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ORIGINAL RESEARCH PAPER

Seismic behavior of two monumental buildings in historical Cappadocia region of Turkey

Baki Ozturk[1]

Abstract This study focuses on a displacement-based approach for the investigation of seismic behavior of two monumental buildings constructed in the historical Cappadocia region of Turkey. The first historic building is located in the town of Konakli, which is 27 km away from the city of Nigde and is situated in north-east of the city. It was built in 1844 and has served as a temple. The second building is located in the town of Fertek, 2 km from the city of Nigde. It has been used as a temple since it was built in 1835. These two buildings are seen as representative of many similar temple buildings constructed in the Cappadocia region during the same era. The seismic behavior of these two monumental buildings is investigated using dynamic analysis procedures. The buildings are subjected to ground motion records obtained during recent earthquakes in Turkey. These ground motions were recorded during the Ceyhan earthquake of 1998, which occurred close to Nigde; the Marmara earthquake of 1999 and the Duzce earthquake of 1999. Dynamic analyses are conducted for two cases with and without walls, as this helps in identification of the effect of structural walls on the seismic behaviors of these buildings. In the light of the dynamic analyses results, the expected level of damage and structural irregularities of the monumental buildings are examined. It is observed that slab discontinuities on the first floors constitute a major element in the expected structural damage for both buildings. Destructive levels of deformation are observed at Konakli in the roof domes of the building. In addition, upon application of certain ground motions, destructive levels of drift are observed, another element contributing to the expected damage.

Keywords Historical monumental building · Cappadocia region · Dynamic analysis · Structural damage · Structural irregularity · Dome

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1 Introduction

This study will help to evaluate the seismic behavior of two historic buildings under different earthquake excitation from a perspective of maximum drift and the resulting damage. Modal, response spectrum and nonlinear dynamic analysis procedures for investigation of seismic behavior of these two monumental buildings are applied here. The buildings were subjected to ground motion records obtained during recent earthquakes in Turkey. The analyses were conducted on the monumental building models with and without walls, helping to define the effect of structural walls on the seismic behavior of these buildings. It is known that the buildings may be exposed to several severe earthquakes during their life spans in which walls may be destroyed and therefore may not function structurally.

Augusti et al. (2001) showed how the probability of collapse and damage of each macro-element of a building and of the whole building, under given horizontal loads, can be assessed. They assumed appropriate probabilistic properties of the relevant quantities and produce "logical diagram" describing the relation between collapse of macro-elements and whole building. Mele et al. (2003) analyzed a basilica-type church in order to assess its structural behavior and seismic vulnerability. They used an effective two-step procedure consisting of (a) 3D static and dynamic linear analyses of the structural complex, and (b) 2D nonlinear push-over analysis of the single macro-elements. Oliveira (2003) provided a review on some of the methods available for assessing seismic vulnerability of historical constructions and how to use them in the case of an earthquake. Cardoso et al. (2005) described a method to evaluate the seismic performance of old masonry buildings, identifying the expected structural collapse mechanism of the structure. Mistler et al. (2006) presented a procedure for assessing the seismic safety of historic masonry buildings based on measurements of their natural frequencies and numerical simulations. Betti and Vignoli (2008a) analyzed a Romanesque masonry church in order to assess its structural behavior and seismic vulnerability with respect to the actual state of conservation. Betti and Vignoli (2008b) described a finite element methodology for static and dynamic nonlinear analysis of historical masonry structures and applied it to a case study. Casarin and Modena (2008) studied a complex building, the Santa Maria Assunta (Our Lady of the Assumption) Cathedral in Reggio Emilia (Italy), in order to evaluate its structural behaviour and defined its seismic vulnerability using different investigation and analysis methods. Mallardo et al. (2008) investigated the seismic behavior of an important Renaissance Palace in Ferrara (Italy), Palazzo Renata di Francia. In first part, a full 3D nonlinear analysis was performed in order to obtain a first insight into the structural performance. Then, three different 2D nonlinear analyses of the main façade were critically compared and discussed. Ozturk (2008) provided a brief explanation on the seismic behavior of the Fertek building in the historical Cappadocia region of Turkey. Ozturk et al. (2010) discussed the effect of retrofitting on the seismic behavior of the Konakli building at Nigde. Roca et al. (2010) presented a review of methods applicable to the study of masonry historical construction, encompassing both classical and advanced constructions. Betti and Vignoli (2011) analyzed the static behavior and the seismic vulnerability of the Basilica of Santa Maria all'Impruneta near Florence (Italy). The static structural behavior and the dynamic properties of the church were evaluated using finite element modelling, where the nonlinear behavior of masonry was taken into account by proper constitutive assumptions. Barbieri et al. (2013) investigated the structural analysis of a historic masonry building, Palazzo del Capitano in Mantua, subject to significant static instabilities due to an

overturning of the longitudinal façades, probably related to ground settlements. Asteris et al. (2014) presented a methodology for earthquake resistant design or assessment of masonry structural systems. The entire process is illustrated using case studies from historical masonry structures in Europe.

In this study, the seismic behavior of two monumental buildings constructed in the historical Cappadocia region of Turkey is investigated using a displacement-based approach. The first building has been used as a temple of worship since it was built in 1844. It is located in Konakli town, Nigde province. The second building, used as a temple of worship since it was built in 1835, is located in the town of Fertek close to the city of Nigde. These two buildings are taken as representative of many similar constructions built as temples in the Cappadocia region during the same era. Investigation into the seismic behaviors of these two buildings is conducted using analytical approaches for modal, response spectrum, nonlinear dynamic, and static analysis procedures. Ground motion records recorded during recent earthquakes in Turkey were used. The indicated ground motions were recorded during the Ceyhan earthquake of 1998, Marmara earthquake of 1999, and Duzce earthquake of 1999. The original models of these monumental buildings, both with and without walls, were analyzed to determine the effect of structural walls on the seismic response. Seismic drift response of building structures at seismically active and near-fault regions has been previously investigated (Ozturk 2003). In this study, the seismic behaviors of these two buildings are evaluated with regard to maximum drift response due to nonlinear dynamic analysis, and the corresponding levels of expected damages are assessed. The importance of considering slab discontinuities in these buildings is noted, in addition to destructive levels of deformation observed in the roof domes of the Konakli building.

