Numerical Modelling for Assessing the Structural Efficiency of CAM® Reinforcement System for Masonry Walls

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Abstract A large portion of the Italian building heritage is made of masonry constructions, which were erected in the first decades of the last century and were conceived to support gravitational loads only. Many research activities have been carried out in order to propose retrofitting techniques aimed at improving seismic behaviour of existing masonry buildings. The present paper focuses on the CAM® reinforcing system. Such a retrofitting technique, which is widely used in Italy, consists in the application of pre-tensioned stainless steel ribbons on existing masonry walls, conferring to them additional strength, ductility and beneficial confinement effects. The preliminary outcomes of a common research activity developed in cooperation between the University of Campania "Luigi Vanvitelli", the University of Palermo, the University of Messina and the EDILCAM® Sistemi Srl, aimed at setting up appropriate design criteria, are provided in this paper. In particular, the calibration procedure of a specific FEM numerical model developed by Abaqus is presented, which then has been applied to interpret the cyclic response of unreinforced and reinforced masonry walls previously tested in within the research project In.CAM.M.I.N.O..

Keywords: Masonry walls, Stainless steel, Ribbons, Shear, Numerical analysis

1 Introduction

The protection of historical masonry buildings is a complex challenge which is promoting the development of many research activities. In fact, the need to preserve such valuable constructions retrieves many difficulties due to their inefficiency to face earthquake loads. In fact, many masonry buildings, due to their structural complexity and the poor mechanical features of the base material (i.e. lack of tensile strength, non-linear behaviour and reduced ductility), are particularly vulnerable against seismic actions.

To define the extent of the problem, it would be enough to mention the notable damage occurred to masonry buildings (both monumental and ordinary ones) after the 2016 Central-Italy earthquake, despite it was not particularly intense in terms of magnitude.

Many research activities have been carried out by various authors, by means of experimental tests and numerical models, with the aim to propose innovative reinforcement techniques suitable to improve the seismic behaviour of masonry structures (e.g. El-Gawady et al., 2006, Campitiello et al., 2007, Bischof et al., 2014, Gattesco et al., 2015, Zizi et al., 2017a). In this context, steel-based retrofitting techniques may result very effective. In fact, the introduction of metallic materials, which are characterized by high mechanical characteristics in terms of ductility, strength and stiffness, could strongly enhance the seismic behavior of masonry structures.

In particular, the CAM® system, which consists in the application of stainless steel ribbons on existing structural elements, is analysed in this paper. Such a reinforcement system is easy to be applied and since it could be embedded in a small layer of external plaster, its application may be suitable for a wide range of masonry buildings, including monumental ones. Although such a system has been already studied in previous works (e.g. Dolce et al., 2002, Spinella et al., 2014), nowadays specific design criteria are not still available.

Based on these premises, in the present paper a specific numerical model, which has been calibrated on available results obtained from an experimental campaign carried out on the CAM® system, is presented. The aim is to lay the groundwork for the definition of more appropriate design criteria.

2 The CAM® System: Description of the System and Reference Experimental Tests

2.1 Description of the CAM® System

The CAM® system is recognized as an efficient retrofitting technique for existing masonry and reinforced concrete structural elements. With regard to masonry buildings, the principles at the base of the CAM® system are essentially: i. to provide additional tensile strength, ductility and stiffness by introducing steel ribbons; ii. to offer a beneficial confinement effects on the existing material due to pre-tensioning of the metallic elements. In practice, it consists of a three-dimensional mesh of pre-tensioned steel ribbons that pierce the masonry throughout transversal holes, which are realized before the installation of the system (Fig. 1).

The pre-tension to the steel elements is conferred by connecting their extremities with specific seals during the closing operations, which are carried out by using a special device. At transversal holes, on the masonry surface, small metallic plates (125x125x4 mm) are applied, in order to distribute and diffuse the stresses transmitted by the steel strips, avoiding local stress peaks in the base material. In addition, angular steel plates are used in the corners where the steel elements and the masonry wall are overlapped. By applying this system, the seismic behaviour of masonry structural elements (i.e. walls or columns) may be strongly improved, as additional stiffness, strength and ductility are provided.

