



Preliminary Seismic Risk Assessment of Monolithic Columns of the Aphaia Temple in Aegina

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Abstract. Studies on the seismic assessment of monumental structures of antiquity are mostly focused on the estimation of the structural behavior and do not involve the pertinent uncertainties. Towards filling this research gap, a preliminary seismic risk assessment of monolithic columns of the Aphaia Temple in Aegina island, Greece, is presented. A comprehensive application of the framework of Performance-Based Earthquake Engineering is carried out. Site-specific seismic hazard estimation is performed at first by employing Probabilistic Seismic Hazard Analysis. The rocking column is analyzed under seismic excitation by numerically solving the equation of motion in order to extract the fragility curves. Finally, the convolving of seismic hazard and structural response yields the seismic risk of the column that is used to estimate the mean annual rate of exceeding predefined limit states that are associated with damage and collapse of the column.

Keywords: Seismic hazard · Record selection · Rocking block · Limit states · Fragility curves · Seismic risk assessment

1 Introduction

The protection of cultural heritage has drawn the attention of engineers, scientists, and the general public during the last decades. In Greece, numerous monuments of classical antiquity are scattered throughout its territory. Most of these monuments have been exposed to various environmental actions during their lifetime and consequently, they are already in some stage of deterioration. The evaluation of their seismic performance is a very complex mathematical and computational problem as the marble structural elements of classical antiquity monuments are typically not rigidly connected and thus susceptible to rocking.

Numerous studies have been conducted on the seismic assessment of monuments, indicatively, Dasiou et al. (2009), Psycharis et al. (2013), Makris and Vassiliou (2013), Psycharis (2018). However, most of these studies are limited to the estimation of structural behavior and do not involve the incorporation of pertinent uncertainties. The aim of the present study is to contribute towards the estimation of the seismic risk of monuments

of classical antiquity. To that effect, a comprehensive application of Performance-Based Earthquake Engineering, initially developed by Cornell and Krawinkler (2000), is carried out, comprising four inter-related steps: (1) estimation of the seismic hazard in terms of a scalar intensity measure via a comprehensive seismic source model, (2) use simplified structural models to enable a multi-fidelity estimation of the response distribution, as characterized by appropriate engineering demand parameters that can be connected to damage, (3) definition of appropriate limit-state thresholds and the corresponding aleatory and epistemic uncertainties to allow determining discrete damage states and associated fragility curves, and (4) assessment of the seismic risk in terms of the mean annual frequency of exceeding a limit state via the convolution of fragility and seismic hazard. The methodology is applied to monolithic columns of the Temple of Aphaia, Greece.

The Temple of Aphaia is located within a sanctuary complex in Aegina island, Greece, which is located 27 km southwest of Athens in the Saronic Gulf. The Temple of Aphaia (Fig. 1) was dedicated to the mother-goddess Aphaia and was founded circa 500 BC in the Late Archaic phase (510 to 470 BC) of the sanctuary. The Temple was a hexastyle peripteral Doric order structure of a 6×12 column plan on a $15.5 \text{ m} \times 30.5 \text{ m}$ platform (www.wikipedia.org). All columns, except three, were monolithic. The remains of the Temple today include free-standing columns, colonnades of two or more columns with epistyles, and an internal segment with a two-story colonnade. In general, the remnants of the Temple have suffered non-negligible damage.



Fig. 1. North view of the Temple of Aphaia, Aegina Island, Greece (courtesy of the authors)

The selected monolithic free-standing column is located in the southeast corner of the Aphaia Temple and has a total height of 5.29 m including the capital. The column is presented in Fig. 2(a) and its drawing is shown in Fig. 2(b). It is noted that within the present preliminary seismic risk assessment, fractures and damages of the column are not considered.

