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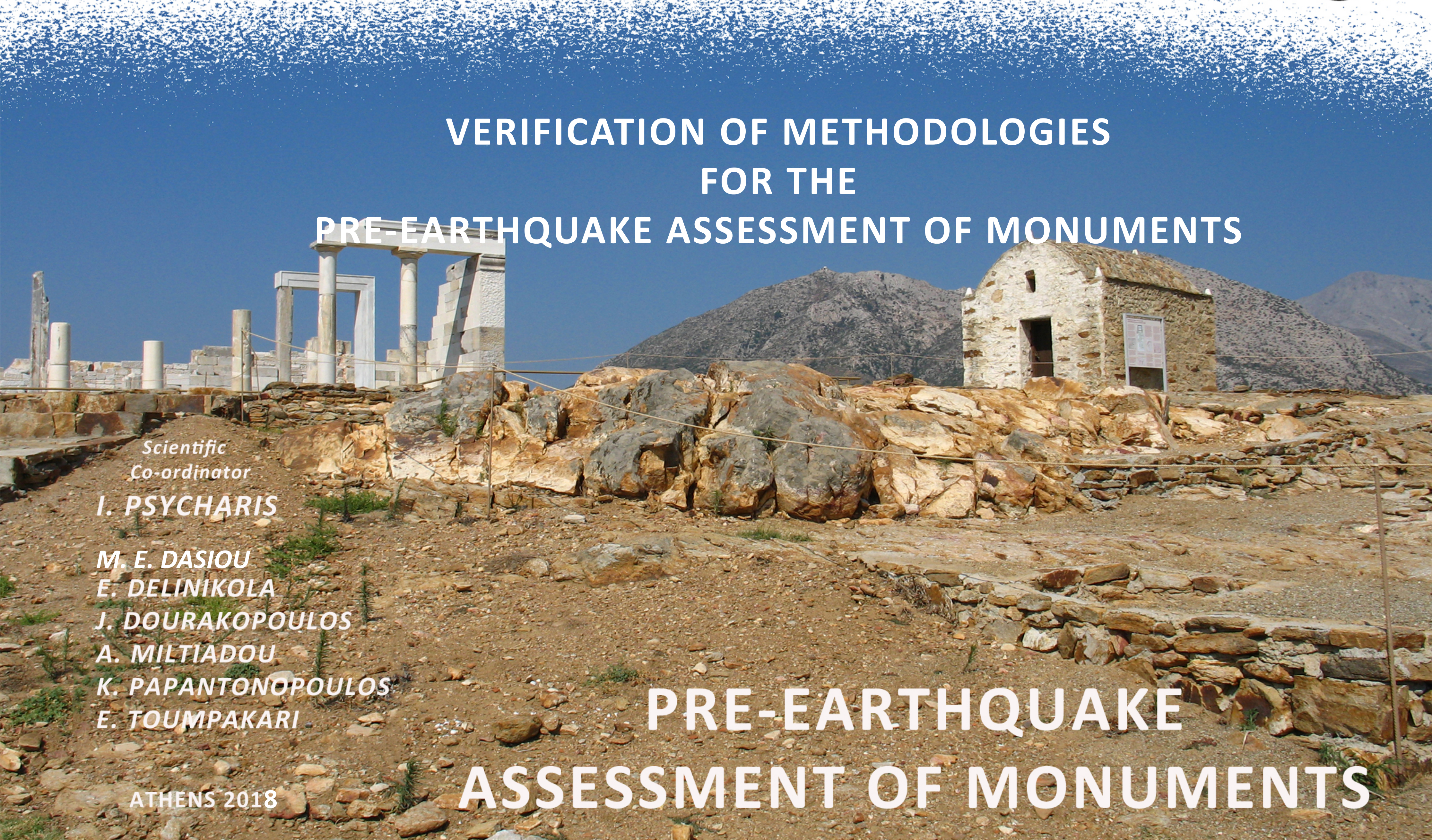

# VERIFICATION OF METHODOLOGIES FOR THE PRE-EARTHQUAKE ASSESSMENT OF MONUMENTS


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# PRE-EARTHQUAKE ASSESSMENT OF MONUMENTS

ATHENS 2018

**Pre – Earthquake assessment of Monuments, Comparison of Empirical and Analytical Methodologies**

# Verification of Methodologies

# for the

# Pre-Earthquake Assessment of Monuments

# PART A

## Single Spaced One Storey Historical Buildings (Structural Category A)

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# 1 INTRODUCTION

## 1.1 Scope of Work

The first degree earthquake assessment of the Single Spaced One Storey Historical Buildings is based on the completion of the documentation sheets and the pre-earthquake assessment data sheet developed in the first phase of the project entitled “Pre – Earthquake assessment of Monuments, Comparison of Empirical and Analytical Methodologies” [1]. The assessment data sheet require information about the structural system, the geometrical characteristics and the pathology of the structures, the seismicity and ground type of the foundation area. Based on the collected information and the rating procedure presented in [1] a vulnerability index is formed for every structure examined, which can be used as an administrative tool for a comparative classification of the historical buildings.

However, the proposed rating procedure [1] is based on empirical methods and should be verified by numerical methods [2]. In the present second phase of the project, several parameters of the pre-earthquake assessment data sheet are further examined and their influence on the seismic response of the structures is evaluated using numerical methods. The methodology and the results presented within the next chapters of the current report are used to determine the final rating methodology of the pre-assessment datasheet.

# 2 NUMERICAL ANALYSIS

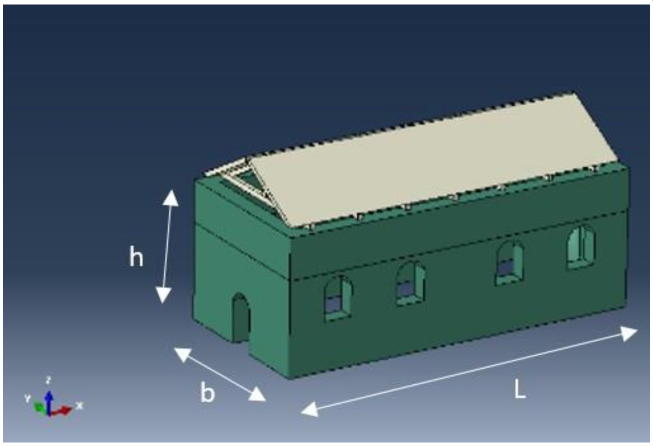
## 2.1 Introduction

To evaluate the individual categories of the parameters included in the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings (EPPO 2016) and verify the rates already set to them, numerical methods were used [2]. Specifically, models of single spaced one storey historical buildings were constructed employing three-dimensional finite elements. The geometrical data used for the design of the structural models (thickness and length of the walls and height of structure) emerged from the examination of the existing basilica churches in the province of Palli in Kefalonia [3,4], representing the majority of the buildings of this typology (Fig. 1).

For the assessment of the seismic response of the structures examined, dynamic elastic analyses during time-history seismic scenarios were used. Specifically, the earthquake of Kefalonia in 2014 (03/02/2014, 38.201N - 20.4373E, Lixouri) was set as the loading scenario and the arising stress condition of the masonry was further examined. Because the tensile strength of the masonry is more critical than the compressive, the principal tensile stresses were computed during the time-history of the earthquake.

| | Church | Location | Municipality | Dimensions |
|---|---|---|---|---|
| 01 | Sotiras | Limin Sotiros | Thinaia | 6.70x8.70 |
| 02 | Aghia Paraskevi | Atheras | Atheras | 7.20x16.50 |
| 03 | Aghios Spiridon | Paralia Athera | Atheras | 5.60x9.30 |
| 04 | Aghia Thekli | Aghia Thekli | Aghia Thekli | 9.20x20.20 |
| 05 | Archaggelos | Aghia Thekli | Aghia Thekli | 7.20x14.90 |
| 06 | Aghia Irini | Vovikes | Aghia Thekli, | 5.50x10.20 |
| 07 | Aghios Dimitrios | Kalata | Aghia Thekli, | 8.00x19.30 |
| 08 | Panagia | Kominata | Aghia Thekli, | 7.50x14.60 |
| 09 | Panagia | Kontogennada | Kontogennada | 7.00x19.90 |
| 10 | Analipseos | Kontogennada | Kontogennada | 5.40x8.90 |
| 11 | Aghios Georgios | Kontogennada | Kontogennada | 4.10x9.50 |
| 12 | Dormition of the Theotokos | Damoulianata | Damoulianata | 10.80x23.70 |
| 13 | Aghios Nikolaos | Rifi | Rifi | 9.50x19.30 |
| 14 | Aghios Georgios | Castle | Rifi | 5.75x8.35 |
| 15 | Aghia Paraskevi | Monopolata | Monopolata | 6.40x20.10 |
| 16 | Panagia | Rogoi | Monopolata | 7.40x15.35 |
| 17 | Aghios Vlassis | Dematora | Monopolata | 6.45x13.90 |
| 18 | Panagia | Dematora | Monopolata | 7.60x16.00 |
| 19 | Kimissi Theotokou | Parissata | Monopolata | 6.45x15.90 |
| 20 | Aghios Ioannis | Favarata | Favarata | 7.35x16.00 |
| 21 | Dodeka Apostoloi | Chavdata | Chavdata | 9.85x21.40 |
| 22 | Sotiras | Chavdata | Chavdata | 6.60x10.50 |
| 23 | Dormition of the Theotokos | Chavriata | Chavriata | 8.80x26.00 |
| 24 | Aghia Marina | Soularoi | Sulari | 9.30x23.00 |
| 25 | Aghios Georgios | Ammoudares | Monopolata | 5.80x11.10 |

(b)



(c)

**Figure 1.** List (a) and Map of the Churches (b) of Palli in Kefalonia [3,4]. Numerical model of single spaced one storey historical building employing three-dimensional finite elements (c).

More specifically, the instants of time during the earthquake (specific seconds of the time-history) when the tensile strength of the masonry is exceeded, were determined. Moreover, the area of the masonry wall where this exceedance appeared ("exceedance area") was also computed. The time of the seismic scenario when the exceedance area received the highest values, were considered as the most critical seconds of the earthquake examined [5,6]. It is noted that two values were set for the tensile strength of the masonry (strength limit), namely 100 kPa and 150 kPa respectively, representing a moderate and a good quality of the existing masonry.

According to the Pre-Earthquake assessment data sheet, each parameter is divided in three or more categories, depending on the geometrical or other structural characteristics of the monument. Therefore, based on the previously presented methodology, a specific rate was assigned to each category describing how vulnerable the monument is with respect to the parameter examined. At first, for the determination of the rate of each Category, the ratio of the largest "exceedance area" of all the categories included in the parameter, to the largest "exceedance area" for the specific category, was computed. Subsequently, based on the aforementioned ratio, a reduction was made so as the most favorable category receive rate 3 (as it had been proposed in the Pre-Earthquake assessment data sheet) and all the other categories were modified respectively. More information on the procedure described is given in the analysis of each parameter set out below.

## 2.2 Parameter 45 (Wall Thickness to Largest Wall Length Ratio)

According to the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, there are three categories for parameter 45 regarding the ratio of the wall thickness to the largest wall length (Fig.1). It should be mentioned that the wall length is defined as the distance between two transversal supports of the wall under study. Examples of existing buildings, that meet the boundaries of each category, are listed below [3,4].

**Category C**: $\frac{wall\,thickness}{wall\,length} = \frac{t}{L} < \frac{1}{12}$

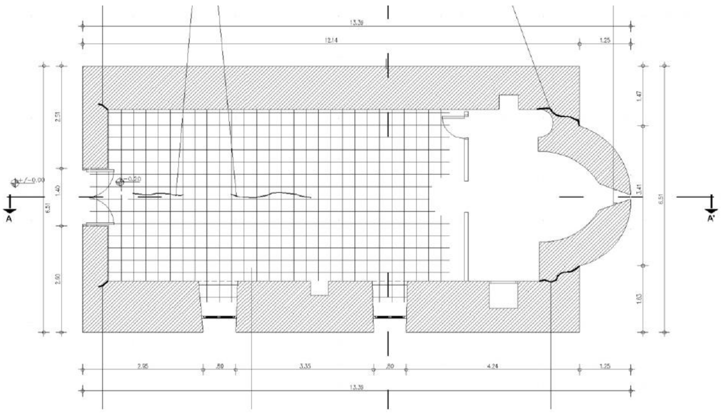


**Figure 2.** Aghios Dimitrios Amarousiou.

**Category B**: $\frac{wall\,thickness}{wall\,length} = \frac{1}{12} < \frac{t}{L} < \frac{1}{8}$

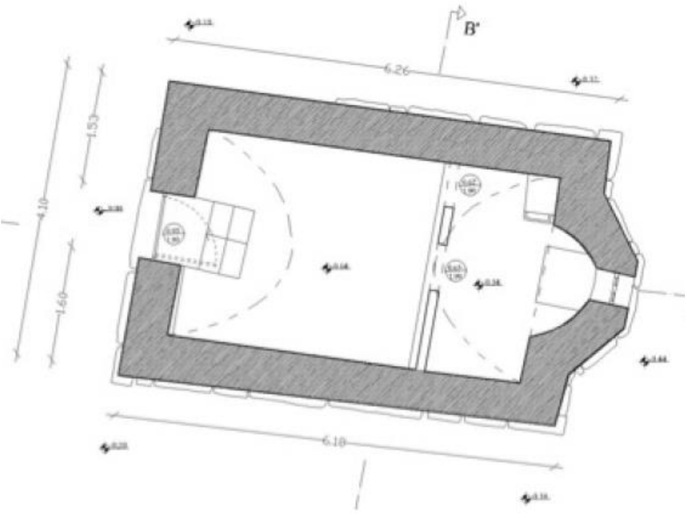


**Figure 3.** Red Church of Aliveri, Euboea (Evia).

**Category A:** $\dfrac{wall\,thickness}{largest\,wall\,length} = \dfrac{1}{8} < \dfrac{t}{L}$

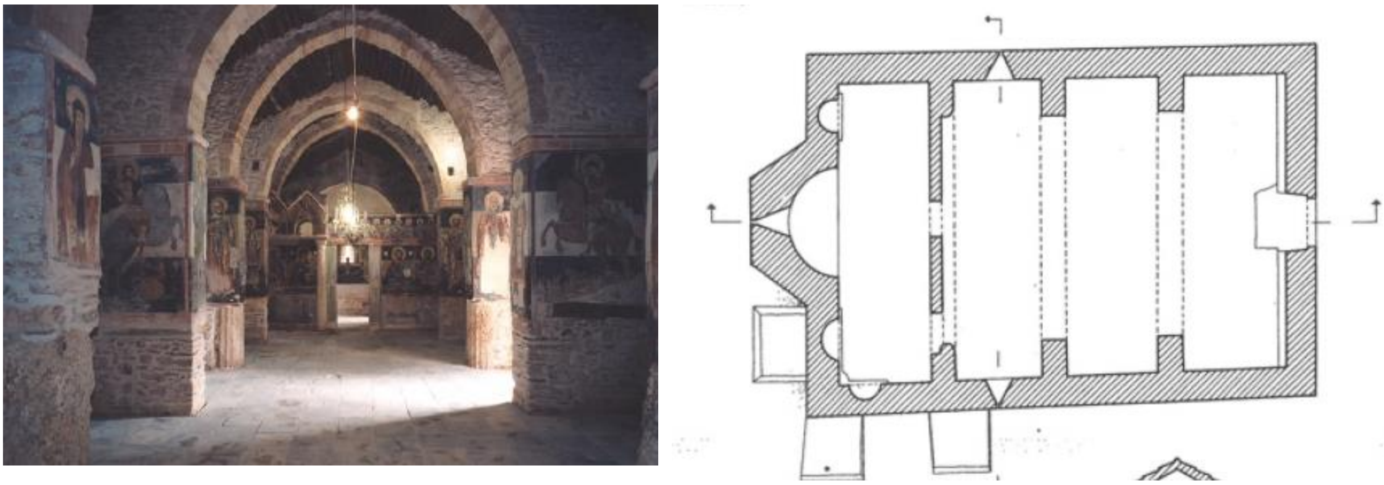


**Figure 4.** Church of Taxiarches, Kalyvia Thorikou in Attica.

In order to examine the aforementioned categories, a structural model of a typical basilica structure with height $h = 5.6m$, length of fixed side $b = 7.0m$ (width of the structure) and wall thickness $t = 0.70m$, was constructed (Fig.1). After substituting the wall thickness, the limits of the various categories with respect to longest walls (without other transversal support) are written as:

**Category C**: $\dfrac{wall\,thickness}{wall\,length} = \dfrac{t}{L} < \dfrac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m$

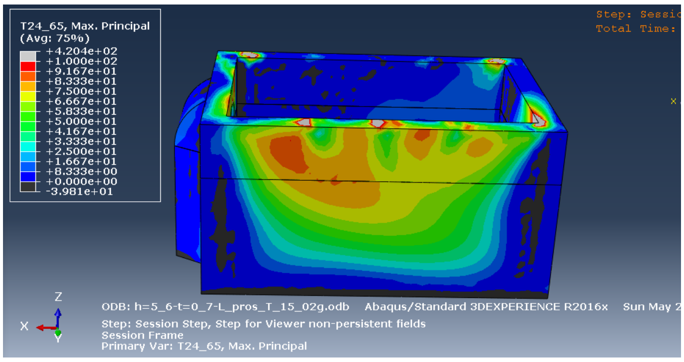
**Category B**: $\dfrac{wall\,thickness}{wall\,length} = \dfrac{1}{12} < \dfrac{t}{L} < \dfrac{1}{8} \Leftrightarrow 8t < L < 12t \Leftrightarrow 5.6m < L < 8.40m$

**Category A:** $\dfrac{wall\,thickness}{wall\,length} = \dfrac{1}{8} < \dfrac{t}{L} \Leftrightarrow L < 8t \Leftrightarrow L < 5.6m$

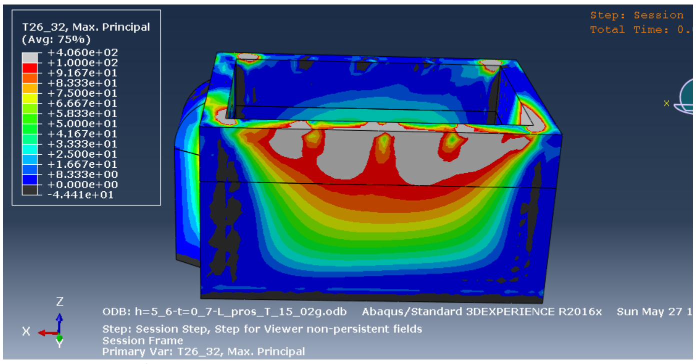
In the following figures, the results of the principal tensile stresses at the most unfavorable seconds of the seismic scenario are presented for each category, while the region, where the tensile strength is exceeded, is computed. It is noted that this region is marked gray to be readily recognizable. Finally, for each time moment, two tensile strength limits (100 kPa and 150 kPa) are employed representing two masonry qualities.

**Category C**: $\dfrac{t}{L} < \dfrac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m$

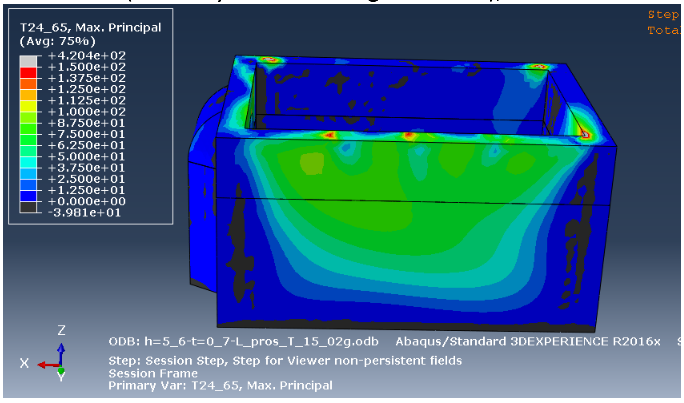
**Case I** $\dfrac{t}{L} < \dfrac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m \Rightarrow L = 10.5m\left(L = 15t\right)$



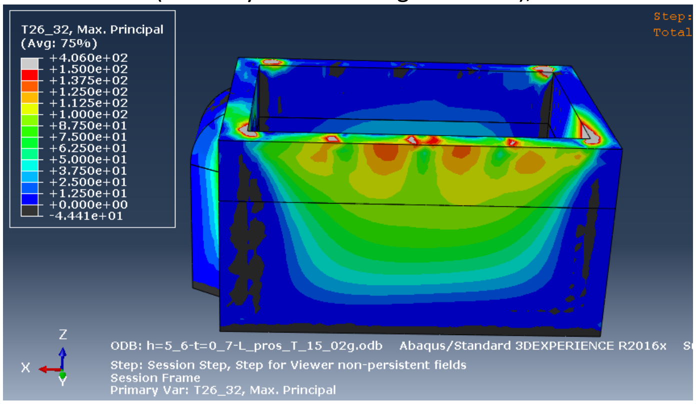

$A_{exc.}$ = 0.00 (masonry tensile strength 100kPa), t=24.65sec




$A_{exc.}$=11.90m$^2$ (masonry tensile strength 100kPa), t=26.32s



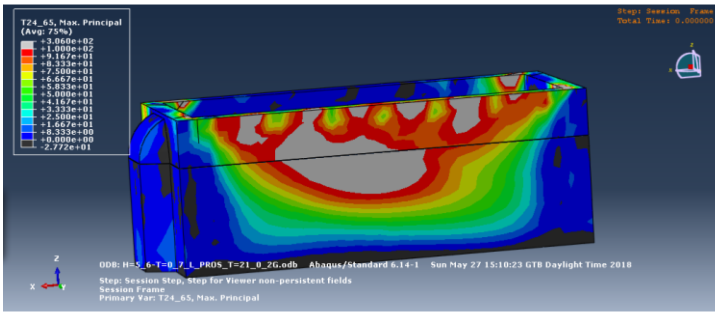

$A_{exc.}$ = 0.00 m$^2$ (masonry tensile strength 150kPa), t=24.65sec



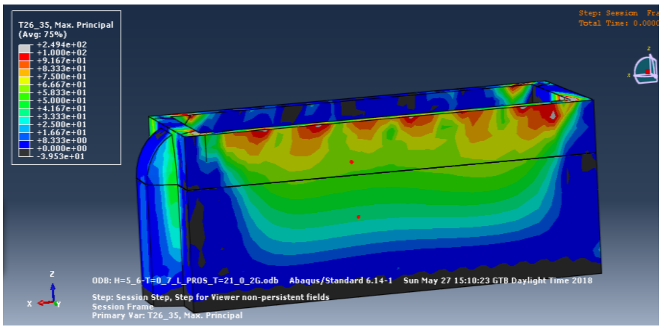

$A_{exc.}$ = 0.03 m$^2$ (masonry tensile strength 150kPa), t=26.32sec

**Category C:** $\frac{t}{L} < \frac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m$

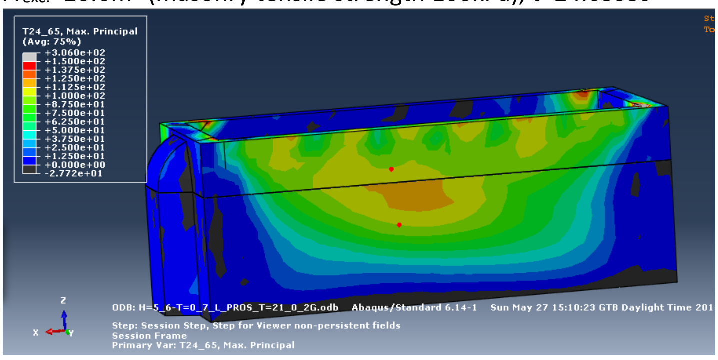
**Case II** $\frac{t}{L} < \frac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m \Rightarrow L = 15m\left(L = 21.4t\right)$



