ASSOCIAZIONE NAZIONALE ITALIANA DI INGEGNERIA SISMICA ANIDIS
Italian National Association of Earthquake Engineering

XVIII CONVEGNO ANIDIS
L'ingegneria sismica in Italia
ASCOLI PICENO 2019
15-19 Settembre

Retrofitting Masonry Buildings by Steel Fiber Reinforced Mortar Coating: from the experimental tests to a practical design approach

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Keywords: masonry, buildings, cyclic tests, Steel Fiber Reinforced Mortar, coating, design, guidelines

ABSTRACT
A comprehensive experimental research has been performed in the last few years at the University of Brescia to develop a Fiber Reinforced Mortar (FRM) coating based technique suitable for retrofitting existing masonry buildings. Unlike traditional strengthening coating techniques (i.e., steel or glass/polymeric fiber reinforcing mesh), the one proposed adopts thin (i.e., 25-50 mm thick) overlays applied on one or both sides of the bearing walls. Moreover, short (30 mm long) high strength steel fibers uniformly spread within the mortar matrix are used as the only reinforcement of the coating layer in place of typical reinforcing meshes.
The paper summarizes the main results provided by the experimental tests performed to assess the effectiveness of the proposed technique. The most significant responses resulting from the material characterization tests and from the cyclic tests on a full-scale two-story building will be briefly described and discussed.
Finally, considering the lack of code recommendations concerning the design of FRM coating, a first proposal of guidelines for the in-plane verification of masonry panels strengthened with FRM coating will be presented. The verification approach is based on a simple strut and tie analytical model calibrated by a numerical parametric study.

1 INTRODUCTION

The high vulnerability of existing masonry buildings to seismic actions have driven the research toward the development of ever new retrofitting techniques as well as the improvement of the traditional strengthening methods.

Among the different techniques available, a possible retrofitting method consists in applying fiber reinforced mortar coating on the UnReinforced Masonry (URM) surface (Hutchinson et al. 1984; Sevil et al., 2011; Facconi et al., 2018). Although some experimental works have been carried out by several researchers (Magenes et al. 1995; Benedetti et al. 1998; Dolce et al. 2008; Mazzon et al. 2010; Magenes et al. 2012), the literature provides a limited number of case studies concerning tests on full-scale buildings made with hollow-clay block masonry. Furthermore, about tests on masonry retrofitted with reinforced coating, the experimental tests reported by literature (Lin et al. 2010; Gattesco et al. 2015; Giaretton et al. 2018) generally focus on small structural elements. On the contrary, the effect of coating on the global response of a full-scale building has not been deeply investigated at the moment.

The experimental and analytical work herein presented focuses on the study of a hollow-clay block masonry building that was tested under quasi-static cyclic loading before and after retrofitting with Steel Fibers Reinforced Mortar (SFRM) coating applied only on the outer surface. The test aimed at assessing the ability of coating to improve the in-plane structural behavior of the masonry walls. Therefore, the building was carefully designed to prevent local mechanisms, including the out-of-plane failure of the bearing walls. The latter part of the paper presents an analytical model able to predict the in-plane resistance of masonry elements strengthened with fiber reinforced mortar/concrete coating. The model is rather simple to use in practical design applications. Moreover, its formulation is suitable for a future implementation in a structural code.

2 EXPERIMENTAL PROGRAM

2.1 *Properties of the full-scale building and description of the retrofitting technique*

To investigate the effectiveness of SFRM coating to retrofit URM buildings, a full-scale building was constructed at the University of Brescia and tested under quasi-static cyclic lateral loading conditions. The test was carried out before and after the retrofitting intervention.