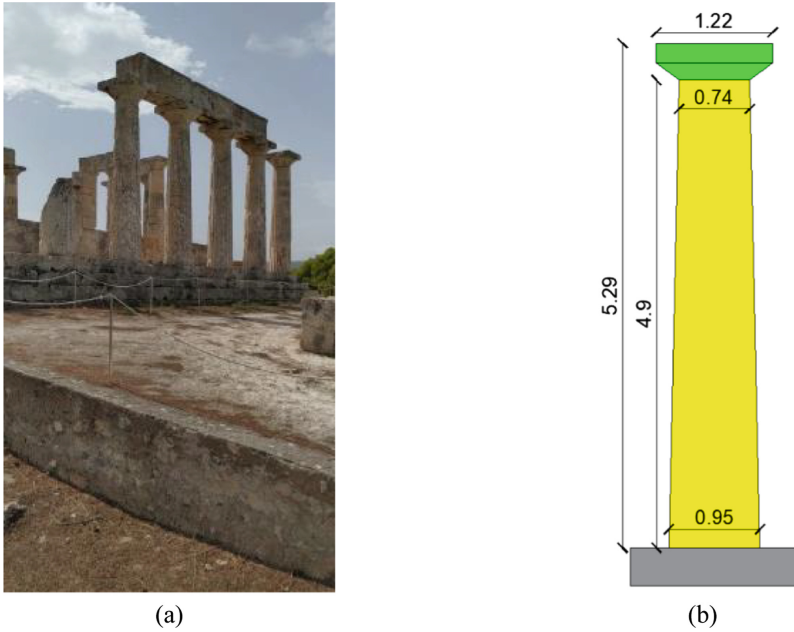


Fig. 2. Selected free-standing monolithic column of the Aphaia Temple: (a) column photo (courtesy of the authors) and (b) drawing

2 Seismic Hazard

Ancient monuments are structures of significant social and cultural importance and when it comes to seismic assessment, the application of probabilistic methods is practically a one-way street. The appropriate tool to estimate the seismic hazard is the Probabilistic Seismic Hazard Analysis (PSHA) (Baker 2008). In PSHA, two approaches are available, the so-called classic (Cornell 1968) and the event-based. Herein, the classical approach is employed, as only a single site is considered. Cotemporaneous consequences over multiple sites can be evaluated via an event-by-event view of seismic ruptures, which is beyond the scope of our study.

PSHA is performed for the IM of $AvgSA$, which is the geometric mean of the log spectral acceleration $Sa(T_1)$ at a set of periods of interest (Bianchini et al. 2009). The range of periods is set from 0.1 s to 1.5 s. It is noted that ancient columns do not possess natural modes in the classical sense since the period of free vibrations is amplitude-dependent (Psycharis 2018) and consequently T_1 is not defined. Still, the lower periods tend to govern early damage, as the peak ground acceleration determines uplift, while longer periods (e.g., moderate periods indicative of peak ground velocity) tend to correlate well with overturning (Giouvanidis and Dimitrakopoulos 2018). Thus, the range of periods selected stands as a reasonable engineering assumption.

The seismic hazard analysis for the site of interest (Temple of Aphaia) is performed using the open-source platform OpenQuake (GEM 2016) and using the latest European seismic source model (SHARE) (Woessner et al. 2015). A comprehensive logic tree

is available in SHARE to account for epistemic uncertainties. In this study, only the area source model is employed together with the Ground Motion Prediction Equation (GMPE) of Boore and Atkinson (2008). Calculations were performed for soil type “rock” ($V_{s30} = 800$ m/s) and the resulting hazard curve is shown in Fig. 3.

Then, 16 hazard levels are considered, corresponding to probabilities of exceedance ranging from 0.0001 to 0.90 in 50 years, or equivalently to mean annual frequencies ranging from 0.046052 to 0.000020. The hazard levels and the corresponding values of AvgSA obtained from the hazard curve (Fig. 3) are tabulated in Table 1.

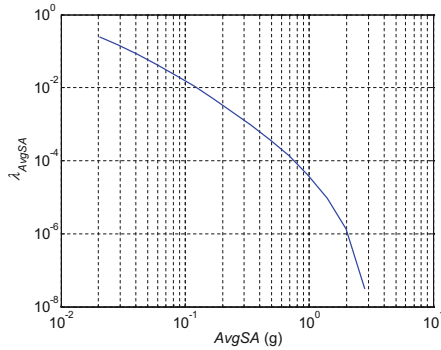


Fig. 3. Seismic hazard curve for the Temple of Aphaia site (island of Aegina), displaying the mean annual frequency (λ_{AvgSA}) of exceeding AvgSA (intensity measure)

