STRUCTURAL RESPONSE OF ORTAKOY BUYUK MECIDIYE MOSQUE IN ISTANBUL

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ABSTRACT: The Büyük Mecidiye Mosque in Ortaköy, Istanbul, is an emblematic building of the city with the unique Bosporus view. The original mosque was built in the 18th century. The current mosque, which was erected in its place, was ordered by the Ottoman sultan Abdülmecid and built between 1854 and 1856. Its contractors were famous Balyan's and its architect is Abdülhalim Efendi [1] .The structure is designed in the Neo-Baroque style. The masonry dome of the structure was substituted with a relatively light reinforced concrete dome in 60's, due to serious structural cracks observed on the dome triggered by soil movements towards the sea [2]. This paper discusses the results of an assessment effort that was focused on defining the seismic response of the structure. The results show that the RC dome transfers limited inertia forces to the substructure. Based on the analyses presented here, the dome is not expected to adversely affect the seismic response of the structure.

1 INTRODUCTION

The Büyük Mecidiye Mosque in Ortaköy (Figure 2,Figure 1), Istanbul, is an emblematic building of the city with the unique Bosporus view. The mosque lays on a platform intending into Bosporus, so a part of the structure is sitting on piles. Through its lifetime the mosque was experienced 5 interventions in 1862, 1866, 1894, 1964 and 1984 [3].

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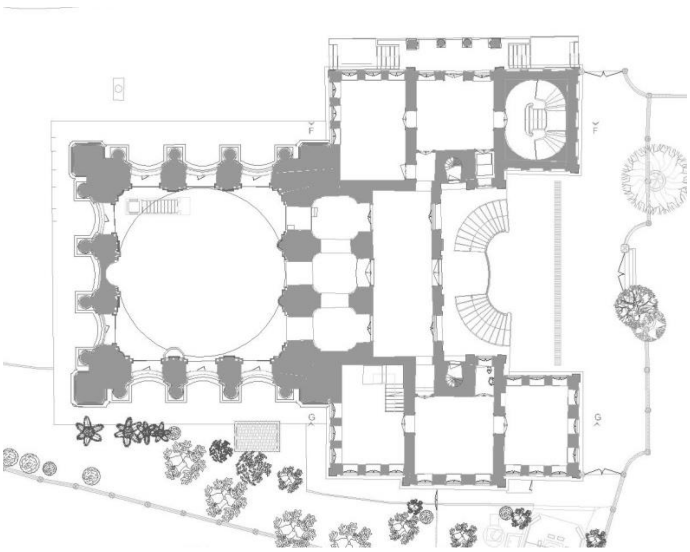


Figure 1. Büyük Mecidiye Mosque in Ortaköy, plan

Figure 2. Büyük Mecidiye Mosque in Ortakoy, and old photo from a postcard

The foundation and the structure experienced a significant level of leaning in 60's, and a rather drastic decision was made to save the mosque. The dome was full of with serious cracks, ready to collapse, so the dome was taken down brick by brick[3]. A light reinforced concrete dome was then built. The new dome consists of 2 slab layers with beams in between the two (Figure 3).

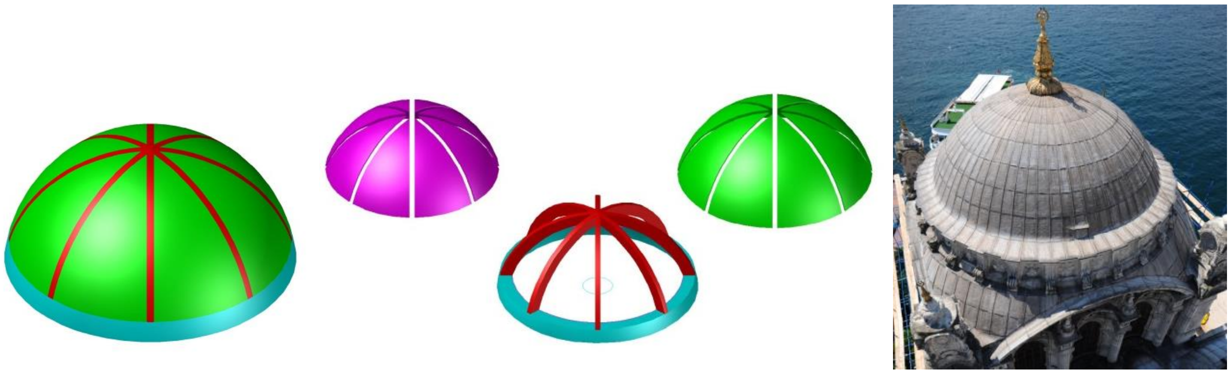


Figure 3. 3d model representation of the concrete dome and the photo

During the restoration works held between 1965 and 1970, soil improvement was realized as the observed damages associated with the soil conditions [4]. Thus, soil piles were used both inside and outside the mosque and a new reinforced concrete foundation was cast. Slab of the main sanctuary saloon was replaced with reinforced concrete slab as well as the harim slabs (welcoming place and private entrance for sultan).

