Chapter 24
Seismic Vulnerability of Classical Monuments

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Abstract Classical monuments are articulated structures consisting of multi-drum columns made of discrete stone blocks that are placed one on top of the other without mortar. Despite the lack of any lateral load resisting mechanism except friction, classical monuments are, in general, earthquake resistant, as proven from the fact that they have survived several strong earthquakes over the centuries. However, in their current condition, they present many different types of damage that affect significantly their stability. This chapter presents the results of theoretical and experimental research on the earthquake resisting features and the assessment of the vulnerability of these structures, which is not straightforward due to the high nonlinearity and the sensitivity of the response. Recent trends towards a performance-based philosophy for the seismic risk assessment of these structures, based on conditional limit-state probabilities and seismic fragility surfaces, are also discussed.

24.1 Introduction

Classical monuments are made of structural elements (drums in case of columns), which lie one on top of the other without mortar. Columns are connected to each other with architraves (also called "epistyles") consisting of stone beams, usually made of marble. A characteristic example is shown in Fig. 24.1 from the Olympieion of Athens, Greece.

Architrave beams are usually connected to each other with iron clamps and dowels. However, in most cases no structural connections are provided between the drums of the columns. Only in few cases, iron shear connectors (dowels) are provided at the joints, which restrict, up to their yielding, sliding but do not affect rocking. The wooden dowels that were usually placed at the joints among the drum of the columns were aiming at

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centring the stones during construction and, practically, do not have any effect on their seismic response.

Despite their articulated construction and the fact that many of them are located in seismically active regions, quite a few classical monuments are standing for more than 2000 years, although with damages in most cases. Of course, many others have collapsed while, in early literature, there are frequent references to extensive repair of these structures because of earthquake damage, whereupon the opportunity to introduce changes in their design and construction that enhanced their earthquake resistance was taken. Thus, while not all classical monuments are intrinsically earthquake-resistant, they are in fact more resistant to earthquakes than might be expected (Psycharis et al. 2000, 2003).

Due to their spinal construction, columns and walls of ancient monuments respond to strong earthquakes with intense rocking and sliding. As a result, the dynamic analysis of ancient monuments and the assessment of their vulnerability to earthquakes is a difficult problem to treat, since their seismic response is nonlinear, complicated and very sensitive to even trivial changes of the parameters of the system or the excitation.

Several investigators have examined the seismic response of classical monuments analytically, numerically or experimentally, mostly using two-dimensional models (e.g. Allen et al. 1986; Sinopoli 1989; Psycharis 1990; Winkler et al. 1995; Psycharis et al. 2000; Konstantinidis and Makris 2005; Papaloizou and Komodromos 2009 among others) and lesser three-dimensional ones (e.g. Papantonopoulos et al. 2002; Mouzakis et al. 2002; Psycharis et al. 2003, 2013; Dasiou et al. 2009a, b). These studies have shown that such structures do not possess natural modes in the classical

Fig. 24.1 Olympieion of Athens, Greece: Left: Columns and architraves at a corner; Right: A free-standing and a fallen column showing their multi-drum construction

sense, since the period of free vibrations is amplitude dependent. Moreover, during a strong earthquake the response alternates between different ‘modes’ of vibration, each one being governed by a different set of equations of motion. As a result, the response is highly non-linear. An example of this non-linearity is that a column may collapse under a certain earthquake motion and be stable under the same excitation magnified to a larger amplitude.

24.2 Main Features of the Rocking Response of Rigid Blocks

The earthquake response of classical monuments is dominated by the rocking that occurs at the joints of the stone elements, following the dynamics of rocking rigid blocks. For this reason, the main features of the rocking response of a single, free-standing block are presented in this section.

The rocking response of a free-standing rigid block, despite its apparent simplicity, is a difficult problem to treat because it is nonlinear and extremely sensitive. Thus, although the problem has been observed since late 19th century (Milne 1885, Milne and Omori 1893, Kirkpatrick 1927) and the first systematic analysis was presented by Housner in 1963, this problem continues to attract the interest of several investigators.

The nonlinear feature of the rocking response of rigid blocks is illustrated in Fig. 24.2, in which the time history of the rocking angle of an orthogonal block with dimensions: base width $b = 0.50$ m and total height $2\,h = 1.50$ m is shown for the El Centro (1940) earthquake amplified to four different values of the peak ground acceleration, namely, $pga = 0.60$ g, 0.70 g, 0.80 g and 0.90 g. In all cases, the coefficient of restitution, which counts for the dissipation of energy during the

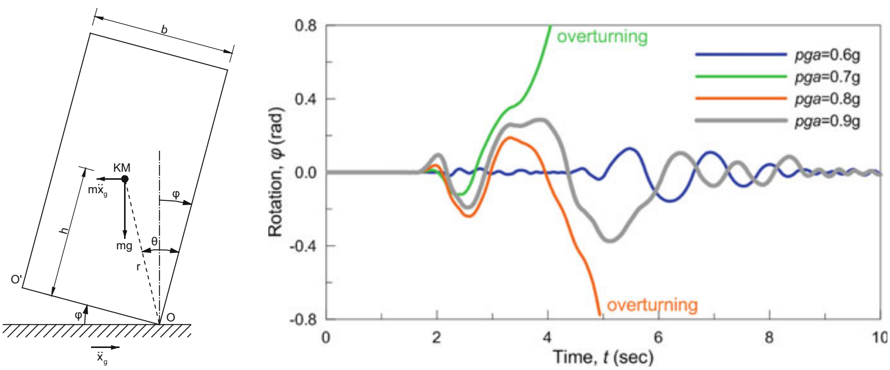


Fig. 24.2 Rocking response of an orthogonal block of dimensions $b = 0.50$ m, $2\,h = 1.5$ m for the El Centro (1940) earthquake amplified to several values of pga ($\varepsilon = 0.85$)

