PROBABILISTIC SEISMIC HAZARD ASSESSMENT THROUGH GEOMETRICALLY NON-LINEAR BACK-ANALYSIS OF BYZANTINE AND ROMAN MONUMENTS

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Keywords: Seismic Response, Rocking, PSHA, Geometric Nonlinearity, Soil-Structure-Interaction.

Abstract. *This paper aims at developing the tools and strategy for assessing the seismic performance of the Byzantine and Roman remains in the city of Thessaloniki, in Greece, as a means to back-evaluate and enrich the seismic microzonation studies available for the Metropololitan area. At first, focus is made on the Walls that have been constructed at the end of the 4th century A.D. in the reign of Theodosius the Great and numerous blocks remain intact widespread within the city grid. The study particularly focuses on a specific Wall residuum, whose small dimensions, simple morpholgy (free-standing, rocking dominated masonry block), availability of nearby strong ground motion recordings and good knowledge of the underlying soil conditions, constitute a well-controlled case-study with the minimum possible numerical modeling (i.e., epistemic), record-to-record and material uncertainty. Secondly, the study focuses on an ancient Roman column, which was reestablished in 1969 after extensive archeological works. For both historical structures, a refined probabilistic dynamic analysis approach is adopted and the structural performance is examined, through a Monte Carlo Simulation scheme, for a number of realistic earthquake scenarios, accounting for geometric nonlinearities (i.e., sliding and rocking) and uncertainties in friction properties. Given the absence of damage, permanent displacement or collapse of the particular bodies, the probability of non-exceedance of a specific intensity measures (for the period that the structures remain intact) is assessed for the Wall residuum and the ancient colonnade, thus implicitly, for the city as a whole. It is also demonstrated that the fragility predicted without dully considering rocking and sliding of the two rigid blocks may lead to misleading results for particular sets of strong ground motions.*

1 INTRODUCTION

The long-term exposure of historical structures to seismic hazard and their response during the centuries has been thoroughly studied during the last decades. Notwithstanding the major research advances already made, it is still the case that the study of monumental and historical structures is hindered by their, typically, complex geometry and the associated uncertainty in terms of boundary conditions, partial inter-surface contacts, material properties as well as their spatial distribution within the structures. As a result, the assessment of the performance of historical structures under earthquake loading requires deep understanding of all those salient features that affect their seismic fragility, while at the same time, requires good knowledge of the seismic history of the monument itself [1]. Until recently, the computational cost for the refined numerical analysis of such structures was prohibitive, even after the development of the necessary tools, thus yielding practically impossible to accurately simulate all the structural features and the phenomena involved. This problem was even more pronounced in case of Classical, Roman or Byzantine Walls whose dimensions were inevitably extended, hence adding up to the computational cost of the analysis. In this framework, a research effort has been undertaken, for the study of the Byzantine Walls of Thessaloniki that aims to take advantage of both the long term experience gained by the study of other Byzantine monuments of the city [2], [3] and the recent advancements in computational earthquake engineering. The Walls, still surrounding partially the old town of Thessaloniki were initially built in 315 B.C. by the Macedonian king Kassandros and were extended at the time of Great Theodosius (379-395 A.D.) at the dawn of the Byzantine era. Nowadays, the Walls extend in kilometers within the civil grid of the modern city but their continuity has been disrupted due to collapse or demolition at numerous locations. Historically, due to their dynamic nature, resulting from their adaptation to the civilian needs, such as repairs or strengthening during sieges and according to the art of war, the Walls of Thessaloniki (as those of Constantinople, Nicaea and others), did in fact changed considerably over the centuries [4] following the heavy fortification requirements that arose. In previous research studies, three different parts of the Wall have been thoroughly studied (Figure 1a), namely:

(a) The Walls circuit in the northern part of the Byzantine fortification [5] currently denoting the limits between the old and the modern city. This part, hereafter denoted as System A, forms a 500m long, statically independent structural system, extending from the beginning of the West Gate (namely "Pyrros Gate") to the main East Gate along the Eptapygriou street, near the Trigonion Tower, inclusive of the two twin gates at the East section (widened and named thereafter by Anna Palaiologina) up to the circle tower constructed later. The particular Walls Section was encircling the Byzantine acropolis thus separating it from the Ancient Acropolis and it consists of numerous rectangular (primarily) and triangle defensive towers. This is essentially a monolithic and straight fortification structure, constructed of masonry made by alternate bands of stones and bricks [6], with its main axis being parallel to the East-West direction.

(b) The Walls complex in the area of the contemporary main City Court which was part of the Southern Gate and city fortification offering protections against invaders arriving from the sea (System B) [7]

(c) A150m Wall straight complex extending from the northern part of Aristotle University Campus up to the city Acropolis (System C).

