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Research Article

Investigation of the earthquake behavior of the historical Adana great clock tower using FEM updated based on environmental vibration data

Hakan Erkek[1], **Musa Yetkin**[2]*, **Yusuf Calayır**[3]

[1] Department of Civil Engineering, Faculty of Engineering, Osmaniye Korkut Ata University, Osmaniye (Türkiye), email: hakanerkek@osmaniye.edu.tr

[2] Department of Civil Engineering, Faculty of Engineering, Fırat University, Elazığ (Türkiye), email: musayetkin@firat.edu.tr

[3] Department of Civil Engineering, Faculty of Engineering, Yalova University, Yalova (Türkiye), email: yusuf.calayir@yalova.edu.tr

*Correspondence: musayetkin@firat.edu.tr (M. Yetkin)

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Abstract: In this study, the historical Adana Great Clock Tower's dynamic characteristics (natural frequencies and mode shapes) were determined using the operational modal analysis (OMA) method. The finite element model (FEM) of the Clock Tower was updated based on the experimentally obtained dynamic characteristics. The update process was performed manually using the material properties of the Clock Tower. Linear dynamic analyses of the structures under earthquake loadings were performed using its updated finite element model. The acceleration records of the 1998 Adana-Ceyhan earthquake, scaled according to the horizontal elastic design spectrum defined in TBEC (2018) were used for dynamic input. The displacements, maximum and minimum principal stresses of the Clock Tower were obtained and evaluated. As a result of the analyses, it was determined that there was a potential for damage to the Clock Tower due to exceeding the tensile strength of the walls, but the potential for damage to the Clock Tower due to exceeding the compressive strength of the walls is weak.

Keywords: Model updating, historical Clock Tower, dynamic characteristics, earthquake behavior, 1998 Adana-Ceyhan earthquake.

1. Introduction

Historical buildings are objects that have a cultural heritage feature. The majority of these historical buildings worldwide are masonry structures. Masonry structures can be counted as castles, bridges, caravanserais, mosques, churches, minarets, bell towers, clock towers, etc. One of the most delicate historical structures is clock towers, structurally similar to minarets and bell towers (Yetkin et al., 2021a). Clock towers are structures that are often symbolic of the regions where they are located and have been used frequently in the past to signal worship, celebration, danger and war. Today, as most people have replaced their timekeeping mechanisms with digital clocks, computers and telephones, clock towers have lost their importance, and these structures remain a historical heritage for societies. However, these structures have been affected by natural and artificial disasters such as earthquakes, floods, fires, winds, and explosions, and some of them have been destroyed, and some of them have been seriously damaged. Some reasons for the collapses/damages are material properties, building geometry, construction workmanship, connections between walls and soil properties (Yetkin et al., 2024). In order to transfer this historical heritage to future generations with its characteristics intact, structural behavior analyses should be carried out by creating models that will reflect the current state of the relevant structures in the most appropriate way, and depending on the results of the analysis, necessary repair and strengthening operations should be carried out.

Analyses to determine the earthquake behavior of structures are frequently performed today with the help of developing package programs. Some of the papers published in recent years can be mentioned here. Türkeli et al. (2015) investigated the dynamic behavior of conventional and buttressed reinforced concrete minarets. In their study, they analyzed four different types of reinforced concrete minarets for two different earthquake acceleration records. They stated that the use of structural load-bearing systems with buttresses in earthquake-prone areas is beneficial and that these structural systems behave better than conventional reinforced concrete systems. Genç et al. (2019) performed a model update of a masonry clock tower in Çorum, Türkiye, based on experimental measurements and then performed dynamic analysis according to the Turkish Building Earthquake Code - 2018 (TBEC-2018). Adam et al. (2020) created a finite element model of a historical masonry minaret in Egypt and determined its structural behavior under various loadings. They evaluated the overall stability and safety of the minaret based on the stresses, deformations and cracks obtained. Bayraktar and Hökelekli (2021) numerically constructed minarets with brick-masonry, stone-masonry and concrete materials and strengthened these models with various methods. They then performed nonlinear seismic analyses for these models and proposed a cost-effective seismic retrofitting technique with minimum intervention, better workability and low cost. Usta (2021) created finite element models of five different minarets in Antalya and determined their behavior under various earthquakes. Usta (2021) determined the drift ratio and stress distribution of the minarets as a result of earthquake analyses, stated that the maximum shear stresses are concentrated in the transition segment and balcony level, and observed that there are damages and deterioration along the minaret in the transition zone. Yurdakul et al. (2021) investigated the earthquake performance of a historical masonry minaret built in Bayburt in the 12th century by creating a finite element model. They used three earthquake acceleration records such as Erzincan, Kocaeli-Düzce and Van-Erciş, to investigate the seismic performance of the historical minaret and stated that some parts of the minaret body may be damaged due to the tensile stresses exceeding the limit tensile stress of the masonry brick. Maraş et al. (2022) evaluated the structural response of a historical mosque by dynamic analysis. The material properties of the mosque were obtained by experimental tests. In order to obtain the seismic behavior, they performed time history analyses using six different earthquake acceleration records. After the analyses, they determined the displacement and stress values in the mosque. They observed that the crack zones formed in the mosque after the 2020-Elazığ earthquake were similar to the possible crack zones that occurred as a result of finite element analysis. Şentürk et al. (2022) investigated the effects of different retrofitting methods on the structural performance of earthquake damaged historical masonry structures. For this purpose, they created a three-dimensional finite element model of the historical Alaeddin Bey Mosque and analyzed it for three different earthquake acceleration records. As a result of the analyses, they reinforced the area where there is a risk of damage in five different ways. Because of the earthquake analyses on the reinforced models, they explained in detail which method would be more effective in strengthening such damages with the help of tables and graphs.

For earthquake analyses of structures using finite element methods to give meaningful results, surveys reflecting the current state of the structures should be created, and the material properties and boundary conditions of the structure should be determined accurately. The most effective method to determine the existing condition of a structure is experimental tests. These tests can be said as tests to determine the material properties and dynamic characteristics of structures. Another name of the tests performed to determine the dynamic characteristics is modal analysis tests. Modal analysis tests are performed experimentally in two different ways. These are forced vibration and operational vibration tests. Forced vibration tests are costly and difficult to apply for large volume structures. For this reason, the forced vibration method is not preferred in modal tests of existing structures. On the other hand, operational vibration tests are the most widely used method in modal testing of existing structures since they are performed under environmental conditions (wind, traffic, human movement, etc.) without any external stimulus. There are studies in which modal tests of many structures, such as mosques, minarets, churches, bridges, bell towers, etc., were performed using the operational modal analysis (OMA) method. Gentile et al. (2015) determined the dynamic characteristics of the historical Chiesa collegiate church (northern Italy) using the OMA method. Based on the OMA results, they performed a model updating for the finite element model of the church. They emphasized that model updating is significant in creating finite element models that reflect the current state of the bell towers. Livaoğlu et al. (2016) investigated the effect of geometric properties on the dynamic characteristics of minarets. Firstly, they determined the modal parameters of seven historical masonry minarets by ambient vibration tests. Then, they created three-dimensional (solid) models of these minarets with the help of finite element software and performed model updating to match the experimentally determined natural frequencies with the numerical results. They derived an analytical formula to determine the fundamental period of such structures depending on the height, cross-section and boundary conditions. Lorenzoni et al. (2017) conducted modal tests of four ancient water towers (Pompeii) using the OMA method. According to the OMA results, they performed model updating of the water towers and evaluated the structural vulnerabilities of these towers. Ubertini et al. (2017) investigated the effects of changes in temperature and humidity on the natural frequencies of fragile masonry buildings based on OMA measurement results for a masonry bell tower at different times. Brownjohn et al. (2018) conducted free and forced vibration tests of 6 lighthouses (British Isles) ranging from 33-49 m in height. De Silva et al. (2018) performed modal tests of the Carmine bell tower, one of the tallest towers in Italy, using the OMA method. Standoli et al. (2021) performed OMA measurements of four masonry bell towers in Ferrara province (Italy). They updated the finite element models they created for the towers depending on the measurement results. They used the Modal Assurance Criterion (MAC) criterion to verify the mode shapes obtained from experimental and finite element solutions. Milani and Clementi (2021) performed the seismic evaluation of these towers using the updated finite element models created by Standoli et al. (2021). Altıok and Demir (2021) investigated the seismic behavior of the historical Lala Mehmet Pasha minaret. They obtained the dynamic properties of the minaret experimentally with the OMA method and updated the initial finite element model. Then, they performed linear and nonlinear time domain analysis of the minaret. As a result of the analyses, they obtained and interpreted the minaret's displacements, damage and stress distributions. Calayır et al. (2021) determined the dynamic characteristics of a masonry minaret using the OMA method. They updated the finite element model of the minaret using the frequencies and mode shapes obtained with OMA. They selected the moduli of elasticity of the minaret and the soil as the effective parameters for the update process and performed the update process manually.

