Seismic assessment of the historical mixed masonry-reinforced concrete Government Palace in La Spezia

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SEISMIC ASSESSMENT OF THE HISTORICAL MIXED MASONRY-REINFORCED CONCRETE GOVERNMENT PALACE IN LA SPEZIA

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Abstract. Government Palace in La Spezia is a six-story mixed masonry-reinforced concrete construction, built in 1926 and today seat of the Prefecture of La Spezia city (Italy). In the paper, after the description of the case study, the attention is focused on the evaluation of the seismic performance of the building. The seismic assessment is performed by means of a numerical hybrid approach: the reinforced concrete frame is modelled according to a concentrated plasticity beam-column element, while the masonry infill is simulated by a plane macro-element which interacts with the surrounding frame, by means of discrete distribution of nonlinear links. The 3D-model has been implemented in the computer code 3DMacro that allows the modelling of both unreinforced masonry and mixed reinforced concrete-masonry structures. According to the performed numerical simulation the building, although not designed to resist to seismic actions, does not possess a high vulnerability with reference to the expected earthquake in the construction site.
1 INTRODUCTION

The adoption of mixed masonry-reinforced concrete as structural seismo-resistant system has been introduced for the first time in the aftermath of Messina-Reggio Calabria earthquake of 1908. The coupling of frames and masonry panels has represented a low-cost constructive technique with a significant quality of performance in occasion of seismic events. This constructive system spread from the southern of Italy, first in the rest of the country and then abroad (e.g. South America and Africa).

The simulation of the nonlinear behavior of this kind of structures is an open problem in which many researchers are involved. Although there are many efficient numerical approaches for modeling both reinforced concrete structures and unreinforced masonry structures, the modeling of mixed masonry-reinforced concrete structure still represents a crucial question in engineering practice. These mixed structural systems are governed by the interaction between frame and infill wall. The highly nonlinear masonry-infill response and the ever-changing contact condition along the frame-infill interfaces make the simulation of the nonlinear behaviour of an entire infilled frame building a challenging problem. Currently, two main strategies can be adopted to numerically model this structural typology, that is nonlinear finite element modeling and simplified approaches.

Detailed finite element analyses generally require constitutive laws for reinforced concrete elements, usually modelled with distributed 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, 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 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 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 last decades many researchers introduced some proposals to consider the effect of infills by means of strut models [1]. The latter approaches are generally based on simplified hypotheses that move from empirical and approximate assumptions. Besides, these models cannot be easily used in presence of openings on the infill.

In the present paper, a simplified analytical approach based on a macro-element, previously presented in the literature [3], is applied to evaluate the seismic performance of mixed masonry-reinforced concrete 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 considered approach, that is able to take into account the interaction between the frames and the infills, is briefly presented. Then it is applied to the seismic vulnerability assessment of a large-scale historical building, currently exploited with strategic purposes. The case study is duly described and the performed numerical applications are re-
ported. Some significant aspects (in particular the strength and ductility properties of the building) are deepened.

2 MIXED MASONRY-REINFORCED CONCRETE STRUCTURES

The first examples of confined mixed masonry-wood structures go back to the nineteenth century and are represented by the so called ‘case baraccate’ (shack houses). Some examples can be found around the area of the Strait of Messina in which the frames are wooden beams enclosed in the masonry walls.

Then, after the catastrophic event of the 1908 earthquake in that area, a new seismic code was introduced in 1909. According to this code the use of mixed masonry-reinforced concrete structures was imposed in seismic area and the constructive technique was standardized. According to the code dispositions the walls have to be raised for first and then they have to be used as formworks for weak reinforced concrete frames, figure 1. The main advantages of this procedure are: the minor cost in terms of formworks and their manufacture, and on the other hand the high seismic performance. This latter aspect is due to the capability of orienting the masonry behavior towards the shear collapse that is the most resistant and dissipative of the failure mechanisms. Another crucial advantage of the introduction of the frames is represented by the quality of the connections among the walls that is left to the columns, thus ensuring a correct interaction between orthogonal walls and preventing out-of-plane collapses.

