Textile Reinforced Mortars (TRM) for repairing and retrofitting masonry walls subjected to in-plane cyclic loads. An experimental approach

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ABSTRACT

Masonry walls exhibit low tensile strength and high material heterogeneity, which makes them especially vulnerable against cyclic loading conditions, such as those typical in earthquakes. This paper presents the experimental results obtained from tests on three masonry walls reinforced with textile reinforced mortar (TRM) materials subjected to in-plane cyclic loading. These full-scale masonry walls were tested in the LARGE laboratory at the University of Alicante (Alicante, Spain). The walls had been built using a traditional construction technique, with solid clay bricks layered with lime mortar. One specimen was tested and damaged by in-plane cyclic loading and was subsequently strengthened by a vertical layer of TRM with an overlapping of 200 mm. It was then tested again until failure in a second test. In addition, another undamaged specimen had been previously reinforced with the same TRM technique and tested until failure, thus providing a third test. A network of sensors and digital image correlation systems were used to monitor displacements and crack patterns. The comparison between these experimental results made it possible to assess the effectiveness of TRM in restoring the structural integrity of damaged masonry walls and almost doubling their load-bearing capacity under cyclic loads. Conclusions obtained here provide valuable information to the scientific community, architects and structural engineers about the strengthening and repair of severely damaged masonry walls.

Keywords: Cyclic loads; Masonry walls; Textile reinforced mortar (TRM); Strengthening of walls; Digital image correlation (DIC).

1. Introduction.

Strengthening and repairing of masonry structures have attracted the attention of the scientific community because numerous earthquakes have highlighted the vulnerability of masonry structures subjected to seismic actions. This vulnerability results in substantial damages, even failures, due to out-of-plane mechanisms [1–4] but also due to in-plane mechanisms such as diagonal cracking, sliding and tow crushing [5]. Although some stringent seismic standards and codes are now available, many heritage structures were designed without any regard to the seismic requirements. In addition, the heterogeneity of masonry as a construction material makes it very difficult to predict its ultimate bearing capacity subjected to seismic actions and cyclic loading conditions. These reasons make it necessary to deepen our knowledge of repairing and retrofitting masonry structures.

To overcome these problems, the most commonly used method is the application of bonded composite materials, also known as FRP (Fiber Reinforced Polymer) [6–8]: a high-performance fiber mesh (usually carbon, glass or aramid) is embedded in an epoxy resin matrix and then installed upon the structural element that requires strengthening. This technique has a number of advantages: light weight with a high mechanical performance, high strength, high elastic modulus, low mass, high ductility and a negligible impact on the structural dynamic properties, in addition to the mechanical capacity improvement. In addition, FRP has a high ability to impregnate porous media such as concrete, timber or masonry [4]. FRP materials seemed to offer an adequate solution for the repair of masonry structures and were first used to strengthen old masonry buildings in seismic areas suffering from structural deficiencies when subjected to horizontal loads [5].

However, FRP-strengthening also has a number of disadvantages that make its application on masonry material not entirely desirable: (i) poor behavior at high temperatures, (ii) problematic application under wet conditions, (iii) strong environmental impact (e.g., non-removability), (iv) potential health risks for workers in charge of their application, (v) condensation problems due to their lack of water vapor permeability and (vi) delamination failures [5]. The latter consists of a detachment of the FRP layer, usually accompanied by the removal of part of the support material [9] and it entails an undesirable situation

because it dramatically affects the mechanical performance of the FRP-strengthened member. Stratford et al. [10] presented some results of in-plane cyclic load tests in masonry walls, in which the FRP-strengthening altered the crack patterns of masonry walls, yet showed some delamination. Because of the said aspects, FRP is arguably no longer considered as the best strengthening solution for its application to architectural heritage and masonry buildings. To solve these drawbacks, a different type of composite was developed. TRM (Textile Reinforced Mortar), also known as FRCM (Fiber Reinforced Cementitious Matrix), is able to overcome most of the limitations associated with FRP. This inorganic solution comprises a fiber mesh (glass, carbon, aramid, steel or basalt) and a cement mortar with different additives for higher ductility. The main advantage of TRM is its better compatibility with masonry, which reduces the risk of the aforementioned delamination problem and also deals with the permeability issues, among others. In addition, TRM allows the masonry to breathe while at the same time ensuring good bonding. Additionally, TRM-strengthening also improves the masonry structure's mechanical capacity and ductility [11–14]. In this aspect, the mechanical properties of the substrates play a crucial role, since cement-based composites are often applied to weak or weathered masonry. Given these advantages, it is not surprising that TRM has gained considerable popularity as a retrofitting solution of masonry structures.

A considerable deal of research has been developed to identify TRM's relevant mechanical properties for strengthening masonry or concrete elements. Three relevant areas of research might be identified: mechanical performance of TRM coupons under direct tensile loading [15–18], testing of TRM-strengthened masonry panels under shear behavior [19–22], testing of TRM-strengthened masonry arches and vault structures [23,24] and experimental research on out-of-plane TRM retrofitting of masonry walls [25–28].

The former area of research aims at characterizing the basic behavior of the TRM material on a mechanical level, isolated from the strengthened member, and identifying the strain ranges associated with cracking of the mortar and transfer mechanism between mortar and fiber mesh. The second area of research showcases the effectiveness of TRM to increase the load-bearing capacity of small specimens subjected to diagonal compression. Their effectiveness depends on their application upon one or two sides, as well as the type of

mortar and fiber used. In the case of [21], reinforced panels exhibited greater shear strength and ductility, with an amplifying factor around 2.5. Finally, the third area of research focused on masonry arches and vault structures and has shown that TRM composites are capable of restoring the integrity of damaged vault structures and increasing their load-bearing capacity. In this aspect, previous research reported in [24] describes a repair technique that uses a radial strengthening configuration installed on severely damaged masonry cross vaults submitted to vertical monotonic or cyclical soil settlements. This procedure has allowed (i) to restore the initial elastic stiffness of the damaged vaults, (ii) to increase by a factor of two the vaults' elastic regime stage and ultimate displacements, and (iii) to increase the load-bearing capacity of the masonry cross vaults, almost doubling the peak reaction forces.

