maximum strength			
			Δ (mm)	Load (kN)	$\frac{\Delta_{cr}^{Retr.}}{\Delta_{cr}^{Non-retr.}}$	$\frac{V_{cr}^{Retr.}}{V_{cr}^{Non-retr.}}$	Δ (mm)	Load (kN)	$\frac{\Delta_{V\max}^{Retr.}}{\Delta_{V\max}^{Non-retr.}}$	$\frac{V_{\max}^{Retr.}}{V_{\max}^{Non-retr.}}$
URM (control)	MC	BF-MC	1.22	121.4	-	-	4.14	140.6	-	-
D-URM-1x300	ICN	PO	2.91	197.4	2.39	1.54	10.05	255.2	2.43	1.81
D-URM-1x200	ICN	PO-FR	2.78	170.4	2.28	1.40	9.12	221.7	2.20	1.58
D-URM-3x100	MCN	PO	2.35	165.2	1.93	1.36	9.68	258.8	2.34	1.84
DR-URM-1x200	MC-ICN	PO	-	-	-	-	9.90	237.2	2.39	1.67
DA-URM-1x200	ICN	PO	2.48	180.4	2.03	1.49	7.15	226.7	1.73	1.61
H-URM-3x150	MCN	DB-PO	2.50	156.8	2.05	1.29	9.00	225.2	2.18	1.60
H-URM-3x100	MCN	DB-PO	2.76	160.4	2.26	1.32	9.10	215.0	2.12	1.53
H-URM-1x300	MC-MCN	PO	2.83	174.5	2.32	1.44	6.99	216.7	1.69	1.54
HR-URM3x100	MC-MCN	DB-PO	-	-	-	-	11.52	226.8	2.78	1.61
HA-URM-3x100	MCN	DB-PO	2.78	166.7	2.28	1.37	9.70	239.1	2.34	1.70
RM (Control)	MC-MCN	BF-MC	2.43	163.0			5.80	193.1	-	-
D-RM-1x100	ICN	PO	2.41	162.5	0.99	1.00	9.84	239.0	1.70	1.24
DE-RM-1x200	ICN	PO	2.51	161.8	1.03	0.99	8.26	242.0	1.43	1.25
D-RM-1x200	ICN	PO-FR	2.24	174.0	0.92	1.07	9.70	259.6	1.67	1.34
DR-RM-1x200	MC-MCN	PO	-	-	-	-	11.15	258.0	1.92	1.34
H-RM-3x100	MCN	DB-PO	2.75	143.3	1.13	0.88	8.76	217.7	1.51	1.13
HE-RM-3x200	MCN	PO	1.36	100.6	0.56	0.61	11.85	197.4	2.04	1.02
HR-RM-3x100	MC-MCN	DB-PO	-	-	-	-	10.00	243.7	1.73	1.26

Notes: *MC*= main crack; *ICN*= network of medium width cracks; *MCN*= network of minor width cracks; *BF*= brittle failure; *PO*= pull off of fibers; *FR*= fiber rupture; *DB*= progressive de-bonding of the CFRP reinforcement.

In the horizontally retrofitted walls with three strips failure started as the bottom strip began to delaminate in important form, generally during the first 14mm cycle, producing a decrease of stiffness and strength of the wall compared to the previous displacement level cycle. As the test went on to the second 14mm cycle the strip continued delaminating and the strength of the walls decreased by more than 50%. After unloading, more than 80% of the strip had delaminated, the stiffness of the wall was very small, and the masonry below the second strip was highly damaged retrofitted with the walls of the bricks buckled outwards. There was no sudden loss of strength. The horizontally retrofitted wall with one strip (H-URM-1x300-1) showed a similar de-bonding mechanism, but in this case failure included a 100% of the CFRP reinforcement strips (Figure 6), which produced a sudden loss of strength. On other hand, the horizontal retrofitted wall with anchorage detail, the total delamination was limited by the anchorage details, reached major displacement capacity. The horizontally repaired walls reached a maximum strength slightly larger than the maximum strength of the corresponding retrofitted walls.

