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A repair method of cracked shear walls with stainless steel rods and fibres

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ABSTRACT

In this paper destructive shear testing was conducted by installing various sensors and instruments at critical locations of full-scale masonry walls. The purpose of this research was to determine through laboratory testing the effectiveness of new repair methods, used as a seismic retrofit of seriously cracked shear walls. Testing was performed on several different types of walls (brickwork and stone masonry). This experimental campaign is divided in two parts: FRCMs (*Fiber Reinforced Cementitious Mortars*) were initially employed to reinforce the walls and, after testing, stainless steel anchors and other materials have been used to "locally" repair the shear walls (crack sealing), and re-tested. The evaluation methods included the analysis of the lateral load capacities and stiffness characteristics, the failure modes, also implemented to detect any debonding phenomenon between the FRCM retrofit system and the masonry substrate. An evaluation of the overall structural behaviour of the walls before and after reinforcement, and again after the repair, is presented. The results indicate that several conditions have an effect on the efficiency of the composite repair. Shear-load capacity, shear strength and stiffness, wall ductility are considered in the analysis.

1. Introduction

Loss in architectural cultural heritage was very high after recent earthquakes in southern Europe and Middle East [1-3]. Historic masonry buildings, often dated back to many centuries ago, have demonstrated to be able to efficiently resist to vertical static loading (typically dead and occupancy loads), but are intrinsically weak when loaded with horizontal dynamic loads [4-7]. Because the main constituent materials of historic masonry are blocks (stone or bricks) and lime mortar, both the shear and tensile strengths of masonry are highly governed by the mechanical properties of the mortar. This is often very weak, sometimes highly inconsistent and powdery, and cracks can easily develop in the mortar or at interface mortar-to-blocks [8,9].

As a consequence, existing masonry constructions often require upgrading, especially for the building stock located in areas where the seismic hazard is high and the old masonry constructions are made of non-worked stones (round, barely cut, pebbles, ashlar). This is frequent in southern-east Europe, Middle-east Asia, south America and many other countries where an important masonry heritage exists, and the seismic hazard is significant.

During the last decades, composite materials have been extensively studied and experimented in the laboratory as a seismic retrofitting method for old masonry buildings because they are light, exhibit high-tensile strength [10-14]. This type of materials can be applied quickly with a minimal disruption in use. FRPs (*Fiber Reinforced Polymers*), typically under the form of sheets or cloths [15,16], were initially glued to the walls' masonry surface using an organic resin (epoxy or polyester) but, unfortunately, these resins may be susceptible to failures or mechanical degradation due to ageing when exposed to humid environments, freeze–thaw cycles, sun radiation [17-19]. The very-low thickness of the composite reinforcements was another area of concern, due to potential weakness udder punching shear that can lead to catastrophic composite failures. Another limitation of the use of glued-composites was their membrane-effect, preventing or reducing the "breathability" of the masonry, which is essential to reduce moisture and humidity from the buildings and the walls. In addition, these epoxy-bonded composite reinforcements are difficult to be removed, if needed: this represents another limitation of their use for rehabilitation of heritage buildings, because conservation bodies prefer to use "removable" solutions [20,21].

When the FRP are used to reinforce masonry members, they can be very effective, because they can highly contribute to provide the needed

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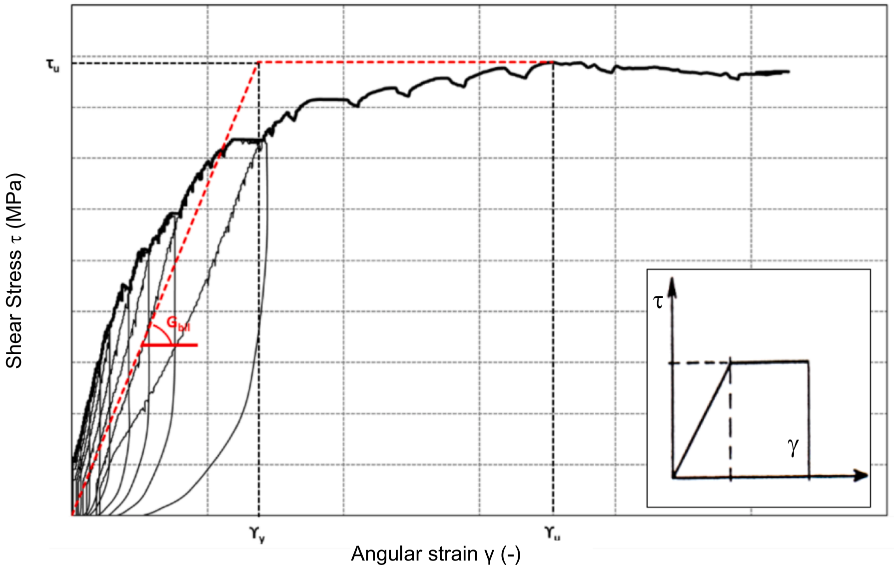


Fig. 1. The method used for the calculation of the shear modulus of elasticity G_{bil}.

tensile strength to masonry. In recent years, extensive research on the use of FRPs in the rehabilitation of masonry structures [22-24]. Different composite reinforcements for masonry shear walls have been developed to cope with these problems, but many of these are only tested in the laboratory with very limited use in real applications.

The use of FRP embedded into an inorganic (mortar) coating could offer a solution for strengthening these walls, as problems resulting from the use of resins are effectively solved. This retrofitting method is known as FRCM (*Fiber Reinforced Cementitious Mortar)*, and typically consists in the use of a flexible or non-flexible (rigid) composite grid embedded into a mortar coating with a thickness of 10–40 mm. In these applications [25-29], composite grids are preferred compared to FRP sheets or cloths because a better connection mortar-to-composite is possible using a grid. Mortar coatings can more durable, more compatible with masonry, more high-temperatures resistant, and their use do not expose the construction workers to chemical risks (toxicity, skin and eye irritation, respiratory tract and allergic respiratory reactions, etc.). While literature is this area is extensive, it should be noted that much of the research concentrated on determining the shear capacity enhancement of undamaged, virgin masonry members [30,31]. A few experimental investigations on the effect of shear wall repair with FRCM have been reported recently [13].

