SAHC2014 – 9th International Conference on
Structural Analysis of Historical Constructions
F. Peña & M. Chávez (eds.)
Mexico City, Mexico, 14–17 October 2014

MULTI-PARAMETRIC SEISMIC VULNERABILITY ASSESSMENT OF FREE-STANDING COLUMNS

Athanasios Pappas[1], Caterina Previato[2], Bruno Q. Silva[3], Francesca da Porto[1], Jacopo Bonetto[2] and Claudio Modena[1]

[1] University of Padova – DICEA

Department of Civil, Architectural and Environmental Engineering
Via Marzolo 9 – 35131, Padova, Italy
e-mail: athanasios.pappas@dicea.unipd.it; francesca.daporto@unipd.it;
claudio.modena@unipd.it[2] University of Padova – DBC

[2] University of Padova – DICEA

Department of Cultural Heritage
Piazza Capitaniato 7 – 35139, Padova, Italy
e-mail: caterina.previato@unipd.it; jacopo.bonetto@unipd.it

[3] University of Porto – DEC

Department of Civil Engineering
Rua Dr. Roberto Frias, s/n 4200-465, Porto, Portugal
e-mail: bruno.silva@dicea.unipd.it

Keywords: Column, DEM, fragility curve, vulnerability assessment, seismic action.

Abstract. *Harmonic and seismic excitation was used for studying the structural behaviour of different free-standing column typologies using 2D discrete element modelling in UDEC 5.0* [1]. *Based on an extensive archaeological survey, a set of realistic parameters was defined for the seismic vulnerability assessment of these elements, namely the soil characteristics, slenderness, height and number of drums.*

It was shown the influence of slenderness, size effect and number of drums. The more slender elements were found to be the less stable, for the same slenderness the taller column is characterised by higher stability. However, since for harmonic analyses the relation between the results of the different elements tested under different frequencies usually do not vary according to a robust law, it was not possible to give quantified results for the effect of each parameter based only on the harmonic excitation results. This quantification of the results became possible using dynamic analyses with earthquake time-histories instead of harmonic excitation. Based on the seismic analyses results, fragility curves were created for all examined soil-column configurations for quantifying the influence of all investigated parameters.

This research is being carried out as part of the Italian project "PROVACI" [2] *that deals with seismic protection and valorisation of cultural heritage.*

1 INTRODUCTION

This parametrical analysis concerns free-standing ancient columns able to perform rocking oscillation. Having as area of interest the Greek and Roman stone columns of the Mediterranean area meant that it was necessary to define some dimension and typology boundaries within which to perform numerical analyses. Those boundaries were defined with the help of archaeological research focused on the dimensions of Corinthian order columns of 50 ancient structures, for which details can be found in a recent study of M. Wilson Jones [3]. The first parameter to investigate was the slenderness of the free-standing element. The archaeological research revealed that the majority of columns had a slenderness up to 8.5-9, (Figure 1). In this study the slenderness value represents the ratio of column height to base diameter without taking into account the height of the capital. Another parameter of interest was the size of the column. As it was shown by the pioneering work of Housner [4], the size effect is an important parameter which makes larger elements more stable than smaller ones with the same geometrical proportions. Based on the archaeological findings, the height boundaries for this study were set between 3m and 12m, (Figure 1).

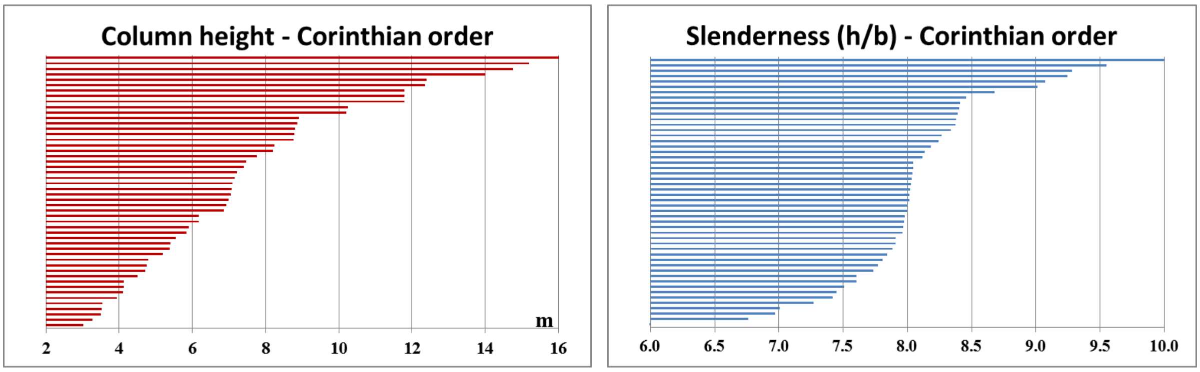
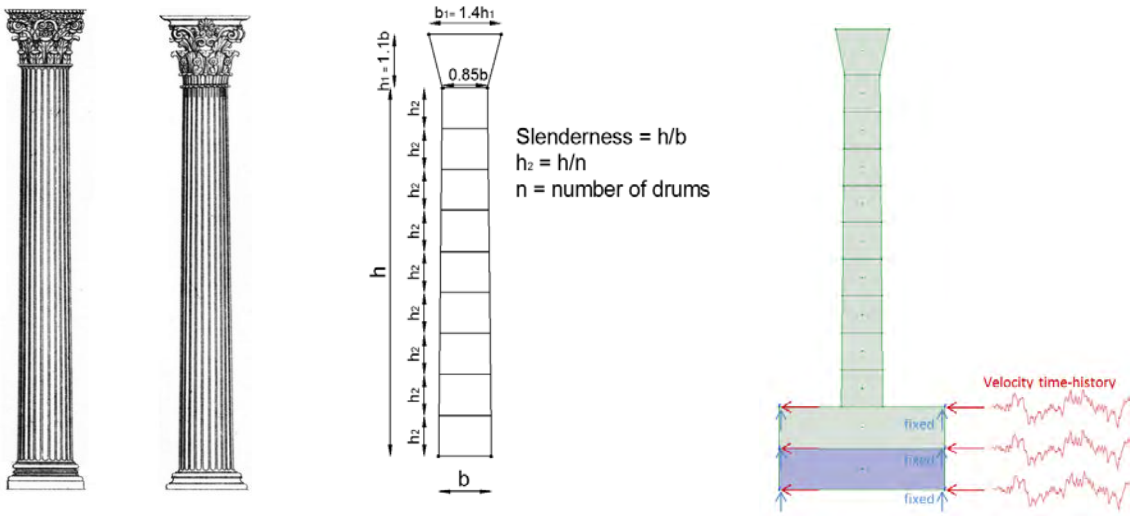