(d) A relatively simple structural system of a statically independent, free-standing, cantilever Wall residuum (System D, depicted in Figure 1), located at the vicinity of Aristotle University. This structure is a part of a Roman tower, which was founded over Byzantine masonry and is studied herein due to its clear geometry, its structural simplicity and its proximity to the location where the 1978 earthquake was recorded.

(e) A similarly simple, monolithic, Roman colonnade of the ancient Agora, reestablished to its original state after archeological works in 1969 (System E, also illustrated in Figure 1). The structure is studied herein due to its slenderness, rigidity and the fact that it is resting on a rigid base.

The scope of the above research framework is to assess the seismic performance of the particular ancient bodies as individual structures and also, through a wider perspective, to back analyze their response during the centuries as a means to complete the picture of the seismic history of the city as a whole. Inevitably, this effort needs to address three major sources of uncertainty that are involved in the assessment process of all structures, but are particularly critical in case of monumental or historical structures, namely the:

- Seismic hazard at the site of interest and the corresponding disaggregation in terms of magnitude M, source-to-site distance R, and peak ground acceleration (PGA) for a given occurrence period, as well as seismic scenarios and resulting ground motions characteristics (frequency content, duration etc), all inducing significant *record-to-record uncertainty* [8–11].
- Material properties in terms of their mean value and dispersion, as well as the spatial distribution of these properties within the structure, at least for the case of the masonry-made, systems A-D (*material uncertainty*) [7], [12].
- Finite element modeling complexity and assumptions, primarily related to the simulation of complex geometries (i.e., for systems A and C), constitutive laws, failure criteria and force redistribution (for systems A-D), soil compliance (for flexibly supported systems B-E) as well as of contact issues (i.e., particularly, uplift and sliding [13–18]) for rocking-dominated, rigid body systems D and E. All the above decisions induce an increased *epistemic (modeling) uncertainty* [19–21] that susbstantially affects seismic response.

To overcome the difficulty to uncouple and quantify the contribution of each one of the above three uncertainty sources to the overall uncertainty induced, this study focuses on the simplest possible, free-standing, rigid structural systems (i.e., the Wall residuum D and the Roman colonnade E, presented in Figure 1) as a means to:

(a) Investigate their seismic capacity and implicitly, predict the minimum level of seismic intensity that is required to trigger collapse. Given the extreme damage state of collapse has not yet been observed, the estimation of their overturning threshold is deemed to correspond to the lower bound of ground motion intensity that has not yet occurred (at least since the systems survive in their present form).

(b) Compare the predicted probability of exceeding (or not exceeding) particular levels of ground motion intensity within a given time frame, with the seismic hazard assessment for the city of Thessaloniki [22], [23]. It is noted that 15 earthquakes have occurred historically since the construction of the Walls [24], while the definition of upper limits on earthquake motion has been identified as the 'missing piece' for both deterministic (DSHA) and probabilistic (PSHA) seismic hazard assessment [25].

The characteristics of the two systems, the methodology adopted and the results obtained from the back-analysis of the systems studied are presented in the following.

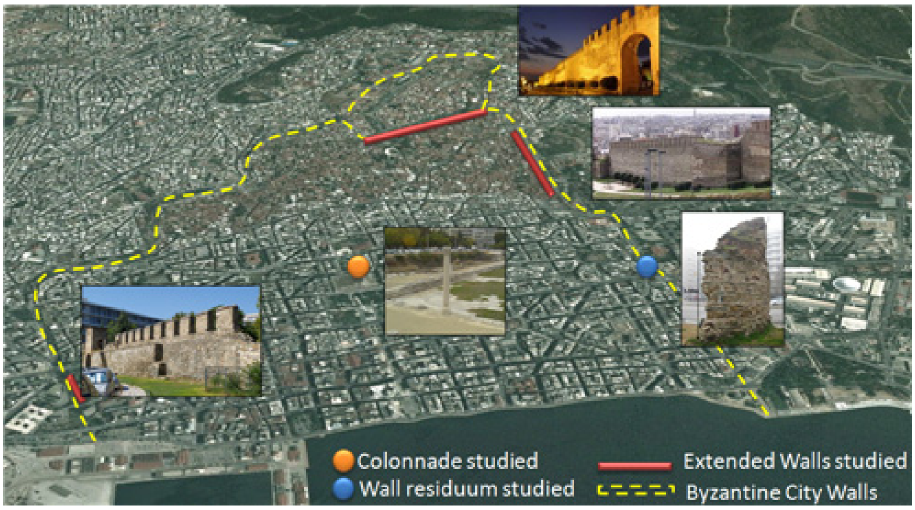

Figure 1. Aerial view of the Thessaloniki area (left) and top view of the Wall residuum studied (right).

