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Shear strengthening of wall panels through jacketing with cement mortar reinforced by GFRP grids

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ABSTRACT

This paper gives the results of a series of shear tests carried out on historic wall panels reinforced with an innovative technique by means of jacketing with GFRP (*Glass Fiber Reinforced Plastics*) mesh inserted into an inorganic matrix. Tests were carried out in situ on panels cut from three different historic buildings in Italy: two in double-leaf rough hewn rubble stone masonry in Umbria and L'Aquila and another with solid brick masonry in Emilia. Two widely-known test methods: the diagonal compression test and the shear-compression test with existing confinement stress. The test results enabled the determination of the shear strength of the masonry before and after the application of the reinforcement. The panels strengthened with the GFRP exhibited a significant improvement in lateral load-carrying capacity of up to 1060% when compared to the control panels. A numerical study assessed the global behavior and the stress evolution in the unreinforced and strengthened panels using a finite element code.

1. Introduction

The European building heritage consists mostly of masonry buildings concentrated in many historic centers and in isolated rural constructions. The succession of earthquakes in recent decades, some of which were catastrophic, has draw attention to the problem of safety in old buildings which are frequently still used as residences, public administrative and religious buildings, offices or for other purposes.

The recent National Codes for Design of Earthquake Resistant Structures in Italy [1], Greece [2] and Israel [3] established principles and rules for the safety assessment, improvement and consolidation of historic buildings against earthquakes. In most cases the competent authorities have encouraged an "active" use of these buildings, not only attributing them with representative functions in consideration of their historic-artistic value and of the art and decorative work they often contain, but also ensuring they continue to serve the purpose for which they were originally built, thus as simple residences or offices. In this way it was attempted to avoid the emptying of the historic centers and particularly of those areas in which the buildings are not of much value.

Seismic upgrading and consolidation of historic masonry is a recurring problem in most work done on existing buildings. In many cases, it is necessary to retrofit very low quality historic masonry walls, where there exists almost no alternative to demolition. Technicians have thus had to seek innovative solutions that were both economical and effective for ordinary historic building applications.

Since the 1990s, numerous retrofitting solutions have been proposed. In many cases, however, the lack of experience has undermined the effectiveness of the work done. The use of particularly stiff and invasive concrete ring beams on low quality masonry, the application of epoxy adhesives in environments that are damp or exposed to sunlight, the use of unprotected steel mesh or the failure to connect the wall leaves transversely for ferrocement, and the use of grout injections into not very injectable historic masonry are a few examples of errors that have been made only too frequently by technicians.

Research have been particularly concerned about the mechanical characterization of masonry frequently used in earthquake prone areas. A vast experimental campaign was carried out by Chiostrini et al. [4] on wall panels cut from buildings in Tuscany (Italy) in the 1990s. Other studies have been done by Turnšek and Čačovič [5], Corradi et al. [6], Borri et al. [7] and most recently by Alecci et al. [8].

Techniques for seismic upgrading of masonry wall panels are widely found in the literature. Fibre Reinforced Polymer (FRP) systems are increasingly used for masonry strengthening. The FRP is usually bonded to the surface of the existing structure,

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where it provides tensile strength and restrains the opening of cracks.

In-plane reinforcement of panels has been studied by Valluzzi et al. [9], Triantafillou [10], El Gawady et al. [11] and Roca and Araiza [12]. Other shear reinforcement techniques using traditional and innovative techniques were analyzed by Modena [13], Binda et al. [14], Corradi et al. [15–16] and Ashraf et al. [17].

The application of FRP composites to masonry structures without epoxy adhesives is less well established. Only in the last few years has the use of non-organic matrixes been the subject of research, and it aims at developing a valid alternative to the use of organic matrixes, especially those based on epoxy resins, which present problems of reversibility, compatibility with historic masonry, durability and poor performance at temperatures higher than 60–80 °C. Many historic buildings are restricted by protection and heritage conservation authorities, which in many cases do not authorize an extensive use of epoxy adhesives.

Recently researchers have focused their interest on GFRP grids coupled with non-polymeric matrixes. In earlier studies a system called "Reticolatus" was proposed [18], which includes the insertion of a continuous mesh of thin stainless steel cords into the mortar joints, the flexibility of which allows reinforced repointing for irregular masonry.

Textile reinforced mortar has been recently investigated by Prota et al. [19] for tuff masonry wall panels and by Papanicolaou et al. [20].

2. Experimental procedures

Two test methods can be used to measure the shear strength of a wall panel: diagonal compression and shear-compression test. These experimental methods, as well as their interpretation, have been widely employed by numerous researchers. For an in-depth description of the test methods, reference should made to ASTM [21] and RILEM [22] standards. Identification of shear parameters has been carried out in [23,24]; the appropriate equations for calculating the shear strength will be presented in this study.