They checked the compatibility of the numerical and experimental modes with the Modal Assurance Criterion values. After the model update, they obtained MAC values of the mode shapes between both solutions above 90%. Akhlaq et al. (2022) determined the dynamic characteristics of a 90 m high reinforced concrete minaret using operational modal analysis. Then, they created a finite element model for the minaret and manually updated it according to these experimentally obtained characteristics. Avcı et al. (2022) performed OMA measurements of a tall tower (230 m), one of the tallest structures in Qatar, and updated the finite element model of the tower based on the measurement results. Bayraktar et al. (2022) determined the natural frequencies, mode shapes and damping ratios of nineteen historical masonry stone cylindrical minarets based on OMA measurements. Considering the obtained natural frequencies, they developed a formula to determine the initial natural frequency depending on the geometrical dimensions of the minaret. They stated that the formula they proposed gave very consistent results with the measurement and literature data. Shabani et al. (2022) performed environmental vibration measurements of the Tønsberg (Norway) stone masonry tower. They used the material properties of the homogenized wall to update the finite element model of the tower. They developed three FEMs to investigate the effect of soil-structure interaction on the FEM update results. They performed a sensitivity analysis to investigate the parameters affecting the dynamic properties of the models. Finally, they performed linear dynamic analyses on the three calibrated models and compared the results with each other. Altunışık et al. (2018) performed ambient vibration tests of a historical masonry armory building with the OMA method for three different measurement schemes and then numerically obtained the dynamic characteristics by creating a finite element model of this structure. They stated that there is a good correlation between the mode shapes for the experimental and numerical results and performed nonlinear analysis for the model they created.

In this study, the dynamic characteristics of the Adana Great Clock Tower were determined using the OMA method. Based on these dynamic characteristics, the finite element model of the Clock Tower was updated. Linear dynamic analysis of the structure was performed using the updated finite element model. The acceleration records of the 1998 Adana-Ceyhan earthquake, scaled according to the horizontal elastic design spectrum defined in TBEC (2018), were used for dynamic input. The top displacement, base shear force, and maximum and minimum principal stresses of the Clock Tower were evaluated.

2. Seismicity of the region

Türkiye is located between the Arabian, African and Eurasian plates and 92% of its territory, 95% of its population and 98% of its industry is in a seismically active earthquake zone. In the last century (1900-2000), 130 damaging earthquakes killed approximately 67000 people (an average of 680 per year), 150000 people were injured and 500000 structures were destroyed/severely damaged (Adalier & Aydingun, 2001; Dedeoğlu et al., 2024; Sezer, 1999). Some of these damaging earthquakes occurred in the Adana-Ceyhan earthquake belt around the city of Adana, where the Adana Great Clock Tower is located. The Adana-Ceyhan earthquake belt is tectonically located in a very complex transition zone between the Cyprus Arc to the west and the Bitlis Thrust and Fault Zone and the Eastern Anatolian Fault to the east (Adalier & Aydingun, 2001) (Figure 1).

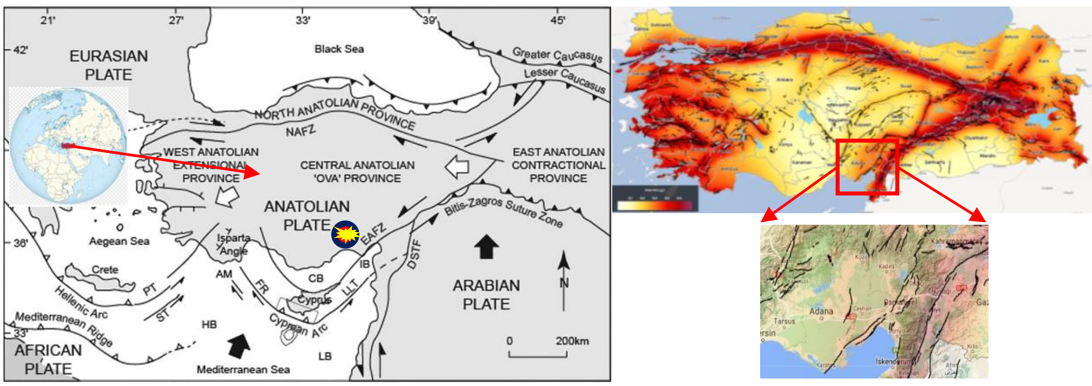


Figure 1. Adana-Ceyhan earthquake zone (DEMA, 2022; Emre et al., 2011; Kinnaird & Robertson, 2013).

The most important destructive earthquakes recorded for Adana-Ceyhan region in the instrumental period are 1945 (Ms=6.5), 1952 (M_s=5.6) and 1998 (M_w=6.2). A total of 169 people lost their lives and 34580 buildings were damaged in these earthquakes (URL-1). The 1998 earthquake (M_w=6.2) was the most destructive of these earthquakes as 146 people lost their lives and 31463 buildings were damaged (Figure 2). The Adana Great Clock Tower was able to survive these earthquakes.

Figure 2. Examples of structures damaged in the 1998 Adana-Ceyhan earthquake (DEMA).

3. Historical Adana Great Clock Tower

Adana Great Clock Tower is located in Seyhan, district of Adana, its construction started in 1879 and was completed in 1882. It is a tower that can be seen from all over the city and is considered the center of the city. The Clock Tower has not suffered any serious damage after the earthquakes it has experienced and was restored in 2014. The Clock Tower is still in service and is important for the historical tourism of the region. The length of the tower is 27.8 meters and it is the tallest clock tower in Türkiye.

The Clock Tower is composed of cut stones up to a height of 2.8 meters from the base and brick units at higher heights. There are a total of 112 stair steps inside the Clock Tower until the section where the clock dial is located. The section where the clock dial is located consists of both stone and brick units. The Clock Tower sits on a 4×4 m^2 base and there is no exact information about the foundation of the Clock Tower. The visuals and geometric features of the Clock Tower are given in Figure 3.

