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Seismic behavior of the cube of Zoroaster tower using the discrete element method

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There are several ancient stone masonry structures of great archeological significance in earthquake prone areas around the world. Ka'ba-ye Zartošt (Cube of Zoroaster) is a 14.2 m square in shape tower, which was built using white limestone blocks and dry joints. The tower dates back to the Achaemenid empire era and is located in the earthquake prone area of Nagsh-e Rustam in Fars, Iran. Although, after approximately 2,500 years the tower is still standing, it is now in a severely deteriorated condition and may be vulnerable against future large in magnitude earthquakes. This paper presents the application of a previously developed three dimensional numerical model based on the discrete element method of analysis to investigate the seismic behavior of the Cube of Zoroaster tower. The tower was represented by a series of distinct blocks separated by zero thickness interfaces. The developed model allows finite displacements and rotations of distinct blocks while new contacts between the blocks are automatically recognized and updated as the calculation progresses. A series of non-liner dynamic analysis have been performed. To this end, the behavior of the tower to different ground shaking motions is discussed and the possible failure modes for each case are explored. The significant advantage of this numerical approach is that discontinuity among stone blocks can be obtained. In addition, reliable prediction of the dynamic behavior of such historic constructions can allow a better understanding of their seismic vulnerability and inform decisions for their maintenance and preservations.

Keywords: stone masonry; ancient monument; Discrete Element Method (DEM); seismic behavior; Ka'ba-ye Zartošt

Introduction

There are many ancient monuments (e.g. towers, multi-drum columns and colonnades) of great architectural and archeological significance around the world. Most of these have experienced major earthquakes during their life. Although some of them are still standing, there are examples where collapses occurred in the past (e.g. the 2,500 years old citadel of the city of Bam, the frescoed vault of the Basilica of St. Francis of Assisi in Perugia, Italy etc.). For engineers, it is important to understand the behavior of historic masonry structures when subjected to strong ground excitations in order to propose options for repair and strengthening for their preservation. There are several historic masonry structures which have been constructed with dry joints (i.e. no mortar between the masonry units) around the world and are composed of rigid blocks stack on top of each other and have been subjected to strong earthquake excitations. According to Psycharis et al. (2011) and Bui et al. (2017), during strong earthquakes, the behavior of masonry constructions with dry joints can be characterized by high non-linearity which is governed by sliding, rocking, wobbling and/or complete detachment of adjacent masonry units. Thus, failure is usually at the block-to-block joint interface rather than in the blocks itself (e.g. cracking or crushing of the masonry units). In addition, in most cases, it is impossible to undertake destructive tests on such constructions since most of them are historical and there are restriction to access and testing on them (Omorfi 1900). Therefore, efforts to develop full-scale experimental tests of scaled historical buildings and monuments or their components have been undertaken over the last years to understand their behaviour when subjected to seismic loading. For example, a series of seismic shaking investigations in historic buildings and monuments is presented at the Krstevska et al. (2010). Tests carried out at the IZIIS' Dynamic Testing Laboratory and the aim was to assess the vulnerability of the structures as well as test the experimental verification of different methodologies for seismic strengthening. Experimental tests to assess the seismic performance of ancient columns were also performed by Drosos and Anastasopoulos (2015) to understand the factors affecting their response. Physical models of multi-drum columns were constructed at reduced scale and tested at the shaking table of the NTUA Laboratory of Soil Mechanics. The difficulties associated with performed such experimental testing was highlighted in their manuscript. Moreover, a state of the art review on the testing of historic masonry elements and buildings is presented by Vintzileou (2010). In particular experimental studies about the behaviour of historic masonry elements in compression, in diagonal compression, in in-plane shear and simultaneous compression, out-of-plane bending, as well as publications related to the behaviour of subassemblies and building models subjected to monotonic, pseudo-dynamic or dynamic tests on earthquake simulator were presented. However, although experimental studies are important to understand how physical models and material will behave when subjected to different types of loading, from the above experimental studies it was evident that experimental studies to represent historical masonry construction are both very expensive and time consuming. Therefore, it is fundamentally important to have available computational tools to predict the in-service and near-collapse behavior of such complex structures with sufficient reliability. Once such a tool has been established, a range of complex problems and scenarios can be investigated in detail.

