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On the stability of colonnade structural systems under static and dynamic loading conditions

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Abstract

The structural behavior of colonnade structural systems subjected to static and dynamic loading is investigated to identify the main factors affecting the stability and to improve our understanding of their behaviour. In particular, the discrete element method (DEM) of analysis is utilised to study the static and dynamic behaviour of a typical section of the two storey colonnade of the Forum in Pompeii. Static analysis indicated that the failure of colonnade structures occur at higher friction angles as the weight above the structure decreases and so a sudden collapse can occur when parts of the monument are disassembled. For the dynamic analysis, the mechanical behavior of the colonnade was investigated for both harmonic and real seismic excitations. For excitations with relatively low dominant frequencies, the primary response is rocking; as the excitation frequency increases, the response becomes more complicated demonstrating both sliding and rocking movements. It was also shown that the construction methods used in ancient times, such as multi-block segmented trabeations and solid block beam, have quite significant impact on the mechanical response of the structures under static and dynamic loading.

Keywords: Ancient monuments; Colonnade structures; Distinct Element Method; Epistyle; Harmonic response; Earthquake response.

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1. **Introduction**

High-intensity earthquake events have caused the destruction and/or damage to several ancient monuments, such as classical columns and colonnades with great archaeological significance. Examples of such damaged structures can be found in high seismicity areas including Greece and Italy (Croci 1998a; Croci 1998b; Macchi, G., 1998). In general, classical monuments which are in their intact condition are not vulnerable to earthquake motions (Papantonopoulos 2002). However, if imperfections (e.g. inclined columns due to foundation failure; damaged drum corners; missing structural components; misplaced drums etc) are present in the structure, then collapse of the monument can occur.

Understanding the seismic behavior of ancient monuments is necessary for the proper selection of rehabilitation and strengthening techniques. Also, as ancient monuments have been exposed to a large number of strong seismic events throughout the years; those survived have successfully passed a natural seismic test which has been extended over several centuries. Therefore, it is very useful to understand the mechanisms that have allowed the surviving monuments to avoid structural collapse and destruction during strong earthquakes. Finally, the study of the earthquake response of classical monuments may reveal important information about past earthquakes which had struck the respective region. For example, a detailed analysis could provide information on the characteristics of past earthquakes, and this information can be used to "re-engineer" the process and bring the monument to its previous condition (before the deformation).

In most cases, classical columns have been constructed by carefully fitted stone drums (usually marble or limestone) placed on top of each other, without connecting material between them. The seismic behavior of these structures is highly non-linear and complex since both rocking and sliding can occur. In fact, the drums may rock either individually or in groups resulting in several different shapes of oscillations. As a result, the analytical investigation of their response is almost impossible. In addition, laboratory tests of large scale models (Papalou et al. 2015) are difficult and costly to perform (Omori 1900; Omori 1902). There are several numerical methods available for the analysis of the seismic behavior of historic masonry constructions (Beskos 1993). The most suitable method, among other factors, depends on the structure under analysis, the available input data, the computational cost, and the analyst experience (Lourenco 2002). It should also be expected that different methods of analysis will lead to different results. According to Lourenço (2001), a more complex analysis tool will not necessarily provide better results than a more simplified tool. Also, most refined theories require the knowledge of mechanical and geometrical parameters. However, it is in general not straightforward to idealize the geometry of historic masonry structures because there is no clear distinction between decorative and structural elements. In any case, there should be a balance between the knowledge level and the complexity of the analysis. Studies carried out by Stefanou et al. (2011b) showed that the presence of imperfections, such as corners cut-off, cracks, geometrical imperfections etc, which are common in ancient structures, have a significant influence on the mechanical behavior and load bearing capacity of the structure. Analytical solutions on the dynamic behavior of infinitely rigid bodies during horizontal excitations was studied by Housner (1963), who estimated the minimum horizontal acceleration of the support base that is required in order to overturn an infinitely rigid body. Later on, other researchers (Makris & Zhang 2001; Manos et al. 2001; Mitsopoulou & Paschalidis 2001; Pompei et al. 1998) used both experimental tests and advanced Finite Element Methods of analysis to investigate further the required conditions to overturn rigid bodies. Although the FEM can be used for the analysis of problems with some discontinuities (de Martino et al. 2006), it is not suitable for the analysis of discontinuous systems that are characterized by continuous changes of the contact conditions among the constituent bodies. On the contrary, Discrete Element Methods (DEM) have been specifically developed for systems with distinct bodies that can move freely in space and interact with each other with contact forces, providing an automatic and efficient recognition of all contacts. Recent research work based on commercial DEM software applications (Alexandris et al. 2001; Lemos 2007; Psycharis et al. 2000; Psycharis et al. 2003; Papantonopoulos 2001; Papantonopoulos et al. 2002; Sarhosis 2012; Sarhosis 2014; Sarhosis 2015), demonstrated that the DEM can be effectively used for the analysis of masonry structures where failure at the joint is the predominant mode of collapse. DEM is well suited for the collapse analysis of historic stone masonry structures under earthquakes since:

a) large displacements and rotations between blocks, including their complete detachment, can be simulated;
b) contacts between blocks are automatically detected and updated as block motion occurs;
c) it is able to simulate progressive failure associated with crack propagation; and
d) interlocking can be overcome by rounding the corners.

Today, many computational tools based on the distinct element method are available for the seismic analysis of masonry structures (Giordano et al 2002; Komodromos et al. 2008; Papaloizou and Komodromos 2009; Sasaki et al 2011). However, UDEC developed by Itasca is the most widely applied DEM code. Extensive and in-depth state-of-the-art reports on the modelling of masonry structures can be found in Lourenço (2002), Asteris et al. (2003), Lourenço et al. (2007), Asteris et al. (2012), Sarhosis (2012) and Asteris et al. (2014).

Also, the dynamic behavior of multi-drum structures such as ancient colonnades does not involve only sliding and rocking, but also wobbling, which is a three-dimensional motion with a strong non-linear character (Stefanou et al. 2011a). Due to wobbling the dissipation of energy is different during seismic excitation, which affects stability and deformation of the structure. Therefore, three dimensional DEM analyses should be better adapted to the real physics of the problem. Also, the out of plane behavior of the colonnade can be modeled when a three dimensional model is adopted. However, studies carried out by Psycharis et al. (2000) and Konstantinidis and Makris (2005) showed that 2D analyses can still be used to capture the overall phenomenon and various parameters that affect the dynamic response of multi-drum columns. More-over, 2D can be used more efficiently and effectively when it is necessary to perform large numbers of simulations to study the effect of various parameters and characteristics, as 2D analysis is much more time efficient and is less sensitive to the contact parameters.

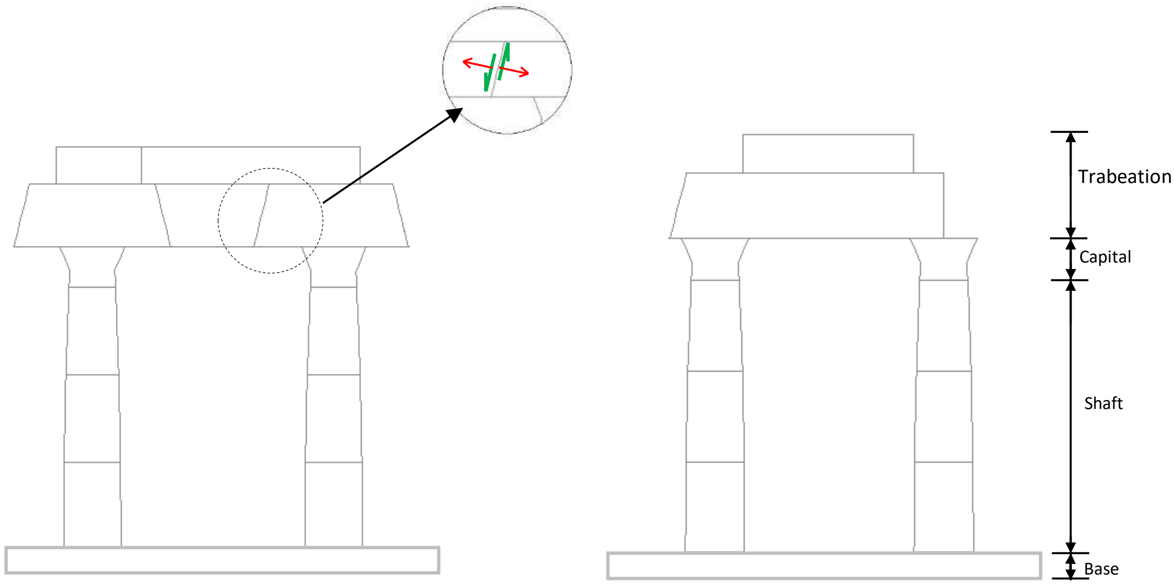
This paper describes the development of a two dimensional computational model using the UDEC software to investigate the seismic vulnerability of a block based frame of architectural heritage at the ancient city of Pompeii in Italy. The structure under investigation is a two series system colonnade consisting of multi-drum columns positioned one over the other. Very few have examined colonnade systems with two rows of columns (Papaloizou and Komodromos 2012), where the lower level columns supported a series of solid beams as it is commonly found in ancient Greek temples such as the temple of Aphaia in Aegina. Conversely, the case study in Pompeii (Italy) is a two series system colonnade where the lower level columns support a series of both segmented and solid beams. Computational studies presented herein, checked both the static condition under the self-weight of the stone blocks and the seismic behaviour of the two storey colonnade. For the static analysis, a sensitivity study on the frictional resistance of the interface has been undertaken. This was to simulate the presence of joint degradation effects and possible water lubrication at the joint. Also, to evaluate the seismic performance of the historic structure, a fully non-linear analysis was conducted. The colonnade 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.