2 ROCKING DYNAMICS OF RIGID STRUCTURES

In contrast to modern structures that dissipate significant amount of energy through inelastic behavior, the dynamic behavior of rigid body systems is ruled by the independent rocking and sliding of the bodies on their base which consists an external energy absorption mechanism. It is known that if the center of gravity coincides with the geometric center of the system and is located at distance R_0 from a base corner, the stockiness angle α of the block is equal to $tan(\alpha)=b/h$ and the rigid body rotation of the block from the vertical axis is equal to θ, then the equation of motion under zero vertical and positive horizontal base acceleration can be written as [26]:

$$\begin{cases} I_o\ddot{\theta} + mgRsin(-\alpha-\theta) = -m\ddot{u}_g Rcos(-\alpha-\theta), \theta<0 \\ I_o\ddot{\theta} + mgRsin(\alpha-\theta) = -m\ddot{u}_g Rcos(\alpha-\theta), \theta\geq 0 \end{cases} \quad (1)$$

where I_o is the moment of inertia of the rigid structure and m is its mass. The angle θ is positive when the rocking takes place around the right base corner of the structure. It is recalled that stockiness α is a measure of the system's tendency to rock; the smallest its value, the more likely the block to uplift [15]. In case of a rigid structure with a single lumped mass, the moment of inertia is equal to $I_0=mR^2$ and equation (1) can be written in the compact form of:

$$\begin{cases} \ddot{\theta} = -p^2\left[sin(-a-\theta) + \frac{\ddot{u}_g}{g}cos(-a-\theta)\right], \quad \theta<0 \\ \ddot{\theta} = -p^2\left[sin(+a-\theta) + \frac{\ddot{u}_g}{g}cos(+a-\theta)\right], \quad \theta\geq 0 \end{cases} \quad (2)$$

where p is an important frequency parameter indicating the tendency of the structure to overturn [27]. Given that the structure is rigid, energy is only dissipated through impact and is expressed by the (geometry dependent) restitution parameter e, which is equal to the ratio of the pre- and post-impact angular velocity. Whether the structure will eventually slide, uplift or overturn depends on its geometrical characteristics (R, α and p), the coefficient of friction μ, the restitution coefficient e, the mass distribution, the foundation compliance and the properties of ground motion (such as the amplitude a_p and the persistence of the pulse $L_p=a_pT_p^2$, T_p being the pulse period of the most energetic pulse of strong ground motion [28]). In case that the oscillating structure is flexible, elastic forces counteract the rocking actions and lead to more complicated transitions from one rocking cycle to the other, hence, to rotational response which is significantly different in shape, amplitude and duration [29]. The dynamic response of multi-drum columns is also more complex as has been shown analytically [13] and experimentally [30], however, is considered to be out of the scope of this study.

3. OVERVIEW OF THE SYSTEMS STUDIED

3.1. Byzantine Wall residuum (squat structure)

The Wall residuum under study, located at the junction of Konstantinou Melenikou and Egnatia Street, belongs to the East part of the Byzantine Walls, at its south lowland part, where once Kassandros Gate stood (Figure 1.1). As mentioned above, the choice of this particular section of the Walls lies in its small size, its static and geometrical simplicity, as well as the lack of structural damage on its body (at least since it obtained its current form). Those features contribute to the minimization of the third source of uncertainty (model uncertainty) and to the more reliable study of the other two sources, namely material and record-to-record uncertainty. The accurate imprinting of the monument geometry was accomplished by combining on-site measurements, satellite images, scaled axonometric views (e.g. Velenis, 1998) and other historic facts. The ground plan of the wall residuum is trapezoidal, with parallel edges corresponding to the north-south axis. The Wall has four vertical faces with base dimensions B=2.05m and 3.60m and W=1.87m and 2.30m. Out of the 5.27m of height, the 3.30m have a constant trapezoidal shape, whereas a stenosis occurs at the upper 1.95m, ending in a trapezoid of dimensions B=1.80m, β=1.30m and υ=1.87m (Figure 2). There is no evidence with regard to the foundation depth, but given the relevant experience from similar parts of the Wall, it is estimated as of 0.50m. Given its geometric characteristics, the stockiness angles of the block are found to vary within α_W = 0.342÷0.608 and α_B = 0.372÷0.829 along the two directions, respectively. Based on earlier research for monuments of the Byzantine era [5], [31], a uniform in space average compression strength of f_{mc}=2.0MPa was adopted for the construction materials, based on the weaker brick masonry while the corresponding tensile strength was set equal to f_{mt}=0.15MPa. The Young's modulus of the masonry was taken equal to E=3500MPa, and the unit weight equal to γ=22kN/m^3. According to the Microzoning study of Thessaloniki [23], [24], the underlying soil can be classified as of "Soil Class B" according to the Greek Seismic Code (highly eroded rocky soils or soils that can be mechanically simulated with grainy), thus "Soil Class B" to "C" according to Eurocode 8 (CEN, 2004). The average shear wave velocity of the upper 30m of soil was estimated as $V_{s,30}$ = 250 m/sec; the unit weight and Poisson's ratio of the soil were set equal to18kN/m^3 and 0.2, respectively.

