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Research Article

ESTIMATION OF SEISMIC DAMAGE PROPAGATION IN A HISTORICAL MASONRY MINARET

Emre ERCAN[1]*, Bengi ARISOY[2], Emin HÖKELEKLİ[3], Ayhan NUHOĞLU[4]

[1] *Department of Civil Engineering, Ege University, Bornova-IZMIR;* ORCID:0000-0001-9325-8534
[2] *Department of Civil Engineering, Ege University, Bornova-IZMIR;* ORCID:0000-0002-2785-0609
[3] *Department of Civil Engineering, Bartın University, BARTIN;* ORCID:0000-0003-0548-5214
[4] *Department of Civil Engineering, Ege University, Bornova-IZMIR;* ORCID:0000-0001-5147-460

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ABSTRACT

This study aims to investigate the seismic damage propagation in historical stone/brick masonry minaret which was built in 1894 under different ground motions by using calibrated finite element model based on Operational Modal Analysis. Surveying measurement and material tests were conducted to obtain 3D solid model and mechanical properties of the components of the masonry Minaret. First, the initial 3D finite element model of the minaret was analyzed and numerical dynamic characteristics of the minaret were obtained. Then operational modal analysis as an ambient vibration test was employed in order to obtain the experimental dynamic characteristics of the minaret. The initial finite element model of the minaret was calibrated by using the experimental dynamic results. Finally, linear and nonlinear time history analyses of the updated finite element model of the minaret were carried out using the acceleration records of two different earthquakes that occurred in Turkey. Concrete Damage Plasticity model was considered in the nonlinear analyses. The analyses indicate that the results of the linear analyses are not as realistic as the non-linear analysis results. According to the nonlinear analysis, earthquake would damage some parts of the elements in the transition segment of the minaret.
Keywords: Masonry minaret; Operational modal analysis; Ambient Vibration Testing; Damage Estimation; Concrete damage plasticity model.

1. INTRODUCTION

Historical structures are the most valuable items of the cultural heritage. Anatolian Peninsula has been the homeland for many civilizations since early ages so it harbors many historical structures that were built in different periods by different civilizations with different techniques. The historical minarets are tower like structures, and like many other historical structures, they are masonry structures in which brick/stone and mortar are used together. Minarets are more vulnerable to natural disasters than others because they have a slender geometry and they are easily affected by wind and earthquakes. As a means to maintain these structures and convey

* Corresponding Author: e-mail: emre.ercan@ege.edu.tr, tel: (232) 311 15 86

them to future generations, various attempts have been made to fix the damages in historical structures.

Large proportio n of historical buildings in Anatolia is located in earthquake regions, and these structures are frequently exposed to moderate and severe earthquakes. For this reason, the protection and evaluation of the structural safety of historical masonry structures have become necessary. Ambient vibration testing and the finite element method are promising techniques for safety and damage evaluation of masonry structures [1], [2]. Many researchers have performed in situ Operational Modal Analysis (OMA) tests to historical towers and minarets and compared the frequency results with initial finite element (FE) model of the same structures to update the FE models [2-8]. Livaoğlu et al. [6], as one of these researchers, studied especially on Ottoman type minarets and obtained numerical correlation between the first mode period and geometric properties, such as height, cross section and boundary conditions. [2], performed a linear time history analysis on the updated FE model of İskenderpasa historical minaret. According to analysis results, the maximum and minimum principle stresses occurred at the intersection region of the transition segment and the cylindrical body of the minaret [2]. Fragonara et al. [8], conducted a study on the bell-tower of S. Maria Maggiorein Mirandola after 2012 Emilia earthquake in Italy. Two ambient vibration tests were performed immediately after the earthquake, and after provisional retrofitting. The authors observed an increase in natural frequencies after provisional retrofitting which verifies success of provisional retrofitting. Altunışık et al. [5], performed linear and nonlinear FE earthquake analyses of Zağanos Bastion in Trabzon, Turkey, but the authors advised to update the initial FE models using experimental testing methods such as the ambient vibration test and reanalyze the structure by updated FE model which represents the actual behavior of the structure for an accurate result. In conclusion, the model updating procedure is confirmed to be an effective tool for investigating the behavior of structures. Furthermore, some researchers aimed to analyze the nonlinear dynamic and static behavior under earthquake loads for structural assessment of structures. In order to predict the nonlinear behavior of masonry these researchers use Concrete Damage Plasticity (CDP) material model which exhibits softening under compression and tension stress [9-12]. Unlike the other studies in the literature, this study focuses on the estimation of seismic damage propagation in historical masonry minarets under earthquakes using a calibrated FE model based on ambient vibration data. Rahmanlar minaret in the city of İzmir, Turkey was selected as the subject of application. Material properties of the minaret were determined by the material tests and FE model was developed using ABAQUS software [13-14]. OMA was performed to update the initial finite element model [14]. Linear and non-linear time history analyses of the updated finite element model of the minaret were carried out using the acceleration records of two different level earthquakes that occurred in, Turkey, İzmir (2012) and Düzce (1999) and nonlinear analysis were performed using Concrete Damage Plasticity model. Seismic damage propagation patterns of the stone masonry minaret were obtained for both earthquake ground motion levels. Düzce earthquake analysis results show that the tensile plastic strain values on the masonry components exceed the critical values at the joint of body and transition segment which is a common failure for most of the minarets.

2. GEOMETRIC AND MECHANICAL PROPERTIES OF RAHMANLAR MINARET

2.1. History of the minaret

Rahmanlar minaret is the only remnant of Hamidiye Mosque which was built in 1894 by Sultan Abdulhamit II. It is located on the 73. km of Tire-Izmir road, Turkey. The mosque was demolished when the road was redirected, and only the minaret survived. Nowadays, the minaret is called as Rahmanlar minaret. The total height of the minaret is 14.5m. Hexagonal base of the minaret is 2m high, and it is made of cut stone. The transition segment contains an entrance which is placed above the base with a conic form. It is 2m high and is coarse stone/brick masonry (Fig.1). The minaret itself was built in brick masonry style and the height from the base to the minaret balcony is 7m. The part above the minaret balcony is 3.5m high. The diameter of the minaret is 1.55m. The stairwa ys of the minaret are helicoid and the steps are in classical form.

