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FINITE ELEMENT ANALYSES FOR SEISMIC PERFORMANCE ASSESSMENT OF HISTORICAL MASONRY BUILDINGS: THE CASE OF OMAR TOSON PALACE (CAIRO, EGYPT)

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ABSTRACT

This paper presents experimental and numerical investigation of a historical masonry building, Omar Toson palace, subjected to static and earthquake loading in order to assess its structural behaviour and seismic vulnerability with respect to its actual state of conservation. The static behaviour and the dynamic properties of the Palace have been evaluated using the finite element modelling technique, taking into consideration all the geometrical and material nonlinearities, in order to investigate the seismic behaviour of the structure by means of nonlinear analysis. A macro-modelling approach has been used based on the concepts of homogenisated material and smeared cracking and crushing constitutive law. The calculations took the values for external loads, earthquake and displacement into consideration. A full 3D non-linear time-history analysis of the entire structure has been done in order to correctly assess the seismic vulnerability of the palace. The numerical analysis has given a valuable picture of possible damage evolution and provides a basic understanding of the structural performance under seismic loads that can support the selection of possible retrofitting measures.

KEYWORDS: Historical buildings; Seismic vulnerability; Nonlinear analysis; finite element method; Time history analysis; Omar Toson palace.

1. INTRODUCTION

The present scientific research discusses the structural behaviour and the seismic vulnerability of the Omar Toson palace, located in Shubra al-kheyma, Egypt. The conservation and the restoration of ancient buildings belonging to the culture heritage and preserving their main architectural features is a very sensitive problem (Betti et al., 2010; Barbieri et al., 2013, Elyamani and Roca, 2018 a, b, c). The assessment of the vulnerability of the historical unreinforced masonry building (HURM) is an essential prerequisite to its seismic risk assessment (Vicente et al., 2013; Asteris et al., 2014). In order to correctly assess and improve the durability of HURM buildings, it is particularly important to understand its structural behaviour under several load conditions with respect to its original characteristics, the most probable causes of damage, the existing damage products, and the changes occurred over the time.

While the structural behaviour of a new masonry construction is a relatively simple task due to the presence of standard codes and available literature (Betti et al., 2011), the predication of the structural response of historical buildings is a more challenging task. In brief, historical buildings are by definition unique buildings and its typological characteristics do not allow us to refer to simplified standard structural scheme because of the uncertainties that affect the structural behaviour (Betti, M., Vignoli, A., 2008) and the mechanical properties of the material lead to a widely non-linear behaviour whose prediction can be very tricky (Giordano et al., 2002). Furthermore, the seismic behaviour of old masonry structures depends on many factors such as geometry of the structure, to be defined by proper surveys; stiffness of the floors (diaphragm effect); connection between orthogonal walls and structural and non-structural elements (Barbieri et al., 2013) and the peculiar properties of masonry structures (low tensile strength and lack of box behaviour) (Lourenço et al., 2001). In particular the key point of masonry behaviour is the mechanical properties of its components and the interfaces between these materials. Additionally, in the non-linear range, masonry behaviour is usually hard to be predicted, because of the combined material performance depends on the properties and failure mechanisms of each constituting material. Masonry non-linear failure mechanisms involve compressive failure of bricks and mortar, tensile and shear failure at the mortar-brick interface where loss of cohesion and frictional slip are resulted. Nevertheless, the non-linear behaviour of masonry buildings is not only attributable to material properties, but it is also attributable to the interaction between the different structural members, i.e. masonry walls, steel or timber beams and columns, timber floors, and their connections (Vera, 2012). The previous considerations enlighten the need of scientific methodology for modelling and analysis of the seismic vulnerability of historic masonry buildings.

The seismic vulnerability assessment of historical masonry buildings is an essential prerequisite to correctly understand its structural performance, extensive damage and possibly to collapse as explained in the works of (Lagomarsino, 1999; Magenes, 2000; Kappos et al., 2002; Mele et al., 2003; Lagomarsino and Podestà, 2004; Lourenço et al.,2007; Betti and Vignoli, 2008; Petrovc and Kilar, 2013; Preciado, 2015; Cakir et al., 2015). In order to correctly assess the structure behaviour and seismic vulnerability of Omar Toson palace different levels of approaches were carried out; the evaluation of the structural performance of the palace mainly depended on the interpretation of various mechanical and physical testing and the structural analysis calculations. The numerical modelling of the palace using FEM was adopted for Linear and Non-linear analyses of static and seismic loads. Various issues concerning the URM building behaviour had to be assessed; the present study concentrates on three of these issues: (i) the construction materials properties, (ii) the structural system of the building, and (iii) numerical modelling and analysis of the palace using FEM under static and seismic loads. Firstly, the construction materials properties affecting the behaviour of Omar Toson palace have been investigated through mechanical and physical testes including, bricks, stones, mortars, and woods using standard experimental tests. Secondly, the structural system of the palace have been investigated; the exact geometry of the structure is acquired; measured drawings were prepared; the structural deficiencies were determined; and finally a 3D model of the structure was assembled together to achieve this goal. Finally, the static and seismic response of the palace had been evaluated using the finite element modelling technique, where the nonlinear behaviour of masonry has been taken into account by proper constitutive assumptions. The calculations took the values for external loads, earthquake and displacement into consideration. 3D nonlinear static and dynamic analyses of the overall structure have been done in order to correctly assess the structural behaviour and seismic vulnerability of the palace. The main information which obtained from the linear analyses of the palace is the interaction, and in particular the stress resultant distribution, among its structural elements in the two principal directions.

2. CASE STUDY

2.1. *Architectural characteristics and materials properties*

Omar Toson place is located in Shubra al-kheyma district, south of Cairo, which was constructed sometime after 1869 A.D during the Ottoman's period in Egypt. The geometry of the palace has been completely acquired and organized within a CAD system. The positions of openings and the variation of the thickness of the palace with respect to that quoted have been carefully recorded. The CAD model served as a basis for the mesh generation needed for further FEM analysis.

The palace is a rectangular masonry construction consists of a basement and two floors. Basement is consists of long rectangular corridors with semi-circular brick masonry vaults based on brick masonry pillars. The upper floors consist of a major corridor, several rooms used for receive visitors, living rooms and a large room which was used as a library (Fig. 1-H). The rigid basement roof, of semi-circular brick-masonry vaults, making load bearing walls are totally braced till the roof of ground level (Fig. 1-F), and the flat wooden roof of two upper floors make the rest of height of these walls are partially braced (Fig. 1-G). The roofs of upper floors are simple flat wooden roof. The whole building was constructed by load bearing wall system. The masonry wall thickness ranges between 0.70 m and 1.20 m. The load bearing walls are variable between solid and multi-leaf type; the internal filling is composed of heterogeneous material (the remainder brick tied by a poor mortar). Brick masonry walls used in the construction of the small rooms on the upper floor and the vaulted roof in the basement were observed; these walls have thickness of 0.40 m. The main dimensions of the palace were a maximum length of about 86.50 m, a maximum width of 38.75 m and a maximum height of about 23.30 m (Fig. 2, 3 and 4). However, despite these differences, the construction was mostly made of irregular limestone masonry with thick lime mortar joints. Laboratory tests have been carried out on limestone, red-bricks, and timber to define its physical and mechanical properties.