This paper addresses the problem of repairing damaged shear walls using new advanced methods and materials: shear cracks have been sealed with grout injections [32,33], combined with stainless steel anchors [34,35] and SRG (*Steel Reinforced Grout*) [36,37]. The topic is interesting because it is very common in post-earthquake interventions to repair damaged walls, but the objective is particularly challenging given the difficulties involved in the repair of passing-through cracks. While is relatively simple to superficially repair these cracks (by bridging the cracks with tensile-resistant materials like composites or metals), this task is very difficult in the walls depth. The continuity of the masonry material (i.e. restoring the original, pre-damage, wall capacity) cannot be efficiently reinstated by only bridging the cracks on the wall faces. In this regard, we have tested at the structure laboratory of the University of Perugia, Italy, several combined solutions, and interesting indications and results have been found.

2. Materials & method

2.1. Test method

The diagonal shear test has been used in this experimental campaign to study the shear response of the wall panel [38]. This test method has been performed by a large array of researchers over the past 40 years. It involves taking a small square portion of a wall (referred to as a "panel", typically 1.2x1.2 m) and applying a diagonal load to opposing corners until failure. The test objective is to determine parameters affecting the shear strength of a masonry (shear strength, tangential modulus of elasticity, angular strain) assemblage, and to allow for assessment of stiffness properties.

The masonry shear strength has been estimated here using the RILEM guidelines [39]:

$$\tau_0 \cong \frac{1}{3}\frac{F_{\max}}{A_n} \tag{1}$$

where F_{max} is the maximum shear, diagonal load, and A_n is the wall cross sectional area (thickness × panel width).

The calculation of the shear modulus can be done using different methods. In this work, this elastic parameter has been calculated assuming a bilinear Stress Block response (rectangular stress block). The shear modulus G_{bil} is given by the slope of the line of the elastic phase. In practice, the actual stress block is replaced by an equivalent rectangular stress block as shown in Fig. 1, which has the same area. In this figure, the shear stress τ is given:

$$\tau \cong 1.05\frac{F_{\max}}{A_n} \tag{2}$$

and.

$$\gamma_Y = 2(\gamma_u - \frac{A}{\tau_u}) \tag{3}$$

where τ_u and γ_u are the maximum shear stress and the corresponding angular strain, respectively, calculated in correspondence of F_{max}.

2.2. The previous experimental program and the new tests

This experimental research is divided in two steps. Initially 12 walls panels (7 brickwork and 5 stone masonry walls) were constructed reinforced and tested at the structures laboratory of the University of

(a) (b)

Fig. 2. (a) The wall panels tested in the experimental work, (b) Test layout.

Table 1
Reinforcement arrangement and test matrix (first experimental campaign).

Test No.	Type of Masonry	Single- or Double-sided Reinforcement
MAT-01-U	**Brickwork** Masonry (wall thickness 240 mm)	Unreinforced
MAT-02-D		Double
MAT-03-S		Single
MAT-04-D		Double
MAT-05-S		Single
MAT-06-S		Single
MAT-07-S		Single
PIE-01-U	**Stone** Masonry (wall thickness 240 mm)	Unreinforced
PIE-02-D		Double
PIE-03-S		Single
PIE-04-D		Double
PIE-05-S		Single

Perugia (Fig. 2). These tests were carried out on undamaged (virgin) wall panels: the outcomes of this initial research activity have been already been published by the authors in [40] in 2020. Recently, it has been decided to repair the cracked masonry panels resulting from the previous experimental activity, and this paper will mainly focus on this second part of the testing campaign.

To understand, assess and study the proposed retrofitting method to repair cracked shear walls, it is preliminarily necessary to summarize the previous experimental activity. This is given in Tables 1 and 2. It can be noted that a total of 7 brickwork panels (MAT series) and 5 stone masonry (PIE series) have been reinforced using the FRCM technique and tested in the first experimental program. With the goal of improving the shear capacity of the tested shear walls, GFRP (*Glass Fiber Reinforced Polymer*) grids have been embedded into a low-cementitious coating applied to one side (single-sided reinforcement, letter designation S) or to both wall sides (double-sided reinforcement, letter designation D), to referred to as “surface mounted” (SM) retrofit in this work. Because the GFRP grids are embedded in mortar coating entirely, it is hypothesized that substantially better bond characteristics will result. Results from this initial study indicate that FRCM method is effective in strengthening shear walls. Both single- and double-sided FRCM reinforcements resulted in strength increases over the control specimen under monotonic conditions. Ultimate load increases of 167% and 203% for double-sided reinforced stone and brickwork panels, respectively. For single-sided reinforcement, increment in shear capacity was 88% and 71%, respectively. 2. The control specimens, MAT-01-U and PIE-01-U, exhibited the most ductile behavior with a high displacement ductility. Furthermore, the method of GFRP-grid application has been shown to be a feasible method for upgrading the strength and stiffness of uncracked shear walls.

2.3. *The repair methods*

The walls panels tested in the previous experimental program (see Section 2.2) have been repaired using different combined materials and methods. It is worth noting that all specimens (unreinforced, single-sided and double sided reinforced) have shown a similar failure mode, i.e. the development of an inclined (45^0), shear crack along the compressed panel’s diagonal in the masonry. This failure mode had a zig-zag

Table 2
Test results (first experimental campaign).

