Structures and Buildings

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ice | proceedings

Proceedings of the Institution of Civil Engineers

http://dx.doi.org/10.1680/jstbu.16.00071
Paper 1600071
Received 04/04/2016 Accepted 13/01/2017
Keywords: columns/seismic engineering

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Institution of Civil Engineers
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Non-linear static behaviour of ancient free-standing stone columns

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This work investigated the non-linear behaviour of ancient natural stone columns in the Mediterranean region made from multiple blocks or 'drums'. A two-dimensional custom-made computational model based on the discrete-element method was employed. In the numerical model, the columns were 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 non-linear static analysis, capacity curves and corresponding failure mechanisms of each of the studied models were obtained. The influence of different parameters (the number of drums, geometrical properties and imperfections at columns) was also 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 also occur, depending on geometric characteristics and the choice of joint material properties. A higher displacement capacity was observed for columns constructed with a larger number of drums. It was also found that imperfections in the ancient free-standing columns have a significant influence on the lateral load resisting capacity. Therefore, structural analysis of undamaged columns may not represent the actual capacity of the columns due to their very sensitive and highly non-linear characteristics.

Notation

c	cohesion
f_t	tensile strength
k_n	normal stiffness
k_s	shear stiffness
Δu_n	change in normal displacement
Δu_s	change in shear displacement
$\Delta \sigma_n$	change in normal stress
$\Delta \tau_s$	change in shear stress
λ	load factor (base shear/self-weight)
ϕ	friction angle
ψ	dilation angle

1. Introduction

The eastern Mediterranean area is the richest region in the world in terms of ancient classical columns and colonnades with 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 that have occurred in earthquake-prone regions such as Italy, Greece, Turkey and Cyprus have 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. Multi-drum columns, which are composed of individual natural stone blocks lying on top of one another, generally do not have mortar or any other type of bonding material between the stone blocks. Also, the geometrical characteristics of ancient colonnades can differ considerably, although they may have the same architectural proportions or orders (e.g. Doric, Ionic and Corinthian).

Facilitation of an appropriate intervention approach for structural repair and strengthening of these historically important structures requires an improved understanding of their dynamic behaviour. Unlike modern forms of construction, historical monuments have often been exposed to seismic loads throughout their lifespan. It is thus important and useful to understand their kinematic mechanisms, which provide a great contribution to their seismic capacity. In this context, non-linear static analyses were performed as an alternative way of obtaining dynamic analyses and analytical solutions, which are not easy to perform and need much computational effort to achieve reliable results.

The motivation to analyse the response of rigid bodies dates back to the end of the nineteenth century. Research on the overturning mechanism of columns of different sizes and shapes was first published by Milne (1881). Peak ground accelerations were used to find the seismic capacity. At the beginning of the twentieth century, the complex nature and highly sensitive response of rectangular columns were studied by Omori (1900, 1902), with emphasis on the effect of input motion on the mode of collapse. Several decades later, the minimum horizontal acceleration required to overturn a rigid body and the influence of geometrical properties were examined by Housner (1963). This pioneering work was further validated and improved by Peña *et al*. (2007) and Makris and Vassiliou (2013).

Over the last three decades, researchers have paid attention to the use of advanced numerical methods to simulate the non-linear behaviour of multi-drum columns under static and seismic excitations. Yim *et al*. (1980) developed a computer program to solve the non-linear equations of motion governing the rocking response of rigid blocks. Small variations in the slenderness ratio and size of the blocks were found to result in significant changes in the response of rigid blocks. Later, Psycharis (1990) presented analytical solutions to examine the non-linear behaviour of two rigid bodies placed on top of one another. A comprehensive body of research to investigate the response of rectangular wooden blocks and block assemblies under harmonic and earthquake base excitation was reported by Winkler *et al*. (1995). To observe the response of single block and block assemblies, Alexandris *et al*. (2014) and Dimitri *et al*. (2011) performed numerical analyses using the discrete-element method (DEM), and the DEM was verified as a powerful method to analyse the stability of free-standing columns and colonnades. The efficiency of the DEM was also presented by Papantonopoulos *et al*. (2002), who compared results predicted from numerical simulations with experimental data obtained from 1:3 scale model tests of a column of the Parthenon. Parametric studies were also carried out to understand the influence of ground motion and geometrical properties on the dynamic response of ancient columns (Psycharis *et al*., 2000, 2003): it was found that the frequency content of seismic excitations had significant consequences on the response of the columns. In the light of experimental and numerical studies, proposed retrofitting solutions for multi-drum columns have been discussed by several researchers (Konstantinidis and Makris, 2005; Psycharis *et al*., 2003).

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

The structural behaviour of a 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 with mortar (Giamundo *et al*., 2014; Sarhosis and Sheng, 2014; Sarhosis *et al*., 2015a). The dynamic behaviour of multi-drum structures such as ancient columns shows three-dimensional (3D) motion with a strong non-linear character. According to Stefanou *et al*. (2011), the seismic behaviour of multi-drum columns is characterised 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, 3D numerical analyses should be better adapted to the real physics of the problem. Also, out-of-plane behaviour of a colonnade can be modelled when a 3D model is adopted. However, 2D 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*., 2016a).

This paper describes the development of a 2D computational model based on custom-made DEM software to investigate the behaviour of blocky ancient columns found in the Mediterranean region. The five columns investigated in this work were 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. Non-linear static analyses were performed on the five selected columns. The progressive contact detachments between each block in the column were captured under incremental uniform horizontal loading. The main motivation to consider non-linear static analysis was to demonstrate the deformation capacities and the lateral load resistance of existing columns. Load–deformation characteristics and inelastic responses of the columns were found by applying a uniform force distribution. In addition, geometrical parametric studies were carried out and the capacity curves and failure modes of the columns were obtained.