2. Description of the monument

The structure under investigation (Figure 1) is a two storey colonnade of the Forum in Pompeii made of white limestone. The colonnade was erected in the main square of the ancient town where political, economic and religious events were taking place. To prevent the passage of the carts, the pave was raised with respect to the height of the adjacent roads by two steps. It is believed that originally (VII-VI century B.C.) the Forum had an irregular shape. Today, the Forum has a rectangular shape with dimensions 143 m long by 38 m wide. Probably, at the end of the II century B.C., a double row colonnade was built which was made of tuff. Some years later in 79 A.D the colonnade was reconstructed and the tuff was replaced with white limestone. The columns of the second storey followed the Ionic order while the columns of the lower storey followed the Doric order. In the second half of the 20th century, only a small part of the second storey was re-erected for educational and touristic purposes. In 1980, an earthquake produced damages in the colonnades of the Forum and it was decided to remove the beam over the second storey.

Figure 1. Two storeys colonnade with multi-drum columns and multi-blocks segmented trabeations

An "innovative solution" was adopted for the construction of the trabeation. To avoid long span beams over the columns, short segments were built up providing opposing inclined patterned edges "flat arch" (Figure 2a). This solution was conceived to simplify construction phases and prevent the lifting of long span heavy beams over the columns. Blocks mutually supporting each other over inclined surfaces (keystone) induced horizontal thrust in the horizontal structures to carry loads without any tensile strength. In a fully functioning structure, each keystone pushes over the two adjacent blocks and this load is counteracted. A stability issue may arise at the corners of the structure, where there is no symmetric mutual interaction and could lead to the overturning of the column. To overcome the above hurdle, the builders avoided the reduced size blocks at the end of the colonnade placing a long block (solid long beam in Figure 2b), above the first two extremity columns. In that way, the horizontal thrust, was counteracted by two columns halving the horizontal thrust. The examination of the methods employed by the ancient builders revealed the continuous research and evolution in design of structures against earthquakes (Adam 1989; Maiuri 1942).



(a) Flat arch (segmented beam) (b) Solid long beam

Figure 2. Different construction methods at the trabeation of the Forum in Pompeii (Sarhosis et al. 2015)

3 Methodology

3.1 Discrete element method

The discrete element method (DEM) falls within the general classification of discontinuum analysis techniques. It is presented in the UDEC (Universal Distinct Element Code) software, developed by Cundall in the early 1970s for numerical research into the sliding of earth and rock masses. Since then, the software has been used for a range of applications including the modelling of classical columns under static and dynamic loading conditions. In UDEC, masonry units are represented as an assembly of rigid or deformable blocks which may take any arbitrary geometry. Rigid blocks do not change their geometry as a result of any applied loading. Deformable blocks are internally discretised into finite difference triangular zones of uniform stress and strain characteristics. These zones 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 and the joints is not required. Mortar joints are represented as zero thickness interfaces between the blocks. Representation of the contact between blocks is not based on joint elements, as occurs in the discontinuum finite element models. Instead the contact is represented by a set of point contacts with no attempt to obtain a continuous stress distribution through the contact surface. The assignment of contacts allows the interface constitutive relations to be formulated in terms of the stresses and relative displacements across the joint. Convergence to static solutions is obtained by means of adaptive damping, as in the classical dynamic relaxation methods.

3.2 *Block representation*

When defining the model in UDEC, a single block covering the region to be analyzed is considered. The model features are then introduced by cutting the block into smaller ones (e.g. block stones) whose boundaries represent discontinuities (e.g. joints). Blocks can take any arbitrary geometry and may be different from each other in size and shape. Individual blocks can be considered as rigid or deformable. Rigid blocks are mainly used when the behavior of the system is dominated by the joints. Alternatively, the blocks can be modelled as deformable and the complexity of the deformation of the blocks depends on the number of zone elements into which they are divided. Zones obey the constitutive model assigned to them and, for each separate block, the strain can be estimated. In UDEC, the deformable block zones can be assumed to be linear elastic or non-linear according to the Mohr-Coulomb criterion (ITASCA, 2004). The material parameters for the linear, isotropic, elastic model are the unit weight of the block (d), the compression modulus (K) and the shear modulus (G).

3.3 *Joint interface representation*

In UDEC joints are represented by zero thickness interfaces between adjacent blocks. 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. This approach is known as the contact hypothesis method (Cundall and Hart, 1989).

For each contact point 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. (3):

$$\Delta\sigma_n = - JKn \cdot \Delta u_n \qquad (1)$$

where JKn 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 JKs using the following expression, Eq. (4):

$$\Delta\tau_s = - JKs \cdot \Delta u_s \qquad (2)$$

where $\Delta\tau_s$ 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 Mohr-Coulomb failure criterion which limits shear stresses along joints. The following parameters are used to define the mechanical behaviour of the contacts: the normal stiffness (JKn), the shear stiffness (JKs), the friction angle (Jfric), the cohesion (Jcoh), the tensile strength (Jten) and the dilation angle (Jdil).

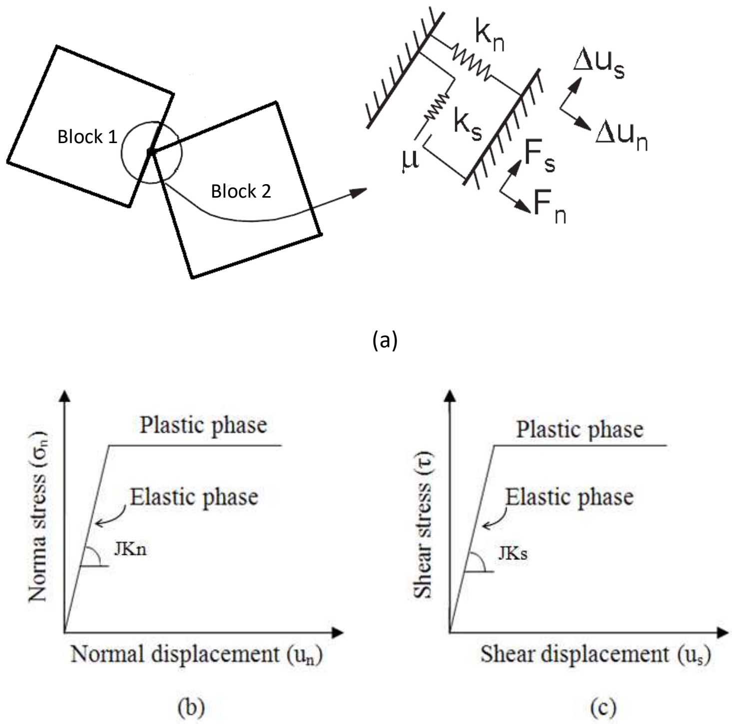


Figure 3. Mechanical representation of the contact between blocks (a); and joint behaviour under normal (b) and shear (c) loads

4 Development of the computational model

4.1 *Model geometry*

Geometric models representing the two storey colonnade of the Forum in Pompeii were created in UDEC (Figure 4). Each stone of the monument (i.e. drum of the column and stone block of the trabeation) was represented by a deformable block separated by zero thickness interfaces at each joint. Also, each block was internally discretized by UDEC into finite-difference zone elements, each assumed to behave in a linear elastic manner, characterized by elastic density and elastic properties. In practice, the stresses in the stone blocks would be well below their strength limit and so no significant deformation would be expected to occur in them. To replicate this, stone material parameters were specified to ensure that no block deformation would occur; yet the software was enabled to calculate the theoretical stresses in each zone element. The zero thickness interfaces between each block were modelled using UDEC's elastic perfectly plastic Coulomb criterion defined by: the elastic normal stiffness (JKn); the shear stiffness (JKs); and the joint angle of friction (Jfric).