Designation	Wall Thickness* (mm)	Single- or Double Sided FRCM Reinforcement	Maximum Diagonal Load F_{max} (kN)	Masonry Shear Strength τ_0 (MPa)	Shear Modulus G_{bil} (MPa)
MAT-01-U	240	–	67.03	0.077	1080
MAT-02-D	291	Double	204.9	0.236	1490
MAT-03-S	260	Single	100.2	0.117	1052
MAT-04-D	297	Double	199.8	0.233	2512
MAT-05-S	270	Single	113.4	0.133	2703
MAT-06-S	267	Single	120.9	0.141	1624
MAT-07-S	265	Single	120.9	0.138	1834
PIE-01-U	245	–	73.80	0.084	2092
PIE-02-D	303	Double	182.3	0.206	2034
PIE-03-S	285	Single	136.1	0.155	954
PIE-04-D	307	Double	209.6	0.242	876
PIE-05-S	293	Single	138.2	0.160	1902

[*] Also considering the thickness of the reinforcement coating.


(a) (b)

Fig. 3. Failure mode of wall panels tested in the first phase of the experimental work: (a) unreinforced specimens, (b) reinforced with FRCM (rigid GFRP grid embedded into a mortar coating).


(a) (b) (c)

Fig. 4. Repair of damaged walls by grout injections: (a) removal of the mortar coating along the diagonal crack, (b) application of the plastic tubes for the injections, (c) manual pump used for grout injections.

pattern and only involved the mortar joints (mortar tension failure or cracking at interface block-to-mortar). For brickwork panels a very small number of bricks (typically 3 or 4 for the entire wall made of about 153 bricks) cracked. For reinforced walls, this was accompanied by cracking and crushing phenomena of the mortar coating and deboning of the GFRP reinforcement along the panel's compressed diagonal. Fig. 3

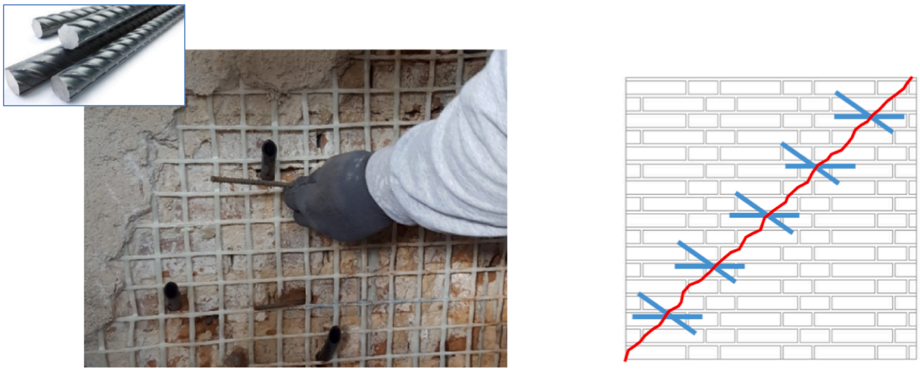

(a) (b)

Fig. 5. Repair of damaged walls with stainless steel anchors: (a) application of the stainless steel rebar into the grouted tubes, (b) Arrangement of the stainless steel ribbed rebar used to bridge the diagonal crack.

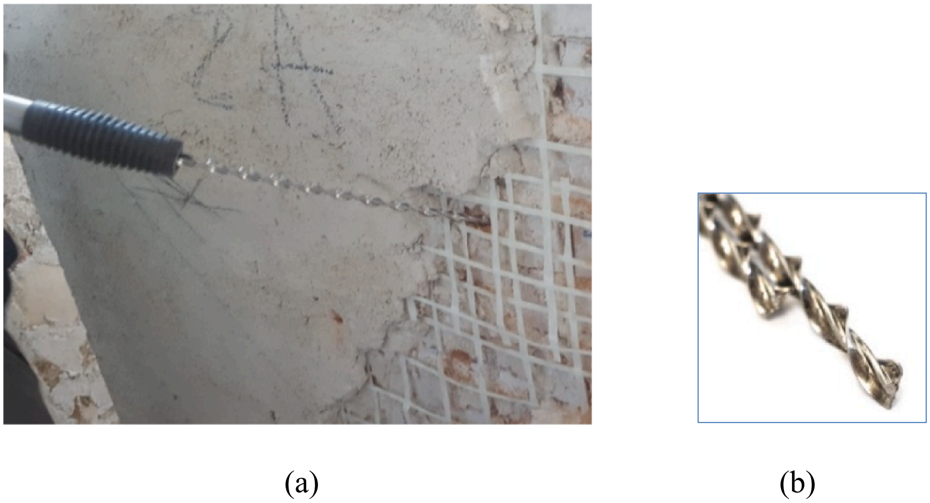


Fig. 6. Repair of damaged walls with “dry” (no-mortar) helical rods: (a) application of the stainless steel rod into the grouted tubes, (b) Detail of the rod (10 mm outer diameter).

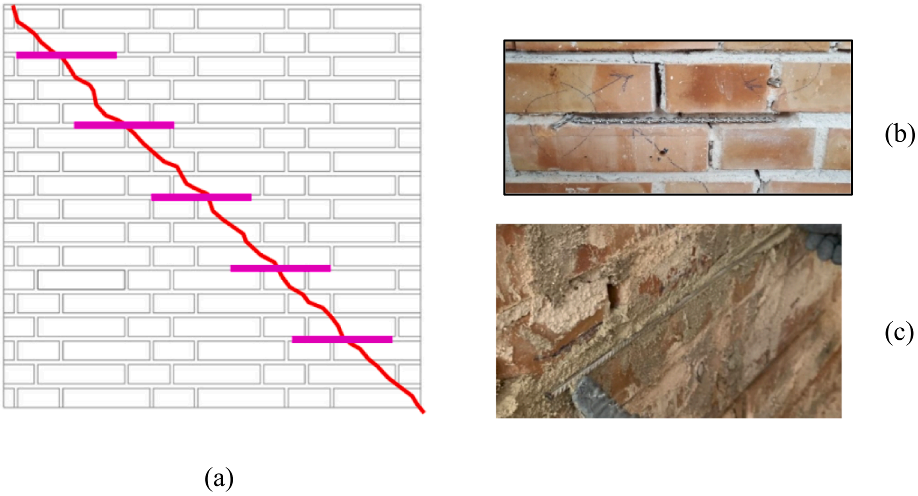


Fig. 7. Repair of damaged walls: (a) Stainless steel rods embedded in the horizontal mortar joints, diameter $\Phi = 6$ mm, length $L = 200$ mm, (b) Helical rods, (c) Ribbed rods.