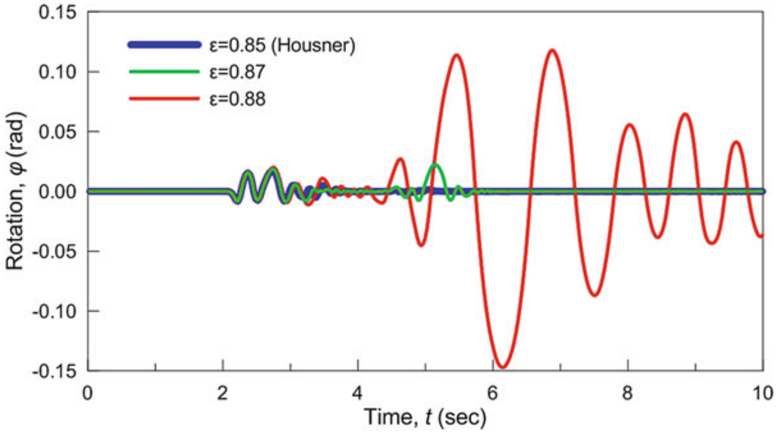


Fig. 24.3 Rocking response of an orthogonal block of dimensions $b = 0.50$ m, $2\,h = 1.5$ m for the El Centro earthquake amplified to $pga = 0.50$ g for various values of the coefficient of restitution ε

impacts of the block with the base, was set to $\varepsilon = 0.85$, which corresponds to the Housner's theoretical value (Housner 1963). It is seen that the response of the block is stable for $pga = 0.60$ g (blue line) while the block overturns in the direction of positive rotations for $pga = 0.70$ g (green line). If the base excitation is increased to $pga = 0.80$ g the block overturns in the opposite direction (negative rotations). However, if the base motion is amplified even more to $pga = 0.90$ g the response is stable again and overturning does not occur (grey line).

Apart from the nonlinearity, another characteristic of the rocking response is its sensitivity to even trivial changes of the parameters. This sensitivity has been proven by the non-repeatability of the same experiment (Yim and Chopra 1984; Mouzakis et al. 2002). In Fig. 24.3, the sensitivity of the response of the above-mentioned block to the value of the coefficient of restitution ε is shown. In this plot, the response of the block is shown for the El Centro record amplified to $pga = 0.50$ g and for three values of the coefficient of restitution: $\varepsilon = 0.85$ (Housner's value), $\varepsilon = 0.87$ and $\varepsilon = 0.88$.

It is seen that the response for $\varepsilon = 0.87$ (green line) is very similar with the one for $\varepsilon = 0.85$ (blue line), except for an additional small rocking response of the block around $t = 5$ s, which does not occur for $\varepsilon = 0.85$. However, if we slightly increase the coefficient of restitution to $\varepsilon = 0.88$, intense rocking occurs after $t = 4$ s with significantly larger amplitude than the amplitude in the time interval $2.0 < t < 3.5$ s when all the rocking response takes place for $\varepsilon = 0.85$. It is interesting to notice that this intense rocking for $\varepsilon = 0.88$ occurs after the strong motion of the ground excitation.

It is worthmentioning that, although, in general, a decrease in the value of ε leads to smaller rocking amplitude, due to the larger dissipation of energy during impact, it is also possible that a smaller coefficient of restitution produces larger rocking response (Aslam et al. 1980). This counter-intuitive phenomenon is attributed to the nonlinearity of the response. Note that the appropriate value of the coefficient of

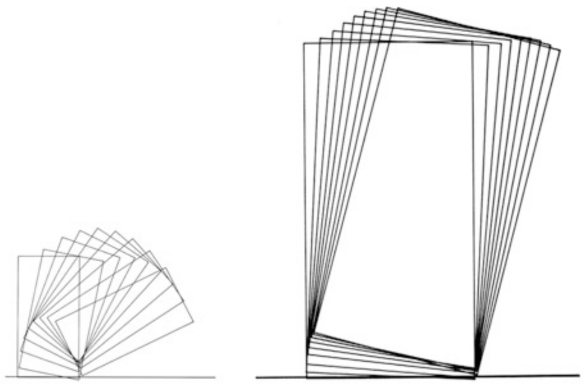


Fig. 24.4 Animated rocking response under the same impulse base excitation of two similar orthogonal blocks of the same slenderness (tanθ = 0.5) and different size: base width $b = 0.50$ m for the left block and $b = 1.50$ m for the right block

restitution is not easy to define, since experimental investigation (e.g., Priestley et al. 1978; Aslam et al. 1980) showed that the actual value of ε might be significantly different than the theoretical one of Housner, depending on the materials of the block and the base.

Concerning the parameters that affect the rocking response, it has been proven that the normalized response under harmonic excitation can be expressed solely by four dimensionless terms (Zhang and Makris 2001; Dimitrakopoulos and DeJong 2012), namely:

- The ratio ω_g/p, where $p = \sqrt{mgr/I_O}$ is the characteristic frequency parameter of the system (r is the half diagonal and I_O is the moment of inertia around point O, refer to Fig. 24.2) and ω_g is the frequency of the harmonic excitation. This ratio increases with the frequency of the excitations and the size of the block (measured through r);
- The ratio $a_g/(g\tan\theta)$, with θ being the slenderness angle (refer to Fig. 24.2) and a_g being the amplitude of the harmonic excitation. This ratio measures the strength of the excitation compared to the critical acceleration $g\tan\theta$ required for the initiation of rocking;
- The slenderness of the block, which is measured with the angle θ; and
- The coefficient of restitution ε.

