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# REINFORCEMENT OF MASONRY PANELS WITH GFRP GRIDS

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**Abstract.** *This paper reports and analyses the results of a series of mechanical tests carried out on historic walls reinforced with an innovative technique by means of jacketing with GFRP (Glass Fiber Reinforced Plastics) grid inserted into an inorganic matrix made of cement-based mortar in order to increase the lateral load-carrying and deformation capacity of unreinforced masonry (URM) walls. The basic construction material of analysed historic buildings are masonry assemblage of barely cut stones of different size and brickwork cemented by weak lime/sand mortar. This heterogeneous masonry typologies constitute a kind of low-strength masonry especially to horizontal seismic actions. Shear tests were carried out in-situ on panels cut from two different historic buildings in Italy: one in double-leaf rough hewn rubble stone masonry in Umbria and the second in the city of L'Aquila. Two widely-known test methods have been used for the experimental work: the diagonal tension test and the shear-compression test. The test results enabled the determination of the shear strength and shear modulus of the masonry wall panels 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 unreinforced panels.*

## 1 INTRODUCTION

Many Un-Reinforced Masonry (URM) historic structures are widely present throughout Europe and other regions around the world. These structures seem to suffer the most severe damage during earthquakes. Not only for the prohibitive cost of replacing all substandard structures, but also for conservation purposes, it is necessary to develop innovative techniques for rehabilitating deteriorating structures.

The behavior of URM buildings in response to dynamic forces provoked by earthquakes is linked to the type of masonry, the poor quality of which is a factor of serious vulnerability that often makes ineffective the interventions. In these cases, the achieving of an adequate degree of safety is connected to the use of reinforcements that are able to ensure an improvement of the mechanical properties of the walls.

Retrofitting of historic masonry is a recurring problem for existing buildings. The aim is often to increase the shear strength of masonry wall panels to achieve a better global behavior for in-plane seismic actions. In many cases, it is necessary to reinforce low quality historic masonry walls, where there exists almost no alternative to demolition. Technicians and researchers have thus had to seek innovative solutions that were both economical and effective for standard historic buildings. In recent years, the technique of strengthening masonry members with externally bonded Fiber Reinforced Polymer (FRP) laminates has been widely investigated and reported. FRP materials are advantageous due to their high tensile strength, low unit weight, resistance to environmental conditions, and flexibility and ease of application during upgrading interventions, although the FRPs have a lower modulus of elasticity and a higher material cost than traditional steel reinforcing. The majority of extant experimental work investigating the retrofit of historic masonry constructions has been conducted on wall panels having an FRP material bonded to the panel surface, where it provides tensile strength and restrains the opening of cracks.

In-plane reinforcement of panels has been studied by Valluzzi et al. [1], Triantafillou [2], El Gawady et al. [3] and Roca and Araiza [4]. Other shear reinforcement techniques using traditional and innovative techniques were analyzed by Modena [5], Binda et al. [6], Corradi et al. [7-8] and Ashraf et al. [9].

Extensive research has been performed on FRP materials applied to masonry walls, and results reveal that shear and flexural strength in masonry walls can be increased through the use of FRP composites externally bonded to masonry elements. However, some areas of concern have been recently highlighted: difficulty in removal of reinforcement, poor behavior of epoxy resins at temperatures above the glass transition temperature, high cost of epoxies, potential hazards for the manual worker and long term behavior of adhesives [10]. Epoxy resins also prevents water-vapor permeability and its fire resistance is very low. In many cases heritage conservation authorities (*Soprintendenze* in Italy, *English Heritage* in England, *CRMH* in France, etc.) do not permit an extensive use of epoxy adhesives on historical listed buildings or monuments.