In all of these previous studies, TRM has been demonstrated to be effective as a strengthening material for masonry structures that could effectively prevent diagonal cracks and shear and bending failures. In addition, the problems associated with FRP-strengthening of masonry elements could be avoided. However, due to their prohibitive cost, few studies have analyzed the performance of TRM on full-scale structures [29–32].

This paper describes the experimental tests performed on full-scale masonry walls to evaluate the effectiveness of a TRM reinforcement in enhancing the seismic performance of masonry walls. There are some previous studies regarding the in-plane shear behavior of masonry walls, in which different dimensions and vertical preloads are combined [29,33,34]. However, there is hardly any information about the use of TRM to repair and test pre-damaged masonry walls. Therefore, the main novelty of this work is that TRM is not only used as reinforcement of undamaged walls, but additionally tested as a repairing technique of severely cracked masonry. In this piece of research we present and discuss the results of three full-scale tests, masonry walls, 3×2 m in size, subjected to in-plane cyclic loading. Two such specimens had been built with solid ceramic brick and lime mortar. One of them was tested until failure (unreinforced wall, UW, first test) and was subsequently repaired by means of crack injection, then reinforced with glass fiber-TRM (Damaged and Repaired Wall, DRW), hence the second tested wall. The third test corresponds to an undamaged glass fiber-TRM-reinforced wall (Non-Damaged Reinforced Wall, NDRW). Their

performance is analyzed and compared in terms of hysteretic behavior, strength and ductility gain, stiffness degradation, energy dissipation capacity and cracking patterns. The experimental results show the effectiveness of the TRM layer in restoring the masonry's integrity and increasing by almost a factor of two the load-bearing capacity of severely damaged masonry walls under cyclic loads.

2. Experimental test.

2.1. Masonry wall geometry and experimental test set-up.

In this research two brick masonry walls were fabricated and tested under in-plane cyclic loads to assess their behavior in the event of earthquakes. One specimen was tested until failure without any kind of strengthening (unreinforced wall, UW). The resulting wall was then repaired and reinforced with TRM and re-tested again until failure (damaged, repaired and reinforced wall, DRW). A second undamaged and untested specimen was reinforced with TRM to evaluate its reinforcement capacity (non-damaged, reinforced wall, NDRW), thus making it possible to compare the effectiveness of the TRM by comparing the results of the three tests.

Fig. 1. Experimental set-up: (a) general view; (b) connection to loading cell and steel plate for pushing forces; (c) steel plate for pulling forces.

Fig. 1(a) shows a general view of the NDRW specimen before the test started, with all the auxiliary elements to apply the vertical preload and the lateral in-plane cyclic force. Fig. 1(b)-(c) show some details of the experimental set-up. Fig. 1(b) shows the steel joint to connect the loading cell to the steel plate that transmits

the horizontal force. In order to apply cyclic loads two steel plates (40 mm thick) were connected by steel bars (prestressed with nuts and washer). Fig. 1(b) shows the plate for pushing forces, and Fig. 1(c) corresponds to the plate for pulling forces.

Fig. 2. Experimental set-up (front and lateral views).

Fig. 2 shows a sketch of the experimental set-up. The dimensions of the walls were $3 \times 2 \times 0.23$ m (length $\times$ height $\times$ thickness). The walls were subjected to horizontal cyclic loads applied at the upper right corner (point D) using a hydraulic loading cell with a maximum capacity of 750 kN. A vertical load (150 kN) was applied and monitored during the test using four Dywidag 32 bars, in order to simulate the weight of a three stories high building. The relative movement between the swaying wall and the upper loading devices was possible thanks to a number of steel beams and cylinders that were used to allow these displacements. Each specimen was embedded on a steel base with rigid plates along its perimeter. This steel base had 150 mm holes to anchor it to the one-meter-deep concrete reaction slab. This experimental setup had been previously checked in tests of masonry walls with windows also reinforced with TRM [35].

The rate of wall's in-plane swaying was fixed during the whole test and the swaying amplitude was progressively increased. Each push-pull cycle was repeated twice with a constant value of amplitude before proceeding to a new stage with greater amplitude. The allowable displacements of the unreinforced wall were expected to be small; hence the cycles' maximum displacement amplitudes were increased in 7 mm

steps. Thus, the target values of the displacement of the loading cell were ± 7 mm, ± 14 mm, ± 21 mm, ± 28 mm and ± 35 mm. However, in the case of both reinforced walls, the cycles' amplitudes were increased in steps of 10 mm, which led to target values of ± 10 mm, ± 20 mm, ± 30 mm, ± 40 mm, ± 50 mm, ± 60 mm and ± 70 mm in the NDRW and DRW tests.

2.2. Mechanical characterization of the materials.

Masonry was built with clay solid clay bricks and lime mortar. This section describes the laboratory tests performed to characterize the mechanical properties of the materials used to build the walls, including the clay bricks tested under compression and a three-point bending test.

Each brick's dimensions were 230×110×55 mm, with a compressive strength of 15 MPa and a density of 1550 kg/m³ (according to the supplier). Lime mortar contained natural pozzolan and had a compressive strength of at least 7.5 MPa (according to the supplier, Puma Group [36]).

Nevertheless, tests were made to check these mechanical properties. In the case of the clay bricks, a total of 4 bending tests and 8 compression tests were carried out. The strength was measured according to the standard EN 1015-11:2000/A1:2007 [37] obtaining an average value of 17.2 MPa at 28 days (with a coefficient of variation of 15%). Bending and compressive strengths of the lime mortar used to lay the bricks were assessed at different ages, in accordance with standard EN 196-1:2005 [38]. A total of 20 bending and 40 compression tests were carried out on lime mortar prismatic specimens. Its compressive strength was equal to or greater than 7.5 MPa at 28 days (which agreed with the supplier's value), but it was also experimentally obtained at 60 days, resulting in 10.4 MPa with a coefficient of variation of 12%. These properties of clay bricks and lime mortar are shown in Tables 1 and 2, with the coefficients of variation reported between parentheses.

