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NUMERICAL ASSESSMENT OF THE OUT-OF-PLANE RESPONSE OF MASONRY PANELS REINFORCED BY MEANS OF FRCM SYSTEMS

B. Pantò[1], M. Malena[1] and G. de Felice[1]

[1] Department of Civil Engineering of Roma Tre University
Via Vito Volterra 62, 00146 Roma
e-mail: {marialaura.malena, bartolomeo.panto, gianmarco.defelice}@uniroma3.it

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Abstract. *The use of Fibre Reinforced Cementitious Matrix (FRCM) materials, is becoming a very common technique of retrofitting for historical and monumental masonry buildings. This technique, if compared to the use of fiber polymeric materials (FRP), is more compatible with the mechanical property of the masonry and more appropriate with the preservation needs of cultural heritage, associated to the historical constructions. The effectiveness of such a type of reinforcement for improving the out-of-plane seismic capacity of masonry walls, in comparison with traditional reinforcements with steel tie-bars, was investigated in a previous experimental campaign. In the present work, a macro-modeling approach, already available in the literature for modeling masonry structures with plane and curved geometry, was used to predicting the out-of-plane response of masonry structures reinforced by means of FRCM. The model was validated by mean of the simulation of the previous cited experimental tests.*

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1 INTRODUCTION

The recent seismic events, such as the Emilia (2012) and the L'Aquila (2009) earthquake, have shown the great vulnerability of existing masonry buildings, not designed to withstand seismic loads. During the seismic event, the building, being characterized by weak connections between the walls at the corners and between walls and diaphragms, does not have a box behavior. Each wall shows an independent response, characterized by a complex non-linear interaction between its in-plane and out-of-plane behavior, that usually end with collapse by overturning. Aiming at reducing seismic vulnerability of existing masonry buildings, several strengthening techniques that make use of composite materials have been developed and investigated by means of experimental tests and numerical simulations. The use of Fabric Reinforced system, applied through Cementitious Matrix (FRCM), has become a common technique for retrofitting existing masonry buildings.

In this paper a macro-modeling approach, already developed for modeling masonry structures [1,2], is used to predict the out-of-plane response of a masonry prototype reinforced by means of FRCM, experimentally investigated through shaking table tests [3,4].

Masonry is considered as made of macro-elements interacting though non-linear interfaces. The reinforcement is modeled by a rigid plate, while the interaction between the reinforcement and the masonry substrate is governed by a discrete zero-thickness interface, able to take into account both the tensile (mode I) and shear (mode II) failure modes [5]. A refined bond-slip constitutive law, suitable for FRCM materials and experimentally validated by means of debonding tests [5], is employed. The capability of the model to predict the effective behavior of masonry repaired with FRCM is evaluated through comparison with experimental tests: the out-of-plane capacity curve of the specimen with and without FRCM reinforcement are obtained and compared with shaking table tests, in terms of ultimate strength and collapse mode.

2 THE DISCRETE MACRO MODELLING APPROACH

The discrete macro modelling approach represents an innovative tool for simulating the non-linear behaviour of masonry structures that requires a limited computational effort when compared to non-linear finite element method. Originally restricted to a 2D-kinematic for simulating the in-plane response of masonry [1,2], the model was extended to simulate the out-of-plane response of masonry panels, while also simulating the response of monumental structures [6,7]. In the present paper, a recent formulation that allows to model the interaction between masonry and the FRCM reinforcement [8] is applied.

2.1 The spatial macro-model for simulating the masonry

The kinematics of the macro-element is governed by 7 degrees of freedom: six DOF associated to the rigid body motions and one associated to the in-plane shear deformability of the element. This latter is described by a diagonal link, while the interactions between two contiguous panels are simulated by means of zero thickness 3D-interfaces. The flexural response is governed by a layer of transversal links, the shear in-plane sliding is controlled by a non linear spring while the out-of-plane sliding and the torsional behaviour of the panel are controlled by two further sliding links (Figure 1). The calibration procedure of the non linear links is based on energy equivalence, as are reported in [1,6].