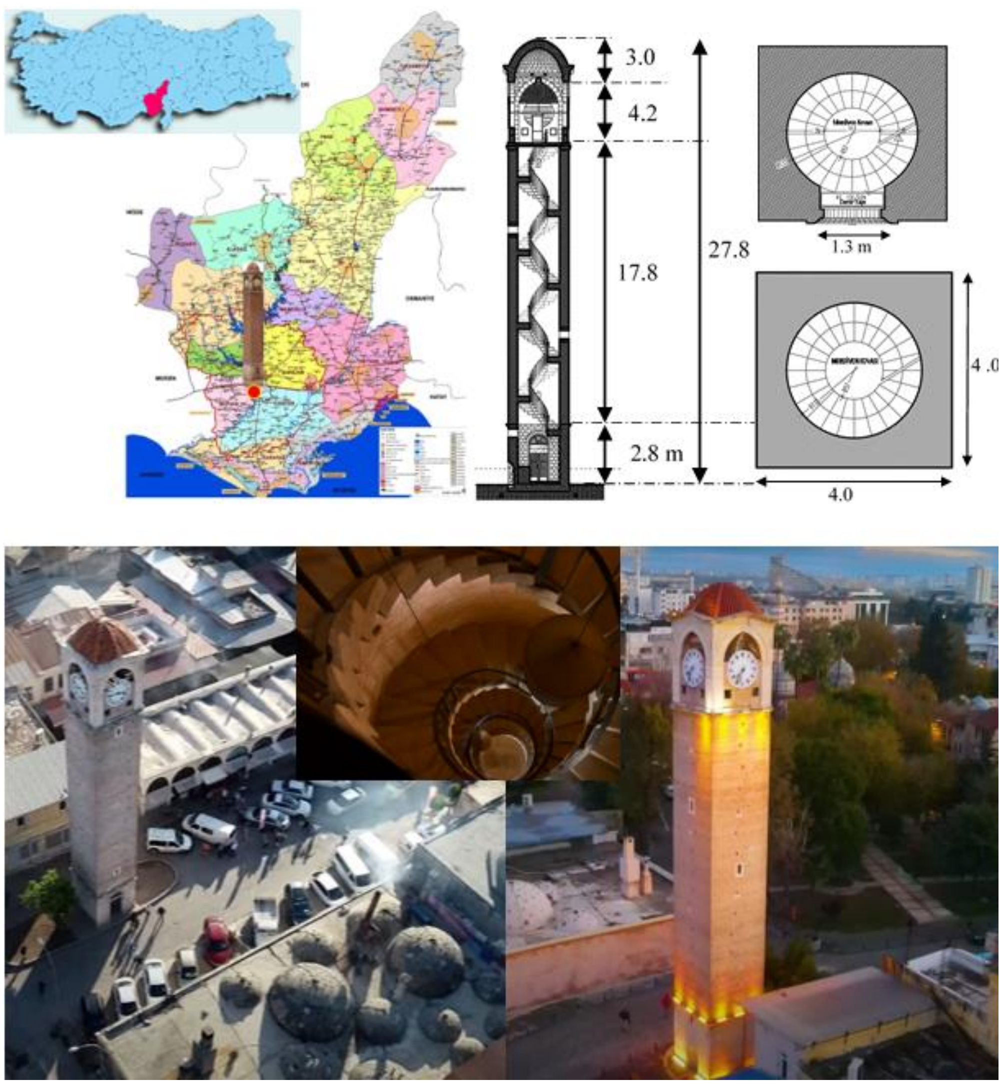


Figure 3. Adana Great Clock Tower.

4. Finite element analyses

The Clock Tower was modeled in ANSYS program using macro modeling technique. In the finite element modeling of the Clock Tower, the interaction of the structure, foundation and soil domains is considered. These domains are assumed to be linear elastic and modeled with the SOLID 65 element. The foundation and soil masses are neglected in the modeling. The foundation of the tower is assumed to be constructed using stone units and its dimensions are selected as 5×5×5 m^3. The soil domain is also chosen as a cube shape and the side lengths of this cube are taken as the height of the Clock Tower. A prism cavity was left inside the soil domain for the foundation domain (Figure 4). Finite element model of the Clock Tower was created with 92046 nodes and 76351 elements, taking into account the spiral staircase, door and window openings. The centers of the projections of all domains in the horizontal plane are at the same point.

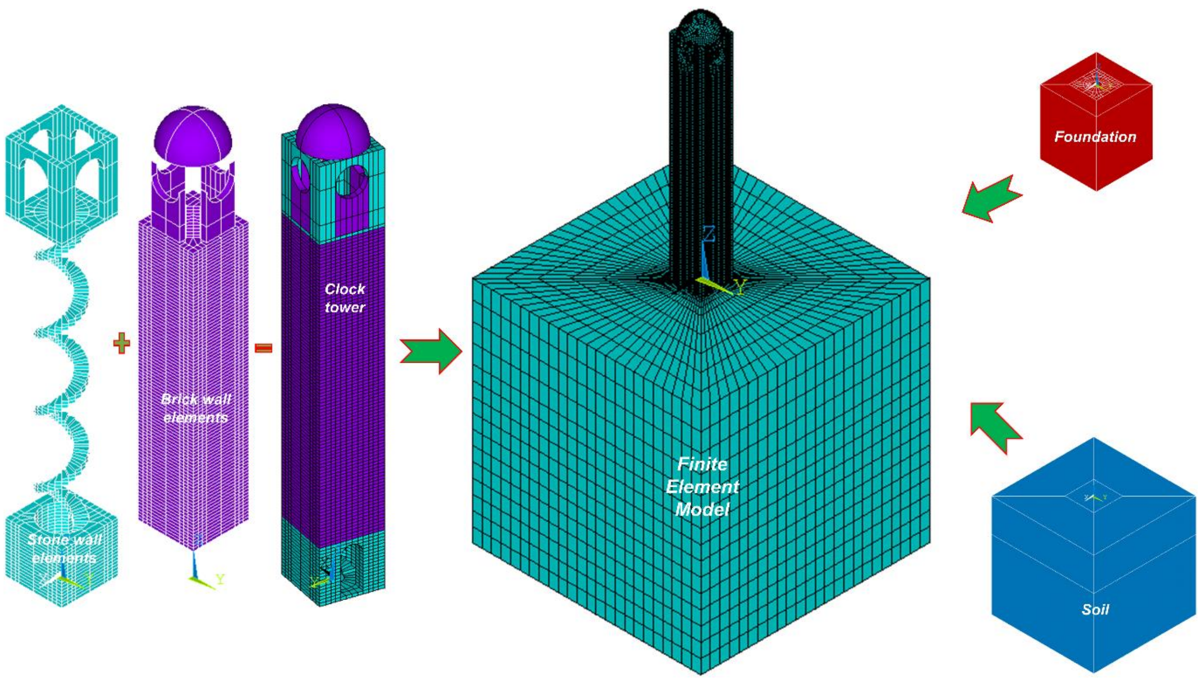


Figure 4. Finite element model of Adana Great Clock Tower.

The Clock Tower consists of stone, brick and mortar units. The compressive strengths of stone, brick and mortar units were assumed to be 80 MPa, 10 MPa and 2 MPa, respectively. In order to determine the compressive strength of the masonry wall, the following equation developed by Tomazevic (1999) was used.

$$f_{m,c} = K\, {f_{b,c}}^{0.65} {f_{mo,c}}^{0.25} \tag{1}$$

Here $f_{m,c}$, $f_{b,c}$ and $f_{mo,c}$ are the compressive strength for masonry wall, masonry unit and mortar unit respectively. K is a constant coefficient and its value can be taken between 0.4-0.6 (Tomazevic, 1999). In this study, the coefficient K is assumed to be 0.5

The value of the elasticity modulus of the masonry wall, taking into account the recommendation in Eurocode 6 (2007), was determined by

$$E = 1000 f_c \tag{2}$$

where E and f_c are elasticity modulus and compressive strength of the wall, respectively.

Accordingly, the elasticity modulus of the stone and brick walls of the initial analytical model can be determined as given in Figure 5.

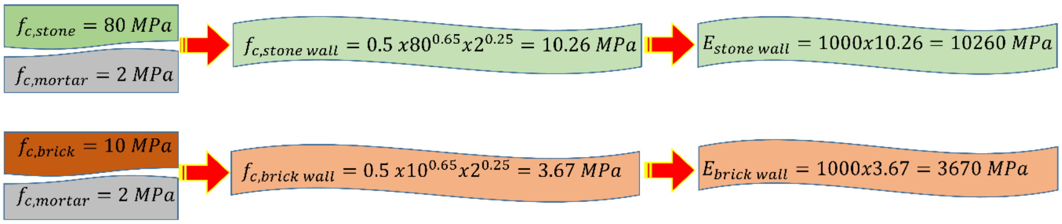
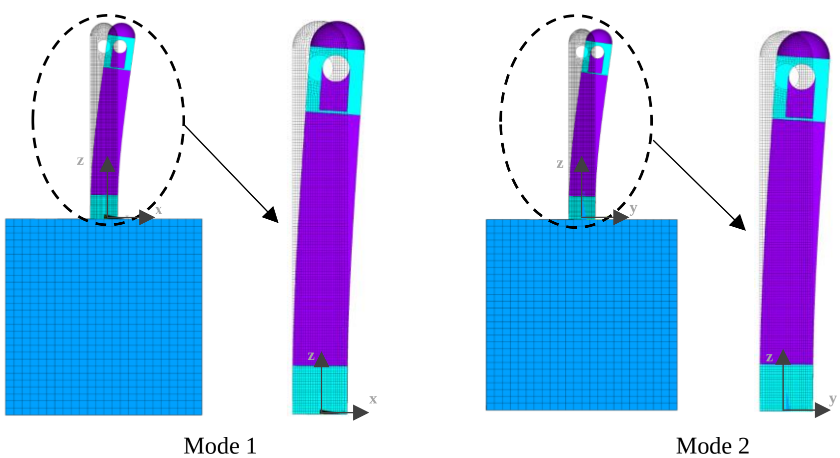


Figure 5. The process of determining the elasticity modulus of the initial analytical model.

Since there is no precise information about the stones used for the foundation domain, it is assumed that the elasticity modulus of for this domain is about 50% higher than the elasticity modulus of the masonry wall used in the upper structure. The elasticity modulus of the soil domain was taken as 150 MPa. The equivalent densities of the stone and brick walls were assumed to be 2200 kg/m^3 and 1800 kg/m^3 respectively.

