New Materials for Strengthening and Seismic Upgrading Interventions

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1. Abstract

This paper focuses on the use of Fiber Reinforced Polymers (FRP) for strengthening and seismic upgrading interventions applied to existing wood and masonry structures. In order to study the performance of wood beams strengthened with FRP materials, several un-strengthened specimens and beams were tested under four point bending up to the point of failure. Other wood specimens and beams were strengthened with externally bonded carbon, glass or aramidic fiber-reinforced polymer (respectively CFRP, GFRP and KFRP) sheets. The second part of the investigation relates to the interventions on masonry buildings. The results of tests carried out on masonry elements show that the application of FRP materials may provide increases in tensile and shear strength. The use of carbon and glass FRP as a reinforcing method for different kinds of masonry elements (vaults, arches, walls, cells, etc) as well as the different possible positions of the FRP sheets, is analyzed, and some solutions are proposed to avoid peeling problems, increase the value of strength and the efficiency of the bond surface. Experimental results are reported along with the case history of the Town Hall of Assisi, where useful indications were obtained on the practical strengthening operations of the masonry vaults. It is therefore important to recognize that the study and the arrangement of the bonding is the key to optimize the structural behavior of strengthened wood and masonry elements.

2. Introduction

Many un-reinforced wood and masonry structures are widely present throughout Italy and other regions around the world. These structures seem to suffer the most severe damage during earthquakes. Not only the prohibitive cost of replacing all substandard structures, but also the conservation of the historic heritage of the country, necessitates the development of innovative techniques for rehabilitating deteriorating structures. As a consequence most of these structures need strengthening or seismic upgrading work in order to ensure their conservation and functional use. Table 1 shows the possible applications of FRP materials for strengthening and seismic upgrading interventions.

3. Innovative materials applied on wood structures

3.1 Introduction

The use of composite materials to reinforce wood elements is not new. Tendons for prestressed elements were developed by Rubinsky and Rubinsky (1954) [1] and by Kaifasz (1960) [2]. High-strength lightweight cables for applications on cable-supported structures, like bridges, were studied by Meier (1987) [3]. Fujisaki et al (1987) [4] and Saadatmanesh and Ehsani (1989) [5] analyzed the possibility to replace the steel classically used as a reinforcement in concrete structures by composite rebars, and demonstrated that the response of these materials under load can be predicted using the analytical methods known to practical engineering. The greatest advantage of these materials was their resistance to corrosion.

TABLE 1: Possible interventions using FRP materials

	Type of intervention using FRP materials	*Related problems*
Wood structures	Flexural reinforcement of wood beams using FRP sheets and re-bars epoxy-bonded or inserted in wood-beams in tension zone; Shear reinforcement of wood beams	Adhesion between FRP materials and wood/masonry surface; Aging behavior of reinforcement and adhesion;
Masonry structures	Shear reinforcement of masonry walls using FRP sheets epoxy-bonded to masonry walls or embedding FRP re-bars mounted near the surface of masonry walls; Flexural reinforcement of masonry walls Seismic upgrading intervention on masonry vaults using FRP sheets; Confinement of masonry columns with FRP sheets; Provisional intervention in order to put in security damaged masonry buildings; Connection of multiple leaf masonry walls (re-bars).	High and low temperatures behavior; Connections of different shapes of FRP materials; Dynamic behavior of the reinforced structures; Types of mortars used in order to leveling masonry surfaces to which the FRP sheets are bonded; Creep behavior; Monitoring methods of reinforced structures (adhesion faults, decreases in mechanical properties of new material, etc)

Ranisch and Rostasy (1986) [6], and Saadatmanesh and Ehsani (1989) [5] analyzed the possibility to increase the strength of existing structural members with epoxy-bonded fiber-composite sheets. This application involved external bonding of thin fiber reinforced composite sheets on the tension face of beams using epoxy resins. These studies demonstrated that monodirectional composite sheets are ideal tension reinforcements, and that they provide an important increase in the stiffness characteristics of a structure. Triantafillou and Deskovic (1991) [7] developed this methodology analyzing the effects obtained by pretensioning the composite sheets before gluing them on the tension face of the beams. They also proposed to apply this technique to increase the strength of existing wood structural members. Before this, all the studied techniques were applied only in the case of concrete and reinforced concrete structures.

Plevris and Triantafillou (1992) [8] focused on the development of a fundamental understanding of the behavior of the fiber reinforced wood systems. They examined the effect of external bonding of composite materials on the failure mechanisms, the stiffness, and the ductility of the hybrid member subjected to a combination of bending and axial loads. Thanks to an incremental numerical procedure they concluded that even very small area fractions of fiber-composite reinforcement result in significant improvements of the member's mechanical behavior. Because this technique allows the structural recovery of also very deteriorated wood, it represents a very interesting possibility for all these country where old and historic monuments are common, which is the case of Italy.

In this section the results of a systematic series of tests on reinforced and non-reinforced wood elements are presented. Three different types of wood, different types of fiber reinforced composite materials (FRP), and samples having different geometry have been used. Samples realized using a technique allowing a pretensioning of the composite sheets have been also tested. The results obtained demonstrated that external bonding of composite materials on wood elements, results in significant improvements of the member's mechanical behavior.

3.2 Materials and experimental procedures

3.2.1 Materials

Chestnut wood (Castanea Vesca), oak wood (Quercus Cerris) and fir wood (Abies Alba) have been used for the experimentation. About chestnut and oak, very old woods obtained from ancient buildings situated in central Italy (Umbria), and classified as 1st type wood (wood having very low percentage of defects) [9], have been used. About fir, wood classifiable as 3rd type wood (wood having diffused defects) has been used.

The hygrometric conditions of these materials have been measured according to the UNI 9091/2 standards [10]; the humidity percentage resulted equal to 9%, 11% and 15% respectively for chestnut, oak and fir woods. The density of these woods resulted respectively equal to 5.40, 8.98 and 4.24 kN/m^3.

The composite reinforcements have been realized using three different types of fibers produced by MAC spa, respectively carbon (CFRP), glass (GFRP) and aramidic fibers (KFRP), having the physical and mechanical characteristics reported in Table 2 as furnished by the producer. Depending on the wood used, small samples or large beams were tested. In the fist case 400 mm long 20 mm large fiber sheets have been used. In the second case 1900 mm long 70 mm large fiber sheets have been used. In the two cases, the thickness of the dry plies was 0.165, 0.118 and 0.070 mm respectively for carbon (CFRP), glass (GFRP) and aramidic fibers (KFRP).

TABLE 2: Mechanical characteristics of the reinforcements (as reported by the producer)

	Carbon Fibres	Aramidic Fibres	E Glass Fibres
Aerial density D_s, [Kg m^{-2}]	0.300	1440	0.300
Ply equivalent thickness, [mm]	0.165	0.070	0.118
Tensile Strength, [MPa]	3430	3150	1550
Tensile Young's modulus, [MPa]	230000	105000	75000
Strain at failure, %	1.5	3.0	2.1

In order to glue composite and wood, a system composed of two curing at room temperature epoxy resins, primer and saturant, has been used. The mechanical and physical characteristics of these materials are reported in Table 3 as furnished by the producer.

TABLE 3: Mechanical properties of the epoxy resins (as reported by the producer)

	Primer	**Saturant**
Tensile Strength (ASTM D638)	>12 MPa	>50 MPa
Tensile Strength from bending test (ASTM D790)	>24 MPa	>120 MPa
Compression Strength (ASTM D695)	-	>80 MPa
Elongation at break, % (ASTM D638)	3.0	2.5
Flexural Young's modulus (ASTM D790)	>580 MPa	>3500 MPa
Tensile Young's modulus (ASTM D638)	>700 MPa	>3000 MPa

The gluing procedure consisted, as a first step, in straining the primer on the wood surface where the reinforcement had to be placed. After the partial polymerization of this layer, the saturant has been strained as a second layer. Then the fibers have been put as a third layer and have been covered by other saturant. The complete polymerization of the saturant has been finally attempted. The complete process has been performed at 20 °C and took around seven days.

3.2.2 Experimental Procedures

Mechanical tests on not reinforced samples, adhesion strength tests, and mechanical tests on small and on real size (beam) reinforced samples have been performed. In the case of chestnut and oak woods, the mechanical properties of the woods have been determined performing compression and bending tests according to the UNI ISO 3132 and UNI ISO 3349 standards respectively [11,12]. In the case of fir wood, four 2000 mm long, 100 mm thick and 100 mm large beams have been tested in bending loading them in four points over a span of 1950 mm.

