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SEISMIC RISK ASSESSMENT OF THE ANCIENT TEMPLE OF APHAIA IN GREECE

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Abstract

The protection of cultural heritage against natural hazards has attracted significant research efforts and funding during the last decades, recognizing its importance in humanity's history and raising public awareness on this issue. In Greece, there are numerous monuments that have been exposed to environmental actions, and, consequently, many are classified as deteriorating structures. In addition, earthquakes pose a significant threat to their structural integrity and contribute to the accumulation of damage. The evaluation of the seismic performance of such heritage assets is a complex computational problem, especially as their structural elements are either not rigidly connected, or connected by weak mortar, and thus prone to rocking due to the seismic excitation.

Research on the seismic assessment of monuments is quite limited to the estimation of the structural behavior, thus excluding the incorporation of pertinent uncertainties. The aim of the study is to contribute to the seismic risk assessment of monuments. The framework of Performance-Based Earthquake Engineering is applied, comprising of four successive and interconnected steps: (1) the European seismic source model is used to estimate the seismic hazard in terms of a scalar intensity measure, (2) the structure is modeled with a discrete element approach. The rocking and/or sliding of the individual stone blocks are accurately addressed by the software since, during the calculation, it locates each contact and computes the motion of each block from the forces that are developed at the joints. Results in terms of maximum displacements are obtained from the analysis and related to damage states. (3) The limit-state and the aleatory and epistemic uncertainties are defined for the determination of discrete damage states and the associated fragility curves, and (4) the seismic risk is calculated in terms of the mean annual frequency of exceeding each limit state. The aforementioned methodology is applied to a free-standing column and a colonnade of two columns with an architrave at the ancient Temple of Aphaia, located on the Greek island of Aegina and built between 510 and 470 BC, comprising a significant example of the Archaic architecture.

Keywords: ancient temple; seismic hazard; discrete element modeling; risk assessment

1. Introduction

The protection of cultural heritage has drawn the attention of the scientific community and society since many decades ago, recognizing its importance in humanity's history and raising public awareness on this issue. In Greece, there numerous monuments of antiquity that have been exposed to environmental actions, such as earthquakes, weathering, etc., and are classified now as deteriorating structures. Their structural integrity is threatened by earthquakes, which also contribute to the accumulation of damage. The evaluation of the performance of such structures is a tricky computational problem, especially when the structural elements are not rigidly connected and consequently are subjected to rocking due to seismic excitation.

The seismic assessment of ancient monuments, in general, has been focused on the estimation of the structural behavior (indicatively [1]-[3]), without considering the pertinent uncertainties. Moreover, qualitative methodologies have been developed to address the issues of disaster risk reduction and disaster risk management. Indicatively, Romão et al. [4] have developed a simplified assessment framework for cultural heritage assets for the preliminary assessment of a portfolio of monuments with limited resources. Lagomarsino and Cattari [5] through the EU-funded research project PERPETUATE have formulated guidelines for the seismic performance-based assessment of cultural heritage masonry structures. Maio et al. [6] have provided a holistic framework for the earthquake risk mitigation by reviewing the available methods for disaster risk mitigation of urban cultural heritage assets located in historical centers. In more studies, researchers have worked towards the minimization of seismic risk of cultural heritage assets (indicatively [7]-[9]). However, most of the available studies concern either mortar and stone/brick masonry structures or do not include all pertinent uncertainties, where typically they adopt an intensity basis or fail to incorporate site characteristics to the hazard.

The aim of the present study is to contribute towards the estimation of the seismic risk in a rigorous way by taking advantage of state-of-the-art tools and methodologies within the framework of Performance-Based Earthquake Engineering (PBEE), initially developed by Cornell and Krawinkler [10]. PBEE application for ancient monuments is comprised of four interconnected steps: (1) the European seismic source model is used to estimate the seismic hazard, (2) part of an existing temple is modeled with the software 3DEC using the discrete element method, (3) limit-states and the aleatory and epistemic uncertainties are defined for the determination of discrete damage states and the associated fragility curves, and (4) the seismic risk is calculated in terms of the mean annual frequency of exceeding a limit state. PBEE is applied within this study to a free-standing column and a colonnade of two columns with an architrave at the ancient Temple of Aphaia, located on the Greek island of Aegina.