Table 1. Hazard levels obtained from seismic hazard curve

No	Mean annual frequency	AvgSA (g)	No	Mean annual frequency	AvgSA (g)
1	0.046052	0.057	9	0.003250	0.212
2	0.032189	0.069	10	0.002107	0.247
3	0.024079	0.081	11	0.001026	0.333
4	0.018326	0.094	12	0.000404	0.477
5	0.013863	0.108	13	0.000201	0.632
6	0.008616	0.136	14	0.000100	0.797
7	0.005754	0.162	15	0.000040	0.991
8	0.004463	0.182	16	0.000020	1.253

Subsequently, hazard disaggregation (Bazzurro and Cornell 1999) is performed using OpenQuake for the predefined hazard levels in order to obtain the magnitude – distance – epsilon distribution of the events that contribute to each hazard level. On the basis of this data, the Conditional Spectrum methodology (Lin et al. 2013) was employed to perform hazard-consistent record selection based on AvgSA (Kohrangi et al. 2017) from the PEER database (<https://ngawest2.berkeley.edu/site>). In particular, 30 records have been selected for each hazard level.

3 Structural Analysis

The response of classical columns to earthquake excitation follows the dynamics of rocking rigid blocks. This rocking response is highly nonlinear and extremely sensitive to initial conditions and system parameters. Housner (1963) was the first to carry out a systematic investigation of the problem and developed the equation for the rocking response of a rigid block:

$$I_o \ddot{\theta} + mgR \sin[\alpha - \theta \operatorname{sgn}(\theta)] \operatorname{sgn}(\theta) = +m \ddot{u}_g R \cos[\alpha - \theta \operatorname{sgn}(\theta)] \quad (1)$$

where I_o is the moment of inertia of the block with respect to its point of rotation, \ddot{u}_g is the earthquake excitation, $\operatorname{sgn}(\theta)$ denotes the signum function, R is the radial distance of the center of mass to the center of rotation, and α is the block (or column) slenderness. Equation (1) is applied for the structural analysis of the column by assuming that neither sliding nor bouncing takes place. Housner (1963) has developed Eq. (1) for rocking rigid blocks.

The shape of the monolithic column including the capital is geometrically transformed into an equivalent rectangular shape, considering that the height difference between the center of mass of the original column and the equivalent shape is lower than 5%, as schematically illustrated in Fig. 4(a), (b). It is noted that this is a typical transformation carried out by researchers in case of examining the rocking response of columns using analytical expressions. Under the assumption that the capital is rigidly connected to the column, Makris and Kampas (2016) and Kavvadias et al. (2017) have, also, examined a monolithic column of the Aphaia Temple using the same approach. Furthermore and regarding the selection of the shape, it has been found that the difference between the rectangular and the cylindrical shape on the mass moment of inertia is below 0.21%. If a truncated cone shape was adopted, as indicatively by Manos and Demosthenous (1996), then the capital would have to be ignored.

The main geometric characteristics of the equivalent block are $R = 2.687$ and $\alpha = 0.178$. For the rocking response, the coefficient of restitution is equal to $1 - 3/2 \sin^2 \alpha \approx 0.96 < 1$ (Housner 1963) and is defined as the ratio between the post-impact and the pre-impact angular velocity. Housner's coefficient of restitution is a geometric property and can be calculated a priori for the rocking block if there is no material dissipation (Čeh et al. 2018). The response of the column is evaluated via the Engineering Demand Parameter (EDP), which is the quantity used to predict the damage. The selected EDP is the peak displacement at the capital (i.e. at the top) normalized by the base diameter (Psycharis et al. 2013):

$$u_{top} = \frac{\max_t(u_{top})}{D_{base}}. \quad (2)$$

The EDP definition is shown in Fig. 4(c), where D_{base} is the diameter at the column base, θ is the column rotation, R is the radial distance of the center of mass to the center of rotation, h is the column height and α is the column slenderness. The adopted EDP after Psycharis et al. (2013) has a physical meaning in terms that collapse is probable to occur for $u_{top} > 1$ because the column's weight becomes a pure overturning force, while

