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Seismic performance of the historical masonry clock tower and influence of the adjacent walls

Ferit Cakir[*1] and Habib Uysal[2a]

[1]Department of Architectural, Amasya University, Amasya, Turkey
[2]Department of Civil Engineering, Ataturk University, Erzurum, Turkey

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Abstract. Ancient masonry towers are regarded as among the most important historical heritage structures of the world. These slender structures typically have orthogonal and circular geometry in plane. These structural forms are commonly installed with adjacent structures. Because of their geometrical shapes and structural constraints, ancient masonry towers are more vulnerable to earthquake damage. The main goal of the paper is to investigate the seismic behavior of Erzurum Clock Tower under earthquake loading and to determine the contribution of the castle walls to the seismic performance of the tower. In this study, four three-dimensional finite element models of the Erzurum Clock Tower were developed and the seismic responses of the models were investigated. Time history analyses were performed using the earthquakes that took place in Turkey in 1983 near Erzurum and in 1992 near Erzincan. In the first model, the clock tower was modeled without the adjacent walls; in the second model, the clock tower was modeled with a castle wall on the south side; in the third model, the clock tower was modeled with a castle wall on the north side; and in the last model, the clock tower was modeled with two castle walls on both the north and south sides. Results of the analyses show that the adjacent walls do not allow lateral movements and the horizontal displacements decreases. It is concluded that the adjacent structures should be taken into consideration when modeling seismic performance in order to get accurate and realistic results.

Keywords: masonry clock towers; seismic assessment; adjacent structures; finite element method; seismic behavior; time history analysis

1. Introduction

Historical masonry towers, such as masonry clock towers, watchtowers, bell towers and minarets, are among the most important ancient structures. They are likely to exist in every ancient city. Today, thousands of these towers are still in use all over the world. Historical masonry towers carry the traces of the previous civilizations by means of sizes, construction style and materials and reflect the characteristics of their period. Therefore, these structures are considered to be a critical part of cultural heritage in the world and their preservation is crucial for future generations.

Masonry tower are generally built in the form of a square and cylinder and are slender and tall

*Corresponding author, Assistant Professor, E-mail: ferit.cakir@amasya.edu.tr
[a]Ph.D., E-mail: huysal@atauni.edu.tr

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structures. These structures are usually installed adjacent to the sides of other structures or are subsequently added to existing structures. Their geometrical forms and structural constraints significantly affect their seismic performance. To better understand the structural behavior of these towers, information about the effects of adjacent structures including structural forms and geometries are very important. Although the structural behavior of masonry structures is a critical research area, masonry towers have been rarely investigated. Some scientists have started to study the seismic performance of historical towers in the last decade. For example, Bernardeschi *et al*. (2004) conducted one of the pioneering studies about historical masonry towers. The structural behavior of Buti's bell tower, located in the Pisa Mountains, and distribution of cracking were investigated using numerical models in their study. Similarly, Carpinteri *et al*. (2005) modeled Torre Sineo masonry tower in Italy and performed an in-situ assessment including possible damage evolution. Moreover, Ivorra and Pallares (2006) analyzed a church bell tower in Spain and investigated the tower's dynamic behavior. Dogangun *et al*. (2008) studied the behavior of masonry minaret structures during the severe 1999 Kocaeli and Duzce, Turkey earthquakes. The authors modeled three different minarets and analyzed their dynamic performance. Similarly, Oliveira *et al*. (2012) focused on minaret behavior under earthquake loading.

In almost all of the previous studies, interactions between the tower and adjacent structures were usually ignored or the adjacent structures were considered as restraints. On the contrary, these interactions need to be taken into account for heavy and tall structures such as masonry towers and minarets. Therefore, the main goal of this paper is to emphasize the global dynamic behavior of Erzurum Clock Tower considering the effect of adjacent walls. This study investigates the dynamic interaction between adjacent walls and the clock tower by determining the physical effects of the adjacent walls. Numerical analysis results including the calculated stress distributions were used to identify critical parts of the structure susceptible to seismic damage.

2. Erzurum clock tower

Erzurum is located on the historical Silk Road in Eastern Turkey and it has been conquered by numerous ancient civilizations throughout its history. Hence, Erzurum has been a host of several centers of civilization and culture and has imposed many of these different cultural effects in its architectural styles. Thus, Erzurum has numerous historical monuments and structures. One of the most notable examples of these cultural heritages is Erzurum Castle. Erzurum Castle is an historical icon of Erzurum and Turkey.

Erzurum Castle has two different parts, which can be defined as the interior citadel and the exterior castle walls. The exterior walls have been mostly ruined while the interior walls citadel remained in good condition. Although the date of is unknown, it is claimed that Roman Emperor Theodosius II built the citadel structure in order to defend the city in the fifth century A.D. They are traces of work that indicate the citadel has been exposed to several attempts of restoration. The citadel consists of a small Castle Mosque (Citadel Mosque) and Clock Tower (Citadel Tower). According to historical inscriptions, Muzaffer Gazi Bin Ebu'l Kasım, Saltukids Amirs in the 12th Century (Gündogdu 2011), subsequently added the small Castle Mosque and Clock Tower to the citadel. These structures were situated in the southwest side of the citadel and the clock tower was originally built as a minaret as shown in Fig. 1. The minaret was carefully installed at the corner point of the citadel and was constructed between two filled, adjacent walls on the north and south

