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Effect of model calibration on seismic behaviour of a historical mosque

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Abstract. The objective of the study is to investigate the effects of model calibration on seismic behaviour of a historical mosque which is one of the most significant Ottomon structures. Seismic analyses of calibrated and noncalibrated numeric models were carried out by using acceleration records of Kocaeli earthquake in 1999. In numerical analysis, existing crack zones on real structure was investigated in detail. As a result of analyses, maximum stresses and displacements of calibrated and noncalibrated numerical models were compared each other. Consequently, seismic behaviour and damage state of historical masonry Hafsa Sultan mosque was determined as more realistic in the event of a severe earthquake.

Keywords: historical masonry structure; operational modal analysis; seismic analysis; time history; seismic safety

1. Introduction

Historical structures are parts of wealth of cities and countries as well as their historical and spiritual assets. Seismic behaviours of these significant structures should be determined better and in a more detailed way to hand down to the next generations safely. Handing down the historical structures having faced a number of natural disasters and earthquakes throughout their lives to the next generations safely is one of the most important issues in the world.

There are many studies for determining seismic behaviours of historical and modern structures by using Operational Modal Analysis (OMA) and finite element analysis methods. In these studies, seismic behaviours of structures such as historical churches, towers, bridges, factory chimneys, minarets and mosques were examined (Gentile and Saisi 2007, Bayraktar, Altunışık *et al*. 2011, Boscato, Dal Cin *et al*. 2012, Votsis, Kyriakides *et al*. 2012, Foti, Diaferio *et al*. 2012, Bartoli, Betti *et al*. 2013, Foraboschi 2013, Bednarz, Jasienko *et al*. 2014, Preciado 2015, Preciado, Orduña *et al*. 2015, Cakir 2015, Bayraktar, Türker *et al*. 2015).

OMA is a modal analysis method which is performed by obtaining real time data from a structure which is in use. The vibrations occuring due to environmental reasons are collected from structure by accelerometers. Then, dynamic modal properties of structure are obtained by using

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different algorithms such as the Enhanced Frequency Domain Decomposition (EFDD) technique, in the Refs. (Jacobsen, Andersen *et al*. 2006) and the Stochastic Subspace Identification (SSI) technique, in the Refs. (Bayraktar, Sevim *et al*. 2009).

Ramos, Marques *et al*. (2010) researched damage in historical clock tower and church by structural health monitoring process that helps in the preservation of historical structures. Dynamical properties of the structures were obtained with OMA method and damage was assessed. Then, the effect of environmental factors on modal parameters was determined and it was observed that temperature and moisture non- ignorably affected dynamic parameters. Osmancikli, Ucak *et al*. (2012) determined dynamic characteristics of the structure by performing pre and post-restoration OMA experiments on Ayasofya Museum Bell Tower in Trabzon and investigated the effect of restoration on dynamic characteristics of the structure. It was established that pre and post-restoration first 5 modes were the same and frequency values were close to the each other. As a conclusion, it was stated that restoration projects should be designed by considering structure response determined with OMA method. Ramos, Aguilar *et al*. (2013) researched dynamic parameters of the Church of Saint Torcato in Portugal with significant structural problems due to soil settlement with OMA method. In the long-term measurements, it was understood that in the event of damage of structure, dynamic characteristics might change and in this way, damage in the church might be controlled with dynamic monitoring system. For this reason, they proposed the estimation of dynamic characteristics before reinforcing the structure and implementation of experimental tests such as finite elements improvement and dynamic monitoring. Calik, Bayraktar *et al*. (2014) determined the dynamic characteristics of Kucuk Fatih mosque in Trabzon, restored in 2012, with OMA method. The various cracks in the structure were observed on masonry arches after plaster was removed. Dynamic characteristics before and after the strengthening of the structure were found and compared. It was observed that frequency values increased and damping rates varied irregularly after the strengthening. Moreover, while the first and second modes were similar in the analysis, other modes were different. Cakir, Bayraktar *et al.* (2015) investigated two historical structures collapsing partially in 2011 Van earthquake. Nathexes of both structures collapsed due to the earthquake. Finite element models of these structures were prepared and reasons for collapse of the structures were revealed with several seismic codes.