Today, there are several approaches to model the mechanical behavior of masonry structures in the literature. An extensive literature review of the different numerical modeling approaches to simulate the mechanical behavior of masonry can be found at Asteris et al. (2015). These approaches include: limit analysis method, finite-element method (FEM), discrete/distinct element method (DEM) and finite-discrete element method (FDEM). Among these approaches, the DEM and FDEM approaches are the best ones to simulate the static and dynamic behavior of jointed discrete blocks media like masonry structures.

Research undertaken in the past (Bui et al. 2017; Azevedo et al. 2000; Psycharis et al. 2003) demonstrated that the discrete/distinct element method (DEM) can be effectively used to simulate the in-plane and out-of-plane nonlinear behavior of masonry structures constructed with dry joints. Discrete/distinct element method developed by Cundall (1971) to investigate the behavior of jointed rocks where continuity between the separate blocks of rock did not exist. Over the last two decades, DEM has been employed with success to study the mechanical behaviour of historic masonry structures and monuments subjected to static and dynamic loads (Azevedo et al. 2000, Psycharis et al. 2000, 2003, 2013, Papantonopoulos et al. 2002, Komodromos et al. 2008, Papaloizou and Komodromos 2009, 2012, DeJong et al. 2012, Sarhosis et al. 2015, Sarhosis et al. 2016 a,b, Çakti et al. 2016, Pulatsu et al. 2017). For example, Psycharis et al. (2000), Papantolopoulos et al. (2002, Psycharis et al. (2013) successfully investigated the seismic behavior of freestanding multi-drum columns. It was found that multi-drum columns are more vulnerable to long-period ground motions than short-period ones. DeJong and Vibert (2012) made use of the three-dimensional software 3DEC based on the DEM to investigate the seismic response of a damaged stone masonry spire in UK. Nayeri (2012) performed a series of parametric DEM analyses under far field and near field earthquakes on the stone Roman Temple in Évora. Later, Çakti et al. (2016) used DEM to simulate the seismic behavior of a scaled stone masonry mosque with dry joints tested in a shaking table. From the results analysis, it was found that the numerical results were in a good agreement with those obtained from the experiment. They also observed that for lower levels of the input motion, the frequency characteristics of the DEM model correlate fairly well with the tested model response characteristics.

Finite-discrete element method is an approach that combines the advantages of the FEM and FEM approaches (Smoljanović, 2013). In this approach, deformable blocks are represented using a mesh of triangular elements between which contact elements can be inserted. Therefore, the discretized elements may split and separate during the analysis. Furthermore, similar to the FEM, nonlinear material models are employed in this approach to simulate the masonry units and mortar failure. Munjiza (2004) developed a method based on fracture mechanics criteria to simulate fracturing problems called the combined finite-discrete element method. Due to the possibility of modelling the fragmentation of each block in the combined FDEM, Smoljanović et al. (2013) recently adopted and developed the FDEM to simulate the monotonic and cyclic shear behaviour of in-plane dry stone masonry walls and monuments. They showed that the FDEM has the ability to simulate the main features of dry stone masonry assemblages such as joints' shear behavior and rocking motion. Smoljanović et al. (2015) using the FDEM proposed a new material model in the contact element between masonry units and mortar. This contact element takes into consideration the possibility of failure and softening behaviour of masonry in tension and shear. Smoljanović et al. (2018) also presented the application of the combined FDEM to simulate the behaviour of 3D dry stone masonry walls and structures.

The purpose of this paper is to investigate the 3D nonlinear seismic behavior of the dry joint stone masonry tower of Ka'ba-ye Zartošt (Cube of Zoroaster) in Nagsh-e Rustam in Fars, Iran. To this end, a previously developed three-dimensional discrete element model was utilized. Use made of the commercial software 3DEC (Itasca, 2013). The intention was to investigate the structural behavior and failure mode of the Cube of Zoroaster tower when subjected to earthquake vibrations with different frequency content. The observations presented herein provide insights into the non-linear dynamic response of masonry structures constructed with dry joints and lead to suggestions for informed decisions for their maintenance and preservation.