Fig. 1 Citadel mosque and clock tower

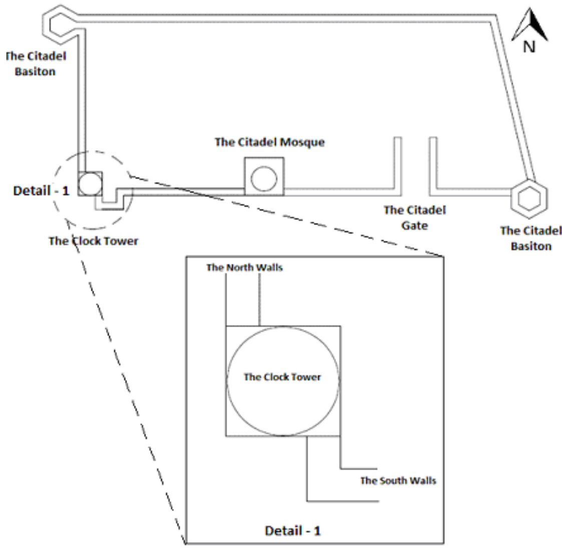

Fig. 2 Citadel and clock tower plan

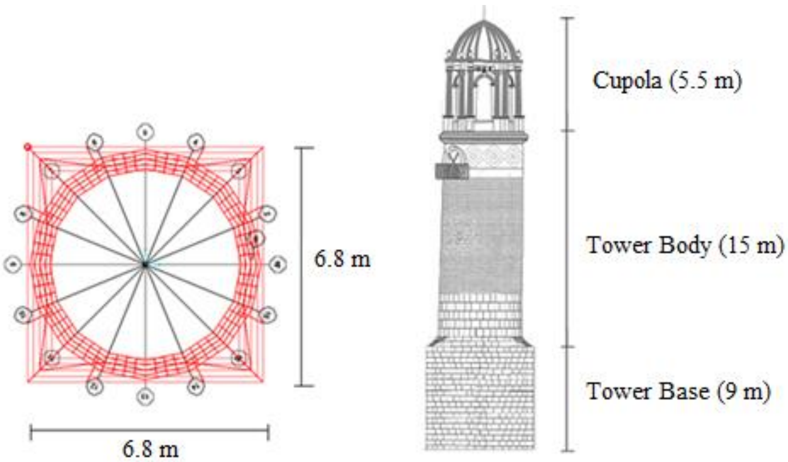

Fig. 3 Sketch of the clock tower

side (Fig. 2). This minaret had a balcony, which was placed on top of the minaret; however, this balcony disappeared for unknown reasons. After the disappearance of the balcony, the minaret served as a watchtower. Moreover, a clock mechanism, cupola (small dome) and bell of the clock were installed in the 19th century and the watchtower turned into a clock tower. (Gündogdu 2011). As of today, the clock mechanism of the body and bell of the clock included in the cupola do not work properly and the tower is visited as a historical heritage site.

The clock tower in the southwest corner of the citadel is composed of three parts. These parts are the square stone tower base, the cylindrical solid brick body, and the semi-sphere timber cupola (Fig. 3). The tower base presents a square plan with 6.8 m per side, a cylindrical tower body that has a 6.6 m diameter and the cupola, a semi-sphere, with a diameter of 6.0 m. The thickness of the walls at tower base and body vary between 60 - 75 cm and the tower base is the lowest part of the tower that meets the ground (Uysal and Cakir 2013). The tower base is composed of cut block, igneous stones, which are 30 × 30 ×5 0 cm in size. Although igneous stones are not malleable, they have been used in early times for structures in Turkey. In addition, traditional mortar, which is generally known as Horasan Mortar, is used for bond between stone surfaces. Mortar is an integral part of masonry structures and generally is used to fill the gaps between construction blocks.

All loads, including dead loads, live loads and lateral loads, are transferred to the supporting soil via the tower base. The 9.0-meter tall tower base is technically the most rigid part of the structure. The body part is composed of the solid baked bricks, which have 5 × 5 × 25 cm dimensions in common. The height of the cylindrical body is 15.0 m and was built with cut stones and solid bricks. There are spiral stairs that proceed in a counter clockwise rotation. The staircases consist of stone steps and surround the internal space of the cylindrical structure. The last part of the tower is the cupola, which has a height of 5.5 m. It is made of a timber shell and is anchored to the body with the timber frames. The dome is covered with lead roofing and the bell of the clock is placed in the cupola.

3. Recursive matrix form

Historical towers are at risk when encountered with seismic events; and these towers are usually deficient in resisting seismic loads. Erzurum and its surrounding cities are in areas prone to seismic activities. Therefore, many destructive earthquakes have taken place in this area in the past. Seismic records show that the most powerful events occurred on October 30, 1983 in Erzurum and March 13, 1992 in Erzincan.

The Erzurum earthquake in 1983, which is also known as Erzurum - Kars earthquake, had an estimated magnitude of 6.9 on the surface wave magnitude scale. The earthquake resulted in 1155 people dead, 537 injuries, 3241 severely damaged buildings, 3000 moderately damaged and 4000 slightly damaged buildings (NEMC 2012). Another destructive earthquake was the Erzincan earthquake, which took place in 1992. Erzincan is located at a seismic prone zone near Erzurum. When the Erzincan earthquake occurred, it not only affected Erzurum city but also other neighboring cities. The earthquake had an estimated magnitude of 6.8 on the surface wave magnitude scale and resulted in 497 deaths, 2000 injuries, 4157 severely damaged buildings in addition to the 5453 moderately damaged and 7867 slightly damaged buildings (Doğangün *et al*. 2008; NEMC 2012).