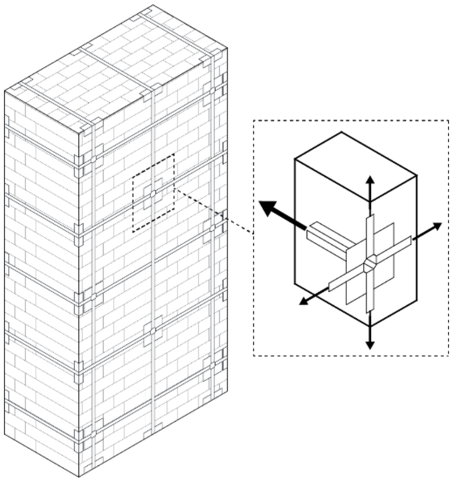

Fig. 1 Example of application of the CAM® system on an existing masonry wall

2.2 The Reference Experimental Test

The experimental campaign adopted as reference test for this study has been carried out in Messina and was part of the framework of the In.CAM.M.I.N.O. research project, which involved various institutes and private companies (namely CHIMETEC s.a.s., ABI s.r.l. of Ragusa and University of Messina). The activities were aimed at investigating the efficiency of the CAM® system on existing masonry panels. The tested masonry specimens had a thickness of 0.55 m and were obtained from a retaining wall, which was shaped in order to define seven different panels with a height of 2.15 m and a width of 1 m for the first panel and 1.1 m for the others. The quality of the masonry was very poor and its arrangement was characterized by stones of different size and crumbly lime mortar.

All tested masonry panels were divided in two parts by inserting two steel beams in the middle height (i.e. UPN 200 with two transversal bolts ϕ24). In particular, on the first panel, double flat-jacks (downer half-panel) and a compression test (upper half-panel) were carried out. The remaining six panels were tested in unreinforced (one) and reinforced (five) configurations by means of Sheppard tests, i.e. in condition of constant vertical compression and increasing horizontal force applied in the middle height of the specimen until the collapse (Turnsek and Sheppard, 1980). The experimental campaign included also a preliminary verification of the mechanical characteristics of the masonry material.

With reference to the reinforced configurations, the CAM® system has been applied in combination with simple external plaster or with polymeric net embedded in it. For the aims of this study, the masonry panel with the reinforcement and simple external plaster are considered. More details on the experimental campaign may be found in Spinella et al. (2014).

2.3 Experimental Results

The preliminary mechanical characterization of the masonry revealed the poor quality of the base material, as it can be also deduced by observing the irregular texture of the wall (Fig. 2a). In particular, by means of a double flat-jack test, an initial Elastic Modulus of E=633 MPa and a compressive strength of f_k=1.07MPa were obtained, while the compressive test, whose stress-strain results are showed in Fig. 2b, provided a compressive strength f_k=0.89 MPa with an ultimate strain of ε_u=0.37 %.

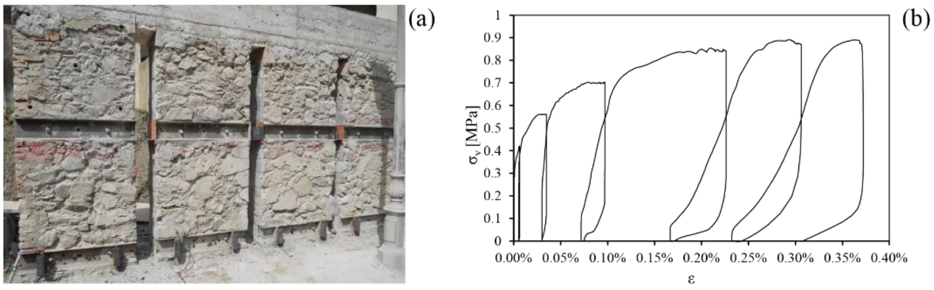

Fig. 2 The tested panel (a) and result of preliminary compressive test (b).

