A new discrete-element approach for the assessment of the seismic resistance of composite reinforced concrete-masonry buildings

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Abstract. In the present study a new discrete-element approach for the evaluation of the seismic resistance of composite reinforced concrete-masonry structures is presented. In the proposed model, unreinforced masonry panels are modelled by means of two-dimensional discrete-elements, conceived by the authors for modelling masonry structures, whereas the reinforced concrete elements are modelled by lumped plasticity elements interacting with the masonry panels through nonlinear interface elements. The proposed procedure was adopted for the assessment of the seismic response of a case study confined-masonry building which was conceived to be a typical representative of a wide class of residential buildings designed to the requirements of the 1909 issue of the Italian seismic code and widely adopted in the aftermath of the 1908 earthquake for the reconstruction of the cities of Messina and Reggio Calabria.

Keywords: Macro-element, composite reinforced concrete-masonry building, framed-masonry, Messina 1908 earthquake.

INTRODUCTION

Composite reinforced concrete-masonry building structures, so defining structural systems where both reinforced concrete elements and unreinforced masonry panels participate to structural strength and stiffness, represent a relevant percentage of the existing building particularly in Italy. These structures are often difficult to be classified because in many cases they represent non-engineered structures built before the emanation of seismic codes and/or they are the by-product of partial modification of pre-existing masonry buildings. Engineered composite reinforced concrete-masonry was introduced in Italy after the 1908 Messina and Reggio Calabria earthquake by the 1909 seismic code. The structural system introduced by this code, named framed-masonry became the most widespread seismic-resistant structural system for residential buildings during the first forty years of the last century [1]. The analytical modelling of the nonlinear behaviour of composite reinforced concrete-masonry structures can be conducted by detailed nonlinear finite element analyses or 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 for 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, on 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 where proposed especially for simple
masonry buildings. Generally these simplified approaches adopt equivalent nonlinear
beam elements, or more complex mechanical sub-assemblages, for the analysis of
unreinforced masonry panels. With reference mixed structures 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 framed and confined masonry systems, play a relevant role to the seismic response
of the structure.

In the present study a new analytical approach for the evaluation of the seismic
resistance of composite reinforced concrete-masonry structures is presented. In the
proposed model, unreinforced masonry panels are modelled by two-dimensional
macro-elements [2, 3, 4], whereas the reinforced concrete elements are modelled by
lumped plasticity elements interacting with the masonry through nonlinear interface
elements. The results of the analysis conducted on the case study of a confined
masonry structure proposed by D’Amore [5] representative of a wide class of
residential structures built in Italy after the 1908 Messina and Reggio Calabria
earthquake are presented.

THE PROPOSED MODEL

The proposed simplified approach is based on a discrete-element originally conceived
for the simulation of nonlinear response of masonry buildings, [2, 3, 4]. The basic
element, i.e. the panel, exhibits a simple and understandable mechanical scheme,
represented in figure 1. It is an articulated quadrilateral with four rigid edges
connected by hinges whose deformability is controlled by two nonlinear diagonal
springs. Each edge of the panel can interact with other elements or external restraints
by means of a discrete distribution of nonlinear springs (interface). Each interface
consists of n springs perpendicular to the edges of the elements which connects
(transversal springs) and a single nonlinear spring which controls the motion in the
direction parallel to the edges which connects (sliding spring).

The proposed mechanical model, in spite of its simplicity, is able to effectively model
the main failure mechanisms of a portion of masonry subjected to in-plane, vertical
and horizontal, action.
The mechanical model of an entire plane masonry wall can be regarded as an assemblage of such panels connected by interfaces. The accuracy of the model can be improved by adopting a refined mesh in order to better describe the kinematics and the failure mechanisms of the masonry wall.

The proposed macro-element is based on a mechanical equivalent scheme where deformability is lumped in the nonlinear springs or \textit{NLinks} (Nonlinear Links). This choice leads to great simplifications, both conceptual and computational, since it is based on uniaxial constitutive laws. The transversal springs have the role of simulating the flexural behaviour of the masonry portion represented by the interface, in which each spring is calibrated according to the corresponding influence volume similarly to the well known fibre models adopted for the simulation of the nonlinear behaviour of reinforced concrete structures. The nonlinear force-displacement relationship of the transversal springs depends on the constitutive law chosen for the axial-flexural behaviour of masonry. Namely in the case of compressive rupture, a \textit{crushing} behaviour has been considered, i.e. the material loses the capacity to resist to further compressive and tensile loads; in the case of tensile failure a \textit{cracking} behaviour is adopted, i.e. the tensile resistance drops to zero when the rupture tensile tension is reached, however the material can still resist to compressive loads.

The simple procedure which permits to transfer the masonry properties to the interface springs is based on the geometry of the portion of masonry which is intended to represent and on the deformability and resistance parameters of the masonry along the main directions. The procedure enforces an equivalence between a single transversal spring and its masonry influence volume. In the considered case an elastic-plastic law with different behaviour in traction and compression has been adopted according to the procedures reported in [3].