Assuming that ε is known and constant, the dimensionless analysis reveals that:

- For a given base excitation (given a_g and ω_g), the response depends on the slenderness θ and the characteristic frequency p. The latter decreases inversely with the size of the block, measured with the half-diagonal r, therefore, for the same slenderness there is an important size effect on the response. Actually, among two blocks with the same slenderness θ but different size, the smaller one will experience more intense rocking than the larger one. This is shown in Fig. 24.4, in which the response of two blocks with tanθ = 0.5 but different size ($b = 0.50$ m for the left block and $b = 1.5$ m for the right one) is shown for the same impulse base excitation. It is seen that the small block overturns while the large one does not.

- For a given block (given θ and p), the rocking response and the overturning risk greatly depend on the predominant period of the base excitation. In general, the required normalized amplitude of the base acceleration, $a_g/(g\tan\theta)$, to cause overturning decreases as the excitation period T_g increases (Zhang and Makris 2001; Dimitrakopoulos and DeJong 2012). In other words, the block is more vulnerable to long-period earthquakes than to high-frequency ones.

It should be noted that, if φ is the angle of rotation, the inequality $\varphi > \theta$ is a necessary but not a sufficient condition for overturning to occur, since it is possible that the rocking angle attains temporarily values larger than θ (i.e. $\varphi_{\max} > \theta$) without overturning. Of course such cases are exceptional, since for $\varphi > \theta$ the weight of the block produces an overturning moment instead of a restoring one; thus the block will not topple only if at the same time a quite large restoring inertial force develops due to the ground motion, capable to reverse this situation and bring the block back to stable state.

The above-mentioned conclusions on the response and toppling of rigid blocks, although they have been derived for harmonic base excitations, apply qualititevely to earthquake ground motions as well, at least near-faults ones containing strong directivity pulses (Fragiadakis et al. 2016a). In this case, the frequency of the ground motion should be set equal to the frequency of the predominant pulse.

24.3 Seismic Response of Classical Monuments

The earthquake response of classical monuments is governed by the motion of the stones they are constructed of, which can rock and slide individually or in groups. In case of columns, wobbling also occurs during rocking due to the cylindrical shape of the drums. Since rocking is dominant in the dynamic behaviour, the earthquake response of classical monuments is characterized by the strong nonlinearity and the sensitivity discussed in the previous section.

A typical example of the seismic response of multi-drum columns is shown in Fig. 24.5, in which snapshots of the response of two columns of the Olympieion of Athens at two different time instances during intense ground shaking are shown. It is evident that rocking dominates the response; however, the response of each column is different, as it is significantly affected by the geometry. In particular, the height of the drums varies, while the left column has 14 drums and the right one has 15 drums.

In general, there are many 'modes' of response in which multi-block systems can respond during an earthquake and the system continuously moves from one 'mode' to another. For example, there are four 'modes' of vibration for two-block assemblies (Psycharis 1990), depicted on Fig. 24.6. For systems with many blocks, the number of 'modes' increases exponentially with the number of blocks.

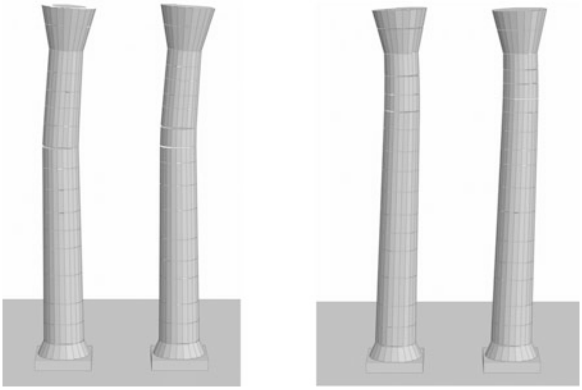


Fig. 24.5 Response of two columns of Olympieion of Athens at two different time instances during intense ground shaking. The geometry of the two columns is different (the left has 14 drums and the right 15) leading to different response (numerical results obtained with 3DEC (Itasca 1998))

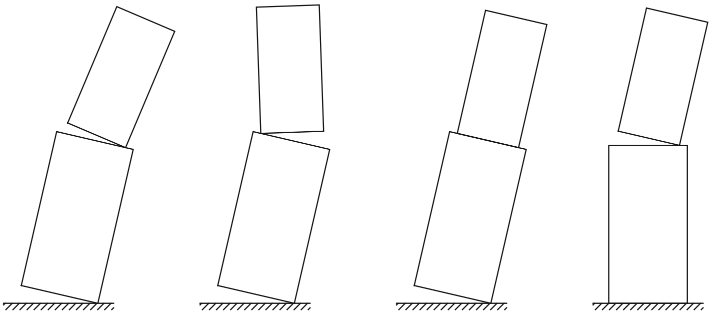


Fig. 24.6 The four rocking 'modes' of vibration of a two-block assembly (Psycharis 1990)

24.4 Vulnerability to Earthquakes

As mentioned above, classical monuments can generally sustain large earthquakes without collapse in their intact condition; however, they are not earthquake proof for all seismic motions. Also, if damage is present, their vulnerability decreases significantly.

The assessment of the seismic reliability of a monument is a prerequisite for the correct decision making during a restoration process. The seismic vulnerability of the column, not only in what concerns the collapse risk, but also the magnitude of the expected maximum and residual displacements of the drums, is vital information that can help the authorities decide the necessary interventions. This assessment is not straightforward, not only because fully accurate analyses for the near-collapse state are practically impossible due to the difficulty in modelling accurately the existing imperfections and the sensitivity of the response to even small changes in the geometry, but also because the results highly depend on the ground motions characteristics.

In general, excluding the effect of damage, the vulnerability of ancient monuments depends on two main parameters: the size of the structure and the predominant period of the ground motion (Psycharis et al. 2000). These issues are discussed in the following.

