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SEISMIC ANALYSIS OF REINFORCED MASONRY BUILDINGS USING A NOVEL MODELLING STRATEGY

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Abstract: *Macroelement models have proved to be particularly useful to study the seismic behaviour of unreinforced masonry buildings, thanks to their low computational burden and simplicity of use. In high seismicity areas, reinforced masonry is often used and the seismic performance of buildings built with this technique needs to be adequately reproduced within the macroelement framework. This work proposes a new macroelement-based strategy to model the in-plane nonlinear behaviour of reinforced masonry piers, starting from the macroelement model implemented in the TREMURI software, based on an equivalent-frame discretization of the walls and widely adopted for unreinforced masonry. The strategy consists in coupling macroelements representative of masonry and horizontal (shear) reinforcement, to nonlinear elements representative of the vertical reinforcement. To test the efficiency of this strategy, in-plane cyclic experimental tests performed on reinforced masonry piers made of clay blocks were simulated. The model allowed to capture well both the flexural and shear failure modes and was then extended to tridimensional structures. A single reinforced masonry building, representative of new buildings located in a high seismicity region, was modelled and nonlinear analyses were carried out to study its seismic vulnerability. In particular, nonlinear static and dynamic analyses were performed to prove the efficiency of the strategy in estimating capacity and demand in the framework of a complete seismic assessment.*

Introduction

Masonry has always represented a sustainable construction technique; however, unreinforced masonry buildings can be particularly vulnerable to earthquakes, particularly if not designed to withstand seismic actions. A possibility for combining the advantage of sustainability with a lower seismic vulnerability, is the adoption, at least for newly designed buildings, of the reinforced masonry technique. Therefore, for the purpose of seismic design and assessment, reliable numerical models are needed.

Macroelement models, together with an equivalent-frame idealisation of the buildings' structure, are widely used to study the behaviour of unreinforced masonry buildings subjected to horizontal actions, due to their proven efficiency and simplicity of use (e.g. Penna et al. 2016). In the framework of extending the capabilities of this modelling strategy (Penna et al. 2022a), the application of macroelement models to the case of reinforced masonry structures surely represents a relevant issue.

Several attempts were proposed in the literature for modelling the behaviour of reinforced masonry elements (e.g. Magenes & Baietta 1998, Maleki et al. 2005, Peruch et al. 2019, Shakarami et al. 2019) or systems (e.g. Mojiri et al. 2015, Abdellatif et al. 2019), typically based on finite element approaches. These models are usually calibrated on experimental tests on masonry components (e.g. Shing et al. 1989, Voon & Ingham 2006, Mosele 2009, Penna et al. 2015) and can be used to define fragility curves of entire buildings (e.g. Lofty et al. 2019).

A complete exploration of the applicability of macroelement approaches to this problem is still missing. With respect to other strategies, this approach potentially has several advantages, since it requires a limited computational time, still assuring reliability and quality of the results.

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This work describes a new macroelement-based strategy to model the in-plane nonlinear behaviour of reinforced masonry piers, by considering a number of macroelements arranged in a series configuration. The model was validated through the comparison with the results of experimental tests on piers and then extended to perform nonlinear analyses of entire buildings.

Modelling strategy

The strategy proposed to model reinforced masonry piers is based on the equivalent-frame schematization of the building, where structural elements are modelled using macroelements. The macroelement model proposed by Penna et al. (2014), implemented in the TREMURI computer program (Lagomarsino et al. 2013) and recently improved by Bracchi et al. (2021) and Bracchi and Penna (2021), has been considered as starting point for this work.

The element is made of three parts (Figure 1): a central body where only shear deformation can occur and two interfaces, where the external degrees of freedom are placed, which can have relative axial displacements and rotations with respect to those of the extremities of the central body. The two interfaces have a negligible thickness and are characterized by infinite shear stiffness. Their axial displacements and rotations reproduce those of a distributed system of zero-length springs, featuring a no tension law with bilinear degrading behaviour in compression.

In this way, flexural and shear response of unreinforced masonry elements can be reproduced using a limited number of degrees of freedom. In particular, the macroelement kinematics can be described by means of eight degrees of freedom, six nodal generalized displacement components (u_i, w_i, φ_i, u_j, w_j, φ_j) and two internal components (w_e, φ_e). The shear response is modelled through a damage model characterized by a Coulomb strength criterion, depending on equivalent values of cohesion and friction coefficient.