For the Sheppard test set-up of the unreinforced panel (Fig. 3a), the masonry panel was connected to a rigid reinforced concrete beam (50x55 cm) at the top, while two UPN 200 steel beams with bolted connections were applied at the basement, similarly to the middle height section. In such a way, perfect restraining condition could be supposed, without allowing rotational and translational movements. A compressive force of 200 kN has been firstly applied on the masonry panel, by imposing tensile stress to dywidag bars (ϕ32, two for each side of the panel), which were connected to the upper and downer rigid beams.

The unreinforced reference panel (URM), whose surface was preliminary treated with a simple external plaster, attained the collapse condition due to a shear failure (diagonal cracking). At the end of the test a more serious damage on the upper half-panel respect to the downer ones was observed, evidently due to the different boundary conditions (Fig. 3c). In fact, since the translational stiffness of the upper beams was higher than the downer system, small displacements in the bolt-masonry interface, as well as rotations of the base section, probably occurred. The panel reached the collapse for maximum horizontal load of 180 kN and ultimate lateral displacement of 18 mm (Fig. 3b).

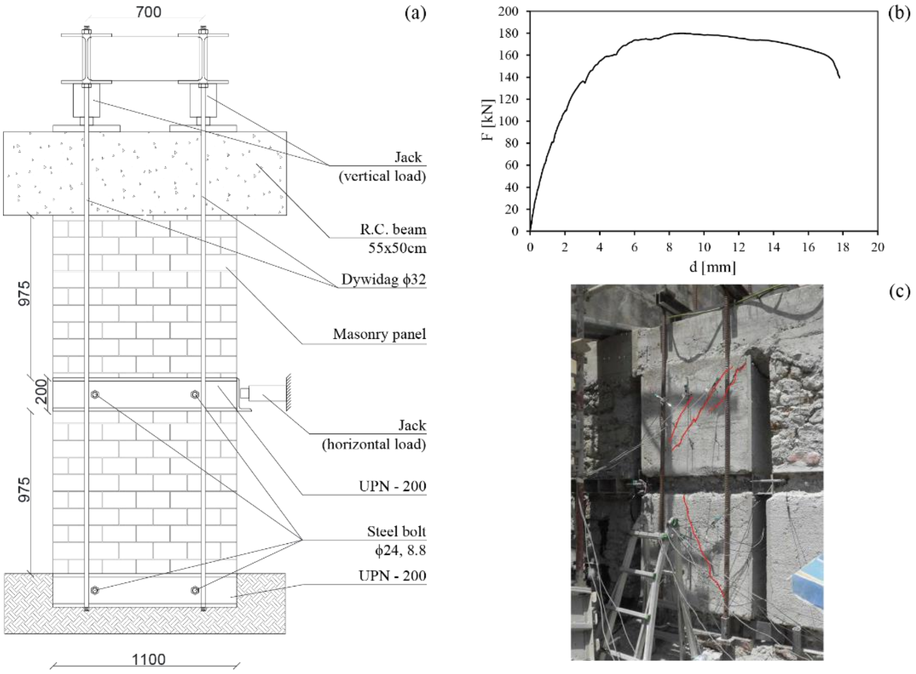

Fig. 3 Test set-up (a), force-displacement curve (b) and cracking pattern (c) of the unreinforced reference panel.

Among the reinforced configurations, the panel reinforced with the CAM® system and with simple external plaster has been selected by the authors for the aims of this study. The test set-up (Fig. 4a) was similar to the one adopted for the unreinforced panel, with rigid elements placed at the top, at the base and in the middle height sections. Nevertheless, in order to avoid displacements and rotations of the downer UPN-masonry system, steel plates welded to the webs of the base beams were introduced during the experimental campaign on the reinforced panels. The panel under consideration was assembled with two overlapped stainless steel ribbons AISI 304 (19x1 mm^2) disposed with a vertical and horizontal distance of 40 cm. At the end of the test a significant increase of the maximum lateral load respect to the unreinforced configuration (about 45%), as well as a lateral stiffness increment were noticed (Fig. 4b). Nevertheless, due to some problems, the test was stopped before reaching the global panel collapse. In particular, the test provided that the horizontal jack was directly supported by the adjacent panel, at whose interface a steel element as contrast was interposed. Therefore, since during the test the adjacent panel showed small cracks and movements of the contrast steel element occurred, the test was interrupted at a lateral displacement of about 17 mm. According to the crack pattern (Fig. 4c), it could be asserted that a maximum horizontal load of 262 kN was reached. Also in this case, the upper part of the panel absorbed a larger force, due to the free rotation that probably occurred at the base.

