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Production Department
NEW METHODS FOR BONDING FRP STRIPS ONTO MASONRY STRUCTURES: EXPERIMENTAL RESULTS AND ANALYTICAL EVALUATIONS

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The paper presents and analyzes the results of an experimental research on masonry walls tested first in unreinforced condition and then in FRP reinforced condition. The specimens were three real-scale perforated brick walls. The experimentation consisted in collapse testings under increasing lateral load and constant vertical load. The reinforced condition consisted in strengthening the masonry walls with Carbon Fiber-Reinforced-Polymeric strips bonded onto the masonry surface. The CFRP reinforcement was applied by using three different techniques. The first technique involved epoxy bonding of strips (common application). The second technique involved the bonding of strips with epoxy resin and strengthening the bond with studs (FRP studs embedded into the masonry and connected with the strips). The third technique consisted in the epoxy bonding of strips under vacuum (i.e., using a special vacuum-packed system to push the resin into the masonry as deep as possible). This research aimed at investigating the differences in the structural behaviors of tested perforated masonry walls due to these three different CFRP application techniques. Actually, the experimental results prove that the application technique influences both the load-carrying capacity and the ultimate horizontal displacement of the perforated tested masonry walls. The comparison between the experimental results and code provisions showed that, as regards the debonding of CFRP strips, the latter underestimate the former excessively.

KEY WORDS: perforated walls, masonry walls, FRP studs, vacuum bonding, in-plane behavior, debonding

1. INTRODUCTION

This paper considers perforated masonry walls strengthened with externally bonded Carbon Fiber-Reinforced-Polymeric (CFRP) strips (Foraboschi, 2009; Vanin and Foraboschi, 2012). The effectiveness of this strengthening technique depends essentially on the capacity of bonding the strips to the masonry surface. The failure of strengthened structures due to debonding is undesirable, because this is a brittle
failure mode (the strain of a strip that debonds is several times lower than the tensile ultimate strain of the strip as a stand-alone element, i.e., of the ultimate strain of the fiber).

In the last decade, the behavior of masonry walls strengthened with externally bonded FRP reinforcement has been widely investigated (Aiello and Sciolti, 2006; ElGawady et al., 2005, 2007; Marcari et al., 2007) with a special emphasis on debonding. In order to comprehend the phenomenon and define basic formulas for governing it, the researches on debonding included experimental tests. Experimental investigations related to the research presented here are that of Capozucca (2010), which was devoted to evaluating the bond of both FRP and steel reinforcements externally epoxy-bonded onto historic brickworks walls; Garbin et al. (2010), which evaluated the local behavior of FRP reinforcement externally epoxy-bonded onto masonry walls made of clay bricks and fitted to experimental data to derive an analytical model; Oliveira et al. (2011), which characterized the tensile and shear bond behavior of FRP sheets externally applied onto masonry prisms (load capacity and stress distribution along the bonded length) and proposed an analytical stress–slip formulation; Grande et al. (2008, 2011), which estimated the effect of the bond length on delamination and developed a simple procedure for evaluating the bond strength of FRP sheets and plates externally bonded onto masonry structures (including the estimation of the fracture energy released during the debonding process).

At the same time, the researches were also aimed at developing new technologies for improving the adhesion of the FRP strip to the masonry support. In particular, Burr (2004) described the use of FRP anchors to improve the connection between externally bonded FRP reinforcement and masonry; Korany and Drysdale (2007) carried out an experimental investigation to evaluate the effectiveness of an innovative technique that consisted in leaving unbonded the horizontal external FRP strips and intermittently bonding the strips in the vertical direction; Tan et al. (2003) completed an experimental research to compare the effectiveness of three different methods for bonding FRP reinforcements, which consisted of: (1) a simple bond, (2) a fiber anchor bolt system, and (3) an embedded bar system. Similar research was developed for a concrete support. Cerioni et al. (2008) evaluated the effectiveness of different systems of end-fixing for FRP sheets onto concrete structures, which consisted of steel or FRP plates glued or bolted, and FRP bars or L-shaped fibers. Cerioni et al. (2010) also developed a simple model to predict the debonding load for these anchorage systems. Moreover, Bilotta et al. (2011) conducted an experimental work to investigate the bonding behavior of two different types of strengthening systems: (1) externally bonded carbon plates, (2) bars or strips externally applied by the Near Surface Mounted (NSM) technique. Willis et al. (2009) executed pull out experimental tests for analyzing the effectiveness of externally bonded and NSM retrofitting techniques for masonry walls as well. The authors also proposed an interface bond-slip model to describe the behavior of the FRP to masonry interface.
The research presented here aims at contributing to the development of innovative solutions for bonding CFRP strips onto masonry surface. In particular, two new techniques were investigated in this research: the first uses CFRP studs as shear connectors between the strips and the masonry, and the second uses a vacuum-packed system to push the resin into the deeper layers of masonry. The research is based on six experimental tests performed on three perforated brick walls, first unreinforced and then reinforced with CFRP strips bonded using the above-mentioned techniques. The paper presents the experimental results as well as their interpretation; moreover, the paper presents comparisons of experimental data with theoretical results obtained using the Italian CNR "Guidelines for strengthening of existing structures with FRP materials" (2005) for the tested FRP reinforced walls.

