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SEISMIC ASSESSMENT OF DAMAGED MARGHERITA PALACE USING APPLIED ELEMENT METHOD

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Abstract

The Margherita Palace is a heritage masonry structure constructed two centuries ago in L'Aquila (Italy) and extensively damaged by the strong 2009 earthquake. The palace was considered unstable and therefore temporarily supported to avoid its collapse. A structural survey of the palace walls and floors was carried out using laser scanning. The laser scanning data was post-processed to create three dimensional model including the cracks and damage in the different structural elements. The paper discusses challenges dealing with both the laser scan data set and detected damage data to create a full 3-D Applied Element Model of the existing conditions of the building with cracking and material damage. The challenges include the removal of noise from the data, meshing challenges to create 3-D elements with acceptable aspect ratio and applying existing cracking and weakening to different components. Nonlinear dynamic analysis is performed for the damaged structure to check its partial or total collapse resistance if similar strong earthquake were to be applied to the structure, without the temporary supports, again. This analysis helps determining the weakest point of the structure which needs special retrofit attention.

Keywords: heritage masonry structure; laser scanning survey; 3-D Applied Element Model; Nonlinear dynamic analysis.

1. Introduction

L'Aquila is a medieval walled city dating from the 13th century and recognized as the main historic and artistic site of Abruzzo (Italy). In April 6, 2009, a devastating earthquake struck the city of L'Aquila and the surrounding villages causing more than 300 fatalities and thousands of injuries. From 10,000 to 15,000 buildings were completely or partially destroyed. Earthquake caused extensive and severe damage to the territory's valuable real estate heritage stemming from the Baroque and Renaissance periods and including eminent churches (St. Mary's church of Collemaggio, St. Bernardino), important palaces (Palazzo Centi, Palazzo Quinzi), and other monumental buildings and structures (Castello Cinquecentesco, Fontana delle 99 Cannelle, etc).

The research work presented in this paper focuses its attention on damages and collapses observed in masonry buildings. In particular, the paper focusses on Palazzo Margherita, a monumental building (XVI century), the town hall of L'Aquila, situated in the heart of the old city center and seriously damaged by the earthquake.

2. Motivation

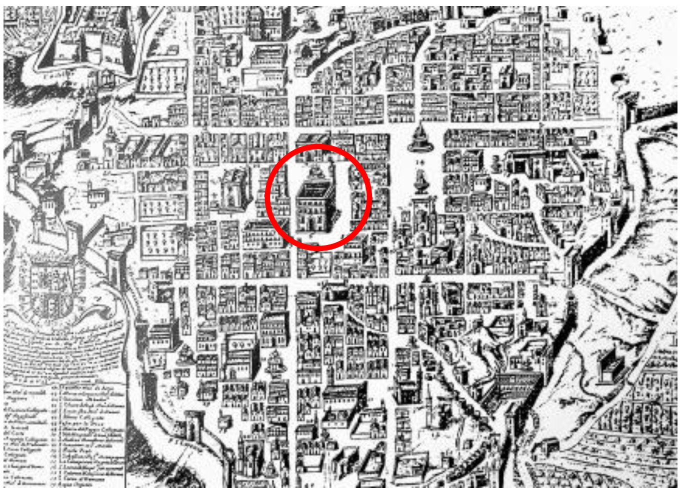
With a magnitude Ml=5.8, April 6, 2009 earthquake represented the main shock of a long lasting seismic sequence including more than 30 minor earthquakes (3.5<Ml<5.0) and several thousand of lower magnitude events [1]. Hypocenters were characterized from being relatively close to the surface (depths ranging from 9 to 15 km respectively) and epicenters were located into an ellipse-shaped region measuring about 15 km in length and 5 km in width, centered on the L'Aquila town and parallel to the Apennines mountain chain. Looking back at the historical seismicity of the area, according to CPTI04 catalog several earthquakes comparable in magnitude to the earthquake of April 2009 struck the town in the past, these main events being characterized by an estimated Richter magnitude as high as Me = 6.5, 6.4 and 6.7 in years 1349, 1461 and 1703 respectively [2]. Geologists state this seismic activity to be the result of a normal fault movement on a NW-SE oriented structure which is part of the 800 km long segmented normal fault system running all along the Apennines mountain chain [3, 4, 5, 6]. The same fault system is also considered responsible for previous recent earthquakes, in Umbria-Marche (1997) and Molise (2002). L'Aquila' seismic sequence never ended, and new minor events continue to be recorded today time as well. This requires rapid intervention on buildings to prevent further damage from and structural collapse. However, time for rehabilitation of the buildings has been estimated in several years, so that restoring interventions have been first addressed to strategic buildings and greatest sights symbol of the town. The need for a variety of different techniques combining both structural controls and the modern methods of geomatic surveys have raised [7, 8]. Currently, engineers and scientist are concentrating a number of investigations to address the geological characterization of the sites and the assessment of the buildings' damage evolution.