Type of test	No. of specimens	Dimensions (mm)	Elastic modulus (MPa)	Strength (MPa)
Compression	8	230×110×55	5832 (17%)	17.2 (15 %)
Three-point bending	4	230×110×55	-	3.8 (11 %)

Table 1. Mechanical properties of clay bricks. Figures between parentheses are coefficients of variation in %.

Type of mortar	Age (days)	Compressive strength (MPa)	Tensile strength (MPa)
Lime	60	10.4 (12 %)	1.9 (17 %)

Table 2. Mechanical properties of lime mortar. Figures between parentheses are coefficients of variation in %.

Further experimental tests were performed to characterize the mechanical properties of the masonry elements. A total of 12 specimens (six for compression tests and six for shear tests) were built for this purpose. The shear behavior of the masonry specimens was measured through diagonal tension tests in accordance with standard ASTM E519/E519M-15 [39] which has also been used in the experimental research reported in [21]. The size of each masonry sample was 710×710×230 mm (see Fig. 3(a)). The compressive behavior of the masonry specimens was measured through compression tests in accordance with standard [40]. The size of each masonry sample was 710×540×230 mm (see Fig. 3(b)). Each specimen was tested in a steel moment resisting frame with a hydraulic actuator of 2500 KN. The test velocity was set at 0.01 MPa/s and the elongation was measured with four LVDTs in both faces of the specimens. A summary of the results can be seen in Table 3.

Fig. 3. General view of the specimens for (a) shear and (b) compression tests.

Type of test	Age (days)	Compressive strength (MPa)	Shear strength (MPa)	E (MPa)	G (MPa)
Compression	60	4.9 (5.2 %)	-	2942 (16.9 %)	-
Shear	60	-	0.72 (16.5 %)	-	1305 (29.0 %)

Table 3. Mechanical properties of the masonry specimens subjected to compression and shear tests. Figures between parentheses are coefficients of variation in %.

Finally, the mechanical performance of the textile reinforced mortar (TRM) was tested in accordance with Annex A of standard AC434 [41]. Six coupons were fabricated with dimensions 400×100×9 mm (length×width×thickness) and tested at an age of 60 days. These TRM specimens were made with a glass fiber mesh (type G220, see main properties in Table 4) embedded within a high ductility lime fiber reinforced mortar (Planitop HDM Restauro, with a compressive strength of at least 15 MPa at 28 days according to the supplier). Both fiber mesh and fiber reinforced mortar were supplied by Mapei [42]. The TRM lime mortar was poured into 40x40x160 mm³ steel formwork for three-point bending and compression tests, as it has been explained in [32]. A total of 6 three-point bending and 12 compression tests were carried out on TRM lime mortar prismatic specimens. Its compressive strength was equal to 18.8 MPa at 60 days with a coefficient of variation of 13.8%. Its tensile strength was equal to 4.3 MPa at 60 days with a coefficient of variation of 11.4%. A summary of the results can be seen in Table 5.

Product Ref.	Type of fiber	Mesh pitch (mm)	Weight (g/m ²)	Tensile strength (MPa)	Elongation at failure (%)	Elastic modulus (GPa)	Load-resistant area (mm ² /m)
G220	Glass	25×25	225	1276	1.8	72	35.27

Table 4. Fiber mesh type properties (as given by the supplier [42]).

Type of mortar	Age (days)	Compressive strength (MPa)	Tensile strength (MPa)
TRM high ductility lime mortar	60	18.8 (13.8 %)	4.3 (11.4 %)

Table 5. Mechanical properties of TRM high ductility lime mortar. Figures between parentheses are coefficients of variation in %.

Each TRM coupon was bolted to steel clamps installed in an electro-mechanical press with a loading cell of 50 kN. The test velocity was set at 0.2 mm/min and the elongation was measured with one LVDT and also through digital image correlation (DIC). The experimental configuration is shown in Fig. 4, which also includes the idealized tensile stress-strain relationship defined in AC434 standard and the curves obtained

experimentally for the six tested coupons. As can be seen in Fig. 4(c), the idealized response from standard is bi-linear, with a transition point (T) that corresponds to the coupon's cracking. On the other hand, Fig. 4(d) includes the experimental curves and clearly shows that the second branch of the diagram presents an almost horizontal slope in most cases. This behavior is directly associated with the type of fibers and the load-resistant area, as has been found in some previous research [43]. Table 6 includes the main results obtained (average value of the six coupons tested plus coefficients of variation, between parentheses), expressed in terms of load-resistant fiber area per unit of width (given in Table 4). The calculation of the average values indicated in the table does not take into account the results that have been considered outliers (in particular, f_{tu} from coupon C1 and E_f^* from coupon C2).

Fig. 4. TRM tensile tests: (a) measurements with LVDT systems; (b) measurements with digital image correlation; (c) idealized stress vs. strain behavior as shown in AC434 standard [41]; (d) test results of six TRM coupons.

Type of test	Age (days)	Uncracked			Cracked		
		f_{ft} (MPa)	ϵ_{ft}	E_f^* (GPa)	f_{tu} (MPa)	ϵ_{fu}	E_f (GPa)
Tensile	60	611.2 (10.4%)	0.0008 (15.4 %)	1052.4 (13.7 %)	644.8 (6.6 %)	0.0123 (13.3%)	33.67 (28.1 %)

Table 6. Mechanical properties of TRM coupons. Figures between parentheses are coefficients of variation in %.