2. Description of the studied columns

Worldwide, there is a great variety of ancient columns with different geometrical characteristics and varying numbers of drums. Some of them are in the form of standalone columns (Figure 1), while others have an architrave on top. Five geometrically different columns were studied in this research (Figure 2).

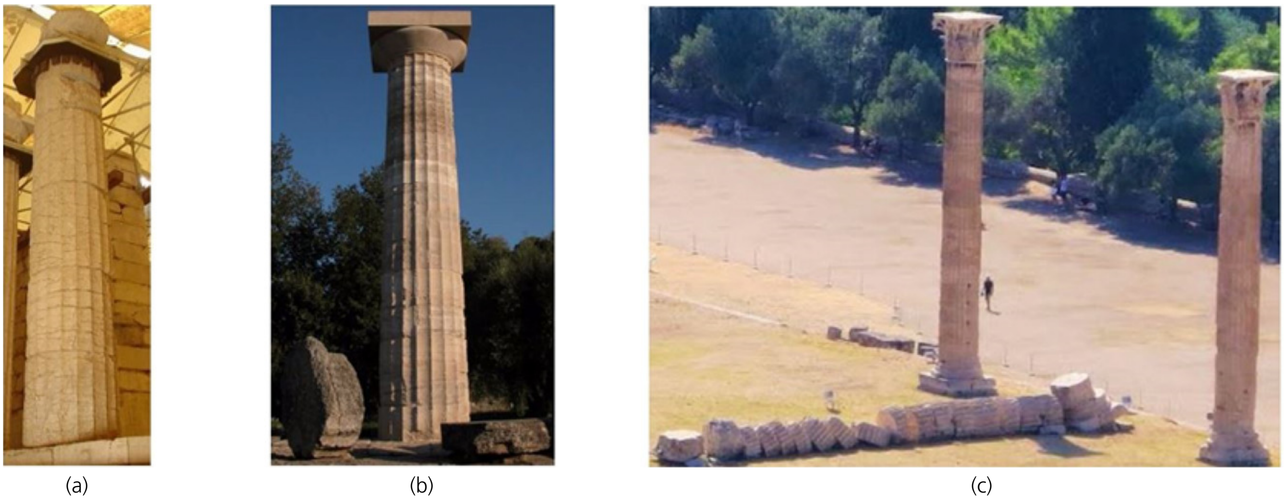

Figure 1. Typical free-standing columns investigated in this study: (a) Temple of Apollo at Bassae; (b) Temple of Zeus at Olympia; (c) Temple of Olympian Zeus (Olympieion)

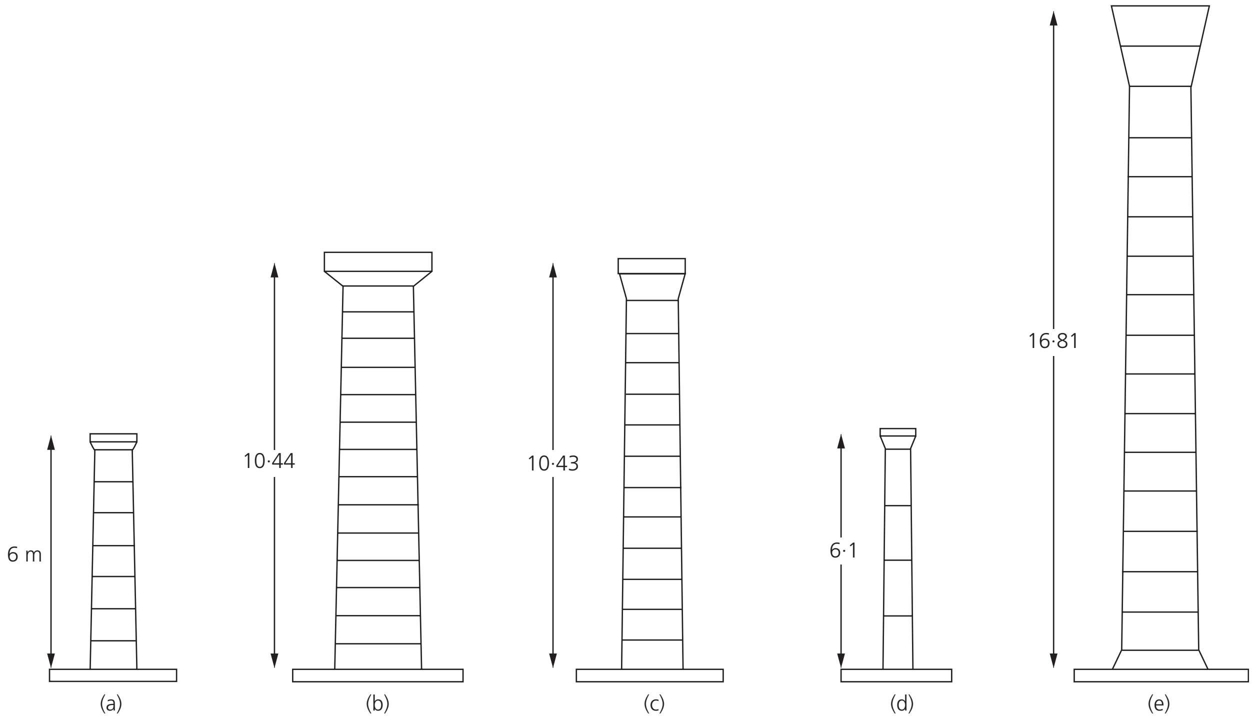

Figure 2. Geometric properties of the ancient columns under investigation (heights in m): (a) Temple of Apollo, Bassae; (b) Temple of Zeus, Olympia; (c) Parthenon *pronaos*; (d) Ancient Agora, Kos; (e) Temple of Olympian Zeus

