Influence of geometrical and mechanical parameters on the seismic vulnerability assessment of confined masonry buildings by macro-element modeling

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

F. Cannizzaro, B. Pantò
University of Catania, Italy

SUMMARY
A new innovative structural system was developed in the reconstruction of Reggio Calabria city after the destructive 1908 earthquake, which was termed "confined masonry". 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. Nowadays many confined masonry buildings still exist in Reggio Calabria city and, in the frame of seismic risk reduction, the evaluation of their seismic vulnerability is of a great importance. However, when performing a large-scale seismic vulnerability assessment of a class of buildings, the mechanical properties of the structural elements cannot be precisely evaluated by means of in-situ experimental tests. These properties can only be estimated within reasonable intervals. In this paper a parametric study have been performed in order to investigate the influence of some mechanical parameters on the seismic response of confined masonry buildings. In particular, the effects of the quality of masonry in terms of shear and compressive strength, deformation capacity and Young and shear modulus have been considered, as well as the influence of the longitudinal bars and stirrups. Finally, the compressive strength of concrete is considered and its importance on the seismic response is estimated. The seismic performances in terms of push-over curves have been evaluated according to the Italian Seismic Code by means of 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.

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

1. INTRODUCTION

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, whereas the reinforced concrete elements are modeled by lumped plasticity elements interacting with the masonry through nonlinear interface elements. 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 modelled 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 modelling 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.

2. CONFINED MASONRY AS SEISMIC RESISTING STRUCTURE

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. 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. Engineered composite reinforced concrete-masonry was introduced in Italy by the 1909 seismic code. The main contribution in terms of the strength and stiffness to the seismic capacity of the structure is due to the masonry walls. Nonetheless, the role of the reinforced concrete elements is very important. First of all there is an effective cooperation and interaction between those two typologies namely the masonry and the reinforced concrete elements; this is also due to the fashion through the framed masonry construction are built: masonry walls are raised firstly and they are used, floor by floor, as formwork to build the reinforced concrete elements. Most importantly, the presence of vertical and horizontal reinforced concrete element plays a significant role in avoiding untimely fracture of the masonry and therefore is helpful in collapse prevention of a single (or group) of masonry panels. In fact, the behaviour of a framed masonry panel has a little more strength and much more ductility (i.e. capacity of dissipating energy in plastic region) with respect to unconfined masonry panel. Besides, the reinforced concrete elements create a very good connections between in plane and orthogonal masonry panels so that first mode collapse of walls (i.e. out of plane collapse) is entreated.

3. CASE STUDY

We focus our attention on the groups of confined masonry buildings namely “isolated 45” afterwards described. 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. 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 1. Original picture: view of the isolated 45

Figure 2. View of the isolated 45 as it is constructed in Reggio Calabria

For “isolated 45” the foundation cross section is variable and it ranges from a 100x130 cm to a 100x180 cm, although in the computational analyses the soil structure interaction, without losing generality, is not considered and the foundation is fixed. For the basement the columns cross section is 40x65 cm whereas for the first and second floor is 27x45 cm; both sections are symmetrically reinforced with 2ϕ22+2ϕ18mm steel bars at the basement, 2ϕ22 at the first floor and 2ϕ20 at the second floor. Beams have also different cross section dimensions: they are 40 cm in width and height ranges from 45 cm to 100 depending on the floor; moreover the height of 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 solid bricks with 40 cm width in the basement and in the first floor, whereas they have 27 cm width at the second floor.
The mechanical characteristics of the materials employed in the computational model al listed in the table below for “isolated 45”.

Table 3.1. Masonry mechanical characteristics of the computational model – Isolated 45

Masonry type	Young’s modulus, MPa	Transvers. modulus, MPa	Compress. strength, daN/cm^2	Tensile strength, daN/cm^2	Shear strength, daN/cm^2	Ultimate shear strain	Density, kN/m^3
Solid brick	1200	400	18	0.5	0.5	0.004	18

Table 3.2. Concrete and steel mechanical characteristics of the computational model – Isolated 45

Material	Compression strength, MPa	Yield strength, MPa	Ultimate strain
Concrete	80.00	-	0.0035
Steel	-	160.00	0.0100

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 3.3. Slab dead and live loads – Isolated 45

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
Beam-plate	4.00	0.77	2.00	0.40	0.50	2.00
Plate	2.50	-	-	0.40	-	0.50
Top	0.70					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.

4. NONLINEAR SEISMIC ANALYSIS THROUGH “MACRO-ELEMENT” MODELING

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*).

