# RETROFITTING OF STONE MASONRY USING INNOVATIVE GRID-BASED COMPOSITES: THE ERIES-RESTORING PROJECT

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***Abstract****: The work presented in this paper, conducted within the ERIES-RESTORING (REtrofitting of STOne masonRy using INnovative Grid-based composites) project, aims at spanning the information gap about the seismic effectiveness of CRM (Composite Reinforced Mortars) applied to existing undressed stone masonry buildings. In-plane quasi-static cyclic shear-compression tests are conducted on four full-scale strengthened piers at the EUCENTRE facilities in Pavia, Italy. CRM, consisting of a glass-FRP mesh embedded in a mortar compatible with historic masonry materials, is applied to one or both sides of the specimen. Piers with two different aspect ratios are investigated under double-fixed boundary conditions, in order to induce flexural or shear failure of the strengthened elements. Two non-retrofitted piers with identical masonry material and a different aspect ratio each are also tested, to provide benchmark responses. The outcomes of the project will provide useful data for the future development of design guidelines and building code requirements for the design of CRM strengthening of existing masonry structures, which are currently lacking. Preliminary results of the material characterization tests and analytical predictions of the pier strength and failure modes are presented in this paper.*

## 1 Introduction

The seismic assessment and retrofit of existing masonry buildings have gained increasing attention in seismically prone regions with a significant presence of built cultural heritage, accentuated by recent and mandatory legal status in several countries worldwide. Moreover, the change in focus from building replacement to existing stock refurbishment is a step towards greater sustainability, which may require structural upgrades in order to provide an acceptable level of structural safety.

The work presented here is part of the ERIES project, which provides transnational access to leading experimental facilities. This experimental programme, named ERIES-RESTORING, is a joint collaboration led by the University of Lisbon (Portugal) with the participation of the University of Pavia (Italy), Universitat Politècnica de Catalunya (Spain), ETH Zurich (Switzerland), EUCENTRE Foundation, and IUSS Pavia (Italy). The campaign is conducted at the EUCENTRE Foundation laboratories in Pavia, Italy, with complimentary work also carried out in the nearby Material and Structural Testing Laboratory of the Department of Civil Engineering and Architecture (DICAr) of the University of Pavia. It aims to bridge the information gap about the seismic effectiveness of Composite Reinforced Mortars (CRM) applied to existing undressed stone masonry buildings.

To achieve this objective, the campaign includes: (i) characterisation tests on mortar samples from the stone masonry and the jacketing systems; (ii) 9 vertical and 9 diagonal compression tests on bare and retrofitted masonry wallettes; and (iii) 6 cyclic shear-compression tests on bare and retrofitted full-scale masonry piers with two different aspect ratios. Mechanical properties of stones, reinforcing meshes, and helicoidal connectors are taken from technical sheets or other official testing certificates. The experimental campaign addresses 3 configurations of undressed stone masonry: unreinforced (bare) condition, serving as a reference; CRM strengthening on one side of the wall; and CRM strengthening on both sides. The 3 retrofit configurations are studied for two height/length aspect ratios of 1.5 (slender piers) and 0.69 (squat piers).

This paper presents preliminary results of the characterization campaign and an analytical prediction of the behaviour of the pier specimens through shear force/axial load interaction diagrams, while the cyclic shear-compression tests are being carried out at the time of writing.

## 2 Material properties

The specimens studied in the experimental campaign are built to represent the current conditions of ancient buildings typical of European and Mediterranean countries, usually constructed of double-leaf natural stone masonry with roughly dressed blocks obtained from sedimentary rocks. The stones are roughly shaped using a hammer, ranging in dimensions from approximately 100 to 300 mm. The specimens exhibit a construction composed of two layers of stone arranged in uneven horizontal courses, separated by mortar layers measuring 5 to 20 mm in thickness. Due to the stone irregularities, the space between the two layers varies and is filled with a mix of mortar and stone fragments. No through stones are provided, except in the edge zones.

### 2.1 Stone characterisation

Natural stones were cut from Credaro-Berrettino calcareous sandstone rocks in the province of Bergamo, Italy. According to the quarry documentation, they are characterised by a mean density of 2580 kg/m$^3$, mean compressive strength of 149 MPa perpendicular and 144 MPa parallel to the sedimentation layers, and mean tensile strength of 19 MPa.

### 2.2 Mortar characterisation

The mortar mix design for the construction of the masonry walls requires special attention in order to obtain a weak hydraulic-lime mortar comparable with the overall masonry properties of historical buildings. For compatibility between the substrate and the strengthening solution, the jacketing mortar also contains a fraction of natural hydraulic lime. The tensile ($f_{mt}$) and compressive ($f_{mc}$) strength of the masonry and strengthening mortars are obtained through standard laboratory tests and are shown in Table 1. According to the procedure established by EN 1015-11 (CEN, 2006), the mortar is cast in prisms with dimensions of 160 x 40 x 40 mm.