The first ancient Doric column is from the Temple of Apollo at Bassae (Figure 2(a)), which was built in the fifth century BC. The column is 6 m high and consists of seven drums, approximately equal in size. The diameter of the base and the top drums are 1·1 m and 0·9 m, respectively. The second column (Figure 2(b)), standing at a classical Greek temple of Doric order (the Temple of Zeus at Olympia) is 10·44 m high, with base and top drum diameters approximately double those of the column at Bassae. The columns of the Temple of Zeus at Olympia consist of 14 drums. The third column is from the *pronaos* of the Parthenon (Figure 2(c)), located in the Acropolis of Athens, regarded as a symbol of the power and architectural miracles of Ancient Greece and one of the greatest cultural monuments in the world. The Parthenon column is

Table 1. Geometrical characteristics of columns

Temple	Total height, *H*: m	Base diameter, *B*: m	Top diameter, *d*: m	Number of drums (without capital)	Aspect ratio, *H*:*B*
Temple of Apollo, Bassae	6·0	1·10	0·90	7	5·36
Temple of Zeus, Olympia	10·44	2·22	1·70	14	4·70
Parthenon *pronaos*, Athens	10·43	1·65	1·25	12	6·32
Arcade of the Ancient Agora, Kos	6·10	0·78	0·64	4	7·82
Temple of Olympian Zeus (Olympieion)	16·81	2·51	1·67	17	6·70

10·43 m high, with 12 drums of the same height excluding the capital; the diameter at the base is 1·65 m, tapering to 1·25 m at the top.

A column at the Arcade of the Ancient Agora (Figure 2(d)) on the island of Kos was also studied. This Doric style 6·1 m height column is nearly the same height as the column at the Temple of Apollo at Bassae, but with a quite different aspect ratio. Consisting of four drums of the same height, the base and the top diameters are 0·78 m and 0·64 m, respectively. The last studied free-standing ancient column is from the Temple of Olympian Zeus, also known as the Olympieion, situated in Athens (Figure 2(e)). This monument is considerably larger than other columns, being 16·81 m in height and with a base diameter of 2·51 m. The geometrical properties of each column are listed in Table 1.

3. Overview of the DEM for modelling blocky structures

3.1 General aspects

Custom-made software (Bretas *et al.*, 2014, 2015) based on the DEM (Cundall, 1971) was used in this study. This software was initially developed to solve structural and hydraulic problems of masonry dams and was later employed to simulate the out-of-plane behaviour of masonry walls (Pulatsu, 2015). Through this research, the application field of the newly developed software was further extended to understand the static behaviour of historical columns. Within DEM, individual blocks can be considered as rigid or deformable. 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 the 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 that 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 when they occur in the finite-element method (FEM). However, unlike the FEM, in the DEM, a compatible finite-element mesh between the blocks is not required.

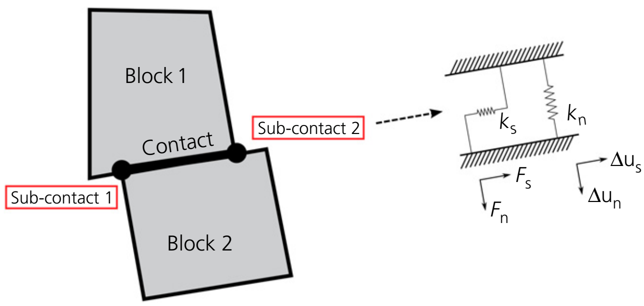

Figure 3. Face-to-face contact type and corresponding sub-contacts where springs are assigned in both orthogonal directions

Mortar joints are represented as zero-thickness interfaces between the blocks. Representation of the contact between blocks is not based on joint elements, as it occurs in the discontinuum FEM. 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 the custom-made software, however, the fundamental contact type is face-to-face (Bretas *et al.*, 2014), which is composed of two sub-contacts (Figure 3). This 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

1. $$\Delta\sigma_n = k_n \cdot \Delta u_n$$

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

2. $$\Delta\tau_s = k_s \cdot \Delta u_s$$

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.

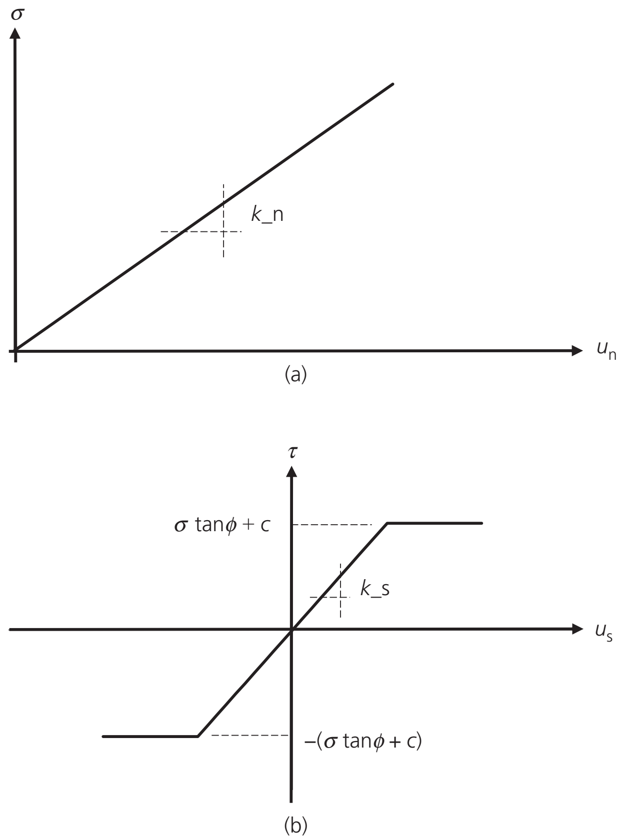

Figure 4. Joint behaviour under (a) normal load and (b) shear load

Stresses calculated at grid points along contacts are submitted to the Coulomb failure criterion, which limits shear stresses along joints (Figure 4). The parameters normal stiffness (k_n), shear stiffness (k_s), friction angle (ϕ), cohesion (c), tensile strength (f_t) and dilation angle (ψ) are used to define the mechanical behaviour of the contacts.