One possible solution to the above problems would be the replacement of epoxy resins with inorganic ones, e.g. lime or cement-based mortars, leading to the replacement of FRP with Fiber Reinforced Mortars (FRM). A relatively small number of studies investigating the behavior of FRM-strengthened masonry wall panels have been conducted. [11-14]. In earlier studies a system called “Reticolatus” was proposed by some of the authors of this article [1r], 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. The consensus is that the effectiveness of fiber reinforced inorganic mortars is governed by properties of the bonding between the reinforcement and the masonry substrate. Textile reinforced mortar has been recently investigated by Prota et al. [16] for tuff masonry wall panels, by Papanicolaou et al. [10] and by Borri et al. [17] on masonry specimens constructed in laboratory.

## 2 OBJECTIVES AND SCOPE

To efficiently determine the shear in-plane retrofitting effects of GFRP reinforcing grids placed in URM wall panels, seventeen specimens were tested. Only twelve specimens were reinforced with GFRP grids, while some remained unreinforced, which would be later repaired by the reinforcing GFRP grids. This approach was taken to evaluate the full retrofitting characteristics of the GFRP reinforcing grids. The strengthened wall panels were chosen in a manner that allowed flexural stability, along with increasing the effects of shear deformations. During testing, the loading was applied slowly using an hydraulic jack and shear deformations were recorded using inductive transducers (LVDT) applied on both masonry surfaces.

## 3 ANALYSIS PROCEDURES

All masonry wall panels underwent shear in-plane loading. Two test methods have been used to measure the shear strength: diagonal compression and shear-compression test. For an in-depth description of the test methods, reference should made to ASTM [18] and RILEM [19] standards. Identification of shear parameters has been carried out in [19, 20]; the appropriate equations for calculating the shear strength will be presented in this study.

The specimen chosen for the first type of shear tests was a 1200x1200 mm wall panel. 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. A special purpose metal element, according to ASTM [18] was designed and fabricated for the diagonal compression tests. It consisted of two welded steel angles with welded steel triangular elements. An overall view of the test apparatus, showing the wall panel under test, is depicted in Figure 1.The shear strength of the masonry at the center of the panel ($\tau_{0D}$) was calculated on the basis of the interpretation of the test reported in the RILEM standards [19]:

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

where $P_{max}$ is the maximum diagonal load, $f_t$ is the masonry tensile strength and $A_n$ is the area of the horizontal or vertical section of the wall square panel. For shear elastic modulus G, this was evaluated at 10% and 40% of the maximum shear diagonal load, using the following:

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

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

For the shear-compression test, the panel, 1800x900 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 900x900 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 An is the area of the horizontal cross-section of the panel. The tensile strength was calculated according to the Turnšek and Čačovič [22] formulation starting from value of the shear load $P_{max}$ on the lower semi-panel, in which the shear crisis is generally reached first:

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

Figure 1: Diagonal compression test.

Figure 2: Shear-compression test.

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 ft in the lower semi-panel was used to determine the shear strength $\tau_{0T}$:

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

## 4 REINFORCEMENT PROPERTIES

The composite material used to reinforce the wall panels consists of a GFRM grid. A single grid (weight density 0.5 kg/m$^2$) was used for panel reinforcement. The reinforcement was manufactured using AR-glass (Alkali-Resistant) fibers and an epoxy resin. Specimens extracted from the composite mesh has been measured with a tensile modulus ranging from 36.1 to 39.8 GPa. The GFRP grid has a cross section area of 7.29 mm$^2$ and 9.41 mm$^2$ respectively in the vertical (weft) and horizontal (warp) direction and has an opening of 99 mm in both directions. The GFRP mechanical properties are described in Table 1.

The choice of the mortar to be used for jacketing is a difficult task; in fact, to have good durability, this mortar must be compatible with the existing masonry from the physical, chemical and mechanical points of view. The mortar should be strong enough but not too stiff, and have good bond with the stones and with the existing mortar. A weak cement-based mortar was used to strengthen the wall panels. Results of mechanical tests carried out on 15 mortar specimens showed a 30-day Young's modulus of 22.53 GPa, compression strength of 21.36 MPa and tensile strength of 2.14 MPa. The mortar's mechanical properties were determined by compression tests [22] and indirect tensile strength tests [23] on cylindrical samples 100 mm in diameter and 200 mm in height.