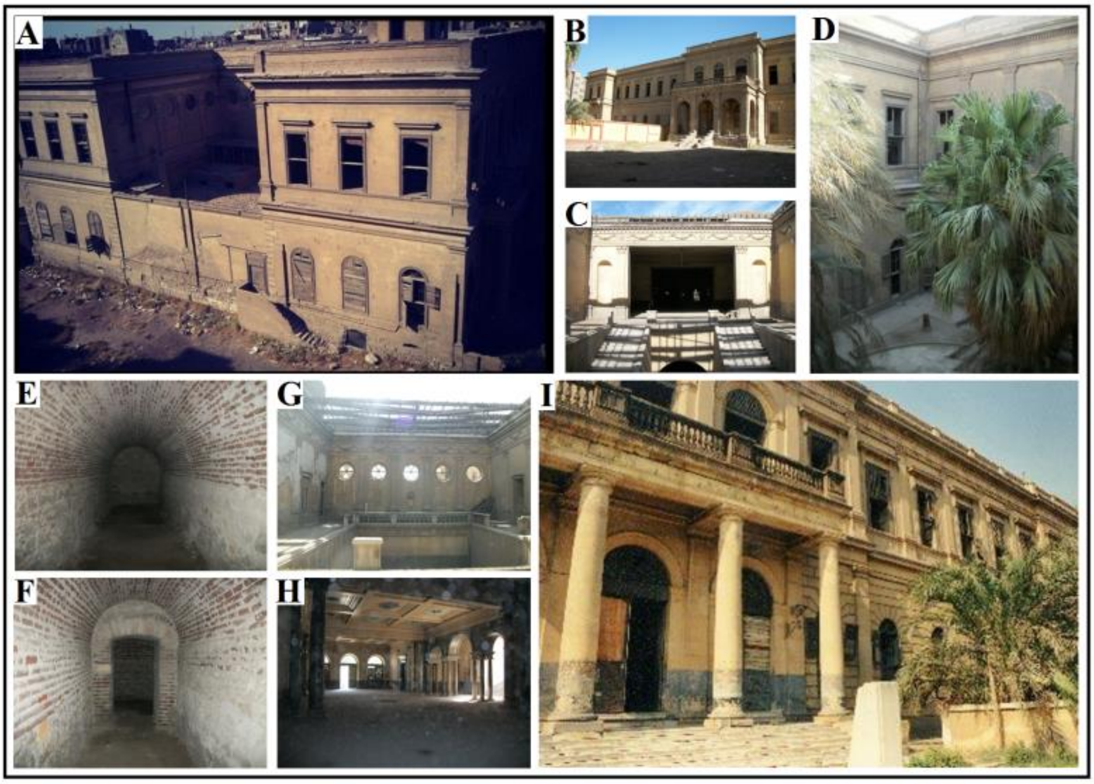

Fig. 1. General views of Omar Toson palace showing: A) North-eastern façade, B) South-eastern façade, C) Stairs to the second floor , D) the internal courtyard , E) long rectangular corridors in basement, F) semi-circular brick masonry vaults, G) damaged roofs in the second floor , H) major corridor in the second floor I) south-eastern (main) façade

In order to obtain basic material parameters which define load bearing capacity, (compressive strength and modulus of elasticity) compression tests were performed. These tests were done on main construction materials to determine the elastic, inelastic and strength parameters required for the present finite element modelling. The specimens were taken from various locations of the Omar Toson palace and tested under standard uniaxial compression test, the specimens were centrally positioned in the testing machine and the applied load was uniformly distributed. In addition, the load increased continuously. The obtained results from the experimental program are reported in Table 1, 2.

Simple calculations, based on the relation between the compressive strength of masonry and the compressive strength of its single components (units and mortar), were used in accordance with the Eurocode 6, CEN (CEN, 2003) to predict the compressive strength fc of masonry walls. The fc value was obtained as follows:

$$f_c = Kf_{cu}^{\ 0.7} f_m^{\ 0.3} \tag{1}$$

Where f_{cu} is the characteristic compressive strength of masonry unite, f_m is the characteristic compressive strength of mortar, and K is a constant depending on the quality of the masonry units. K factor ranges between 0.40-0.60. These equations can be used where the thickness of the mortar joints, both horizontally and vertically, is between 8 and 15mm. According to (Abdel-Aty, 2004), in the case of stone masonry, the value of K factor can be estimated as 0.5; and for mortar joint width of 1 cm, the fc can be approximated to:

$$f_c = 0.35f_{cu} \tag{2}$$

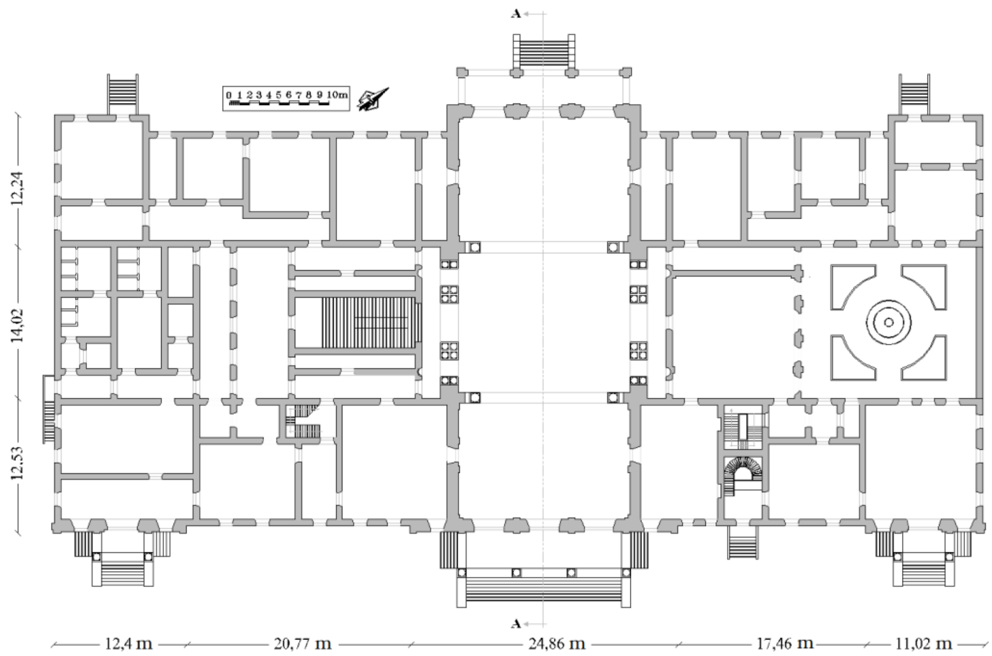

Fig. 2. Ground floor layout

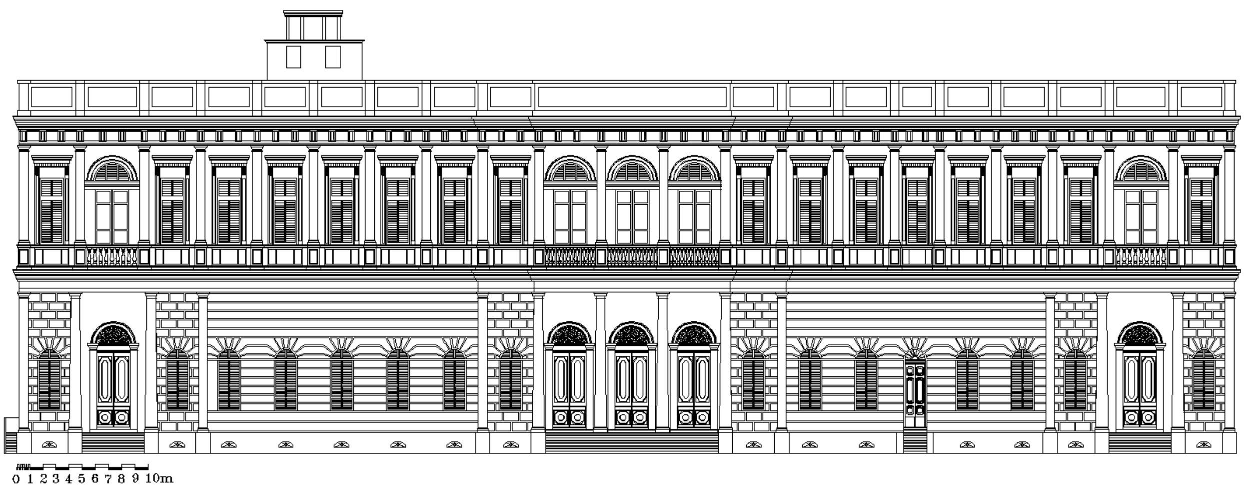

Fig. 3. North-western elevation of Omar Toson palace

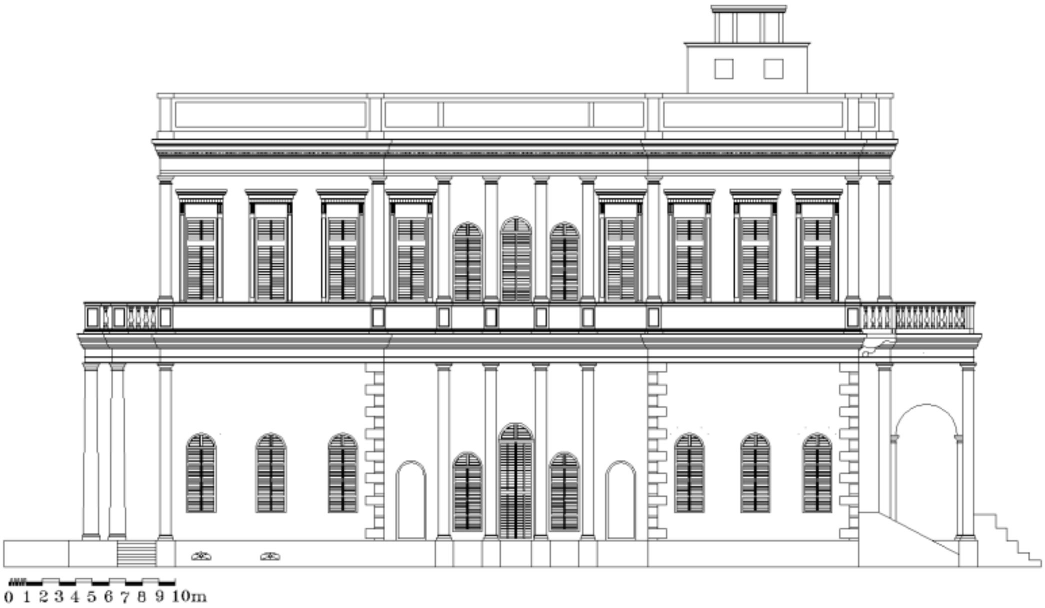

Fig. 4. South-western elevation

Table 1. Experiments results for stone and bricks

S. No		γ (kN/m³)	W. A %	η %	f_c N/mm²	f_t N/mm²	E N/mm²
stone	1	22.13	6.16	4.6	36.14	2.9	5526.14
	2	20.20	5.07	3.3	28.57	2.2	5311.32
	3	19.02	2.03	1.8	26.14	1.9	4480.87
Bricks	1	21.03	18,95	11.4	6.51	2.9	3634.46
	2	19.41	12.54	10.5	4.45	2.7	3602.73
	3	19.68	18.80	11.2	8.20	2.8	4729.95

Table 2. Experiments results for timber

S. No		Weight per unit Volume (kN/m³)	W. Ab %	f_c N/mm² Parallel to grain	f_c N/mm² Perpendicular to grain	f_t N/mm² (in fibre direction)	E N/mm²
Timber	1	7.8	12.1	25.3	44.6	**Timber**	1
	2	6.3	11.7	22.4	48.4		2
	3	7.6	12.3	24.7	43.2		3

Where, γ: unit weight, ***WA***: water retention ratio, ***η***: porosity, f_c: compressive strength, f_t: tensile strength, ***E***: modulus of elasticity

While the modulus of elasticity E_m and shear modulus G_m were estimated, according to Eurocode 6, CEN (CEN, 2003), as follows:

$$E_m = 1000 f_c \quad (3)$$

$$G_m = 0.4 E_m \quad (4)$$

2.2. Structural deficiencies and damage survey

Diagnosis is the first phase of any study and represents the judgment on the cause and nature of damage, cracks, decay and the other phenomena that have affected a building. Safety evaluation is the subsequent judgment on the capacity of the structure to resist specific actions such as loads, earthquakes, etc., and the potential risk involved (Croci, G., 2000).

The cracks patterns of the palace are quite complex. Some of the upper floor walls and roofs are greatly damaged; there are several structural cracks through the top of the walls in many locations (Fig. 5-A, B, C, H and G). According to the historical sources, the palace was used as a secondary school and some windows in various sizes were opened while some others were closed (Fig. 1-A); which cause the existing cracks getting denser in the north-western and south-eastern parts. The palace was exposed to a fire, which caused the destruction of the wooden roof of the south-western area of the upper floor. Moreover, the weathering factors have decreased the compressive strength of the stones. Similarly, the current mortar used in the joints bonding the stones is extremely cracked and the stones are practically held on under gravity. Major cracks are observable on the north-western and south-western façades and on the corresponding internal masonry walls. In contrast, the north-eastern and the south-eastern facades are substantially in a good state of preservation. Fig. 5-A shows cracks on the north-western facade which are concentrated on the central part that is the most damaged area. Fig. 5-G shows cracks on the internal wall parallel to the south-western facades. Fig 5-D, H, J show the main cracks in the internal walls on the first floor, while Fig 5-C reports details of the cracks present at the basement in the eastern area of the palace. The ground floor tiles are have sever settlement in many location that reach in some parts to about 12cm depression (Fig. 5-F). The internal walls have high dampness and covered with efflorescence salts. It is noteworthy to underline that, it is possible to observe that the damage does not affect the whole palace, but mainly the western and southern parts of the building that can slight to moderate impact on the structural stability of different elements in the building.