Modal analysis of the Clock Tower-foundation-soil system was performed using the initial finite element model. The first four natural frequencies of the system were determined as 1.506 Hz, 1.526 Hz, 6.771 Hz and 6.871 Hz, respectively. The mode shapes of these natural frequencies are given in Figure 6. Modes 1 and 4 show bending behavior around the y axis and modes 2 and 3 show bending behavior around the x axis. While modes 1 and 2 are single-zone, modes 3 and 4 are two-zone. Since the system is approximately symmetrical in plan (x-y plane) at each elevation level along the height, the centers of stiffness and mass are approximately at the same point. In addition, since the plan section is square, the x and y direction stiffnesses of the system are almost the same. For these reasons, the frequencies of modes 1 and 2 of the system are very close to each other. Similarly, this situation is valid for the frequencies of modes 3 and 4.



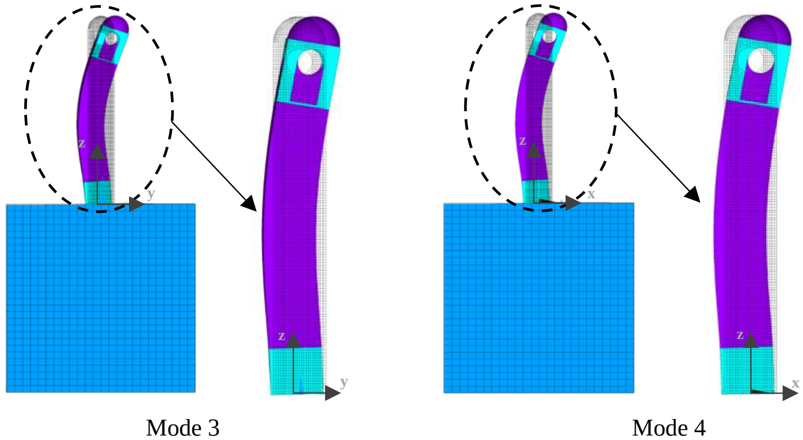


Figure 6. First four mode shapes of the initial analytical model of the Clock Tower-foundation-soil system.

5. Environmental vibration analyses

The vibration characteristics of the Clock Tower were obtained experimentally using the OMA method. OROS-OR36 Multichannel Noise and Vibration Analyzer and KB12VD type uniaxial accelerometers were used in the experimental measurements. These accelerometers were placed at four different points of the Clock Tower on the same vertical direction in x and y directions (in the horizontal plane) and measurements were made with a total of eight accelerometers in the frequency range of 0-10 Hz, spectral lines 6401, average number 5 and total 3200 s (Figure 7).

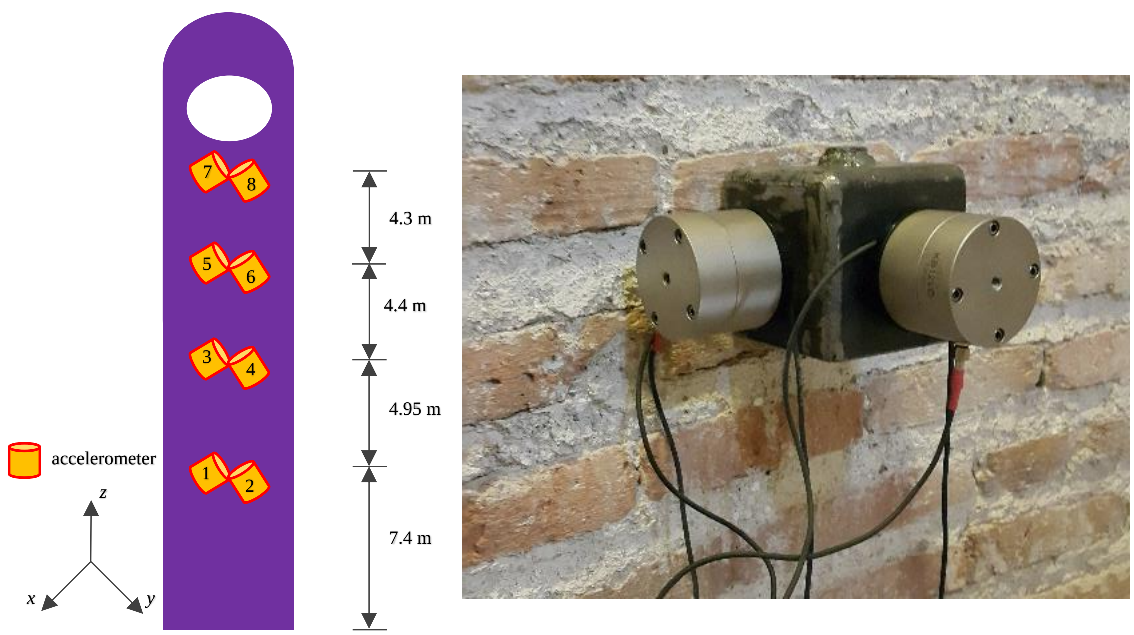


Figure 7. Accelerometer layout in the Clock Tower.

The raw signals from the accelerometers were recorded by applying FFT (fast Fourier transform) in OROS-NVGATE software and these data were filtered according to the weight functions in OROS-Modal software. Then, the dynamic characteristics were obtained using the EFDD (enhanced frequency domain decomposition) method, which gives the spectral density functions of the signals in each channel. The spectral density function and the selected natural frequency values are given in Figure 8.

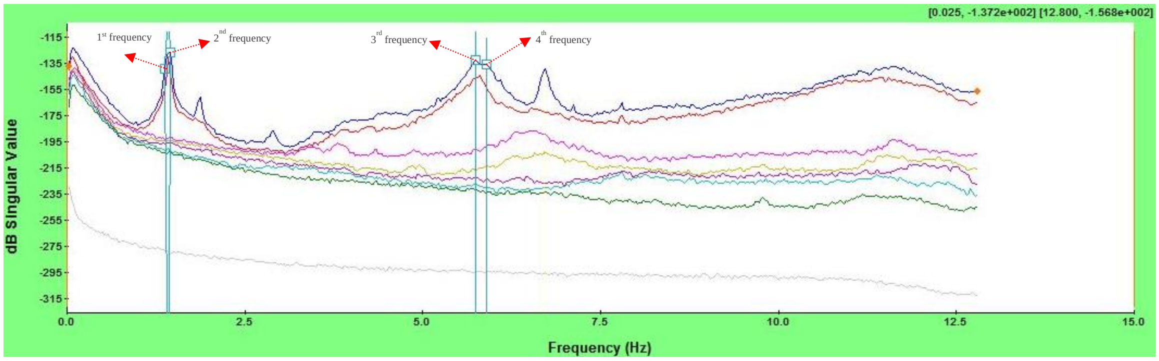


Figure 8. Spectral density function and selected frequency values.

The differences between the frequency values and the MAC values of the mode shapes were used to compare the initial FEM solutions with the experimental results. Experimental frequencies were taken as reference for the calculation of the differences between frequency values. MAC values are determined by following equation (Allemang, 2003)

$$MAC\ (\psi_a\ ,\psi_e) = \frac{\left|{\psi_a}^T\ \psi_e\right|^2}{({\psi_a}^T\psi_a)({\psi_e}^T\psi_e)} \tag{3}$$

here ψ_a and ψ_e are the FEM and experimental mode shapes, respectively.

When the FEM solutions and experimental results are compared (Table 1), it is seen that the MAC values are high. This indicates that the mode shapes of both solutions are similar. However, the same case is not valid for the differences between frequency values. Especially, the differences are high for the third and fourth frequencies. Model updating is required to reduce these differences between frequencies to acceptable levels (Altunişik et al., 2018; Erkek and Yetkin, 2023; Yetkin, 2018a; Yetkin et al., 2021).

Table 1. Comparison of initial finite element model solutions and experimental results.

Mode Number	FEM Frequencies (Hz)	Experimental Frequencies (Hz)	Frequency Difference (%)	MAC value (%)
1	1.506	1.41	6.809	96.021
2	1.526	1.44	5.972	95.006
3	6.771	5.75	17.757	93.755
4	6.871	5.89	16.655	96.056

6. Model updating

In this study, the model updating of the tower-foundation-soil system is performed using the dynamic characteristics obtained from the OMA method. The elasticity modulus is selected as the effective parameter for the model updating of the system. The values of the elasticity modulus of the stone wall, brick wall and soil that reduce the differences between the OMA results and FEM solutions to acceptable levels were determined by a parametric study. Accordingly, the elasticity modulus values for stone wall, brick wall and soil were taken as 14800 MPa, 2000 MPa and 90 MPa, respectively. In this case, according to Equation (2), the compressive strengths of stone and brick wall materials can be determined as 14.8 MPa and 2.0 MPa, respectively. If it is accepted that the tensile strengths of the stone and brick wall materials are 10% of the compressive strengths (Erkek and Yetkin, 2023; Yetkin et al., 2018b), the tensile strength values will be 1.48 MPa and 0.2 MPa, respectively.