3. Seismic Behavior of Historical Masonry Buildings

Main purpose of aftershock investigation is to identify main causes of the building damages and failures and to recognize recurrent collapse mechanisms associated with the different structural types. Monitoring of the damage evolution and survey activity are part of the inquiries and provide useful data to evaluate residual structural safety and strategies of intervention. The poor connections among orthogonal masonry walls, reduced floors stiffness, wrong mass distribution and absence of restrains to wall over-turning have been already recognized as causes for collapse of most masonry buildings (especially in traditional palaces and high-density residential quarters in the old city center). In L'Aquila, all historical buildings dating back to the 15th and 16th century are frequently characterized from a traditional masonry structure consisting of load-bearing masonry walls not well orthogonal to each other and not re-producing a perfect three-dimensional box behavior. Actually, these vertical systems are frequently combined to curved structural elements, such as arches and barrel/cross vaults, that sustain the horizontal floors in place of rigid plane structures. Typical masonry patterns feature an irregular stonework, in which stone units laid in approximately parallel horizontal courses and are often jointed with poor quality mortars. Concerning the historical buildings that survived the earthquake, their collapse was in most cases avoided by tie rods, anchor plates and other simple but effective earthquake-resistant measures. Recent strengthening interventions may have modified the building structural behavior by application of ring beams at the floor levels and replacing of the vaulted structures with reinforced concrete slabs. A debate is currently underway concerning the actual effectiveness of such measures (typical retrofitting techniques of past decades). Scattered distribution of the heaviest damage recognized from site to site, suggests a crucial role in the amplification of the seismic action may depend on site-specific conditions as well.

3.1 Palazzo Margherita: Site and Building Description

Margherita Palace is a monumental masonry building standing in the old city of L'Aquila, almost at the center of the old fortified town (Fig. 1). The main front of the building faces Palazzo Square. On the back side, the building faces S. Margherita Square. Next to Palazzo Margherita are several other historical buildings including Palazzo Camponeschi (XVI century), the old Jesuits' Church (XVII), Palazzetto dei Nobili (XVII century). Original site plan probably dates XIII century, although profound intervention were made on the building during XVI century.



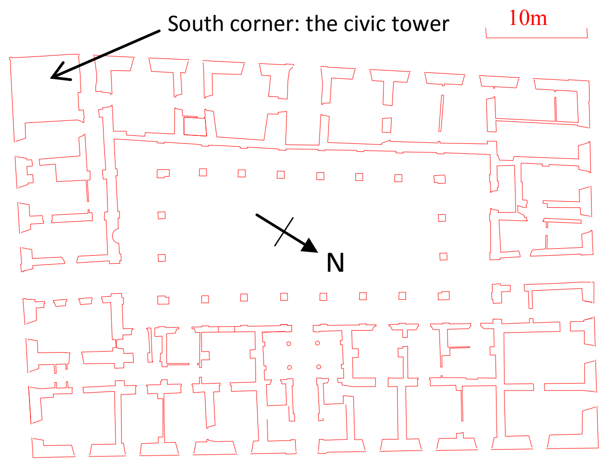


Fig. 1– On the Left: an old drawing of L'Aquila town. Margherita Palace (into the red circle) in axonometric view; on the right: the plan of the building at the first level.

Building geometry is currently characterized by an rectangular shaped plan measuring about 40x60 m with the longer sides aligned in SE-NW direction. The four sides of the building surround an internal court yard. At the south corner of the building stands a 41 m high civic tower, characterized by an almost square plan, and consisting of stone made walls measuring about 1.70 m in thickness at the base (Fig. 1). The palace consists of three stories, each of them measuring about 5 m in height. The first level is partially set below the sloping ground level, with the highest fronts of the building at the North corner of the building (Fig. 2). The building's facades present a regular vertical and horizontal distribution of three orders of rectangular windows. A detailed geometric documentation was needed to better understand the entire structural volume, the nature of its constructive elements (walls, arcs, roofs and attics) and the building's spatial distribution. At the basement, the fronts facing the internal court-yard are characterized by regular arches supported by masonry columns (Fig. 2c). A laser scanning survey was then carried out to create 2D and 3D models of the structure (Fig. 3). An extensive study of both the internal and external fronts of the building was performed by means of thermographic end ultrasonic investigations as well. This allowed for a better damage analysis (see next section), and helped in documenting all visible cracks and minor collapses as well [7, 8].

