Available online at www.sciencedirect.com

ScienceDirect

Structural Integrity Procedia 00 (2022) 000–000

Structural Integrity Procedia

www.elsevier.com/locate/procedia

XIX ANIDIS Conference, Seismic Engineering in Italy

Investigating the effectiveness of a CRM system: full scale reverse cyclic tests on a two-storey rubblestone masonry building

N. Gattesco[a], E. Rizzi[a]*, L. Facconi[b], F. Minelli[b], A. Dudine[c]

[a]University of Trieste, Department of Engineering and Architecture, Via Alfonso Valerio 6/1, 34127 Trieste, Italy
[b]University of Brescia, Department of Civil, Environmental, Architectural Engineering and Mathematics, via Branze 43, 25123 Brescia, Italy
[c]Fibre Net S.p.a., Via Jacopo Stellini, 3 – Z.I.U., 33050 Pavia di Udine (UD), Italy

Abstract

The research work aims at investigating the effectiveness of a minimally invasive strengthening technique used to repair a full scale two-storey building, consisting of 350 mm thick two-leaf rubblestone masonry walls, a timber floor, and a timber roof with clay tiles. The strengthening technique consists in applying on the external façade of the building a single layer of a Composite Reinforced Mortar (CRM) System, which consists in a mortar coating containing a Glass Fiber Reinforced Polymer (GFRP) mesh; artificial diatons connecting both wythes of the walls were also applied. The reverse cyclic load was applied to each of the two longitudinal walls of the building by means of a servo-controlled hydraulic jack pinned to a vertical steel beam. This beam allowed splitting the total lateral force into two forces, acting at first story and roof levels; these forces were calculated proportional to the product of the floor mass with the floor level.
The first cyclic test was carried out on the Unreinforced Masonry (URM) building up to attaining a damage level quite close to that corresponding to the ultimate limit state of the structure. Then, the building was repaired (RM) with the proposed technique and tested again, up to reaching a near-collapse condition. The experiments proved the effectiveness of the proposed strengthening method showing a resistance increment of 240%, a larger displacement capacity (150%) and a significant increase of the total dissipated energy. The importance of the artificial diatons to prevent the separation of the masonry leaves in strengthened walls was also clearly highlighted.

Keywords: Composite structures; Glass Fiber; Masonry structures; Earthquake engineering

* Corresponding author. Tel.: +39 339 7057482.
E-mail address: emanuele.rizzi@units.it

2452-3216 © 2022 The Authors. Published by ELSEVIER B.V.
This is an open access article under the CC BY-NC-ND license (https://creativecommons.org/licenses/by-nc-nd/4.0)
Peer-review under responsibility of the scientific committee of the XIX ANIDIS Conference, Seismic Engineering in Italy

1. Introduction

Unreinforced stone masonry buildings represent traditional residential constructions in hilly and mountain areas of different European regions and countries worldwide. Most of these buildings have a maximum of three storeys; the floors are made mainly with timber joists and perpendicular boards or with concrete/steel beams and brick labs laid on the masonry bearing walls. Existing buildings typically present a high seismic vulnerability, as they were designed before the introduction of seismic provisions in structural codes. Therefore, many studies have focused on the development of new retrofitting techniques able to improve the structural safety under seismic events.

Various technologies for strengthening unreinforced masonry (URM) buildings are available. The traditional strengthening techniques are usually time consuming and require the users to temporarily move out of their buildings, thus new single side reinforcement approaches were developed. Different studies have investigated on the effectiveness of fiber-reinforced coatings, applied in single and double-sided configurations. In particular Lucchini et al., 2021 investigated on the effectiveness of a steel fiber-reinforced mortar (SFRM) coating, applied only on the outer surface, evidencing a significant improvement of the seismic behaviour of a hollow clay block masonry building. Other authors (Gattesco and Boem, 2015; Del Zoppo et al., 2019) have studied a CRM (Composite Reinforced Mortar) System applied in single and double-sided configurations, which evidenced a significant increase of the resistance and the ductility of the masonry elements tested. Compared to traditional steel reinforced cement coatings, the use of glass fiber reinforced polymer (GFRP) meshes guarantees increased structure durability, in terms of corrosion potential and ease of application. Many historical buildings present masonry units arranged with a multi-leaf layout. On these structures, single-side reinforcement configurations have presented important criticalities (i.e., out of plane displacements; separation of wall leaves) that need to be mitigated through other interventions, thus other studies (D'Antino, Carozzi and Poggi, 2019) have evidenced the effectiveness of artificial diatons in preventing the wall leaves separation.

