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Effect of Seismic Horizontal Loads on the Structure of the Taj al-Molk Dome in Jameh Mosque of Isfahan

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Abstract

The external forces acting on the structures are divided into two categories based on the direction of application: a. Loads applied in the vertical direction or gravity loads such as: dead loads, live loads, snow load, etc. b. Loads applied in the horizontal direction or lateral loads such as: earthquake load, wind load, etc. Therefore, in this study, the behavior of the northern dome of Isfahan Jameh Mosque (Taj al-Molk dome) was examined under horizontal seismic loads using a finite element three-dimensional model (3D FE) (nonlinear dynamic analysis) with one level of numerical complexity with the aim to cover the structure of the dome, which has a medium to high complexity in operation. The results indicated that the structure of the Taj al-Molk dome is vulnerable to horizontal loads, so that if the residual deformation occurs in the range of 0.4% and 0.8%, it will cause the structure to collapse. On the other hand, geometric features (irregularity in the plan and height of the dome, the presence of large openings, height changes in the cross section as well as the presence of the dome or an internal inclined arch) play an essential role in the seismic performance of the building.

Keywords: Taj al-Molk Dome; Seismic Horizontal Loads; Geometric Characteristics; Abacus Software

1. Introduction

Mosques, as the most important structures built in all historical periods of Islamic architecture, have a unique place in research and investigations (Ebad, 2013: 940). The Jameh Mosque or the Atiq Mosque, also known as the Old Mosque, is known as the oldest mosque in Isfahan, Iran, and represents the history of construction of the mosque in Iran that is more than a thousand years old.

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The building, which displays all kinds of scientific methods, styles, and decorations of Iranian architecture at the same time, was recognized and registered as an international heritage site in 2012. This historical complex covers an area of 170 * 140 m^2 in the northeast of Isfahan and near the Old Square and includes parts such as the domes of Nizam-ol-Molk and Taj al-Molk, the four-porch courtyard, the Shabestans around the mosque, Muzaffari School, and the famous altar of Oljaito. The oldest date discovered inside the mosque is related to its northern dome, i.e. the dome of Taj al-Molk or Khaki Dome, based on an inscription dated 481 AH (1088-1089 AD) and this date has been engraved at the end of the circular inscription on the neck of the smaller of the two Seljuk domes. In the present study, the behavior of the historical dome of Taj al-Molk is analyzed under the horizontal seismic loads using a three-dimensional finite element (3D FE) model. Nonlinear dynamic analysis on the model was carried out using a real accelerograph with different peaks of the Earth acceleration. For the above structure, a model with plastic failure with the hardening property in both compressive and tensile stress states was used. Given the importance of preserving historic buildings, the investigation of the factors affecting the destruction and the creation of mechanisms of destruction is a priority (Andreea clim et al, 2017: 65). Protecting historic buildings against seismic loads is of great strategic importance, which should be given serious consideration by the relevant organizations in order to keep these precious heritages from being destroyed.

2. Materials and Methods

This study was developed based on the descriptive-analytical method and the information required for the analysis of the architecture of the Taj al-Molk Dome was collected in a documentary-library manner as well as field observation and photography. In the following, the finite element method (numerical method of structural analysis) was implemented in Abaqus/Standard CAE software to investigate the horizontal seismic loads on the dome.

3. Selected Properties of Materials

A masonry structure is formed by a regular construction to behave in a suitable and in a preferably complex way. Along with its main axes, this material exhibits orthotropic characteristics in the form of tension with very low strength and consequent tensile damage, differences in tensile and compressive fracture levels, plastic deformation, and failure during the emergence of cracking due to compression, re-hardening, and the like. Some of the main causes of such complex behavior have been previously demonstrated with good approximation in experimental samples under static loads and in numerical models. One of the main obstacles in the behavioral estimation of a masonry structure stems from the fact that the masonry structure has a strange and unknown behavior in relation to its texture (regular or irregular, and if regular, in its dependence on the arrangement and formulation of its brick parts), typology of the units (concrete, clay, perforated or non-perforated, and the shape of each unit), the arrangement of units, and other items. The reaction of a brick wall to horizontal forces parallel to it, except for the strength and homogeneity of the wall structure, depends on the location and the size of the openings (Chini, 2004: 310). In fact, a wall with its openings can behave like a frame with short and thick beams or like a set of independent vertical beams rigid in base. Given such conditions, it seems that the use of complex models established in the specialized literature is effective in certain cases, but it is not possible as a common general design. Therefore, plastic failures, which are mainly used in isotropic[†] models mainly for concrete, are also accepted in the case of masonry structures.

On the basis of this behavioral model of materials, during the application of loads in an increasing manner, orthotropic[‡] behavior is completely eliminated, but the average strength and stiffness of the structure along the main axes of the constituent materials still remain the same, in a way that the structural behavior is marginally affected by this inaccuracy. Besides in this model, both failure in tension and cracking in compression are well coordinated with the material model framework, which leads to good development in model stability, in a variety of different problems, including seismic excitation, a subject that is of paramount importance for the masonry structures.

It is worth noting that for the analysis of masonry structures, only the use of compatible 3D FE models is recommended. Such models must be capable of reproducing the strength and stiffness of a material after destruction, which emerge by masonry materials in the non-linear range. Based on such an approach in analysis, and considering the geometric effects of large deformations and nonlinear behavior of materials, the Concrete Damage Plasticity (CDP) model, which is fully available in standard analysis and design packages in the software like Abacus is used for masonry structures. The CDP model is based on the assumption of linear isotropic failure of materials, along with separate criteria of compressive and tensile failure. This model is generally suitable for functions in which materials, especially under loading-unloading modes, exhibit failure, and are therefore suitable for seismic analysis of the structure. Therefore, in this model, there will be a separate nonlinear behavior for tensile and compressive stress, as shown in the figure below.