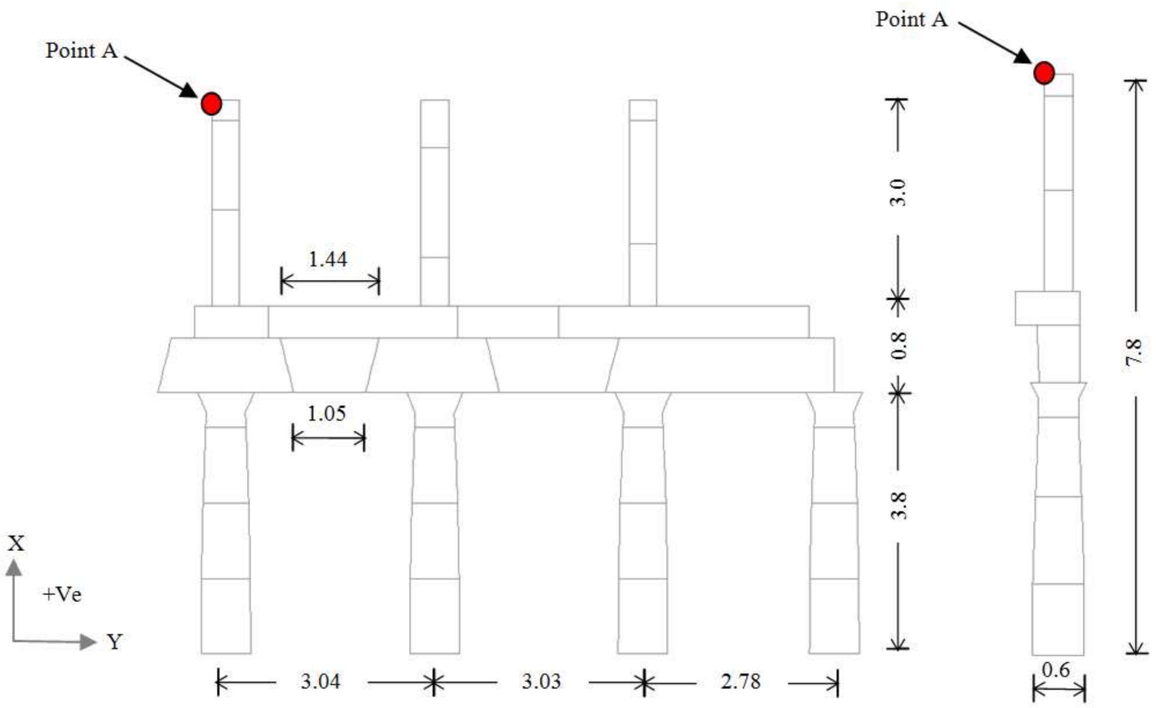


Figure 4. Geometry of the two storey colonnade of the Forum of Pompeii (Sarhosis et al. 2015)

4.2 *Material Properties*

The material properties of the numerical model are important for the accurate prediction of the mechanical behaviour of structures subjected to external loads. Since mechanical tests on archaeological structures are not permitted, material properties for the stone blocks and joints were obtained from experimental tests carried out on similar stones and joints (Kastenmeier et al. 2010 and Angrisani et al. 2010). The material parameters used for the development of the computational model are shown in Tables 1 and 2. Since the colonnade of the Forum is a mortarless (dry-stacked) block masonry system, the joint tensile strength (Jten), joint cohesive strength (Jcoh). Also, the joint dilation angle (Jdil) were assumed to be equal to zero. A sensitivity analysis on the frictional performance of joints was undertaken. The joint friction angle varied from 14° to 36.9° degrees. This was to simulate potential joint degradation effects and/or possible water lubrication at the joint.

Table 1. Properties of the limestone blocks (Kastenmeier et al. 2010 and Angrisani et al. 2010).

Unit Weight d [kg/m^3]	Young Modulus E [GPa]	Shear Modulus G [GPa]	Bulk Modulus K [GPa]	Poisson's Ratio v [-]
2680	40	16	27	0.25

Table 2. Properties of the joint interfaces (Kastenmeier et al. 2010 and Angrisani et al. 2010).

Normal Stiffness JKn [GPa/mm]	Shear Stiffness G [GPa/mm]	Joint friction angle Jfric [degrees]	Joint tensile strength Jten [MPa]	Joint cohesive strength Jten [MPa]	Joint dilation angle Jdil [degrees]
4	2	14° to 36.8°	0	0	0

4.3 Loading procedure

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, external loading has been applied to the structure by increasing uniformly horizontal accelerations in the in plane direction (non-linear pushover analysis). Horizontal displacements at the upper part of the colonnade (ref. Figure 4, Point A) have been recorded. The results of the response of the structure under static and dynamic load are presented below.

5 Response to Static loading conditions

Since the original columns that were placed adjacent to the structure have collapsed and thus the continuity of the “flat arch” beam above the columns has been lost, mechanisms can develop between inclined faces of the blocks which results in thrust forces and hence sliding of the blocks and opening of the joints. The response of the structure under gravity load has been examined.

To simulate joint degradation, a series of computational tests were undertaken where values of joint friction angle ranged from 14 to 36.9 degrees. From the results analysis, it was found that sliding of blocks and opening at the joints increases as the joint friction angle decreases. Also, stability of the historical structure occurs for values of friction angle greater or equal to 15 degrees. It is worth mentioning that the angle of each of the inclined blocks is 14 degrees.

Figure 5 shows the displacement vectors in the colonnade when the joint friction angle is 15 degrees. The displacement vectors clearly show the opening of the columns in the lower storey which is a result of the thrust of the central blocks of the segmented beam. The block at the central bay slides down and pushes the contiguous blocks apart. Also, due to the sliding, the upper block tends to rotate and further pushes down the former. The upper storey columns just follow the sliding of the blocks of the beam. From the above, stability in this case is mainly a geometrical problem since the loss of support is the reason of collapse.

Figure 6 shows the location of the joint opening for different values of friction angle. For a joint friction angle equal to 36.8 degrees, the maximum opening at the joints is in the order of 0.085 mm (Figure 6a). For a joint friction angle equal to 15 degrees, the maximum opening at the joints is 0.37 mm (Figure 6b). When the joint friction angle is 14 degrees or below, equilibrium of the colonnade cannot be granted and failure occurred (Figure 6c). This is due to the fact that sliding of the blocks over the columns is not able to hold in place the central block which is flowing down. It is worth mentioning that the angle of the inclined faces of the segmented blocks in the trabeation is also 14 degrees.

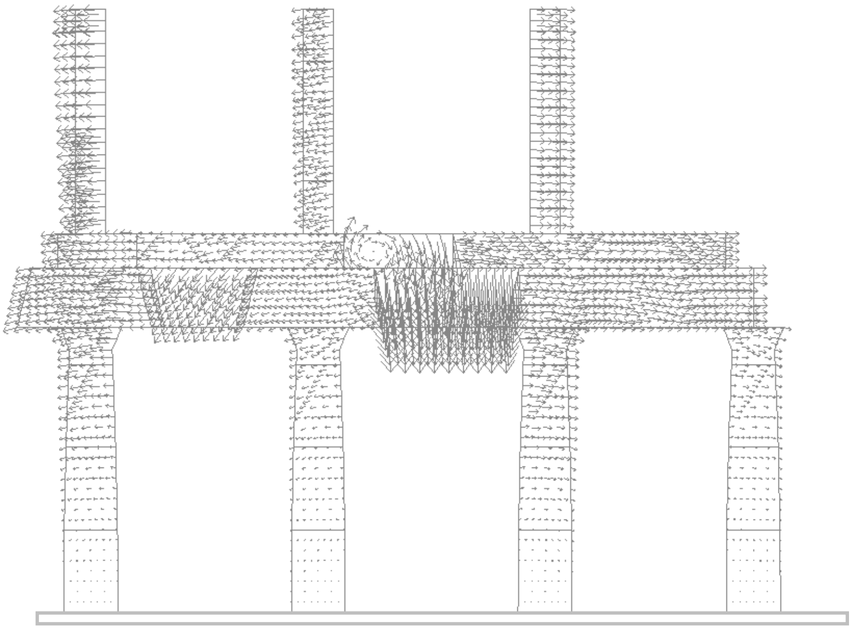
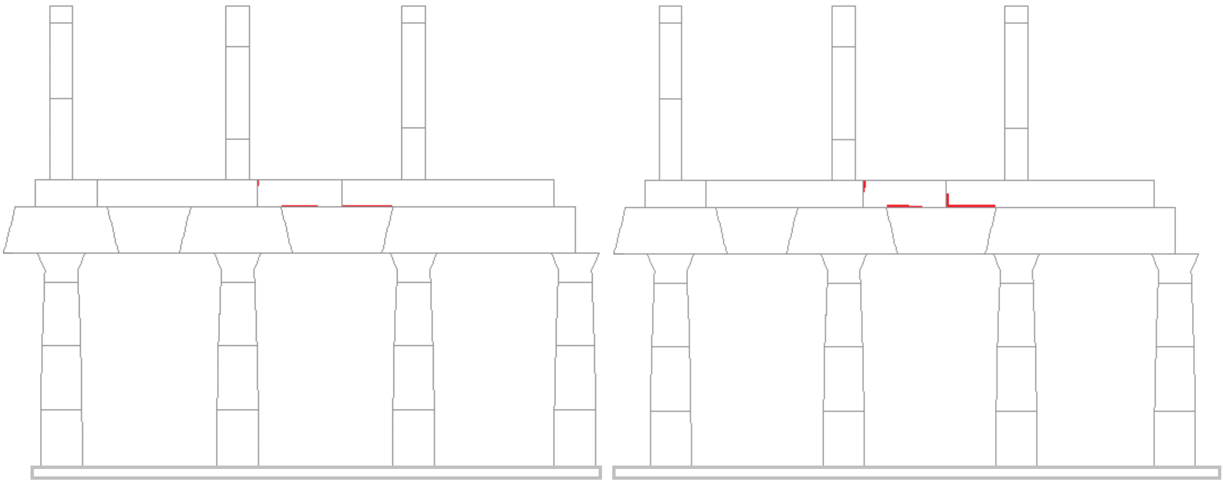
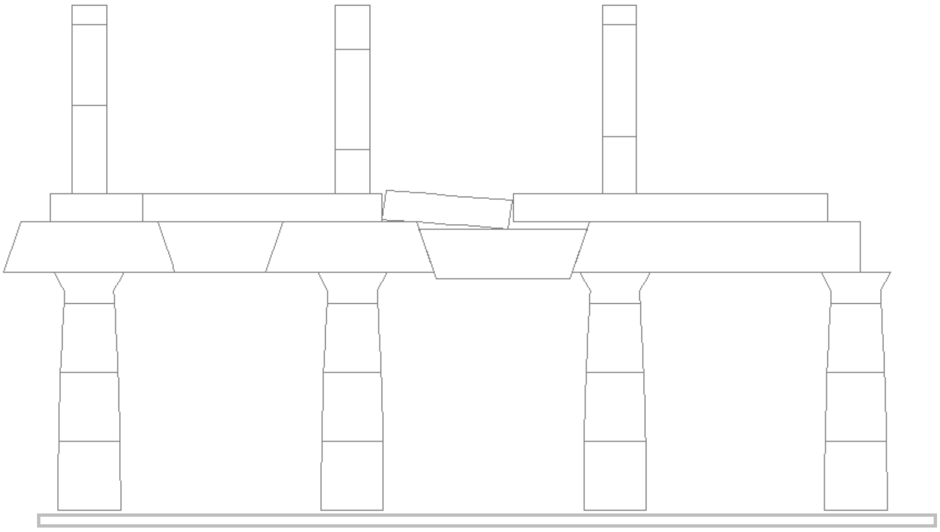