Figure 1: Summary of archaeological research results on 50 existing Corinthian order columns. Height (left) and slenderness (right).

For homogenising the parametrical study, all columns were considered with a capital of constant proportions approximating the ones of the Corinthian order as shown in Figure 2. The Corinthian order capital is the one with the highest slenderness and therefore it was adopted as the less favourable configuration.



(a) (b) (c)

Figure 2: Corinthian columns (a); adopted typical geometry (b); numerical model base excitation (c).

The number of drums can also influence the structural behaviour of a column. Multi-block elements tend to present a “pseudo-flexible” behaviour in comparison with monolithic columns. In reality, the number of drums may vary significantly even between the columns of the same monument. In order to simplify the analyses, 9 drums were selected as the highest number for the parametrical analysis. Another important and influential parameter which is often underestimated or even neglected is the soil effect. The same earthquake will have very different characteristics in terms of peak ground acceleration and frequency domain on different soil types. As a consequence the rocking behaviour of the elements will be strongly influenced. Here, for the numerical analyses under seismic loads there were considered the five different soil typologies as they are defined in EC8. The investigated parameters are presented in Figure 3. For instance, the “A.S8.H6.D9” analysis concerns a column on soil A, with slenderness 8 (S8), height 6m (H6) and 9 drums (D9). The same codification is used for naming all analyses. The soil prefix is used only for the seismic analyses.

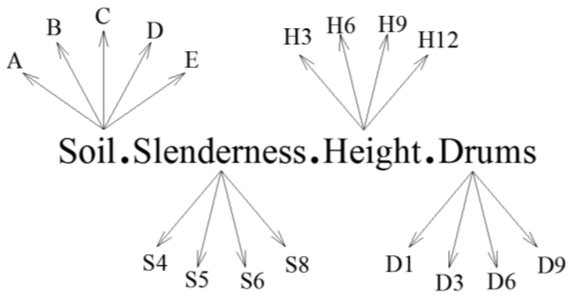


Figure 3: Analyses codification

The columns were modelled in UDEC 5.0 [1] as discrete and rigid blocks. For the joints between the blocks, Coulomb friction law was implemented with no tensile strength simulating in this way the behaviour of a dry joint. The damping of the system is stiffness proportional Rayleigh damping and its parameters are calculated as proposed by DeJong [5]. Critical damping for the natural frequency of the edge impact was selected and it was calculated for the dimensions of the typical drum of each model. The excitation of the columns is induced at the base of the system as velocity time-histories in the horizontal direction. The configuration of the system and the system parameters are shown in Figure 2c and Table 1 respectively.

Table 1: Numerical model parameters

Property	**Value**
Density [kg/m^3]	2560
Joint normal stiffness [Pa/m]	$5 \cdot 10^{11}$
Joint shear stiffness [Pa/m]	$5 \cdot 10^{11}$
Joint friction angle [°]	35
Joint tensile strength [Pa]	0
Joint cohesion [Pa]	0
Rayleigh damping	100% for edge impact natural frequency

2 HARMONIC EXCITATION

Although harmonic excitation is an oversimplified type of ground motion which is rather impossible to occur, it can give valuable results in estimating the structural behaviour against more complicated excitations that lay in seismic motion. For the harmonic numerical analyses a total of 10 sinusoidal acceleration cycles was imposed at each frequency of interest. The range of periods between 0.3 s and 2.0 s was investigated. As reported by Manos and Demosthenous [6] and Psycharis et al. [7] a safe-unsafe boundary appears in the results of such dynamic analyses. It also appears a safe-unsafe area at which the column is possible to collapse for a certain level of acceleration but it may survive for a higher acceleration at the same frequency of excitation

In general terms, the columns are able to resist much higher acceleration levels for low periods of excitation than for large periods. It is noticed that for high frequencies the columns present intense sliding and relative displacement between drums. This behaviour is continuously reducing and disappears for low frequencies where the multi-drum column tends to behave like a single rigid body performing a rocking oscillation. The numerical analyses allowed concluding that for failure at high frequencies usually a high number of column drums survives.

It is also worth mentioning the existence of a "no rocking area". For long periods, usually around 2.0s depending also on the type column, the acceleration that is demanded for overturning is almost the same with the acceleration found from limit kinematic analysis. This is the lowest possible acceleration for which overturning is possible. Accelerations under that level do not result in rocking but in translational movement of the column along with the base. For slightly higher accelerations the oscillation usually begins very weakly during the first cycles and continues with a gradual build-up of rocking that finally results in overturning (See Figure 4).