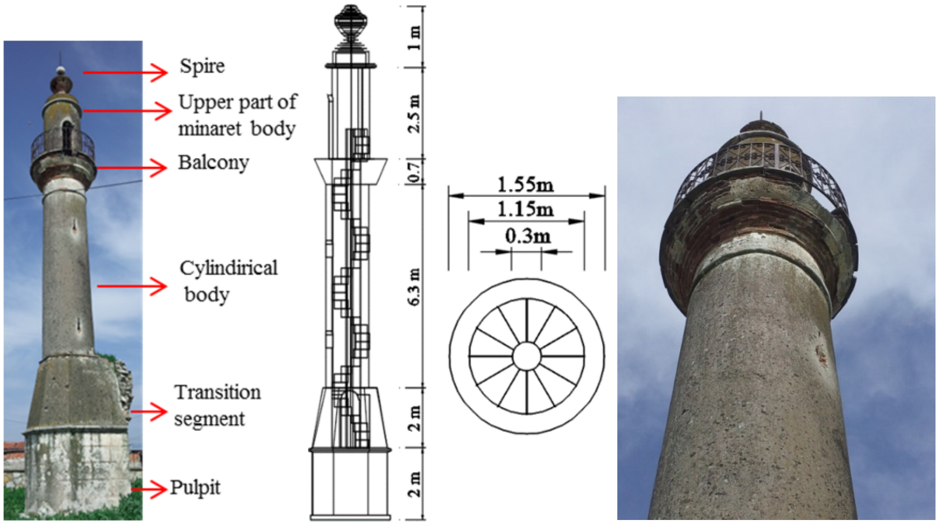


Figure 1. Rahmanlar Minaret [14]

2.2. Material properties of the minaret

The minaret has a stone masonry base and brick masonry body. Since it is considered as a historical heritage, core sampling for the tests was not allowed. Some of the fallen stones were taken and tested with density-porosity and load point tests. L and LB type Proceq Schmidt Hammers were used to detect the surface hardness values (rebound value) of the stone and clay brick samples, respectively as shown in Fig. 2. The material properties were determined using density porosity tests and rebound values [15-16]. Collected properties are given in Table 1.

Figure 2. Schmidt hammer test [14]

Table 1. Material properties of the stone, brick and mortar according to nondestructive tests [14]

Materials constitutive masonry	Bulk density (kg/m^3)	Schmidt Hammer test surface hardness values	Compressive strength (MPa)	Tensile strength (MPa)
Stone	3020	47.2	200	20
Brick	1730	21.5	14	1.4
Mortar	1920	-	7	0.7

Masonry is considered as a composite material that consists of different elements, namely brick or/and stone units and mortar joints. During the implementation of the homogenization method, the stone/brick and mortar were assumed to behave as a mono block therefore the overall behavior of the composite media was taken into account [17]. The minaret has three types of masonry: cut stone, coarse stone and clay brick masonry. The material properties were determined by Equations described in Eurocode 6, Kocak 1999 and Lorenço 2004 [18-20] and in Eq. 1and 2. Material properties calculated are given in Table 2.

Modulus of elasticity of the masonry unit is calculated as follows:

$$E = \frac{t_m + t_u}{\frac{t_m}{E_m} + \frac{t_u}{E_u}} \rho \tag{1}$$

where t_m is average thickness of the mortar, t_u is average thickness of the stone block or masonry brick, E_m is the modulus of elasticity of mortar, E_u is the modulus of elasticity of the stone block or masonry brick, ρ is an efficiency factor associated with the deficient bond between two materials, for this study it is assumed to be 0.5.

Compressive strength of the masonry unit is calculated as follows:

$$f_k = 0.5 f_b^{0.65} f_m^{0.25} \tag{2}$$

where f_m and f_b are the compressive strength of the stone block and masonry brick, respectively. Tensile strength of the wall units is calculated 10% of compressive strength.

Table 2. Initial Material Parameters of Masonry Walls according to homogeneous technique

	Compressive strength (MPa)	Tensile strength (MPa)	Modulus of elasticity (MPa)	Bulk density (kg/m^3)	Poisson's ratio
Brick masonry	5.42	0.54	5420	1750	0.20
Cut stone masonry	30.6	3.06	10000	2500	0.17
Coarse stone masonry	15.6	1.50	7000	2000	0.17
Stairs	30.6	3.06	30600	2500	0.20

3. INITIAL FINITE ELEMENT MODEL OF THE MINARET

Three-dimensional solid and finite element models of the minaret were prepared with ABAQUS [13] software using in situ survey results (Fig. 3). Convergence analysis was conducted for the purpose of determining the most appropriate range of mesh in the finite element model of the minaret. The coverage graphics is given in Fig. 4. According to the convergence analysis, the range of the mesh in numerical analysis was chosen as 0.1 m. A total of 208290 four-node tetrahedral (C3D4) solid elements and 499896 nodes were used for the initial finite element model. The modal analysis was carried out and the first five numerical frequency values and mode shapes are presented in Fig. 5. The first and the fourth mode shapes were in the x direction whereas the second and the third mode shapes were in the y direction and the fifth mode shape was in the torsional type.