Nowadays the mosque is having another comprehensive restoration activities executed by Gürsoy Restoration Company. Within these works, decomposition of concrete slabs were realized but for the dome an investigation was studied in order to evaluate the effect of reinforced dome to the existing structure and if it is convenient to reconstruct or maintain.

The mosque is located in a well-known place, Ortaköy on a small natural cape like land projected on sea. It was constructed by Nigogos Balyan who is a registered Armenian origin architect of Ottoman Empire also known as the architect of Dolmabahçe Palace.
Praying space of the mosque has a square shaped plan and in Hunkar section a complex of rooms placed in U shape plan.
Structural bearing system of the mosque is composed of four main pillars made of hewn limestone at the corners. The dome rests on four stone arches but unusually the span of the arch is filled with limestone with some carved decorations.
The corner pillars are supported with merged additional rectangular shaped pillars in diagonal direction outside as if buttresses. With this structural and architecturally effective touch it is resulted with obtaining supported pillars but aesthetically realized as thin pillars outside. Each facade also has three large openings with concave longitudinal windows. In all facades in between the windows, smaller pillars are located, shaped with the same approach applied to the main pillars at corners so the pillars look much thinner than their exact section. Mihrab wall has a solid section till the mid height of the elevation (Figure 4).

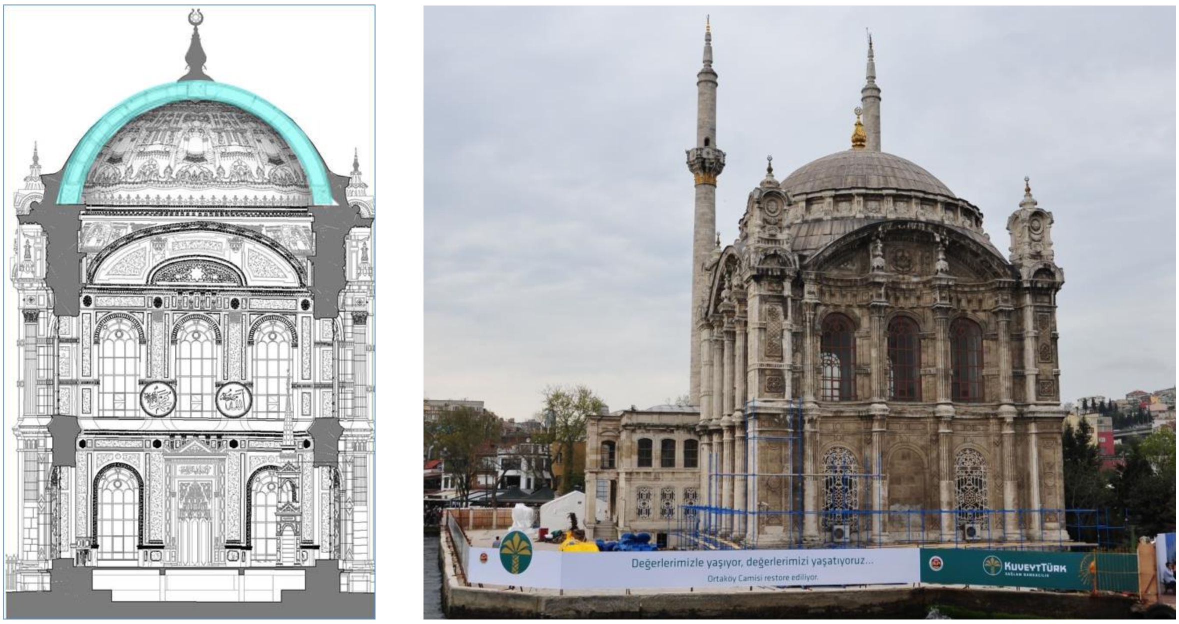


Figure 4. Elevation through mihrab and mihrab facade

It is to say that sanctuary has no continuous walls but pillars with 1.90 m in width, 2.30 m in depth pillars and 0.80 in diameter circular supports outside. The skew of arches starts at 16.80 m high from courtyard and the span of the arches is 12.80 m. Thickness of the arches and the filling is 1.40 m. Span of the dome is 14 m in diameter and the total thickness is around 90 cm.

2 STRUCTURAL ANALYSES AND SEISMIC RESPONSE OF THE MASONRY BEARING SYSTEM

2.1. Motivation and the preparatory studies

The motivation of the structural analyses and assessment studies presented in this paper is answering the question whether the modern reinforced concrete dome has an adverse effect on the overall seismic response of the structure or not.

