Seismic vulnerability assessment of confined masonry buildings by macro-element modeling: a case study

F. Nucera, A. Santini, E. Tripodi
Mediterranean University of Reggio Calabria, Italy

I. Caliò
University of Catania, Italy

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SUMMARY
Confined masonry structures are seismic resisting structures, where masonry walls are confined by reinforced concrete beams and pillars. During the construction of a confined masonry structure, masonry walls are used as formworks to build the reinforced concrete elements. The reinforced concrete frame plays the important role of confining masonry walls, and therefore helps in increasing the ductility of the whole structure; this implies better performances of the confined masonry with respect to the traditional masonry structures. In confined masonry structures, openings are confined by reinforced concrete frames, while wall intersections and floor slab-wall connections are realized by means of reinforced concrete elements. As observed after several severe earthquakes, confined masonry structures showed a reliable anti-seismic behavior due to several reasons. Among them, the confinement action of the reinforced concrete frames, the in-plane floor stiffness, the plan and elevation regularity. In this paper a case study is considered, which is a building located in Reggio Calabria and built in 1932 during the reconstruction following the 1908 earthquake, which struck and destroyed a large part of the city. The original plan of this building has been found in a historical archive. Therefore information is available about the mechanical characteristics of the materials and some structural details. The seismic vulnerability of the structure is assessed according to the Italian Seismic Code by using 3DMacro structural analysis software. Masonry walls are modeled through an innovative "macro-element", which allows to take into account different collapse mechanism: bending failure (rocking), shear failure by diagonal cracking, shear failure due to sliding. Secondary element, such as pillars, beams, architraves, are modeled by using nonlinear frame element having concentrated plasticity at their ends. A critical analysis of the results has been performed, regarding both the seismic vulnerability in terms of push-over curves and the techniques to improve the anti-seismic performances.

Keywords: seismic vulnerability, confined masonry structures, macro-element modelling

1. INTRODUCTION

The analytical modelling of the nonlinear behaviour of reinforced concrete-masonry structures can be conducted by detailed nonlinear finite element analyses of by simplified approaches. Detailed finite element analyses generally require constitutive laws for reinforced concrete elements, usually modeled with diffused or lumped plasticity elements, and for the masonry elements taking into account the limited tensile strength of simple masonry. Two- or three-dimensional inelastic elements are usually adopted for the modeling of unreinforced masonry. A detailed finite element approach, even though capable at giving a deep insight on the nonlinear behaviour of the component materials, or their interaction and on the local and global collapse mechanisms, is extremely time consuming during both the modelling and the results interpretation phases. Moreover, its complexity and some convergence issues, usually make the described approach not suitable for nonlinear dynamic analysis of real three-dimensional buildings. To partially overcome the complexity of detailed finite element analysis, some simplified analytical models were proposed especially for simple masonry buildings. Generally these simplified approaches adopt equivalent nonlinear frame elements, or more complex mechanical sub-assemblages, in the modelling and analysis of unreinforced masonry panels. With reference to mixed structures (i.e. confined masonry buildings) the simplified models usually consider reinforced concrete and masonry elements arranged in series or in parallel, without taking into account for the confining effects, which, at least in case of confined masonry systems, play a crucial role on the seismic performance of the structure. In the present study, a simplified analytical approach, previously presented in the literature (Caliò et al., 2012), is applied to evaluate the seismic resistance of confined masonry structures. According to this model, unreinforced masonry panels are modeled by two-dimensional macro-elements (Caliò et al., 2008), whereas the reinforced concrete elements are modeled by lumped plasticity elements interacting with the masonry through nonlinear interface elements.

2. CASE STUDY DESCRIPTION: "ISOLATED 78"

After the 1908 Messina and Reggio Calabria earthquake which was a quite destructive event, the reconstruction of the cities was started in organic fashion by employing the confined (i.e. framed) masonry building technique. This type of construction is widely present in this named cities but also in many other areas in Italy as well as in several other countries all over the world. Composite reinforced concrete-masonry structures are particular structural systems such that both unreinforced masonry panels and reinforced concrete elements give contribution to the overall stiffness and strength. Engineered composite reinforced concrete-masonry was introduced in Italy by the 1909 seismic code. We consider a set of buildings named "isolated 78" made by seven framed- masonry structure. The case study is the building identified by the marker "A2", its shape in plan is rectangular with 20,30 m length 11,40 width and 11,40 m height. The building has three floors with the lower of them partially basement; the top floor is accessible and usable.