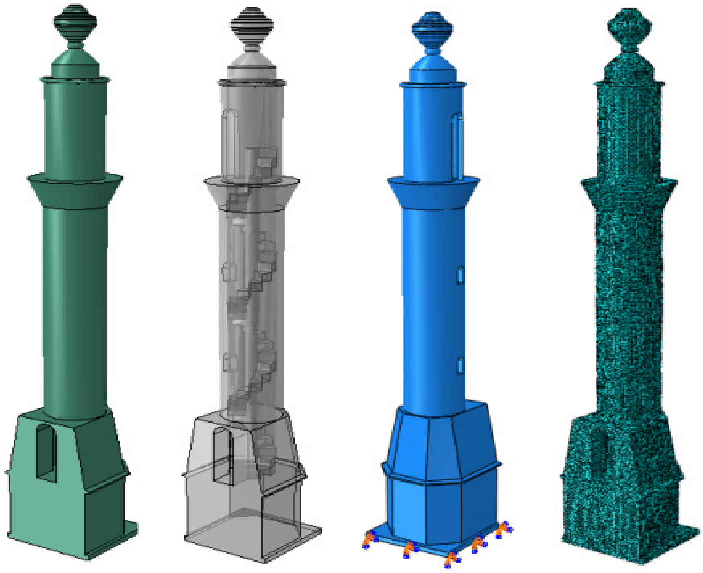


Figure 3. Three-dimensional solid model and finite element model of minaret

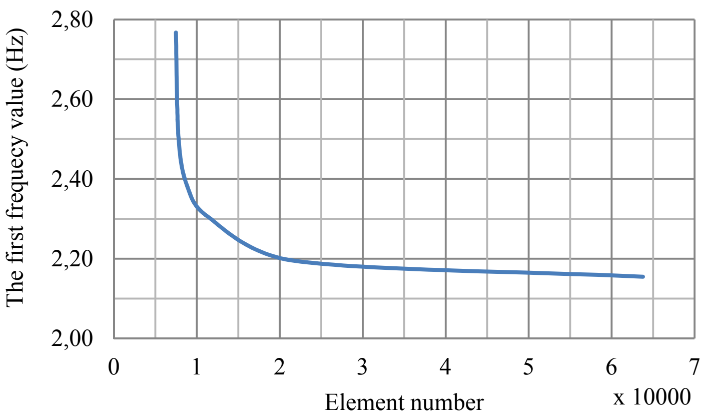


Figure 4. Frequency and mesh size convergence graphic

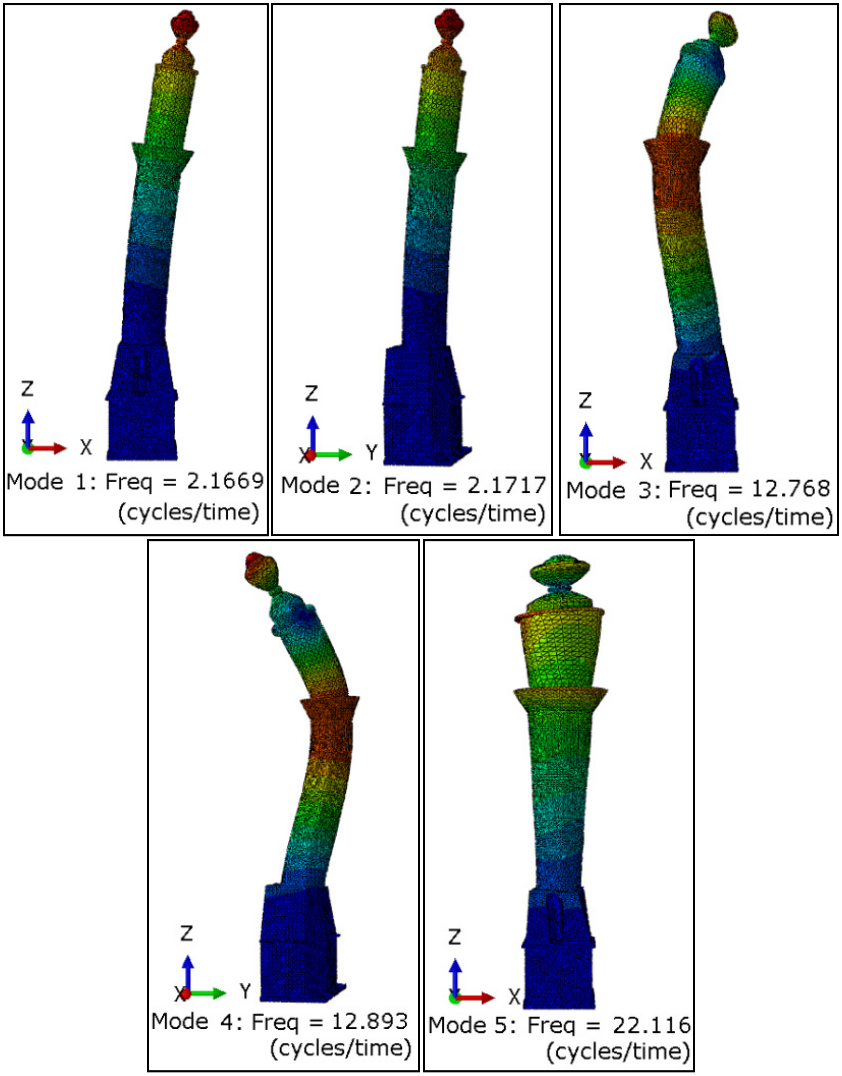


Figure 5. Mode shapes and frequency values from the initial finite element model

4. FE MODEL UPDATING OF THE MINARET VIA OMA

Operational Modal Analysis (OMA) is an output only method. In the OMA method, vibrations generated by environmental impacts on a structure are transferred to data collection devices by means of accelerometers placed on the structure and they are recorded accordingly. These vibration data are computerized for analysis and the dynamic modal parameters of the structure are obtained. In the study, Stochastic Subspace Identification (SSI) method is used in OMA for the determination of modal parameters. Since the SSI method uses time data, it requires no conversion throughout the processing steps [21-25].

In order to collect dynamic characteristics of the minaret and to decrease the measuring errors, three ambient vibration tests were conducted (Fig. 6). Total of 14 uniaxial accelerometers were used, their locations are shown in Fig. 7. Acceleration data acquired from accelerometers were gathered in a 16 channel data acquisition system Testbox 6501 (Fig. 7). Modal parameters were then extracted using ARTeMIS Modal Pro software by using the SSI method [26].

Figure 6. OMA test of the minaret.

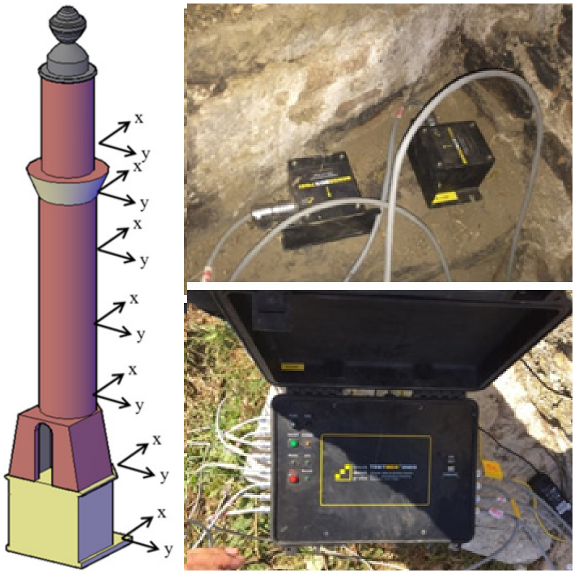


Figure 7. Locations and directions of the accelerometers on the minaret and data accusation system.