*Table 1. Mortar mechanical properties.*

| | **Masonry walls** | | **CRM strengthening** | |
|---|---|---|---|---|
| | $f_{mc}$ **[MPa]** | $f_{mt}$ **[MPa]** | $f_{mc}$ **[MPa]** | $f_{mt}$ **[MPa]** |
| **Mean** | 0.81 | 0.17 | 22.6 | 5.8 |
| **C.o.V.** | 33% | 72% | 12% | 9.1% |

## 2.3 GFRP mesh and helicoidal connector characterisation

The GFRP meshes embedded in the CRM strengthening of the specimens were characterised by mean tensile strengths of 74 kN/m and 86 kN/m in the weft and warp directions ($f_{weft}$ and $f_{warp}$), respectively, and an ultimate strain $\varepsilon_{fu}$ of 1.5%. Based on the manufacturers' technical sheets, the mean tensile strength is 16 kN for the helicoidal steel connectors.

## 2.4 Masonry characterisation

For this part of the experimental campaign, 9 wallettes are tested in vertical compression and another 9 in diagonal compression. For each retrofit configuration, 3 wallettes are tested. The wallettes are saw-cut from two long walls, discarding a 400 mm length at the extremities to avoid the confining effects of through stones necessary at the wall edges for construction. For vertical compression test specimens, reinforced concrete (RC) spreader beams are provided at the top and bottom of the wall to distribute the load during testing and to facilitate transportation. Spreader beams were not provided to the wall for diagonal compression test specimens because they would have interfered with the test setup and execution.

The dimensions of the specimens designated for vertical compression tests are defined to align with the specifications outlined in the European standard EN 1052-1 (CEN, 1998), with modifications made to accommodate the average size of irregular stones instead of standard brick block dimensions. Conversely, the dimensions of specimens intended for diagonal compression tests follow the specified in ASTM (2015) and RILEM (1991). In both testing scenarios, a force-controlled universal testing machine and displacement transducers constitute the testing apparatus. To ensure accurate measurements, the mounting rods of the transducers are selectively installed in stones of sufficient size, avoiding mortar joints. Consequently, the nominal length between the mounting rods is adjusted on a case-by-case basis, and the actual lengths are measured for each individual specimen. Loading and unloading cycles are characterised by amplitudes set at 1/6, 1/3, and 1/2 of the expected strength, with corresponding application durations of 2.5, 5.0, and 7.5 minutes, respectively, prior to reaching failure. Maximum and zero loads of each cycle are held constant for about 10 seconds before the unloading or reloading phase to stabilise the stress state.

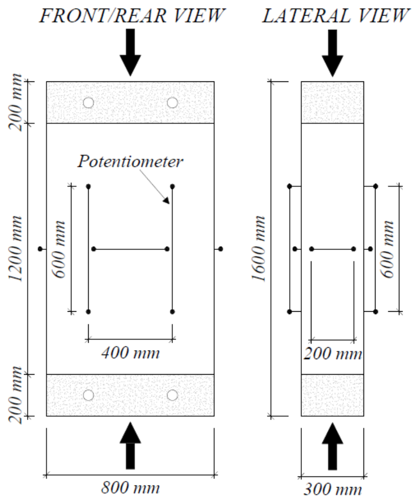
Regarding the vertical compression tests, the applied axial compression is centred on the wallette and distributed as uniformly as possible on the cross-section by spreader beams and tie-beams. Longitudinal and transverse deformations are measured by eight 25-mm-stroke potentiometers, located as shown in Figure 1. The instrumentation is installed within the middle half of the panel, where it is assumed that the boundary conditions (concrete tie-beams) do not have any influence on the state stress. The compressive strength $f_c$ is evaluated by testing the specimen up to failure and taking the maximum applied force. The applied force is then divided by the nominal cross-section area of 300 x 800 mm. Assuming that masonry behaves as a homogeneous, isotropic, and linearly elastic material at low-stress levels, Young's modulus $E$ and Poisson's ratio $\nu$ are evaluated for each specimen between 10% and 33% of the measured compressive strength, while the shear modulus $G$ is derived from its relationship with $E$ and $\nu$.

In the diagonal compression tests, compressive force $P$ is applied along one diagonal of the square masonry panel, leaving the opposite diagonal unloaded. To capture deformations, four 25-mm-stroke potentiometers are strategically positioned along both diagonals, as illustrated in Figure 2. The ASTM (2015) and RILEM (1991) standards interpret the results assuming a pure-shear stress state at the centre of the panel. The corresponding Mohr circle (Figure 3a) is centred on the origin of the $\sigma$-$\tau$ plane, resulting in principal tensile and compressive stresses and pure shear stress, all equal to the radius of the circle, with a nominal area $A_n$ of 300 x 1000 mm subjected to uniform shear stress. The shear strength $\tau_{max}$ is then evaluated by testing the specimen up to failure and taking the maximum applied force:

$$\tau_{max}=0.707\frac{P_{max}}{A_n} \quad (1a)$$

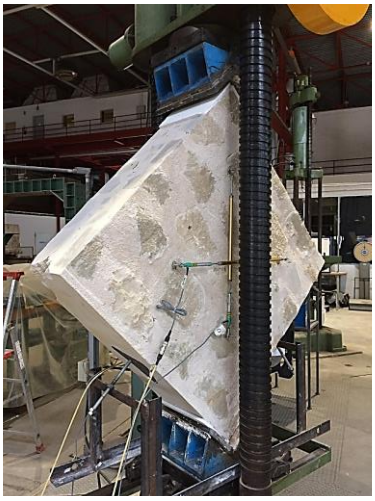
However, the actual distribution of shear stresses is far from uniform, and the panel is not subject to pure shear. Addressing this concern, various authors, employing both analytical and numerical approaches (Frocht, 1931; Brignola et al., 2009), have investigated the issue. They propose more accurate formulations to assess the principal stresses at the centre of the panel, as illustrated in Figure 3b. Following these formulations, the masonry tensile strength $f_t$ is calculated as:

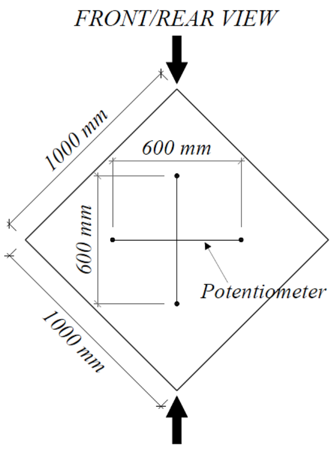

(a)



(b)

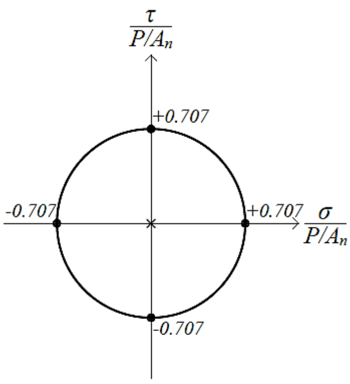
*Figure 1. Vertical compression test: (a) test set up, and (b) specimen dimensions.*


(a)

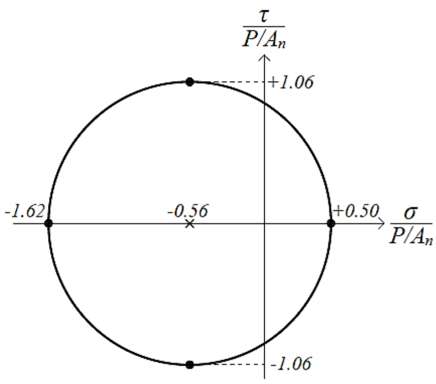


(b)

*Figure 2. Diagonal compression test: (a) test setup, and (b) specimen dimensions.*



(a)



(b)

*Figure 3. Mohr circles of the diagonal compression test: (a) standard interpretation, and (b) refined interpretation. Tension positive. (Senaldi et al., 2018)*

$$f_t = 0.5\frac{P_{max}}{A_n} \tag{1b}$$

From the vertical (CEN, 1998) and diagonal compression tests (Frocht, 1931; RILEM, 1991; ASTM, 2015) it is possible to obtain the mean masonry properties on bare stone masonry wallettes summarised in Table 2. Compressive strength ($f_c$), elastic modulus ($E$), Poisson's coefficient ($\nu$), and shear modulus ($G_{vc}$) are all obtained from vertical compression tests, while the tensile strength ($f_t$) and shear modulus ($G_{dc}$) are evaluated from diagonal compression tests.

*Table 2. Bare masonry mechanical properties.*

| | $f_c$ [MPa] | $f_t$ [MPa] | $E$ [MPa] | $v$ [-] | $G_{vc}$ [MPa] | $G_{dc}$ [MPa] |
|---|---|---|---|---|---|---|
| **Mean** | 1.98 | 0.092 | 3077 | 0.13 | 1365 | 1096 |
| **C.o.V.** | 2.2% | 6.6% | 13.5% | 45.8% | 18.9% | 53.8% |

## 3 Quasi-static cyclic shear-compression tests on piers

### 3.1 Specimen description

The experimental campaign is centred on six pier specimens built with the same natural stone masonry previously described. No through stones are provided, except at the pier or spandrel edges on alternated courses. Two different pier geometries are considered with height/length aspect ratios of $h/l$ = 1.5 (slender piers, Figure 4a) and $h/l$ = 0.69 (squat piers, Figure 4b). The slender specimens include portions of the adjacent spandrels, to allow CRM anchorage, as in the case of masonry piers between windows. The squat piers correspond to walls without openings or with large spacing between them, and are completed by top and bottom RC beams where the GFRP mesh is anchored. Combining retrofit configurations and pier geometries results in 6 specimens, as presented in Table 3.

The CRM solution consists of a glass-FRP (GFRP) mesh embedded in a mortar with a nominal thickness of 30 mm; however, it should be considered that the masonry's irregular surface makes it impossible to keep a constant thickness throughout the wall. The weft of the GFRP mesh is spaced 120 mm apart and the warp 80 mm apart for a total mesh weight of 400 g/m$^2$. The meshes are applied with the warp in the vertical direction for the squat piers and in the horizontal direction for the slender ones, to force the squat piers to fail in shear and the slender ones in flexure.