Except for the results provided by the abovementioned tests, no information is available on the structural performance of full-scale buildings retrofitted with the CRM strengthening technique applied only on one side of the masonry walls. Thus, in the present research, quasi-static reverse cyclic tests were carried out on a pre-damaged full-scale two-leaf stone masonry building, retrofitted with a CRM System (30÷40 mm thick mortar coating) applied only on the outer surface, with the addition of artificial diatons connecting both wall leaves.

A fundamental premise is that with the application of the CRM System only on the external façade, even if more economical, and practical, the attainable seismic performance is governed by the limited availability of bearing walls that can be strengthened. Therefore, the advantages resulting from the adoption of the technique are closely related to the seismic hazard of the building site.

2. Test program

As observed, the CRM coating is applied only on the outer surface of the building so that, in real case scenarios, the retrofitting intervention avoids the need to find a different accommodation for people.

The experimental tests presented in this paper were carried out at the University of Brescia (Italy). In addition to the tests on a full-scale masonry building, the experimental program included a series of structural and material tests on stone and clay brick masonry samples, either unreinforced or strengthened with single or double-sided configurations, such as shear-compression and out of plane bending tests on walls, shear-bending tests on spandrels and out of plane bending tests on tie-beams (Gattesco et al., 2022).

The full-scale structure was designed to represent a typical stone masonry building constructed in most parts of Italy between the 1920 and 1980. The test was subdivided into two stages to simulate a repairing intervention on a building that was pre-damaged by a significant seismic event.

3. Specimen description and masonry properties

The two-story test building consisted of four unreinforced masonry (URM) walls (i.e., North, West, South, and East wall) with wooden floor and roof. A view of the whole building is displayed in Fig. 1. The construction of the building was completed in 65 days. During that period, the mortar used to construct the walls and the coating was sampled and then tested to assess the compressive strength.

The structure has in-plane dimensions 5,750 mm by 4,350 mm with a total height of 6733 mm (Fig. 2). The aspect ratio (height to width) of the piers ranged from 0.7 to 2.4 to promote, as much as possible, both the shear and flexural collapse mechanisms. The loading was applied in the plane of West and East walls (North-South direction).

(a) (b)

Fig. 1. View of the unreinforced test building: (a) East and North walls; (b) North and West walls.