Stabilization diagram of the estimated state space models, obtained using the SSI technique, are presented in Fig. 8 [14]. This technique estimates the modal parameters directly from the raw measured time series. For this study, the number of modes was selected obtained to be two. Thus, the frequencies are read 1.673 Hz for mode 1, 1.715 Hz for mode 2, the mode shapes of the minaret gathered from testing is given in Fig. 9.

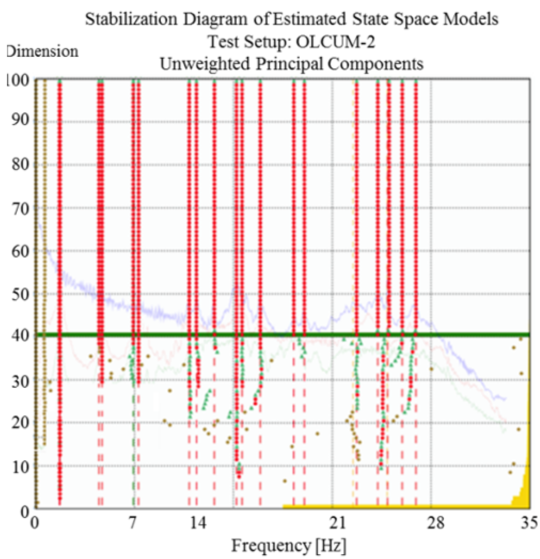


Figure 8. Stabilization diagram of estimated state space models obtained from the SSI technique [14]

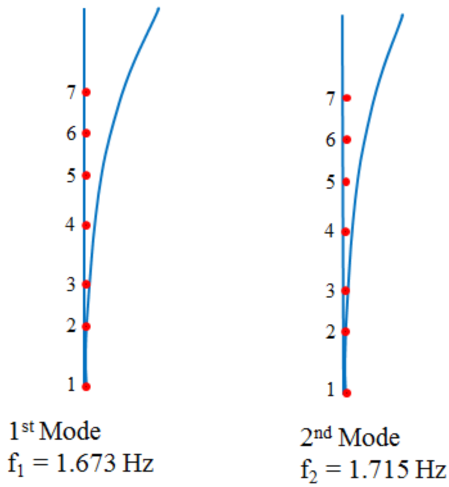


Figure 9. The mode shapes of the minaret created using OMA testing results

In Table 3, frequencies determined from FE analysis and OMA are exhibited. Frequencies obtained with the SSI method were found to be different from the ABAQUS [13] eigenvalue analysis results. The differences between the analytical and experimental results are described to some non-conclusive parameters of the masonry. Usually, upgrading is carried out by modifying the mass, stiffness parameters and boundary conditions of the FE model until an improved agreement between FE analysis data and test data are met. In this study, calibration of the FE model of the minaret is performed by calibrating all mentioned parameters. In order to meet frequencies collected from ambient vibration test for numeric analysis, FE analysis was repeated several times. Thus, calibrated FE model of the minaret is obtained.

The modulus of elasticity of the brick masonry, pulpit, transition element and stairs, and boundary conditions of the FE model are updated according to environmental vibration test results. Table 3 presents the first two numerical and experimental frequency values obtained before and after the model calibration of the minaret. The calibrated frequencies are close to the experimental frequencies. Hereby, the FE model which presents the actual behavior of the minaret was built.

Table 3. Comparison of numerical and experimental frequency values.

	Frequency [Hz]			Error (Ratio of differentiation)(%)	
Mode	FEM initial	FEM update	OMA EFFD	Before FE updating	After FE updating
1	2.167	1.679	1.673	29	0
2	2.171	1.694	1.715	27	1

Seismic behavior of the minaret did not changed regarding mode shapes but the frequency values. Since the minaret is tower like structure behaved as cantilever beam, the first mode shape would exhibit same form both un-calibrated and calibrated FEM model. On the other hand frequency values of the calibrated model are smaller than the un-calibrated model.

5. SEISMIC DAMAGE PATTERN ESTIMATION OF THE MINARET

5.1. Constitutive law for masonry

The seismic loading is considered to be a cyclic process and causes progressive damages on the structure because of its back and forth effect. Masonry is a load bearing structural system composed of brittle components, namely brick and/or stone and mortar. Since its all components are brittle, masonry is also brittle material, like concrete, with very low tensile stress and it is vulnerable to seismic loads. In previous studies, seismic response of concrete and the model was developed for concrete in order to model masonry for seismic response. Thus, Concrete Damage Plasticity (CDP) model was used to model masonry structure [27-28]. The mentioned model is suitable for nonlinear analysis for brittle materials and it is available in ABAQUS software [13]. In order to use CDP model, the compressive stress-strain curve for masonry should be provided. On the other hand, the subject structure is a historical structure, destructive tests were not performed, and accordingly necessary stress-strain curve was adopted from literature [29-31].
The nonlinear analyses were performed using the Concrete Damage Plasticity (CDP) model. Material parameters for masonry in CDP model are summarized in Table 4.

Table 4. Material properties adopted in the analysis

	Ultimate compressive strength (MPa)	Inelastic strain	Ultimate tensile strength (MPa)	Crack strain
Brick masonry	5.42	0.0022	0.54	0.0002
Cut stone masonry	30.6	0.0081	3.06	0.0008
Coarse stone masonry	15.6	0.0053	1.50	0.0005
Stairs	30.6	0.0037	3.06	0.0003

The other parameters used for masonry in CDP are dilation angle was taken 10°, flow potential eccentricity was taken 0.1, ratio of initial equi-biaxial compressive yield stress to initial uni-axial compressive yield stress was taken 1.16, ratio of second stress invariant was taken 0.667, viscosity parameter was taken 0 [9].