On-site investigations, on the dome and the masonry-RC interface have been conducted to better understand the material characteristics of the added dome as well as the boundary conditions valid during a lateral loading case. The bore-hole tests showed that the concrete of the dome has 32 MPa average compressive strength [4]. Smooth rebars with 220MPa characteristic tensile strength was used for reinforcing. The details of the rebars placed in the sections can be seen in (Figure 5).

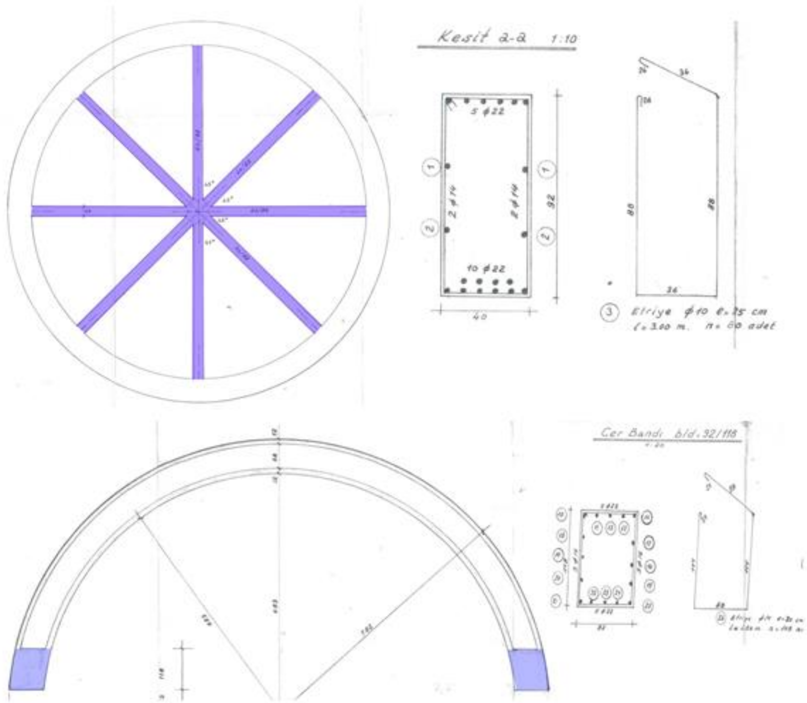


Figure 5. Details of the new RC dome

Shop drawings were accessible during the study. It was observed that the execution of the existing RC dome was realized accordingly with the approved shop drawings, which was controlled by Vakıflar (the public institutions that in charge of management and maintenance of monuments) and noted that the design is suitable and the construction is done as per the drawings (1967) [5].

The on-site investigations were also used for determining the actual reinforcement of the dome. Non-destructive tests such as rebar scanning with Bosch-Dtect-150 equipment and accessive semi destructive scraping were used to map the existing rebar. Scanning was done by placing a transparent paper on dome surface in order not to disturb the existing hand-drawings. Lateral and longitudinal places of rebar were marked on the protective paper layer. Rebar span and quantities were noted (Figure 6).

Figure 6. Rebar scanning equipment and application

Although span of rebar varies along the perimeter the average span was acceptable. The place of main girders that divides the dome into eight was detected that are in direction of mihrap (southeast-northwest direction). However due to dense content of rebar, exact span and quantity could not be determined and deemed as it is defined on drawings. On inner slab the average rebar span was determined as 30 cm, in relevance with the drawings. Outer slab rebar content couldn't be checked so deemed to be executed as per the drawings.

Relevancy of the detections was also checked by removing the concrete cover and the actual condition of rebar is checked. That information was crosschecked with the gathered original project and coherency is determined.

In order to observe the connection between concrete dome and the masonry structure, concrete cover was removed near to the cat-walk where no hand-drawing exist. After removal, it was determined that the bottom girder of the dome is placed 30 cm higher than the cat-walk without any anchorage. Thus the connection between concrete beam and masonry structure is provided by the by the interlocking through the friction of these surfaces (Figure 7).