The sliding spring has the role of simulating the failure mechanisms associated to the sliding of the masonry portion. This mechanism is ruled by a rigid-plastic Mohr-Coulomb law.
The diagonal springs simulate the in-plane shear deformability and the diagonal cracking failure mechanism. The ultimate load relative to this mechanism can be evaluated according to a specific Mohr-Coulomb law or according to the Turnsek-Cacovic criterion [6], or with reference to other criteria proposed in the literature. In the analyses presented in the following an elastic-plastic law with limited deformability according to the Mohr-Coulomb criterion has been considered. For mixed masonry-reinforced concrete structures, beam/column elements are included in the model as uniaxial finite elements with lumped plasticity. Each of these frame elements interacts with the adjacent masonry for its whole length by means of the interfaces. The proposed modelling approach allows the simulation of the confinement effect in framed masonry.

THE CASE-STUDY

The case-study building, described in full detail by D’Amore [5] was conceived on the basis of detailed investigation on eleven two-storeys framed-masonry residential buildings, built in Reggio Calabria (Italy) after the 1908 Messina Earthquake. The typical floor plan of the case-study building, shown in figure 2, is rectangular with dimensions of 22.84 m and 11.78 m alongside the longitudinal and transversal directions respectively. The case-study structure represents a wide class of confined masonry residential buildings widely adopted in the cities of Messina and Reggio Calabria after the 1908 earthquake.

THE MODEL OF THE BUILDING

The three-dimensional model of the building has been realised with the computer software 3DMacro [7] developed by a research group of the University of Catania. This computer code allows the implementation of the approach previously described for composite reinforced concrete-masonry buildings. In order to investigate different classes of structures, two different models have been considered: (1) the SH model, in
which solid brick masonry have been considered at the first level of the building and hollow brick masonry has been considered at the second level; (2) the SS model, where solid brick masonry has been considered at both levels.

The mechanical characteristics of the masonry considered in the two models are reported in Table 1; the friction angle $\varphi$ has been set to 0.3.

| TABLE 1. Mechanical characteristics of the masonry considered in the numerical analyses. |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| | Young's modulus, $E$ MPa | Transv. elastic mod., $G$ MPa | Compr. strgth., $\sigma_c$ MPa | Tens. strgth., $\sigma_t$ MPa | Shear strgth., $\tau_o$ MPa | Ultim. shear strain, $\gamma_u$ | Density Kg/m$^3$ |
| Solid bricks | 2000 | 350 | 3 | 0.2 | 0.1 | 0.006 | 1800 |
| Hollow bricks | 900 | 150 | 1.5 | 0.1 | 0.05 | 0.003 | 810 |

In the following table 2 the mechanical characteristics of concrete and steel rebars are summarised.

| TABLE 2. Mechanical characteristics of concrete and steel considered in the numerical analyses. |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| | Young's modulus of concrete, $E$ MPa | Cubic resistance of concrete, $R_{ck}$ MPa | Ultim. strain of concrete, $\varepsilon_{cu}$ | Yield. strgth of steel, $f_y$ kN/cm$^2$ | Ultimate strain of steel, $\varepsilon_{yu}$ |
| | 20000 | 1.2 | 0.003 | 24 | 0.01 |

**RESULTS OF NUMERICAL ANALYSES**

For the estimation of the seismic resistance of the considered models, nonlinear static (pushover) analyses have been performed; two load distributions have been considered: (1) a mass-proportional distribution and (2) an inverse-triangular distribution.

In order to investigate the resistance capacity as a function of the earthquake direction, many loading directions and verses have been considered. Namely the principal directions of the building (X and Y) and the directions which form a 45° angle with respect to the principal directions in both verses have been taken into account.

In the following, the results obtained for the two considered models loaded along the principal directions are reported together with some images of the collapse mechanisms. The results are expressed as storey shears as a function of the relative inter-storey drifts.

**The SH Model**

In figures 3 and 4 the results of the analyses obtained for the SH model loaded in its two principal directions according to the two load distributions considered are reported. Each plot reports the storey shear, normalized respect to the building weight $W=6662$ kN, as a function of the corresponding relative inter-storey drift $\Delta u_i/h_i$. 


The obtained results highlight that the weaker direction is $Y$ and that the collapse mechanisms is localized at the second level.

**The SS Model**
In figures 5 and 6 the results relative to the SS model are presented. The total weight of the SS model has been estimated equal to $W=7363$ kN.

From the observation of the results, it is apparent that in this case the collapse mechanism is concentrated at the first level for both the load distributions considered.

**Three-Dimensional Dominia**

The pushover curves obtained for different loading directions and verses can be synthetically represented by means of the three-dimensional capacity dominia [8].
In such a 3D plot (figure 7) each push-over curve is represented along the input direction, in a plane perpendicular to the XY plane, so that the base shear coefficient is reported in the Z axis. Such a representation allows to identify the stronger and weaker directions of the building. Furthermore the resistance levels is represented in a colour scale. The ductility of the structure in all the investigated directions is also readable in the graph. The directions that has not been investigated has been linearly interpolated.

ACKNOWLEDGEMENTS

This work has been financially supported by the Executive Project 2005-2008 of the ReLUIS consortium (Rete dei Laboratori Universitari di Ingegneria Sismica), research line 1: “Evaluation and reduction of the vulnerability of masonry buildings”, coordinated by Professors S. Lagomarsino and G. Magenes.

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