The acceleration records of the earthquakes that took place in Çay-Sultandağı and Düzce were used to determine the seismic damage patterns of Rahmanlar Minaret. Time histories of two components of the earthquakes are depicted in Fig. 10. The Çay-Sultandağı (Mw 6.0) and Düzce (Mw 7.2) earthquakes occurred in 2002 and 1999, respectively [32]. The N-S and S-E acceleration records were applied to the minaret in x-x and y-y horizontal directions, respectively.

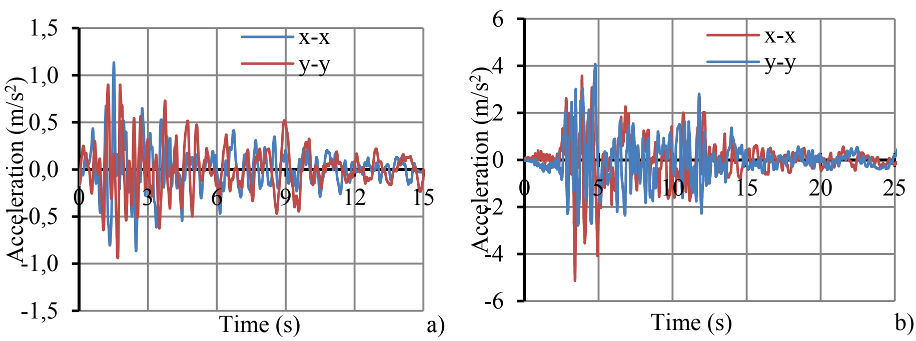
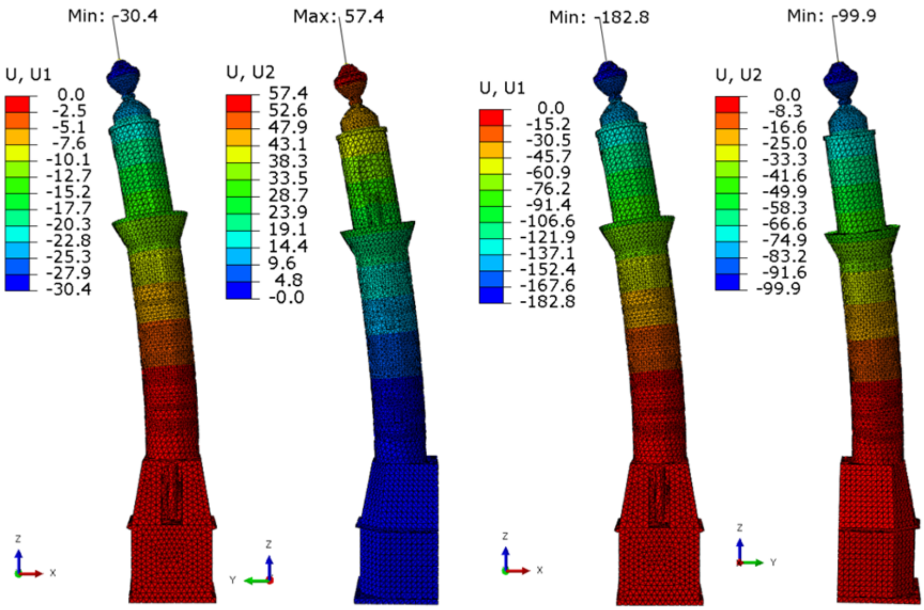


Figure 10. (a) Çay-Sultandağı, **(b)** Düzce earthquake acceleration records [32]

6. LINEAR ANALYSIS OF MINARET UNDER MANISA AND DÜZCE EARTHQUAKES

The maximum lateral displacement values in U1 (x) and U2 (y) directions are 3.04 cm and 5.74 cm under Çay-sultandağı earthquake, and 18.28 cm and 9.99 cm under Düzce earthquake, respectively (Fig. 11). The lateral displacement values throughout the height of the minaret under linear analysis for Çay-Sultandağı and Düzce earthquakes are presented in Fig. 12.



Çay-Sultandağ earthquake Düzce earthquake

Figure 11. Maximum displacement contour shapes under the linear time history analyses (mm)

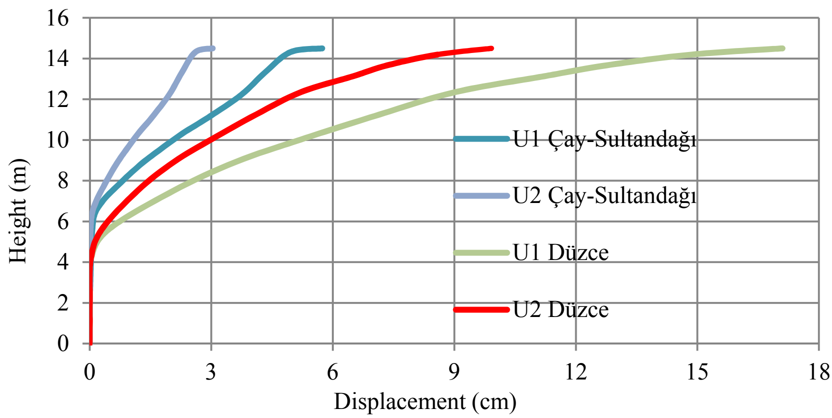


Figure 12. Comparison of lateral displacements throughout the height for linear analyses

Under Çay-Sultandağı earthquake, it was observed that the maximum principal stress was concentrated around the entrance region in the stone core of the stairs while the minimum principal stress was concentrated on the opposite side of the stone core (Fig. 13). Likewise, under Düzce earthquake the maximum and minimum principal stresses occurred in the same location as shown in Fig. 14.