2 Building descriptions

2.1 Konakli monumental building

The monumental building in Konakli was built as a church in 1844. It was converted into a mosque in the 1920s, and is still used as a temple. Konaklı, 27 km from city center, is located in north-east of Nigde province. It has two storeys, the entrance floor and the gallery floor. It must be considered that this temple is the second largest building in Anatolia and was originally built as a church along the lines of the Hagia Sophia church in Istanbul.

Originally, it was built as a basilica type church, with three semicircular apses forming the east walls of the building. There is a narthex with five segments at the west entrance (Parman 1988). Inside, the gallery is composed of columns connected to each other with rounded arches in the east, west, north and south. The main area is a "U" shape. Yellow or dark gray colored straight-cut stones from the Cappadocia region are used in all the external walls. The side and inside views are shown in Fig. 1.

Nigde andesite cut stone, easily available and commonly used in the Cappadocia region, was used for its construction. The stone has a density, γ of 22 kN/m^3, a modulus of elasticity of 20×10^6 kN/m^2, and a Poisson's ratio, ν of 0,2 (Korkanc 2013).

The top of the gallery level elevation is 10.20 m, while the entrance storey has a height of 7.10 m, the gallery storey has a height of 3.10 m. There are ten hemi-spherical domes located on the roof of the building. The maximum height is 13.70 m to the roof top. Total

Fig. 1 Side (**a**) and inside (**b**) views
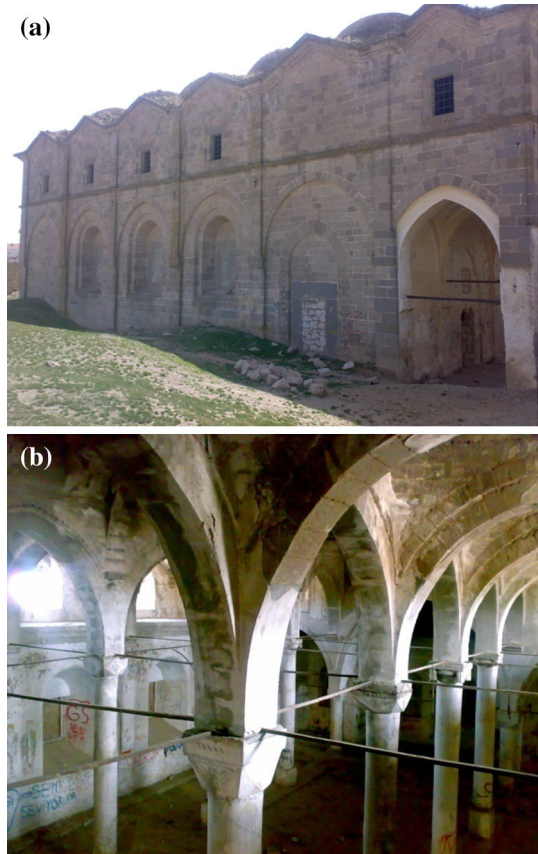
weight of the building is calculated to be around 20,300 kN. A three-dimensional model of the building is given in Fig. 2.

The diameters of inner and outer circular columns are 60 cm (Ø 60) and 70 cm (Ø 70), respectively; there are additional rectangular columns of 75 cm × 80 cm embedded in the outer walls. The building extends for 35.55 m in the east–west direction and 26.30 m north–south. The inner and outer walls have thicknesses of 80 and 90 cm, respectively. A plan of the entrance and gallery levels is shown in Fig. 3.

The building has seven spans in the east–west direction. Four of which are 5.20 m in length while the remaining three spans are 3.20, 2.82, and 2.10 m. There are five spans in the north–south direction: two with lengths of 5.28 m while the remaining three spans are 6.00, 2.82 and 2.10 m.

2.2 Fertek monumental building

The monumental building in Fertek was built as a church in 1835. It was converted into a mosque in 1924, and is still being used. It is 2 km. away from the city center of Nigde County. It is a two storey structure with an entrance floor and a gallery floor.

It was built in the form of a basilica type church. There are three semicircular apses at its east end. There is a "U" shaped narthex at the west entrance and a circular curve outside
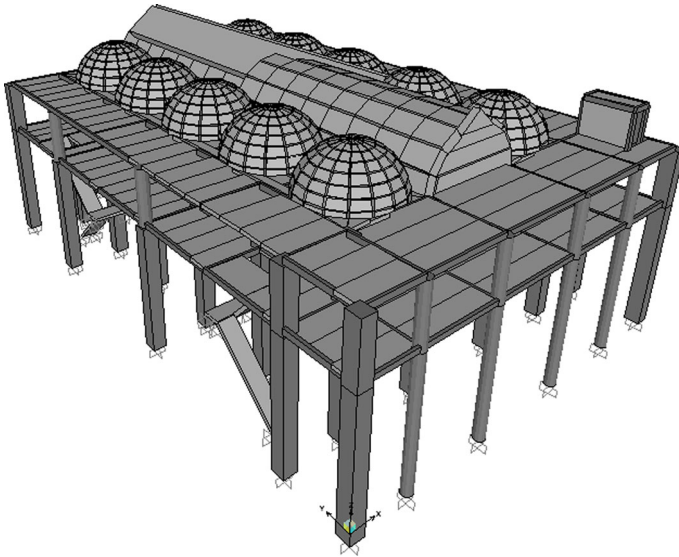
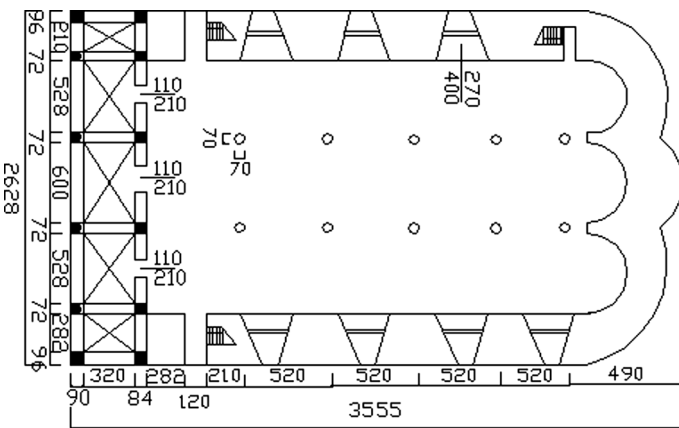**Fig. 2** Three-dimensional model without walls

Fig. 3 Building plan with all dimensions in cm

(Parman 1988). Inside, the gallery is composed of columns connected to each other by rounded arches in the east, west, north and south; the main area is a "U" shape.

