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EARTHQUAKE ANALYSIS AND STRENGHTHENING OF THE HISTORICAL MEHMET AGA MOSQUE IN ISTANBUL

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ABSTRACT :

Istanbul is one of the oldest cities in the world and it was the capital of several empires. The city has many historical buildings from various civilizations, especially from the Ottoman Empire, including mosques, mausoleums, bathhouses, bridges and aqueducts. On the other hand Istanbul is very close to a major earthquake source of Turkey, which is called the North Anatolian Fault Line. There are several estimations stating that a major earthquake is likely to strike Istanbul in a short time. Several works to mitigate the effect of a probable earthquake have been done by the Istanbul Metropolitan Municipality and one of them is related to the seismic evaluation of the historical buildings in the city. The present study was carried out within this work and dealt with the seismic evaluation of a historical mosque in Istanbul. Mehmet Aga Mosque was constructed in 1585 by Architect Davud Aga. The building was subjected to various strong ground motions several times without having a major damage. For the detailed investigation of the building existing damages were determined by in situ survey. A 3-D model of the building was prepared and structural and seismic analyses were performed for various characteristic values of the material and seismic excitations. The paper presents a summary of the results of the structural response analyses of the building, including its response under gravity and seismic loads. The stress concentrations in the walls and the crack patterns were compared with the existing damages for interpretation of the results and various strengthening methods for increasing the structural and seismic performance of the building were discussed.

KEYWORDS: historical structures, modeling, seismic analysis, repair and strengthening

1. INTRODUCTION

Mehmet Aga mosque located in Fatih province of Istanbul, which is one of the oldest cities in the world. Istanbul was the host of several empires such as Byzantine Empire and Ottoman Empire and the city has many historical heritages from several civilizations, especially from Ottoman time, such as mosques, churches, bathhouses, bridges and aqueducts. The city and consequently the building is very near to the North Anatolian Fault Zone, which is one of the most important seismic sources in Turkey. The building was subjected to several seismic effects so far; the most important ones are 1766, 1894 and 1999 earthquakes. Two interventions were performed ever since in the building; a repair in 1743 and a restoration in 1982 (Branch of Fatih of P.R.A, 1991). Existing damages of the building were investigated by in situ survey. In the present study, gravity and earthquake analyses were carried out using design and maximum earthquakes defined by the Turkish Seismic Code (2007) and the results are given comparatively and damage reasons were investigated. In fact, in the present situation, the existing damages do not threat the integrity of the building. However, some repair and strengthening were recommended in order to prevent the possible future damages (Gedik, 2008).

2. DESCRIPTION OF THE MOSQUE

The mosque was constructed in 1585 by Architect Davud Aga, who is a student of the great Ottoman Architect Sinan. It was commissioned by Mehmet Aga. The building has been used ever since. The mosque has a main area of square shape in plan. The main area is covered by a main dome having a height of 17.3 m and a diameter of 11 m. There are pendentives and tromps as secondary components for covering the mosque. The main dome is supported by four pilasters and by four corners of the external walls. There is a wooden balcony having a height of 2.8 m inside the mosque. The external walls of the historical building consist of one stone row alternated with three brick rows having an average thickness of 1.3 m. The main dome is made of brick. The cloister of the mosque consists of five parts which are covered by five small domes. The domes are supported by the six marble columns. All arches of the mosque have iron lateral ties. The mosque has a minaret having a height of 29 m and it is made of ashlars. There are still original nice ceramics, beautiful calligraphies and handicrafts on the walls; however most of them were changed (Figure 1 and 2).