In the diagonal compression tests, 1200 × 1200 mm panels were isolated from the surrounding walls by making four cuts with a circular saw. In cases where it was possible to obtain samples underneath existing openings, it was possible to reduce the number of cuts made to three (Fig. 1). The shear strength of the masonry at the center of the panel (τ_{0D}) was calculated on the basis of the interpretation of the test reported in the RILEM standards [22]:

$$\tau_{0D} = \frac{f_t}{1.5} = \frac{P_{\max}}{3A_n} \tag{1}$$

where f_t is the tensile strength of the masonry, $P_{\max}$ is the maximum diagonal load and A_n is the area of the horizontal section of the panel. As regards the tangential elastic modulus, secant elasticity modulus was computed using two points located along the stress–strain curve at 10% and 40% of the maximum shear stress:

$$G = \frac{1.05(0.4P_{\max} - 0.1P_{\max})}{A_n(\gamma_{0.4P\max} - \gamma_{0.1P\max})} \tag{2}$$

where $\gamma_{0.4P\max}$ and $\gamma_{0.1P\max}$ are the angular strains at 40% and 10% of $P_{\max}$ respectively.

For the shear-compression test, the panel, 1800 × 900 mm in size, was obtained by two vertical cuts in the masonry (Fig. 2), i.e. letting the vertical compression load from the remaining part of the building act on the top of the sample. The horizontal force is applied to the midpoint, and in this way the panel can thus be schematized as two superimposed 900 × 900 mm semi-panels. The vertical stress is estimated based on the analysis of the loads weighing on each sample:

$$\sigma_0 = \frac{N}{A_n} \tag{3}$$

where N is the maximum vertical compression load and A_n is the area of the horizontal cross-section of the panel. The tensile strength was calculated according to the Turnšek and Čačovič [5] formulation starting from value of the shear load $P_{\max}$ on the lower semi-panel, in which the shear crisis is generally reached first:

$$P_{\max} = f_t \frac{Bt}{b}\sqrt{1 + \frac{\sigma_0}{f_t}} \tag{4}$$

where B and t are the width and the thickness of the panel respectively, b is the shape factor, which in this case is assumed to be equal to one. The tensile strength value of the masonry f_t in the lower semi-panel was used to determine the shear strength τ_{0T}:

$$\tau_{0T} = \frac{f_t}{1.5} \tag{5}$$

3. Strengthening technique

The strengthening technique tested in this study may be classified as the Near Surface Mounted (NSM) reinforcing methods. Compared to the traditional ferrocement technique, instead of metal bars a GFRP grid is inserted into a low cement content mortar jacketing. The use of composite materials provides a solution to the problems usually encountered in traditional ferrocement: the

Fig. 1. On-site diagonal compression test (San Felice building).

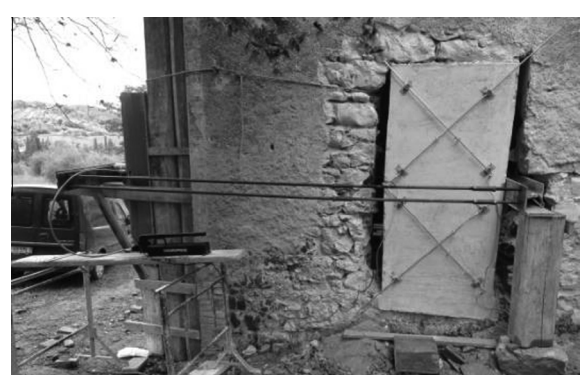

Fig. 2. On-site shear-compression test (Colle Umberto building).

rusting of the steel rebars, the excessive stiffness of the reinforcement, the limited reversibility of the work.

The GFRP grid used is manufactured by Fibre Net S.r.l. and is characterized by square mesh with nominal dimensions of 66 × 66 mm. It is made up of AR (*Alkali Resistant*) glass fiber with a zirconium content equal to or greater than 16% pre-impregnated with thermosetting epoxy vinyl ester resin (Fig. 3). The geometric and mechanical properties, measured with tensile tests are shown in Table 1.

The strengthening of the panels obtained from existing walls called for the complete removal of the lime plaster on both sides and the elimination of any loose materials with compressed air. 12 mm-diameter holes were made through the wall for the connectors according to the scheme shown in Fig. 4. The GFRP grid and connectors were then installed. Each connector consisted of two unidirectional fiberglass L shaped bars joined together by injecting epoxy paste into the hole.

Lastly mortar was applied by hand in a thickness of about 30 mm. Despite the presence of the composite grid, the application of mortar was not difficult, thanks to the large mesh size adopted (Fig. 5).

A cement-based mortar was applied to all of the reinforced samples; this mortar had the following composition by volume: 1 part of hydraulic lime, 1 part strength grade 32.5 OPC (Ordinary Portland Cement); 2½ parts of dry sand.

The choice of a mortar with a high cement content was dictated by the need to have to carry out the tests just 28 days after the application of the reinforcement. The mortar's mechanical properties, determined by compression tests [25] and indirect tensile strength tests [26] on cylindrical samples 100 mm in diameter and 200 mm in height, are given in Table 2. Compressive strength of mortar at 6, 10 and 30 days after casting has been measured. Fifteen cylindrical samples were tested and the average 30-day strength of mortar was 21.36 MPa.

4. Experimental tests

4.1. The Colle Umberto building

This building, constructed in the early 19th century as a farmhouse and currently unused, is located in Umbria, in the countryside between Lake Trasimeno and the town of Perugia in Italy. The building has two floors: the ground floor, designed to be used for storage, is divided into three rooms by stone masonry walls consisting of two weakly connected leaves.

