“Earthquake Resistance of Beyazit II Mosque, Istanbul”

in

IASS 2007 Conference, Shell and Spatial Structures: Structural Architecture - Towards the future looking to the past, December 2007, Venice, Italy.

by

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EARTHQUAKE RESISTANCE OF BEYAZIT II MOSQUE, ISTANBUL

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Summary

Beyazit Mosque is a classical type Ottoman structure built between 1501 and 1506. Its original structural system is a combination of Hagia Sophia consisting of four main arches and semi-domes, and Ottoman construction techniques. The mosque contains four great brick and cut-stone composed arches springing from stone piers that offer primary support for a central dome of 16.78m diameter and 36.5m height and two semi-domes. The mosque experienced a strong earthquake just 3 years after its opening and was repaired immediately. After some strong earthquakes in the following years, Sinan, known as the master of the Ottoman architecture, retrofitted the mosque by adding some external jacketing to the columns, with anchorages and a modified cut-stone arch system. The importance of the mosque also lies on the fact that it is the first mosque which was constructed with the same structural type (two semi-domes) as Hagia Sophia after the conquest of the city in 1453.

In this paper, the intervention applied by Sinan has been investigated in detail. Additionally, in order to complete the 3D elastic model of the monument, the effect of the additional arches and the column jacketing have been examined by employing nonlinear analysis techniques. The conditions of the 1509 earthquake have been simulated by using a reasonably similar scenario earthquake triggered by the Marmara segments of the North Anatolian Fault. The reasons caused the partial collapse of the dome in 1509 and the effects of the retrofitting by Sinan to improve the stability of the central dome and semi-domes have been examined.

Keywords: Beyazit Mosque; earthquake resistance; masonry dome; historical monument; 1509 Istanbul earthquake.

1. Introduction

Built between 1501 and 1506, Beyazit is the oldest royal mosque in Istanbul that follows the pattern of the Byzantine monument Hagia Sophia. The main structural shape of the mosque, consisting of two semi-domes and a main dome, springing on four massive columns, reminds the innovative layout of Hagia Sophia.

The mosque experienced a strong earthquake and repaired in 1509. 62 years after that earthquake, it was retrofitted by Sinan. It has been intended by the authors to simulate the effects of the 1509 earthquake which shook the building and caused substantial damages. Retrofitting of Sinan has also been tested with the same simulation in order to compare the earthquake resistance of the mosque before and after his retrofitting.

2. Structural Properties of the Monument

The mosque contains four great brick and cut-stone composed arches, springing from four stone piers that offer primary support to a central dome with 16.78m diameter and 36.5m height and to two semi-domes (Fig. 1). The main arches under semi-domes initially had 90cm depth; however, the section depth was increased to 180cm at the crown level during retrofitting. It is noted by several researchers that one of the main deficiencies of Hagia Sophia, repeated in Beyazit Mosque, is the weaker arches under the semi domes [1-4]. It is stated by the authors [5,6] that the retrofit of Sinan, adding stone arches under the semi-domes, was aiming to balance the strength of the both main frames under the main dome. Strong and weaker (retrofitted) arches are marked in Fig.1a.