The first four frequencies and corresponding mode shapes obtained from the modal analysis of the updated FEM are presented in Table 2 and Figure 9, respectively. After the model updating process, the differences between the natural frequencies are significantly reduced. There is an increase in MAC values of the modes 1 and 2 after the model updating. In MAC values of the modes 3 and 4, there is a slight decrease and these differences are negligible since the MAC values for all modes are above 90%.

Table 2. Comparison of the updated finite element model solutions and experimental results.

Mode number	FEM frequencies (Hz)	Experimental frequencies (Hz)	Frequency difference (%)	MAC value (%)
1	1.367	1.41	-3.050	96.371
2	1.378	1.44	-4.306	95.175
3	6.038	5.75	5.009	90.611
4	6.09	5.89	3.396	95.866

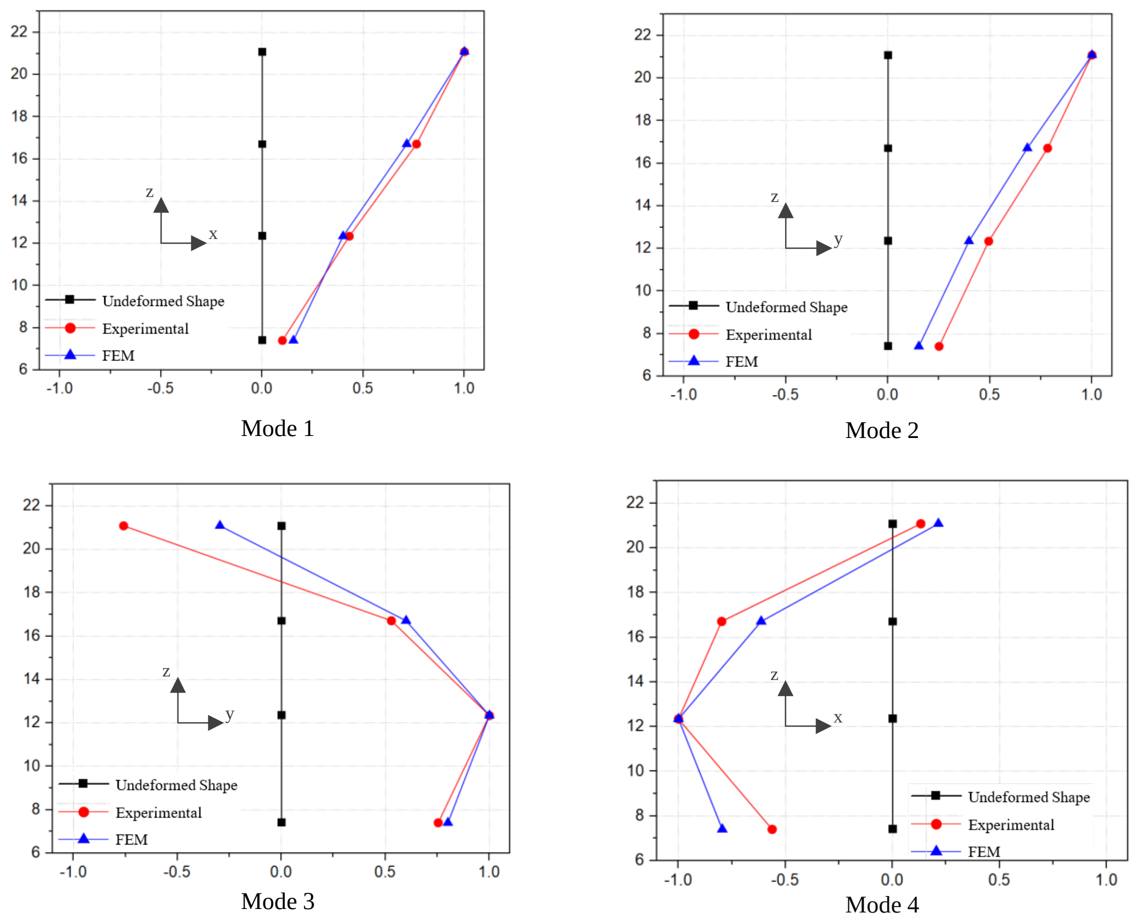
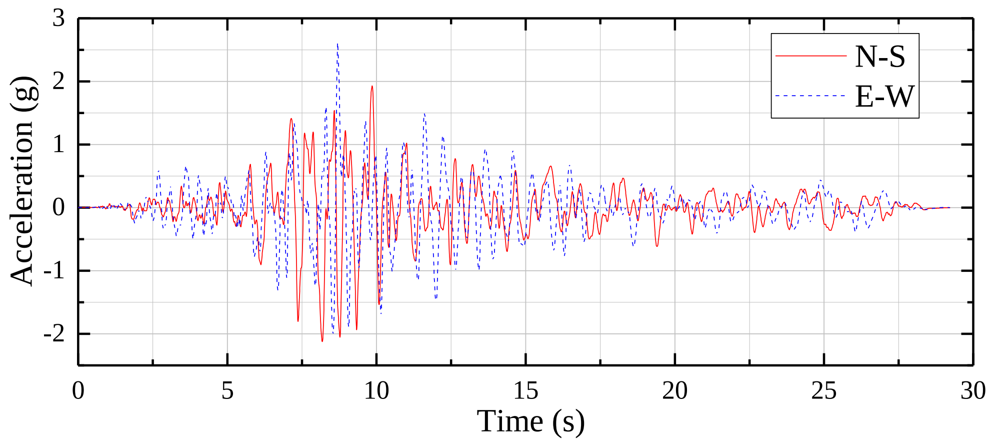


Figure 9. Spectral density function and selected frequency values.

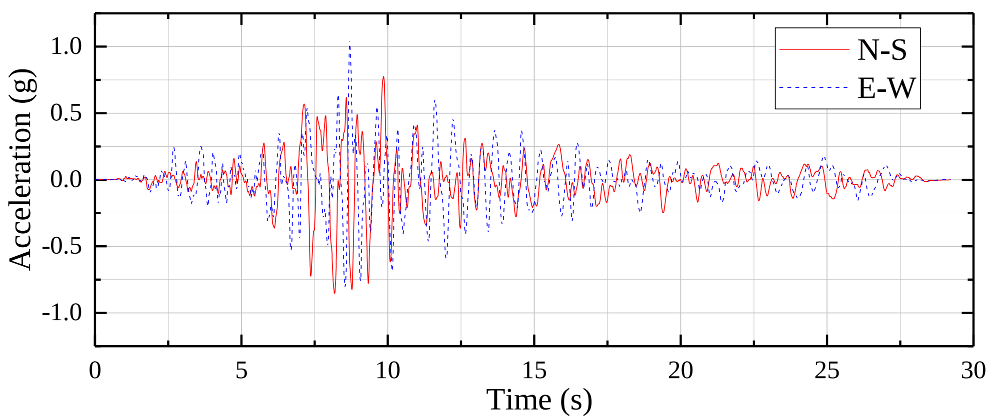
7. Time history analyses

The seismic performance of the Clock Tower was evaluated by linear time history analyses. For this purpose, considering the location of the Clock Tower, acceleration records of the North-South and East-West components of the 1998 Adana-Ceyhan earthquake were selected for the dynamic input. These acceleration records were taken from RESORCE (Reference Database for Seismic Ground-Motion Prediction in Europe) database.

Two horizontal components of the 1998 Adana-Ceyhan earthquake acceleration record were scaled separately according to the horizontal elastic design spectrum defined in TBEC (2018) for earthquake levels which probability of exceedances in 50 years are 10% and 50%. The acceleration amplitudes of each earthquake acceleration record component were first scaled so that the amplitudes of the spectrum of each acceleration record component between the periods $0.2T_p$ and $1.5T_p$ are compatible with the amplitudes of the design spectrum defined for the relevant earthquake level in the same period range, where T_p is the dominant period of the structure. Then, to ensure that the scale coefficients of both acceleration components are the same, each acceleration component was rescaled by taking into account the average of the scale coefficients obtained for these acceleration components. The scaled earthquake acceleration records for different earthquake levels are presented in Figure 10. In Figure 10, the curves labeled N-S and E-W represent the North-South and East-West components of the scaled earthquake acceleration records, respectively.



a) Scaled earthquake acceleration records for DD2 earthquake level.



b) Scaled earthquake acceleration records for DD3 earthquake level.