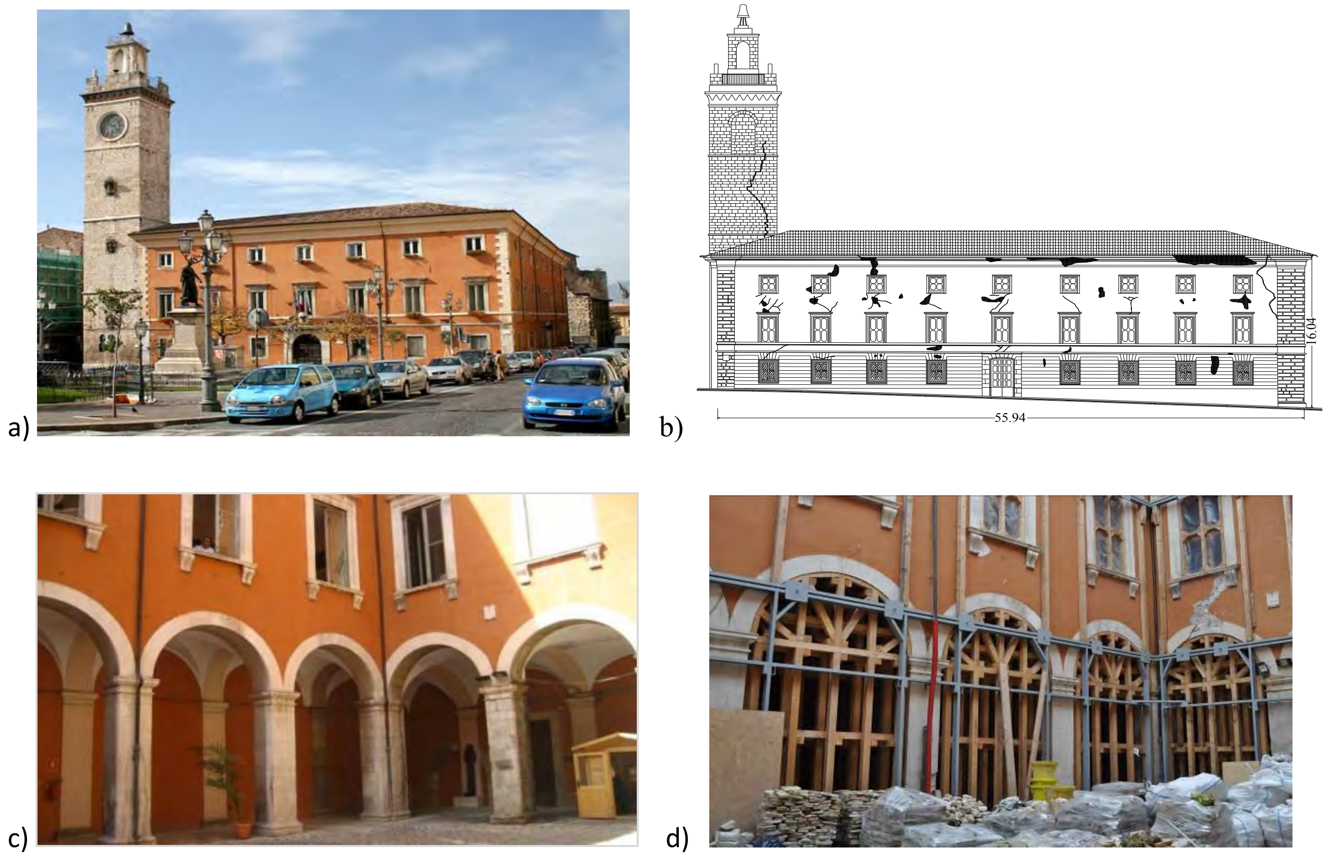


Fig. 2 – a) View of the S-E front of the building from Palazzo Square (before the April 2009 earthquake); b) Drawing of the N-E front of the building; c)-d) View of the building from the internal courtyard before and after the 2009 earthquake; d) Shoring system.

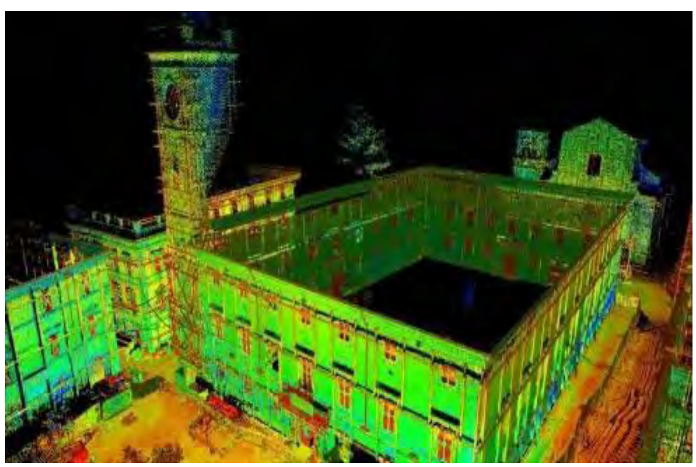


Fig. 3 – A view of the three-dimensional model of the building created with laser scanning.

3.2 Seismic Damage Analysis

From a structural viewpoint, Palazzo Margherita's original resistant system consists of vertical masonry walls composed of irregular stone units and poor lime-clay mortar. A chaotic (irregular) masonry texture is generally recognized in the building. Several thermographic analysis and ultrasonic tests were carried out to better assess the quality and the homogeneity of the masonry from wall to wall of the building. A number of different masonry typologies were recognized, probably as a result of restoring and retrofitting interventions done during the past centuries. The overall poor connection among orthogonal walls and the inhomogeneous masonry in the walls thickness were certainly causes for the heavy structural damages suffered during the April 2009 earthquake (Fig. 4a). At the first level, all the rooms present a roof consisting of single or a double curvature vaults made of bricks. Trough vaults, groined (cross) vaults, cloister vaults and ribbed vaults are largely diffused in the building (Fig. 4b). These structural systems are part of the floor package usually including a variable thickness of filling material, a few centimeter thick layer of poor concrete, a few centimeter thick layer of mortar, then a marble pavement. The vaults system suffered heavy damage (Fig. 4c), with longitudinal cracks frequently due to the unrestrained displacement (overturning) of the sustaining walls. At the second level, all the rooms have a flat roof which structurally consists of steel joists (spaced each 0.8m-1.0m) and supporting long hollow blocks. Simply supported steel joists are oriented parallel to the shorter dimension of the rooms. At the two ends, the joists are inserted into the wall thickness for about 0.25 m. Nothing, other than simply friction, prevents the lateral disengagement at the extremities of the steel joists. Actually, this lack of restraint caused the collapse of floors during the earthquake (Fig. 4d). A traditional gable roof consisting of wood truss, secondary wood elements, planking, and brick tiles covers the building.

Fig. 4 – a) Diagonal cracks due to in plane action resulted in extended portions of the resisting walls: out of plane displacement have been recognized in few front walls of the building; b) typical geometry of cross vaults system; c) partial collapse of a trough vault; d) Part of a floor collapsed as consequence of the joist ends disengaged from the sustaining wall.