The CRM layers are mechanically connected to the wall by 5 helicoidal steel bars per square metre of façade, providing also transverse confinement to the masonry (Figure 5a and b). Connectors pass through both masonry leaves when the CRM is applied on both sizes, but are embedded to about ¾ of the specimen thickness when the CRM is applied to only one side. In the case of the squat specimens, some connectors are applied to the top of the bottom RC beam to provide flexural anchorage to the strengthening. Where necessary, the mesh is lap-spliced for at least 300 mm (Figure 5c).

*Table 3. Test specimen combinations.*

| Retrofit Configuration | Aspect Ratio (h/l) |
|---|---|
| Bare masonry | 0.69 |
| | 1.5 |
| Masonry with CRM on one side | 0.69 |
| | 1.5 |
| Masonry with CRM on both sides | 0.69 |
| | 1.5 |

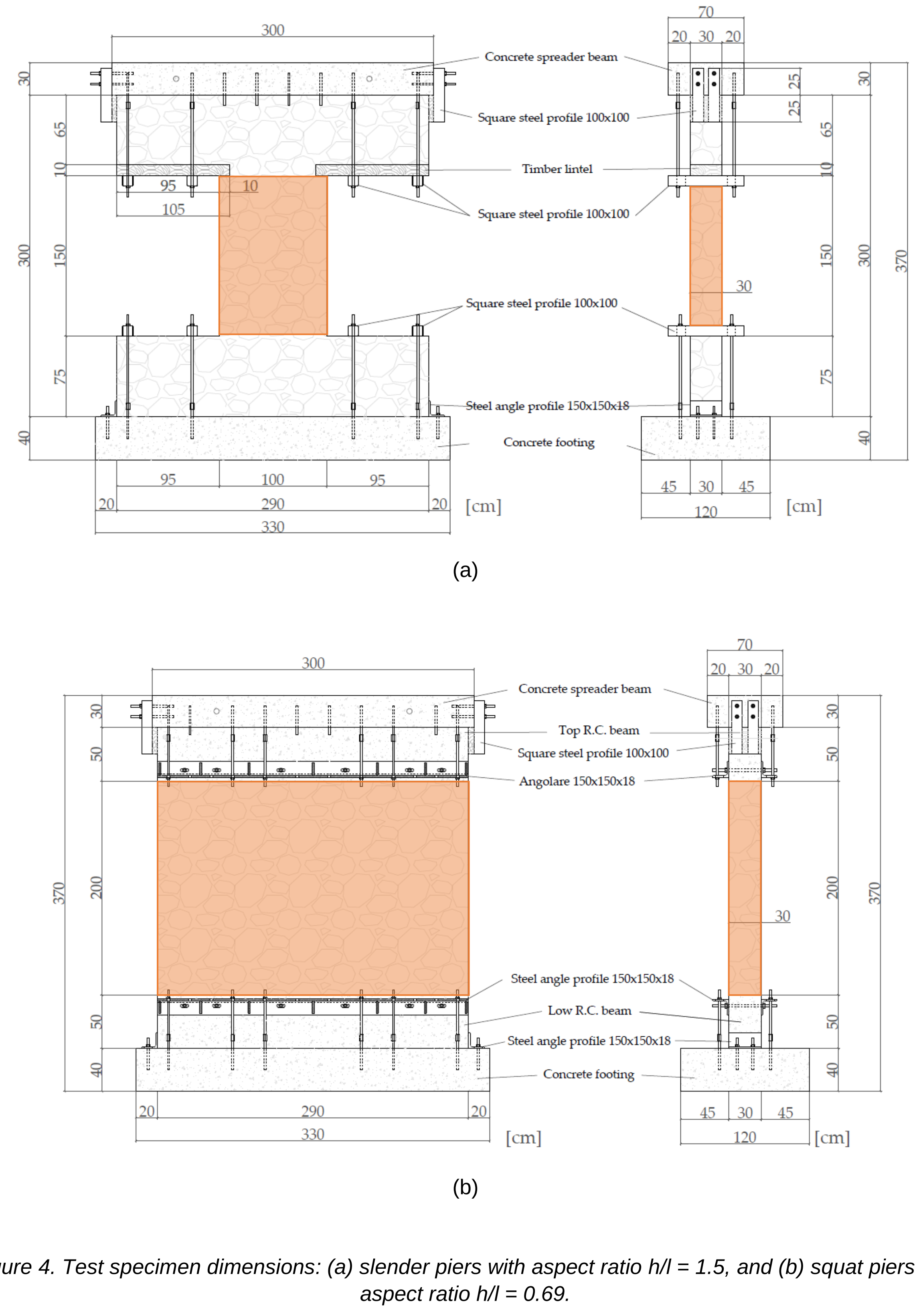



*Figure 4. Test specimen dimensions: (a) slender piers with aspect ratio h/l = 1.5, and (b) squat piers with aspect ratio h/l = 0.69.*