(a) East view (b) West view (c) North view (d) South View

Figure 2. Various perspectives of the Wall residuum studied.

3.2. Ancient Roman Colonnade (slender structure)

The Roman Agora was constructed in the late 2nd century AD and it constituted the administrative centre of the town till the 5th century. The excavations started in 1962, while the restoration works began in 1989 and created an important archaeological site to visit in the centre of the town. The complex of the ancient Agora was situated in the heart of Roman Thessaloniki; it occupied a surface of about twenty acres and constituted for three centuries the city's administrative center. The Agora's facilities had been developing around a large, rectangular, paved square of 146 meters length and 97 meters width. Along with the three sides of the square, the eastern, the southern and the western, there were double lines of Corinthian order's columns that formed arcades; behind the arcades, there were different kinds of public areas, serving various needs of Thessaloniki's citizens. The colonnade is monolithic, without flouting, with a height of 6.0m (including the capital) and a diameter of 0.8m, corresponding to a stockiness angle α equal to 0.109. The diameter of the capital varies from 0.6 to 0.8m along the height. The colonnade is supported on a 1.0x1.0m rectangular base. The particular colonnade was reestablished at its original state in 1969.

Figure 3. Overview of the Roman Colonnade studied in the Ancient Agora of Thessaloniki.

Table 1. Dynamic and rocking characteristics of the two systems studied

System studied	Dimensions [m]	Stockiness $\alpha = tan^{-1}(W/H)$	Size parameter R_0 [m]	Frequency parameter $p = \sqrt{mgR/I_0}$ [rad/sec]	Natural periods of the fixed-base deformable system T_s [sec]
Wall residuum	W=1.87÷2.30 B = 2.05÷3.60 H=3.30÷5.25	α_W = 0.342÷0.608 α_B = 0.372÷0.829	R_w = 1.89÷2.86 R_B = 1.94÷3.18	$p = \sqrt{3g/4R}$ p_w = 1.60÷1.96 p_b = 1.52÷1.94	E_s=3.5GPa [5] T_s =0.09sec
Colonnade	D=0.66 H=6.0	α_D = 0.109	R_d = 3.01	p_d = 1.56	E_s=40GPa T_s =0.15sec

4. METHODOLOGY

4.1 Selection of ground motion scenarios

For the execution of the dynamic time-history analyses a set of 57 earthquake records was selected from the PEER-NGA database, based on the results of the disaggregation of seismic hazard for the city of Thessaloniki [22]. In particular, four sub-sets were developed containing records compatible to magnitude 6.0<M<6.5, source-to-site distance 10km<R<30km, the epicentral distance, average shear wave velocity 200m/sec<$V_{s,30}$<300m/sec and peak ground acceleration within the intervals [0,0.1g] (corresponding to the PSHA period of recurrence of 50years), [0.1g,0.28g] (corresponding to the PSHA period of recurrence of 475years), [0.28g,0.50g] and [0.50g,1.50g]. An additional analysis was performed using the (single) strong-motion record available from the Thessaloniki earthquake of 20/6/1978, a shallow-depth, normal event of magnitude M_w = 6.5 with a duration of 10sec, originating from a blind fault at a source distance of about 8-10 km from the center of the city. The particular ground motion with PGA=0.15g and a predominant period of 0.4-0.5sec, was recorded at the basement of City Hotel at a distance of 900m from the Ancient Agora and 1.5km from the Byzantine Wall residuum.

4.2 Finite Element Modeling and Stability Analysis

Due to the geometrical nonlinearities, it was deemed necessary to establish a rigorous framework for the seismic evaluation of the Byzantine and Roman structures, which can be applied uniformly in all similar monumental structures within the city of Thessaloniki. A basic assumption was that the Roman colonnade is expected to translate with the ground, slide or rock about a center of rotation, which is assumed to lie along an edge of their base. The Byzantine Wall residuum can only move with the ground or rock, given the fact that its 0.4m deep foundation prevents relative sliding. ABAQUS [32] was used to simulate the geometrically nonlinear deformation of the system, considering frictional sliding, rocking and complete separation. An extensive parametric scheme was followed to verify the accuracy of the numerical procedure based on known analytical solutions [33]. These comparisons produced satisfactory agreement in terms of maximum displacements, frequency characteristics of the response and the time of overturn. Both structures were deemed as rigid resting on a rigid base with a coefficient of friction exponentially decaying from a static value μ_s at the initiation of sliding, to a lower value, μ_k:

$$\mu = \mu_k + (\mu_s - \mu_k)e^{-d_c\gamma'_{eq}} \quad (3)$$

where γ'_{eq} is the equivalent slip rate and d is the decay coefficient from static state to kinetic state. Due to the uncertainty in the value of μ_k, two (equally probable) cases were examined: μ_s=μ_κ=0.7 and μ_s=0.7, μ_k=0.3 with d=0.05, based on relevant data from the literature [13], [34].