2. EXPERIMENTAL TESTS

The specimens in experimental tests were three full-scale masonry walls with a central opening (perforated walls), made of bricks. Two spandrel concrete beams were placed at the top and the bottom of each wall, to spread the loads. Each specimen was loaded in-plane by two vertical forces applied at the top of the upper spandrel, whose value was maintained constant, and by a lateral force applied at the side of the upper spandrel, whose value was increased up to reaching the failure. The loading details are provided in the following test description.

Each specimen was tested first under unreinforced condition. The load was increased up to full activation of the kinematic mechanism but was stopped before the disintegration of the wall. Subsequently, each specimen was strengthened with CFRP strips externally bonded onto the masonry surface and then it was tested under reinforced condition. No crack provoked by the first test was repaired for the second test of the specimen. The load was increased up to the failure, which was due to debonding.

The CFRP strips were applied by using three different techniques, two of which are innovative, as mentioned in the Introduction: (1) strips epoxy-bonded onto the masonry surface, which consisted of the common bonding technique; (2) strips both epoxy-bonded onto the masonry and pinned with CFRP studs embedded into the masonry, which consisted of providing the strips with anchorages; (3) strips epoxy-bonded onto the masonry using a technique that pushed the resin into the deeper layers of masonry by means of a vacuum-packed system, which consisted of a means for increasing the masonry layer whose detachment due to peeling stresses causes debonding to occur. The details of these bonding techniques are provided in the following paragraph.

The three reinforced tests maintained the same arrangement of the CFRP strips. Thus, the reinforced walls differed from one another only for the technique by which the strips had been attached (whereas their tests differed also for the vertical load).
The tests aimed at comparing:

1) the behavior of each FRP reinforced wall with the behavior of the same unreinforced wall, in order to evaluate the mechanical efficiency of strengthening the structure with CFRP strips;

2) the behavior of the three FRP reinforced walls, in order to evaluate the efficiency of the different bonding techniques;

3) the ultimate load for debonding each FRP reinforced wall obtained in the experimental tests to the ultimate load for debonding obtained by using the Italian CNR "Guidelines for strengthening of existing structures with FRP materials" (CNR, 2005), in order to evaluate the safety margins guaranteed by code provision (i.e., if and how preservative it is).

2.1 Specimens

This paragraph describes the main characteristics of the specimens, to provide details with respect to what was anticipated in the previous paragraph.

2.2 Geometry, Materials, Construction Technique

Each masonry wall consisted of two masonry piers and two masonry spandrels. This configuration implied a central significant opening. The overall dimensions of the specimens are given in Fig. 1. The masonry structure of the specimens, including the texture, is shown in Fig. 2. The walls were made of bricks assembled with mortar. As already mentioned, two spandrel concrete beams were placed at the top

![FIG. 1: Geometrical shape of the specimens](image-url)
and bottom of each wall, respectively. The concrete spandrel beams are not part of the specimens, but only a means for smearing the stresses due to the load applied at the top and to the restraint at the bottom. The bricks used were procured from the demolished ancient buildings, and were of size $240 \times 116 \times 55$ mm. The composition of the mortar was $3/12$ of lime, $1/12$ of Portland cement, $1/3$ of sand, and $1/3$ of water. The horizontal and vertical mortar joints, made flush with the faces of bricks, were 10 mm thick. The masonry texture (Fig. 2) alternated as follows: one brick parallel to the wall plane and the next one orthogonal to the wall plane.

The concrete spandrel beam at the bottom of the wall enabled the specimens to be clamped to the ground floor of the laboratory. The concrete spandrel beam at the top of the wall allowed the vertical and horizontal forces to be applied to the specimens without any stress localization in the masonry wall, i.e., the top spandrel beam smeared the forces applied during the tests along the whole top side of the wall.