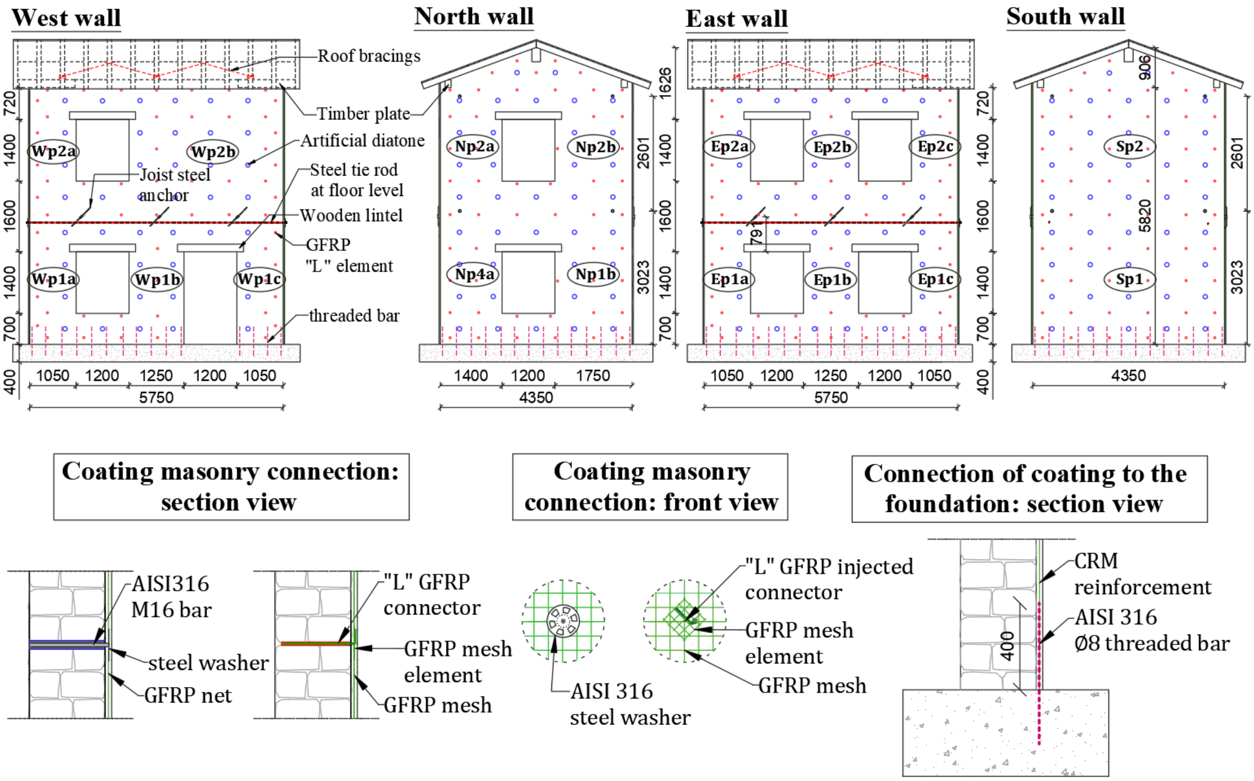


Fig. 2. Drawing of the building walls and details of the retrofitting technique (dimensions in millimeters).

The 350 mm thick bearing walls are made of sandstone masonry units with approximate dimensions of 210 (length) x 120 (width) x 100 (height) mm, laid in a two-leaf masonry configuration. Both head and bed joints have an average thickness of 10 mm, and they are filled with a lime-based mortar with a mix ratio of 1:7 (i.e., 200 kg hydraulic lime and 1400 kg sand per m^3 of mortar). A grain size curve of sand typically used for mortar in historic masonry buildings was adopted. The characterization tests performed on mortar samples resulted in an average compressive strength of 1.5 MPa and an average flexural tensile strength of 0.5 MPa, both measured at 28 days air curing. The average specific weight of masonry was about 21.0 kN/m^3. The first masonry row of the building was effectively connected to the foundation by 150x150x120 mm^3 reinforced concrete blocks, which were cast in-situ and embedded within the wall thickness to prevent potential sliding phenomena occurring at the masonry-concrete interface (the foundation is actually made of masonry as that of the wall).

The wooden floor consists of 9 solid timber long joists, which are located into masonry pockets providing a support length of 150 mm. Timber boards with a thickness of 25 mm are nailed to the top of the joists. Three of the joists composing the floor are connected to East and West walls by means of steel joist anchors, with a center distance of 1800 mm, passing through the masonry wall and clamped by a steel wedge. The wooden pitched roof consists of 26 solid timber joists connected on one end to a solid timber ridge beam. The other end of the joists is laid on the longitudinal walls. Timber boards with a thickness of 25 mm are nailed on top of the joists. All wooden elements are made of red spruce. Two parallel 1500 mm long wooden lintels, with a section of 170 x 170 mm, are placed over each opening and laid on masonry. The lintels have an end support of 150 mm on each side.

It is worth remarking that the wooden floor, in absence of stiffening elements, allows for the uneven distribution of seismic actions towards the bearing walls.

4. Details of the retrofitting technique and properties of CRM

After the test on the URM building, the construction was strengthened using 50 mm diameter artificial diatons and a 30÷40 mm thick layer of a reinforced coating applied on the outer surface of the perimeter walls (Fig. 2), whereas cracks on the inner surface were superficially repaired by the injection of low viscosity cementitious mortar. The GFRP mesh reinforcement embedded within the coating thickness has a 66 x 66 mm^2 grid dimension and is formed by wires with a cross-section area of 11.6 mm^2 and 8.9 mm^2 for parallel and twisted fiber mesh wires, respectively.

The L-shaped connectors having a cross section of 7 x 10 mm^2 and a nominal fiber cross area equal to 32.4 mm^2 were injected into masonry to a depth of 300 mm with an epoxy resin. Artificial diatons were made with a 16 mm diameter threaded steel bar centered in a 50 mm diameter hole and embedded in a high resistance thixotropic cement-based mortar.