The building is characterized on the ground floor by two types of walls built in different periods. The first type, which presumably goes back to the early 1800s, is 560–570 mm thick and was made with very poor lime-based mortar. The stones are up to 350 mm in size, and are well squared (Fig. 6). The second type, built in the early 1900s, is 480 mm thick and has lime-based mortar with better mechanical properties (Fig. 7). However, the stones are very rough hewn and almost square in shape, with sides of not more than 250 mm. In both wall types there are no through stones.

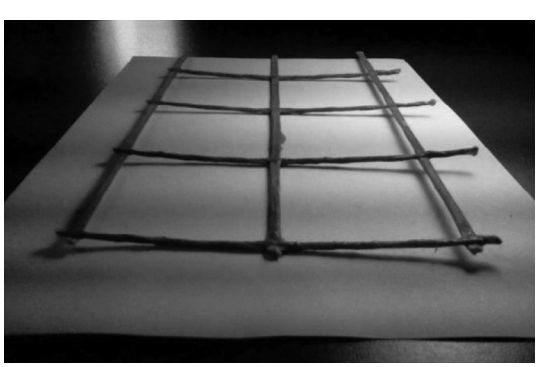

Fig. 3. Detail of the GFRP grid.

Table 1
Mechanical characteristics of glass FRP mesh.

Horizontal direction	Tensile strength (MPa)	530
	Sample size	10
	Cross section (mm^2)	7.29
	Elongation at failure (%)	1.73
	Young's modulus (GPa)	36.1
Vertical direction	Tensile strength (MPa)	680
	Sample size	10
	Cross section (mm^2)	9.41
	Elongation at failure (%)	1.93
	Young modulus (GPa)	39.8
Weight density	[kg/m^2]	0.5

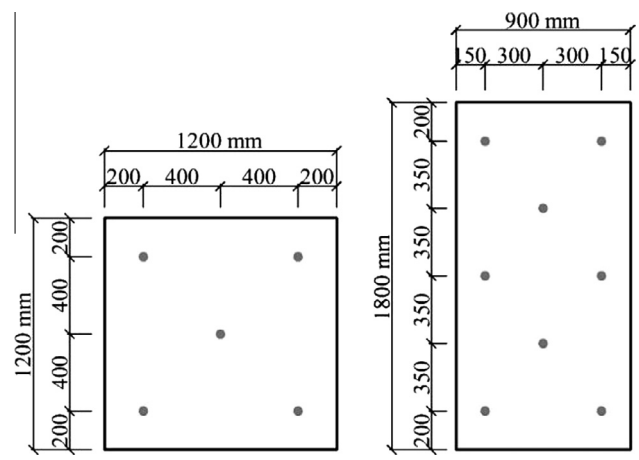

Fig. 4. Anchor position.

The test panels were taken from the ground floor. 3 panels were cut for diagonal compression tests (1 in the 19th-century wall and 2 in the 20th-century wall) and 4 panels for shear-compression tests (2 from each wall type).

4.2. The San Felice sul Panaro building

The building is located in the countryside near the village of San Felice, in the province of Modena, Italy and has been severely damaged by the recent earthquake in Emilia in 2012, having suffered a partial collapse of the floors and the overturning of an external perimeter wall.

The building is rectangular in shape and completely isolated from other structures. It has three floors (ground floor, first floor and an attic) and is made entirely of solid brick masonry. The wall structure, however, is unusual in that the walls, which are about 300 mm thick, have few connecting bricks (headers) between the leaves (8–12 bricks/m^2), almost as if it were a wall with two separate leaves, each made from solid bricks (Fig. 8). The floors are made with wooden beams. Three tests were carried out on this building: two compression and one shear-compression test. All tests were done on the external walls on the ground floor.

4.3. Palace Pica Alfieri in L'Aquila

The stone-wall panels of the L'Aquila building were made of a double-leaf wall with a thickness between 58 and 61 cm. The

Fig. 5. Application of mortar jacketing and detail of connection between anchors and GFRP grid.

Table 2
Mortar mechanical properties.

Days of curing	Young modulus (GPa)	Compression strength (MPa)	Tensile strength (MPa)
6	–	18.40	–
10	–	20.64	–
30	22.53	21.36	2.14

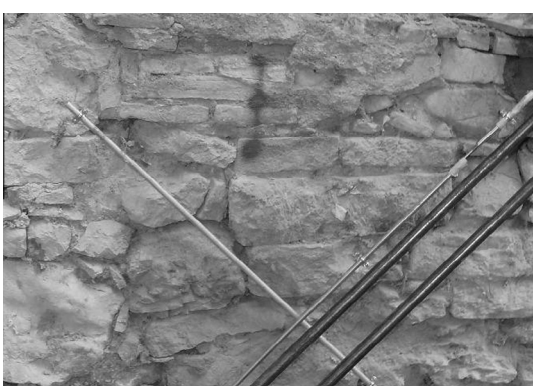

Fig. 6. Stonework of Colle Umberto farm house (19th-century).