3. STRUCTURAL ANALYSIS METHODOLOGY

As already discussed in the introduction, the seismic behaviour of old masonry structures is particularly difficult to be investigated, It depends on many factors such as material properties, to be characterized by direct inspection, geometry of the structure, to be defined by proper surveys; stiffness of the floors (diaphragm effect) and connection between orthogonal walls and structural and non-structural elements (Barbieri et al., 2013).

The assessment methodology in the present study is mainly the achievement of an adequate knowledge of structural behaviour. A preliminary in situ survey of the palace was made to obtain basic information on the geometry, the structural details and any irregularities. The paper approaches the numerical problem using the FE technique to assess the seismic vulnerability of the palace. In order to assess the seismic vulnerability of the structure 3D static and dynamic analyses of the whole structure have been done taking into account the masonry nonlinear behaviour.

The numerical model has been prepared according to the geometrical information gathered from the historical documents and by the measurement of the actual conditions of the structure with focusing on the variations in the wall thickness, in the geometrical and structural irregularities and on the wall connections. Finally, the major openings in the building have been reproduced as it shown in (Fig. 6). The 3D numerical model was developed, using the F.E. ANSYS software package. According to (Betti et al., 2011), the nonlinear behaviour of the masonry elements was reproduced assuming a Mohr–Coulomb type failure criterion with tension cut-off type behaviour, a Drucker–Prager (DP) perfectly plastic material criterion has been joined with the Willam and Warnke (WW) failure surface.

Table 3 reports values assumed for both linear and non-linear characterization of masonry materials. According to (Lourenco P.B, 2002; Senthivel & Lourenço, 2009; Asteris & Giannopoulos, 2012; Betti M., Galano L., Vignoli A., 2015), macro-modeling approach was used (Fig. 7), masonry is considered as a homogeneous anisotropic continuum, stone, mortar joints and interfaces are globally represented by single continuous elements (Masonry as an one-phase material).

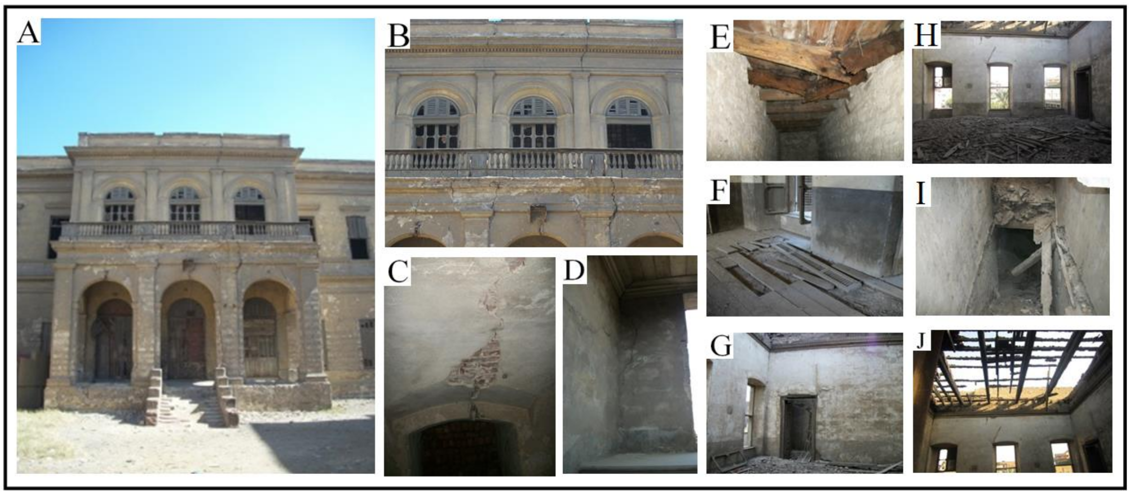

Fig. 5. Structural deficiencies and Cracking pattern of Omar Toson palace

The behaviour of the masonry wall is replicated by use of Solid65 elements (three-dimensional eight noded isoparametric elements) and its stiffness is modified by the crack and crushing development; Shell143 elements (isoparametric two-dimensional element with four nodal points) have been used to model the vaults in basement. After meshing, the final 3D model consisted of 22,887 joints and 75,952 elements that correspond to 73,512 degrees of freedom (Fig. 6). All the variations in wall thickness, geometrical irregularities and wall connections were taken into account, and the deterioration and damage states of the structural elements have been considered in the modelling phase. In both the static and dynamic analyses the models have been subjected to the vertical loads deriving from the masonry own weight and from the roof loads. The whole palace was analysed under the following load cases: static load case, modal analysis case (in order to determine the fundamental participation factors and natural frequencies for the palace which will be used for the dynamic analysis), and time-history case to evaluate the seismic behaviour and resulting internal stresses under the dynamic actions.