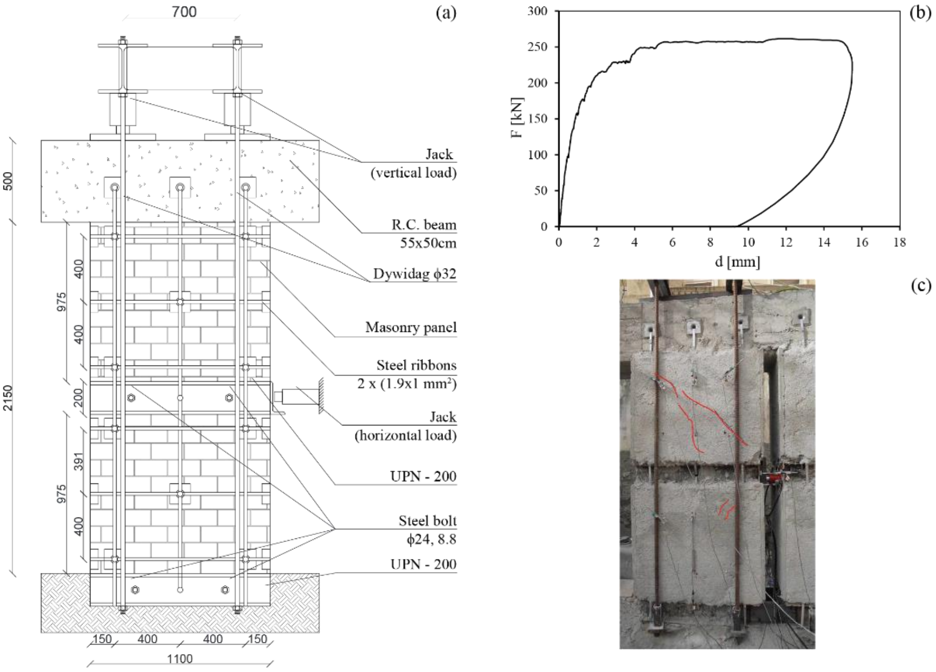

Fig. 4 Test set-up (a), force-displacement curve (b) and cracking pattern (c) of the reinforced reference panel.

3 Numerical Modelling

3.1 Introduction

A numerical model have been developed in order to define a tool suitable to interpret and predict the effect of the CAM® system when applied on existing masonry walls. The Abaqus software with standard solution has been used for the FE Models, which have been calibrated on the basis of the aforementioned experimental tests. The adopted modelling technique and the results related to the calibration procedure with a real test referred to both unreinforced and reinforced configurations are shown in the following.

3.2 The FE Model for the Unreinforced Panel URM

The unreinforced panel has been modelled by using 8-node linear brick with reduced integration (C3D8R). Since the contribution of the external plaster has been neglected in terms of both strength and stiffness, it has not been included in the numerical model. The masonry in the unreinforced FE Model (URM) has been modelled by adopting a macro-modelling approach, which is suitable for the description of the global response of masonry walls (Lourenço, 1996). In order to faithfully reproduce the set-up test and simulate the different boundary conditions, the upper and the downer nodes of the panel have been rigidly linked to control nodes (reference points RPs) by means of internal constraints (i.e. rigid body). In such a way external constraints and loading conditions applied to the RPs are directly transmitted to the nodes of the panel. Moreover, the same technique has been adopted for taking into account the presence of the intermediate steel beams. The vertical dywidag bars have been also considered in the numerical model by introducing between the upper and downer nodes four axial springs, whose stiffness K=78.5 kN/mm has been evaluated according to real axial stiffness of the bars.