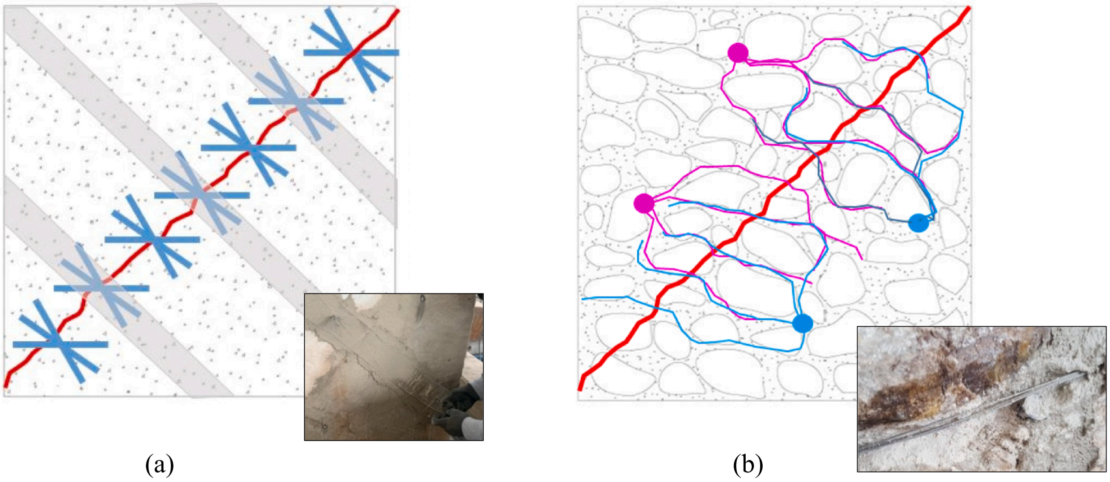


Fig. 8. Repair methods: (a) using unidirectional steel fibres sheets (grey) and stainless rods (blue) , (b) embedding the steel cords into the mortar joints (blue and pink).

Table 3
Test matrix and repair methods.

Designation	MAT-01-U	MAT-02-D	MAT-03-S	MAT-04-D	MAT-06-S	PIE-01-U	PIE-03-S	PIE-05-S
Removal of the mortar coating along the crack (width 200 mm)	No	Yes	Yes $^{+}$	Yes	Yes $^{+}$	No	Yes $^{+}$	Yes $^{+}$
Sealing of the diagonal crack by grout injections	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Anchors, stainless steel rebars, Φ6 mm, 300 mm, holes filled with mortar	No	No	Yes	$10 + 10^{\circ}$	$10 + 10^{\circ}$	$10 + 10^{\circ}$	No	$10 + 10^{\circ}$
Dry anchors, helical rods, Φ10 mm, 45° inclined, length 300 mm	$10 + 10^{\circ}$	$21 + 21^{\circ}$	$10 + 10^{\circ}$	No	No	No	$21 + 21^{\circ}$	No
Local lateral jacketing with SRG (Steel Reinforced Grout)	No	Yes	No	Yes	No	No	Yes	Yes
Bed joint reinforcement stainless steel rebars, Φ6 mm, length 200 mm	No	No	No	No	5*	No	No	$5 + 5^{\circ}$
Bed joint reinforcement helical rods, Φ10 mm, rod length 200 mm	5 + 5	No	5*	No	No	No	$7 + 7^{\circ}$	No
New mortar coating along the diagonal (width 200 mm)	No	Yes	Yes $^{+}$	Yes	Yes $^{+}$	No	Yes $^{+}$	Yes $^{+}$

* Applied only at the unreinforced wall side (single-sided retrofit), ° number of rods applied on each face (for example: 10 + 10 = 10 rods/face), $^{+}$ applied only at the reinforced wall face, Φ = diameter.

Table 4
Mechanical properties of repair materials (*producer data sheet, $^{+}$ mechanical characterization).

	Compressive Strength (MPa)	Young's Modulus (GPa)	Tensile Yielding Strength (MPa)	Notes
Injection grout	> 15*	5*	–	Natural hydraulic lime mix (no cement)
Mortar used for application of stainless steel rebars	> 18*	5*	–	Low water-soluble salts, natural hydraulic lime mix (no cement)
Stainless steel rebars, diameter Φ6 mm	–	180*	571.6^{+}	Ribbed
Helical stainless steel rods, Φ10 mm		>150*	955*	Failure tensile load 15.1 kN, tensile strength 1164 MPa
Steel fibers Φ1 mm, weight density 800 g/m^2	–	210*	1184*	Sheet made of unidirectional steel fiber cords, sheet width 100 mm

Φ = diameter.

shows the typical failure mode of both reinforced and unreinforced wall panels.

Two classes of repair methods have been used: first class included methods to restore the continuity "inside the thickness" of the cracked walls, and it consisted in grout injections, inclined anchors using stainless rebar and helical rods. The second class included "surface mounted" repair methods, aimed at bridging the crack: this consisted in the use of SRG sheets, stainless steel rebars and helical rods embedded in the horizontal mortar joints.

For previously-reinforced specimens, the mortar coating of the FRCM system was initially removed along the diagonal crack, taking care not to damage the fiberglass grid (Fig. 4a). The width of the removed layer of coating was about 20–25 cm. Plastic tubes were subsequently applied in holes drilled across the crack. These tubes have been used to inject a lime grout: injection was carried out manually at low pressure (1 atm) not to damage further the wall panels. Fig. 4b and 4c shows this preliminary repair method, used for all walls.

In addition, several panels have been repaired with stainless steel anchors (ribbed rods), very similar to the ones normally used for reinforced concrete (Fig. 5). These anchors have been inserted in inclined holes, drilled diagonally across the crack and filled with the grout.

