Article:

Pulatsu, B, Sarhosis, V, Bretas, EM et al. (2 more authors) (2017) Non-linear static behaviour of ancient free-standing stone columns. Proceedings of the Institution of Civil Engineers - Structures and Buildings, 170 (6). pp. 406-418. ISSN 0965-0911

https://doi.org/10.1680/jstbu.16.00071

Nonlinear static behaviour of multi-drum ancient columns

B. Pulatsu[1], V. Sarhosis[2], E. Bretas[3], N. Nikitas[4], P.B. Lourenço[5]

[1]Architectural Engineering, University of Nebraska-Lincoln, Omaha, NE, USA
[2]School of Civil Engineering and Geosciences, Newcastle University, Newcastle upon Tyne, NE1 7RU, UK, email: vasilis.sarhosis@newcastle.ac.uk
[3]Department of Infrastructure, Materials and Structures, Northern Research Institute, Narvik, Norway
[4]School of Civil Engineering, University of Leeds, LS2 9JT, Leeds, UK
[5]ISISE, Department of Civil Engineering, University of Minho, Guimarães, Portugal

Abstract

This paper investigates the nonlinear static behaviour of blocky multi-drum ancient columns. A two-dimensional custom-made computational software based on the Discrete Element Method developed. In the computational model, the columns represented as an assemblage of distinct blocks connected together by zero thickness interfaces, which can open and/or close depending on the magnitude and direction of the stresses applied to them. Through nonlinear static analysis, capacity curves and corresponding failure mechanisms of each of the studied models obtained. The influence of different parameters, namely number of drums, geometrical properties and imperfections at columns, assessed to observe their influence on the response of drum assemblies. The results of analyses revealed that rigid overturning is the main collapse mechanisms under uniform horizontal forces. A combination of rigid and shear failure mechanisms might be obtained depending on geometric characteristics and choice of joint material properties used. Higher displacement capacity observed for columns constructed with larger number of drums. It was found that imperfections at the ancient columns have a significant influence on the lateral load resisting capacity. Therefore, structural analysis of undamaged state of the columns may not represent the actual capacity of the columns due to their very sensitive and highly nonlinear characteristics.

Keywords: Ancient monuments; Multi-drum column; Discrete Element Method; Nonlinear push-over analysis.

1 INTRODUCTION

The Eastern Mediterranean area is the richest region in the world in terms of ancient classical columns and colonnades, having a significant archaeological and architectural importance. Today, some of these monuments and their columns, which have the typical structural forms of the ancient Greek and Roman temples, do not maintain their full structural integrity. High seismic events which occurred in earthquake prone regions, including Italy, Greece, Turkey and Cyprus, caused damage to these ancient constructions and monuments through the centuries (Ambraseys 2009).

In general, ancient colonnades can be found as monolithic or multi-drum free standing columns that may or may not have an architrave on top. The multi-drum columns, which are composed of individual stone blocks, lying on top of one another, generally do not have mortar or any other bonding type of material between stone drums. Also, the geometrical characteristics of ancient colonnades may considerably differ from each other, although they may have the same architectural proportions or orders (e.g., Doric, Ionic and Corinthian).

The facilitation of an appropriate intervention approach for structural repair and strengthening of these historically important structures require an improved understanding of their dynamic behaviour. Unlike modern forms of construction, historical monuments have often been exposed to seismic loads throughout their life span. Thus, it is important and useful to understand their kinematic mechanisms, which provide a great contribution to their seismic capacity. In this context, nonlinear static analyses were performed as an alternative way to dynamic analysis and analytical solutions which are not easy and which need high computational effort to get reliable results.

The motivation to analyse the response of rigid bodies dates back to the end of the 19th century. Research on the overturning mechanism of columns, having different sizes and shapes, was first undertaken by Milne (1881). Peak ground accelerations were used to find the seismic capacity. At the beginning of the 20th century, the complex nature and high sensitive response of rectangular columns were studied by Omori (Omori 1900; Omori 1902) and the effect of input motion on the mode of collapse was emphasized in his experimental research. After several decades, minimum horizontal acceleration to overturn a rigid body and the influence of geometrical properties were examined by Housner (Housner 1963). Housner's pioneering work was further validated and improved within time by Peña et al. (2007) and Makris and Vassiliou (2013). Over the last two decades, researchers paid attention to the use of advanced numerical methods to simulate the nonlinear behaviour of multi-drum columns under static and seismic excitations. Yim et al. (1980) developed a computer program to solve the nonlinear equations of motion governing the rocking response of rigid blocks. Significant changes in the response of rigid blocks were noticed by small variations in slenderness ratio and size of the blocks. Later, analytical solutions to examine the nonlinear behaviour of two rigid bodies, placed on top of one another, were presented by Psycharis (1990). A comprehensive body of research to investigate the response of rectangular wooden blocks and block assemblies under harmonic and earthquake base excitation was published by Winkler et al. (1995). To observe the response of single block and block assemblies, numerical analyses were performed using the Discrete Element Method (DEM) (Dimitri et al. 2011, Alexandris et al. 2014, Sarhosis et al. 2016a. Sarhosis et al. 2016b). In these studies the DEM was verified as a powerful method to analyse the stability of freestanding columns and colonnades. Furthermore, the efficiency of DEM was presented by Papantonopoulos et al. (2002) where results predicted from numerical simulations were compared with experimental ones obtained from 1:3 scale model tests of the column of the Parthenon. Also, parametric studies were carried out to understand the influence of ground motion and geometrical properties on the dynamic response of ancient columns (Psycharis et al. 2000; Psycharis et al. 2003). Based on the results, it was found that the frequency content of seismic excitations has significant consequences on the response of columns. In the light of experimental and numerical studies, proposed retrofitting solutions for multi-drum columns were discussed by several researchers (Psycharis et al. 2003; Konstantinidis & Makris 2005).

Experimental tests using small scaled models consisting of marble stone blocks to replicate the Parthenon columns were conducted by Mouzakis et al. (2002). Although overall seismic response of colonnades was revealed through the physical experiment, the experimental testing was found to be highly sensitive to boundary conditions applied, which makes it impossible to replicate even identical experimental setups and perform sensitivity studies. Recently, Drosos and Anastaspoulos (2014) undertook experimental tests on 1:5 scale models of a multi-drum portal frame. The sensitive seismic performance of portal frames was examined under idealized Ricker pulses and real seismic records. Advantage of the architrave in terms of restoring capacity was observed, and main features of dynamic response, such as rocking, sliding or a combination of two, were captured (Drosos & Anastasopoulos 2014). In addition, comprehensive numerical simulations, including parametric studies related with the geometrical properties of ancient columns and colonnades with an architrave, were performed using custom-made software by Papaloizou and Komodromos (2009).

The structural behaviour of multi-drum masonry column differs from the behaviour of typical masonry walls panels and prisms, which consist of numerous blocks (bricks) that are usually bonded by mortar (Sarhosis & Sheng 2014; Giamundo et al. 2014; Sarhosis et al. 2015a; Sarhosis 2016a). The dynamic behaviour of multi-drum structures such as ancient columns shows a three-dimensional motion with a strong nonlinear character. According to Stefanou et al. (2011), the seismic behaviour of multi-drum columns is characterized by rocking, sliding and wobbling motions that can occur within individual stone units or in groups in the form of monolithic behaviour. Due to wobbling, the dissipation of energy is different during seismic excitation, which affects the stability and deformation of the structure. Therefore, three dimensional numerical analyses should be better adapted to the real physics of the problem. Also, the out of plane behaviour of the colonnade can be modelled when a three dimensional model is adopted. However, two dimensional analyses can still be used at the initial stage since they provide significant information relating to the dynamic behaviour of the structure (Dimitri et al. 2011, Sarhosis et al. 2015b, Sarhosis et al. 2015c).

This paper describes the development of a two-dimensional computational model based on a custom-made DEM software to investigate the behaviour of blocky ancient columns found in the Mediterranean region. The columns under investigation consist of varying geometries, with multi-drum stones positioned one over the other. The colonnade was represented as an assemblage of distinct blocks connected together by zero thickness interfaces, which can open and/or close depending on the magnitude and direction of the stresses applied to them. The nonlinear static analyses were performed on the five selected columns. The progressive contact detachments, between each block at the column, were captured under incremental uniform horizontal loading. The main motivation to consider nonlinear static analysis was to demonstrate the deformation capacities and the lateral load resistance of existing columns. Load-deformation characteristics and inelastic response of the columns were found by applying uniform force distribution. In addition, geometrical parametric studies were carried out and both the capacity curves and failure modes of the columns are obtained.