To describe multidimensional behavior in the nonlinear range, it is assumed that the masonry structure follows the Drucker-Prager resistance criterion with the unrelated flow rule. The resistance range of a standard Drucker-Prager level, modified by a concept called the Kc parameter, is equal to the ratio of yield stress in the three-axial tensile test to its corresponding value in compression mode (Figure below). The Kc parameter is taken as 0.666 in numerical modeling.

[†] With similar physical characteristics

[‡] Is a special state of isotropic, namely their properties depend on the direction on which they are measured. The orthotropic materials have two or three orthogonal axes and in general, the mechanical properties of materials along each axis differ from those along the other axis.

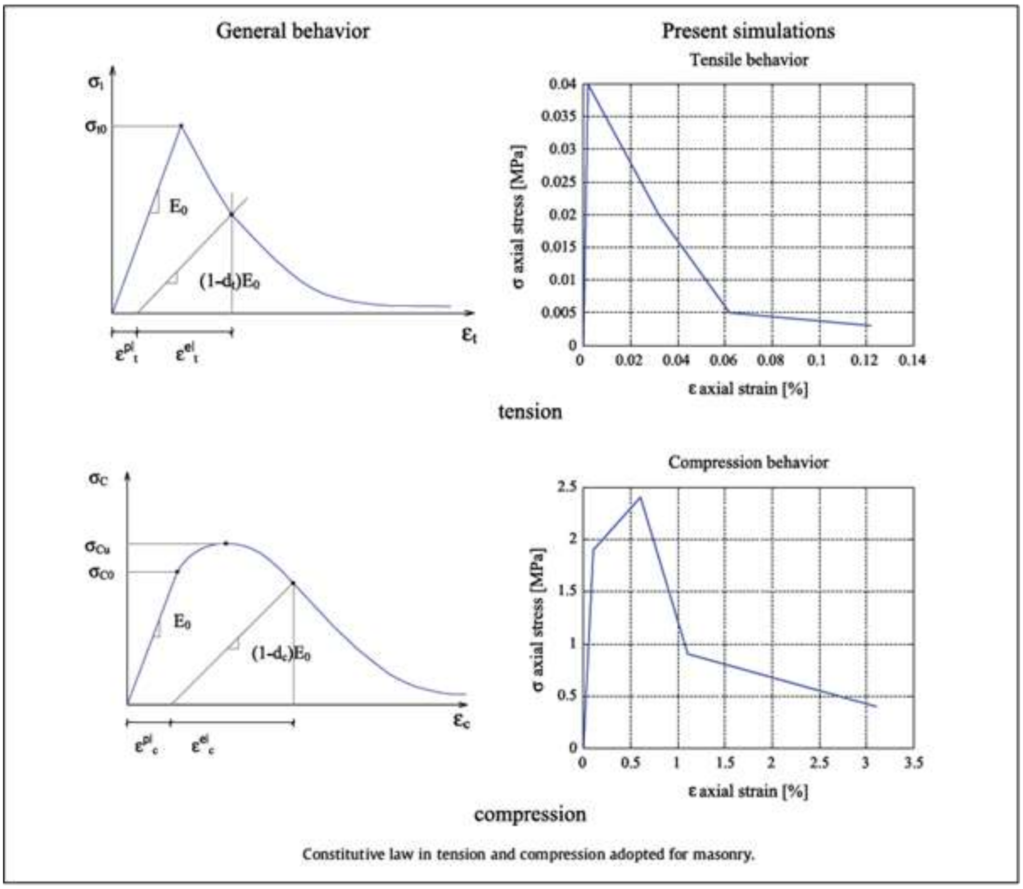

Fig 1 Plastic strain and non-elastic strain in the graph of the concrete response to a uniaxial load. Above: Tensile, below: Compressive (Rafati and Toosi, 2014: 3)

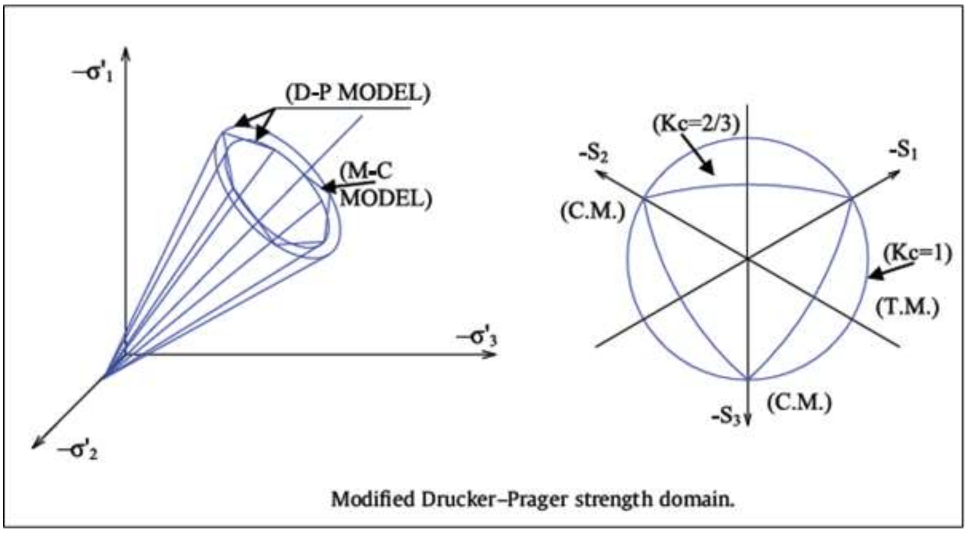

Fig 2 Drucker-Prager boundary levels. Right: General shape. Left: Deviatoric plane (Shakibapour and Raouf Sheibani, 2018: 6)