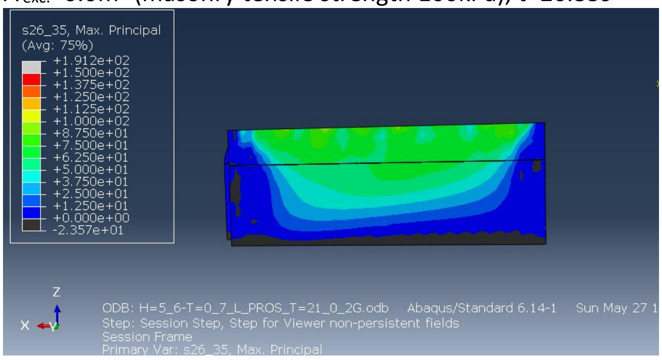

$A_{exc.}$=20.6m$^2$ (masonry tensile strength 100kPa), t=24.65sec




$A_{exc.}$=0.0m$^2$ (masonry tensile strength 100kPa), t=26.35s

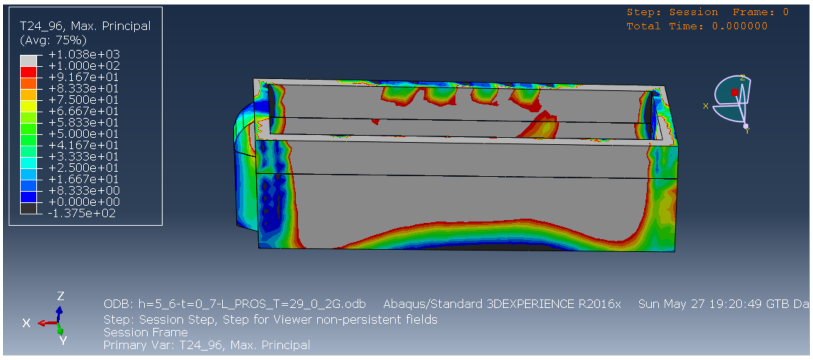



$A_{exc.}$=0.m$^2$ (masonry tensile strength 150kPa), t=24.65sec

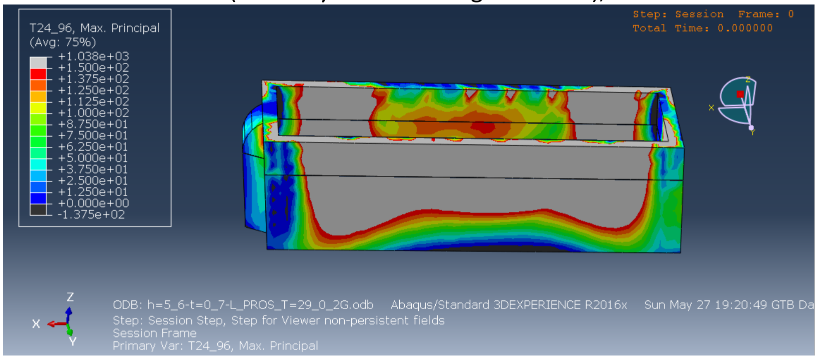



$A_{exc.}$=0.m$^2$ (masonry tensile strength 150kPa), t=26.35sec

**Category C:** $\frac{t}{L} < \frac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m$ **, Case III** $\frac{t}{L} < \frac{1}{12} \Leftrightarrow L > 12t \Leftrightarrow L > 8.40m \Rightarrow L = 20m\left(L = 28.6t\right)$

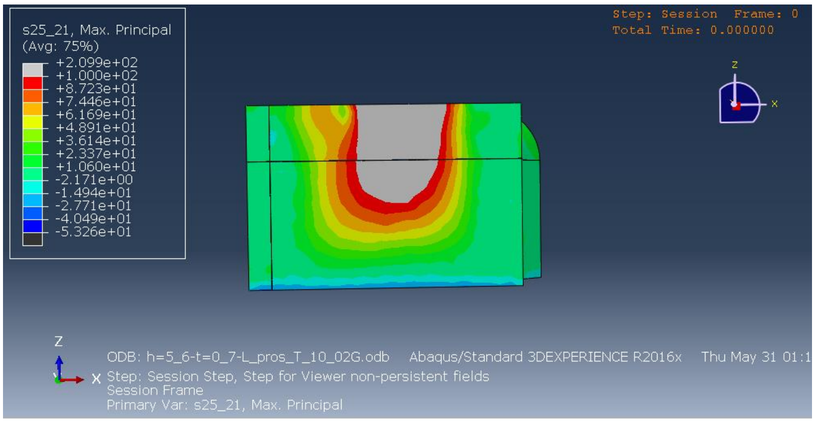



A $_{exc.}$=112 m$^2$ (masonry tensile strength 100kPa), t=24.96sec

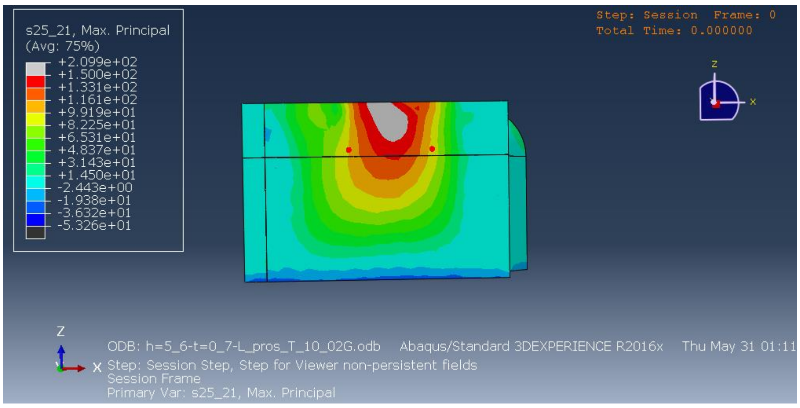



A $_{exc.}$=100m$^2$ (masonry tensile strength 150kPa), t=24.96sec

**Category B:** $\frac{1}{12} < \frac{t}{L} < \frac{1}{8} \Leftrightarrow 8t < L < 12t \Leftrightarrow 5.6 < L < 8.4 \Leftrightarrow L = 7m\ \left(L = 10t\right)$

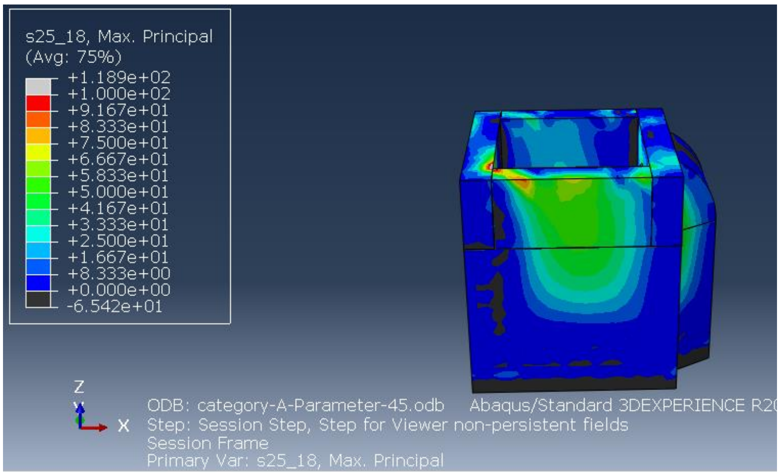



A $_{exc.}$=7.95 m$^2$ (masonry tensile strength 100kPa), t=25.21sec

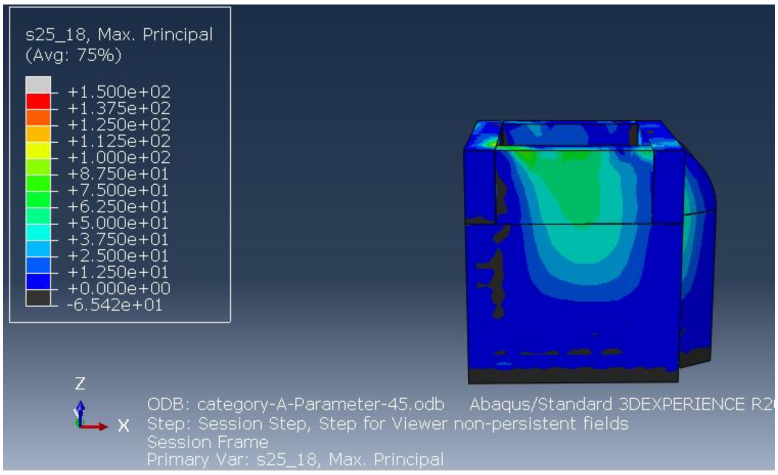



A $_{exc.}$=1.2 m$^2$ (masonry tensile strength 150kPa), t=25.21sec

**Category A:** $\frac{1}{8} < \frac{t}{L} \Leftrightarrow L < 8t \Leftrightarrow L < 5.6 \Rightarrow L = 5.0m$ In this case, the width of the structure (length of fixed side) was changed from b=7.0m to b=4.9m so as the wall along direction East-West (normal to the horizontal bearing elements) to remain the longest of the structure.



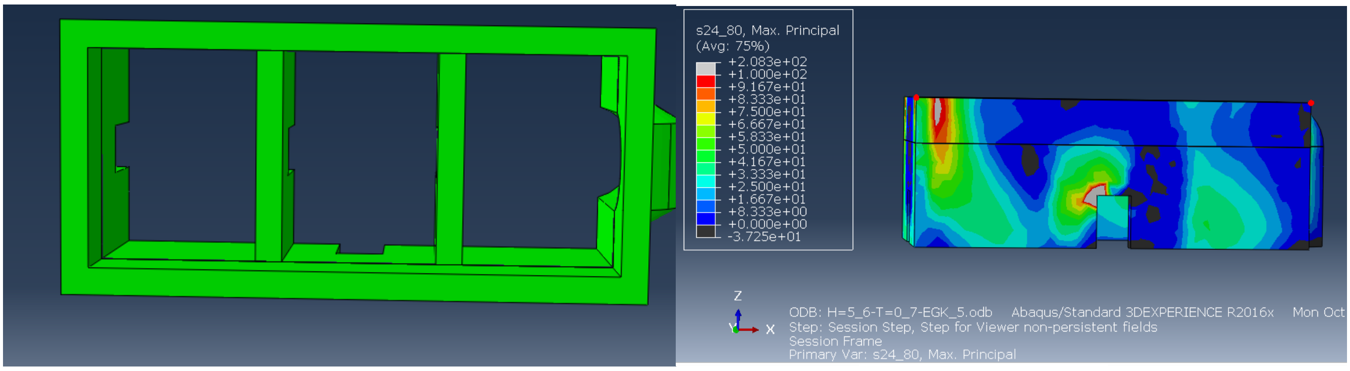

A $_{exc.}$=0.05 m$^2$ (masonry tensile strength 100kPa), t=25.18sec



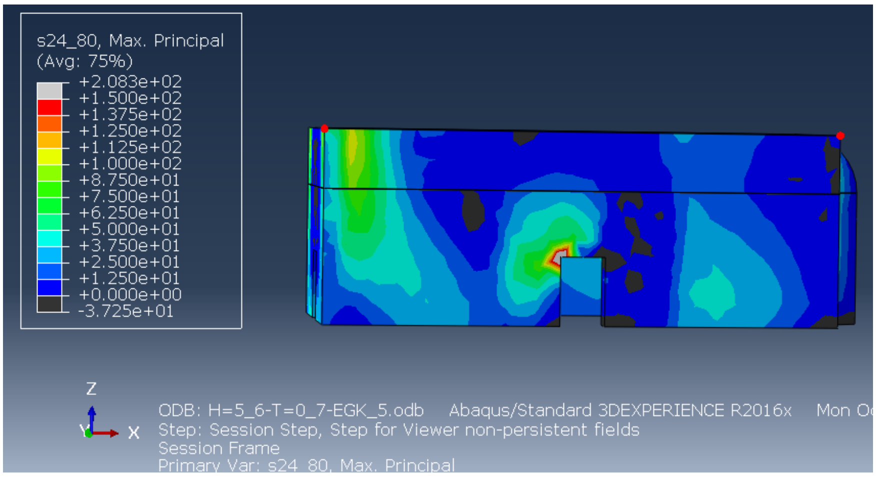

A $_{exc.}$=0.0 m$^2$ (masonry tensile strength 150kPa), t=25.18sec

It should be mentioned that a structure with largest wall length equal or bigger than 8t also belongs to Category A, if transversal restraints (walls, arches) exist every 8t (where t is the thickness of the wall). These transversal restraints should not allow the out of plane motion of the largest wall. Therefore, a structure with largest wall length equal to 15m and transversal walls every 5.0 m which support both large walls, is examined. As it is easily concluded, the results of both the structures belonging to Category A are similar.

**Category A:** $L = 5.0m\left(L_{total} = 15.0m\right)$




A $_{exc.}$=1 m$^2$ (masonry tensile strength 100kPa), t=24.80sec




$A_{exc.}$=0.2 $m^2$ (masonry tensile strength 150kPa), t=24.80sec

Concluding, the exceedance area can become rather important when the ratio of the maximum length to the wall thickness increases significantly (Case III of Category C: $L = 20m$ namely $L/t = 28.6$). Even though case III of category C is considered to be rare (largest length of walls without support or other transversal restraints equal or larger than 20m), the division of parameter 45 is suggested to be made in four categories (contrary to the three categories already suggested in the Pre-Earthquake assessment data sheet) namely:

**Category D**: $\frac{t}{L} < \frac{1}{25} \Leftrightarrow L > 25t \Leftrightarrow L > 17.5m\left( \text{ for } t = 0.70m \right)$

**Category C**: $\frac{1}{25} < \frac{t}{L} < \frac{1}{17} \Leftrightarrow 17t < L < 25t \Leftrightarrow 12m < L < 17.5m$

**Category B**: $\frac{1}{17} < \frac{t}{L} < \frac{1}{12} \Leftrightarrow 12t < L < 17t \Leftrightarrow 8.4m < L < 12.0m$

**Category A:** $\frac{1}{12} < \frac{t}{L} \Leftrightarrow L < 12t \Leftrightarrow L < 8.4m$

The rate of each category results from the “exceedance area” ratio, namely the ratio of the largest “exceedance area” computed for all the categories, belonging to the specific parameter, to the largest “exceedance area” of the category examined. It is easily observed that case III of the Category C ( $L = 20m\left(L = 28.6t\right)$ ) is the worst of all the cases examined, regarding the stress condition of the walls during the seismic scenario. As a result, case III of the Category C (also called Category D according to the present proposal) is assigned with rate 1. Therefore, the rates for the rest Categories can be written as

**Category D: rate** $\frac{112}{112} = 1$

**Category C: rate** $\frac{112}{20.6} = 5$

**Category B: rate** $\frac{112}{7.95} \approx 14$

**Category A: rate** $\frac{112}{2} = 56$

Due to the significant difference between the new Category D and the most favorable Category A (with smallest "exceedance area"), a reduction is made by setting the maximum rate equal to 3 (Category A) following therefore the suggestions of the Pre-Earthquake assessment data sheet. It is noted that according to the data sheet, the worst case of each category is represented by the smallest rate. Therefore, the final rates are written as

**Category D: rate** $\frac{112}{112}\cdot\frac{3}{56}=0.05$

**Category C: rate** $\frac{112}{20.6}\cdot\frac{3}{56}=0.30$

**Category B: rate** $\frac{112}{7.95}\cdot\frac{3}{56}\approx 0.80$

**Category A: rate** $\frac{112}{2.0}\cdot\frac{3}{56}=3.0$

## 2.3 Parameter 46 (Wall Thickness to Wall Height Ratio)

According to the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, there are three categories for parameter 46. Examples of existing buildings, that meet the boundaries of each category, are listed below.

**Category C**: $\frac{wall\,thickness}{wall\,height}=\frac{t}{h}<\frac{1}{9}\Leftrightarrow h>9t$

**Figure 5.** Damoulianata, Kefalonia

**Figure 6.** Apostoloi Chavdata, Kefalonia

**Category B**: $\frac{wall\,thickness}{wall\,height} \Rightarrow \frac{1}{9} < \frac{t}{h} < \frac{1}{7} \Leftrightarrow 7t < h < 9t$

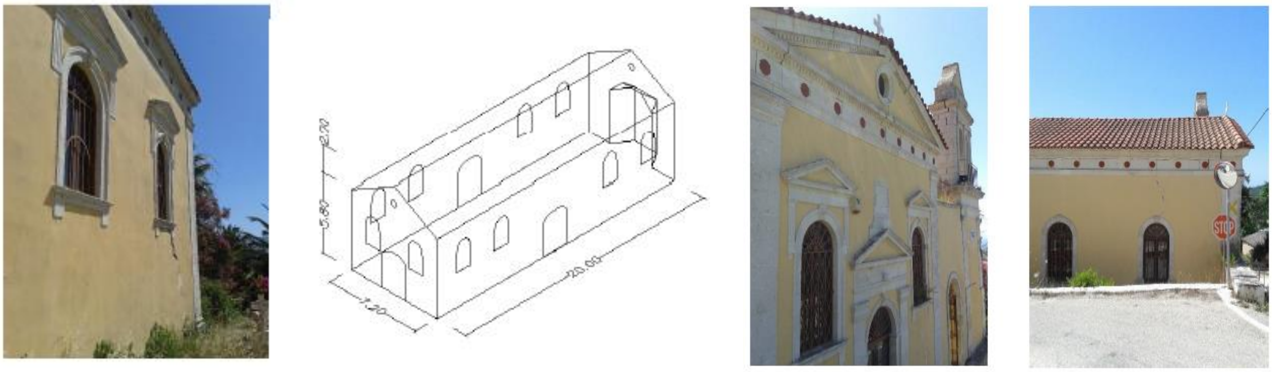



**Figure 7.** Panagia Kontogennada, Kefalonia


**Figure 8.** Aghia Thekli, Kefalonia

**Category A**: $\frac{wall\,thickness}{wall\,height} \Rightarrow \frac{t}{h} > \frac{1}{7} \Leftrightarrow h < 7t$


**Figure 9**. Panagia Kalata, Kefalonia

In order to examine the aforementioned categories, a typical basilica structure was simulated with largest and smallest length of walls equal to $L = 15.0m$ *and* $b = 7.0m$ respectively and wall thickness *t* equal to 0.70m. As a result, after substituting the wall thickness to the definition of each category, the limits of the various categories with respect to the height of the walls (without transverse walls) are written as:

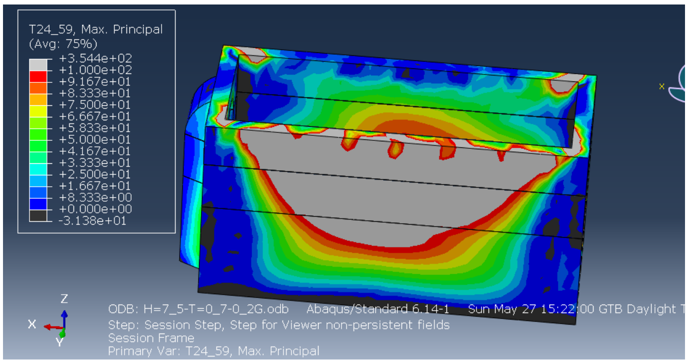
**Category C**: $\frac{t}{h} < \frac{1}{9} \Leftrightarrow h > 9t \Leftrightarrow h > 6.3 \Rightarrow h = 7.5m\left(namely\ \ 10.7t\right)$

**Category B**: $\frac{1}{9} < \frac{t}{h} < \frac{1}{7} \Leftrightarrow 7t < h < 9t \Leftrightarrow 4.9 < h < 6.3 \Rightarrow h = 5.6m\left(namely\,8t\right)$

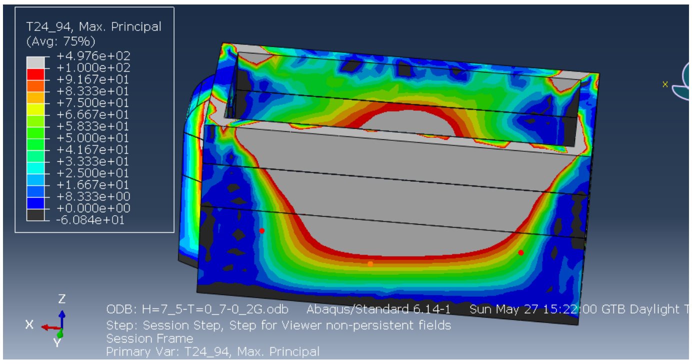
**Category A**: $\frac{t}{h} > \frac{1}{7} \Leftrightarrow h < 7t \Leftrightarrow h < 4.9 \Rightarrow h = 4.5m\ \left(namely\,6.4t\right)$

In the following figures, the principal tensile stresses for each category at the most unfavorable seconds of the seismic scenario are presented, while the region, where the masonry tensile strength is exceeded, is computed (gray area). Finally, for each time moment, two tensile strength limits (100 kPa and 150 kPa) are employed representing two masonry qualities.