Figure 10. Scaled acceleration-time graphs of the 1998 Adana-Ceyhan earthquake.

Time history analysis of the Clock Tower was performed by simultaneously applying two earthquake acceleration components scaled according to an earthquake level to the structure. Then, these acceleration components were rotated by 90º and the analyses were repeated. The analyses in which the North-South component was applied in the x direction and the East-West component was applied in the y direction were called N-S(x)/E-W(y), while the analyses in which the North-South component was applied in the y direction and the East-West component was applied in the x direction were called N-S(y)/E-W(x).

7.1. Analysis results and discussions

The linear seismic solutions of the Clock Tower were obtained using the acceleration records of the 1998 Adana Ceyhan earthquake scaled for the DD2 earthquake level. The variations of the displacements of the top node in the x and y directions with time are presented in Figure 11-12 for N-S(x)/E-W(y) and N-S(y)/E-W(x) solutions, respectively. The displacement amplitudes also increased in the time interval in which the earthquake acceleration amplitudes were large. Absolute maximum values of the displacements are obtained as 61.25 mm from the N-S(y)/E-W(x) solution for the x direction and 60.48 mm from the N-S(x)/E-W(y) solution for the y direction at 8.705 s.

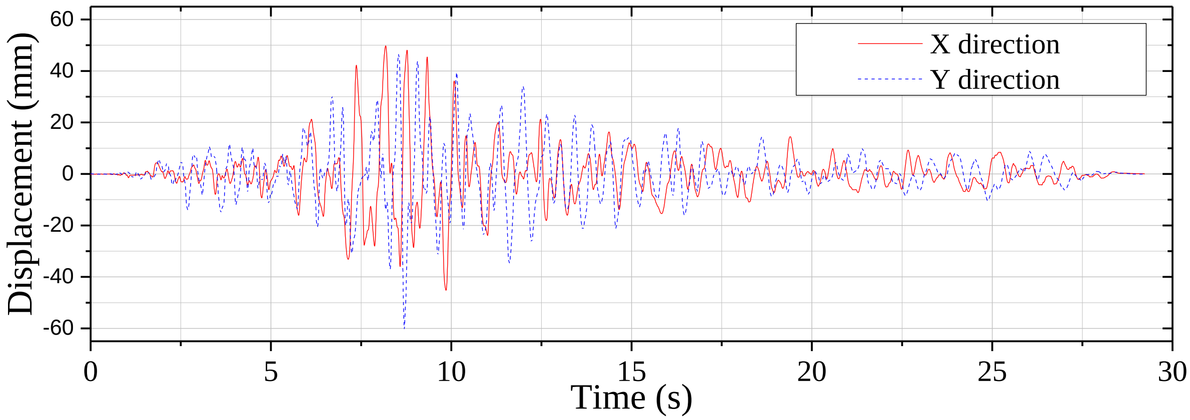


Figure 11. The variations of the displacements of the top node in the x and y directions with time for N-S(x)/E-W(y) solutions.

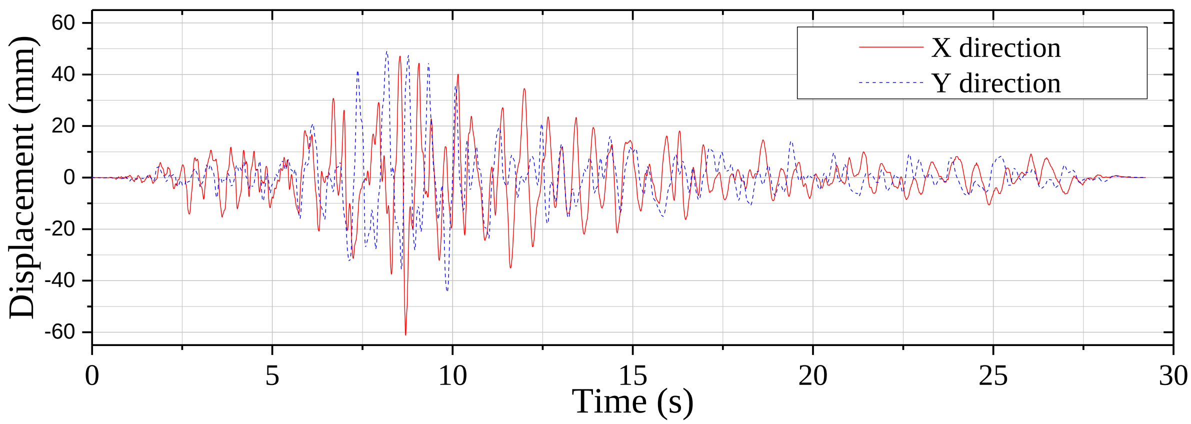


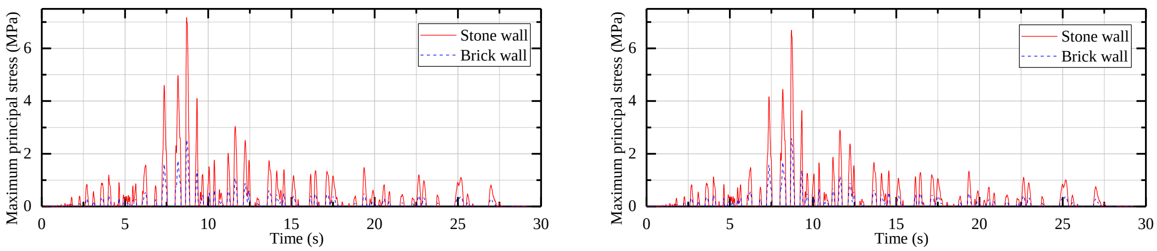
Figure 12. The variations of the displacements of the top node in the x and y directions with time for N-S(y)/E-W(x) solutions.

The absolute maximum values for the top displacement and base shear force of the Clock Tower are given in Table 3. The absolute maximum top displacement values in x and y directions for N-S(x)/E-W(y) solutions are almost the same as the absolute maximum top displacement values in y and x directions for N-S(y)/E-W(x) solutions, respectively. This is also the case for the base shear force values. This similarity between the solutions is due to the fact that the stiffnesses of the system in x and y directions are approximately the same.

Table 3. The absolute maximum values for the top displacement and base shear force of the Clock Tower.

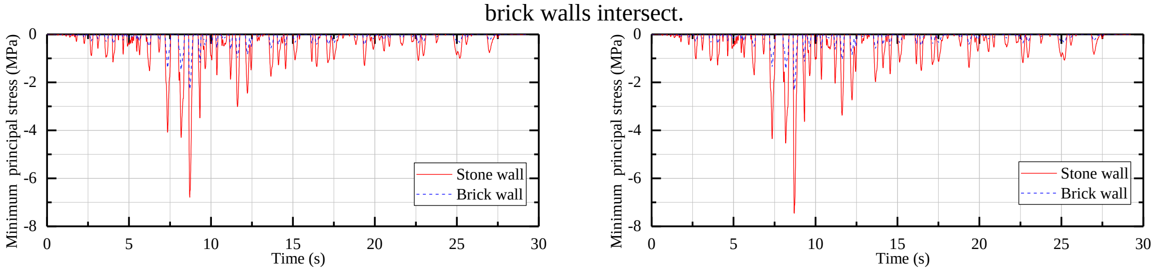
Solution name		Maximum displacement		Base shear force	
		δ_x (mm)	δ_y (mm)	V_x (kN)	V_y (kN)
DD2	N-S(x)/E-W(y)	49.77	60.48	933.33	1149.53
	N-S(y)/E-W(x)	61.25	49.07	1149.53	933.33

For the N-S(x)/E-W(y) and N-S(y)/E-W(x) solutions, the principal stresses in the brick wall reach the absolute maximum values at the corner point just above the left side of the entrance door of the Clock Tower where the stone wall and the brick wall intersect. For the same solutions, the principal stresses in the stone wall reach absolute maximum values at the corner point to the left of the entrance door at the base of the Clock Tower. Time variations of maximum and minimum principal stresses at these relevant points where the principal stresses in the stone and brick walls reach the absolute maximum values are given in Figure 13-14. As observed in the displacement amplitudes (Figure 11-12), the maximum and minimum principal stress amplitudes also increase in the time interval in which the earthquake acceleration amplitudes are large. Absolute maximum of the principal stresses for the stone and brick walls of the Clock Tower are given in Table 4.



a) N-S(x)/E-W(y) b) N-S(y)/E-W(x

Figure 13. Variations of maximum principal stresses with time at the corner point where the stone and brick walls intersect.



a) N-S(x)/E-W(y) b) N-S(y)/E-W(x)

Figure 14. Variations of minimum principal stresses with time at the corner point where the stone and brick walls intersect.

