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Research Paper

SEISMIC BEHAVIOR OF HERITAGE MASONRY STRUCTURES

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Seismic assessment of historical buildings is a complex problem due to the wide variety of involved aspects, such as the quality of masonry, the structural systems, the large effort in inspection and diagnosis, the economic and cultural implications. The dynamic behavior of block masonry minaret of a historical mosque in Cairo is analyzed, and a seismic assessment is proposed. Under seismicity of the Cairo region and Egyptian loading code, a 3D finite element model is used to determine lateral displacements and failure modes under seismic load. The analyses show that the highest damage usually occurs at the base and the lower part of the minaret. Wall-Hungarian eyes which is another a heritage structure is analyzed. The dynamic properties were evaluated using ambient vibration field measurements. The field measurements were used to update the finite element model analysis. The analysis based on updating finite element model was carried out to evaluate the seismic performance for this structure.

Keywords: Historic masonry structure, Minaret, Masonry building seismic assessment

INTRODUCTION

Earthquakes are one of the most dangerous factors of mechanical damage which affects buildings were severely damaged, as do many cities and buildings to ruins and ruins, severe earthquakes may be leading to the demolition of the building entirely, although sometimes lead to falling upper parts as walls, minarets, domes and terraces. Well established historically that the lighthouse of Alexandria, which was one of the seven wonders of the world was established in 280 BC, in the era of "Ptolemy II collapsed completely due to earthquake in 1303 CE during the reign of Sultan Al-Nasir Muhammad Ibn Qalawun", hit the Eastern Mediterranean destroyed the forts of Alexandria and its walls and its lighthouse. Therefore, we must maintain the vestiges in Egypt and try to study the current behavior under the influence of earthquake protection or restoration if needed. As Egypt is located in one of the moderate seismically active regions of the world, it is quite understandable that a considerable attention is given to the

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problem of seismic protection of historical heritage. Structural engineers always find the analysis and design of such structures quite challenging, especially because of highly complex behavior of materials these structures are formed of. The problem becomes even more complex when dynamic behavior is included in the analysis. The earthquakes that occurred in the Dahshour on October 12, 1992 (M_w= 5, 9) caused considerable casualties, damage and structural failure of various buildings, including many minarets and mosques. Much older historical structures also experienced different levels of damage during major earthquakes that occurred in a more distant past.

Significant results have been achieved in the study of mechanical behavior of historic (masonry) structures (Doherty *et al.*, 2002; Laurent *et al.,* 2008; Sezen *et al.,* 2008; Lourenco, 2006; Lourenco *et al.,* 2005; Turk and Cosgun, 2010), Dogangun *et al.,* 2008). These studies are crucial not only from the point of view of protection, but also for analysis of ground motion that occurred during past earthquakes. Within this framework, dynamic properties of old masonry structure, which usually exhibit vulnerable behavior under seismic load, are investigated.

A contribution to dynamic characterization and seismic assessment of medieval masonry structures is provided in a representative single case study (Pineda *et al.,* 2011), the Árchez tower, located in the active seismic area of Málaga, Spain. This study follows a multidisciplinary approach, in order to identify architectural, historical and structural features. The tower exhibits high vulnerability under seismic action, mainly due to its slenderness, low shear strength, low ductility and its possible lack of effective connections among structural elements. To assess its safety, transient and incremental static analyses are performed, aimed at predicting the seismic demand as well as obtaining the expected plastic mechanisms, the distribution of damage and the performance of the building under future earthquakes. A number of three-dimensional linear and non-linear finite element models with different levels of complexity and simplifications are developed, using 3-D solid elements, 3-D beams and macro-elements. All the models assume that the masonry structure is homogeneous, and the material non-linear behavior- including crushing and cracking is simulated by means of different constitutive models. Comparison among the different models are discussed, in particular as predicted local and global collapse mechanisms is concerned, to evaluate the suitability, accuracy and limitations of each analysis.

The static behavior and the seismic vulnerability of the Basilica of Santa Maria allImpruneta near Florence (Italy) have been evaluated using the finite element modelling technique (Michele and Andrea, 2011), where the nonlinear behavior of masonry has been taken into account by proper constitutive assumptions.

Complete 3D models of Un-Reinforced Masonry (URM) structures have been obtained assembling 2-nodes macro-elements (Alessandro *et al.*, 2004), representing the non-linear behavior of masonry panels and piers.

