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A DISCRETE ELEMENT APPROACH FOR THE EVALUATION OF THE SEISMIC RESPONSE OF MASONRY BUILDINGS

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ABSTRACT:

The evaluation of the seismic response of masonry buildings represents a relevant topic both in the field of the research and in the engineering practice. In the paper a discrete element approach for the evaluation of the seismic behaviour of masonry buildings is presented. The proposed methodology is based on a simple mechanical nonlinear scheme that, with a reduced computational cost with respect to a nonlinear finite element simulation, is able to predict both the in-plane and the out-of-plane nonlinear behavior of a masonry portion and, for assemblage, of a masonry building. The results appear to be suitable for the use of the proposed methodology in order to predict the nonlinear seismic response of masonry buildings by means of nonlinear static analyses.

KEYWORDS: Masonry buildings, numerical methods, discrete element method.

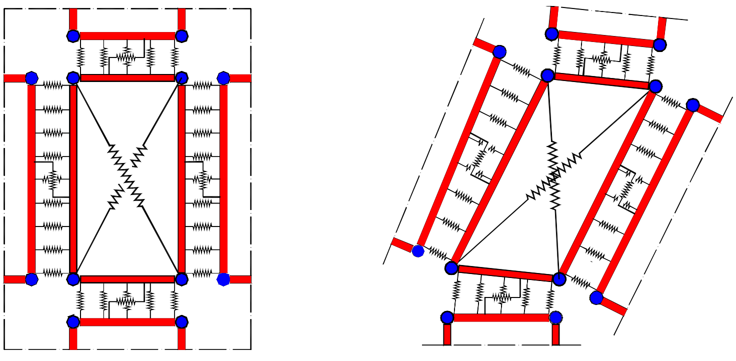
1. INTRODUCTION

The problem of the evaluation of the seismic vulnerability of an existing masonry building represents a subject of high practical relevance but, at the same time, a very difficult task. In order to estimate the seismic vulnerability of an existing building and to ascertain if a structure requires a seismic upgrade, the structural analyst needs simple and efficient numerical tools whose complexity and computational demand must be appropriate for the practical engineering purposes. However, the simulation of the nonlinear dynamic behaviour of a masonry structure still represents a very challenging problem which rigorously requires the use of computationally expensive nonlinear finite element models and, above all, very expertise judgments. For this reason, in recent years, many authors developed simplified methodologies that, with a reduced computation effort with respect to a nonlinear finite element simulation, should be able to simulate the nonlinear seismic behaviour of masonry buildings and to provide numerical results that can be considered sufficiently accurate for engineering practice purposes. An overview of the recent code developments of the state-of-the-art methods for the earthquake resistant design of masonry building is reported in [Tomaževič 1999] where the experimental results are also used in order to justify the analytical approaches.

In this paper a three-dimensional discrete-element, derived by an upgrade of a plane macro-element previously introduced by the same authors [Caliò *et al.* 2005, Pantò 2006], is presented. The basic element, developed for the simulation of the in-plane response, is constituted by an articulated quadrilateral (*panel*) with four rigid edges and four hinged vertices connected by two diagonal nonlinear springs; each of the rigid edges can be connected to other elements by means of discrete distributions of nonlinear springs with limited tension strength (*interfaces*). This plane discrete-element has been applied for the simulation of the nonlinear behaviour of masonry buildings in which the masonry walls are subjected to in-plane forces, without taking into account the out-of-plane response of masonry walls. In order to overcome this significant restriction, the plane macro-element has been upgraded by introducing a third dimension and the needed additional degrees-of-freedom, with the aim to describe the out-of-plane kinematics of masonry walls. This enrichment of the element obviously produced an increased computational effort, due to the larger number of degrees-of-freedom and to the introduction of additional non-linear elements needed for describing the out-of-plane mechanical behavior of the element. Nevertheless, the increased complexity of the model is balanced by the fact that both the in-plane and out-of-plane mechanisms are considered in a unique model. In order to show the potentiality of the proposed approach the well-known Rondelet out-of-plane mechanism are numerically simulated together with an example of the global behaviour of a typical masonry single nave church.