For stone panels, in order to prevent detachment and different behaviors of the wall panel leaves, it is necessary to apply transversal connectors. The two reinforced faces were connected to each other by means of 2 GFRP L shaped bars joined together by injecting epoxy paste.

Table 1: Mechanical characteristics of glass FRP mesh.

| | Horizontal direction | Vertical direction |
|---|---|---|
| Tensile strength [MPa] | 530 | 680 |
| Sample size | 10 | 10 |
| Cross section [$mm^2$] | 7.29 | 9.41 |
| Elongation at failure [%] | 1.73 | 1.93 |
| Young modulus [GPa] | 36.1 | 39.8 |

In this method the outer layers of plaster is removed until the surface composes of stones and/or bricks. The elimination of any loose materials with compressed air is important because this will result in a better bonding between the GFRP grid and masonry surface. Appropriate implementing of this method will result in proper transfer of shear stresses between the masonry surface and the GFRP grid. Compared to the traditional ferrocement technique, instead of metal bars a GFRP grid is inserted into a low cement content mortar jacketing.

Figure 3: Joint repointing with new mortar.

Figure 4: Application of the GFRP grid and anchors.

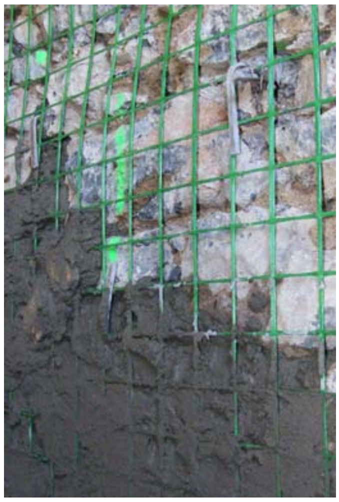

Figure 5: Application of second layer of mortar.

12 mm-diameter holes were made through the wall for the connectors as shown in Figures 3-5. 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).

## 5 IN SITE TESTS

The first part of the experimental campaign was carried out in-site. Masonry brick- and stone-panels were cut off from two buildings in Italy. A brief description of the masonry typologies is reported in the following sections.

This building named after the village of Colle Umberto in Umbria (Italy) was constructed in the early 19th century as a farmhouse and the wall are made of stone masonry 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 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. The second type is 480 mm thick and has lime-based mortar with better mechanical properties. 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.

The second building is located in L'Aquila, Italy and has been severely damaged by the earthquake in Abruzzo in 2009, having suffered a partial collapse of the floors and the overturning of an external perimeter wall. The wall structure, however, is unusual in that the walls, which are about 560-600 mm thick, have any connecting stones (headers) between the leaves. The floors are made with wooden beams. Three tests were carried out on this building: two compression and one shear-compression test.

## 6 TEST RESULTS AND ANALYSIS

A total of 14 shear tests were carried out in-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); 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 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, P or A= Aquila Building) 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 2. The unreinforced rough hewn stone panels at the Colle Umberto building gave fairly similar shear strength values, 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).

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.

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% Figures 6-9 and Table 2 present the recorded behavior of the monotonic specimens tested in this study.

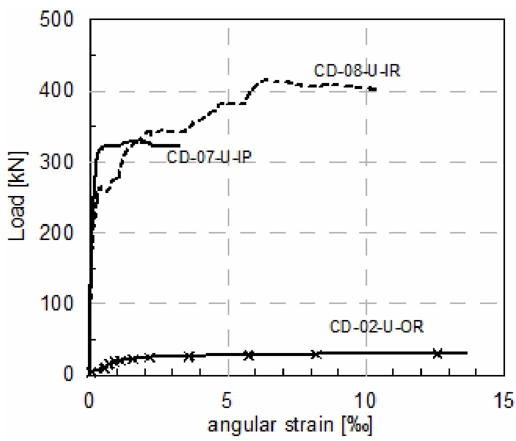


Figure 6: Curves of the shear load-angular strain response for stone masonry panels for the 20th-century masonry typology.