A finite element methodology for the static and dynamic nonlinear analysis of historical masonry structures is described and applied to the case study of a Romanesque masonry church (Michele and Andrea, 2008). A quasi-static approach (the seismic coefficient method) for the evaluation of the seismic loads has been used (as indeed is common in many analyses of the seismic behavior of masonry structures). The comparison demand vs. capacity confirms the susceptibility of this type of building to extensive damage and possibly to collapse, as frequently observed.

In the present study, two structures build with masonry in Cairo are presented in Figure 1 and Figure 2. The first is a wall - Hungarian eyes built by Sultan Al-Ghouri 800 years ago and was the target of the wall is to extend Citadel of Salah al-Din water by raising the waters of the Nile Balsoaki to the course of the fence, so that the water running to be up to the castle, because the castle was the seat of power in Egypt since the Ayyoub period and then moved the headquarters after to Abdeen Palace.

Figure 1: View of Wall-Hungarian Eyes (from 1798-2012)

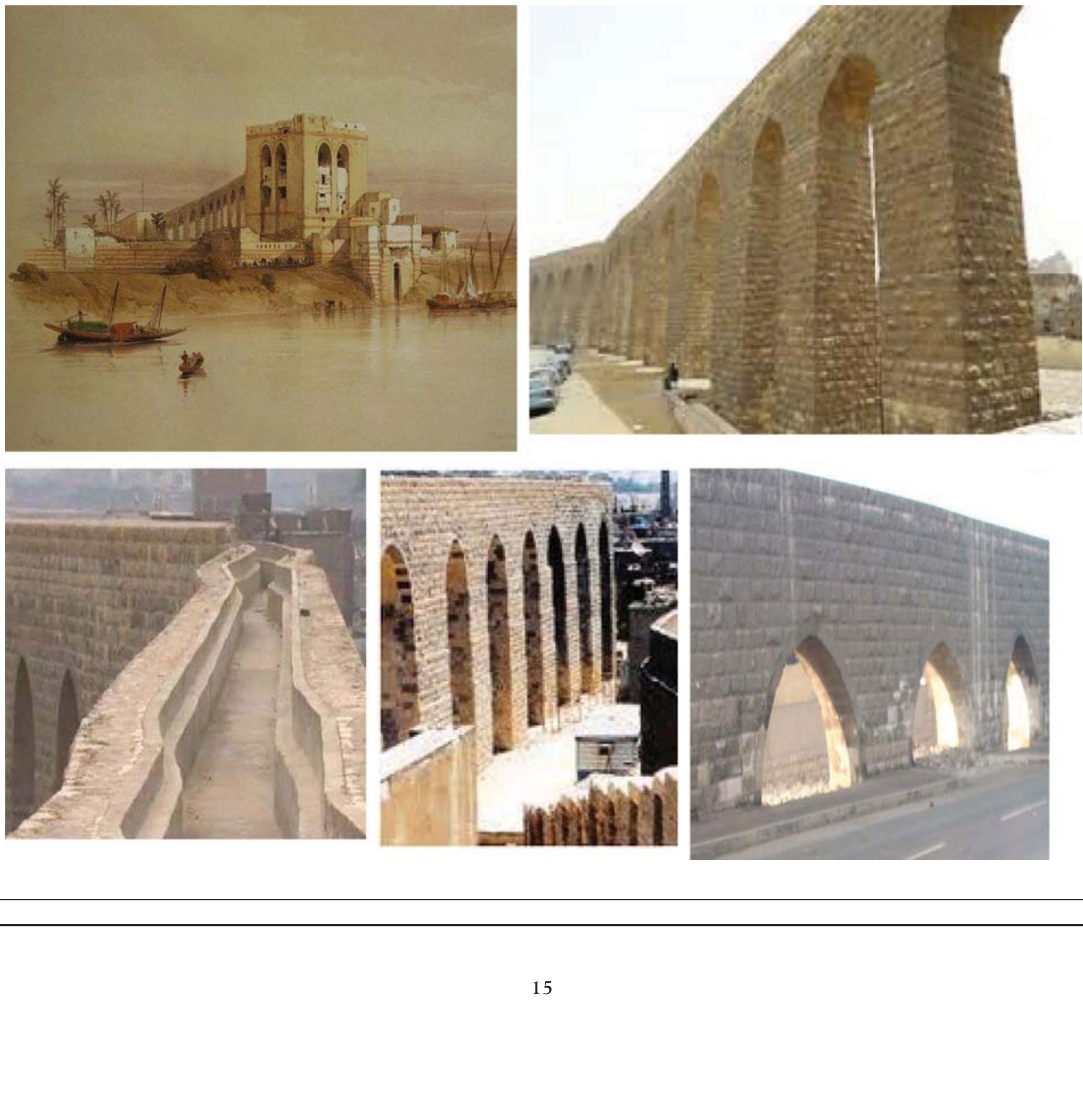
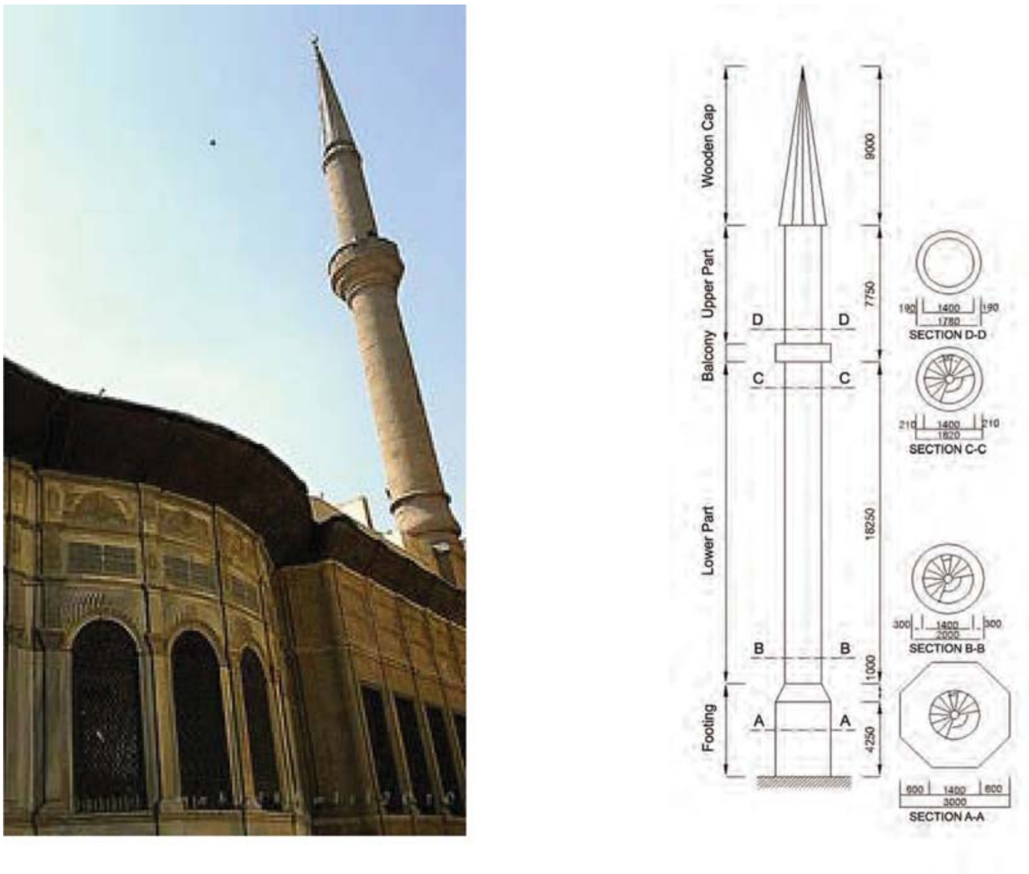


Figure 2: View of Solomon AGA Jagan Mosque Minarets with Their Main Features (dimensions in mm)



The length of the remaining portion of the Aqueduct of water about three kilometers away, and this is the Aqueduct water from the barrages of the most beautiful examples of water not only in Egypt but also in the whole Islamic world.

The second on is a minaret of Prince Suleiman AGA mosque Jagan which was constructed from the year 1250 Hijri, completed in the year 1255 Hijri and is one of the finest and rarest ancient mosques architecture make it the pearl of the region with Islamic antiquities, and is divided into three halls and a standpipe and a book to teach the Koran. The mosque is Locate left stepper towards Bab Al-futuh and minaret shape as other Ottoman cylindrical minarets and have one session and ending with the conical obelisk.