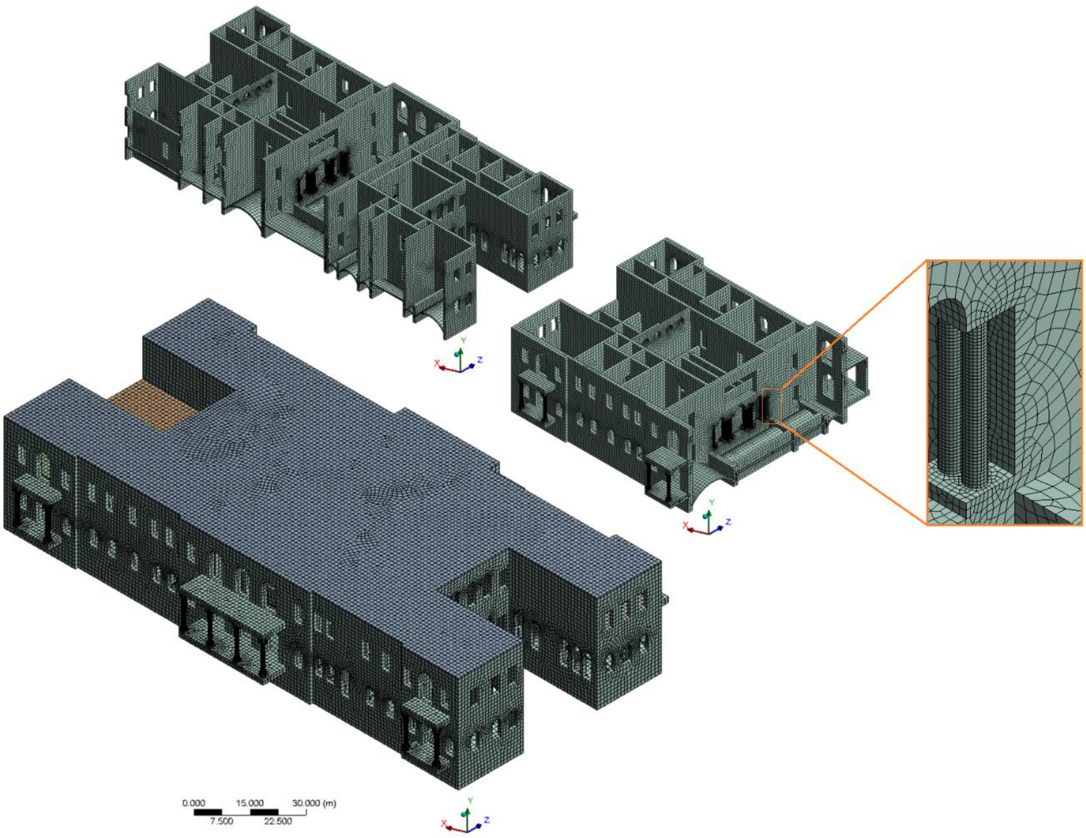

Fig. 6. Geometrical model and finite element discretization of the Palace in its current state

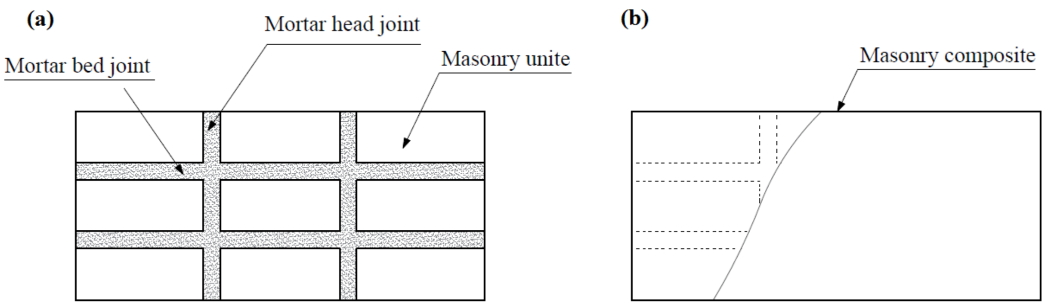

Fig. 7. Macro-modelling strategy for masonry structures. (A) masonry sample, (B) macro-modelling technique

4. STATIC STRUCTURAL ANALYSIS

The static structural behaviour of the palace has been analysed in the nonlinear range under constant vertical loads deriving from the own weight of masonry walls and from the roof loads, beside the differential settlement of supports in order to determine the various forces and actions that caused the structural deficiencies in the whole structure. The main goals in the static analysis is to solve the equilibrium conditions, compatibility and to analyse the structure reactions, stresses, strains and displacements when it is subjected to its own weight to identify the weak points of potential failure. Results of the static analyses on the 3D palace model are reported in Fig. 8 and 9.

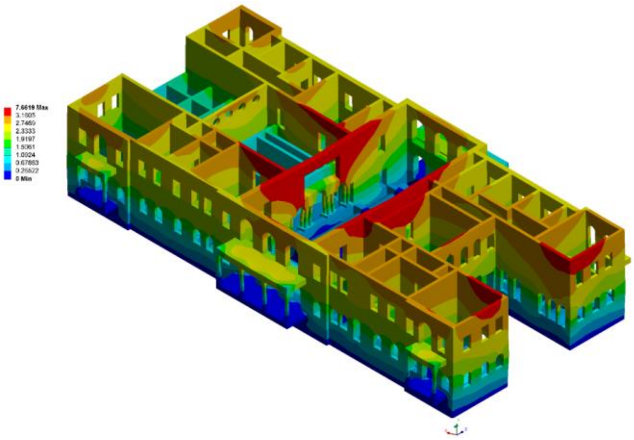

Fig. 8. Vertical displacement (mm)

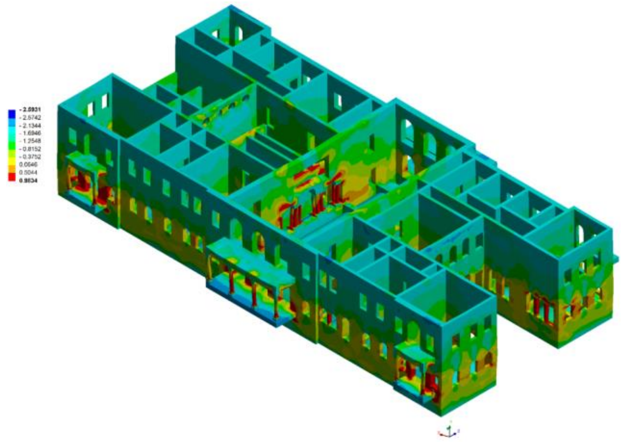

Fig. 9. Vertical stress state σ_y (N/mm^2): global view

The obtained results (deformed shape and stress distribution) give realistic values. The total weight of the entire palace is 269427 KN, The structure is almost everywhere compressed and only in a few points the analysis detects the presence of tensile stresses, the maximum value of tensile stress appears on the top surface of the main walls, due to the timber roof loads but this is mainly a local numerical effect depending on the punctual connection between beam and solid elements. The maximum value of compressive stresses ranged from (1.5) to (2.0) N/mm2. These results confirm that the palace is adequate to withstand vertical static loads in a very safe and stable way; this is a quite common result for historical masonry structures because its structural system (bearing wall system) is perfectly able to transfer vertical compressive static loads.

5. MODAL ANALYSIS

The 3D model was investigated for the modal eigenvalue analysis, to determine their modal natural frequencies and the fundamental participation factors. The first 100 modal shapes of the palace have been evaluated with the aim to assure that the total effective modal mass of the model is at least 90% of the actual mass. It has been found that the 90% of the total mass is accounted for by using the first 71 modes.

The first mode of vibration acts along the weakest transversal direction (Z-direction) involving a rotation about X-direction (Fig. 10-A). This mode has a participating mass ratio of about 36.76%. The second mode involves translation in longitudinal direction with out-of-plane deformation of the orthogonal elements, with a participating mass ratio of about 31.37% (Fig. 10-B). The first frequency (Z-direction) is about 5.4% more than the second one (X direction); although this difference is relatively low, it could be evidences a tendency of the building to be more deformable in direction Z that is parallel to the shorter side. While the third modal shape displays torsional deformations (Fig. 10-C).

The remained modes are less important, due to the lower mass contribution and basically excite the structure in various torsional modes. The distribution of the modal shapes demonstrates that the palace displays low transversal and torsional stiffness, with significant out-of-plane deformations of the orthogonal structural elements. Table 4 reports the natural frequencies and effective masses for the first 10 vibration modes in the transversal, longitudinal and vertical direction.