![Figure 1: Constructive phases of confined masonry buildings typical of the reconstruction of Messina and Reggio Calabria after the 1908 earthquake.](image)

The adoption of this structural typology spread then on a large part of Italy, and is currently adopted in many countries, especially South America and North Africa.

3 CASE STUDY: GOVERNMENT PALACE IN LA SPEZIA (ITALY)

The Government Palace of La Spezia (Italy) was built in 1926 according to the project of the architect Franco Oliva. The building, which occupies an entire block, is constituted by six floor levels with a total height of 25.8 m and a rectangular base of 37.3 x 44.5 m. It is a mixed masonry-reinforced concrete structure although it has not been designed according to the 1909 seismic code. The ground and first floors are weakened by two asymmetrical porticoes on North and Eastern sides with a height of about 8 m, figure 2.

Two small courtyards of about 60 m² each are inside the building, starting from 2nd level (western side) to 3rd level (eastern side).
The Palace has been classified as strategic, in the emergency plan of the city, due to its destinations (Prefecture, Civil Protection, Province offices). It is also covered by specific historic regulation requirements, for the precious architectural features of the building, which strongly influence the choices in case of structural retrofitting. The façades are typical of the beginning of 19th century. The presence of statues, a double-height hall with coffered ceiling and valuable stained glass windows constitute elements characterizing the construction style.
The seismic resistance of the mixed masonry-reinforced concrete structure takes a great advantage from the interaction of the external frame with masonry infills. The building is quite regular and no significant structural variations occurred over the years.

Figure 4: Examples of original drawings of r.c. structures

Figure 5: Examples of vulnerable non structural elements

A low destructive testing campaign, using strategy in [10] together with the availability of several original drawings (figure 4), have allowed the estimation of the main mechanical parameters and the definition of the main structural details needed for the definition of a 3D-model of the building. Compressive tests on concrete cylindrical specimen established a cy-
lindrical compression strength of about $f_{cc}=15$ Mpa. Penetrometric PNT-G tests on mortar joints together with compressive tests on stone blocks permitted to adopt a compression strength for masonry of about $f_{m}=1.5$ Mpa as in [9]. Non destructive tests, as sclerometric measurements, infrared image monitoring and electromagnetic analysis to detect steel bars in r.c. elements, have been performed to individuate geometry and further information on mechanical properties of structural materials, as in [11].

4 THE ADOPTED NUMERICAL MODEL

The proposed model is base on a discrete element to model masonry panels, recently introduced by Caliò et al. [2], and initially conceived for the simulation of the nonlinear in-plane behaviour of unreinforced masonry walls. This element is characterized by a simple mechanical scheme, Figure 6, constituted by an articulated quadrangle with rigid edges connected by four hinges and two nonlinear diagonal springs. Each side of the quadrangle can interact with other elements or supports by means of a discrete distribution of the nonlinear springs, denoted as interface. Each interface is constituted by $n$ nonlinear orthogonal springs, perpendicular to the panel side, and an additional longitudinal spring, parallel to the panel edge.

![Figure 6: The basic macro-element for masonry: (a) undeformed configuration; (b) deformed configuration.](image)

The mechanical scheme thus conceived, providing a correct calibration procedure of the nonlinear links, is kinematically able to simulate the three main in-plane collapse failure modes of a masonry panel, Figure 7.

![Figure 7: Main in-plane collapse failure modes of a masonry panel: (a) flexural mechanism; (b) shear-diagonal mechanism; (c) shear-sliding mechanism.](image)

These well-known collapse mechanisms, that are the *flexural failure*, the *diagonal shear failure* and *sliding shear failure*, are approximately represented in Figure 7 where the typical crack patterns, together with the qualitative kinematics of masonry portion, are also sketched according to a brick walls. Figure 8 shows how the proposed macro-element allows a simple
and realistic mechanical simulation of the corresponding failure mechanisms of a masonry wall in its own plane

![Figure 8: Simulation of the main in-plane collapse failure modes of a masonry panel: (a) flexural mechanism; (b) shear-diagonal mechanism; (c) shear-sliding mechanism.](image)

Each discrete element exhibits three degrees-of-freedom, associated to the in-plane rigid-body motion, plus a further degree-of-freedom, needed for the description of the shear deformability. The deformations of the interfaces are associated to the relative motion between corresponding panels; therefore no further Lagrangian parameters have to be introduced in order to describe their kinematics.