Alternatively, other wall panels have been repaired using helical stainless steel rods (with an outer diameter of 6 mm), applied in a similar way to the ribbed rods, but without filling the holes with the grout (dry application, Fig. 6).

With regard to "surface mounted" repair methods, stainless steel ribbed rods and helical stainless steel rods (identical to the ones used for "inside" repair) have been installed into the horizontal bed joints to bridge the diagonal crack (Fig. 7). After the application of the steel rods (length = 200 mm), the mortar joints have been finally repointed with new mortar.

Steel fibres (SRG method) have been also used to repair the walls: steel fibres were applied perpendicularly to the diagonal crack on the surface of the wall in such a way to bridge the crack (Fig. 8a). For reinforced panels, SRG was applied under the form of unidirectional sheet applied diagonally (along the tensile diagonal) over the mortar coating, while, for unreinforced walls, the steel fibres have been embedded into the mortar joints with new mortar: this was necessary in order to preserve the fair face aspect of the masonry (Fig. 8b).

Table 3 summarizes the repair solutions used for all the wall panels, while Table 4 provides the main mechanical and geometrical properties of the used repair materials.

3. Experimental results and analysis

The analysis and interpretation of the test results are not immediate and easy to conduct, given the large number of variables and the limited number of experimental results. However, some interesting conclusions can be drawn. In the following we will use the term "reinforced" to identify virgin (uncracked) wall panels reinforced with FRCM (a GFRP grid embedded into a mortar coating (first experimental campaign)) and the term "repaired" to designate the wall panels tested after having been repaired according the description given in Table 3 (second experimental campaign).

It is preliminarily important to highlight that only 8 wall panels (5 brickwork and 3 stone masonry specimens) were repaired. The damage in the remaining 4 walls was too extensive and serious, and it was not possible to repair these panels: stones and mortars were diffusely disconnected at the end of the first experimental work.

Fig. 9 shows the stress–strain curves for the repaired brickwork specimens and Table 5 the corresponding test results. The following is the comparison between the lateral load capacity from the reinforced and the repaired brickwork panels: it can be preliminarily noted that the average lateral load capacity of unreinforced, single-sided and double-sided reinforced walls were about 67, 110.6 and 202.4 kN, respectively (first experimental campaign).

With regard to MAT-01-U, MAT-03-S, MAT-06-S test results seem to demonstrate that ribbed rods were more effective than helical ones, as a bed joint repair method. These three panels have been repaired using helical (MAT-01-U, MAT-03-S) and ribbed rods (MAT-06-S), applied at

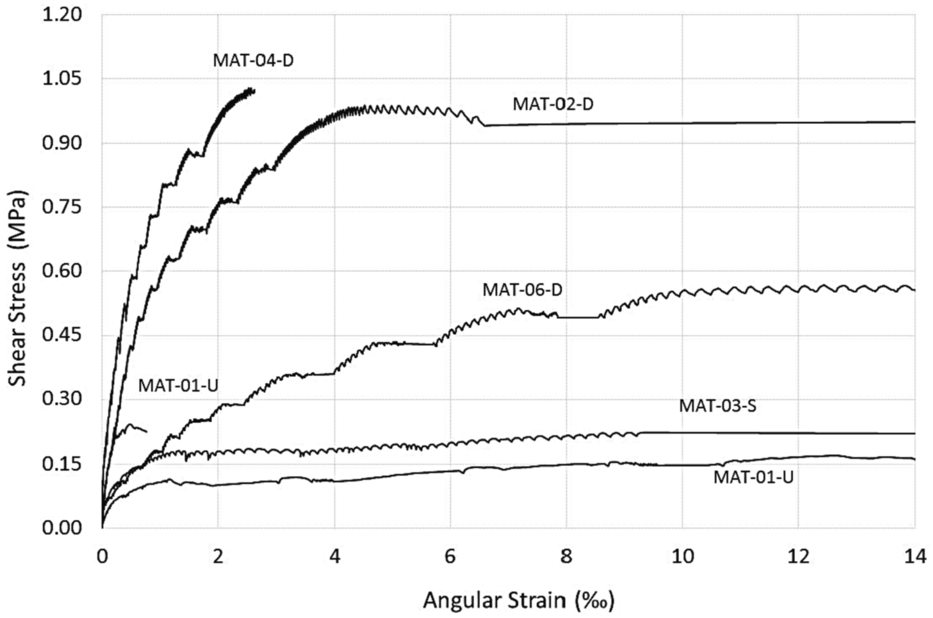


Fig. 9. Repaired brickwork wall panels: shear stress vs. angular strain envelope curves.

Table 5
Test results for unreinforced, reinforced and repaired wall panels.

		F_{max} (kN)	f_t (MPa)	τ_0 (MPa)	$\gamma_{(Fmax)}$ (MPa)	$\gamma_y x10^{-3}$ (-)	G_{bil} (MPa)	$G_{1/3}$ (MPa)
MAT-01-U	Unreinforced*	67.0	0.116	0.077	0.483	–	1080	–
	Repaired	45.9	0.081	0.054	12.66	5.83	29	354
MAT-02-D	Reinforced*	205.0	0.353	0.236	0.653	–	1490	–
	Repaired	283.0	0.471	0.314	4.847	2.39	413	959
MAT-03-S	Reinforced*	100.3	0.175	0.117	1.001	–	1052	–
	Repaired	64.5	0.108	0.072	20.61	3.55	64	676
MAT-04-D	Reinforced*	199.8	0.350	0.233	0.405	–	2512	–
	Repaired	281.7	0.490	0.327	2.532	1.14	904	1864
MAT-06-S	Reinforced*	120.9	0.211	0.141	0.708	–	1624	–
	Repaired	158.6	0.270	0.180	12.43	6.41	89	178
PIE-01-U	Unreinforced*	73.8	0.126	0.084	1.153	–	2092	–
	Repaired	135.6	0.213	0.142	4.840	1.633	274	1994
PIE-03-S	Reinforced*	136.1	0.233	0.155	1.568	–	954	–
	Repaired	181.0	0.306	0.204	7.489	1.738	370	997
PIE-05-S	Reinforced*	138.2	0.240	0.160	0.631	–	1902	–
	Repaired	155.3	0.272	0.181	1.300	0.748	764	2066