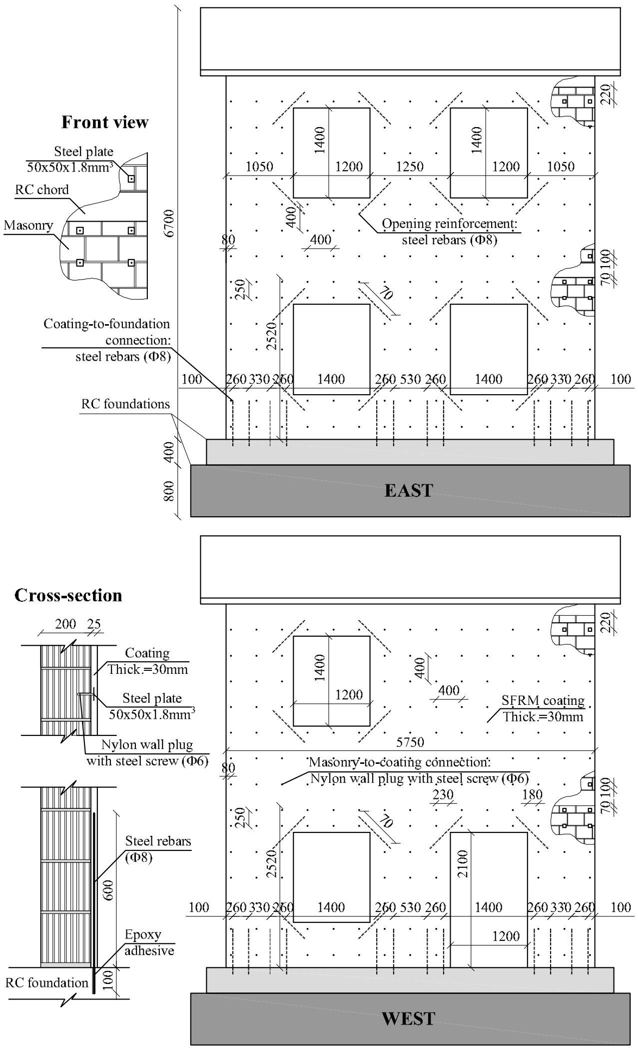

Figure 1: Main details of the retrofitting intervention.

The structure was designed to represent the behavior of an existing building in which floors have a high in-plane stiffness and an effective connection with the bearing walls. Such an assumption aims both at preventing any out-of-plane mechanism and at promoting a global-type response of the building when subjected to earthquake actions. The masonry building had a total length of 5750 mm, a width of 4250 mm and a maximum height of 6700 mm. The 200 mm thick single wythe walls were made of vertically perforated block masonry. Each floor consisted of 120 mm (width) x 140 mm (height) wooden joists provided with a 20 mm thick wooden plank. A total of twenty 200 mm spaced steel dowels (diameter=16 mm; length = 130 mm) were inserted into pre-drilled holes to connect each joist to the 60 mm thick Reinforced Concrete (RC) slab laid on the plank. The 500 mm spaced joists were placed perpendicular to the longitudinal walls, namely East and West façade. The RC chords placed at the floor levels allowed connecting the simply supported floor (i.e., the joist and the RC slab) with the four walls. The second wooden-concrete floor supported the pitched wooden roof that covers the whole building. Finally, all the masonry walls were provided with strip RC foundation connected to the strong floor.

After pre-damaging the building, a 30 mm thick SFRM coating was applied on the external surface of the four façades. The procedure adopted to apply the SFRM coating is fully described in Lucchini et al. (2018). Figure 1 shows the main details of the retrofitting technique, including the layout of the masonry-to-coating connections as well as the properties of the rebars used to connect the coating layer to the RC foundation.

2.2 *Mechanical characterization tests*

The masonry walls were made with hollow clay blocks having dimensions 250x200x190 mm^3 and a high percentage of holes (62%) oriented normally to mortar bed joints. The compression tests performed on 32 clay units according to EN 772-1:2015 provided a mean compressive strength parallel and perpendicular to perforations equal to 12.7 MPa (CV=13.5%) and 2.5 MPa (CV=10.1%), respectively. A ready-mix commercial cement-lime-sand mortar was used to fill the 10 mm thick bed and head joints. Flexural and compressive tests performed on 30 40x40x160 mm^3 mortar prisms according to EN 1015-11:2007, provided a mean flexural strength of 3.4 MPa (CV=7.2%) and a mean compressive strength of 8.9 MPa (CV=10.3%).