5. NUMERICAL APPLICATIONS

In the present section the results of the numerical simulation of the case study are reported. In particular, the load direction influence on the strength and ductility of the structure has been investigated as well as the influence of the shear strength of masonry.

A three-dimensional global model has been implemented by using the well known 3DMacro software. Nonlinear static analyses have been performed with a force distribution proportional to the masses. The nonlinear static analyses have been conducted by imposing force increments till the conventional collapse of the structure. Four control points of the model have been monitored, one for each floor of the building.

The numerical model take into account a lumped plastic model for the reinforced concrete beams. However the formation of plastic hinges is allowed at each point of the beam length. The constitutive laws of the plastic hinges are elastic-perfectly plastic for the beams, whereas the PMM (axial load-bending moment) interaction is considered for the columns.

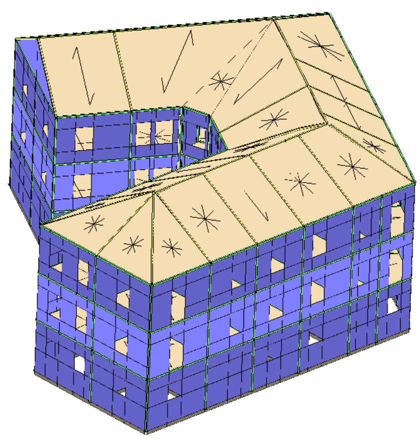

Figure 3. Three-dimensional global model: axonometric view

6. INFLUENCE OF THE LOAD DIRECTION

Eight nonlinear static analysis have been performed according to the four main directions and the four intermediate ones (i.e. with an shift angle of 45°).

The results are reported in terms of deformed shape of some relevant walls of the model, and by means of an original representation of the eight pushover curves.

In the following Figures the deformed shape of representative walls of the structure in the direction of the load are reported, with reference to the numerical analysis in the main directions.

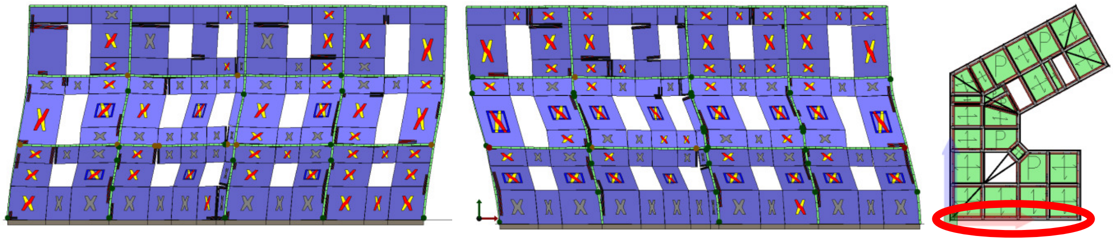

Figure 4. Deformed shape at collapse stage of a wall:
numerical analyses along the X axis of the building (0° and 180°)

The deformed shapes show the damage configuration of the structure at collapse. In particular the in the represented step the damage is mainly concentrated in the piers of the first two levels due to shear behaviour. Some of the masonry panels have reached the limit drift and their ruptures occurred. The collapse mechanism of masonry and the reinforced concrete frames are coupled, and some plastic hinges are opened. In the third level some panels are damaged due to rocking motion, especially in correspondence of the openings; in fact the presence of the reinforced concrete frame inhibit the mechanism of rocking in correspondence of the confined masonry.

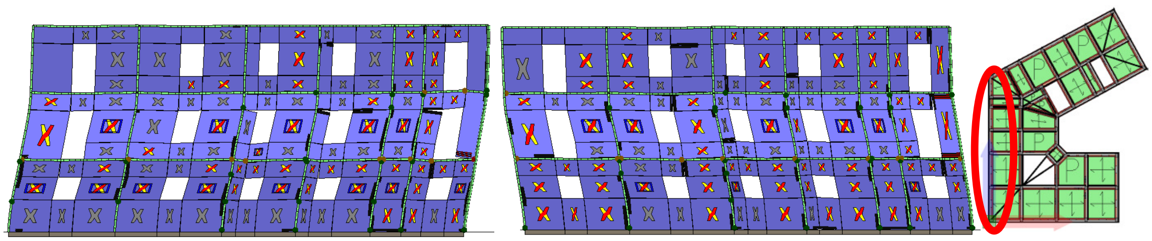

Figure 5. Deformed shape at collapse stage of a wall:
numerical analyses along the Y axis of the building (90° and 270°)

It is interesting to underline the mutual influence of masonry panels and reinforced concrete frame. In the following some pictures which show this effect are reported. In Figure 6 on the left, the bending moment in the beams with reference to the 90° analysis is showed; the bending moment trend demonstrates the need of considering potential plastic hinges not only at the beam ends but also all along its length. On the right, a plastic hinge history of a column is reported: after the elastic phase, the boundary of the yielding dominium has been reached and the plastic phase is modelled.