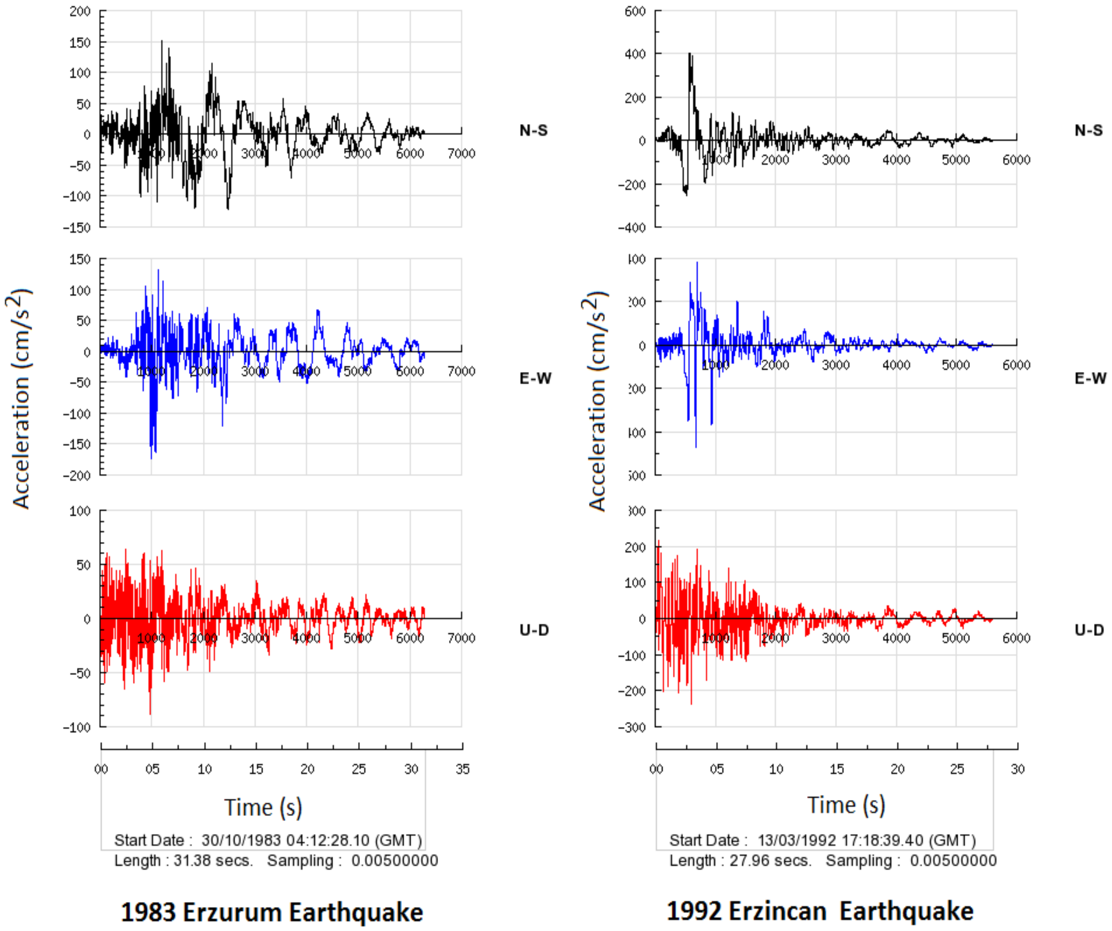

Fig. 4 Ground motions on Erzurum and Erzincan earthquakes (SGMD 2012)

Earthquake loads include ground movement and the formation of seismic waves. In this study, the horizontal ground motions record (North-South) are taken into account for the dynamic analysis of numerical models. Fig. 4 presents the ground motion records of Erzurum and Erzincan earthquakes for North – South (N-S), East – West (E-W) and Up – Down (U-D) directions.

4. Numerical models and dynamic analysis of the clock tower

Many masonry towers are located in the earthquake prone regions of the world; and the majority of these towers are considered to be seismically unsafe. They need to be retrofitted with convenient restoration methods to repair and prevent further earthquake damage. To determine the seismic protection requirements for these towers, a better understanding of their behavior, structural integrity and failure mechanisms are needed. Engineers have retrofitted structures to withstand earthquakes for many years. These ancient seismic retrofits have been discovered when

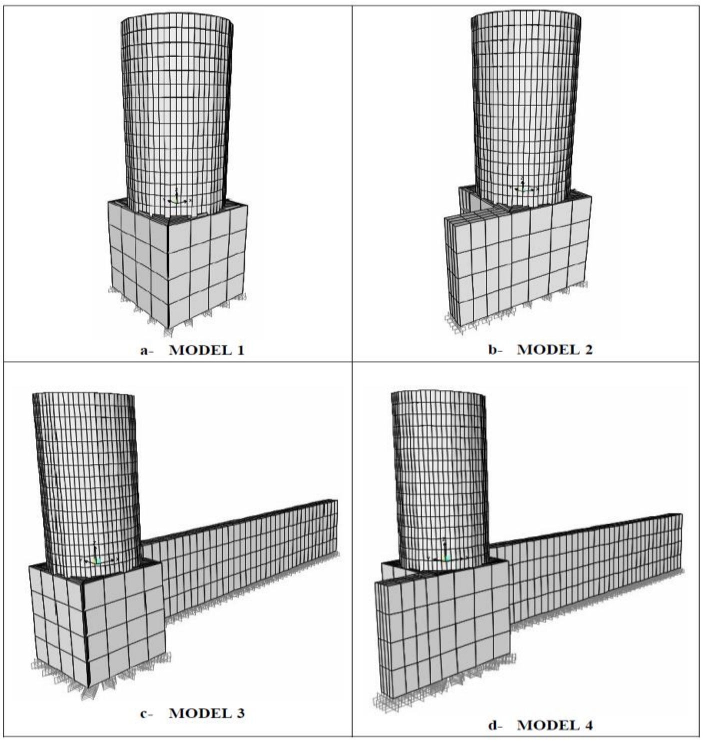

Fig. 5 Four types of three-dimensional finite element models for Erzurum Clock Tower

historical buildings in earthquake areas were inspected. Those ancient examples of seismic protection have contributed to the current retrofitting projects and revealed the use of appropriate restoration methods for many structures (Ahunbay 2009). Seismic retrofitting of masonry structures has significantly advanced in recent years through the better understanding of their structural behavior, appropriate analysis methods, and seismic design practices. Therefore, this study essentially focuses on conducting a comprehensive structural analysis with the use of finite element method in order to investigate the seismic behavior of Erzurum Clock Tower.