2 DESCRIPTION OF COLUMNS UNDER INVESTIGATION

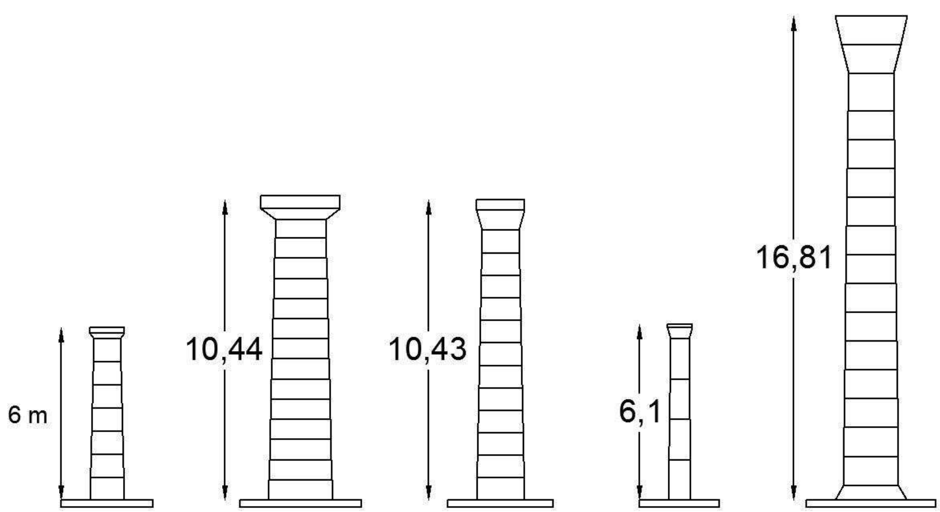
There are a great variety of ancient columns with different geometrical characteristics and varying number of drums worldwide. Some of them are in the form of standalone columns (Figure 1), while others have an architrave on top. Five geometrically different columns have been studied in this research (Figure 2). The first ancient Doric column is from Temple of Apollo at Bassae (Figure 2a), which was built in the 5th century BC. The column is 6 m in height and consists of seven equal in size drums. The diameter of the base and the top drums are 1.1 m and 0.9 m, respectively. The second column (Figure 2b), standing at the classical Greek temple of Doric order, namely Temple of Zeus at Olympia, is 10.4 m in height, with approximately two times larger base and top diameters of than the former. The columns of the Temple of Zeus at Olympia consists of 14 drums. The third column is from the Parthenon’s Pronaos (Figure 2c), located in the Acropolis of Athens, and is regarded a symbol of power and architectural miracle for Ancient Greece and one of the greatest cultural monuments in the world. The column of the Parthenon is 10.4 m in height and the diameter at the base is 2.22 m, tapering to 1.25 m at the top. The column of the Parthenon has twelve drums of the same height excluding the capital. Furthermore, a column at the Arcade of the Ancient Agora (Figure 2d) in the island of Kos, which consists of four drums of same height, was studied. This Doric style 6.1 m height column has nearly the same height but quite different aspect ratio from the column of Temple of Apollo at Bassae. The base and the top diameters are 0.78 m and 0.64 m, respectively. The last studied free standing ancient column belongs to Temple of Olympian Zeus, also known as Olympieion, situated in Athens (Figure 2e). This monument is considerably larger than other temples and has 16.81 m height and 2.51 m base diameter. Thus, all the geometrical properties of each column are shown in Table 1.

Table 1 Geometrical characteristics of colonnades

Name of the Temple	**Total Height (m)**	**Base Diameter (m)**	**Top Diameter (m)**	**Number of Drums (without capital)**	**Aspect Ratio (height over width of the column, H:d)**
Temple of Apollo at Bassae	5.95	1.11	0.90	7	5.36
Temple of Zeus, Olympia	10.44	2.22	1.70	14	4.70
Parthenon Pronaose, Athens	10.43	1.65	1.25	12	6.32
Arcade of the Ancient Agora, Kos	6.10	0.78	0.63	4	7.82
Temple of Olympian Zeus (Olympieion)	16.81	2.51	1.67	17	6.70

Temple of Apollo at Bassae   Temple of Zeus at Olympia   Temple of Olympian Zeus (Olympieion)

Figure 1. Typical free standing columns used in this study



(a) Temple of Apollo, Bassae   (b) Temple of Zeus, Olympia   (c) Parthenon Pronaos   (d) Ancient Agora, Kos   (e) Temple of Olympian Zeus

Figure 2. Geometric characteristics of the ancient multi-drum columns under investigation (Heights are in meter)

3 OVERVIEW OF THE DISCRETE ELEMENT METHOD FOR MODELLING BLOCKY STRUCTURES

3.1 General aspects

In this study, a custom-made software (Bretas et al. 2014, Bretas et al. 2015) based on the DEM (Cundall 1971) was used. This custom-made software was initially developed to solve structural and hydraulic problems of masonry dams and later it was employed to simulate the out of plane behaviour of masonry walls (Pulatsu 2015). Through this research, application field of newly developed software was further extended to understand the static behaviour of historical columns. According to the model, individual blocks can be considered as rigid or deformable. Since the behaviour of masonry structures is dominated by the joints, rather than stone units, rigid blocks are used in the numerical models. Moreover, rigid blocks have computational advantages, especially in explicit dynamic analysis, because the equations of motion are established only in the centroid of the elements. Alternatively, the blocks can be modelled as deformable. In this case, blocks are divided into finite elements which follow the constitutive model assigned to them. Hence, for each separate block, strain can be estimated. Deformable blocks can be assumed to be linear elastic or non-linear according to the Mohr-Coulomb criteria. These blocks are continuum elements as they occur in the Finite Element Method (FEM). However, unlike FEM, in the DEM a compatible finite element mesh between the blocks is not required.

Representation of the contact between blocks is not based on joint elements, as it occurs in the discontinuum finite element models. At the interfaces, the blocks are connected kinematically to each other by sets of point contacts. These contact points are located at the outside perimeter of the blocks and are created at the edges or corners of the blocks and the zones based on the contact hypothesis method (Cundall and Hart 1992). In this custom-made software, the fundamental contact type is face-to-face (Bretas et al. 2014), which is composed of two sub-contacts (Figure 3). The face-to-face contact type allows for the use of different stress integration schemes to determine the contact forces, statically consistent with the stress diagrams and bending stiffness. For each sub-contact, there are two spring connections (Figure 3). These can transfer either a normal force or a shear force from one block to the other. In the normal direction, the mechanical behaviour of joints is governed by Eq. (1):

$$\Delta\sigma_n = k_n \cdot \Delta u_n \tag{1}$$

where k_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 behaviour of joints is controlled by constant shear stiffness k_s using the following expression, Eq. (2):

$$\Delta\tau_s = k_s \cdot \Delta u_s \tag{2}$$

where k_s is the shear stiffness, $\Delta\tau_s$ is the change in shear stress and Δu_s is the change in shear displacement. Stresses calculated at grid points along contacts are submitted to the Coulomb failure criterion, which limits shear stresses along joints (Figure 4**Error! Reference source not found.**). The following parameters are used to define the mechanical behaviour of the contacts: the normal stiffness (k_n), the shear stiffness (k_s), the friction angle (ϕ), the cohesion (c), the tensile strength (f_t) and the dilation angle (φ).