3.2 Numerical Simulation of the Building Dynamic Response

Several in situ mechanical tests (double flat jack) were carried out on masonry wall to determine current stress, elastic properties and strength of the masonry. Combined to data concerning the building geometry, this information allowed preparing a detailed numerical model of the building at the computer (Fig. 5). The civic tower was excluded from the model. Mechanical properties assigned to masonry are summarized in Table 1. These values agree well with those suggested by Italian Rules and Recommendations with respect of a chaotic masonry typology [9, 10].

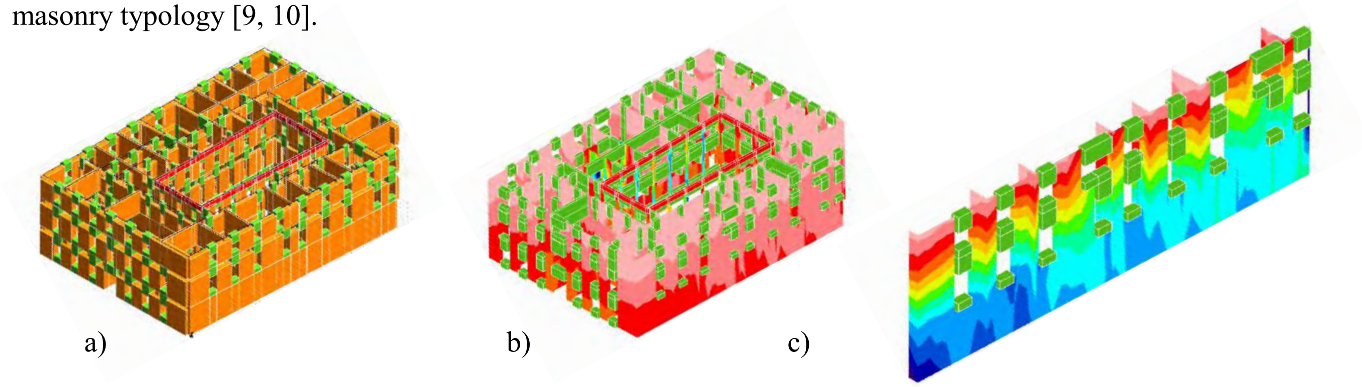


Fig. 5 – a) Numerical model of the building; b) Elastic Analysis under vertical load: predicted stress in the structural elements; c) Qualitative stress distribution concerning the SW front of the building.

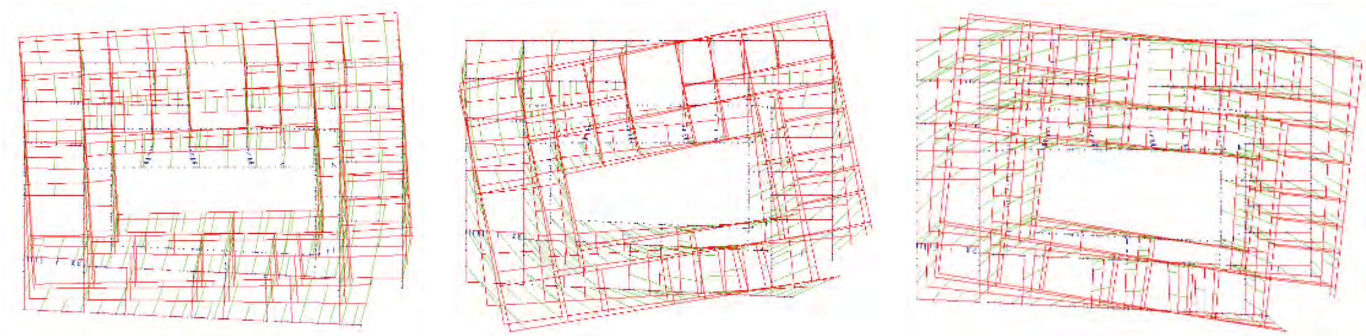
Table 1 – Mechanical properties assigned to masonry.

Compressive Strength f_m (N/mm^2)	τ_0 N/mm^2	E N/mm^2	G N/mm^2	Unit weigth kN/m^3
1.4	0.026	870	290	19

First, an elastic analysis was carried out to evaluate the stress concentration in the structures subjected to vertical loads only (gravity and service loads). The predicted stress level into material was qualitatively compared to actual stresses measured in situ. Although more sophisticated analyses were possible (see next section), hypotheses concerning the poor connections among orthogonal masonry walls, the reduced in plane stiffness of the floors and the real mass distribution were assumed in a simplified manner when considering the elastic numerical model of the building. Numerical simulations confirmed all these hypotheses strongly influence the predictions of the local and global form of collapses of the building, also predicted. Modal properties of the building are summarized in Table 2 and in Fig. 6.

Table 2 – Modal properties of the building.

Mode n.	Period [sec]	Mass X [%]	Mass Y [%]
1	0.270	0.43	82.67
2	0.238	18.90	6.23
3	0.228	69.85	0.34



First mode Second mode Third mode

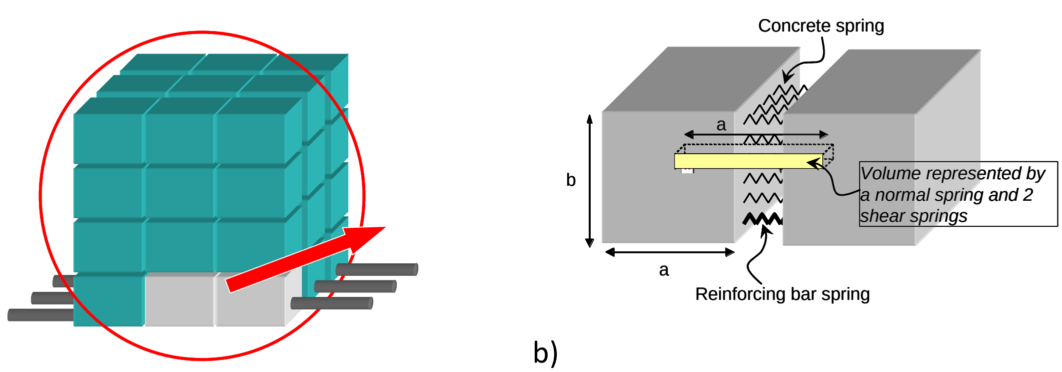
Fig. 6 – Modal properties of the structure.