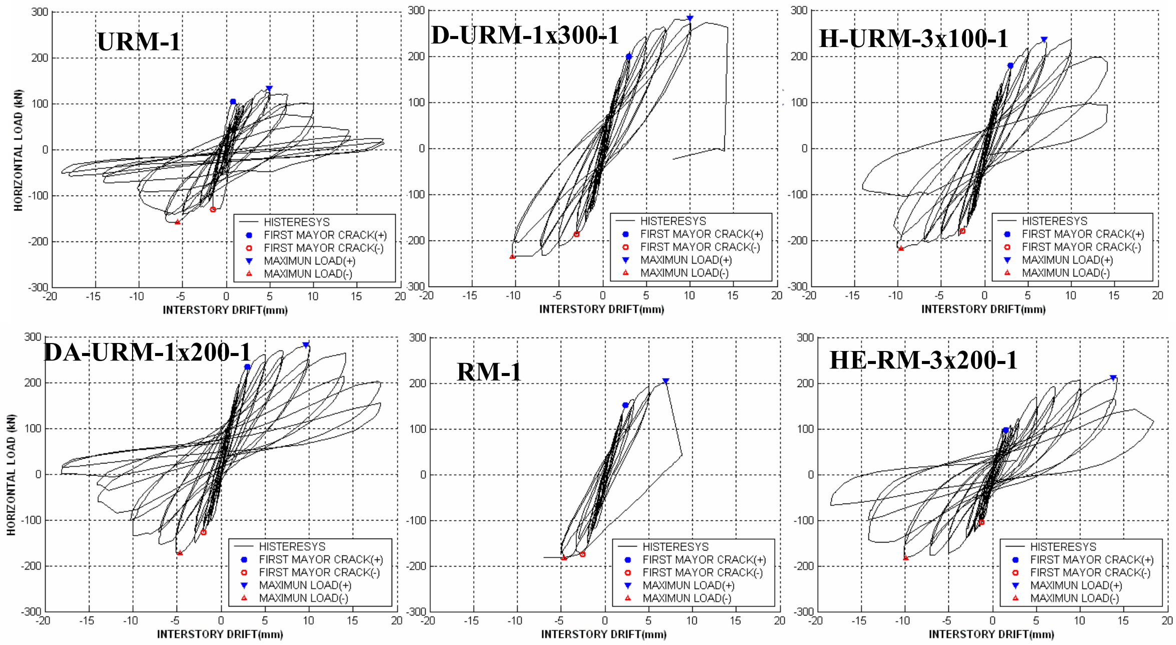

Figure 4. Hysteretic response of walls with different retrofitted configuration.

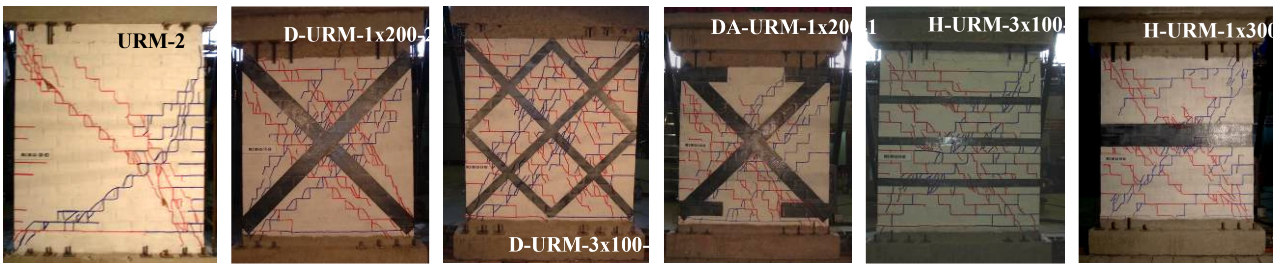

Figure 5. Cracking pattern of walls with different retrofitted configuration at displacement of 7mm

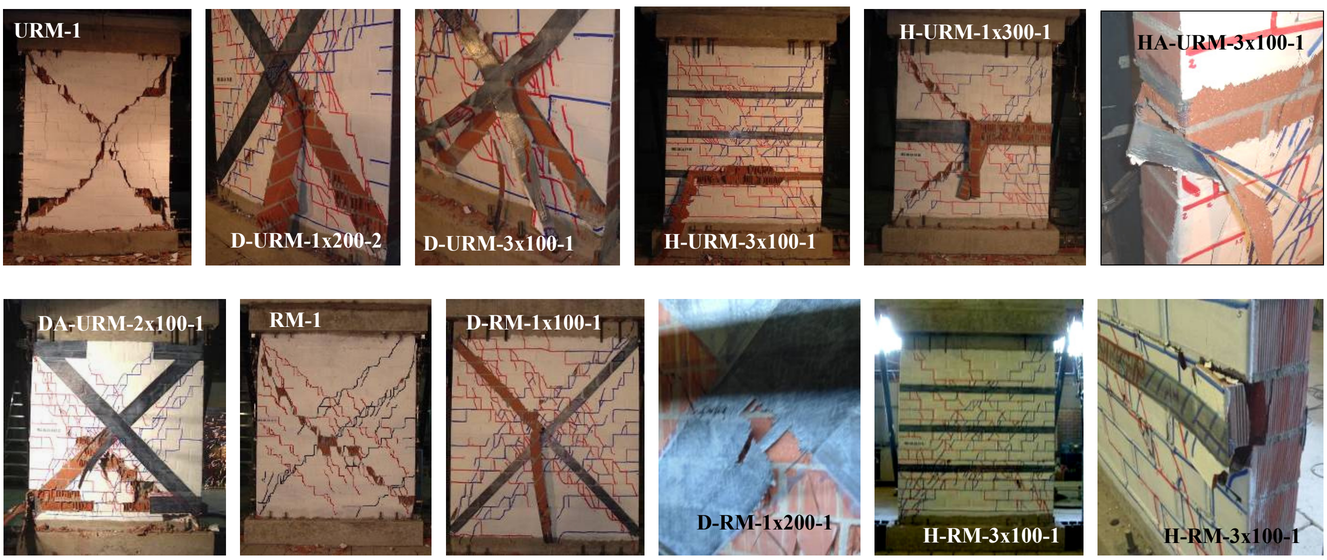

Figure 6. Failure modes of walls with different retrofitted configuration.