Three-dimensional, finite element models have been developed based on the structural state and the geometrical constraints of Erzurum Clock Tower. In the first case the clock tower has been modeled without the adjacent walls. In the second case the clock tower has been modeled with a citadel wall at the south side that has a length of 4 m in the -X direction. In the third case the clock tower has been modeled with a citadel wall at the north side, which has a length of 40 m in the X direction. In the fourth case the clock tower has been modeled with two citadel walls at both north and south sides, which are also X and –X directions.

For all the models, the computer software, SAP2000 (2012), was used and the models have been analyzed during the 1983 Erzurum and the 1992 Erzincan earthquakes. The clock tower was discretized with 4352, 4416, 4992 and 5056 solid elements with corresponding 5836, 5956, 6856 and 6976 nodes for the 1, 2, 3 and 4 cases, respectively (Fig. 5). The real solid elements are used in order to observe the stresses and deformations along the wall thickness.

For a better investigation of the tower’s behavior, the cupola, itself, was not modeled. The copula's mass was added to each model though. The main objective of this paper is to determine the contribution of the adjacent walls to the seismic performance of the tower. Therefore, the staircase, the cupola and their influence on the main structure were not taken into account in this study. Thus, the staircase and the cupola were neglected and this simplification leaded to easier interpretations of the results obtained by FEA. Furthermore, the internal spiral stairs were not considered because Turkish code standards and Eurocode standards do not include specific guidelines for including spiral stairs (Dogangun *et al*. 2008).

4.1 Material properties

According to the Masonry Standards Joint Committee (MSJC 2005), the modulus of elasticity is determined as a function of masonry compressive strength for clay masonry. Thus, modulus of elasticity values (E_m) for the design of clay masonry are calculated based on $E_m = 700f'_m$ for clay masonry; where f'_m is the specified compressive strength of masonry in MPa.

According to the Turkish Earthquake Code (TEC 2007), the modulus of elasticity (E_d) for new masonry units used in wall construction can be calculated by $E_d = 200f_d$, where f_d is the pressure strength of masonry wall in MPa. In addition, according to Ü nay (2002), the modulus of elasticity (E_m) can be computed based on the following equations as well $E_m = 1000f_k$, where f_k is characteristic pressure resistance of masonry material in MPa.

However, masonry structures are generally made of stones and bricks and these construction materials are connected with mortar, which has significantly smaller modulus of elasticity (Seker 2011, Camlibel 1998). Therefore, new composite materials should be derived and new engineering properties should be counted considering bricks, stones, and mortar with regard to their properties and dimensions. Hence, according to Lourenço *et al*. (2001), the modulus of elasticity for new composite material might be obtained from homogenization procedures such as

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

where t_m represents the thickness of the mortar, t_u represents the height of the brick or stone and ρ states an efficiency factor regarding the deficient bond between the two materials. Therefore, the modulus of elasticity for cut stones + mortar and solid bricks + mortar can be calculated with the formula above. Hence, in all three-dimensional finite element models, engineering properties of materials such as the modulus of elasticity, poisson's ratios and mass per unit volume were adopted from similar research studies and were calculated using the above formulas. The scope of this paper deals mainly with solids and structures of elastic materials. In addition, this paper considers only the problems of very small deformations where the deformation and the load have linear relationship. Therefore, our problems will mostly be linear elastic. Hence, linear elastic material behavior is considered and the stiffness degradation is ignored in this study. The engineering properties used in all numerical analyses are summarized in Table 1.

Table 1 Engineering properties of the materials (Seker, 2011, Dogangun *et al*. 2008)

Materials	Modulus of elasticity (kN/m^2)	Poisson's ratio	Mass per unit volume (kg/m^3)
Stone + Mortar	8.0E6	0.28	2500
Brick + Mortar	3.8E6	0.18	1800

4.2 Finite element analysis

Modal analysis and dynamic time history analysis were used for all models. The obtained analysis results were too complicated to present each node or element. Because of this, contour pictures, bars and scale tables were used to present analysis results. Primarily, all models were analyzed by modal analysis method before time history analysis. The first five mode periods and directions, which were determined by modal analysis, are given in Table 2. According to obtained modal analysis results, the lateral translation has been decreased; torsion has occurred at lower frequencies of vibration. In addition, mode shapes have changed in the case of adding the adjacent walls to the tower.

After modal analysis, all models were subjected to time history analysis during the Erzurum and the Erzincan earthquakes. In all models maximum stresses occurred in between the tower base and the tower body, just above the tower base. This section is also the transition zone for two different geometries and materials. According to Dogangun *et al*. (2008) and Sezen *et al.* (2008), this transition zone is prone to failure for masonry structures and reinforced concrete. Because of that, this part of the tower is the most critical section for seismic behavior.

According to element local coordinate system, S11, S22 and S33 are the direct stresses acting on the faces in 1, 2 and 3 axis direction respectively. $S_{\max}$ is the maximum principal stress and $S_{\min}$ is the minimum principal stress. In this study, the maximum value of the maximum principal stress is the tensile stress and the minimum value of the minimum principal stress is the compression stress. The maximum and minimum principal stress, $S_{\max}$ and $S_{\min}$, contours for all tower models for the Erzurum earthquake were given in Figs. 6-7 and for the Erzincan earthquake in Fig. 9-10. In addition, the maximum principal stresses, the minimum principal stresses, and the direct stresses were given as stress bars for the Erzurum and Erzincan earthquakes in Fig. 8 and Figure 11, respectively. As seen from the figures, the maximum and minimum principal stresses were calculated around the transition zone of the all models. It was observed that stress values increased in the first case (model 1) and in uniform on stress distributions occurred in the case of adding a wall to the initial case in the –X (south), +X (north) directions or both directions.