In all external walls, yellow or dark gray colored straight-cut stones from the Cappadocia region are used. Timber is used internally at the top of the columns and in the entrance hall at the top of the narthex. The front and inside views are given in Fig. 4.

Construction material is Nigde's yellow tuff cut stone, commonly used in the Cappadocia region. The stone has a density, γ of 15 kN/m^3, modulus of elasticity of 18×106 kN/m^2 and a Poisson's ratio, ν of 0,3 (Korkanc 2013).

The roof top elevation is 11.60 m. The top of the gallery elevation is 8.00 m, while the entrance storey has a height of 4.50 m, the gallery storey has a height of 3.50 m. Total

Fig. 4 Front (**a**) and inside (**b**) view

weight of the building is calculated to be around 9070 kN. A three-dimensional model of the building is given in Fig. 5.

The diameter of inner columns is 60 cm (Ø 60) while the outer columns are 50 cm (Ø 50). The building extends for 34.55 m east–west and 23.90 m north–south. The inner walls are 60 cm thick and the outer walls are 85 cm. The plan of the entrance and gallery levels is given in Fig. 6.

The building has seven spans in the east–west direction, four of which are 4.40 m in length while the remaining three are 5.20, 4.70 and 4.10 m. In the north–south direction there are five spans: two with a length of 4.00 m, two with 4.35 m, and one at 6.20 m.

3 Analyses of the monumental buildings

The finite element method (FEM) presents many various possibilities regarding the description of the masonry structures within the frame of detailed non-linear analysis. An application of Mark et al. (1993a, b) can be regarded as a pioneer example for analysis of Hagia Sophia allowing different portions of the structure to weaken at different levels of tension. Macro-modelling and micro-modelling can be regarded as modern possibilities based on FEM fall within two main approaches. Owing to its lesser calculation demands,
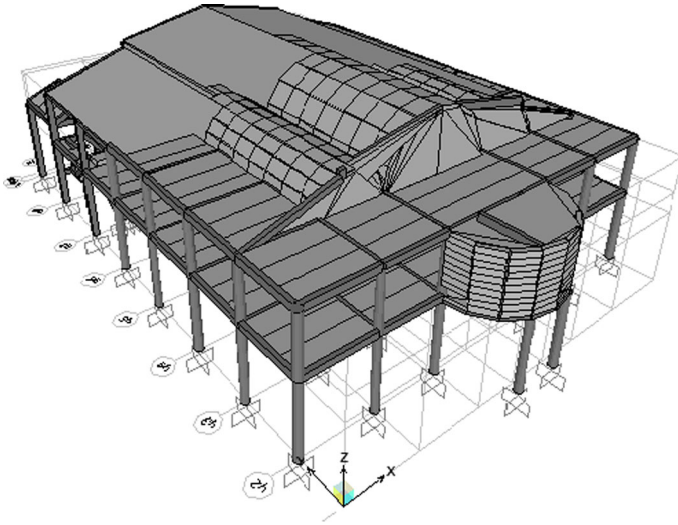
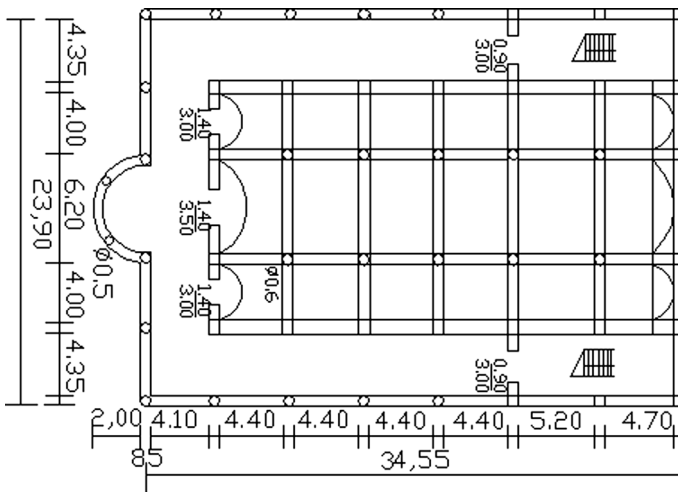**Fig. 5** Three-dimensional model without walls

Fig. 6 Building plan (all dimensions in m)

macro-modelling is probably the most popular and common approach. A detailed description of the interaction between units and mortar may not be seen as necessary in practice-oriented analyses on large structural members or full structures. In these cases, macro-modelling, which ensures balance between units and joints, may suggest a proper approach for the characterization of the structural response. The macro modelling strategy regards the material as a fictitious homogeneous orthotropic continuum.

A proper relationship is built between average masonry strains and average masonry stresses. A complete macro-model must explain various tensile and compressive strengths along the material axes as well as various inelastic properties along each material axis. The continuum parameters must be determined using tests conducted on sufficiently large size

examples which are subjected to homogeneous states of stress. Macro-modelling presents significant practical advantages as it is compared to more detailed approaches covering the description of discontinuities. Due to the fact that finite element meshes do not have to accurately describe the internal structure of masonry, they are particularly simpler. When a compromise is required between accuracy and efficiency, that kind of a modelling is considerably substantial.

A relation can be established between the macro-models, which is also described as *Continuum Mechanics* finite element models, and plasticity or damage constitutive laws. The work of Lourenço (1996) and Lourenço et al. (1998) suggesting a non-linear constitutive model for in-plane loaded walls based on the plasticity theory, stands for an example of the former approach.

Due to their mathematical simplicity and the need for only few material parameters, isotropic criteria have been mostly favored in the case of continuum damage of finite element models. Nevertheless, a number of orthotropic models have been also introduced (Papa 1996; Berto et al. 2002).

The macro-models have been widely adopted in an attempt to analyze the seismic response of complex masonry structures, such as historical buildings (Mallardo et al. 2008), and mosques and cathedrals (Roca et al. 2004; Martínez et al. 2006; Murcia-Delso et al. 2009).

A disadvantage of the macro-modelling approach arises from its description of damage as a smeared property spreading over a large volume of the structure. In real unreinforced masonry structures, damage appears normally localized in isolated large cracks or similar concentrated lesions. A smeared modelling of damage does not give a quite realistic description of damage and may result in either inaccurate or complex predictions in order to associate with real observations. While macro-modeling procedure may be favored in the analysis of large masonry structures, it is not applicable for the detailed stress analysis of a small panel, due to the fact that capturing all the possible failure mechanisms is not easy and the influence of the mortar joints which act as planes of weakness cannot be addressed.