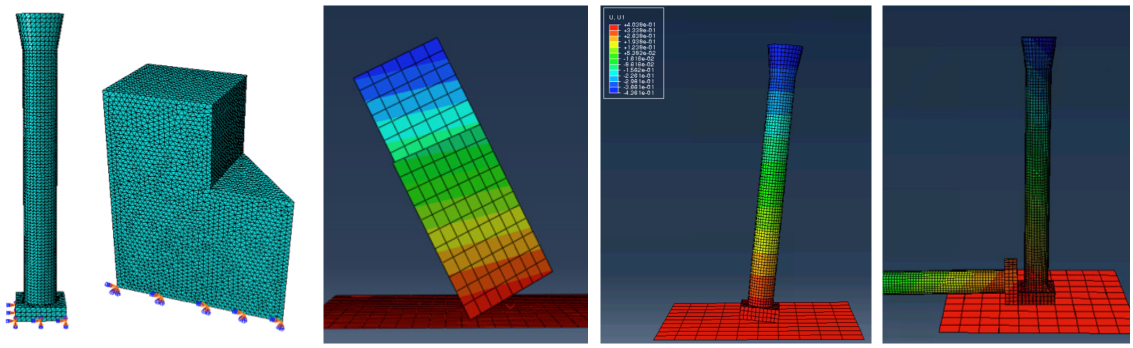

Figure 4. Overview of the finite element models of the Byzantine residuum and the Ancient Roman Colonnade.

5. RESPONSE UNDER HARMONIC EXCITATION

Harmonic excitation was used first, to determine the response of the rocking structures studied and to investigate the frequency ranges that can drive them to overturning instability. The primary modes of failure for both systems are the ones identified in [35], that is, overturn after a single impact and overturn after multiple impacts. However, for illustration purposes, additional modes of vibration were also considered for the colonnade, such as, no rocking, rocking without sliding, as well as rocking with sliding. From the rocking spectra presented in Figure 5 it is clear that the coupled rocking-sliding mode of vibration is dominant for the ancient colonnade is compared to the Wall residuum as a result of the restrain of the latter to translate horizontally ue to embedment in the soil. It is also shown that the slender structure overturns directly (i.e., without prior impacts) for ground motion amplitudes that exceed 1g but can also overturn after multiple impacts at lower amplitudes at low frequencies. The distinct overturning mechanisms and amplitude/frequency dependence of the slender and the squat structure is deemed to constitute a useful indicator of rocking susceptibility when selecting additional free standing monuments in the city of Thessaloniki to back-analyze the seismic history of the city.