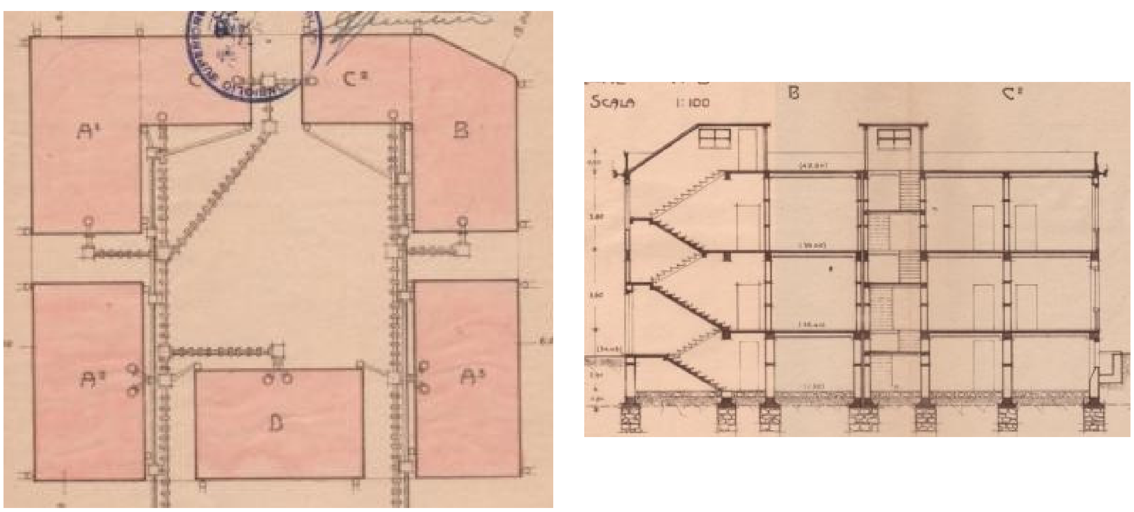


Figure 1. Original drawings: plan view of the isolated 78 and section of the building

Foundation of the building is constituted by a reinforced concrete system of orthogonal beams whose bases are stone masonry walls. Masonry panels are in both directions, x and y in plan, framed and thus confined by reinforced concrete elements connected horizontally and vertically between them; also doors and windows openings are framed by reinforced concrete elements. At the basement the cross section of columns is 50x55 cm and the reinforcement bars 2ϕ30mm are placed symmetrically. For the first floor the columns cross section is 40x50 cm whereas for the second floor is 30x38 cm; both are symmetrically reinforced with 2ϕ28mm and 2ϕ20 steel bars, respectively. Beams have also different cross section dimensions: they are 50 cm in width at the basement floor, 40 cm and 30 cm at the first and second level, respectively. The height of beam cross section changes across the beam length being larger in the neighborhood of the clamped ends and smaller in the middle span; they also have different reinforcement percentage depending either on their position in plan and depending on the floor they belong to. Walls are made by stone masonry with 50 cm width in the basement, by solid brick having 40 cm width at the first level and 27 cm at the second floor. In the basement the partition walls are made by stone masonry whereas at the first and second level they are made by hollow brick masonry whose width ranges from 30 cm to 40 cm and from 20 cm to 30 cm at the first and second level, respectively. Masonry panels are framed and so confined by reinforced concrete elements; intersection between walls, jambs and architraves are reinforced concrete elements. The first and second floor slabs are constituted by a reinforced concrete plate with intrados beams; the third floor slab is simple reinforced concrete plate.