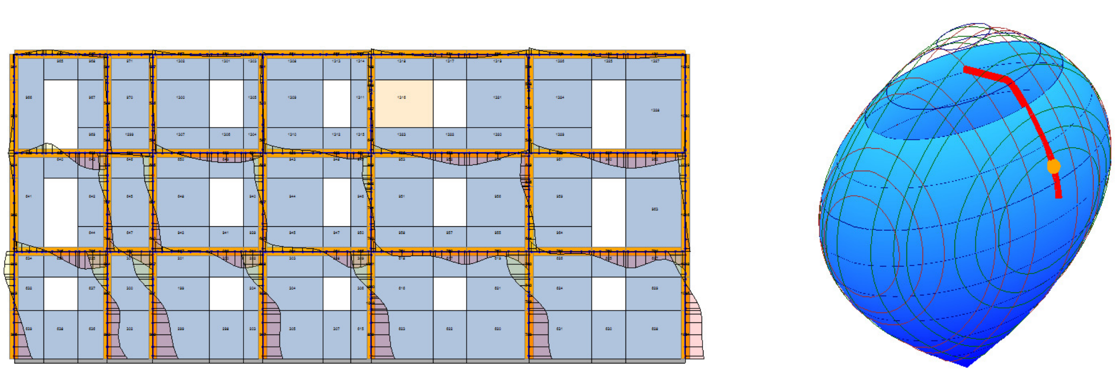

Figure 6. Bending moment diagram and plastic hinge history

With reference to the masonry panels two pictures are reported: on the left the shear force distribution in diagonal springs of the panels of a wall is considered, while on the right the interface force distribution is reported. It is clear that the bottom a generic panel is, the highest shear force it exhibits; while according to the interface the force distribution depends on the forces exchange with the reinforced concrete frame and among the masonry panels.

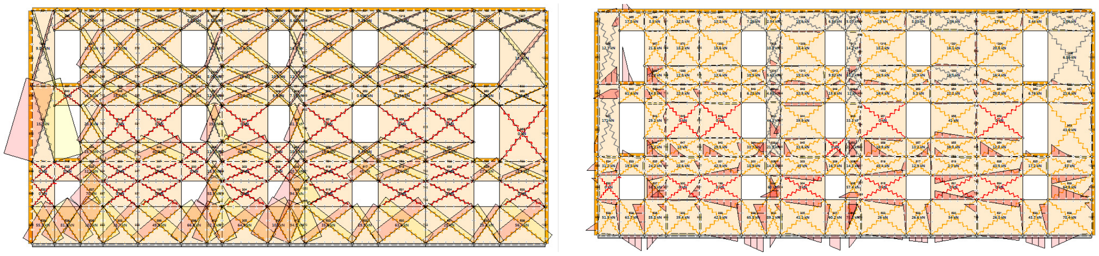

Figure 7. Shear force distribution and interface force distribution on the masonry panels

Some of the diagonal springs do not exhibit any force at the considered step. This is due to the reaching of the limit drift considered; in fact whenever a panel reaches its limit drift the corresponding diagonal springs lose its current force; this latter is locally redistributed to the structure. The following picture shows the force-displacement history of a diagonal spring in which the rupture occurs. In the plastic branch it is shown how the current strength changes due to the fluctuation of the confinement action during the numerical analysis.

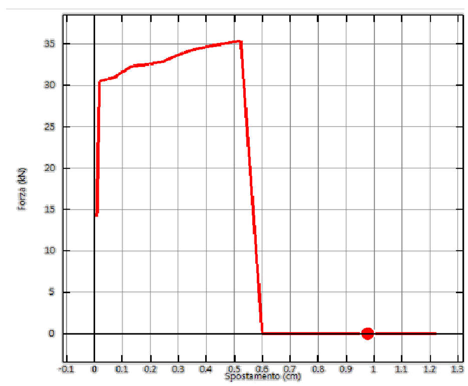

Figure 8. Force-displacement history in the diagonal spring of a masonry panel

The collapse mechanisms are similar whichever direction of load is considered, since the presence of rigid floor is considered. However, the structure along the intermediate directions seems to be more resistant than along the main directions, probably due to a higher number of hired walls. An original representation of the pushover curves, the "capacity basket", proposed for the first time in Caliò et al. (2008), is reported in Figure 9.