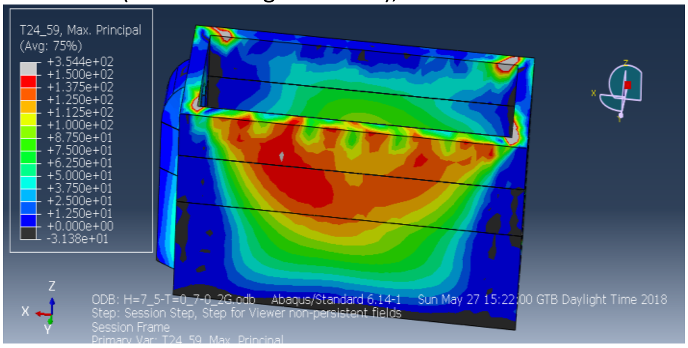
**Category C**: $\frac{t}{h} < \frac{1}{9} \Leftrightarrow h > 9t \Leftrightarrow h > 6.3 \Rightarrow h = 7.5m\left(10.7t\right)$




$A_{exc.}$=41m$^2$ (tensile strength 100kPa), t=24.59sec




$A_{exc.}$=52 m$^2$ (tensile strength. 100kPa), t=24.94sec

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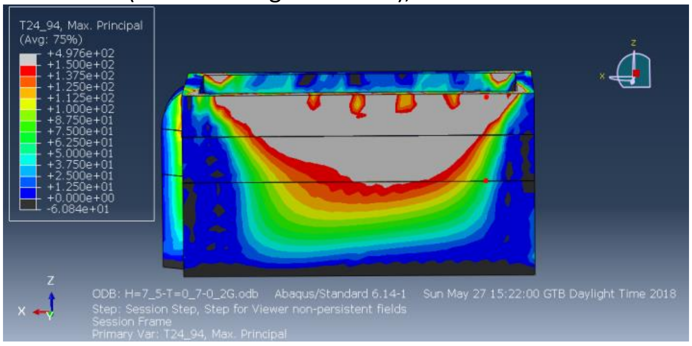
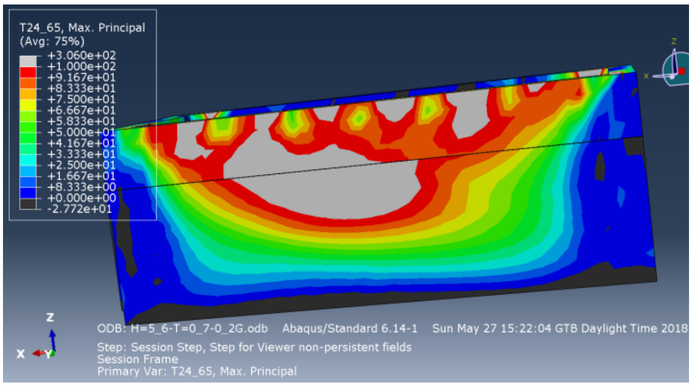
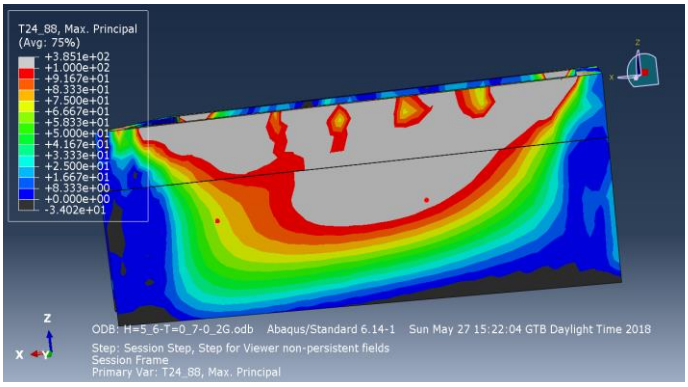



$A_{exc.}$ =44m$^2$ (tensile strength 150kPa), t=24.94sec

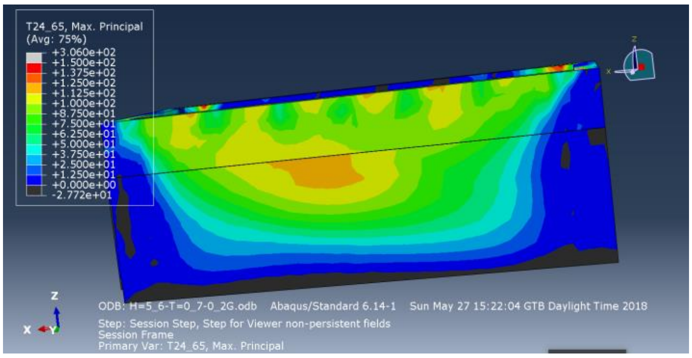
**Category B**: $\frac{1}{9} < \frac{t}{h} < \frac{1}{7} \Leftrightarrow 7\tau < h < 9t \Leftrightarrow 4.9 < h < 6.3 \Rightarrow h = 5.6m$



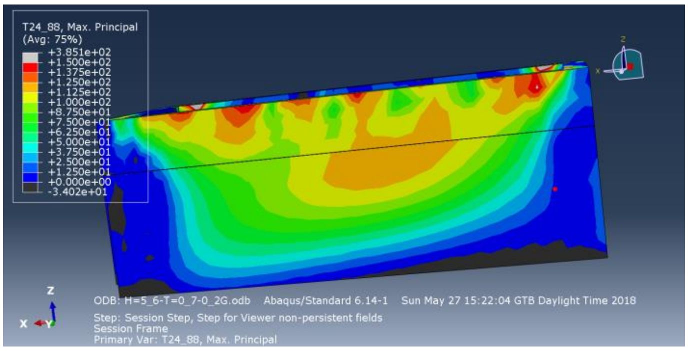

$A_{exc.}$=26.6 m$^2$ (masonry tensile strength 100kPa), t=24.65sec




$A_{exc.}$=27.75 m$^2$ (masonry tensile strength 100kPa), t=24.88sec

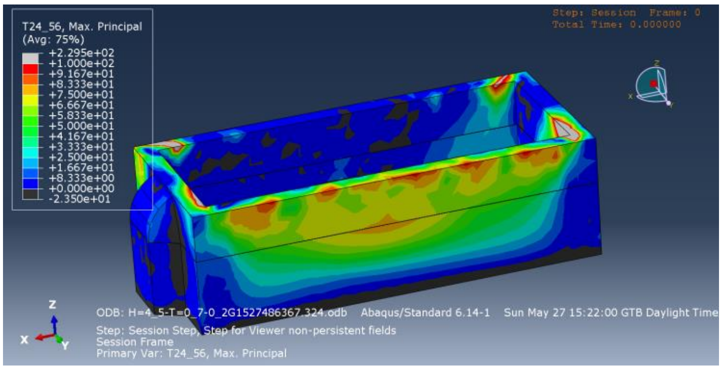



$A_{exc.}$=0.0 m$^2$ (masonry tensile strength. 150kPa), t=24.65sec

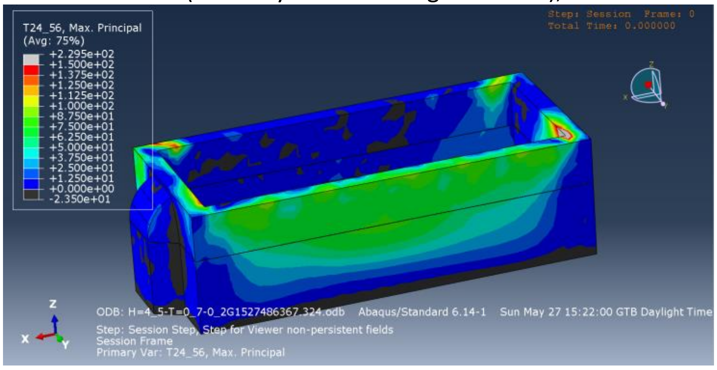



$A_{exc.}$=0.5m$^2$ (masonry tensile strength 150kPa), t=24.88sec

**Category A:** $\frac{t}{h} > \frac{1}{7} \Leftrightarrow h < 7t \Leftrightarrow h < 4.9 \Rightarrow h = 4.5m$




A $_{exc.}$ =0.00 m$^2$ (masonry tensile strength 100kPa), t=24.56sec




A $_{exc.}$ =0.00 m$^2$ (masonry tensile strength 150kPa), t=24.56sec

Based on the results presented where the largest exceedance area is computed 52m$^2$ for masonry tensile strength 100kPa, the rates for each Category obtain the following values

**Category C: rate** $\frac{52}{52} = 1$

**Category B: rate** $\frac{52.0}{27.75} = 1.87 \approx 2$

**Category A: rate** $\frac{52.0}{0.00} \Rightarrow \textit{because the ratio becomes indefinite, grade is set to a large value, namely } 5.0$

Following the same methodology presented in the previous parameter, a normalization is needed so as a rate equal to 3.0 is set to the most favorable case. Therefore, the final rates of all the Categories are written as

**Category C: rate** $\frac{3}{5}\cdot 1 = 0.6$

**Category B: rate** $\frac{3}{5}\cdot 2 = 1.20$

**Category A: rate** $\frac{3}{5}\cdot 5 = 3$

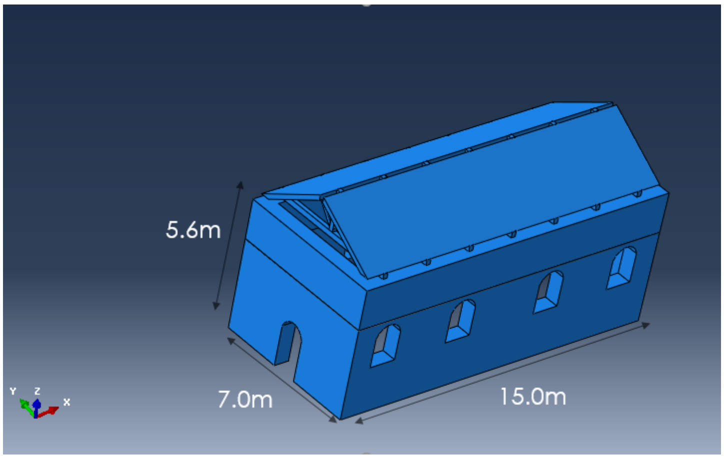
## 2.4 Parameter 49: Openings near the Corners (Distance from the Corner)

According to the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, there are three categories concerning parameter 49.

**Category C**: Openings at a distance less than 1.2m from the corners

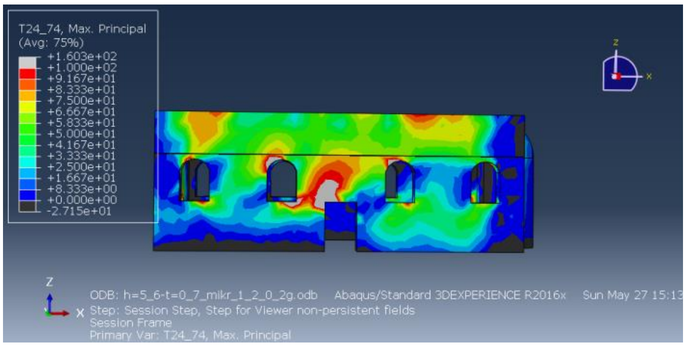
**Category B**: Openings at a distance between 1.2m and 2.0m

**Category A**: Openings at a distance larger than 2.0m

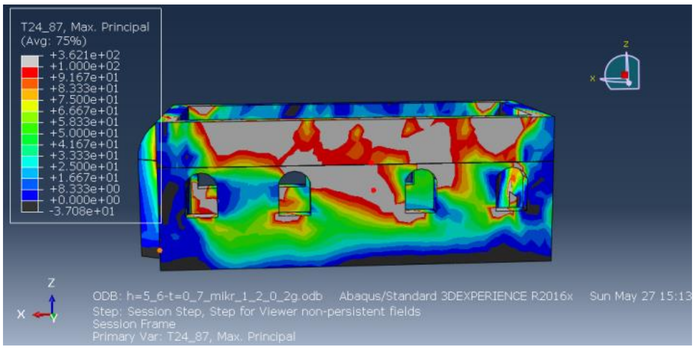



In order to examine the aforementioned categories, a typical basilica structure was simulated with longest and smallest walls equal to $L = 15.0m \; and \; b = 7.0m$, respectively, wall thickness $t = 0.70m$ and height $h = 5.6m$. The parameter examined concerns the distance of the closest opening to the corner of the structure. In the following figures, the results of the maximum principal tensile stresses during the time-history of the examined scenario are presented.

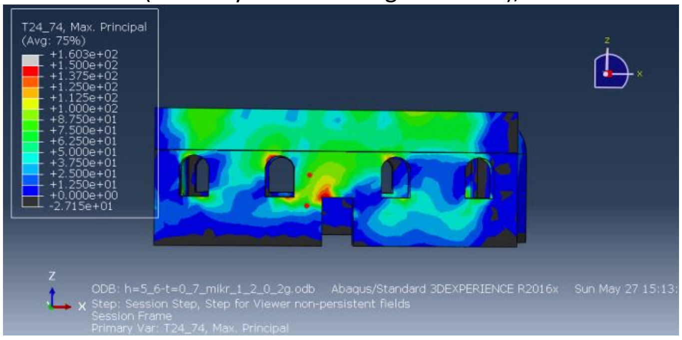
**Category C**: Openings near the corners at a distance smaller than 1.2m



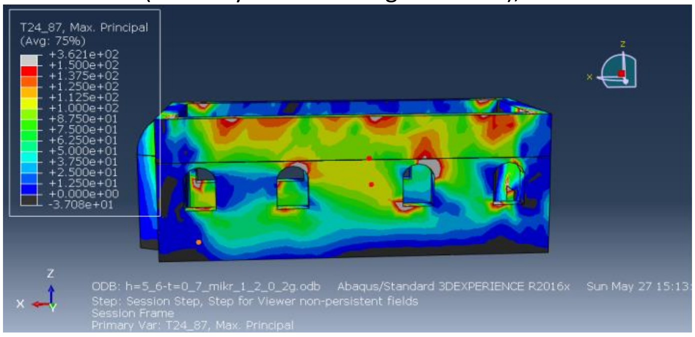

$A_{exc.}$=0.91m$^2$ (masonry tensile strength 100kPa), t=24.74sec




$A_{exc.}$=33.0m$^2$ (masonry tensile strength 100kPa), t=24.87sec

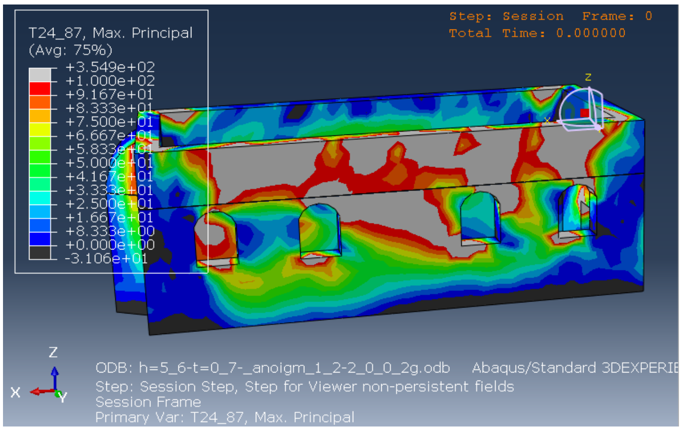



$A_{exc.}$= 0.0m$^2$ (masonry tensile strength 150kPa), t=24.74sec

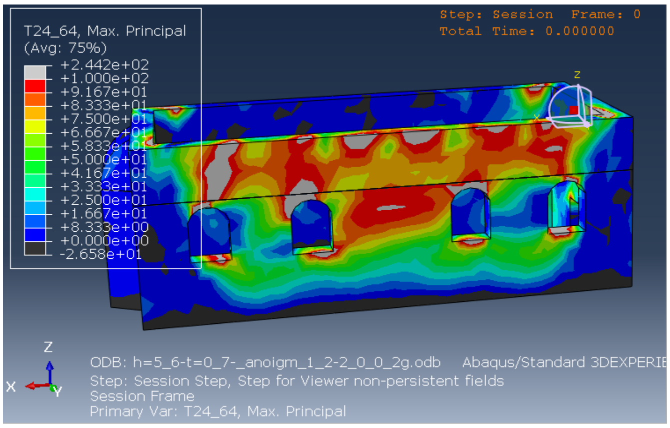



$A_{exc.}$=0. 5m$^2$ (masonry tensile strength 150kPa), t=24.87sec

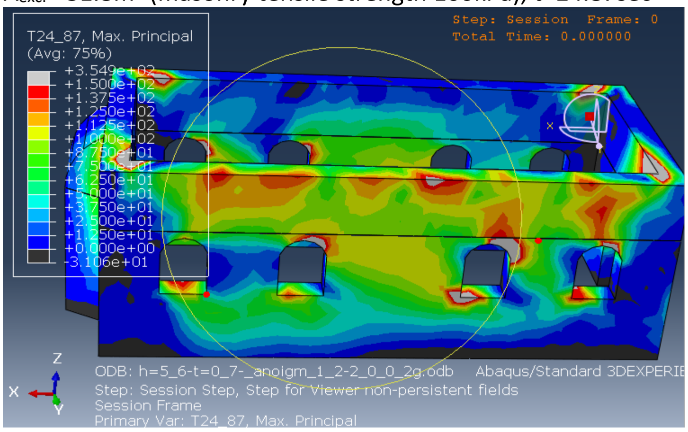
**Category B**: Openings near the corners at a distance between 1.2m and 2.0m



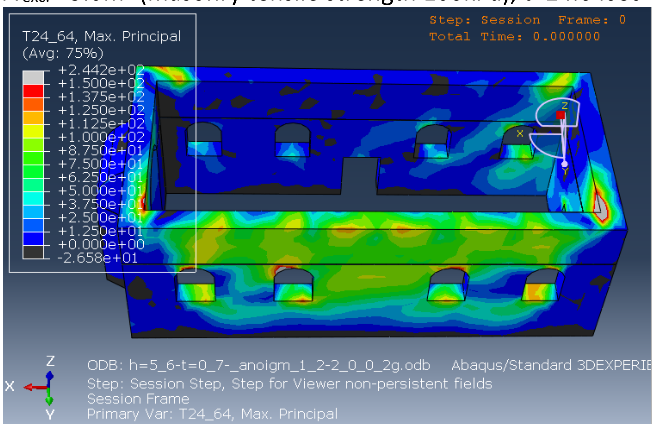

$A_{exc.}$= 31.8m$^2$ (masonry tensile strength 100kPa), t=24.87sec




$A_{exc.}$= 5.0m$^2$ (masonry tensile strength 100kPa), t=24.64sec

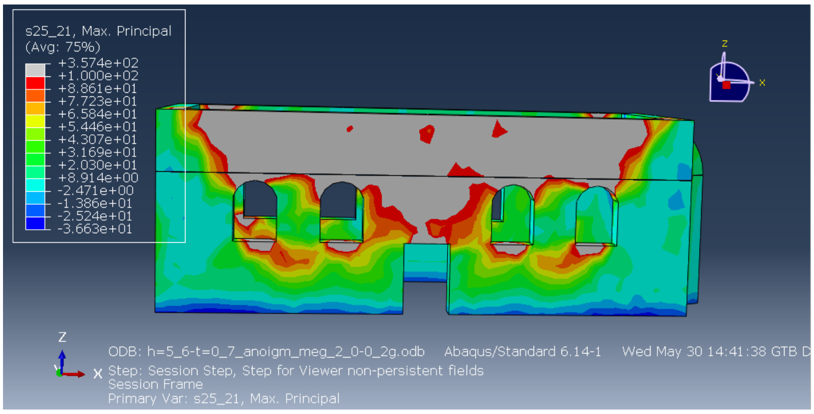



$A_{exc.}$= 0.5m$^2$ (masonry tensile strength 150kPa), t=24.87sec

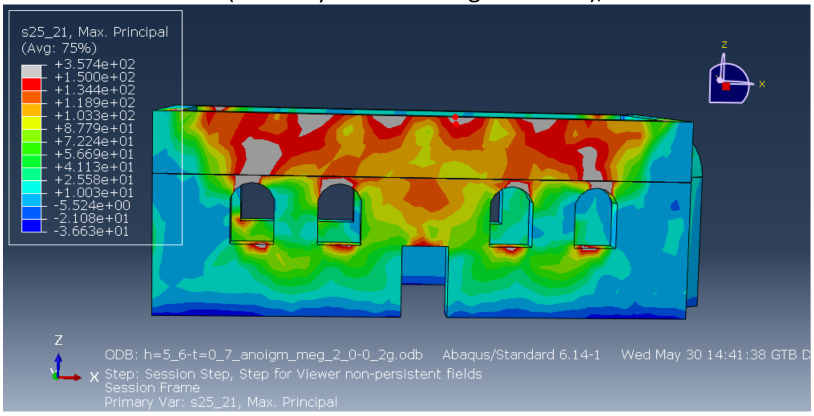



$A_{exc.}$= 0.0m$^2$ (masonry tensile strength 150kPa), t=24.64sec

**Category A**: Openings near the corners at a distance more than 2.0m




$A_{exc.}$=31.6m$^2$ (masonry tensile strength 100kPa), t=25.21sec




$A_{exc.}$=2.96m$^2$ (masonry tensile strength 150kPa), t=25.21sec

The rating of the categories results from the "exceedance area" ratio, namely the ratio of the largest "exceedance area" as computed from all the categories of the parameter examined to the largest "exceedance area" of the category examined. As it was expected, Category C is more critical and Category A is the most favorable. Therefore, the rate for Category A is set to 3.0.

**Category C: rate** $\frac{33.0}{33.0}\cdot\frac{3.0}{1.05}=2.86\approx 2.90$

**Category B: rate** $\frac{33.0}{31.8}\cdot\frac{3.0}{1.05}=2.97\approx 3.0$

**Category A: rate** $\frac{33.0}{31.6}=1.05\Rightarrow\frac{33.0}{31.6}\cdot\frac{3}{1.05}=3.0$

Based on the results presented, it is easily concluded that the differences between the three categories are ignorable. The effect of the distance of the closest opening to the corner of the wall, where more than one openings exist, is not significant. On the contrary, this effect may become more important, when the wall has only one opening. However, this case was not studied because almost all the basilica structures have more than one openings in the perimetric walls.

## 2.5 Parameter 52 (Ground type)

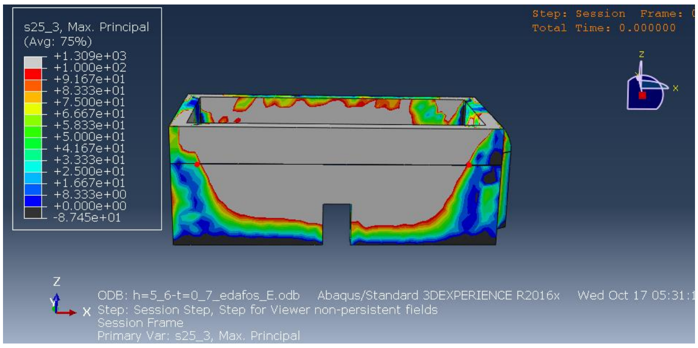
According to the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, the ground type is divided into four categories (A – [B, C] – D – E, ground types B and C were set in a group) as compared to EC8 where ground type is divided in 5 categories as shown in Table 1.