To connect the mortar coating to the diatons, perforated stainless steel washers (Fig. 2) with a 16 mm nut welded to them were fixed on the diaton threaded bar at half thickness of the CRM coating, above the GFRP mesh. Holes were drilled using a core drilling machine. The arrangement of L-shaped connectors ($4/m^2$) and artificial diatons ($2/m^2$) is illustrated in Fig. 2. For the CRM coating, a Natural Hydraulic Lime (NHL) mortar was used, with an average compressive strength of 15.3 MPa and an average flexural tensile strength of 3.4 MPa, measured at 48 days air curing, The average specific weight of the mortar was about 18.0 kN/m^3.

Before applying the CRM coating, the mortar joints were removed for a depth of 10 mm and the masonry surface was washed with a high-pressure water cleaner to remove the white hydraulic lime paint and to moisten the masonry surface, to promote a better adhesion of the CRM coating. Note that the white paint was applied on the façade to make the detection of cracks easier during the URM test.

Threaded stainless 8 mm diameter steel bars, (characteristic yield strength 200 MPa) were anchored to the RC foundation by an epoxy resin, injected in predrilled holes having a depth of 250 mm. As shown in Fig. 2, these bars were put along the entire perimeter of the building at about 10 mm from the masonry wall surface arranging three per meter, to provide a good connection between the CRM coating and the concrete foundation. The threaded bars were installed beneath the GFRP mesh to avoid possible splitting failure mechanisms of the mortar coating. Previous pull-out tests (Boem and Gattesco, 2021) carried out on 6 mm diameter threaded steel bars showed that an approximate length of 50 times the diameter is enough to anchor the bars in the reinforced mortar coating. Along the four external corners of the building, GFRP angular preformed meshes with a width of 330 mm were used to guarantee the continuity of the reinforcement.

5. Experimental apparatus and test procedure

Fig. 3 illustrates the loading setup adopted. A couple of in-plane lateral forces were applied on the East and West walls at the first and second story levels. To enable lateral load reversal, the lateral force was transferred from the South to the North side by eight (bar diameter equal to 27 mm) steel rods placed on both sides of the two longitudinal walls. Two 1,500 kN hydraulic servo actuators connected to a reactive RC wall were used to apply the total lateral force (F_{tot}). A force equal to $0.49 \cdot F_{tot}$ was applied at the top and a force equal to $0.51 \cdot F_{tot}$ was applied at the first story. These forces were applied to the building through a vertical steel device hinged to the actuator. The adopted vertical distribution of lateral forces was proportional to the product of the floor mass with the corresponding floor level.

Added gravity loads by means of concrete blocks on the first story and clay bricks on the second were used to simulate a typical seismic combination load. By combining the loads reported with the self-weight of masonry, the expected value of the vertical stress acting at the base of the building ranged from a minimum of 0.16 MPa (South wall) to a maximum of 0.22 MPa (East wall), while at the first story ranged from a minimum of 0.08 MPa to a maximum of 0.11 MPa.

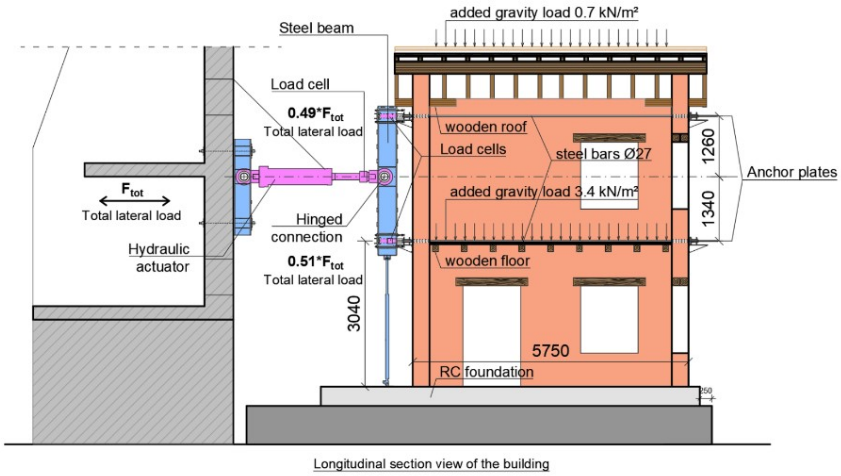


Fig. 3. Loading system apparatus.