Figure 1 Exterior and interior view of the mosque

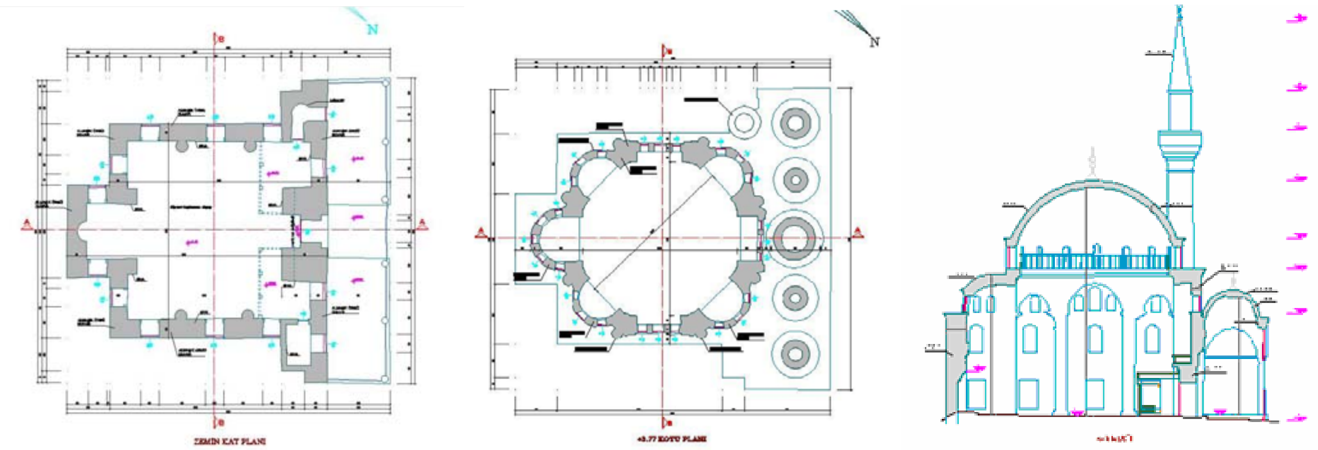


Figure 2 Sections and elevation of the mosque

3. EXISTING DAMAGES

The mosque has experienced several earthquakes including the great earthquakes of 1766, 1894 and 1999. It was repaired in 1743 and restored in 1982 (Branch of Fatih of P.R.A., 1991). Moreover, the mosque is still in service and regular maintenance works for the building has been done continuously. Consequently, the damages of the past earthquakes could not be identified easily by simple in situ investigations. In addition, no damage originating from the soil is observed. Damages appear at the edges of window openings as cracks. Some ashlars of the cloister are deteriorated. One of the tromps has slight radial cracks and damages. Buckling and corrosion are observed in several iron ties of the arches (Figure 3).

Figure 3 Observed damages

4. SEISMICITY OF THE SITE

The mosque is located in a highly active seismic region of Turkey. It is very close to the North Anatolian Fault Line, which runs at the south of the city and is one of the major earthquake sources of Turkey. Several estimations predict that a great scale earthquake will probably strike Istanbul in a very near future. Thus, seismic safety of the mosque is important issue. The mosque is located in a region having the second highest seismic risk according to the Turkish Seismic Zone Map (http://deprem.gov.tr/linkhart.htm). Nevertheless, the mosque was assumed to be located in the highest seismic zone having a maximum effective ground acceleration of 0.4g according to the Turkish Seismic Code (2007) due to the historical value of the mosque.

5. SOIL CONDITION

Subsoil of the mosque does not have any slope. Geotechnical microzonation investigation, which has been carried out by Istanbul Metropolitan Municipality, was used in this study for determining the soil properties. The data were obtained by the consultation with the municipality officers. The subsoil of the mosque located on Gungoren Formation that is one of the main soil formations of Istanbul. The subsoil of the mosque consists of firm-stiff consistency, high plasticity clay (CH). In addition, there are partial gravel and sand strata in part. SPT values are given to be in between 20 to 40. Specific gravity, cohesion coefficient and shear friction angle of the subsoil are assumed to be γ = 18 kN/m^3, c = 60 kN/m^2 and $\phi = 10^0$ respectively. Vertical soil stiffness is assumed to be K = 20 MN/m^3 and used in the structural analyses by considering the soil properties to obtain the dynamic response of the building. The soil is is considered to be as Z3 class having the characteristic soil periods of T_A= 0.15 s and T_B = 0.60 s according to the Turkish Seismic Code.