Clemente et al. (2006) recently put forward a considerable improvement to smeared damage representation. In order to combine non-linear constitutive equations with solid 3D element meshes, quite a few attempts have been formulated. Oñate et al. (1995) and Hanganu (1997) suggested a 3D continuum damage model regarding masonry and concrete under physical and environmental conditions and applied it to the study of the Domes of the San Marco's Basilica located in Venice. The model was subsequently put into use to study Barcelona Cathedral (Roca et al. 1998).

The software SAP2000 Nonlinear (SAP2000 2000) is the program simulating the structure at structural modeling stage. A proper FEM model in an attempt to calculate the response of the structure was formed with the aim of this software. The improvement of the finite elements mesh was observed from that the optimal concentration of masses at the nodes simulates the real mass distribution very well (Fig. 7). This provides a reliable simulation of the inertial loads regarding dynamic analysis. Six degrees of freedom for every one of the nodes were considered to completely determine the deformation of the system. The six degrees of freedom equals to three translations, along the axes x, y, z and three rotations of vectors, parallel to the same axes. Shell internal forces regarding (a) membrane forces and (b) plate bending and twisting moments are given in Fig. 8.

The geometrical simulation was carried out with the help of isotropic surface members (shell elements) and isotropic linear members (frame elements), which are considered to represent with sufficient reliability the properties of the real body. The model which is used
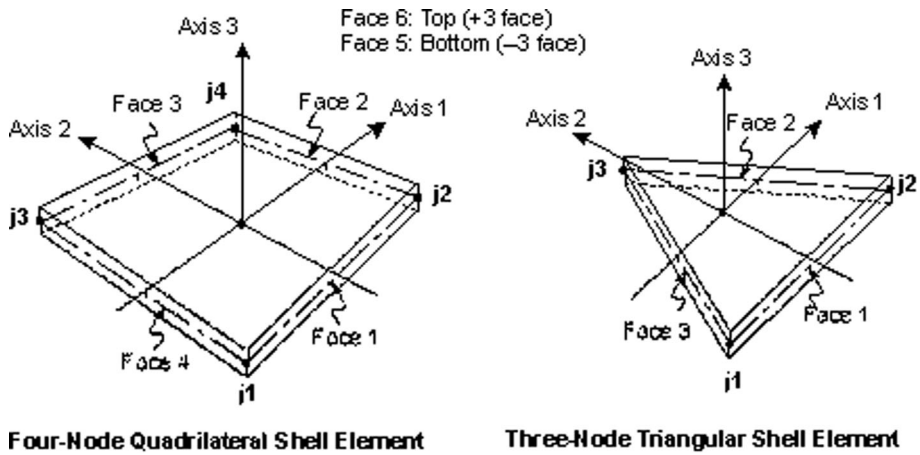
Fig. 7 General quadrilateral or triangular shell elements with orthotropic material properties (SAP2000 2000)
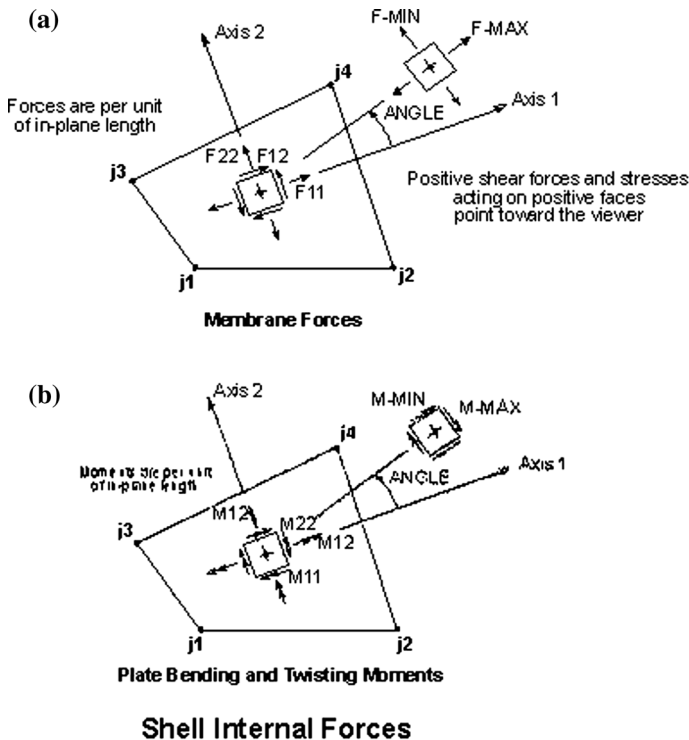

Fig. 8 Shell internal forces considering **a** membrane forces and **b** plate bending and twisting moments (SAP2000 2000)

with the aim of analyzing the building is spatial. Flat quadrilateral and triangular elements were active in discretization of the finite element mesh. The condensation of the data was adopted depending on the geometry and loading conditions prevailing at each region of the

model. By that way, the anisotropic behavior of the masonry structure was better simulated.

3.1 Konakli monumental building

3156 nodes were defined in the structural system of the building with walls and 2293 nodes defined for the building without walls. The periods of the first three modes of the original building were obtained by modal analyses (SAP2000 2000). These periods are provided for both the building with walls and without walls (Tables 1). In the analyses, a damping ratio of 5% was considered.

The building is located on a soil type Z3 with $T_A = 0.15$ s and $T_B = 0.6$ s. The corresponding spectrum function is given in Fig. 9. The effective ground acceleration constant, A_O was 0.1, while the building importance constant, I value was 1.0. The ductility factor, R was taken as 2.0 referring to requirements in the Turkish Earthquake Code (2007). The spectrum constant, K was calculated to be 0.98 m/s^2 using Eq. (3.1). The parameters and calculations given in this paragraph for Konakli monumental building also applies for Fertek monumental building defined in Sect. 3.2 below.

$$K = \frac{A_O \cdot I \cdot g}{R} \tag{3.1}$$

The spectrum constant, S(T), evaluated according to Eq. (3.2), was calculated to be 4.13 for the building with walls and 3.09 for the building without walls.

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \tag{3.2}$$

The analyses were conducted both in X and Y directions for the ground motion records provided in Table 2. The building, soil, and seismic regional properties of the structure explained above were used. The ground motion records used were: Ceyhan EW and Ceyhan NS records (Adana Ceyhan Earthquake of 1998), Izmit EW and Izmit NS records (Marmara Earthquake of 1999), and Duzce EW-NS and Bolu EW-NS records (Duzce Earthquake of 1999). The maximum ground acceleration values (PGA) for the given earthquake data set are provided in Table 2.