To determine the compressive strength of masonry, whose unit weight was 745 kg/m^3, a total of 6 uniaxial tests on masonry prisms were carried out according EN 1052-1:2001. Specimens with both vertical and horizontal holes were investigated (3 specimens for type). The wallets dimensions were 820x510x200 mm^3 and 950x590x200 mm^3, respectively. The former were characterized by a mean compressive strength of 2.9 MPa (CV=28.8%) and an elastic modulus of 8980 MPa. The latter presented a mean compressive strength of 0.6 MPa (CV=22.1%) and an elastic modulus of 1610 MPa. Note that the elastic moduli were calculated as the secant slope of the compressive stress-strain curve from the origin to 1/3 of the peak strength. Diagonal compression tests were also performed on $1200x1200x200\ mm^3$ square panels according to ASTM E 519-02:2002. Those tests provided a mean shear strength of 0.26 MPa (CV=24.0%) and a corresponding shear strain of 0.28 mm/m.

The SFRM coating applied on the building façades consisted of a ready-mix cement-based mortar containing 60 kg/m^3 (0.76% by volume) of randomly diffused high-strength double hooked-end steel fibers having a minimum tensile strength of 2800 MPa, a diameter of 0.4 mm and a length of 32 mm (fiber aspect ratio=80). The obtained SFRM had a density of about 2000 kg/m^3. The tests performed on 42 $40x40x160\ mm^3$ SFRM prisms according to EN 1015-11:2007 provided a mean flexural strength of 8.8 MPa (CV=19.2%) and a mean compressive strength of 35.1 MPa (CV=15.6%). Compressive tests performed on mortar cylinders according EN 12390-13:2013 provided a mean elastic modulus of 20430 MPa (CV=8.4%). The tensile fracture behavior of the SFRM was investigated by three-Point Bending Test (3PBT) on notched beams (40(width) x 150(height) x 500(length) mm^3) performed under CMOD (Crack Mouth Opening Displacement) control according to EN 14651-5:2005. The 3PBTs provided a limit of proportionality $f_{L,m}$ of 3.76 MPa (CV=18.9%) and residual tensile strengths $f_{R1,m}$=5.80 MPa (CV=26.6%), $f_{R2,m}$=6.87 MPa (CV=26.8%), $f_{R3,}$m=6.68 MPa (CV=27.6%) and $f_{R4,}$m=6.09 MPa (CV=28.2%).

Uniaxial compression tests on masonry prisms, with vertical and horizontal holes, strengthened with SFRM coating on both sides, provided a mean compressive strength of 4.8 MPa (CV=6.1%) and 1.7 MPa (CV=25.0%), respectively. The corresponding elastic moduli were 12400 MPa and 6110 MPa, respectively.

According to the results of the diagonal compression tests performed on 3 masonry panels strengthened on both sides, the mean shear strength and the corresponding strain were respectively equal to 1.0 MPa (CV=17.3%) and 0.41 mm/m. The latter are respectively 3.8 and 1.5 times higher than that detected from the unstrengthened panels.

2.3 Test set-up and instrumentation

A detailed description of the whole test rig was reported in Lucchini et al. (2018). For the sake of brevity, a summary of the main features of the test set-up is reported in the following.

A 1500 kN electromechanic jack, fixed to a reaction wall, was used to apply the lateral load to the structure. A vertical steel distributor beam allowed to subdivide the total lateral force into two concentrated loads acting in the middle of the North and South façades of the structure. Each concentrated load was applied to the floors by the RC chords placed along the short sides of the building. A double-hinged connection was installed between the jack and the distributor beam to enable relative rotation in the vertical as well as in the horizontal plane. Concrete blocks were spread on the first floor to get a uniformly distributed overload of 2.6 kN/m^2. In order to ensure a load distribution proportional to the building masses, the distributor beam was designed so that the 60% of the total lateral load was applied to the first floor. A couple of load cells were connected to the distributor beam to measure the actual lateral loads acting at the two floor levels. A total of 28 potentiometric transducers were installed on the East and West façades of the building to monitor crack formation. Horizontal displacements were detected by a total of 8 Linear Variable Differential Transformers (LVDTs) placed in correspondence of the floors and of the slab foundation. During the test on the retrofitted building, 4 potentiometric transducers were added to monitor crack formation on the inner surface of the central piers of East and West façades. The quasi-static tests were performed under displacement control, by progressively increasing the average top floor displacement.