2. THE PLANE MACRO-ELEMENT

The plane discrete-element has been conceived in order to simulate the mechanical behavior of masonry walls when subjected to in-plane loading. It is constituted by a simple mechanical scheme, that must represent a portion of masonry, consisting in an articulated quadrilateral with rigid edges and hinged vertices connected by diagonal springs, which simulates the shear deformability. The quadrilateral (*panel*) can interact with other elements along each edge by means of discrete distributions of nonlinear springs with limited tension strength (*interfaces*), as it is shown in figure 1. Each interface includes springs orthogonal to the rigid edges which it connects (*transversal springs*) and a spring parallel to these edges (*sliding spring*). The whole set of nonlinear springs (*NLinks*) allows an effective simulation of the main in-plane collapse mechanisms of the masonry, figures 2, as reported in figures 3.



(a) (b)

Figure 1: The plane discrete-element: (a) undeformed configuration; (b) deformed configuration.

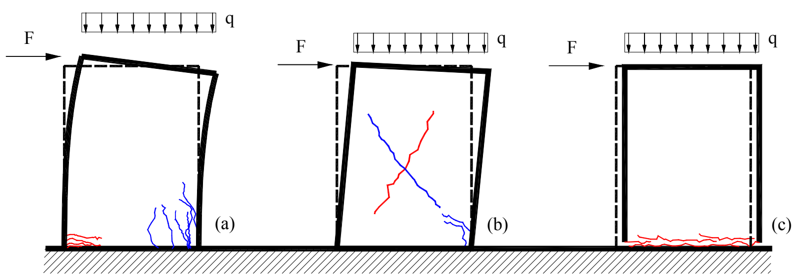


Figure 2: Main in-plane failure mechanisms of a masonry panel:
(a) flexural failure; (b) shear-diagonal failure; (c) shear-sliding failure.

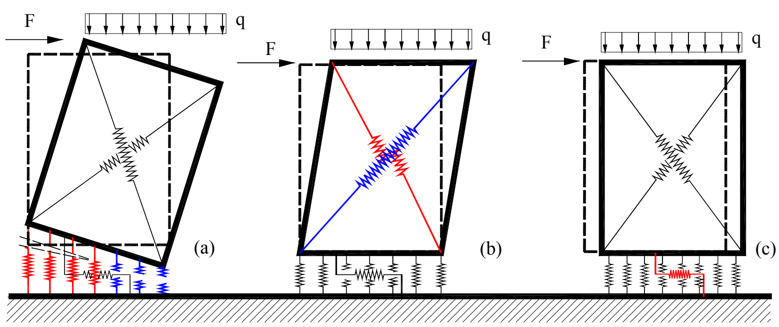


Figure 3: Simulation of the main in-plane failure mechanisms of a masonry panel by means of the proposed macro-element: (a) flexural failure; (b) shear-diagonal failure; (c) shear-sliding failure.

An original aspect of the proposed plane macro-model is that the panel can interact with other panels or elements along its whole perimeter. This is a great advantage, since it allows an accurate modeling of the floor spandrels and permits the realization of a mesh of macro-elements that, in some cases, may be useful for a more accurate simulation of the collapse mechanisms. The basic macro-element possesses three degrees-of-freedom associated to the in-plane kinematics and an additional degree-of-freedom associated to the articulated movement of the panel. Therefore the kinematics of a structure with N panels is completely described by $4N$ Lagrangian parameters. The in-plane behaviour of a masonry wall can be represented through a mesh of macro-elements and an assemblage of masonry walls leads to the simulation of an entire masonry building. However this simplified approach, based on the plane element, is not able to simulate the out-of-plane behavior of the masonry walls. In order to overcome this limit the plane element has been upgraded by introducing a third dimension.

3. THE 3D MACRO-ELEMENT

The three-dimensional macro-element represents the natural upgrade of the plane macro-element described in the previous paragraph. In particular three additional degrees-of-freedom have been considered for the description of the out-of-plane kinematics and further nonlinear springs have been introduced in the interfaces in order to account for the three-dimensional mechanical behavior (figure 4).