A 10-degree angle is chosen for the angle of dilation, which is acceptable for a masonry structure exposed to vertical pressure in the range of medium to low stress. This value is in line with the empirical evidence available in the literature of this type of structure. To prevent cases and problems of non-convergence in the analysis, the tip of the Drucker-Prager cone is softened using a hyperbolic curve (Cong Lu, 2009: 35). Abacus software allows for the softening of the resistance range by a concept called the eccentricity parameter. This parameter provides the distance between the points of contact of the cone and the hyperbola with the p-axis on the p-q plane, where the p and q axes are the compressive hydrostatic stress and the equivalent Von-Mises stress, respectively (Figure below). In numerical modeling, a value of 0.1 was accepted for the eccentricity parameter.

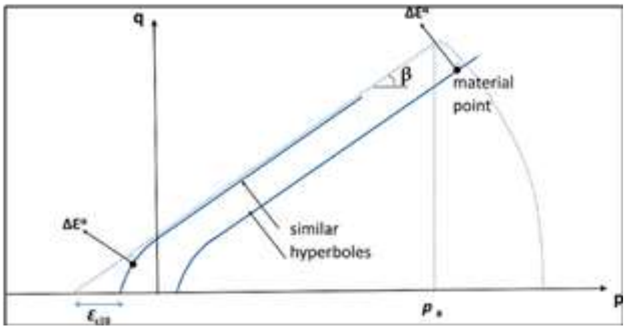

Fig 3 p axis: compressive hydrostatic stress, q axis: equivalent Von-Mises stress (Cong Lu, 2009: 35)

The available experimental results for conventional masonry structures show a moderate orthotropic ratio (in the range of 1.2) under biaxial stress states in compressive-compressive regions. Obviously, such a feature cannot be considered where in an isotropic model like the current ones.

Of course, in the literature of this type of structure, the use of other isotropic models (such as the model with the behavioral function of concrete smeared crack), as a behavioral model definable in both Ansys and Abina software is generally accepted after matching the parameters, which is performed to calibrate the average behavior in the vertical and horizontal compressive stress states.

An appropriate model also counts the ratio between the ultimate compressive strength in the biaxial stress states to the corresponding value in the uniaxial stress state. Such a ratio, which is the same for concrete and masonry structures, is logically considered equal to 1.16. The values of the various parameters accepted for numerical modeling of the masonry structure are reported in table 1.

The stress-strain relation in the final state accepted for dynamic analysis consists of a linear elastic section to the maximum σt stress, which is then directed to the microscopic hardening stage with the emergence of formation of hairline cracks and their development within the material. In the case of compressive stress, the reaction of the material to stress is linear until the yield stress σc. A linear hardening is then assumed until the cracking stress of σcu, followed by the non-linear hardening section. The failure variables in tension dt and compression dc are defined by the following standard relations:

$$\sigma_t = (1 - d_t)E_0(\varepsilon_t - \varepsilon_t^{pl})$$
$$\sigma_c = (1 - d_c)E_0(\varepsilon_c - \varepsilon_c^{pl})$$

Table 1 Values of the parameters accepted for numerical modeling of the masonry structure (Source: Authors)

Values of the parameters accepted for numerical modeling	
Parameter	Value
Poisson ratio	0.2
Angle of dilation	10
Eccentricity	0.1
σbo/σco	1.16
k	0.666
Viscosity	0.002

4. Analysis

In this project, nonlinear dynamic analysis was selected with one level of numerical complexity in order to cover the structure of the dome, which has an average to high complexity in function. In this regard, the following should be regarded: The structure of the dome for the analysis of a three-dimensional meshed model with modified four-sided elements included Quadratic tetrahedron, type C3D10M elements with 30994 nodes and 17595 elements. In this modeling, the dome structure-foundation interaction was ignored and the foundation was considered to be rigid. Generally, the damping of the masonry structures is taken about 2 to 5%. Therefore, in this project, the damping property of the materials of this building was considered to be approximately 3% for the first mode of vibration of the structure. Assuming the Riley damping method, the β factor for the natural frequency of the first mode of vibration of the structure for 4.7217 was equal to 2*.03/4.7217 = 0.0.0127. Since significant nonlinear states, such as the probability of instability due to cracking, are expected in the structure, general convergence in analysis is expected to be non-uniform. In such cases, the automatic option should be selected to adjust the parameters related to the time components in order to prevent untimely interruption of the analysis process. In this regard, the related parameters were set as follows:

Tolerance of the half-increment residual = 1e07

Maximum time increment = 0.02

While pre-seismic analysis mode can be run in abaqus/explicite, the quasi-static analysis mode is much more efficient in abaqus/standard. On the other hand, abaqus/explicite requires a very high number of time components because the stable time component (6e-10) is much smaller than the total time of the earthquake. Moreover, the use of double precision and mass scaling affects the dynamic reaction of the structure. The implementation cost in abaqus/explicite is very high due to the very small size of the element. Moreover, because the earthquake load is of a relatively long duration relative to the time components, the time components in this type of analysis will be very large. Therefore, for this type of problem, abaqus/standard is more effective than the abaqus/explicite analysis. So here the solution starts with gravity loads in the abaqus/standard analysis and the results with continue with abaqus/explicite. A significant number of nonlinear functions have symmetrical Jacobean matrices, and this matrix does not change much in one resumption of the analysis compared to the next resumption. Implicit dynamic problems with smaller time components relative to the natural vibration period of the structural response are examples of this case. In this case, in particular when the problem is of a large dimension, using an alternative to the Newton method approximation in solving nonlinear equations will reduce the running costs. The quasi-newton method is such an alternative that has been presented by various researchers for systems with symmetrical Jacobean matrices. This method can be used in abaqus/standard that can be replaced by the Newton method, which is the default of this case. In case of selecting this method, a value must be selected for the allowable resumption of the analysis before the base matrix is deformed. The maximum resumption value is 25 and the default value is 8.

5. Nonlinear Dynamic Analysis

In the first stage, gravitational loads are slowly applied to the structure, without the structure being subject to gravitational acceleration. In the second stage, the horizontal movement of the earth on the structure base (the interface of the structure and the earth) are applied to the structure in the presence of gravitational loads. However, in several technical studies, it has been indicated that the application of the vertical component of the earth's motion does not have a significant effect on the masonry structures. Therefore, its effects have been neglected in the present study. In this study, the acceleration curve obtained from real accelerographs in the Tabas Earthquake, Iran, was used to evaluate the seismic response of the dome. This curve was applied to the structure after normalization to gravity acceleration (according to code 2800, the maximum acceleration will be equal to the gravity acceleration) with a coefficient of 0.3 g (PGA = 0.3 g). In summary, the structure was first subjected to the gravitational load due to the weight of the structure itself in STEP1 and then in STEP2 was exposed to the lateral load of the earthquake. The gravity acceleration coefficient in the application of the inertial seismic load on the structure is equal to 0.3*g. The acceleration applied to the structure is presented in the following figure.

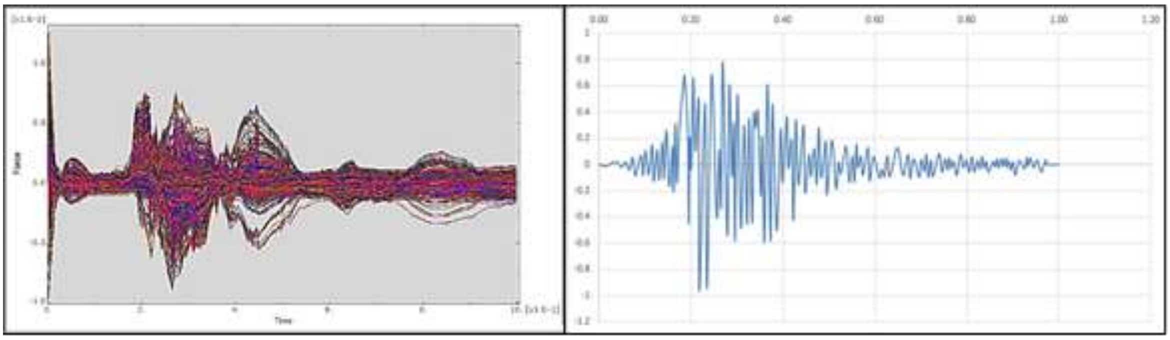

Fig 4 Right: STEP1 (Graph of acceleration applied to the structure); Left: STEP2 (Graph of distribution of the support force in the nodes of the structure floor); Reference: Authors

The provisions of Code 2800[§] in this regard are as follows:

2.5.3.3. A pair of accelerographs selected for three-dimensional analysis of structures should be scaled as follows:

[§] Building Design Regulations against Earthquake, Standard 2800, Road, Housing and Urban Research Center, Edition 4, Publication Number: D-253

A. Each pair of accelerographs is scaled to their maximum value. This means that the maximum acceleration in the component with the larger maximum value is equal to the gravity acceleration g.

B. The acceleration response range of each pair of the accelerographs scaled should be determined considering 5% damping ratio.

C. The response ranges of ... are compared with the standard design range. The scale factor should be determined so that ... in no case is less than 1.4 times the corresponding value of the standard range.

D. The scale factor determined must be multiplied by the accelerograph of paragraph A and used in dynamic analysis.

At the end of the time history analysis, the horizontal displacement of some control points during the occurrence of the failure phenomenon, generally the higher nodes of the structure, were qualitatively examined in order to determine whether the structure was in the initial stage of destruction or not. The numerical analysis was conducted with a dynamic approach based on implicit integration over time and using a time interval of 0.005 seconds, which corresponds to the time interval for recording the acceleration function. The results of the nonlinear dynamic analysis resulting from the implementation of the program are presented in the next section.