2.4 Test results

The total lateral load (V) - displacement (δ) curves obtained from the tests on both the unstrengthened and the retrofitted building are plotted in Figure 2. Moreover, Table 1 reports the main test results related to both the positive (+) and the negative (-) loading direction: the initial secant stiffness (evaluated in the displacement range 0-0.6 mm (K^+_s, K^-_s)), the lateral load at first cracking (V^+_{crack}, V^-_{crack}) and at peak (V^+_{peak}, V^-_{peak}) and the lateral deflection at peak (δ^+_{peak}, δ^-_{peak}). Note that the specimen used in the first test (unstrengthened building) was re-used in the second tests (retrofitted building) after repairing the building.

The test performed on the unstrengthened building showed that the behavior of the structure was governed by the in-plane response of the longitudinal walls, particularly by the flexure-shear mechanisms of the piers at the ground floor. The latter were characterized by flexural cracks starting from the corners of the openings and diagonal shear cracks placed in the middle of the piers. The maximum lateral capacity was 180kN.

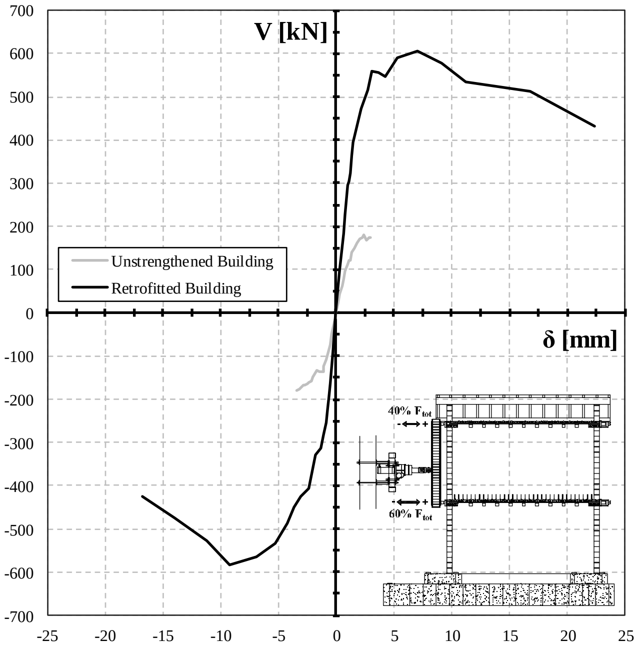

Figure 2: Lateral load-displacement envelops.

Compared to the unstrengthened building, the retrofitted one presented a more spread ultimate crack pattern (Figure 3). In fact, the high post-cracking toughness of the SFRM promoted the internal stress redistribution after cracking, leading to the formation of small cracks diffused on the coating surface. The overall behavior of the structure was still governed by the in-plane flexure-shear mechanisms of the piers at the ground floor. In addition to these mechanisms, at the end of the test, the inner side of the walls not provided with coating exhibited toe-crushing phenomena that involved the units placed close to the windows of the ground floor.

The SFRM coating led to an important improvement of the lateral displacement capacity of the building as compared to the reference one. Furthermore, the initial lateral stiffness and the maximum base shear provided by the coating was respectively 2.3 times and 3.3 times higher than that exhibited by the reference building.. The lateral drift corresponding to 20% loss of lateral resistance on the post-peak branch, was 0.4% and 0.3% respectively in the positive and negative loading direction. No detachment of coating from masonry surface was observed, proving the effectiveness of the steel connectors as well as the good bond properties that characterized the mortar-to-coating interface. Furthermore, when considering low loading levels significant for the Serviceability Limit State condition, the initial secant stiffness of the retrofitted building resulted more than double compared to that of the reference building and no crack was visible to the naked eye. This proved the effectiveness of fibers in controlling the cracking process and promoting a higher structure durability.