Description of the monument

Ka'ba-ye Zartošt (Cube of Zoroaster), as shown in Figure 1, is a square tower built of white limestone blocks placed on top of each other. The tower was built during the pre-Persepolitan phase of the Achaemenid monumental architecture (ca. 559-511 B.C.) (Schmidt 1970) in Nagsh-e Rustam, Fars, Iran. The tower has been constructed in two levels. The lower level has no voids in it. Stone unit were placed in regular interval and contain the entire space of the lower level of the tower. The upper part of the tower contains a thick stone wall around the perimeter of the tower as well as some false/blind windows of dark gray limestone and an entrance in the north side. The slightly pyramidal roof of the tower was made of stone blocks tied together using metal clamps. However, there is no steel clamp in its current state. The foundation of the tower is an approximately square terraced pyramid of three steps made of large stone slabs (Schmidt 1970). Although the foundation slabs have different thicknesses, they have been laid in level surface.

Prior to the archeological excavations of Prof. Schmidt from the Oriental Institute, University of Chicago in June 1939, the lower part of the Tower was buried beneath debris (Schmidt 1970). The staircase which gave access to the main room of the Tower, was built of stone blocks. However, the exposed blocks of the upper part of the staircase have been demolished. Today, only part of the staircase is preserved. The portion of the north wall behind the upper part of the staircase has been mostly destroyed. The Cube of Zoroaster tower is square in plan with dimensions of 7.30 m on each side (Schmidt 1970). However, there are four ornamental piers in its corners of approximately 1.06 m on each side. The total maximum height of the Cube of Zoroaster tower including the steps of the base is 14.12 m. The average height of the stone courses is 0.5 m. The upper room of the tower is almost square with dimensions of 3.72m × 3.74 m. The walls' thickness ranges from 1.54 to 1.62 m. The height of the tower room is 5.54 m.

Overview of the discrete element method for modeling dry masonry

3DEC is a numerical modeling code based on DEM for discontinuous modeling and can simulate the three dimensional response of discontinuous media, such as masonry, subjected to either static or dynamic loading. When used to model masonry, the units (i.e. stones) are represented as an assemblage of rigid or deformable blocks which may take any arbitrary geometry. Typically, rigid blocks are adequate for structures with stiff, strong units, in which deformational behavior takes place at the joints. For explicit dynamic analysis, rigid block models run significantly faster. For static problems, this computational advantage is less important, so deformable blocks are preferable, as they provide a more elaborate representation of structural behavior. In the analyses reported herein, rigid blocks were used. Joints are represented as interfaces between blocks. These interfaces can be viewed as interactions between the blocks and are governed by appropriate stress-displacement constitutive laws. These interactions can be linear (e.g. spring stiffness) or non-linear functions. Interaction between blocks is represented by set of point contacts, of either vertex to face or edge to edge type (Itasca 2013). In 3DEC, finite displacements and rotations of the discrete bodies are allowed. These include complete detachment between blocks and new contact generation as the calculation proceeds. Contacts can open and close depending on the stresses acting on them from the application of the external load. Contact forces in both the shear and normal direction are considered to be linear functions of the actual penetration in shear and normal directions, respectively (Itasca 2013). In the normal direction, the mechanical behavior of joints is governed by the following equation:

$$\Delta\sigma_n = - JK_n \cdot \Delta u_n \tag{1}$$

where JK_n is the normal stiffness of the contact, $\Delta\sigma_n$ is the change in normal stress and Δu_n is the change in normal displacement. Similarly, in the shear direction the mechanical behavior of mortar joints is controlled by a constant shear stiffness JK_s using the following expression:

$$\Delta\tau_s = - JK_s \cdot \Delta u_s \tag{2}$$

where $\Delta\tau_s$ is the change in shear stress and Δu_s is the change in shear displacement. These stress increments are added to the previous stresses, and then the total normal and shear stresses are updated to meet the selected non-elastic failure criteria, such as the Mohr-Coulomb model.