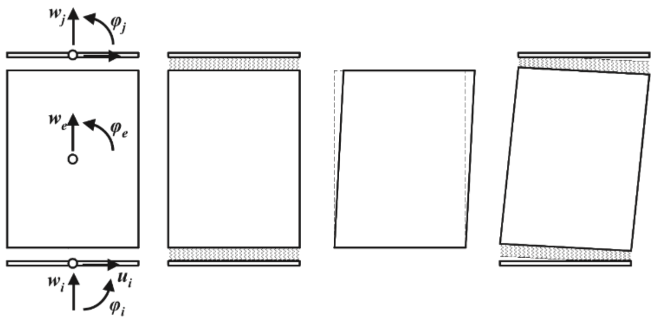

Figure 1. Macroelement proposed by Penna et al. (2014).

To model reinforced masonry piers, Bracchi et al. (2020) recently proposed a strategy consisting in assembling various sub-macroelements behaving in series, to create a larger macroelement representing a masonry member with shear reinforcement, coupled with nonlinear beam elements, modelling longitudinal (vertical) reinforcement. A proper choice of the equivalent shear strength parameters of the sub-macroelements allows to consider the contribution of transversal shear reinforcement: these parameters are calculated starting from the strength criterion prescribed by the Italian building code (NTC18 2018), according to the strategy proposed by Bracchi and Penna (2021).

The sub-macroelements modelling masonry features a no-tension behaviour, with possible cracking of the end sections, whereas the nonlinear beams modelling longitudinal reinforcement have a strength limited by the yield strength of the steel. The division into sub-macroelements allows to better model the flexural deformed shape and, therefore, the contribution of the longitudinal reinforcement. Figure 2 shows the scheme of a single pier modelled with the developed strategy. The model is also suitable to model uplift phenomena occurring in case of rocking behaviour, since in cracked section conditions, axial displacements and rotations are coupled.

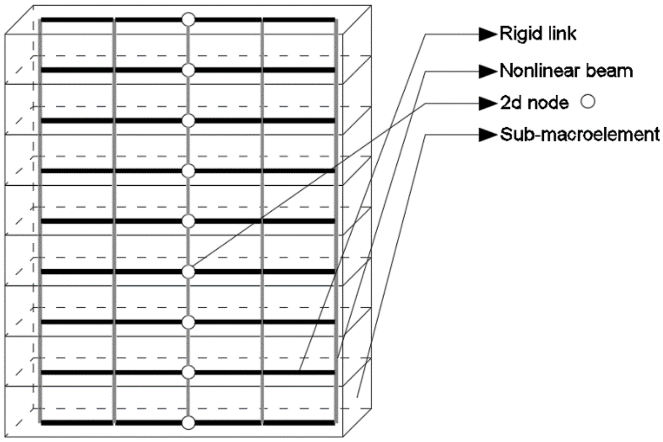

Figure 2. Scheme of the adopted modelling strategy for piers, with indication of the sub-macroelements in which the pier is subdivided and the nonlinear beams representing the longitudinal reinforcement (Bracchi et al. 2020).

The Italian building code (NTC18) prescribes a shear strength of a reinforced masonry pier equal to:

$$V_s = dtf_v + 0.6d \frac{A_{sw}}{s} f_y \leq 0.3 f_m td \tag{1}$$

where f_v is the contribution due to masonry, i.e.:

$$f_v = f_{v0} + 0.4\sigma_n \leq 0.065 f_b \tag{2}$$

t is the thickness of the element, d is the distance between the compressed side of the section and the center of mass of the bars in tension, A_{sw} and s are the area and step of the transversal reinforcement, respectively, f_y is the yield strength of the steel of the transversal reinforcement, f_m is the masonry compressive strength, f_{v0} is the initial shear strength (cohesion) of masonry, f_b is the compressive strength of the unit and σ_n is the axial stress acting on the considered section over the area dt.

The equivalent cohesion and friction coefficient to be used in the strength criterion implemented in the macroelement of Penna et al. (2014) are obtained linearizing the strength criterion of Equation 1, as discussed by Bracchi et al. (2020), as a function of the acting axial load. In addition, flexural and shear drift capacities of the assemblage of sub-macroelements had to be properly defined.

Simulation of experimental tests

With the aim of testing the efficiency of the developed modelling strategy, the simulation of experimental tests performed at the University of Pavia (Italy) within the DREMAB project (Magenes 1998) on single reinforced masonry piers, was carried out.