Experimental tests were performed to determine the compressive strength, modulus of elasticity, and indirect tensile strength of the bricks and mortar. Experimental tests were also performed to determine the compressive strength and modulus of elasticity of the masonry, while the masonry tensile strength was determined based on the tensile strength of the bricks and mortar, by means of the analytical formula proposed by Tassios (1988).

Samples of mortar and masonry were tested 28 days after their construction, during which they were submitted to the same environmental conditions of the walls. The mean values of the mechanical properties of materials, experimentally obtained or analytically deduced, are reported in Table 1.

**FIG. 2:** Masonry structure of the specimens
2.3 Strengthening with FRP Strips

As stated above, after the first loading each masonry wall was strengthened with CFRP strips. We adopted the manufactured tissue with fibers lying in one direction, i.e., FRP one-way CFRP fabric. The fabric was impregnated with resin at the job site to support the masonry, i.e., a wet lay-up system was used, which was impregnated with the matrix of epoxy resin when it was laid onto the film of resin smeared onto the masonry.

More specifically, initially basic repairs were made to the masonry surface, but not to the cracks, which were neither replaced by new brick textures nor injected. In particular, the masonry surface was abraded to smooth out irregularities, remove contaminants, and radius sharp corners. This was performed by shot and sand blasting, water jet, and grinder. Then, in order to promote adhesion, a low viscosity epoxy primer was applied with a roller, until the substrate was locally saturated. Subsequently, an adhesive, high viscosity putty, was applied to the surface to fill in "bug holes" offsets and voids. In the clean area of the laboratory, away from the resins, the carbon fabric was measured and cut in accordance with the specifications. Then, the surface was coated with resin, and dry fabric was applied, i.e., pre-wetted tissue was laid onto the surface and smoothed out to remove air bubbles and ensure that the fibers were straight. Finally, a layer of epoxy resin was applied to the wall surface.

In so doing, carbon FRP fabrics saturated with saturating resin were applied in strips to the masonry wall surface. During the cure, the fabrics were checked to ensure that all air bubbles had been removed and that each fabric was not sagging (a trained, qualified inspector monitored the applications).

Each strip was 50 mm wide and 1.7 mm thick. The mechanical properties of the CFRP strips are reported in Table 2.

<table>
<thead>
<tr>
<th>Table 1: Mechanical properties of the materials of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Masonry</strong></td>
</tr>
<tr>
<td>Compressive strength $f_M$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Compressive strength $f_b$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Compressive strength $f_m$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Modulus of elasticity $E_M$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Modulus of elasticity $E_b$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Modulus of elasticity $E_m$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Tensile strength $f_{Mt}$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Tensile strength $f_{bt}$ (N/mm$^2$)</td>
</tr>
<tr>
<td>Tensile strength $f_{mt}$ (N/mm$^2$)</td>
</tr>
</tbody>
</table>
TABLE 2: Physical, geometrical, and mechanical characteristics of the CFRP strips

<table>
<thead>
<tr>
<th>Weight of CFRP on m² of tissue (N/m²)</th>
<th>3.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of the strip (mm)</td>
<td>200–1000</td>
</tr>
<tr>
<td>Thickness of carbon for analytical evaluations (mm)</td>
<td>0.177</td>
</tr>
<tr>
<td>Failure load on a strip of unit length (N/mm)</td>
<td>≥640</td>
</tr>
<tr>
<td>Modulus of elasticity (N/mm²)</td>
<td>240.000</td>
</tr>
</tbody>
</table>

The strips were arranged so as to prevent the opening of the cracks formed during the first loading (Figs. 3 and 4). Here, the mechanism that dictated the load-carrying capacity of the unreinforced walls was prevented from being activated.

FIG. 3: Kinematic mechanism shapes of the unreinforced specimens

FIG. 4: Reinforcement of the specimens with FRP strips
The least possible amount of fabric was used. Accordingly, the strips were applied only to one side of each specimen and arranged in one layer only.

Not only the cracks appearing due to the first loading were not repaired but also each wall was left in the state attained by the mechanism activated by the previous loading. It means that the application of the CFRP strips was the only action taken for improving the structural behavior of each masonry wall, i.e., the only action taken after the first loading was to improve the masonry surface that the strips had to be bonded onto.

The details of what was anticipated after the three previously considered different techniques were used to bond the CFRP strips onto the three specimens are the following:

**Specimen 1.** The strips were bonded onto the masonry surface simply by means of epoxy resin, i.e., the traditional bonding technique was used. However, the masonry surface was adequately prepared prior to bonding the FRP strips, to provide the reinforcement with appropriate support. This means that not only the debonding had to occur due to the detachment of a masonry layer, but also that the support provided the bond with adequate strength as regards the peeling stresses.