3.2 Fertek monumental building

For the modal and response spectrum analysis of the structure, SAP2000 Program (2000) was used. The three structural periods obtained as a result of modal analyses for both the building with and without walls are provided in Table 3. For the building with walls the first three periods were 0.31, 0.28 and 0.27 s, while for the building without walls the first

	Modes	Periods of building with walls (s)	Periods of building without walls (s)
Table 1 Periods of first three modes of the original building (Ozturk et al. 2010)	1st	0.32	0.46
	2nd	0.26	0.45
	3rd	0.13	0.34

Fig. 9 Spectrum function
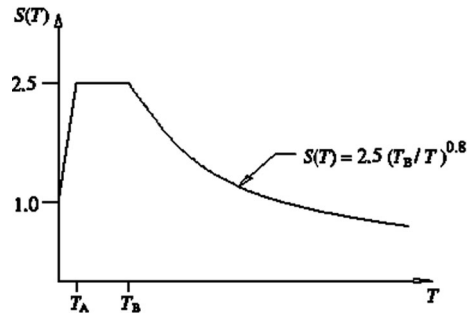(Turkish Earthquake Code 2007)

$$S(T) = 2.5\, (T_B/T)^{0.8}$$

Table 2 Ground motion records and maximum ground acceleration (PGA) values

Ground motion record	Maximum ground acceleration (PGA) (g)
Ceyhan EW (Ceyhan 1998)	0.23
Ceyhan NS (Ceyhan 1998)	0.28
Izmit EW (Marmara 1999)	0.23
Izmit NS (Marmara 1999)	0.17
Duzce EW (Duzce 1999)	0.51
Duzce NS (Duzce 1999)	0.41
Bolu EW (Duzce 1999)	0.82
Bolu NS (Duzce 1999)	0.75

Table 3 Periods of first three modes of the building (Ozturk 2008)

Modes	Periods of building with walls (s)	Periods of building without walls (s)
1st mode	0.31	0.63
2nd mode	0.28	0.46
3rd mode	0.27	0.44

three modes were 0.63, 0.46 and 0.44 s. There were 1881 nodes defined in the structural system of the building with walls and 814 nodes defined for the building without walls. In the analyses, a damping ratio of 5% was considered.

Using Eq. (3.2), the spectrum constant, $S(T)$ was calculated to be 4.24 for the building with walls and 2.40 for the building without walls.

4 Nonlinear analyses of the monumental buildings

SAP2000 Program (2000) enables to model material's nonlinear behaviour through either link/support elements or plastic hinges. Plastic hinges are used in nonlinear analyses; for the advantage of using link elements depends on the possibility of modelling hysteretic behaviour under cyclic loadings. Plastic hinges enable to model the yielding and post-yielding behaviour of frame elements by defining independent parameters for momentum, shear and axial loading. A rigid-perfectly plastic behaviour is considered for plastic hinges (Fig. 10).

The seismic characterization regarding the nonlinear behavior of the structure must be accomplished by carrying out a time-history analysis. In SAP2000 Program, the solution of the movement equation system is done by modal superposition using Fast Nonlinear Analysis (FNA), which is an overly quick and efficient method considering structures with a limited number of non linear elements (SAP2000 2000). Non linearity is confined to link/support elements in FNA. Every link consists of six springs, one for each degree of freedom, where various stress-deformation curves may be defined. The multi-linear plastic pivot model is favored because of its allowing hysteretic behavior among all the available links. The loading degradation is characterized by a set of parameters which stem from experimental observation that loading and unloading from any displacement level tends to particular points and that every stress-deformation curve tends to cross the elastic loading curve in a particular point as shown in Fig. 11 (Dowell et al. 1998).

In order to define the hysteretic behavior of the link, SAP2000 Program (2000) uses a base loading curve and the parameters α and β. By simulating, in SAP2000 (2000), an experimental proceeding of a wall submitted to a cyclic horizontal loading, stress-deformation curves which were identical to the ones from the actual proceeding (Magenes et al. 2000) come up as the work of Pasticier et al. (2008). This was provided by changing the loading and unloading parameters until both curves were identical. It was found as $\alpha_1 = \alpha_2 = \beta_1 = \beta_2 = 0.45$ (Fig. 12) in that wall and with that masonry.

The best method to define the link parameters is experimental simulation. Accordingly, the parameters which were put through by Pasticier et al. (2008)were adopted. The links are set as shear plastic hinges. The yielding and post-yielding parameters are also the same. The only difference occurs in the post-yielding behaviour, in which it is regarded that a 30% decrease in shear resistance realizes for each 6 mm of horizontal displacement (Fig. 13).

If one conclusion is drawn from these non linear dynamic analyses, it will be the significant role of the hysteretic behaviour on a building's resistant capacity. This reveals the significance of considering hysteretic behaviour on a dynamic non linear analysis. It is shown that SAP2000 (2000) enables us to estimate the dynamic non linear response of an ancient masonry building (Pasticier et al. 2008).

4.1 Konakli monumental building

The nonlinear analyses were carried out both in X and Y directions for the ground motion records provided in Table 2. The building, soil and seismic region properties of the structure explained above were used for the nonlinear analyses. The ground motion records were the same as those above. The given earthquake data maximum ground acceleration values (PGA) are in Table 2.

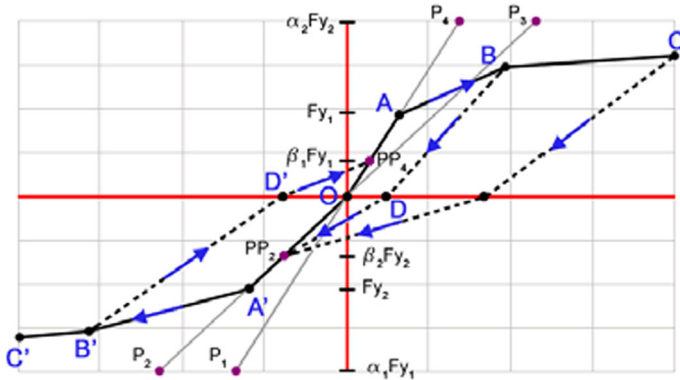Fig. 10 Behavior adopted for hinges: rigid-perfectly plastic (Pasticier et al. 2008)
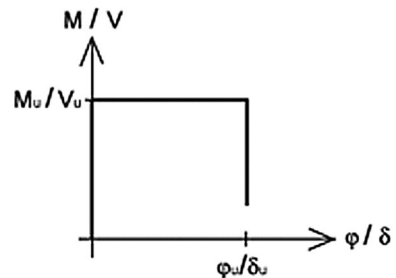
Fig. 11 Multi-linear plastic pivot model (SAP2000 2000)