In order to examine the effect of the aforementioned ground types, a typical basilica structure was simulated with length of the longest and smallest walls equal to $L = 15.0m$ and $b = 7.0m$ , respectively, wall thickness $0.70m$ and height $h = 5.6m$ . Subsequently, the maximum principal tensile stresses and the corresponding moment during the examined seismic scenario are presented. Moreover, ground types B and C were examined separately in order to verify the validity of the assumption made in the Pre-Earthquake assessment data sheet.

**Table 1.** Ground types

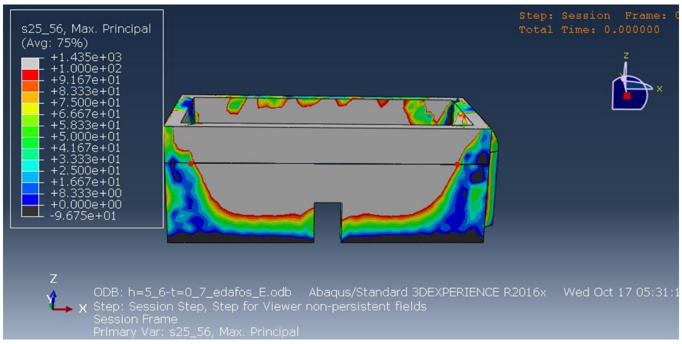
| Ground type | Description of stratigraphic profile | Parameters | | |
|---|---|---|---|---|
| | | $v_{s,30}$ (m/s) | $N_{SPT}$ (blows/30cm) | $c_u$ (kPa) |
| A | Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface. | > 800 | _ | _ |
| B | Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth. | 360 – 800 | > 50 | > 250 |
| C | Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres. | 180 – 360 | 15 - 50 | 70 - 250 |
| D | Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil. | < 180 | < 15 | < 70 |
| E | A soil profile consisting of a surface alluvium layer with $v_s$ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s$ > 800 m/s. | | | |
| $S_1$ | Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content | < 100 (indicative) | _ | 10 - 20 |
| $S_2$ | Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or $S_1$ | | | |

**Category E**: ground type E

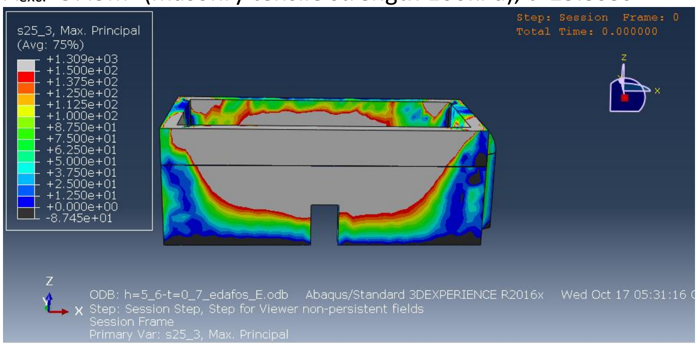



A<sub>exc.</sub>

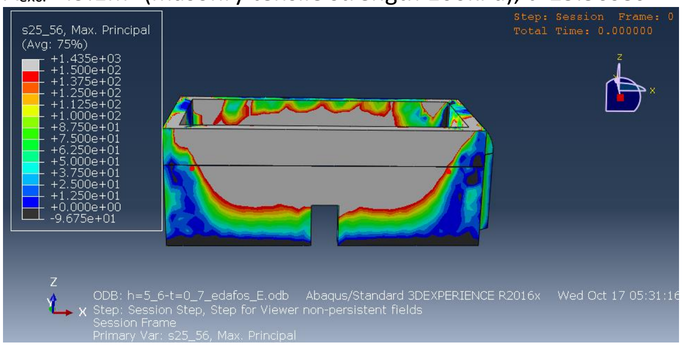
$A_{exc.}$=57.5m$^2$ (masonry tensile strength 100kPa), t=25.3sec




$A_{exc.}$=49.1m$^2$ (masonry tensile strength 100kPa), t=25.56sec

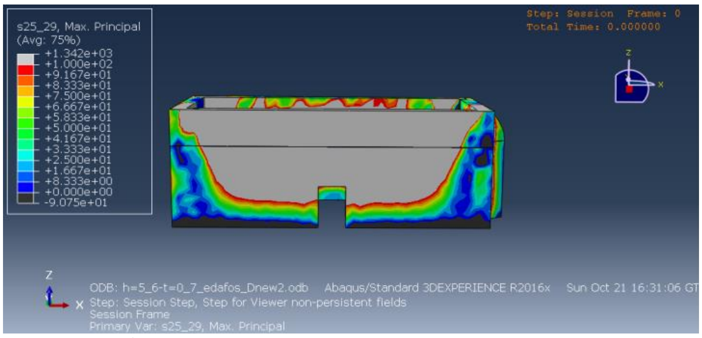



$A_{exc.}$=49.5m$^2$ (masonry tensile strength 150kPa), t=25.3sec

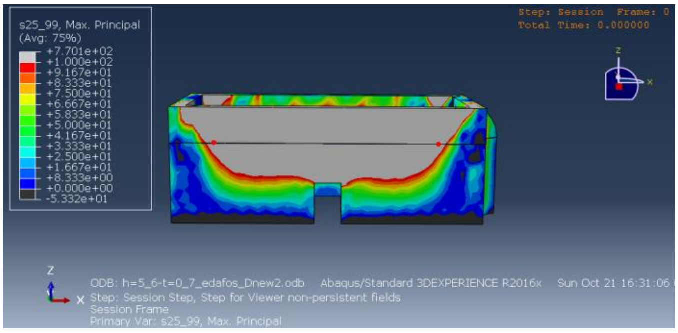



$A_{exc.}$=40.7m$^2$ (masonry tensile strength 150kPa), t=25.56sec

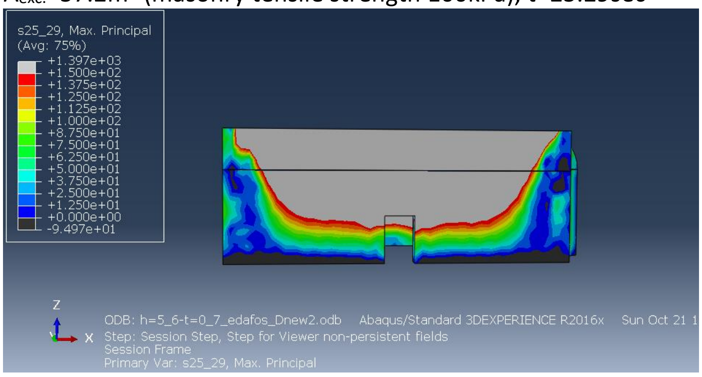
**Category D**: ground type D



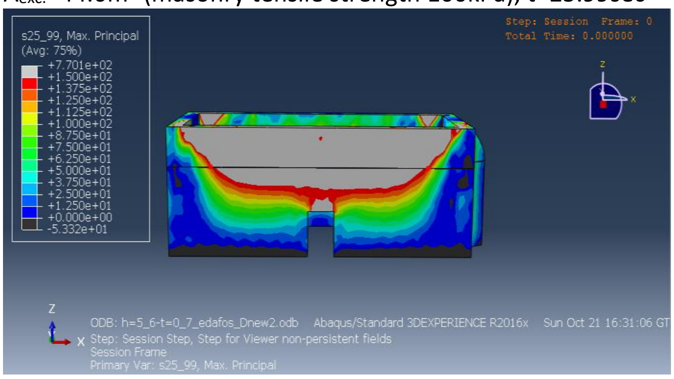

$A_{exc.}$=57.2m$^2$ (masonry tensile strength 100kPa), t=25.29sec




$A_{exc.}$=44.6m$^2$ (masonry tensile strength 100kPa), t=25.99sec

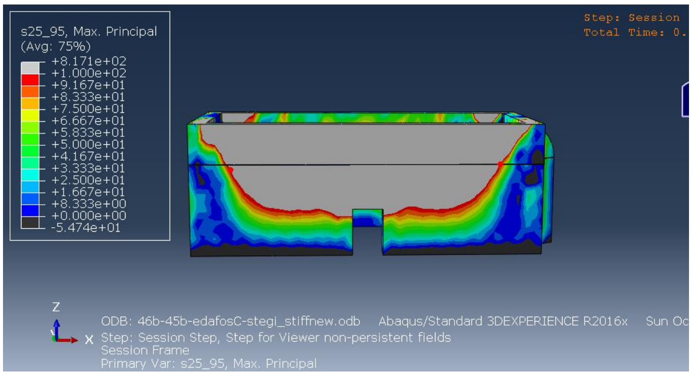



$A_{exc.}$=47.9m$^2$ (masonry tensile strength 150kPa), t=25.29sec

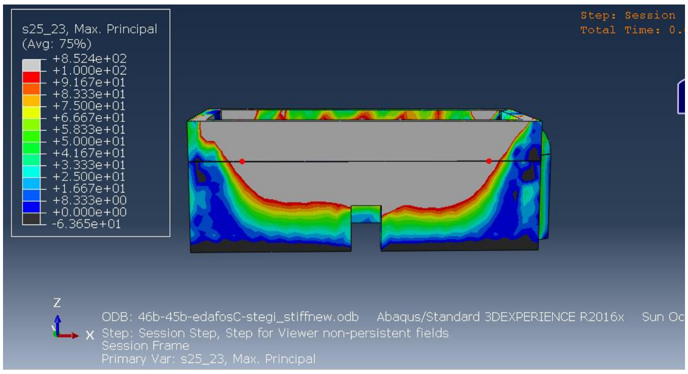



$A_{exc.}$=38m$^2$ (masonry tensile strength 150kPa), t=25.99sec

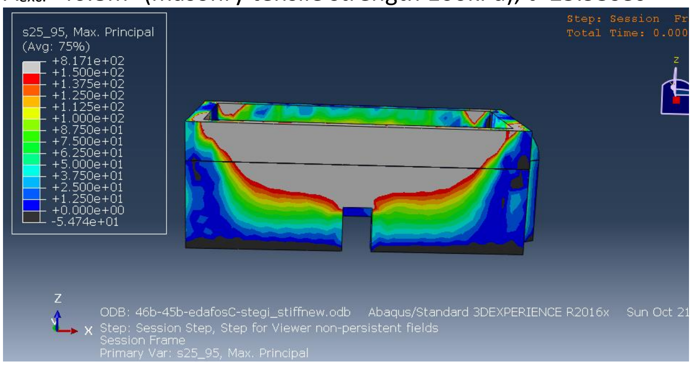
**Category C**: ground type C



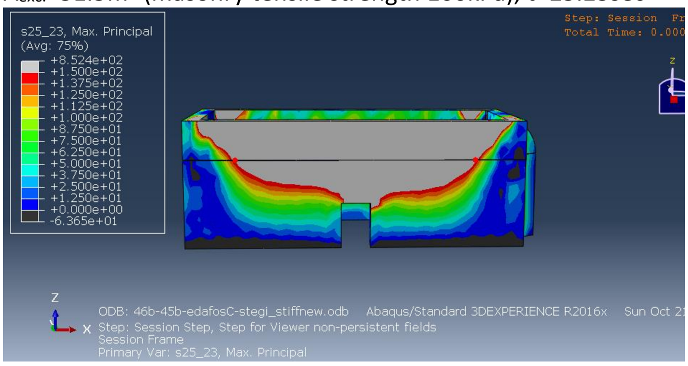

A$_{exc.}$=46.8m$^2$ (masonry tensile strength 100kPa), t=25.95sec




A$_{exc.}$=51.5m$^2$ (masonry tensile strength 100kPa), t=25.23sec

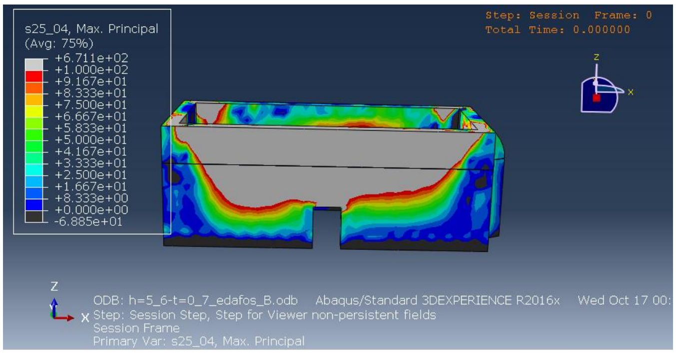



A$_{exc.}$=40.3m$^2$ (masonry tensile strength. 150kPa), t=25.95sec

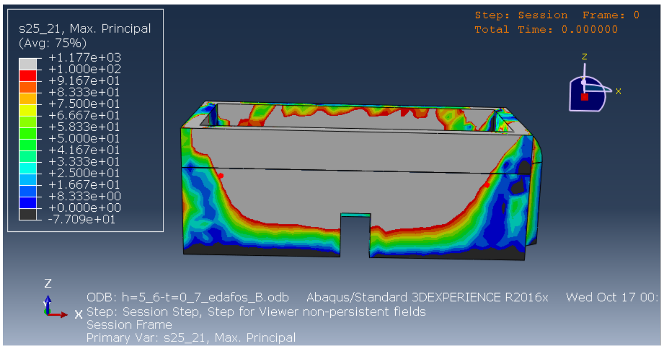



A$_{exc.}$=42.9m$^2$ (masonry tensile strength 150kPa), t=25.23sec

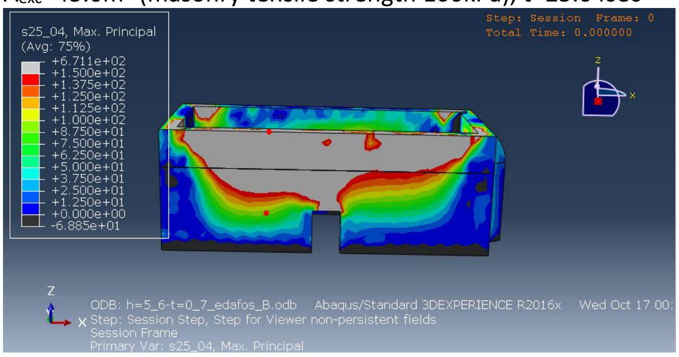
**Category B**: ground type B



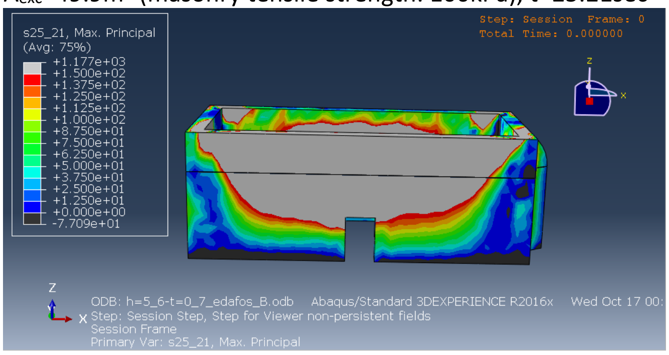

A$_{exc}$=45.0m$^2$ (masonry tensile strength 100kPa), t=25.04sec




A$_{exc}$=49.9m$^2$ (masonry tensile strength. 100kPa), t=25.21sec

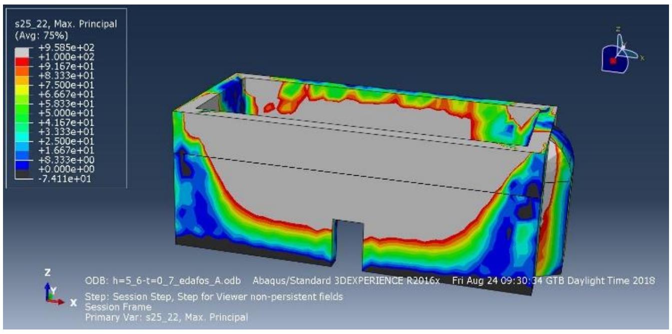



A$_{exc}$=33.7m$^2$ (masonry tensile strength 150kPa), t=25.04sec

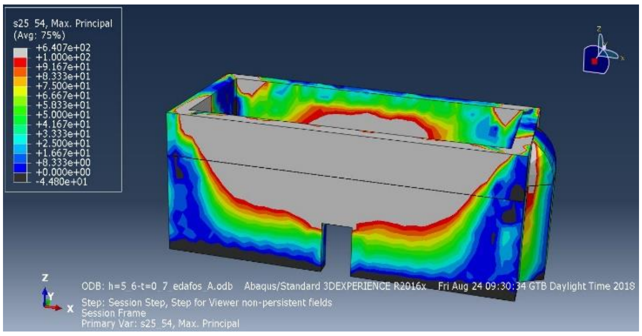



A$_{exc}$ =42.9m$^2$ (masonry tensile strength 150kPa), t=25.21sec

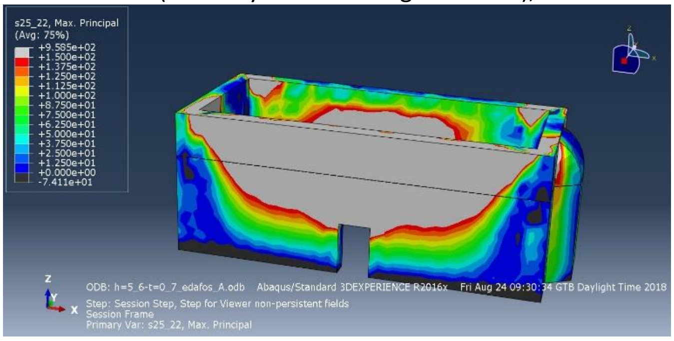
**Category A**: ground type A



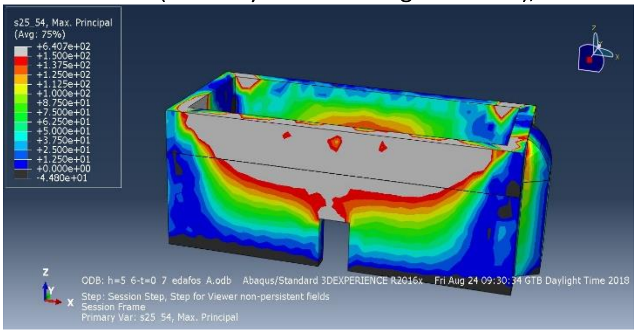
$A_{exc}$ =43.80m$^2$ (masonry tensile strength 100kPa), t=25.22sec



$A_{exc}$ =32.41m$^2$ (masonry tensile strength 100kPa), t=25.54sec



$A_{exc}$ =32.3m$^2$ (masonry tensile strength 150kPa), t=25.22sec



$A_{exc}$ =25.62m$^2$ (masonry tensile strength 150kPa), t=25.54sec

The rating of each category results from the "exceedance area" ratio, namely the ratio of the largest "exceedance area" as computed from all the categories of the parameter examined to the largest "exceedance area" of the category examined. Therefore, after setting rate 3.0 to the most favorable Category A, the rates for the other Categories are written as

**Category E: rate** $\frac{49.5}{49.5}\cdot\frac{3.0}{1.5}=2.00$

**Category D: rate** $\frac{49.5}{47.9}\cdot\frac{3.0}{1.5}=2.08$

**Category C: rate** $\frac{49.5}{42.9}\cdot\frac{3.0}{1.5}=2.30$

**Category B: rate** $\frac{49.5}{42.9}\cdot\frac{3.0}{1.5}=2.30$

**Category A: rate** $\frac{49.5}{32.6}=1.5 \Rightarrow \frac{49.5}{32.6}\cdot\frac{3.0}{1.5}=3.00$

Due to the similar rate, it is verified that Categories B and C can be grouped in one Category. It is noted here that in the Pre-Earthquake assessment data sheet, the following rates had been assigned to the ground types which prove to be similar after the required normalization with respect to the maximum rate 3.0 of the most favorable category.

| Type of ground (according to the Data sheet) | | | | |
|---|---|---|---|---|
| A | B,C | D | E | S1, S2* |
| 0.85 | 1.00 | 1.15 | 1.25 | - |

| Type of ground (according to the Data sheet after reduction) | | | | |
|---|---|---|---|---|
| A | B,C | D | E | S1, S2* |
| 3.00 | 2.55 | 2.22 | 2.04 | - |

## 2.6 Parameter 40 (Connection of Horizontal or Inclined Bearing Elements with Vertical Walls)

According to the Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, there are three categories for parameter 40.

**Category C**: Bearing elements simply supported at the edges of long walls

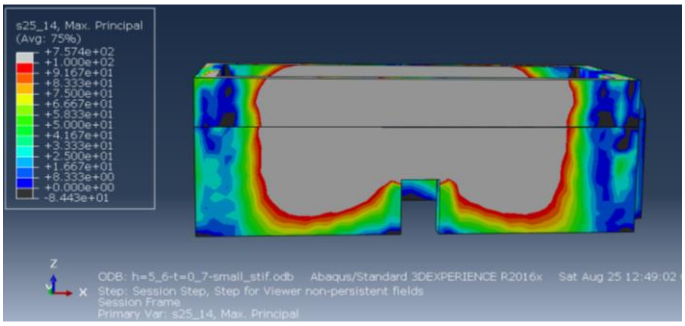
**Category B**: Well-connected only to the long walls, anchored on a proper chainage

**Category A**: Well connected on the whole perimeter of walls, anchored on a well-constructed chainage.

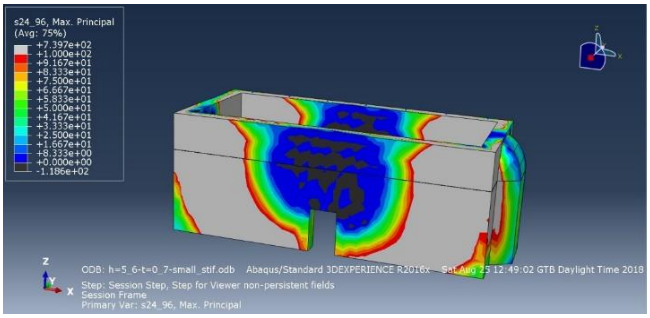
In order to examine the aforementioned categories, a typical basilica structure was simulated with length of longest and smallest walls equal to $L = 15.0m\ and\ b = 7.0m$ , respectively, wall thickness $0.70m$ and height $h = 5.6m$ . In the following figures, the results of the maximum principal tensile stresses during the time-history of the examined scenario are presented.