The numerical analysis has been developed in two different steps, which correspond to the application of vertical loads (first step) and horizontal displacement (second step). In particular, during the first step, to the upper RP only vertical degree of freedom (DOF) was left free, while the others movements were constrained; then vertical concentrated forces (F=50 kN) have been applied to the upper nodes in correspondence of the four axial springs, in order to simulate the total compressive load of 200 kN. In the second step, the boundary condition of the top RP was changed: an entire DOF restriction was introduced in order to fix the vertical displacements obtained in the first step. Hence, an increasing horizontal displacement was applied to the central RP. The boundary condition of the bottom RP has been assumed equal for both steps. In particular, only rotations in the plane of the panel and horizontal translations have been permitted, while the other DOFs have been blocked. In order to take into account that small horizontal movements could occur due to the not infinite stiffness offered by the bolts-masonry system, a translational external spring has been introduced. Since in the bolt-masonry a shear-out failure did not occur during the test, the stiffness of the introduced spring has been evaluated considering an elastic behaviour. Therefore, according to Gelfi and Giuriani (1987), who proposed a theoretical model for the concrete-steel bolt connections, the translational stiffness K_t of the connection has been defined as K_t=7.6 kN/mm.

A picture of the URM FE Model is provided in Fig. 5.

The masonry has been modelled by adopting a plastic-damage model material, namely concrete damage plasticity (CDP), similarly to other research experiences carried out by the authors of this study (Campitiello et al., 2007, De Matteis and Mazzolani, 2010, Zizi et al., 2017a). In particular, the CDP, which is available in the software material library, refers to the amendments proposed by Lee and Fenves (1998) to the plastic-damage model originally implemented by Lubiner et al. (1988).

Since they do not influence significantly the global response at the end of the analysis (Zizi et al. 2017b), the base parameters of the adopted material model have been defined according to the default values proposed in software guide (Abaqus Inc., 2014).

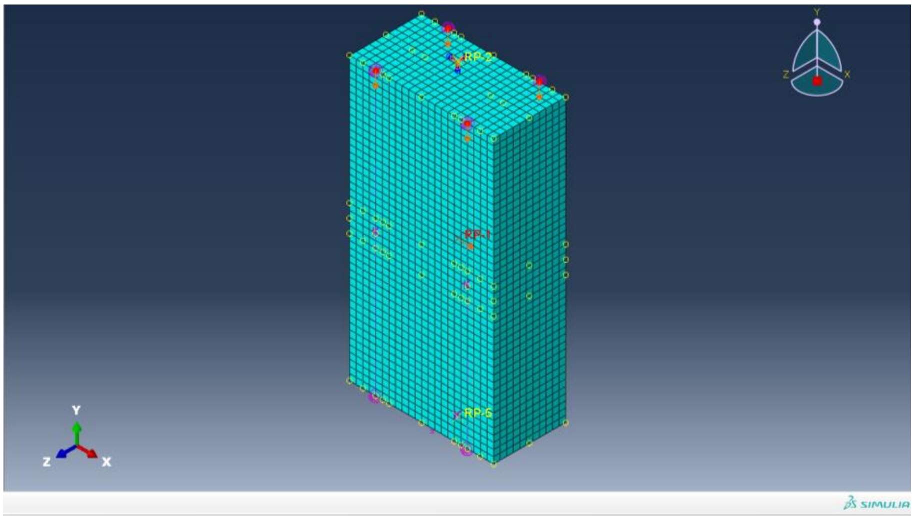

Fig. 5 The FE Model for the unreinforced system (URM)

Therefore, the following data have been assumed: i. dilation angle ψ=35°; ii. ratio between biaxial and uniaxial compressive strength f_{bo}/f_{co}=1.16; iii. eccentricity ε=0.1; iv. parameter that influence the yield surfaces in the deviatoric plane K_c=2/3.