6. STRENGTH MODEL

A simple equation is proposed to estimate the strength of the retrofitted walls (V_{max}). From the horizontal free body equilibrium shown in Figure 7 the strength of a retrofitted wall can be written as is in Eqn. 6.1, where V_m is the shear in the masonry (assumed equal to the maximum strength of the non-retrofitted walls); T_f is the force in the fabric reinforcement, calculated as the product between the total width of FRP reinforcement and the maximum bond strength per unit width of the FRP strips (0.24kN/mm); θ is the angle between the FRP reinforcement and the courses; and α is a coefficient of efficiency. Previous investigations have shown that the efficiency of horizontal reinforcement is inversely proportional to the amount of the shear reinforcement, proposing a value of 0.55 (Lüders and Hidalgo 1986) for horizontally retrofitted walls, while α is assumed equal to 1.0 for diagonally retrofitted walls.

$$V_{max} = V_m + \alpha \cdot T_f \cdot \cos(\theta) \quad (6.1)$$

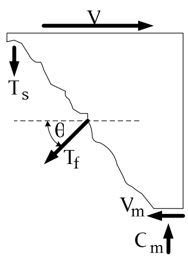

Figure 7. Free body horizontal equilibrium of a wall subject to shear

In Table 3 are shown the average measured strengths and the strengths calculated using Eqn.6.1. It can be seen that Eqn. 6.1 is conservative in estimating the strength of the diagonally retrofitted walls, but is slightly non conservative in the estimation of the strength of the horizontally retrofitted walls. The latter may be because the efficiency coefficient was originally obtained for steel reinforcement, not for FRP strips.

Table 3. Comparison of calculated and average measured loads

Specimen ID	Maximum strength			Specimen ID	Maximum strength		
	Meas. Str. V_m^M (kN)	**Calc. Str. V_m^C (kN)**	$\frac{V_m^M}{V_m^C}$		**Meas. Str. V_m^M (kN)**	**Calc. Str. V_m^C (kN)**	$\frac{V_m^M}{V_m^C}$
D-URM-1x300-1	259.4	241.8	1.07	H-URM-3x150-1	223.7	259.4	0.86
D-URM-1x300-2	251.0	241.8	1.04	H-URM-3x150-2	226.7	259.4	0.87
D-URM-1x200-1	229.6	208.1	1.10	H-URM-3x100-1	209.9	219.8	0.95
D-URM-1x200-2	213.8	208.1	1.03	H-URM-3x100-2	220.1	219.8	1.00
D-URM-3x100-1	258.8	241.8	1.07	H-URM-1x300-1	216.7	219.8	0.99
DR-URM-1x200-1	237.2	241.8	1.14	HR-URM-3x100-1	226.8	219.8	1.03
DA-URM-1x200-1	226.7	241.8	1.09	HA-URM-3x100-1	239.1	219.8	1.09
D-RM-1x100-1	239.0	226.8	1.05	H-RM-3x100-1	217.7	242.1	0.90
DE-RM-1x200-1	242.0	226.8	1.07	HE-RM-3x100-1	197.4	242.1	0.82
D-RM-1x200-1	259.6	260.6	1.00	HR-RM-3x100-1	243.7	242.1	1.01
DR-RM-1x200-1	258.0	260.6	0.99				
Average	-	-	**1.06**	**Average**	-	-	**0.95**
Standard deviation	-	-	**0.04**	**Standard deviation**	-	-	**0.09**
				Total Average	-	-	**1.01**
				Total Standard deviation	-	-	**0.09**

7. CONCLUSIONS

The contribution of two different configurations of CFRP reinforcement to the in-plane shear response of hollow clay brick walls was experimentally studied. The main conclusions are summarized as follows:

1. Externally bonded CFRP strips on URM walls increase the displacement and load at which the first major crack occurs.
2. Externally bonded CFRP strips on URM and RM walls increase the shear strength and the maximum displacement before failure.
3. The anchorage details were very effective in improving the deformation capacity and failure mode of the walls.
4. The walls with CFPR strips showed several spread cracks, with small thickness.
5. The walls with more parallel strips in each direction had a less brittle failure mode.
6. Previously damaged walls repaired with external CFRP strips can reach the same strength as walls with the same amount of CFRP and no initial damage.
7. A simple model to calculate the maximum strength of masonry walls with CFRP was proposed. The model is reasonably accurate.

9. ACKNOWLEDGMENTS
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