2.3. Textile Reinforced Mortar (TRM) strengthening.

In order enhance and assess the wall's behavior subject to cyclic loading a TRM layer was applied to both 3×2 m² surfaces. The reinforcement was made by means of vertical bands with a single layer of mesh and 200 mm overlaps, embedded within a mortar layer with a thickness of 10-15 mm. Fig. 5 shows a sketch of

the reinforcements applied onto the damaged, repaired and reinforced wall (DRW specimen) and onto the non-damaged and reinforced wall (NDRW specimen). Fig. 5 also includes the crack patterns in the original unreinforced wall (UW specimen) after testing it until failure.

Fig. 5. Crack patterns (UW specimen) and TRM strengthening (DRW and NDRW specimens).

The TRM application technique was done in three phases (Fig. 6). In the first place, the masonry assemblage of the UW specimen after testing was repointed by means of a super-fluid injection slurry applied in the cracks and joints, as shown in Fig. 6(a). Next, a 5-7 mm thick layer of high ductility lime mortar was applied onto the wall's surface (Fig. 6(b)) and the fiber glass mesh was then placed across the fresh lime mortar's surface (Fig. 6(c)). Both wall's surfaces were thus reinforced with 900 mm wide, vertically placed mesh strips with overlaps of 200 mm. The glass fiber mesh strips were not anchored to the steel base. Finally, another 5-7 mm thick mortar layer was extended thus embedding the glass fiber mesh inside the TRM composite (Fig. 6(d)).

Fig. 6. Repairing of UW specimen and TRM application.

2.4. Monitoring strategy.

Eight LVDTs were attached to the structure on one side of the wall. Six of them were used to register crack openings, and two of them were used to measure horizontal displacements, in the base (C) and the top (D), as shown in Fig. 2. On the other hand, the opposite side was monitored with a digital image correlation (DIC) system. DIC is a non-contact optical technique used for measuring strain and displacement which has been previously and successfully used in experimental research of masonry structures [21]. The position of each object point in the image can be identified by applying a correlation algorithm using a stochastic intensity pattern (speckle) painted across the specimen's surface. Using this technique, deflections, axial deformations, local strains and crack patterns can be determined. The resolution of the DIC depends on the pixel density of the camera, the size of the area of interest and the quality of the speckle. For the current DIC measurements, a 16 MP camera and the GOM Correlate software were used. Only one camera was used because out-of-plane displacements were considered negligible compared to the in-plane components. A crucial feature of DIC measurements is the creation of the stochastic pattern. Fig. 7 shows the stochastic pattern used in the UV (Fig. 7(a)) and DRW specimens (Fig. 7(b)). In order to optimize the DIC analysis, contrast in the images was improved by painting the walls in white before applying a black speckle [21,44].

Fig. 7. Stochastic pattern in UV (a) and DRW (b) specimens.

3. Results and discussion.

The main objective of this work is to contrast the efficiency of a TRM layer as reinforcement of an intact, uncracked masonry wall with its performance as a retrofit measure of a repaired masonry wall previously damaged due to the effect of in-plane shear forces. A summary of the main results of the three tested situations can be found in Table 7. The particular presentation and discussion of each parameter is addressed in the following subsections.

Table 7. Summary of the main results. Brackets show the values in [pushing, pulling] directions.

Properties	UW	DRW	NDRW
Displacements in elastic behavior (mm)	[1.5, -1]	[2, -1.7]	[2, -1.5]
Displacements at failure (mm)	[11, -12]	[16, -15]	[20, -7.5]
Ultimate displacement during push-over test (mm)	[11, < -20]	[55, -15]	[> 50, -7.5]
Maximum loads (kN)	[145, -136]	[320, -359]	[415, -264]
Energy dissipation CED_{TRM}/CED_{UW} at ultimate displacement	-	20.2	22.6
Residual stiffness $K_{TRM}/K_{UW,0}$ at [10, -10] mm	-	[0.95, 1.15]	[0.8, 0.75]
Stiffness loss (%) at displacement level	[56, 76] % at [10.4, -16.9] mm	[72, 71] % at [28, -17.4] mm	[74, 68] % at [25, -10] mm
Ductility $\mu = \delta_u/\delta_y$	[8, 12.5]	[27.5, 11.2]	[26, 5.7]
Maximum crack width (mm) and position	12.5 mm in LVDT5 for pulling direction	46 mm in LVDT6 for pushing direction	20 mm in LVDT5 for pushing direction

3.1. Hysteretic response.

Specimens were subjected to push-pull cycles by means of a horizontal hydraulic actuator on the in-plane longitudinal upper edge of the walls. The cycles were generated by imposing positive displacement (pushing) and negative displacements (pulling) on the specimens, as shown in Fig. 2 and Fig. 5.

Fig. 8 shows the displacement measured in the wall, representing the walls' swaying or drift in the vertical axis. The drift is obtained as the difference between the lateral movements of the upper and lower edges of the wall, which were measured, respectively, with sensors LVDT8 and LVDT7 shown in Fig. 2. These were the real displacements sustained by the wall, which were different from the real displacements applied by the actuator. The differences are due to small gaps between the load application elements and walls. As

commented earlier in this paper, the application of the pulling force was possible thanks to six prestressed steel bars, 20 mm in diameter, which connected both upper corners of the wall. It was observed that, for high pulling loads, these bars exhibited an elongation that was another source of discrepancy between the applied real displacements and the relative displacements really sustained by the wall. Because of this, although the behavior at the beginning of the test was quite symmetrical in both directions, the walls stopped behaving symmetrically as the test progressed on to greater loads. This may be attributed to the fact that, after the first signs of damage appeared, the wall's stiffness along one direction was different from the opposite one.

Fig. 8. Horizontal displacements measured in the upper part of the tested specimens.