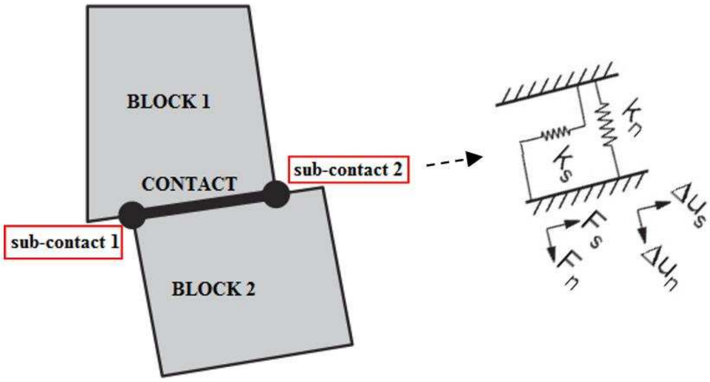


Figure 3. Face-to-face contact type and corresponding sub-contacts where springs are assigned in both orthogonal directions

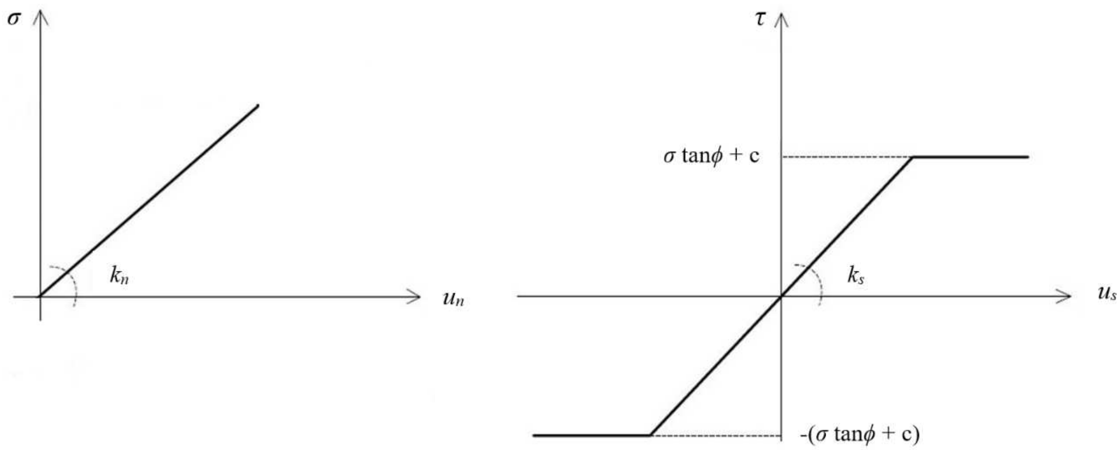


Figure 4. Joint behaviour under normal and shear loads

3.2 Validation Study

The validation of the custom-made software was done by performing pushover analyses on a historical masonry tower, namely Qutb Minar in New Delhi, India. The results of the analyses, demonstrating the lateral load-deformation behaviour of masonry tower, were compared with different numerical analysis approaches such as Finite Element Method (FEM) and Rigid Element Method (REM), which were comprehensively studied by Peña et al. (2010). Nonlinear static analyses were applied considering a uniform force distribution along the height of the tower, where histories of the top corner of the tower were recorded. The capacity curves in terms of lateral displacement versus load factor λ (base shear/self-weight) were generated. Although the results of discrete and rigid element models were found very close to each other, approximately 25-30% difference with finite element model was observed in terms of the maximum load leading to failure and corresponding displacement capacity, as shown in Figure 5. On the other hand, the same collapse mechanism, namely overturning failure, was obtained for different numerical models. Therefore, the result of the custom-made software was validated on the existing masonry tower and good agreement was obtained with other numerical approaches (Pulatsu 2015).

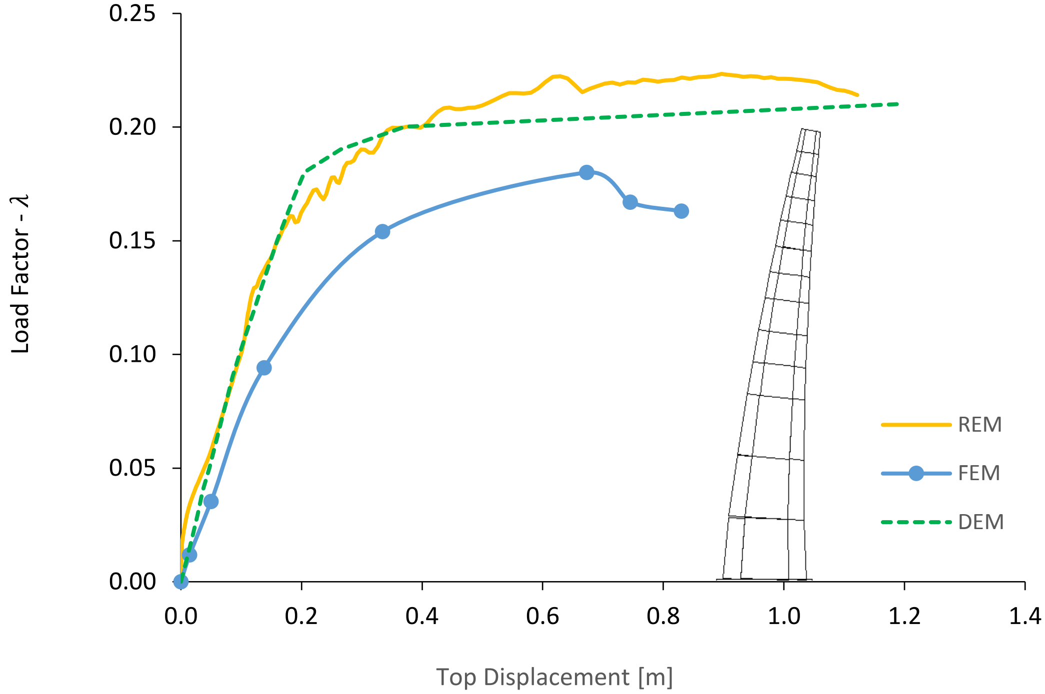


Figure 5. Capacity curve of the custom-made DEM software with other capacity curves obtained by different numerical approaches (Pulatsu 2015)

4 MATERIAL PROPERTIES, BOUNDARY CONDITIONS AND APPLICATION OF LOAD

The material properties of the numerical models are important for the accurate prediction of the lateral behaviour of structures subjected to external loads. Since intrusive tests on archaeological structures are not permitted, in most of the cases, material properties for the stone blocks and joints were obtained from previous small scaled laboratory works and related experimental studies (Papantonopoulos et al. 2002; Drosos and Anastasopoulos 2014). The material parameters used for the development of the numerical models are shown in Table 2. Since the columns are a mortarless (dry-stacked) block masonry system, the joint tensile strength and joint cohesive strength were assumed to be zero. The joint dilation angle was also assumed to be equal to zero. In the normal direction, relatively high compressive strength was assigned to the computational model, since compression failure (e.g. crushing of the stone units) under lateral loading is not expected. Moreover, the unit weight of drums was assumed to be equal to 2,400 kg/m^3 (Drosos and Anastasopoulos 2014). All columns were assumed to sit on a rigid base and can move in horizontal and vertical directions.

Table 2 Properties of the joint interfaces

Normal Stiffness K_n [GPa/m]	Shear Stiffness K_s [GPa/m]	Joint friction angle [degrees]
1	1	37°

Self-weight effects were assigned as gravitational load. At first, the model was brought into a state of equilibrium under its own weight (static gravity loads). Then, uniform acceleration pattern was considered through the analyses. The applied accelerations were multiplied by the mass of each block and turned into uniform horizontal forces acting (nonlinear pushover analysis) on each block, as presented in Figure 6a. The static solutions were obtained by a process of dynamic relaxation, using scaled masses and artificial damping. Viscous mass proportional damping was used, with an adaptive scheme that updates the damping coefficient step-by-step based on the dominant frequency of the structure from the Rayleigh quotient (Sauvé & Metzger 1995). Also, in the proposed numerical model, the default ratio of the damping and the rate of change of nodal kinetic energy is 0.5. However, from the numerical simulations it was found that with this default ratio the convergence of the solution was very slow near collapse: the structure already unstable under the given loads, but the damping did not allow the development of the failure. Therefore, in this study, the ratio was decreased to 0.1. In addition, horizontal displacements at the upper part of each drum of the colonnade were recorded at each loading step (Figure 6) giving rise to the capacity curves. The results from nonlinear pushover analysis of existing columns were compared with different monolithic or multi-drum conditions in terms of displacement capacity and failure mechanism.