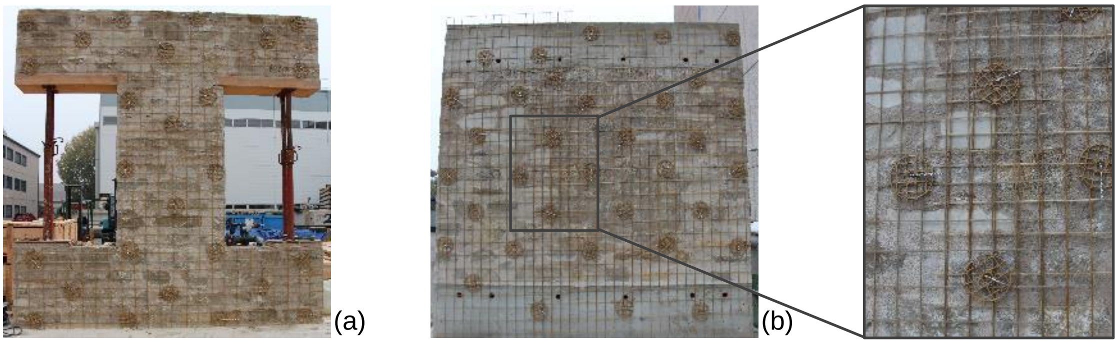



*Figure 5. Jacketing system: (a) slender piers, (b) squat piers, and (c) detail of connectors and mesh lap-splice.*

### 3.2 Testing protocol and set-up

The experimental setup for in-plane cyclic tests exploits the three-dimensional strong-wall/strong-floor configuration available at the EUCENTRE laboratory (Magenes et al., 2010; Guerrini et al., 2022). Three servo-hydraulic actuators are linked to a steel beam securely attached to the RC spreader beam: two vertical actuators apply the axial load and boundary conditions, while the horizontal one induces lateral displacements on the top RC spreader beam (Figure 6).

Two sets of actuators are employed to accommodate the large range of capacity between the tested specimens. For most specimens, a horizontal actuator with a maximum force capacity of 500 kN in tension/compression and vertical actuators with a maximum force capacity of 250 kN in tension and 500 kN in compression are used. However, for the two strengthened squat piers, a horizontal actuator with a maximum force capacity of 1000 kN and vertical actuators with a maximum force capacity of 500 kN in tension/compression are required.

To ensure a double-bending configuration, the vertical rotation of the pier top is restrained through a hybrid control of the vertical actuators. This control mechanism compels the actuators to apply a constant combined axial load while elongating or shortening by the same amount. Additionally, specific restraints are in place to prevent out-of-plane displacements of the pier top, allowing only longitudinal translation. A connection system, which is shown in Figure 4, was designed to prevent rocking or sliding between each specimen and the RC foundation or spreader beam.

40 displacement transducers are installed mainly on the hidden side that faces the strong wall and on the thickness of the specimens, to derive relative and absolute displacements and local deformations. A Digital Image Correlation (DIC) method is also adopted to measure the deformation and strain fields of the specimens. For that, a white and black pattern is created on the masonry surface using brushes. Forces are measured by load cells applied to each actuator head.

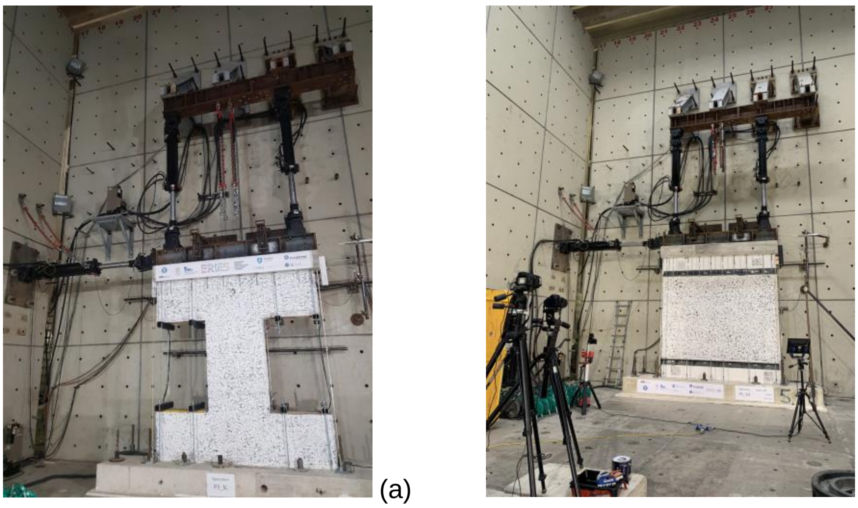



*Figure 6. Quasi-static cyclic shear-compression test setup: (a) slender specimen, and (b) squat specimen.*

An axial force equal to 20% of the pier's compressive strength is desired at its base. Given the pier nominal cross-section (Figure 4) and the masonry compressive strength of 1.98 MPa, this corresponded to a force of 119 and 344 kN for the slender and the squat specimens, respectively. Subtracting the weights of the masonry pier (8.2 kN for the slender or 32 kN for the squat specimen), top spandrel or RC beam (11 kN), RC spreader beam (16 kN), loading steel beam (7.2 kN), and half horizontal actuator (3.6 kN for the 500-kN capacity actuator or 11 kN for the one with 1000-kN capacity), the combined resultant force to be constantly applied by the pair of vertical actuators can be found depending on the pier aspect ratio and actuator set.