The tests were displacement-controlled. The horizontal load applied by the hydraulic actuator was recorded by means of a load cell. Fig. 9 depicts the load-displacement curves obtained in the tested specimens with the horizontal force (shear) in the vertical axis and the recorded drift from Fig. 8 in the horizontal axis. The hysteretic behavior of the walls is thus illustrated by Fig. 9. A symmetrical behavior of both directions can be easily observed in the initial cycles. As the tests progressed, however, symmetry was lost. The final cycles are completely asymmetric because the cracking failure that compromised the overall stability occurred in a single direction, although cracks had appeared and evolved in both directions earlier on.

Fig. 9. Hysteretic load-displacement curves for tested specimens.

The drift amplitudes of the UW specimen were distinctly smaller, with a final drift of +11 and -12 mm for shear forces of +141 and -136 kN, respectively. In this case, the failure that compromised the overall stability occurred along the pulling direction (negative) and the test was stopped. The DRW specimen showed a completely different behavior, even modifying the failure pattern. In this case, failure took place along the pushing direction, despite the fact that previously this wall had failed along the pulling direction. In this case, TRM-strengthening not only restored the original load-bearing capacity of the UW specimen, but also increased it by 129%. In addition, the hysteretic curve of the DRW specimen showed how the TRM had improved the displacement capacity of the masonry wall under push-pull loads: the final drift amplitudes were +23 and -15 mm. A greater increase might have been expected for the pulling direction, but once the collapse along the pushing direction occurred, it was decided to stop the test.

Regarding the NDRW specimen, no differences were initially observed with respect to the DRW specimen. Along the pulling direction, the DRW specimen surprisingly exhibited greater load-carrying capacity. Nonetheless, the NDRW specimen showed a greater load-carrying capacity along the pushing direction compared to the DRW specimen (28% higher) and also greater displacement capacity. The NDRW was able to sustain a drift of +50 mm, although Fig. 9 only represents values up to +25 mm. As in the case of the DRW specimen, a greater drift increase might have been expected along the pulling direction, but it was decided to stop the test once the failure occurred along the pushing direction.

The ultimate load-carrying capacity of the specimens with TRM (both DRW and NDRW) was significantly greater than that of the UW one, as are the corresponding ultimate displacements. Once the failure occurred, a certain degree of ductility was observed in the specimens with TRM that had not been observed in the UW specimen.

3.2. Envelopes and mechanical performance.

The complete envelope curves, for both pushing and pulling directions, are given in Fig. 10. Envelope curves are obtained connecting peak loads and the associated drift displacements of the hysteretic curves. A reasonably symmetrical behavior is apparent in these envelopes. For all specimens, this symmetry can be observed up to a certain level of drift, beyond which the degradation of the specimen provokes the general collapse.

Fig. 10. Envelope curves for tested specimens.

Different strength degradation trends can be detected in the tested specimens. The UW specimen showed a reduction in the peak load corresponding to a pulling drift over -11 mm. Beyond this point, the UW specimen was not capable of assuming greater loads or greater displacements: the wall behaved as two independent structures because a tilted crack evolved across the wall's surface and both parts were detached - the upper mid wall continued moving while the lower part remained still, hence widening the crack. This failure compromised the overall stability of the test.

Nevertheless, specimens strengthened with TRM (DRW and NDRW) were able to show a residual behavior, with a progressive reduction of the peak loads as drifting increased under pushing forces. In the case of the DRW specimen, the critical drift value was around +16 mm and, in the case of the NDRW specimen, it was around +20 mm. Beyond these drift values, TRM-strengthened specimens were not capable of sustaining greater loads though they were indeed capable of exhibiting greater displacements, up to +50 mm for both specimens. Along the pulling direction, negligible strength degradation was observed because failure took place along the opposite direction, although stiffness reductions were observed as the drift increased.

TRM-strengthened specimens showed very similar behavior up to pushing drifts of around +12 mm. Beyond this point, clear differences were observed since there was a significant degradation in the DRW specimen. The fact that the DRW had been previously damaged and repaired did not seem to have an impact on its response in terms of hysteretic behavior up to a drift of +12 mm. It should be remembered that the DRW specimen had previously been damaged along the pushing direction, so the brick's assemblage repointing, the injection of the cracks and the TRM-strengthening seemed to restore the original load-bearing capacity and increase it. Moreover, the DRW responded with even more stiffness than NDRW along the pulling direction.

These experimental results may prove the effectiveness of the TRM-strengthening layer on the mechanical performance of undamaged and damaged masonry walls, which could be assessed in terms of both strength and displacement capacity. All specimens presented linear behavior during the first load cycles. Besides, no damage occurred during this stage as confirmed by visual inspection. Specimens with TRM (DRW and NDRW) showed a linear load/drift ratio in the interval [-1.5, +2] mm, approximately. Beyond this range, behavior continues to be linear but loses stiffness. For drifting values over 16 mm, a non-linear behavior showcased a certain degree of ductility. On the other hand, UW show a similar trend with an initial linear behavior in the interval [-1, +1.5] mm, followed by a linear behavior with less stiffness until failure at around the interval [-11, +10] mm.

The behavior of DRW and NDRW under pushing loads showed quite a large improvement of the peak load, which increased from 145 kN in the UW to 320 kN and 415 kN in the DRW and NDRW specimens, respectively. Under pulling forces, an increase was also observed but no definite conclusion can be obtained, since failure along that direction did not take place, so greater load-bearing capacity values might have been reached. Both reinforced walls, DRW and NDRW, showed a strengthening behavior with a certain residual ductility after crack initiation. In order to discuss the response of the TRM layer, Fig. 11 shows crack development at different drift levels along the test of DRW and NDRW. The first image of each test, Figs. 11 (a) and (d) are close to the maximum strength of each test, in which the main diagonal crack is fully developed. At this stage, TRM performed correctly and TRM debonding was not observed during the test. Further lateral displacements initiated the stress decay, shown above in Fig. 9 and Fig. 10. The loss of effectiveness of the TRM layer can be related with two different phenomena. First, Fig. 11(e) shows a wide mortar spalling of the TRM. In this case, it appeared while applying loads in the pulling direction, approximately at a drift of -9 mm, in which a change in the behavior may be seen in hysteresis curves of Fig. 10. A similar effect could be observed when a second mortar spalling occurred, Fig. 11(f). The second mechanism that affected the mechanical strength of the TRM was the rupture of the fibers in the glass fiber mesh. TRM layers were capable of maintaining the mechanical response of the walls for severe crack openings, which were several centimeters wide. However, further widening of the cracks provoked rupture of the fibers, as illustrated in Fig. 12. These failures were located at the knots of the mesh -the points at which orthogonal fibers cross and are bonded together. Therefore, the progressive failure of fibers may have been responsible for the slow softening of the mechanical behavior of reinforced walls.