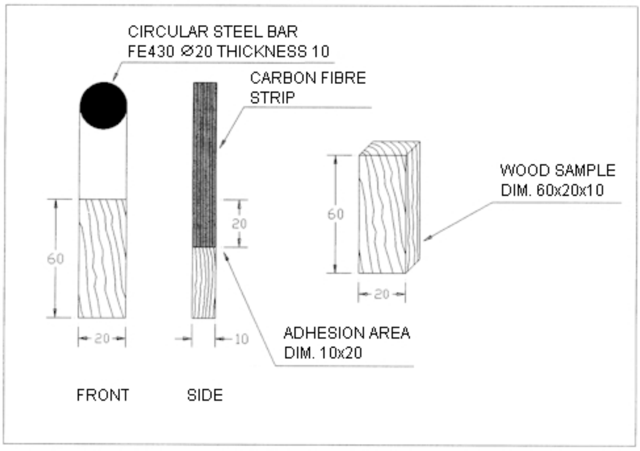


FIGURE 1: Sample geometry of adhesion strength tests

Adhesion strength tests have been performed on samples realized bonding small carbon and glass reinforced composite laminates on 60 mm long, 20 mm large, 10 mm thick wood elements. In the case of carbon fibers, a 20x10 mm bonding area has been used, but in the case of glass fibers, in order to obtain the crisis of the adhesion surface before the crisis of the fibers, it has been necessary to reduce the gluing surface up to 10x10 mm. The described sample geometry is illustrated in Figure 1.

Adhesion strength tests have been realized in speed control mode, imposing, up to the adhesion crisis, a constant displacement rate equal to 0.2 mm/min. These tests involved the use of only the 1st category woods (oak and chestnut), free of knots and other defects. Five samples for each wood – fiber combination have been used.

As in the case of not reinforced woods, bending tests on reinforced woods have been performed using two different types of samples depending on the wood used. In the case of chestnut and oak wood, 350 mm long, 20 mm large, 20 mm thick samples have been used. They have been reinforced with thin carbon, glass and aramidic composite layers positioned in their bottom part (the side stressed in tension). The samples have been tested according to the UNI ISO 3349 standard. Young's modulus and strength have been measured.

Because the three reinforcing materials employed had different thickness, the area ratios, defined as the ratio between the wood section and the reinforced section, were also different. Thus, the obtained results were not directly comparable [8].

$$\rho_f = \frac{tb_f}{bh} \tag{1}$$

Where t and b_f are respectively the thickness and the width of the fiber and h and b are respectively the height and the width of the cross section of the wood sample.

In the case of fir wood, twenty two 2000 mm long 100 mm large 100 mm thick beams have been reinforced in different ways and tested in bending: Eighteen samples (six for each kind of fiber) have been strengthened with external composite reinforcements on the sides loaded in tension. For each kind of used fiber, four beams have then been wrapped up with small composite strips as shown in Figure 2. This further reinforcement, which can be realized only with some difficulty in the case of put in place structures, has been realized in order to avoid peeling and other problems that can happen in case of not perfect fiber-wood adhesion.

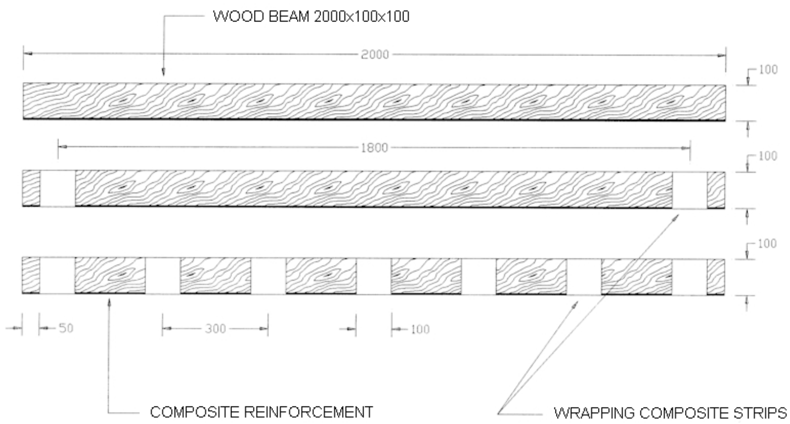


FIGURE 2: Fir wood beam samples

The samples have been tested in four-point bending, over a span of 1950 mm, using roller supports having diameters equal to 300 mm. Again, because the three reinforcing materials used had different thickness, the results obtained were not directly comparable [8].

Except the case of bending tests on beam samples, for which a GEO E 05 manometer produced by NUOVA FIMA has been used, mechanical tests have been performed on a Lloyd instruments LR 30K dynamometer, using the controlling and measuring system R-Control Lloyd.

3.3 Results and discussion

About mechanical tests, the characteristics of the used woods were first of all analyzed. The results obtained with the tests on not reinforced wood samples, illustrated in Table 4, show that Oak and Chestnut woods have very similar mechanical properties. On the other hand Fir wood is characterized by lower modulus and strength.

TABLE 4: Mechanical properties of different wood types used

	Chestnut wood	Oak wood	Fir wood
COMPRESSION TESTS			
Number of tested samples	10	10	10
Load velocity [mm/min]	2	2	2
Young's modulus [MPa]	11900	12650	4538
Failure stress [MPa]	50.10	55.40	32.70
Sample dimensions [mm]	60x20x20	60x20x20	60x20x20
BENDING TESTS			
Number of tested samples	30	30	4
Young's modulus [MPa]	5992	6760	6639
Failure load [N]	1844	2164	14528
Sample dimensions [mm]	20x20x350	20x20x350	100x100x2000

The results of the adhesion strength test, illustrated in Table 5, show that the crisis of the adhesion may occur in two different ways, depending on the direction of the wood fibers with respect to the direction of the composite ones.

When the composite fibers were tangent to the wood fibers, in fact, the crisis happened into the wood fiber: the composite remained attached to the wood material and the sample failed for the delaminating of the wood fibers. In this case the measured tensions resulted equal to around 7 MPa, with not measurable differences for the different kinds of wood and of composite. When the composite was glued perpendicularly to the wood fibers, on the other hand, the crisis occurred at higher tensions (around 10 MPa), and it provoked the separation of wood and composite. Also in this last case the values of the joint strength did not depend on the different kinds of fiber and wood involved.

TABLE 5: Adhesion test results

	Chestnut wood	Oak wood
Type of reinforcement	CFRP	CFRP
Number of tested samples	5	5
Load speed [mm/min]	0.2	0.2
Adhesion surface [mm]	20x10	20x10
Failure stress [MPa]	9.76	10.20
Wood sample dimensions [mm]	60x20x10	60x20x10
Type of reinforcement	GFRP	GFRP
Number of tested samples	5	5
Load speed [mm/min]	0.2	0.2
Adhesion surface [mm]	10x10	10x10
Failure stress [MPa]	8.32	9.13
Wood sample dimensions [mm]	60x20x10	60x20x10

The results obtained with the bending tests on small-reinforced samples (chestnut and oak wood, 350 mm long, 20 mm large, 20 mm thick samples) are reported in Table 6. The carbon fiber produced the highest increase of strength: 88% and 83% respectively for chestnut and oak wood. This result was not unforeseen, in fact the thickness of this reinforcement was bigger than in the case of glass fibers, and carbon fiber strength and modulus are higher than glass and aramidic ones.

TABLE 6: Wood FRP samples results.

Wood type Sample dimensions [mm]	Chestnut wood 350x20x20	Oak wood 350x20x20
Reinforcement	CFRP	CFRP
Number of tested samples	10	10
Area fraction of fiber composite ρ	0.00825	0.00825
Stiffness increment (%)	+28.9 %	+22.8 %
Failure load [N]	3474	3964
Failure load increment (%)	+88.4 %	+83.2 %
Reinforcement	GFRP	GFRP
Number of tested samples	10	10
Area fraction of fiber composite ρ	0.00590	0.00590
Stiffness increment (%)	+27.9 %	+16.5 %
Failure load [N]	3054	3405
Failure load increment (%)	+65.6 %	+57.3 %
Reinforcement	KFRP	KFRP
Number of tested samples	10	10
Area fraction of fiber composite ρ	0.00350	0.00350
Stiffness increment (%)	+14.9%	+13.9%
Failure load [N]	2822	3133
Failure load increment (%)	+53.0%	+44.8%

In Figure 3 the behaviors of the load-deflection curves of two different samples are reported: the sample was not reinforced while the second was reinforced with carbon fibers. As one can see, the two curves are both at the beginning near the linear behavior. In the case of not reinforced sample, the curve remains in the elastic field and not yielding point is evident up to the failure point. In the case of the reinforced sample, on the other hand, the yielding point is evident: after this point the slope of the curve is decreasing because of wood plastic deformation. When the composite reaches its crisis point, the sample fails in the zone loaded in tension.

The way in which the crisis occurs is linked to the wood used, and in particular to its strength, compression behavior and tension behavior. In general, if loaded in compression, after having reached the maximum load, wood has a residual strength of 70-80%. But in tension the material breaks in fragile way. During the experimental work, the first signs of the crisis have been normally noted in the compressed zone, where small wrinkling appeared if the load was sufficiently high. But because the wood can plastically deform if it is loaded in compression, the samples failed at the side loaded in tension. In particular, the failures started at the sides, which were loaded in tension.