F_{max}, = Maximum diagonal Load, f_t = Masonry Tensile Strength, τ_0 = Masonry Shear Strength, G_{bil} and $G_{1/3}$ = Masonry Shear Moduli, $\gamma_{(Fmax)}$ = angular strain at F_{max}, * First experimental campaign.

the un-reinforced side (on both sides for MAT-01-U, and on a single side for MAT-03-S and MAT-06-S). The lateral load capacity decreased of 33.6% (from 67 to 45.9 kN - MAT-01-U, and from 100.3 to 64.5 kN - MAT-03-S) for brickwork wall panels repaired with helical rods, while it increased of 31.2% (120.9 to 158.6 kN – MAT-06-S). The typical failure mode at the unreinforced side was due to ejection of mortar cover near the stainless steel rods and subsequent re-formation of the diagonal crack (Fig. 10).

Table 5 also shows that the repair operations were able to significantly increase the original lateral load capacity for wall panels MAT-02-D (from 205 to 283 kN, Fig. 11) and MAT-04-D (119.8 to 281.7 kN, Fig. 12) compared to the lateral load capacity of reinforced walls. Both these panels were double-sided reinforced, but different repairs were used: panel MAT-02-D was repaired by sealing the diagonal crack with helical anchors, while ribbed stainless rods were used for MAT-04-D. The repair procedure was completed for both panels with a SRG jacketing of the area near the diagonal crack. While the first repair methods (helical anchors or ribbed stainless rods) were also used for the repair of the other panels, the SRG jacketing was only applied to these panels: it can be concluded that the repair method is more effective when this includes the SRG jacketing (steel fibers applied diagonally over the shear crack to bridge it).

The SRG jacketing seems to play a critical role in the resisting mechanism under shear loading: SRG was able to contrast the re-formation of the shear cracks, transferring the tensile stresses between both sides of the diagonal crack. SRG was also (Fig. 8a) applied using an adequate bonding length and this also prevented phenomena of sheet debonding.

On opposite, the repair method resulted non-effective when applied to unreinforced wall panels: MAT-01-U was initially tested in unreinforced configuration. A pass-through shear crack developed along the compressed diagonal in the mortar joints, while the lateral load capacity and shear modulus G_{bil} were 67 kN and 1080 MPa, respectively. This panel was subsequently repaired using helical rods, installed both horizontally in the bed joints and diagonally “inside the wall thickness” to bridge the shear crack. The shear crack was also injected with new lime grout. During the test, the panel could withstand only 68% (45.9 kN) of the original lateral load: the shear crack re-formed/re-opened and large

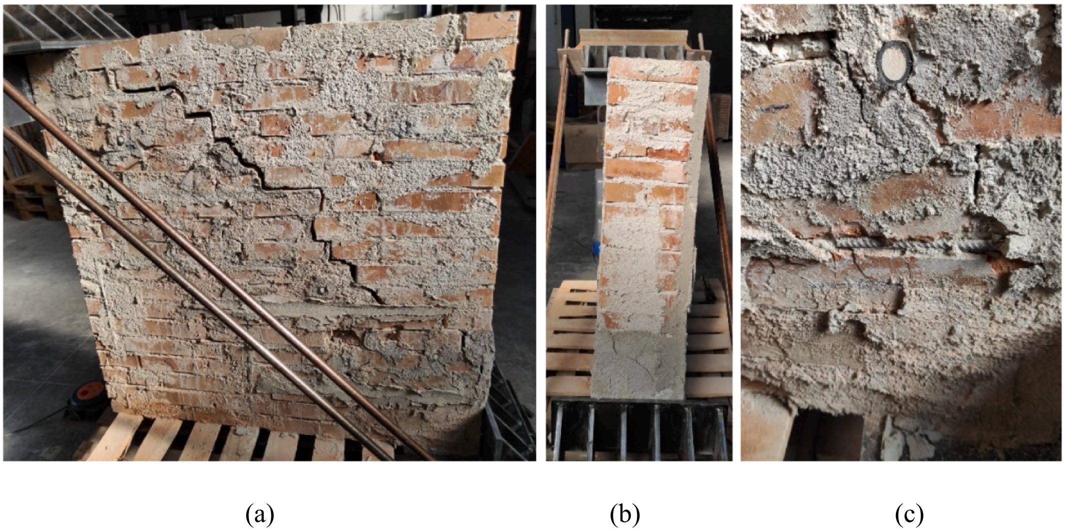

(a) (b) (c)

Fig. 10. Repaired MAT-06-S wall (unreinforced side): (a) Failure mode, (b) Lateral view, (c) detail of the ejection of the mortar cover near the stainless steel bars.