24.4.1 Size Effect

Similarly to the single rocking block, the size of ancient monuments affects their dynamic response and their vulnerability to earthquakes, with larger structures being more stable than smaller ones. This is shown in Fig. 24.7, in which the minimum acceleration amplitude of a harmonic excitation of varying period, required to cause collapse (stability threshold), is shown for two cases: (a) the columns of the temple of Apollo at Bassae, Greece, of height 5.95 m; and (b) the columns of the temple of Zeus at Nemea, Greece, of height 10.33 m (Psycharis et al. 2000). Results are given for the free-standing column and the set of two columns connected with an architrave. It is seen that, for the same period of excitation, significantly larger acceleration is needed to overturn the larger columns of Zeus compared with the smaller columns of Apollo.

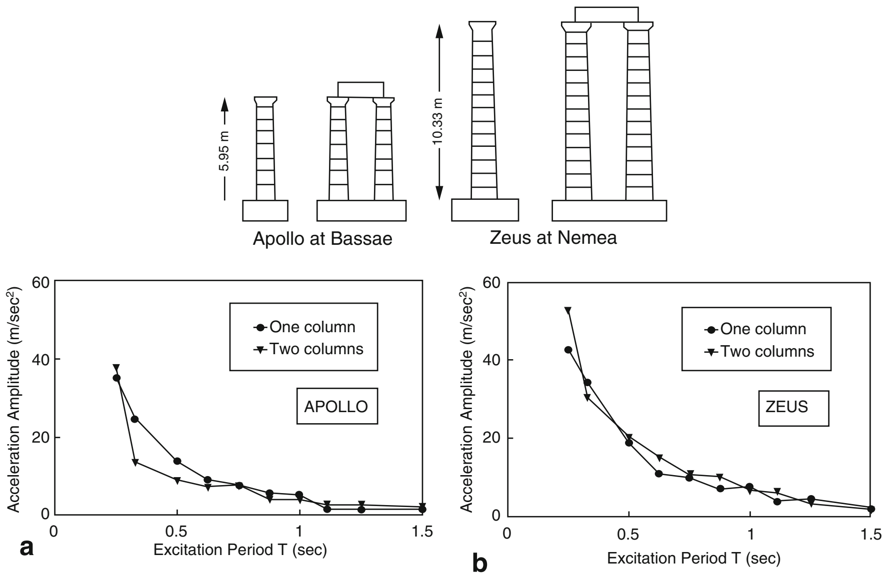


Fig. 24.7 Minimum acceleration amplitude of harmonic excitations required for the collapse of free-standing columns and sets of two columns connected with an architrave: (**a**) columns of the temple of Apollo; (**b**) columns of the temple of Zeus (Psycharis et al. 2000)

The results depicted on Fig. 24.7 also show another interesting observation: the stability threshold of each monument is similar for the free-standing column and the set of two columns. This means that restoration of fallen architraves does not necessarily lead to enhanced stability of the monument against future earthquakes. Figure 24.7 shows that such restoration of the architraves might be favourable or unfavourable depending on the characteristics of the structure and the excitation: in case of Apollo, it was generally unfavourable while in case of Zeus, it was generally favourable.

It should be mentioned that the above observation concerns the in-plane collapse of the columns (2D analyses). However, shaking table tests on sets of columns connected with architraves in line or in corner have shown that the architrave beams are quite vulnerable in the out-of-plane direction, being the first pieces that fall down. The collapse of the architraves endangers the stability of the whole monument, since it is possible that they hit the columns during their fall.

24.4.2 *Effect of Predominant Period of Ground Motion*

The earthquake response of ancient monuments is dominated by the rocking of the drums of the columns, therefore, it is greatly affected by the predominant period of the excitation with low-frequency earthquakes being much more dangerous than high-frequency ones. In this sense, near field ground motions, which contain long-period directivity pulses, might bring these structures to collapse.

The effect of the period of excitation to the risk of collapse in case of harmonic excitations is shown in Fig. 24.7 for the columns of the Temples of Apollo and Zeus. It is seen that, in all cases examined, the stability threshold decreases exponentially as the period of excitation increases. The same trend is also observed in Fig. 24.8, in which the stability of a free-standing column of the Parthenon of Athens, Greece is examined under near-fault earthquake excitations containing a directivity pulse of frequency f_p. In this plot, the threshold between safe (non-collapse) and unsafe (collapse) regions on the PGA–f_p plane for 3500 near-fault simulated earthquake motions with magnitudes M_w ranging from 5.5 to 7.5 and epicentral distances ranging from 0 to 20 km is shown (Psycharis et al. 2013). It is seen that the minimum required PGA for collapse of the column decreases for smaller f_p (larger predominant period).

In general, previous analyses (Psycharis et al. 2000) have shown that low-frequency earthquakes force the structure to respond with intensive rocking, whereas high-frequency ones produce significant sliding of the drums, especially at the upper part of the columns.