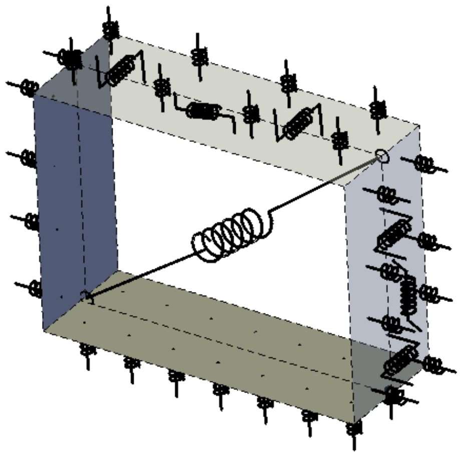


Figure 1. geometric representation of the spatial macro-model.

2.2 The macro-model for simulating the reinforcing

Following [8], the presence of composite reinforcing, applied by unidirectional strips, 2D webs or grids, is simulated by introducing plate elements bonded to masonry with non-linear transversal and longitudinal links. A further set of springs represents the membrane behavior of the reinforcement (Figure 2).

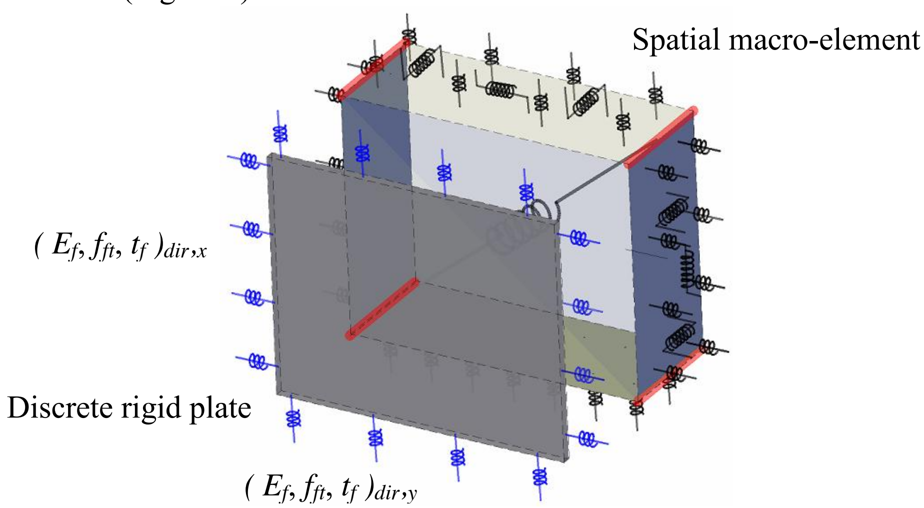


Figure 2. modeling of reinforcing by discrete rigid plate [8].

The macroscopic tensile behaviour of the reinforcement, comprehensive of the fabric and the matrix, is assumed as elastic-perfectly brittle, with equivalent elastic modulus E_f, and ultimate tensile strength f_{ft}. These properties are referred to the net area of the reinforcement that is defined by the equivalent thickness t_f. The model accounts for the anisotropic behaviour of the reinforcement when the reinforcement amount is different along the two directions. The non-linear links that connect two plates are calibrated through a simple elastic equivalence between the continuous elastic solid and the discrete equivalent model. The normal and tangential interaction between the reinforcement and the masonry substrate are modeled by means of a zero-thickness discrete interface, composed by transversal N-links for simulating the normal interaction and two N-links for simulating the bond-slip constitutive law. The transversal N-links are assumed as elastic, while the longitudinal links are modeled through a refined elasto-plastic bond-slip constitutive law with a post-peak softening branch as proposed in [5], to take into account the cracking process within the cementitious matrix.

3 CALIBRATION OF THE BOND-SLIP CONSTITUTIVE LAW

In the case of FRCM systems made of steel cords, failure usually occurs within the mortar matrix at the interface with the reinforcement [5,9]. Aiming at describing the debonding process, the key element is the adoption of a reliable bond shear stress-slip law [10,11,12].

In the present work, the bond law in the tangential direction, expressed in terms of slip displacement (d_t) and tangential stress (p_t), is described by an ascending elastic branch ending at the point (d_{tm}; p_{tm}), a linear softening branch ending at the tangential debonding displacement d_{td} and a plane frictional branch, with constant stress p_{t0}. This latter branch describes the interlocking phenomenon between the fibers and the surrounding matrix (see Fig. 3), typical of such a type of reinforcement.