When an adjacent wall was added to the tower on one side (either –X or X direction), the stress distribution became non-uniform. However, when two adjacent walls were added to the tower on both sides, the stress distribution became better in comparison to the first model case. In addition, stress distribution in model 4 was much better and stress values were lower than other models. Consistent with the observed performance, the stress contours show that both the largest compressive and tensile stresses were concentrated in the tower body within a few meters above the transition segment. Results of the analyses prove that the most valuable section of failures is the transition zone during the earthquakes.

The lateral displacements at the top of the tower, which were calculated by time history analysis of all models, are given in Fig. 12. The maximum lateral displacement was calculated as 0.114 m in Model 1 for the Erzincan earthquake and the minimum lateral displacement was calculated as 0.024 m in Model 4 for the Erzurum earthquake. When the displacements of the tower were examined, the lateral displacements of the first case were the calculated maximum of all models. As the first adjacent wall (south side) was added to the tower, the lateral displacements decreased almost 25%. When the second wall was applied (third case), the lateral displacements decreased almost 35% and, finally, when two adjacent walls were added (south and north sides), it was reduced to almost half the original lateral displacements. Indeed, as the adjacent walls were

Table 2 The first five mode periods and directions

		1st	2nd	3rd	4th	5th
Model 1	Period (s)	0.20249	0.19757	0.09262	0.06718	0.06683
	Direction	Y	X	Torsion	Y	X
Model 2	Period (s)	0.20038	0.17602	0.09256	0.08067	0.06897
	Direction	Y	X	Torsion	Torsion	Torsion
Model 3	Period (s)	0.19803	0.17064	0.16255	0.13107	0.09640
	Direction	Y	X	Y	Torsion	Torsion
Model 4	Period (s)	0.19634	0.16260	0.15874	0.13102	0.09633
	Direction	Y	Y	X	Torsion	Torsion

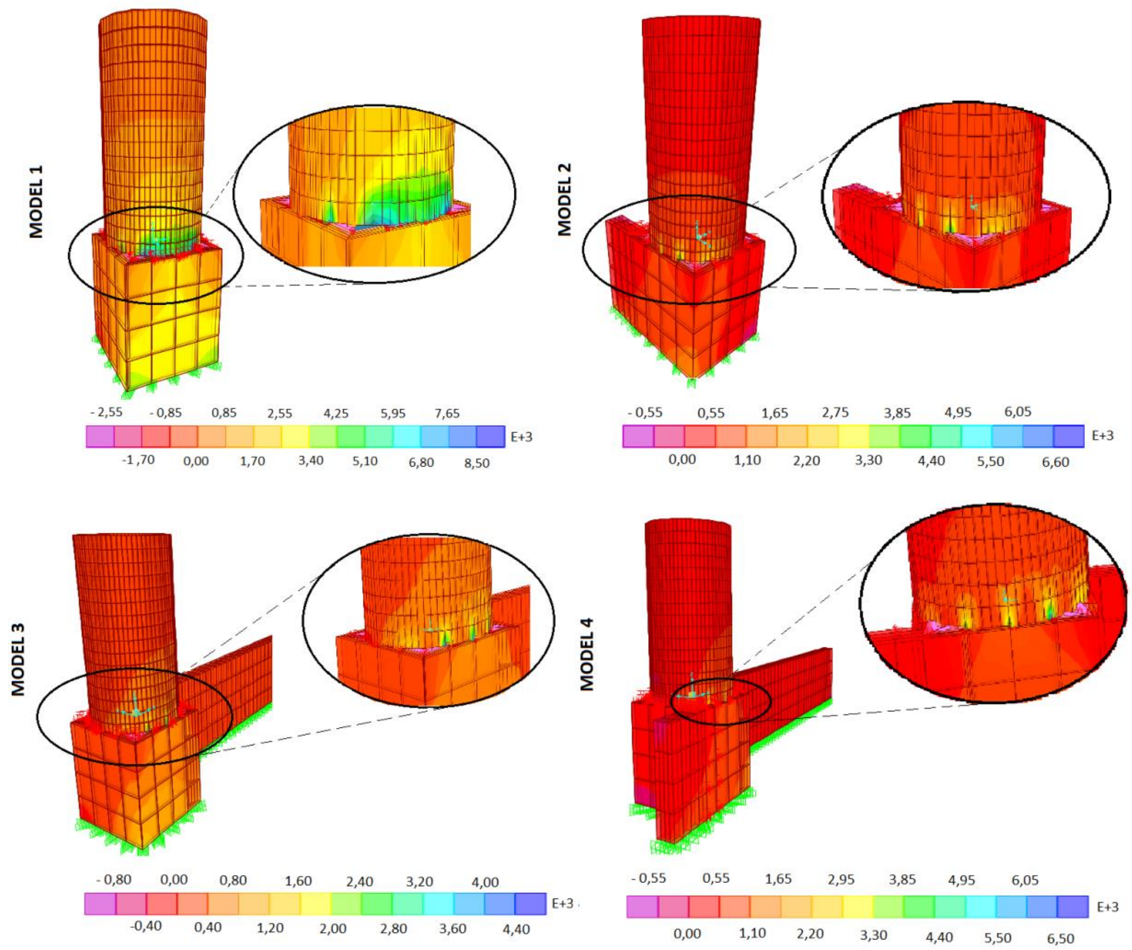

Fig. 6 Maximum principal stress contours, S_{max}, for Model 1, Model 2, Model 3 and Model 4 for 1983 Erzurum earthquake records (kN/m^2)