Fig. 11. Crack development and TRM spalling at different drift levels of the DRW (a) 15.7 mm, (b) 31.3 mm, (c) 52.2 mm, and the NDRW (a) 15 mm, (b) -8 mm, (c) 33 mm.

Fig. 12. Fiber rupture in the TRM glass fiber mesh.

3.3. Stiffness and strength degradation.

The stiffness of masonry walls is analyzed in terms of the secant stiffness, defined as the ratio of 70% of the peak load in a cycle to the associated drift displacement [5,29,45,46]. The hysteresis loops show that the secant stiffness decreases with an increase in the applied displacement. In the literature, some authors [47,48] are more interested in the secant stiffness at each cycle, normalized with respect to the secant stiffness at ultimate load. However, in this paper the stiffness corresponding to a certain drift value was normalized with respect the secant stiffness of the original UW specimen, $K_{0,UW}$. Stiffness was determined for both directions of displacement –push and pull–. Fig. 13(a) graphically represents the evolution of the stiffness vs. the drift of each cycle. For a better comparison, Fig. 13(b) represents the stiffness as a normalized residual value (i.e., with respect to the initial stiffness of the unreinforced wall).

Fig. 13. Evolution of the shear stiffness: (a) stiffness degradation; (b) residual stiffness.

As confirmed by visual inspection, no damage occurred during the initial elastic phase and specimens with TRM seemed to present a similar degradation trend, Fig. 13(a). The values at the start of each test (closest drift values to 0), confirmed the effectiveness of TRM in restoring the UW specimen's original stiffness and even increasing it by a factor over two, Fig. 13(b). It is also noteworthy that in the case of the pull direction (negative values of drift), the DRW specimen (previously damaged and repaired) exhibited an even greater initial stiffness than the NDRW specimen (without any prior damage), despite its previous cracked condition along the pushing direction. For positive drift values (push direction), the difference

between the behavior of the stiffness of DRW and NDRW specimens was negligible, which seems to confirm that TRM-strengthening is able to restore the original integrity of a severely damaged masonry wall.

Fig. 14. (a) Cyclic strength degradation and (b) in-cycle strength degradation for different drift values. Positive drifts correspond to push displacements and negative values to pull direction.

Strength degradation is another important parameter of the discussion of experimental results. In accordance with FEMA P440A [49], cyclic and in-cycle degradations should be differentiated. Cyclic strength degradation accounts for the strength loss between cycles made with the same displacement amplitude. On the other hand, in-cycle degradation represents the difference at every cycle between the maximum stress and the minimum stress for the maximum displacement of the iteration. Fig. 14 represents both values assessed for the registered behavior during the cyclic loading of three tests (UW, DRW and NDRW). Therefore, the final push-over phases have not been considered in the current analysis. Fig. 14(a) includes the cyclic strength degradation represented as the ratio between the maximum stress in the second cycle of each amplitude with respect to the corresponding value in the previous cycle. In general, all cycles presented a degradation below 5%, i.e., the strength at the second iteration was at least 95% of that of the first cycle. This damage can be easily observed in the DRW test, Fig. 9(b), in which strength degradation increased rapidly for drift values beyond 15 mm –a decrease between 29% and 66% for cycles with greater amplitudes. In that case, the strength degradation was coupled with an increase in the energy dissipation, as it will be discussed in the next section.

In masonry structures, in-cycle degradation can be initially attributed to the different behavior between bricks and mortar joints. Fig. 14(b) showed small degradation values for all specimens, below 5%, which is arguably a reasonable value for masonry. During the second iteration at each amplitude, the in-cycle strength degradation was slightly smaller. Once again, damage was concentrated in the pushing cycles of the DRW, as shown in the hysteresis curves of Fig. 9(b). Cycles with drift values between 20 and 30 mm showed in-cycle degradations between 10% and 25%. Wider cycles have not been included in Fig. 12(b), but in-cycle strength degradation reached a maximum 32.5% value at a drift of +55 mm. These degradation values may have been related to the aforementioned rupture of fibers in the TRM layer, Fig. 12.

3.4. Energy dissipation capacity.

The dissipated energy during a cycle can be defined as the area within the hysteretic loop in the load–displacement curve (Fig. 9), which may be calculated using the trapezoidal rule. The evolution of the dissipation energy has been represented in Fig. 15(a). A detail of the evolution for smaller drift displacements, up to 12 mm, is shown in Fig. 15(b).

Fig. 15. Energy dissipation capacity in each cycle: full test (a) and drift under 12 mm (b).

As expected, at the end of the test, the DRW and NDRW specimens were capable of dissipating a substantially greater amount of energy than the UW specimen. In addition, the amount of energy increased for wider cycle amplitudes due to the cracking mechanism. All specimens show a similar trend of energy dissipation for drift displacements under 11 mm, although UW presented a more severe final cracking

pattern that threatened the wall's overall stability. Beyond this point, UW was not able to sustain further cyclic loading and test was stopped. However, TRM-strengthened walls kept on sustaining new load cycles because the TRM layers made it possible to dissipate greater amounts of energy. It means that these materials succeed in limiting masonry damages at low displacements thus generating a lesser energy dissipation.