These results show that the choice of the earthquakes that will be used in time-history analyses is very important, as the dynamic response and the risk of collapse are sensitive to the energy and frequency content of the time history of the input ground motion. Apart from the above-mentioned strong effect of the predominant period of the ground motion, the time sequence of the various phases in the record

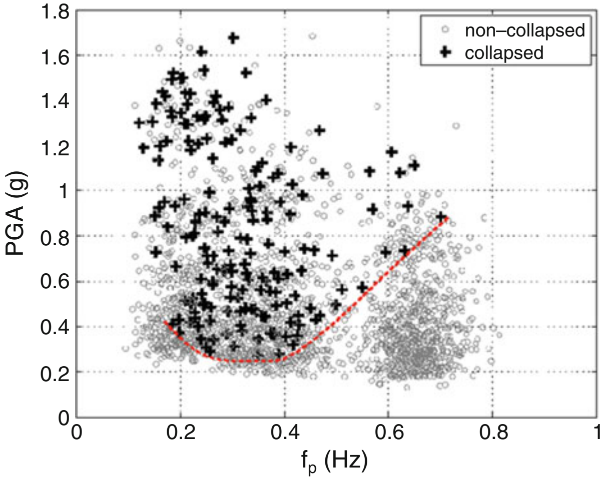


Fig. 24.8 Threshold (red line) between safe and unsafe regions on the *PGA*–f_p plane for a free-standing column of the Parthenon subjected to 3500 near-fault simulated earthquake motions (f_p is the frequency of the predominant pulse contained in each record). (Psycharis et al. 2013)

might also be significant. In this sense, it is essential to constrain the selection of the base excitations to what one may call suitable surrogate ground acceleration time histories that could replicate as closely as possible the time histories of past and anticipated earthquakes.

It is evident, therefore, that the choice of which time-histories to include and which to exclude in order to constrain ground motions is an important decision. There is a balance to be struck between being not restrictive enough in the time histories used, leading to unreliable results and hence predictions due to errors and uncertainties; and being too restrictive, which leads to a too small set of time histories and hence non-conclusive results.

24.4.3 Effect of Existing Damage

Although classical monuments without significant damages are, in general, not vulnerable to usual earthquake motions, collapse can occur much easier if imperfections are present. In their current condition, ruins of ancient structures present many different types of damage (Fig. 24.9). Most common are: missing pieces (cut-offs) that reduce the contact areas, foundation problems resulting in tilting of the columns, dislocated drums from previous earthquakes and cracks in the structural elements that, in some cases, split the blocks in two parts. Such imperfections may endanger the safety of the structure in future earthquakes.

An example of the effect of existing imperfections on the stability of ancient columns is shown in Fig. 24.10 for the free-standing column of the Parthenon of

Fig. 24.9 Classical columns with significant drum dislocations. Left: Columns at Propylaia of the Acropolis of Athens, Greece; Right: Column of the temple of Hera in Samos, Greece

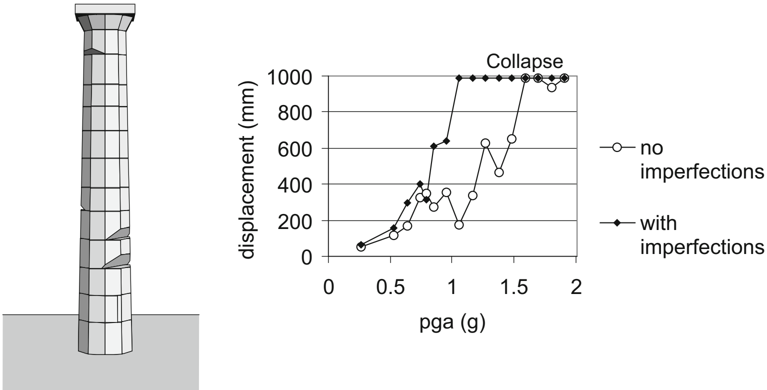


Fig. 24.10 Maximum permanent displacements of a free-standing column of the Parthenon under the Aigion, Greece (1995) earthquake amplified to several values of *PGA* without and with the imperfections shown in the left diagram (Psycharis et al. 2003)

Athens (Psycharis et al. 2003), where the maximum permanent displacement of the column is plotted versus the *PGA* of the ground motion. It is seen that the presence of the imperfections shown in the left drawing of Fig. 24.10 leads to larger displacements and significantly earlier collapse.

Similar results were obtained when the column of the Propylaia of the Acropolis of Athens with the dislocated drums (left photo in Fig. 24.9) was

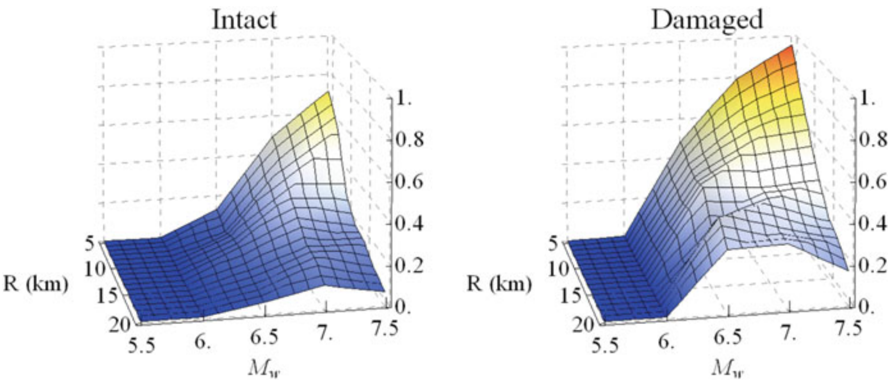


Fig. 24.11 Collapse probabilities for the intact and the damaged column of the Propylaia of the Acropolis of Athens (left photo in Fig. 24.9). The damaged column is evidently more prone to collapse (Fragiadakis et al. 2016b)

subjected to 3500 near-fault simulated earthquake motions with magnitudes M_w ranging from 5.5 to 7.5 and epicentral distances ranging from 0 to 20 km (Fragiadakis et al. 2016b). In Fig. 24.11, the collapse probabilities of the intact and the damaged column are presented as function of earthquake magnitude and distance. Evidently, the damaged column is clearly more prone to collapse compared to the one that is intact.

24.5 Performance-Based Reliability Assessment

Performance-Based Earthquake Engineering (PBEE) and seismic risk assessment combine computational tools and reliability assessment procedures to obtain the system fragility for a wide range of limit states. The seismic risk assessment requires the calculation of the failure probabilities of a pre-set number of performance objectives. According to PBEE, the acceptable level of damage sustained by a structural system depends on the level of ground shaking and its significance. Thus, the target in risk assessment is to obtain the probabilities of violating the stated performance levels, ranging from little or no damage for frequent earthquakes to severe damage for rare events.