Each pushover curve of the eight analyses is reported with an orientation which corresponds to the load direction of the related numerical simulation. The pushover curves are linked by patches whose colour depends on the corresponding base shear coefficient. This representation can be regarded as capacity dominium of the whole structure. Along the Z-axis the base shear coefficient can be inferred, while in the XY view the displacement capacity, that is ductility data, can be obtained. Therefore in an unique representation the main data for each load direction are reported. It is clear that the more direction analyses are available the more the capacity basket is detailed.

The capacity basket shows that the intermediate directions exhibit an higher strength while in the main directions seem to be more ductile.

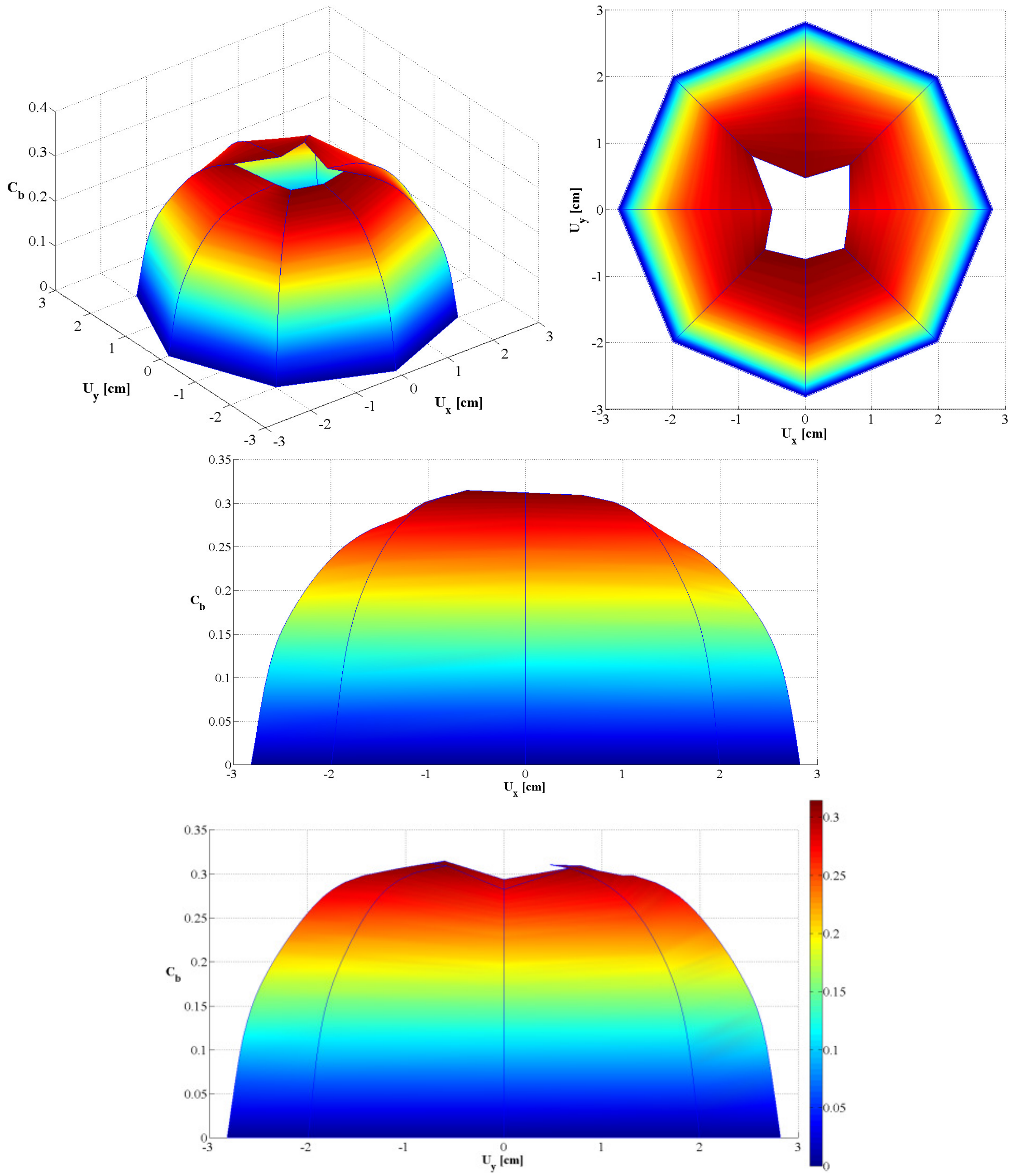

Figure 9. Capacity basket: axonometric and orthogonal views

7. INFLUENCE OF THE SHEAR STRENGTH

The influence of the mechanical parameters on the global response of the structure has been investigated. Since the collapse is mainly due to shear damage in masonry panels, the influence of the masonry shear strength has been considered.