The seismic behaviour of historical Hafsa Sultan mosque in Manisa, one of the historical cities of Turkey, was investigated in this study. Dynamic characteristics of the structure were determined with a study carried out by Nohutcu, Demir *et al*. (2015) by using OMA method. Scaled acceleration records of 1999 Kocaeli earthquake were applied to calibrated and noncalibrated finite element models of the structure and linear analysis in time history of the structure was performed. In this way, the effect of model calibration on seismic behaviour of the structure was determined. Maximum stresses and displacements in the structure were presented comparatively. At the same time, locations of existing cracks in the structure were found and these zones were investigated in the numerical analysis in a more detailed way.

2. Material and methods

2.1 Geometric and structural characteristics of Hafsa Sultan Mosque

Hafsa Sultan Mosque in Manisa, one of the historical cities of Turkey, is also one of the most significant Ottoman structures built by Kanuni Sultan Suleyman in the 16^{th} century. Face stone,

Fig. 1 Hafsa Sultan mosque

Table 1 Mechanical parameters of masonry wall for numerical analysis (Nohutcu, Demir *et al*. 2015)

Mechanical parameter	Masonry Wall
Compressive strength (MPa)	7.42
Tensile strength (MPa)	0.74
Young's modulus (MPa)	1500
Shear modulus (MPa)	600
Density (kg/m^3)	2200
Poisson's ratio	0.17

brick and Khorasan mortar were used for construction. Walls constituting the carrier system are 1.40 m in width. Lay-out of the mosque is rectangular and 29.325×21.33 m in size. There are a great dome and four small domes at the sides of the great dome. Besides, there are 5 small domes and 6 marble circular columns carrying them in the courtyard (Fig. 1). The great and small domes are 11.8 m and 5.25 m in width, respectively.

Walls of Hafsa Sultan Mosque were built with rough-hewn grey/pink andesite stone and mortar between them and arches were built with clean cut andesite stone. Horizontal lengths of stones used in the structure vary between 25 cm-70 cm. Thicknesses of stones vary between 20 cm and 30 cm. The use of pink and grey andesite in the structure is equal and at acceptable level. Stone/brick and mortar constituting the masonry structure was thought as a whole and modeled with homogenization approach. Material values found in the tests and empirical formulas are presented by Nohutcu, Demir *et al*. (2015) in detail (Table 1).

2.2 Performance concept for historical structures

Some methods have been developed for investigating of seismic safety of historical structures (Terenzi and Sorace 2002, Lourenço, Oliveira *et al*. 2013, Asteris, Chronopoulos *et al*. 2014). Terenzi and Sorace (2002) proposed three performance levels for the structures. The performance levels are no damage expectancy (ND), conservation of cultural values (CVS), prevention of collapse (CP). Terenzi and Sorace (2002) proposed to use displacement instead of crack width for structural assessment. Because crack width is difficult to exactly determine by finite element programs.

It is suggested that first of all, material models and ground motion parameters should be defined and then defined limits on the basis of dislocations (d_{lim}) for performance of the structure in the analysis performed, ND and CVS performance levels should be controlled and maximum moment (*Mu*) method will be used for CP performance level and finally capacity investigations should be performed.

For displacement limits, d is for displacement and h is for member height, for d/h the tangent value of angle with vertical of member, value for ND level was suggested as 0.0005 and as 0.001 for CVS (Terenzi and Sorace 2002).