Regarding the stiffness increase because of the reinforcement, it can be noted that it has in general been not very high. As showed in Figure 4, it is possible to obtain an increasing of the stiffness of the samples, simply pre-loading them. The curve represented in Figure 4, in fact, shows the bending behavior of a not reinforced sample, together with the behavior of a sample made gluing the composite reinforcement when the wood sample was under flexural load applied in the direction opposite to the test one (the flexural load was equal to 0.25 times the ultimate load of the not reinforced sample). As one can see, in the second case the increase of the stiffness has been equal to 80%, while in the first one it has only been equal to 28% (see Table 6, Chestnut wood, for CFRP and GFRP fibers). All this without affecting the sample's strength. But more tests should be necessary in order to better qualify such type of behavior.

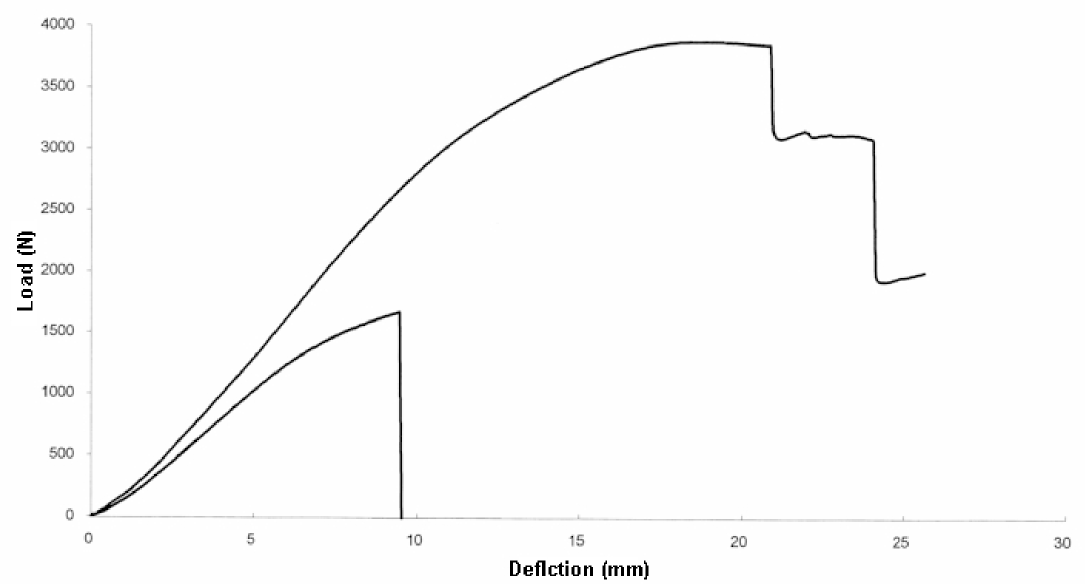


FIGURE 3: Typical bending behavior of a un-reinforced and a reinforced sample

The results obtained with the bending tests on large reinforced samples (fir wood, 2000 mm long, 100 mm large and 100 mm thick beams) are reported in Table 7. These results confirmed the data obtained in the case of small samples in terms of strength and Young modulus increasing because of the reinforcement.

The experimental results show that the composite strips used to wrap up the beams did not determine further strength or stiffness. This is because the gluing system is sufficiently good to guarantee an optimum adhesion without the use of wrapping strips or similar techniques.

TABLE 7: Results of bending tests on large samples

Reinforcement material	KFRP	GFRP	CFRP
Sample dimensions [mm]	2000x100x100	2000x100x100	2000x100x100
Wood type	Fir	Fir	Fir
Number of tested samples	6	6	6
Area fraction of fiber composite ρ	0.00056	0.00118	0.00165
Stiffness increment (%)	+4.2%	+29.7 %	+48.4 %
Failure load [N]	18524	19437	24502
Failure load increment (%)	+27.5%	+33.8 %	+68.7 %

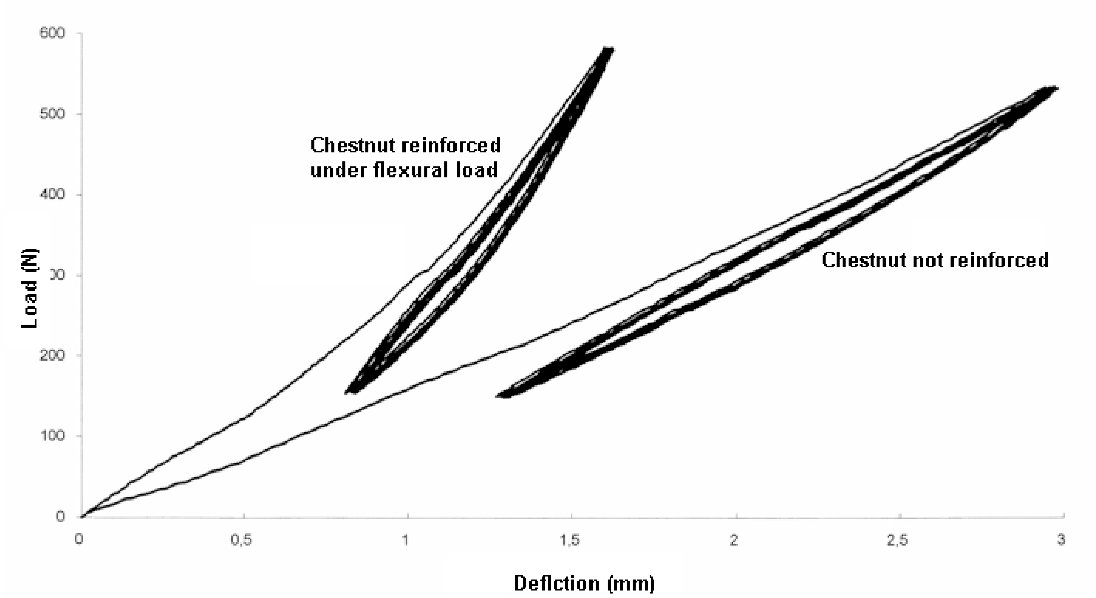


FIGURE 4: Bending behavior of a not reinforced sample and of a sample made gluing the composite reinforcement when the wood was under flexural load

4. Innovative materials applied on masonry structures

A considerable amount of research has been conducted in the past to improve the mechanical behavior of masonry structures using a variety of reinforcement techniques. The shear reinforcement of walls was realized by employing metallic bars, or injecting lime or cement based mortars.

In recent years, reinforcing concrete structures by the wrapping and bonding of Fiber-Reinforced Polymers made of sheets, strips, belts or procured shells has became increasingly popular, but limited research has been conducted on the use of FRP reinforcement in masonry structures. In countries such as Italy, the use of brickwork as a building material has prompted a wide range of studies to investigate its structural behavior in a variety of applications. As an example, Di Tommaso and others first applied FRP in order to upgrade the seismic behavior of bell-towers [13]. This paper contains an abstract of the most part of research carried out by the authors during the years following the Umbrian-Marchigiano earthquake of 1997. References are reported at the end of this paper.

4.1 Reinforcing of masonry structures by the wrapping of FRP

Fiber wrapping, or encasement of masonry buildings in fiber-reinforced polymers (FRP) shells, may significantly enhance the strength and ductility of masonry structures. Most masonry walls are not correctly connected to each other, and this makes these structures particularly vulnerable to seismic action. The walls orthogonal to the direction of the seismic action often collapse following out-of-plane mechanisms.

In order to connect masonry walls different procedures have been introduced during the last decades. The most popular technique consists of the realization of a reinforced concrete ring beam along the perimetrical walls. Ten masonry panels were assembled to form two masonry cells: six of the them had a masonry texture of roughly cut calcareous stones with lime-based mortar (panel thickness 50 cm), while the remaining four panels had a texture made up of hollow bricks (panel thickness 25 cm). The complete results of this experimental research are reported in [14]. The walls were constructed using lime based mortar and sand in volume ratio 1:2. The height of the walls from the base is 150 cm and the length is 90 cm.

FIGURE 5: The containing action of FRP sheet

Preparation of the wall surface for the strengthening operation is a quick procedure, consisting of the use of a grinder to remove the external plaster. The application of CFRP reinforcement is carried out after having spread an epoxy-primer and an epoxy-mortar (putty) on the surface of panels. Two bands of CFRP are applied along the external perimeter in order to wrap the masonry cell with one band near the base and with another near the upper border. A hydraulic jack was introduced inside the masonry cells at the level of the upper FRP sheet in order to stress the masonry panels The masonry cells were loaded monotonically up to failure with load increments of 10 kN. In order to distribute the load, two metallic T shapes were introduced between the masonry panels and the hydraulic jack.

The solid brick and the stone masonry cells failed respectively at a load of 70 kN and of 90 kN. The low flexural stiffness of the CFRP sheet caused large lateral deformations at the maximum load. The experiment showed that the FRP reinforcement maintains its effectiveness until the crushing strain of masonry is attained. A significant consequence of the presence of the FRP reinforcement was the absence of the out-of-plane collapse of the masonry wall. The FRP sheet was in fact able to contain the stones and bricks resulting from the crumbling of the masonry wall (see Fig. 5).