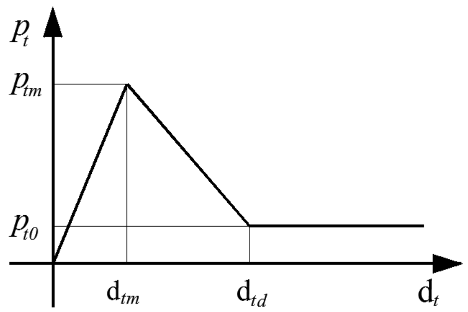


Figure 3. Trilinear shear stress-slip interface law.

4 SIMULATIONS

Aiming at showing the capability of the macro-modeling approach, to reproduce the out-of-plane response of masonry structures reinforced by means of FRCM, the results of the experimental campaign described in [3,4], were numerically simulated.

4.1 The prototype

The structural prototype modelled in the present work consists of a façade wall and two transverse walls, with an overall U-shaped configuration. The façade is 3.30m long, while the transverse walls are 2.3m long. All the walls are 3.44m high and 0.25m thick (Fig.4). The façade is connected to the transverse walls by a vertical mortar bed joint in order to simulate a weak connection in view of the out-of-plane overturning of the façade.

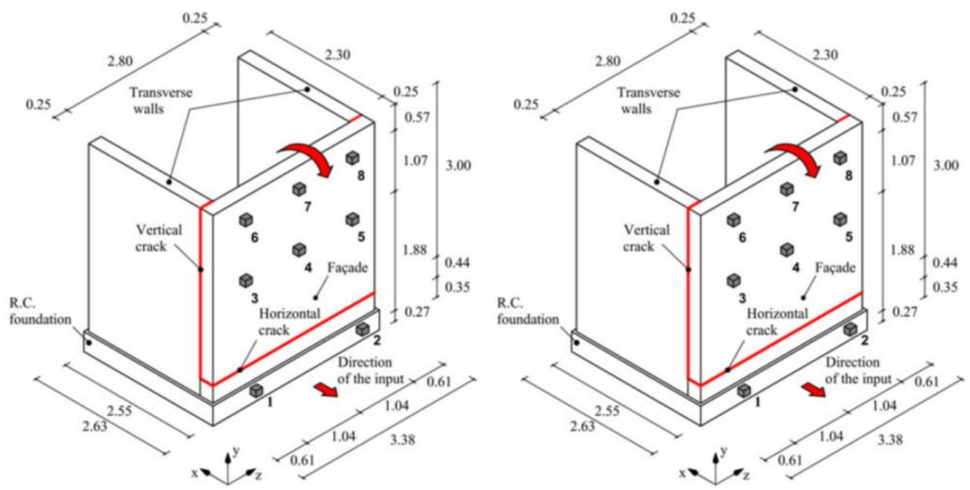


Figure 4. Geometry of the prototype [3].

The walls are made of tuff units (250x370x110mm) and hydraulic lime mortar. The mean mechanical properties of the masonry components are: unit weight 12.06 kN/m3, cubic compressive strength 5.98 MPa, Young Modulus 1575 MPa for the blocks; cubic compressive strength 4.08 MPa, Young Modulus 2038 MPa, tensile strength 0.84 MPa for the mortar.

The specimen was initially subjected to a shake table test series without reinforcement [3], which caused the detachment of the façade from the transverse walls and its out-of-plane overturning (Fig. 5(a)).

In order to investigate the effectiveness of traditional retrofitting devices, a further test series was carried out on the specimen retrofitted with two Ø20-mm steel tie-bars installed at a distance of 0.85m from the top of the wall and 0.30m from the corner. The height of the rods (about 75% of the whole height of the façade) maximized the stabilizing effect and, at the same time, prevented failure by overturning. A vertical crack developed in the middle section of the façade, induced by the out-of-plane horizontal bending (Fig. 5(b)) and some diagonal cracks formed at the top and mid-height of the front wall, probably related to the impact and the punching effect of the end-plates (Fig. 5(c)).