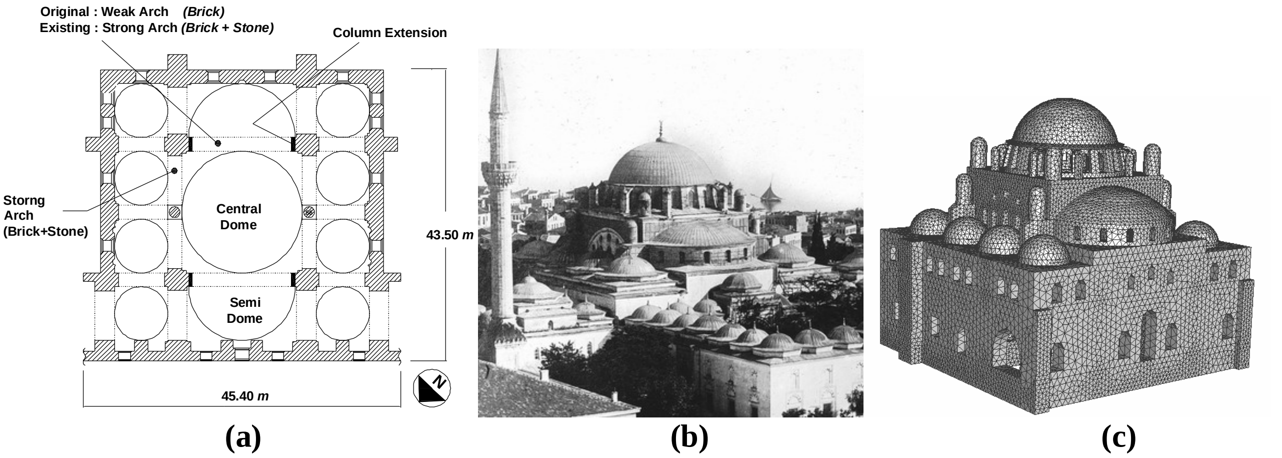

Fig. 1 (a) Layout plan (b) photo and (c) 3D modelling of the structure

The material properties of the piers, which are constructed by traditional limestone material (küfeki), have been obtained from experimental results [7]. Brick masonry properties are not available for the mosque. Therefore, the average properties of Hagia Sophia and Süleymaniye Mosques have been applied considering that the construction technique and the used material close enough. The granite columns are red granite, mostly found in central Turkey, namely as Aksaray Red Granite. Their characteristics are provided by the producers. Finally, the iron ties are assumed to be close to cast iron and to low quality steel. Brick arches, domes and pendentives are assumed to have the modulus of elasticity of 3000 MPa. The modulus of elasticity of the cut-stone elements (i.e. piers) is assumed the same as indicated in the above-mentioned experiments, 14830 MPa.

The depth of the foundation of the mosque cannot be detected in any source to the authors' knowledge. Besides that, the practical issues related to the monumental structures render the determination of the foundation depth rather difficult. Instead of defining the foundation depth in an observational way (i.e. digging out observation holes), ambient vibration mode shapes have been employed in this study [8]. The ambient vibration deformed shape starts with a non-zero value at the bottom of the structure, implying that the zero-deflection point of the structure lies somewhere deeper. The deflected shapes have been extrapolated up to zero deflection line, claiming that the level of the intersection point represents the real foundation depth. Considering the average results obtained from the aforementioned technique, the foundation depth has been assumed as 4.0 m in this study [6].

8 dome supports are covered by cut-stone material and a thick iron strut is hidden inside aiming to confine the dome at the level of top of the windows. The struts are strong enough even to distort the circular form of the dome.

Between 1571 and 1574, the mosque was extensively retrofitted by Sinan. The scope of the retrofit, as reported in the existing sources, can be summarised in the addition of strong arches in northeast-southwest direction and in the extension of main columns, however, details still need to be discovered by careful investigation in the Ottoman archives.

2.1.1 Damage Caused by 1509 Earthquake

The sources report that during the 10th of September, 1509 earthquake, the main dome of the mosque was shattered, other domes and arches of the complex split, its store-room collapsed and the newly built structure was repaired rapidly. Although earthquake catalogues lay emphasis on some other earthquakes between 1509 and 1571, the starting date of the mosque retrofitting, no other data can be found about any damage until 1719 earthquake, which had surface wave magnitude Ms=7.6 [9]. Historical data provide that the dominant earthquake direction including stronger component parallel to the retrofitted arches results in higher damage of the mosque, as shown in Fig.2 [6].

3. Seismic Parameters of 1509 Earthquake and the Simulative Scenario

1509 earthquake is one of the largest and most destructive of the last five centuries in the Eastern Mediterranean; it occurred in the Sea of Marmara, and was felt over a very large area. The magnitude of the earthquake is predicted between 7.2 and 7.6. The distance of the shake to Beyazit Mosque was around 47km. The damage of the mosque, using the historical descriptions, is predicted as 4 above 5 according to European Macroseismic Scale [10].