4. The Applied Element Method (AEM)

In this study, the Extreme Loading® for Structures (ELS) software [11], which is based on the AEM, is used. The AEM was initially developed at the University of Tokyo in 1998 by Tagel-Din and Meguro (2000, a,b) solving a two-dimensional plane stress problem. Later on, in beginning of current century, it was expanded to solve three-dimensional problems with the ELS software. The AEM is an innovative modeling method that adopts the concept of discrete cracking.

In AEM, structures are modeled with element assembly, as shown in Fig. 7a. The elements are connected along their joint surfaces through a set of normal and shear springs. Those springs are responsible for transfer of normal and shear stresses among adjacent elements. Each spring represents stresses and deformations of a certain volume of the material, as shown in Fig. 7b. Each two adjacent elements can be completely separated once the springs connecting them fail. Fully nonlinear path-dependent constitutive models are adopted in the ELS software, as shown in Fig. 7c. For concrete in compression, an elasto-plastic and fracture model is adopted Maekawa and Okamura (1983) [12]. When concrete is subjected to tension, the linear stress-strain relationship is adopted until cracking, where the stresses degrade to zero.

Since AEM adopts a discrete crack approach, the reinforcing bars are modeled as bare bars for the envelope (Okamura and Maekawa, 1991[13]), while the model of Ristic et al. Ristic et al. [14](1986) is used for the interior loops. AEM is a stiffness-based method, in which an overall stiffness matrix is formulated and the equilibrium equations including each of stiffness, mass and damping matrices are nonlinearly solved for the structural deformations (displacements and rotations). The solution for equilibrium equations is an implicit one that adopts a dynamic step-by-step integration (Newmark-beta time integration procedure) (Bathe, 1995 [15], Chopra, 1995[16]).



a) b)