3.2 Validation study

Validation of the custom-made software was conducted 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 the tower, were compared with different numerical analysis approaches such as the FEM and the rigid element method (REM), which were comprehensively studied by Peña *et al.* (2010). Non-linear 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. Capacity curves in terms of lateral displacement versus load factor λ (base shear/self-weight) were generated. Although the results of the DEM and REM were found to be very close, for the FEM a difference of approximately 25–30% was observed in terms of the maximum load leading to failure and corresponding displacement capacity, as shown in Figure 5.

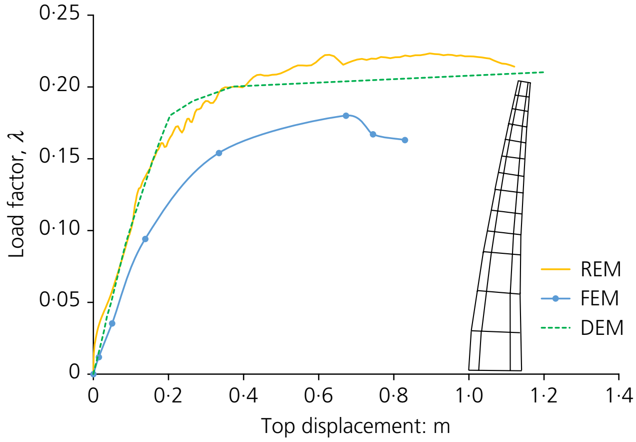

Figure 5. Capacity curve of Qutb Minar (New Delhi, India) from the custom-made DEM software and other numerical approaches (Pulatsu, 2015)

However, the same collapse mechanism, namely overturning failure, was obtained for all the different numerical models. Therefore, the results of the custom-made software were validated on the existing masonry tower and good agreement was obtained with other numerical approaches (Pulatsu, 2015).

4. Material properties, boundary conditions and application of load

The material properties used in numerical models are important for an accurate prediction of the lateral behaviour of structures subjected to external loads. Since intrusive tests on archaeological structures are not permitted in most cases, material properties for the stone blocks and joints were obtained from previous small-scale laboratory works and related experimental studies (Drosos and Anastasopoulos, 2014; Papantonopoulos *et al.*, 2002). The material parameters used for the development of the numerical models are shown in Table 2. Since the columns are mortarless (dry-stacked) block masonry systems, the joint tensile strength and joint cohesive strength were assumed to be zero. The joint dilation angle was also assumed to be zero. In the normal direction, a relatively high compression strength was assigned to the numerical models, 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 2400 kg/m^3, according to Drosos and Anastasopoulos (2014). All the columns were assumed to sit on a rigid base and can move in both horizontal and vertical directions.

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 load). Then, a 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 (non-linear pushover analysis) on each block, as presented in Figure 6(a). 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 and Metzger, 1995). In addition, horizontal displacements at the upper part of each drum of the colonnade were recorded at each loading step (Figure 6(b)), giving rise to the capacity curves. The results from non-linear pushover analysis of existing columns were compared with different monolithic or multi-drum conditions in terms of displacement capacity and failure mechanism.

Table 2. Properties of the joint interfaces

Normal stiffness, k_n: GPa/m	Shear stiffness, K_s: GPa/m	Joint friction angle, ϕ: degrees
1	1	37

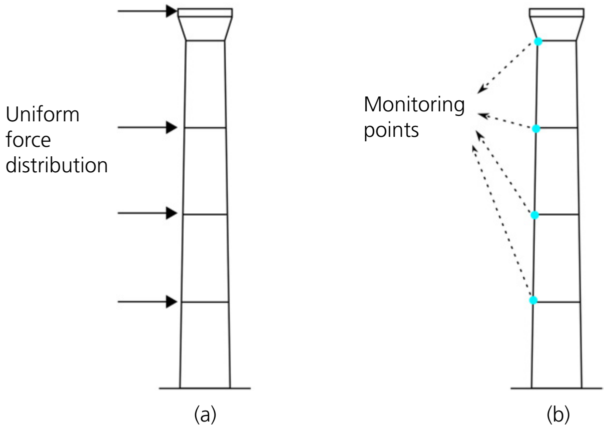

Figure 6. (a) Applied force pattern. (b) Points where displacements were recorded

5. Capacity curves

The obtained capacity curves for the five ancient columns studied 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. The column of the Ancient Agora is able to carry the lowest load (16 kN) and has the highest aspect ratio (7·82) of the columns analysed. Furthermore, although the column of the Temple of Zeus in Olympia and the column of the Parthenon *pronaos* are very nearly identical in height, their lateral load–deformation behaviour was found to be dissimilar. Thus other geometrical properties such as the base diameter of the column and the number of drums must affect the capacity and behaviour of these historical colonnades, as represented by the capacity curves for the five different standalone columns (Figure 7).

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 Parthenon column. The obtained capacity curve comprises three phases, similar to the other columns investigated in this work.

(*a*) 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. The deformation is a function of contact stiffness.
(*b*) In the second phase, with increasing load, contacts between drums detach as degradation under uniform loading occurs, with a clear loss of global stiffness.
(*c*) Finally, in the third phase, the column fails as a result of excessive shear sliding and/or overturning. Once shearing or opening of the drums has occurred, the sequence of