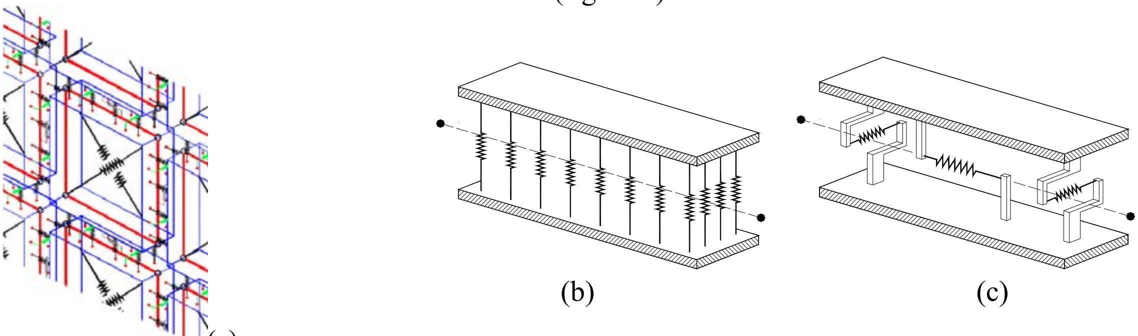

(a) (b) (c)

Figure 4: (a) The 3D macro-element; the 3D interface: (b) longitudinal springs (c) sliding springs.

The kinematics of the spatial macro-element is therefore governed by 7 degrees-of-freedom able to describe both the rigid body motions and the shear deformability of the base element. The 3D-interface possesses m rows of n *longitudinal* (*i.e.* perpendicular to the planes of the interface) NLinks each. Therefore each of the planes of the interface is subdivided, similarly to what is done in classical fibre models, in $m \times n$ sub-areas (figure 4b). Each longitudinal spring of the 3D interface represents a column of masonry with base area equal to that of the sub-area which pertains to it. The number of rows m and the number of springs in each row n must be selected according to the desired level of detail of the nonlinear response. It is apparent that, in order to consider the flexural out-of-plane behavior of the panel, at least two rows of transversal springs must be provided. The span distance between the rows is assigned by enforcing an out-of-plane flexural equivalence between the masonry wall and the discrete model. It is trivial to observe that the proposed model allows a simply approach to the problem of the nonlinear biaxial bending, including the influence of the axial force. Beside the transversal springs and the in-plane sliding spring, similar to that in the plane interface, the 3D-interface possesses also further *sliding springs*. These are required to control the relative displacement of the panels perpendicular to their plane. To this purpose, two out-of-plane sliding NLinks, perpendicular to the plane of the panel and contained in the plane of the interface, have been provided (figure 4c). These NLinks control the out-of-plane sliding mechanisms of the panels and the torsion of the panels about the axis perpendicular to the plane of the interface.

3.1. Interaction between intersecting walls

The interaction between walls in the corners or, more generally, of intersections requires *ad hoc* modeling. Within a simplified approach, the detachment of a wall in the case of a corner intersection can occur according to two fundamental mechanisms: the sliding mechanism (figure 5a) and the flexural mechanism due to the excessive tension force (figure 5b).

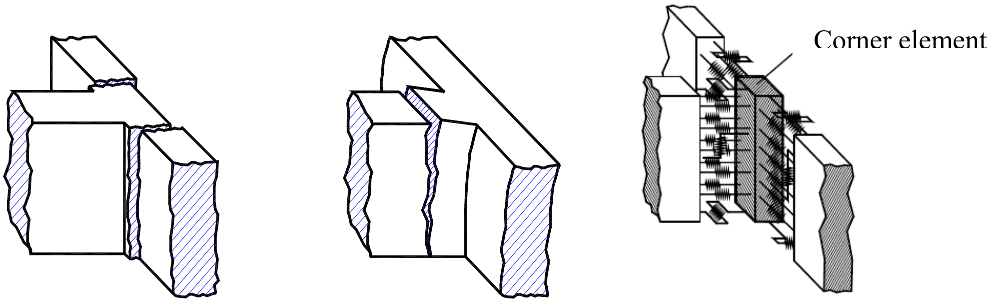


Figure 5: Collapse mechanisms in the intersections with masonry walls: (a) sliding collapse; (b) flexural collapse; (c) the connection between three macro-elements by means of a corner element.