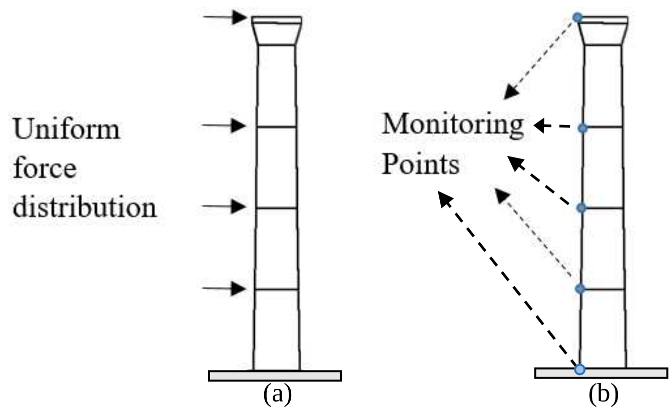


Figure 6. (a) Applied force pattern (b) Points where displacements were recorded

5 CAPACITY CURVES FOR EACH OF THE COLONNADES STUDIED

The obtained capacity curves for the five ancient columns under consideration are shown in Figure 7. The column of the Temple of Zeus in Olympia can carry the largest load (106 kN) and has the lowest aspect ratio (4.7) among the columns studied in the present study. On the contrary, the column of the Ancient Agora carries the lowest load (16 kN) and has the highest aspect ratio (7.82) among the columns analysed here. Furthermore, although the column of the Temple of Zeus in Olympia and the column of the Parthenon Pronaose have identical height, their lateral load-deformation behaviour is dissimilar. Hence, other geometrical properties such as base diameter of the column and number of drums affect the capacity and behaviour of these historical colonnades as represented in capacity curves for the five different standalone columns (Figure 7).

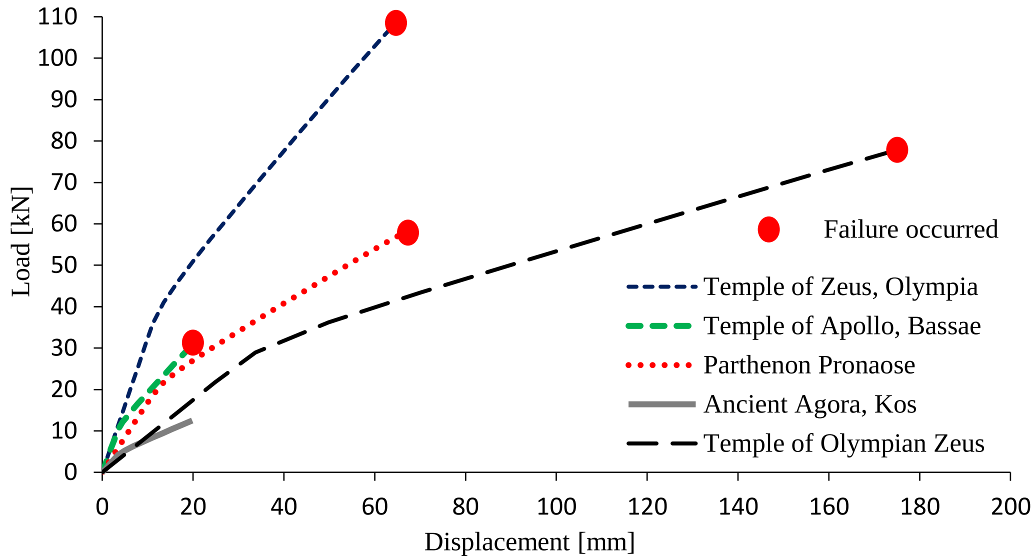


Figure 7. Capacity curves of the five different ancient columns investigated in this study

Figure 8 shows the capacity curve in detail, the resultant horizontal loads, obtained from each load increment through the pushover analysis, against displacement of the column of the Parthenon. The obtained capacity curve is composed of three phases, similar to the rest of the columns investigated in this study. The three phases of the response of columns observed numerically are:

a) **First phase:** The first phase describes an elastic response of the structure, in which all distinct bodies of the column (i.e. drums) are in contact with each other.
b) **Second phase:** With increasing load, contacts between drums detach under uniform loading.
c) **Third phase:** Finally, colonnade fails as a result of excessive shear sliding and/or overturning. Once shearing or opening of the drums has occurred, the sequence of events leading to collapse can be very quick with little warning of impending collapse. The final point, indicating the collapse load and corresponding maximum displacement of the numerical model, is represented by a collapse point (Figure 8).

It is demonstrated that the nonlinear response of drum assemblies is directly controlled by the geometric configuration (e.g. number of drums, size of drums and height of the columns) and joint properties which allow joint opening and closure during the application of external load. From Figure 8, obtained pushover curves have a bilinear fashion, since the considered constitutive laws for the springs at the sub-contacts are simple and the failure mechanism is governed by the lack of tensile capacity at the joints. The apparent difference between elastic limit strength and the maximum horizontal load that causes to failure is shown in Table 3.

Table 3. Failure load and load at first damage at which first opening occurred in ancient columns.

Name of the Temple	***Minimum Horizontal Load (kN) to exceed the elastic response of ancient column (load at first damage)***	***Maximum load (kN) leading to failure***
Temple of Apollo at Bassae	12.1	30.3
Temple of Zeus, Olympia	38.4	108.9
Parthenon Pronaose, Athens	23.8	58.2
Arcade of the Ancient Agora, Kos	4.7	13.5
Temple of Olympian Zeus (Olympieion)	32.6	78.9

Furthermore, it was noticed that there is a certain displacement limit to observe the first detachment between stone units which is around 15% to 25% of the total displacement capacity (Figure 8). For instance, in case of the Column of the Parthenon, the maximum elastic displacement was found as 13.5 mm while, the total displacement capacity, obtained at the end of the pushover analysis, was around 66 mm (Figure 8).

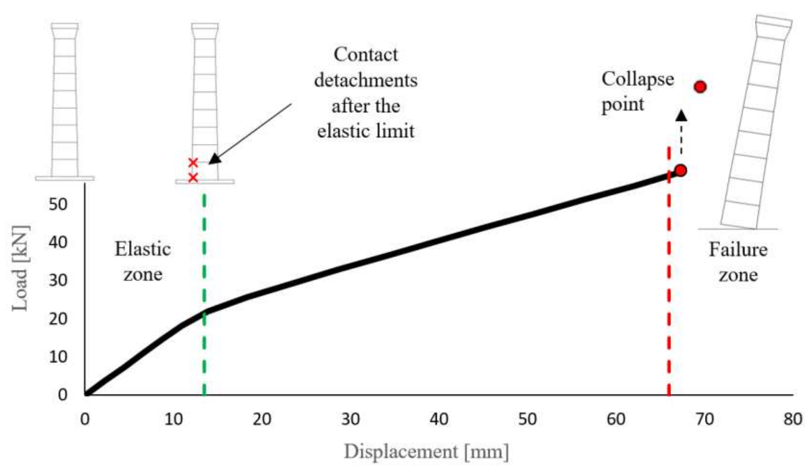


Figure 8. Capacity curve of the column of the Parthenon.

The capacity curves and deformed shapes were further investigated to understand the effect of geometrical parameters on failure modes of the columns. Therefore, the contact points of discrete element models were monitored through each loading step to understand the contact conditions of the drums during the pushover analysis. The contact conditions are important especially for discrete element models to understand the behaviour of structure since the force transmission occurs within the contact points. As a result, the instant contact conditions, e.g. sliding and opening, are captured through the analyses. The main action at the contacts are observed as contact opening. The contact detachments of the column of the Temple of Apollo at Bassae under lateral loading is indicated in Figure 9 **Error! Reference source not found.Error! Reference source not found.**with a cross mark. As the columns start to overturn under applied loading, contact detachments or openings may appear at the joints where tensile forces exist. It was noticed that drums can lose partial face-to-face contact due to lack of tensile strength at the joints under horizontal static loading. The first contact detachment occurred at the bottom drum then went through the height of the column sequentially until the maximum displacement capacity was reached (Figure 9)**Error! Reference source not found.**.