The simulation of the connection between the horizontal bearing elements and the vertical walls was realized with springs, whose stiffness was parametrized with respect to the category examined. Specifically, in case of poor connection along a direction, a low value of stiffness was assigned to the springs along that direction, allowing a relative motion between the horizontal bearing elements and the masonry walls. More specifically, in Category B, the motion of the bearing elements normal to the largest walls is not restrained while the relative displacements between the bearing elements and the masonry along the direction which is parallel to the largest walls are null. Finally, in Category C, the bearing elements are based on the long walls but can move relatively to it (simply supported)

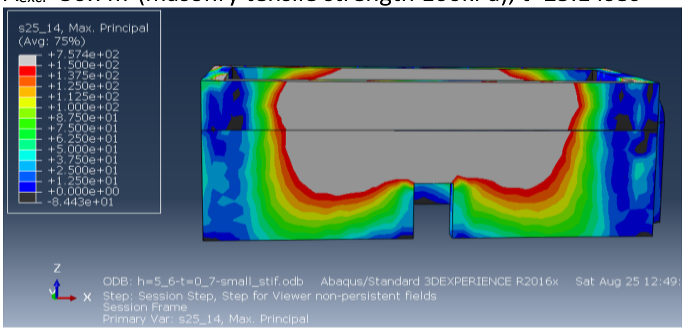
**Category C**: Bearing elements simply supported at the edges of long walls



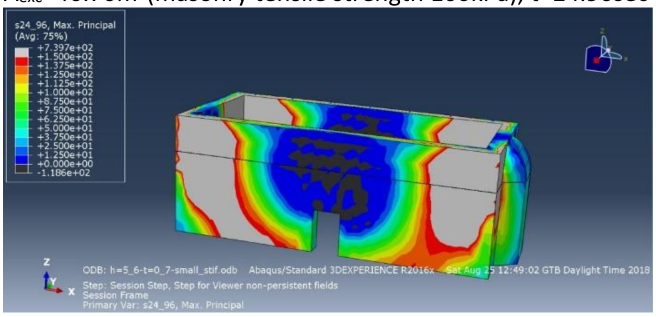

$A_{exc.}$=50.7m$^2$(masonry tensile strength 100kPa), t=25.14sec




$A_{exc}$=46.76m$^2$(masonry tensile strength 100kPa), t=24.96sec

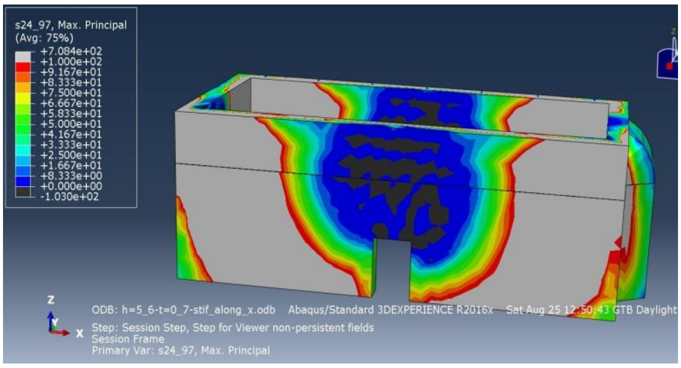



$A_{exc.}$=35.10m$^2$ (masonry tensile strength 150kPa), t=25.14sec

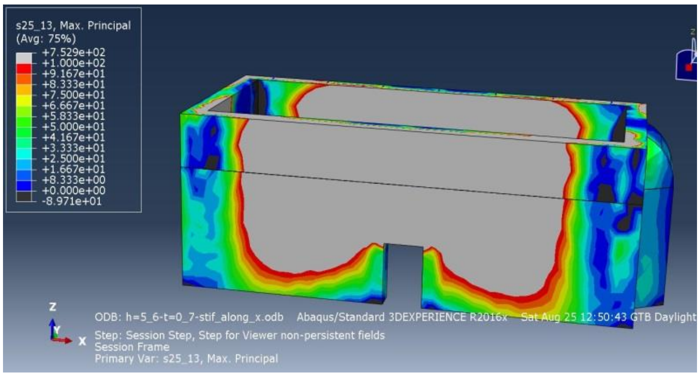



$A_{exc}$=27.58m$^2$ (masonry tensile strength 150kPa), t=24.96sec

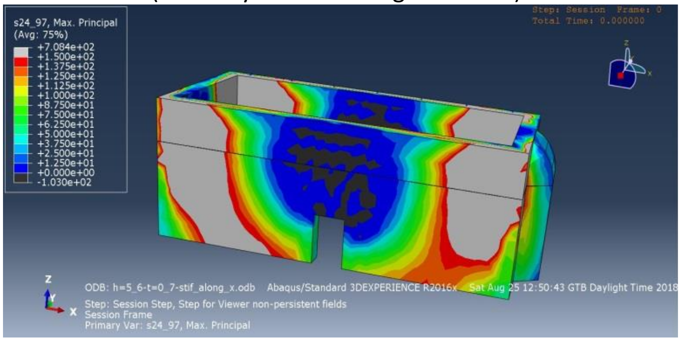
**Category B**: Well-connected only along the largest walls, anchored on a proper chainage (the horizontal elements are not constrained normal to the longest length walls)



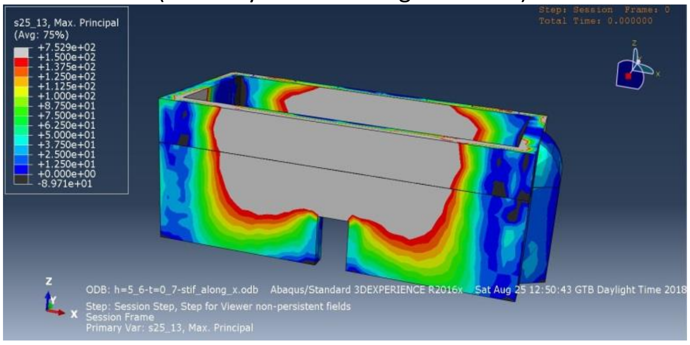

$A_{exc.}$=48.49m$^2$(masonry tensile strength 100kPa) t=24.97sec




$A_{exc.}$=46.96 m$^2$(masonry tensile strength 100kPa) t=25.13sec

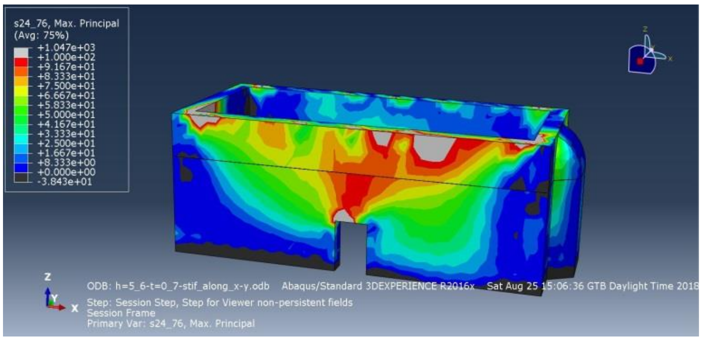



$A_{exc.}$=27.58m$^2$(masonry tensile strength 100kPa) t=24.97sec

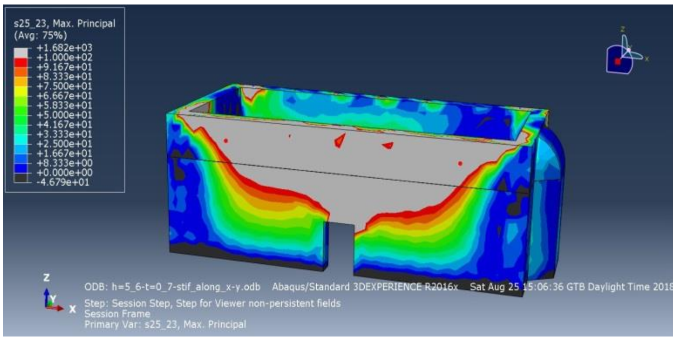



$A_{exc.}$=34.04 m$^2$(masonry tensile strength 150kPa) t=25.13sec

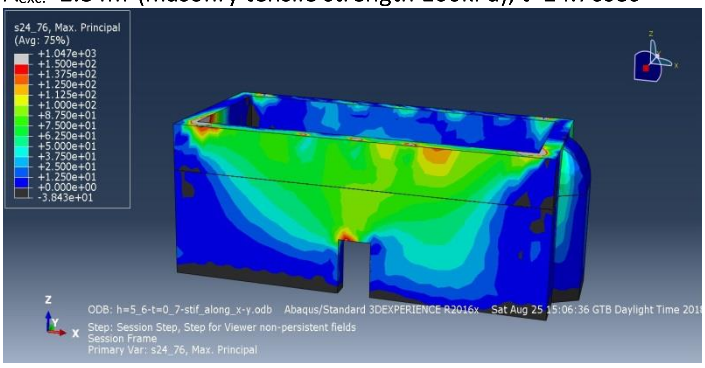
**Category A**: Well connected on the whole perimeter of walls, anchored on a well-constructed chainage.



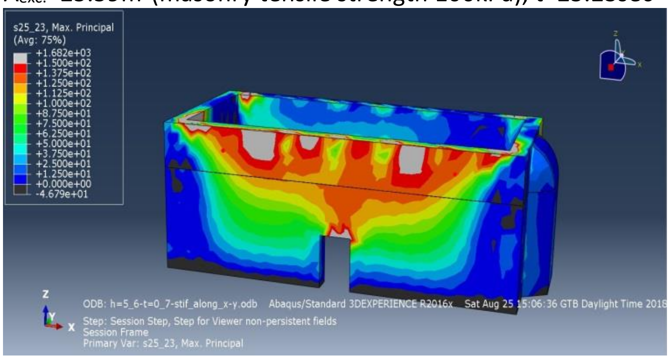

$A_{exc.}$=2.84m$^2$(masonry tensile strength 100kPa), t=24.76sec




$A_{exc.}$=25.59m$^2$(masonry tensile strength 100kPa), t=25.23sec




$A_{exc.}$=0 m$^2$(masonry tensile strength 150kPa), t=24.76sec




$A_{exc.}$=3.57 m$^2$(masonry tensile strength 150kPa), t=25.23sec

The rating of each category results from the "exceedance area" ratio, namely the ratio of the largest "exceedance area" as computed from all the categories of the parameter examined to the largest "exceedance area" of the category examined. Therefore, after setting rate 3.0 to the most favorable Category A and considering that the maximum exceedance area (assuming tensile strength of masonry equal to 150kPa) has been found equal to 35.10m$^2$ for Category C, the rates for the other Categories are written as

**Category C: rate** $\frac{35.10}{35.10}\cdot\frac{3.0}{9.83}=0.31$

**Category B: rate** $\frac{35.10}{34.04}\cdot\frac{3.0}{9.83}=0.32$

**Category A: rate** $\frac{35.10}{3.57}\cdot 1=9.83\Rightarrow\frac{35.10}{3.57}\cdot\frac{3.0}{9.83}=3$

Based on the presented results, the poor connectivity of the horizontal elements normal to the largest walls has a significant effect to the behavior of the structure. On the other side, the poor connectivity along the direction of the largest walls slightly increased the exceedance area. Therefore, the difference between Categories B and C is ignorable, while the rate of Category A shows the improvement of structural seismic behavior. Specifically, apart from the out of plane bending of the large walls, the stress condition in the corners of Structures B and C reflects the vulnerability of the largest wall in the absence of transversal connection elements.

Finally, it should be mentioned that for the determination of the rates of each category, the results for masonry tensile strength equal to 150kPa (not 100kPa as used in the other parameters), have been used since the arising tensile stresses in Categories B and C are much higher than 100kPa contrary to Category A which are close to the aforementioned limit.

# 3 VULNERABILITY INDEX

## 3.1 Determination of Participation Factor of Each Parameter

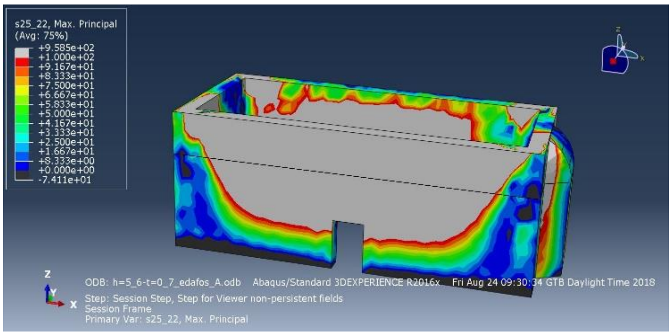
After the in-situ completion of the pre-earthquake assessment data sheets, the responsible Service goes on to the rating procedure in order to estimate the vulnerability index of the monument visited. Due to the determination of the effect using numerical methods of some of the parameters included in Pre-Earthquake assessment data sheet for single spaced one storey historical buildings, in the following section, the final vulnerability index is computed according only to the parameters analyzed in this report.

For the determination of the participation factor of each parameter in the computation of the final vulnerability index of the structure, a basic numerical model (**reference structure**) employing three-dimensional finite elements was constructed. The geometrical characteristics of the reference structure correspond to the most common dimensions of a Basilica structure (as it was concluded from the examination of the Basilica churches existing in the province of Palli [2,3]) namely length of largest and smallest masonry wall equal to $L_{\max}=15.0m$ and $b=7.0m$ respectively, masonry thickness $0.70m$, and height $h=5.6m$. Subsequently, each parameter of the reference structure was changed, one at a time and to a less favorable category, and the corresponding change in the exceedance was calculated. This change represents the effect of the specific parameter in the computation of the vulnerability of the structure.

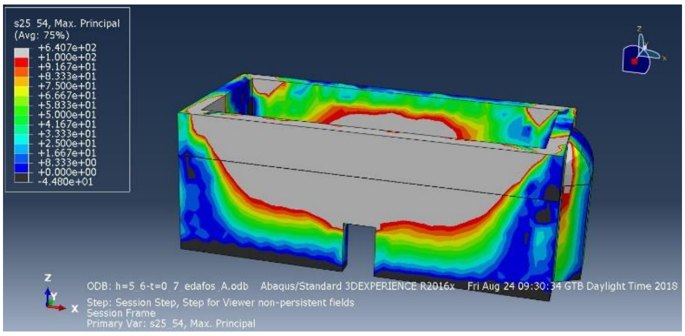
**Table 2.** Parameters' details of the reference structure and their changes

| | **Length of largest wall (Parameter 45)** | **Height of structure (Parameter 46)** | **Ground type (Parameter 52)** | **Connection of horizontal or inclined bearing elements with vertical walls (Parameter 40)** | **Openings (Parameter 49)** |
|---|---|---|---|---|---|
| Reference structure | $L = 15m$ $(L = 21.5t\ Category\ C)$ | $h = 5.6m$ $(h = 8t\ Category\ \mathrm{B})$ | Ground Type A | Well connected in both directions | None |
| Change 1 (change of parameter 45) | $L = 20m$ $(L = 28.6t\ Category\ D)$ | $h = 5.6m$ $(h = 8t\ Category\ \mathrm{B})$ | Ground Type A | Well connected in both directions | None |
| Change 2 (change of parameter 46) | $L = 15m$ $(L = 21.5t\ Category\ C)$ | $h = 7.5m$ $(h = 10.7t\ Category\ C)$ | Ground Type A | Well connected in both directions | None |
| Change 3 (change of parameter 52) | $L = 15m$ $(L = 21.5t\ Category\ C)$ | $h = 5.6m$ $(h = 8t\ Category\ \mathrm{B})$ | Ground Type B,C | Well connected in both directions | None |
| Change 4 (change of parameter 40) | $L = 15.0m$ $(L = 10t\ Category\ C)$ | $h = 5.6m$ $(h = 8t\ Category\ \mathrm{B})$ | Ground Type A | Well-connected only to the long walls anchored on a proper chainage | None |
| Change 5 (change of parameter 49) | $L = 15m$ $(L = 21.5t\ Category\ C)$ | $h = 5.6m$ $(h = 8t\ Category\ \mathrm{B})$ | Ground Type A | Well connected in both directions | Existing at a distance smaller than 1.2m |

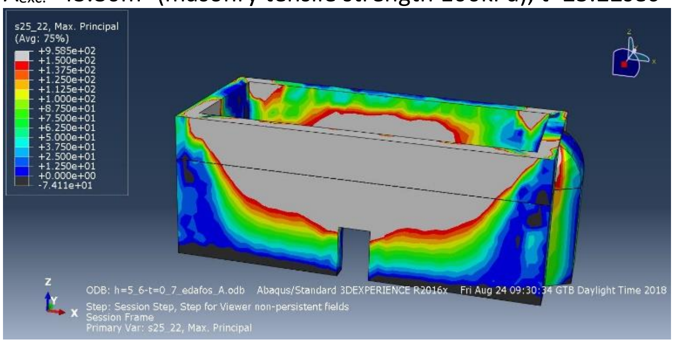
- **Reference Structure**



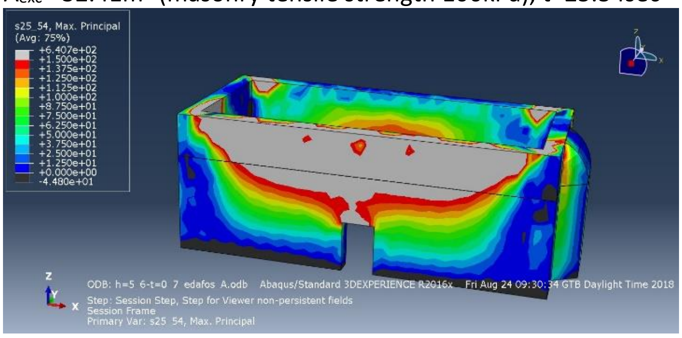
$A_{exc.}$=43.80m$^2$ (masonry tensile strength 100kPa), t=25.22sec



$A_{exc}$ =32.41m$^2$ (masonry tensile strength 100kPa), t=25.54sec

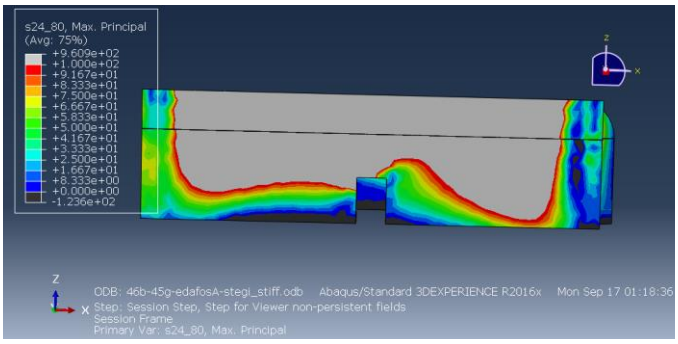


$A_{exc}$ =32.34m$^2$ (masonry tensile strength. 150kPa), t=25.22sec

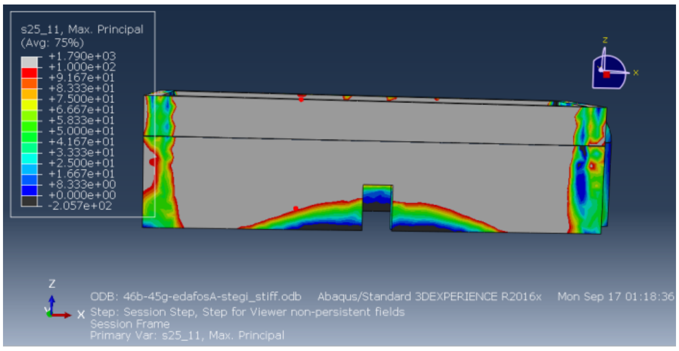


$A_{exc}$ 25.62m$^2$ (masonry tensile strength 150kPa), t=25.54sec

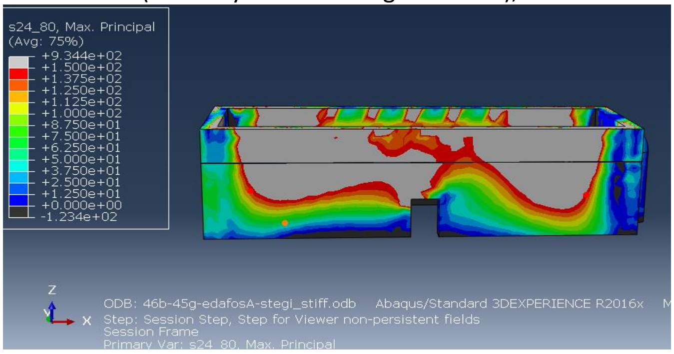
- **Change 1 (change of parameter 45** $L = 15.0m \Rightarrow L = 20m$ namely change of Category C => D**)**



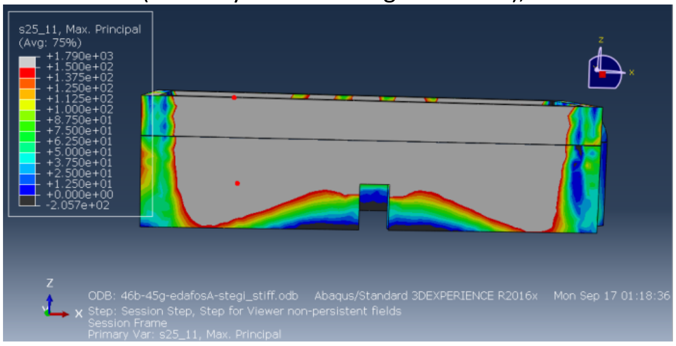

A $_{exc.}$=75.2m$^2$(masonry tensile strength 100kPa), t=24.80sec




A $_{exc.}$=86.5m$^2$(masonry tensile strength. 100kPa), t=25.11sec

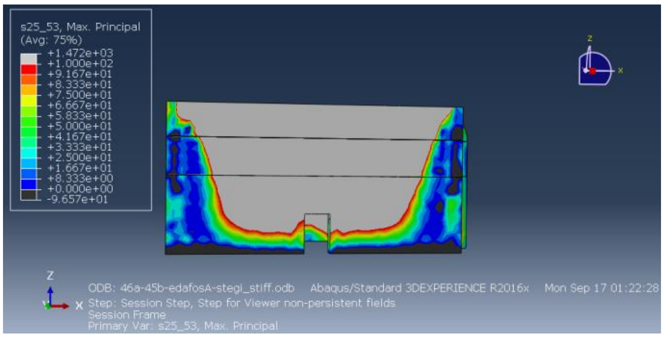



A $_{exc.}$=47.5m$^2$(masonry tensile strength 150kPa), t=24.80sec

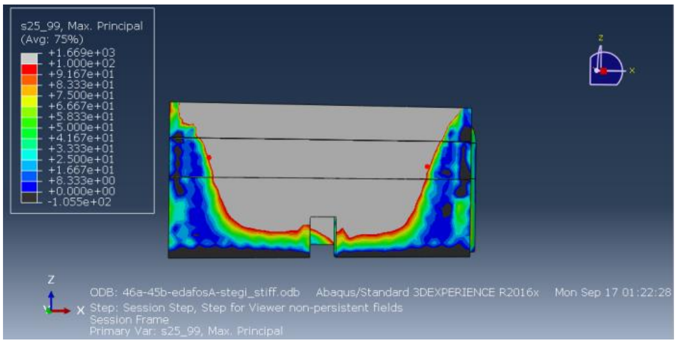



A $_{exc.}$=78.5m$^2$(masonry tensile strength 150kPa), t=25.11sec

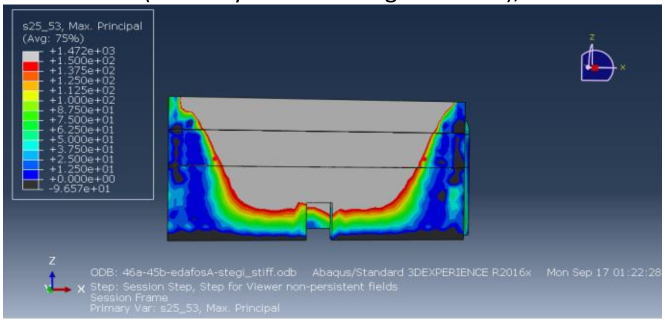
- **Change 2 (change of parameter 46** $h = 5.6m \Rightarrow h = 7.5m$ namely Category B => C**)**