Figure 5. Displacement vectors(Jfric=15°)



(a) Jfric = 36.8° (Max joint opening 0.085 mm)

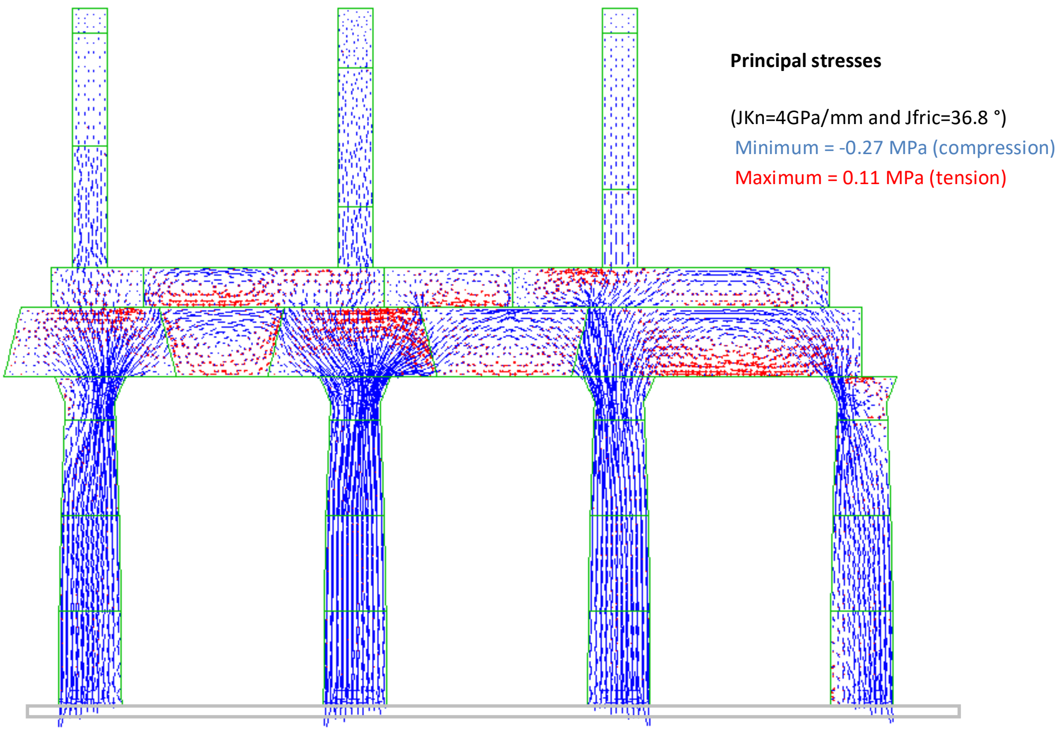
(b) Jfric = 15° (Max joint opening 0.37 mm)



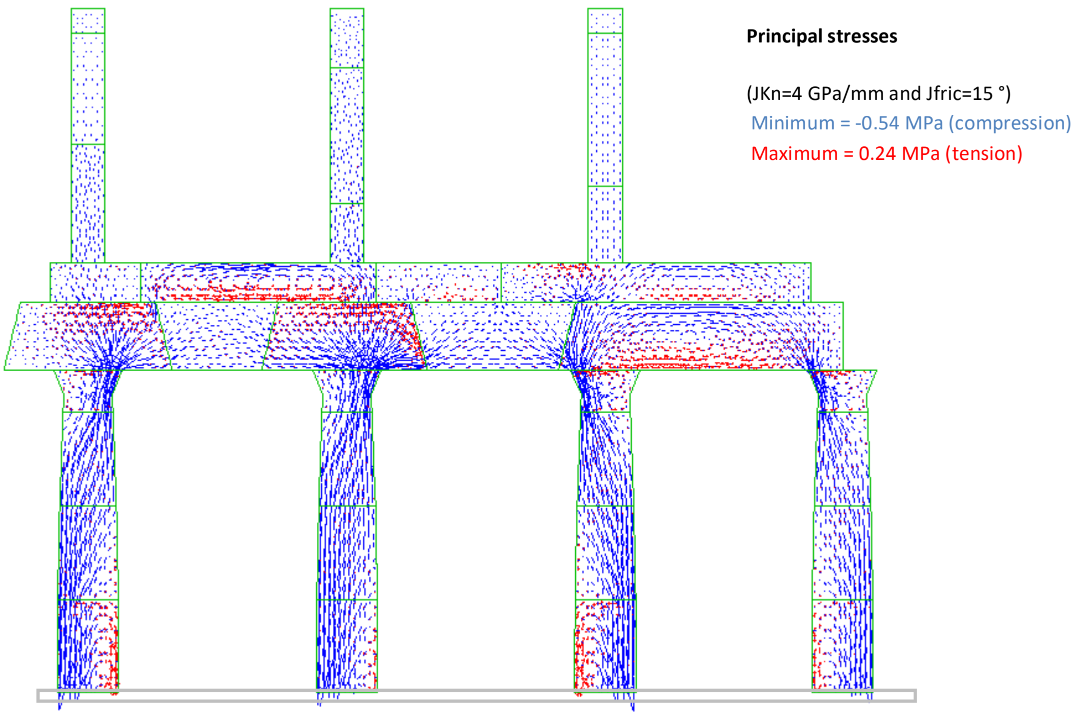
(c) Jfric = 14° (collapse state)

Figure 6. Joints opened in the structure

Figures 7a and 7b show the principal stress distribution when joint friction angle is 36.8 and 15 degrees respectively. From Figure 7, stress intensities are reducing as the joint friction angle increases. When the joint friction angle is 36.8 degrees, the maximum principal compressive stress in the structure is 0.27 MPa and the minimum principal tensile stress is 0.11 MPa. Similarly, when the joint friction angle is 15 degrees, the maximum principal compressive stress in the structure is 0.54 MPa and the minimum principal tensile stress is 0.24 MPa. These values corroborate the idea that blocks are rigid and no failure can be expected to occur in them. In fact, the strength in the blocks is expected to be at least one order of magnitude higher, both in tension and in compression (Angrisani et al. 2010). Also, for the continuous beam (left part of the structure), high tensile stresses occurred at the bottom of the beam while compression occurred at the top of the overlapped elements. Conversely, the left layout provides a "flat-arch"; having almost negligible tractions and widespread compression stresses.



(a)



(b)

Figure 7. Principal stress distribution for different values of joint friction (tensile: red, compression: blue)

A sensitivity study on the disestablishment of the geometric components of the two storey colonnade in Pompeii has been investigated under static loading conditions. The stability has been checked for the following two cases:

a) Case 1: The second row of columns is missing (Figure 8); and
b) Case 2: The second row of columns and the upper part of the beam supported with the columns is missing (Figure 9).