The main simplified models for the simulation of masonry walls in the literature are the so called ‘equivalent frame models’ in which a masonry panel is simulated with an equivalent beam that can interact with other elements at its ends. On the contrary the adopted model has the advantage to allow the interaction of the element along the four sides, thus allowing the possibility of being used according to different mesh discretization and to account for the interaction between the reinforced concrete frame and the masonry infilled also in presence of openings, Figure 9. The considered approach has been introduced for both composite reinforced concrete-masonry structures [3] and for infilled frame structures [4].

![Figure 9: Modelling of infilled frame with and without a central door opening. (a) the geometrical layout; (b) model corresponding to the basic mesh; (c) model corresponding to a more refined mesh resolution.](image)

In order to make effective the simulation of the nonlinear behaviour of a masonry structure, it is necessary to infer the mechanical parameters of the model by an equivalence between the masonry media and a reference continuous model characterized by simple but reliable constitutive laws. This equivalence is based on a straightforward fiber calibration procedure, and is based only on the main mechanical parameters that characterize the masonry according to an orthotropic homogeneous medium [2]. For sake of conciseness the details of the calibration procedures are not reported in this paper. It is worth to notice that each macro-element inherits the plane geometrical properties of the corresponding modelled masonry portion, as a con-
sequence, differently from the simplified models based on equivalent strut element approach; the definition of an effective dimension of the element is not needed.

The reinforced concrete beams are modeled according to a classical approach that makes use of monodimensional elements with lumped plasticity. The inelastic behaviour of the element, concentrated at the rigid-plastic hinges, take into account only the bending moment for the beams, while the interaction between the axial force and the bending moments along the main axis of the sections by means of a PMM hinge is considered in the columns.

Each beam of the frame interacts with the masonry infill by means of the nonlinear-link distribution along the macro-element interfaces. For each interface a general layout discretization, of $n$ orthogonal and a single longitudinal nonlinear springs, has been considered coherently with what is done for unreinforced masonry structures. The nonlinear behaviour of the frame element is modelled by rigid-plastic hinges occurring along the beam span at different cross sections. Once the constitutive laws have been defined, both force and displacement controlled processes can be performed according to the procedures currently used in finite element analyses.

A further extension of the considered model has dealt with curved masonry structures [5], and it is suitable for monumental structures (in particular arches and vaults).

5 NUMERICAL APPLICATIONS

The numerical approach, described in the previous paragraphs, has been applied to the case-study under investigation. Namely, the seismic resistance of the existing building has been assessed. The seismic assessment of the structure has been performed through nonlinear static analyses. In the following the results corresponding to eight directions of the building are summarized.

The three-dimensional global model (Figure 10), has been implemented by using the computer code 3DMacro [6], whose capability to simulate mixed masonry-reinforced concrete structures has been recently investigated [7-8]. The nonlinear static analyses have been performed with reference to mass and inverse triangular force distributions.

According to the in situ masonry tests the mechanical properties reported in Table 1 have been assumed, while the mechanical properties of the concrete and steel are summarized in Table 2.
Seismic assessment of the existing building

In order to assess the seismic resistance of the building, sixteen nonlinear static analyses have been performed according to the eight equally spaced directions (with an angular scan- sion of 45°). For each direction two force distributions have been considered (mass-proportional an inverse triangular).

For the sake of conciseness the results are expressed in terms of push-over curves and the collapse behavior is highlighted by means of some deformed shape of few relevant walls. The push-over curves are expressed in terms of base shear (equal to the ratio between the horizontal base reaction and the total weight of the structure) as a function of the roof displacement. The total seismic weight has been estimated in 171405 kN.

In the following Figures 11 and 12 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.

The collapse damage scenarios show global failure mechanisms characterized by shear diagonal failure of the masonry infills and rocking mechanisms for slender walls.