**Specimen 2.** CFRP studs were plunged into the masonry before bonding the strips. The length of these studs was 100 mm and the diameter was 15 mm. The studs were inserted into holes made preliminary. Each stud was inserted into the hole made in two narrow CFRP strips which were much longer than the hole. Thus, the introduction of a stud into its hole caused the narrow strip to be introduced into the hole too and to let it be bent onto the external masonry surface as well, where it was superimposed by the strip (more specifically, each narrow strip was sufficiently long to come out from the hole for more than 100 mm). Therefore, the two narrow strips per stud were bonded onto the masonry surface by means of epoxy resin in the same direction in which the CFRP strips are applied subsequently (the latter overlapped the former).

The studs of specimen 2 were placed in correspondence with all the intersections between the strips (Fig. 6). As a consequence, the CFRP strips were continuously

![FIG. 5: Strengthening of specimen 1. Application of cellulose paper (left), covering with nylon (centre), final result (right)](image-url)
bonded onto the masonry structure of the specimen and discontinuously pinned at the masonry (i.e., anchored at each intersection between the strips). Note that a stud improves the adhesion of the strip to the masonry not only at the point where it is placed, but over the entire strip, since the debonding strength depends on the process zone (i.e., the masonry involved in the fracture) and the process zone is longer if one end of the strip is prevented from translating.

The application of the strips of specimen 3 consisted of the same application of specimen 1 but under vacuum. More specifically, first a thin layer of cellulose paper was applied onto each strip and attached to it by epoxy resin. Then, the specimen was entirely covered with a nylon, attached at its edge, to prevent any air transpiration (Fig. 5). At that point, a suction opening had been made, where a suction pump took the air away until the vacuum was reached. Moreover, the nylon strongly pushed the strip onto the masonry. This extra-pressure made the resin penetrate deep into the masonry (much deeper than at atmospheric pressure). Then, debonding was to involve detachment of a thicker layer of masonry, which means that the strip is provided with a better bond.

2.4 Test Setup

The test setup is shown in Fig. 7. Four vertical hydraulic jacks were placed along the two masonry piers of each wall, one per side of each pier. Iron chains connected these jacks from the top to the bottom of concrete spandrel beams. Steel plates were interposed between the concrete spandrel beam placed at the top and the iron chains, to prevent local failure due to compressive stress localization. A load cell was connected to the jacks, to provide the wall with the amount of vertical load that had been calculated for having a given lateral load carrying capacity.
(the latter depends on the former, of course). The vertical load was kept constant during each test.

Two horizontal hydraulic jacks were placed parallel each other and put on a special steel plate at the top of the masonry wall (at the level of the top spandrel concrete beam). A load cell was connected to these two hydraulic jacks to provide the masonry wall with an increasing lateral load. The steel plate put off the horizontal force from the hydraulic jacks to the concrete beam and then transferred it to the wall.

The load cells were connected to a central hydraulic apparatus, which was connected with an electronic system and a computer. This instrumentation provided the control system that kept the vertical load constant and increased the lateral load by the given load steps.

A transducer, mounted on telescopic rods, was placed between the contrast wall of the laboratory and the specimens. This transducer, connected with the same electronic system, recorded the lateral displacements of the top of the wall.

2.5 Test Description

The execution of the experimental tests is shown in Fig. 8. All the tests can be divided into (1) unreinforced specimens tests and (2) reinforced specimens tests. Each test with an unreinforced specimen differed from the tests on other unreinforced specimens by the value of the constant vertical load $F_v$ (Table 3). Each test on a reinforced specimen differed from the tests on other reinforced specimens by the
value of the constant vertical load $F_v$ (Table 3), as well as by the technique of application of reinforcement. Note that each specimen was tested under the same applied vertical load in unreinforced and reinforced condition, as shown in Table 3.

The lateral force was increased monotonically from zero up to the failure of a specimen. The failure of unreinforced specimens was caused by the kinematic mechanism, while the failure of the reinforced specimens, was caused by debonding (Fig. 8). The debonding mechanisms are described in the next paragraph.

### 2.6 Experimental Results

The development of cracking in unreinforced specimens is described in Fig. 9, where the numbers indicate the progression of cracks in each specimen.

The failure caused by debonding of the CFRP strips in reinforced specimens is described in Fig. 10, where the numbers indicate the progression of the debonding of strips. Figure 11 show some particulars of detachment of the strips.