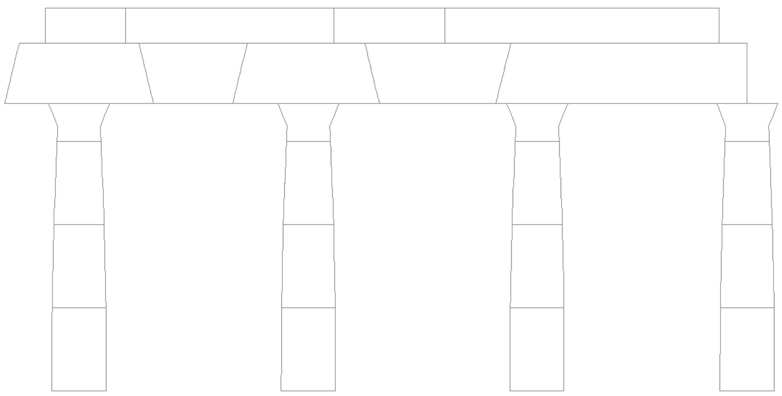


Figure 8. Geometry of the colonnade when the upper columns disestablished

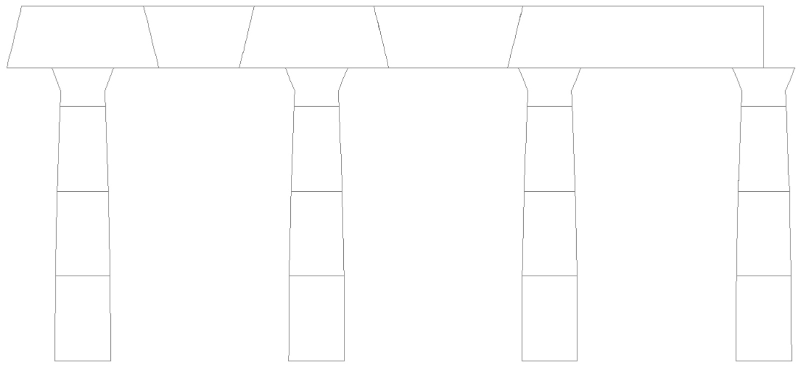


Figure 9. Geometry of the colonnade when the upper columns and the upper part of the beam disestablished

For each case, the angle of internal friction varied until the structure reached equilibrium. For Case 1, stability achieved when the joint friction angle of the interface was greater or equal to 16 degrees (Figure 10). The absence of the upper columns reduces the friction in the column/beam joint. Therefore, the vertical sliding of the key stone is more prominent when the upper columns are missing since the keystone is moving downwards while the supports (due to the self-weight of the columns) are spread. In addition, for the Case 2, stability achieved when the joint friction angle of the interface was greater or equal to 24 degrees (Figure 11). By comparing Case 1 and Case 2, it is apparent that failure of the system-structure occurs at higher friction angle as the weight above the structure decreases. Therefore, care should be taken when a monument is disassembled (i.e. dead load is reduced) since this may lead to sudden collapse.

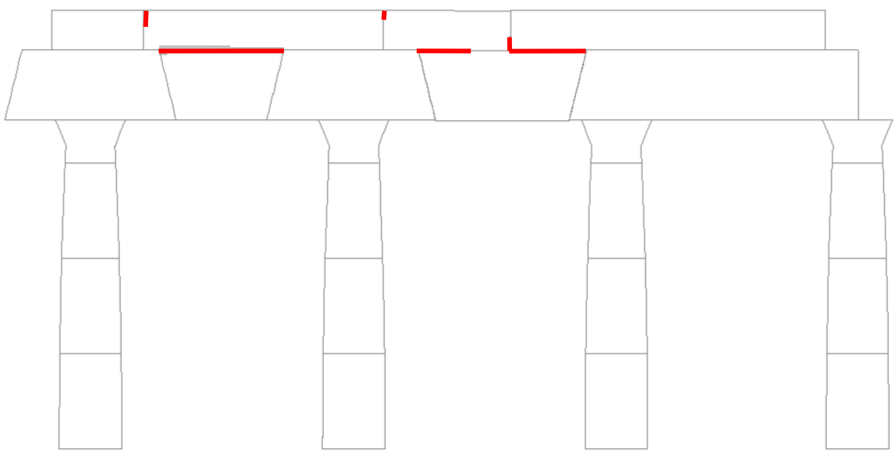


Figure 10. Joints opened (in red) at the structure; Jfric = 16° (collapse state)

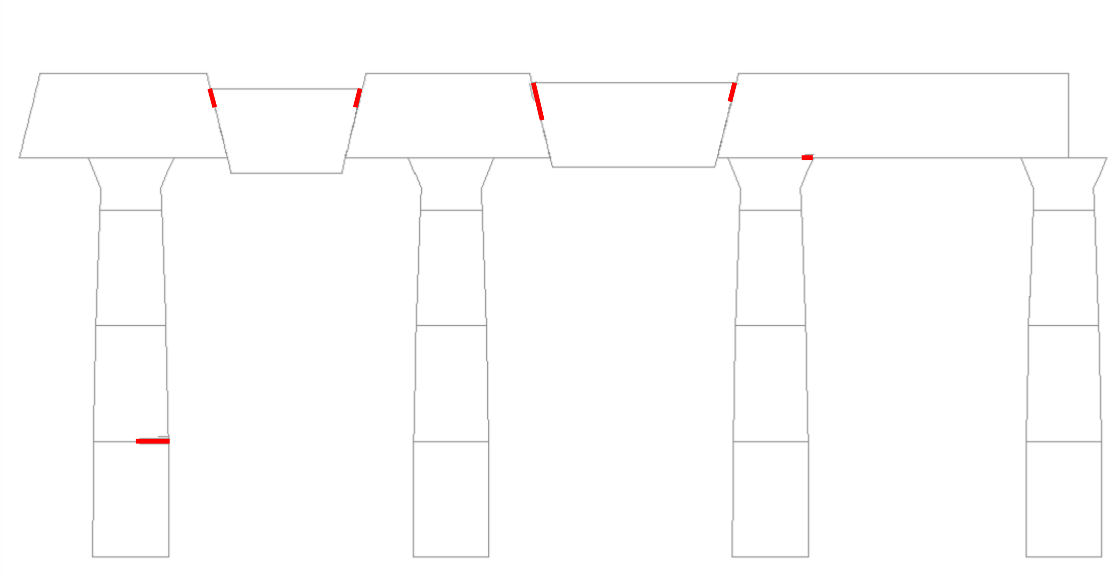


Figure 11. Joints opened (in red) at the structure; Jfric = 26° (collapse state)

6 Response to dynamic loading conditions

6.1 Response to harmonic excitations

The role of the excitation frequency in the collapse mechanism was examined using sinusoidal excitations. The frequency of the excitation varied from 1 to 7 Hz and the amplitude of the base acceleration ranged from 1.7 to 80 m/s^2. From the initial results analysis, it became apparent that high frequency motion requires large base acceleration amplitude to cause rocking motion and deformability between the individual blocks of the colonnade leading to collapse. Alternatively, a low frequency of excitation resulted in a monolithic response of the structure leading to collapse. Also, a parametric investigation has been carried out and the amplitude of the base acceleration was increased by small steps until collapse of the structure occurred. It was observed that the frequency plays a dominant role in the dynamic response of a structure. Harmonic wave with larger acceleration amplitude leads to larger deformations (Figures 12 and 13). However, apart from the amplitude, the frequency of motion is also playing a dominant role in the dynamic response of a structure. Also, the safe-unsafe threshold of the acceleration amplitude of the two storey colonnade in Pompeii was determined for each excitation period (Figure 14). Figure 14 shows for each pair of values for the excitation period and amplitude, whether the colonnade collapsed or not. The amplitude required to cause collapse should be larger than the safe-unsafe threshold. Based on this restriction, the safe-unsafe boundary has been estimated. The threshold of acceleration amplitude causing collapse significantly decreases as the frequency of the harmonic excitation reduces. Low-frequency harmonic excitations are more prominent to cause collapse of the two storey colonnade than the high-frequency ones.

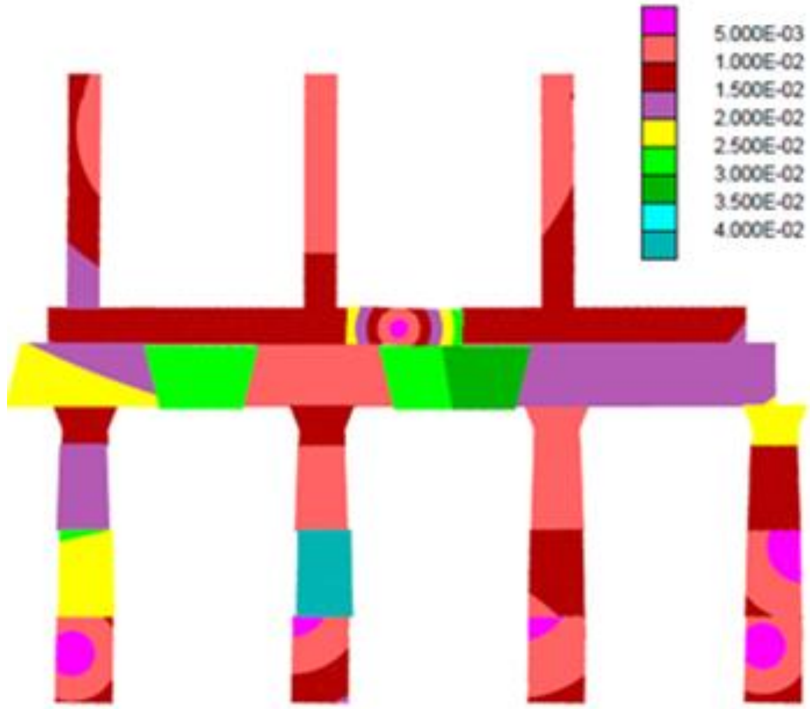


Figure 12. Displacement (m) magnitude contour of model with segmental beam (f=7Hz, A=20m/s^2)

Figure 13. Displacement (m) magnitude contour of model with segmental beam (f=7Hz, A=47.5m/s^2)