6. Numerical Results of Nonlinear Dynamic Analysis

The time history of acceleration, velocity, and displacement of the control node (above the dome) in the direction of the y-axis and under the ground acceleration with PGA = 0.3g under dynamic nonlinear analysis are presented in the following figures:

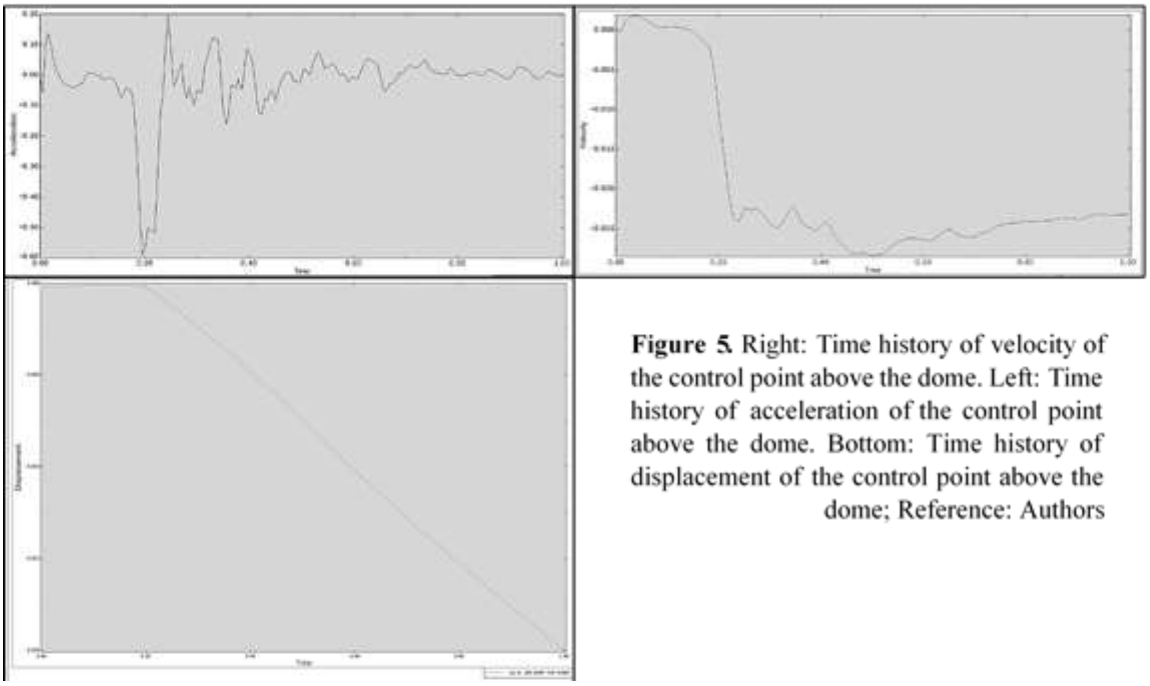

Figure 5. Right: Time history of velocity of the control point above the dome. Left: Time history of acceleration of the control point above the dome. Bottom: Time history of displacement of the control point above the dome; Reference: Authors

It can be roughly stated that if the residual deformation, defined as the ratio of the non-elastic residual horizontal displacement to the height of the structure, is between 0.4% and 0.8%, the structure can be logically considered to be in the threshold of the collapse state. This is clearly the case with the dome structure. The residual deformation values of 0.4% and 0.8% were adopted by referring to the failure of foundations with masonry materials, respectively, under the shear and bending applied on their planes, which match with the international design codes related to this type of structure. However, the choice of these figures is debatable, because the structural behavior of the dome with the masonry materials cannot be compared with the behavior of individual foundations with the masonry materials. This can only be considered a sign of effort being made in this direction. The dynamic nonlinear analysis is capable of calculating the effects of higher modes that may occur due to local irregularities. The distribution of tensile failure corresponding to displacement is usually similar to the failure pattern observed at the end of the nonlinear dynamic analysis, however numerical values of failure usually show lower values. The only exception is the severe failure observed in the structure base. The strange geometric features and structural configurations of the structure are the main causes of high seismic resistance in masonry structures. Additionally, having thick surrounding walls, the absence of large openings that can facilitate the vulnerability of that part of the structure compared to the whole structure, symmetry in the plane and in height, and providing a regular internal distribution in the structure, are among the reasons for the stability of the Taj al-Molk Dome. In this structure, geometric features and the presence of irregularities in the plan and height have caused failure mechanisms that have been highlighted in the results of the analysis. Relative problems associated with the configuration of the structure, especially the cases related to the symmetry and insufficient regularity in the openings, have affected the structure failure level. The non-uniform distribution of stiffness and strength in the plan and height and the resulting torsional effects can be among the main causes of extensive failure and collapse of the masonry structure.

7. Conclusion

A comprehensive numerical study was carried out on the structure using FE modeling (nonlinear dynamic analysis) with the main objectives as follows:

1. In order to identify some of the general effects due to the geometric and morphological characteristics of the structure, such as openings, wall thickness, and horizontal and vertical irregularities, in the seismic performance of the structure using the finite element 3D model.
2. In order to evaluate the seismic safety of the structure using nonlinear dynamic analysis.

Based on the general analysis and using the results obtained in this study, the following results were obtained:

- The results of the nonlinear dynamic simulation of the structure revealed the high vulnerability of the Taj al-Molk dome structure under horizontal loads. As mentioned earlier, it can be roughly shown that if the residual deformation occurs in the range of 0.4% to 0.8%, the structure can be logically considered to be close to collapse.
- A number of geometric features, such as irregularity in plan and height, large openings, accidental changes in cross section, and the presence of the dome (or an internal sloping arch in the structure) play an essential role in seismic operation. In the analysis performed, the correlation between local geometric items and possible failure modes was clearly revealed by the numerical analyses.
- Furthermore, the results in the frequency analysis of the structure are listed in figure fre-1 presented in the appendix. The results of the first mode were used to calibrate the damping of the materials used during the seismic dynamic analysis of the structure. Figures fre-b (1) to fre-b (20) and fre-f (1) to fre-f (20) in the appendix demonstrate the vibration modes of the structure from the back and front views. The nonlinear dynamic modeling of the structure suggests that the structure of the Taj al-Molk dome is very vulnerable to horizontal loads.