2.3 Scaling earthquake acceleration by spectrum curve

Scaling real acceleration records by design acceleration spectrum was carried out with time history method. In time history method, frequency content of the record is not changed, only amplitude of record is changed. Response spectrum of records is grouped by the type of soils and design spectrum of the relevant soil type is used and scaling coefficient α of record is calculated with the Eq. (1). Properties of the structure soil were determined as group C soil and Z2 soil class as a result of soil tests performed.

$$\alpha = \frac{\sum_{T=T_1}^{T_2} (S_a^{actual}(T) S_a^{target}(T)}{\sum_{T=T_1}^{T_2} (S_a^{actual}(T)^2} \tag{1}$$

Where, S_a^{target} is the target acceleration response spectrum, S_a^{actual} is the acceleration spectrum of actual earthquake record and α the linear scaling coefficient, T is period of structure, T_1 is the lower limit of period for scaling, T_2 is the upper limit of period for scaling. As a result of calculations, scaling coefficient α was calculated as 2.75.

The region of Hafsa Sultan Mosque is active in terms of seismicity. The last moderate earthquake in Manisa Center occurred on 28th January 1994 with the magnitude of its was 5.1. As a result of seismicity study of the region, it was determined that it is possible that severe earthquake may occur in Manisa Center. For this reason, acceleration records of Kocaeli earthquake were used in seismic analysis of Sultan Mosque. The records were provided by Pacific Earthquake Engineering Research Center (PEER).

The east-west component with the biggest horizontal values in Kocaeli earthquake records was applied to direction *y*, being the weak direction of Sultan Mosque and north south component was applied to direction *x* and vertical component was applied to direction *z*.

2.4 Existing cracks in Hafsa Sultan Mosque

Modal parameters of structure are significantly affected with cracks which were occurred due to severe earthquakes in the past. The cracks were marked on the structure as shown in Fig. 2. 11 significant cracks were observed on the structure. Other than these, nodal point no 12 represents

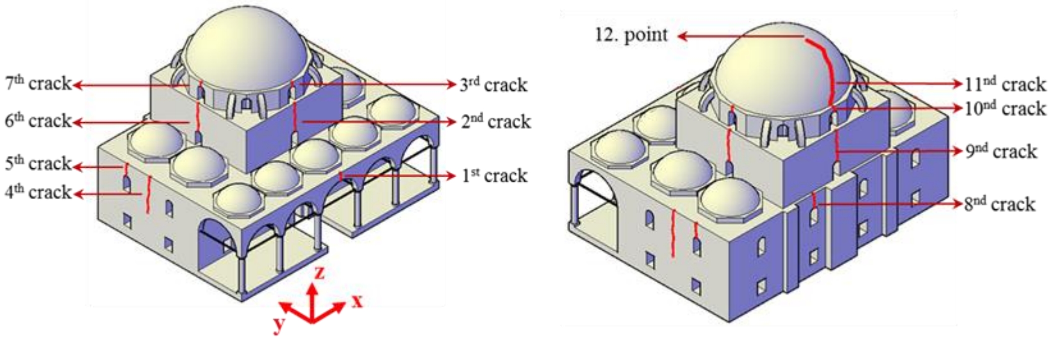

Fig. 2 Existing cracks on Hafsa Sultan mosque

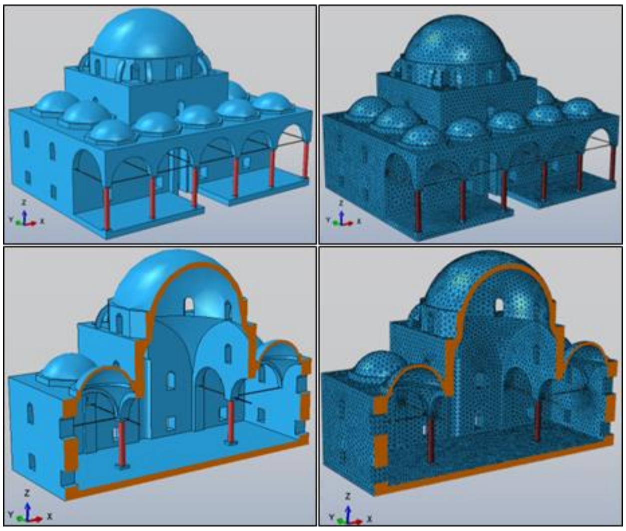

Fig. 3 Three-dimensional finite element model of Hafsa Sultan mosque

the point at the top level of the dome. Assessments were carried out based on zones of existing cracks in the structure in order to control and compare maximum displacements, compression and tensile stresses found as a result of analysis.