Four load cells C1SW, C2SW, C1SE, C2SE allowed to measure the applied load at first floor and roof, respectively, for the West and East walls. Two load cells of the same type were installed on the actuators as well as two LVDT (liner variable displacement transducer) to serve as displacement control instruments. Horizontal linear potentiometer transducers were arranged both at the first floor (i.e., H1SW-H1SE) and at the roof level (i.e., H2SW-H2SE), to survey the lateral displacement of the building. Many linear potentiometers were installed on masonry piers and spandrels to check in plane vertical and diagonal displacements. On the East wall façade, a digital image correlation system (DIC) was adopted to record the in-plane displacements and to identify the formation of cracks.

For the test, the horizontal displacement at top level of the building was cyclically varied between increasing couples of opposite sign displacements (H2SW, H2SE). Each loading cycle between a couple of opposite values of the displacements were repeated two or three times. The experiment was governed through a computer arranged with a software to control both actuators and to force the same top displacement at WEST and EST wall at the same time. The URM test was stopped when the damage level was quite close to that corresponding to the ultimate limit state of the structure; then the retrofitting operations were carried out. A second test on the RM building was carried out with a similar procedure as for URM up to reaching a level of damage quite close to collapse.

6. Experimental results

The global response of the building may be summarized by the diagram of the total base shear load (V_b) against the 2nd story lateral displacement δ_2 determined with the average measure of instruments H2SW and H2SE.

The 2nd story drift (γ_2) was determined with the average measure of instruments H2SW and H2SE divided by their heigth from the foundation, equal to 5640 mm (Fig. 3). The maximum values of V_b, δ_2 and γ_2 obtained in URM and RM building tests are summarized in Table 1.

Table 1. Maximum values of the base shear (V_b), 2nd story lateral displacement (δ_2) and 2nd story drift (γ_2) measured in positive and negative directions.

Test building	Sign	$V_{b,max}$ [kN]	$\delta_{2,max}$ [mm]	$\gamma_{2,max}$ [%]
URM	+	267	19.68	0.35%
	-	256	17.17	0.30%
RM	+	645	78.95	1.55%
	-	590	45.35*	0.89%*

(*) the values of the displacement and drift do not correspond to the maximum ones because the test was stopped after loading in the positive loading direction.

6.1. URM building

The URM building was tested after about six months from the construction. It exhibited the formation of single diagonal cracks on the slender outer piers as well as horizontal cracks both at the top and at the bottom end of the central squat pier of the East wall (Fig. 4b). The most significant cracks affected the piers of the first story whereas those detected at the ground story were clearly smaller. The path of the cracks generally followed mainly the mortar joints.

The experimental curve presented in Fig. 4a remained linear until the first flexural cracks occurred at top and bottom toes of the pier Ep2b (Fig. 2) for a total base shear load V_b of about ±70 kN (δ_2 = ±0.9 mm). Once a displacement of around δ_2=1.2 mm was attained (V_b = ±77 kN), the stiffness of the building gradually reduced as cracks developed close to the corners of the openings and horizontal cracks occurred at the top of the pier Ep2a. A further reduction of the lateral stiffness was caused by shear diagonal cracks grown on the West and East wall spandrels (V_b = ±110 kN, δ_2=2.1 mm). In particular, the shear crack of the spandrel below the pier Wp2b extended to the corner of the 2nd floor window. With increasing lateral positive displacements, shear cracks formed on piers Ep2a and Ep2b, that led the building to the attainment of the maximum capacity, which was equal to +267 kN. After that it slightly decreased to +251 kN as a diagonal crack formed in the pier Ep2b. Such a behavior prevented the structure to further increase the global resistance. Based on the damage propagation discussed above, it may be observed that a 2nd story collapse mechanism regulated the response of the structure.

During the test, no up-lifting phenomena of the building were observed at the base of the walls.

6.2. RM building

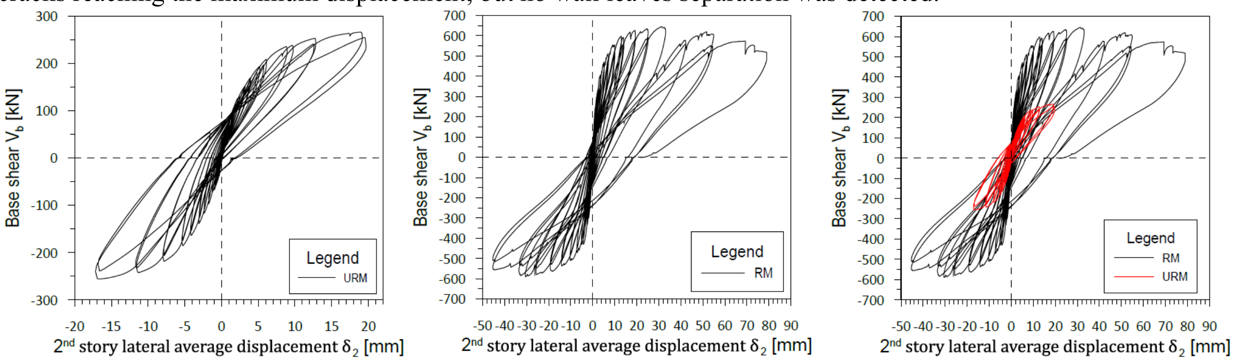
After about four months from the end of the first test, the building was retrofitted and then tested again. Some minor microcracks were detected around the openings on the coating, due to plastic shrinkage.