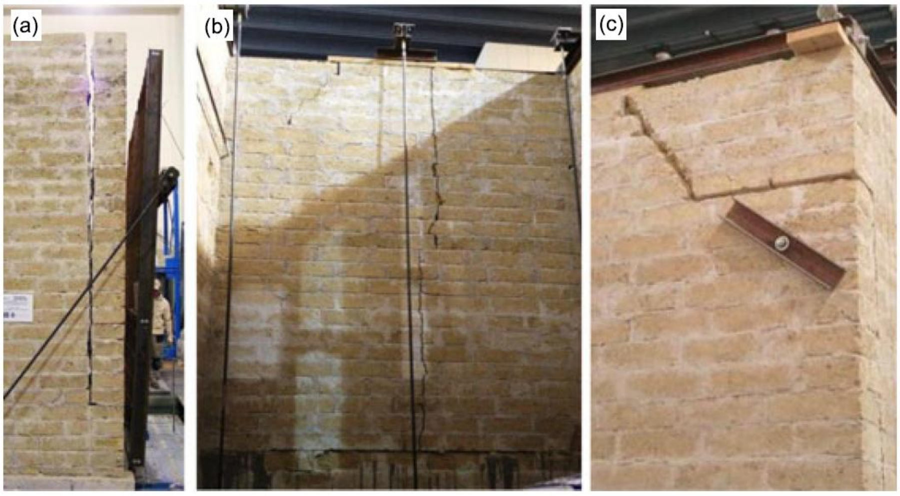


Figure 5. Damage pattern before the installation of SRG reinforcement [3]: lateral view (a), back view of the façade (b), detail of the anchorage of the reinforcing tie rod (c).

A last test series was carried out on the specimen retrofitted with FRCM made of unidirectional Ultra High Tensile Strength Steel (UHTSS) cords [3] that currently are referred as Steel Reinforced Grout (SRG). The strengthened system is made of 12 steel connectors retaining the out-of-plane overturning of the façade, four horizontal SRG strips applied to the side walls to transfer the load from the connectors to the masonry, two horizontal SRG strips applied to the front wall to transfer the retaining effect of the connectors to the façade. These strips are installed to provide also an increase of the seismic capacity of the wall with respect to horizontal bending. The textile has density of 4 cords/in, a design thickness equal to 0.084 mm and a mass density equal to 640 g/m^2. A mineral-NHL mortar was used as a matrix having compressive strength, Elastic Modulus, tensile strength and grain size range, equal to 20.6 N/mm^2, 11.42 KN/mm^2, 5.42 N/mm^2 and 0-1.4 mm, respectively.

4.2 Numerical modeling of the structural prototype

Three different models, representative of the unreinforced, tie rods reinforced and SRG reinforced prototypes, were developed and implemented in the code Histra [13]. The First step was the calibration of the model in term of mechanical characteristics of masonry.

The compression of masonry was fixed equal to the compressive strength of the tuff units, while the tensile strength and the tensile fracture energy, along the direction orthogonal to the bed mortar joints (f_{tv}, G_{tv}), were determined by fitting the experimental results relative to the unreinforced prototype. With this aim, Figure 6 shows the influence of the tensile strength on the prototype response, keeping constant the fractural tensile energy equal to 0,06 N/mm according to the mechanical properties of the mortar [2]. From such a parametric analysis, the value of the tensile strength that allows to the numerical model to best fit the experimental results was 0,015Mpa. The corresponding collapse mechanism was reported in Figure 7 in which a representation of the flexural damage was included by using a gray-scale proportional to the cumulate plastic energy [6]. The sliding is governed by a Mohr-Coulomb yielding surface with cohesion (c) equal to 0,15Mpa, friction factor μ=0,57 and fracture energy in sliding $G_s = 0.3N/mm$. The tensile strength along the direction parallel to the horizontal bed joints (f_{th}) was evaluated by the expression $f_{th} = f_{tv} + \frac{b}{2h}c$, where b and h are respectively the base and height of the brick units. Analogously, the fracture energy along that direction was determined by the expression $G_{th} = G_{tv} + \frac{b}{2h}G_s$. The masonry parameters of the masonry, adopted in the analyses, are reported in Table 1.

Direction	E [N/mm^2]	G [N/mm^2]	f_c [N/mm^2]	f_t [N/mm^2]	G_c [N/mm]	G_t [N/mm]	c [Mpa]	μ [-]	G_s [N/mm]
Vertical (v)	1614	645	5,98	0,40	0,50	0,50	0.15	0,57	0,30
Horizontal (h)	1606			0,15		0,06			

Table 1: Mechanical parameters of the masonry used in the simulations.