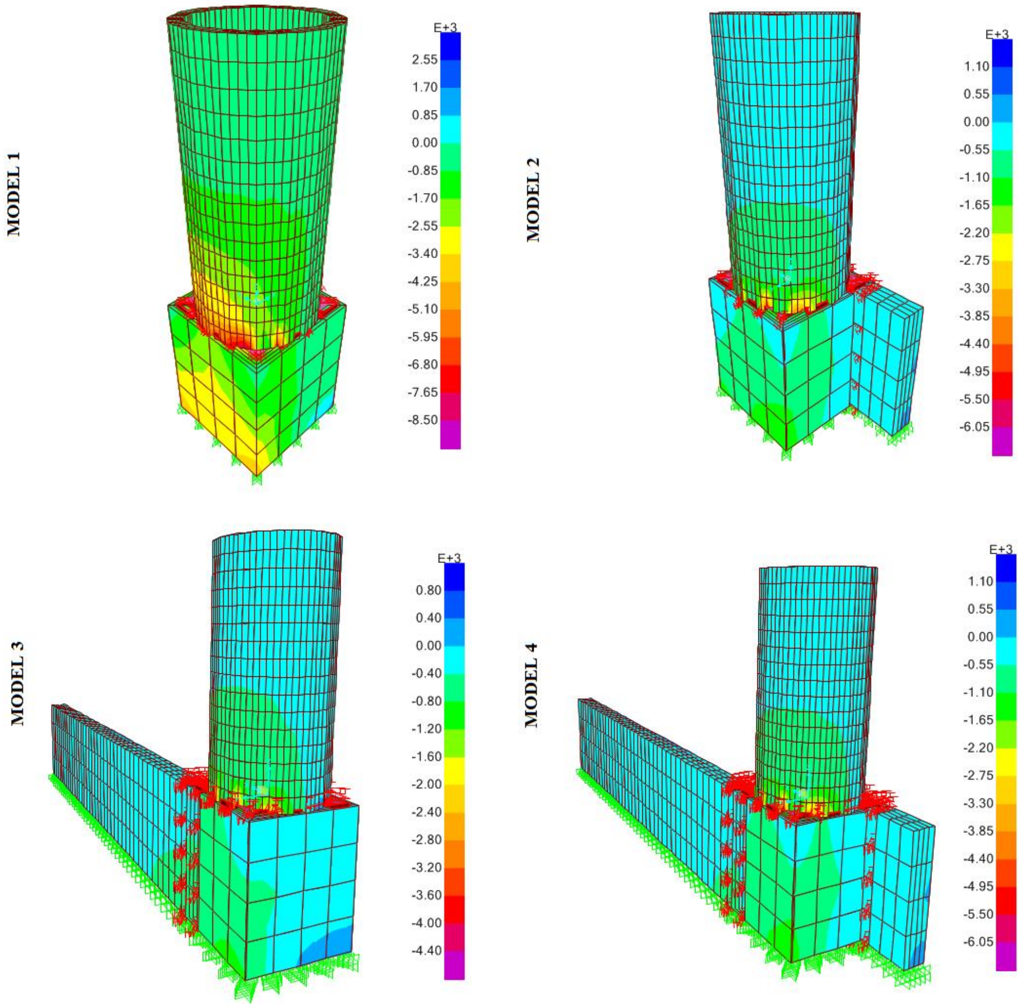

Fig. 7 Minimum principal stress contours, S_{min}, for Model 1, Model 2, Model 3 and Model 4 for 1983 Erzurum earthquake records (kN/m^2)

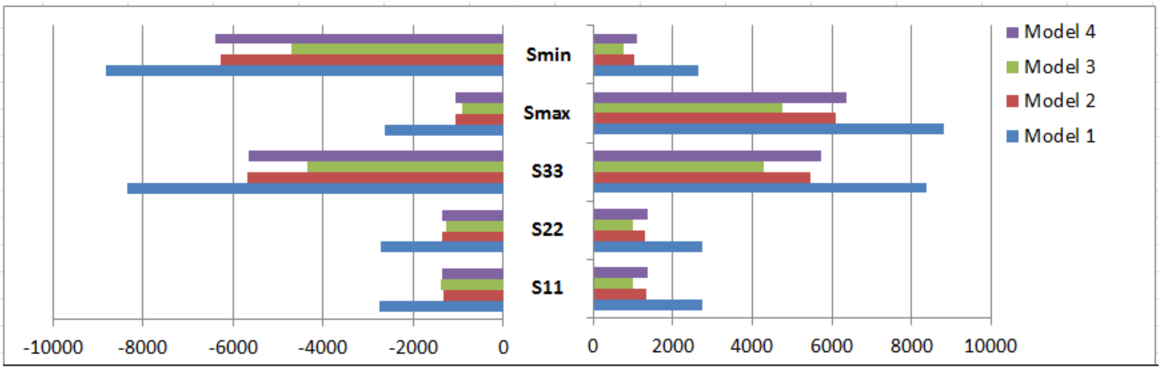

Fig. 8 Stress bars for all models for 1983 Erzurum earthquake (kN/m^2)