Fig. 12 Definition of the *link* parameters (Pasticier et al. 2008)
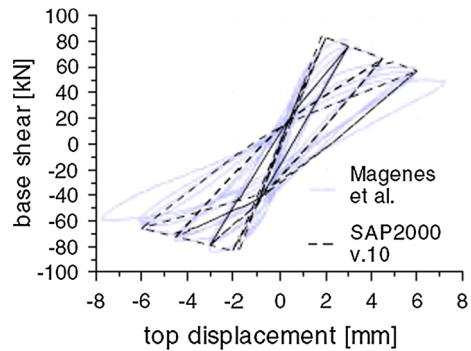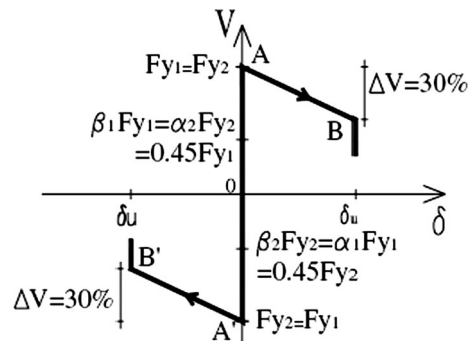

Fig. 13 Shear-displacement curve (Pasticier et al. 2008)

The results of dynamic analyses revealed that Bolu-EW and Bolu-NS were the most demanding ground motions in means of maximum displacement response of the building. The deformed three-dimensional model with walls under Bolu-NS in the X-direction (Fig. 14) and the deformed three-dimensional model without walls under Bolu-NS in X-direction (Fig. 15) are given below. In addition, it is observed that slab discontinuities on the gallery floor constitute a major element in the structural damage expected.

The nonlinear displacement–time histories for the given ground motions are provided in Fig. 16 while the maximum displacement response values at the gallery top elevation for

all six ground motions applied are in Table 4. Maximum displacement response values were observed at nodes located at the gallery top. The expected destructive levels of deformation under earthquake excitation which can be observed in the roof domes of the Konakli building are shown below in Figs. 14 and 15.

Nonlinear displacement–time histories of joints show that the maximum drift in both X and Y directions is infinitesimal in the building with walls.

Gallery storey top displacements in the Y direction were infinitesimal while those in the X direction are given below in Fig. 16. The nonlinear displacement–time histories for Izmit EW and NS ground motions are not given below, since the corresponding displacement values are quite small.

4.1.1 Nonlinear displacement–time histories of gallery storey top elevation joints with maximum displacement in building without walls

See Figure 16 and Table 4.

4.2 Fertek monumental building

The nonlinear analyses were carried out both in X and Y directions for the ground motion records provided in Table 2. The building, soil, and seismic regional properties of the structure explained above were used for the nonlinear analyses. The ground motion records used were as above. The given earthquake data maximum ground acceleration values (PGA) are in Table 2.

The results of dynamic analyses revealed that Bolu-EW and Bolu-NS were the most demanding ground motions in means of maximum displacement response of the building. The deformed three-dimensional model with walls under Bolu-NS in the X-direction
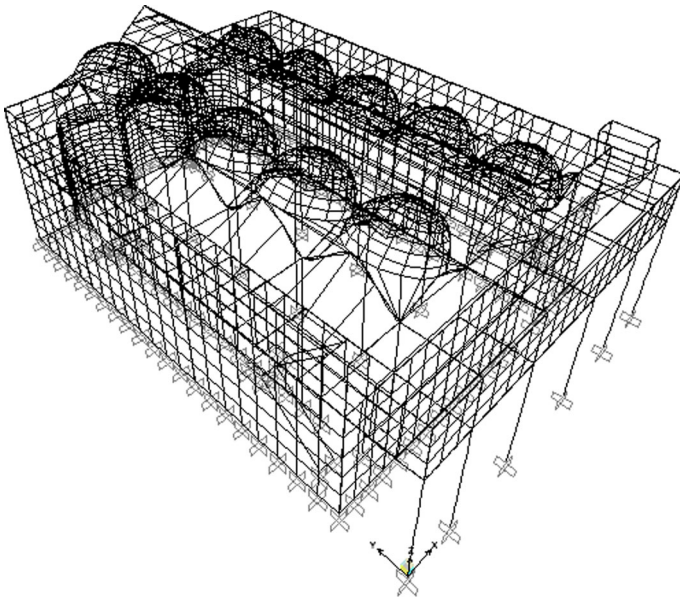

Fig. 14 Three-dimensional model with walls under Bolu-NS in X-direction (extreme deformation)
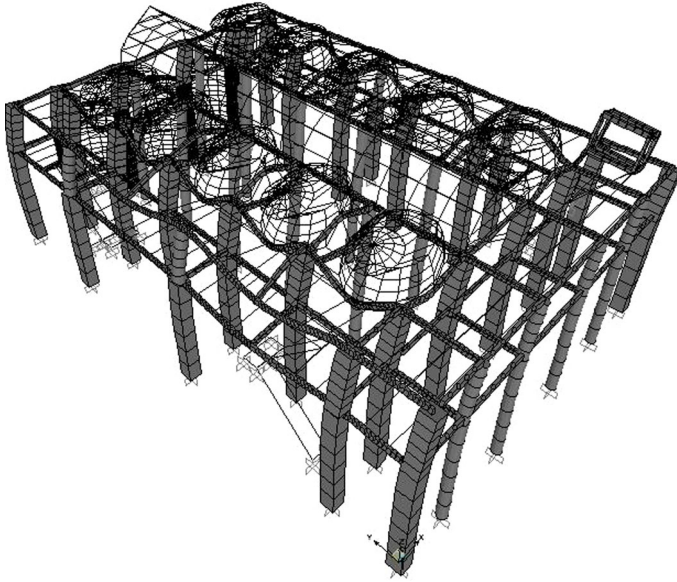
Fig. 15 Three-dimensional model without walls under Bolu-NS in X-direction (extreme deformation)