Fig. 7. Stonework of Colle Umberto farm house (20th-century).

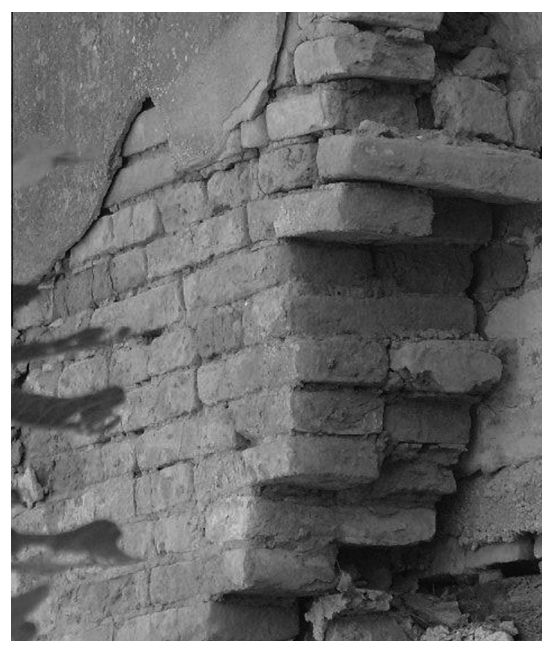

Fig. 8. Brickwork of San Felice building.

panels were cut from undamaged walls of Palazzo Pica Alfieri, an aristocratic residence reconstructed after the 1703 earthquake. The stone bonding pattern is made with rubble stones with a maximum length of 200–250 mm and the wall is a double-leaf masonry with through stones. Four panels were cut off. One of them was tested without any kind of strengthening technique: all were subjected to a diagonal compression test.

4.4. Laboratory tests

Twenty-two diagonal compression tests were carried out in laboratory. Test results are presented in detail in [27–28] and partially reported here only to be compared with on-site and numerical results. More specifically, tests were done on: (a) 14 panels of double-leaf rough hewn stone masonry, with a thickness of 400 mm

and (b) 8 panels of uncut rounded stones (pebbles), with a thickness of 400 mm.

In order to simulate a historic lime-based mortar, stone and brick panels were built with a hydraulic lime-based mortar characterized with low mechanical properties. For pebble stone panels, a weaker lime-based mortar has been used.

5. Test results

A total of 17 shear tests were carried out on-site. The number of tests is greater than that of the panels because in the Colle Umberto building, when possible, the samples tested in their original state were repaired and then tested again. In this way it was possible to evaluate the effectiveness of the proposed reinforcement technique not only for preventive application, but also when used as a repair technique. The in-site test program was as follows:

1) For the Colle Umberto building: (a) 5 diagonal compression tests: 2 on unreinforced panels (CD-02-U-OR, CD-06-U-OR), 1 on a panel with preventive reinforcement (CD-07-U-IP) and 2 on repaired panels (CD-08-U-IR, CD-10-U-IR) (Figs. 9 and 10) and (b) 5 shear compression tests: 2 on unreinforced panels (TC-16-U-OR, TC-17-U-OR), 2 on panels with preventive reinforcement (TC-19-U-IP, TC-20-U-IP), 1 on a repaired panel (TC-21-U-IR);
2) For the San Felice building: (a) 2 diagonal compression tests: 1 on a unreinforced panel (CD-09-S-OR), 1 on a panel with preventive reinforcement (CD-11-S-IP) and (b) 1 shear compression test on a unreinforced panel (TC-18-S-OR);
3) For the L'Aquila building: 4 diagonal compression tests: 1 on a unreinforced panel (cut from the adjacent building of S. Maria Misericordia) (CD-11-A-OR), 3 on panels with preventive reinforcement (CD-12-P-IP, CD-13-P-IP, CD-14-P-IP).

Each test is identified with a code of four indices, the first of which indicates the test type (CD = diagonal compression, SC = shear-compression), the second a progressive number identifying the panel, the third the location where the test was done (U = Colle Umberto, S = San Felice, P = Pica Alfieri) and lastly the fourth index identifies the type of shear strengthening done (OR = unreinforced panel, IP = preventive reinforcement, IR = panel repaired).

The results obtained from the diagonal compression tests are given in Table 3.

The unreinforced rough hewn stone panels at the Colle Umberto building gave fairly similar shear strength values τ_{0D}, between 0.018 MPa and 0.021 MPa. The highest shear strength value was measured for the oldest masonry (19th-century) having a thickness of 600 mm (test CD-06-U-OR).

Fig. 9. Un-reinforced panel tested at Colle Umberto.

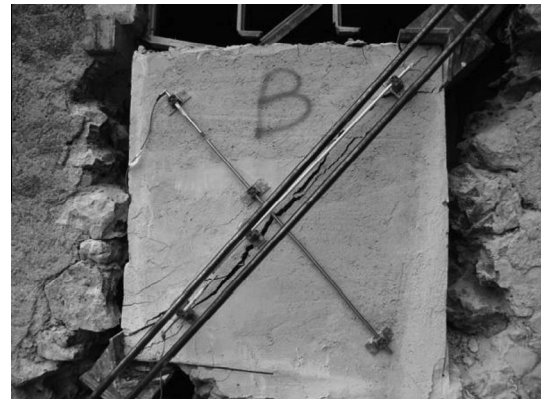

Fig. 10. Crack pattern of a reinforced panel tested at Colle Umberto.