Experimental campaign

In the considered experimental campaign, seven walls made of perforated clay bricks (void ratio of 45%) were tested in cantilever boundary conditions, applying a cyclic horizontal shear force with constant axial load. Bricks of 300 mm width, 230 mm length, 185 mm height were adopted, together with a cement/lime/sand mix mortar with a 1.5/1/4 proportion (in volume). Walls' thickness was equal to 300 mm. Vertical reinforcement was inserted in grip holes, whereas horizontal reinforcement was laid in bed-joints; in both the cases, high bond bars of steel with characteristic yield strength f_{yk} equal to 430 N/mm^2 were used.

By varying the aspect ratio (and therefore the shear ratio) and the amount and distribution of vertical and horizontal reinforcement, it was possible to design specimens characterized by a specific failure mode. Table 1 reports the geometry and the applied axial stress at the top of each wall.

Walls 4c and 4s were designed to exhibit pure flexural failure. They had the same amount of total vertical reinforcement, but different distribution along the length of the section. Walls 9, 10 and 11 were designed to exhibit shear failure and they differ only for the spread reinforcement ratios.

Walls 2 and 3 were designed to have similar flexural and shear capacities and hence a mixed flexure-shear failure mode.

Wall	l [m]	h [m]	t [m]	$\sigma_{0,top}$ [MPa]
2	2.23	3.015	0.3	0.250
3	2.23	3.015	0.3	0.278
4c	2.23	3.015	0.3	0.285
4s	2.23	3.015	0.3	0.472
9	2.71	1.615	0.3	0.238
10	2.71	1.615	0.3	0.244
11	2.71	1.615	0.3	0.235

Table 1. Geometry and applied axial load, for each wall.

Closed hoops were adopted as horizontal reinforcement, with the exception of wall 11, which had a single 6 mm diameter bar every course. The vertical bars were continuous along the wall height and were anchored into the reinforced concrete foundation at the base and into the r.c. spread beam at the top. Table 2 reports the reinforcement configurations of the walls.

Wall	Longitudinal reinf.		Transversal reinf.
	Spread	Concentrated	Spread
2	1φ10/48 cm	2+2φ18	2φ6/40 cm
3	1φ14/48 cm	2+2φ18	2φ6/20 cm
4c	1φ6/48 cm	1+1φ18	2φ6/40 cm
4s	1φ12/48 cm	-	2φ6/40 cm
9	1φ10/48 cm	2+2φ20	2φ6/40 cm
10	1φ14/48 cm	2+2φ20	2φ6/20 cm
11	1φ10/48 cm	2+2φ20	1φ6/40 cm

Table 2. Reinforcement of each wall.

As expected, flexural and shear cracks appeared during tests on Wall 2 and Wall 3: in particular, corner-to-corner diagonal shear cracks formed during the test, followed by toe crushing. Wall 4c was characterized by flexural cracks with very moderate shear cracking. A flexural response was also present in Wall 4s, where failure was attained with crushing of the compressed masonry corners.

In Walls 9 and 11, characterized by lower spread reinforcement ratios, a major diagonal crack developed from the upper corners down to the base in each direction of loading. On the contrary, Wall 10, having the highest spread reinforcement ratio, featured a higher number of inclined shear cracks of lower width.

Numerical simulation

The proposed modelling strategy was used to simulate all the tests carried out by Magenes (1998). Each wall was discretized into various sub-elements of equal height with mechanical properties of masonry assumed equal to the values derived from the characterization tests performed by Magenes et al. (1996). In particular, for all the sub-elements of all the walls, the following values were adopted: elastic modulus E 11218 MPa, shear modulus G 240 MPa, compressive strength f_m 10.76 MPa, density ρ 900 kg/m^3, initial shear strength f_{v0} 0.586 MPa, compressive strength of bricks f_b 17.8 MPa. The shear deformability parameters χGc_t and β were assumed equal to 8 and 0.4, respectively. Steel strength was differentiated among the different bar diameters (Table 3), according to the values obtained from the characterization tests.

φ [mm]	f_y [MPa]
6	592.9
10	589.7
12	597.3
14	487.0
18	608.3
20	510.6

Table 3. Strength of reinforcement, as a function of the bar diameter φ.