6. NUMERICAL ANALYSES

6.1. Modeling

Micro and macro modeling strategies for masonry structures exist due to various accuracy level of the analysis. Macro modeling is better in case of necessity compromising between efficiency and accuracy and it is used when the building has solid walls with large dimensions (Lourenco, 2002). Thus, the mosque was assumed to be made of a unified material that has an elastic modulus E = 2 GPa, a unit weight γ = 20 kN/m^3 and Poison's ratio υ = 0.2. These values were determined by literature survey and by comparing the properties with similar type structures which belong to the same era. The three-dimensional analysis model is developed by using SAP2000 structural analysis software (Habibullah and Wilson, 1998) and architectural plans and sections of the mosque, which are drawn in AutoCAD format. Three different types of elements are used for preparing the analysis model. The external masonry walls were modeled by three-dimensional solid elements, whereas two-dimensional shell elements are used for modeling the domes, tromps, thick arches and pendentives. Pilasters, thin arches and iron ties were modeled by frame elements. Totally, 2418 solid, 1530 shell and 201 frame elements were used for the analysis model (Figure 4). It is important to use reasonable number of finite elements to obtain response of the structure having acceptable level of accuracy. Using large number of elements may yield results where the global behavior of the building can be difficult to extract. However, the very small number of elements is generally not able to capture the behavior of the building with efficient level of accuracy. Various minor simplifications are carried out in order to avoid geometrical complexity of the structure and the number of the finite elements is kept at a minimum level as possible. However, it is elaborated for obtaining the response of the structure with a reasonable accuracy.

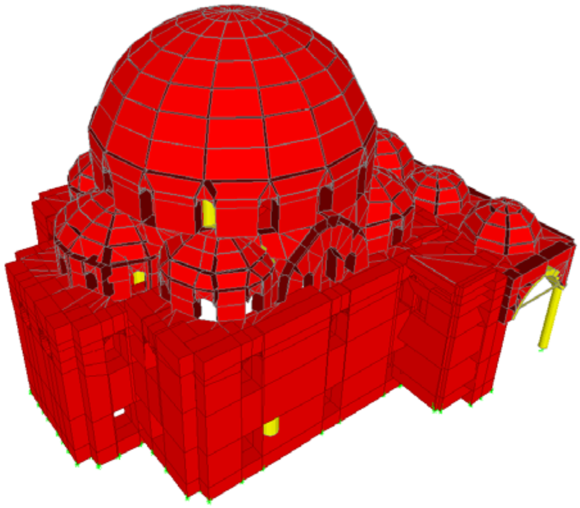


Figure 4 Structural analysis model

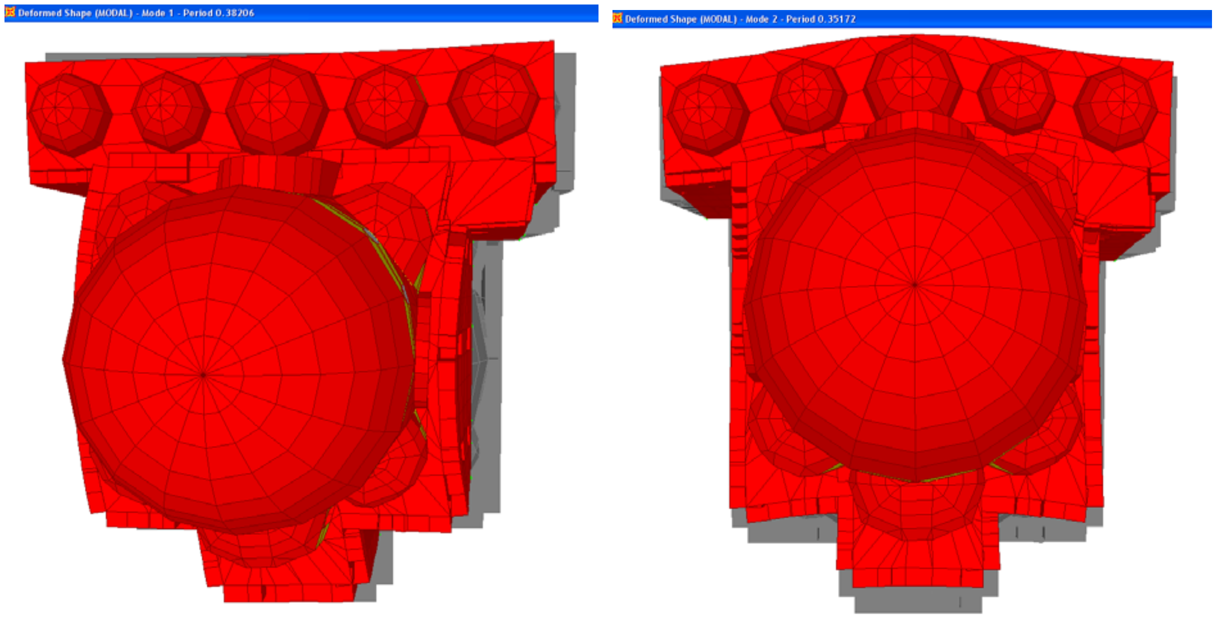
6.2. Self-weight and earthquake analyses