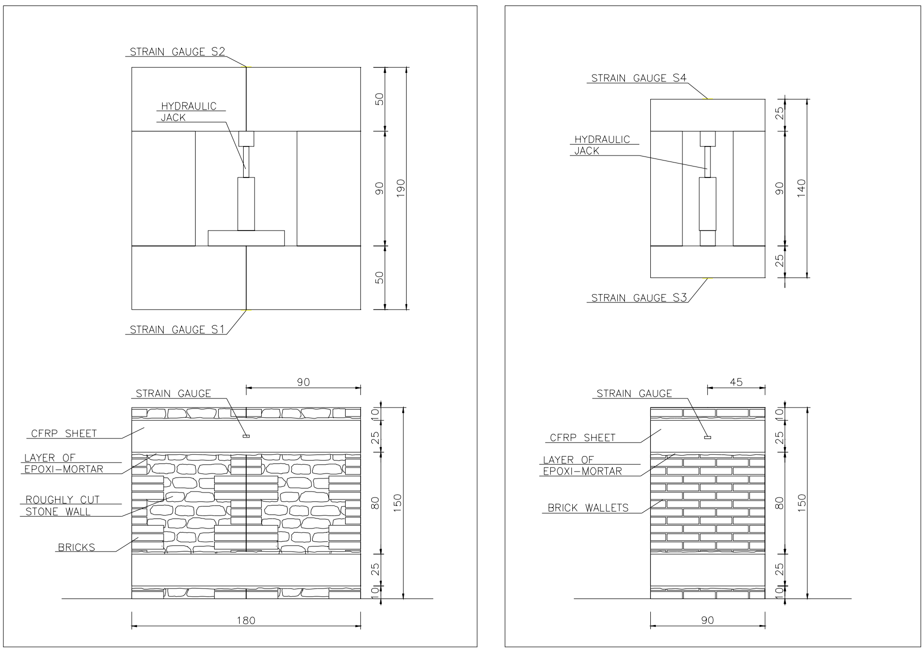


FIGURE 6: Layout of masonry cells with the position of CFRP sheets

The process of the crumbling of the masonry panel started long before the maximum load was reached. At the maximum load, large deformations were measured highlighting the high ductility characteristics of the masonry reinforced with CFRP polymers. The use of the two CFRP sheets placed one upon the other (the total number of sheets used is four, considering that two bands of two sheets were placed around the masonry cell) did not cause problems at the four angles.

Several strain gauges were fixed along the FRP sheets in order to measure the stress tensile status of the composite (see Fig. 6). This allowed us to confirm that the FRP reinforcing technique carried out its strengthening action, as well as to measure the degree of utilization of the FRP. However, the abrupt variation in curvature occurring at the four angles of the masonry cells should be avoided. CFRP sheets have a high degree of weakness due to the onset of stress at right angles to the fibers. In this case, the problem was partially solved thanks to the introduction of steel L shapes characterized by a radius of curvature of 2 cm.

Moreover, the use of L shapes with a height greater than the height of the CFRP sheet causes a better distribution of compressive stresses on the masonry surface and in part avoids the masonry crushing at the angles of the cell (see Fig. 7).

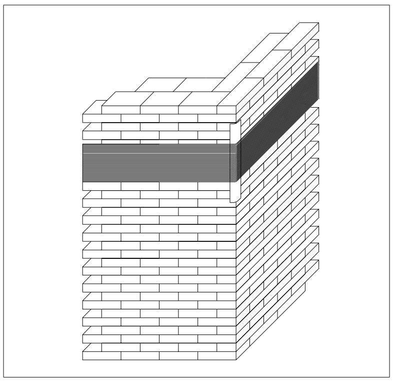
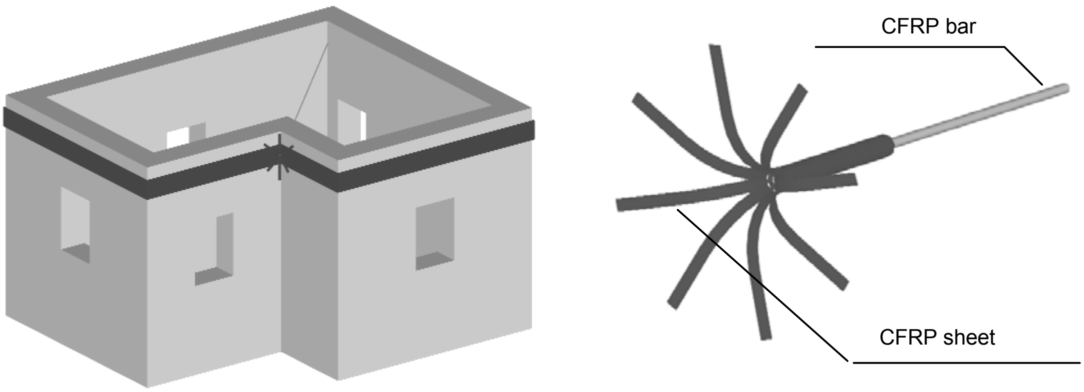


FIGURE 7: The use of metallic L shapes in order to distribute stresses along the masonry

However the effect of using CFRP strengthening on corrosion confinement of steel elements should be studied and carried out with care. Many researchers have attempted to characterize the performance of corrosion damaged RC structures reinforced with CFRP sheets, showing the importance of the resin layer in separating steel and carbon reinforcements.



FIGURES 8-9: A proposed solution to the problem of non-rectangular buildings. Research is in progress in order to connect different FRP materials used for seismic upgrading of masonry structures

The use of FRP to wrap masonry buildings may be a provisional or a definitive intervention. Even in the case of a provisional use the advantages are significant, considering the rapidity with which it can be realized and the possibility of putting in security damaged masonry structures.

This technique may also be applied to non-rectangular structures. In these cases a possible solution is shown in Figures 8 and 9: a tendon composed of a CFRP bar (or a metallic one) is connected, by means of FRP strips, to the external FRP band in order to avoid debonding of the FRP sheet from the surface of the masonry structure. The connection of CFRP bars to orthogonal sheets is now studied and preliminary results indicate its effectiveness.

4.2 Shear reinforcement of masonry walls

Various kinds of intervention may be carried out to upgrade a wall. Currently, the most widespread technique consists of injecting lime-based grouts or fixing a metal net on masonry surfaces under a concrete jacket. Borri, Corradi and Vignoli applied this technique in order to upgrade several two-leaf and three-leaf walls. Other authors applied different techniques in order to seismic upgrade masonry panels [15] [16]. The results substantially showed that while the injection technique can be effective when used as a repair technique, its adoption as a preventive measure necessitates a previous, careful analysis of the masonry texture and of its characteristics, in order to understand if the grout can, in effect, penetrate it well.

The use of FRP on similar walls was also studied. These walls were tested under diagonal compression and shear-compression. This involved the use of panels of various dimensions. Five of ten panels were tested without strengthening, to determine the mechanical characteristics of the masonry. The remaining five was consolidated respectively with monodirectional carbon fibers and monodirectional fiberglass. Strengthening with one sheet of monodirectional carbon fiber or fiberglass was carried out on both sides of the panel, following the scheme presented in Figure 11. Due to the great irregularity in the masonry texture, the strengthening of the panels was carried out only after spreading a layer of cement based mortar to create a sufficiently plane, uniform surface on which the composite was then applied. The mortar used was placed directly on the masonry once the original plaster had been removed

4.2.1 Diagonal Compression Tests

The diagonal compression test was carried out on panels 120 x 120 cm (according to ASTM specifications [17]) with cross-sections of varying thickness, depending on the structure on which the intervention was affected. The aim of this test was to determine the shear-strength and the shear elastic modulus of the masonry. The panel was isolated from the rest of the masonry by means of the diamond wire cutting technique in order to leave it undisturbed (see Fig. 10).

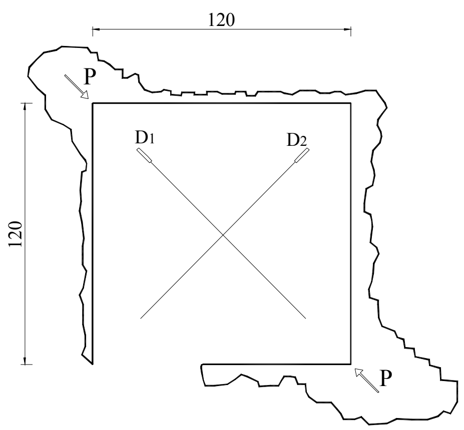


FIGURE 10: Layout of the diagonal compression test

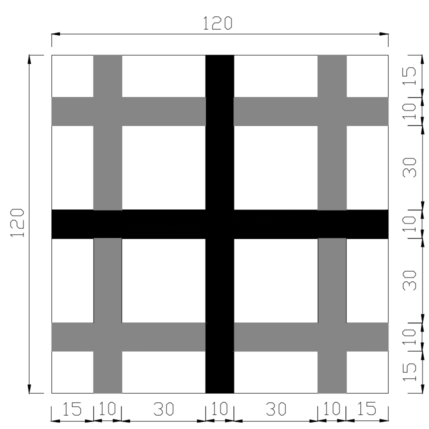


FIGURE 11: A masonry panel strengthened with CFRP

The test mechanism was composed of a set of metallic elements joined together to obtain a closed system. The stress was applied to the panel by means of a compression load, coplanar to the panel itself and directed along one of its two diagonals. A jack placed along the diagonal, external to the panel, was positioned between two metallic elements which permitted it, on the one hand, to act directly on an edge of the panel, while at the same time to remain rigidly connected to an analogous metal element located along the opposite edge. In correspondence to this edge the situation was analogous, except for the absence of the jack: the two most external metal elements were rigidly connected by means of two steel rods of equal length to allow a uniform distribution of the load throughout the thickness of the panel. The panel was instrumented on both sides with LVDT inductive transducers to measure the deformations occurring along the four diagonals. It was therefore possible to acquire the strains during the test as a function of time and the value of the load applied to the jack. The test procedure consisted of loading and unloading cycles of increasing maximum values, until the point of failure was reached. The load was applied following a series of equal couples of cycles, with increases of 10 kN, up to the point of failure. In the case of strengthened panels with failure loads greater than 80 kN, the interval between one couple and the other was increased to 20 kN.