Today, these concepts are well understood among earthquake engineers, but when classical monuments are considered the performance-based criteria may differ considerably. For example, to retrofit an ancient column one has to decide what is the 'acceptable level' of damage for a given intensity level. The approach for making such decisions is not straightforward. A consensus among various experts in archaeology and monument preservation is necessary, while a number of non-engineering decisions have to be taken.

In order to assess the risk of a monument, the performance levels of interest and the corresponding levels of capacity of the monument need first to be decided. Demand and capacity should be measured with appropriate parameters at critical locations, in accordance to the different damage (or failure) modes of the structure. Subsequently, this information has to be translated into one or a combination of engineering demand parameters (*EDP*s), e.g., permanent or maximum column deformation, drum dislocation, foundation rotation or maximum axial and shear stresses. For the *EDP*s chosen, appropriate threshold values that define the various performance objectives e.g. light damage, collapse prevention, etc. need to be established.

In case of classical columns, two engineering demand parameters (*EDP*s) have been suggested by Psycharis et al. (2013) for the assessment of their vulnerability, namely: (a) the maximum displacement at the capital normalized by the base diameter; and (b) the relative residual dislocation of adjacent drums normalized by the diameter of the corresponding drums at their interface. The first *EDP* is the maximum of the normalized displacement of the capital (top displacement) over the whole time history and is denoted as u_{top}, i.e. $u_{top} = \max[u(\text{top})]/D_{base}$. This is a parameter that provides a measure of how much a column has been deformed during the ground shaking and also shows how close to collapse the column was brought during the earthquake. Note that the top displacement usually corresponds to the maximum displacement among all drums. The second *EDP* is the residual relative drum dislocations at the end of the seismic motion normalised by the drum diameter at the corresponding joints and is denoted as u_d, i.e. $u_d = \max(\text{res}u_i)/D_{i.}$ This parameter provides a measure of how much the geometry of the column has been altered after the earthquake increasing thus the vulnerability of the column to future events.

These *EDP*s have a clear physical meaning and allow to easily identify various damage states and set empirical performance objectives. For example a u_{top} value equal to 0.3 indicates that the maximum displacement was 1/3 of the bottom drum diameter and thus there was no danger of collapse, while values of u_{top} larger than unity imply intense shaking and large deformations of the column, which, however, do not necessarily lead to collapse. It is not easy to assign a specific value of u_{top} that corresponds to collapse, as collapse depends on the 'mode' of deformation, which in turn depends on the ground motion characteristics. For example, for a cylindrical column that responds as a monolithic block with a pivot point at the corner of its base (Fig. 24.12a), collapse is probable to occur for $u_{top} > 1$, as the weight of the column turns to an overturning force from a restoring one when u_{top} becomes larger than unity. But, if the same column responds as a multi-drum one with rocking at all joints (Fig. 24.12b), a larger value of u_{top} can be attained without threatening the overall stability. In fact, the top displacement can be larger than the base diameter without collapse, as long as the weight of each part of the column above an opening joint gives a restoring moment about the pole of rotation of the specific part.

Based on the above defined *EDP*s, the performance criteria of Tables 24.1 and 24.2 have been adopted. For u_{top}, three performance levels were selected (Table 24.1), similarly to the ones that are typically assigned to modern structures.

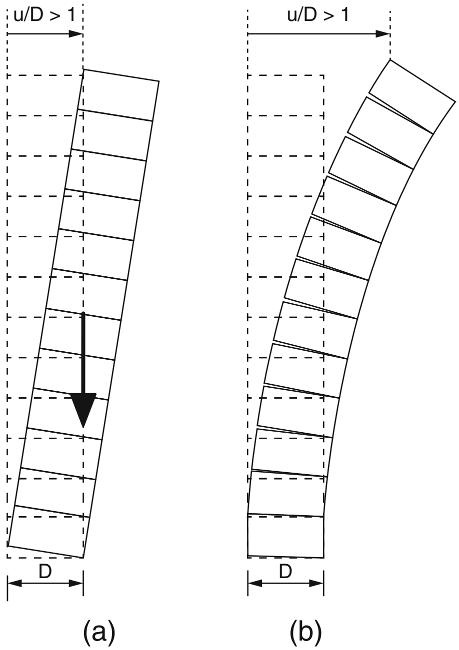


Fig. 24.12 Top displacement for two extreme modes of rocking: (**a**) as a monolithic block; (**b**) with opening of all joints (displacements are shown exaggerated) (Psycharis et al. 2013)

Table 24.1 Performance criteria concerning the risk of collapse (Psycharis et al. 2013)

u_{top}	Performance level	Description
0.15	Damage limitation	No danger for the column. No permanent drum dislocations expected.
0.35	Significant damage	Large opening of the joints with probable damage due to impacts and considerable residual dislocation of the drums. No serious danger of collapse.
1.00	Near collapse	Very large opening of the joints, close to partial or total collapse.

The first level (*damage limitation*) corresponds to weak shaking of the column with very small or no rocking. At this level of shaking, no damage, nor any severe residual deformations are expected. The second level (*significant damage*) corresponds to intense shaking with significant rocking and evident residual deformation of the column after the earthquake; however, the column is not brought close to collapse. The third performance level (*near collapse*) corresponds to very intense shaking with significant rocking and probably sliding of the drums. The column does not collapse at this level, as $u_{top} < 1$, but it is brought close to collapse. In most cases, collapse occurred when this performance level was exceeded. The values of u_{top} that are assigned at every performance level are based on the average assumed risk of collapse.