Free vibration, self-weight and spectral earthquake analyses are carried out in order to determine the structural response and the stress variation of the building. Analyses are performed under the assumptions of the linearly elastic material as it is done generally in solid masonry structures in order to obtain the overall structural behavior.

Free vibration analysis is performed and the natural periods of the building are obtained. Additionally, sensitivity analysis is carried out by using various values of modulus of elasticity (E) and of that foundation stiffness (K) in order to determine the effect of these values on the analysis results. Natural periods are given in Table 1 for various E and K values. Table 1 show that the periods are approximately in between the soil characteristic periods. In this interval, the spectral coefficient is constant having the maximum value S (T) = 2.5. It means that the modulus of elasticity and the foundation stiffness do not affect the seismic loads significantly. The first two mode shapes of the building occur in the two orthogonal directions (Figure 5). First and second natural periods are close, which shows the participating mass of the first two modes and the structural rigidity of the two directions are approximately equal to each other (Güler et al., 2004).

Table 1 Free vibration periods of the building (s)

Mode	1	2	3	4	5
T (K = 20 MN/m^3, E = 2GPa)	0.382	0.352	0.201	0.193	0.185
T (K = 20 MN/m^3, E = 1GPa)	0.446	0.418	0.254	0.244	0.213
T (K = 20 MN/m^3, E = 3GPa)	0.354	0.323	0.197	0.159	0.153
T (K = 10 MN/m^3, E = 2GPa)	0.480	0.437	0.277	0.193	0.185
T (K = 30 MN/m^3, E = 2GPa)	0.339	0.315	0.187	0.180	0.167
T (Fixed support, E = 2GPa)	0.229	0.221	0.174	0.147	0.140



(a) (b)

Figure 5 Mode shapes; a) first mode shape, b) second mode shape

Variation of the vertical normal stress due to the self-weight is presented in Figure 6. As expected, vertical compressive stress increases downwards and it reaches about 0.20 MPa at the lowest wall level. As seen in the figure, stress concentrations occur around the openings and the stress at the edges of the lower openings increases up to 0.28 MPa. Small tensile stresses occur at the top levels of the walls due to the out-of-plane loads originating from the dome and tromp. The compressive stress along meridian direction at the bottom level of the main dome is obtained about 0.2 MPa where the maximum tensile stress along circumferential direction at the lower part of the dome is about 0.05 MPa under vertical loads.