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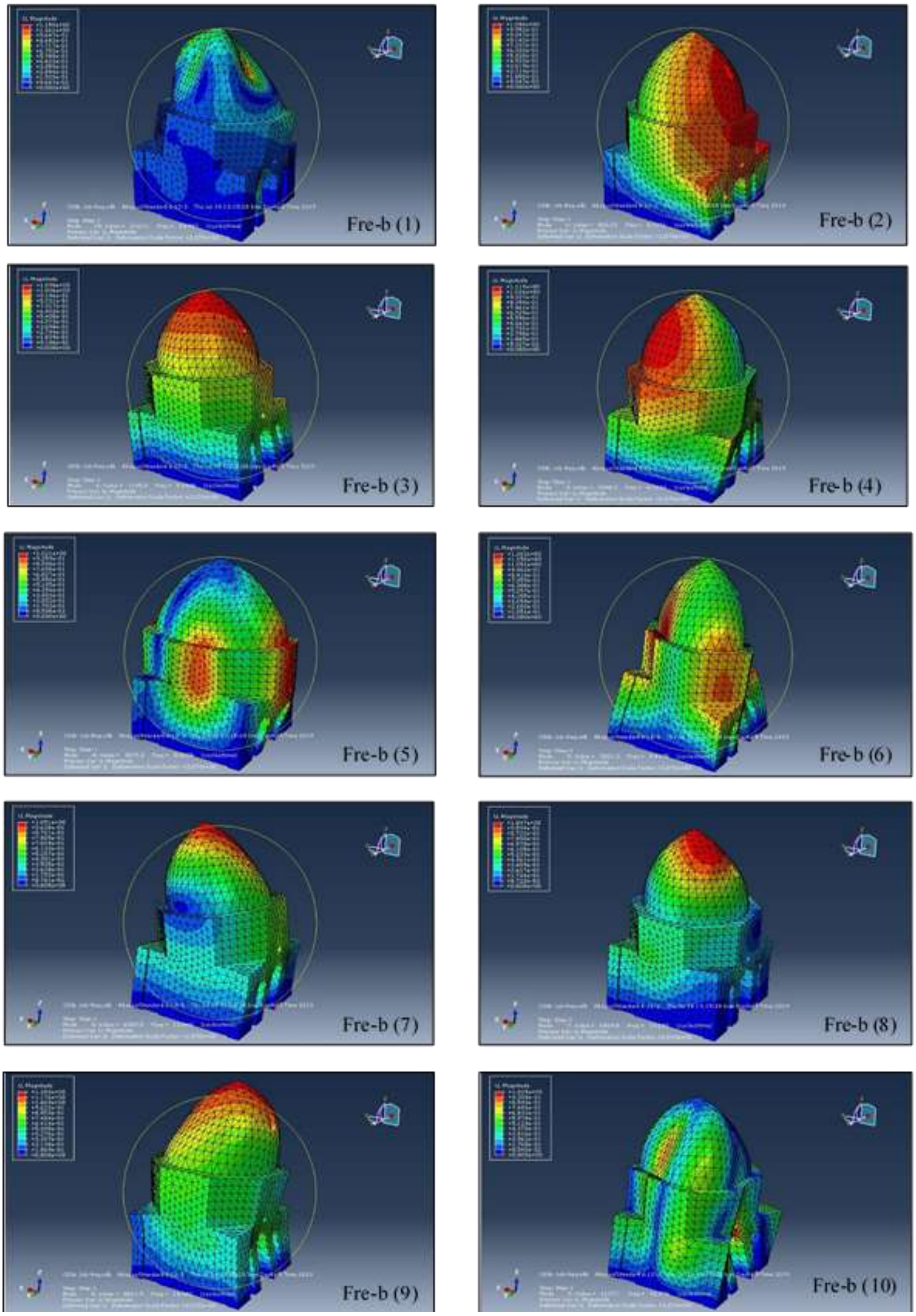
Appendix


Fre-b (1)
Fre-b (2)
Fre-b (3)
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Fre-b (10)