The experimental curve presented in Fig. 4b is almost linear until the first flexural cracks occurred at top and bottom of the piers Ep1a, Ep1b, Ep1c, Wp1a, Wp1b on the unreinforced side, for a total base shear load V_b of about ±90 kN (δ_2 = ±0.3 mm). Once a displacement reached almost δ_2=0.4 mm (V_b = ±110 kN), the stiffness of the building gradually reduced as cracks developed on the coating, close to the corners of the first story openings. With increasing lateral deflection, the same type of cracking occurred near the second story openings, while shear diagonal cracks have grown on the coating of the West and East wall spandrels (V_b = ±235 kN, δ_2 = ±1.2 mm), which caused a further reduction of the lateral stiffness.

At a lateral deflection of about δ_2 = ±3.2 mm (V_b = ±390 kN), the flexural cracks of the first story piers and the shear diagonal cracks of the spandrels began to spread locally, while shear cracks involved the first story piers. This caused an increasing reduction in stiffness with each new cycle. After reaching a lateral deflection of about δ_2 = ±9.2 mm (V_b = ±525 kN), cracks widely spread on piers and spandrels, which caused a significant reduction of the global lateral stiffness. The complete formation of the flexural and shear diagonal cracks on the coating (Fig. 5a, b) of the first story piers caused the attainment of the maximum shear capacity, equal to +645 kN and -590 kN in the positive and negative loading directions, respectively. After the peak load, a gradual decrease in shear capacity was observed. Once the displacement reached δ_2 = -33.0 mm, the first story piers suffered a significant opening of their cracks, promoting a sharp reduction of the global lateral stiffness in the subsequent cycles. Once reaching an after peak resistance decrease of about 15% (δ_2 = ±45.0 mm, V_b = ±520 kN), the test was conducted monotonically, in the positive loading direction, up to near collapse, that happened for a lateral displacement δ_2 = ±69.3 mm and a total base shear load V_b = +573 kN. After that it decreased to +509 kN as the GFRP mesh failed at the base of the piers Ep1b and Wp1a. At the end of the test, the pier Wp2b did not show any damage. Close to the maximum displacement reached a horizontal crack formed at the bottom of the piers Np1b and Wp1a, causing a horizontal slippage of the whole North-West corner.

In the elastic phase, up to a lateral deflection of δ_2 = ±3.2 mm, the global overturning of the building led to an uplift of about 0.1–0.2 mm detected by the potentiometers V1W, V13W, V33E. In the plastic phase, some vertical splitting cracks occurred (δ_2 = +33.0 mm) along the threaded bars connecting the CRM coating of the South wall to the building foundation. The vertical uplift detected by the instrument V1W, was about 16.6 mm at the highest measured top lateral displacement. Based on the damage propagation discussed above, it may be observed that a 1st story collapse mechanism regulated the response of the structure.

Only two artificial diatons, located at half height of the piers Ep1b and Wp1b were interested by the diagonal shear cracks reaching the maximum displacement, but no wall leaves separation was detected.



(a) (b) (c)

Fig. 4. Shear against 2nd story average lateral displacement curves: (a) URM sample; (b) RM sample and (c) comparison between URM (red) and RM (black) samples.