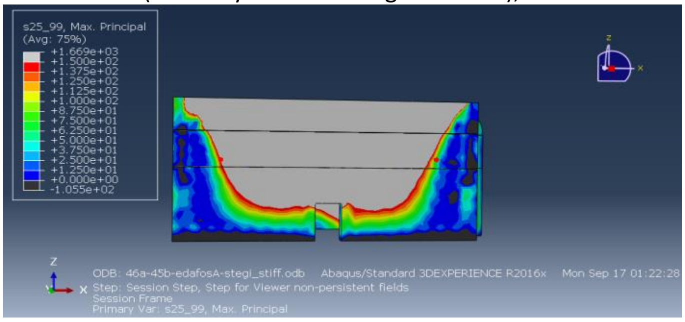

A $_{exc}$ =66.8m$^2$(masonry tensile strength 100kPa), t=25.53sec




A $_{exc}$ =63.1m$^2$(masonry tensile strength 100kPa), t=25.99sec

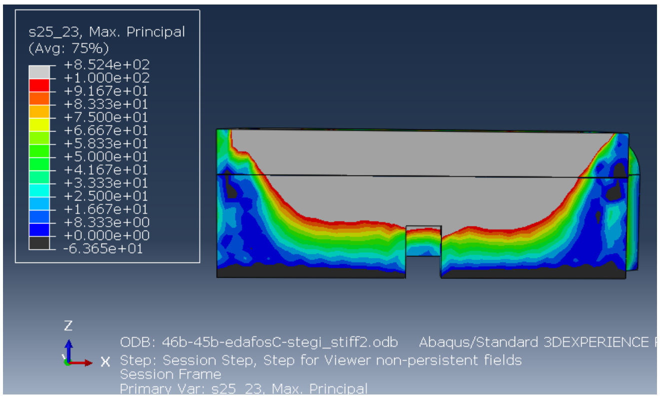



A $_{exc}$ =64.1m$^2$(masonry tensile strength 150kPa), t=25.53sec

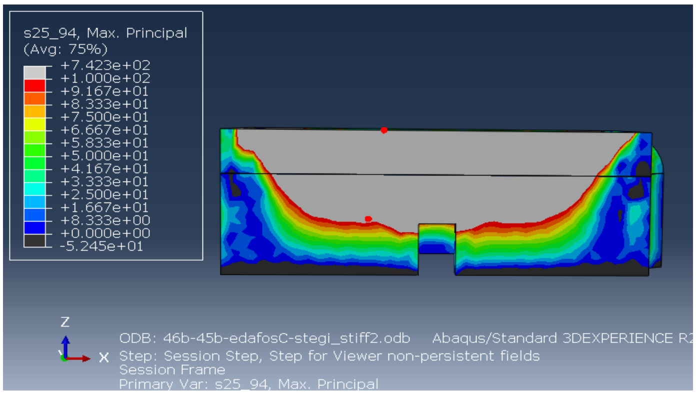



A $_{exc}$ =63.7m$^2$(masonry tensile strength 150kPa), t=25.99sec

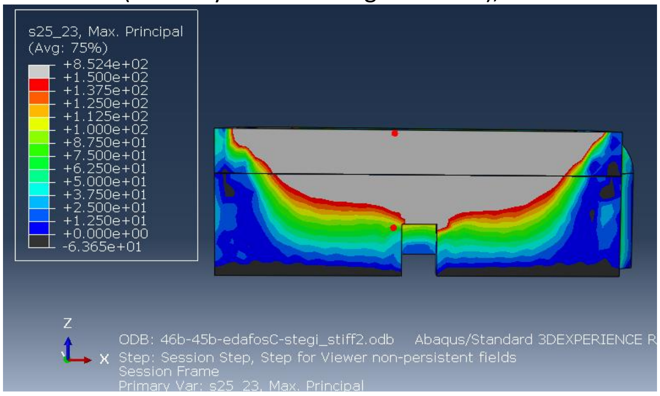
- **Change 3 (change of parameter 52** from ground type A to ground type B, C**)**



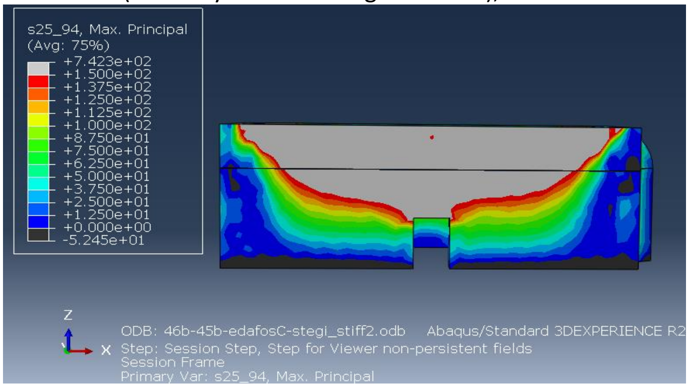

A exc =52m$^2$(masonry tensile strength 100kPa), t=25.23sec




A exc =49m$^2$(masonry tensile strength 100kPa), t=25.94sec

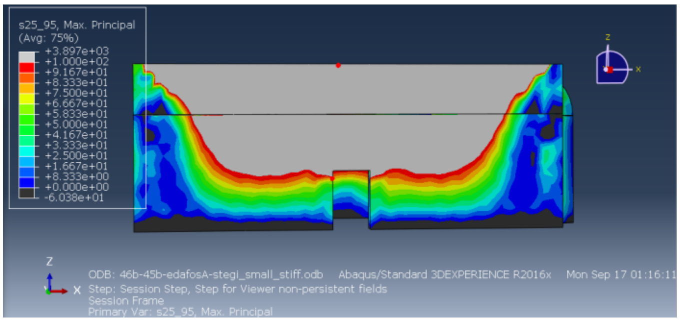



A exc =43m$^2$(masonry tensile strength 150kPa), t=25.23sec

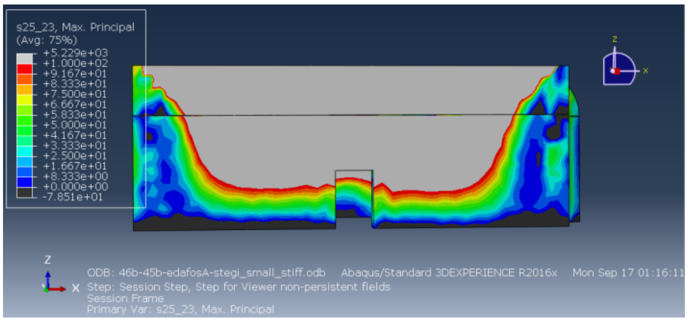



A exc=40m$^2$(masonry tensile strength 150kPa), t=25.94sec

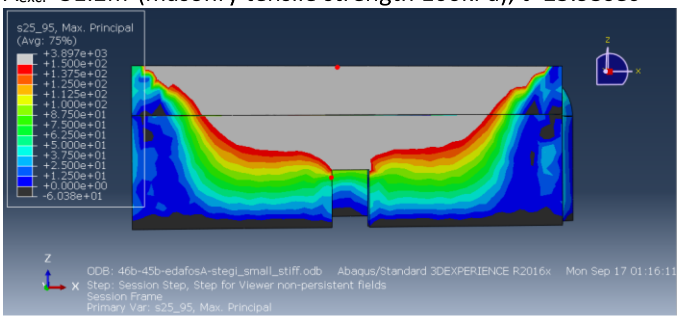
- **Change 4 (change of parameter 40 from Category A "Well connected in both directions" to Category B "Well-connected only to the long walls anchored on a proper chainage")**



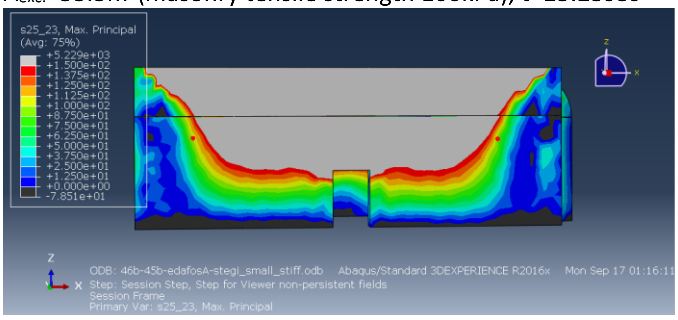

A$_{exc.}$=51.2m$^2$(masonry tensile strength 100kPa), t=25.95sec




A$_{exc.}$=53.3m$^2$(masonry tensile strength 100kPa), t=25.23sec

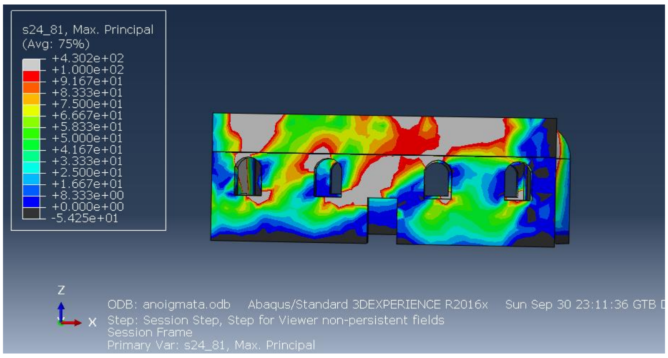



A$_{exc.}$=41m$^2$(masonry tensile strength 150kPa), t=25.95sec

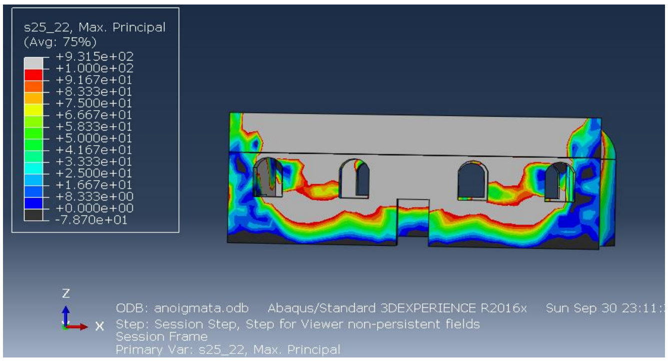



A$_{exc.}$=41.3m$^2$(masonry tensile strength 150kPa), t=25.23sec

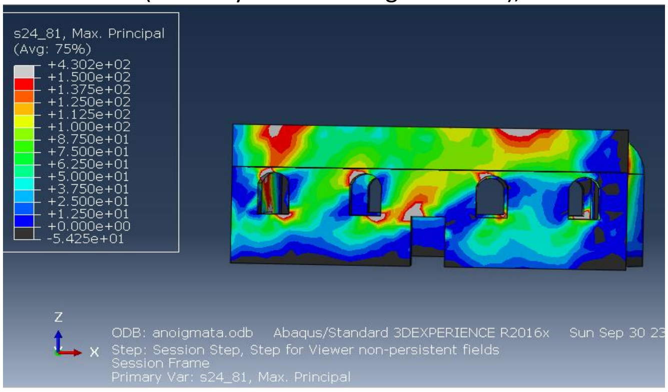
- **Change 5 (Distance of openings from the corners):** The participation factor computed from the following analysis represents the effect of the existence of openings in the perimetric walls (reference structure has no openings along largest walls) since as it was shown in previous paragraph that the effect of the distance of the closest opening to the corner can be ignored.



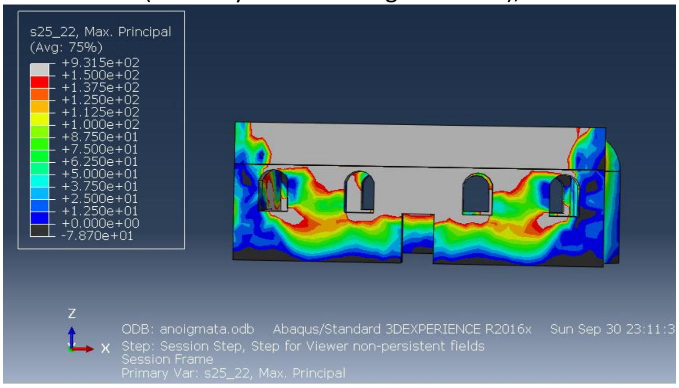

$A_{exc.}$=9.27m$^2$ (masonry tensile strength 100kPa), t=24.81sec




$A_{exc.}$=61.4m$^2$ (masonry tensile strength 100kPa), t=25.22sec




$A_{exc.}$= 0.0m$^2$ (masonry tensile strength 150kPa), t=24.81sec




$A_{exc.}$=38.5m$^2$ (masonry tensile strength 150kPa), t=25.22sec

The determination of the participation factors of each parameter will result from the change of exceedance area of the reference structure for the case of 100kPa masonry tensile strength which was computed equal to $A_{υπέρβ}$=43.80m$^2$.

**Parameter 45 (Wall Thickness to Longer Length ratio):** $Change\,rate: \dfrac{86.5}{43.8} \approx 2.0\ (200\%)$

**Parameter 46 (Wall Thickness to Wall Height ratio):** $Change\,rate: \dfrac{66.8}{43.8} = 1.53 \approx 1.5\ (150\%)$

**Parameter 52 (Ground type):** $Change\,rate: \dfrac{52.0}{43.8} =\approx 1.2\ (120\%)$

**Parameter 40 (Connections of horizontal or vertical elements with masonry**):

$$Change\,rate: \frac{53.3}{43.8} = 1.22 \approx 1.2\ (120\%)$$

**Parameter 49 (Openings near the corners - distance from the corner):** $Change\,rate: \dfrac{61.4}{43.8} \approx 1.4\ (140\%)$

Based on the aforementioned analysis, the participation factors with respect to unity are written as

**Participation factor of parameter 45** $a_{45} = \dfrac{2.0}{2.0} = 1.0$

**Participation factor of parameter 46** $a_{46} = \frac{1.5}{2.0} = 0.75$

**Participation factor of parameter 52** $a_{52} = \frac{1.2}{2.0} = 0.60$

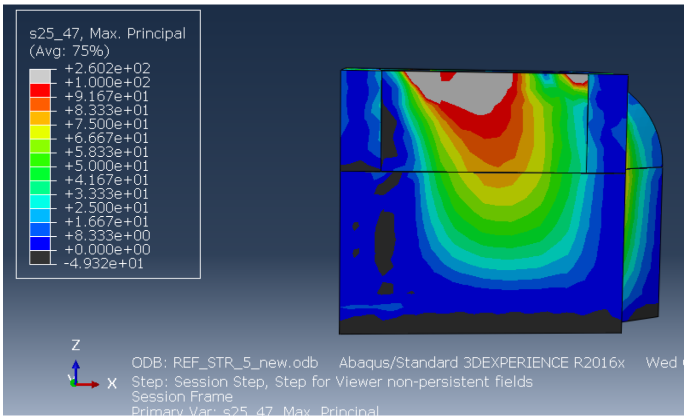
**Participation factor of parameter 40** $a_{40} = \frac{1.2}{2.0} = 0.60$

**Participation factor of parameter** $a_{49} = \frac{1.4}{2.0} = 0.70$

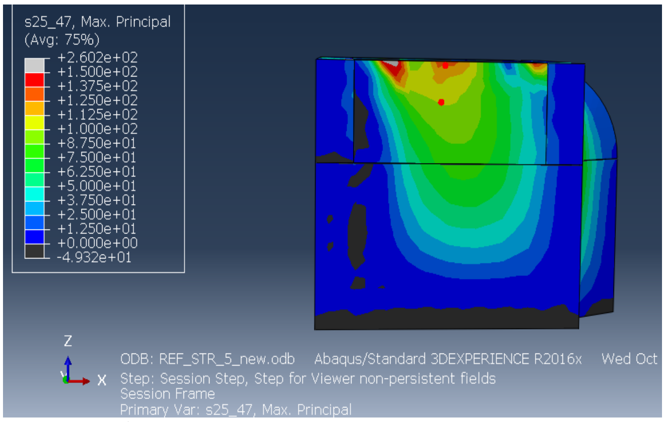
In order to verify the validity of the aforementioned participation factors, a new geometry of basilica structure is studied, whose dimensions are much smaller and comply with the limits of the category A of all aforementioned parameters.

| | **Length of largest wall (Parameter 45)** | **Height of structure (Parameter 46)** | **Ground type (Parameter 52)** | **Connection of horizontal or inclined bearing elements with vertical walls (Parameter 40)** | **Openings (Parameter 49)** |
|---|---|---|---|---|---|
| Reference structure | $L = 5.0m$ $(L < 8t\ Category\ \mathrm{A})$ | $h = 4.5m$ $(h < 7t\ Category\ \mathrm{A})$ | Ground Type A | Well connected in both directions | None |
| Change 1 (change of parameter 45) | $L = 7.5m$ $(8t < L < 12t\ Category\ \mathrm{B})$ | $h = 4.5m$ $(h < 7t\ Category\ \mathrm{A})$ | Ground Type A | Well connected in both directions | None |
| Change 2 (change of parameter 46) | $L = 5.0m$ $(L < 8t\ Category\ \mathrm{A})$ | $h = 5.6m$ $(7t < h < 9t\ Category\ \mathrm{B})$ | Ground Type A | Well connected in both directions | None |
| Change 3 (change of parameter 52) | $L = 5.0m$ $(L < 8t\ Category\ \mathrm{A})$ | $h = 4.5m$ $(h < 7t\ Category\ \mathrm{A})$ | Ground Type B,C | Well connected in both directions | None |
| Change 4 (change of parameter 40) | $L = 5.0m$ $(L < 8t\ Category\ \mathrm{A})$ | $h = 4.5m$ $(h = 8t\ Category\ A)$ | Ground Type A | Well-connected only to the long walls anchored on a proper chainage | None |
| Change 5 (change of parameter 49) | $L = 5.0m$ $(L < 8t\ Category\ \mathrm{A})$ | $h = 4.5m$ $(h < 7t\ Category\ \mathrm{A})$ | Ground Type A | Well connected in both directions | Existing at a distance between 1.2m and 2.0m |

- **Reference Structure**

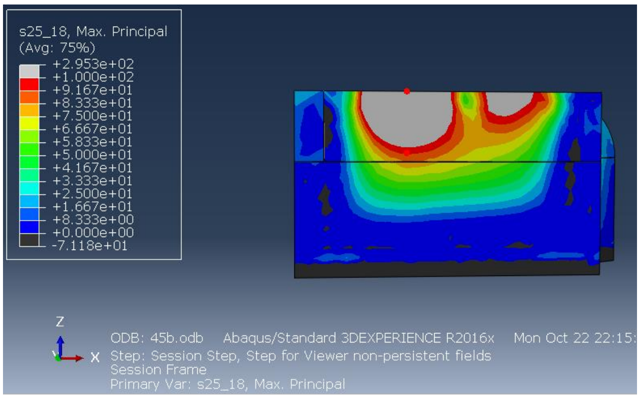



$A_{exc}$=1.1m$^2$ (masonry tensile strength 100kPa), t=25.47sec

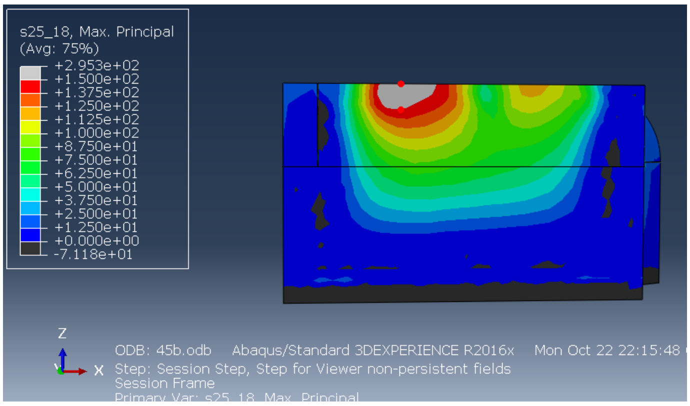



$A_{exc}$=0.01m$^2$ (masonry tensile strength 150kPa), t=25.47sec

- **Change 1 (change of parameter 45** $L = 5.0m \Rightarrow L = 7.5m$ **namely change of Category A => B)**

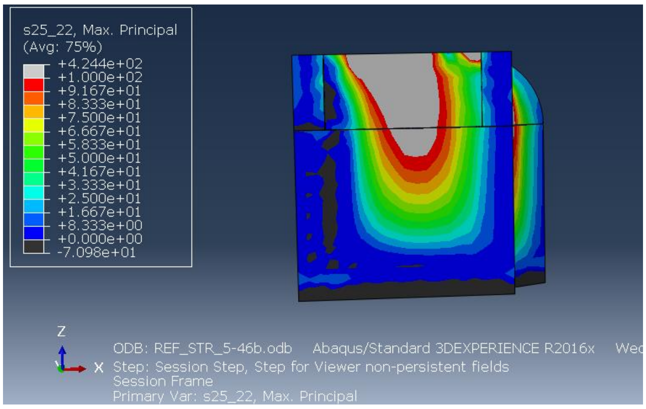



$A_{exc}$=3.3m$^2$ (masonry tensile strength 100kPa), t=25.18sec

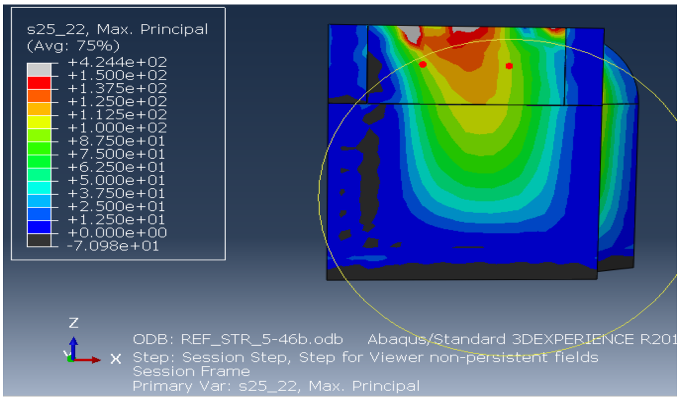



$A_{exc}$=0.60m$^2$ (masonry tensile strength 150kPa), t=25.47sec

- **Change 2 (change of parameter 46** $h = 4.5m \Rightarrow h = 5.6m$ **namely Category A => B)**