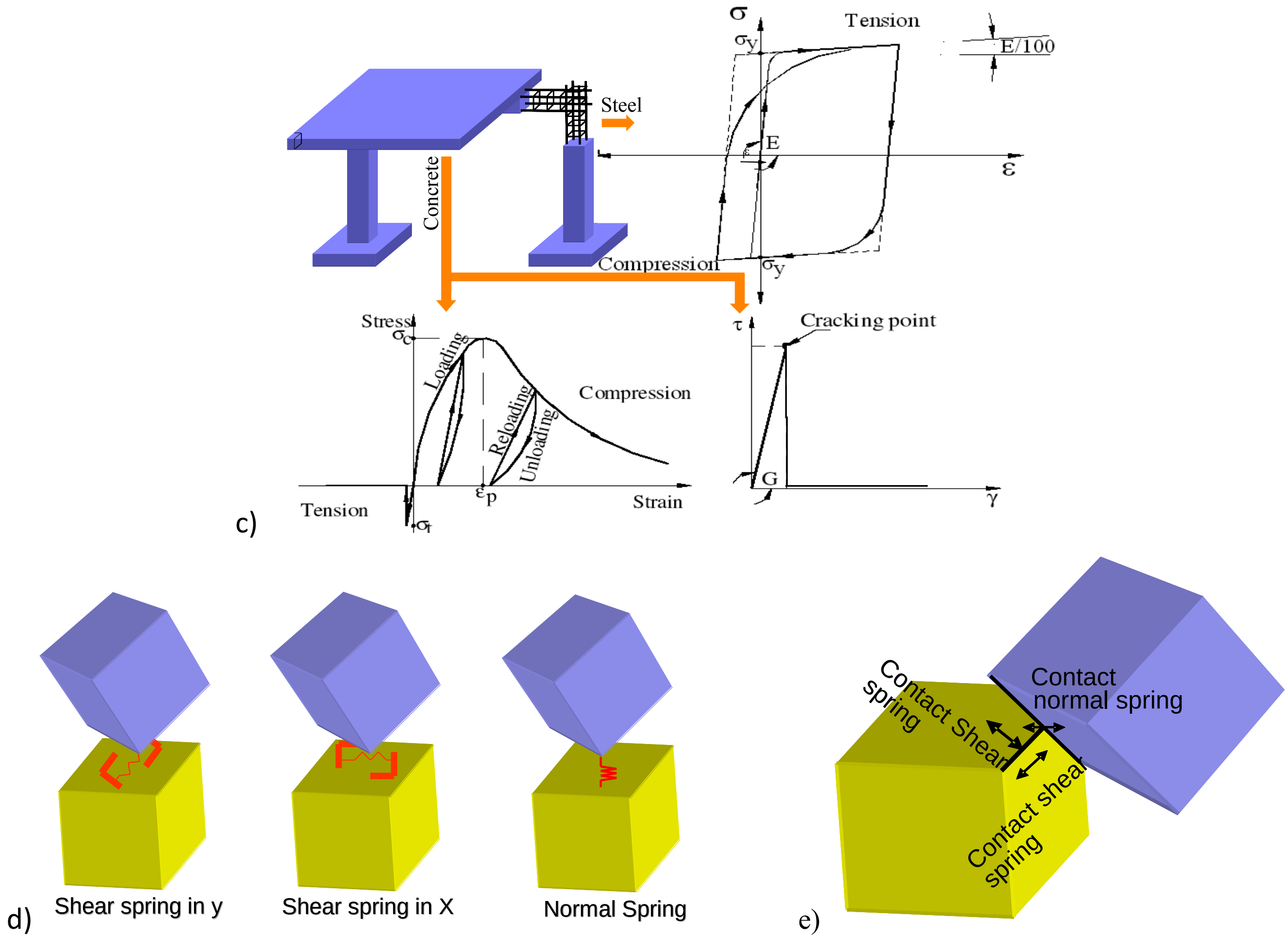


Fig. 7 – Schematic modeling of a structure with the AEM: a) Element generation for AEM; b)Spring distribution and area of influence of each pair of springs; c) Examples of constitutive laws; d) Corner-to-face or corner-to-ground contact; e) Edge-to–edge contact.

In AEM, two neighboring elements are separated from each other if the matrix springs connecting them are ruptured. Elements may automatically separate, re-contact or contact other elements. Figures 7d) and e) illustrate the different types of element contact, where contact springs are generated at contact points.

The AEM was proven to be capable of following the deformations of a structure subjected to extreme loads to its total collapse[17-32] Therefore, since the goal of the current study is to investigate the behavior of the historical building under severe seismic action, it was decided that the AEM is the most appropriate numerical tool for such an investigation.

5. Structural Model of the Palace

The structural model of the palace is shown in Fig. 8. The walls were modelled as bricks while the floors are modelled as reinforced concrete slabs, either single-curved, double-curved or horizontal slabs according to the given survey of the palace. The total number of elements in the model is 120,000 elements with 720,000 degrees of freedom. Damage existing in the structural elements is taken into consideration in the form of pre-cracking of the cracked elements. Figure 9 shows a sample for cracks and damage modelling in ground floor roof, while Fig. 10 shows a sample for cracks and damage modelling in ground floor walls. Pre-cracks are modelled in their exact locations as per the structural survey of the damage. No tensile strength of concrete or bricks is considered at the locations of the cracks.

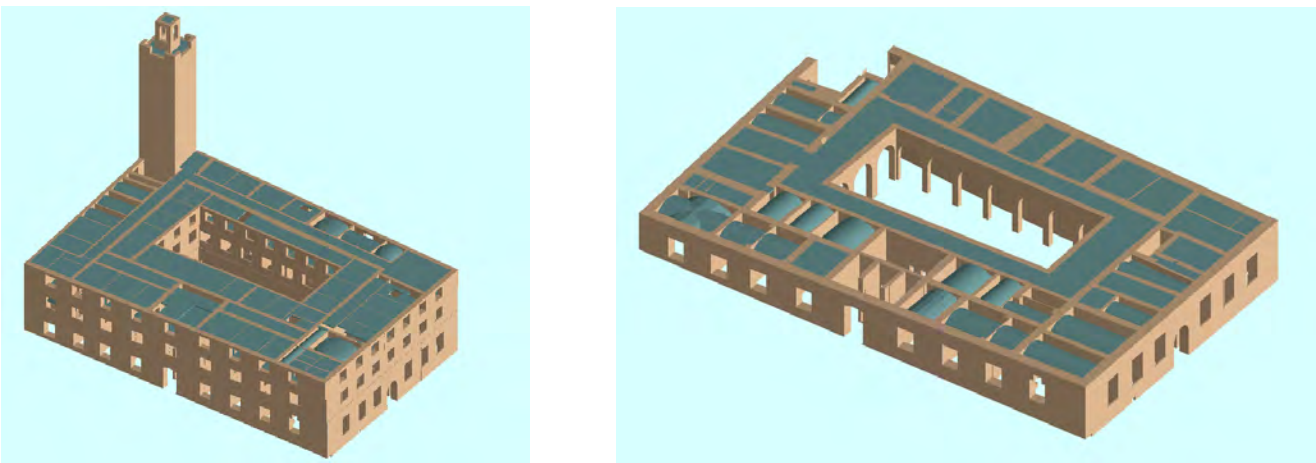


Fig. 8: 3D model of the palace (on the left) and model of the ground floor (on the right).