The analysis of the results of the diagonal compression tests is the object of interpretation differing from Author to Author. This test was introduced to simulate a pure shear stress state. In these conditions the Mohr circle of the stress state is centered in the origins of σ–τ axes and the value of the average shear stress τ , equal to the principal tensile stress σ_I, is given by:

$$\tau = \sigma_I = \frac{P}{A\sqrt{2}} \tag{2}$$

in which P is the diagonal compression load, A the cross-horizontal section of the panel and τ is the average shear stress in the panel. According to this interpretation, which is that more frequently used [18], the shear strength is evaluated as:

$$\tau_k = \frac{P_{\max}}{A\sqrt{2}} \tag{3}$$

Where P_{max} is the maximum compression load applied during the diagonal compression test and τ_k is the average shear strength.

Both the shear modulus of elasticity $G_{1/3}$ and the shear strain $\gamma_{1/3}$ are measured at 1/3 maximum load. The value of shear modulus of elasticity $G_{1/3}$ is given by the slope of secant line matching the shear stresses τ_i and $\tau_{1/3}$.

$$G_{1/3} = \frac{\tau_{1/3} - \tau_i}{\gamma_{1/3}} \tag{4}$$

Where τ_i is the shear stress at $\gamma = 0$, $\tau_{1/3}$ is the shear stress at 1/3 maximum load, $\gamma_{1/3}$ is the shear strain at 1/3 maximum load.

$$\gamma = |\varepsilon_c| + \varepsilon_t \tag{5}$$

Where ε_t and ε_c are the average normal strain of the four diagonals in compression and in traction.

4.2.2 Shear-Compression Tests

The shear-compression test was carried out on panels of dimension 180x90 cm, with a maximum section thickness of 65 cm. Eight transducers were placed along the diagonals of the four square surfaces, obtained by subdividing each of the two vertical sides of the panel in two equal parts. Three additional transducers were positioned on each side - to measure transversal movements - along the centerline, and at the top of the panel. Two other transducers were placed vertically at the edge of one side of the panel to measure eventual rotations at the top of the panel. The shear-compression tests, analogous to the diagonal compression tests, were designed to determine the shear strength and shear elastic modulus characteristics of the masonry. The test consists of a monotonic loading up to the point of failure, under a state of constant compression for the duration of the test. The compression stress was applied up to a vertical stress of 0.3 MPa.

The horizontal shear force was applied by two steel rods, which acted on a special metal element positioned at the centerline of the panel. This element consists of two C shapes, coupled with plates welded to the webs, whose function is to distribute the concentrated load throughout the panel thickness. An oleo-dynamic jack is interposed between an I shape, used as a contrast element (positioned parallel to the height of the panel at a lateral opening in the masonry), and a metal beam similar to that at the centerline of the panel. The actuator loads, on one side, the I shape, whose job is to distribute the load concentrated at the base and at the top of the opening, and on the other side, the metallic element to which are attached the two steel ties which thus result in tension.

The upper part of the panel was contrasted by means of a couple of 100 kN jacks positioned horizontally, in order to avoid flexural failure mechanisms. This couple also allowed measurement of the horizontal reaction at the top of the panel. The condition of double bending results only in the hypothesis that the upper edge is prevented from translating and rotating.

This condition is in part supplied thanks to the presence of the jack couple, blocked on the load assigned, and of the horizontal plate placed on the panel. Nevertheless, the stiffness of the apparatus at the top of the panel and the vertical rods is not sufficient to act as a perfect constraint, as requested. This allowed a lack of symmetry in the distribution of the shear stresses between the upper and lower halves of the panel, which was taken into account during the elaboration of the data.

In addition to the displacements of the sixteen inductive transducers, measurements were also acquired of both the time and the pressure in the two vertical and three horizontal jacks, for a total of twenty-one channels of acquisition.

During the shear-compression test, the failure condition occurs when the principal tensile stress σ_I in the center of the panel is equal to the tensile strength of the masonry. In order to evaluate the shear strength of the masonry, the well-known Turnsek and Cacovic formulation is assumed:

$$\tau_{\max} = \tau_k \sqrt{1 + \frac{\sigma_0}{b\tau_k}} \tag{6}$$

Where $\tau_{\max}$ is the shear strength, τ_k is the shear strength without normal stress and σ_0 is the compressive stress. b is a shape factor that takes into account the variability of the shear stresses on the horizontal section of the wall. The Italian Standards and the well-known POR method assume this parameter equal to 1.5.

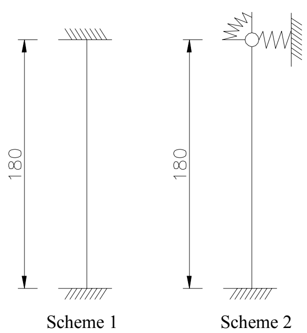


FIGURE 12: The two schemes used for elaboration of the data of the shear-compression test.

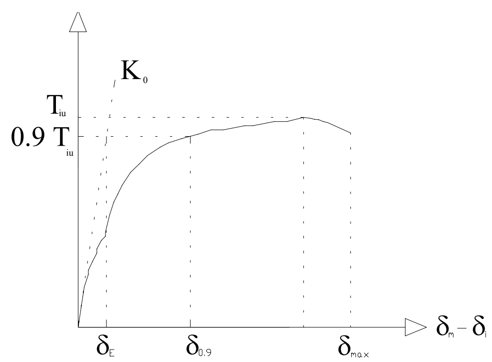


FIGURE 13: Procedure applied to calculate δ_E

With regard to the Scheme 1, shown in Figure 12, the shear modulus of elasticity G is given by:

$$\frac{1}{K_0} = \frac{\delta_E}{0.9T_{iu}} = \frac{1.2h}{GA}\left[1 + \frac{G}{1.2E}\left(\frac{h}{d}\right)^2\right] \tag{7}$$

Where:

T_{iu} is the maximum shear load in the lower halve,

h is the height of the lower halve panel,

E is the Young's modulus of elasticity obtained from compression tests,

d is the width of the panel,

δ_E is evaluated as showed in Figure 13,

A *is the horizontal cross-section of the panel.*

4.2.3 Evaluation of the Results

Eight, of the ten panels, were double-leaf masonry walls, made of weakly connected roughly cut stone, while two were of solid brick. The cross section thickness of the panels in stone also turned out to be more or less constant with values varying from 48 to 57 cm, while differences of properties were noted in the mortars (all lime-based), depending also on the period in which the building was constructed.

For the diagonal compression tests the interpretative models of structural behavior are the classic ones [17], while for the shear-compression tests Sheppard's model [19] is used, taking into account hypothesis and interpretative choices as in [20].

The results of shear-compression tests (see Tab. 8) show an increase in the strength, τ_k, of the strengthened panels, relative to those non-strengthened, of approximately 55%, independent of the type of fibers used. This essentially depends on the particular modality of failure, which did not involve the crisis of the composite, except at a limited area on some of the panels. In fact, the fibers remained attached to the cement-based mortar (used to create a uniform surface on which the FRP was applied), which in turn had detached from the masonry. Therefore, since the state of fiber stress was well below its failure level, it was only marginally engaged. For this reason, no difference resulted from the use of carbon fibers or fiberglass. However, in the successive diagonal compression tests the layer of mortar was replaced by a thinner layer of epoxy putty, which bonded more efficiently with the masonry substrate and determined a better utilization of the fibers, which did not detach until the masonry-composite system failure was reached.