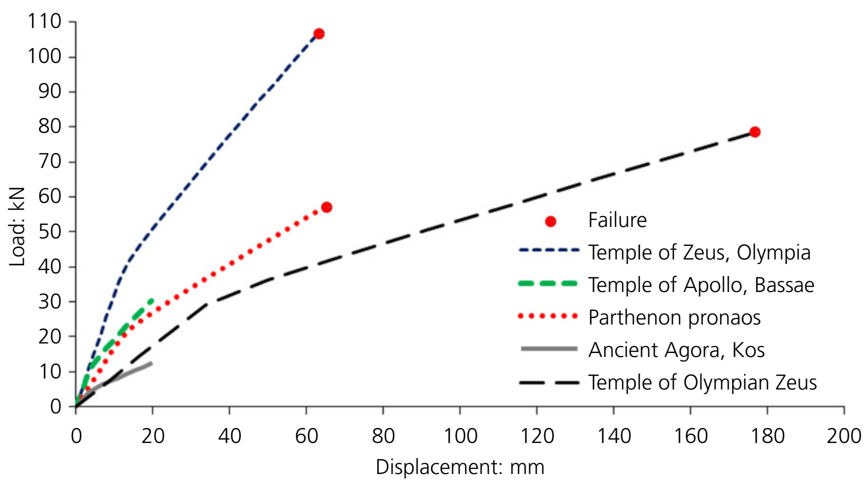

Figure 7. Capacity curves of the five different ancient columns investigated in this study

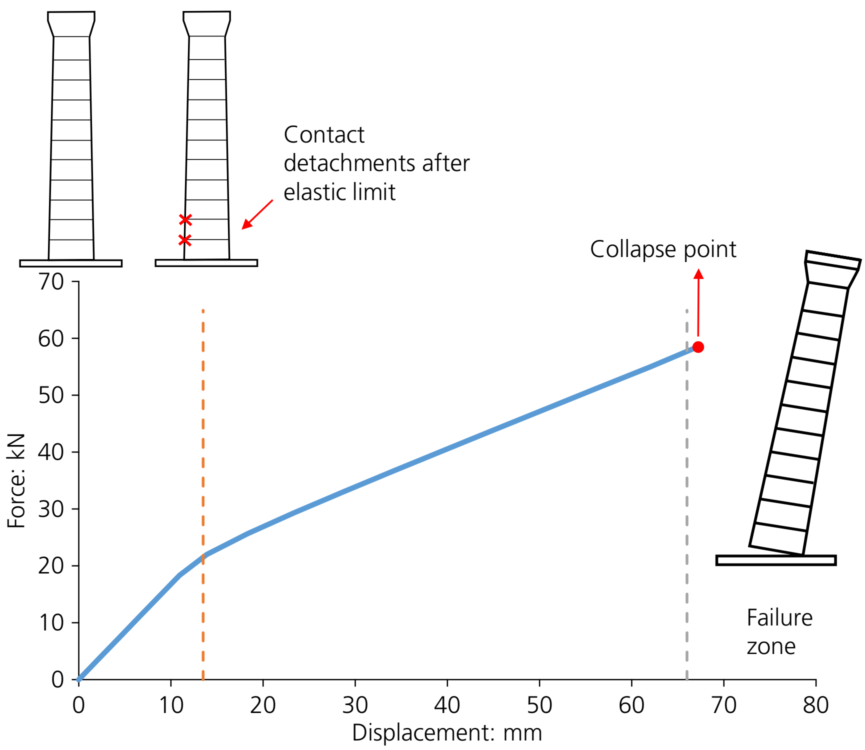

Figure 8. Capacity curve of the column of the Parthenon *pronaos*

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

Temple	Minimum horizontal load to exceed elastic response (load at first damage): kN	Maximum load leading to failure: kN
Temple of Apollo, Bassae	12·1	30·3
Temple of Zeus, Olympia	38·4	108·9
Parthenon *pronaos*, Athens	23·8	58·2
Arcade of the Ancient Agora, Kos	4·7	13·5
Temple of Olympian Zeus (Olympieion)	32·6	78·9

events leading to collapse can be very quick with little warning of the 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).

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

Furthermore, it was noticed that there is a certain displacement limit before the first detachment between stone units; this is around 15–25% of the total displacement capacity. For instance, in case of the Parthenon column, the maximum elastic displacement was 13·5 mm while the total displacement capacity, obtained at the end of the pushover analysis, was around 66 mm (Figure 8).

The capacity curves and deformed shapes were further investigated to understand the effects of geometrical parameters on the failure modes of the columns. The contact points of the DEMs were monitored through each loading step to understand the contact conditions of the drums during the pushover analysis. The contact conditions are especially important for DEMs to understand the behaviour of a structure since force transmission occurs within the contact points. As a result, the instant contact conditions (e.g. sliding and opening) were

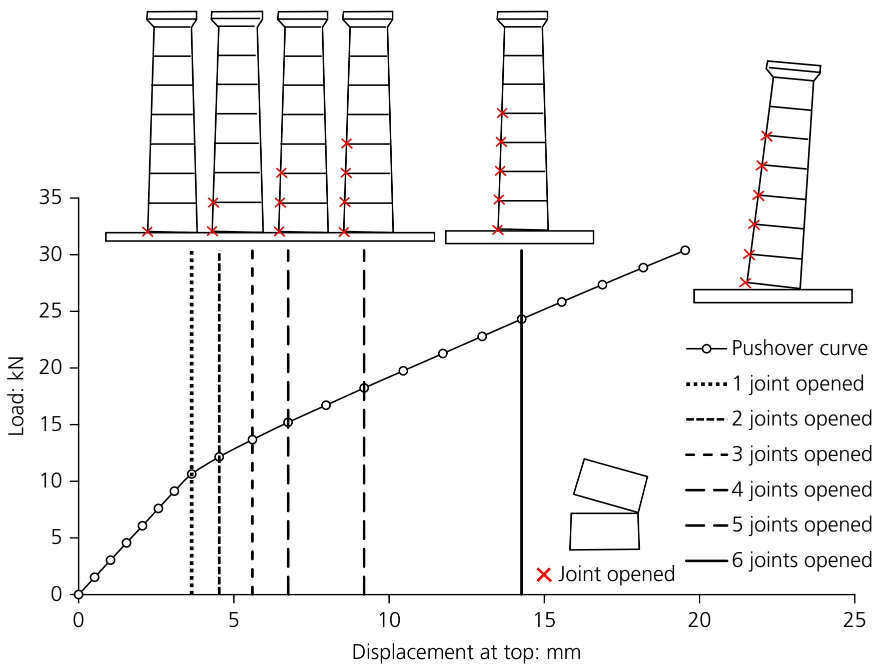

Figure 9. Number of open joints of the column of the Temple of Apollo at Bassae through pushover analysis

captured through the analyses. The main action at the contacts was observed to be contact opening. The contact detachments of the column of the Temple of Apollo at Bassae under lateral loading are indicated in Figure 9 with cross symbols. 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. As shown in Figure 9, the first contact detachment occurred at the bottom drum then detachments went through the height of the column sequentially until the maximum displacement capacity was reached.