6. SEISMIC ANALYSIS

Historical unreinforced masonry buildings are perfectly able to transfer vertical compressive static loads and its own weight in a very safe and stable way; it mainly consists of vertical load-bearing elements designed primarily to resist in-plane compressive forces, while they are rather sensitive, from a structural point of view, and not adequate to withstand horizontal loads deriving from seismic actions. The high seismic vulnerability of this type of building is due to both the specific mechanical properties of masonry materials (highly non-linear behaviour and very small tensile strength) and the particular configuration of the buildings itself that are characterized by an open plan layout often with perimeter slender walls (Tzamtzis and Asteris, 2004; Betti and Vignoli, 2008; Betti and Vignoli, 2011).

As already stated in the premise, the analysis of the seismic behaviour of the URM buildings is a quite difficult task. So that it is necessary to start from the assessment of the seismic performance of URM buildings, the determination of mechanical and physical properties of construction materials, the characterization of the most vulnerable elements of the structure, and the identification of the possible mechanisms of failure.

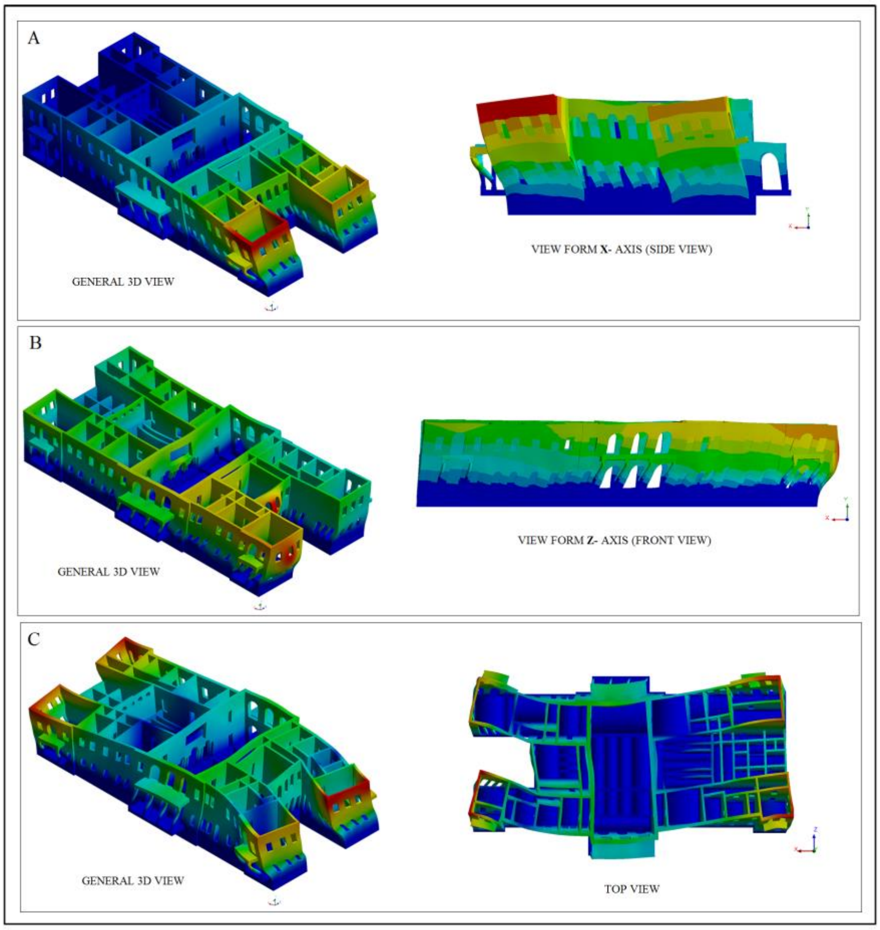

Fig. 10. The three primarily modes of vibration of the palace: A) Rotational about X-direction; f1 = 3.44 Hz, B) Translation in longitudinal direction; f2 = 3.87Hz, C) Torsional deformations; f3 = 4.0Hz

The seismic loads have been evaluated according to the Egyptian code (ECP, 2012). For the case study the response spectrum has been assumed with a soil type B. The following expressions have been used to evaluate the elastic response spectrum:

$$
\begin{aligned}
&0 \le T \le T_B && \Rightarrow S_e(T) = a_g . \gamma_t . S . \left[1 + \frac{T}{T_B}(2.5_\eta - 1)\right] \\
&T_B \le T \le T_C && \Rightarrow S_e(T) = 2.5 . a_g . \gamma_t . S . \eta \\
&T_C \le T \le T_D && \Rightarrow S_e(T) = 2.5 . a_g . \gamma_t . S . \eta . \left[\frac{T_C}{T}\right] \\
&T_B \le T \le 4_{sec.} && \Rightarrow S_e(T) = a_g . \gamma_t . S . \eta . \left[\frac{T_C . T_D}{T^2}\right]
\end{aligned}
\quad (5)
$$

In Eq. (5), *Se(T)* is the elastic response spectrum; T is the fundamental period of vibration of the building for lateral motion in the direction considered; T_B is the lower limit of the period of the constant spectral acceleration branch; T_C is the upper limit of the period of the constant spectral acceleration branch; T_D is the value defining the beginning of the constant displacement response range; a_g is the design ground acceleration for the reference return period; γ is the importance factor of the building; η is the design damping correction factor for the horizontal elastic response spectrum where a reference value of $\eta = 1$ corresponds to a normal 5% viscous damping ratio, according to (ECP, 2012). The peak ground acceleration (PGA) for the reference return period is denoted by a_g and for the case study it is equal to 0.12g according to the ECP. S is a factor depending on the ground type that in this case (soil type B) is assumed equal to 1.35.

Although nonlinear TH analysis is very time consuming, it is an accurate and reliable analysis to predict all possible failure mechanisms due to seismic actions, as a result of its capability to evidence both

Table 3. Elastic parameters, Yield criterion and Failure surface of main element

	Walls	Columns	Vaults
E_m (Young's modulus)	1460 N/mm^2	1500 N/mm^2	1900 N/mm^2
ν (Poisson's ratio)	0.2	0.2	0.2
γ_m (Specific weight)	2300 kg/m^3	2400 kg/m^3	1600 kg/m^3
c (Cohesion)	0.1 N/mm^2	0.1 N/mm^2	0.1 N/mm^2
η (Flow angle)	15°	15°	15°
φ (Friction angle)	38°	38°	38°
f_c (Uniaxial compressive strength)	8.67 N/mm^2	9.47 N/mm^2	9.32 N/mm^2
f_t (Uniaxial tensile strength)	0.86 N/mm^2	0.94 N/mm^2	0.93 N/mm^2
β_c (shear transfer coefficient across close crack)	0.75	0.75	0.75
β_t (shear transfer coefficient across open crack)	0.15	0.15	0.15

in-plane and out-of-plane, local and global failure mechanisms of the structures. In this paper, a TH analysis was performed to assess the seismic vulnerability of Omart Toson palace. In the current analysis the vertical loads combined with seismic horizontal actions along the transversal and longitudinal directions are considered.