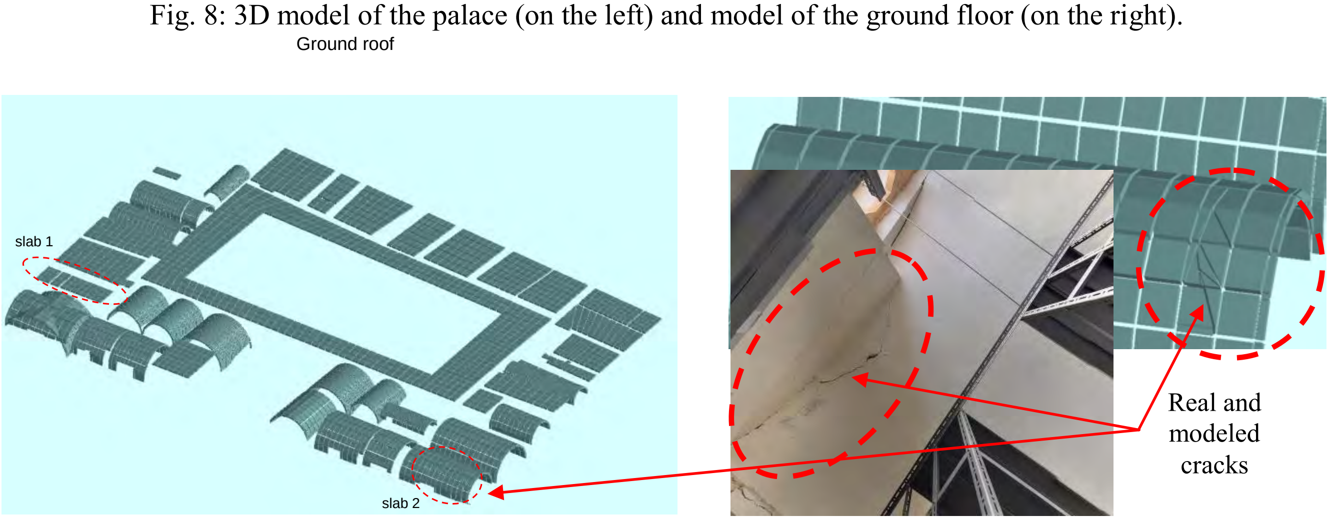


Fig. 9– Sample for cracks and damage modeling in ground floor roof.

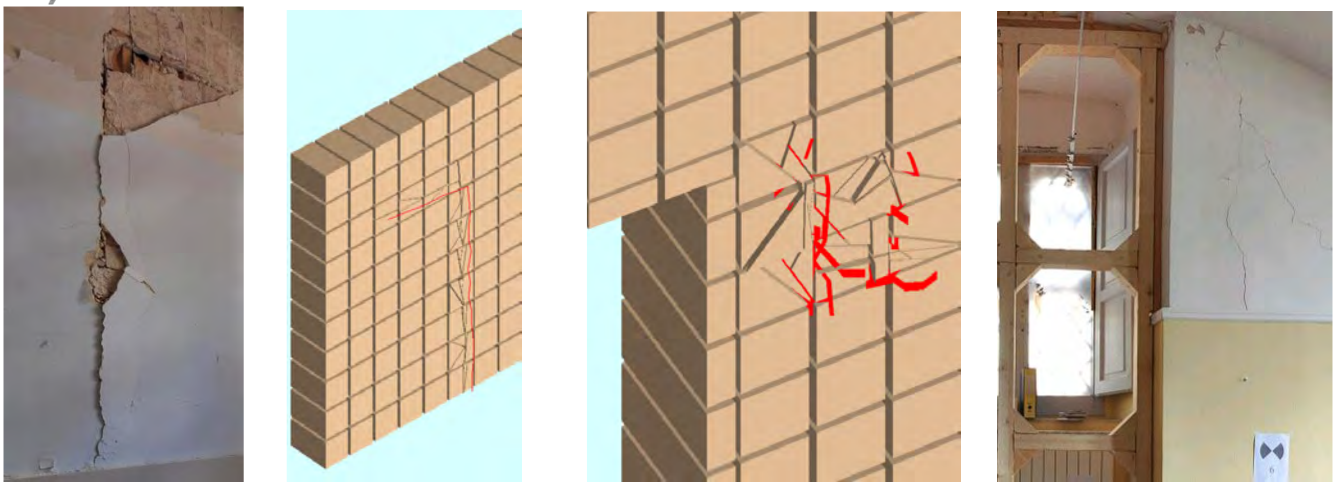


Fig. 10 – Sample for cracks and damage modeling in ground floor walls.

The mechanical properties of constituting materials of the palace are shown in Table 3. The earthquake record used in analysis is shown in Fig. 11. The accelerograms used were selected among a group of 7 different records found to be in agreement with the response spectrum compatibility criterion.

Table 3 – Mechanical properties of constituting materials.

Material	Brick	Concrete	Steel
Yield stress (MPa)	----	----	360
Tensile strength (MPa)	0.3	1.0	504
Compressive strength (MPa)	3	20	504
Young's modulus (MPa)	2500	26716	203000
Shear Modulus (MPa)	1000	10686	81556

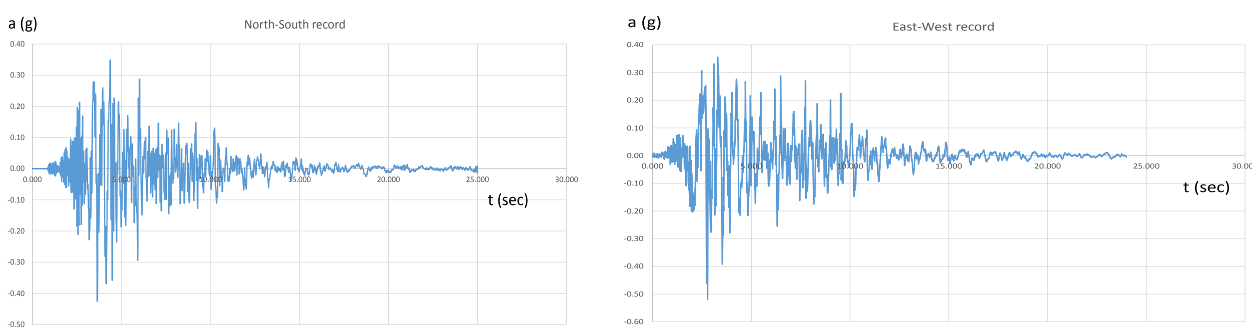


Fig. 11 – Earthquake records used in dynamic analysis.