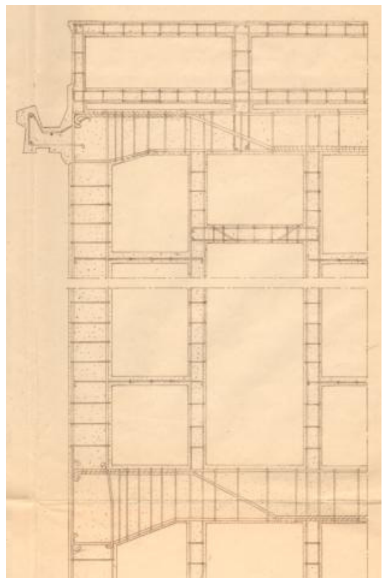


Figure 2. Particular of the reinforced concrete element: beams, columns and confining elements

3. ANALYTICAL MODELING THROUGH MACRO-ELEMENTS

The simulation of the nonlinear dynamic behaviour of a masonry building represents a challenging problem which rigorously requires the use of computationally expensive nonlinear finite element models and, above all, expert judgment. The different behaviour of masonry structures, compared to ordinary concrete and steel buildings, requires ad hoc algorithms capable of reproducing the nonlinear behaviour of masonry media and providing reliable numerical simulations. Refined finite element numerical models, such as the smeared cracked and discrete crack finite element models (Penelis, 2006; Seible et al., 1991) able to predict the complex nonlinear dynamic mechanical behaviour and the degradation of the masonry media, require sophisticated constitutive laws and a huge computational cost. As a consequence, these methods are nowadays not suitable for practical application and extremely difficult to apply to large structures. An alternative approach to the nonlinear FEM is represented by the rigid-body spring models. In the last three decades, many authors have developed simplified or alternative methodologies that, with a reduced computational effort, should be able to predict the nonlinear seismic behaviour of masonry buildings and to provide reliable numerical results for engineering practice purposes. The most commonly used practical approach for the analysis of masonry structures is the so called 'equivalent frame model', in which the masonry building is represented by an equivalent nonlinear frame structure constituted by nonlinear beam elements and rigid offsets. An overview on recent code developments and state-of-the-art methods of earthquake resistant design of masonry buildings is reported in (Tomazevic, 2006) where the experimental results are also used in order to justify the analysed numerical approaches. In this paper a new modeling approach for the simulation of the seismic behaviour of masonry buildings, suitable for current engineering practice applications, is employed (Caliò et al., 2012). The proposed approach is based on the concept of macro-element discretization (Lourenço, 2002) and has been conceived with the aim of capturing the nonlinear behaviour of an entire masonry wall and of the entire building, as an assemblage of several walls. The model is based on a plane nonlinear discrete element, able to simulate the behaviour of masonry wall in its own plane. The basic macro-element consists of an articulated quadrilateral with rigid edges in which two diagonal springs govern the shear behaviour. The flexural and sliding shear behaviour is governed by discrete distributions of springs in the sides of the quadrilateral that preside over the interaction with the adjacent macro-elements. The calibration of the model require only a few parameters to define the masonry material based on results from current experimental tests. The computational cost of the proposed numerical approach is greatly reduced, compared to a traditional nonlinear finite element modelling. Since the equivalence between the masonry portion and the macro-element is based on very simple physical considerations, the interpretation of the numerical results is simple and straightforward. This novelty approach is intended as a tool, which requires low computational resources, for investigating the nonlinear behaviour of masonry buildings. The reinforced concrete elements are modelled by lumped plasticity elements interacting with masonry panels through nonlinear interface elements (Caliò et al., 2008). Each edge of the panel can interact with other elements or external restraints by means of discrete distribution of nonlinear spring (*interface*).

4. THE SEISMIC PERFORMANCE OF THE BUILDING

The software 3DMacro, developed at the University of Catania, has been used to built the computational model of the case study building. This software allows the implementation of the approach previously described for composite reinforced concrete-masonry structures. The geometrical and mechanical features of the case study are reported from the original design report found in the historical archives. Although this kind of (widely diffused) structures were designed according to very primitive seismic codes, it turns out that they have, generally speaking, fairly good seismic performances. Therefore we have investigated the seismic behavior and response according to the actual Italian seismic code, i.e. the DM 14.01.2008 and Circolare n. 617, which are performance based codes. The mechanical characteristics of the materials employed in the computational model al listed in the table below.