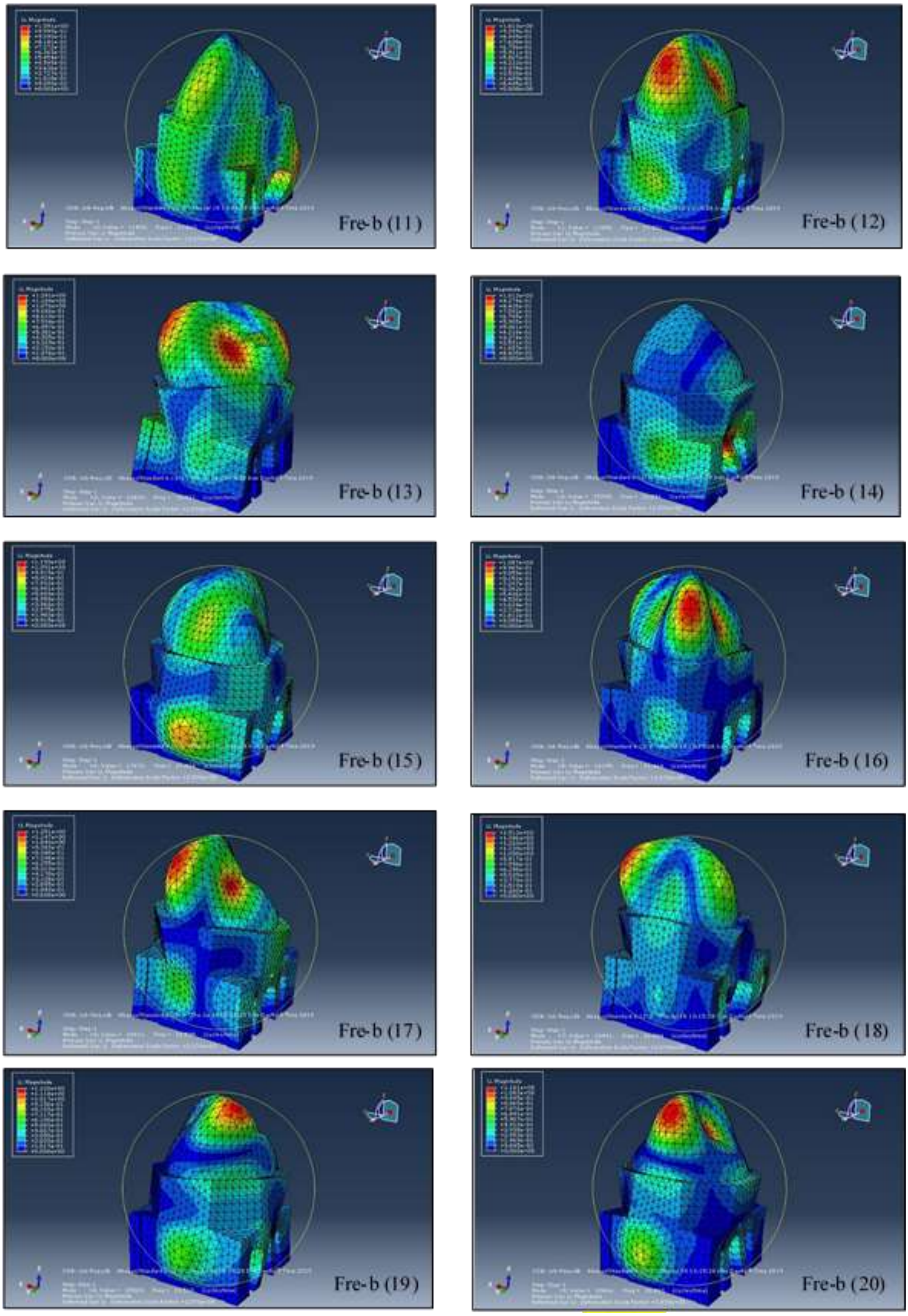
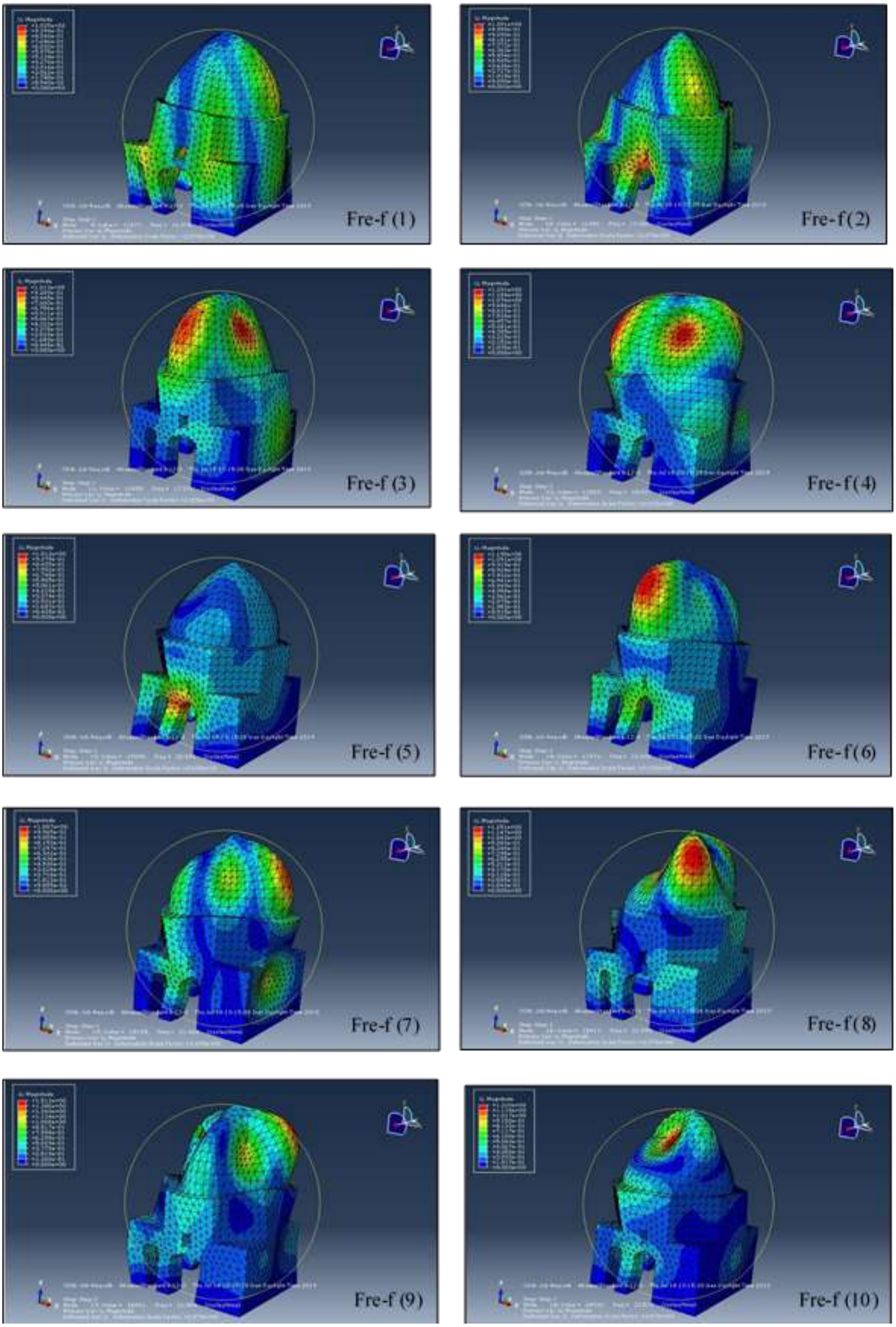