6. Parametric studies

6.1 Influence of 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 was varied from monolithic to 4, 8 and 12 drums. An example of the geometric parametric study for the column of the Arcade of the Ancient Agora is presented in Figure 10.

Figure 11 shows the capacity curves for each column with different numbers of drums. The results show that the number of drums has a significant effect on the capacity curves. For each of the columns studied, the column develops a larger displacement capacity as the number of drums was increased. Furthermore, columns with 12 drums have a displacement capacity 2·5 to 4 times higher than their monolithic forms, given the fact that joints have some elastic deformability and this extends to the non-linear range. However, the number of drums does not have any noticeable influence on the ultimate strength of the columns, as indicated in Figure 11.

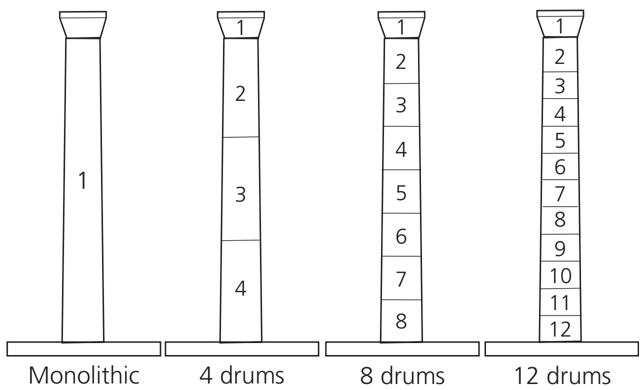

Figure 10. Geometries of the column of the Arcade of the Ancient Agora in Kos used in the sensitivity study

Figure 12 shows the deflected shapes of the columns of the Ancient Agora in Kos and the 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 the investigated columns exhibited less brittle behaviour and higher deformability with a larger number of drums (Figure 12).

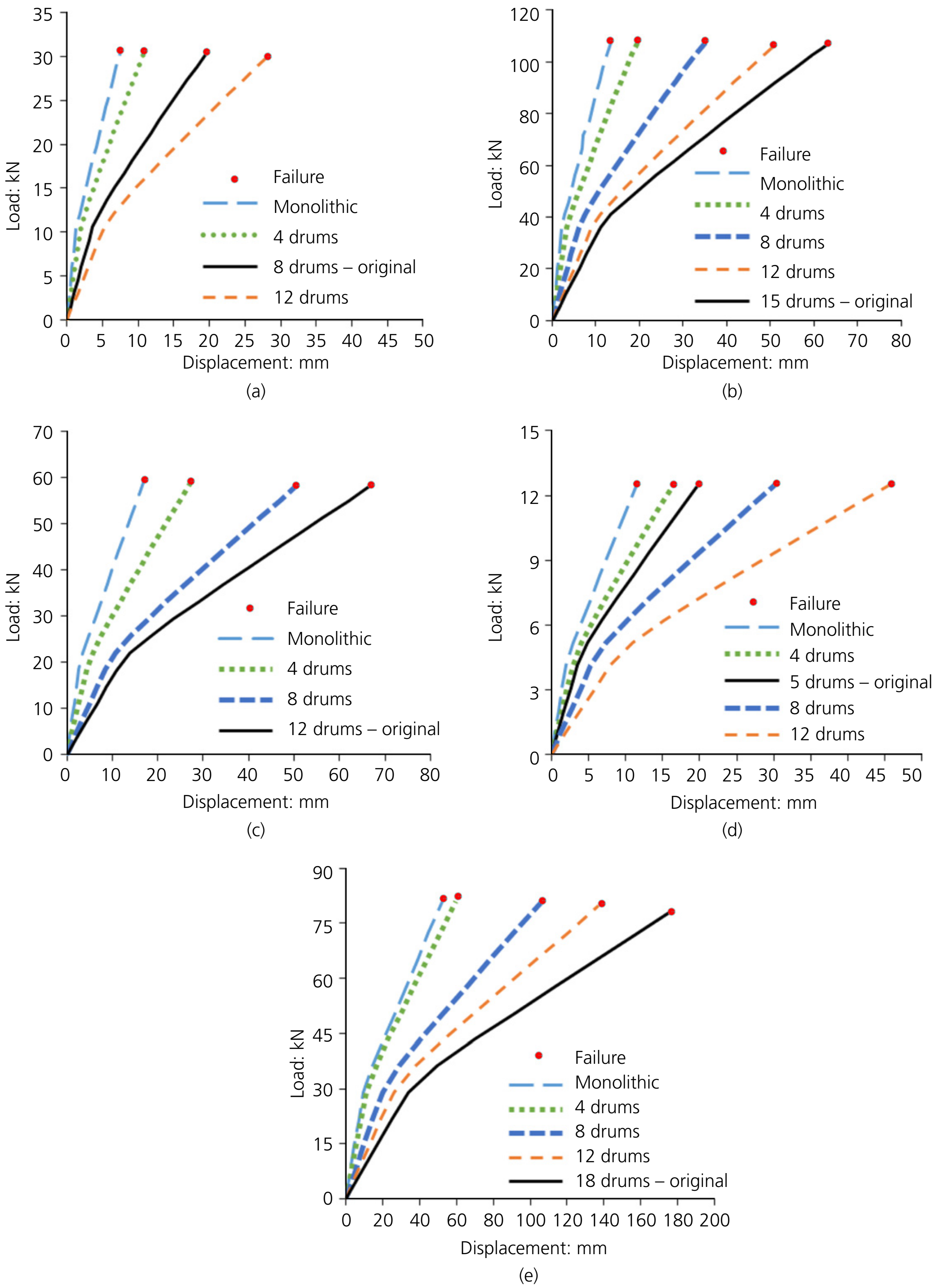

Figure 11. Capacity curves, representing the influence of the number of drums: (a) Temple of Apollo at Bassae; (b) Temple of Zeus, Olympia; (c) Parthenon *pronaos*; (d) Ancient Agora in Kos; (e) Temple of Olympian Zeus (Olympieion)