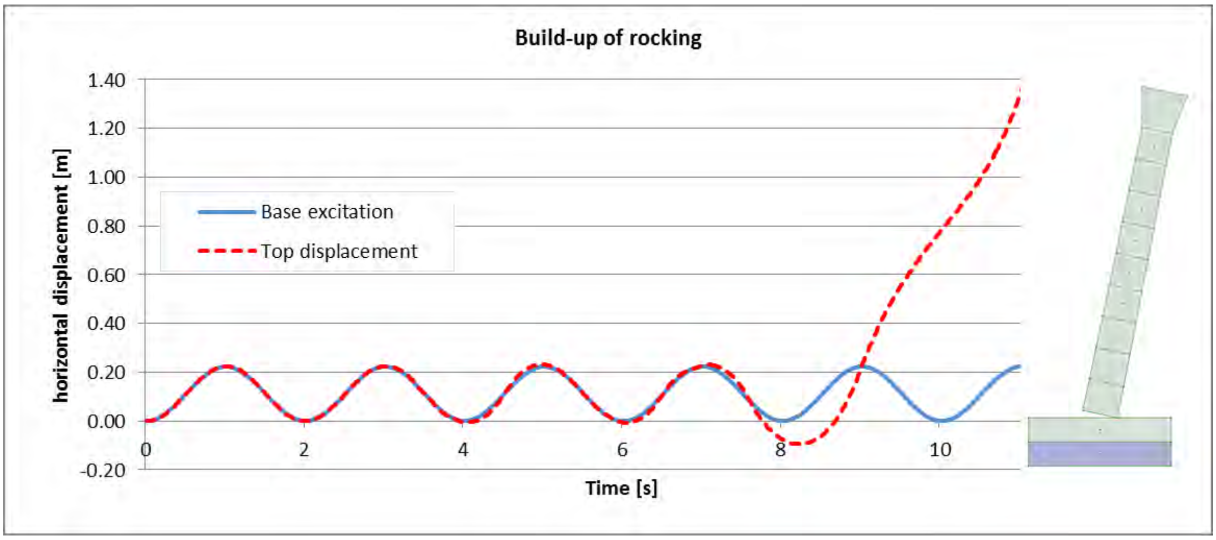


Figure 4: Gradual build-up of rocking and overturning of the S8.H6.D9 column subjected to harmonic excitation with period 2.0s and peak acceleration 0.11g.

Concerning the effect of the slenderness, it was found that, as it was expected, the stockier columns present higher stability. The safe-unsafe boundary for the tested ratios of slenderness (4, 6, 8) is presented in Figure 5 for different slenderness ratios of 6 m height columns. All columns have similar shape of curves and the modes of failure are usually the ones described before for a typical multi-drum column. However, the ratio between curves of different slenderness columns is very scattered for the different frequencies. Thus, it is not possible to define a law according to which it would be possible to predict the response of a column using as point of reference the results of another column with different slenderness.

From the investigation of the height influence it was confirmed the "size effect" theory of Housner [4]. As shown clearly in Figure 6, for the same slenderness, the tallest elements are more stable under dynamic excitation. The same behaviour was noticed for the 3-drum and monolithic columns. It has to be noted though that this effect tends to disappear for periods around 2.0 s. In such low frequencies the numerical results almost coincide with the static limit analysis results, which depend only on the element proportions and not on its size. In this frequency domain the multi-drum column behaves and overturns like a single body.

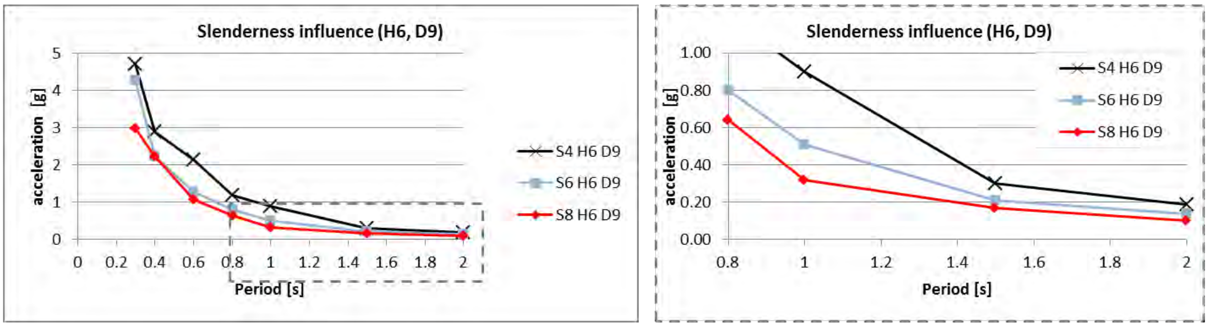


Figure 5: Safe-unsafe boundary curves for different slenderness ratios of 6m height columns with 9 drums.