The calculations are made using the force-displacement law at all contacts and the Newton's second law of motion at all blocks. The force-displacement law is used to find contact forces from known displacements, while the Newton's second law governs the motion of the blocks resulting from the known forces acting on them. Convergence to static solutions is obtained by means of adaptive damping, as in the classical dynamic relaxation methods.

DEM modeling of the tower

Development of the geometric model and material properties

A geometric model to represent the current condition of the tower has been developed (see Figure 2). Blocks assumed to behave as rigid elements connected together by a zero thickness interface. Thus, the nonlinear behavior of the structure is caused by geometrical nonlinearity and friction between the stone blocks in contact and not with the material nonlinearity. The generated model had about 2,433 rigid blocks.

The unit weight of the limestone masonry blocks was considered to be 2,680 kg/m^3. The joints' normal and shear stiffness parameters (i.e., JK_n and JK_s) can be evaluated from the stiffness of the real joint under the assumption of stack bond (Lourenço et al. 2005). However, due to the lack of experimental data, typical values of 4×10^9 and 2×10^9 N/m^3 for the joints' stiffness in the normal and shear directions respectively have been used (Sarhosis et al. 2016b). The zero thickness interfaces between adjacent blocks were modelled using the Mohr-Coulomb slip model (Itasca 2013). The angle of internal friction at the block-to-block interfaces were assumed 30°, which is typical for low bond strength limestone construction with dry mortar. Since the tower has been constructed with dry joints, both the cohesion and tensile strength at the interfaces were set to zero.

Seismic input and damping

Initially, gravitation load has been assigned into the tower system. Then, the system brought into equilibrium under its own weight where the unbalance forces checked whether they are constant and almost equal to zero. Strong ground motions obtained from the Tabas earthquake in three different stations (i.e. Tabas, Ferdows and Kashmar) were applied at the base of the tower.

The Tabas earthquake occurred on the 16th of September 1978 in central Iran. The earthquake had 7.35 Richter magnitude. The PGA, PGV, PGD and duration of longitudinal and transverse and vertical components of the selected records are given in Table 1. Figure 3 shows the longitudinal components of the earthquakes acceleration along with their Fourier amplitude spectra. The longitudinal and transverse components of the earthquakes were applied parallel (x-axis) and perpendicular (y-axis) to the staircase direction, respectively (see Figure 2). It should be mentioned that the peak ground acceleration of the far-filed Ferdows and Kashmar records were scaled to reach the Tabas record PGA (i.e., L: 0.85g, T: 0.86g and V: 0.64g) as shown in Table 1. Such scaling was implemented to investigate merely the effect of earthquake's frequency contents on the tower vulnerability. The dominant frequency of the records are also given in Table 1. All three components of the earthquake records were applied simultaneously to the tower base block as velocity records in DEM model. During the dynamic analysis, no viscous damping was assumed. The only dissipation being due to frictional sliding on the joints. This conservative assumption is often used in simulating stone masonry structures containing dry joints (e.g. Papantopoulos et al. 2002). However, it should be mentioned that considering no viscous damping is just suitable for the analysis of masonry structures in which frictional sliding is the dominant failure mechanism. This is the case occurred in the monument investigated in this paper. However, if rocking motion failure is the governing mechanism, contact-impact energy mechanism should also be considered.