The horizontal actuator is initially set in force control, and the specimens are subjected to three push-and-pull cycles with a force amplitude of about 1/4 of the analytically predicted shear strength. The following sequence of three cycles has an amplitude equal to 1.5 times the previous one. Then, a protocol consisting of displacement-controlled sequences of increasing amplitude is followed. The test is stopped when the specimen reaches near-collapse conditions in terms of potentially dangerous damage patterns, a significant drop of lateral strength, or unstable behaviour under constant vertical load.

## 4 Prediction of the masonry piers' behaviour

The Italian guidelines (CNR, 2018) provide design provisions for FRCM systems used to strengthen existing masonry and RC buildings. Since there are no guidelines for the design of CRM systems, the formulations provided in CNR (2018) are adapted by using the tensile strengths per unit length $f_{weft}$ and $f_{warp}$, and the ultimate strain $\varepsilon_{fu}$ defined in part 2.3. Moreover, it is assumed that the effective height $h_{eff}$ is equal to the net height of the pier (as illustrated in Figure 4), i.e., without considering the upper and lower masonry spandrels for the case of the slender specimens.

The lateral strength of piers associated with flexural failure, $V_{Rf}$, considering two specific cases: (i) toe-crushing of the masonry (reaching the ultimate compressive strain $\varepsilon_m = \varepsilon_{mu} = 0.35\%$); (ii) brittle tensile failure of the GFRP mesh (reaching the ultimate tensile strain $\varepsilon_f = \varepsilon_{fu}$). The resisting moment $M_R$ is calculated for cases (i) and (ii) as per the following equations (2) and (3):

$$M_R = \frac{\alpha\beta f_c \mathrm{t}\, y_n}{2} \cdot (\ell - \beta y_n) + \frac{\varepsilon_{mu}}{y_n} \cdot \frac{\left(d_f - y_n\right)^2}{12} \cdot \frac{f_{vert}}{\varepsilon_{fu}} \cdot \left(2y_n + 4d_f - 3\ell\right) \tag{2}$$

$$M_R = \frac{\alpha\beta f_c \mathrm{t}\, y_n}{2} \cdot (\ell - \beta y_n) + f_{vert} \cdot \frac{d_f - y_n}{12} \cdot (2y_n + \ell) \tag{3}$$

In expressions (2) and (3), the neutral-axis depth $y_n$ is calculated respectively with equations (4) and (5):

$$y_n = \frac{\sigma_0 tL - \frac{f_{vert}}{\varepsilon_{fu}} \cdot d_f \varepsilon_{mu} + \sqrt{(\sigma_0 t\ell)^2 + 2 \cdot \frac{f_{vert}}{\varepsilon_{fu}} \cdot d_f \varepsilon_{mu}\left(\alpha\beta f_c t\, d_f - \sigma_0 t\ell\right)}}{2\alpha\beta f_c t - \frac{f_{vert}}{\varepsilon_{fu}} \cdot \varepsilon_{mu}} \tag{4}$$

$$y_n = \frac{f_{vert} d_f + 2\sigma_0 t\ell}{2\alpha\beta f_c t + f_{vert}} \tag{5}$$

In equations (2) through (5) $\ell$ is the length of the pier, *t* is the thickness of the pier, $d_f$ is the distance between the compressed edge and the fibre of the GFRP mesh furthest from it, $\sigma_0$ is the average vertical compressive stress on the section, $f_{vert}$ is the tensile strength per unit length of the mesh vertical threads, which for a slender pier corresponds to $f_{weft}$ and for a squat pier to $f_{warp}$, and $\alpha = 0.9$ and $\beta = 0.8$ are the rectangular stress-block factors for masonry in compression. After attaining the resisting moments for both the top and bottom sections of the pier, it is possible to obtain $V_{Rf}$ as:

$$V_{Rf} = \frac{M_{R,top} + M_{R,top}}{h_{eff}} \tag{6}$$

The lateral strength of a strengthened pier failing in shear is given by the sum of two contributions, that of the unreinforced masonry obtained according to current standards (MIT, 2018; MIT, 2019) and that of the strengthening system (CNR, 2018). Shear failure in undressed masonry typically involves two main mechanisms, sliding over a flexural crack $V_{m,sl}$ and diagonal cracking $V_{m,dc}$, as calculated per equations (7) and (8), respectively:

$$V_{m,sl} = \ell_c t f_{v0} + \mu\sigma_0 \ell t \tag{7}$$

$$V_{m,dc} = \frac{\ell t f_t}{b} \cdot \sqrt{1 + \frac{\sigma_m}{f_t}} \tag{8}$$