The Temple of Aphaia, dedicated to the mother-goddess Aphaia, was built between 510-470 BC and is located within a Sanctuary complex in Aegina island, Greece (Fig. 1). The Temple is of peripteral Doric order and is considered as one of the most important and well preserved monuments of archaic architecture. Made of porous limestone, the temple consists of the Opistodomos, the Cella, the Pronaos and the external colonnade. The crepidoma is composed by three layers of squared stone blocks and its uppermost level (15.5m x 30.5m) provides the surface on which the columns and the walls are placed. All columns, except three, are monolithic. The remains of the Temple nowadays include free-standing columns, colonnades of two or more columns with architraves, parts of the Cella walls and an internal segment with a two-story colonnade. In general, the remnants of the Temple have suffered non-negligible damage due to environmental actions, aging, human activity, and perhaps past earthquakes.

Two sub-assemblages of the Temple, namely a free-standing monolithic column (Fig. 2a) and a colonnade of two monolithic columns with an architrave (Fig. 2b), which are part of the external colonnade, are selected for the preliminary seismic risk assessment. The total height of the columns is approximately 5.30m, including the capital, while the dimensions of the two-segment architrave are 2.63x0.61x0.82m (length x width x height). The two segments of the architrave are not structurally connected. Finally, it is noted that within the present preliminary seismic risk assessment, fractures and damages of the columns and the architrave are not considered.

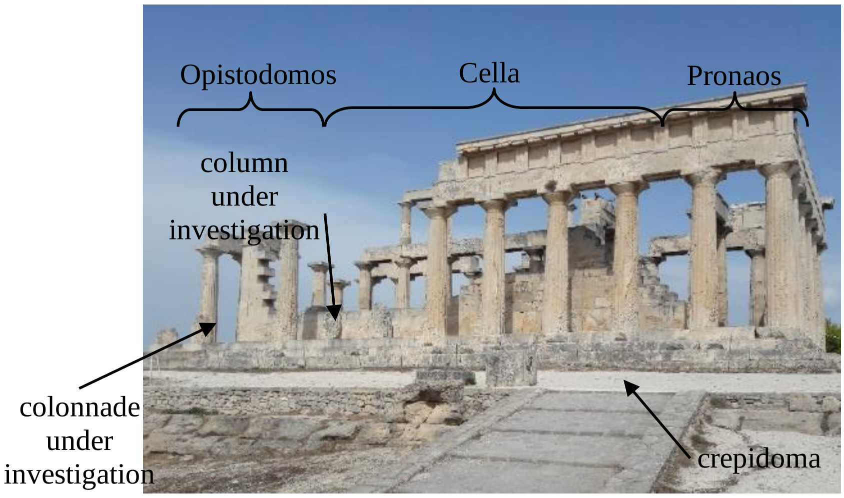
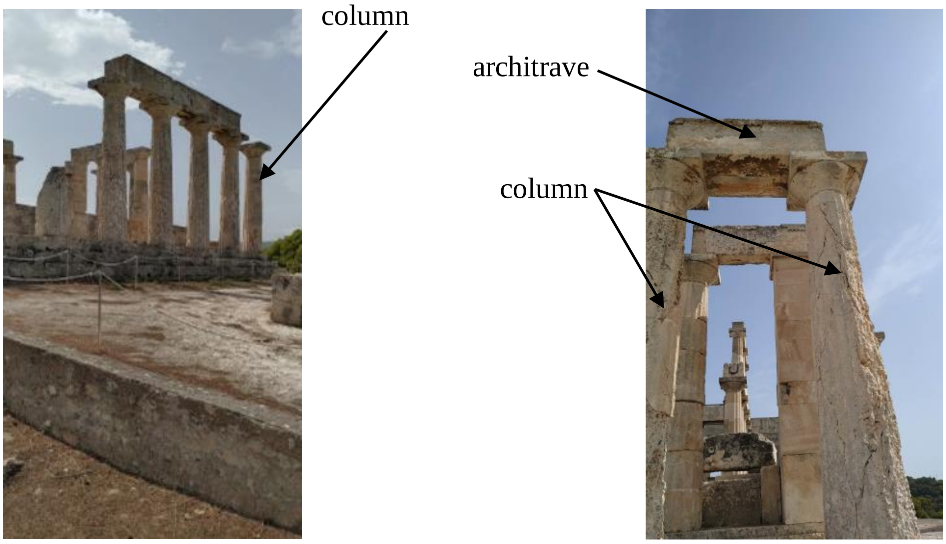