Experimental field dynamic measurements of these two structures are present. Results of the eigenvalue analysis of numerical models were compared to the natural frequencies extracted from the in situ measurements. The field measurements were used to update the finite element model analysis.

Numerous material tests were performed on limestone specimens taken from residues of old historical structures. Typical mechanical properties of the limestone are given in Table 1. Mechanical properties of limestone are: modulus of elasticity of un-cracked stone section E = 8856 MPa, Poisson ratio n = 0,24, and unit weight g =22 kN/m^3. While calculating the elastic modulus of the limestone material it was assumed that the elastic modulus to compressive strength ratio is E/f_c=720, where F_c=12.30 MPa (minimum compressive strength of the tested lime stone).

Table 1: Mechanical Properties of Limestone

Physical Properties	Max.	Min.	Average
Density (dry,kN/m^3)	25.0	22.8	23.9
Density (fullysaturated,kN/m3)	25.3	23.7	24.5
Uniaxial Compressive Strength (MPa)	19.2	12.3	16.7
Uniaxial Tensile Strength (MPa)	0.95	0.88	0.9
Modulus of Elasticity (GPa)	7.36	4.30	5.84

WALL-HUNGARIAN EYES

Descriptions of the Structure

The height of wall-Hungarian eyes ranging from 4.0 m to 22 m above adjacent road level next to the River Nile and then gradually to less than 4.0 m when intersect to Salah Salem Street. The wall was built of stone columns up through decades of stones. The column dimensions are approximately 2.5 x (2.5-3.0 m). The Foundation layer is weak soil due to the nature of the region and it's close to adjacent waterways. Due to clear difference in wall height, in different regions, leading to different stiffness and therefore the stability of the stone itself. Given the great wall length with more than 8 km, the highest wall part it is taken for study and evaluates field dynamic measurement as this part has less stiffness compared with the remaining parts of the wall.

Ambient Vibration Testing

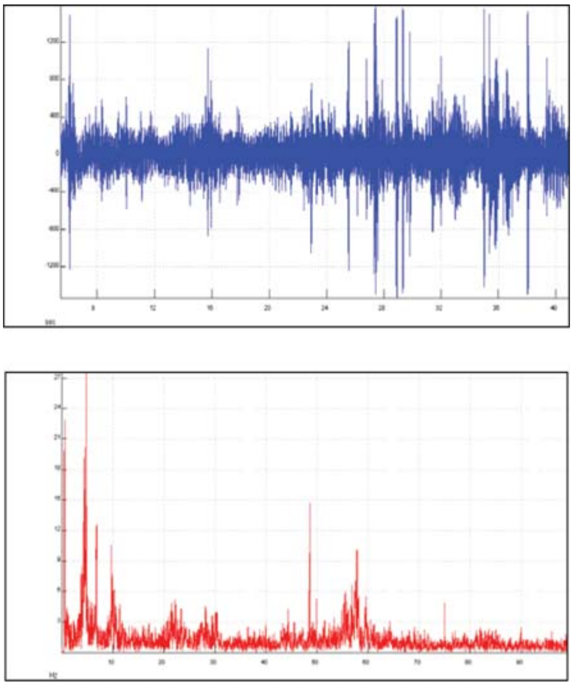
Ambient vibration tests were conducted on the Wall-Hungarian eyes at the beginning of July 2010 to measure the dynamic response in 6 different points, with the excitation being associated to environmental loads and traffic load from adjacent roads. The test was conducted using an 8 channel data acquisition system with uniaxial piezoelectric sensors (Figure 3); these sensors allowed acceleration or velocity responses to be recorded.

Figure 3: Accelerometers

Figure 4 shows the recorded time history obtained by ambient vibration test. Since the wall was repaired many times and difficulties arises from exciting of many cracks, the finite element model was updated by variation in mechanical properties of lime stone only based on mechanical field measurements. Then the Eigen problem was solved and the new natural frequencies for the lower mode shapes are compared with the experimental results in Tables 2.

From these results, it can be concluded that the Ambient Vibration tests (AVM) on historic structures, can provide valuable data for the validation of the detailed finite element models.

Figure 4: Some Recorded Data and Corresponding FFT



Dynamic Analysis of Structure and Evaluation of Results

The 3D finite element model, developed to study dynamic behavior of the minaret, is shown in Figure 5.