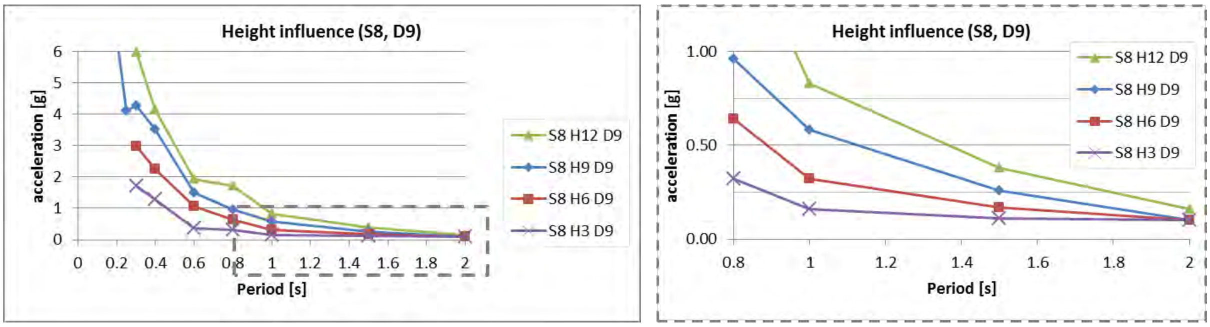


Figure 6: Safe-unsafe boundary curves for columns of different height with 9 drums and slenderness 8.

The findings regarding the influence of the number of drums are more complex. The numerical results for different height elements of slenderness 8 are presented in Figure 7 - Figure 8 for monolithic (D1), 3-drum (D3) and 9-drum (D9) columns. For analysing the results it is necessary to split the response into high frequency and low frequency area. For low frequencies it was noticed that the D9 columns are considerably more stable than the D1 and D3 which appear results very close to each other. This behaviour changes for high frequency domain. There, the monolithic column is able to resist to higher levels of acceleration. The failure of a multi-block column at those frequencies is mainly governed by the high sliding and relative displacement between drums and usually by collapse of the upper blocks. Such sliding behaviour is not possible for monolithic columns. Thus, a source of instability is eliminated. Similar higher stability of single-block columns over multi-block columns at high frequencies was also seen at the numerical results of Psycharis et al. [7].

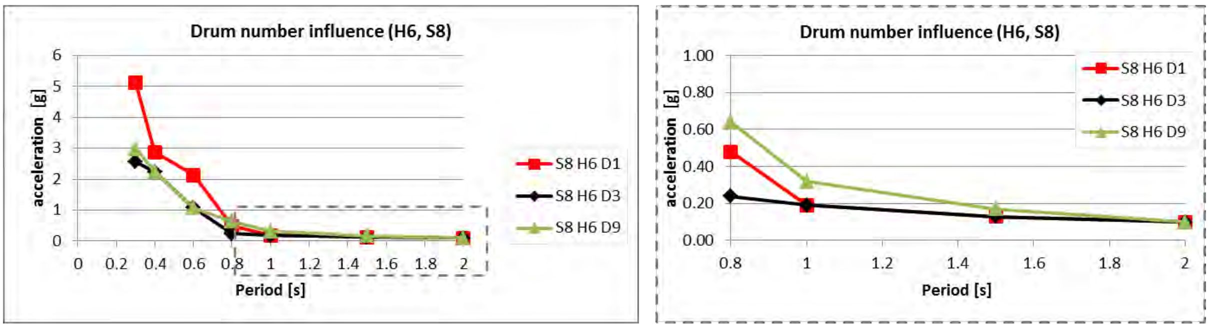


Figure 7: Safe-unsafe boundary curves of different drum number columns of height 6m and slenderness 8.

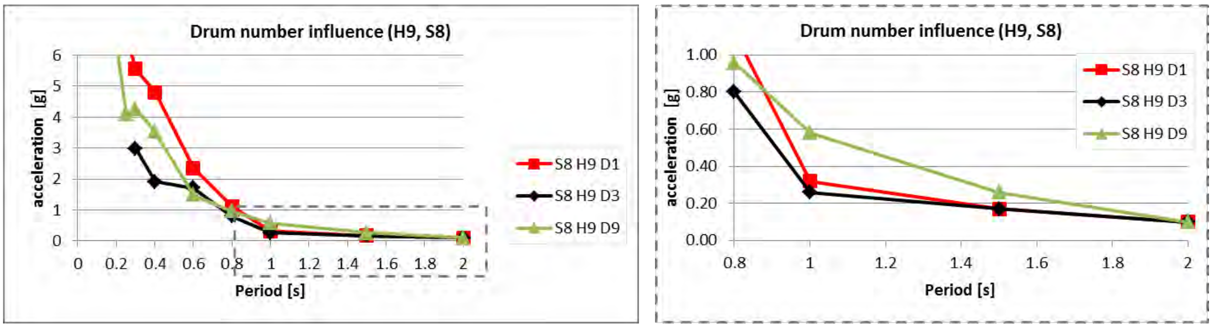


Figure 8: Safe-unsafe boundary curves of different drum number columns of height 9m and slenderness 8.

Summarising the findings of the parametrical analysis of columns subjected to 10-cycle harmonic excitation, it was shown the influence of the slenderness, the size effect and the influence of the number of drums. Slender elements were found to be less stable while for two elements of the same slenderness the tallest is characterised by higher stability. However, since the relation between the results of the different elements tested under different frequencies usually do not vary according to a robust law, it was not possible to give quantified results for the effect of each parameter based only on the harmonic excitation analyses. This results quantification may be proven possible with the use of dynamic analyses with earthquake time-histories instead of harmonic excitation.

3 SEISMIC EXCITATION

The most challenging part of this research was to yield results on the vulnerability of ancient columns based on seismic excitation parametrical numerical analyses. Apart from the vulnerability assessment of existing columns, this research can also find use, even between limits, in the estimation of the intensity of past earthquakes based on actual elements that collapsed or not. Nevertheless, the seismic motion is a far more complicated dynamic phenomenon than the previously described harmonic excitation. This causes an even more complex response which depends also on additional parameters like the soil effect which influences the frequency content of the earthquake. However, this multi-parametric challenge can be faced with the help of a probabilistic approach in order to obtain quantified results regarding the effect of each parameter.