Three performance levels were also assigned to the normalised residual drum dislocation, u_d (Table 24.2). This *EDP* is not directly related to how close to collapse

Table 24.2 Performance criteria concerning permanent deformation (residual drum dislocations) (Psycharis et al. 2013))

u_d	Performance level	Description
0.005	Limited deformation	Insignificant residual drum dislocations without serious effect to future earthquakes
0.01	Light deformation	Small drum dislocations with probable unfavourable effect to future earthquakes
0.02	Significant deformation	Large residual drum dislocations that increase significantly the danger of collapse during future earthquakes

the column was brought during the earthquake, since residual displacements are caused by wobbling and sliding and are not, practically, affected by the amplitude of the rocking. However, their importance to the response of the column to future earthquakes is significant, as previous damage/dislocation has generally an unfavourable effect to the seismic response to future events (Psycharis 2007).

The first performance level (*limited deformation*) concerns very small residual deformation which is not expected to affect considerably the response of the column to future earthquakes. The second level (*light deformation*) corresponds to considerable drum dislocations that might affect the dynamic behaviour of the column to forthcoming earthquakes, increasing its vulnerability. The third performance level (*significant deformation*) refers to large permanent displacements at the joints that increase considerably the danger of collapse to future strong seismic motions. It must be noted, however, that the threshold values assigned to u_d are not obvious, as the effect of pre-existing damage to the dynamic response of the column varies significantly according to the column properties and the characteristics of the ground motion. The values proposed are based on engineering judgment taking into consideration the size of drum dislocations that have been observed in monuments and also the experience of the authors from previous numerical analyses and experimental tests.

This approach was applied to the free-standing column of the Parthenon of Athens subjected to 3500 near-fault simulated earthquake motions with magnitudes M_w ranging from 5.5 to 7.5 and epicentral distances ranging from 5 to 20 km (Psycharis et al. 2013). The comparison of the two proposed *EDP*s is shown in Fig. 24.13 for all ground motions considered excluding those that caused collapse. Although there is a clear trend showing that, generally, strong ground motions lead to large top displacements u_{top} during the strong shaking and also produce large permanent deformation u_d of the column, there is significant scattering of the results indicating that intense rocking does not necessarily imply large residual dislocations of the drums and also that large drum dislocations can occur for relatively weak shaking of the column. This was also observed during shaking table experiments (Mouzakis et al. 2002) where cases of intense rocking with very small residual drum displacements have been identified.

The proposed fragility assessment methodology can be applied to derive fragility curves or surfaces. For example, for the free-standing column of the Parthenon

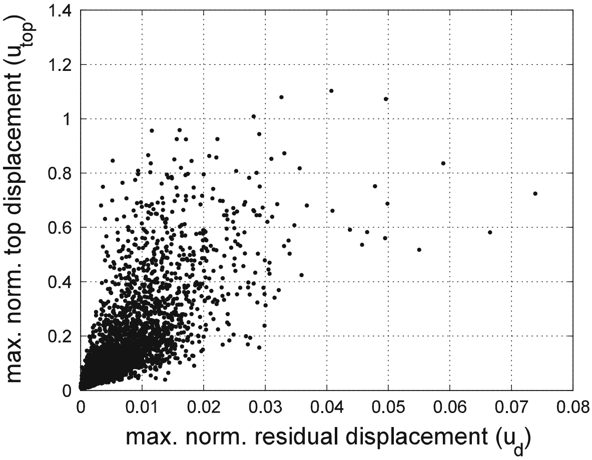
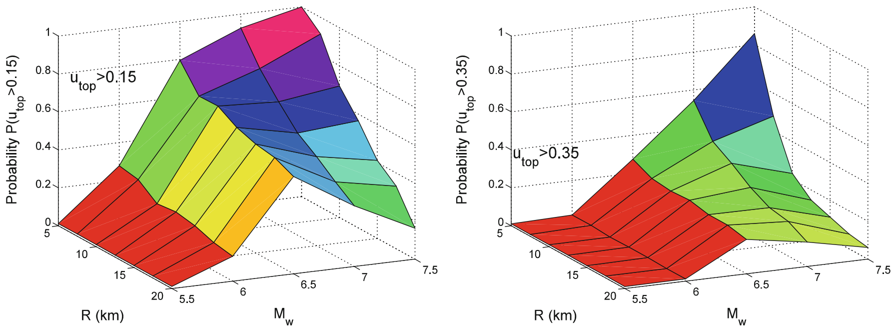


Fig. 24.13 Comparison of u_d versus u_{top} for the free-standing column of Parthenon subjected to 3500 synthetic near-fault ground motions, excluding ground motions causing collapse (Psycharis et al. 2013)



(a) (b)

Fig. 24.14 Fragility surfaces of the Parthenon column with respect to the maximum capital displacement u_{top} for the performance levels of Table 24.1: (**a**) $u_{top} > 0.15$; (**b**) $u_{top} > 0.35$ (Psycharis et al. 2013)

Fig. 24.14 shows the fragility surfaces for two performance levels of Table 24.1 corresponding to the above-mentioned 3500 simulated near-fault ground motions. It is seen that for both *damage limitation* and *significant damage*, the exceedance probability generally increases for ground shakings of larger magnitude. However, the exceedance probability decreases with magnitude in the range $M_w = 6.5$–7.5 and $R > 15$ km. This counter-intuitive response, which was verified for real earthquakes as well, is attributed to the saturation of the *PGV* for earthquakes with magnitude larger than $M_{sat} = 7.0$ (e.g. see Rupakhety et al. 2011) while the period of the pulse is increasing exponentially with the magnitude. As a result, the directivity pulses haves small acceleration amplitude for large magnitudes, which is not capable to produce intense rocking.