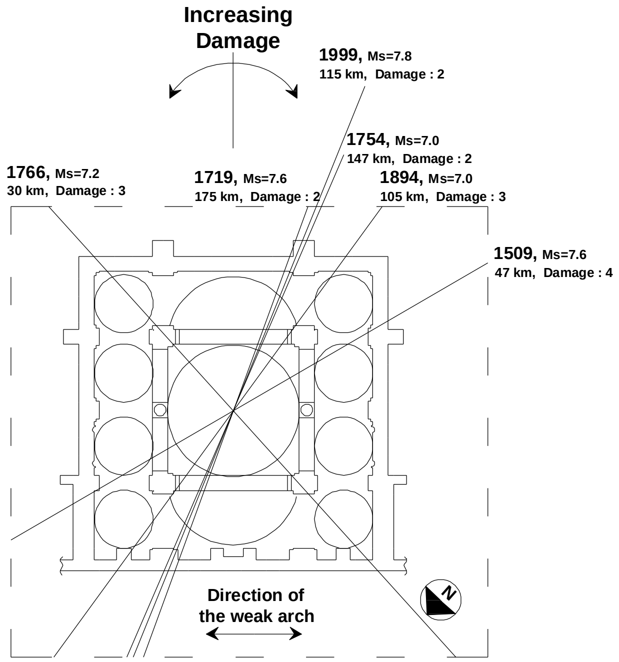

Fig. 2 Historical major earthquakes (larger than magnitude 7.0) having hit the structure so far

3.1 Simulative Scenario Similar to 1509 Earthquake

Simulation of the effects of the 1509 earthquake is one of the most important ways to understand the weaknesses of the structure and the reasons behind the retrofitting by Sinan.

1509 earthquake occurred along the fault segments 7 and 8 shown in Fig 3. A reasonably similar earthquake scenario [11] has been used in this study in an effort to reproduce the 1509 earthquake action. The scenario earthquake occurs along the faults segments 5, 6, 7 and 8 with a magnitude of 7.4. According to this scenario, considering the attenuation and the soil conditions of Beyazit Mosque, the expected PGA is around 0.27g below the area of the monument.

4. Structural Modelling

Analyses mentioned in this study have been conducted by using FEM package of ABAQUS 6.6. Reader is referred to Sadan et al. [5] for more detailed discussion on the linear analysis results of the structure.

4.1.1 Mesh Size

One of the most important aspects of the 3D FEM analyses is the mesh size. A preliminary study has been conducted in order to discuss this issue in more detail in [6]. The underlying objective of this study was to identify the trends of the crown displacement for a series of analyses with different mesh sizes. In the results the difference is found to be in the order of 45% between the two extreme cases of mesh sizes applied, indicating that an average meshed volume between 0.01 and 0.02 m^3 for 0.65m edge length of tetra-faced elements is appropriate.

4.1.2 Dynamic Properties and Calibration of the Model

Dynamic properties of the mosque have been obtained by ambient vibration tests (wind forced) [8] including mode shapes, modes and damping coefficients. The first three modes of the structure have been found as 0.38, 0.38 and 0.25sec experimentally while analytically computed first three modes are found equal to 0.34, 0.32 and 0.21sec. The latter are slightly lower than the former due to the existence of cracks and the soil-structure interaction which have not been included in the model. The main dome, Northern semi dome and its arch crown and some of the secondary domes have been severely cracked, mostly due to the recent earthquakes of 1999. Briefly, the 3D elastic model has been calibrated by using the experimentally defined dynamic properties.