Table 4.1. Masonry mechanical characteristics of the computational model

Masonry type	Young's modulus, MPa	Transvers. modulus, MPa	Compress. strength, MPa	Tensile strength, MPa	Shear strength, MPa	Ultimate shear strain	Density, kN/m^3
Stone	1500	500	1.93	0.20	0.042	0.005	22
Solid brick	1200	400	1.78	0.20	0.040	0.006	18
Hallow brick	854	280	1.68	0.10	0.038	0.003	12

Table 4.2. Concrete and steel mechanical characteristics of the computational model

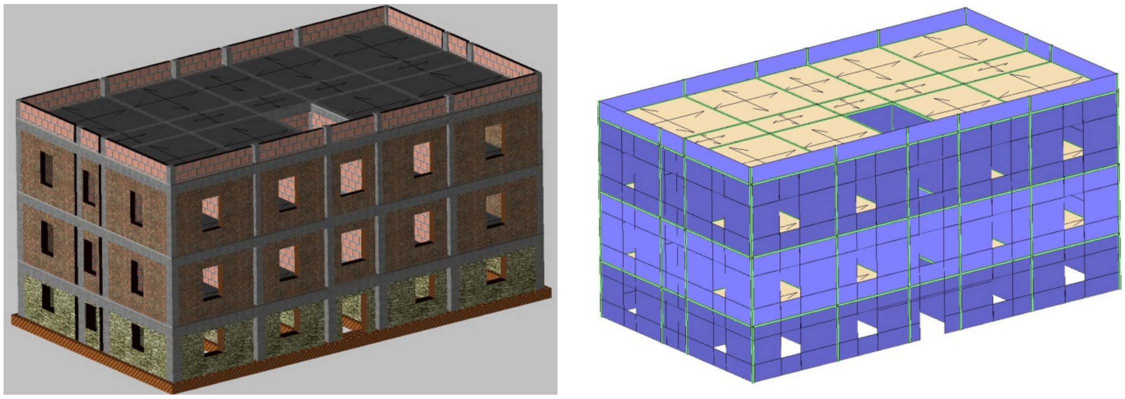
Material	Compression strength, MPa	Yield strength, MPa	Ultimate strain
Concrete	15.00	-	0.003
Steel	-	220.00	0.010

According to the Italian seismic code the soil is defined as class C and the usage class of the building is II. We have considered the following dead loads

Table 4.3. Slab dead and live loads

Slab type	Self weigth, kN/m^2	Floor, kN/m^2	Floor rough, kN/m^2	Plaster, kN/m^2	Partition walls, kN/m^2	Live load, kN/m^2
1st and 2nd	2.65	0.77	2.00	0.40	2.00	2.00
Top	2.50	0.77	1.00	0.40	-	1.00

We investigate the seismic performance of the case-study structure through nonlinear static (pushover) analysis; two load set have been considered: (1) mass proportional and (2) inverse-triangular distribution. Considering that the earthquake hits randomly the structures without any preferential direction, we considered the principal x and y as principal load directions, having both positive and negative sense and also with and without adjunct eccentricity.

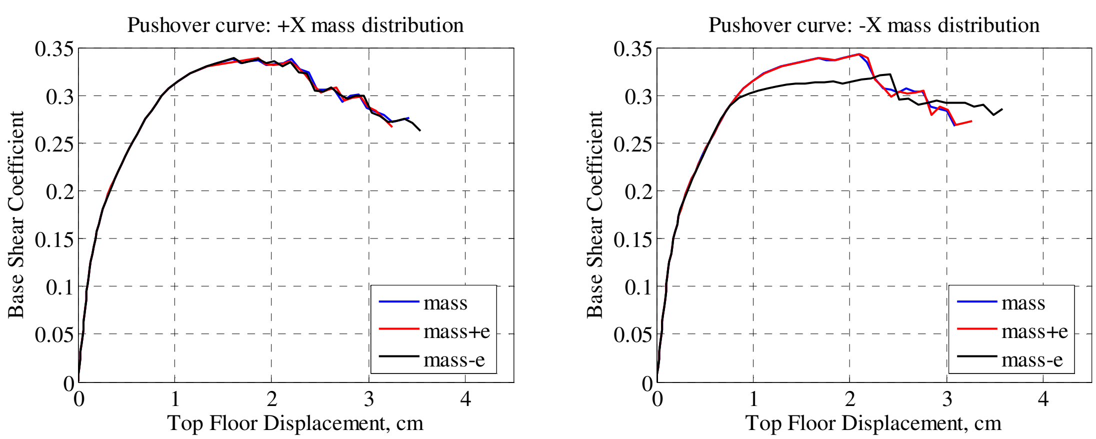


a) b)