2.5 Numerical analysis of the structure and calibration of numerical model

Numerical model of the structure was created with ABAQUS program by use of tetrahedral solid elements (C3D4). At the modeling stage, 673527 solid elements and 145442 nodal points were used. There are rings with the property of hinges in lower and upper parts of the columns. Columns were modeled independently from the structure at the stage of modeling and interaction was defined between joint regions of column and the structure. By means of interaction, columns of the structure work for the compression only. Three-dimensional finite element model of Hafsa Sultan mosque is shown in Fig. 3.

Nohutcu, Demir *et al*. (2015) determined dynamic characteristics of historical Hafsa Sultan mosque with the OMA method and calibrated numerical model of the structure. OMA were

Table 2 Experimental and numerical frequencies before and after calibration of the structure (Nohutcu, Demir *et al*. 2015)

Mod	Frequencies with FEM		Operational Modal Analysis			
			Frequencies with EFDD (Hz)		Frequencies with SSI (Hz)	
	Before calibration	After calibration	Test 1	Test 2	Test 1	Test 2
1	3.197	2.908	2.902	2.908	2.902	2.903
2	5.092	4.633	4.577	4.580	4.574	4.590
3	5.176	4.703	4.648	4.675	4.669	4.680
4	5.984	5.454	5.510	5.494	5.502	5.480
5	6.656	6.059	6.179	6.133	6.098	6.112

EFDD: Enhanced Frequency Domain Decomposition, SSI: Stochastic Subspace Identification

Table 3 Comparison of displacements before and after calibration

Node points	Before OMA	After OMA	Ratio (Calibrated/ Noncalibrated)
	y-*y* direction (m)		
1	0.085	0.212	2.49
2	0.096	0.262	2.73
3	0.099	0.269	2.72
4	0.024	0.061	2.54
5	0.025	0.059	2.36
6	0.081	0.201	2.48
7	0.092	0.232	2.52
8	0.071	0.190	2.68
9	0.099	0.257	2.60
10	0.102	0.261	2.56
11	0.102	0.257	2.52
12	0.108	0.250	2.53
	Average		2.56

performed by using EFDD and SSI algorithms with ARTEMIS software program. Young's modulus of the structure was reduced to 1210 MPa from 1500 MPa following a number of tests and calibrated model was constituted. Calibrated and noncalibrated frequencies of the model are presented in Table 2.

3. Seismic behaviour of historical Hafsa Sultan mosque

3.1 Displacements

It was seen that maximum displacements in the structure were in the direction of y in the seismic analysis performed before and after calibrating the numerical model of the structure. Maximum displacements occurring in the structure are observed in crack zones numbered 2, 3, 9, 10, 11 and main dome peak point numbered 12 (in Table 3).