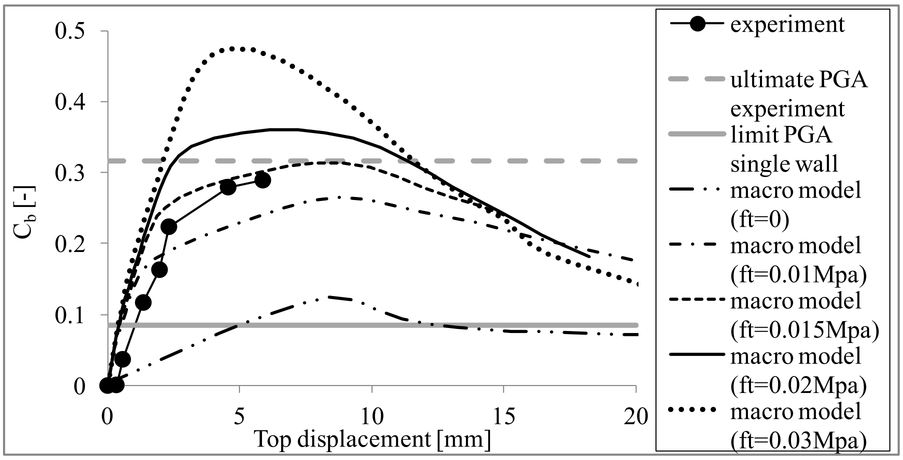


Figure 6. Unreinforced model – Load versus displacement curve.

In Fig. 6 the gray continuous line represents the theoretical limit strength corresponding to the out-of-plane overturning of the façade considered isolated, and considering a no-tension material, while the dashed gray line represents the ultimate lateral strength registered during the experiment. The push-over capacity curve is represented in terms of the maximum out-of-plane displacement of the façade as function of the base shear coefficient (C_b), corresponding to the base shear of the structure dimensionless by the total weight (W=108 kN). The results of the dynamic analyses, expressed in terms of maximum displacement versus PGA, are included in the graph and superposed to the capacity curve.

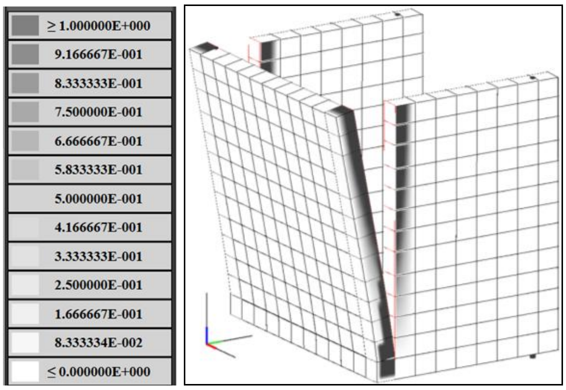


Figure 7. Numerical collapse mechanism obtained for the unreinforced prototype.

The results of the numerical analyses performed on the prototype reinforced by means of the two horizontal tie rods, are reported in Figure 8, both in terms of collapse mechanism and capacity curve. It can be observed a good agreement with the experimental analyses.