The fundamental natural frequencies of the structure are evaluated using original Finite Element Model (FEM). Then, FE model updated with the measured field ambient vibration test and the Eigen problem was solved and hence, the new natural frequencies for the lower mode shapes are compared with the experimental results in Tables 2. From these results, it can be concluded that the ambient vibration tests (AVM) on historic

Figure 5: 3D Finite Element Model

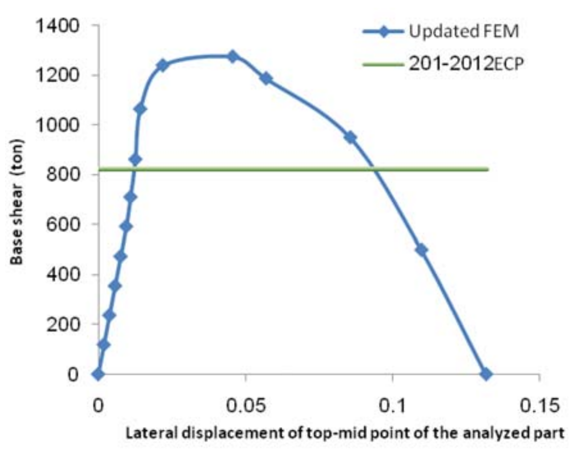
Table 2: Comparison of Wall-Hungarian Eyes Natural Frequencies

Vibration mode	Natural frequency (sec)		
	Experimental measured	FE model(without update)	UpdatedFE model
1	0.49	0.62	0.51
2	0.41	0.49	0.42
3	0.37	0.42	0.39
4	0.34	0.40	0.33
5	0.26	0.33	0.28

structures, have provided valuable data for the validation of the detailed finite element models. This helps to have accurate analysis which is essential for these types of structures.

The base shear-top lateral displacement relationships obtained by nonlinear analysis for Wall-Hungarian eyes are presented in Figure 6. In this figure, the base shear forces calculated according to Egyptian loading code (ECP201-2012) are also plotted. In comparison with the updated FEM model, the Wall-Hungarian eyes can resist safely the seismic loads as per ECP201-2012.

Figure 6: Base Shear-top Lateral Displacement Relationship



SOLOMON AGA JAGAN MOSQUE MINARET

The understanding of dynamic behavior of masonry minarets is of great significance for proper preservation and strengthening or retrofitting of historical monumental structures. Sezen *et al*. (2003, 2008) discuss vulnerability of and damage to 64 masonry and reinforced-concrete minarets after the 1999 Kocaeli and Duzce earthquakes, and investigate seismic response of reinforced-concrete minarets. In Dogangun *et al*. (2008) analyzed and evaluated behavior of reinforced masonry minarets subjected to dynamic earthquake load. A finite element model of the minaret based on shell elements was prepared using the SAP2000 (2009) software.

In this study, the dynamic behaviour of a representative minaret made of natural block stone is investigated using finite element software [Sap2000]. The structure is modelled and examined using the response spectrum analysis described in the ECP201-2012 loading code .

Description of Minaret Structure

As shown in Figure 2, the 40.25 m high minaret is formed of the following segments: base (4.25 m), transition segment (1 m), cylindrical body (26 m), and wooden cap (9 m). The outer diameter and thickness of the cylinder differ in each part of the minaret. The minaret wall thickness is 30 cm at the lower part, and this thickness gradually comes down to 21 cm at the upper part of the minaret, i.e., the thickness reduces by 1 cm for each 2 m of the height. At the base, the outer diameter and thickness of the wall are 3 m and 0.8 m, respectively, while in the lower part they are 2 m and 0.3 m, in the balcony part 1.8 m and 0.2 m and, finally, in the upper part 1.78 m and 0.19 m, respectively. The minaret footing is made of very thick stone blocks, and is connected with the exterior wall of the mosque. It can be identified as a slender cantilever structure. The lower part, from bottom to the gallery, is formed of the wall envelope, stairs and the core. In this segment, the thickness of the masonry wall decreases along the height. The interior of the upper part, from gallery to the top of minaret, is empty and has no practical use. Here, the wall thickness is constant along the entire height. Balconies are mostly used for prayers, which create a mass concentration along the minaret's height, and affect its dynamic structural response. This part is narrower when compared to the bottom part of the minaret. The conical top of the minaret is made of zinc-coated timber (Figure 2).