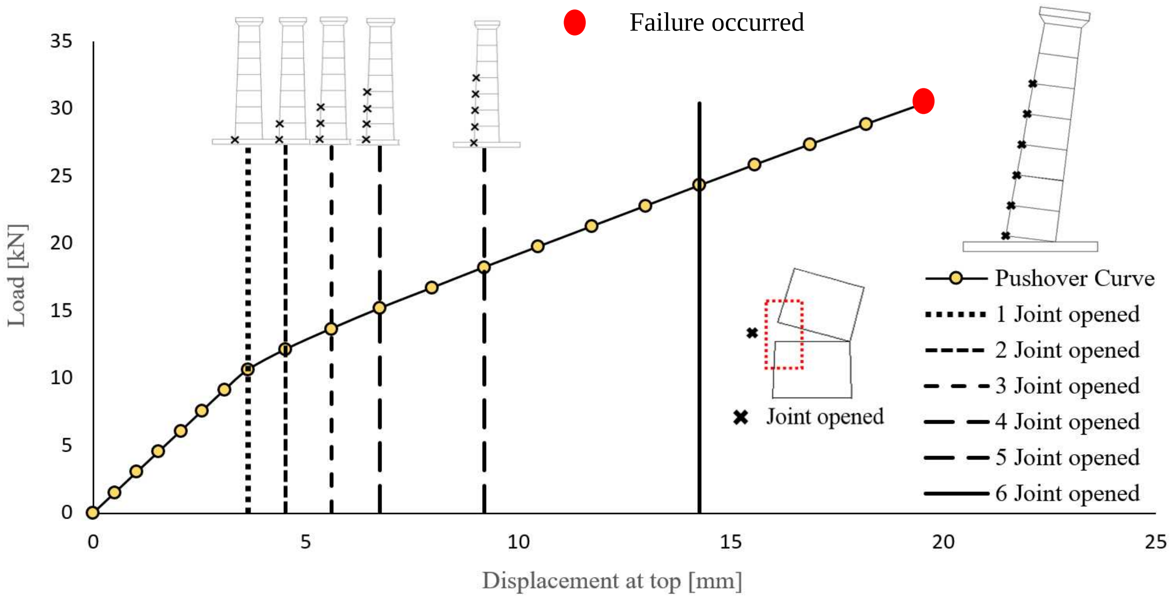


Figure 9. Number of open joints of the column of the Temple of Apollo at Bassae through the pushover analysis

6 PARAMETRIC STUDIES

6.1 Influence of the number of drums

The influence of the number of drums was investigated by examining the displacement capacity of each column subjected to external horizontal loading. The geometry of each column varies from monolithic to 4, 8 and 12 number of drums. An example of the geometric parametric study for the case of the column of the Arcade of the Ancient Agora (Figure 10).

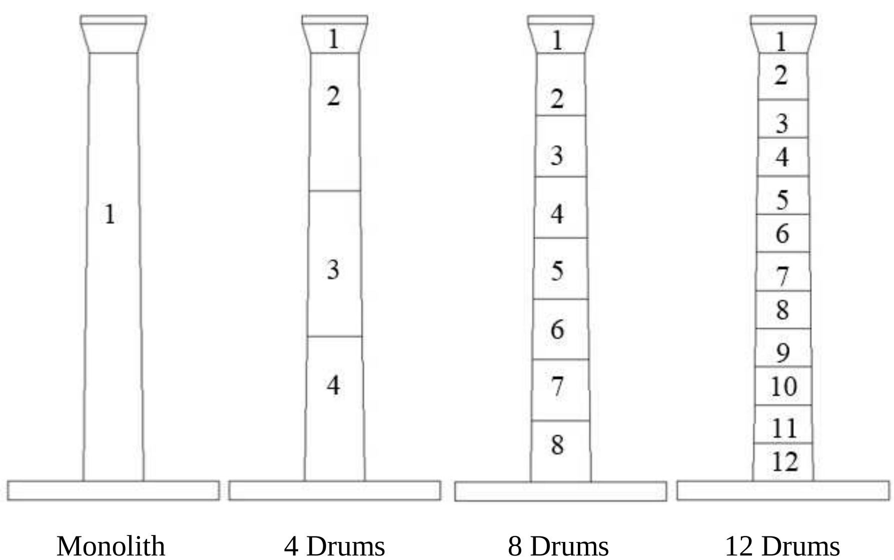
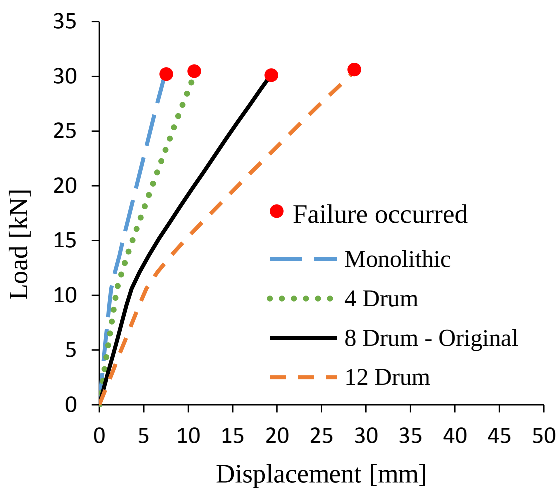
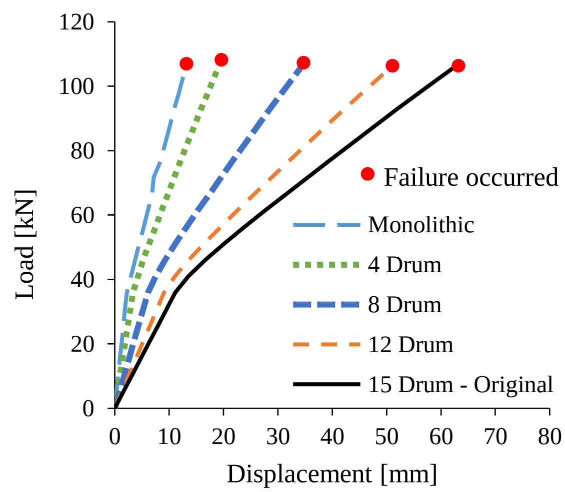


Figure 10. Geometries of the colonnade of the Arcade of the Ancient Agora in Kos used in the sensitivity study.

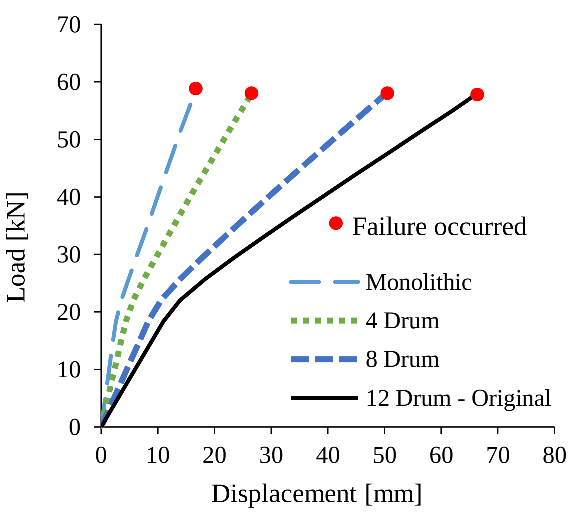
Figure 11 shows the capacity curves for each studied column consisting of different number of drums. The results indicated that the number of drums has a significant effect on the capacity curves of studied models. For each of the columns studied, as the number of drums increases, the column develops a larger displacement capacity. Also, it was observed that columns composed of 12 drums have 2.5 to 4 times higher displacement capacity than their monolithic forms, given the fact that joints have some elastic deformability and this extends to the nonlinear range. However, the number of drums do not have any noticeable influence on the ultimate strength of the columns as indicated in Figure 11.