3.1 Methodology

The use of seismic time-histories was required for studying the effect of the soil in the seismic vulnerability of columns. Therefore, it was necessary to apply seismic excitations corresponding to all 5 types of soil described in EC8. In order to have a more uniform sample of earthquakes it was decided to use artificially generated accelerograms for each type of soil. With this approach some of the bias attributed to the selection of the accelerograms were reduced. Eight accelerograms were created for each type of soil. Those reference accelerograms were scaled to PGA levels from 0.2 g to 0.8 g with step of 0.1 g. The artificial accelerograms had 20 s duration each and they were created with the use of SIMQKE_GR [8].

The definition of fragility curves of different types of ancient columns is a fundamental component for their seismic risk assessment. The process of the initial numerical results and the creation of the initial fragility curves was based on the approach of Psycharis et al. [9] who proposed the separation of the results of numerical analyses that caused the collapse from the analyses for which the columns survived modifying in this way the traditional fragility curves methodology. A fragility curve F(I) depicts the probability that an engineering demand parameter (EDP) exceeds a certain threshold value (edp) for the different values of a specific earthquake parameter (I).

$$F(I) = P(EDP \geq edp|I) \quad (1)$$

The PGA of the scaled accelerograms was selected to be the earthquake parameter (I). Alternatively, it would be possible to use other parameters (magnitude, characteristic intensity etc.) or a combination of two earthquake parameters that would result in the creation of fragility surfaces instead of fragility curves. The current parametrical analysis focuses on estimating the seismic vulnerability of multiple columns using earthquake input data which are easy to be found and can cover broad geographical areas. Thus, the level of PGA, influenced by the soil type which changes the frequency contents of the accelerograms, was selected mainly because it is a parameter already known for most regions.

As EDP, for the current work it was taken the exceeded level of capacity in terms of horizontal displacement measured on column top. It was assumed that the maximum displacement of top required for overturning is equal to the base width. Since the scope of the current study was to investigate the seismic vulnerability of multiple column typologies, it was necessary to compare the fragility curves of the different soil-column systems. Therefore, the fragility curves were smoothened in order to be directly comparable. For the smoothing methodology, which however is not going to be presented here, lognormal distribution of the seismic response data was assumed.

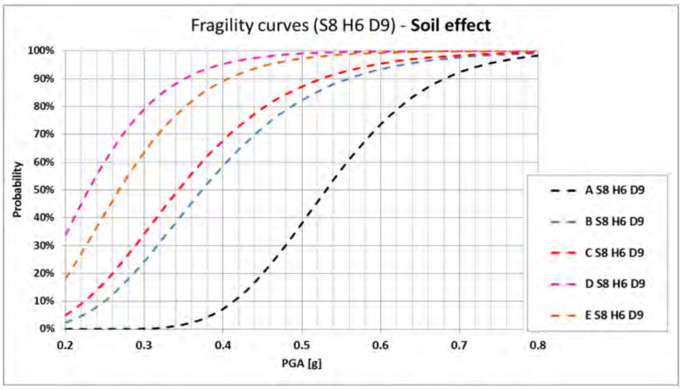
3.2 Seismic excitation results

The numerical results presented here are valuable data for estimating the seismic vulnerability and the influence of the soil, slenderness, height and drums number of free-standing columns. A total of 26 different soil-column configurations was analysed (See Table 2). Since 56 time-history analyses were performed for each configuration, 1456 numerical analyses were demanded in total. The main output data recorded and analysed were the maximum horizontal displacement of the top (the highest point without considering the capital) and the number of remaining drums for analyses where collapse occurred. After creating the elaborated fragility curves, the influence of each parameter was calculated as the ratio of PGA, for a specific probability level, that was demanded for the collapse of the different columns when the only variable was the investigated parameter. As probability level at which the results were compared was selected the collapse probability of 25% ($P_{25\%}$).

Table 2: List of tested elements for the parametrical seismic excitation analysis.

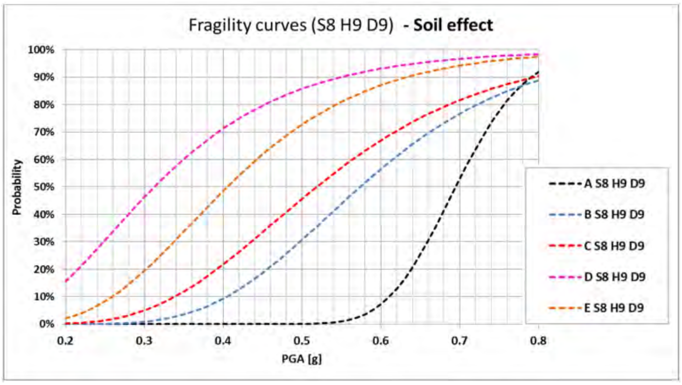
Num.	Soil	Slenderness	Height	Drums	Num.	Soil	Slenderness	Height	Drums
1	A	S8	H6	D9	14	C	S6	H9	D9
2	A	S8	H9	D9	15	C	S6	H3	D9
3	C	S8	H6	D9	16	C	S4	H6	D9
4	C	S8	H9	D9	17	C	S4	H3	D9
5	A	S8	H6	D1	18	C	S5	H6	D9
6	A	S8	H9	D1	19	A	S8	H9	D3
7	B	S8	H6	D9	20	A	S8	H9	D6
8	B	S8	H9	D9	21	D	S8	H9	D9
9	B	S8	H9	D1	22	D	S8	H9	D1
10	C	S8	H9	D1	23	D	S8	H6	D9
11	C	S8	H9	D3	24	E	S8	H9	D9
12	C	S8	H9	D6	25	E	S8	H9	D1
13	C	S6	H6	D9	26	E	S8	H6	D9