Fig. 1 – North view of Temple of Aphaia [courtesy of the authors]



(a) (b)

Fig. 2 – Selected (a) free-standing monolithic column and (b) colonnade of two monolithic columns with architrave [courtesy of the authors]

2. Seismic Hazard

The seismic risk assessment of ancient structures and especially cultural heritage assets of the antiquity necessitates the implementation of state-of-the-art probabilistic tools. Such appropriate tools to estimate the seismic hazard are provided by the Probabilistic Seismic Hazard Analysis (PSHA) [11]. The so-called classic PSHA [12] is adopted because a single site is considered. The selected intensity measure (IM) is *AvgSA* that is the geometric mean of the log spectral acceleration $S_a(T_1)$ at a set of periods of interest [13]-[17]:

$$AvgSA = \prod_{i=1}^{N} \left[Sa_{gm}(T_{Ri}) \right]^{1/N} \tag{1}$$

where $Sa_{gm}(T_{Ri})$ is the geometric mean of both horizontal components at the i-th reference period T_{Ri}, which is directly approximated by modern Ground Motion Prediction Equations (GMPEs) [18]. The range of periods is set from 0.1sec to 1.5sec. Ancient columns do not possess natural modes in the classical sense because the period of vibration is amplitude-dependent [2] and consequently a fundamental period T_1 cannot be defined. In general, the lower period ordinates affect early damage, similar to the effect of peak ground acceleration to uplift, while longer periods have been found to be correlated to overturning [19]. Therefore, the selected range of periods stands as a reasonable engineering assumption.

The seismic hazard analysis for the site of interest (Temple of Aphaia) is carried out using the open-source platform OpenQuake [20]. The European seismic source model of SHARE [21] is employed with a single area source model together with the GMPE of Boore and Atkinson [22]. The soil type "rock" with $V_{s,30}$ = 800m/sec is considered in the analysis. The resulting site-specific seismic hazard curve is illustrated in Fig. 3 from which three hazard levels are considered for the record-selection procedure. The hazard levels are listed in Table 1 and correspond to 2%, 10% and 20% probability of exceedance in 50 years (P_{50}).

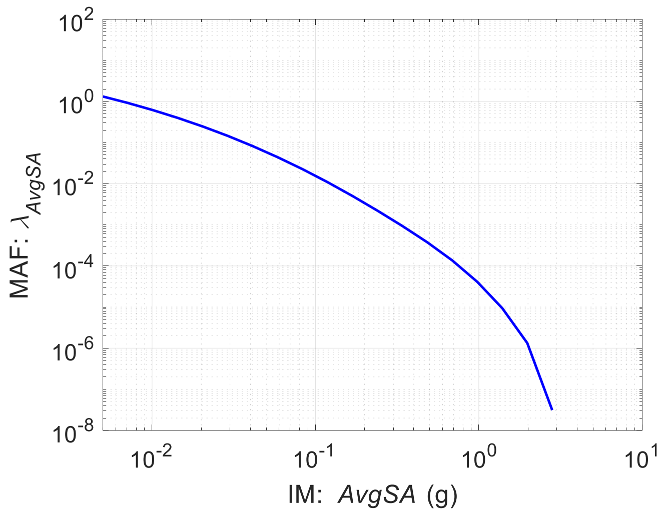


Fig. 3 – Seismic hazard curve for the Temple of Aphaia site (island of Aegina), displaying the mean annual frequency (λ_{AvgSA}) of exceeding given values of *AvgSA*