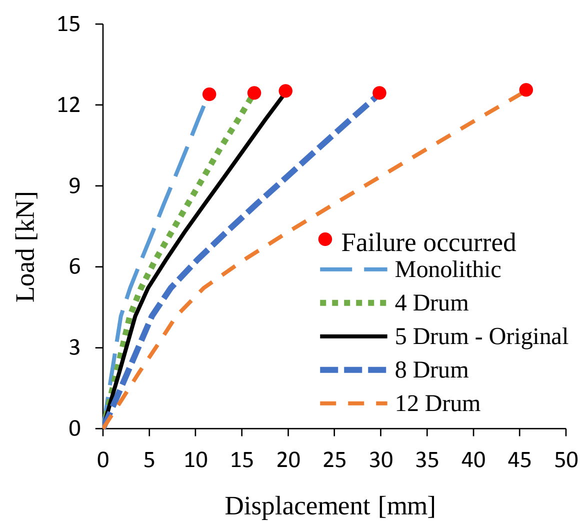


(a) Temple of Apollo at Bassae



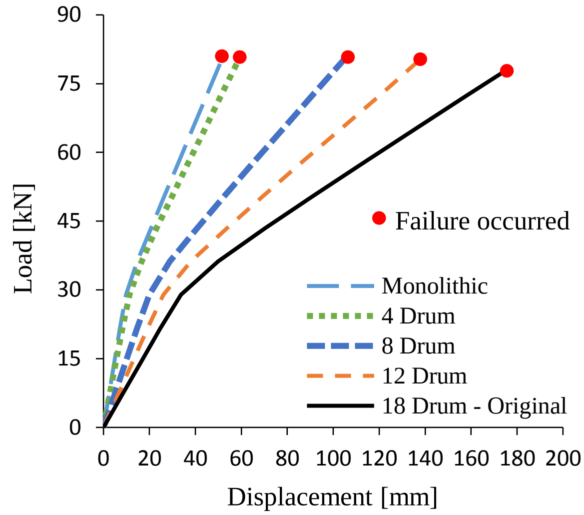
(b) Temple of Zeus, Olympia





(c) Column of the Parthenon Pronaos

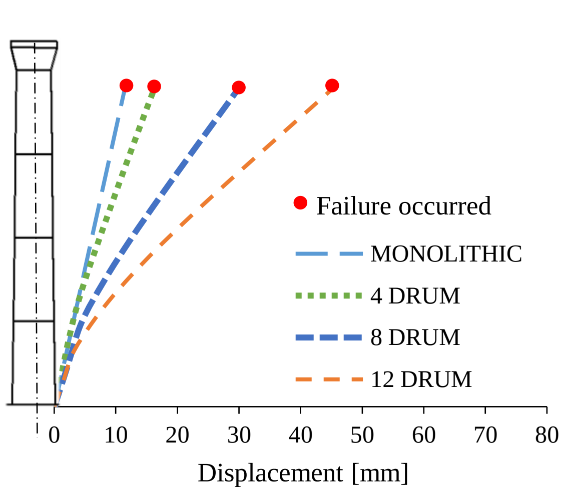
(d) Arcade of the Ancient Agora in Kos



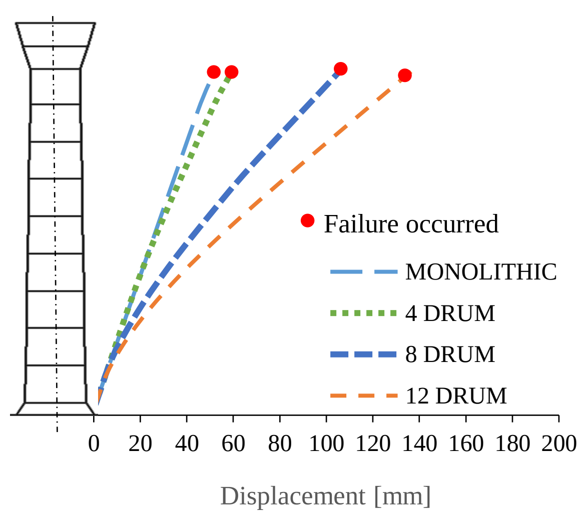
(e) Temple of Olympian Zeus (Olympieion)

Figure 11. Capacity curves, representing the influence of the number of drums

Figure 12 shows the deflected shapes of the column of the Ancient Agora in Kos and Temple of Olympian Zeus, depending on the number of drums just before failure. According to Figure 12, each column has an overturning mechanism with different displacement capacities depending on the number of drums. However, all investigated columns exhibit less brittle behaviour and higher deformability when they consist of larger number of drums (Figure 12).



Column of the Ancient Agora in Kos



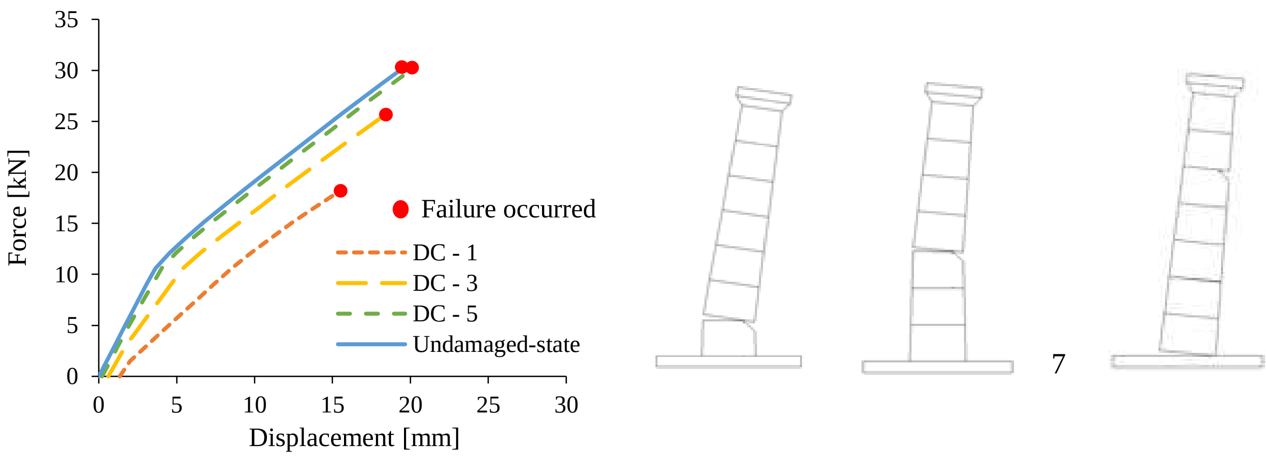
Temple of Olympian Zeus (Olympieion)

Figure 12. Deflected shapes of the ancient columns depending on the number of drums

6.2 Influence of the imperfections at the drums

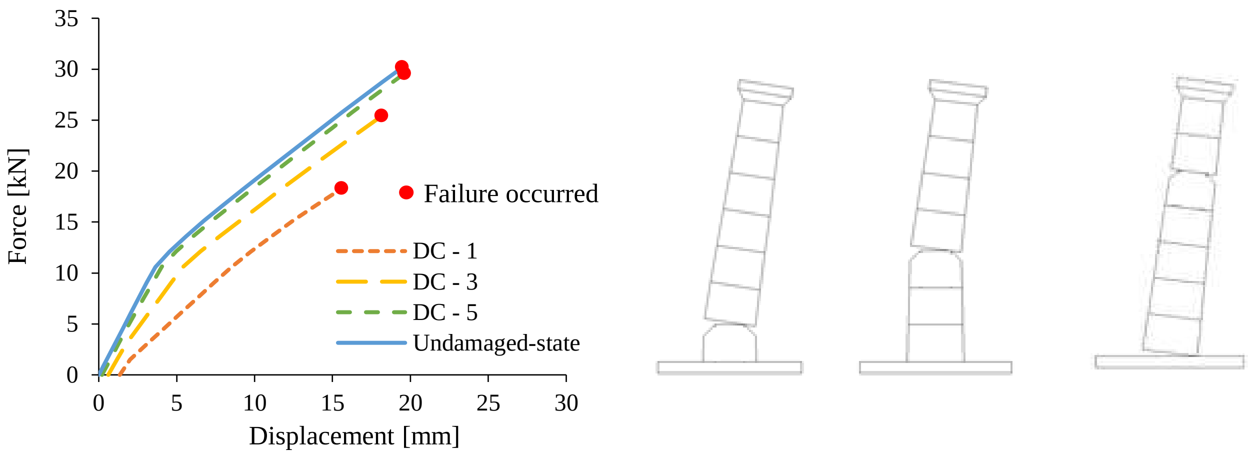
Over the years, strong earthquakes, stone deteriorations, vandalism attacks as well as inappropriate intervention techniques have led to geometrical imperfections of ancient columns. Therefore, it is almost impossible to categorize the imperfections due to the unique characteristics of each structure. As a result of this, different scenarios were prepared for the column of Temple of Apollo at Bassae to demonstrate the influence of imperfections on the load carrying capacity and failure mode. The type and location of the imperfections (in this case it is localized at the corners) have significant consequences on the ancient columns in terms of the maximum displacement capacities and failure mechanisms. From Figure 13, the location of the imperfections in the drums has a remarkable influence on the load carrying capacity and failure mode of the free standing columns.