The uniaxial behaviour of the masonry has been defined according to the results obtained by the preliminary material tests and following an iterative calibration procedure. In particular, an initial Elastic Modulus E=633 MPa has been assumed, as obtained in the double flat-jack test. Hence, a parabola-rectangular compressive stress-strain law has been adopted. The compressive strength (σ_k) has been evaluated according to the results obtained in the preliminary tests (f_k=0.89 MPa and f_k=1.07 MPa for double flat-jack and compressive test, respectively), while an elastic limit stress (σ_y) corresponding to the 30% of the ultimate one has been assumed. Therefore σ_k=1 MPa and σ_y=0.3 MPa, with an ultimate strain ε_u=0.37%, have been fixed (Fig. 6a). The uniaxial behaviour in tension has been defined according to an analytical and then an iterative procedure, which for the sake of brevity are not reported in the present paper. In particular, a fracture energy approach has been used, with a strength of σ_t=0.23MPa and a stress-displacement linear constitutive law with an ultimate displacement u_{t0}=2.5mm (Fig. 6b).

At the end of the analysis, the adopted URM allowed a correct interpretation of the experimental results both in terms of quantitative behaviour and failure mode.

In fact, the obtained force-displacement curve (Fig. 7a) is able to reproduce the experimental test result with a good accuracy. Furthermore, as shown in Fig.7b, 7c and 7d, the crack pattern at the failure point, which has been conventionally assumed at the maximum displacement of 18mm, revealed also a good agreement with the reference experimental test. Indeed, the numerical analysis revealed a more serious damage level of the upper half panel.

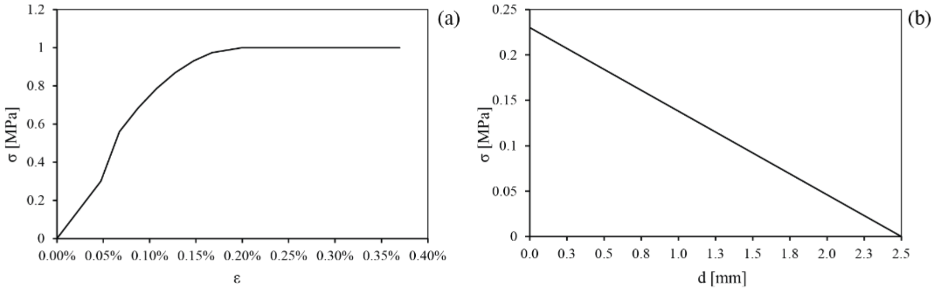

Fig. 6 Uniaxial stress-strain law in compression (a) and uniaxial stress-displacement post-elastic law in tension (b).

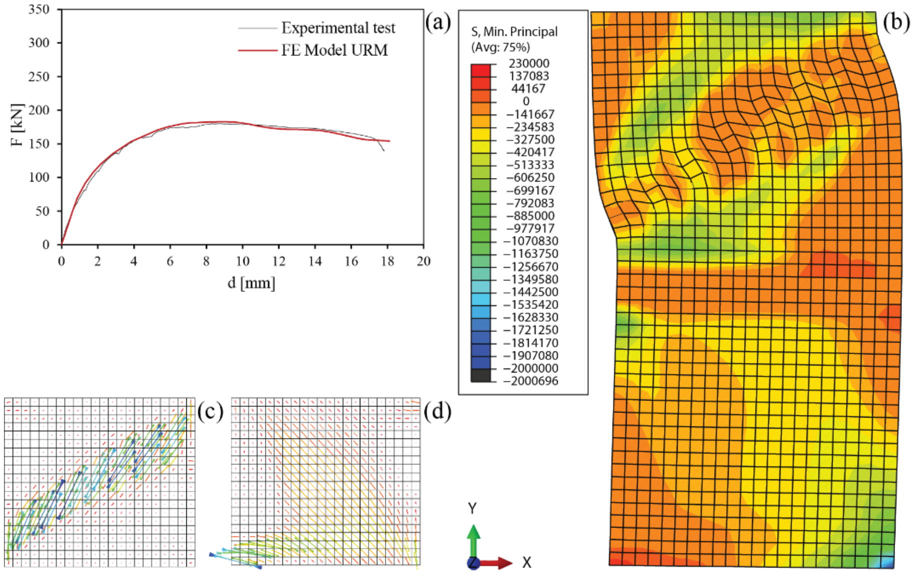

Fig. 7 Results of the FE Model analysis for the unreinforced panel (URM): force-displacement curve (a), compressive stress at the collapse point (deformed shape amplified 5 times) (b) and plastic compressive strain of the upper (c) and downer (d) half panels