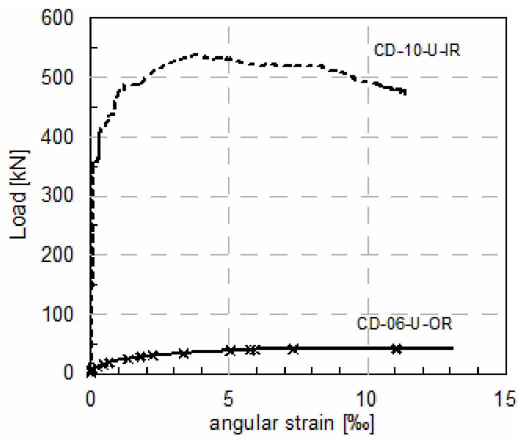


Figure 7: Curves of the shear load-angular strain response for stone masonry panels for the 19th-century masonry typology.

The results of the shear-compression tests, given in Table 3, show increases in shear strength $\tau_{0T}$ 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. Figures 8 and 9 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.

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. 10). 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.

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). Since many retrofit applications result in an over-reinforced condition, the behavior of masonry in traction and partially in compression is critical to the response of the reinforced member. As a result, ductility is often sacrificed. However the load-angular strain curves observed explains its relatively ductile ultimate behavior due to crushing and cracking of masonry material and the superior bond characteristics of the NSM GFRP grids help the specimens attain a better value of displacement ductility. The reinforced panels exhibited a smaller deformation capacity compared to unreinforced ones.

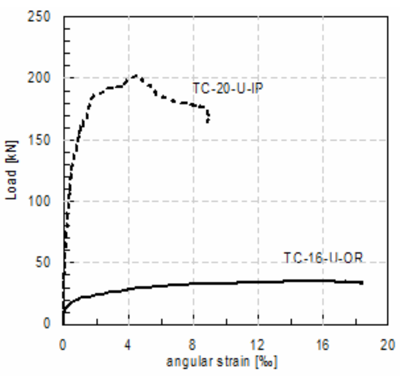

Figure 8: Curves of the shear load-angular strain response for stone masonry panels (shear compression test method, Colle Umberto) for the 20th-century masonry typology.

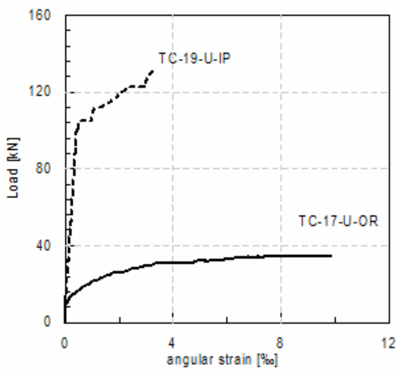

Figure 9: Curves of the shear load-angular strain response for stone masonry panels (shear compression test method, Colle Umberto) for the 19th-century masonry typology.

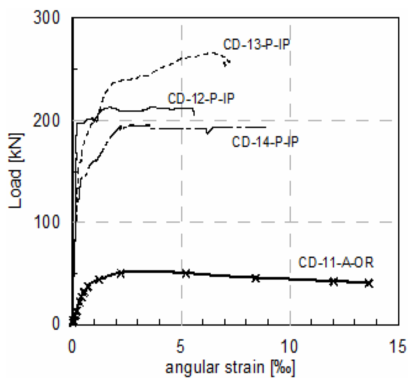

Figure 10: Curves of the shear stress-angular strain response for stone masonry panels (Diagonal compression test method, L'Aquila).