Only the results of the numerical simulation of two walls are reported in this work, one characterized by flexural behaviour (Wall 4s) and the other failing in shear (Wall 9). The comparison between the hysteresis curves, i.e. base shear versus displacement curves, obtained from the experimental tests and the numerical simulations, is reported in Figure 3. It can be observed that the numerical model is able of reproducing the experimental response in terms of strength, displacement capacity and hysteretic behaviour. The dissipated energy is slightly underestimated, particularly in the initial phases for the Wall 4s, i.e. the one with a flexural failure mechanism.

The damage mechanisms obtained from the simulations of the two tests (Figure 4) prove the ability of the numerical model of reproducing a flexural mechanism in case of Wall 4s, consistent with the actual experimental behaviour. No shear cracks are hence visible in the mechanism; moreover, the adopted discretization into sub-elements allows to reproduce the variation of cracked length of the section along the height, as indicated by the nearly horizontal lines. For the case of Wall 9, experimentally failing in shear, the damage mechanism obtained from the simulations actually shows a shear failure, as indicated by the “X” marker in the top element. An increment of damage moving from bottom to top is evident, as graphically indicated by the darker colour of the damage mechanism of the sub-elements. The legend indeed reports the colour scale with reference to a parameter of the macroelement (α), which represents the level of shear damage within the element (Penna et al. 2014). When this parameter reaches the unit value, the element fails in shear and a “X” marker appears. In the numerical results, it is evident that the failure is concentrated in a single sub-element (the top one in this case), due to the discretization of the pier in different sub-macroelements.

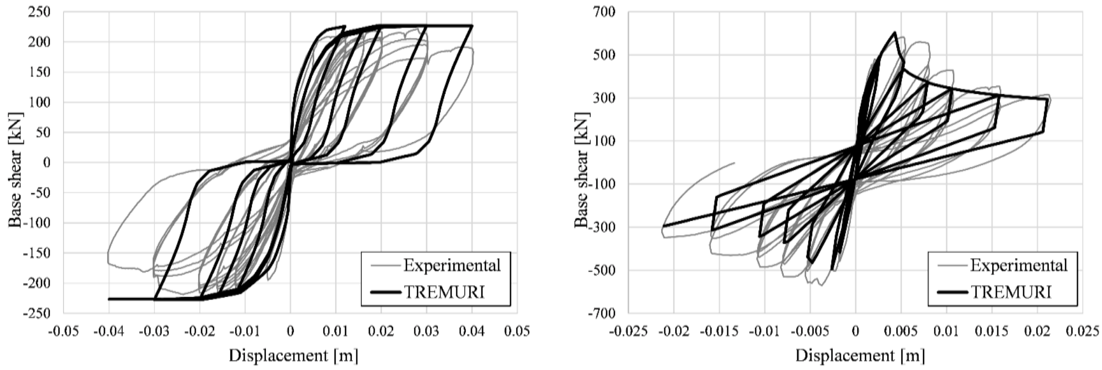

Figure 3. Comparison between the force vs. displacement curves of experimental tests and numerical simulations, for Wall 4s (left) exhibiting a flexural failure mode and Wall 9 (right), failing in shear.

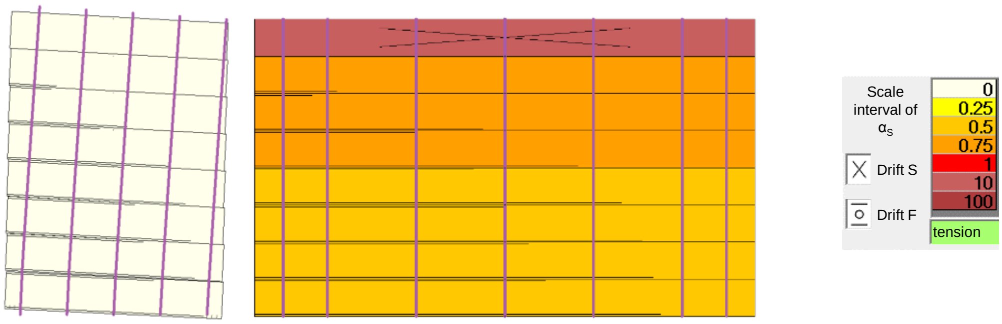

Figure 4. Damage mechanism obtained from the numerical simulation of the tests on Wall 4s (left) and Wall 9 (right); the colour legend indicates the level of shear damage in terms of the internal damage parameter α.