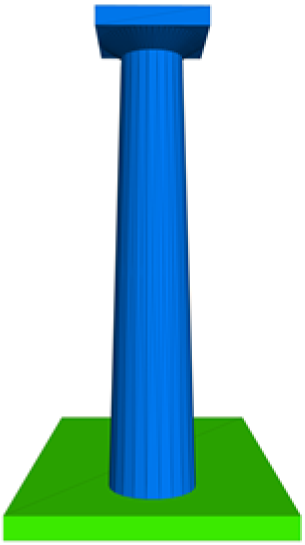
Table 1 – Hazard levels obtained from seismic hazard curve

No	P_{50}	**Return Period**	***AvgSA* (g)**
Hazard Level 1	20%	225 years	0.182
Hazard Level 2	10%	475 years	0.247
Hazard Level 3	2%	2475 years	0.477

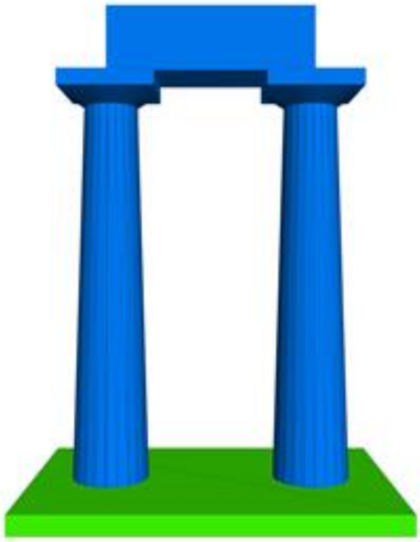
Subsequently, hazard disaggregation [23] is carried out using the OpenQuake platform [20] for the predefined seismic hazard levels in order to obtain the magnitude – distance – epsilon distribution of the events that contribute to each hazard level. Then, using this data, the Conditional Spectrum methodology [24] was utilized to perform hazard-consistent record selection based on *AvgSA* [25] from the PEER database [26]. In particular, 9 records with horizontal and vertical components have been selected.

3. Numerical Modeling

The response of the structural sub-assemblages under investigation follow the dynamics of rigid blocks. The rocking response of rigid bodies is highly nonlinear and extremely sensitive to initial conditions and system parameters. Housner [27] was the first to carry out a systematic investigation of the problem, which still attracts the attention of researchers through analytical and numerical studies, for example, [28]-[30]. The numerical modeling of the sub-assemblages has been carried out using the 3DEC [31] software, which is based on the distinct element method. The structures under investigation were simulated as rigid blocks, while friction and normal and shear stiffness are assigned at the joints with values obtained from Dasiou et al. [32]. The first model, namely the free-standing column is composed of two rigid blocks, the column shaft and the capital, while the second model that represents the colonnade that consists of two monolithic columns and the architrave that is composed of two approximately identical rectangular beams set side-by-side, is simulated with five rigid blocks. It should be noted that the discrete element models of the sub-assemblages, illustrated in Fig. 4, are being developed with the following assumptions: (1) the architectural details of the architrave and the flutes of the columns and (2) the fractures and damages have been disregarded.



(a)



(b)

Fig. 4 – Discrete element models of (a) the free-standing column and (b) the colonnade under investigation

The response of the structures under investigation is evaluated via an Engineering Demand Parameter (EDP) that 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 of the column [3]:

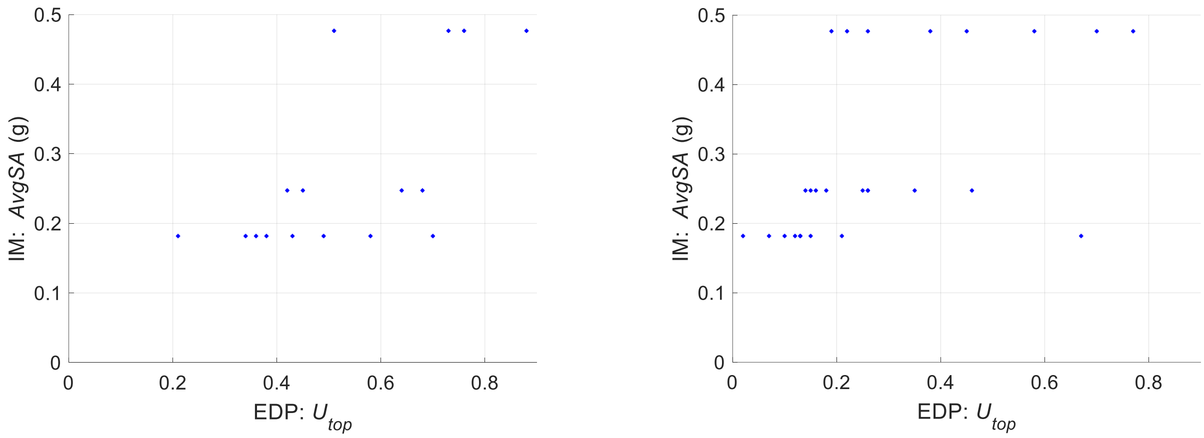
$$U_{top} = \max(u_{top}) / D_{base} \tag{2}$$

where $\max(u_{top})$ is the maximum displacement obtain from the time-history response of the capital for every single time-history analysis of the column and the colonnade and D_{base} is the base diameter of the column (both columns have the same dimensions). The damage of the structures is evaluated through distinct limit states (LS), each of them having distinct consequences to the structure [33]. The performance criteria proposed in [3] are adopted and listed in Table 2. It is noted that these criteria have been proposed for multidrum columns, rather than monolithic, yet they are still indicative of similar levels of damage.

Table 2 – Performance criteria and associated limit states for classical antiquity columns after [3]

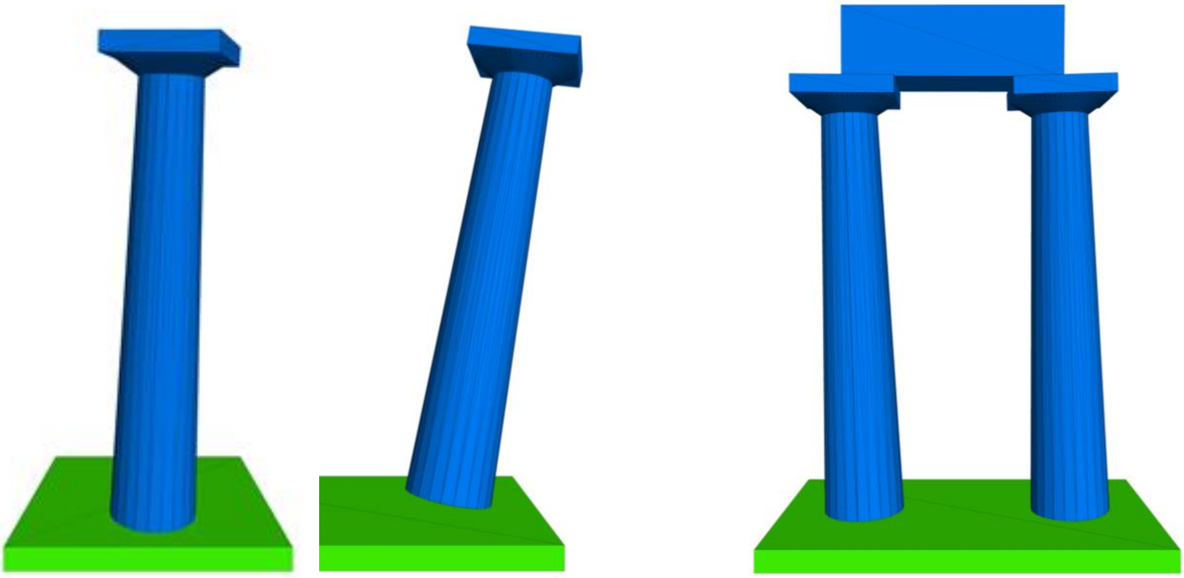
Limit state (LS)	U_{top}	**Performance level**
LS1	0.15	Damage limitation
LS2	0.35	Significant damage
LS3	1.00	Near Collapse