Table 2: Diagonal compression test results.

| Panel No. | Wall section [cm] | Load $P_{max}$ [kN] | Tensile strength $f_t$ [MPa] | Shear strength $\tau_{0D}$ [MPa] | Shear Modulus G [MPa] | $\tau_{0D,R}/\tau_{0D,UR}$ |
|---|---|---|---|---|---|---|
| CD-02-U-OR | 48 | 31.2 | 0.028 | 0.018 | 29 | - |
| CD-06-U-OR | 60 | 44.1 | 0.031 | 0.021 | 35 | - |
| CD-07-U-IP | 57 | 333.4 | 0.244 | 0.162 | 2787 | 9.0 |
| CD-08-U-IR | 56.5 | 422.3 | 0.314 | 0.209 | 2458 | 11.6 |
| CD-10-U-IR | 70 | 543.6 | 0.321 | 0.214 | - | 10.2 |
| CD-11-A-OR | 62 | 53.0 | 0.034 | 0.023 | 83 | - |
| CD-12-P-IP | 72 | 215.8 | 0.125 | 0.083 | 668 | 4.1 |
| CD-13-P-IP | 64 | 269.2 | 0.175 | 0.117 | 732 | 5.1 |
| CD-14-P-IP | 64 | 204.1 | 0.133 | 0.089 | 895 | 3.8 |

A comparison between the results obtained by the diagonal compression and shear-compression tests (Tabs. 2-3) 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 [7-8], the problem arises again regarding the choice of the shear test that best simulates the behavior of masonry subjected to horizontal lateral forces.

Table 3: Shear-compression test results.

| Test No. | Wall section [cm] | Compression stress $\sigma_0$ [MPa] | Load $P_{max}$ [kN] | Shear Strength $\tau_{0T}$ [MPa] | $\tau_{0T,R}$/ $\tau_{0T,UR}$ |
|---|---|---|---|---|---|
| TC-16-U-OR | 48 | 0.100 | 36.1 | 0.032 | - |
| TC-17-U-OR | 60 | 0.100 | 36.7 | 0.023 | - |
| TC-19-U-IP | 67 | 0.100 | 131.6 | 0.116 | 5.0 |
| TC-20-U-IP | 56.5 | 0.100 | 203.8 | 0.236 | 7.4 |
| TC-21-U-IR | 56.5 | 0.100 | 155.3 | 0.173 | 5.4 |

## 7 COMPARISIONS

The results of previous experimental campaigns will be reported in this section with the aim to allow a comparison. In particular two different retrofitting methods applied on stone masonry wall panels of the same dimensions used here for diagonal compression tests (1200x1200 mm) will discussed. A full description of the results is reported in [7] and [8]. The “traditional” ferrocement retrofitting method and the application of GFRP sheets with epoxy resins will be discussed and compared.

Figure 11: A panel subjected to diagonal compression test retrofitted with ferrocement.

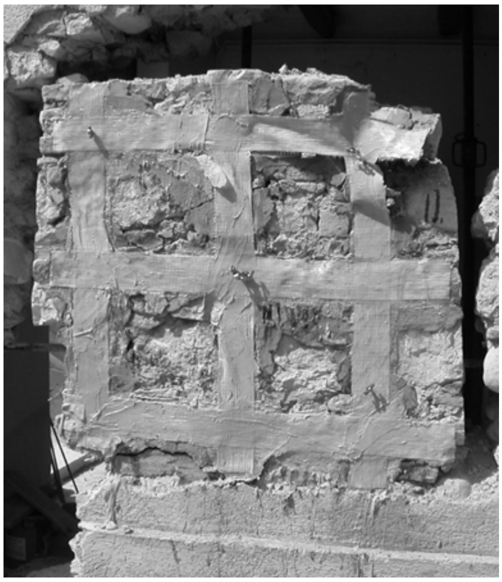

Figure 12: A panel subjected to diagonal compression test retrofitted with GFRP sheets applied with an epoxy resin.