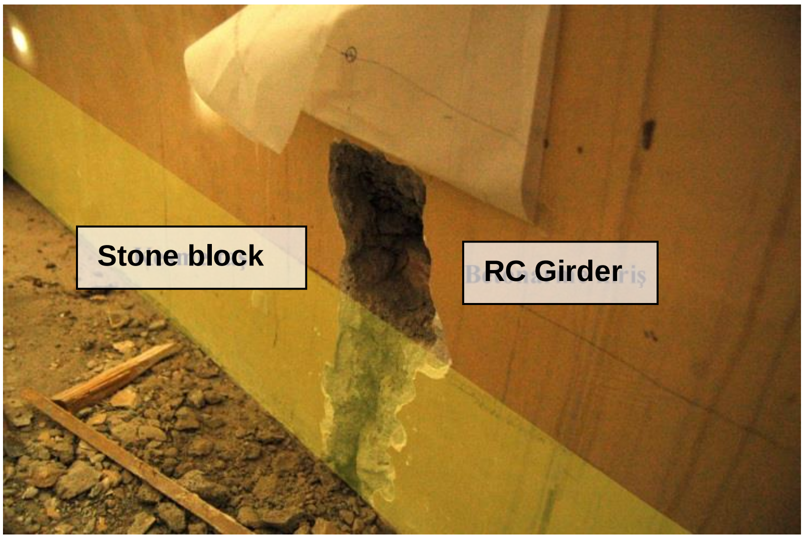


Figure 7. Examination of interaction between RC dome and masonry

Removal of concrete cover allowed us to measure the diameter of rebar to check its relevancy with shop drawings and evaluate the material condition. In this examination it was observed that smooth steel was used and no critical corrosion was occurred.

2.2. Analyses conducted

Based on existing survey, 3d solid model was build up with common CAD software. However to some constraints of structural analysis programs, geometry of the structure was simplified with acceptable errors such as keeping the dimension of load bearing sections constant and by eliminating the architectural elements which has no structural contribution such as ornamental elements (Figure 8).

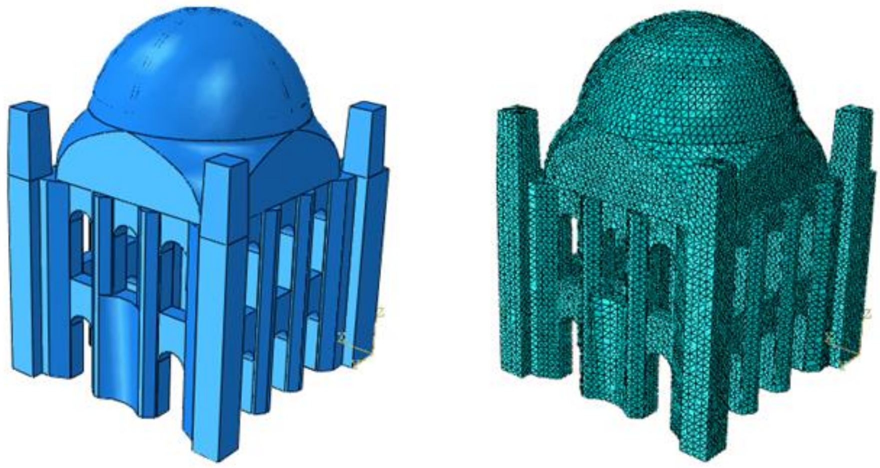


Figure 8. Finite element model, geometry and meshed model

The finite element model is composed of 160781 tetrahedral elements (linear tetrahedral element C3D4) and have 36637 unique nodal. Each node has 3 DOF (Figure 9).

Figure 9. Mesh type used in finite element model (Abaqus, C3D4 tetra)

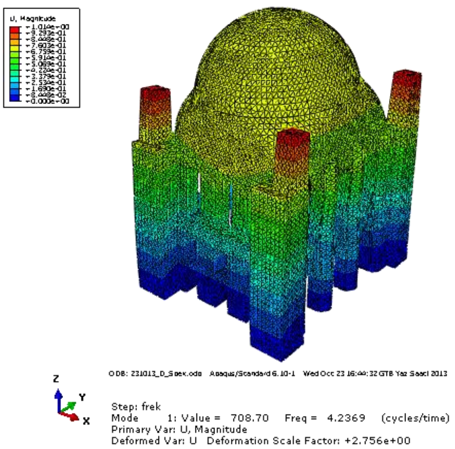
Related freedoms for each node are translations and rotations in three different axes. At the bottom, interaction of superstructure with soil was defined as pinned connection by constraining the translations of these nodes in 3 axes but allowing rotations.

Material properties of masonry were extracted from previous studies on similar type of structures (Table 1).

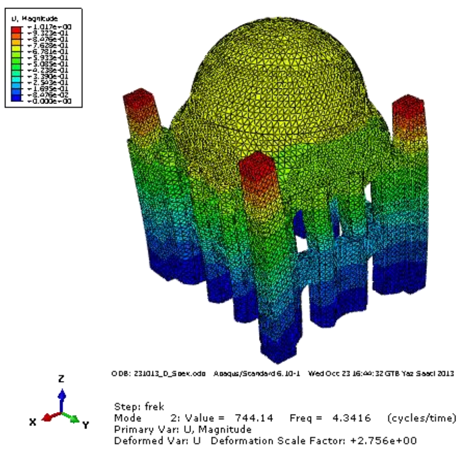
Table 1. Material properties used in the model

Material	Modulus of Elasticity [Gpa]	Unit Weight [kg]
Limestone	5.00	2500
Stone-Brick	4.00	2200
Concrete	27.00	2400