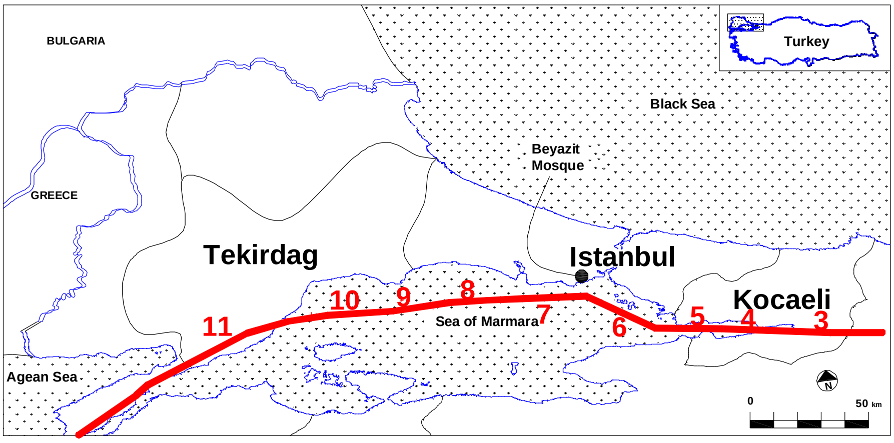

Fig. 3 Fault segments in Marmara Sea (extracted from [11])

4.1.3 Application of the Earthquake Load

The application of the earthquake loading is one of the most important issues for the seismic analysis of historical structures. Force based applications cause the problem of overestimating the forces, and thus stresses. Implementation of a load behaviour factor, as applied for RC buildings, is rigorously questioned due to the complexity and unique behaviour of monumental buildings.

Fully elastic load can be applied together with dead load if the aim is to estimate displacements only (not forces or stresses), assuming that the structure follows the equivalent displacement rule.

The purpose of the elastic model used in this study is to obtain only the displacement profile below the dome (along the perimeter of the circumferential drum) in order to predict the effects of the movement on the main dome, thus the use of fully elastic earthquake loading used in this study is justified.

The earthquake loads are applied as body forces (force per unit volume) in the model, avoiding the stress and deformation concentration on the concentrated load application points.

4.1.4 Earthquake Intensity and Direction

Earthquake intensity used in this study is compatible with the before-mentioned scenario earthquake of 7.4 in Marmara Sea. The obtained PGA value, 0.27g, is assumed to be the resultant one applied in the same dominant direction with the 1509 earthquake. The acceleration component of the earthquake in retrofitted arch direction is 86% of PGA while the perpendicular direction is only 50% of PGA.

4.1.5 Nonlinear Analysis

The characteristics of the masonry material render the nonlinear analysis of the historical masonry rather difficult. The main difficulty is to define the nonlinear material behaviour for such orthotropic materials. It has been shown that the orthotropic material presents different failure planes for different angles of loading, thus, a 3D failure surface having as third parameter the angle of loading [12-14] is required. Focused on the masonry infill panels in reinforced concrete frames, recent studies tend to solve the problem of orthotropic nonlinearity in a more simple way (i.e. single or double strut representation). Additionally, the dimensions of the infill panel itself simplify the definition of the loading angle, reducing thus the problem in the use of the corresponding "slice" of the 3D failure surface. Unfortunately, the inherent difficulty in the definition of the arbitrary loading angle for the historical masonry strictly diminishes the advantage of employing 3D failure surfaces in a simpler manner.

Another serious uncertainty faced in the nonlinear analysis is the loading pattern, which is still an open research subject even for modern reinforced concrete buildings. Furthermore, the concentration of mass at floor levels in modern structures determines the points of application of loading, whereas the similar idealisation is not applicable to historical masonry structures. Finally, the transition to the nonlinear behaviour is accompanied by elongation of periods during the loading. Recent studies attempt to cover these important issues, but the application is still limited to modern structures and only tentative steps have been made for the study of historical masonry structures. The solution of the problem is hindered by a number of aspects of substantial importance such as geometric nonlinearities, uncertainties caused by the material properties etc.

However, in few benchmark studies attention has been paid on the material behaviour. Indicatively, the step-by-step approach applied by Şahin and Mungan [15] for Hagia Sophia and by Bal et al. [6] for Beyazit Mosque are mentioned.