Ferrocement: This method is consists of applying reinforced cement onto both faces of a wall. The reinforcing material can be steel rebars mesh (Fig. 11). The concrete was applied by shotcreting and a total thickness of coating of 40-50 mm was used. The coating was applied in two layers. After completing the first layer of shotcrete the steel (B450C type) mesh (100x100 mm, rebar diameter 6 mm) was installed and then was applied the second layer. The steps for ferrocement reinforcement, applied to rubble or cut-stone stone masonry walls, were: a) Removal of existing plaster; b) Loose stones were first removed and then fixed with cement-based mortar; c) Through holes are drilled for 6 mm-diameter rebars anchors $9/m^2$; d) first layer of coating; e) application of steel mesh; f) second layer of coating.

GFRP jacketing: Uni-directional glass fiber was used to retrofit stone wall panels (Fig. 12). The GFRP laminates involved in the experimental work consist in glass unidirectional fibers embedded in epoxy resin, according with the wet lay up technique. Strengthening with one sheet of GFRP was carried out on both sides of the panel. Through holes were drilled for 12 mm-diameter GFRP rebars anchors $5/m^2$. Rebars were wrapped with GFRP sheets connected to the external reinforcements.

As expected, the panel reinforced with ferrocement resulted very stiff (G=261 MPa). Shear strength was 0.120 MPa (with an increment of 328% compared to unreinforced panels). The stress-strain curve shows a quasi-elastic behavior with a weak yield plateau. The failure mechanism consisted in sudden loss of collaboration between reinforcement (concrete) and substrate (masonry) with no cracks along the compressed diagonal observed on concrete surface. All the un-reinforced panels present the failure along the compressed diagonal. Cracking appeared suddenly in the mortar joints and in the bricks for single-leaf solid brick masonry panels. The shear strength was 0.028 MPa while the shear modulus G was 38.5 MPa.

For panels retrofitted with GFRP sheets, two panels, with a thickness of 450 mm, were repaired by GFRP sheets in order to check the effect of this method when used as a repair. In this case the shear strength change from the un-repaired panel to the panel after repair, is from 0.019 to 0.063 MPa, which is a significant increase in value, but smaller compared to the increase when ferrocement or FRM with GFRP grids is applied. On the contrary, the shear stiffness G decreased from 79.6 to 29 MPa due to the fact that the panel was internally cracked. Thus, a first consequence of the reinforcement is the increase of the strength of the wall while restoring only a part of the initial in-plane stiffness. The adhesion between the panel and the epoxy plaster, used as a base for the fibers, did not fail, but the aesthetic aspect could be considered a problem. The crisis resulted from a separation of the two masonry leaves and from masonry local cracking at un-reinforced zones

From these results, a clear tendency is shown: ferrocement alone can significantly increase the shear stiffness of the masonry, while a very large increase in shear strength is obtained (for tested wall panels from 38.5 to 261 MPa). A similar results can be reach using GFRP grids embedded into inorganic matrices without dealing with typical rusting problems of steel bars A high increase in shear strength was found in this experimental campaign with a significant increment of the shear modulus. On the contrary, the application of GFRP sheets with organic (epoxy) adhesives has the positive effect to increase the shear strength without affecting the masonry shear modulus.

## 8 CONCLUSIONS

The following conclusions can be made from this study:

- 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 Near Surface Mounted reinforcements (NSM).
- Results from preliminary studies indicate that significant shear strength increases are possible through the use of this system in specimens tested in-site under monotonic conditions. Although the results were differentiated depending on the different masonry types tested and the procedures for application of reinforcement as a preventive tech-

nique 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.

- 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.
- The application of FRM with GFRP grids causes significant increments in both shear strength and modulus similar to the effect of ferrocement method. The comparison with results of panels reinforced with GFRP sheets applied with organic matrices highlighted an higher effectiveness of this retrofitting technique, but large increments of shear modulus were measured.

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