Table 4. Absolute maximum of the principal stresses for the stone and brick walls of the Clock Tower.

Solution name		Maximum principal stress (MPa)		Minimum principal stress (MPa)	
		Stone Wall	Brick Wall	Stone Wall	Brick Wall
DD2	N-S(x)/E-W(y)	7.175	2.535	-6.786	-2.289
	N-S(y)/E-W(x)	6.689	2.596	-7.457	-2.333

As can be seen from Table 4, the differences between the principal stress values of N-S(x)/E-W(y) and N-S(y)/E-W(x) solutions are around 2% for the brick wall and approach 10% for the stone wall. The stress contour graphs of the N-S(x)/E-W(y) solutions of the Clock Tower at 8.705 s are presented in Figure 15, separately for the stone and brick walls.

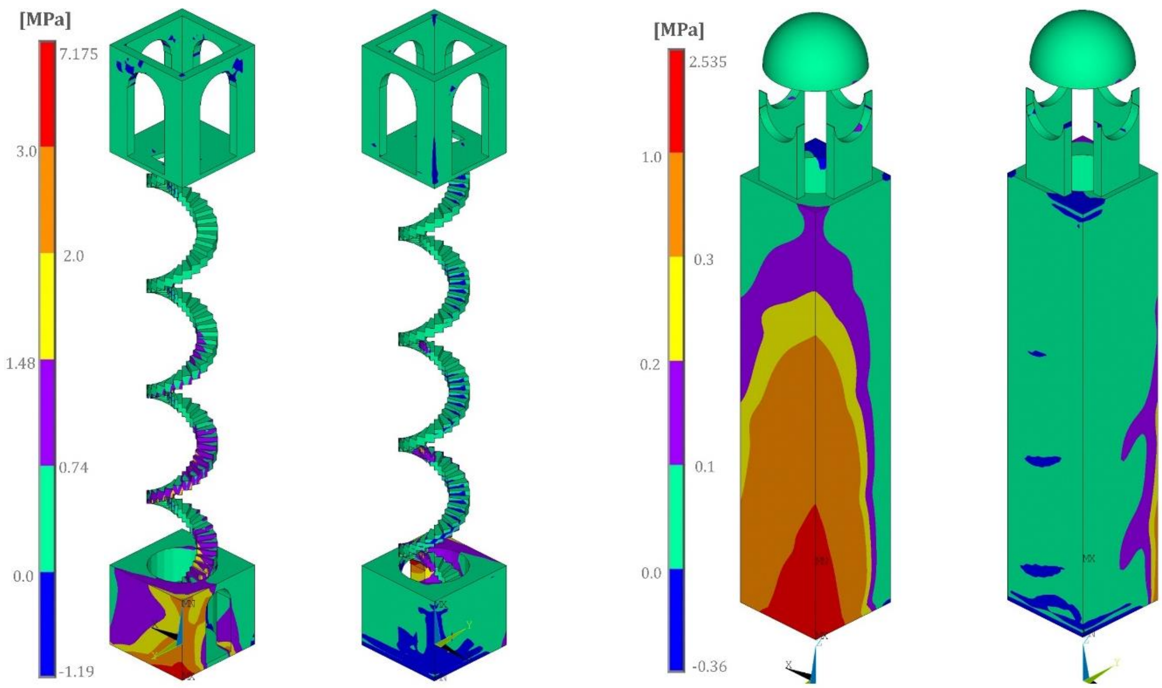
When the maximum and minimum principal stress contour graphs of the stone walls of the Clock Tower are examined:

- The maximum principal stresses reach the largest values at the corner to the left of the entrance door at the base and decrease with distance from the corner. At the corner diagonal to the relevant corner, the maximum principal stresses reach the smallest values. In the region where the maximum principal stresses reach large values, the stone wall tensile strength of 1.48 MPa is generally exceeded. This indicates that the damage potential is high in this region for a possible earthquake at DD2 earthquake level.

- The largest absolute values of the minimum principal stresses occur in the base corner region where the maximum principal stresses reach the smallest values. The minimum principal stresses reach the compressive strength of the stone wall (14.8 MPa) in a narrow region near the base. In the corner diagonal to this corner, the minimum principal stresses are the smallest in absolute value.

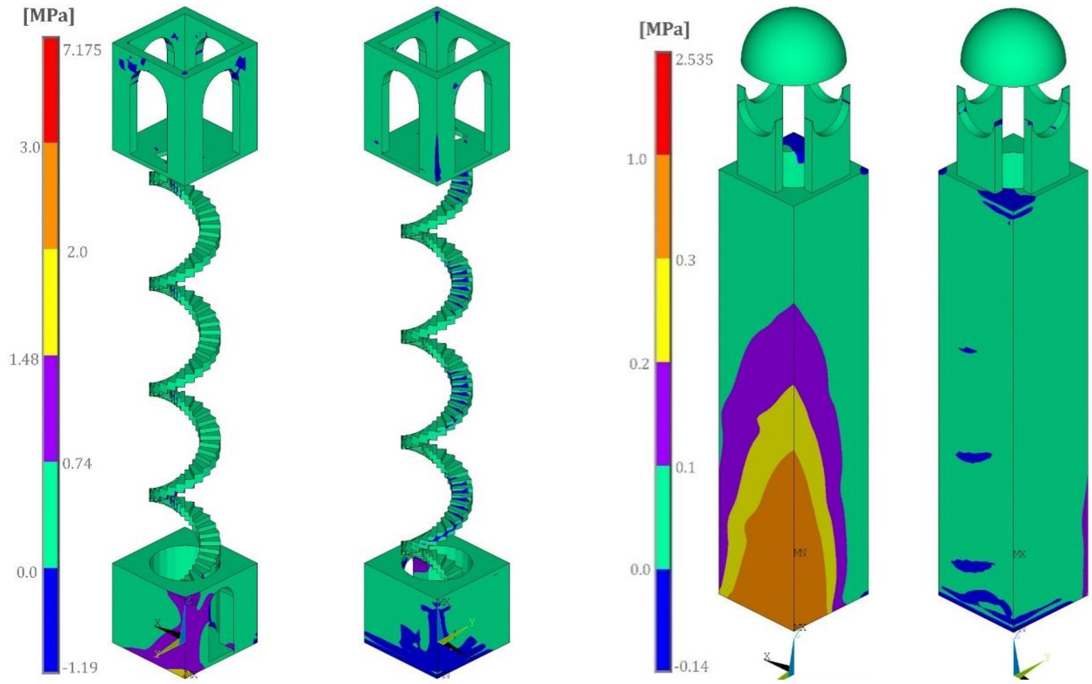
When the maximum and minimum principal stress contour graphs of the brick walls of the Clock Tower are examined;

- The maximum principal stresses reach the absolute maximum values at the connection region of the stone and brick walls at the left side of the entrance door of the Clock Tower. These stresses decrease with distance from the relevant region. At the corner diagonal to the relevant corner, the maximum principal stresses reach their smallest values. In the region where the maximum principal stresses reach large values, the brick wall tensile strength of 0.20 MPa is generally exceeded. This indicates that the damage potential is high in this region for a possible earthquake at DD2 earthquake level.
- The largest absolute values of the minimum principal stresses occur at the corner region where the maximum principal stresses reach the smallest values. The minimum principal stresses reach the compressive strength of the brick wall (2 MPa) in a narrow area around the connection region of the stone and brick walls. At the corner diagonal to the relevant corner, the minimum principal stresses reach their smallest values.



Stone wall elements *Brick wall elements*

(a) Maximum principal stresses.