5. Use of Linear and Nonlinear Analysis Results

Two types of analyses have been combined in this study in order to understand the behaviour of the main dome. The first analysis is the linear analysis with fully elastic earthquake loads. Whole structure has been modelled in this case. The sole aim of this analysis was to define the deformed shape below the dome for retrofitted and strong arch directions in order to have a picture of the differential displacements developed at the base of the dome. It should be noted that the nonlinear analysis mentioned hereinafter covers only the frames, leading thus to definition of the displacement profile above the arches, not along the perimeter of the dome.

The nonlinear analysis results have been used to define more accurately damage behaviour of the arches below the dome. It is known that the linear analysis results do not provide realistic definition of the damage just below the dome, even though they are rather reliable in terms of displacements. The damage is expected to concentrate at the crown point of the main arches. A simplified step-by-step nonlinear approach has been applied to obtain more realistic deformation values at the crown level. The level of loading consistent with the scenario earthquake defined previously. Details of the analysis are given below.

Finally, the displacement values obtained from the nonlinear analysis are applied to the base of the dome as support settlements, combined with the deformed shape obtained from 3D linear analysis. The stress states of the dome for original (before 1509) and retrofitted by Sinan (after 1574) are presented (Figs. 5 and 6).

5.1 Outcomes of the Full Linear Analysis

The first outcome of the linear analysis results is the deformed shape. As clearly seen in Fig.5 the deflection of the weaker arch causes differential movement of the circumferential drum along the weaker arch. The deformation profile of the drum along the strong arch exhibits more uniform distribution of displacements compared to that of the original (before retrofitting) case.

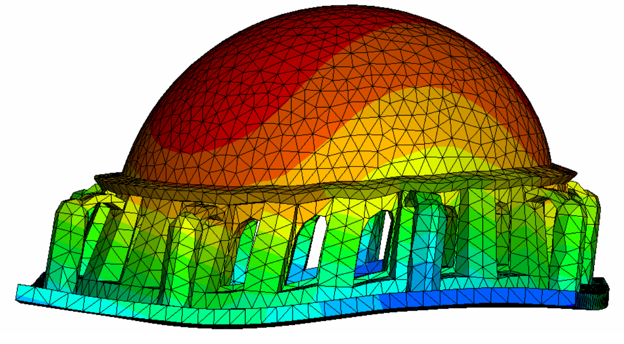

Fig. 4 250 times scaled dome deflection along the weaker arch before the retrofitting (only the main dome is in view)

5.2 Outcomes of the Nonlinear Benchmark Analysis

The nonlinear analysis results cover the main frames under the main dome, which are modelled with 8-node brick elements. The analysis starts with imposing 80% of the self-weight in order to capture the initial cracks in masonry. Following steps include the combination of the constant dead load with the incremental earthquake load. The earthquake loading is increased step by step by scaling up the imposed earthquake spectrum. In every successive step, the analysis has been restarted by deleting manually the elements that reached their limit states. The material properties are represented by single modulus of elasticity values, instead of failure surfaces, however, bi-linear stress-strain relationships are created by considering the material limit states. Details of the proposed nonlinear method can not be discussed here in detail, due to lack of space, readers are referred to Bal et al. [6].

The brick masonry of the mosque is identical with the thick mortar brickwork, widely used in Roman, Byzantine and Ottoman structures, which has ratio joint/brick thickness ≥ 1.0. The puzzolanic mortar (horasani harç) used in this type of masonry is known since the Roman era. The mortar a lime-based material owes its strength mostly to the lime preserved under earth for some years without having contact with air. Based on the previous experimental studies [16], the brick masonry material is assumed to have 5.0MPa compressive and 1.0MPa tensile strength.

Considering the experimental values, the limit-state in compression is assumed to be equal to 25MPa. The tensile strength between the dry stone units is provided by the iron connectors. An average iron connector section has 1.5cm×5cm dimensions. The tensile strength is assumed to be equal that of very low quality steel of 140MPa. However, since the mathematical model has continuous cut-stone surfaces rather than distinct tie-rod links between elements, this limit force must be divided to the surface of an average stone in order to create a smeared composition of the tie rods and the average stone units, which have the dimensions of 30cm×20cm. The tensile limit strength for the stone masonry is calculated as 140×(1.5×5)/(30x20)=1.8MPa. The shear strength limit can also be calculated similarly, however, it should be noted that the shear strength is much more complicated since it also includes the frictional behaviour.