Modelling of buildings

The developed strategy was then extended to model entire buildings. To this purpose, a case-study building, previously designed according to the Italian building code NTC08 (2008) in a high seismicity site (RINTC Workgroup 2018) was selected.

Masonry is made by H-shaped clay units, with a characteristic compressive strength f_{bk} equal to 10 MPa and M10 mortar; C25/30 concrete tie beams are also present at each story level. Design of the reinforced masonry building resulted in the adoption of a longitudinal reinforcement made by 1φ16 at each end or wall intersection, with a distance not larger than 4 m and transversal stirrups of φ6/40 cm, i.e. every two courses; furthermore, B450C steel was adopted.

In addition to f_{bk}, the following values of mechanical parameters were adopted in the numerical analyses: elastic modulus E 5300 MPa, shear modulus G 2120 MPa, compressive strength f_k 5.3 MPa, density ρ 900 kg/m^3, initial shear strength f_{vk0} 0.2 MPa. With the aim of performing nonlinear analyses, mean values corresponding to the aforementioned characteristic values were adopted. Drift limits corresponding to shear failure of the entire pier were defined starting from the average drift leading to shear collapse of the piers tested by Magenes (1998): in particular, a value equal to 1.36% was adopted. Drift limits associated with flexural failure of the entire pier were assumed equal to 1.6% (i.e. value suggested by NTC18). The TREMURI model of the building was developed according to the proposed strategy (Figure 5); however, spandrels were not modelled, since in newly designed buildings, these elements are usually conceived as not contributing to the seismic response.

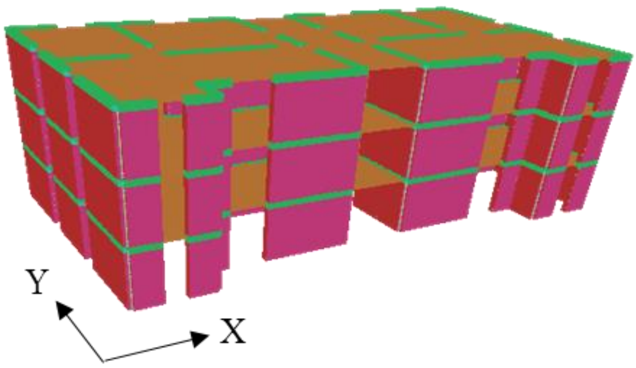

Figure 5. TREMURI model of the case-study building.

For the purposes of quantifying the improvement of performance due to the presence of reinforcement, the same structure was modelled also in the unreinforced configuration. Nonlinear static analyses were then performed. In case of the unreinforced masonry configuration, piers were modelled using the macroelement recently proposed by Bracchi et al. (2021) and Bracchi and Penna (2021), adopting as shear strength criterion the formulation corresponding to sliding along the cracked section, together with its upper limit depending on the compressive strength of bricks (Bracchi and Penna 2021). The same values of E, G, f_m, f_{vm0} used for the reinforced configuration were adopted, whereas μ and f_b were assumed equal to 0.4 and 14.3 MPa, respectively. Macroelement drift limits were set equal to 0.54% and 1.22% for shear and flexural mechanisms, respectively, as done in other works dealing with newly designed clay bricks masonry buildings (Cattari et al. 2018).

Nonlinear static analysis with an inverse triangular force distribution was performed in the positive X direction. Pushover curves obtained (Figure 6) showed a significant increase of base shear when adding reinforcement, since maximum strength of the reinforced masonry configuration is about three times the one of the unreinforced structure. An improvement of displacement capacity is also evident, especially with reference to the attainment of a significant decay of base shear (e.g. 50%). In both the configurations, as shown in Figure 6, the failure mechanism is characterized by shear failure located at the ground level. However, the unreinforced configuration is characterized by a significant shear damage also at the higher levels, whereas in the unreinforced one only a slight damage is present at the first and second level.

To prove the ability of the strategy to perform nonlinear dynamic analyses, the case study building was subjected to two ground motions in the X direction, corresponding to different seismic intensities, exciting the building both in the linear elastic and nonlinear regime. The hysteresis curves obtained (Figure 7) allow to conclude that the strategy is suitable to model both the linear elastic and nonlinear dynamic response of the building.