Figure 24.15 shows the fragility surfaces when the *EDP* is the normalized permanent drum dislocation, u_d, and considering the performance levels of

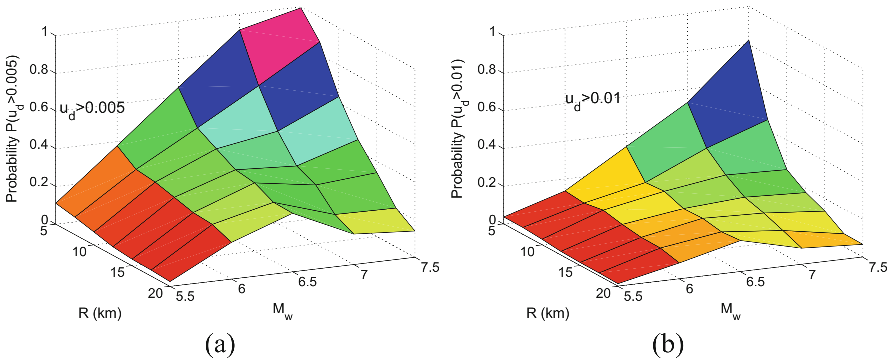


Fig. 24.15 Fragility surfaces of the Parthenon column with respect to the permanent drum dislocations, u_d for the performance levels of Table 24.2: (**a**) $u_d > 0.005$; (**b**) $u_d > 0.01$ (Psycharis et al. 2013)

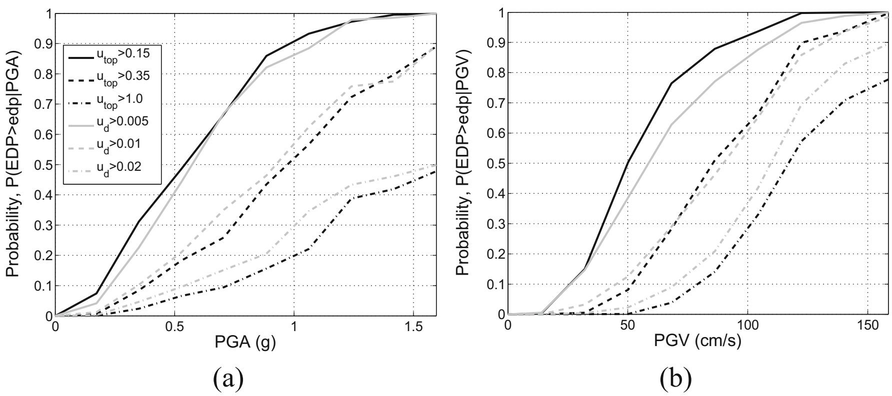


Fig. 24.16 Fragility curves of the Parthenon column using different intensity measures: (**a**) peak ground acceleration; (**b**) peak ground velocity

Table 24.2. For the *limited deformation* limit state ($u_d > 0.005$), probabilities around 0.3 are observed for magnitudes close to 6. Note that, for the column of the Parthenon with an average drum diameter about 1600 mm, $u_d > 0.005$ refers to residual displacements at the joints exceeding 8 mm. The probability of exceedance of the *light deformation* performance criterion ($u_d > 0.01$), which corresponds to residual drum dislocations larger than 16 mm, is less than 0.2 for all earthquake magnitudes examined and for distances from the fault larger than 10 km.

Finally, fragility curves for the *EDP*s thresholds defined in Tables 24.1 and 24.2 and using *PGA* and *PGV* as intensity measures, are shown in Fig. 24.16. It is seen that the probability that a moderate earthquake with *PGA* ~ 0.3 g and *PGV* ~ 40–50 cm/s has only 10% probability to cause considerable rocking to the column with $u_{top} > 0.35$ and to produce permanent dislocations of the drums that exceed 1% of their diameter.

24.6 Summary

In this chapter the main parameters that affect the vulnerability of classical monuments to earthquakes are presented and discussed. Based on the results of previous studies, the main features of the response can be summarized as follows:

- Owing to rocking and sliding, the response is nonlinear. The nonlinear nature of the response is pronounced even for the simplest case of a rocking single block. In addition, multi-drum columns can rock in various 'modes', which alternate during the response increasing thus the complexity of the problem. The word 'mode' denotes the pattern of rocking motion rather than a natural mode in the classical sense, since rocking structures do not possess such modes and periods of oscillation.
- The dynamic behaviour is sensitive to even trivial changes in the geometry of the structure or the base-motion characteristics. The sensitivity of the response has been verified experimentally, since 'identical' experiments produced significantly different results in some cases. The sensitivity of the response is responsible for the significant out-of-plane motion observed during shaking table experiments for purely planar excitations.
- The vulnerability of the structure greatly depends on the predominant period of the ground motion, with earthquakes containing low-frequency pulses being in general much more dangerous than high-frequency ones. The former force the structure to respond with intensive rocking, whereas the latter produce significant sliding of the drums, especially at the upper part of the columns.
- The size of the structure affects significantly the stability, with bulkier structures being much more stable than smaller ones of the same slenderness.
- Classical monuments are not, in general, vulnerable to earthquakes. However, their stability might have been significantly reduced in the damaged condition that they are found today. Types of damage that might increase their vulnerability to earthquakes include cut-off of drums, displaced drums, inclined columns due to foundation failure, cracks in the stones, etc.
- Two engineering demand parameters (*EDP*s) are adopted for the assessment of the vulnerability of classical columns in terms of PBEE: (a) the maximum displacement at the capital normalized by the base diameter; and (b) the relative residual dislocation of adjacent drums normalized by the diameter of the corresponding drums at their interface. Three performance levels are assigned to each *EDP* and the values of the corresponding thresholds are proposed.

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