This failure mechanisms can be effectively simulated by means of the introduction of a specific *corner element*. This element actually represents a spatial constraint imposed to a group of 3D interfaces which interact with the panels of the intersecting walls. As an example, figure 5c reports the model of the intersection between three panels each belonging to a different wall.

4. MECHANICAL CALIBRATION OF THE MODEL

The proposed macro-element is based on an equivalent mechanical scheme where the constitutive behaviour is lumped in a discrete number of nonlinear NLinks. This allows the use of mono-axial nonlinear constitutive laws instead of multi-axial ones, with a significant conceptual and computational simplification. The mechanical properties of the nonlinear springs are calibrated by imposing an equivalence between the macro-model and an appropriate corresponding homogeneous continuum model. For the purpose of the calibration, both the models are subjected to suitable simple tensional states in order to analyse separately each aspect of the masonry behavior: flexural, shearing and sliding. The effectiveness of such an approach has been tested by comparing the obtained results with those of more refined nonlinear finite element methods [Pantò 2006, T.R.E.M.A. Project 2007]. In the following the calibration criteria adopted for each group of NLink are briefly illustrated.

4.1. Interface Elements

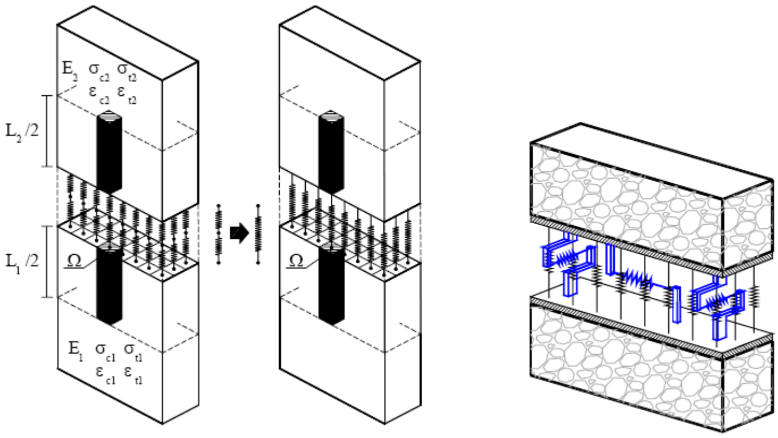
4.1.1 Transversal spring of the interface

The 3D-interface is constituted by a discrete set of nonlinear transversal springs, each of them plays the role of modeling a masonry strip according to a simple influence volume criterion, as shown in figure 6a.

The calibration criteria for the transversal springs of the interface depends on the adopted constitutive law for the axial-flexural behavior of masonry. Herein a procedure based on the hypothesis of elastoplastic orthotropic behavior with limited displacement in tension and compression is briefly described. The considered constitutive law is characterised, along each principal direction, by the elastic modulus (E), the compressive and tensile strength (σ_c, σ_t) and the limit compressive and tensile strain (ε_c, ε_t); when the limit strain is achieved, a fragile rupture is experienced so the stress in the considered direction is suddenly reduced to zero.

For what concerns the post-failure behavior, two options are available: a *crushing* behavior, providing that the material after a compressive failure is not able to carry further compressive and tensile loads, and a *cracking* behavior, providing that the material after a tensile failure retains the capacity to carry compressive loads.

A two-step procedure is adopted for the calibration of the interface springs: in the first step the deformability of each panel is simulated by an equivalent spring; then the two nonlinear springs in series, relative to the two adjacent panels, are combined in a single interface spring. The procedure is formally analogue to that reported in [Caliò *et al.* 2005] for the plane macro-element. In order to accurately simulate the out-of-plane behavior, the value of the NLink row-span must be set according to a stiffness equivalence criterion between the discrete-element and the continuum model.



(a) (b)

Figure 6: Calibration of the springs of the interfaces: (a) transversal springs; (b) sliding springs.

4.1.2 Sliding springs of the interface

The in-plane and out-of-plane sliding mechanisms of the panels are governed by three nonlinear springs which are calibrated according to a Mohr-Coulomb law (figure 4c). The single spring laying in the longitudinal direction plays the role of simulating the sliding mechanism in the plane of the wall, while the other two rule the sliding of the element with reference to the out of the plane behaviour (figure 6b). The mechanical properties of the springs can be derived directly from masonry properties by considering the influence area of each NLink.