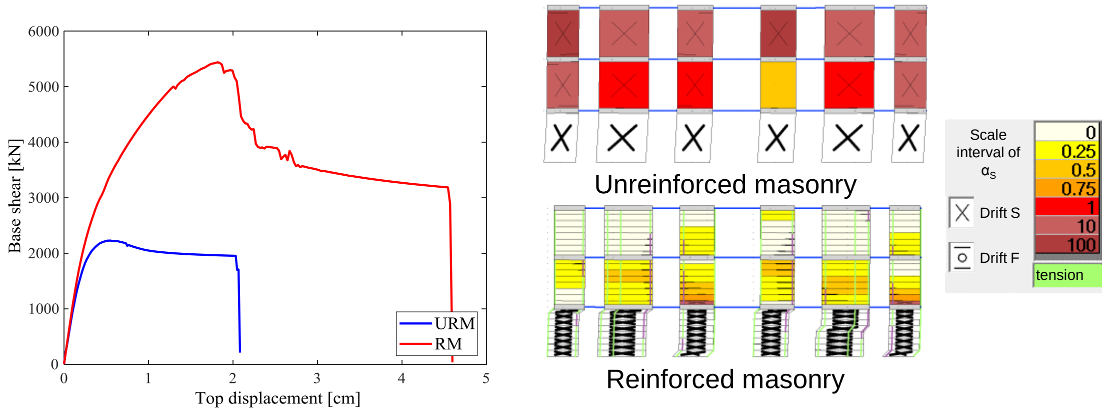

Figure 6. Comparison among pushover curves (left) and failure mechanisms (right) obtained for the unreinforced and reinforced configuration of the case-study building.

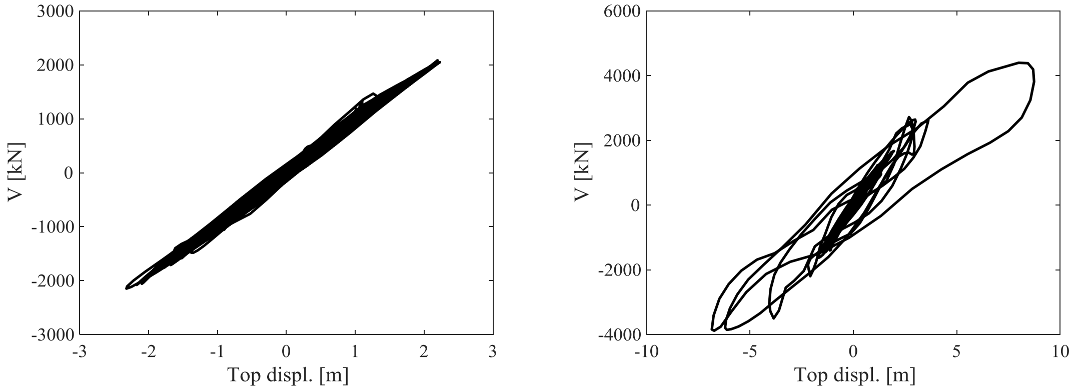

Figure 7. Hysteresis curves obtained from nonlinear dynamic analysis of the case-study building with ground motions exciting the structure in the linear elastic (left) and nonlinear (right) regime.

Conclusions

This paper proposed a novel strategy to model reinforced masonry buildings, developed starting from the macroelement proposed by Penna et al. (2014) and implemented in the software TREMURI, in the framework of an equivalent-frame modelling approach. A new element was developed, assembling a certain number of sub-elements representing masonry, with nonlinear beams representing the longitudinal reinforcement; transversal reinforcement was indirectly accounted for by means of properly selected values of the equivalent shear strength parameters of the sub-elements.

The response of a series of experimental tests on reinforced masonry piers, with different geometries and reinforcement configurations, was then simulated. The model was able to satisfactorily reproduce the experimental response, in terms of hysteresis curves and damage patterns, both for walls dominated by a flexural failure mechanism and walls exhibiting a shear failure mechanism.

The proposed strategy was then applied to model entire buildings. In particular, a case-study structure was modelled and nonlinear static analyses were performed in both the unreinforced and reinforced configuration, followed by nonlinear dynamic analyses. Results obtained allowed to prove the efficiency of the developed strategy in modelling the behaviour of entire buildings and to quantify the improvement of seismic performance of a building when reinforcement is added. Since the computational burden proved to be still low, this strategy appears to be suitable for the performance of nonlinear analyses in the framework of vulnerability and risk studies, similarly to what done for unreinforced masonry buildings (e.g. Bracchi et al. 2015, Bracchi et al. 2016, Cattari et al. 2018, Penna et al. 2022b, Lagomarsino et al. 2022).

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