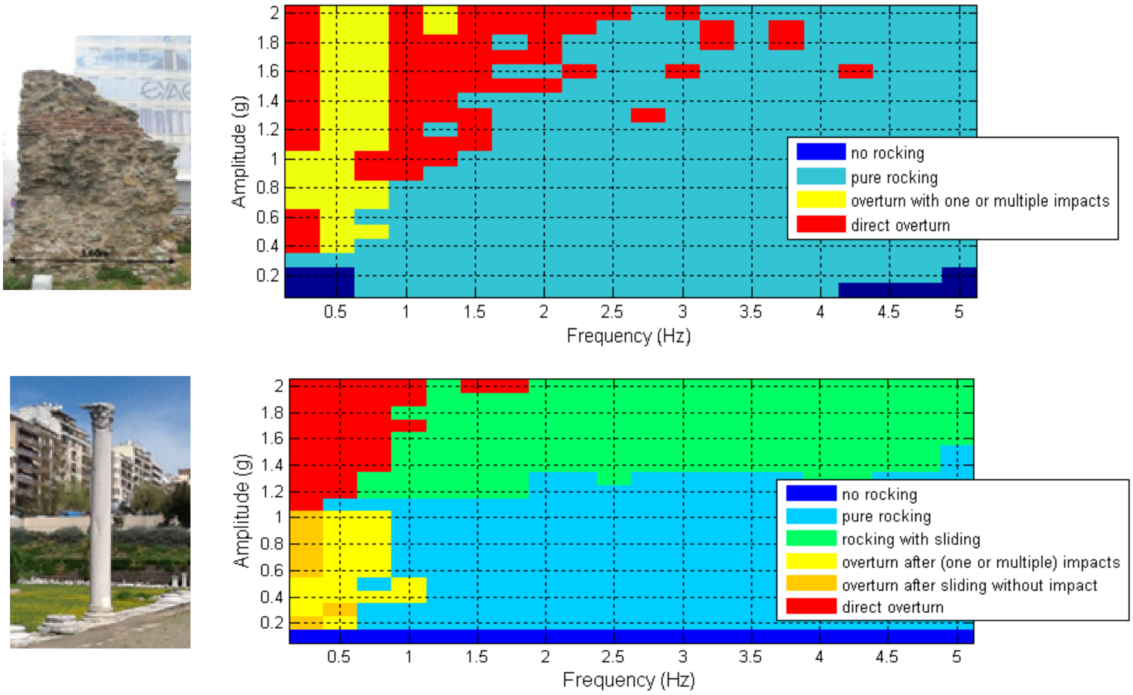

Figure 5. Rocking spectra for the two systems under harmonic excitation *f(t)=cos(ωt)*.

6. RESPONSE UNDER SEISMIC EXCITATION

From the two structures studied under harmonic excitation, the ancient colonnade was selected to be further examined under the ground motion scenarios of Section 3. This was primarily due to the fact that the colonnade presented more "visible" signs of permanent dislocation (as the Wall residuum was partially restrained against sliding by its surrounding foundation soil). The results of the colonnade response to bi-directional seismic excitation are presented in Figure 7 in the form of the SRSS of the peak ground accelerations of the two horizontal components applied versus their average mean frequency. It is recalled that the mean period parameter (T_m) quantifies the predominant frequency content of the records used [36] by weighting the amplitudes over a specified range of the Fourier Amplitude Spectrum:

$$T_m = \frac{1}{f_m} = \frac{\sum C_i^2 \cdot \frac{1}{f_i}}{\sum C_i^2} \qquad \text{for } 0.25\ Hz \le f_i \le 20\ Hz \text{ with } \Delta f \le 0.05\ Hz \qquad (4)$$

It is noted that the mean period is not necessarily a representative indicator of the persistence T_p of the pulse along the entire period range [37]; however, it was deemed preferable to illustrate the results on the basis of the mean period T_m, to facilitate the comparison with the code-defined spectrum. What can be seen by Figure 6 is that for the Thessaloniki seismic hazard–compatible ground motion scenarios studied, no visible permanent displacement or torsion should be expected for amplitudes lower than 0.52g independently of the frequency content of ground motion. Given that the colonnade is standing intact since 1969, it is also evident that such an amplitude has not been exceeded for the last 45 years. By plotting the results versus the maximum PGA of the two components instead of the SRSS of the two maxima, the amplitude threshold drops to 0.47g. This is equivalent to implying that, according to the colonnade's recent seismic history, the probability of exceeding 0.47g within a period T_L of 45 years is essentially P_R=0% (as it is also evident by the actual recorded data for this period which confirm that the maximum PGA recorded since 1969 was 0.14g). It is also shown that for the 475 years seismic scenario for the city of Thessaloniki (i.e., 0.20g<a_g<0.28g), the probability of permanent dislocation of the ancient colonnade is approximately 30% (corresponding to 4/14 analysis cases). It is recalled that the present Eurocode 8 specifies in its Greek National Annex a probability of 10% to exceed 0.16g within 50 years (corresponding to a period of recurrence of 475 years). Clearly, a greater sample is required before drawing stable statistical estimates, hence, the above can only be seen as a rough approximation.

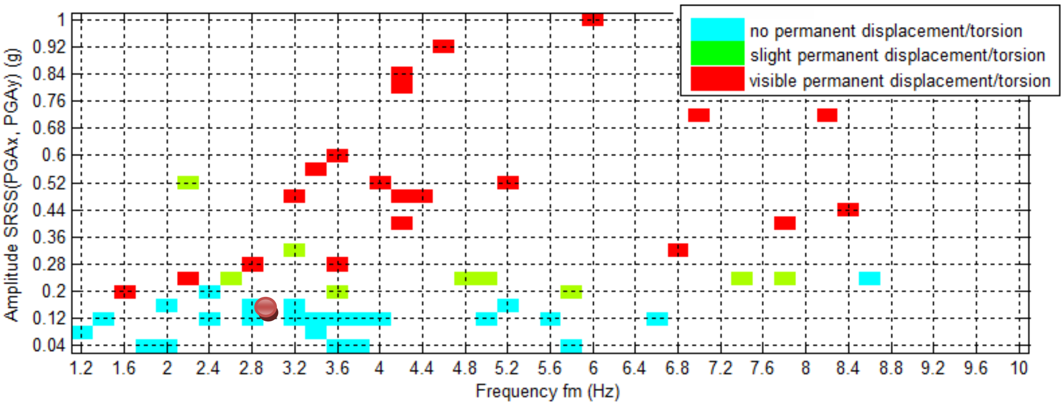

Figure 6. Rocking spectra for the Roman Colonnade under seismic-hazard compatible ground motion scenarios. Thessaloniki (1978) earthquake strong ground motion record depicted as a circle.