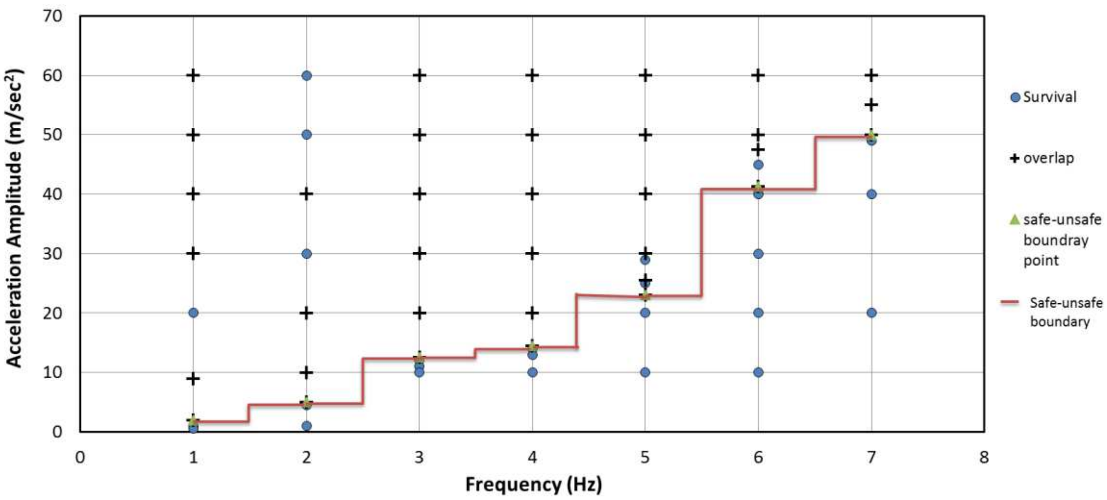


Figure 14. Safe-unsafe boundary of segmental beam model

Figure 15 shows that the blocks of the lower beams and columns have most of the times larger displacements than the upper columns. The existence of the beams increases the stability of the upper columns. In addition, for the structure under consideration, the use of solid beams instead of segmental ones has been assessed with respect to harmonic excitation loads..

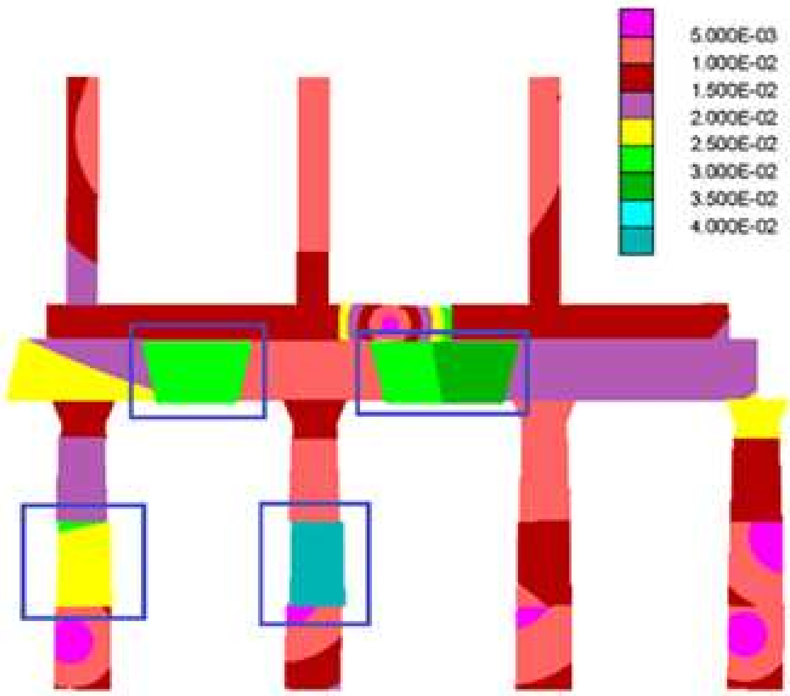


Figure 15. Location of the blocks with large displacement.

Using the same model but substituting the segmental beams with solid ones and using the same excitation showed that higher frequency leads to larger threshold of acceleration amplitude, which means that lower frequency is more prone to cause collapse.

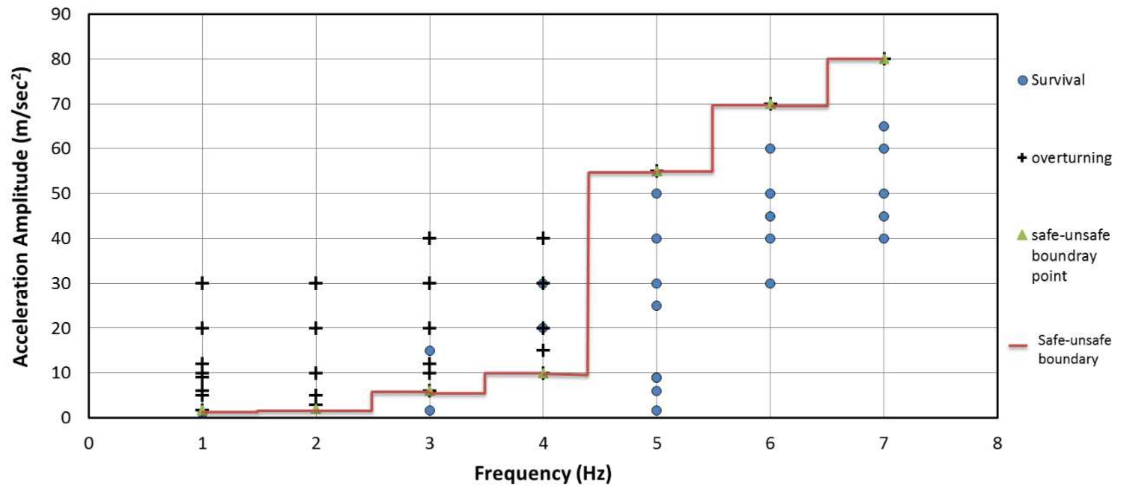


Figure 16. Safe-unsafe boundary of solid beam model

From figure 17, the stability of solid beam model differs from that of the segmental beam model when frequency is greater than 4 Hz but it is similar when the frequency is less than 4 Hz. So, the colonnade with the segmental beam is more prone to collapse when compared to the one with a solid beam for frequencies ranging greater than 4 Hz.

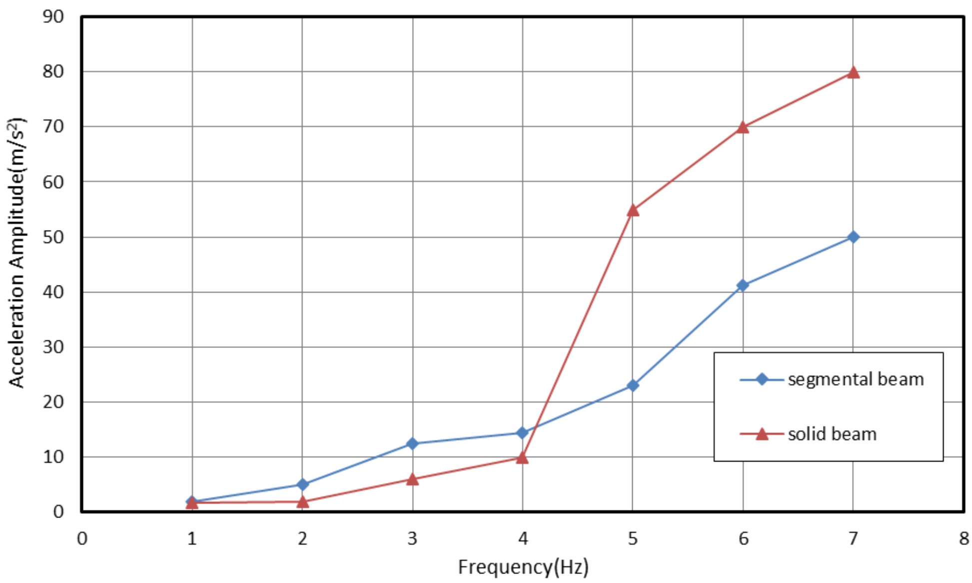


Figure 17. Comparison of segmental beam model and solid beam model

6.2 Response to seismic excitations

Four earthquake records with different characteristics such as the magnitude, the predominant period, the frequency and the type of seismic wave (sinusoidal or not) were used in order to investigate how they affect the stability of the two storey colonnade. The records selected are shown in Figures 18 and 19 and based on the same acceleration and time scales. The four records selected for this study are: the earthquake of Bucharest 1977, Irpinia 1980, Kalamata 1986, and L'Aquila 2009. The 2007 Bucharest earthquake took place in the region of Vrancea and it was one of the worst earthquake disasters of the 1970s causing 1,578. This earthquake, of the 7.2 magnitude and 94 kilometers local depth, was selected because of its large-amplitude quasi-sinusoidal pulse. The Irpinia earthquake occurred in the Irpinia region in Southern Italy in 1980 with 2,914 casualties. Its magnitude was 6.9 and it was the result of the Irpinia fault activation which is known as one of the most active natural earthquake laboratories around the world. The Kalamata (Greece) 1986, record of 6.2 magnitude contains near field strong motion data that caused considerable damage to the buildings of the city of Kalamata. The 2009 L'Aquila earthquake occurred in the region of Abruzzo, in central Italy. The main shock was rated 6.3 on the moment magnitude scale; its epicentre was near L'Aquila, the capital of Abruzzo, which together with surrounding villages suffered considerable damage.