The cracks produced in the unreinforced panels were exclusively in the mortar joints and involved the entire thickness of the wall panel along the compressed diagonal. A similar shear strength (0.020 MPa) was obtained also for the unreinforced panel made with solid bricks at the San Felice building.

As regards the panels reinforced in advance or as a technique for repairing pre-damaged masonry, the results showed a substantial effectiveness of the technique tested. The results obtained for Colle Umberto panels (20th-century masonry wall panels) were of particular significance. In this case a shear strength of 0.162 MPa was measured for the panel with preventive reinforcement and 0.209 MPa for the repaired panel, compared to a shear strength of 0.018 MPa in the same panel unreinforced, an increase varying between the 800% and 1060% (Fig. 11).

The two diagonal compression tests (one on an unreinforced panel (CD-09-S-OR) and one on a panel with preventive reinforcement (CD-11-S-IP) at the brick building in San Felice sul Panaro have essentially confirmed the results obtained on the stone panels, although the effectiveness of the reinforcement was less significant. In this case, there was an increase in shear strength from 0.020 MPa to 0.086 MPa for the reinforced panel (Table 3, Fig. 12). Due to the limited number of headers in the brickwork, the two leaves tend to separate and deform differently during the shear test.

The results of the shear-compression tests, given in Table 4, show increases in shear strength τ_{0T} that are similar to those obtained in the diagonal compression tests.

For the reinforced Colle Umberto 20th-century masonry panel (with an original thickness of 480 mm), a shear strength increase of 638% was measured, going from 0.032 MPa (unreinforced) to 0.236 MPa (reinforced), while the shear strength of the panel repaired reached 0.173 MPa. Fig. 13 shows a comparison between the graphs of the maximum shear loads of the tests on unreinforced, reinforced and repaired stone panels.

In the case of 19th-century stone masonry wall (600 mm thick), a less significant increase was measured in shear strength, which went from 0.023 MPa (unreinforced) to 0.116 MPa (reinforced). The lower increase in shear strength can be explained by the lower ratio between the thickness of the two GFRP jacketings and the thickness of the wall cross section. As this ratio decreases, the effectiveness of the reinforcement tends to diminish. It should be pointed out, however, that in this case the strength value measured for the reinforced panel does not represent the failure value of the panel, but only that corresponding to the maximum applied load, since, for security reasons, it was not possible to test the panel to failure.

Table 3
Diagonal compression test results.

Panel no.	Wall section (cm)	Bond pattern	Load P_{max} (kN)	Tensile strength f_t (MPa)	Shear strength τ_{0D} (MPa)	Shear modulus G (MPa)	$\tau_{0D,R}/\tau_{0D,UR}$
CD-02-U-OR	48	1	31.2	0.028	0.018	29	–
CD-06-U-OR	60	1	44.1	0.031	0.021	35	–
CD-09-S-OR	28	2	19.6	0.029	0.020	150	–
CD-07-U-IP	57	1	333.4	0.244	0.162	2787	9.0
CD-08-U-IR	56.5	1	422.3	0.314	0.209	2458	11.6
CD-10-U-IR	70	1	543.6	0.321	0.214	–	10.2
CD-11-S-IP	38	2	112.1	0.129	0.086	795	4.3
CD-11-A-OR	62	1	53.0	0.034	0.023	83	–
CD-12-P-IP	72	1	215.8	0.125	0.083	668	4.1
CD-13-P-IP	64	1	269.2	0.175	0.117	732	5.1
CD-14-P-IP	64	1	204.1	0.133	0.089	895	3.8

Bonding patterns: (1) stones and (2) bricks.

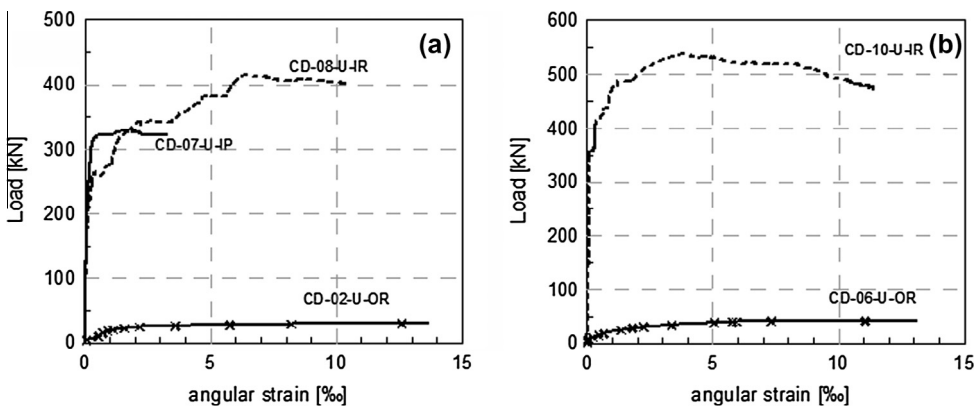

Fig. 11. Curves of the shear stress-angular strain response for stone masonry panels, (a) 20th-century masonry and (b) 19th-century masonry (Diagonal compression test method, Colle Umberto).