In equations (7) and (8), $b$ is a shape factor which takes a value of 1.5 for piers having $h/l \geq 1.5$ (slender piers) and a value of 1 when $h/l \leq 1$ (squat piers), $\sigma_m$ refers to the average vertical compressive stress at mid-height of the pier, $\sigma_0$ is the average vertical compressive stress on the sliding section, $f_t$ refers to the tensile strength of masonry evaluated using the diagonal compression tests, $\mu = 0.7$ is the masonry inherent friction coefficient, $f_{v0} = 1.25 f_t$ is the masonry cohesion, and $\ell_c$ is the length of the compressed area, which is defined in equation (9), being $\alpha_v = M/(V \cdot H)$:

$$\ell \cdot \frac{\sigma_0}{\alpha\beta f_c} \leq [\ell_c = 1.5\ell(1 - \alpha_v \frac{3f_{v0} + 2\mu\sigma_0}{\sigma_0 + 3f_{v0}\,\alpha_v})] \leq \ell \tag{9}$$

The shear contribution given by the CRM system $V_{CRM}$ is calculated per equation (10):

$$V_{CRM} = \frac{1}{\gamma_{Rd}} f_{horiz} n_f b_f \, \alpha_t \tag{10}$$

In equation (10), $n_f$ is the number of GFRP layers arranged on each wall face; $f_{horiz}$ is the tensile strength per unit length of the mesh horizontal threads, which for a slender pier corresponds to $f_{warp}$ and for a squat pier to $f_{weft}$; $b_f$ is the dimension of the CRM measured orthogonally to the shear force, which shall not be taken greater than the length of the pier $\ell$.

However, the shear strength of the retrofitted pier is limited by the following value calculated per equation (11), corresponding to the crushing of the diagonal masonry strut, $V_{Rmax}$:

$$V_{Rmax} = 0.25 \cdot f_c \cdot t \cdot d_f \tag{11}$$

As a consequence, the capacities associated with the two shear failure modes in a strengthened pier, $V_{Rsl}$ and $V_{Rdc}$, can be obtained as:

$$V_{R,sl} = V_{m,sl} + V_{CRM} \leq V_{Rmax} \tag{12}$$

$$V_{R,dc} = V_{m,dc} + V_{CRM} \leq V_{Rmax} \tag{13}$$

Given the strengths corresponding to the three mechanisms discussed above (i.e., $V_{Rf}$, $V_{R,sl}$, $V_{R,dc}$), the lateral strength of the pier can be taken as their minimum:

$$V_R = min\{V_{Rf};\, V_{R,sl};\, V_{R,dc}\} \tag{14}$$

The analytical $V_R - \sigma_{0,bot}$ interaction diagrams of the specimens are presented in Figure 7 and Figure 8, where the vertical stress level at the base is normalized by the masonry compressive strength $f_c$. It is worth noting that, when applying the previous equations to the strengthened specimens, the masonry compressive strength was magnified by a factor of 1.5 when the CRM was provided on both sides, and by a factor of 1.25 when it was provided on one side only, to account for the CRM confining effect. These values were supported by the preliminary results of vertical compression tests on strengthened wallettes.

The vertical blue dashed lines in Figure 7 and Figure 8 represent the vertical stress level that is applied to the piers during the tests. For the bare slender pier, failure occurs at the transition between the flexural and diagonal cracking modes, while for both strengthened slender piers it is clearly associated with flexure and controlled by tensile fracture of the mesh. In the cases of the squat piers, instead, failure is due to diagonal cracking, independent of the CRM application.

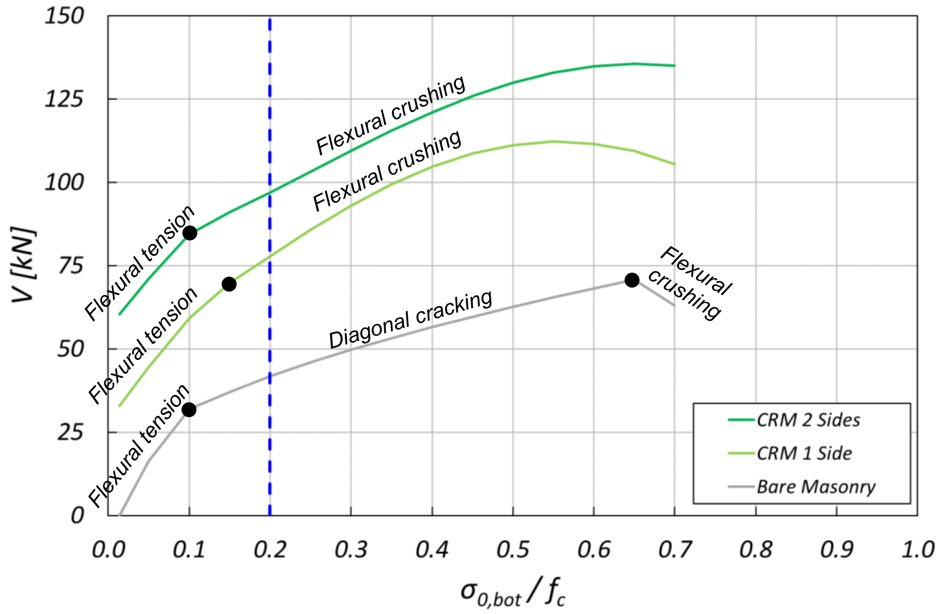



*Figure 7. Interaction diagrams for the bare and retrofitted slender piers with h/l=1.5.*