Fig 6 Fre-b(1-20); Vibration modes of the structure on the back view (Source: Authors)


Fre-f (1)
Fre-f(2)
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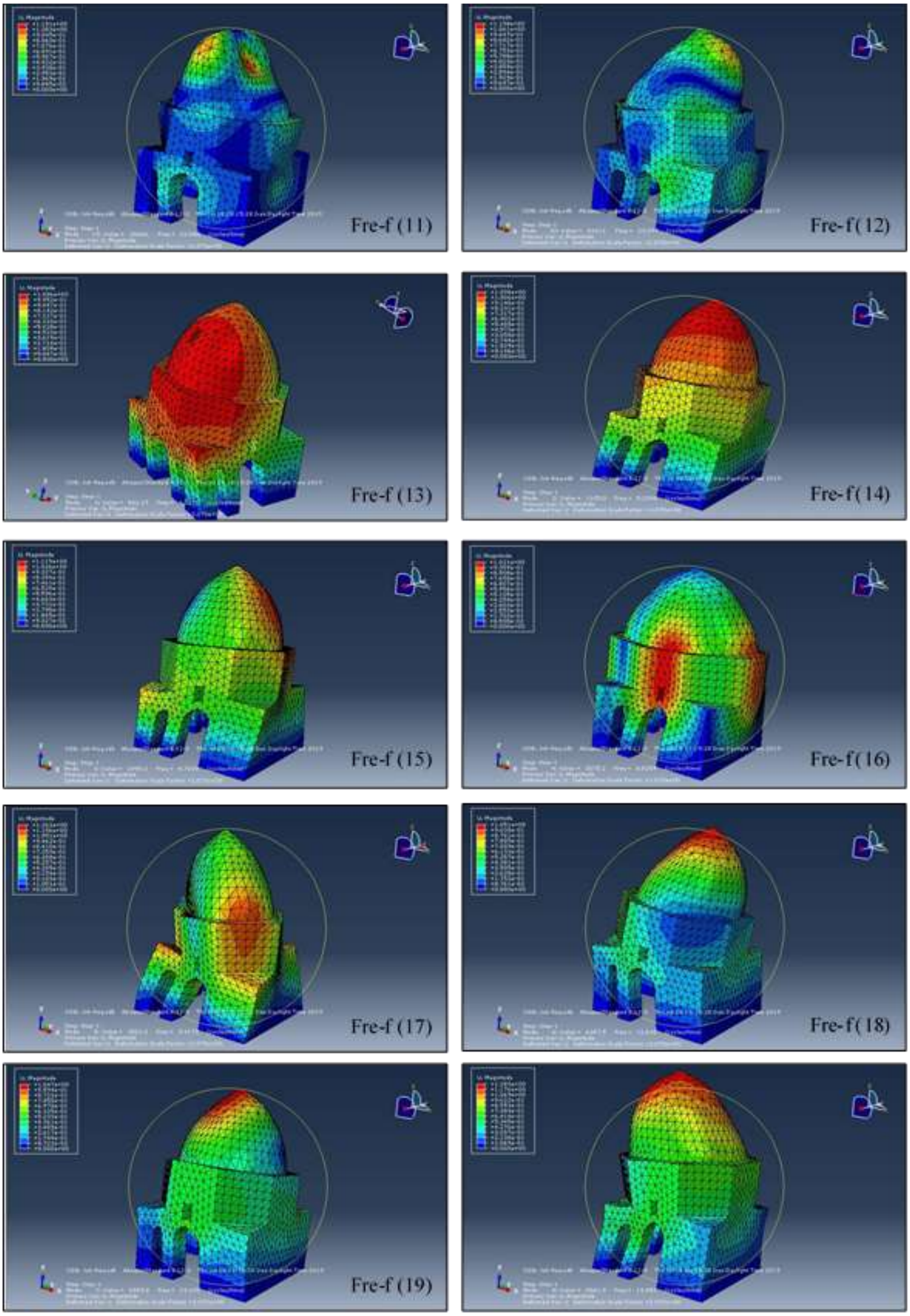

Fig 7 Fre-b(1-20); Vibration modes of the structure on the front view (Source: Authors)