7. CONCLUSIONS

This paper aims at developing the tools and strategy for assessing the seismic performance of the Byzantine and Roman remains in the city of Thessaloniki, in Greece, as a means to back-evaluate and enrich the earthquake historical data available for the Metropolotitan area. A first observation is that the seismic response of a slender and a squat monumental structure studied and their respective fragility is clearly different when they allowed to uplift compared to the case that limit states are strength-dependent and geometrical nonlinearities are neglected. Moreover, based on the refined numerical study of the two systems studied, it is back-verified that a PGA=0.47g has not been exceeded within the last 45 years, while the probability of permanent dislocation of the ancient colonnade for the 475 years scenario of the city is found approximately equal to 30%. From one point of view, the above observations might seem rather obvious, however, it is the first time that they are quantified implicitly through a numerically rigorous procedure that takes into consideration the actual performance of rocking-dominated monuments in time. As such, it deemed as a promising tool towards the improvement of our understanding of historical seismic events, particularly when focusing to structures which stand still for significantly longer periods in time, an extension which is currently the focus of this ongoing work.

REFERENCES

[1] T. P. Tassios, "Seismic engineering of monuments," *Bull.of Earthquake Engineering*, vol. 8, no. 6, pp. 1231–1265, 2010.

[2] G. G. Penelis, K. Stylianidis, M. Karaveziroglou, and D. Leontaridis, "Strengthening the Rotonda Monument in Salonica," in *IABSE Symposioum, Venezia.*, 1983.

[3] A. J. Kappos, G. Panagopoulos, and G. G. Penelis, "Development of a seismic damage and loss scenario for contemporary and historical buildings in Thessaloniki, Greece," *Soil Dynamics and Earthquake Engineering*, vol. 28, no. 10–11, pp. 836–850, Oct. 2008.

[4] C. Bouras, C. Morrison, N. Oikonomides, and C. Pitsakis, *The Economic History of Byzantium: From the Seventh through the Fifteenth Century*. Washington, D.C., Trustees for Harvard University, 2002.

[5] K. Stylianidis and A. G. Sextos, "Back Analysis of Thessaloniki Byzantine Land Walls as a Means to Assess its Seismic History," *Int. Journal of Architectural Heritage*, vol. 3, no. 4, pp. 339–361, 2009.

[6] G. Velenis, *The Walls of Thessaloniki: From Kasandrus to Heraklios, University Studio Press, Thessaloniki.* University Studio Press, 1998.

[7] A. G. Sextos, K. Stylianidis, and K. Mykoniou, "Sensitivity of the Seismic Response of Long Medieval Walls to Earthq. and Material Uncertainty," *Advanced Materials Res*, vol. 133–134, pp. 689–695, 2010.

[8] J. J. Bommer and H. Crowley, "The Influence of Ground-Motion Variability in Earthquake Loss Modelling," *Bulletin of Earthquake Engineering*, vol. 4, no. 3, pp. 231–248, Mar. 2006.

[9] A. R. Kottke and E. M. Rathje, "A semi-Automated procedure for selecting and scaling recorded earthquake motions for dynamic analysis," *Earthquake Spectra*, vol. 24, no. 4, pp. 911–932, 2008.

[10] E. I. Katsanos, A. G. Sextos, and G. D. Manolis, "Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective," *Soil Dynamics and Earthquake Engineering*, vol. 30, no. 4, pp. 157–169, Apr. 2010.

[11] A. G. Sextos, E. I. Katsanos, and G. D. Manolis, "EC8-based earthquake record selection procedure evaluation: Validation study based on observed damage of an irregular R/C building," *Soil Dynamics and Earthquake Engineering*, pp. 1–15, Dec. 2010.

[12] O.-S. Kwon and A. S. Elnashai, "The effect of material and ground motion uncertainty on the seismic vulnerability curves of RC structure," *Engineering Structures*, vol. 28, no. 2, pp. 289–303, Jan. 2006.

[13] N. N. Ambraseys and I. N. Psycharis, "Earthquake Stability of Columns and Statues," *Journal of Earthquake Engineering*, vol. 15, no. 5, pp. 685–710, Jun. 2011.

[14] E. Voyagaki, I. N. Psycharis, and G. E. Mylonakis, "Rocking Response and Overturning Criteria for Free Standing Blocks to Single – Lobe Pulses," in *15th World Conf. on Earthq. Eng., Lisbon*, 2012.

[15] D. Konstantinidis and N. Makris, "Experimental and analytical studies on the seismic response of freestanding and anchored laboratory equipment," PEER Center, Report 2005/07, 2005.

[16] N. Makris and M. F. Vassiliou, "Planar rocking response and stability analysis of an array of free-standing columns capped with a freely supported rigid beam," *Earthquake Engineering & Structural Dymamics*, vol. 42, no. 3, pp. 431–449, 2013.