The lateral force–displacement curves obtained for all the specimens, under both unreinforced and reinforced conditions, are shown in Figs. 12–14.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vertical load $F_v$ on each pier (kN)</th>
<th>Vertical load on a specimen (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>3</td>
<td>70</td>
<td>140</td>
</tr>
</tbody>
</table>

**FIG. 8:** Execution of the experimental tests for specimens in unreinforced (left) and reinforced (right) conditions

**TABLE 3:** Constant vertical forces applied to the specimens
FIG. 9: Progression of the cracking frame for specimens in unreinforced condition

FIG. 10: Progression of detachment of CFRP strips for specimens in reinforced condition

FIG. 11: Detachment of CFRP strips from the masonry surface of the specimens
2.7 Unreinforced Specimens

The crack patterns in unreinforced specimens are very similar and develop in the same way, despite the different values of the vertical load each specimen was subjected to (Fig. 9).

The first crack occurred in the top spandrel, just above the opening, and developed at an angle of 45°. The second and third cracks occurred at the base of the
right and left piers, respectively; both of them pointed to the bending failure of the piers. The third crack also hinted at the attainment of the kinematic mechanism.

The uncoupled displacements of the two piers involved in the kinematic mechanism led the masonry top spandrel to losing the equilibrium. For this reason, other cracks developed in the masonry spandrel, which produced the detachment of several bricks (these bricks slid towards the opening).

The lateral force–displacement curves of unreinforced specimens display a first rising branch, which starts from the origin and continues up to the manifestation of local crushing (Figs. 12–14). Then, the curves show a second rising branch, which continues up to the first crack of the top spandrel above the opening. This crack implies the first significant reduction of the stiffness of the specimens, which is shown by the curves.

The curves of specimens 1 and 2 exhibit a third branch, whose slope is lower than that of the second branch. This third branch continues up to the cracking of the pier subjected to tensile axial force due to the global bending moment.

The curve of specimen 3 exhibits a third branch, whose slope is nil, followed by a fourth branch, whose slope is lower than that of the second branch. This fourth branch continues up to the cracking of the pier subjected to a tensile axial force due to the global bending moment.

In all the specimens, the reduction of the stiffness in consequence of the cracking of the pier subjected to a tensile axial force is significant.

The peaks of the curves indicate the cracking of the pier on application of a compressive axial force, as well as the attainment of the kinematic mechanism by the walls.

The lateral load–displacement curves present some significant values (Table 4):

FIG. 14: Horizontal force–displacement curve of specimen 3 in unreinforced and reinforced conditions
- the value of the horizontal force $F_{h_{unr}}$ that identifies the load-bearing capacity peak;
- the ratio $\gamma_{hv_{unr}}$ between $F_{h_{unr}}$ and the vertical load $F_v$ applied to each specimen (Table 2);
- the value of the horizontal force $F_{h_{unr-1}}$ that corresponds to the first cracking;
- the value of the horizontal displacement $\delta_{h_{unr}}$ corresponding to $F_{h_{unr}}$;
- the value of the horizontal displacement $\delta_{h_{unr-1}}$ corresponding to $F_{h_{unr-1}}$.

Specimens 1 and 3 exhibit the same value of $\gamma_{hv_{unr}}$, while specimen 2 exhibits a different value of $\gamma_{hv_{unr}}$ although the difference is marginal. Consequently, it can be deduced that the differences between the values of $F_{hv_{unr}}$ are due mainly to the different values of the vertical force $F_v$ applied to the specimens.

### 2.8 Reinforced Specimens

The debonding failure of the reinforced specimens developed in the same way in all the specimens, although the CFRP strips had been applied by different techniques (Figs. 10 and 11). In particular, the first detached CFRP strip was that which stitched the crack of the top spandrel, whose debonding occurred above the opening. The second and third detached CFRP strips were those at the base of the piers (first the strips attached to the piers on application of a tensile axial force and then the other one). Thus, on the whole, the debonding developed in the same way as cracking, for all the specimens, independently of the technique by which the strips were bonded.

After the detachment of the third strip, the specimens attained the kinematic mechanism, since the CFRP strips that remained still attached could not prevent the activation of the mechanism.

According to the CNR (2005), three types of fracture may occur in an adhesive joint: cohesive, adhesive, and mixed (Fig. 15). Cohesive fracture occurs inside one of the two materials that form the adhesive joint. Thus, the two surfaces of frac-
ture are made of the same material. The structures onto which the epoxy resin is correctly applied exhibit cohesive fracture (Fig. 15a).

Adhesive fracture occurs at the interface between the adhesive and the support. Thus, the two surfaces of fracture are made of the two materials that compose the joint. The adhesive fracture occurs in the structures whose interface strength (adhesion force) is lower than the strength of the support (Fig. 15b).