Validation study

Prior to the analysis of the tower under the selected earthquakes, the validity of the adopted three dimensional DEM model was investigated. The Ka‘ba-ye Zartošt (Cube of Zoroaster) is a historic monument which is impossible to undertake in-situ destructive tests or to remove and test samples of the tower which are large enough to be representative. Consequently, the validity of the numerical model undertaken using the shaking table experimental results of a stone masonry column tested in the laboratory by Mouzakis et al. (2002). Mouzakis et al. (2002) investigated the seismic behavior of a 1:3 scale model of a stone multi-drum column of the Pronaos of the Parthenon on a 6-DOFs shaking table at the National Technical University of Athens. The total height of the column was 3.34 m and was made of marble. The column had 12 drums and a capital resting on a marble base as shown in Figure 4a. The column was tested under the scaled strong ground motions of Argostoli, Kalamata and Edessa. For the purpose of this study, numerical investigations of the dynamic behavior of the column subjected to the earthquake record of Edessa (i.e. specimen EQ17) with a scale factor of 2.0 and longitudinal, transverse and vertical PGAs of 0.26g, 0.15g and 0.09g, respectively, was simulated using the distinct element method software 3DEC. It should be noted that the above mentioned PGAs are the actual input seismic motions given to the shaking table which are slightly different from the real recorded Edessa motions. The stone blocks were assumed to be rigid elements connected together by zero-thickness interface elements. The nonlinear behavior of the column was considered at the block-to-block interfaces using the Mohr-Coulomb slip model with the angle of internal friction taken equal to 35° and zero cohesion and zero tensile strengths. The joints’ normal and shear stiffness parameters (i.e., JK_n and JK_s) were assumed to be 2×10^9 N/m^3. The stiffness and mass damping were both assumed equal to zero as discussed earlier in Section 4.2. It should be mentioned that specimen EQ17 has been analysed previously in Papantonopoulos et al. (2002) using the same software. In Papantonopoulos et al. (2002), in order to match better the numerical data with the experimental one, the friction angle and the stiffness parameters at the joints were chosen slightly varying for different specimens. In fact in Papantonopoulos et al. (2002), the joints' normal and shear stiffness parameters were assumed to be 1.0 or 2×10^9 N/m^3 and the friction angle was considered equal to 35 or 37°. However, the exact assumed values of these parameters used for each specimen model have not been reported in their study. The experimental and numerical (DEM) lateral displacement histories of the monitoring point K3 (see Figure 4) of the column have been plotted in Figure 5. In Figure 5, the numerical analysis results conducted by Papantonopoulos et al. (2002) were also plotted for comparison reasons and denoted as "3DEC". From Figure 5, the numerical simulation of this study denoted by "Present study" can capture the overall seismic response of the column with a good agreement which denotes the validity of our developed model.

Results and discussions

During the analyses, the time-history of lateral displacements of eight monitoring points around the tower at two levels as shown in Figure 6 were recorded. The tower response to each earthquake record is presented and discussed below.

Dynamic system identification of the tower

In order to identify the fundamental frequency of the tower, it was first excited by the transverse component of the Tabas earthquake applied in Y direction, since this was decided to be the most vulnerable situation. Initially, the record PGA was set to a small value (i.e. 0.2g) to ensure monolithic and elastic behavior of the tower during the dynamic analysis. During the analysis, the velocity histories of monitoring point 5 in both X and Y directions (see Figure 6) were recorded and analyzed. To identify the fundamental frequency, the velocity histories were transferred from the time domain to the frequency domain using the Fourier Amplitude Spectrum (FAS) as shown in Figure 7. The fundamental frequencies of the tower in the X and Y directions obtained by the FAS of the recorded velocity histories are presented in Figure 7. As it is seen, the tower has approximately the same fundamental frequencies in both principal directions (i.e. 3.75 Hz vs 3.70 Hz). However, it should be noted that if the intensity of input motion increases, the vibrational frequencies would decrease due to nonlinear behavior of the tower.

Response of the tower to Tabas Earthquake

The histories of relative lateral displacements of the monitoring points of the roof slab along both principal directions of X and Y to the tower base are shown in Figure 8. From Figure 8a, for the first 11 seconds of the earthquake excitation, the X-axis horizontal displacement of all the monitoring points are the same, while just after the 11th second of the excitation, each corner of the roof moves independently to each other. This is an indication that the isolated slabs of the roof has been detached. However, at the end of excitation, the lateral displacements at the control points 6, 7 and 8 approach more or less to each other, while the X-axis horizontal displacement for the control point 5 oscillates around a permanent lateral displacement of about 0.064 m to the right.