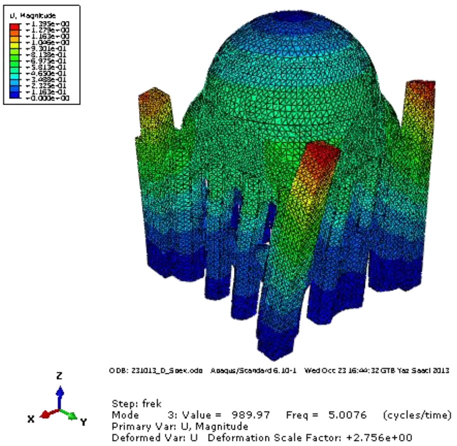
The first set of analyses was the eigenvalue analyses where the free vibration mode shapes as well as the relevant period values have been found. Ambient vibration tests have been conducted on the structure by the authors, the results of which could not be prepared until the time of writing of this paper, unfortunately. The period values, however, was in agreement with those found in the eigenvalue analysis, leading thus to a level of confidence about the numerical model created. The estimated mode shapes and periods can be found in Figure 10. It is seen that the first two mode shapes are translational, very close to each other, while the 3rd and 4th modes are dominated by torsion (Figure 10).



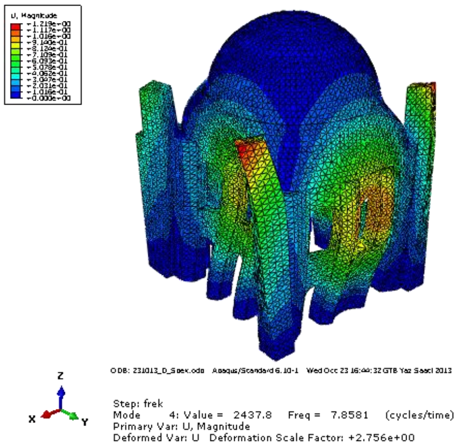
Mode 1 – 0.236 sec



Mode 2 – 0.230 sec



Mode 3 – 0.199



Mode 4 – 0.127 sec

Figure 10 : Modes shapes and the periods of the mosque

Another observation with the eigenvalue analyses is that the dome, with its current boundary conditions represented in the model, do not create local modes, in other words, it rather moves synchronized with the rest of the bearing system. This very preliminary observation is already a good sign on the integration of the dome with the masonry bearing system.

The lateral force that may act on the structure was applied statically in addition to its dead load. Applied load was calculated according to the definition given in our national disaster code (2007) depending on the expected spectral acceleration, weight of the structure and elastic reduction factor (1).

$$V_t = \frac{W A(T_1)}{R_a(T_1)} \geq 0.10 A_0 \, I \, W \qquad \textbf{(1)}$$

The equivalent earthquake force was calculated with the given formula where *W* is the weight, *A(T1)* is the value calculated depending on *I,* importance coefficient, $A_{0,}$ effective ground acceleration, and *S(T)*, spectral acceleration, R is the elastic response coefficient**Error! Reference source not found.** (2).

$$A(T) = A_0 \, I \, S(T) \tag{2}$$

Main parameters used in the equivalent earthquake calculations are given below (Table 2).

Table 2. Equivalent earthquake force parameters

Total volume	2213	m3
Total weight	5406	ton
Structural Importance coefficient	1.4	
Spectral magnification factor	2.5	
Peak Ground Acceleration	0.3	(II.Degree Earthquake Zone)
Elastic Seismic Response Coefficient	1	

Estimation of elastic response coefficient in such historical structures is practically not possible where it is widely used in design of modern structures. Even if it is possible to define the response coefficient correctly, calculated stress values wouldn't match with the real conditions, where tension cracks develops and spread even in low level of tension force were occurred. These phenomena can't be defined with elastic material properties.

Because of the stated reasons above, elastic response coefficient was assigned as 1 and the equal displacement rule was considered. So that the deformation records obtained from the analysis realized where R=1 were compared with limit deformation values. Although this approach brings some uncertainties and depends on some assumptions it was preferred to the elastic response coefficient which has no base.