[17] N. Makris and D. Konstantinidis, "The rocking spectrum and the limitations of practical design methodologies," *Earthquake Engineering & Structural Dynamics*, vol. 32, no. 2, pp. 265–289, 2003.

[18] E. G. Dimitrakopoulos and M. J. DeJong, "Revisiting the rocking block: closed-form solutions and similarity laws," *Proceedings of the Royal Society*, no. January, Apr. 2012.

[19] M. Mistler, C. Butenweg, and K. Meskouris, "Modelling methods of historic masonry buildings under seismic excitation," *Journal of Seismology*, vol. 10, no. 4, pp. 497–510, Nov. 2006.

[20] D. Celarec, P. Ricci, and M. Dolšek, "The sensitivity of seismic response parameters to the uncertain modelling variables of masonry-infilled reinforced concrete frames," *Engineering Structures*, vol. 35, pp. 165–177, 2012.

[21] A. Der Kiureghian and O. Ditlevsen, "Aleatory or epistemic? Does it matter?," *Structural Safety*, vol. 31, no. 2, pp. 105–112, Mar. 2009.

[22] K. D. Pitilakis, G. Cultrera, B. Margaris, G. Ameri, A. Anastasiadis, G. Franceschina, and S. Koutrakis, "Thessaloniki Seismic Hazard Assessment: Probabilistic and deterministc approach for rock site conditions," in *4th Int. Conference on Earthquake Geotechnical Engineering,* , 2007, no. 1701.

[23] A. Anastasiadis, D. Raptakis, and K. D. Pitilakis, "Thessaloniki's detailed microzoning: Subsurface structure as basis for site response analysis," *PAGeophysics*, vol. 158, no. 12, pp. 2597–2633, 2001.

[24] G. A. Leventakis, "Leventakis G.-A., Microzonation Study of the city of Thessaloniki, PhD.Thesis (in Greek with an English abstract)," Aristotle University Thessaloniki, 2003.

[25] J. J. Bommer, "Deterministic vs. probabilistic seismic hazard assessment: an axaggerated and obstructive dichotomy," *Journal of Earthquake Engineering*, no. Special Issue, pp. 43–73, 2002.

[26] N. Makris and Y. S. Roussos, "Rocking response of rigid blocks under near-source ground motions," *Géotechnique*, vol. 50, no. 3, pp. 243–262, Jan. 2000.

[27] G. W. Housner, "Behaviour of inverted pendulum structures during earthquakes," *Bulletin of the Seismological Society of America*, vol. 53, no. 2, pp. 403–417, 1963.

[28] N. Makris and C. J. Black, "Evaluation of Peak Ground Velocity as a 'Good' Intensity Measure for Near-Source Ground Motions," *Journal of Engineering Mechanics*, vol. 130, no. 9, p. 1032, 2004.

[29] S. Acikgoz and M. J. Dejong, "The interaction of elasticity and rocking in flexible structures allowed to uplift," *Earthquake Engineering & Structural Dynamics*, 2012.

[30] G. C. Manos and M. Demosthenous, "Dynamic response of rigid bodies subjected to horizontal base motion," in *10th World Conference on Earthquake Engineering, Madrid, Spain*, 1992.

[31] G. C. Manos, V. J. Soulis, and A. Diagouma, "Preliminary numerical investigation of the dynamic characteristics of historic monuments," in *13th World Conf. on Earthq. Eng., Vancouver, Canada*, 2004.

[32] K. Hibbit and N. Sorenson, "ABAQUS ver. 6.6, User's Manual," Pawtucket, USA, 2006.

[33] D. Konstantinidis and N. Makris, "Seismic response analysis of multidrum classical columns," *Earthquake Engineering & Structural Dynamics*, vol. 34, no. 10, pp. 1243–1270, 2005.

[34] P. Komodromos, L. Papaloizou, and P. C. Polycarpou, "Simulation of the response of ancient columns under harmonic and earthq. excitations," *Engineering Structures*, vol. 30, no. 8, pp. 2154–2164, 2008.

[35] Y. Zhang and N. Makris, "Rocking response of free-standing blocks under cycloidal pulses," *Journal Engineering Mechanics, ASCE*, vol. 127, no. 5, pp. 473–483, 2001.

[36] E. M. Rathje, F. Faraj, S. Russell, and J. D. Bray, "Empirical relationships for frequency ground motions," *Earthquake Spectra*, vol. 20, no. 1, pp. 119–144, 2004.

[37] M. F. Vassiliou and N. Makris, "Estimating Time Scales and Length Scales in Pulselike Earthquake Acceleration Records with Wavelet Analysis," *BSSA,* vol. 101, no. 2, pp. 596–618, 2011.