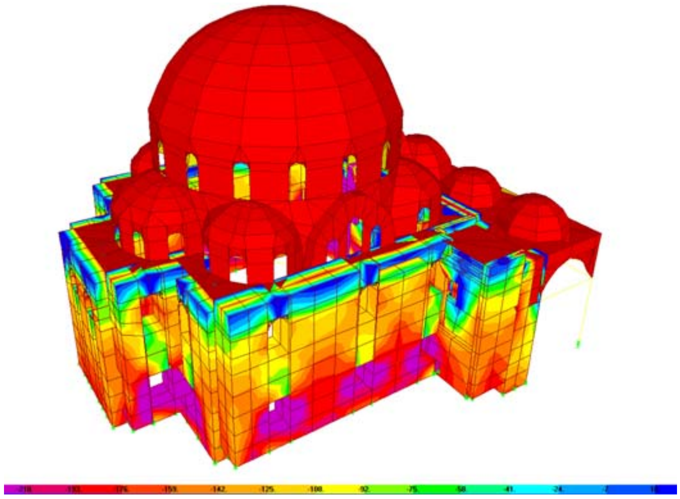
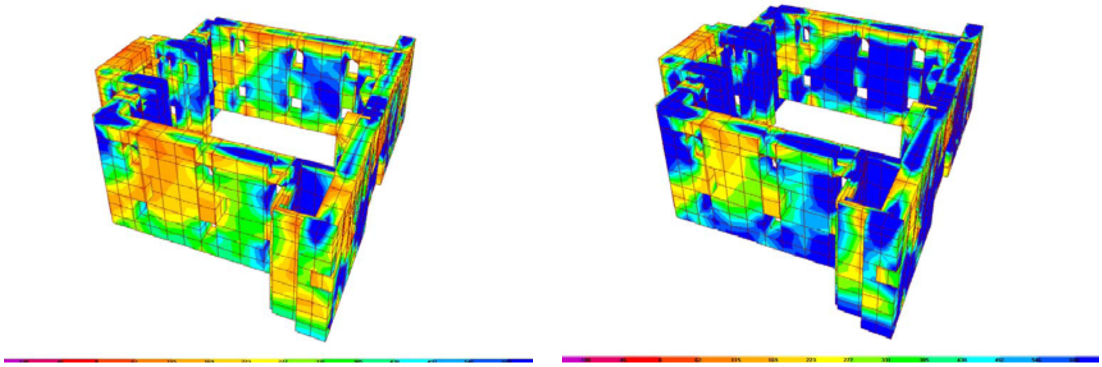


Figure 6 Self-weight analysis- Vertical normal stresses (kN/m^2)

Spectral earthquake analysis are carried out by using the spectra defined in the Turkish Seismic Code (2007) where the design spectra are given for an earthquake having an exceeding probability of 10% in 50 years which corresponds to an event having return period of 475 years and called the Design Earthquake. However, since the present building has an historical value, the spectra is increased by the factor of 1.5 which approximately corresponds to the maximum probable earthquake having an exceeding probability of 2% in 50 years and a return period of 2475 years (Sesigür et al., 2007).

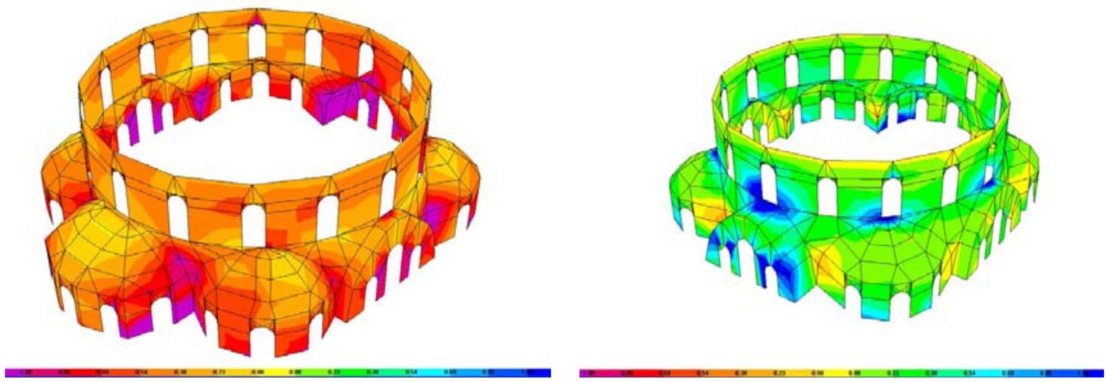
In the numerical analysis, the base shear reduction factor of R_a=2 and the live load participating factor of n=0.3 are adopted, whereas the second factor is of very low effect on the results. For comparison the numerical analysis is carried out for the two spectra, which correspond to the design earthquake and the maximum earthquake. Comparison of the results for the two spectra is given in Figure 7. As it is expected, the vertical normal stress is increases up to 40-50%, when the spectrum of the maximum earthquake is considered.