TABLE 8: Results of the shear compression tests (static scheme 1: double fixed end; static scheme 2: fixed end and fixed end with determinate stiffness to 2 D.O.F.: horizontal translation and rotation in the panel's plane)

Index Code	*Strength technique*	*Texture*	*Section [cm]*	P_{max} *[kN]*	τ_{max} *[MPa]*	*G [MPa] Scheme 1*	*G [MPa] Scheme 2*	τ_k *[MPa] b=1.50*
B-T-04-OR	None	double-leaf roughly cut stone masonry	48	180.70	0.219	546	328	0.130
B-T-05-FC	CFRP	double-leaf roughly cut stone masonry	48	241.40	0.352	467	771	0.262
V-T-06-FV	GFRP	double-leaf roughly cut stone masonry	48	231.30	0.334	245	249	0.245
P-T-15-OR	None	double-leaf roughly cut stone masonry	48	100.40	0.172	216	-	0.136

TABLE 9: Results of the diagonal compression tests.

Index Code	*Strength technique*	*Texture*	*Section [cm]*	P_{max} *[kN]*	τ_k *[MPa]*	$G_{1/3}$ *[MPa]*	$\gamma_{1/3}$
B-D-02-OR	None	one-leaf solid brick masonry	48	34.31	0.069	131	0.136
B-D-03-FC	CFRP	one-leaf solid brick masonry	48	188.25	0.373	100	1.240
G-D-11-OR	None	double-leaf roughly cut stone masonry	57	51.14	0.053	26	0.643
G-D-12-FC	CFRP	double-leaf roughly cut stone masonry	57	121.53	0.127	55	0.699
P-D-13-OR	None	double-leaf roughly cut stone masonry	48	47.66	0.059	37	0.533
P-D-14-FC	CFRP	double-leaf roughly cut stone masonry	48	141.61	0.173	117	0.497

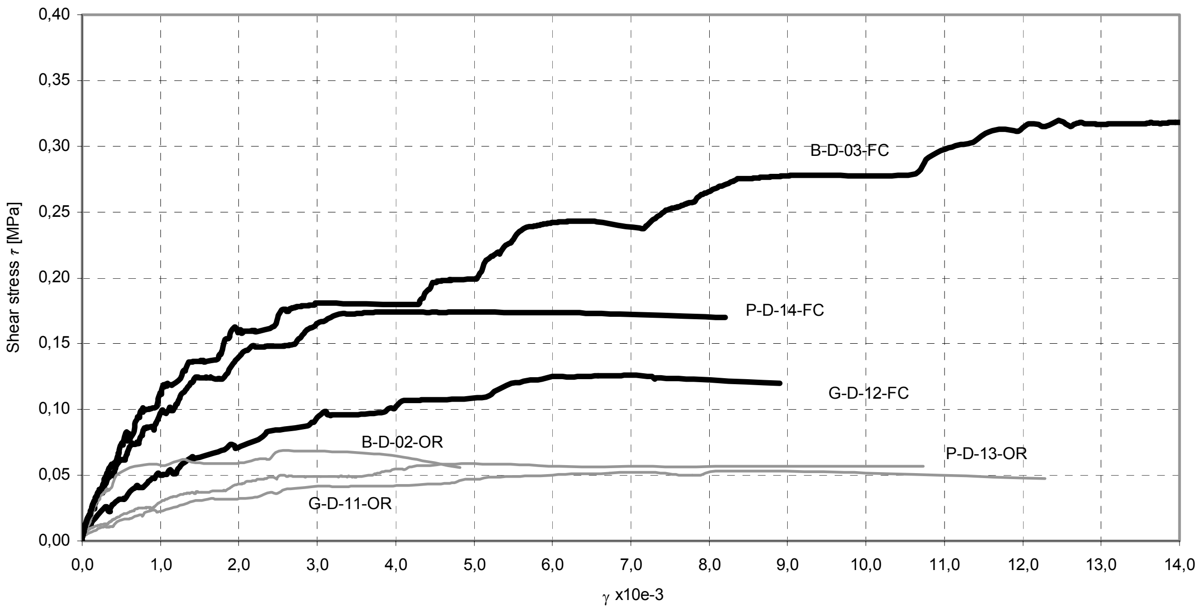


FIGURE 14: Diagonal compression tests (gray lines: panels un-strengthened, black lines: panels strengthened with CFRP)

For diagonal compression tests, the use of epoxy putty in place of the cement-based mortar determined an increase in shear strength of 240% compared to the non-strengthened panels. The masonry panel(code: B-D-02-OR), made of solid brick, presented a notably low failure stress of 0.069 MPa for the non-strengthened panel (see Tab. 9). Instead, the masonry-composite system of the panel strengthened with CFRP was shown to be very resistant, with a failure stress of 0.373 MPa, thanks to the effect of confinement the fibers induced on the masonry as well as to the presence of diatons, which acted as connectors. At the failure load, the masonry fractured in compression, determining the expulsion of small portions of brick from the panel, while the CFRP applied on a thin layer of epoxy putty remained attached to the panel

The results for the shear elastic modulus G, measured following ASTM specifications at 1/3 maximum load, were between 26-131 MPa for non-strengthened panels subjected to diagonal compression tests. Strengthening by means of fibers in composites does not result in sharp increases in the shear elastic modulus: panels for diagonal compression tests, strengthened with carbon fibers, resulted stiffer, however, with average values of approximately 250%, while in one case there was a practically negligible increase. The composite does not alter the static scheme of the structure nor does it determine major redistribution of stiffness (above all for fiber glass), but mainly shows up as an increase in the shear strength of the masonry by means of the mobilization of the fiber traction strength.

Comparisons in the behavior of panels subjected to diagonal compression tests are shown on a τ–γ graph in Figure 14. Both the similarity in behavior among the non-strengthened panels, as well as the significant increase in shear strength of the strengthened panels, can be noted. This latter aspect turns out to be particularly evident for the injected panels.

4.3 Seismic upgrading of masonry vaults or arches

In general vaults adapt well to variations in geometrical configuration, being able to distribute deformations along mortar joints without significant cracks forming and a behavior that may be partly compared to that of the isostatic structures. The onset of damaging kinematic motions is not so much conditioned by the exceeding of the limits of strength of the materials, as by the incapacity of the constraints to counteract the actions transmitted by the vault. In many cases the structural damage is therefore due to indirect causes. The pressures exerted against the walls produce the reciprocal moving away of the imposts or springers, something which generally happens through a rotation of the external walls and sagging of the keystone. A somewhat frequent type of damage is distinguished by the formation of cylindrical hinges in the sections at the haunches to the extrados and at the intrados or soffit crown, where the concentration of stress leads to the plasticitation of the material and the consequent accentuation of deformation. Rotation of the voussoirs around the haunches results in their detachment from the walls and the formation of cracks, through which the filling material "crumbles". Worsening of damage may be effectively countered with suitable work, while the correction of the deformative state is more difficult despite the fact some valid strengthening techniques have as their operating strategy the restoring of the original geometrical configuration. To recover the profile of the crown, it is generally necessary to integrate the brickwork bond in order to compensate for the aperture caused by the moving away of the imposts. Other causes of damage are due to the differentiated yields of the support, which in the case of cross vaults are caused by localized thrust near the imposts. The sagging effect is shown in Figure 15, where the cracks around the intrados at crown are highlighted as well as at the extrados of the haunches opposite the corner involved in the shift.

4.3.1 Aseismic work with composite materials

This section gives the theoretical results and details of some work a site regarding the aseismic strengthening of historical vaulted structures. In particular, a working method is proposed based on the use of sheets of composite material (FRP), the outlines of which have been explained by the same Authors in previous publications [21] [22] and by other Authors in [23] [24]. The aim was to study, design and accomplish aseismic protective systems for vaulted structures without altering the static set-up. The reason was to maintain the capacity of the vaults to adapt to the settling of the load-bearing structure and to the deformation induced by seismic stress. It must be specified, however, that the proposed work cannot be applied to all situations; the type of fault shown by the vault being of crucial importance:

- Statically acceptable configuration: the profile of the vaulted structure is sufficient to limit the line pressures with an adequate margin of safety. The local cracks do not jeopardize overall operation, in that the vault is, in short, still structurally efficient.
- Statically unacceptable configuration: the deformations of the profile or profiles are accentuated to the extent that the static of the vaulted structure in critical conditions; in the case of an arched structure, this is equivalent to the risk of collapse.

The proposed method of operation is referred to the first case as well as to the situations in which the vaulted structure has no damage. Other indications which influenced the project decision and which led to cladding the extrados with FRP sheets were as follows:

- The need to work on the extrados avoiding any percolation of material (e.g. due to the presence of frescoes);
- The need to avoid introducing connections between the vault and the outer wall (reinforcing injections) in order not to alter the static operation of the vault;

- The need not to create superstructures over the vault aimed at relieving it of its static function;
- The need to replace the supports, if present, with similarly effective devices (low brick walls).

Regarding the thrusts, which the vault exerts on the walling, it should be noted that the cladding of the extrados with the fibers does not substitute classic operations (tie-rods on the haunches or abutments). There is, however, an exception to this rule if it is possible to encircle the vaulted premises externally at the height of the centre of thrust; a somewhat rare possibility due to the difficulties tied to a historical site. The following summarizing chart is given to describe the efficacy of the cladding with FRP sheets and the way in which their effect is combined with that of the tie-rods. The chart in Figure 15 presents some states of damage related to the mechanism of damage that caused it in the case of vaults with or without tie-rods. The forces indicate the prevailing effect of the earthquake responsible for amplifying the vertical loads and of appearance of horizontal forces in proportion to the weight of the vault and of superstructures. This stress obviously depends on the magnitude of the seismic accelerations at the base of the piers.