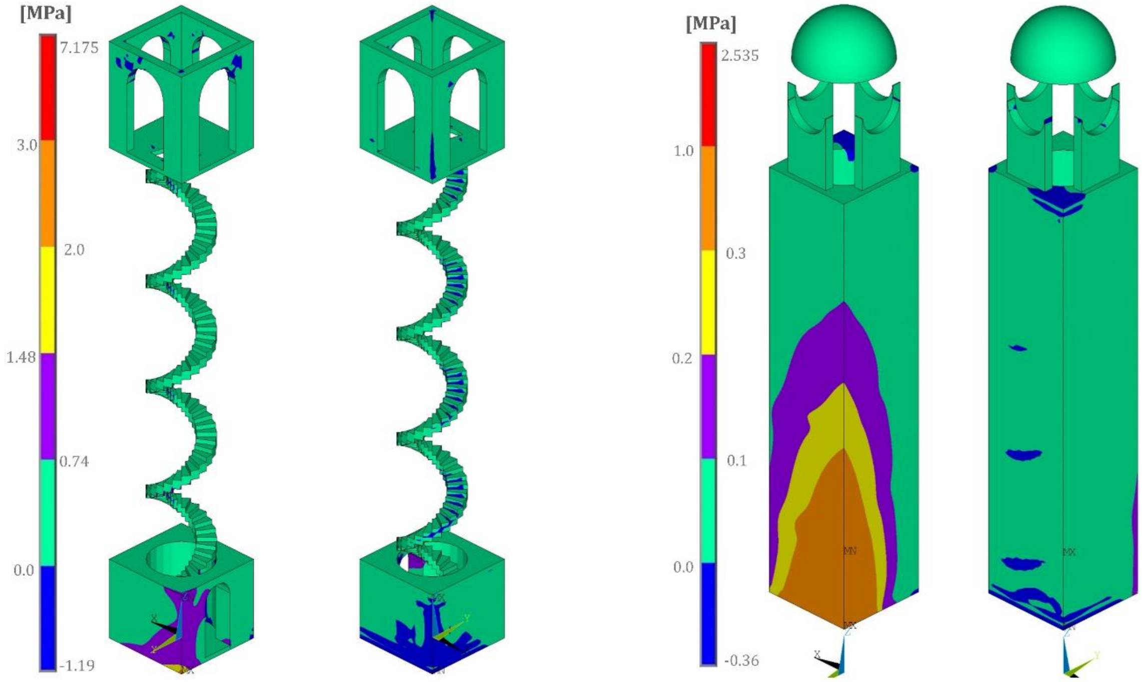
Stone wall elements *Brick wall elements*

(b) Minimum principal stresses.

Figure 15. Maximum and minimum principal stress contour graphs for N-S(x)/E-W(y) solutions with DD2 earthquake level.

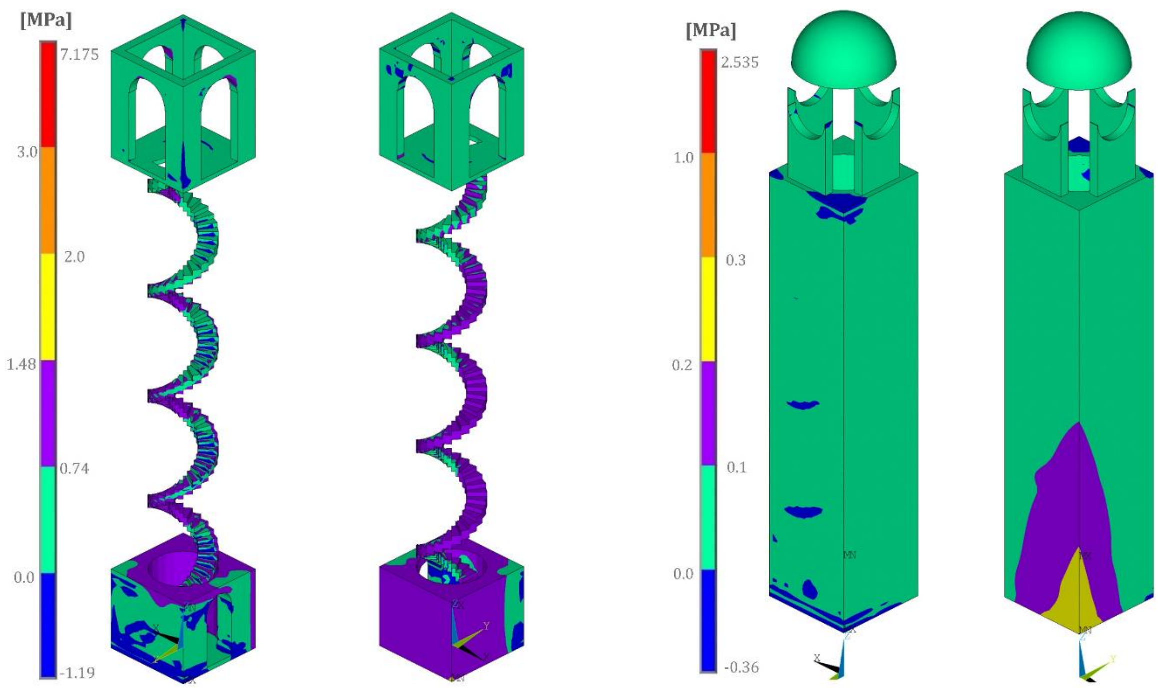
From the above evaluations, it is seen that damages will occur in the Clock Tower at DD2 earthquake level due to the exceedance of tensile stresses. In order to see whether the tensile stresses are exceeded in the Clock Tower at the DD3 earthquake level, which is lower than the DD2 earthquake level, linear seismic solutions of the Clock Tower were performed using the acceleration records of the Adana Ceyhan earthquake (Figure 10.b) scaled according to the DD3 earthquake level. Since it is a linear analysis, the time variations of the response quantities under the scaled records of the same earthquake for different earthquake levels will show similar characteristics. As in the DD2 earthquake level solutions, the maximum and minimum principal stress values in absolute value occur at 8.705 s for the stone and brick walls in the DD3 earthquake level solutions. The stress contour graphs of the N-S(x)/E-W(y) solutions of the Clock Tower with DD3 earthquake level at 8.705 s are presented in Figure 16-17, separately for the stone and brick walls. When these stress contour graphs are examined:

- The variations of the maximum and minimum principal stresses on the walls are similar to the DD2 earthquake level solutions. However, the stresses are lower for DD3 earthquake level solutions.
- Maximum principal stresses in the stone wall exceed the tensile strength in a narrower region than those of DD2 solutions. Similar situation is also observed for the brick wall. This indicates that there is a damage potential in the relevant region for a possible earthquake with DD3 earthquake level.



Stone wall elements *Brick wall elements*

Figure 16. Maximum principal stress contour graphs for N-S(x)/E-W(y) solutions with DD3 earthquake level.



Stone wall elements *Brick wall elements*

Figure 17. Minimum principal stress contour graphs for N-S(x)/E-W(y) solutions with DD3 earthquake level.

8. Conclusions

In this study, the dynamic characteristics of the historical Adana Great Clock Tower were determined based on environmental vibration data. The finite element model of the Clock Tower was updated considering these dynamic characteristics. Linear dynamic analysis of the Clock Tower was performed using the updated finite element model. For dynamic input, acceleration records of the 1998 Adana-Ceyhan earthquake scaled according to the horizontal elastic design spectrum defined in TBEC (2018), taking into account DD2 and DD3 earthquake levels, were used. The top displacement, base shear force, and maximum and minimum principal stresses of the Clock Tower were evaluated.

It was observed that the maximum and minimum principal stresses in the stone and brick walls reached large values in absolute at the regions close to the base of the Clock Tower.

In the regions where the maximum principal stresses reach large values, the tensile strengths of the stone and brick walls are exceeded in wide regions at the DD2 earthquake level, while they are exceeded in relatively narrower regions at the DD3 earthquake level. This indicates a potential for damage to the Clock Tower due to exceeding the tensile strengths of the walls in possible earthquakes with DD3 and DD2 earthquake levels. However, the damage potential is higher at the DD2 earthquake level. In the regions where the minimum principal stresses reach the largest values in absolute value, the stress values are obtained around the compressive strengths of the stone and brick walls. Therefore, it can be said that the probability of damage to the Clock Tower due to exceeding the compressive strengths of the walls in possible earthquakes with DD3 and DD2 earthquake levels is weak.

Author contributions: Hakan Erkek: Conceptualization, methodology, experimental investigation, visualization, review and supervision and enhancement of manuscript. Musa Yetkin: Methodology, visualization, writing, review and supervision, enhancement of manuscript. Yusuf Calayır: Methodology, review and supervision, enhancement of manuscript

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