3 DESIGN APPROACH

3.1 Analytical model

The in-plane shear resistance of URM panels strengthened either on one or both sides with Fiber Reinforced Mortar (FRM) coating can be calculated by using the superposition principle. The total shear resistance of the retrofitted masonry element (V_R) can be calculated as the sum of the shear strengths respectively provided by URM ($V_{R,m}$) and FRM coating ($V_{R,r}$):

$$V_R = V_{R,m} + V_{R,r} \tag{1}$$

Table 1. Main results obtained from the cyclic tests on the full-scale masonry building.

Specimen name	K^+_S [kN/mm]	K^-_S [kN/mm]	V^+_{crack} [kN]	V^-_{crack} [kN]	V^+_{peak} [kN]	V^-_{peak} [kN]	δ^+_{peak} [kN]	δ^-_{peak} [kN]
Reference	125	132	120	128	180	179	2.4	3.4
Retrofitted	283	318	515	405	605	584	7.0	9.2

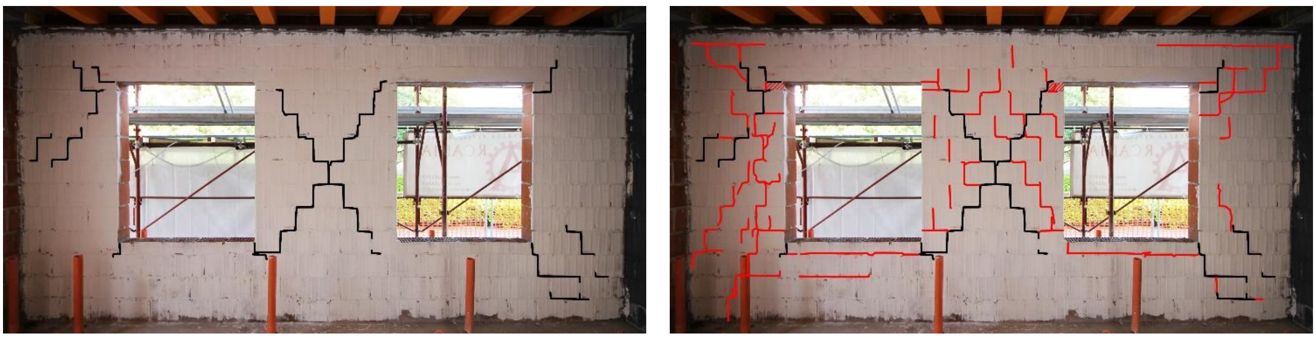
(a) (b)

Figure 3: Final crack pattern of East façade at ground floor (inside view); reference (a) and retrofitted (b) building.

The shear resistance of masonry was estimated by means of the model suggested by the commentary (Circ. 21 Gennaio 2019 n°7) to the Italian code NTC2018 for existing structures made of masonry with a regular pattern.

The shear resistance of the FRM overlay was evaluated by a strut and tie plane-stress model develop within the present research. The model assumes to concentrate the principal stresses into an idealized strut and a tie place along the panel diagonals. Both the strut and the tie have the same thickness of coating (t_r), a constant width named as b_r and same inclination (θ) to the horizontal axis. The angle θ depends on the panel aspect ratio (l/h), which is defined as the panel total length (l) to height (h) ratio:

$$\begin{cases} \theta = \tan^{-1}\frac{h}{l} & for\, \frac{l}{h} \le 1.5 \\ \theta = \tan^{-1}\frac{1}{1.5} \cong 34\, deg & for\, l/h > 1.5 \end{cases} \tag{2}$$

The shear strength ($V_{R,r}$) of the FRM coating depends on the resisting contribution provided by the strut ($V_{R,r\text{-}S}$) and the tie ($V_{R,r\text{-}T}$) according to the following relationships:

$$V_{R,r}=\min\left(V_{R,r\text{-}S};V_{R,r\text{-}T}\right) \tag{3}$$

$$V_{R,r\text{-}S}=f_c\cdot\alpha_S\cdot b_r\cdot t_r\cdot 2\cos\theta \tag{4}$$

$$V_{R,r\text{-}T}=f_{Ft\text{-}0.25}\cdot\alpha_T\cdot b_r\cdot t_r\cdot 2\cos\theta \tag{5}$$

where:

- f_c is the cylindrical compressive strength of FRM.
- α_S=0.4 is an efficiency factor that considers the cracking of the strut, the small thickness of the FRM coating layer and the uncertainty on the subdivision of the horizontal shear force between the strut and tie.
- $f_{Ft\text{-}0.25}$ is the residual tensile strength of the FRM coating corresponding to a crack width of 0.25 mm.
- α_T=0.85 is a correction factor related to the actual stress distribution within the FRM coating.

The width of the idealized band (b_r) depends only on the panel aspect ratio (l/h) and on the length of its diagonal (d) according to the following relation:

$$b_r = \left(0.25 + 0.2\cdot\frac{l}{h}\right)\cdot d \tag{6}$$

The experimental test on the retrofitted building and the numerical study performed to calibrate the analytical model herein proposed showed that the cracks on the coating surface were generally characterized by a width lower than 0.5 mm. In view of this, it was assumed to consider a tensile post-cracking strength $f_{Ft\text{-}0.25}$ of FRM corresponding to a crack width of 0.25 mm. The latter can be easily obtained from the uniaxial tensile stress-crack width law as the stress corresponding to a crack opening of 0.25 mm (see Figure 4). In more detail, the bilinear curve of Figure 4 was obtained by defining the tensile strength (f_t) as well as the post-cracking residual strengths f_{Fts} and f_{Ftu} reported by the fib Model Code 2010.

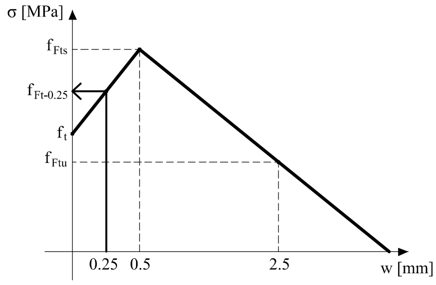

Figure 4: Bilinear uniaxial tensile stress-crack width law.

3.2 *Application to the full-scale building*

The analytical model herein presented was applied to the six longitudinal piers of the ground floor of the retrofitted building to determine their capacity. The piers were schematized by using the approach proposed by (Dolce 1989) to determine their own effective height. Furthermore, based on the experimental evidences, the global resistance of the building was estimated as the sum of the resistance of each pier. The analytical prediction was mainly governed by a shear failure, as observed during the experimental test.

The predicted overall resistance of the building was equal to 449 kN and was 25% lower than that exhibited by the test specimen. This means that the prediction of the analytical model was on the safe side.

4 CONCLUDING REMARKS

The results of two quasi-static cyclic tests on a full-scale hollow clay unit masonry building were presented in this paper. The two tests were performed before and after retrofitting the building with a 30 mm thick layer of SFRM applied only on the outer surfaces. Moreover, an analytical model to predict the in-plane shear resistance of masonry panels strengthened with FRM coating was presented. The experimental and the analytical results addressed the following main conclusions:

1. The proposed technique proved to be effective although the SFRM coting was applied only on the external surface. As compared to the unstrengthend building, the retrofitted one exhibited an average stiffness and capacity improvement of 134% and 231%, respectively.
2. The construction details of the retrofitting technique allowed to prevent local mechanisms, including coating delamination as well as failure at the corners of the openings.
3. The application of the analytical model herein presented to the retrofitted building led to an underestimation of the global capacity of about the 25%.

ACKNOWLEDGMENTS

The research is part of the project “Sismacomf” supported by the University of Brescia and by Tecnologia e Ricerca Italiana S.r.l. (TRI S.r.l), which is gratefully acknowledged. A special acknowledgment goes also to Eng.s Silvia Martini, Roberto Vecchi and Jessica Paterlini for their assistance in carrying out the cyclic tests on the full-scale building and to Eng. Alessandro Cavalleri for his contribution in developing the analytical model.

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