2.3. Results of the structural analyses

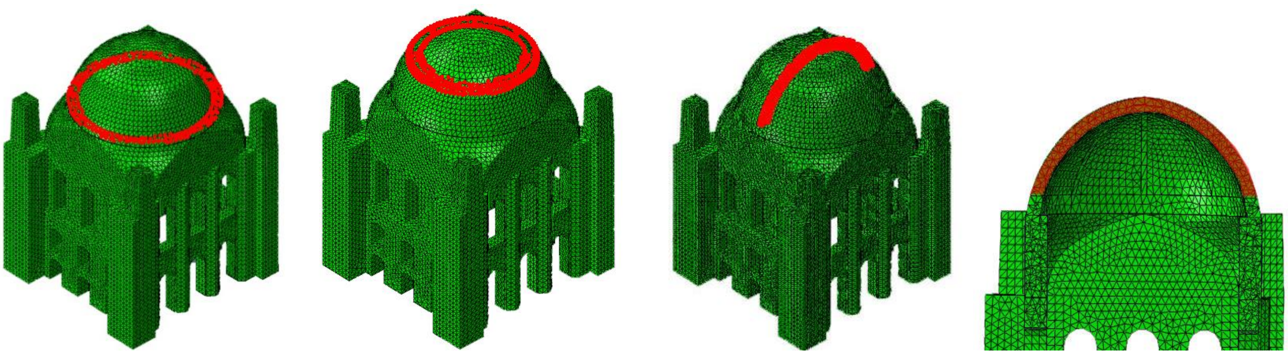


Figure 11. Nodes that the strain values are read on dome

Strain values were read on dome where the shear force, compressive and tensile stresses are expected to be maximum so that the strain values as well. Under the dead and earthquake load combination, in direction of lateral load, strain values read about 10^{-5} - 10^{-6} at the bottom surface of the main girder and it was observed that lower than the considered damage limit of concrete as 0,002. For the rebar controls, 4 cm concrete cover was considered and rebar were evaluated for limit yielding strain and seen that the read values are lower the limits (Figure 11,Figure 12, Figure 13).

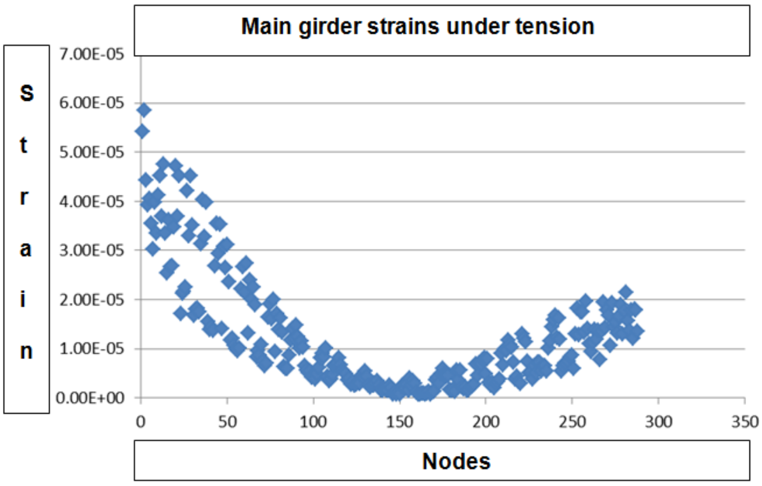


Figure 12. Strain values of the main beam on dome under tension

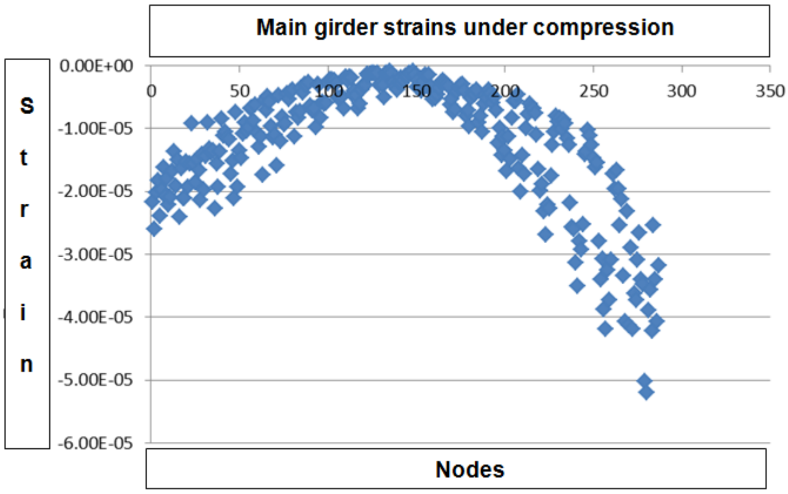


Figure 13. Strain values of the main beam on dome under compression

In addition to the strain records plotted, strain values on masonry were picked where it is supposed to have maximum values. So that for masonry strain records were read on masonry pillars on mihrab facade (Figure 14).