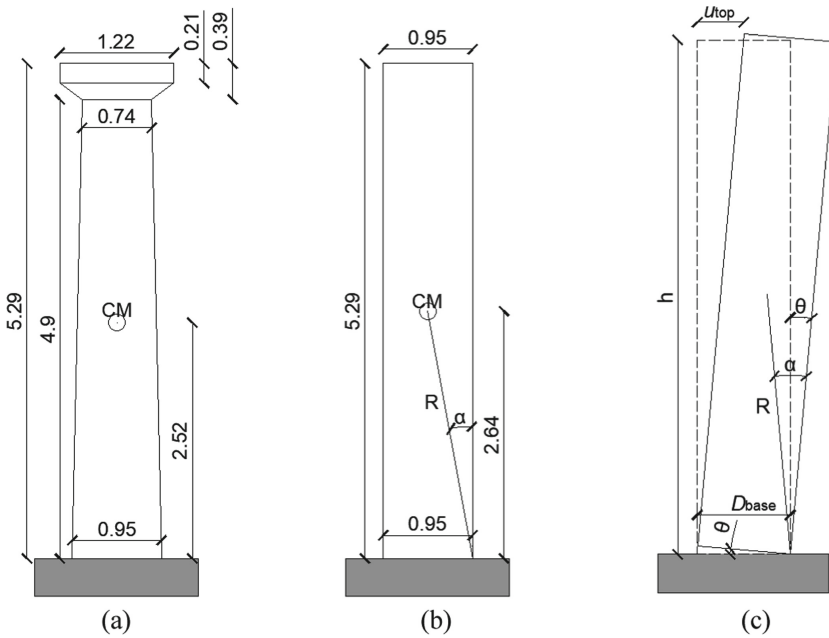


Fig. 4. (a) Original column shape and (b) equivalent rectangular shape, (c) EDP definition for the monolithic column

for $u_{top} = 0.30$ the maximum displacement at the top is one-third of the base diameter, indicating no danger of collapse.

In order to characterize the damage of a structure, it is necessary to introduce discrete damage states or limit states (LS), each with distinct consequences to the structure (Bakalis and Vamvatsikos 2018). The performance criteria (limit states) proposed by Psycharis et al. (2013) for the collapse risk of ancient columns are adopted (Table 2). It is noted that these performance criteria have been proposed for multidrum columns, rather than the monolithic one employed here, yet they are still indicative of similar levels of damage.

Table 2. Performance criteria and associated limit states for single classical antiquity columns after Psycharis et al. (2013)

LS	u_{top}	Performance level
LS1	0.15	Damage limitation
LS2	0.35	Significant damage
LS3	1.00	Near collapse

Structural analysis of the column is performed by solving Eq. (1) for every record selected (30 in total) at each hazard level (16 in total). The numerical solution of Eq. (1) has been carried out using the software of Prof. Michalis Vassiliou at ETH Zurich (<https://n.ethz.ch/~mvassili/scripts.html>). In fact, a stripe analysis is performed, where each of the 30 records selected per IM level is scaled to match said IM level and used to run response history analysis. The results are shown in Fig. 5, where the intensity measure (AvgSA) is plotted on the vertical axis and the engineering demand parameter (u_{top}) on the horizontal axis. Each one of the 16 stripes (horizontal discrete set of results) corresponds to a single hazard level and it comprises 30 EDP values from the 30 ground motion records employed.

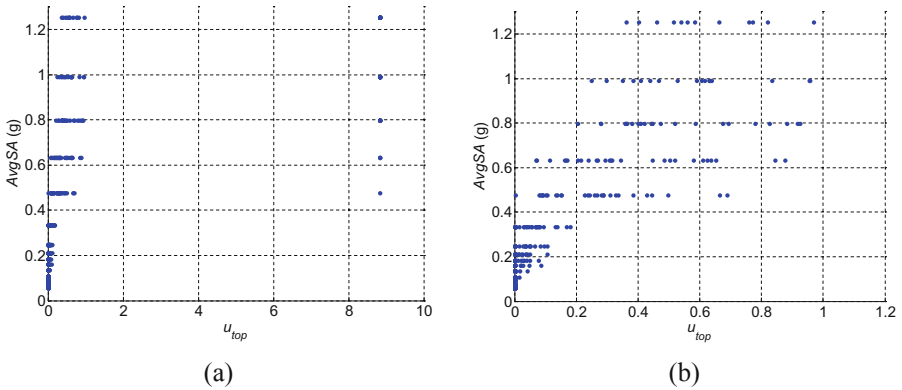


Fig. 5. Stripe analysis results for (a) the full range of EDP values, where the values at $u_{top} = 9$ correspond to collapse, and (b) curtailed range of EDP values, excluding collapse points