Similarly, from Figure 8b it can be seen that for the first 10 seconds of the earthquake excitation, the Y-axis horizontal displacement of all the monitoring points are the same, while just after the 10th second of the excitation, the two corner points located on a common roof stone slab unit vibrate with the same amplitudes. However, it is apparent from the Figure 8b that the two parallel roof slabs vibrate independently to each other.

The maximum lateral displacement under this record occurred along the Y-axis direction (i.e. direction perpendicular to the staircase direction). The occurrence of low lateral displacements along the X-axis direction compared to the other direction, may be attributed to the bracing effect of the damaged staircase along the X-axis direction and the presence of no opening in this direction, as well.

Response of the tower to Ferdows Earthquake

The histories of relative lateral displacements of the monitoring points of the roof slab under the Ferdows earthquake are plotted in Figure 9. From Figure 9a, it is seen that for the first 8 seconds of the earthquake excitation, the X-axis horizontal displacement of all the monitoring points are the same, while just after the 8th second of the excitation, the monitoring points tend to vibrate independently to each other until the 20th second which they start approaching to each other again. As the earthquake load progresses, and after the 20th second of the earthquake, the monitoring points separate again from each other and vibrate independently around different permanent displacements up to the end of record without collapse.

Similarly, from Figure 9b it can be seen that for the first 13 seconds of the earthquake excitation, the Y-axis horizontal displacement of all the monitoring points are the same, while just after the 13th second of the excitation, the two corner points located on a common roof stone slab vibrate with the same amplitudes. Also, from Figure 9b, it is apparent that the two parallel roof slabs vibrate independently to each other.

Comparison of inter-story drift demands and failure mode of the tower subjected to Tabas and Ferdows EQs

The maximum inter-story drift demands of the tower corner points under the Tabas earthquake are shown in Figure 10a. From Figure 10a, the tower sustains considerably large lateral drifts in each direction with a maximum value of 5.5 % without collapse. While the ordinary masonry towers can hardly tolerate lateral drifts of more than 1% as reported in the literature (Sarhosis et al. 2018). This large lateral drift may be attributed to the governing failure mode of the tower which is a sliding shear failure mode throughout the tower height. This mode is a deformation-controlled (ductile) mode which makes the structure to sustain large lateral drifts without global instability. On the other hand, the phenomenon of shear sliding along the bed joints would act like a frictional damper which can benefit the stability of the tower because of increasing energy dissipation during strong motion reversal.

Furthermore, it should be noted that large pieces of stone blocks compared to the tower overall plan dimensions have been used in its construction. As it was mentioned earlier, the plan dimension of the tower is 7.30 m by 7.30 m and the horizontal length of smallest and largest stone blocks are 1.38 m and 4.40 m, respectively. Therefore, the ratio of stone blocks horizontal dimension (b) to the tower horizontal dimension (B) varies between 0.19 and 0.6 which is significantly larger to the ratio of conventional masonry towers/buildings constructed with much shorter masonry units. The ratio of b/B highlights the "blocks size effect" on the lateral load behavior of this unique in construction masonry tower. As this ratio increases in a masonry wall/tower, lateral deformation capacity increases as a result of increasing interlocking between the adjacent stone blocks. Also, from 12b, during the Ferdows earthquake, the tower sustains considerably larger lateral drifts in each direction with a maximum value of 10.1 % without collapse. In all cases, and as expected the maximum inter-story drift observed at the second story of the tower.

Figure 11 shows the deformed geometry of the tower when subjected to Tabas and Ferdows EQs. From Figure 11 it is evident that the tower undergoes shear sliding failure mode along bed joints. Furthermore, it is seen that the tower deforms simultaneously in complex lateral and torsional deformation modes.

Figure 12 shows the lateral displacement profile of the tower confirming its complex and irregular deformed shape at the end of the earthquakes. This complexity may be attributed to the vertical stiffness irregularity of the tower as well to the presence of the door opening in the upper level. From Figure 12a, the maximum deformation at the tower at the end of the Tabas earthquake was equal to 45 cm. Also, from Figure 12b, the maximum deformation at the tower at the end of the Ferdows earthquake was equal to 38 cm. Also, as expected, the maximum deformation in the tower observed in the second story.