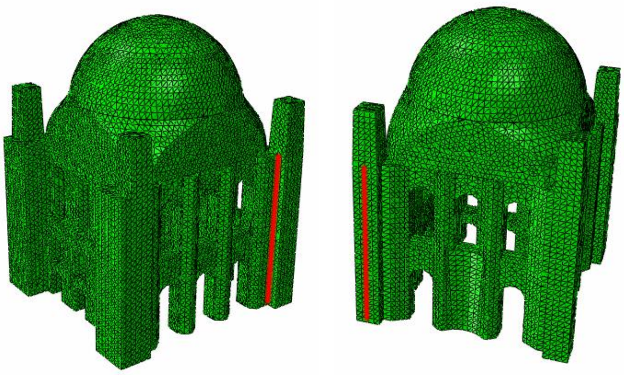


Figure 14. Strain records taken on the corner pillar

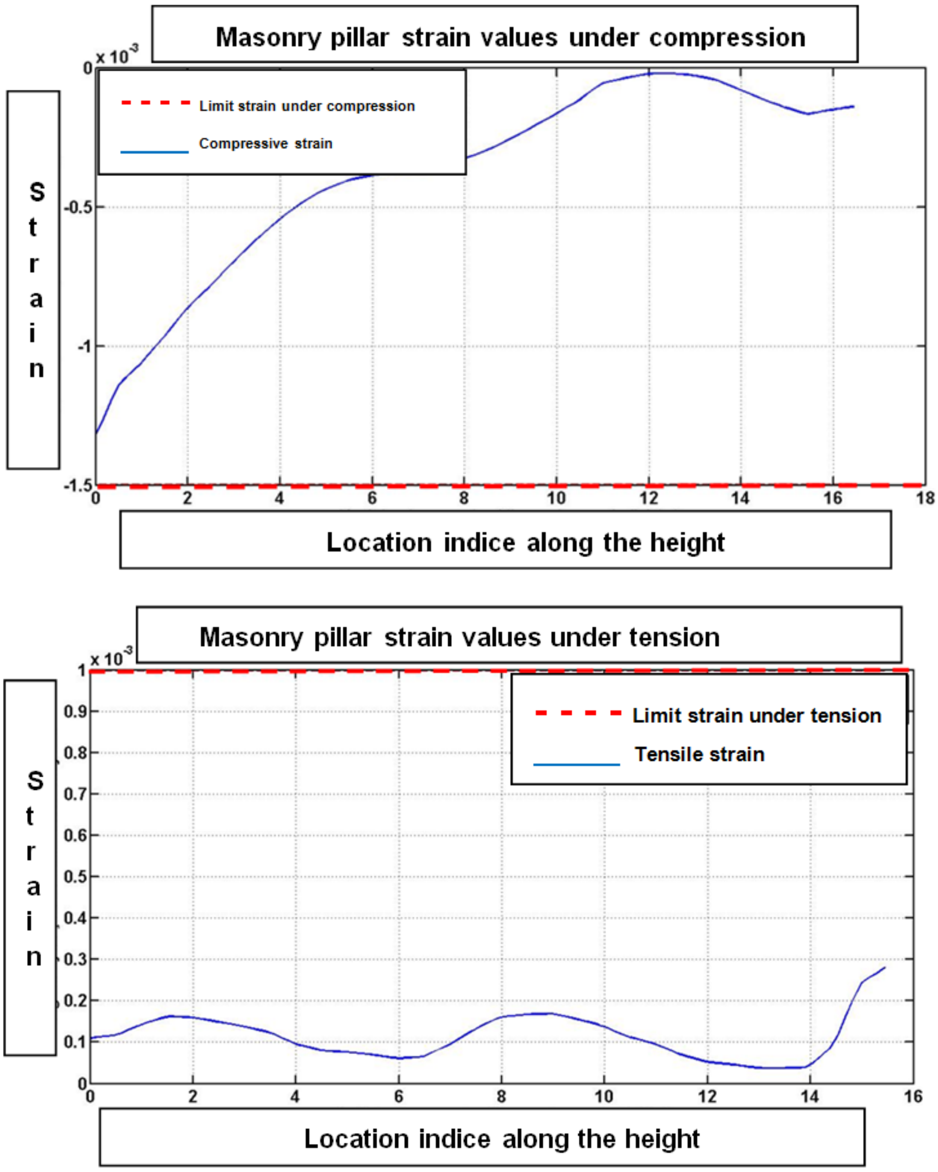


Figure 15. Top; Strain values under compression, Bottom; strain values under tension

Strain values recorded on pillars exhibited higher values than the dome as expected. However, under the considered earthquake excitation the values determined to be under the considered damage limit of masonry.

As it was stated above sections, the main consideration for the conditional assessment of the structure was considered to be the elastic deformation capacity of the material. Some of the comparisons by means of strains are given. As the stress level and strain values are dependent, the stress level on overall structure is below the limits. Given stress diagrams are plotted only for general understanding.