6. Analysis, Results, and Discussion

The application of the earthquake loading to the palace showed a partial collapse of its structure as shown in Fig. 12. The collapse took place in the roofs of the ground, first and second floors with minor or almost no damage in the palace walls.

Fig. 12 illustrates the progression of collapse of a part of the roof of first floor. In this slab, pre-existing cracks are there on three sides of the slab, totally penetrating concrete. Due to these through cracks, when the earthquake was applied to the palace, a stress concentration was generated at the pre-existing cracks locations leading to slippage of the steel joists out of the walls, and consequently the fall of the slab to the lower floor. The collapsed roof impacted with the roof of the ground floor eventually causing the collapse of this roof as shown in Fig. 12. It should be mentioned also that the analysis revealed that the tower of the palace did not experience any obvious damage due to the earthquake as shown in Fig. 12.

As a whole, the seismic analysis of the palace showed that the palace structural integrity would be significantly affected by the earthquake only with respect of precise portions of the building.

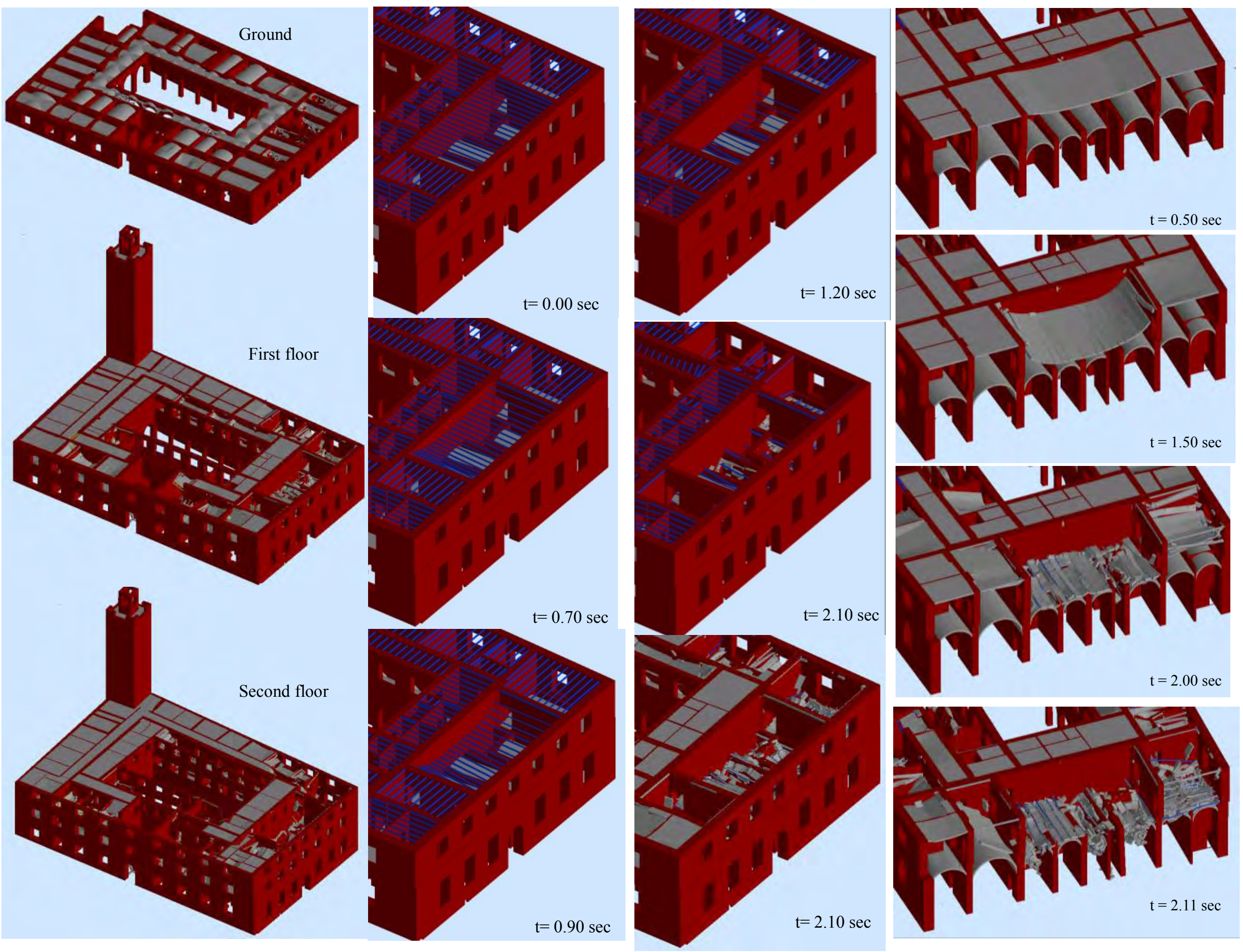


Fig. 12 –Damage of the palace obtained from AEM analysis. On the left: damage of different floors; In the middle: progression of failure through slippage of steel beams out of the walls; on the right: damage of the roof of the ground floor due to falling of the upper roofs.

7. Summary and Conclusions

A structural survey of the Margherita Palace walls and floors was carried out using laser scanning, and then scanning data was post-processed to create three dimensional model of the palace. The model included the cracks and damage in the different structural elements. 3-D nonlinear dynamic analysis using the Applied Element Model is performed for the damaged structure to check its partial or total collapse resistance if similar strong earthquake were to be applied to the structure, without the temporary supports, again. The analysis results showed that the palace would be subjected to a partial progressive collapse which would take place in the floors with minor damage evolution in the walls and the tower. This analysis helps determining the weakest point of the structure which needs special retrofit attention.

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