Mixed fracture occurs alternatively as cohesive and adhesive fracture. The two surfaces of fracture are made of the two materials. Mixed fracture occurs in the structures whose support is not coherent or whose application is not correct (Fig. 15c).

In all the tested specimens, the debonding exhibited the cohesive fracture (Fig. 16), which proves that the application of the reinforcement had been correct.

All the lateral force–displacement curves of the reinforced specimens show an initial rising branch, which starts from the origin and continues up to local crush-

FIG. 15: Types of fracture of the adhesive joint

FIG. 16: Particular of the detachment of CFRP strips from the masonry surface
ing (Figs. 12–14). Then, the curves exhibit a second rising branch, which continues up to detachment of the CFRP strip that had stitched the crack of the top spandrel. More specifically, this detachment was initiated above the opening, where the crack which stitched the strip was initiated. This detachment implied a first reduction of the wall stiffness. The second rising branch is followed by the third rising branch, which continues up to the detachment of the CFRP strip that bonded the crack at the base of the pier in tension, and by the fourth rising branch, which continued up to the detachment of the CFRP strip that bonded the crack at the base of the pier in compression. The second and third detachments of the strips cause a further reduction of the walls stiffness.

The lateral load–displacement curves present some significant values (Table 5):

- the value of the horizontal force \( F_{h,r} \) that identifies the load-bearing capacity peak;
- the value of the ratio \( \gamma_{hv,r} \) between \( F_{h,r} \) and the vertical load \( F_v \) to which each specimen was subjected constantly during the test (Table 2);
- the value of the horizontal force \( F_{h,r-1} \) that identifies the first detachment of the CFRP strips;
- the value of the horizontal displacement \( \delta_{h,r} \) corresponding to \( F_{h,r} \);
- the value of the horizontal displacement \( \delta_{h,r-1} \) corresponding to \( F_{h,r-1} \).

### 3. ANALYTICAL EVALUATION OF DEBONDING

The ultimate load that provokes debonding of the CFRP strips in each tested specimen was evaluated according to code provisions (CNR, 2005).

When debonding involves only the superficial layer of the masonry (normal application of the FRP strip, as that in specimen 1), the characteristic value of the specific fracture energy, \( \Gamma_{FK} \), is

\[
\Gamma_{FK} = c_1 \sqrt{\frac{f_{tm}}{f_{mtm}}},
\]

### TABLE 5: Reinforced specimens: experimental values

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( F_{h,r} ) (kN)</th>
<th>( \gamma_{hv,r} ) (ad)</th>
<th>( F_{h,r-1} ) (kN)</th>
<th>( \delta_{h,r} ) (mm)</th>
<th>( \delta_{h,r-1} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19</td>
<td>0.475</td>
<td>15</td>
<td>28</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>54</td>
<td>0.725</td>
<td>42</td>
<td>65</td>
<td>18</td>
</tr>
<tr>
<td>3</td>
<td>84</td>
<td>0.6</td>
<td>72</td>
<td>75</td>
<td>29</td>
</tr>
</tbody>
</table>
in which $f_{nk}$ is the characteristic value of the masonry compressive strength, $f_{ntm}$ is the average value of the masonry tensile strength, and $c_1$ is a coefficient provided by the CNR (2005). In particular, this code prescribes $c_1 = 0.015$ without any specific experimental evaluation, differently from $f_{nk}$ and $f_{ntm}$, which have to be determined experimentally.

Then, according to the CNR (2005), the stress that provokes debonding, $f_{fdd}$, is

$$f_{fdd} = \frac{1}{\gamma_{f,d} \cdot \sqrt{\gamma_M}} \cdot \sqrt{\frac{2 \cdot E_f \cdot \Gamma \cdot F_K}{t_f}} ,$$  \hspace{1cm} (2)

where $\gamma_{f,d}$ and $\gamma_M$ are two safety coefficients related to the uncertainties of FRP and masonry, respectively, $E_f$ is the modulus of elasticity of the FRP, and $t_f$ is the thickness of the CFRP strip. The comparison made in this paragraph considers two different evaluations. The first evaluation (EV.1) assumes that $\gamma_{f,d} = 1.2$ and $\gamma_M = 2$, as well as the characteristic value of the masonry compressive strength given by the CNR guidelines (CNR, 2005), while the second evaluation (EV.2) assumes that $\gamma_{f,d} = 1$, $\gamma_M = 1$, as well as the average value of the masonry compressive strength.