Dynamic Analysis of Structure and Evaluation of Results

The 3D finite element model, developed to study dynamic behavior of the minaret, is shown in Figure 7. The model includes spiral stone-made stairs, which are fixed to external minaret walls. The dead load of the wooden cap (upper part of minaret) is uniformly distributed along the top of minaret wall.

As to boundary conditions, the base of the minaret is considered as a fixed support. As can be seen in Figure 1, the base part of the minaret is connected with the thick external wall of the mosque, which is why no soil-structure interaction, nor rotation of minaret base, were taken into account. The linear elastic material behavior is assumed in the structural model, while the changes in stiffness are neglected. It is assumed that minaret is located in a zone 3 seismic region (0.15 g [14]) with weak soil class "C".

The behavior factor q (similar to factor R) for masonry tower walls according to Eurocode 8-1998 (2006), Part 6, Annex E amounts to 1.5. The 2% damping ratio is assumed for the dynamical analysis of such structures. The second order effect (P-delta) is ignored in the analysis. Dynamic analysis of the minaret model is carried out using the response spectrum defined in ECP201-2012.

The first five modal periods of the minaret model (determined through modal analysis), and contribution of individual modes to dynamic response, are presented in Table 2. The fundamental period of the minaret obtained through modal analysis is very similar to that obtained through ambient vibration measurements.

The first four modes greatly contribute to the overall response, with the first one participating with as many as 34% in the total response. The torsional mode (5th mode) has practically no effect on the response of the minaret. Micro tremors caused by ambient vibrations were measured on the minaret structure, and the fundamental frequency of 0.95 Hz and the period of 1.053 s were obtained. In this study the first period amounted to 1.125 s (Table 3). The about 6% difference can be assumed as negligible for practical design purposes.

Lateral displacements at the top of the minaret, calculated during the response spectrum analysis, are shown in Figure 8. The maximum calculated displacement was 213 mm for the design spectrum corresponding to Zone 3 type soil conditions (C). The deflected shape of the minaret points to mostly lateral flexural deformations, with largest displacement calculated at the roof. The height of minaret roof amounts to 31.25 m, not including the wooden cap. Although the minaret acts as a cantilever structure, the deformation is smaller over the height of a relatively stiff 4.25 m high base. Displacements start to increase above the transition segment at about 4.95 m in height.

Table 3: First Five Modes and Their Participation Factors

Modes	**1**	**2**	**3**	**4**	**5**
FEM Period (s)	1.125	1.125	0.198	0.196	0.099
Measured Period (s)	1.053	1.051	0.205	0.204	0.853
Modal participation factor(%)	28.0	28.0	12.0	12.0	2.0

The calculated roof drift index (d/h) amounts to 0,0068, which is less than the value of 0,002 specified in [SAP 2009] as maximum roof drift ratio for building structures. In FEMA 273 guidelines, 0.4 % limit drift (bed joint sliding behavior) is proposed for preventing collapses of unreinforced masonry walls (for walls made of hollow or solid bricks and clay/concrete units).

It can be seen from previous studies investigating causes of post-earthquake failure of minarets that most of masonry minarets usually fail at the bottom part of the cylindrical body, just above the transition zone (Turk 2010). For the minaret considered in this study, the maximum stress values determined through FEM analysis amount to 11.67 MPa (compression), 6.20 MPa (tension) and 0.72 MPa (shear), as shown in Figures 9 and 10. Tensile stresses greatly exceed the tensile strength capacity of the limestone, the maximum capacity of which is estimated at 1.0 MPa. High tensile stresses mainly occur in lower segments of the minaret, which is why its structure is highly susceptible to seismic load. In addition, in terms of roof drift, the design value of 0.0068 (213 mm of roof displacement) is higher when compared to similar types of slender masonry structures. Under these circumstances, it is clear that historic structures of this type are greatly vulnerable to strong seismic action. On the other hand, the seismic resistance of such structures additionally decreases because of complex behaviour of stone material, and due to interaction between stone blocks. Furthermore, when conducting any structural intervention of this kind, it is highly significant to preserve initial appearance of the structure.