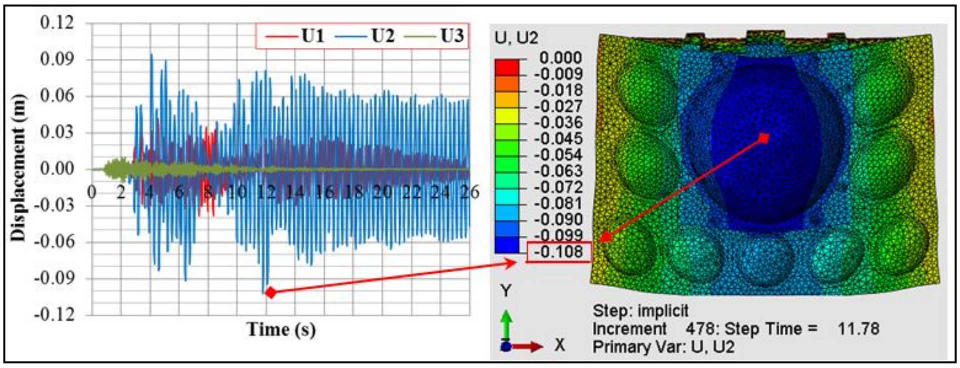

Fig. 4 The maximum displacements occurring before calibration

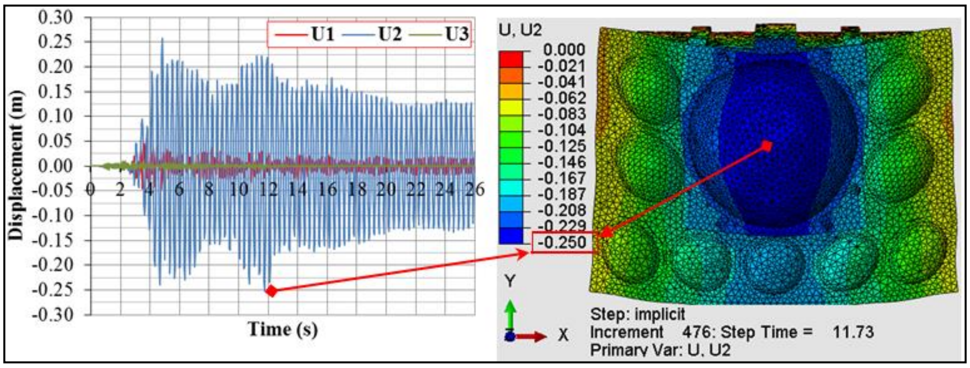

Fig. 5 The maximum displacement occurring after calibration

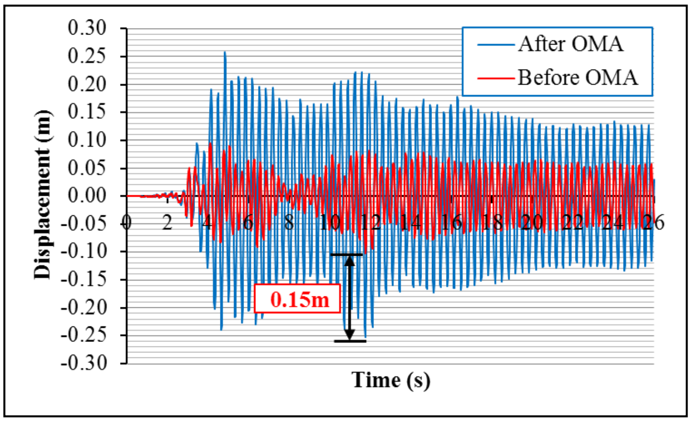

Fig. 6 Comparison of maximum displacement time graph in the main dome peak point

Displacements in the calibrated numerical model increased average 2.56 times than noncalibrated model. The displacement value in main dome peak point was calculated as 0.108 m before the calibration (in Fig. 4) and as 0.250 m after the calibration (in Fig. 5) and the difference was about 0.15 m as shown in Fig. 6. It is seen that calibration of model significantly affected displacements.

Table 4 Comparison of the tensile stresses in the calibrated and noncalibrated models

Node points		Stress before calibration (MPa)			Stress after calibration (MPa)			Ratio (Calibrated/ Noncalibrated)		
		σ_{11}	σ_{22}	σ_{33}	σ_{11}	σ_{22}	σ_{33}	σ_{11}	σ_{22}	σ_{33}
1	Tensile	5.92	0.11	0.29	11.04	0.89	0.66	1.86	2.63	2.28
2	Tensile	4.56	0.62	2.24	8.32	0.99	3.05	1.82	1.60	1.36
3	Tensile	4.32	0.93	1.92	7.77	1.54	2.04	1.80	1.66	1.06
4	Tensile	2.12	1.84	0.47	0.55	2.58	0.42	0.26	1.40	0.89
5	Tensile	0.42	2.61	1.85	0.84	4.48	3.59	2.00	1.72	1.94
6	Tensile	0.87	2.76	1.83	1.89	3.67	3.72	2.17	1.33	2.03
7	Tensile	0.42	4.09	1.77	0.81	3.85	3.14	1.93	0.94	1.77
8	Tensile	5.23	0.73	1.72	10.22	1.16	2.82	1.95	1.59	1.64
9	Tensile	5.81	0.68	2.44	10.25	1.93	3.67	1.76	2.84	1.50
10	Tensile	4.81	0.73	1.7	8.64	1.27	2.41	1.80	1.74	1.42
11	Tensile	0.64	0.52	0.81	1.41	0.64	0.80	2.20	1.23	0.99