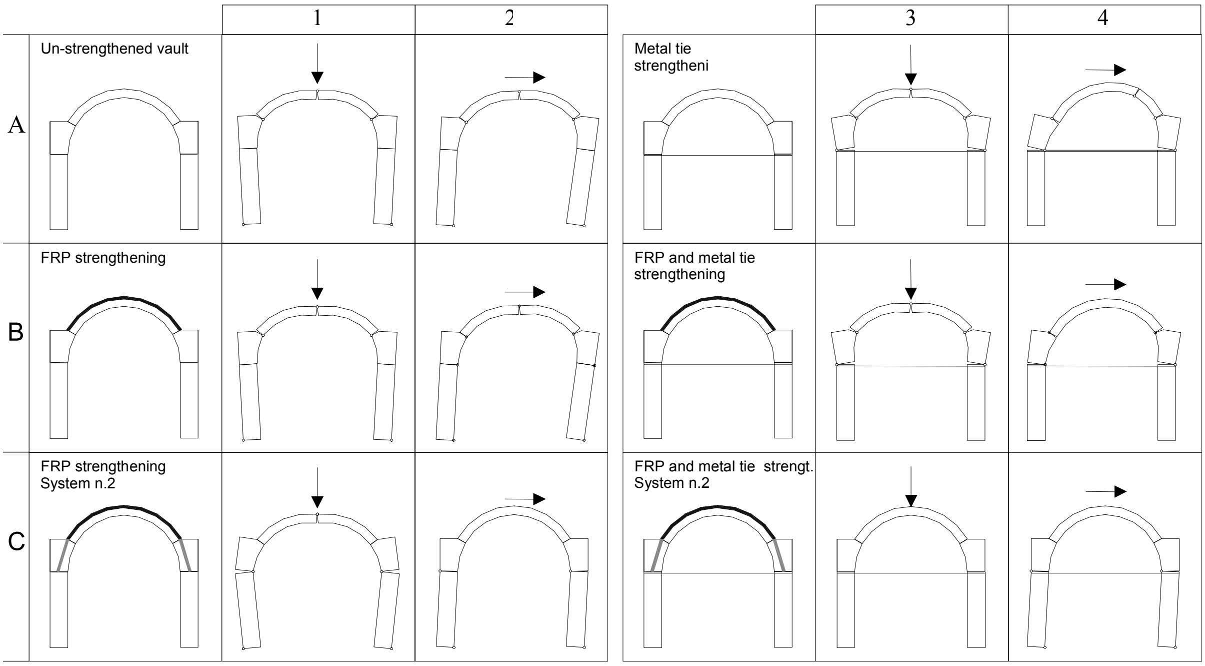


FIGURE 15: Collapse mechanisms for different strengthening systems

The bottom two lines show the fault in the presence of just and with sheets plus anchoring devices. It should be specified that the extrados sheets are only effective as aseismic protection with regard to important movements of the profile of the arch on which they rest.

4.3.2 Project criteria for work on cross vaults

The following overall indications may be given with regard to the design and the site operations. The first "shoot" of concrete should be high enough to reach the area of the arch haunches. The height of the closing jet and therefore the height of the slab are just above the haunches. It is advisable to use a double layer of sheets. A third layer is recommended on half-brick vaults with over 6-meter span. The ideal width for the outer sheets is 25 cm. A rod FeB44K-ϕ24 with threaded head is recommended for the anchorage. Once the fibers have been laid, a plug is made at the cross between the median lines of the two sheets with a common cutter. A hole of at least 4 cm of diameter is made and should be carefully cleaned. A suitable grout for the particular masonry is injected and then the rod or bar is inserted.

After a few days the head of the bar is fixed to the slab by means of a nut and load-distributing wedge. Torque wrenches are not required (see Figures 16-17).

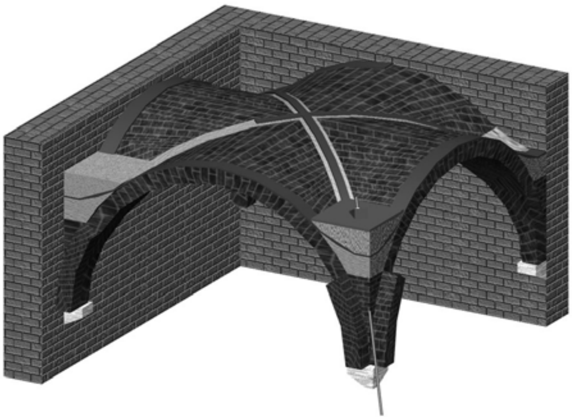


FIGURE 16: Position of FRP on the Assisi Town Hall cross vaults

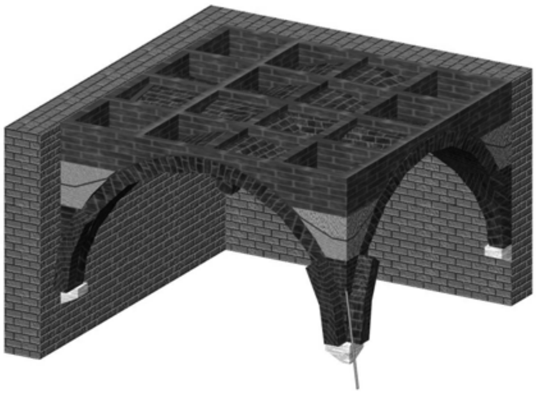


FIGURE 17: Intervention is completed with hollow-brick walls

4.3.3 A study case: seismic strengthening of the vaults of the Town Hall of Assisi

The design criteria described above are applied here to the vaults of the Town Hall of Assisi. This important complex, which covers a vast portion of the main square of Assisi, consists of many building units built at different times and joined together over the centuries. Unlike past work, which altered the spatial layout and therefore the actual load-bearing structures, this work was mostly concerned with seismic strengthening. To achieve this aim, an attempt was made to avoid any work, which would be particularly traumatic for the structure and in particular for the vaults, favouring the introduction of aseismic protective devices. Even though the complex is not organised according to what would today be called a correct engineering design logic (the various parts have been aggregated in a often chaotic manner), the current static set-up, as often happens in buildings in historical centres, is basically not negative thanks to the capacity of the walls to adapt to the different structural situations. In other words, the equilibrium that the structure has reached may be considered statically satisfactory, even if poor from an aseismic point of view. The building contains vaults of different types and different eras with problems tied to the bad state. The most frequent type is cross vaults, but there are also barrel vaults with lunettes. In the top floor rooms there are Welsh cavetto vaults both with half-brick thickness and with bricks laid on edge. Analysis of the damage produced by the Umbria-Marche earthquake of 1997 highlighted a particular weakness of the seismic actions on the brick-on-edge vaults and the half-brick vaults covering the reception rooms on the top floors. Less severe damage, but un-doubted worthy of aseismic engineering attention, was found on the cross vaults of the ground floor rooms, while no particular structural problem was obvious on the massive vaulted structures of the underground floors. The considerable variability of the different types of vaults in the complex, the presence of frescoed surfaces on the intrados of the vaults in the reception rooms and the particular site conditions resulted in a variety of structural situations for each of which *ad hoc* solutions were studied. Within this study, several cases are highlighted in this paper where the strengthening technique using sheets of composite material seemed preferable to “traditional” methods. A brief description of the type of vault and the damage is given for each work, followed by the criteria, which guided the project decision. Lastly, a list of the working stages is given with special attention paid to the site measures made necessary for correct results.

4.3.4 Application of FRP to the extrados of half-brick cross vaults

The work involved removal of the support up to the haunches, where the bricks of the arched lintel are inserted into the outer wall. A layer of concrete was created above these areas of insertion, with sufficient thickness to join the curvature of the outer arches of the vault with the area of the lintels. Since the vaults have frescoes on the intrados, a rheoplastic mortar with coarse sand as filler was used.

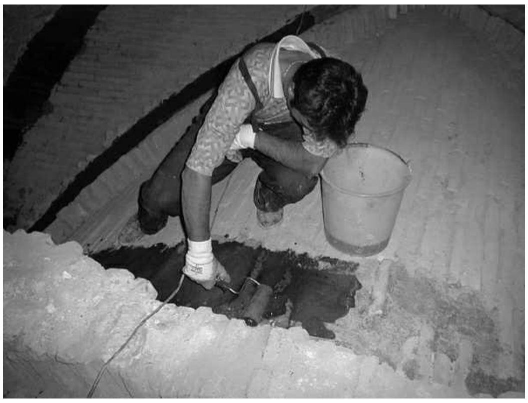


FIGURE 18: Primer application

FIGURE 19: The fixing operation at the extrados of the vault

After having thoroughly cleaned (by sanding and water cleaning) and then leveled the surface of the outer vault area, 4 bedding bands were created using suitable mortar. After the few days required for curing of the mortar, the surface was prepared with a suitable primer and the first monodirectional layer of FRP was laid with epoxy-resin. A second layer was also prepared (see Figures 18-19).