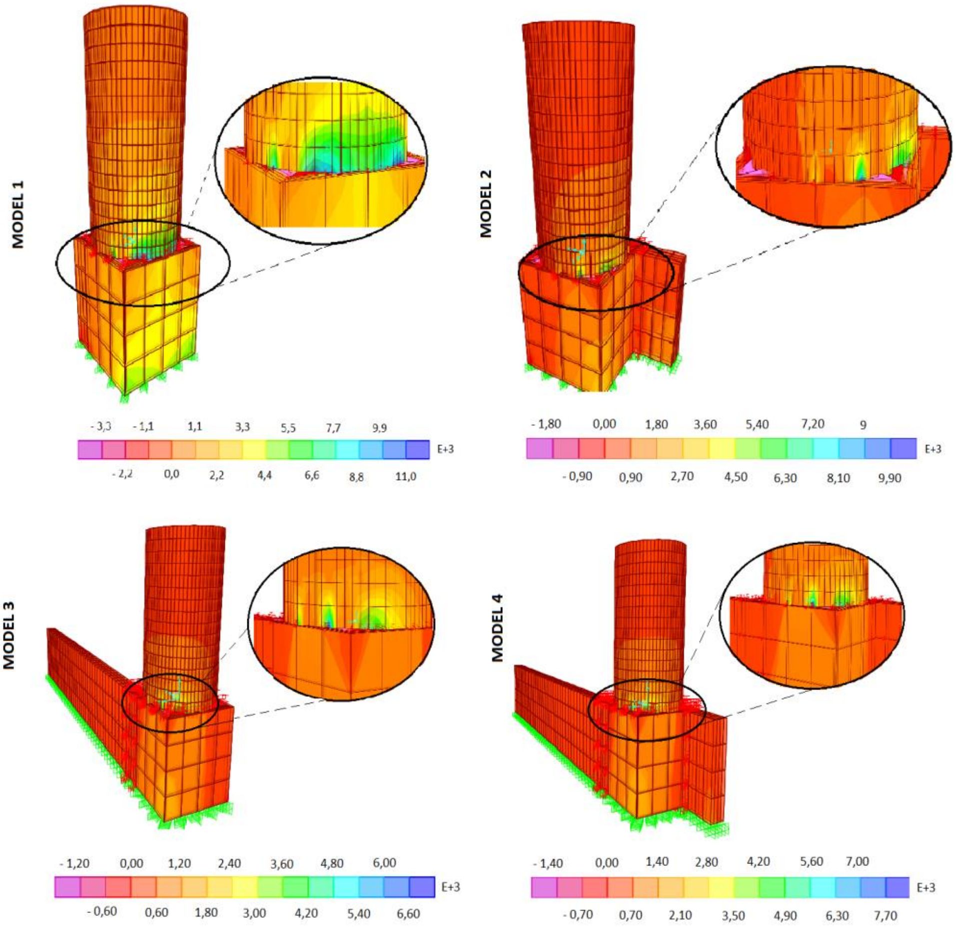

Fig. 9 Maximum principal stress contours, S_{max}, for Model 1, Model 2, Model 3 and Model 4 for 1992 Erzincan earthquake records (kN/m^2)

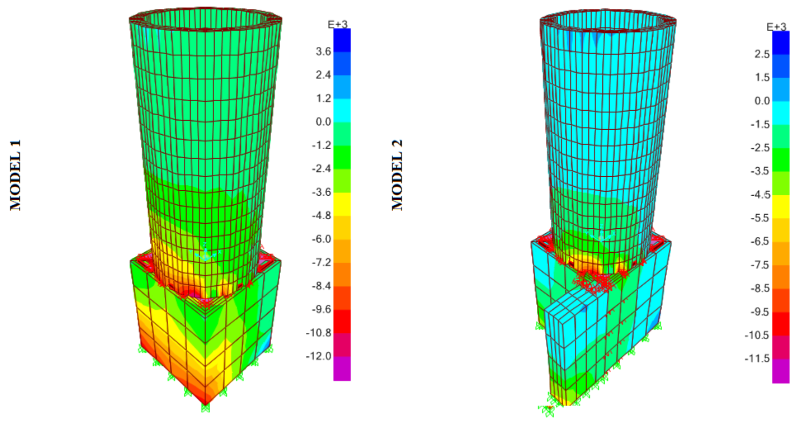

Fig. 10 Minimum principal stress contours, S_{min}, for Model 1, Model 2, Model 3 and Model 4 for 1992 Erzincan earthquake records (kN/m2)

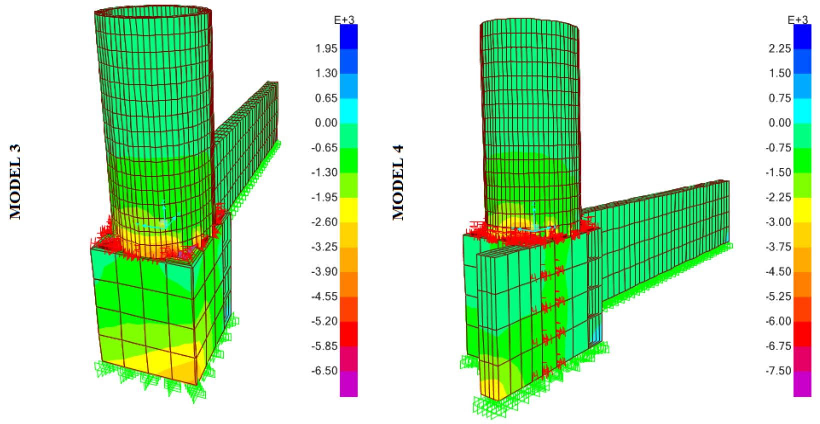

Fig. 10 Minimum principal stress contours, S_{min}, for Model 1, Model 2, Model 3 and Model 4 for 1992 Erzincan earthquake records (kN/m2)

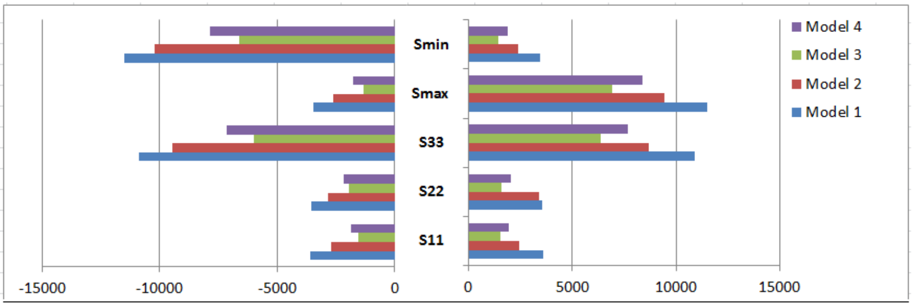

Fig. 11 Stress bars for all models for the 1992 Erzincan earthquake (kN/m^2)