Structural time-history analyses of the sub-assemblages have been performed for every record selected (9 in total) at each hazard level (3 in total). In fact, a stripe analysis is performed, where each of the 9 records selected per IM (*AvgSA*) level is scaled to match said IM level and used to run response time-history analysis. The results are depicted in Fig. 5, where the IM (*AvgSA*) is plotted on the vertical axis and the EDP (U_{top}) on the horizontal axis. Each one of the 3 stripes (horizontal discrete set of results) corresponds to a single hazard level and it comprises 9 EDP values from the 9 ground motion records employed. An indicative shape of the deformed free-standing column and the colonnade are portrayed in Fig. 6a and Fig. 6b, respectively.



(a) (b)

Fig. 5 – Stripe analysis results: (a) free-standing column and (b) colonnade



(a) (b)

Fig. 6 – Indicative shape of the deformed: (a) free-standing column and (b) colonnade during time-history analyses

The next step of the risk assessment is the definition of the fragility that is the probability function of violating a predefined limit state given IM [33] and is calculated via Eq.(3), taking into account that LS violation is defined only by a single EDP, namely U_{top}:

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

where *IM* is the intensity measure, while *D* and *C* are the structure's demand and capacity, respectively. The resulting fragility curves for the limit states of Table 2 are shown in Fig. 8. It is noted that record-to-record variability has been introduced in the structural response by considering a set of 9 analyses at each hazard level. This source of uncertainty is incorporated via the lognormal fitting [34] of the discrete probability data points back into a single continuous cumulative distribution function (CDF) per each LS using the maximum likelihood algorithm. Fragility curves of the free-standing column and the colonnade are displayed in Fig. 7 and Fig. 8, respectively, where the IM is plotted on the horizontal axis and the probability of limit state violation via Eq.(3) is plotted on the vertical axis. The near-collapse limit state has not been examined for the colonnade as it requires rare ground motions of considerable intensity, which have not been considered in the analysis. On the other hand, and regarding the free-standing column, LS1 (damage limitation) is exceeded for all cases examined and the corresponding fragility curve is not presented. Pertinent fragilities may still be estimated for both of these cases if a regression of the EDP-IM relationship ([33],[35]) is employed, to overcome this weakness of fitting a fragility function via the maximum likelihood approach.

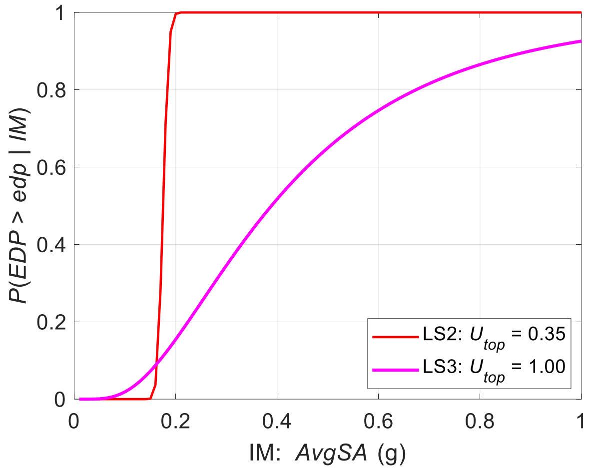


Fig. 7 – Free-standing column fragility curves: Median *AvgSA* = 0.18g / 0.39g, and dispersion β = 0.05 / 0.65 for LS2 and LS3, respectively