The first important influence noticed in this parametrical analysis was the soil effect. As already shown from the harmonic excitation tests, the frequency content of the base excitation is a crucial parameter. The columns on rock (soil A) were proved to be much more stable and able to resist higher levels of base acceleration than columns on soft soils. The worst soil typology was found to be soil D followed by soil E. The numerical results for 9-drum columns of height 9 m and 6 m are shown in Figure 9 and Figure 10. The same behaviour was noticed for the monolithic column of height 9 m (See Figure 11). However, comparing the results of the multi-block and monolithic columns it was noticed that the soil effect was slightly more intense for multi-block configurations. This can be shown in Table 3 where the results for the $P_{25\%}$ are presented normalised for soil C. For the 9-drum element the influence varies from 1.70 (soil A / soil C) to 0.54 (soil D / soil C) while for the monolithic column the same influences are between 1.55 and 0.65.



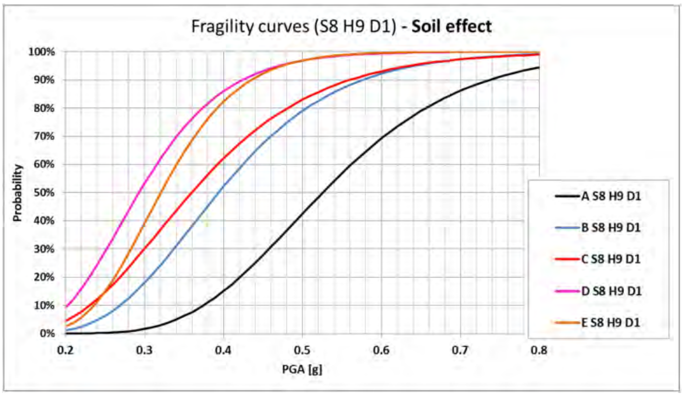
Soil effect					PGA [g] $P_{25\%}$	normalised for soil C
1	A	S8	H6	D9	0.465	1.70
7	B	S8	H6	D9	0.302	1.10
3	C	S8	H6	D9	0.274	1.00
23	D	S8	H6	D9	0.147	0.54
26	E	S8	H6	D9	0.215	0.78

Figure 9: Fragility curves and PGA levels for collapse with $P_{25\%}$ - soil effect for S8 H6 D9 columns.



Soil effect					PGA [g] $P_{25\%}$	normalised for soil C
2	A	S8	H9	D9	0.649	1.57
8	B	S8	H9	D9	0.477	1.15
4	C	S8	H9	D9	0.414	1.00
21	D	S8	H9	D9	0.232	0.56
24	E	S8	H9	D9	0.320	0.77

Figure 10: Fragility curves and PGA levels for collapse with $P_{25\%}$ - soil effect for S8 H9 D9 columns.



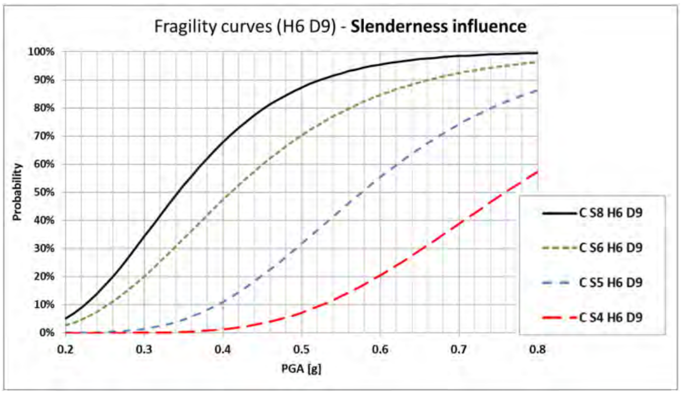
Soil effect					PGA [g] $P_{25\%}$	normalised for soil C
6	A	S8	H9	D1	0.440	1.55
9	B	S8	H9	D1	0.322	1.13
10	C	S8	H9	D1	0.284	1.00
22	D	S8	H9	D1	0.241	0.85
25	E	S8	H9	D1	0.273	0.96

Figure 11: Fragility curves and PGA levels for collapse with $P_{25\%}$ - soil effect for S8 H9 D1 columns.

Table 3: Soil influence for collapse with $P_{25\%}$ normalised for soil C.

Soil influence normalised for soil C					
	H6	H9			
	D9	D9	D1	Mean	St.Dev
A/C	1.70	1.57	1.55	1.60	0.081
B/C	1.10	1.15	1.13	1.13	0.026
C	1.00	1.00	1.00	-	-
D/C	0.54	0.56	0.85	0.65	0.173
E/C	0.78	0.77	0.96	0.84	0.104

Regarding the slenderness, as it was expected, it was also proved to be a very influencing parameter. Numerical analyses were performed for 9-drum elements of height 3 m, 6 m and 9 m on soil C (See Figure 12). However, it is worth mentioning that this influence in terms of PGA level demanded for collapse probability $P_{25\%}$ is not proportional to the variation of slenderness. In Figure 12 it can be seen that the S8 and S6 curves appear relatively close to each other while when the elements become stockier, for S6 and S4, the PGA required for collapse increases drastically. Finally, based on the numerical results it has been possible to quantify the influence of the slenderness on the seismic vulnerability of multi-drum elements. The results of the slenderness influence, normalised for slenderness S6, are presented in Table 4.