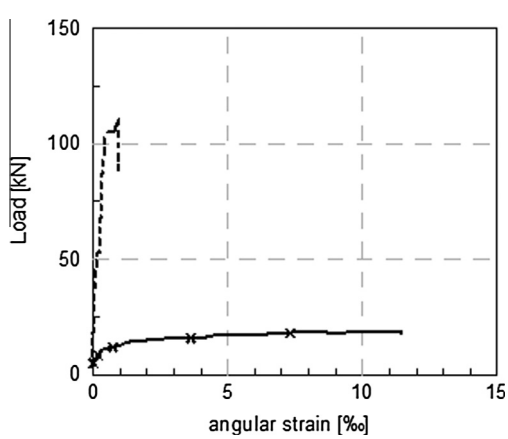

Fig. 12. Curves of the shear stress–angular strain response for brick masonry panels (Diagonal compression test method).

At the San Felice building, just one shear-compression test was carried out on unreinforced masonry, and a shear strength of 0.062 MPa was measured.

Like that observed in the diagonal compression tests, this type of test as well the unreinforced samples showed cracks only in the mortar joints along the compressed diagonals. However a partial change in the type of failure was observed in the reinforced panels. The shear cracks along the compressed diagonals of the lower and upper square semi-panels were formed after evident horizontal cracks due to in-plane bending action at the midpoint of the panel (Fig. 14).

For panels retrofitted in L'Aquila, the strengthening constrained the development of the cracks, and failure was confined by the GFRP jacketing. The average panel lateral strength increased to 0.0963 MPa (average maximum load 229.7 kN), i.e. the GFRP enhanced the lateral resistance by a factor of approximately 4.33 compared to the control panel (Fig. 15). The ultimate limit state was clearly a shear failure that was initiated by tensile rupture in grid fiber reached when the masonry cracked in tension.

Table 4
Shear-compression test results.

Test no.	Wall section (cm)	Bonding pattern	Compression stress σ_0 (MPa)	Load P_{max} (kN)	Shear strength τ_{0T} (MPa)	$\tau_{0T,R}/\tau_{0T,UR}$
TC-16-U-OR	48	1	0.100	36.1	0.032	–
TC-17-U-OR	60	1	0.100	36.7	0.023	–
TC-18-S-OR	28	2	0.200	40.6	0.062	–
TC-19-U-IP	67	1	0.100	131.6	0.116	5.0
TC-20-U-IP	56.5	1	0.100	203.8	0.236	7.4
TC-21-U-IR	56.5	1	0.100	155.3	0.173	5.4

Bonding patterns: (1) stones and (2) bricks.

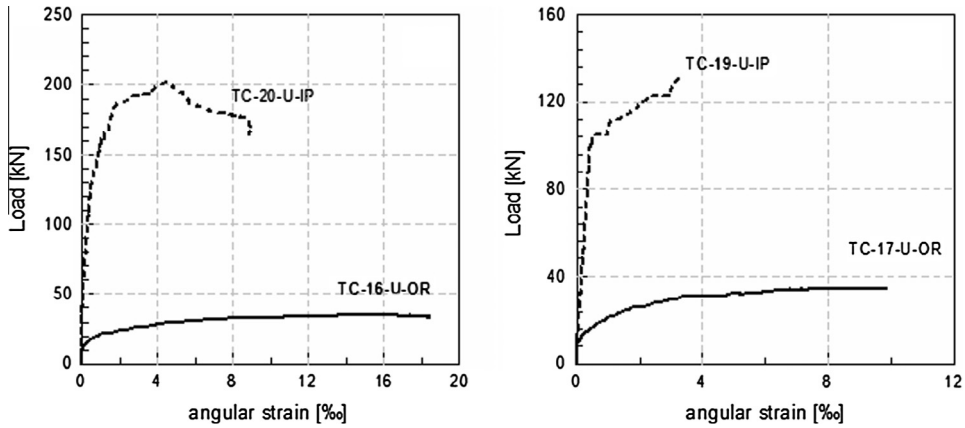

Fig. 13. Curves of the shear stress–angular strain response for stone masonry panels (shear compression test method, Colle Umberto).

The elastic phases of the curves of the reinforced panels are characterized by a steeper slope as those obtained in the case of the unreinforced ones, regardless to the type of the stonework masonry. In-plane stiffness (shear modulus, G) increased significantly due to reinforcement (from an average value of 32 to 2622 MPa respectively for unreinforced and reinforced panels in Colle Umberto and from 83 to 765 MPa in L'Aquila). Moreover, we remark an important deformation capability of the unreinforced walls, emphasized by the presence of a relevant post-elastic plateau. The reinforced panels exhibited a smaller deformation capacity compared to unreinforced ones. However this is usual for reinforced concrete/mortar jacketing: if compare this behavior with the one of steel mesh reinforced concrete jacketing we note an important increase in the deformation capacity [15].