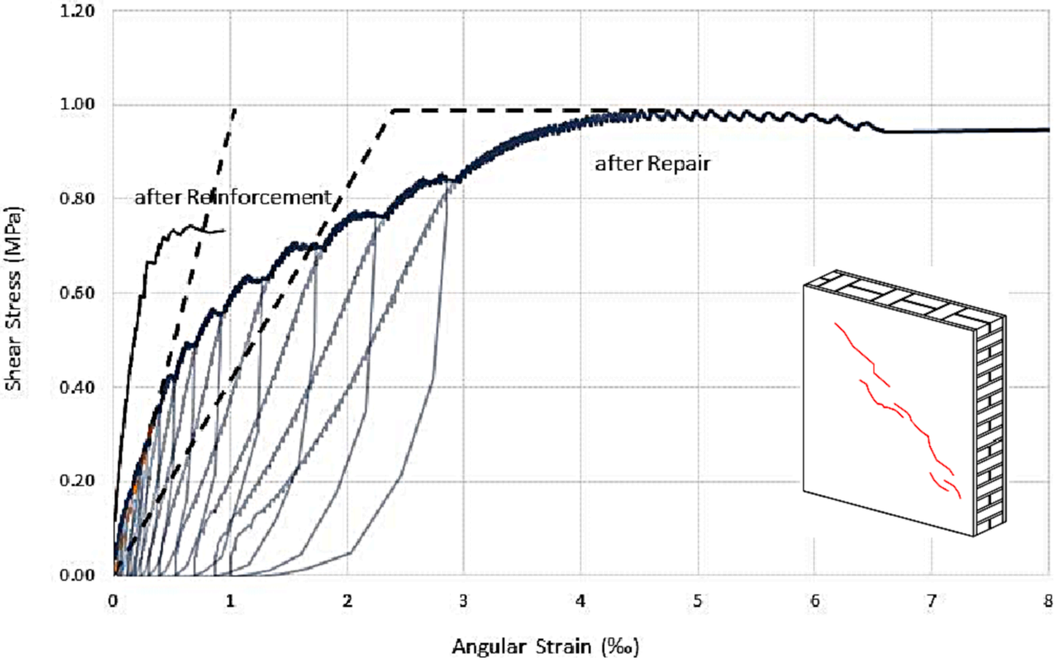


Fig. 11. MAT-02-D tests: comparison between structural response after FRCM reinforcement and after repair.

angular deformations occurred.

In general, phenomena of re-formation/re-opening of the shear cracks happened in all new tests: as a consequence, given the non-elastic character of the wall deformation, the values of the shear moduli shown in Table 5 should be interpreted with this limitation in mind. Both G_{bil} and $G_{1/3}$ (i.e. the shear modulus calculated according the method shown in Fig. 4, and the secant one at 1/3 of the panel lateral ultimate capacity) of repaired panels were always significantly smaller than the ones calculated for reinforced (undamaged) panels. For example, for test MAT-01-U, the shear modulus G_{bil} was 1080 and 29 MPa for the panel in reinforced and repaired configurations, respectively.

The large magnitude of the shear deformations and the low values of shear moduli, compared to wall panels tested in the first experimental work can be interpreted in two different ways: for those tests where an increment of lateral load capacity was recorded, this is indication that repairs start contributing to the resisting mechanism only after the walls were highly deformed or cracked. The different repair materials have different deformation capacities, especially in tension: very low for mortars and injection grouts, and very large for steel fibres. The consequence of this is a premature tensile failure of the mortars/grouts with a progressive redistribution of the stresses between the masonry and repair materials. The resisting mechanism of a repaired wall panels is similar to the one typically exhibited by an indeterminate structure at ultimate load: its response is highly non-elastic, and it is governed by the progressive failure of redundant constraints. On opposite, for those walls where the lateral load capacity wasn't restored with the repair, this is a clear indication of the inability of the repairs to bridge the shear cracks.

For stone work panels, the repair solutions resulted more effective, with load capacity increments ranging between 12.4% and 83.7% (Figs. 13 and 14). This seems the effect of the application of the SRG reinforcement. The fundamental contribution of the SRG is also confirmed by the analysis of the wall failure modes: only when the SRG

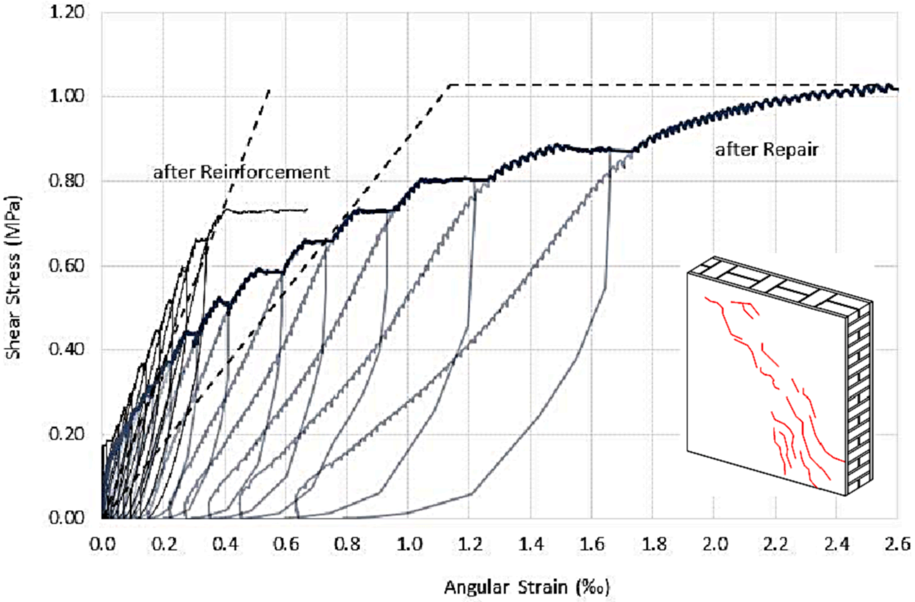


Fig. 12. MAT-04-D tests: comparison between structural response after FRCM reinforcement and after repair.