Three different values of the shear strength have been considered, that is τ_0=0.5 daN/cm^2, τ_0=1.0 daN/cm^2 and τ_0=1.5 daN/cm^2. In order to appreciate the influence of this parameter a nonlinear static analysis in the X direction has been considered. The load distribution is proportional to the masses. Moreover, the numerical analyses have been conducted throughout two phases: first constant increments of loads have been applied, till the reaching of the maximum strength of the structure; then,

in order to evaluate the softening branch of the seismic response of the structure, the analyses have been continued by imposing displacement at each seismic level of the structure.
In terms of collapse mechanisms, an increase of the shear strength seems to reduce the level of damage in the reinforced concrete frame. The picture below shows the deformed shapes of a wall for the three levels of shear strength at the same level of top displacement (2.5 cm). Moreover, in the third picture it is shown how a pier has changed its failure mechanism into a rocking collapse.

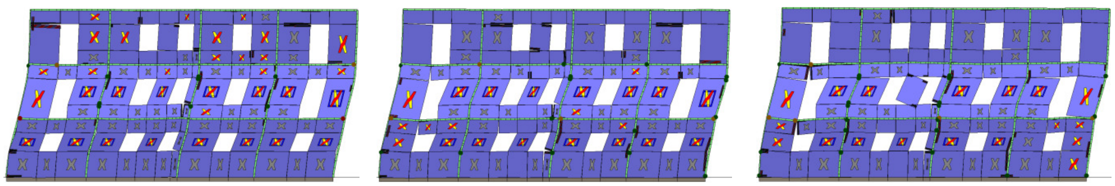

Figure 10. Collapse mechanism of a wall for three different levels of shear strength: τ_0=0.5 daN/cm^2, τ_0=1.0 daN/cm^2 and τ_0=1.5 daN/cm^2

Finally the three pushover curves have been compared in the following picture. The graph demonstrates the high influence of the shear strength in the global strength of the structure. In terms of maximum base shear coefficient ranges from 0.3, which corresponds to the lowest level of shear strength, to 0.5 in the case of τ_0=1.5 daN/cm^2, as shown in the picture below.

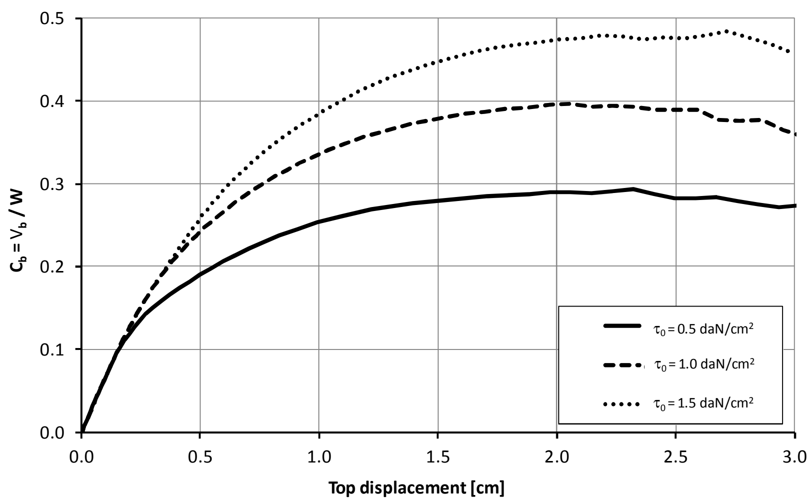

Figure 11. Pushover curves of the structure for three different levels of shear strength: τ_0=0.5 daN/cm^2, τ_0=1.0 daN/cm^2, τ_0=1.5 daN/cm^2

8. CONCLUDING REMARKS

We have considered framed masonry structures as a performing anti-seismic resisting system. We have focused our investigation on cases study in Reggio Calabria identified as "isolated 45". 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.
A numerical model of the case study has been implemented in the code 3DMacro (2009), in which the masonry panels and the confinement frame are modelled explicitly, and their mutual interaction is considered.