The main imperfection in Figure 13(a) is considered as a deterioration at the right corner, while in Figure 13(b), imperfections are assigned to both left and right edges. The location of the irregular drum units was changed through the height in order to investigate the effect of the deteriorations on the load carrying capacity of the standalone columns. A drastic decrease in strength was observed, when the irregular stone units, or drums, are located at the bottom. Furthermore, collapse mechanism may change depending on their location and type of the imperfections at the column, as represented here. Therefore, it is important to take into account the current structural condition of ancient columns in order to estimate the load carrying capacity precisely.



(a) Capacity curves for the imperfection at one edge of the drum

DC – 1 DC - 3 DC – 5



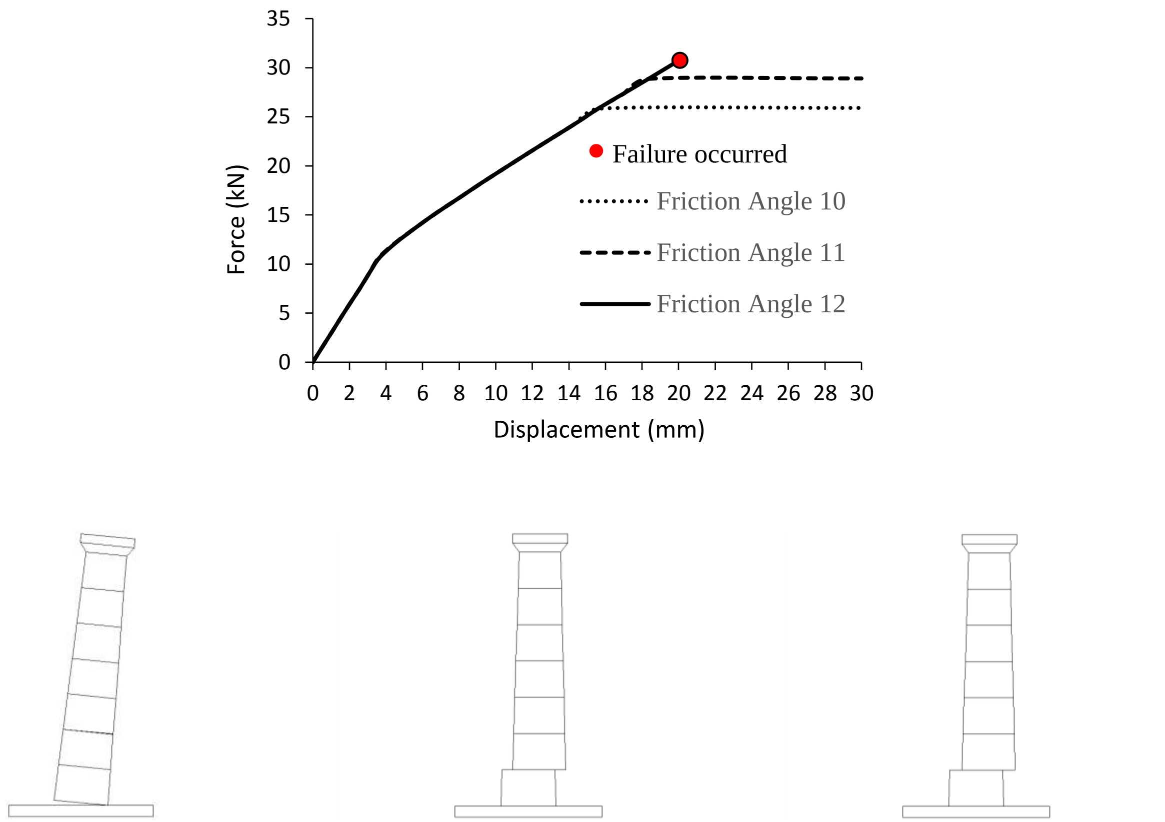
(b) Capacity curves for the imperfections at both edges of the drum

DC - 1 DC - 3 DC – 5

Figure 13. Scenarios of different imperfections for the column of the Temple of Apollo at Bassae and corresponding failure and capacity curves (DC: Damage Condition)

8 INFLUENCE OF THE FRICTION ANGLE BETWEEN DRUMS

The type of rock used for the construction of the ancient columns varies and has different properties. The value of roughness between individual drums in a column is an additional parameter that may lead to higher or lower values of coefficient of the friction. There may be cases, where ancient columns were repaired and the old drums were replaced. Also, joint degradation effect and/or water ingress between the drums of a column may be present. These conditions might result in different coefficients of friction between drums even in the same column. In the present study, a parametric study was carried out on the influence of friction angle on the pushover response of the columns under investigation. The friction angle between drums varied from 10 to 40 degrees (Dimitri et al. 2011, Sarhosis et al. 2016). As shown in Figure 14, the friction angle has some influence on the collapse mechanism and ultimate load carrying capacity. Lower joint friction angles lead to sliding under uniform horizontal loading, whereas the higher friction angles lead to overturning failure.



(a) Friction angle: 12 degrees (b) Friction angle: 11 degrees (c) Friction angle: 10 degrees

Figure 14. Influence of the friction angle on collapse mechanism and capacity curve of the column of the Temple of Apollo at Bassae

9 CONCLUSIONS

A two-dimensional custom-made computational model developed based on the DEM to investigate the static nonlinear behaviour of blocky ancient columns commonly found in the Mediterranean region. The ability to simulate such complex systems of multi-drum columns is crucial to better understand how ancient monuments have experienced and survived strong earthquakes throughout centuries. Five ancient columns with different geometries consisting of multi-drum stones positioned one over the other were studied. In the numerical model, the columns were represented as an assemblage of distinct blocks connected together by zero thickness interfaces, which could open, and/or close depending on the magnitude and direction of the stresses applied to them. Through nonlinear static analysis of the models, capacity curves and corresponding failure mechanisms were obtained. Rigid overturning was found as the governing failure under uniformly distributed load. As pointed out in the previous sections, general kinematic mechanism starts with small openings at the contact points due to lack of tensile capacity, and ends up with an overturning mechanism. A sensitivity study undertaken to assess the influence of the number of drums under lateral loading. Lateral loads against displacement curves were obtained depending on the size and number of drums. The columns consisting of higher number of drums developed higher deformation capacities than monolithic ones, which have more brittle failure. It is recommended that in order to assess the seismic response of the ancient columns, the exact geometry (including geometrical imperfections and/or damage) should be considered. Otherwise, depending on the level of existing damage and/or imperfections, the results may not represent the real behaviour and capacity of the columns. In addition, a sensitivity study carried out to assess the influence of the friction angle of the drum-to-drum interface. From the analysis of results, lower values of the coefficient of friction increase the dominance of sliding between the drums.

References

Alexandris, A., Psycharis, I. & Protopapa, E., 2014. The collapse of the Ancient Temple of Zeus at Olympia revisited. In *2nd European Conference on Earthquake Engineering and Seismology*.

Ambraseys, N., 2009. *Earthquakes in the Mediterranean and Middle East: A Multidisciplinary Study of Seismicity up to 1900*, Cambridge University Press.

Bretas, E.M., Lemos, J. V. & Lourenço, P.B., 2014. A DEM based tool for the safety analysis of masonry gravity dams. *Engineering Structures*, 59, pp.248–260. Available at: http://linkinghub.elsevier.com/retrieve/pii/S0141029613005233.

Bretas, E.M., Lemos, J. V. & Lourenço, P.B., 2015. Seismic analysis of masonry gravity dams using the Discrete Element Method: Implementation and application. *Journal of Earthquake Engineering*, 2469(October), p.150901094713002. Available at: http://www.tandfonline.com/doi/full/10.1080/13632469.2015.1085463.