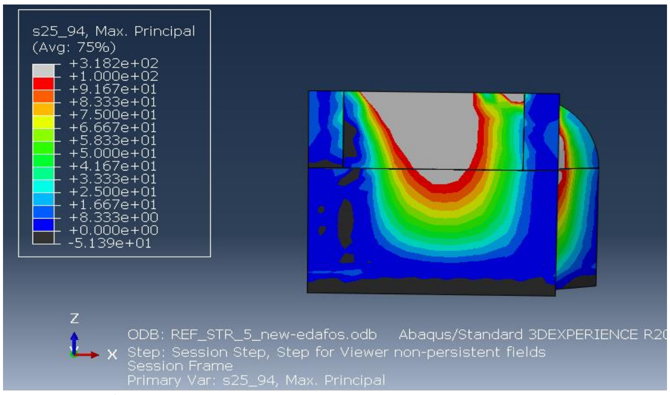



$A_{exc}$=3.1m$^2$ (masonry tensile strength 100kPa), t=25.22sec

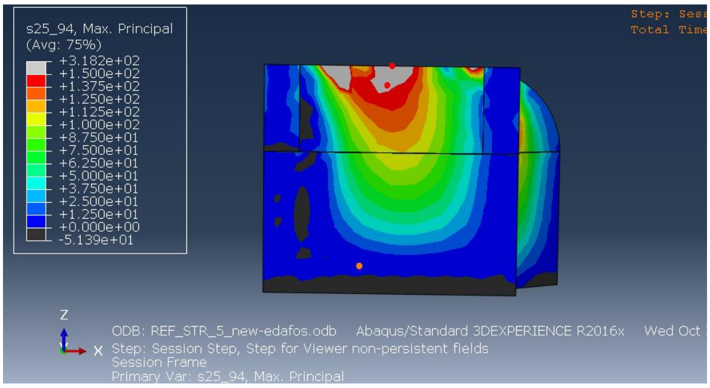



$A_{exc}$=0.30m$^2$ (masonry tensile strength 150kPa), t=25.22sec

- **Change 3 (change of parameter 52 from ground type A to ground type B, C)**

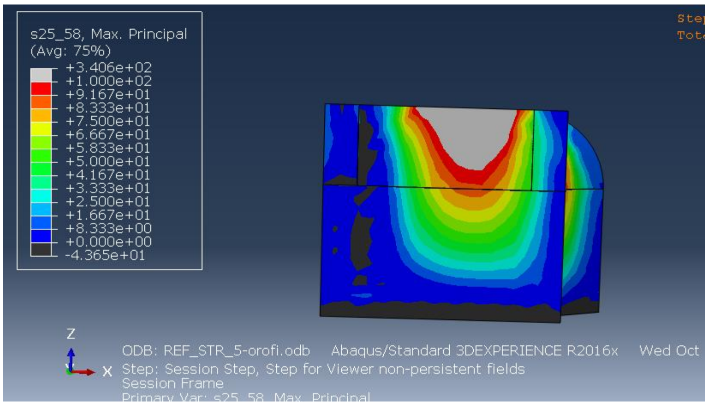



$A_{exc}$=3.6m$^2$(masonry tensile strength 100kPa), t=25.94sec

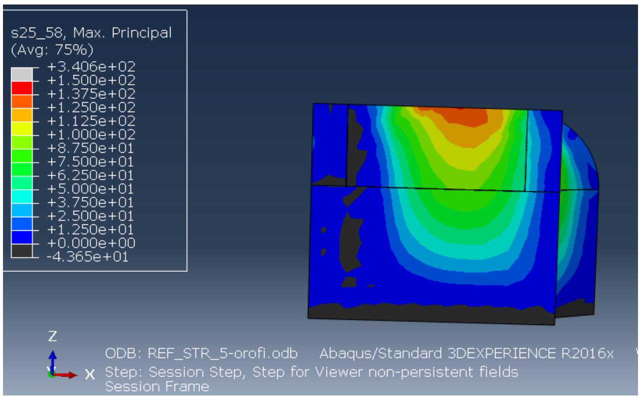



$A_{exc}$=0.7 m$^2$(masonry tensile strength 150kPa), t=25.94sec

- **Change 4 (change of parameter 40 from Category A «Well connected in both directions» to Category B « Well-connected only to the long walls anchored on a proper chainage »)**

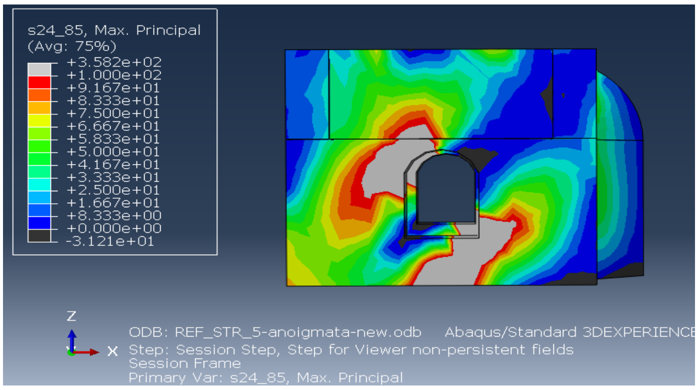



$A_{exc}$=1.9m$^2$ (masonry tensile strength 100kPa), t=25.58sec

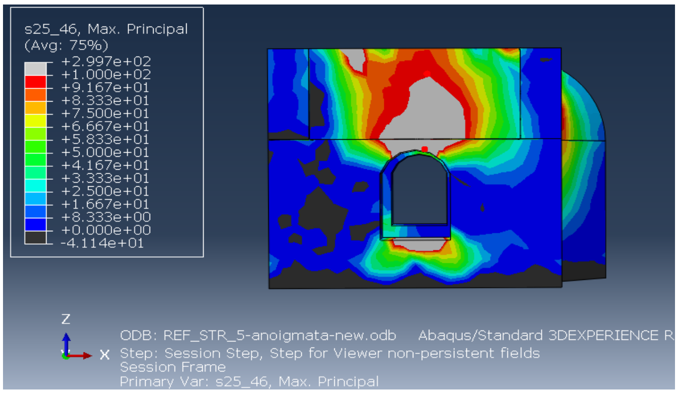



$A_{exc}$=0.01m$^2$ (tensile strength 150kPa), t=25.58sec

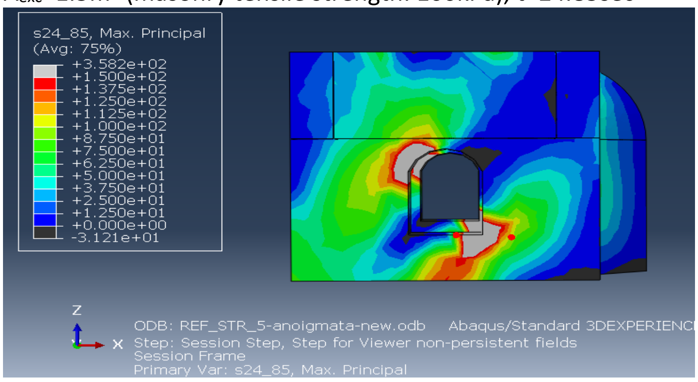
- **Change 5 (change of parameter 49 - Distance of openings from corners):**



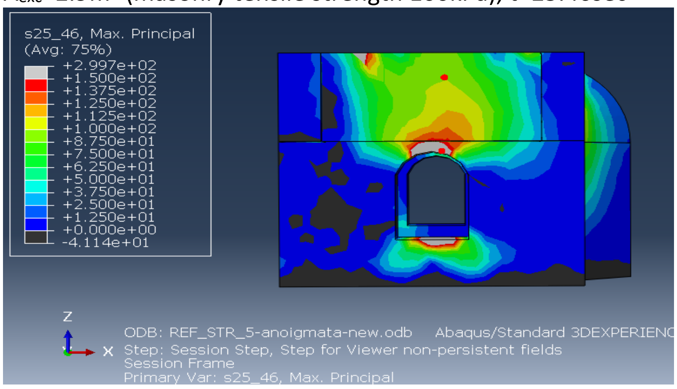

$A_{exc}$=1.5m$^2$ (masonry tensile strength. 100kPa), t=24.85sec




$A_{exc}$=2.9m$^2$ (masonry tensile strength 100kPa), t=25.46sec




$A_{exc}$= 0.7m$^2$ (masonry tensile strength 150kPa), t=24.81sec




$A_{exc}$=0.3m$^2$ (masonry tensile strength 150kPa), t=25.46sec

The determination of the participation factors of each parameter will result from the change of the principal tensile stresses of the reference structure and more specifically from the change of the exceedance area of the masonry tensile strength 100kPa (largest exceedance are of reference structure $A_{υπέρβ}$=1.1m²).

More specifically:

**Parameter 45 (Wall Thickness to Longer Length ratio):** Increase of the largest masonry wall length from $L = 5.0m \Rightarrow L = 7.5m$ namely from Category A => B

$$Change\,rate: \frac{3.3}{1.1} = 3.0\ (300\%)$$

**Parameter 46 (Wall Thickness to Wall Height ratio):** Increase of the largest masonry wall height from $h = 4.5m \Rightarrow h = 5.6m$ namely from Category A => B

$$Change\,rate: \frac{3.1}{1.1} = 2.8(280\%)$$

**Parameter 52:** Change of Ground Type from Category A to B, C

$$Change\,rate: \frac{3.6}{1.1} = 3.3\ (330\%)$$

**Parameter 40 (Connections of horizontal or vertical elements with masonry**): Well-connected only to the long walls anchored on a proper chainage

$$Change\,rate: \frac{1.9}{1.1} \approx 1.7(170\%)$$

**Parameter 49 (Openings near the corners - distance from the corner):**

$$Change\ rate: \frac{2.9}{1.1} = 2.6\ (260\%)$$

Based on the aforementioned analysis, the participation factors with respect to unity are written as

**Participation factor of parameter 45** $a_{45} = \frac{3.0}{3.3} = 0.9$

**Participation factor of parameter 46** $a_{46} = \frac{2.8}{3.3} = 0.85$

**Participation factor of parameter 52** $a_{52} = \frac{3.3}{3.3} = 1.0$

**Participation factor of parameter 40** $a_{40} = \frac{1.7}{3.3} = 0.52$

**Participation factor of parameter 49** $a_{49} = \frac{2.6}{3.3} = 0.8$

Concluding, the final participation factors will result as the average of the presented values of the two groups of analyses examined

**Participation factor of parameter 45** $a_{45} = \dfrac{1.0 + 0.9}{2.0} = 0.95$

**Participation factor of parameter 46** $a_{46} = \dfrac{0.75 + 0.85}{2.0} = 0.80$

**Participation factor of parameter 52** $a_{52} = \dfrac{0.60 + 1.0}{2.0} = 0.80$

**Participation factor of parameter 40** $a_{40} = \dfrac{0.6 + 0.52}{2.0} = 0.56$

**Participation factor of parameter 49** $a_{49} = \dfrac{0.7 + 0.8}{2.0} = 0.75$

For comparison reasons, the maximum and minimum rating which represents the most and less favorable case of basilica structure, respectively, as obtained from the previously presented parameters are computed as.

**Maximum rating (Category A of all parameters):**

$$\alpha_{45} \cdot 3.0 + \alpha_{46} \cdot 3.0 + \alpha_{52} \cdot 3.0 + \alpha_{40} \cdot 3.0 + \alpha_{49} \cdot 3.0 =$$
$$0.95 \cdot 3.0 + 0.80 \cdot 3.0 + 0.80 \cdot 3.0 + 0.56 \cdot 3.0 + 0.75 \cdot 3.0 = 11.6$$

**Minimum rating (Category C of all parameters):**

$$\alpha_{45} \cdot 0.05 + \alpha_{46} \cdot 0.60 + \alpha_{52} \cdot 2.0 + \alpha_{40} \cdot 0.31 + \alpha_{49} \cdot 2.9 =$$
$$0.95 \cdot 0.05 + 0.80 \cdot 0.60 + 0.80 \cdot 2.0 + 0.56 \cdot 0.31 + 0.75 \cdot 2.9 = 4.5$$

# 4 CASES STUDIED

In this section, the Pre-Earthquake Assessment Data Sheet for Single Spaced One Storey Historical Buildings is filled for four cases of monuments with respect to the parameters examined in the previous paragraphs.

| TABLE OF FINAL RESULTS OF APPLICATIONS | | | | |
|---|---|---|---|---|
| | CHURCH OF THE VIRGIN'S DORMITION<br>PANAGIA AGRILOU -<br>KONTOGENNADA CEFALONIA | CHURCH OF ST BASIL-<br>KONTOGENNADA CEFALONIA | CHURCH OF ST. DEMITRIOS –<br>MAROYSI ATTICA | CHURCH OF ST. TAXIARCHES –<br>SARONIKO ATTICA |
| Parameter 45 | **Category C** | **Category C** | **Category B** | **Category C** |
| Parameter 46 | **Category C** | **Category A** | **Category B** | **Category B** |
| Parameter 52 | **Category A** | **Category B,C** | **Category E** | **Category A** |
| Parameter 40 | **Category B** | **Category C** | **Category B** | **Category B** |
| Parameter 49 | **Category A** | **Category C** | **Category B** | **Category A** |
| **Rating based on the previous parameters** | | | | |
| | **5.59** | **6.87** | **5.75** | **6.07** |
| **Rating according to all the parameters of the Pre-Earthquake Assessment Data Sheet** | | | | |
| **Vulnerability Index of the Method of Construction** | 53.13 | 44.74 | 42.50 | 56.25 |
| **Vulnerability Index of the Seismic Action** | 27.00 | 25.50 | 45.90 | 75.00 |
| **Vulnerability Index of the Pathology** | 25.00 | 25.00 | 25.00 | 75.00 |
| **Final Vulnerability Index** | **105.13** | **95.24** | **113.40** | **206.25** |

Although three of the four monuments maintain their turn (1. Church of the Virgin's Dormition, 2. Church Of St. Demitrios, 3. Church of St. Taxiarches) regarding the necessity of interventions based on the criterion of the vulnerability index, there is a significant deviation in the case of the 4th monument (Monastery of Saint Basil-2$^{nd}$ column). More specifically, according to the present analysis, the monument shows the smallest priority for interventions as it receives the highest score, contrary to the investigation carried out in the Pre-Earthquake Assessment Data Sheet, where it received the lowest score (highest priority). This deviation is resulted due to the ignorance of many parameters in this report such as the pathology and the seismic zone which in the case of this monument are the most unfavorable. Therefore, it is inappropriate to draw conclusions from the study of only five parameters, but it is necessary to extend the research presented here to the rest parameters of the Pre-Earthquake Assessment Data Sheet for Single Spaced One Storey Historical Buildings as well.

# 5 REFERENCES

[1] Delinikola E., Dourakopoulos J., Miltiadou A., Papantonopoulos K., Toumpakari E., Psycharis I. (Scientific Co-ordinator), 2016, 'Pre-earthquake Assessment of Monuments', Report

[2] ABAQUS 6.9 (2011). User Documentation. Internet Manual. Simulia. Retrieved 10 September.

[3] Greek Bibliography: Η.Μπεριάτος «Παραδοσιακά Εκκλησιαστικά Κτίσματα της Επαρχίας Πάλλης» από τον 2ο τόμο «Κεφαλλονιά, ένα μεγάλο μουσείο», Εκκλησιαστική Τέχνη, Αργοστόλι 1996.

[4] Greek Bibliography: Ε.Κοζικόπουλος Αριθμητική μελέτη δομικών συστημάτων εκκλησιών που υποβάλλονται σε σεισμικές φορτίσεις.

[5] Psycharis, I. N., Mouzakis, H. P., Miltiadou, A., Pavlopoulou, E. P., Taflampas, I. M., "Investigation of the seismic response of a Byzantine church and evaluation of the effectiveness of proposed interventions", Earthquake Spectra, Vol. 26, No. 2, pp. 473-497, 2010.

[6] Miltiadou–Fezans A., Dourakopoulos J.A., Giannopoulos P., Delinikolas N., Vintzileou E. and Mouzakis H. (2016), "Numerical analyses of the structural behaviour of the Katholikon of Daphni Monastery after grouting application", 10th International Conference on Structural Analysis of Historical Constructions (SAHC 2016), Leuven, Belgium, 13-15 September.

# Verification of methodologies for the Pre-Earthquake Assessment of Monuments

# PART B

## Single Free-Standing Columns (Structural Category F)

# TABLE OF CONTENTS

# 1 INTRODUCTION

## 1.1 SCOPE OF WORK

In the frame of the Project 'Pre-Earthquake Assessment of Monuments' an initial approach towards determining the criteria for the evaluation of the stability of single free-standing columns is presented (structural category F).

In the first part of the Project an extended research of the geometrical configurations, the pathology and environmental conditions of ancient columns, found in Greece, has been made. In addition, a pre-assessment datasheet for a preliminary rating of their seismic vulnerability has been created. The rating is established by taking into account geometrical parameters (i.e. height and diameter of the columns, the number of drums) as well as soil category and seismic hazard zone of the location of the examined monument [1]. In specific, a vulnerability index is calculated based on the rating of the following geometrical and environmental parameters:

1. Diameter of bedding of the lowest drum
2. Number of Drums
3. Existence of empolia or dowels
4. Seismic Hazard Zone (according to the Greek National Annex of Eurocode 8)
5. Existence of Near Faults
6. Soil Category
7. Environmental Danger Assessment
8. Loss of Mass
9. State of preservation of the joints
10. Maximum relative drum displacement
11. State of preservation of the foundation
12. Column standing since the Antiquity

In the second part of the Project, the influence of some of the above-mentioned parameters is examined with the use of numerical analysis. The results, presented within the next chapters of the current report, are evaluated and used to determine the final rating methodology of the pre-assessment datasheet.

## 1.2 SELECTION OF EXAMINED PARAMETERS

Due to the fact that not all of the geometrical and environmental parameters could be examined within the project, a selection of the most critical ones was made. Thus, it was decided to determine the influence of the size of the column, the column's slenderness and the number of the drums composing the column and evaluate the effect of the Seismic Hazard Zone of the location of the column. For all the parameters, upper and lower margins were set based on realistic data obtained from [1] and the values in-between were used for the numerical analysis models. In detail the following values for each parameter were examined:

1. 'Size effect'

   For the evaluation of the effect of the size of a column on its stability during a seismic event, numerical analyses with five different base diameters (diameter of lowest drum) were performed. In specific the diameters D=0.45m, 0.60m, 0.80m, 1.00m and 1.50m were examined for several column slenderness's, meaning for several different column heights.

2. Column slenderness (Height/Diameter)

   The non-dimensional column slenderness is the ratio of the height to the base diameter. For each aforementioned column size, numerical models were created for non-dimensionless slenderness varying from 5.5 to 9.5 (with a step of 0.5).

3. Number of drums

   The effect of the number of drums composing a column is examined by modeling the middle sized column (of a 0.80m base diameter) with 4, 5 and 6 drums and comparing their results in terms of maximum displacement.

4. Monolithic columns

   For each column size and all aforementioned slenderness ratio numbers, analyses for monolithic columns were performed. The results were compared to those of the multi-drum columns.

5. Seismic Hazard Zone

   Based on the Greek National Annex of Eurocode 8, the Greek area is divided into three zones of seismic risk. The ground acceleration values are 0.16g for the first zone, 0.24g for the second zone and 0.36g for the third zone. In order to evaluate the effect of the location of the column all the seismic records used for the parametric analyses were amplified with the EPA of the three Seismic Hazard Zones of the Greek National Annex of Eurocode 8.

It should be noted that the parametric analysis is implemented on single free-standing columns without any damages. Damages such as drum shifts, which lead to loss of contact between the drums, cracks of the construction material and inclination of the column to the vertical are parameters crucial for the stability of the free-standing columns that should be evaluated in a future research project.

# 2 NUMERICAL ANALYSIS

## 2.1 INTRODUCTION

For the parametric numerical analyses presented in this report, the code 3DEC by Itasca Consulting Group, Inc. was employed. The code is based on the discrete element method and was initially designed for the analysis of assemblies of discrete rigid bodies [2]. The discontinuities are considered as boundary conditions, large displacements and rotations as well as detachment of the blocks are permitted, while the code automatically detects new contacts during the response. For rigid block problems, the code gives an accurate contact formulation, in which the interaction takes place at a number of contact points. Each contact is assigned a contact area, which is used to calculate the local point stiffness, in terms of the user-defined normal and shear stiffness of the discontinuity surface. In addition, a damping coefficient is introduced. Thus, the code is suitable for the numerical analysis of single free-standing columns, since due to their spinal construction, their behavior during a seismic event is mostly governed by the rocking and/or sliding of the drums, individually or in groups that lead to large displacements and rotations.

The values of the joint stiffness used in the models were $k_N = 5\times10^9$ Pa/m in the normal direction and $k_S = 1\times10^9$ Pa/m in the tangential direction. These values were derived by comparing numerical results with corresponding experimental data obtained from shaking table tests performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens [3, 4, 5].