Response to Kashmar Earthquake

The histories of relative lateral displacements of the monitoring points of the roof slab under the Kashmar earthquake are plotted in Figure 113. From Figure 13a, for the first 6 seconds of the earthquake excitation, the X-axis horizontal displacement of all the monitoring points are the same, while just after the 6th second of the excitation, the monitoring points of the different roof slabs separate from each other and their lateral displacement increase rapidly with time. Similar behavior was observed for the Y-axis displacements, as illustrated in Figure 13b. In fact, this earthquake could destroy the upper level of the tower as shown in Figure 14. From Table 1, it can be seen that the Kashmar record is the only record which has low-frequency content in all components. Therefore, it seems that the tower is vulnerable to such low-frequency content far-field earthquake records. This behavior is somehow similar to the vulnerability of multi-drum stone columns to low-frequency content earthquakes as reported in the literature by Psycharis (2013). However, there is a major difference between the failure modes of this tower and stone columns. According to Psycharis (2003), multi-drum stone columns under low-frequency content earthquakes fail in a rocking mode like a monolithic stone block. While, the considered low-frequency content records in this study led to the collapse of this stocky stone tower in which the upper story blocks displaced excessively and toppled over (Figure 14).

Conclusions

Historical masonry constructions have inevitably suffered damage with time. Earthquakes, soil settlements, material degradation and lack of maintenance are the main reasons for that. Careful and periodic assessment of such structures is necessary in order to evaluate their structural capacity and safety levels. However, performing the structural analysis of historical masonry construction is not an easy task. Historic masonry is a material which is characterized by high non-linearity and discontinuity; it is the mortar joints in a masonry structure which act as a plane of weakness. The need to predict the in-service behavior and load carrying capacity of masonry structures has led researchers to develop several numerical methods and computational tools which are characterized by their different levels of complexity. For a numerical model to adequately represent the behavior of a real structure, both the constitutive model and the input material properties must be selected carefully by the modeler to take into account the variation of masonry properties and the range of stress state types that exist in masonry structures. A broad range of numerical methods is available today ranging from the classical plastic solution methods to the most advanced non-linear computational formulations (e.g. finite element and discrete element methods of analysis).

In this study, an investigation has been undertaken into the seismic behavior of the Ka‘ba-ye Zartošt (Cube of Zoroaster) tower located in Fars, Iran. Use made of the three dimensional software based on the discrete element method of analysis 3DEC. Since, in-situ destructive tests or the testing of samples of masonry removed from the structure that are large enough to be representative, is not usually possible, the model validated against a series of experimental tests obtained from the literature. Within the model, the tower was represented by a series of distinct blocks separated by zero thickness interfaces.

Ground motions obtained from the Tabas, Ferdows and Kashmar earthquakes were used and applied at the base of the tower in the longitudinal, transverse and vertical directions, simultaneously. Also, the time-history of horizontal displacements at the corner points in first story and roof slab of the tower were recoded. From the results analysis, it can be observed that the seismic behavior of the tower is highly non-linear and influenced by the frequency content of the earthquake applied. In particular, when the tower subjected to Tabas EQ, it sustained later drifts up to 5.5% without collapse. However, when the tower subjected to Ferdows EQ, it sustained considerably larger lateral drifts in each direction with a maximum value of 10.1 % without collapse. In both cases studied, and as expected the maximum inter-story drift observed at the second story of the tower. Moreover, the failure mode of the tower was a combination of both shear and torsional failure. Such failure mode allowed the structure to sustain large lateral drifts without global instability. This ductile behavior could be due to the relatively large size of the stone blocks compared to the size of the tower. In addition, when the Kashmar earthquake applied, which is a much lower frequency earthquake compared to Tabas and Ferdows EQs, it was found that the stone tower couldn't sustain such load and collapsed. Therefore, from this present study it is evident that this particular dry joint ancient stone tower requires earthquakes of low-frequency content to cause global failure of its upper part.

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