An earthquake has a much richer frequency spectrum than sinusoidal signals. Moreover, the four earthquakes that were taken into account can be hardly considered as representative given the complex dynamic and strongly non-linear behavior of this kind of systems. Of course these approaches were relatively recently applied to this kind of structures, but when general conclusions are to be made regarding the dynamic behavior and resistance to collapse, they are necessary.

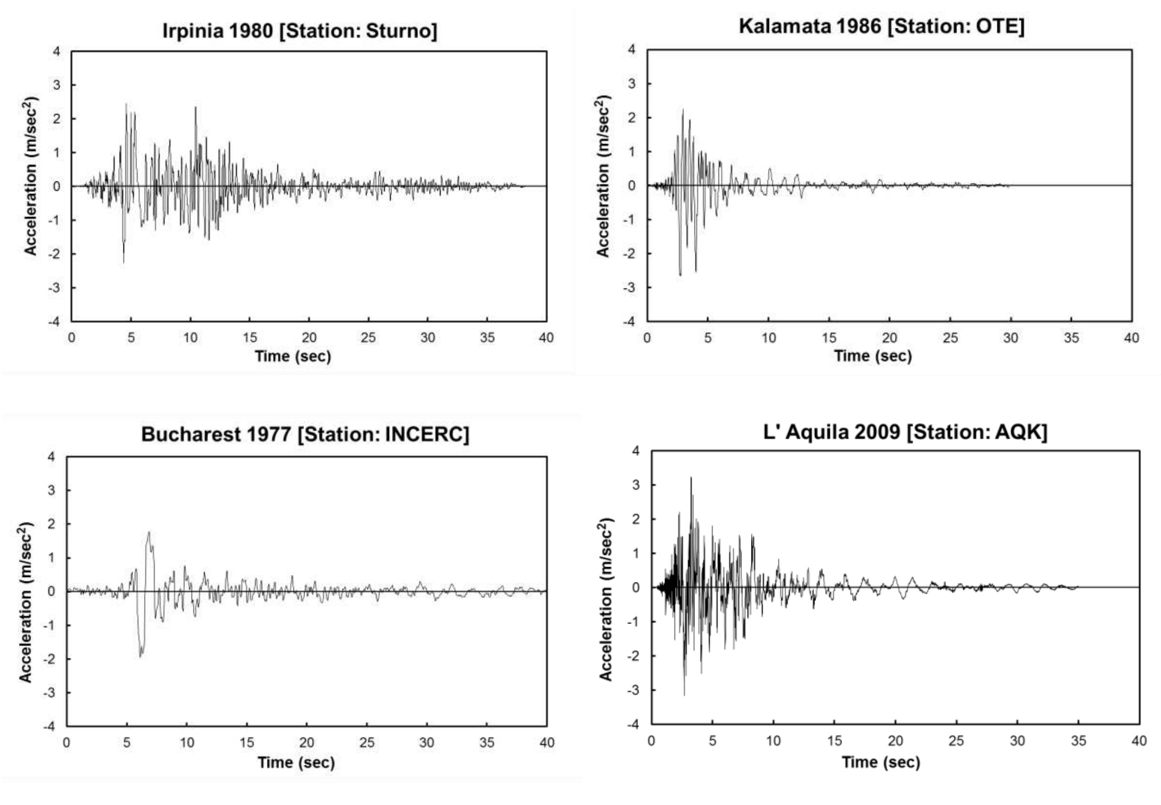


Figure 18. Acceleration Time History for selected earthquakes

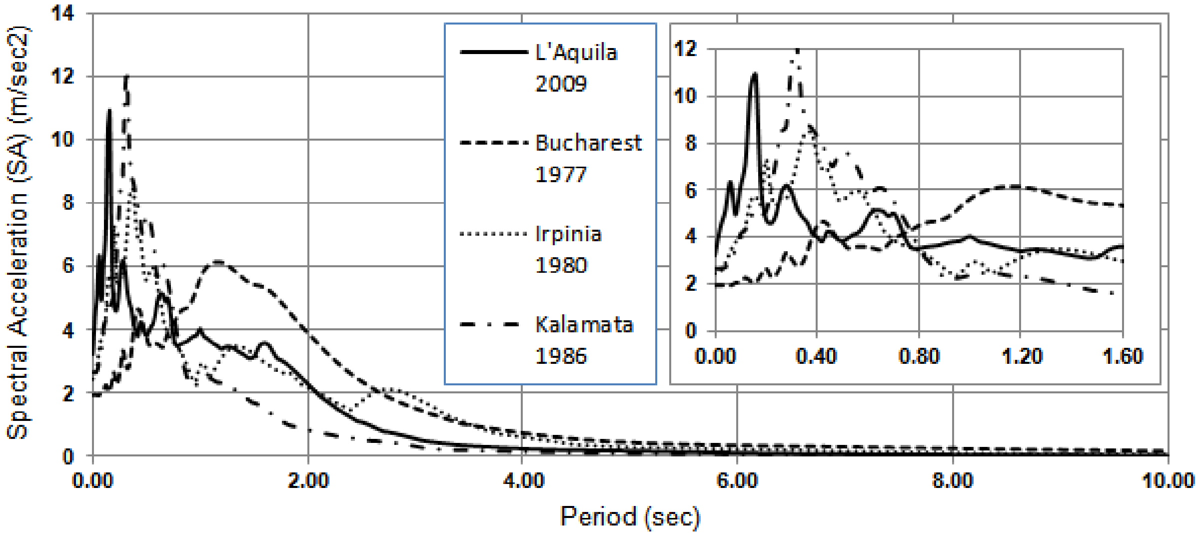
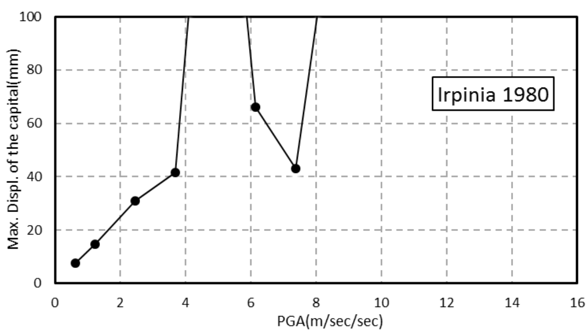
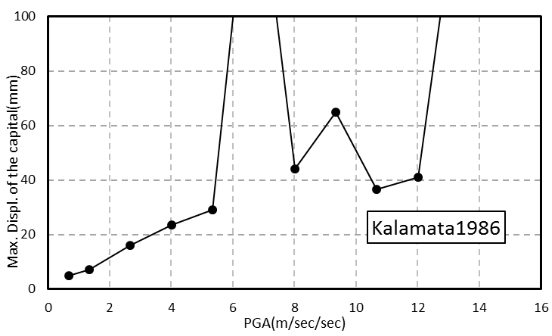


Figure 19. Acceleration response spectrum for 5 % damping ratio

Seismic loads were applied to the model of the two storey column of Pompeii scaling if needed the records to a specified level of seismic intensity in order to cause collapse. The influence of the acceleration levels and pulse period in the seismic response was investigated. Figures 20 and 21 show that there are certain PGA values where collapse occurs. Below these values the maximum displacement of the left corner increases as the PGA increases. Above the PGA values that causing collapse the response is less predictable. Similar observations can be made for all earthquakes with predominant period less than 0.4s. Larger displacements occur for Bucharest earthquake whose predominant period is 1.2s. This helps us to conclude that the fundamental period of the structure is approximately 1.2s. It is worth mentioning that this earthquake caused considerable damages to buildings of 12 to 18 floors which also corresponds to a fundamental period of about 1.2s (Armas 2006). Also, it can be stated that the response of the colonnade structure is crucially affected by low-frequency earthquakes, like the Bucharest earthquake (Figure 18). In such cases of low-frequency earthquakes, in accordance with the results presented in the previous section in which the structure was studied under harmonic excitations, the number of drums that assemble a colonnade does not affect the seismic response of the system, since all of the drums of the columns tend to rotate in a single group, similar to a monolithic column.





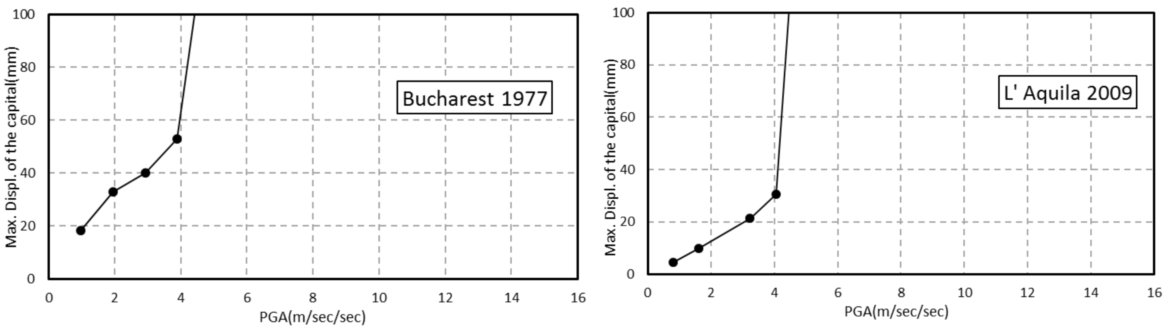


Figure 20. Maximum displacement of top left corner of system under scaled earthquake motion records with segmental.