Figure 7: 3D Minaret Model with the Close-up View of the Balcony

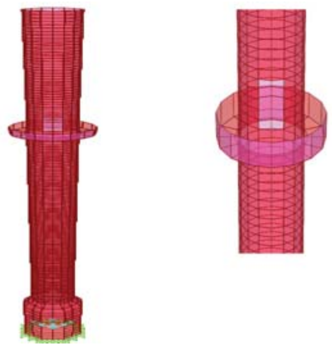


Figure 8: Deflected Shape and Lateral Displacement over the Minaret Height (in mm)

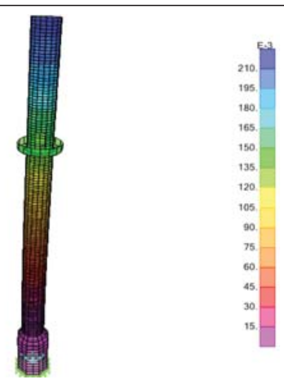


Figure 9: Axial Stress Distribution over the Minaret Height (in MPa)

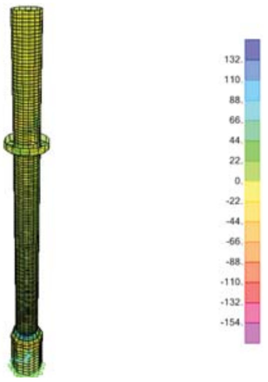
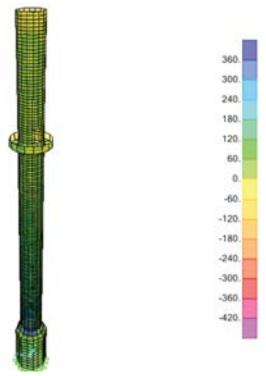


Figure 10: Shear Stress Distribution Along the Minaret Height (MPa)



The pushover curves for the studied minaret in x-directions, the demand and capacity spectra in y direction, are shown in Figures 11 and 12, respectively.

CONCLUSION

This paper presents possible failure modes for a typical historic structures located in Cairo. The results obtained from ambient vibrations, and previous material tests cited in the paper, show that the behaviour under seismic action can be predicted quite accurately, and that the mentioned measurements can be used in the assessment of these structures.

Figure 11: Base shear-top Displacement Curve

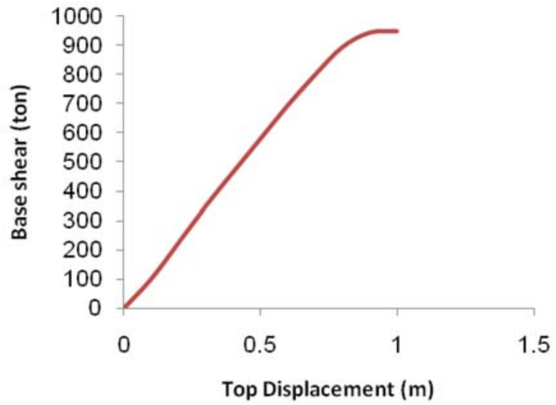
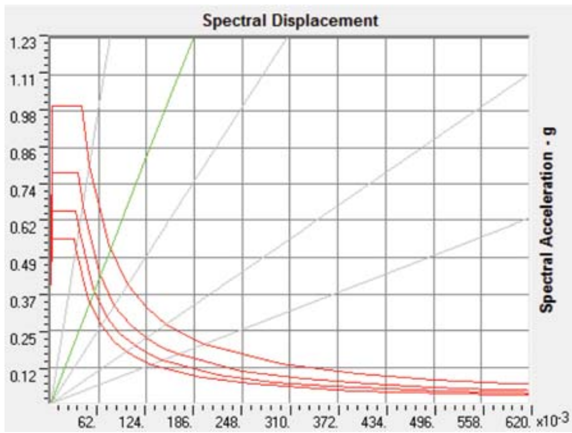


Figure 12: Capacity Spectrum as ATC-40



The 3D analysis presented in the paper enables assessment of structural behaviour of heritage structures subjected to seismic action, with determination of failure mode, and definition of possible failure zones. Additional investigations should be undertaken to determine those design response spectra that are particularly adapted to this type of structures. More realistic values of R factor (seismic reduction factor) and damping ratio should be studied both experimentally and analytically.

The results obtained may be used for making retrofitting decision and for solving problems relating to seismic protection of these and of similar historic structures.

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