More specifically, EV.2 adopted the values of $f_M$ and $f_{Mt}$ of Table 1, and assumed that $E_f = 240.000$ N/mm$^2$ together with $t_f = 0.177$ mm (Table 2). Hence, EV.1 includes all the safety coefficients prescribed by the codes, and thus it consists of the design debonding strength. On the contrary, EV.2 considers the average values of the parameters involved and thus it gives the average values of the debonding strength (expected debonding strength). More specifically, the probability that the actual debonding strength is lower than EV.1 and EV.2 are about 0.005 and 0.50, respectively. Accordingly, two types of comparisons can be made between experimental results and code provisions, one using the design values and the other using the expected values.

The force at debonding, i.e., the maximum force that is provided to the masonry wall by the strip, is

$$N_{fdd} = f_{fdd} t_f h_f K_{cr} ,$$  \hspace{1cm} (3)

where $h_f$ is the height of the strip, which in this case is 50 mm, and $K_{cr}$ is the coefficient depending on the type of failure. According to the CNR (2005), two types of debonding may occur in this case:

- plate end debonding, which develops from one of the two ends of the strip;
- intermediate crack debonding, which develops far from the ends of the strip.

Code provisions yield $K_{cr} = 1$ for the plate end debonding and $K_{cr} = 3$ for the intermediate crack debonding.

Equations (1)–(3) allow the theoretical force that provokes debonding to be calculated for each specimen. More specifically, the theoretical values of $N_{fdd}$ were obtained following the two calculations indicated above, i.e., EV.1 and EV.2.
The experimental value of $N_{fdd}$ was obtained from the values of the horizontal force that made the reinforced specimens fail, $F_{hdb}$, by means of the Virtual Work Theorem (Table 6).

### 4. DISCUSSION

The first comparison was made between the experimental results obtained for the unreinforced and reinforced conditions for each specimen.

The relationship between the crack development and debonding development shows that the arrangement of the strips does not modify the structural behavior of each masonry wall. For each masonry wall, the unreinforced and reinforced conditions exhibited the same displacements, which is proved by the fact that the sequence of opening of the cracks under unreinforced conditions was the same under which the strips were debonded.

Strengthening the masonry walls with CFRP strips provided an increase in the load-bearing capacity of walls, as proved by the experimental values of $F_{hr}$ (Table 5) vs. those of $F_{h unr}$ (Table 4). Moreover, the strengthening of the masonry walls provided the structures with greater ultimate lateral displacement, as is seen from the experimental values of $\delta_{hr}$ (Table 5) vs. those of $\delta_{h unr}$ (Table 4).

These results are significant, especially considering that the specimens were not repaired after the first loading (i.e., the CFRP strips were applied to severely cracked walls) and that the least possible amount of fabric was used for strengthening the specimens (i.e., only the amount strictly necessary for preventing the activation of the kinematic mechanisms observed in the previous unreinforced tests). In particular, the strips were applied to one side of the specimens only and each strip was composed of one layer only. Thus, the action of the CFRP strips improved considerably both the load-bearing capacity and ultimate lateral displacement of the reinforced specimens.

These comparisons show that the strengthening techniques used are viable means for increasing both the load-bearing capacity and the ultimate lateral displacement of masonry perforated walls.

### TABLE 6: Theoretical values of $F_{hdb}$

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$F_{hdb}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plate end debonding</td>
</tr>
<tr>
<td></td>
<td>EV.1</td>
</tr>
<tr>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
</tr>
</tbody>
</table>

The experimental value of $N_{fdd}$ was obtained from the values of the horizontal force that made the reinforced specimens fail, $F_{hdb}$, by means of the Virtual Work Theorem (Table 6).
The ratio between $F_{h,r}$ (Table 5) and $F_{h,unr}$ (Table 4) is equal to 1.06 for specimen 1, to 1.125 for specimen 2, and to 1.33 for specimen 3. Moreover, the ratio between $\delta_{h,r}$ (Table 5) and $\delta_{h,unr}$ (Table 4) is equal to 1.22 for specimen 1, to 1.25 for specimen 2, and to 1.27 for specimen 3.

These ratios indicate that the technique of application of CFRP strips in the vacuum-packed system (specimen 3) provided the load-bearing capacity with a greater increase, and that the technique of application of CFRP strips with FRP studs (specimen 2) provided the load-bearing capacity with a greater value than the traditional application technique that uses epoxy resin only (specimen 1).

Moreover, these ratios indicate that the application technique that used the vacuum-packed system (specimen 3) provided the ultimate displacement with a greater increase, although this increase was moderate, and that the technique of application of CFRP strips with FRP studs (specimen 2) provided the ultimate displacement only with a marginal increase as compared to the traditional application technique (specimen 1).