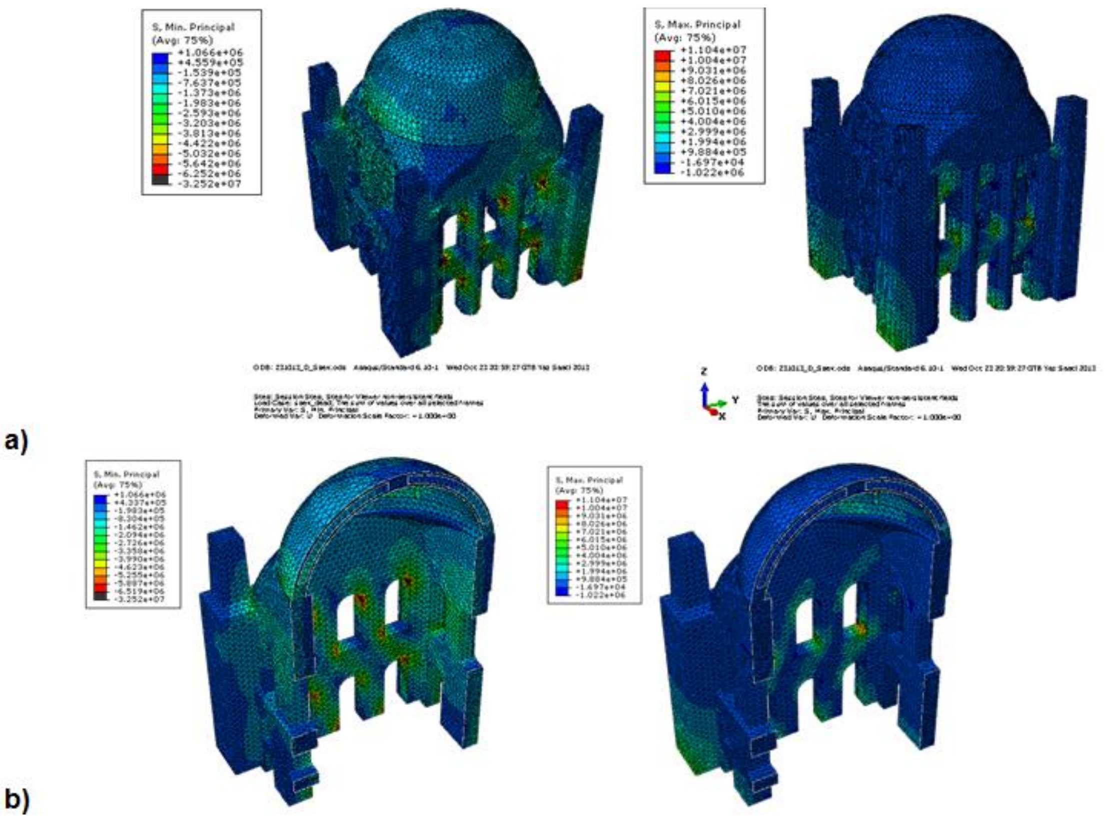


Figure 16. Left (a & b): Stress distribution in pressure (Smax = 6,65 MPa) , right (a & b): Stress distribution in tension (Smin=5.77 MPa)

After the evaluation of existing structure a comparative study was realized to understand the effects of alteration of original dome with a lighter concrete dome. Main parameters are compared and seen that the use of reinforced concrete dome decreases the weight and earthquake loads so slight decrement is observed by means of stress and strain (Table 3).

Table 3. Comparison of concrete dome structure with masonry dome structure

	Concrete Dome	**Masonry Dome**
Total Volume	2213 m3	2422 m3
Total Weight	5406 ton	5925 ton
Dmax	0.031 m	0.045 m
Smax	6.65 Mpa	7.57 Mpa
Smin	5.75 Mpa	9.2 Mpa
Emax	0.00122	0.0014
Emin	0.00107	0.0017
Frequency		
F1	4.2369 Hz	4.1916 Hz
F2	4.3416 Hz	4.2697 Hz
F3	5.0076 Hz	5.3881 Hz
F4	7.8581 Hz	8.2357 Hz

3 CONCLUSIONS

In the light of analyses and experiments it was observed that the reconstructed dome in concrete exhibits total body response with the masonry as the friction satisfies interlocking forces at the interface.

It’s observed that after the structural analysis, weakly recognized structural system is stronger than its appearance with additional pillars at the corners and the façade pillars that carry the weight of the dome transferred by the masonry wall under the arch. That masonry load bearing wall under the arch allows distribution of dead loads to all pillars different than a classic arch system that concentrate the weight of the dome on main pillars.

The strain values read on different sections of the concrete dome are all below the limits and stress distribution is much less than the masonry part. Thus in earthquake response of the structure the contribution of the dome considered to be minor. Even the replacement of original brick dome with a concrete one decreased the total weight of the structure so that the earthquake forces.
During an earthquake no damage is expected on the dome but likely to be on main pillars and at the corners of window openings on facades.

In anyhow, at least for the bearing system of the very mosque presented here, the dome is a load more than a component of the load bearing system, thus decreasing the inertia loads by replacing the dome with RC had a positive effect.

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