Two limit-states have been defined for the tie-rods, one is for buckling under compression and the other one concerns anchorage slip under tension. A detailed look through the damaged historical buildings, mostly to the post-earthquake photographs from previous century, reveals that the tie-rods do not fail easily even if the surrounding material fails. In some limited cases, a shear type failure has also been observed. The tie-rods of Beyazit Mosque have 15cm×10cm dimensions and 6.60m length and therefore Euler's buckling load is calculated as 560KN for fixed-end boundary conditions in both extremities. In fact, one end of the tie-rods is installed into brick masonry, while the other end is embedded in the granite or limestone columns. As known from past experience, the end of the tie-rod is bent and driven into the granite, marble or stone columns according to the Ottoman construction practice, rendering thus the pullout failure less probable compared to the other end embedded in brick masonry. Specifically, if the slip failure at the end embedded into stone and granite is ignored, the end of the tie-rod in the brick masonry is expected to have a failure close to "pullout cone failure" (failure surface follows a cone having 45^o developing angle) of anchorages embedded in concrete. The tensile limit of the expected cone is given in Eq (2), where L is the anchorage length, A is the projection area of the failure cone and f'_t is the tensile strength of the mortar.

$$A = L \times \tan(45/2) \times \pi / 4 \tag{1}$$

$$F_u = A \times f_t^{'} = 390KN \tag{2}$$

It should be noted that the expression is modified from the suggestions of the ACI for anchoring concrete [17].

5.3 Dome Stresses as a Combination of Linear and Nonlinear Analyses

The displacements below the perimeter of the dome have been obtained by combining the results of the above-mentioned linear and nonlinear analyses and are applied as support settlements to the models which include only the dome.

Elements which reached to 1/3 of their tensile strength (0.33 MPa) are assumed to be cracked due to tensile failure. These regions are represented in gray in the Fig. 5 and 6. It can be claimed that the 1/3 is a conservative value, however, considering that the earthquake is a cyclic loading and the tension cracks do not just open up and close during the cycles but propagate progressively, a lower value (i.e. 1/3 or 1/4) would then be justified.

It can be clearly seen from Fig. 5 that the possible tensile failure zones are much bigger in the original case, before Sinan's retrofitting, leading to a possible collapse mechanism of the dome. On the contrary, Fig. 6 represents the tensile zones almost parallel to the earthquake loading direction; however, tensile stresses do not penetrate below the dome.

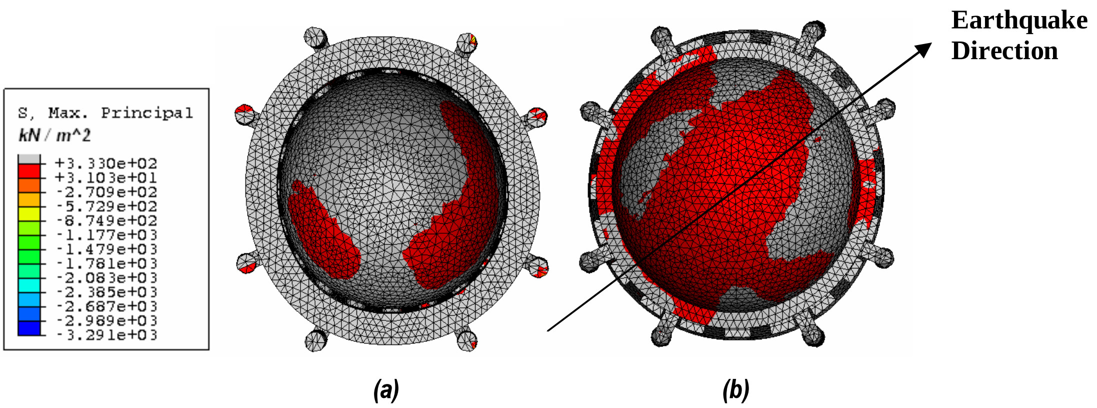

Fig.5 Max. principal stresses of the dome before retrofitting by Sinan (a)view from bottom and (b) view from top (gray zones of the dome represent the possible tensile failure zones)