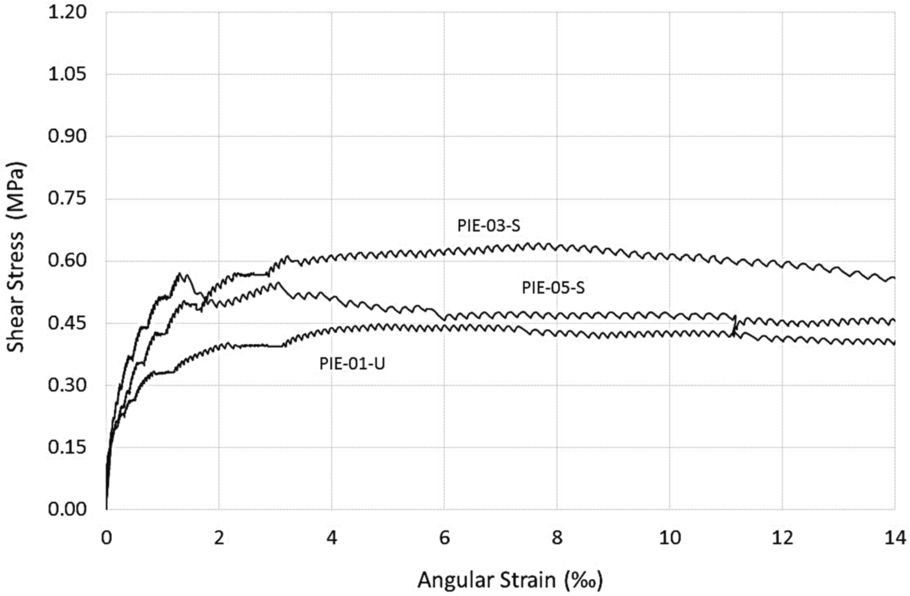


Fig. 13. Repaired stone masonry panels: shear stress vs. angular strain envelope curves.

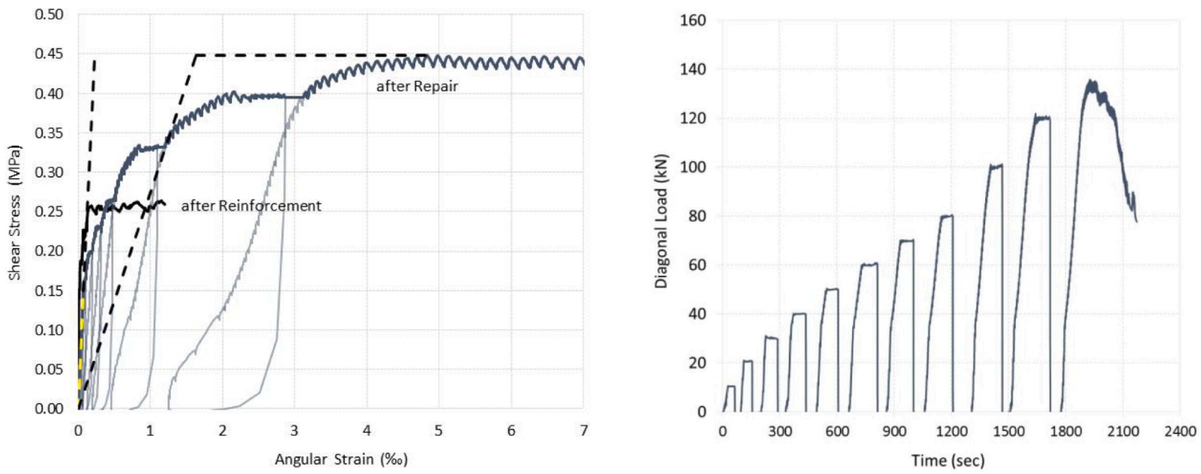
failure (i.e. tensile crisis of the steel fibers), the classic shear cracks in the masonry material could develop. For the shear stiffness, it was again noted that none of the proposed repair methods could restore the original wall shear stiffness. Test results show significant reductions in shear stiffness (40–87%). The proposed repair solutions are unable to restore the masonry continuity, and the used repair materials contribute to the resisting mechanism only after the wall panel is significantly deformed.

4. Conclusions

The shear behaviour of reinforced and repaired masonry wall panels was experimentally studied in this paper. This investigation is divided in two steps: wall panels have been initially reinforced with FRCMs, and tested in shear (diagonal tension test): substantial increases in shear strengths and reductions in shear strains were obtained through external strengthening with rigid GFRP grids embedded into a cementitious coating. The strength increases and strain reductions were much greater when the GFRP grids were applied on both panel sides (double-sided reinforcement). More details of this first experimental work are given in [40].

In the second phase of the investigation, new solutions and materials were successfully used to repair cracked brickwork and stone shear walls. Two classes of repair methods were used: “surface mounted” and “inside the thickness” of the walls. Both methods are aimed at restoring the continuity of the cracked shear walls. With regard to “inside the thickness” methods, shear cracks have been sealed using grout injections, stainless steel inclined anchors (ribbed or helical rods). For “surface mounted” solutions, steel fibres and stainless steel rods (now embedded in the horizontal mortar joints) were used.

The following conclusions can be drawn:



(a) (b)

Fig. 14. PIE-01-U tests: (a) comparison between structural response after FRCM reinforcement and after repair, (b) loading protocol.

1. "Inside the thickness" methods (inclined anchors and grout injection of the cracks) resulted partially ineffective: in many cases this repair methods were unable to restore the original lateral load capacity of the shear walls. It was also difficult to apply the inclined anchors across the crack to bridge it or to uniformly distribute the injection grout along the surface of the crack.
2. "Surface mounted" methods: repair using steel fibres has demonstrated to be effective. This repair solution not only restored the original lateral load capacity of the shear walls, but a significant enhancement in capacity has been recorded.
3. "Surface mounted" methods: stainless steel rods embedded into the bed joints seem to produce a limited repair effect. This could be due to the limited length of the rods (200 mm), the low strength of the repointing mortar, or the unsatisfactory rod-to-mortar or mortar-to-block (brick or stone) bonding characteristics.
4. All repaired walls exhibited large shear deformations and shear moduli, compared to un-damaged wall panels tested in the first experimental work: for those tests where an increment of lateral load capacity was recorded, this is indication that repairs start contributing to the resisting mechanism only after the walls were deformed. For those walls where the lateral load capacity wasn't restored with the repair, this is a clear indication of the inability of the repairs to bridge the shear cracks.

Additional optimization of the repair solutions will be carried out to make them more effective, flexible and more durable.

It is important to highlight that a limited number of walls has been tested in this experimental work. The studied repair solutions were numerous and combined together, and this represent an important limitation of this investigation. This has been done in order to simulate the interventions that could be likely implemented in a real application. More tests and analysis will be necessary to confirm the results of this experimental campaign.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data Availability Statement

The raw/processed data required to reproduce the findings of this experiment cannot be shared at this time as the data also forms part of an ongoing study.

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