A comparison between the results obtained by the diagonal compression and shear-compression tests (Tables 3 and 4) allows one to observe the differences in the values obtained. In view of this phenomenon, observed previously in experimental investigations carried out by the authors (Corradi et al., 2003; Borri et al., 2013), the problem arises again regarding the choice of the shear test that best simulates the behavior of masonry subjected to horizontal lateral forces.

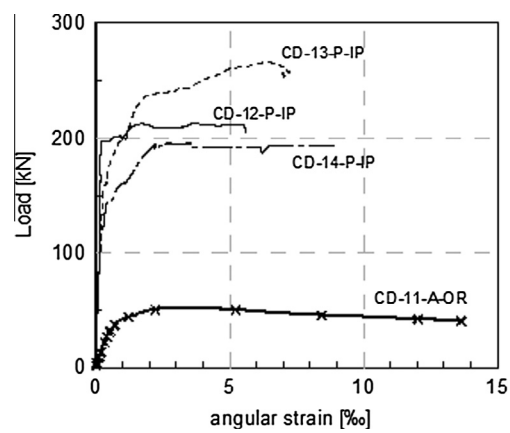

Fig. 15. Curves of the shear stress–angular strain response for stone masonry panels (Diagonal compression test method, L'Aquila).

Fig. 14. Crack pattern of a reinforced wall panel tested at Colle Umberto.

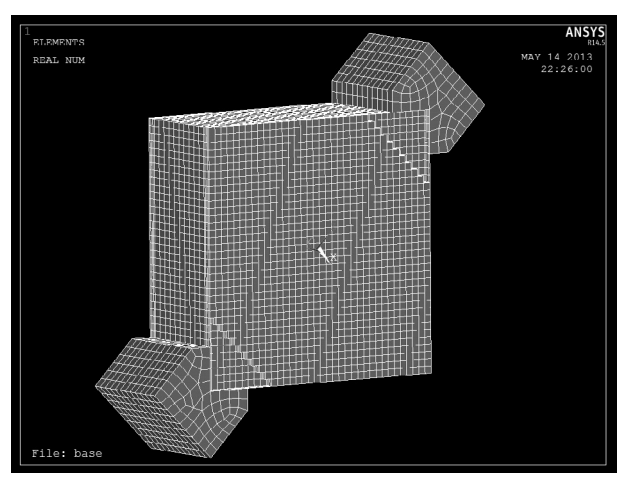

Fig. 16. Model and FE mesh adopted in the numerical simulations.

6. Finite element analysis

In this section a simulation of panels tested under diagonal-compression is presented. Simulation is based on the laboratory