6.2 Influence of drum imperfections

Over the years, strong earthquakes, stone deteriorations, vandalism and inappropriate intervention techniques have led to geometrical imperfections of ancient columns and it is almost impossible to categorise these imperfections due to the unique characteristics of each structure. Different scenarios were thus

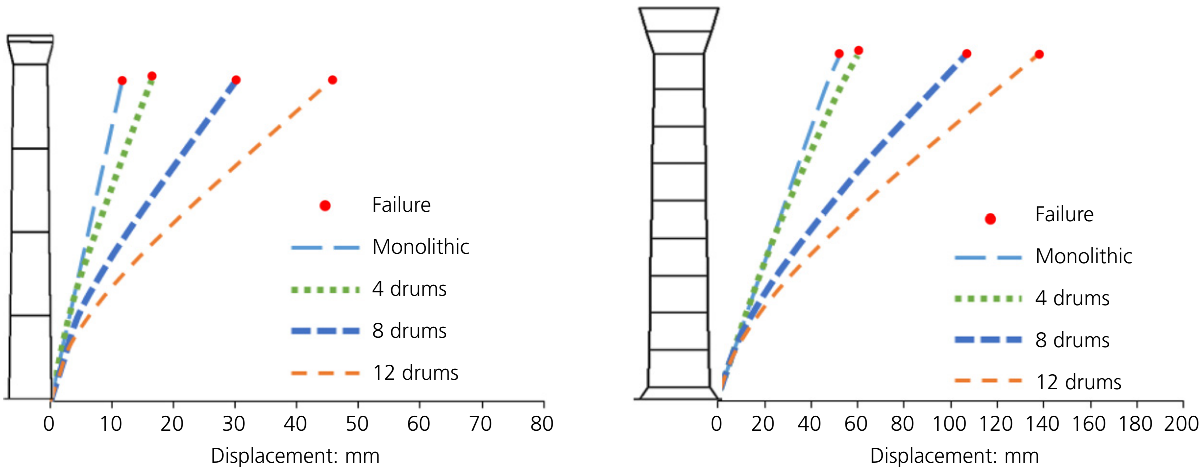

Figure 12. Deflected shapes of ancient columns depending on the number of drums: (a) Ancient Agora in Kos; (b) Temple of Olympian Zeus (Olympieion)