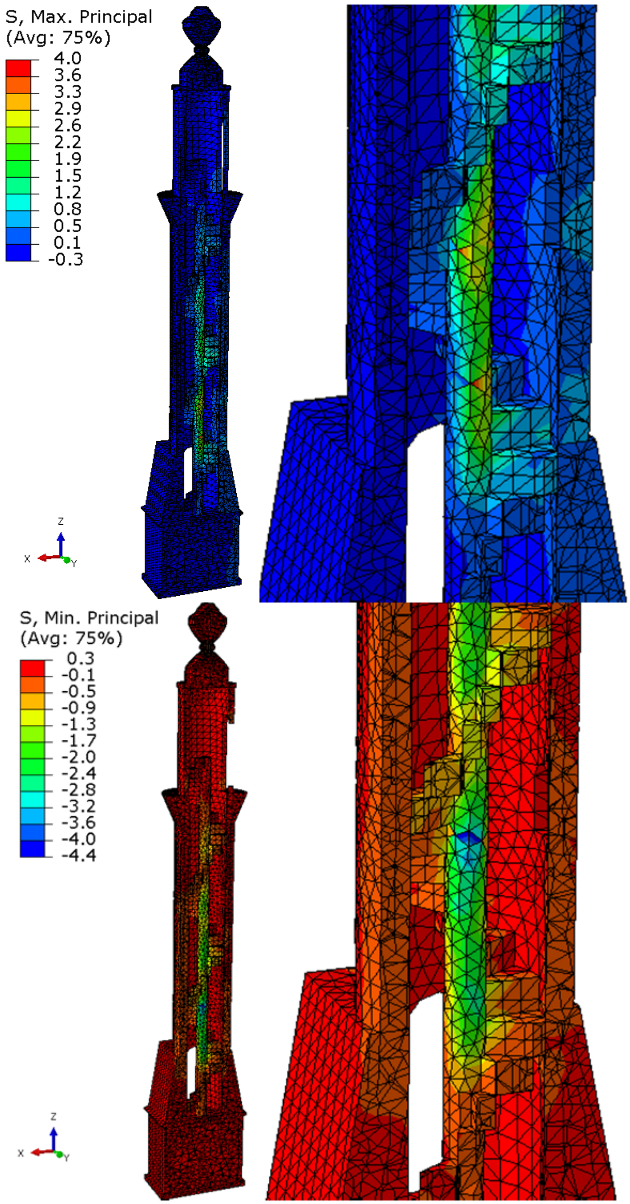


Figure 13. Principal stresses distribution on minarets under Çay-Sultandağı earthquake (MPa) (Linear analysis).

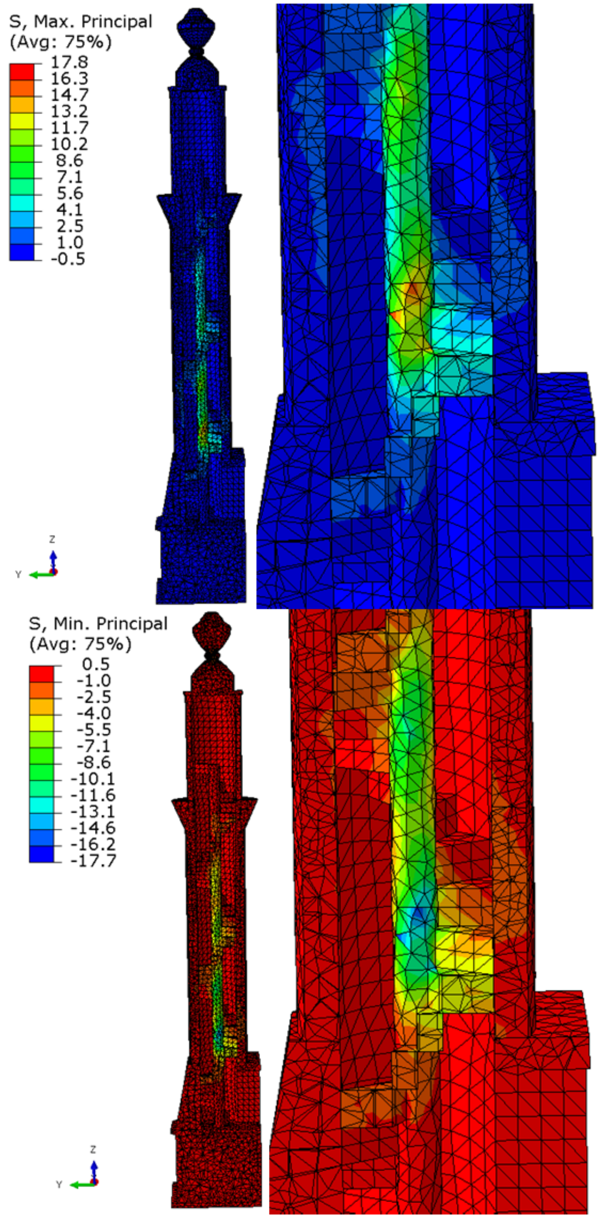


Figure 14. Principal stresses distribution on minarets under Düzce earthquake (MPa) (Linear analysis).

7. NONLINEAR ANALYSIS OF MINARET UNDER ÇAY-SULTANDAĞI AND DÜZCE EARTHQUAKES

According to nonlinear time history analysis results for Çay-Sultandağı and Düzce earthquakes, the maximum lateral displacement values is obtained as 4.46 cm in U2 direction and 10.86 cm in U1 direction, respectively (Fig. 15). The lateral displacement values throughout the height of the minaret under non-linear analysis for Çay-Sultandağı and Düzce earthquakes are presented in Fig. 16.

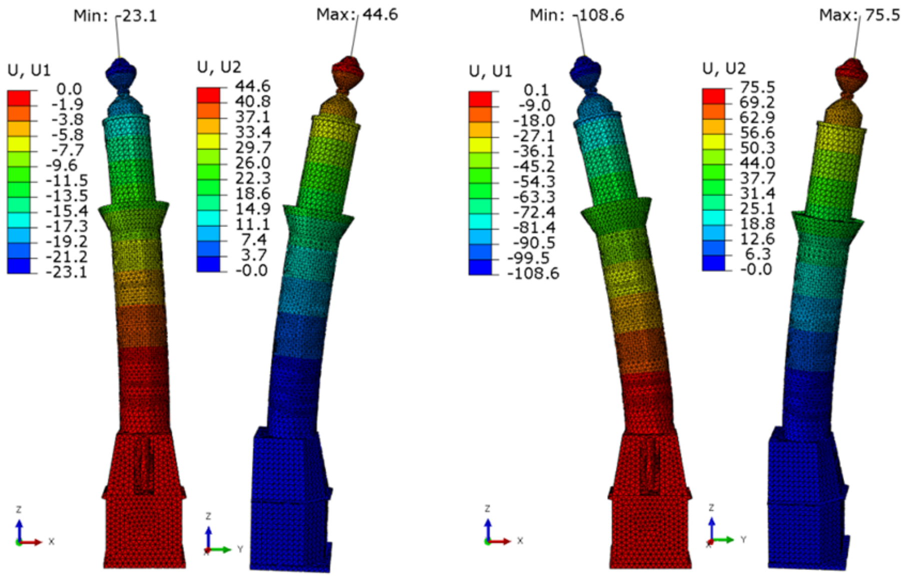


Figure 15. Time history contour graphs of displacements in the minaret earthquakes (mm) (Nonlinear).