The reason for this does not lie in the strength of the material (one layer is more than enough in terms of resisting capacity), but in the greater reliability of the pack made up of two layers, which is far less sensitive to local effects connected with the irregularity of the laying surface. It should be noted, in fact, that despite careful preparation of the extrados of the vault, areas with abrupt variations in curvature may occur. In these cases experimental tests showed high degree of weakness of the individual monodirectional sheet due to the onset of stress at right angles to the fibers; the pack with two sheets instead proved far less sensitive to this effect, also thanks to the imperfect alignment of the direction of the fibers and due to the distribution effect exerted by a thicker layer of resin. After the sheets have been laid, a small amount of concrete was cast over each lintel, onto which a steel plate fitted with a wedge (also in steel) was placed (see Fig. 20). The latter was designed to house the anchoring rod, inserted diagonally, in order to reach the height of the springer. The work was completed in the traditional way with the construction of low hollow-brick walls, arranged at a distance apart equivalent to that of the overlaying hollow floor slab (approx 80-100 cm).

4.3.5 Application on FRP to the extrados of half-brick cloister vaults

The work involved four Welsh cloister vaults in as many rooms. The vaults only have a decorative function and the space separating them from the roof is simply a garret with just enough height to carry out routine maintenance. That is why the vaults do not have supports but just voussoirs, which have a modest bracing function in any case limited by the excessively low profile. All the vaults were half a brick thick, even if an extensive portion in the central part of one of them had bricks laid on edge (thickness 5 cm). It was probably a reconstruction following a collapse. The greater constructional simplicity of the brick-on-edge vaults compared to the half brick ones may have justified the choice. The crack due to the Umbria-Marche earthquake in 1997 was particularly evident around the ribs. In all four cases highly visible cracks had formed at the corners to the intrados where there were frescoes (see Fig. 21). There was more damage to the larger vaults and in any case the cracks had occurred over pre-existing fissures, symptom of an intrinsic weakness of this type, also obvious in static conditions. Damage to the top (at the crown) was only important in one case, where there was sagging of a central longitudinal portion about 50 cm wide.

The work with FRP strips in these cases was quite easy due to the absence of filling above the vaults. Even the difficulties tied to the restricted working space were limited due to the lightweight and relatively small size of the material used. This peculiarity of the cladding work using sheets of composite material was of fundamental importance in the garrets and in general in unsuitable spaces for normal site facilities. As for the cross vaults, also in this case the preliminary operation was to clean the vault and level the areas to be clad with special mortar.

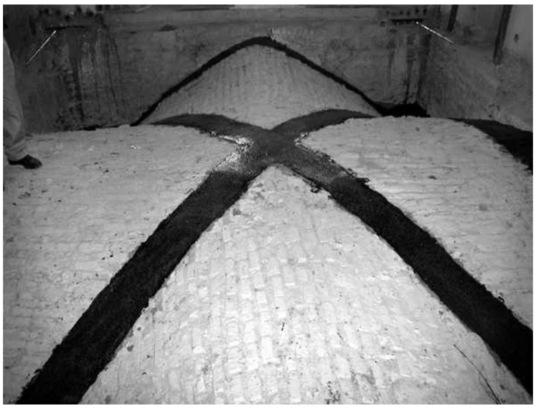


FIGURE 20: The strengthening intervention of the cross vaults

FIGURE 21: Frescoes on the intrados

5. Mechanical characterization of used innovative materials

The reinforcements were made using monodirectional FRP sheets and re-bars epoxy-bonded to wood and masonry elements. According to ASTM specifications all the epoxy-resins (primer, saturant and putty) were characterized in order to find the tensile and compressive value of strength and modulus of elasticity. Carbon fiber materials (sheets and re-bars) were also characterized. Epoxy-resins and fibers are produced by Mac S.p.a. (Mbrace reinforcement system).

5.1 Characterization of epoxy-resins

According to ASTM D638 and D695 specifications a mechanical characterization of the epoxy-resins was performed in order to find the tensile and compressive value of strength and modulus of elasticity. Tests were performed on a Lloyd Instruments LR30K dynamometer using the controlling and measuring system R-Control Lloyd.

The three different epoxy-resins characterized are:

- Primer
- Saturant
- Putty

Table 10 and 11 shows the results of tests.

TABLE 10: Results of compression tests of used epoxy-resins.

Epoxy-resin	*Number of samples*	*Compression strength [Mpa]*	*Young Modulus E [Mpa]*
Primer	5	26.15	-
Saturant	5	56.54	2081
Putty	5	65.54	4634

TABLE 11: Results of traction tests of used epoxy-resins.

Epoxy-resin	*Number of samples*	*Tensile strength [Mpa]*	*Young Modulus E [Mpa]*
Primer	5	12.69	426
Saturant	5	23.43	2144
Putty	4	25.21	4510

5.2 Characterization of fibers

The CFRP sheets were characterized following ASTM D 3039 specifications. The results of the tests are reported in Table 12.

TABLE 12: Results of traction test carried out on CFRP samples (monodirectional sheets)

Number of samples	Failure stress [MPa]	Young modulus of elasticity E [MPa]	Failure stress (dry fiber thickness t=0.165mm) [MPa]	Young modulus of elasticity E (dry fiber thickness t=0.165mm) [MPa]
5	958	117855	3388	417624

6. Prospective of the experimental research

An analytical model suitable for design purposes is now developed for the prediction of the behavior of shear walls, masonry vaults and wood beams reinforced with FRP sheets. Based on the test results, it is possible to conclude that the application of externally bonded carbon fiber sheets seems to be an effective seismic strengthening and repair procedure. However it will be necessary further research in order to determine the efficiency of the proposed intervention methods. In particular the long-term performance will be analyzed.

In order to verify the effective behavior of the composite reinforcements, a series of simple traction, compression and bond tests will be carried out on fiber, resin, masonry and wood test pieces. The results of these tests may be useful for numerical modeling in order to find the appropriate parameters to use for analytical analyses.

7. Conclusions

Mechanical tests on the reinforced wood showed that external bonding of thin composite sheets on the side loaded in traction of flexed wood parts produces very interesting effects. In the case of carbon fibers, which demonstrated to be the most performing ones, results showed increases in flexural strength up to 90% compared to the results of un-strengthened specimens. Glass and, even more, aramidic fibers, produced smaller strength increases, but if taking into account its low price, glass fibers demonstrated to be very good candidates. On the other hand the Young modulus increase was not so high.

It has than been illustrated how it is possible to obtain an important increasing of the stiffness simply pre-loading the samples. Gluing the composite reinforcement during the wood pieces were under a bending load applied in the opposite direction than during the tests, produced a maximum stiffness increase as high as 80%, without affecting the sample's strength. But more tests should be necessary in order to better qualify such a behavior. The used technique resulted easy and fast to realize, even if on in situ parts. In particular it demonstrated to be very promising in many cases of old and historical structural wood parts restoration.

The second part of the experimental work relates to a study on the seismic upgrading and repair interventions of masonry buildings, walls and vaults using externally bonded carbon and glass fiber sheets.

Fiber wrapping of masonry buildings in FRP may inhibit the out-of-plane mechanisms of masonry walls and permits the transfer of seismic loads to the wall parallel to the direction of seismic action. This technique may be used for provisional or definitive interventions and it shows positive characteristics with regard to several aspects, such as the increase in ductility of the masonry structure.

With regard to the shear strengthening of masonry panels with sheets of monodirectional carbon or glass fiber, the shear compression tests carried out on panels of varying dimensions have highlighted the fact that the interface between the panel and the mortar used as a base for the fibers was the weakest element in the system. When a cement-based mortar was applied, failure resulted from the separation of the layer of the mortar from the panel, with almost equal increases in strength and stiffness for both carbon fibers and fiberglass. The subsequent diagonal compression tests, carried out using carbon fibers on a thin layer of epoxy plaster applied directly on the surface of the panels, demonstrated greater increases in both strength and stiffness. Failure resulted, in the case of stone panels, from a separation of the two masonry leaf walls, while in the case of the panel in brick diatons it resulted from the crushing of the bricks at high load values. For this masonry texture a comparison of the results with those for non-strengthened panels gives very high increases in shear strength. The application of this type of intervention resulted more effective in the case of panels made of adequately connected masonry multiple-leaf walls or, as in the case of the brick panel, consisting of a single leaf masonry wall, so that crisis mechanisms due to bending moment and axial forces were avoided. Several specimens have been tested in the present study and an interesting increase in shear strength, shear elasticity and ductility was obtained.

The reinforcing of masonry arch bridges or vaults seems to be more efficacious compared to traditional reinforcing procedures. This technique was applied on a historic building located in Assisi, Italy, and its effectiveness is now under laboratory-testing on full-scale prototypes with dynamic and static loading tests up to collapse. In addition, the problem of fixing the FRP sheet to the masonry structure was studied, in order to avoid debonding problems.

Acknowledgements

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