Table 5 Comparison of the compression stresses in the calibrated and noncalibrated models

Node points		Stress before calibration (MPa)			Stress after calibration (MPa)			Ratio (Calibrated/ Noncalibrated)		
		σ_{11}	σ_{22}	σ_{33}	σ_{11}	σ_{22}	σ_{33}	σ_{11}	σ_{22}	σ_{33}
1	Compression	-5.51	-0.13	-0.28	-12.56	-0.25	-0.61	2.28	1.92	2.18
2	Compression	-4.56	-0.59	-1.99	-9.85	-1.09	-3.07	2.16	1.85	1.54
3	Compression	-4.21	-0.84	-2.04	-8.94	-1.85	-2.07	2.12	2.20	1.01
4	Compression	-2.16	-1.97	-0.47	-0.62	-2.99	-0.43	0.29	1.52	0.91
5	Compression	-0.39	-2.52	-1.84	-0.75	-4.29	-3.36	1.92	1.70	1.83
6	Compression	-0.93	-2.11	-1.74	-1.88	-3.76	-3.75	2.02	1.78	2.16
7	Compression	-0.42	-3.56	-1.68	-0.89	-3.58	-3.12	2.12	1.01	1.86
8	Compression	-4.85	-0.67	-1.79	-10.36	-1.29	-3.11	2.14	1.93	1.74
9	Compression	-5.43	-0.66	-2.44	-10.36	-1.16	-3.73	1.91	1.76	1.53
10	Compression	-4.57	-0.68	-1.98	-8.36	-1.25	-2.15	1.83	1.84	1.09
11	Compression	-0.69	-0.51	-0.82	-1.29	-0.65	-0.79	1.87	1.27	0.96

3.2 Stresses

As a result of seismic analysis, tensile and compressive stresses of calibrated and noncalibrated models is presented in Table 4 and Table 5, respectively.

It was observed in calibrated model that tensile stress was 1.78 times higher in the direction of x and 1.70 times in the direction of y and 1.54 times higher in the direction of z compared to noncalibrated model.

It was seen in calibrated model that compressive stress was 1.88 times higher on average in the direction of x and 1.71 times in the direction of y and 1.53 times higher in the direction of z compared to noncalibrated model. Change of stresses in the direction of x in the first crack zone by

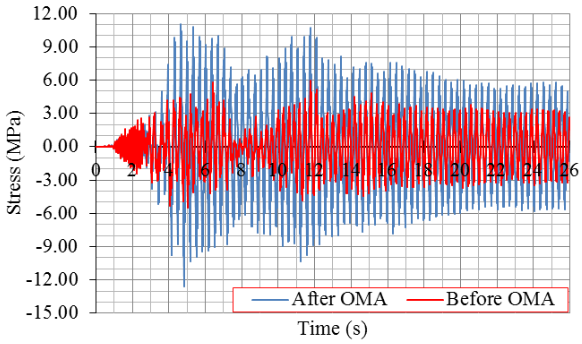

Fig. 7 The stresses in the direction x in the first crack zone (σ_{11})

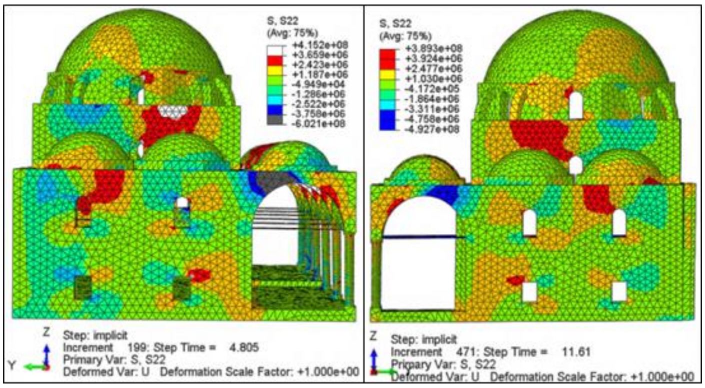

Fig. 8 Stress concentrations on model calibrated of Hafsa Sultan mosque (σ_{22})