3.3 The FE Model for the Reinforced Panel RM

The reinforced RM FE Model has been defined on the same basis of the URM FE model. Also in this case the presence of the external plaster has been neglected, while the masonry solid elements has been modified in order to simulate the presence of holes for the application of steel ribbons. The overlapped stainless ribbons have been considered as a unique element, considering bi-dimensional reduced integration 4 node elements (CPSR4) with a thickness of 2 mm.

The material model adopted for the reinforcement elements has been defined according to the experimental tensile tests preliminary carried out (Spinella et al., 2014). In particular a typical bi-linear material law has been adopted, with an Elastic Modulus E=212000 MPa, a yielding stress of f_y=317 MPa, an ultimate strength f_k=567.3 MPa and an ultimate strain ε_u=5.4%.

Moreover, the pre-tension to the stainless steel ribbons, which could be approximatively estimated equal to σ_p=200 MPa, has been conferred by means of thermal loads. Therefore, an *expansion* behaviour has been settled for the material assuming the typical thermal coefficient of the steel $\alpha_c=1.2\cdot10^{-5}$ °C^{-1}. The difference of temperature ΔT to assume in order to obtain the aforementioned stress state has been evaluated according to Eq. (1):

$$\sigma_p = \alpha_c \cdot E \cdot \Delta T \cdot \tag{1}$$

Hence, an additional step has been introduced in the RM before the application of the compressive load, in which the temperature of the steel elements has been varied from a value of 0°T of the initial state, to -80°C.

The *interaction* between the CAM® system and the existing masonry panel has been defined with a *hard contact* for the normal behaviour, and a *penalty* with a friction coefficient of 0.3 for the tangential one. Moreover, rigid bodies have been introduced in correspondence of the small metallic plates at masonry holes. Aimed at simulating the perfect connection of the ribbons with the top, middle and bottom rigid beams, the terminal nodes of the steel elements have been included in the existing rigid bodies.

In order to take into account the additional contribute offered by the steel plates welded to the UPN 200, the horizontal stiffness of the translational spring adopted for the downer reference nodes has been modified in the RM. In particular a value K_t=10 kN/mm has been adopted on the basis of a preliminary calibration procedure carried out with the aim to obtain the same lateral stiffness of the real test.

In Fig. 8 a picture of the implemented RM FE Model is provided.

At the end of the analysis, the RM revealed a good correspondence with the experimental test in terms of global response, initial stiffness and maximum horizontal force (Fig. 9a).

Since the real test was interrupted before the complete collapse of the wall, the failure point in the numerical analysis has been conventionally assumed when the horizontal force decreased more than 20%. This condition corresponded to a maximum displacement of 30 mm.

It is important to note that after the elastic branch of the obtained force-displacement curve, there is a disagreement between the two curves. This is probably due to some local failures that occurred in the experimental test. In fact, in the experimental curve, it is possible to observe some parts where there is increment of displacement with a constant force. The stress and strain states at the failure point plotted in Fig. 9b, 9c and 9d showed a very similar damage state of the both half masonry panels with small rotation at the base.