4.2.Panel Element: Diagonal Springs

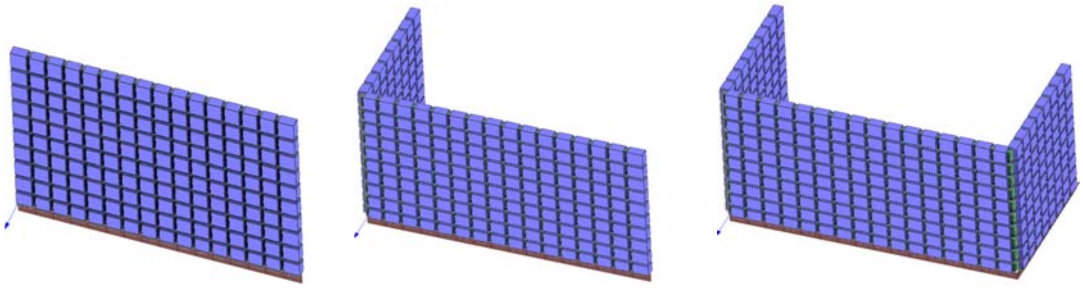
The diagonal springs of the panels must simulate the shear deformability and control the shear diagonal failure mechanism; the corresponding ultimate load can be evaluated according to a specific criterion of the Mohr-Coulomb or Turnšek-Cacovič type [Turnšek & Cacovič 1971]. The mechanical characteristics of the springs are determined by enforcing an equivalence in terms of displacement in the case of pure-shear loading between the panel, considered as an elastic continuum, and the discrete model [Caliò *et al.* 2005, Pantò 2006].

5. NUMERICAL APPLICATIONS

In the following the results of some numerical applications, performed by using the computer code *3DMacro* [Caliò *et al.* 2008], are reported in order to show the effectiveness of the proposed modeling approach. The first series of application is relative to simple masonry panels subjected to different restraint conditions and loaded in order to activate the well-known out-of-plane mechanisms of collapse identified by Rondelet [Rondelet 1834]. Another application is relative to the results of pushover analyses of a single nave church subjected to in-plane and out-of-plane loading.

5.1. Simulation of Rondelet's collapse mechanisms

The present paragraph refers to the results of a masonry wall with dimensions 5 m × 8 m and thickness 25 cm subjected to a uniform distribution of out-of-plane loading. The panel is fully restraint at the base and subjected to various restraint conditions at the vertical edges due to the presence of orthogonal walls. Namely three cases, reported in figure 7, have been analysed: the first case considers the wall constrained only on the base, the second case is relative to the wall constrained in one vertical edge, the third case is representative of the wall constrained by two orthogonal wall at its both vertical edges. The obtained collapse mechanisms can be qualitatively compared to those described by Rondelet [Rondelet 1834] and reproduced by many authors both experimentally [Giuffrè 1993, Lagomarsino *et al.* 2002a, Lagomarsino *et al.* 2002b] and numerically [Orduña & Lourenço 2005, Restrepo-Vélez & Magenes 2005]. The masonry panel has been schematized by means of a mesh of macro-elements of 50 cm × 50 cm. The main mechanical properties of the considered masonry are reported in table 1.



(a) (b) (c)

Figure 7: The cases considered for the simulation of the Rondelet out-of-plane mechanisms.