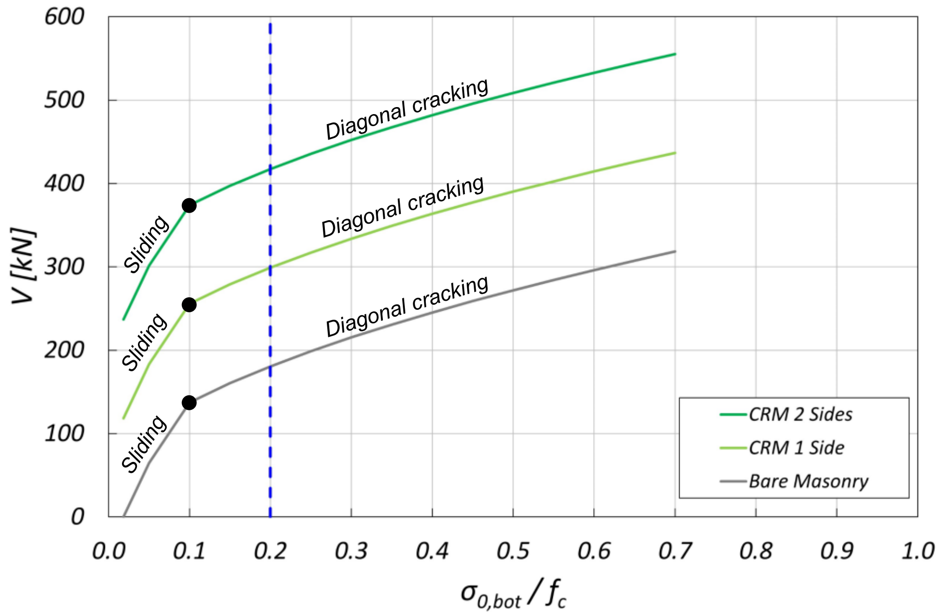



*Figure 8. Interaction diagrams for the bare and retrofitted squat piers with h/l =0.69.*

## 5 Concluding remarks and future developments

This paper presents an overview of the ERIES-RESTORING project that comprises an experimental campaign to study the behaviour of existing undressed stone masonry buildings when strengthened with CRM, through quasi-static cyclic in-plane shear-compression tests of 6 full-scale pier specimens. The characterisation tests are first carried out by the Department of Civil Engineering and Architecture of the University of Pavia, and the quasi-static cyclic in-plane shear-compression tests at the EUCENTRE Foundation laboratories in Pavia, Italy.

The experimental campaign addresses 3 configurations: unreinforced (bare) masonry, serving as a reference; CRM strengthening on one side of the wall; and CRM strengthening on both sides. The CRM solution consists of a glass-FRP mesh embedded in a mortar compatible with the substrate of historical buildings, with connectors distributed across the facade to provide transverse confinement. Each retrofit configuration is tested under double-fixed conditions for two height/length pier aspect ratios, i.e. 0.69 (squat) and 1.5 (slender). The slender specimens include parts of the adjacent spandrels to allow CRM anchorage, as in the case of masonry piers between windows. On the other hand, the squat piers correspond to walls without openings or with large spacing between them, and have top and bottom RC beams where the GFRP mesh is anchored.

The characterisation of the materials, specimen description, test set-up and procedure are presented in detail. The analytical estimates of the lateral strength and failure modes of the bare masonry and retrofitted specimens are presented for the two aspect ratios. The formulations used to calculate these estimations were adapted from the Italian guidelines for the design of FRCM systems applied to masonry and reinforced concrete buildings, since there are no available guidelines for the design of CRM retrofits. After testing the specimens, the analytical estimates will be validated with the experimental results.

The outcomes of this experimental campaign will serve as a basis for the future development of building codes or design guidelines for the CRM retrofit of masonry. This will be particularly valuable for engineers aiming to design effective CRM retrofit solutions using composite meshes and inorganic mortars, ensuring compatibility with the preservation of historical buildings. Such an approach promotes more sustainable decision-making compared to partial or total replacements.

The results of the experimental campaign will also provide insights into the effects of CRM application on single or both sides of undressed stone masonry walls. This information will allow engineers to make well-informed decisions, striking a balance between conservation and safety considerations. While most studies address only the case of strengthening both sides of the walls, as advised by manufacturers, practical constraints in real buildings may limit this approach, especially when one side is inaccessible or covered by decorative elements.

## 6 Acknowledgements

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