Figure 3. View of the case study. a) solid model; b) computational model

To estimate the seismic resistance of the considered model, nonlinear static (*pushover*) analyses have been carried out; two load distribution have been considered: (1) a *mass-proportional* distribution and (2) an *inverse-triangular* distribution. X and Y principal directions of the building with and without artificial eccentricity have been considered as load directions. The results are give in terms of capacity curves with the base shear, normalized respect to the building weight, as a function of the top floor displacement.

By observing the following figures, from 4 to 7, we note that the displacement capacity, with reference to a control point at the top floor, ranges from 3 to 4.5 cm which corresponds to a 0.4% of drift. The maximum base shear coefficient ranges from 0.25 to 0.35 which confirm that in terms of strenght to horizzontal force a simmetric confined masorny structure (the case study) show an acceptable seismic performance.



a) b)

Figure 4. Pushover curves with and without artificial eccentricity. a) Mass proportional load distribution along X direction in positive sense; b) Mass proportional load distribution along X direction in negative sense

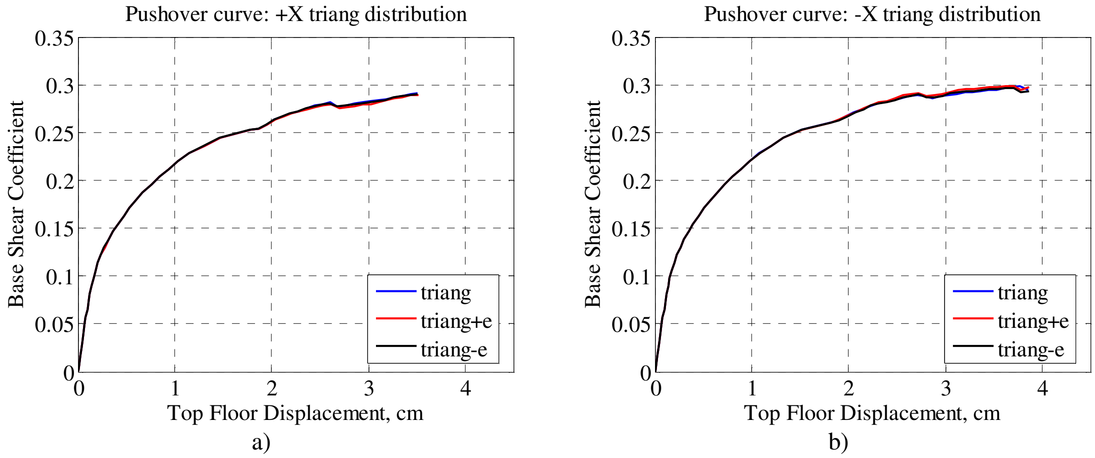


Figure 5. Pushover curves with and without artificial eccentricity. a) Inverse triangular load distribution along X direction in positive sense ; b) Inverse triangular load distribution along X direction in negative sense