Nonlinear dynamic analysis was carried out using an artificial accelerogram compatible with the elastic response spectrum of the Egyptian codes (ECP). The corresponding acceleration history applied at the basis of the building, having a total duration of 15 sec., is visualized in Fig. 11. The seismic behaviour of the palace, the seismic horizontal actions along the transversal (Z) and longitudinal (X) directions are considered. The TH analysis results show that the most critical direction is the transversal direction, the damage is mainly concentrated in the structural elements in the south- western parts exhibit major problems of stability, due to excessive damage and consequent exhaustion of the material strength. This result was expected due to the remarkable out-of-plane deformations of the orthogonal structural elements in the transversal direction. The main façade shows diagonal and vertical cracks, the most damage is concentrated in the openings and near the springers of the arches. The unsafe horizontal stresses of the TH analysis have reached the most of the cracks in the palace structure. The maximum lateral displacements in transversal direction reached from 21 to 34mm. In Fig. 12 a and b the displacements' time histories in transversal and longitudinal direction of the considered control points shown, respectively. While Fig. 13 reports the obtained base shear and *Fig. 14* plots the base shears vs. the displacement of the considered control points in both transversal and longitudinal direction. While Fig. 15 a and b report the principle stresses distribution under the time history cases in transversal and longitudinal direction respectively. As expected, the Top point (located on the top of the main façade) presented largest displacements. With the exclusion of the initial transitory, the base shear oscillates at the principal frequency corresponding to the first fundamental frequency of the structure, the same at which the displacement of the considered points oscillates, and at the frequency of the second vibration mode of the structure. Therefore, while the displacement is dominated by the first modal shape, the base shear is also conditioned by the effects of the higher modes.

Table 4. Natural frequencies and mass participation ratio for transversal, longitudinal and vertical direction

Mode no.	F (Hz)	Z- D (transversal) Meff (%)	X-D (longitudinal) Meff (%)	Y-D (Vertical) Meff (%)
1	3.44744	36.76	0.127	0.00
2	3.87416	0.35	31.37	0.002
3	4.00956	29.57	21.45	0.010
4	4.08532	14.26	0.022	0.000
5	4.57087	1.02	0.636	0.003
6	4.89072	0.29	2.504	0.000
7	5.15817	2.16	0.736	2.031
8	5.22415	0.00	4.729	0.014
9	5.3887	2.02	3.112	0.074
10	5.71642	0.00	0.127	0.003

Depending on the non-linear behaviour of the palace synthesized by the relationship between the base shear, that is the resultant of the horizontal forces, and the displacement of the considered control points of the building, the palace showed different capacities regarding the direction of the seismic loads. The location of hinges in the longitudinal direction obtained by the nonlinear TH analysis were clearly less damage than those obtained in the transversal direction. Besides, lower capacity was noticed in the transversal direction than that found in the longitudinal one, as the absolute maximum resisted load in the longitudinal direction was 0.218g, while in the transversal direction the absolute maximum resisted load was 0.1418g, which relatively was 11.7% and 39.2% less than those resisted in +X and –X, respectively.

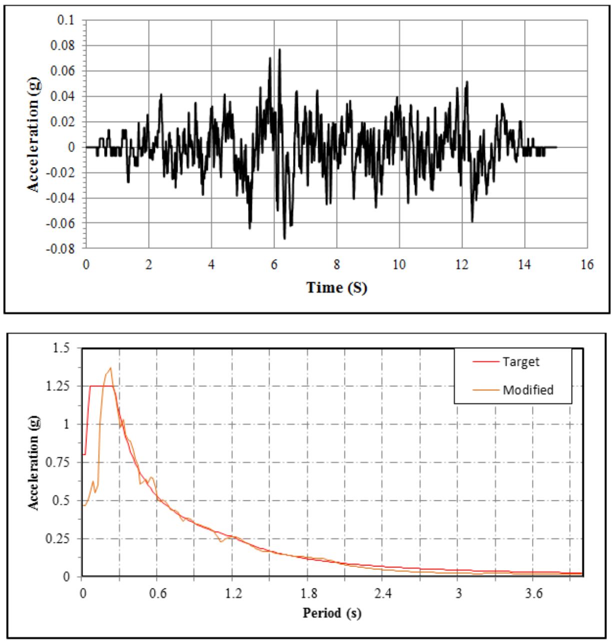

Fig. 11. Acceleration history artificially generated: A) accelerogram; B) Comparison between artificial time history and return period

In the transversal direction, the observed collapse mechanism for the seismic analysis was the overturning of the main facades. The damage concentrated around the openings and the springers of the arches as well as the top and the bases of the columns.

7. CONCLUSIONS AND RECOMMENDATIONS

The present paper contains the results of laboratory testing and numerical investigation to investigate the structural behaviour and seismic vulnerability of an important historical uninformed masonry structure, Omar Toson Palace in Egypt, in order to evaluate its structural performance and deficiencies under earthquake loading. The importance of considering the exact nonlinear and three dimensional behaviour of the masonry structure is shown, in order to determine all the structural deficiencies of the palace under seismic loads. For this purpose, 3D FE static and dynamic with time integration analysis of the overall structure have been done in order to correctly assess the structural behaviour and seismic vulnerability of the palace. The numerical model has been prepared according to the geometrical information gathered from the historical documents and by the measurement of the actual conditions of the structure.

The static structural behaviour of the palace has been analysed under constant vertical loads deriving from the own weight of masonry walls and from the roof loads. The static analysis shows that the structure is almost everywhere compressed and only in a few points the analysis detects the presence of tensile stresses.

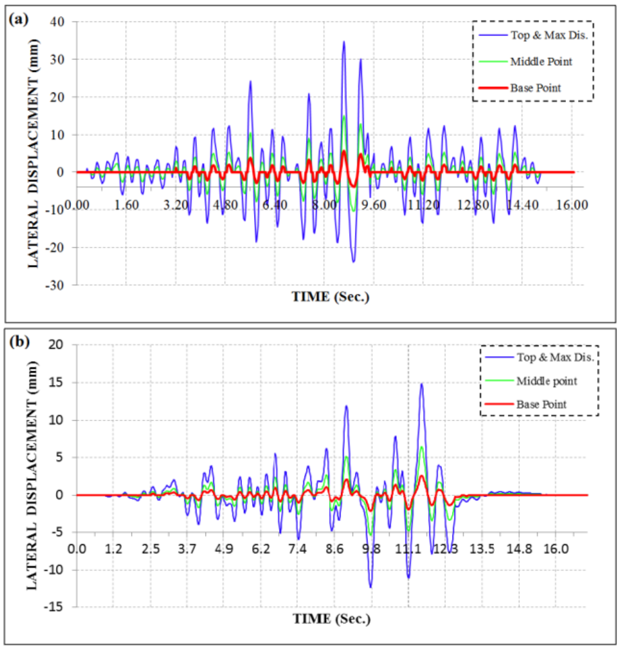

Fig. 12. Time histories of the displacements of the considered control points in (a) transversal and (b) longitudinal direction