After performing the structural analysis of the column, the next step is the fragility definition. A fragility curve is defined as the probability function of violating a predefined limit state given the IM (Bakalis and Vamvatsikos 2018) and is mathematically expressed as follows taking into account that LS violation is defined only by u_{top} that is a single EDP:

$$FS_{LS}(IM) = P[LS\text{violated}|IM] = P[D > C|IM] \tag{3}$$

where IM is the intensity measure, while D and C are the structure’s demand and capacity, respectively. The fragility curves are depicted in Fig. 6 for the limit states of Table 2. At each hazard level, a set of 30 analyses has been carried out, introducing record-to-record variability in the structural response. This source of uncertainty is incorporated via the lognormal fitting of the discrete probability data points back into a single continuous cumulative distribution function (CDF). The AvgSA (intensity measure) is plotted on the horizontal axis and the probability of limit state violation via Eq. (3) is plotted on the vertical axis as $P(EDP > edp | AvgSA)$, where EDP is the demand and edp is the threshold (capacity).

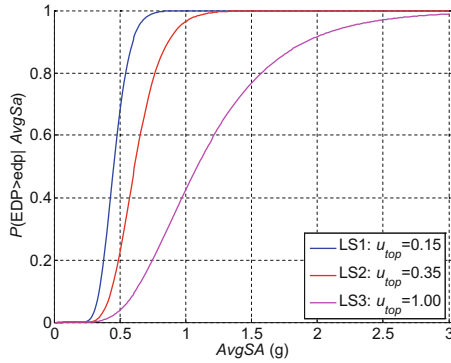


Fig. 6. Column fragility curves

4 Seismic Risk

The seismic risk assessment is based on the simplified expression of the Cornell-Krawinkler framing equation adopted by the Pacific Earthquake Engineering Research (PEER) Center (Cornell and Krawinkler 2000), which can be reduced to a single integral equation (Vamvatsikos and Cornell 2006) for convolving seismic hazard with the fragility in order to estimate the mean annual frequency (MAF):

$$\lambda_{LS} = \int P(EDP > edp | IM) |d\lambda(IM)| \tag{4}$$

where EDP is u_{top} , IM is AvgSA and $P(EDP > edp | IM)$ is the fragility corresponding to limit state LS. The resulting MAF values are listed in Table 3. It is noted that demand uncertainty is indirectly incorporated via the lognormal fitting at the fragility curves, while capacity uncertainty is not considered due to the lack of data. When more information is available, this source of uncertainty can also be incorporated by appropriately inflating the dispersion of the fragility curve (Bakalis and Vamvatsikos 2018).

In order to make results more comprehensible, the MAF of exceeding LS is converted into the probability of exceedance (POE) in 50 years (P_{50}) using the exponential distribution CDF and assuming a Poisson process:

$$P_{50} = 1 - e^{-50MAF} \tag{5}$$

Table 3. Mean annual frequency and probability of exceeding limit states

LS	Performance level	P_{50}	MAF of exceeding LS
LS1	Damage limitation	2.59%	0.000524
LS2	Significant damage	1.20%	0.000251
LS3	Near collapse	0.37%	0.000074

5 Conclusions

Seismic risk assessment within the framework of Performance-based Earthquake engineering is the appropriate tool to estimate the seismic vulnerability of ancient monuments. This approach can provide the necessary information in quantified terms regarding the actual seismic risk of monumental structures, helping to support decision making for any given monument. Herein, this was employed to study a free-standing monolithic column from the Temple of Aphaia in Aegina island, Greece. Its application can be based on a simple model of response, but it requires comprehensive incorporation of all sources of uncertainty, including both epistemic and aleatory sources influencing demand and capacity. Most importantly, though, to control bias, one needs to carefully choose ground motion records, as well as employ an efficient intensity measure to connect hazard and fragility.

The same framework can be extended to cover all adjacent monuments, e.g., the entire Temple and Sanctuary and the nearby museum, where identical ground motions and fully correlated hazard is expected. Extension to multiple sites over an extended region can be based on the same fragility analyses yet also requires further considerations of spatial variability of hazard, to be treated by the more comprehensive event-based approach of PSHA.

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