Low level of damage is encountered in the reinforced concrete frames, however their contribution has the valuable effect to guide the failure of masonry panels towards a diagonal shear collapse that is the most dissipative among the possible masonry failures.

![Figure 11: Damage scenarios of the (a) Northern and (b) Southern prospects for the analysis in +X direction (force distribution proportional to the masses).](image)
According to the strength and the ductility of the structure, the Y direction seems to be stronger than the X direction; furthermore for both directions positive and negative ways leads to comparable behaviors hence confirming a substantial structural double symmetry of the building.

Figure 12: Damage scenarios of the (a) Western and (b) Eastern prospects for the analysis in +Y direction (force distribution proportional to the masses).

The estimated base shear coefficient associated to the performed analyses spreads in the range 0.13-0.24, and assumes its minimum for the analyses with inverse triangular force distribution in the X direction.

Figure 13: Capacity basket for the two force distributions: (a) proportional to the masses and (b) according to an inverse triangular distribution.

In order to provide a synthetic view of the performed push-over analyses, the results are reported in terms of three-dimensional domains called capacity baskets [3]. In such a three-dimensional representation (Figure 13) each push-over curve is represented along the input direction, in a plane perpendicular to the horizontal XY plane, so that the base shear coeffi-
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cient is always reported in the Z axis. The radius of the capacity basket is equal to the maximum displacement achieved among all the considered directions (in this case it is 7.3 cm in Fig. 13a and 9.7 cm in Fig. 13b).

Such simple representation allows an easy identification of the stronger and weaker directions of the building. Furthermore the base shear levels are also represented in colored scale.

The ductility of the structure in all the investigated directions is also easily readable in the graph by considering that in the following representations the curve corresponding to the higher displacement touches the vertical axis. The directions which have not been investigated have been linearly interpolated by considering the results obtained in the nearest directions.

In particular, from Figure 13-b the extreme regularity of the structure both in terms of strength and ductility for the inverse triangular force distribution is clear, all the push-over curves are characterized by the same level of maximum displacement that is associated to the reached maximum rotation in the plastic hinges of the reinforced concrete frame structure.

A further representation of the results obtained by this conventional representation is reported in Figure 14 in which the displacement and the strength domains are shown for both the considered force distributions. Namely, in the polar axes the maximum top displacement (in cm) and the maximum base shear coefficient, for the considered analysis, are reported.

It is interesting to highlight how the domains associated to the force distribution proportional to the masses are larger in terms of strength and smaller in terms of displacement capacity than those associated to the inverse triangular distribution.

Figure 14: (a) Displacement and (b) strength domains for the two force distributions.

6 CONCLUSIONS

The adoption of mixed masonry-reinforced concrete as structural seismo-resistant system has been introduced for the first time in the aftermath of Messina-Reggio Calabria earthquake of 1908. The coupling of frames and masonry panels has represented a low-cost constructive technique with a significant quality of performance in occasion of seismic events. The simulation of the nonlinear behavior of this kind of structures is an open problem in which many researchers are involved. Although there are many efficient numerical approaches for modeling both reinforced concrete structures and unreinforced masonry structures, the modeling of mixed masonry-reinforced concrete structure still represents a crucial question in engineering practice. These mixed structural systems are governed by the interaction between frame and infill wall. The highly nonlinear masonry-infill response and the ever-changing contact condition along the frame-infill interfaces make the simulation of the nonlinear behaviour of an entire infilled frame building a challenging problem. In the present paper the seismic assessment of a real mixed masonry reinforced concrete historical building has been performed. It is a six-story mixed masonry-reinforced concrete construction, built in 1926 and today seat of the Prefecture of La Spezia city (Italy). The building has been modeled according to a hybrid ap-
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A computational approach in which the unreinforced masonry panels has been modeled by two-dimensional macro-elements, whereas the reinforced concrete elements has been modeled by means of lumped plasticity elements interacting with the masonry through nonlinear discrete interface elements. The seismic resistance of the building has been estimated by means of nonlinear static analyses performed in several input direction by considering two force distributions consistently with the Italian seismic code.

REFERENCES