The energy dissipation capacity of the DRW specimen was greater than that of the NDRW one in the drift range between 11 mm and 41 mm. Due to the DRW's previous damage, energy dissipation was provided solely by the TRM layers, which then suffered greater degradation than in the NDRW test. The fact that the later specimen had not gone through any prior damage could explain that the masonry material was able to provide some degree of contribution to the energy dissipation.

Subsequent push-pull cycles dissipated less energy than the first cycle for a given drift value, because of the cracked condition induced by that first cycle. This observation is illustrated in Fig 15(b), especially in the curves' discontinuities corresponding to UW and DRW –NDRW's curve is practically continuous between 6 mm and 11 mm. The most notorious example is illustrated in Fig. 15(a) for a drift value of 22 mm in the DRW specimen, where a large difference of energy dissipation is observed between the first and second cycles (around 1250 kN·mm and 800 kN·mm, respectively). These discontinuities are less noticeable in the NDRW wall, which exhibited a better linear-elastic behavior without degradation.

Fig. 16 represents the relationship between the cumulative energy dissipation (CED) and the stiffness degradation. It is possible to observe a drastically different behavior between the UW test and the TRM-strengthened specimens (DRW and NDRW). The UW exhibited a linear increase of the energy dissipation as more damage was produced. However, the DRW and NDRW registered an exponential growth of the cumulative energy dissipation -NDRW showed a very good fit with $R^2=0.9252$.

Fig. 16. Cumulative energy dissipation (CED) vs. stiffness loss.

TRM-strengthening enhanced the behavior in terms of total energy dissipation and higher ductility, which may prove the TRM as an effective solution to delay or control crack growth, thus improving the behavior of severely damaged masonry walls. As stated in previous sections, this would mean that it might be possible to restore and even improve the original mechanical capacity of masonry.

3.5. Lateral displacement ductility.

As seen in a previous section, the UW test showed a significant stiffness loss at relatively low drift values and this compromised the overall stability of the element. The UW specimen simply did not have enough ductility to maintain the lateral load and the associated increasing displacements.

This section shows the ductility obtained for each specimen. Ductility μ was evaluated according to Eq. (1), where δ_y is the displacement related to the first cracking and δ_u the final lateral displacement reached by each specimen. Following the recommendations given in [50], polygonal curves were traced from the envelope curves shown in Fig. 17 in order to evaluate δ_y . These polygonal curves were drawn by highlighting three main points: (i) first cracking, (ii) maximum strength, and (iii) ultimate state.

$$\mu = \frac{\delta_u}{\delta_y} \quad (1)$$

Fig. 17. Polygonal curves of tested walls.

Table 8 shows the ductility obtained for each specimen. It has to be noted that in the case of specimens with TRM layers, the increase in ductility was over 200% along the pushing direction. There are negligible differences between the DRW and NDRW tests because both showed almost the same strength. In the specific case of the previously damaged wall, this would demonstrate that TRM was able not only to restore the original capacity but to increase it by a factor over three and equaling the capacity of the retrofitted wall without initial damage. Along the pulling direction, it is important to point out that DRW and NDRW would have withstood even greater loads and sustained larger displacements. However, once the failure had already occurred along the pushing direction, it did not seem safe to provoke an opposite failure for pull direction. For this reason, ductility obtained along the latter direction does not indicate a reliable value, which would probably have been much higher (these values are indicated with * in Table 8).

Direction	UW			DRW				NDRW			
	δ_y (mm)	δ_u (mm)	μ	δ_y (mm)	δ_u (mm)	μ	μ_{DRW} / μ_{UW}	δ_y (mm)	δ_u (mm)	μ	μ_{NDRW} / μ_{UW}
Push	1.5	12.0	8.0	2.0	55.0	27.5	3.43	2.0	52.0	26.0	3.25
Pull	-1	-12.5	12.5	-1.7	-19.0*	11.2	-	-1.5	-8.5*	5.7	-

Table 8. Ductility obtained for each tested specimen.

3.6. Crack patterns.

The displacement recordings from all LVDTs installed onto the UW, DRW, and NDRW specimens are depicted in Fig. 18, 19 and 20. LVDT sensors were installed on one side of the wall, perpendicularly to the two diagonals which would almost coincide with the principal directions, right where main cracks were expected. In addition, a digital image correlation (DIC) system was used on the opposite side of the wall in order to (i) monitor the main cracks' widths, (ii) estimate the main strain and (iii) visualize all cracks, regardless of their width. On the other hand, since DIC is fundamentally a non-contact technique, it poses a number of advantages in the monitoring of loaded structural elements. Whole members are photographed, thus allowing for deformation measurement beyond the limited areas covered by other sensors, such as the aforementioned LVDTs. Another important advantage is that DIC allows registering the post-crack behavior, because traditional LVDTs often have to be removed in later stages of the test to avoid being damaged when the ultimate brittle collapse takes place. Furthermore, in the particular case of masonry structures, the distribution of mortar and brick joints might make the behavior unpredictable and a traditional sensor may not yield useful information if failure takes place in a different area than the one initially expected. This eventuality is arguably entirely managed by using a DIC setup, which makes it possible to observe and register the whole specimen in terms of displacement and deformations. It would be possible to detect effects not considered in the design of the experimental test, such as yielding of the steel bars and sliding or rotation of the wall. Each graph in Fig. 18, 19 and 20 includes the three sensors located perpendicularly to one diagonal direction or the other, as shown in Fig. 2. Theoretically, push loads should generate elongations (positive values) in LVDT4, LVDT5 and LVDT6 and shortenings (negative values) in LVDT1, LVDT2 and LVDT3. An opposite behavior was then expected for pull loads.