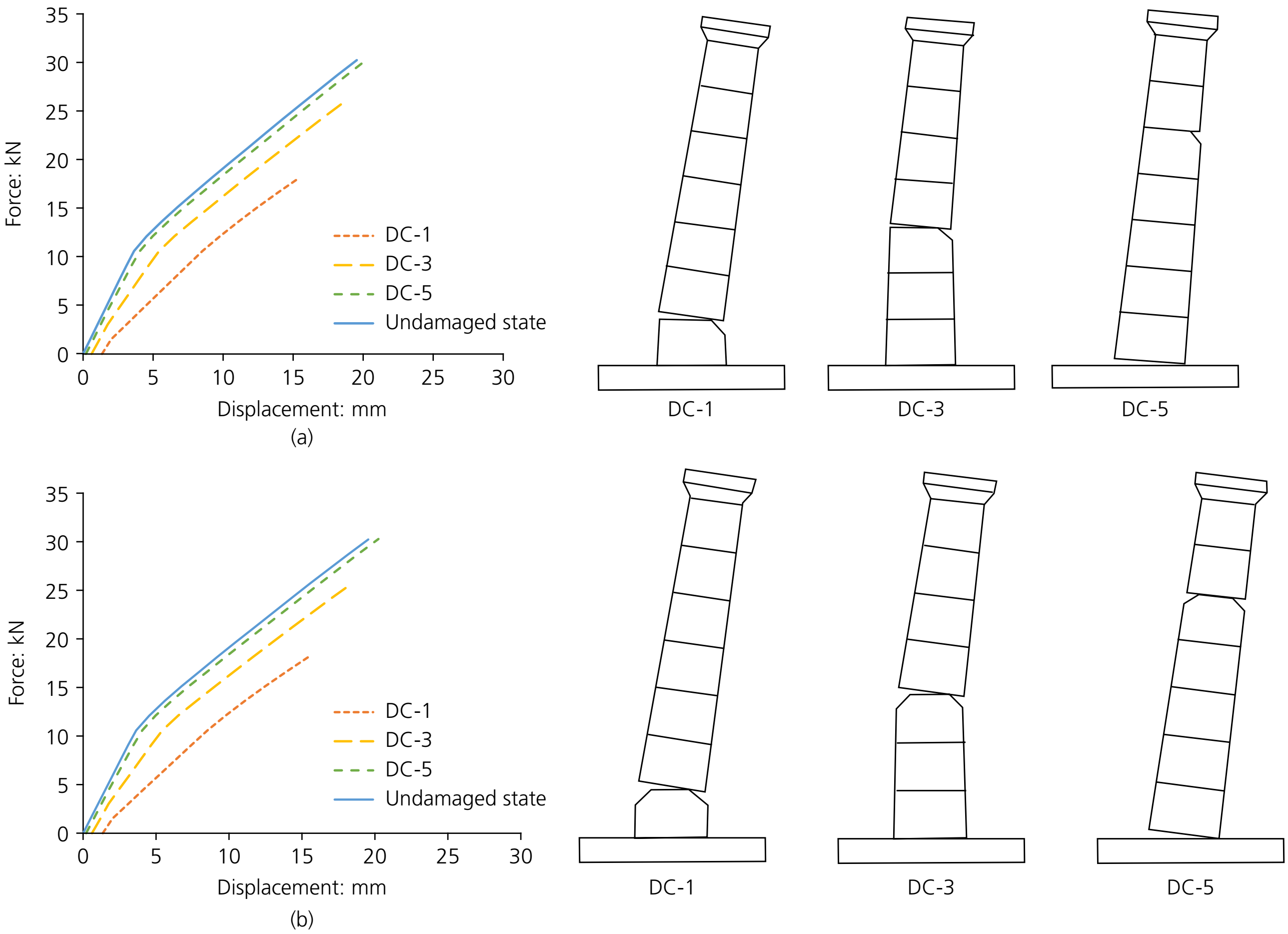

Figure 13. Scenarios of different imperfections for the column of the Temple of Apollo at Bassae and corresponding failure and capacity curves (DC = damage condition): (a) an imperfection at one edge of the drum; (b) imperfections at both edges of the drum

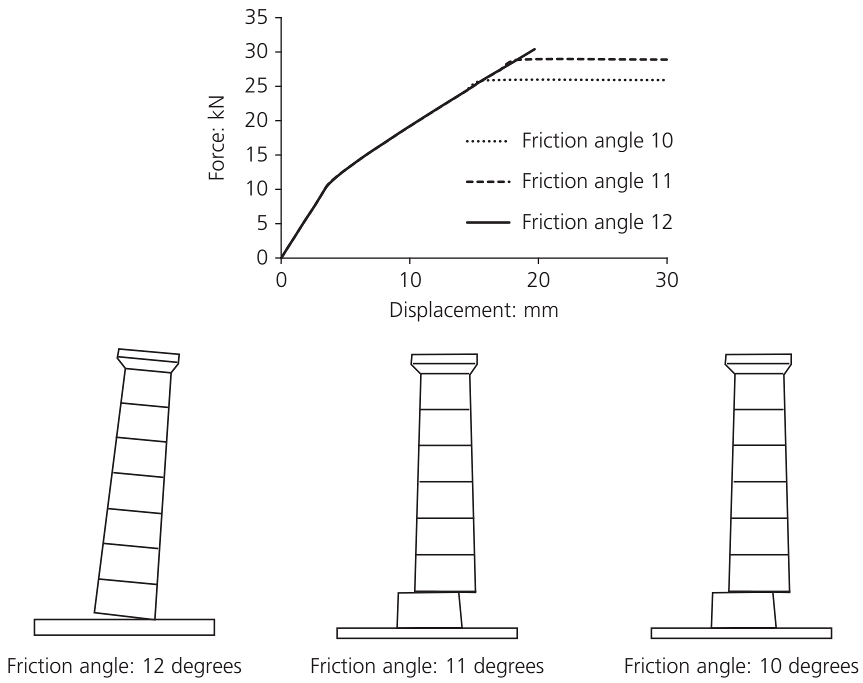

Figure 14. Influence of friction angle on collapse mechanism and capacity curve of the column of the Temple of Apollo at Bassae

prepared for the column of the Temple of Apollo at Bassae to demonstrate the influence of imperfections on the load-carrying capacity and failure mode. The type and location of imperfections (in this case localised at the corners) have significant consequences on the ancient columns in terms of the maximum displacement capacities and failure mechanisms. Figure 13 shows that the location of imperfections in the drums has a remarkable influence on both the load-carrying capacity and the failure mode of free-standing columns.

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

6.3 Influence of the friction angle between drums

The type of rock used for the construction of ancient columns varies and thus 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 the coefficient of the friction. There may also be cases where ancient columns have been repaired and old drums replaced. Joint degradation effects and/or water ingress between the drums of a column may also be present. These conditions can result in different coefficients of friction even between drums in the same column. A parametric study of the influence of friction angle on the pushover response of the columns under investigation was thus carried out. The friction angle between drums varies from 10° to 40° (Dimitri *et al.*, 2011; Sarhosis *et al.*, 2016b). As shown in Figure 14, the friction angle was found to have some influence on the collapse mechanism and ultimate load-carrying capacity. Lower joint friction angles lead to sliding under uniform horizontal loading, whereas higher friction angles lead to overturning failure.

7. Conclusions

A 2D computational model was developed based on a custom-made discrete-element method to investigate the static non-linear behaviour of blocky ancient columns in the Mediterranean region. The capability to simulate such complex systems of multi-drum columns is crucial to better understand how ancient monuments have experienced and survived strong earthquakes throughout the centuries. Five ancient columns with different geometries consisting of stone drums positioned one over the other were examined. In the numerical model, the columns were represented as an assemblage of distinct blocks connected together by zero-thickness interfaces that could open and/or close depending on the magnitude and direction of the stresses applied to them. Through non-linear static analysis of the models, capacity curves and corresponding failure mechanisms were obtained. Rigid overturning was found to be the governing failure under a uniformly distributed load. The general kinematic mechanism was found to start with small openings at the contact points due to lack of tensile capacity and end with an overturning mechanism.

A sensitivity study was carried out 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. Columns with more drums developed higher deformation capacities than monolithic columns, which showed more brittle failure.

It is recommended that, in order to explore the seismic response of ancient columns, their current state should be taken into account. Otherwise, depending on the level of existing damage, the results may not represent the real behaviour and capacity of the columns. A sensitivity study on the influence of the friction angle of the drum to drum interface was also carried out. The analyses indicated that lower coefficients of friction increase the dominance of sliding between drums.

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