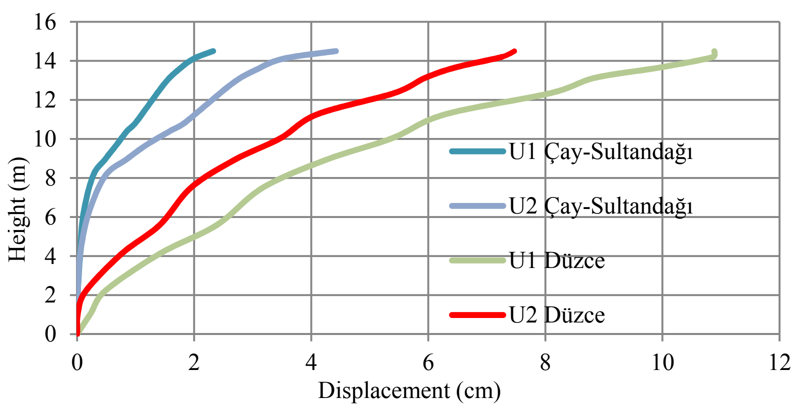


Figure 16. Comparison of lateral displacements throughout the height for linear analyses.

7.1. Çay-Sultandağı earthquake

Critical maximum and minimum principle stress contours under Çay-Sultandağı earthquake are presented in Fig. 17.

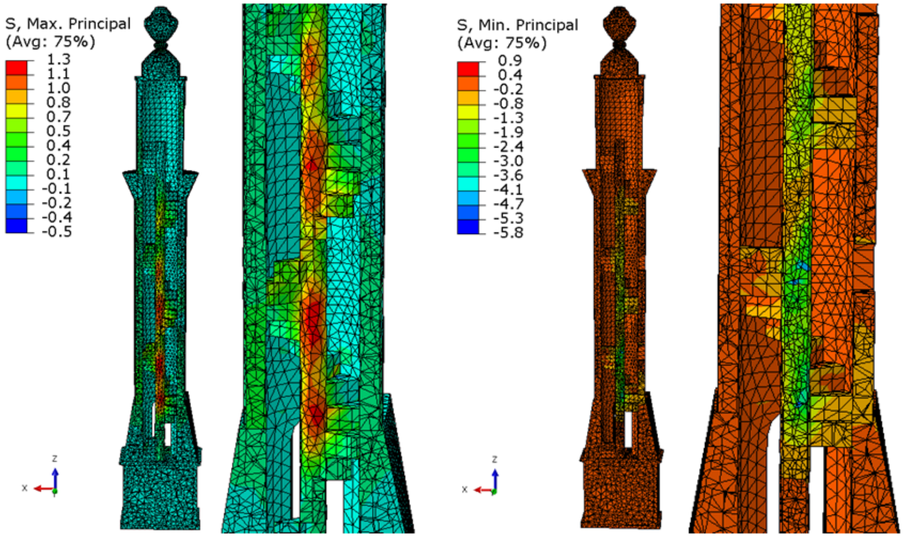


Figure 17. Maximum and minimum principal stresses in minaret under Çay-Sultandağı earthquake.

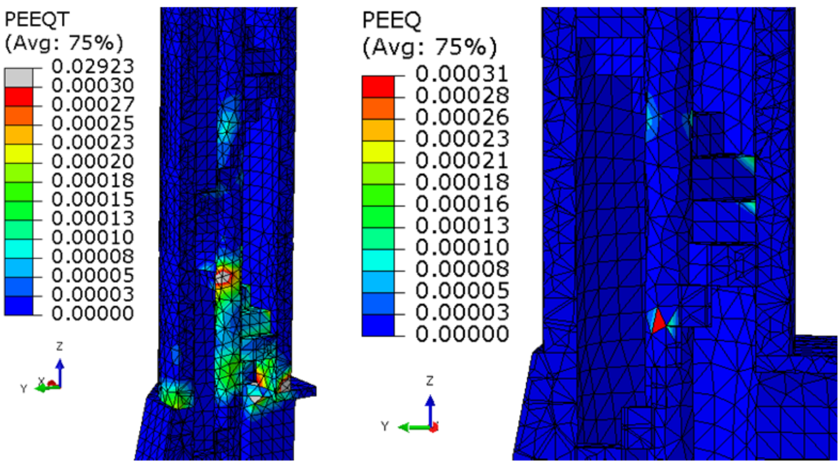


Figure 18. Tension and compression plastic strain contour for minaret under Çay-Sultandağı earthquake

Tension and compressive plastic strain on the minaret is calculated by CDP analysis (Fig. 18). Some part of the minaret, core of stairs has plastic strain 0.029 and exceeds the tension critical damage strain value of 0.0002 [33]. However the compressive plastic strain value of 0.00031 occurred on the opposite side of the core do not exceed the critical value of 0.0022. The masonry system can be considered safe under Çay-Sultandağı earthquake as the tension area covers only a small region.

7.2. Düzce earthquake

Critical maximum and minimum principle stress contours under Düzce earthquake are presented in Fig. 19.