Slenderness influence						PGA [g] $P_{25\%}$	normalised for S8	normalised for S6
3	C	S8	H6	D9		0.233	1.00	0.73
13	C	S6	H6	D9		0.319	1.37	1.00
18	C	S5	H6	D9		0.471	2.02	1.48
16	C	S4	H6	D9		0.626	2.69	1.96

Figure 12: Fragility curves and PGA levels for collapse with $P_{25\%}$ - slenderness influence for H6 D9 columns.

Table 4: Slenderness influence for collapse with $P_{25\%}$ normalised for slenderness S6.

Slenderness influence normalised for S6					
	D9			Mean	St.Dev
	H3	H6	H9		
S8/S6	-	0.73	0.74	0.73	0.003
S6	1.00	1.00	1.00	1.00	-
S5/S6	-	1.48	-	1.48	-
S4/S6	1.53	1.96	-	1.75	0.309

It was proven both for multi-drum and monolithic columns for different types of soil and slenderness ratios that bigger elements are less vulnerable than smaller elements with the same proportions. The fragility curves for columns with a height of 6 and 9 m and slenderness S8 for all types of soil are presented in Figure 13. Moreover, it is worth remarking that the fragility curves corresponding to a specific soil and two different heights of columns have always a noticeable shape similarity. The same response was reported for the monolithic columns and for the slenderness S4 and S6 multi-drum columns (Figure 14). In terms of quantified height influence for the H6 and H9 elements, it was found that this influence is more or less in the same level for all types of soil-column configurations. More precisely, in Table 5 are presented the results of PGA for H6 over the PGA for H9 which varies from 0.57 (for C S6 D9) to 0.75 (for A S8 D1). The comparison between different soil types for S8 D9 columns shows results with even lower variation, between 0.63 and 0.72.

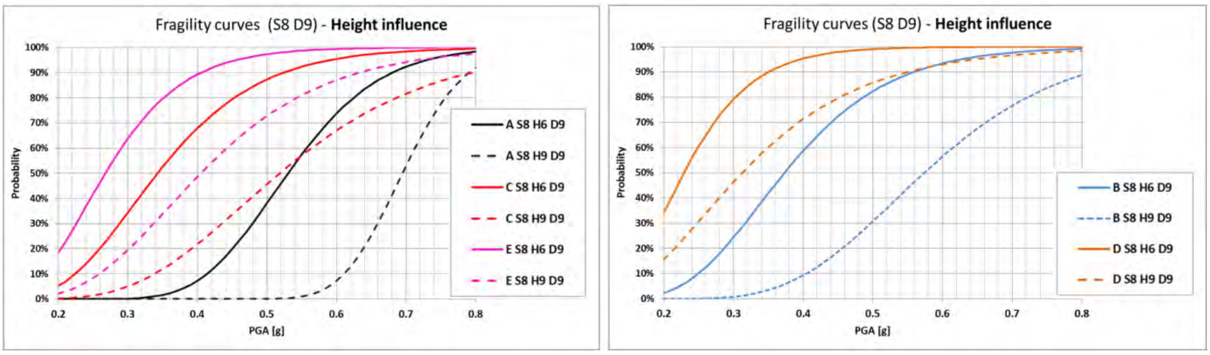


Figure 13: Fragility curves for investigation of the height influence for S8 D9 columns and soils A, C, E (left) and B, D (right).

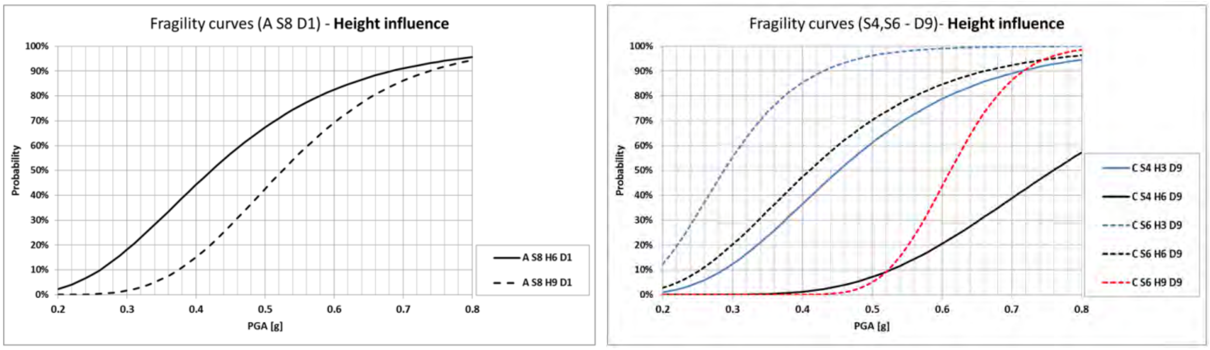


Figure 14: Fragility curves for investigation of the height influence for S8 D1 columns on soil A.

Table 5: Height influence (H6/H9) for different soil-column configurations with collapse probability $P_{25\%}$.

Height influence					PGA [g] $P_{25\%}$	H6/H9
1	A	S8	H6	D9	0.465	0.72
2	A	S8	H9	D9	0.649	
7	B	S8	H6	D9	0.302	0.63
8	B	S8	H9	D9	0.477	
3	C	S8	H6	D9	0.274	0.66
4	C	S8	H9	D9	0.414	
23	D	S8	H6	D9	0.147	0.63
21	D	S8	H9	D9	0.232	
26	E	S8	H6	D9	0.215	0.67
24	E	S8	H9	D9	0.320	
5	A	S8	H6	D1	0.328	0.75
6	A	S8	H9	D1	0.440	
13	C	S6	H6	D9	0.319	0.57
14	C	S6	H9	D9	0.563	
					Mean	0.66
					St.Dev	0.059

4 CONCLUSIONS

The harmonic excitation and the seismic excitation results are complementary findings for the seismic vulnerability assessment. From the harmonic tests we are able to obtain results for the whole range of the investigated parameters due to the fact that all possible column configurations were analysed. However, it is not possible to make a direct estimation of the seismic vulnerability and of the influence of each parameter based only on the harmonic excitation. Performing time-history analyses with artificially generated accelerograms it is possible to assess the vulnerability of the tested elements. Moreover, with a correlation of the harmonic with the seismic analyses results, it is possible to extrapolate the results of the second ones even for soil-column configurations not tested using earthquake accelerograms.