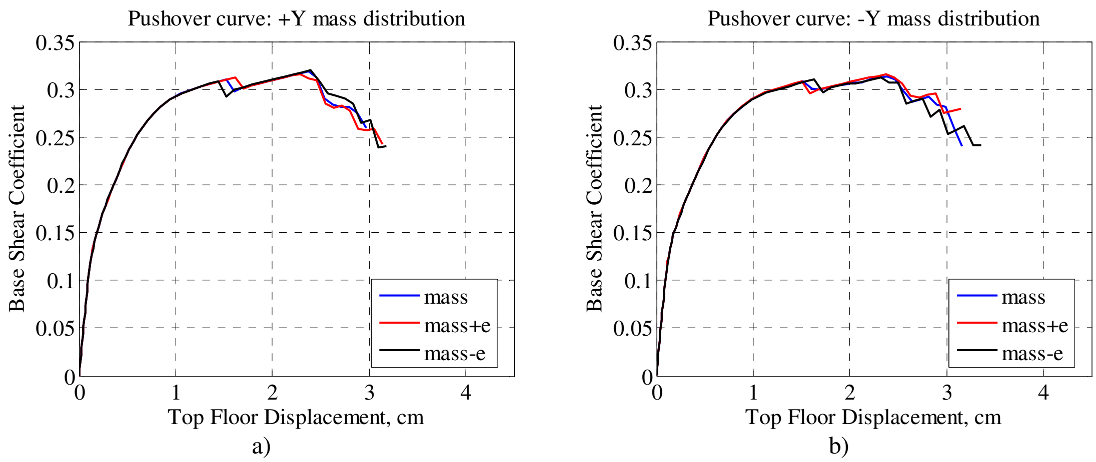


Figure 6. Pushover curves with and without artificial eccentricity. a) Mass proportional load distribution along Y direction in positive sense; b) Mass proportional load distribution along Y direction in negative sense

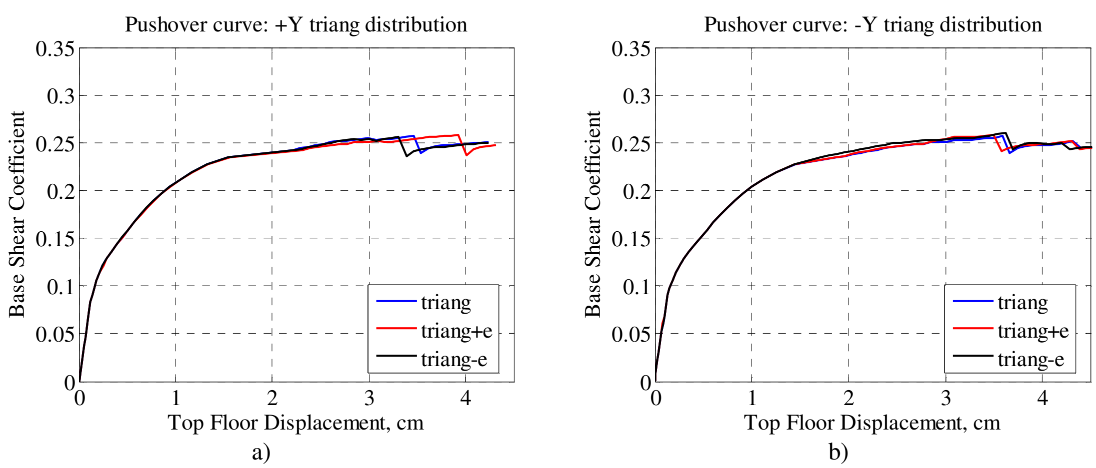


Figure 7. Pushover curves with and without artificial eccentricity. a) Inverse triangular load distribution along Y direction in positive sense ; b) Inverse triangular load distribution along Y direction in negative sense

In the worst load combination (cfr Figure 8), which turns out to be the inverse triangular load distribution along the positive X direction with negative artificial eccentricity, the structure seismic performance satisfy the Italian code requirements. In particular, for the Damage Limit Stage (SLD) the ratio displacement capacity over demand is equal to 228.5% which largely satisfy the requirement; with reference to the Life Safety Limit Stage (SLV) the ratio capacity over demand is approximately 100% which means that the capacity requirement for an existing construction (not new built) is satisfied.