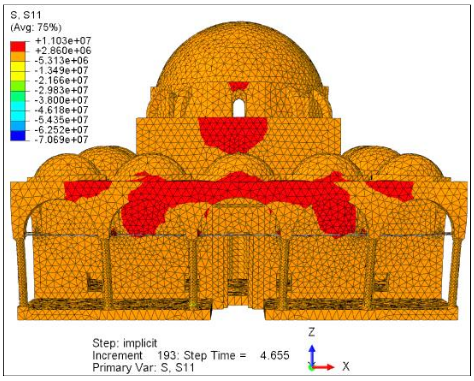

Fig. 9 Stress concentrations on model calibrated of Hafsa Sultan mosque (σ_{11})

the time is comparatively presented in Fig. 7.

Compressive and tensile stresses were compared with limit values in Table 1. Stresses exceeding limit value following model calibration of Hafsa Sultan Mosque become intense in domes and around the windows as seen in Fig. 8. It is seen that the stresses are in accordance with 4th, 5th, 6th, 7th existing cracks in Fig. 2.

Stresses exceeding limit values following model calibration of Hafsa Sultan Mosque become intense around windows and entry arch where the existing cracks as seen in Fig. 9. It is seen that the stresses are in accordance with 1^{st}, 2^{nd}, 3^{rd}, 8^{th}, 9^{th}, 10^{th}, 11^{th} existing cracks in Fig. 2.

4. Discussion

d/h ratio for CVS performance level is determined as 0.001 for historical structures. If displacement of main dome peak point before the calibration is 0.108 m and height of structure is 22.4 m, the value will be 0.108/22.4=0.0048. If displacement of main dome point following the calibration is 0.250 m and height of structure is 22.4 m, the value will be 0.250/22.4=0.0112. It is seen that above values exceed limit value of performance level of CVS.

Considering the seismic analysis results of Hafsa Sultan Mosque, it was seen that σ_{11} compressive stresses in the crack zones numbered 1, 2, 3, 8, 9 and 10 before the calibration did not exceed masonry wall compressive strength however, this value exceeded after the calibration. σ_{22} and σ_{33} compressive stresses in all crack zones did not exceed masonry wall compressive strength before and after the calibration. σ_{11}, σ_{22} and σ_{33} tensile stresses in the structure exceeded masonry wall tensile strength in almost all crack zones. Especially, it is clearer in the calibrated model.

5. Conclusions

In this study, it is aimed to estimate more accurately damages occuring under seismic loads in the model calibrated with OMA method. Experimental (Nohutcu, Demir *et al*. 2015) and numerical analyses were carried out to determine the behaviour of historical Hafsa Sultan mosque in a possible severe earthquake in the region. Dynamic characteristics of the structure were calibrated with the OMA method and 1999 Kocaeli earthquake was applied to the structure by using linear time history analysis. Displacements and stresses at various points of the numerical model, noncalibrated and calibrated with the OMA, were obtained with ABAQUS program. Selected points are existing crack zones. It was seen that compressive stresses exceeded limit value in some zones in calibrated model. However, limit value was not exceeded in these points of noncalibrated model. Besides, it was similar for tensile stress. Displacements increased on average 2.56 times, compressive stresses increased 1.67 times and tensile stress increased 1.71 times in calibrated model. It shows that state of damage of the structure may be determined more accurately as a result of calibrating dynamic characteristics of the structure with OMA. The results show that calibrated structural model represents the existing structure more realistically. Most of the cracks in the structure took place due to tensile stresses. Increases of compressive stresses may result in compressive crush in critical points of the structure in the future. As a conclusion, it is estimated that the structure may collapse by expanding existing cracks in the event of a severe earthquake.

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