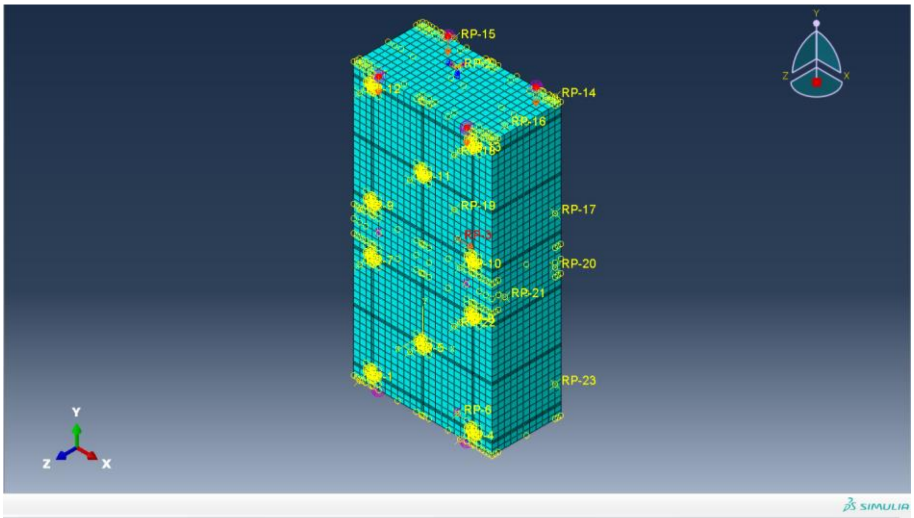

Fig. 8 The FE Model for the reinforced system (RM)

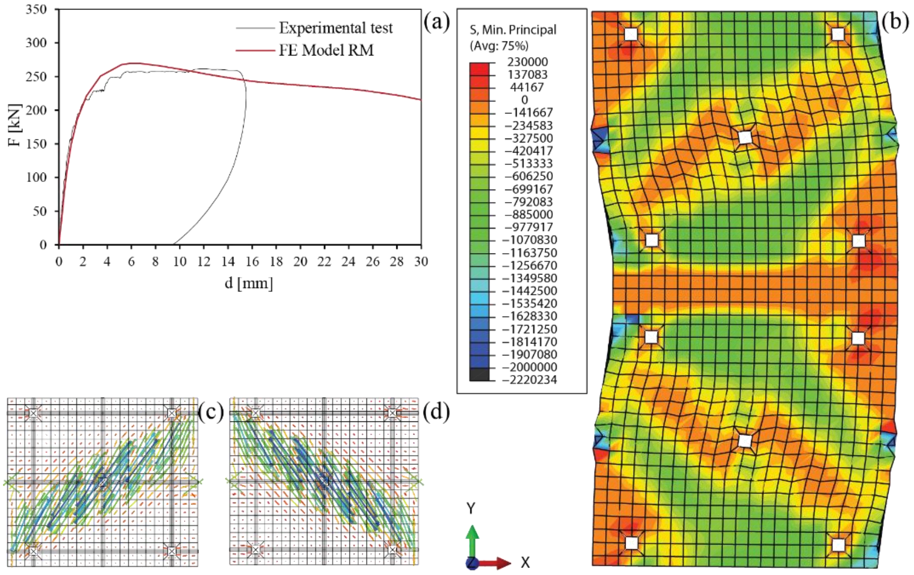

Fig. 9 Results of the FE Model analysis for the reinforced panel (RM): force-displacement curve (a), compressive stress at the collapse point (deformed shape amplified 2.5 times) (b) and plastic compressive strain of the upper (c) and downer (d) half panels

4 Conclusions and Future Developments

In this paper, the problem of structural preservation of the existing masonry buildings has been faced by analysing innovative steel based retrofitting techniques suitable for improving the seismic behaviour of masonry walls under compressive and shear forces. In particular, the potentialities of the CAM® system, patented by the Italian company EDILCAM® Sistemi Srl, has been examined based on a previous experimental and research activity.

To this purpose, a FEM model able to reproduce the shear behaviour of a masonry walls retrofitted with the CAM® system has been set up, considering as reference tests the experimental results on full scale masonry panels obtained within the research project In.CAM.M.I.N.O..

The obtained outcomes show that there is a very good agreement between numerical and experimental results, in terms of strength, ductility and stiffness, evidencing that the implemented numerical tool could be profitably adopted in the future in order to define suitable design criteria for this retrofitting technique when applied to simple and complex structures.

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