All sensors showed clearly a cyclical behavior. Damage was rapidly produced in the UW specimen, and the maximum capacity was reached even at relatively low displacements. Cracks were observed along the mortar joints following the main diagonals, corresponding to maximum values in LVDT2 and LVDT5, right across the central area of the wall, with a value around 10 to 12.5 mm, Fig. 18(a). However, there was an even wider crack in the lower part cutting the wall into two independent elements. This continuous crack

–a typical failure mode of masonry walls [29]– disconnected the two parts, both of which moved separately during post-failure. This can be easily observed in Fig. 18(b), which shows the horizontal displacements distribution obtained by DIC analysis for a drift of -20 mm (pull direction). Its maximum width horizontal value measured by DIC system is around 15 mm.

Fig. 18. Results for UW: (a) LVDT measurements; (b) horizontal displacements obtained by DIC analysis.

During the DRW specimen's test, the maximum crack width was around 46 mm (Fig. 19(a)) and it was measured by LVDT6 along a pushing phase. This value was affected by its proximity to the load application point, which exhibited a great degradation because of the concentrated force. Also along the pushing direction, a shortening of around 30 mm can be observed in LVDT2 towards the end of the test, but this value was also affected by another crack produced during a pulling phase, located at the anchorage of the sensor. Fig. 19(b) illustrates the results of the DIC analysis for a drift of +20 mm (push direction). Red lines represent areas where tensile strain values went over 0.8%. It can be seen that, despite the TRM layer,

failure cracks evolved along the mortar joints (red lines in Fig. 19(b)). This crack pattern was concurrent with the cracks that had been previously injected during the repair.

Fig. 19. Results for DRW: (a) LVDT measurements; (b) tensile strain distribution obtained by DIC analysis.

During the NDRW test there was a connection problem with sensors LVDT3 and LVDT4. Therefore, their corresponding measurements could not be recorded. The maximum crack width was around 20 mm (Fig. 20(a)) and it was measured by LVDT5 during a pushing stage. Along the pulling direction, values were lower –around 5 mm along both main diagonals. Fig. 20(b) illustrates the results of the DIC analysis for a drift of +20 mm applied along the pushing direction. Red lines represent again areas with tensile strains in excess of 0.8%. The overall behavior was very similar to the one obtained in the DRW test. Nonetheless, there was a clear difference in the pattern followed by cracks. In addition, cracking initiation associated to the pulling direction was also observed. In addition, Fig. 20(b) is almost concurrent with the failure instant, with a 30 mm wide tilted crack running across the entire surface of the wall.

Despite the smeared cracking –slightly more smeared in the NDRW test–, ductile behavior was observed at the highest shear forces in both DRW and NDRW specimens. Cracks along the principal diagonal followed a relatively similar path in both specimens. Therefore, the fiber mesh of the TRM served as a crack growth controlling device, improving the behavior of masonry wall structural members subjected to in-plane cyclic shear forces. It is also noteworthy that no debonding, fiber-to-matrix slippage or tensile failures of the fiber mesh were detected.

Fig. 20. Results for NDRW: (a) LVDT measurements; (b) tensile strain distribution obtained by DIC analysis.

Finally, Fig. 21 shows the envelope of the crack widths during the three tests. The one with the largest crack width was the DRW specimen, whilst the lack of ductility of the UW specimen barely allowed any deformation. During the interval between 30 and 50 minutes, the DRW showed the greatest increase in crack widths, which was associated to the opening of those that had been initially repaired. Thus, in the NDRW test, the crack growth process took place in a more gradual manner throughout the test.

Fig. 21. Envelopes for crack width evolution.

4. Conclusions.

This paper describes the experimental tests performed on full-scale masonry walls to evaluate the effectiveness of a TRM reinforcement in enhancing the seismic performance of masonry walls. For this purpose, the laboratory tests consisted of three full-scale tests masonry walls subjected to in-plane cyclic loading. Specimens had been built with solid ceramic brick and lime mortar. One of them was tested until failure (unreinforced wall, UW) and was subsequently repaired by means of crack injection, then reinforced with glass fiber-TRM (Damaged and Repaired Wall, DRW). The third test corresponded to an intact, undamaged glass fiber-TRM-reinforced wall (Non-Damaged Reinforced Wall, NDRW).

The repair technique was evaluated in terms of hysteretic behavior, strength and ductility gain, stiffness and strength degradation, energy dissipation capacity and cracking patterns. These experimental results allow us to elicit some conclusions, which could be summarized as follows:

- The behavior of DRW and NDRW under pushing loads showed quite a large improvement of the peak load of the UW, which was increased by 148% and 186% in the DRW and NDRW, respectively.

- TRM also improved the displacement capacity of the masonry walls under cyclic in-plane loads: the final drift amplitudes of the UW were increased by 35% and 20% in the DRW and NDRW, respectively.
- DRW and NDRW showed a strengthening behavior with a certain residual ductility after crack initiation: the NDRW and DRW specimens were able to sustain a drift equivalent to 1/40 of the walls height during post-failure. TRM performed correctly and mesh debonding was not observed during the test.
- TRM-strengthening enhanced the behavior in terms of total energy dissipation and higher ductility, which may prove its efficiency in delaying crack growth, thus improving the behavior of severely damaged masonry walls.
- Although both TRM-strengthened walls were capable of dissipating a substantially greater amount of energy than the original unreinforced structure, the energy dissipation capacity of the DRW specimen was found to be greater than that of the NDRW one. Due to the DRW's previous damage, energy dissipation was provided solely by the TRM layers, which then suffered greater degradation than in the NDRW test.

An important highlight of this research for designers and researchers is that historical in-plane damaged heritage can be repaired and subsequently retrofitted with TRM so that its structural performance is significantly improved to resist eventual catastrophic events, and it would be close to that resulting from retrofitting of an intact structural masonry element. Secondly, as noted in [28], the calibration of statistically based coefficients for analytical design models of TRM-strengthening and their validation depend on the availability of a large number of experimental tests. Therefore, this experimental report supplies a full characterization of the strengthening materials' properties and their outcome in retrofitting full-scale masonry walls for in-plane cyclic loading.

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