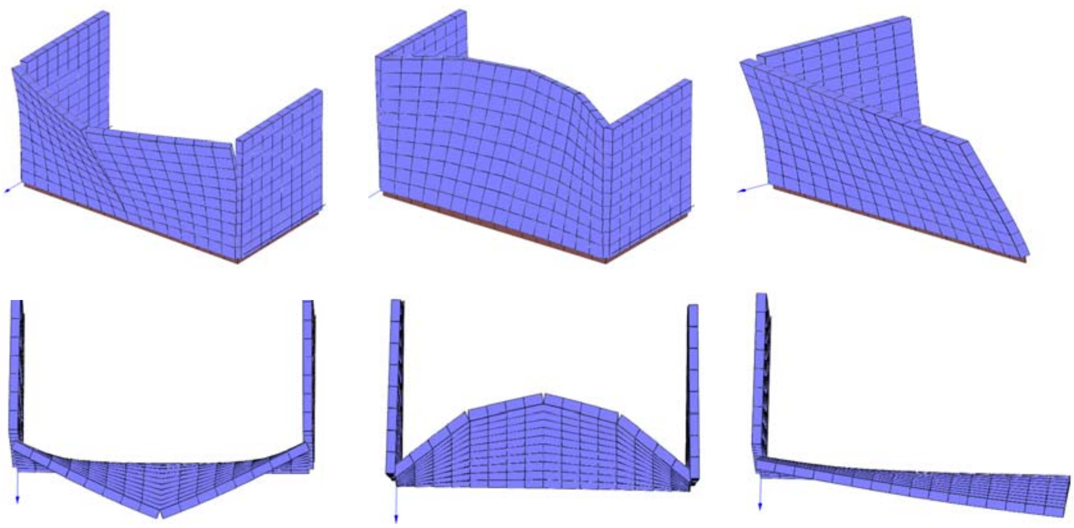
Table 1: Main material properties.

Bending			Shear			Sliding	
Young's modulus E (MPa)	compressive strength σ_c (MPa)	tensile strength σ_t (MPa)	transversal modulus G (MPa)	shear strength τ_k (MPa)	friction angle φ	cohesion c (MPa)	friction angle φ
2500	5.00	0.05	500	0.15	0.15	0.01	0.15

Static nonlinear analyses have been performed by applying the gravity loads and an horizontal mass-proportional load distribution. In figures 8, for each considered case, the collapse mechanisms obtained by pushover analyses are reported. It is apparent that the classical Rondelet's mechanisms are clearly reproduced by the considered models.

5.2. Simulation of the response of a masonry church

In the following the results of the numerical analyses relative to a single nave rectangular-plan church are presented. This structural typology, studied by several researchers [Lagomarsino 2002a, Lagomarsino 2002b] is greatly vulnerable with respect to the out-of-plane mechanisms. The considered case-study is the St. Michael church built in the town of Noto, near Syracuse (Italy), in 1713 (figure 9). The properties that has been assumed for the materials are reported in table 2.



(a) (b) (c)

Figure 8: Collapse mechanisms obtained for the three considered cases.

Table 2. Main material properties.

Façade				Cantonals				Lateral Walls			
E (MPa)	ν	σ_c (MPa)	w (kN/m^3)	E (MPa)	ν	σ_c (MPa)	w (kN/m^3)	E (MPa)	ν	σ_c (MPa)	w (kN/m^3)
1200	0.2	1.8	17.0	1400	0.2	2.0	18.0	900	0.2	1.2	15.5
Triumphal Arch				Lateral Arches				Apses			
E (MPa)	ν	σ_c (MPa)	w (kN/m^3)	E (MPa)	ν	σ_c (MPa)	w (kN/m^3)	E (MPa)	ν	σ_c (MPa)	w (kN/m^3)
1000	0.2	1.4	18.0	1000	0.2	1.4	16.5	900	0.2	1.2	15.5

In figure 10 the collapse mechanisms of the church subjected to mass-proportional distributions of horizontal loads acting along the longitudinal and the transverse direction are reported. The corresponding pushover curves, in terms of the base-shear coefficient as a function of the displacement of a control point located at the top of the façade, are shown in figure 11.