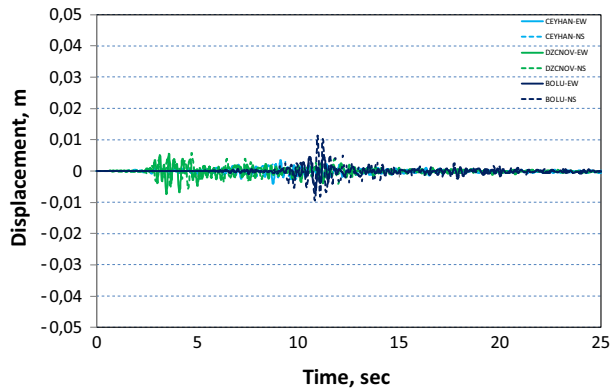**Fig. 16** Gallery storey top displacements in X direction

Table 4 Maximum displacements at gallery storey top for building without walls

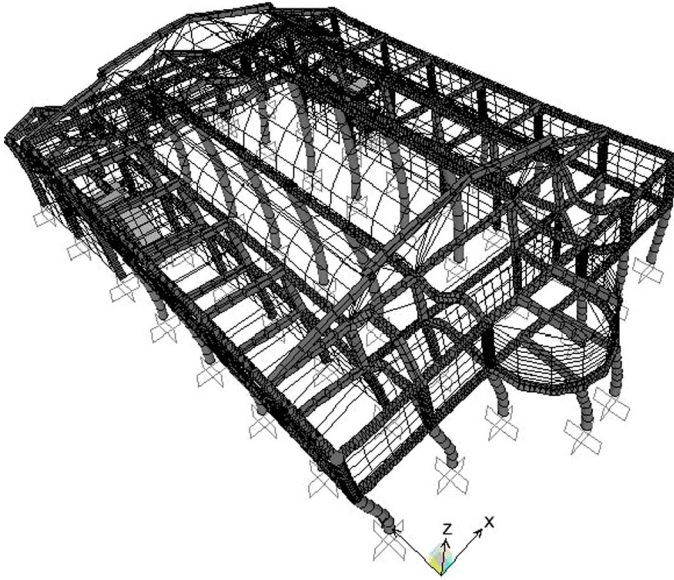| Ground motion	X direction (mm)	Mean drift in X direction (%)
Ceyhan EW (Ceyhan1998)	4	0.04
Ceyhan NS (Ceyhan1998)	3	0.03
Duzce EW (Duzce 1999)	7	0.07
Duzce NS (Duzce 1999)	6	0.06
Bolu EW (Duzce 1999)	6	0.06
Bolu NS (Duzce 1999)	11	0.11

Gallery storey top elevation is 10.20 m

Fig. 17 Three-dimensional model with walls under Bolu-NS in X-direction (extreme deformation)
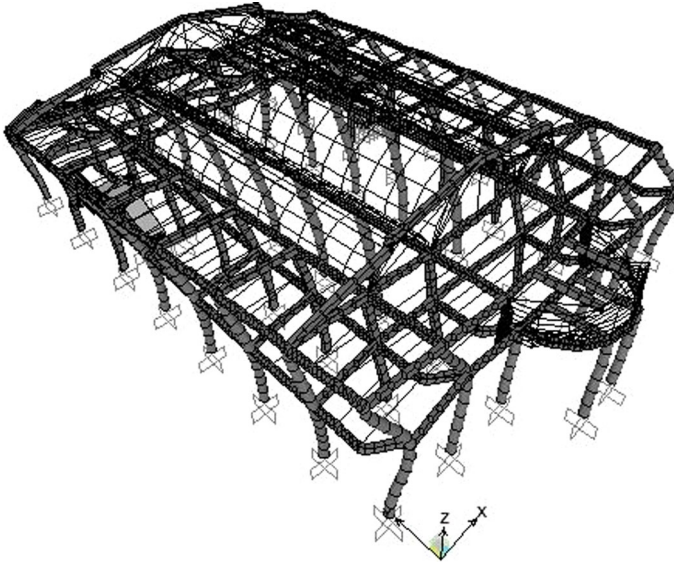

Fig. 18 Three-dimensional model without walls under Bolu-NS in X-direction (extreme deformation)

(Fig. 17) and that without walls (Fig. 18) are given below. In addition, it is observed that slab discontinuities on the gallery floor constitute a major element in the structural damage expected.

   The nonlinear displacement–time histories for the given ground motions are provided in Fig. 19 while the maximum displacement response values at the gallery top elevation for

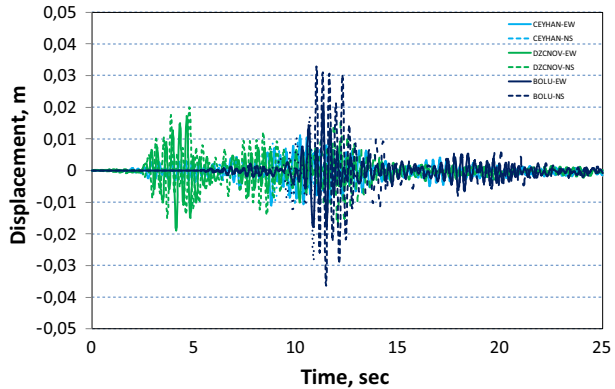**Fig. 19** Gallery storey top displacements in X direction

Table 5 Maximum Displacements at Gallery Storey Top for Building with Walls

Ground Motion	X direction (mm)	Mean drift in X direction (%)
Ceyhan EW (Ceyhan1998)	11	0.14
Ceyhan NS (Ceyhan1998)	10	0.13
Duzce EW (Duzce 1999)	19	0.24
Duzce NS (Duzce 1999)	20	0.25
Bolu EW (Duzce 1999)	20	0.25
Bolu NS (Duzce 1999)	35	0.44

Gallery Storey Top Elevation is 8.00 m

all of the ground motions applied are in Table 5 for the three dimensional model with walls. The nonlinear displacement–time histories for the given ground motions are provided in Figs. 20 and 21, while the maximum displacement response values at gallery top elevation for all of the ground motions applied are in Table 6 for the three dimensional model without walls. Maximum displacement response values are observed at nodes located at the gallery storey top elevation.

Gallery storey top displacements in Y direction were infinitesimal while those in the X direction are given below in Fig. 19.

The nonlinear displacement–time histories for Izmit EW and NS ground motions are not given below since the corresponding displacement values are quite small.