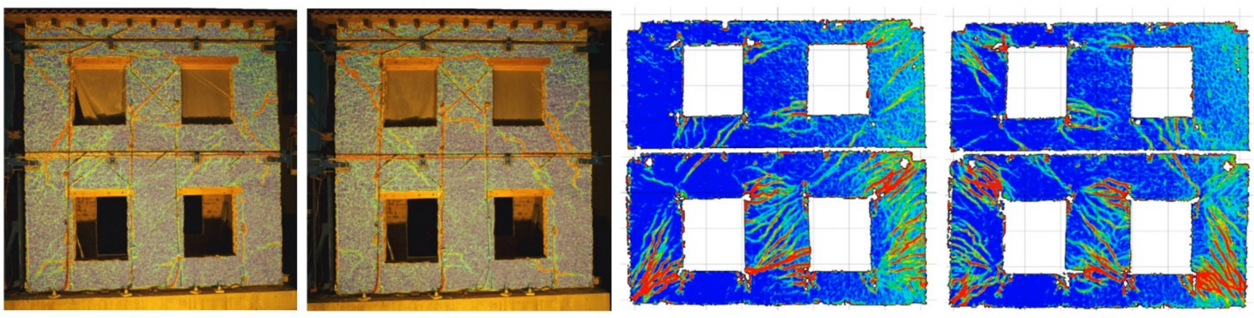

(a) (b) (c) (d)

Fig. 5. Crack configuration on the East wall at the end of the test (DIC System): (a) URM sample in negative and (b) positive load directions, (c) RM sample in negative and (d) positive load directions.

7. Conclusions

A 30 mm thick CRM coating, made with a GFRP mesh reinforcement (66 x 66 mm^2 grid dimension) embedded within a natural hydraulic lime mortar (15 MPa), was applied on the outer surface of a two-leaf rubblestone unit masonry building. To avoid the separation of leaves, artificial injected diatons were applied. After strengthening, the structure was tested under quasi-static reverse cyclic loading, by taking into account a displacement-controlled test protocol. By comparing the results of the tests carried out on the URM building with those obtained on the RM construction, the following main considerations can be drawn:

- The lateral resistance of RM with respect to URM building increased by about 240%, whereas the displacement capacity was 150% larger and, as evidenced by the wider hysteretic cycles, the total dissipated energy increased considerably.
- In the URM masonry, the damage (shear and bending cracks) occurred mainly in the second floor walls, because of the low value of the axial load in the piers. Differently, in the RM sample the flexural resistance of the reinforced piers is mainly due to the CRM System, thus the collapse occurred at the first story, where the force applied is higher.
- Based on the present test as well as in previous experiences (Gattesco et al., 2022), the adoption of 2 artificial diatones per square meter proved to be effective in preventing masonry leaves separation, as no wall bulging phenomena were observed in the RM test.
- The reinforced building test evidenced the importance of the connection between the coating and the foundation. The connection detail should be further improved to prevent the vertical uplift phenomena of the building.

The results discussed herein provided important information concerning the structural performance of the proposed strengthening technique.

Acknowledgements

The experimental tests presented have been developed within the project CONSTRAIN, funded by the Interreg Italy-Slovenia Cooperation Programme 2014-2020; leaded by the University of Trieste (Italy), alongside with the University of Ljubljana (Slovenija) and the companies FibreNet S.p.A., Igmat d.d., Veneziana Restauri Costruzioni S.r.l. and Kolektor CPG d.o.o..

References

Boem, I. and Gattesco, N. (2021) 'Cyclic behavior of masonry barrel vaults strengthened through Composite Reinforced Mortar, considering the role of the connection with the abutments', *Engineering Structures*, 228, p. 111518.

D'Antino, T., Carozzi, F.G. and Poggi, C. (2019) 'Diagonal shear behavior of historic walls strengthened with composite reinforced mortar (CRM)', *Materials and Structures*, 52(6), p. 114.

Del Zoppo, M. *et al.* (2019) 'In-plane shear capacity of tuff masonry walls with traditional and innovative Composite Reinforced Mortars (CRM)', *Construction and Building Materials*, 210, pp. 289–300.

Gattesco, N. *et al.* (2022) 'Study on the effectiveness of a CRM system: in-plane and out-of-plane cyclic tests on masonry piers', *Structural Integrity Procedia* [Preprint].

Gattesco, N. and Boem, I. (2015) 'Experimental and analytical study to evaluate the effectiveness of an in-plane reinforcement for masonry walls using GFRP meshes', *Construction and Building Materials*, 88, pp. 94–104.

Lucchini, S.S. *et al.* (2021) 'Cyclic Test on a Full-Scale Unreinforced Masonry Building Repaired with Steel Fiber-Reinforced Mortar Coating', *Journal of Structural Engineering*, 147(6), p. 04021059.