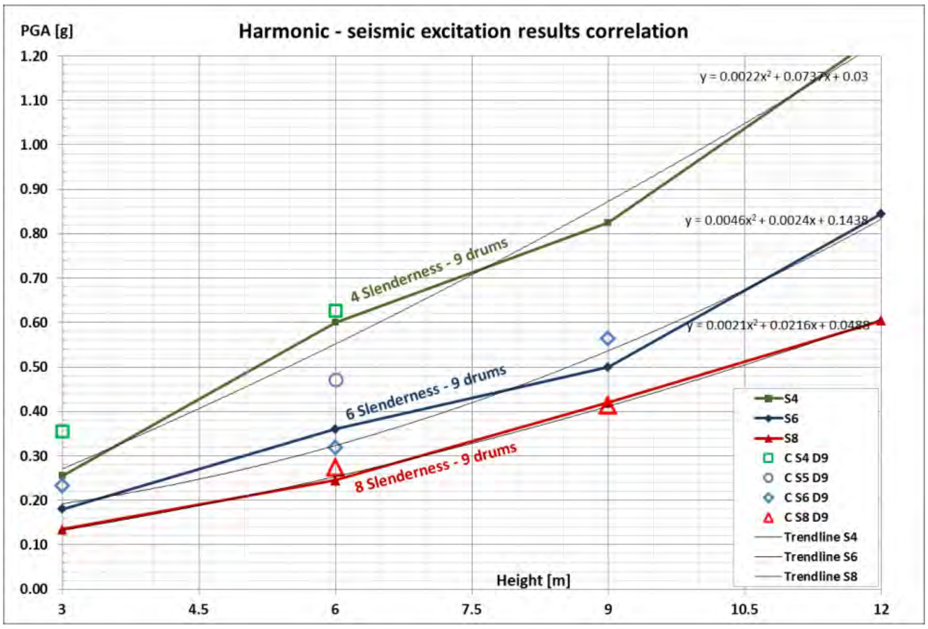


Figure 15: Comparison and correlation between the harmonic and the seismic excitation results for 9-drum columns on soil C.

The first step for finding a correlation between the two methods was to find a range of frequencies for the harmonic excitation where the required PGA for inducing collapse is similar to the PGA level for a specific collapse probability taken from the fragility curves. It was noticed that for the majority of the analysed columns, the collapse probability $P_{25\%}$ of the fragility curves for soil C corresponds to a PGA level of the 10-cycle harmonic excitation around the periods of 1.0 s and 1.5 s. The average PGA for the periods of 1.0 s and 1.5 s for all column tested with harmonic excitation are plotted in Figure 15 with continuous lines. The independent points in the same figure represent the numerical results corresponding to 9-drum columns on soil C for collapse probability $P_{25\%}$. The horizontal axis represents the height of the columns while the vertical axis is the PGA level. The three different colour lines stand for the different slenderness ratios. It can be seen that although the independent points and lines do not match exactly, the correlation between them is quite strong. Therefore, it can be assumed that the continuous lines can represent in a reliable way the overall tendency of the structural behaviour.

The future research within the framework of the project “PROVACI” [2] will aim in creating an easy to use form for the seismic vulnerability assessment of free-standing columns using this type of analyses and quantified results.

5 REFERENCES

[1] Itasca Inc., “UDEC-Universal Distinct Element Code, Version 5.0-User’s Guide.” 2011.

[2] PROVACI, “Tecnologie per la Protezione sismica e la Valorizzazione di Complessi di Interesse culturale (Technologies for the seismic protection and the development of Cultural Heritage sites), PON01_02324 MODEMIUR11.” 2012.

[3] M. Wilson Jones, Principles of Roman Architecture. 2003.

[4] G. Housner, “The behavior of inverted pendulum structures during earthquakes,” Bulletin of the Seismological Society of America, vol. 53, no. 2, pp. 403–417, 1963.

[5] M. J. DeJong, “Seismic Assessment Strategies for Masonry Structures - Ph.D. Thesis,” Massachusetts Institute of Technology, 2009.

[6] G. C. Manos and M. Demosthenous, “Study of the dynamic response of models of ancient columns or colonnades subjected to horizontal mase motions,” in 11th World Conference on Earthquake Engineering (WCEE), 1996.

[7] I. N. Psycharis, D. Y. Papastamatiou, and A. P. Alexandris, “Parametric investigation of the stability of classical columns under harmonic and earthquake excitations,” Earthquake Engineering & Structural Dynamics, vol. 29, pp. 1093–1109, 2000.

[8] P. Gelfi, “SIMQKE_GR - SIMulation of earthQuaKE ground motions (version 2.7).” 2007.

[9] I. Psycharis, M. Fragiadakis, and I. Stefanou, “Seismic reliability assessment of classical columns subjected to near source ground motions,” in Computational Methods in Structural Dynamics and Earthquake Engineering COMPDYN 2013, 2013, no. June.