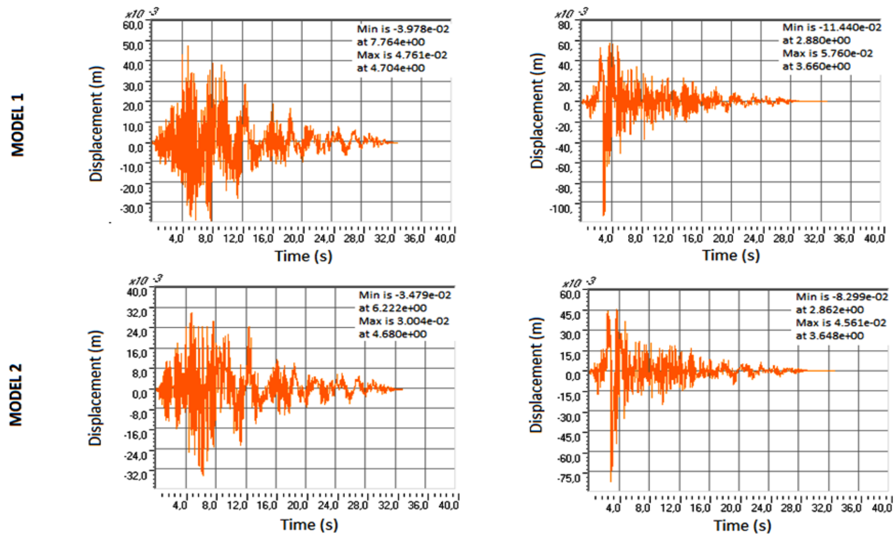

Fig. 12 Continued

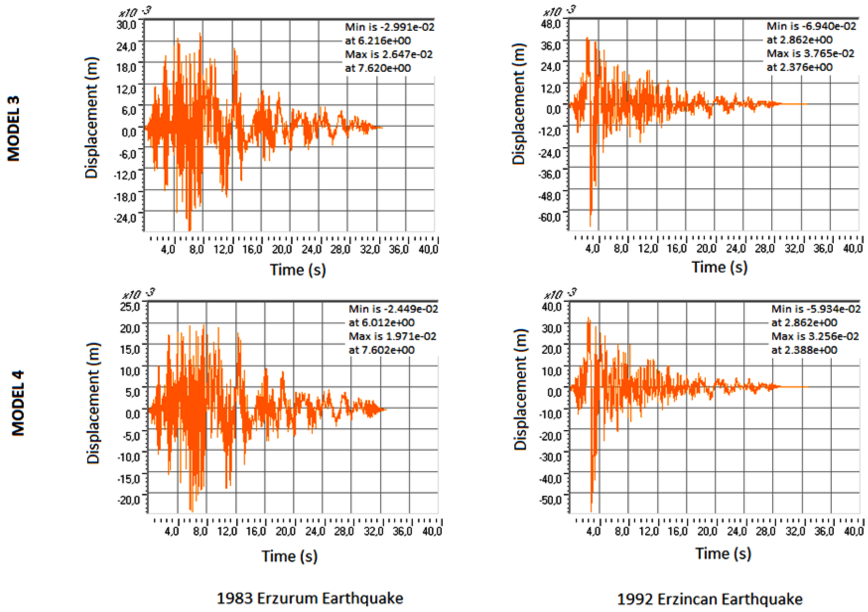

Fig. 12 Lateral displacement at the top of the towers subjected to (a) Erzurum, (b) Erzincan earthquakes

added to the tower, the lateral displacements became subsequently smaller. According to Dogangun *et al*. (2008) and Uysal and Cakir (2013), all displacements are below allowed maximum displacement value for slender and tall masonry structures.

$$\Delta_{i\max} \leq \frac{0.02 \cdot h_i}{R} \tag{2}$$

where h_i is the tower height, and R is the behavior factor related to the ductility of structure. If this requirement is applied to the Erzurum Clock Tower (h_i = 24 m, R = 2), the corresponding maximum allowable top displacement is 0.24 m. The static and dynamic displacements achieved are lower than the values generated through the formula. In this respect, it is seen that the maximum displacement values are in the allowable limits.

5. Conclusions

Historical masonry towers are one of the most important structures in terms of their age, intended uses, heritage value, and structural properties. Tower structures are usually attached to other structures. The seismic behavior of masonry towers depends mainly on the material properties, geometries, adjacent structures, and earthquake characteristics. This study investigated Erzurum Clock Tower in Turkey, which was constructed integral with adjacent walls. The effect of adjacent structure was determined under different earthquake scenarios. Four different three-dimensional models were developed and dynamic behavior of the models was investigated through time history analysis.

Results of the analyses show that the dynamic interaction between the tower and adjacent walls plays an important role in the dynamic behavior of the structure. Since the adjacent walls do not allow lateral movement, the adjacent walls added to the tower decreased the horizontal displacements. Critical stresses were calculated in the region between the tower base and tower body, typically a few meters above the top of adjacent walls. In previous studies, the effects of adjacent structures were generally ignored or they were modeled as constraints. However, the tower structure and adjacent structures and their geometries should be modeled together for seismic analysis in order to better understand the dynamic interaction and behavior. It is concluded that the adjacent structures should be taken into consideration into the models in order to get more accurate and realistic results. Different materials, geometrical forms, and different earthquake ground motions must be studied further to better understand main and adjacent structures' dynamic effect and interaction.

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