(a) (b)

Figure 7 Comparison of the vertical normal stresses for the NE-SW earthquake directions (kN/m^2), a) design spectrum, b) maximum spectrum

As it mentioned above, stress concentrations occur at the corners, around the openings of the walls, which support of the main dome, under the self weight and the earthquake loading. These stress concentrations around the openings may explain the cracks that are observed in situ investigations. The maximum normal stress in the structure is around 2 MPa both in tension and compression. The maximum normal stresses in the ties of the arches under the design earthquake loading are around 44 MPa in compression and 80 MPa in tension. Existing buckling damages at the iron ties indicate that the ties are overstressed. The maximum compressive stress along meridians of the main dome appears to be about 0.3 MPa where the maximum circumferential tensile stress is over 0.35 MPa on the lower part of the main dome. Stress concentrations are formed on the pendentives and around the openings in the tromps, pendentives and drums as well (Figure 8). The maximum soil settlement was obtained as 2 cm under the design earthquake loading and the self-weight, it increases up to 2.8 cm for the maximum earthquake loading.



(a) (b)

Figure 8 Stress contours along meridians under the design earthquake (kN/m^2); a) compressive stress distribution, b) tensile stress distribution

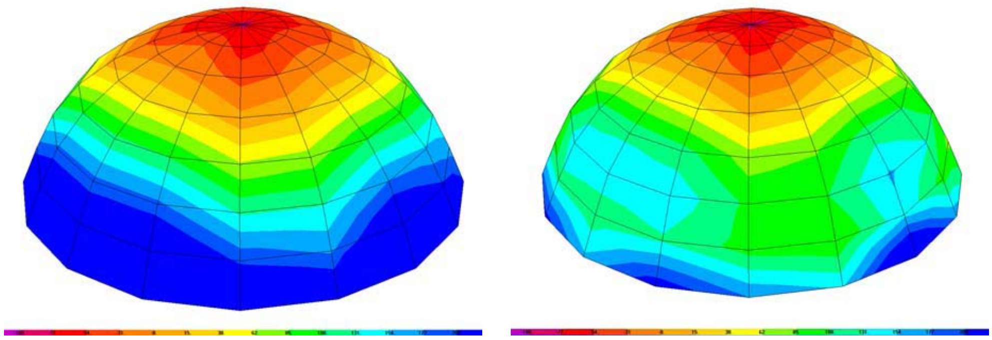
7. REPAIR AND STRENGTHENING

Repair and strengthening of historical structures require special attention. Preservation of aesthetic and historic value of historical structures is primarily important issue (Penelis, 2002). Thus, it is not always possible to apply all intervention methods to historical structures used for ordinary structures. All interventions to historical structures should satisfy the requirements of the Venice Charter 1964 that gives the basic intervention principles for historical heritages.

Venice Charter requires that all interventions for historical structures should be kept at a minimum level as possible. In principle interventions should be reversible. Reversible interventions can be replaced without any reasonable damage if their inefficiency and low durability are proven or better techniques or materials are improved later. Irreversible techniques are permitted in case the reversible techniques are not applicable or sufficient. Material for repair and strengthening must be compatible with original material and it must have enough durability (Penelis, 2002 and Osman, 2005).

7.1. *Strengthening of the main dome*

As it is well-known, tensile stresses are formed along the circumferential direction at lower part of domes. One of the strengthening methods for domes is addition a tensional ring around the bottom level of the domes for reducing the tensile stress and providing an additional measure for ensuring the integration of the dome. In order to determine the effect of this intervention in the present numerical model, the main dome is strengthened by a tensional ring and stress distributions with and without the tensional ring were compared. As seen in Figure 9, the circumferential tensile stresses were decreased significantly under the self-weight and the design earthquake loading when the tensional ring was used.