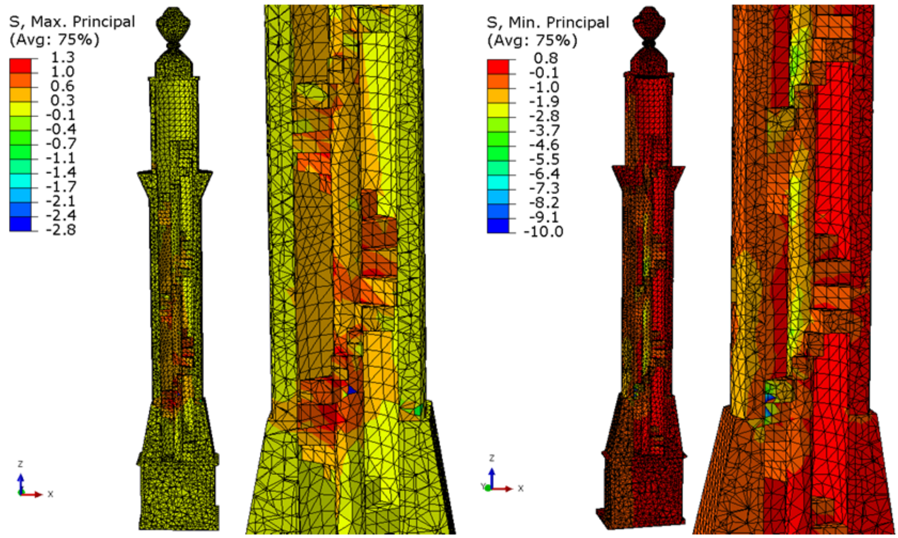


Figure 19. Maximum and minimum principal stresses in minaret under Düzce earthquake

The compression plastic deformation value reaches 0.0089 at the end of the analysis (Fig. 19). Only a small part exceeds the compressive plastic strain value of 0.0022 which is the critical damage strain value. This region (green locations) will be damaged, but damaged area is limited and the system can be considered safe. The tension plastic deformation value reaches to 0.117 at the end of the analysis (Fig. 20). Upper part of the transition segment and stone core of stair exceeds the value of 0.0002 which is the critical damage strain value. This critical region will be damaged for this reason the masonry system is not safe under Düzce earthquake.

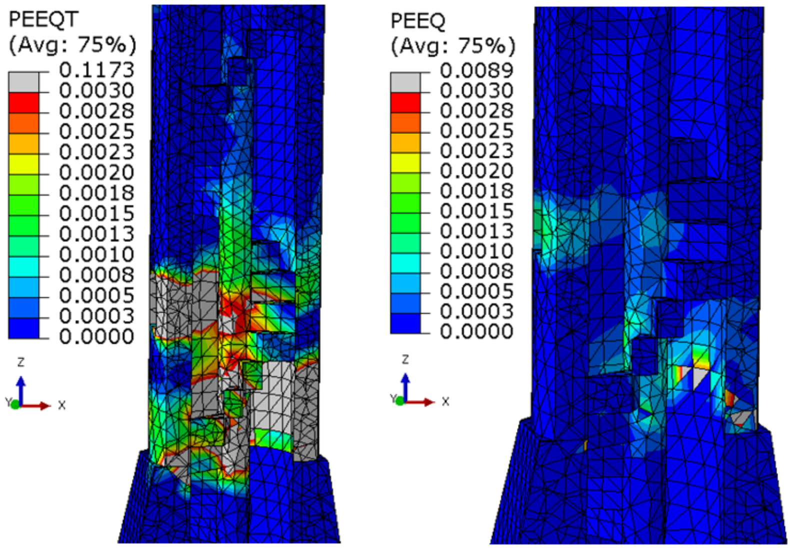


Figure 20. Tension and compressive plastic strain contour under Düzce earthquake

8. DISCUSSION

Both in linear and in nonlinear analysis, the largest tensile and compressive stresses in the structure occurred at the same regions. Besides, the expected damages in the analysis and the existing cracks overlapped as seen in Fig. 21. This result indicates that both methods, linear and nonlinear, are successful to determinate possible damage locations. Overlapping of existing damages and damages obtained from FE analysis indicates that calibrated FE model by OMA is realistic to exhibit actual behavior. The tensile stresses obtained from the linear analysis were found to be 13 times higher than the material strength for Çay earthquake, and 58 times higher than the material strength for Düzce earthquakes. On the other hand, the tensile stresses obtained from the nonlinear analysis were found to be only 4 times higher and 9 times higher than the material strength for Çay and Düzce earthquakes, respectively. It is seen that the minaret was damaged under the effect of Çay earthquake in linear FE analysis, whereas only few members passed critical plastic strain level in nonlinear FE analysis, and in fact, the minaret would not be damaged, at all. On the other hand, the tensile plastic strains gathered from nonlinear FE analysis under the effect of Düzce earthquake exceeded the critical material strain value indicating that there would be serious damages in the minaret in case of severe earthquake. Contrary to that, the compressive strength gathered from both analysis methods, linear and nonlinear, would not have caused damage, so the minaret is safe against compressive stresses.

Figure 21. Existing cracks and damages on the minaret

9. CONCLUSIONS

Seismic damage propagation estimation in historical clay brick masonry minarets under different ground motion levels were implemented in this paper by using updated finite element analysis. Çay-Sultandağı (2002) and Düzce (1999) earthquakes were considered in the linear and nonlinear time history analyses. 3D initial finite element model of the minaret was improved using the ambient vibration test results. The difference between the experimental and numerical frequencies is about 28% before the model calibration and 1% after the calibration. The first and the second modes of calibrated model are in harmony with experimental results. Linear time history analysis results exceeded the allowable tensile strength of the masonry quite a few and it is not valid reasonable. Neither of the minimum principal stresses obtained from linear and nonlinear analyses exceeded the compressive strength values of the masonry. The maximum tensile strength obtained from both linear and nonlinear analyses concentrated on stone core region of the stairs and brick masonry wall near the entrance of the minaret above the transition segment. Previously experienced dramatic events have shown that thin and tall masonry structures like minarets are damaged particularly at the transition segment. Occurrence of stress concentration in the regions where section cracks and damages take place indicates that the updated FE model via OMA represents the behavior of the minaret as compatible to reality as possible. Accordingly, it is estimated that in case of a major earthquake, the minaret will be damaged particularly at stone core region of the stairs and brick masonry wall near the entrance of the minaret above the transition segment.

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