The comparison between the experimental values of $F_{h,r,1}$ (Table 5) and $F_{h,unr,1}$ (Table 4) shows that the failure of any reinforced specimens due to the first detachment of the CFRP strips occurred for a value of the lateral force greater than that for which the same specimens failed in unreinforced condition due to the first cracking.

The comparison between the experimental values of $\delta_{h,r,1}$ (Table 5) and $\delta_{h,unr,1}$ (Table 4) shows also that the first detachment of the CFRP strips occurred at a value of the lateral displacement greater than the lateral displacement at which the first cracking occurs.

These comparisons demonstrate that not only did all the proposed CFRP application techniques increase the ultimate lateral force and displacement, but also that they increased the lateral force and displacement at failure of the walls tested.

The comparison between the experimental and theoretical results obtained for reinforced specimens permits the estimation of code provisions, in particular, their preservation level. This comparison considered the experimental values of $F_{h,r,1}$ and the theoretical values of $F_{hdb}$ both for plate-end and intermediate crack debonding.

All the specimens, which used all the adopted application techniques, exhibited a debonding strength $F_{h,r,1}$ considerably greater than the theoretical values of $F_{hdb}$ (Table 6) both for plate-end debonding and intermediate crack debonding. With reference to EV.1 and to plate-end debonding, the ratios between $F_{h,r,1}$ and $F_{hdb}$ are 1.88, 2.63, and 2.57 for specimens 1, 2, and 3, respectively; with reference to intermediate crack debonding, the ratios are 1.67, 2.47, and 2.48, for specimens 1, 2, and 3, respectively. With reference to EV.2 and to plate-end debonding, the ratios between $F_{h,r,1}$ and $F_{hdb}$ are equal to 1.07, 1.56, and 1.5 for specimens 1, 2, and 3, respectively; with reference to intermediate crack debonding, the ratios are equal to 1.04, 1.45, and 1.47, for specimens 1, 2, and 3, respectively.
These comparisons prove that the code provisions underestimate the actual debonding strength, both in the case of plate-end debonding and intermediate crack debonding, also without using the safety coefficients. Consequently, incorporating the safety coefficients, the code provisions give a design value of the debonding strength whose probability that the actual value is a lower value is excessively small. At least, the code should define more accurately $\gamma_{d}$ and $\gamma_{M}$, in order to avoid excessive underestimation of $F_{\text{hdb}}$ in strengthening design.

5. CONCLUSIONS

This paper presents and discusses the results of some experimental tests performed in laboratory on three specimens of perforated masonry walls, in unreinforced and FRP reinforced conditions. The reinforced condition was obtained by strengthening the masonry walls with Carbon FRP strips. The strips were bonded onto the masonry surface by using different techniques, namely: (1) the common bonding with epoxy resin, (2) the bonding with epoxy resin and FRP studs inserted into the masonry, (3) the bonding with epoxy resin and the use of a vacuum-packed system that provides a pressure normal to the masonry surface to make the resin penetrate deeper into the masonry.

The results provide a comparison between the effectiveness of the different bonding techniques as well as the comparison with code provisions for debonding strength.

The technique that uses CFRP studs, on the one hand, is not significantly time-consuming, but, on the other hand, provides the wall with a higher ultimate deformation, whereas the lateral load-bearing capacity (i.e., the peak of the lateral force — displacement curve) exhibits only a slight increase. Thus, on the whole, this technique turns out to be an effective mean for improving the seismic behavior of a masonry building, since the increase in the ultimate deformation of the walls entails a greater value of the peak ground acceleration that a building can tolerate.

The technique that uses the vacuum-packed system turns out to be an effective means for improving the bond. Consequently, this technique provides an increase in the lateral load-bearing capacity. To get the greatest benefits with the lowest costs, this technique can be applied partially. In fact, the bond demand varies significantly along the strip, so that the vacuum-packed system can be used only in the zones of the strips where the bond demand is more severe. For instance, this is the case of strips bonded onto the intrados of masonry vaults, where the demand is particularly severe at the crown, whereas the other parts of the strip can be bonded by the common technique of bonding.

The comparison between experimental results and code provisions showed that the latter excessively underestimate the former. More specifically, the underestimation drastically exceeds the conservative position that codes have to assume, and may cause design solutions that can be economically unsustainable and/or unacceptable from the conservation point of view.
If, on the one hand, the number of specimens involved in the experimentation is limited, on the other hand, the specimens were full-scale. So the experimental results are reliable, although their statistics needs to be enlarged. This will be the further step of this research, which will be devoted to full-scale walls with different geometries.

REFERENCES


*Composites: Mechanics, Computations, Applications. An International Journal*