(a) (b)

Figure 9 Circumferential stress variations along circumferential direction under the design earthquake loading (kN/m^2); a) without the tensional ring b) with the tensional ring

8. CONCLUSION

Masonry walls, domes, arches and vaults generally consist of masonry units and mortar in between them. There are macro and micro modeling techniques for masonry structures. Proper modeling technique is important due to the interpretation of the results. Macro modeling is recommended, when the building has solid walls with large dimensions for necessity of compromising between efficiency and accuracy. Furthermore, the building may be assumed as a unified material having the same material properties. Modeling of historical masonry structures requires reasonable geometrical simplifications. It is shown that, overall structural behavior of the building can be obtained by using a reasonable number of finite elements. Furthermore, the numerical results yield that the stress variations are insensitive to the modulus of elasticity and the foundation stiffness that is uniformly distributed under the foundation of the building in this study. The first two mode shapes of the building come into being as a result of lateral displacements and occur in two orthogonal directions. However, higher mode shapes are more complex to make simple comment. Historical structures are expected to sustain larger earthquake compared to the ordinary structures due to their highly cultural and aesthetic values. Therefore, in the numerical analysis, the maximum earthquake spectra of the Turkish Seismic Code (2007) is employed, which is assumed to produce 1.5 larger forces that the design earthquake spectra. Then, the two analyses results were compared. As expected, stress concentration occurs around the edge of the openings. Small tensile stresses are formed at the top wall levels. The vertical normal stress increases up to 40-50%, when the maximum earthquake is considered instead of the design earthquake. Under the self weight and the earthquake loading, stress concentrations occur at the corners, around the openings of the wall, which support the main dome. This finding may explain why the edges of the openings have small cracks. Existing buckling damages at the iron ties of the arches may be explained by the existence of larger compressive stresses under the self-weight and the earthquake loadings. These iron ties may be replaced by the new ones as a simple repair intervention. The maximum stress varies around 2 MPa in the structure both in tension and in compression. Especially, tensile stress of 2 MPa is very high value for this type of masonry structures. However, it is believed that there are wooden lintels and iron elements and other connection components within the structural walls that are used to resist the tensile stresses and to provide the structural integrity for this type of historical structures. It is the most probably why there is no damage in these parts, although high tensile stresses are expected.

Repair and strengthening of historical structures require great care and it is not possible to use all repair and strengthening methods applicable to the ordinary structures. Venice Charter 1964 states the basic intervention principles for the historical buildings. All interventions for historical structures should be kept at minimum level as much as possible and should be reversible. One of the simple and widely used strengthening methods for domes is to create a tensional ring in order to decrease the tensile stress that occurs at lower part of the dome along the circumferential direction. The main dome is considered to be strengthened by a tensional ring and it is observed that the tensile stresses decreases at lower part of the dome significantly.

REFERENCES

Gedik, Y.H. (2008), Analysis, repair and strengthening of historical masonry structures; case study: Mehmet Aga Mosque, *MSc Thesis,* Istanbul Technical University, Istanbul.

Sesigür, H., Çelik, O.C., Çılı, F. (2007), Load-bearing components, damage patterns, repair and strengthening in historical structures, *Journal of the Chamber of Civil Engineering of Istanbul,* **89**, 10-21

Osman, A.M.G.D (2005), Structural Preservation of Historical Buildings in Seismic Area, *Cultural Heritage Without Borders*, Kotor, pp. 19-26

Venice Charter (1964), Venice

Penelis, G.G.(2002), Structural restoration of historical buildings in seismic areas, *Prog. Struct. Engng Mater.*, **4**, pp. 64-73.

Lourenco, P.B. (2002), Computations on historic masonry structures, *Prog. Struct. Engng. Materials*, **4**, 301-319, John Wiley & Sons ltd.

Turkish Seismic Zone Map (1996), Ministry of Public Works and Settlement, Government of Republic of Turkey, <http://deprem.gov.tr/linkhart.htm> (03.05.2008).

The Branch of Fatih of the Presidency of Religious Affairs (1991), The mosques of Fatih and other historical monuments, Istanbul.

Habibullah, A., Wilson, E. L.(1998), SAP2000, Structural Analysis Program, Computers and Structures, Inc.

Turkish Seismic Code (2007), Ministry of Reconstruction and Resettlement, Ankara

Güler, K., Sağlamer, A., Celep, Z., Pakdamar, F. (2004), Structural and earthquake response analysis of The Little Hagia Sophia Mosque" Paper no. 2652, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada. August 1-6.