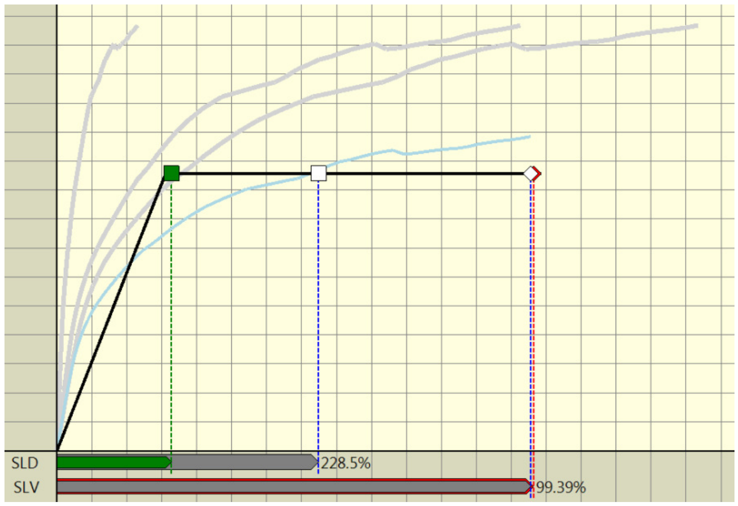


Figure 8. Performance based verification according to the Italian code: case of the inverse triangular load distribution along the positive X direction. Damage (SLD) and Life Safety (SLV) limit state verifications.

For the sake of brevity we show only some confined masonry walls condition in correspondence of the life safety limit state. The 3DMacro software takes into account several different masonry panel behaviour and different way of failure like those summarized in the following figure

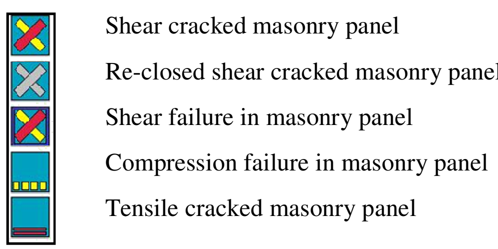


Figure 9. Nonlinear masonry behaviour considered in the analyses

Moreover, also the reinforced concrete elements are considered to have nonlinear behaviour. The plasticity is concentrated at the end on the frame elements; in particular, green highlights the opening stage of the hinge, yellow colour means that the hinge has reached three quarters of the ultimate rotation capacity and red colour denotes a hinge completely collapsed.
In figure 10 it showed the overall picture of masonry wall number 11 at the life safety limit stage; it is possible to understand as the second floor shows insufficient shear strength; in fact all the shear walls at that level are undergoing a shear failure. There are also many masonry panel which show failure due to shear and due to overcoming of the maximum tensile strength. The reinforced concrete elements show mainly green and yellow end hinges. In figure 11 it showed the overall picture of masonry wall number 11 at the life safety limit stage corresponding to a mass proportional load condition along the positive X principal direction of the building; it is possible to understand as both the first and the second floor shows insufficient shear strength; in fact all the shear walls at that level are undergoing a shear failure. There are also many masonry panel which show failure due to shear and due to

overcoming of the maximum tensile strength. The reinforced concrete elements show mainly green and yellow end hinges but also a certain number of red hinges denoting a collapse condition.