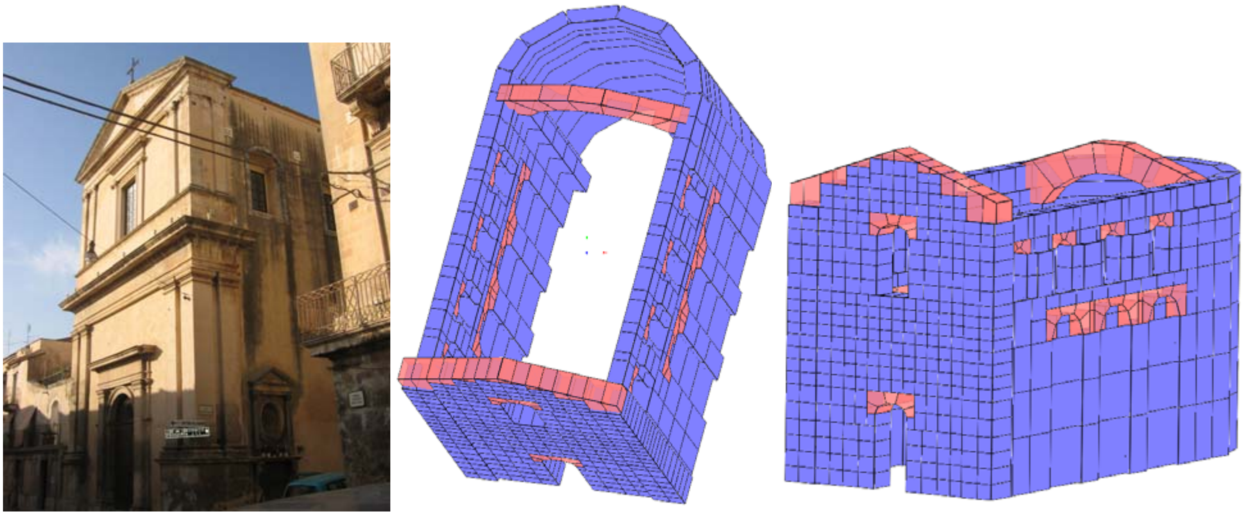
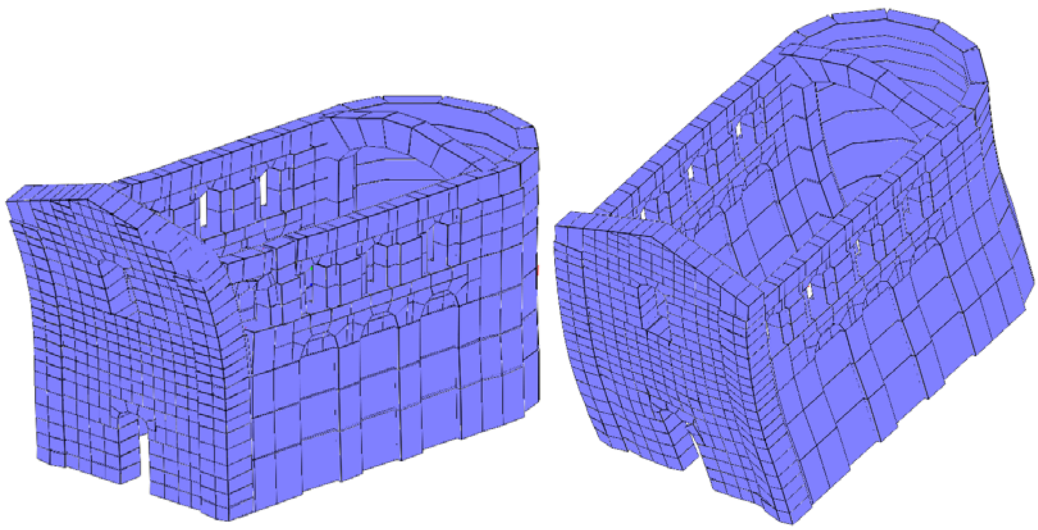
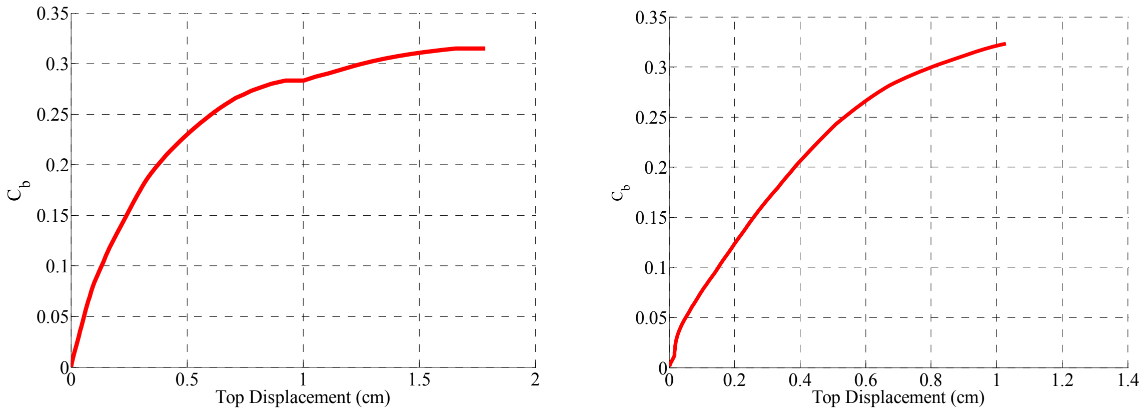