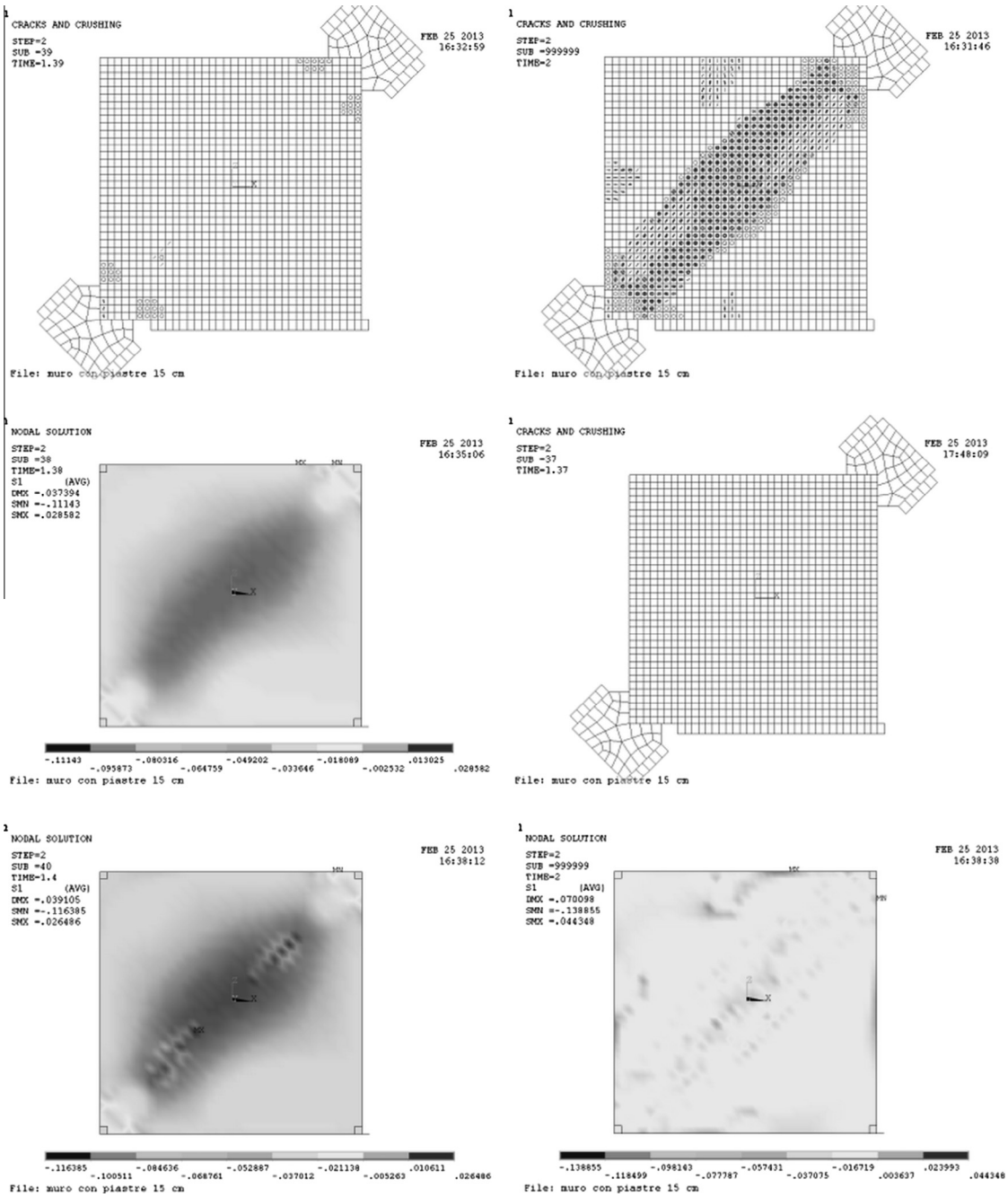

Fig. 17. Tensional results vs crushing and cracking path obtained in the non-linear analysis due to increasing shear load.

Table 5
Numerical simulation results.

Sample	Ultimate load (kN)		Normalized load (numerical/experimental)
	Experimental	Numerical	
MC21-22	218.3	133.3	0.61
T-MC-1-CHL8	375.4	312.5	0.83
MP-1A-F66s	349.1	349.3	1.00
MP-1B-F66s	381.0	363.1	0.95
T-MP-1-NHL6	438.6	312.5	0.71
T-MP-1-CHL8	461.1	372.3	0.81
T-MP-1-NHLZ12	473.4	399.9	0.84

results. The aim is to evaluate the reliability of the use of such simulations in predicting the shear behavior of strengthened wall panels.

In view of this, reinforced panels are described by means of a three-dimensional finite element model, in which masonry and mortar jackets are modeled separately (the interfaces being merged with joints). The internal reinforcement (GFRP mesh) was modeled using three dimensional spar elements with plasticity embedded within the solid mesh. This option was favored over the alternative smeared stiffness capability as it allowed the reinforcement to be precisely located whilst maintaining a relatively coarse mesh for the surrounding mortar medium. The inherent assumption is that there is full displacement compatibility between the reinforcement and the mortar jacketing with no occurrence of bond slippage.

Non-linear constitutive laws are adopted for both masonry and mortar jacketing: values of the physical properties of both materials have been established on statistical analysis of experimental data obtained from characterization tests [27,28]. They have been implemented in a general purpose commercial code (ANSYS). Eight-noded hexahedron elements have been adopted. The average size of the hexahedron elements was chosen so as to have ten and four elements across the panel and mortar jacketing thickness respectively: this allows the more critical details to be captured. The complete finite element model (FEM) is shown in Fig. 16.

All the materials forming the structure were assumed to be isotropic. This approximation is partially due to the lack of information about the properties of the materials along different directions. Another reason for this is that taking anisotropy into account would make the numerical mode, which is definitely cumbersome, even heavier.

The analyses carried out on the non-linear 3D F.E. model have been used to evaluate the shear strength capacity of the tested specimens (Fig. 17).

The comparison of the experimental and the predicted values of the shear strength is reported in Table 5. It can be observed that, except for sample MC21-22 and T-MP-1-NHL6, the estimation errors are always within 20%; moreover, it can also be seen how in all cases the proposed approach leads to conservative values when compared with the experimental results. Therefore, until new approaches to predict the strength of confined masonry are available, the proposed model appears to be adequate for design purposes.

7. Conclusions

The experimental results reported in this paper form a base of knowledge on the effectiveness of an innovative technique for shear strengthening of masonry walls. A series of tests on historic masonry wall panels reinforced with GFRP grids inserted into an inorganic matrix made with a cement-based mortar was carried out. The technique is to be classified as a Near Surface Mounted reinforcements (NSM) of masonry walls.

The increase in strength following reinforcement with GFRP grids was highly significant, and although the results were differentiated depending on the different masonry types tested and the procedures for application of reinforcement as a preventive technique or for repairing damaged masonry, this method can be considered a viable solution to problems of strengthening and seismic upgrading of some types of historic masonry. Even though more tests are needed, it can be concluded that the specimens retrofitted or repaired with GFRP grids behaved satisfactorily.

The problem of wall leaves connection was dealt with through the use of composite bars inserted in holes in the masonry and connected to the GFRP grid applied to the surface of the panels. For masonry walls of limited thickness the applications tested were able to bring about a significant increase in the shear strength.

Conclusions obtained from these experimental results are valid for actual materials and construction techniques. Results presented in the paper should be interpreted taking into account this variability. The finite element modeling can be a helpful tool when the reinforcement of these structures with GFRP grids is proposed. However, substantial work is still needed to validate the proposed model, considering different masonry types. Nevertheless, general conclusions stated in the paper are not expected to be affected by the observed experimental variability.

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