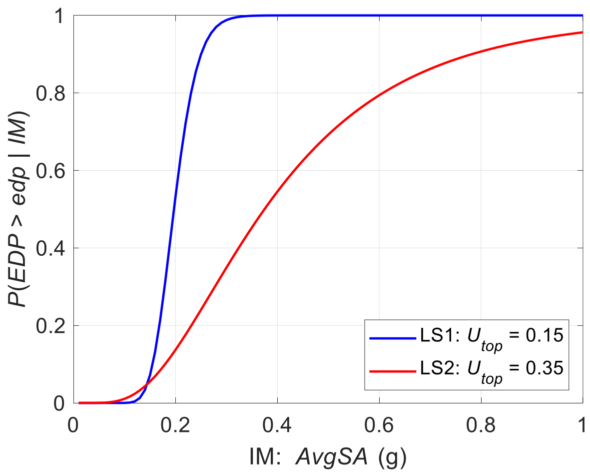


Fig. 8 – Colonnade fragility curves: Median *AvgSA* = 0.20g / 0.37g, and dispersion β = 0.19 / 0.57 for LS1 and LS2, respectively

4. Risk Assessment

The expression of the Cornell-Krawinkler [10] framing equation adopted by the Pacific Engineering Research (PEER) Center is used for the estimation of the seismic risk. This expression has been simplified in [36] in order to convolve seismic hazard with the fragility in order to estimate the mean annual frequency (MAF) of exceeding a predefined limit state:

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

where EDP is U_{top}, IM is *AvgSA* and $P(EDP > edp \mid IM)$ is the fragility corresponding to limit state LS. The resulting MAF values for the limit states are listed in Table 3 for the free-standing column and in Table 4 for the colonnade. It is noted that the demand uncertainty is not directly considered but indirectly incorporated via the lognormal fitting at the fragility curves. Although easily incorporated in Eq.(3), capacity uncertainty is not considered due to a lack of data. Finally, the MAF of exceeding LS is converted into the probability of exceeding (POE) in 50 years using the exponential distribution CDF and assuming a Poisson process. In general, the free-standing column is significantly more vulnerable to earthquake loading than the colonnade with architrave, as also experimentally observed in [37].

$$P_{50} = 1 - \exp(-50MAF) \tag{4}$$

Table 3 – Free-standing column: mean annual frequency and probability of exceeding limit states

Limit state (LS)	**Performance level**	P_{50}	λ_{LS}
LS1	Damage limitation	—	—
LS2	Significant damage	20.71%	0.0046
LS3	Near collapse	9.95%	0.0021

Table 4 – Colonnade: mean annual frequency and probability of exceeding limit states

Limit state (LS)	**Performance level**	P_{50}	λ_{LS}
LS1	Damage limitation	17.41%	0.0038
LS2	Significant damage	8.62%	0.0018
LS3	Near collapse	—	—

5. Conclusions

Performance-based Earthquake Engineering (PBEE) is the appropriate tool for the seismic risk assessment of ancient monuments as it allows the quantifiable estimation of seismic vulnerability. Also, it provides the necessary information to assist and support decision making for any given monument. This procedure has been employed in the present study to evaluate a free-standing column and a colonnade of two columns with architrave, being sub-assemblages of the Temple of Aphaia in Aegina island, Greece. The PBEE framework can be based on a simplified or a more detailed model, as the one employed here. However, it requires the comprehensive incorporation of all sources of uncertainty, including both epistemic and aleatory that influence demand and capacity.

The PBEE framework is an established approach since the 2000s. However, every single application of PBEE to a non-ordinary structure brings new challenges. For a monument of classical antiquity, such as the Temple of Aphaia examined in the present study, two major points of contention appear: (1) the structure consists of rigid blocks that are not structurally connected for either one of which or their ensemble, a period of vibration cannot be defined. So, an essentially period-free intensity measure is needed to base the hazard assessment, record selection, and fragility estimation. Whereas PGA was a typical choice in the past, the average spectral acceleration (*AvgSA*) can be a viable candidate. (2) The numerical model of the structure is especially demanding computationally, raising new issues in the classical question of model complexity versus the number of ground motions to consider for optimal fidelity. The resulting procedure constitutes a fair proposal that can be extended to cover all adjacent monuments, for example, the entire Temple and Sanctuary and the nearby museum, where identical ground motions and fully correlated hazard are expected.

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