Figure 9: The façade of the St. Michael church in Noto and two views of the model of the church implemented in the software *3DMacro* based on the proposed approach.



(a) (b)

Figure 10: Collapse mechanisms for mass-proportional loading:
(a) longitudinal direction; (b) transverse direction.



(a) (b)

Figure 11: Capacity curves obtained from pushover analyses:
(a) longitudinal direction; (b) transverse direction.

5. CONCLUSIONS

In the paper a new discrete element approach for the evaluation of the seismic behaviour of masonry buildings is presented. The proposed methodology is based on a simple mechanical non-linear scheme that, with a reduced computational cost with respect to a nonlinear finite element simulation, is able to predict both the in-plane and the out-of-plane nonlinear behavior of masonry building. Some relevant applications have been reported, showing the capability of the proposed approach as an advanced-but-simple tool for the estimation of the seismic vulnerability of masonry structures, also in the cases where the out-of plane mechanisms cannot be ignored.

REFERENCES

Caliò, I., Marletta, M., Pantò, B. (2005), A simplified model for the evaluation of the seismic behaviour of masonry buildings. *Tenth International Conference on Civil, Structural and Environmental Engineering Computing*, August 30 – September 2, Rome, Italy.

Caliò, I., Marletta, M., Pantò, B., Cannizzaro F. (2008), The *3DMacro* computer code: http://www.3dmacro.it.

Giuffè, A. (editor) (1993), Sicurezza e conservazione dei centri storici – Il caso Ortigia, Ed. Laterza (In Italian).

Lagomarsino, S., Podestà, S., Resemini, S., Curti, E., Parodi, S. (2002a), Mechanical models for the seismic vulnerability assessment of churches. In: C. Modena, P.B. Lourenço, P. Roca (editors), *Structural Analysis of Historical Constructions, Proc. of IV Intl. Seminar SAHC*, Padova, Italy, A.A. Balkema, London (UK), ISBN 04 1536 379 **9**:2, 1091-1101.

Lagomarsino, S., Podestà, S., Resemini, S. (2002b), Seismic response of historical churches, *Proc. 12th European Conference on Earthquake Engineering, London*, September 2002, 10 pp. (On CD-ROM, ISSN 0080440495).

T.R.E.M.A. Project (2007), Tecnologie per la Riduzione degli Effetti sismici sui Manufatti Architettonici in muratura ed c.a. Italian National Research Project Report: http://www.unibas.it (In Italian).

Tomaževič, M. (1999), Earthquake-resistant design of masonry buildings, Imperial College Press, London (UK).

Turnšek, V., Cacovič, F. (1971), Some experimental results on the strength of brick masonry walls. *Proc. of the 2nd Intern. Brick Masonry Conference*, Stoke-on-Trent, 149-156.

Orduña, A., Lourenço, P.B. (2005), Three-dimensional limit analysis of rigid blocks assemblages. Part II: Load-path following solution procedure and validation, International Journal of Solids and Structures, **43**, 5161-5180.

Pantò, B. (2006), La modellazione sismica degli edifici in muratura: un approccio innovativo basato su un macro-elemento spaziale. Doctoral dissertation, Department of Civil and Environmental Engineering, University of Catania, Italy (In Italian).

Rondelet, J. (1834), Trattato teorico-pratico dell’arte di edificare, Libro IX, Teoria delle Costruzioni, F.lli Negretti, Mantova (Italy) (In Italian).