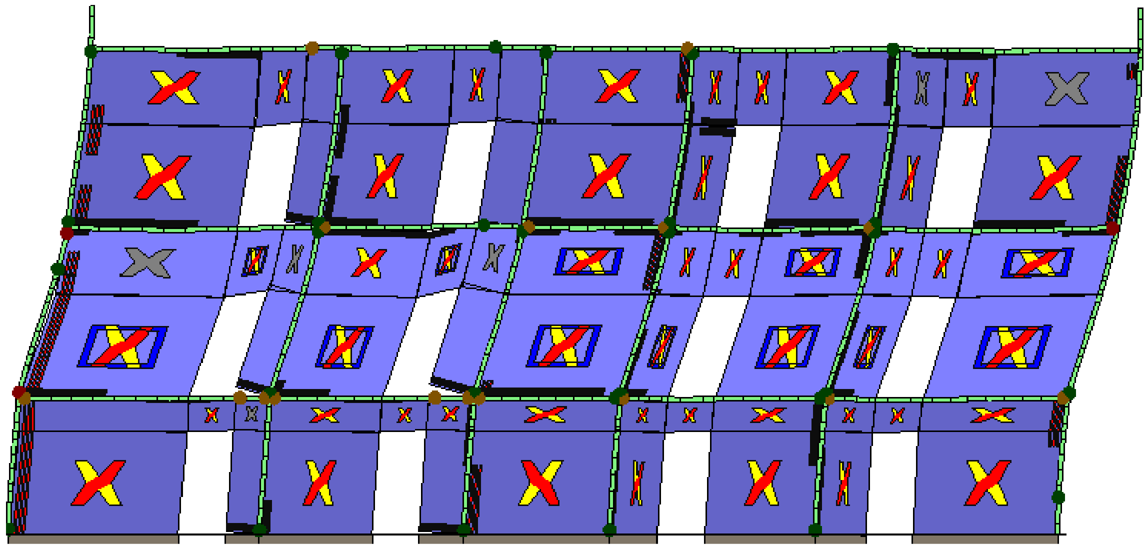


Figure 10. Pushover +X inverse triangular load condition. Configuration of wall n. 11 at the life safety limit stage (analysis step n. 55)

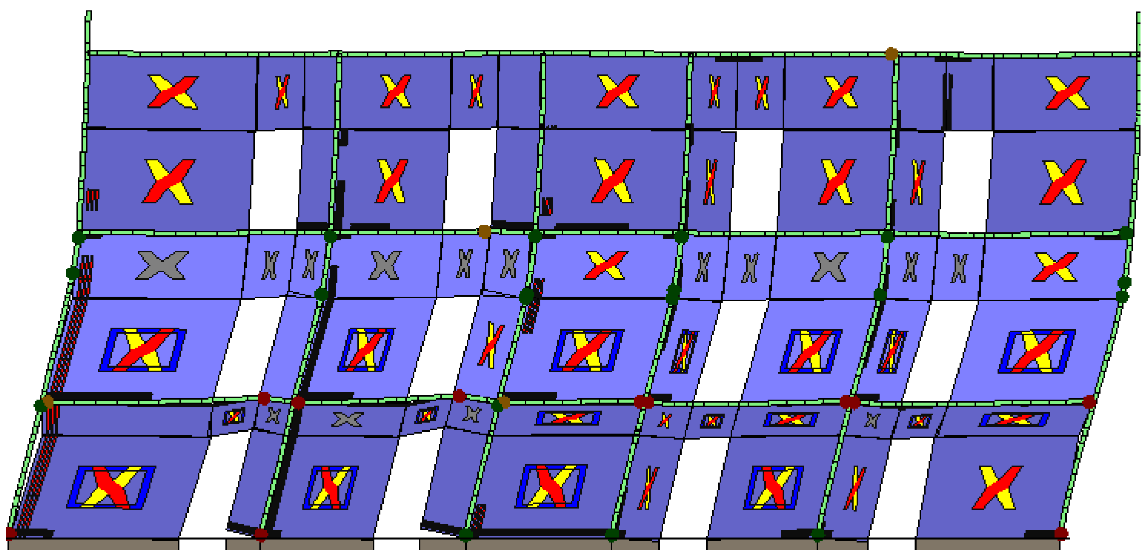


Figure 11. Pushover +X mass proportional load condition. Configuration of wall n. 11 at the life safety limit stage (analysis step n. 68)

5. CONCLUDING REMARKS

We have considered a build system quite widely present in Italy and also in other cities all over the world namely the confined/framed masonry structure. We have focused our investigation on a case study in Reggio Calabria identified as "isolated 78". Having described geometry, mechanical characteristics of materials and load conditions according to the actual Italian seismic codes (i.e. D.M. 14.01.2008 and Circolare n. 617) the seismic response and performance has been studied through nonlinear static analyses with different load cases and combinations. Generally, such kind of seismic resistant structures, despite the fact that they were designed according to very primitive seismic codes over the first decades of the past century, have desirable anti-seismic performances. We performed pushover analyses following two different path load: mass proportional and inverse triangular load distribution along the height of the building. It turns out that this engineered typology of construction, i.e. the confined masonry structure, offers reasonably good anti-seismic performances according to the Italian seismic code. The 3DMacro software allows the user to take into account several types of failure in the masonry panels and also the confining effects of the reinforced concrete elements. This features are useful for a deep understanding and study of the anti-seismic capacity of confined masonry structures and to design suitable seismic retrofitting interventions.

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