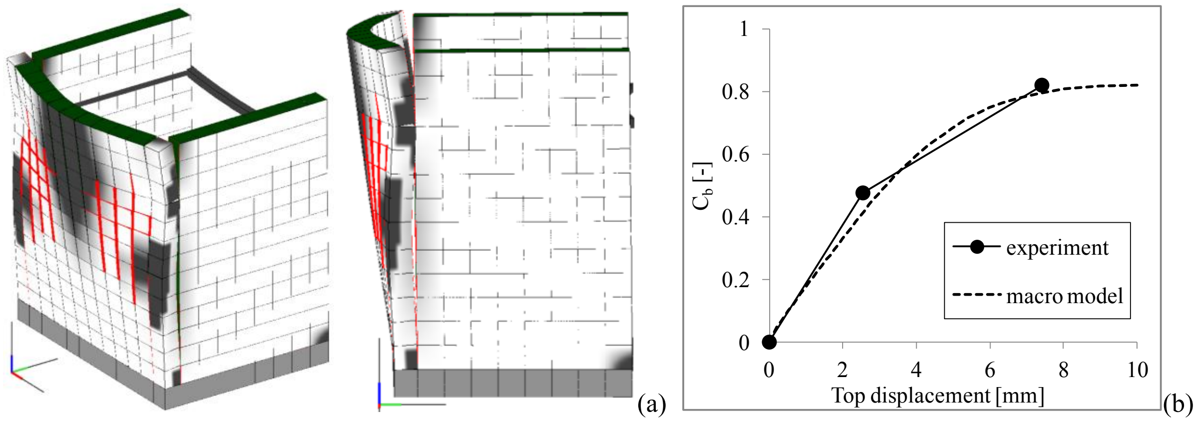
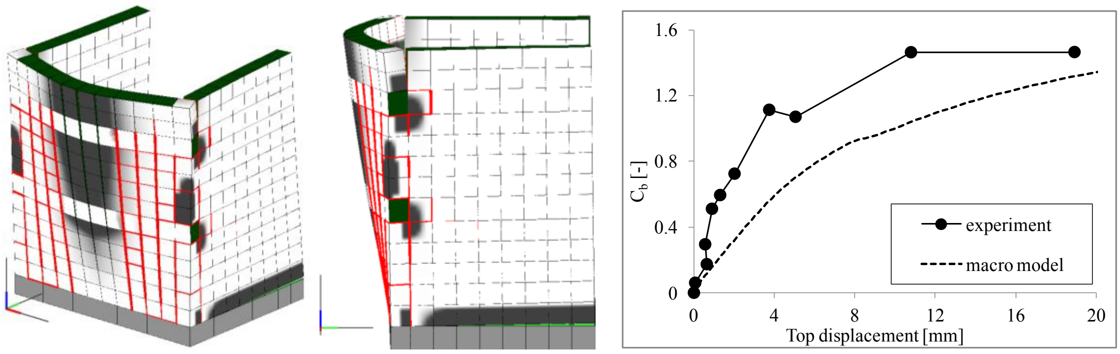


Figure 8. Prototype reinforced by tie rods: deformed mesh at the failure (a) and capacity curve (b).

It is worth noting that the model was able to represent the variation of the collapse mechanism from the unreinforced prototype to the reinforced one. In the latter case the numerical damage was concentrated mainly at the central part of the façade (Figure 8) according to the experimental observations [3,4].

Afterwards, aiming at reproducing the results of the last series of tests, the behaviour of the prototype reinforced by two horizontal SRG composite strips, was numerically analyzed. The trilinear bond slip law, defined in the previous section, was derived by mean of the procedure proposed in [11] (p_{tm}=1.267 N/mm, d_{tm}=0.005 mm, d_{td}=1.1 mm, p_{t0}=10%p_{tm}). In Figure 9, the numerical results related to such a simulation are represented in terms of plastic strain concentration and global load-displacement curve.

The ultimate experimental PGA of the reinforced system was about 1,4g against the ultimate value of 0,3g observed for the unreinforced prototype. The model is able to reproduce the increasing of the global strength, as shown in Figure 9c. The numerical failure mechanism is characterized by the damage of the façade, mainly concentrated in the central zone between the two reinforcing strips (Figure 9a). Also, the transversal walls were damaged at the corners and at the base section (Figure 9b).



(a) (b) (c)

Figure 9. Failure mechanism of the SRG reinforced prototype: plastic damage at the façade (a) and at the transversal wall (b); capacity curve compared to the experimental results (c).

5 CONCLUSIONS

In this paper the out-of-plane behaviour of masonry structures strengthened with FRCM is numerically investigated. A discrete macro-modeling approach, able to simulate the non-linear behaviour of the masonry and its bond-slip interaction with the composite reinforcing is employed. A refined bond-slip interaction between the composite reinforcement and the mortar matrix is considered. The model is used to simulate the response of a three-dimensional masonry prototype composed by a façade wall and two transversal walls experimentally investigated in the literature by means of dynamic tests. Three different configurations were numerically reproduced: the unreinforced prototype, the prototype reinforced by tie rods and the prototype reinforced by SRG composite strips. The experimental behaviour of the prototype was strongly characterized by the presence of the reinforcing system: the simple out-of-plane overturning of the façade, observed in the unreinforced configuration, moved to a global mechanism after the addition of the reinforcement. The numerical simulations presented in the paper are based on static push-over analysis, carried out on a three-dimensional model in the three tested configurations. The numerical results are in a good agreement with the experimental ones, both in terms of capacity curves and failure mechanisms, demonstrating the capability of the model in predicting the efficacy of SRG reinforcement against out-of-plain failure of masonry structures.

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