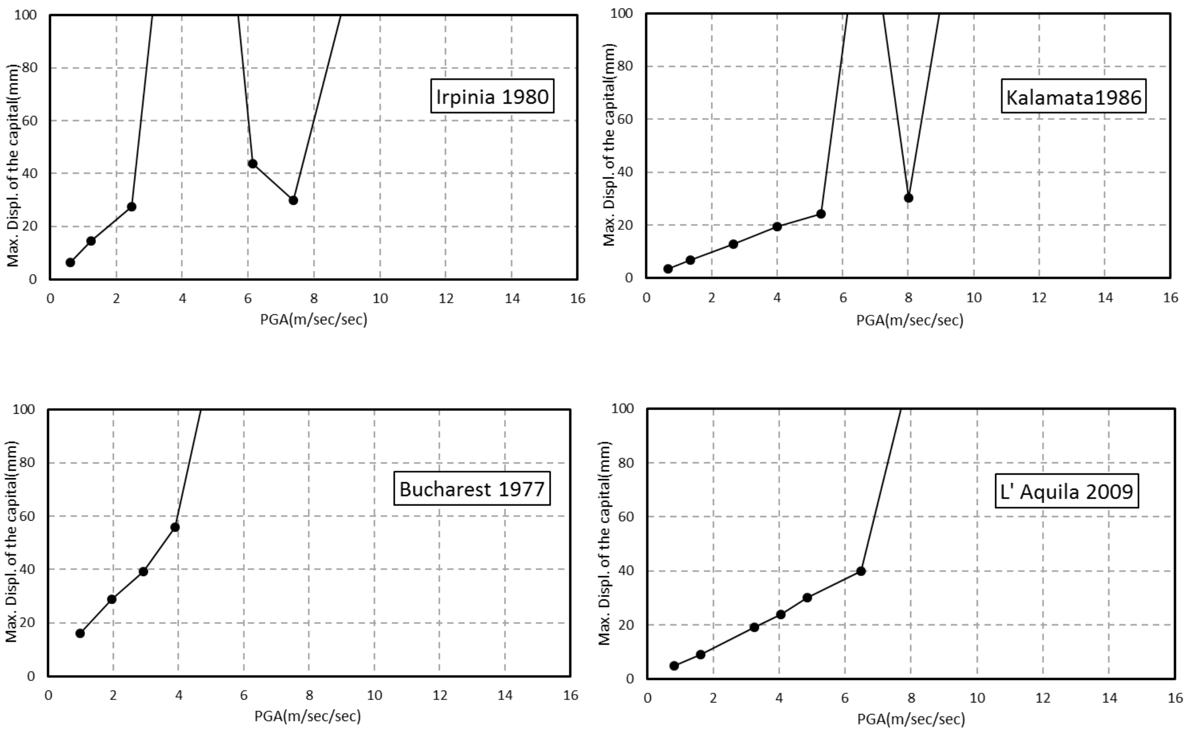


Figure 21. Maximum displacement of top left corner of system under scaled earthquake records with solid and segmental beam.

Figure 22 presents the seismic response of the colonnade containing segmental beams compared to that with solid beam. From the numerical analysis, it is observed that the colonnade containing a solid beam may affect the seismic response differently and depending on the earthquake excitation. For Irpinia, Kalamata, and L'Aquila- records the seismic response is quite similar for low acceleration levels but differs for high ones. It can also be noticed that the displacements are lower when the solid beam is used except when collapse occurs. For the Bucharest record containing a long period pulse of 1.2s, the influence of the structure of the beam is not significant.

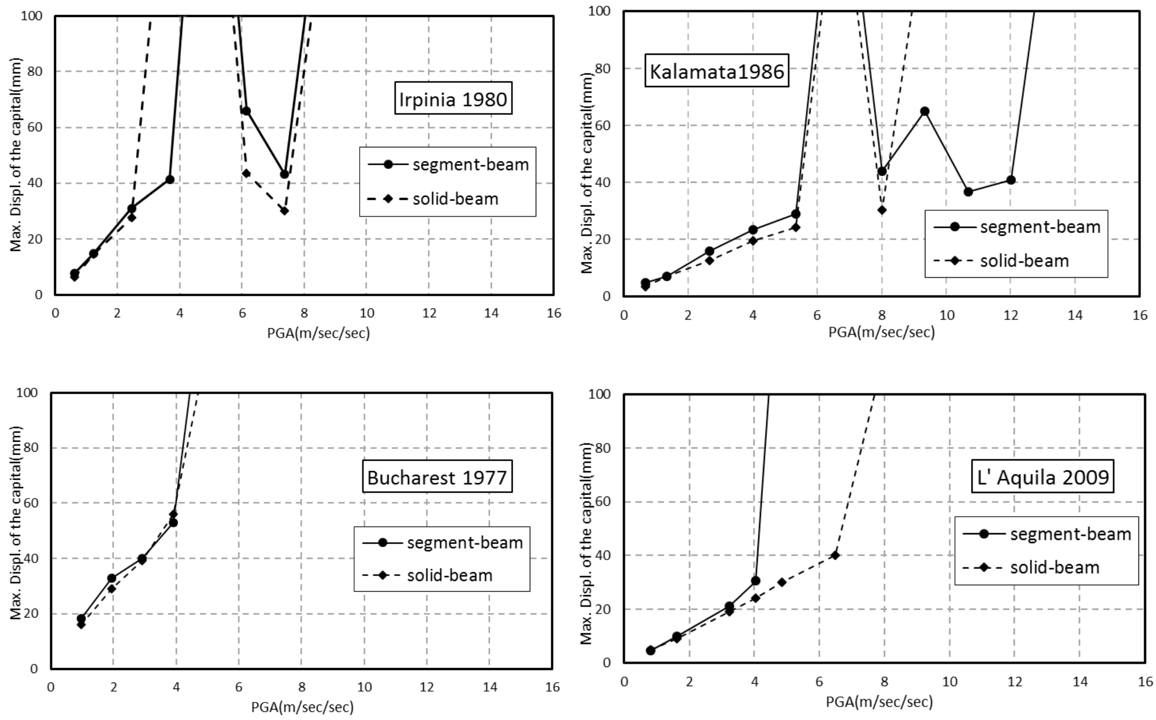


Figure 21. Maximum displacement of top left corner of system under scaled earthquake records with solid and segmental beam.

7 Conclusions

Earthquakes represent one of the major threats to the stability of certain world architectural heritage landmarks, such as classical columns and colonnades. The seismic behaviour of these structures is highly nonlinear and complex since both rocking and sliding phenomena can occur. As a result, numerical modelling seems to be the most promising approach to understand their behaviour during strong earthquakes. Using software based on the Distinct Element Method (DEM) of analysis, the parameters affecting the seismic behavior of colonnades' structural systems have been examined. A typical section of the two storey colonnade of the Forum in Pompeii has been modelled and studied parametrically, in order to identify the main factors affecting its stability. The major findings based on the two dimensional model developed are presented below:

- The present condition of the partially collapsed colonnade presents higher vulnerability because the system of horizontal thrust is not perfectly balanced on one side, hence providing non-symmetric behaviour
- Joint openings are considered to be detrimental for the structure since they may lead to water leakage and lubrication of the potentially sliding planes. The analysis showed that if the joint friction angle is 14 degrees, equilibrium of the structure cannot be granted and failure occurs.
- Also, as the weight above the structure decreases, failure of the system occurs at higher friction angle. Therefore, care should be taken when parts of the monument are disestablished (i.e. dead load is reduced) since this may lead to sudden collapse.

- The techniques used for the construction of the trabeations (beams/epistyle) affect the response of the structure. The colonnade systems with segmented trabeations are more vulnerable than those constructed by solid blocks.
- Harmonic and seismic analysis showed that that ground motions with large frequences are more threatening to multi-block systems than short-period ones.
- The structure responds with larger displacements to earthquake motions, which are characterized by large predominant period. The response of the colonnade structure is significantly affected by low-frequency earthquakes, like the Bucharest earthquake.
- Despite that these structures exhibit high vulnerability under low-frequency earthquakes, they successfully withstand strong earthquakes with high-frequency content.
- Modeling colonnade structures is still a challenging task despite the long and rigorous attention that it has received from the research community over the last twenty years. The large number of parameters involved, and in particular the discontinuous nature of the structure makes the modeling of this "structural system" very difficult. Further studies are currently conducted by the authors on the modeling of such structural systems aiming to a more reliable modeling as the 3D modeling.

In the future, the three dimensional behaviour of the colonnade will be studied. In this way, the out of plane behaviour of the colonnade subjected to different magnitudes of loading and seismic excitations will be studied since the structure might be more vulnerable for out-of-plane loadings (dynamic or not). Also, stochastic analyses will be carried out to obtain a better estimation of the vulnerability of the ancient colonnade to earthquake. Such approaches are relatively recent and have been carried out by (*Psycharis et al. 2013; Stefanou et al. 2015).*

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