4.2.1 Nonlinear displacement–time histories of gallery storey top elevation joints with maximum displacement in building with walls

(See Fig. 19; Table 5).

4.2.2 Nonlinear displacement–time histories of gallery storey top elevation joints with maximum displacement in building without walls

(See Figs. 20, 21; Table 6).

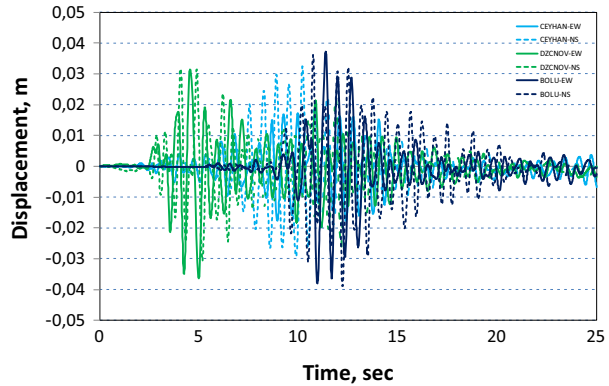**Fig. 20** Gallery storey top displacements in X direction

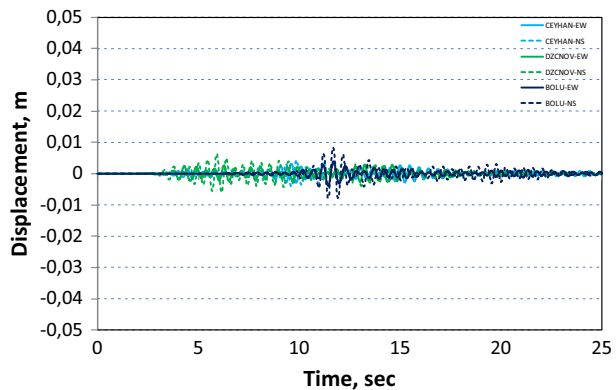**Fig. 21** Gallery storey top displacements in Y direction

Table 6 Maximum displacements at gallery storey top for building without walls

Ground motion	X direction (mm)	Mean drift in X direction (%)	Y direction (mm)	Mean drift in Y direction (%)
Ceyhan EW (Ceyhan1998)	18	0.23	3	0.04
Ceyhan NS (Ceyhan1998)	32	0.40	4	0.05
Duzce EW (Duzce 1999)	37	0.46	3	0.04
Duzce NS (Duzce 1999)	32	0.40	6	0.08
Bolu EW (Duzce 1999)	38	0.48	4	0.05
Bolu NS (Duzce 1999)	40	0.50	9	0.11

Gallery storey top elevation is 8.00 m

5 Discussion and comparison of the results

In this manuscript, periods of first three modes of the original Konakli monumental building (Ozturk et al. 2010) are provided in Table 1 in addition to ground motion records and maximum ground acceleration (PGA) values (Table 2). Meanwhile, periods of first three modes of the original Fertek monumental building are provided in Table 3 (Ozturk 2008).

Nonlinear analyses of the investigated monumental buildings are explained in Sect. 4. Considering Konakli monumental building, three-dimensional model with walls under Bolu-NS in X-direction (Extreme deformation) is given in Fig. 14 while three-dimensional model without walls under Bolu-NS in X-direction (Extreme deformation) is provided in Fig. 15. Gallery storey top displacements in the Y direction are infinitesimal while those in the X direction are given in Fig. 16 in the form of time-history analyses. Maximum mean drift values, which are in the range of 0.03–0.11% at gallery storey top for building without walls are given in Table 4. It is observed that maximum mean drift occurs under Bolu-NS ground motion while minimum mean drift occurs under Ceyhan-NS ground motion.

Considering Fertek monumental building, three-dimensional model with walls under Bolu-NS in X-direction (Extreme deformation) is given in Fig. 17 while three-dimensional model without walls under Bolu-NS in X-direction (Extreme deformation) is provided in Fig. 18. Gallery storey top displacements in X direction for building with walls are given in Fig. 19 in the form of time-history analyses. Maximum mean drift values, which are in the range of 0.13–0.44% at gallery storey top for building with walls, are given in Table 5. It is observed that maximum mean drift occurs under Bolu-NS ground motion while minimum mean drift occurs under Ceyhan-NS ground motion. Gallery storey top displacements in both X and Y directions for building without walls are given in Figs. 20 and 21, respectively in the form of time-history analyses. Maximum mean drift values in X-direction are in the range of 0.23–0.50% while in Y-direction the range is in between 0.04 and 0.11% at gallery storey top for building without walls (Table 6). It is observed that maximum mean drift occurs under Bolu-NS ground motion while minimum mean drift occurs under Ceyhan-EW ground motion.

6 Conclusions

This study focuses on a displacement-based approach for analysis of the seismic behavior of two monumental buildings located in historical Cappadocia region of Turkey. First one of these is located in the town of Konakli while the second is located in Fertek. These two buildings are representatives of many similar buildings constructed as temples during the same era in this region. It must be considered that the temple in Konakli is the second largest building in Anatolia and was originally built as a church in the style of the Hagia Sophia church, located in Istanbul. Modal, response spectrum, and dynamic analyses for different ground motions were applied to these building models. The ground motions used in the analyses are shown in Table 2.

As nonlinear dynamic analyses were applied, the expected levels of damage and the structural irregularities of the buildings were examined. The ground motion records which were used, were Ceyhan EW and Ceyhan NS, recorded during the 1998 Adana Ceyhan earthquake; Izmit EW and Izmit NS, recorded during the 1999 Marmara earthquake; and Duzce EW-NS and Bolu EW–NS, recorded during the 1999 Duzce earthquake. These two buildings were analyzed with and without walls to account for the fact that they may be exposed to several earthquakes during their life spans; and their walls may be destroyed andmay not function structurally.

Considering the ground motion set which was applied, the Konakli building model without walls may be exposed to a mean drift value up to 0.11% (Table 4), while for the building with walls the mean drift values are quite small. It must be considered that destructive deformationwas observed in the domes located in the roof of the Konakli

building. The Fertek building model with walls may be exposed to a mean drift value up to 0.44% (Table 5) while without walls the mean drift reached 0.5% (Table 6), in the range of comparable damage to the structure. It has to be noted that excessive levels of drift mainly occur upon application of certain demanding ground motions (Bolu-EW and Bolu-NS). In addition, it is observed that slab discontinuities on the gallery floor constitute a major element in the structural damage expected for the Fertek building.

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