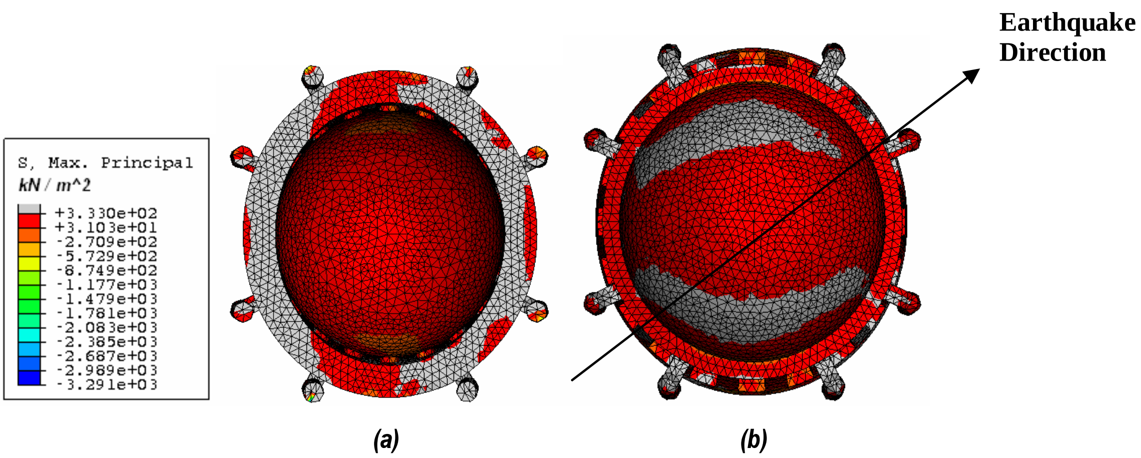

Fig.6 Max. principal stresses of the dome after retrofitting by Sinan (a) view from bottom and (b) view from top (gray zones of the dome represent the possible tensile failure zones)

6. Conclusions

The earthquake resistance of Beyazit Mosque in Istanbul has been evaluated in this study. The deficiencies which lead substantial failure of the structure in 1509 earthquake have been investigated. September 1509 earthquake has been simulated by using a reasonably similar earthquake scenario, assumed to be triggered along the Marmara segments of the North Anatolian Fault with a magnitude of 7.4. Linear and nonlinear analyses techniques have been combined to get the deformational behaviour below the circumferential drum of the main dome.

Along with the linear 3D analysis, material behaviours are discussed in detail and limit states are defined for brick masonry, stone masonry and iron tie-rods. A simple nonlinear analysis has been introduced as a benchmark study. The proposed method needs manual manipulation of the loading and deletion of the elements which reached the pre-defined limit states. Difficulties of creating nonlinear models for the historical masonry have been discussed. Advantages and disadvantages of the method have also been evaluated.

The drum of the dome is found to experience differential deformations along its perimeter. It has been shown in this study that the retrofitting by Sinan has improved the support conditions of the dome by decreasing the effects of the differential settlements along the perimeter of the drum of the main dome.

It is deduced from the retrofitting strategy that maybe not the stresses but the displacements and deformations have been more accurately evaluated by Sinan and most probably by contemporary masters of him as well.

7. Acknowledgements

This study has been financially supported by the Metropolitan Municipality of Istanbul, “Projem Istanbul” and is gratefully acknowledged. Additionally, authors would like to thank to Prof. Ihsan Mungan, Prof. Gorun Arun, Prof. Fevziye Akoz and Tomasz Garbowski for their tremendous contribution.

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