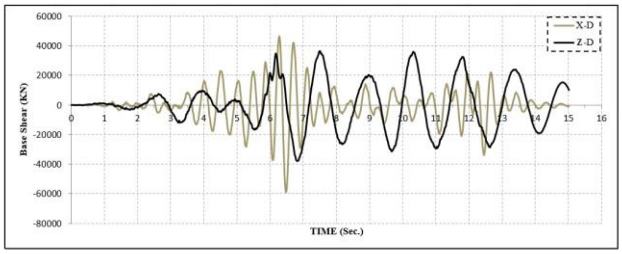

Fig. 13. Time histories of the obtained base shear in transversal and longitudinal direction

The seismic analysis has been evaluated according to the Egyptian codes (ECP-2008); the RS has been assumed with a soil type B and an artificial accelerogram compatible with the seismicity of the palace site have been used to performer the TH analysis. The non-linear seismic analysis prove that the seismic resistance of the palace in the transversal direction is lower than that in the longitudinal direction. This is due to the fact that in the weakest/shorter transversal direction, the orthogonal elements is highly exhibited to out-of-plane deformation due to applied horizontal seismic loads acting in ±Z direction results these elements tend to overturning about X direction. The most tension unsafe zones obtained by the present models are in the currently cracked sections. According to the analysis results, the dynamic behaviour of the palace is very sensitive to the structure damage and vulnerable to seismic actions.

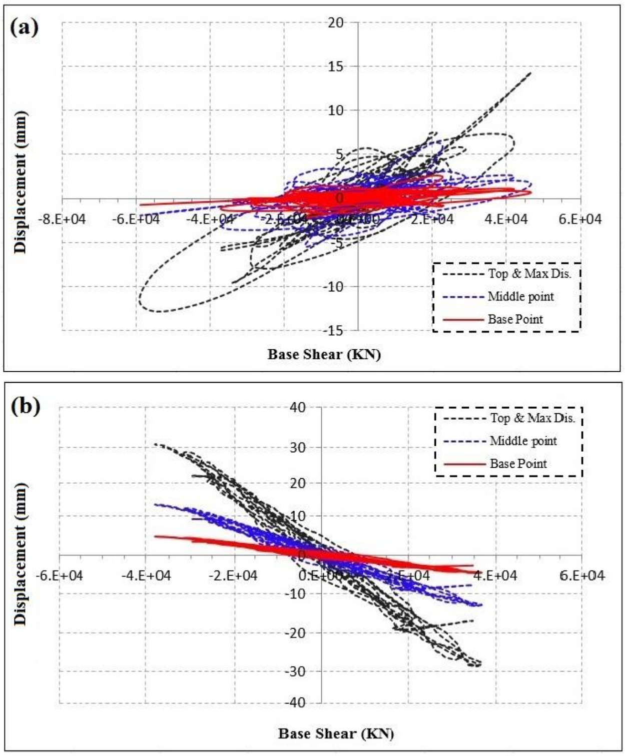

Fig. 14. Base shear vs. displacement curves of the considered control points in (a) transversal, (b) longitudinal direction

Therefore, scientific intervention methodology is necessary to be performed to reach a certain level of structural safety and to achieve successful conservation processes taking into account all criteria and requirements regarding preservation of the architectural value of the building.

The performed study underlines the importance of the nonlinear three-dimensional dynamic analysis in the evaluation of the actual seismic performance of a masonry structure in order to investigate all possible failure mechanisms under seismic loading. Besides, the examination of the final deformed configuration and stresses distribution helps to understand the global behaviour of the structural elements. As well as the accurate seismic assessment allows identifying essential and appropriate intervention strategies to increase the seismic capacity of the historic masonry structures. Additionally, the macro-element technique for modelling the nonlinear response of masonry structures is particularly efficient and suitable for the seismic performance investigation. The paper also proves that the nonlinear behaviour of masonry structures and the number of integration points both in-plane and in the thickness directions plays an important role for the accuracy of the results.

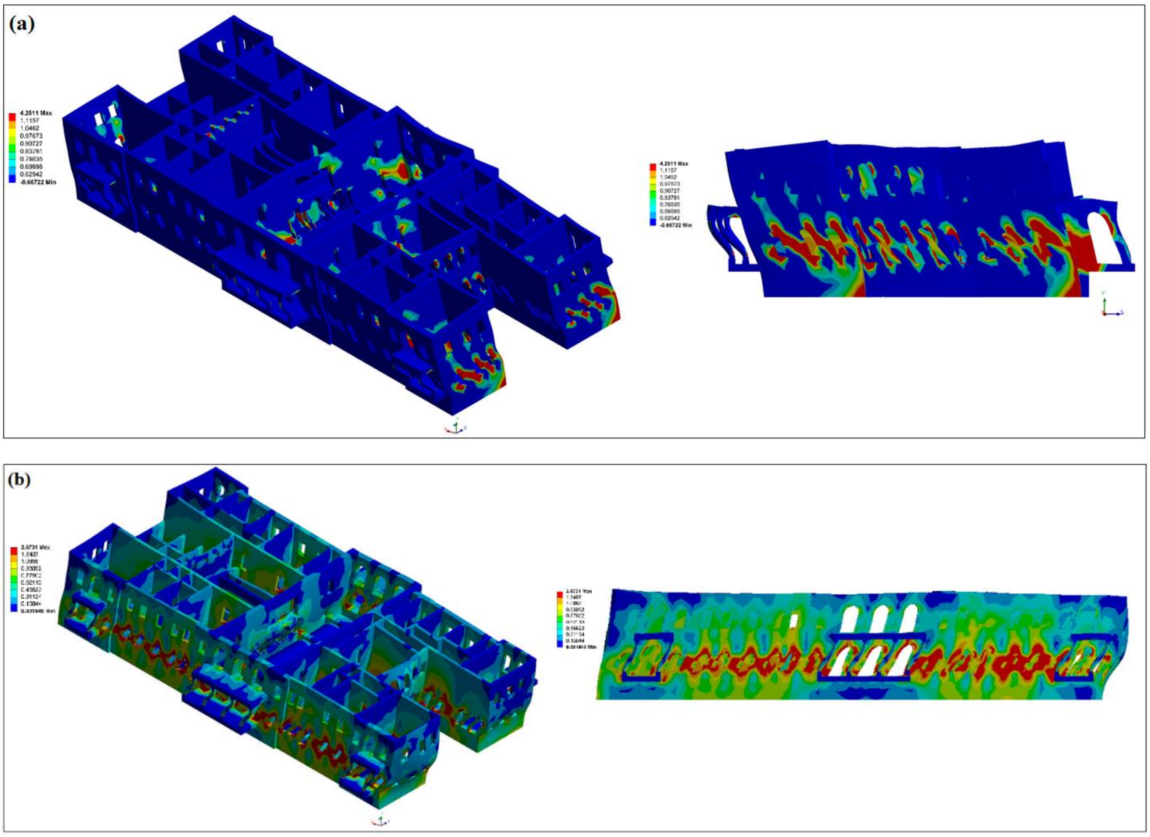

Fig. 15. Deformed configuration and principal stress distribution due to the artificial earthquake accelerogram in (a) transversal, (b) longitudinal direction (N/mm2)

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