The direction load has been investigated both along the main directions (0°, 90°, 180°, 270°) and along the intermediate ones (45°, 135°, 225°, 315°). The influence of the confinement has been shown by means of the deformed shapes of the structure and force diagrams at collapse stage. An original representation of the pushover curves is reported to give a general overview of the strength and the ductility of the structure along each direction.
Finally, the influence of the shear strength on the global response of the structure has been investigated. The results show the high influence of this mechanical parameter in the seismic response of confined masonry structure.

REFERENCES

Caliò, I., Marletta, M. And Pantò, B. (2012). A new discrete element model for the evaluation of the seismic behaviour of unreinforced masonry buildings. *Engineering Structures* **40** (2012) 327–338.

Chen, S..Y, Moon, F.L. and Yi, T. (2008). A macroelement for the nonlinear analysis of in-plane unreinforced masonry piers. *Engineering Structures* 2008;**30**:2242–52.

Moon, F.L., Yi, T., Leon, R.T. and Kahn, L.F. (2006). Recommendations for the seismic evaluation and retrofit of low-rise URM structures. *ASCE J Struct Eng* 2006;**132(5)**:663–72.

Penelis, G. G., (2006). An efficient approach for pushover analysis of unreinforced masonry (URM) structures. J Earthq Eng 2006;**10(3)**:359–79.

Seible, F. and Kingsley, G.R. (1991). Modeling of concrete and masonry structures subjected to seismic loading. *Experimen Numer Meth Earthq Eng*; 1991.

Casolo, S. and Peña, F. (2007). Rigid element model for in-plane dynamics of masonry walls considering hysteretic behaviour and damage. *Earthq Eng Struct Dynam* 2007;**36(8)**:1029–48.

Casolo, S. and Sanjust, C.A. (2009). Seismic analysis and strengthening design of a masonry monument by a rigid body spring model: The ''Maniace Castle'' of Syracuse. *Eng Struct* 2009;**31(7)**:1447–59.

Lourenço, P.B. (2002). Computations on historic masonry structures. *Progr Struct Eng Mater* 2002;**4(3)**:301–19.

Magenes, G. and La Fontana, A. (1998). Simplified nonlinear seismic analysis of masonry buildings. *Proceedings of British masonry society*, **vol. 8**; 1998. p. 190–5.

Kappos, A.J., Penelis, G.G. and Drakopoulos, C.G. (2002). Evaluation of simplified models for lateral load analysis of unreinforced masonry buildings. *J Struct Eng* 2002;**128(7)**:890–7.

D'Asdia, P. and Viskovic, A. (1995). Analyses of a masonry wall subjected to horizontal actions on its plane, employing a non-linear procedure using changing shape finite elements. *Trans Modell Simul* 1995;**10**:519–26 [WIT Press].

Braga, F., Liberatore, D. and Spera, G.A. (1998). Computer program for the seismic analysis of complex masonry buildings. *Computer methods in structural masonry*, **vol. 4** p. 309–16.

Vanin, A. and Foraboschi, P. (2009). Modelling of masonry panels by truss analogy – part 1. *Masonry Int.* 2009;**22(1)**:1–10.

Tomazevic, M. (2006). Earthquake-resistant design of masonry building. *Series on innovation in structures and construction*, **vol. 1**. London: Imperial College Press; 2006.

Caliò, I., Cannizzaro, F., D'Amore, E., Marletta, M. and Pantò, B. (2008). A new discrete-element approach for the assessment of the seismic resistance of mixed masonry and reinforced concrete buildings. *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake* (MERCEA '08). Reggio Calabria, Italy, 8-11 July 2008.

Caliò, I., Marletta, M., Pantò, B., (2008). A discrete element approach for the evaluation of the seismic response of masonry buildings. *Proceedings of 14th World Conference of Earthquake Engineering*, Beijing, China, 12-17October 2008.

Caliò, I., Cannizzaro, F. and Marletta, M. (2010). A discrete element for modeling masonry vaults. *Proceedings of 7th International Conference on Structural Analysis of Historical Constructions*, Shanghai, China,6-8 October 2010.

Magenes, G. and Calvi, G.M. (1997). In plane seismic response of brick masonry walls. *Earthquake Engineering and structural Dynamics*, **Vol. 26**, 1091-1112.

Massonet, C.H. and Save, M. (1980). *Calcolo Plastico a Rottura delle Costruzion*. CLUP, Milano.