Cundall, P.A., 1971. A computer model for simulating progressive, large-scale movements in blocky rock systems. In *The International Symposium on Rock Mechanics*. pp. 47–65.

Cundall, P.A. & Hart, R.D., 1992. Numerical modelling of discontinua. *Engineering Computations*, 9(2), pp.101–113. Available at: http://www.emeraldinsight.com/doi/abs/10.1108/eb023851.

Dimitri, R., De Lorenzis, L. & Zavarise, G., 2011. Numerical study on the dynamic behavior of masonry columns and arches on buttresses with the discrete element method. *Engineering Structures*, 33(12), pp.3172–3188. Available at: http://linkinghub.elsevier.com/retrieve/pii/S0141029611003336.

Drosos, V. & Anastasopoulos, I., 2014. Shaking table testing of multidrum columns and portals. *Earthquake*

Engineering & Structural Dynamics, 43(11), pp.1703–1723. Available at: http://onlinelibrary.wiley.com/doi/10.1002/eqe.2230/full.

Giamundo, V. et al., 2014. Evaluation of different computational modelling strategies for the analysis of low strength masonry structures. *Engineering Structures*, 73, pp.160–169. Available at: http://www.sciencedirect.com/science/article/pii/S0141029614002855.

Housner, G.W., 1963. The Behavior of Inverted Pendulum Structures During Earthquakes. *Bulletin of the Seismological Society of America*, 53(2), pp.403–417.

Konstantinidis, D. & Makris, N., 2005. Seismic response analysis of multidrum classical columns. *Earthquake Engineering & Structural Dynamics*, 34(10), pp.1243–1270. Available at: http://doi.wiley.com/10.1002/eqe.478.

Makris, N. & Vassiliou, M.F., 2013. Planar rocking response and stability analysis of an array of free-standing columns capped with a freely supported rigid beam. *Earthquake Engineering & Structural Dynamics*, 42(3), pp.431–449. Available at: http://doi.wiley.com/10.1002/eqe.2222.

Milne, J., 1881. Experiments in observational seismology. *Transactions of the Seismological Society of Japan*, 3, pp.12–64. Available at: http://repository.dl.itc.u-tokyo.ac.jp/dspace/handle/2261/25098.

Mouzakis, H.P. et al., 2002. Experimental investigation of the earthquake response of a model of a marble classical column. *Earthquake Engineering & Structural Dynamics*, 31(9), pp.1681–1698. Available at: http://doi.wiley.com/10.1002/eqe.184.

Omori, F., 1902. On the overturning and sliding of columns. *Publications of the Earthquake Investigation Committee*, 12, pp.8–27.

Omori, F., 1900. Seismic Experiments on the Fracturing and Overturning of Columns. *Publications of the Earthquake Investigation Committe in Foreign Language*, 4, pp.69–141.

Papaloizou, L. & Komodromos, P., 2009. Planar investigation of the seismic response of ancient columns and colonnades with epistyles using a custom-made software. *Soil Dynamics and Earthquake Engineering*, 29(11-12), pp.1437–1454. Available at: http://dx.doi.org/10.1016/j.soildyn.2009.06.001.

Papantonopoulos, C. et al., 2002. Numerical prediction of the earthquake response of classical columns using the distinct element method. *Earthquake Engineering & Structural Dynamics*, 31(9), pp.1699–1717. Available at: http://doi.wiley.com/10.1002/eqe.185.

Peña, F. et al., 2010. Numerical models for the seismic assessment of an old masonry tower. *Engineering Structures*, 32(5), pp.1466–1478. Available at: http://dx.doi.org/10.1016/j.engstruct.2010.01.027.

Peña, F. et al., 2007. On the dynamics of rocking motion of single rigid-block structures. *Earthquake Engineering & Structural Dynamics*, 36(15), pp.2383–2399. Available at: http://onlinelibrary.wiley.com/doi/10.1002/eqe.2230/full.

Psycharis, I.N., 1990. Dynamic behaviour of rocking two-block assemblies. *Earthquake Engineering & Structural Dynamics*, 19(4), pp.555–575. Available at: http://www.scopus.com/inward/record.url?eid=2-s2.0-0025199326&partnerID=40&md5=d18f407e249f614de1aa1d3e84d6bfe5.

Psycharis, I.N. et al., 2003. Numerical study of the seismic behaviour of a part of the Parthenon Pronaos. *Earthquake Engineering & Structural Dynamics*, 32(13), pp.2063–2084. Available at: http://doi.wiley.com/10.1002/eqe.315.

Psycharis, I.N., Papastamatiou, D.Y. & Alexandris, a. P., 2000. Parametric investigation of the stability of classical columns under harmonic and earthquake excitations. *Earthquake Engineering & Structural Dynamics*, 29(8), pp.1093–1109. Available at: http://doi.wiley.com/10.1002/1096-9845(200008)29:8<1093::AID-

EQE953>3.0.CO;2-S.

Pulatsu, B., 2015. *Simulation of the failure of masonry walls subjected to out of plane loading using the discrete element method*. MSc thesis, Universidade do Minho.

Sarhosis V, Bagi K, Lemos JV, Milani G. (2016a). Computational modelling of masonry structures using the discrete element method. USA: IGI Global.

Sarhosis V, Asteris P, Wang T, Hu W, Han Y. (2016b). On the stability of colonnade structural systems under static and dynamic loading conditions. Bulletin of Earthquake Engineering, 14(4), 1131-1152.

Sarhosis V, Lignola GP, Asteris GP. (2015a) Seismic Vulnerability of Ancient Colonnade: Two Story Colonnade of the Forum in Pompeii. In: Panagiotis G. Asteris, Vagelis Plevris, ed. Handbook of Research on Seismic Assessment and Rehabilitation of Historic Structures. Hershey, Pennsylvania: IGI Global, 2015, pp.331-358.Sarhosis, V., Garrity, S.W. & Sheng, Y., 2015. Influence of brick–mortar interface on the mechanical behaviour of low bond strength masonry brickwork lintels. *Engineering Structures*, 88, pp.1–11. Available at: http://linkinghub.elsevier.com/retrieve/pii/S014102961400755X.

Sarhosis, V., Lignola, G.P. & Asteris, P.G., 2015b. Seismic Vulnerability of Ancient Colonnade. In *Handbook of Research on Seismic Assessment and Rehabilitation of Historic Structures*. IGI Global, pp. 331–358. Available at: http://services.igi-global.com/resolvedoi/resolve.aspx?doi=10.4018/978-1-4666-8286-3.ch011.

Sarhosis, V. & Sheng, Y., 2014. Identification of material parameters for low bond strength masonry. *Engineering Structures*, 60, pp.100–110. Available at: http://linkinghub.elsevier.com/retrieve/pii/S0141029613006007.

Sarhosis, V.. et al., 2016. On the stability of colonnade structural systems under static and dynamic loading conditions. *Bulletin of Earthquake Engineering*, 14(4), pp.1131–1152. Available at: http://link.springer.com/10.1007/s10518-016-9881-z.

Sauvé, R.G. & Metzger, D.R., 1995. Advances in Dynamic Relaxation Techniques for Nonlinear Finite Element Analysis. *Journal of Pressure Vessel Technology*, 117(2), p.170. Available at: http://link.aip.org/link/JPVTAS/v117/i2/p170/s1&Agg=doi.

Stefanou, I., Psycharis, I. & Georgopoulos, I.-O., 2011. Dynamic response of reinforced masonry columns in classical monuments. *Construction and Building Materials*, 25(12), pp.4325–4337. Available at: http://dx.doi.org/10.1016/j.conbuildmat.2010.12.042.

Winkler, T., Meguro, K. & Yamazaki, F., 1995. Response of rigid body assemblies to dynamic excitation. *Earthquake Engineering & Structural Dynamics*, 24(10), pp.1389–1408. Available at: http://doi.wiley.com/10.1002/eqe.4290241008.

Yim, C.-S., Chopra, A.K. & Penzien, J., 1980. Rocking response of rigid blocks to earthquakes. *Earthquake Engineering & Structural Dynamics*, 8(6), pp.565–587. Available at: http://doi.wiley.com/10.1002/eqe.4290080606.