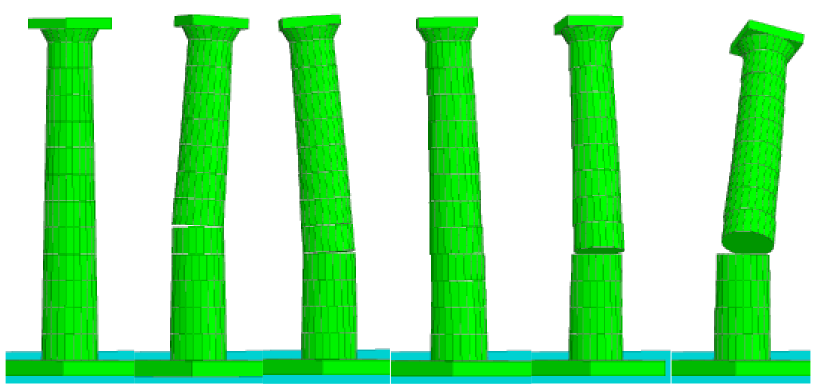


*Figure 2.1. Rocking and sliding of the drums according to the numerical analysis (3DEC model).*

## 2.2 DETERMINATION OF SEISMIC INPUT MOTIONS

As already mentioned, due to their spinal construction, the dynamic behaviour of ancient monuments is highly nonlinear and very sensitive to even trivial changes in the geometry or the base motion characteristics. The determination and selection of the seismic input motions is crucial. Variation in terms of frequency content (especially the predominant period) and amplitude are needed in order to obtain 'secure' conclusions from a parametric analysis. Thus, for the selection of the earthquake motions, the results of previous research projects were used. Three seismic motions of earthquakes recorded on rock and stiff soil were selected based on records proposed by I.N. Psycharis [6, 7] for the numerical analysis and design of the Acropolis monuments:

- The seismic input motion recorded at the at the first basement of the Syntagma Metro station (B) at a depth of approximately 7.0 m below the ground surface during the 5.9 magnitude earthquake in Athens, Greece, on the 7th of September 1999. The station - approximately 20 km away from the epicenter- is located on stiff soil. The record shows a maximum acceleration of 0.11g.
- A record from the 13-05-1995 Kozani (Greece) Earthquake. The earthquake was of a $M_w$ = 6.5 magnitude. The selected seismic input motion was recorded in the prefecture building of the town located approximately 14 km away from the epicenter. The input motion has a maximum acceleration of 0.21g and period of about 0.2 sec.
- The seismic input motion recorded at Wonderland Ave during the 6.7 magnitude earthquake in Northridge California (USA) on the 17th of January 1994. The station - approximately 23 km away from the epicenter- is located gave a maximum acceleration of 0.17g.

All records were amplified with the EPA of the three Seismic Hazard Zones of the Greek National Annex of Eurocode 8 in order to determine the influence of the Seismic Zones to the column's stability.

Both horizontal components of the considered records were applied simultaneously as the base motion to the numerical model. The main parameters of the seismic input motions are shown in Table 2.1. In Figures 2.2 and 2.3 the time histories and the response spectrum of each seismic motion are given.

*Table 2.1 Data of selected earthquake excitations*

| No | Earthquake | Date | Record | Soil | pga [g] | |
|---|---|---|---|---|---|---|
| 1 | Northridge, California, USA | 17-01-1994 | Los Angeles Wonderland Ave. | A | L: | 0.112 |
| | | | | | T: | 0.172 |
| 2 | Athens Greece | 07-09-1999 | Syntagma Metro B | Stiff soil | L: | 0.109 |
| | | | | | T: | 0.086 |
| 3 | Kozani, Greece | 13-05-1995 | Kozani Prefecture Blg. | A | L: | 0.208 |
| | | | | | T: | 0.143 |

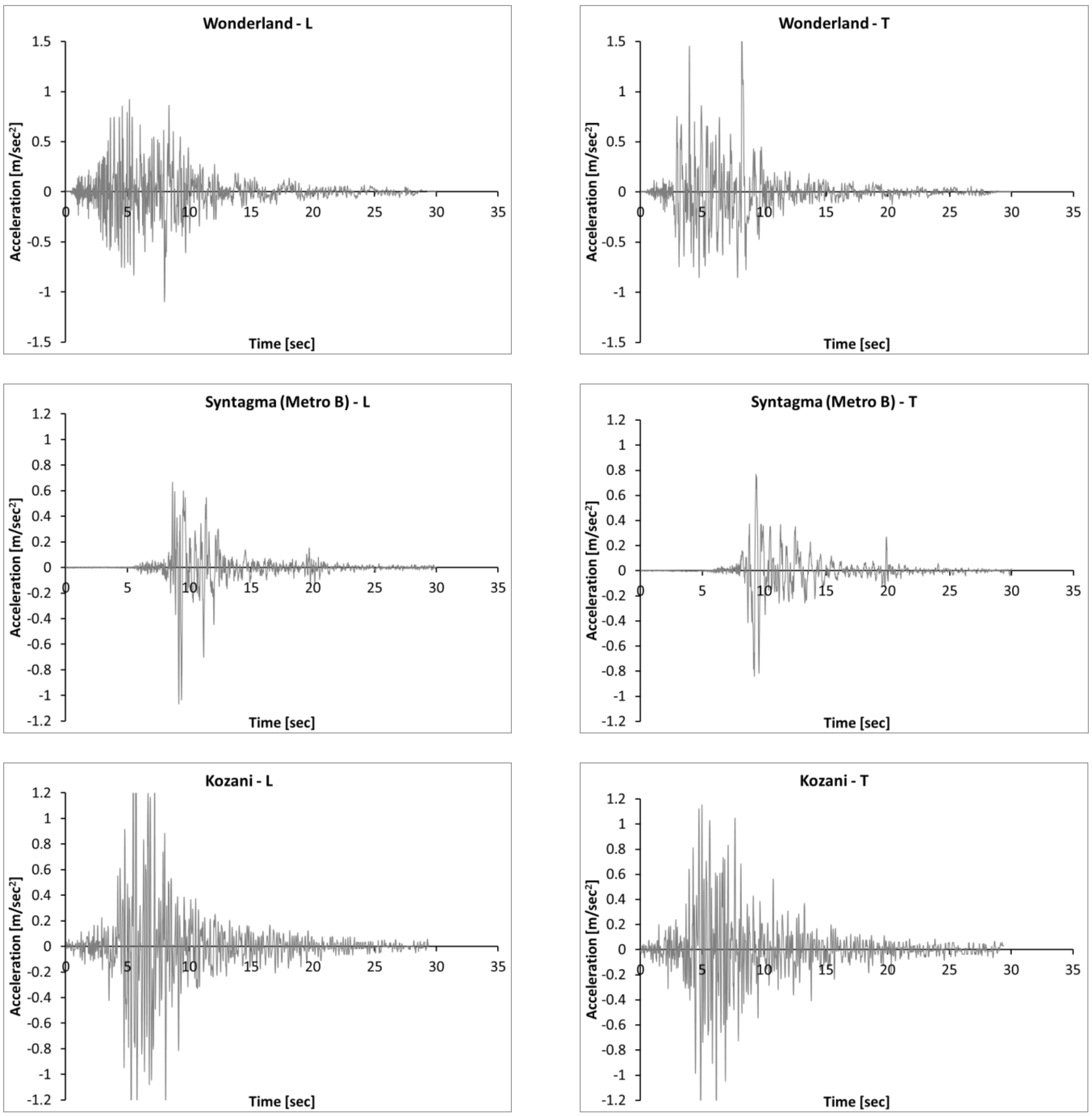



*Figure 2.2 Time histories of the selected records*

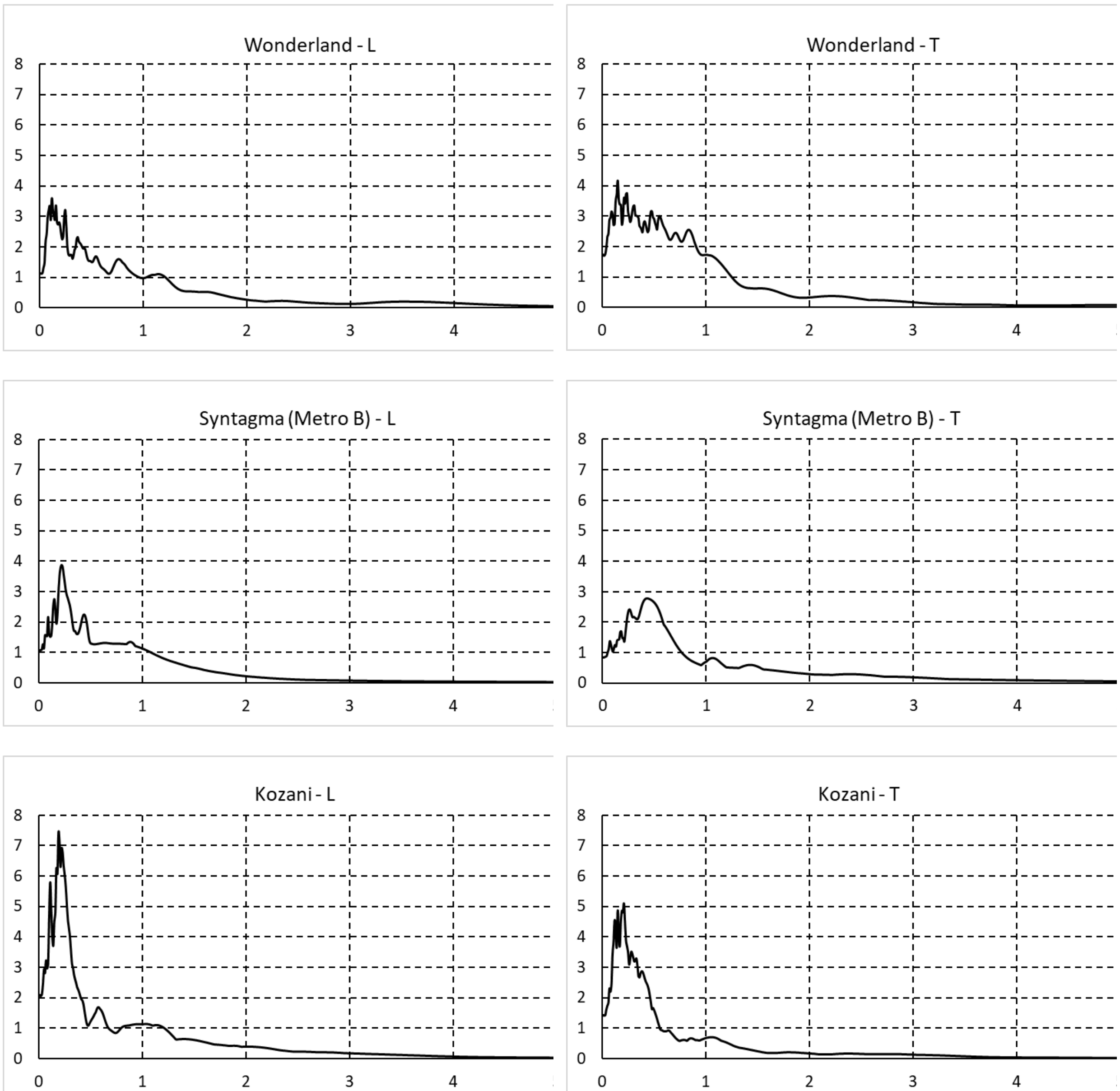



*Figure 2.3 Acceleration response spectra for 5% damping of the selected records*

## 2.3 PRESENTATION OF THE RESULTS

### 2.3.1 General

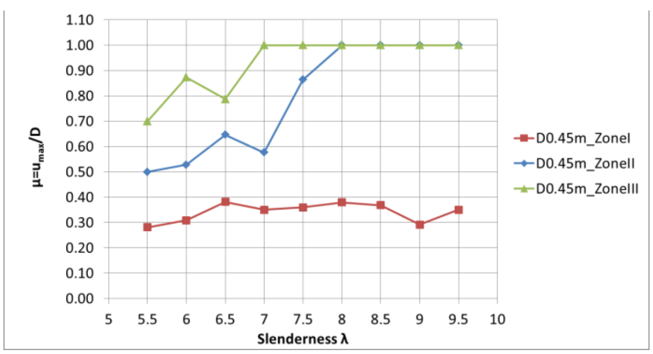
In order to evaluate the effect of the aforementioned parameters on the dynamic behavior of a single freestanding column the ratio of the maximum displacement, calculated during the seismic excitation at the top of the column, to the base diameter was used:

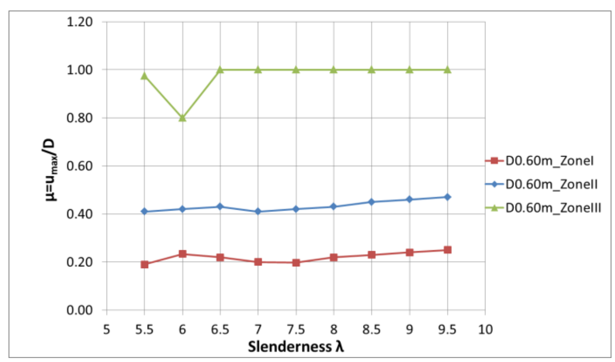
$$\mu = \frac{u_{\max}}{D_{base}} \qquad \text{[Ex.2.1]}$$

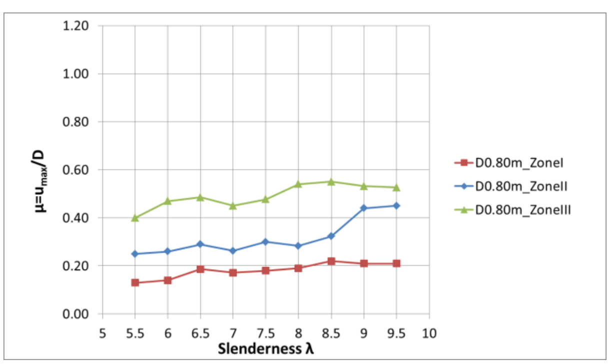
In total 594 numerical analyses were performed and the results are summarized in the following diagrams (Figure 2.4-2.9).

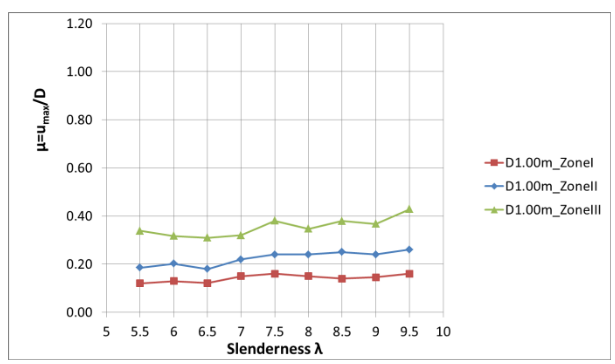
### 2.3.2 Results

In the following figures (2.4 -2.9) the results from the analyses of the multi-drum as well as the monolithic columns with the seismic input motions of the Syntagma, Kozani and Northridge Earthquake, amplified to match the EPA of the three seismic zones, are given. Each diagram contains the results for a specific size of column, meaning a specific diameter of the column's base. The vertical axis of the diagrams represents the ratio of the maximum displacement, calculated during the seismic excitation at the top of the column, to the base diameter, while the horizontal axis shows the slenderness of the column. When during the analysis overturning of the column was observed the ration μ was set to zero.



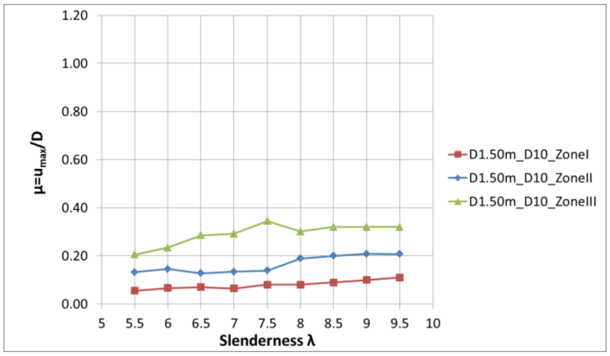



*Figure 2.4. Results from the analyses of the multi-drum column with the seismic input motion of the Syntagma Earthquake*

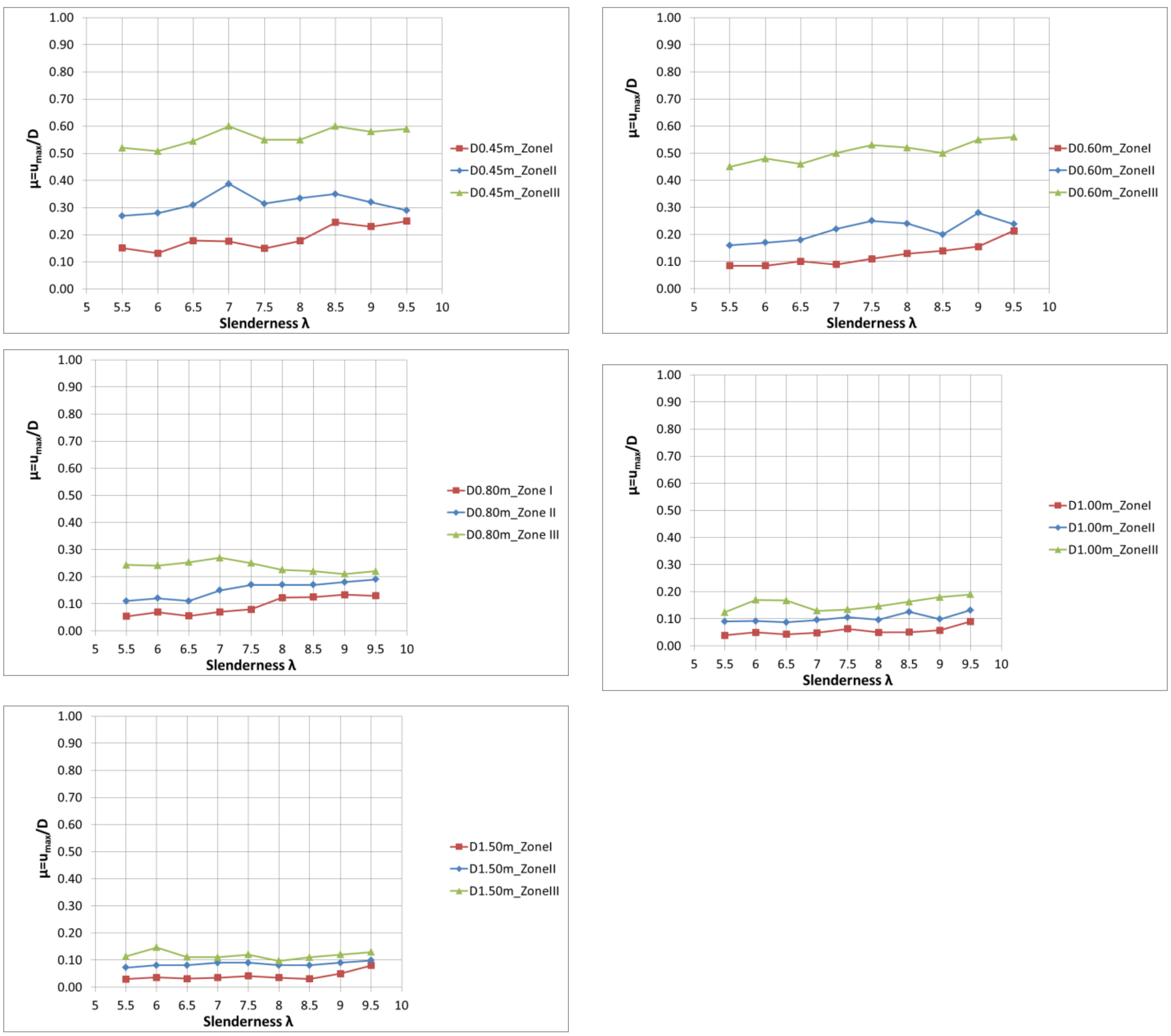



*Figure 2.5. Results from the analyses of the multi-drum column with the seismic input motion of the Kozani Earthquake*

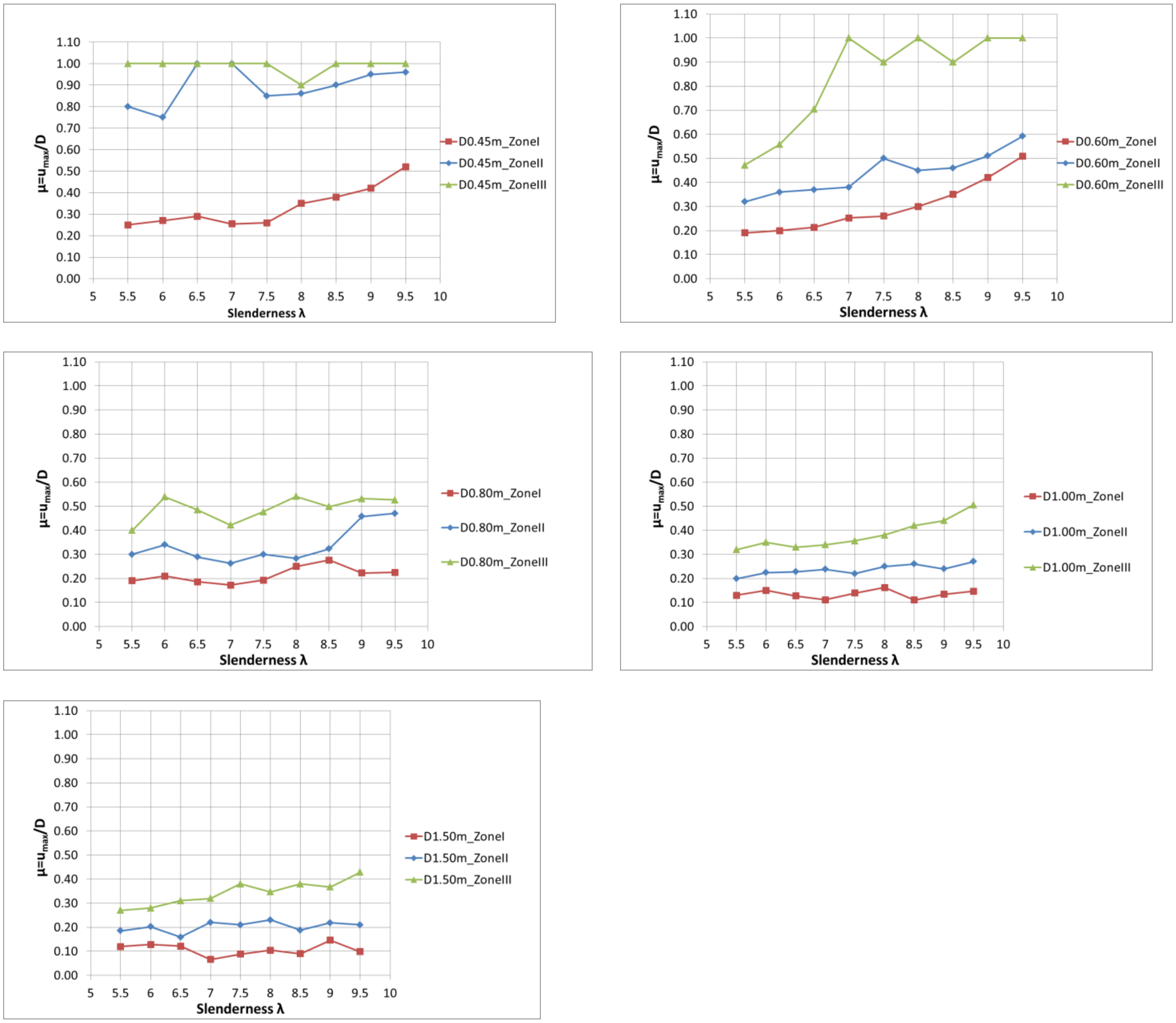



*Figure 2.6. Results from the analyses of the multi-drum column with the seismic input motion of the Northridge Earthquake*

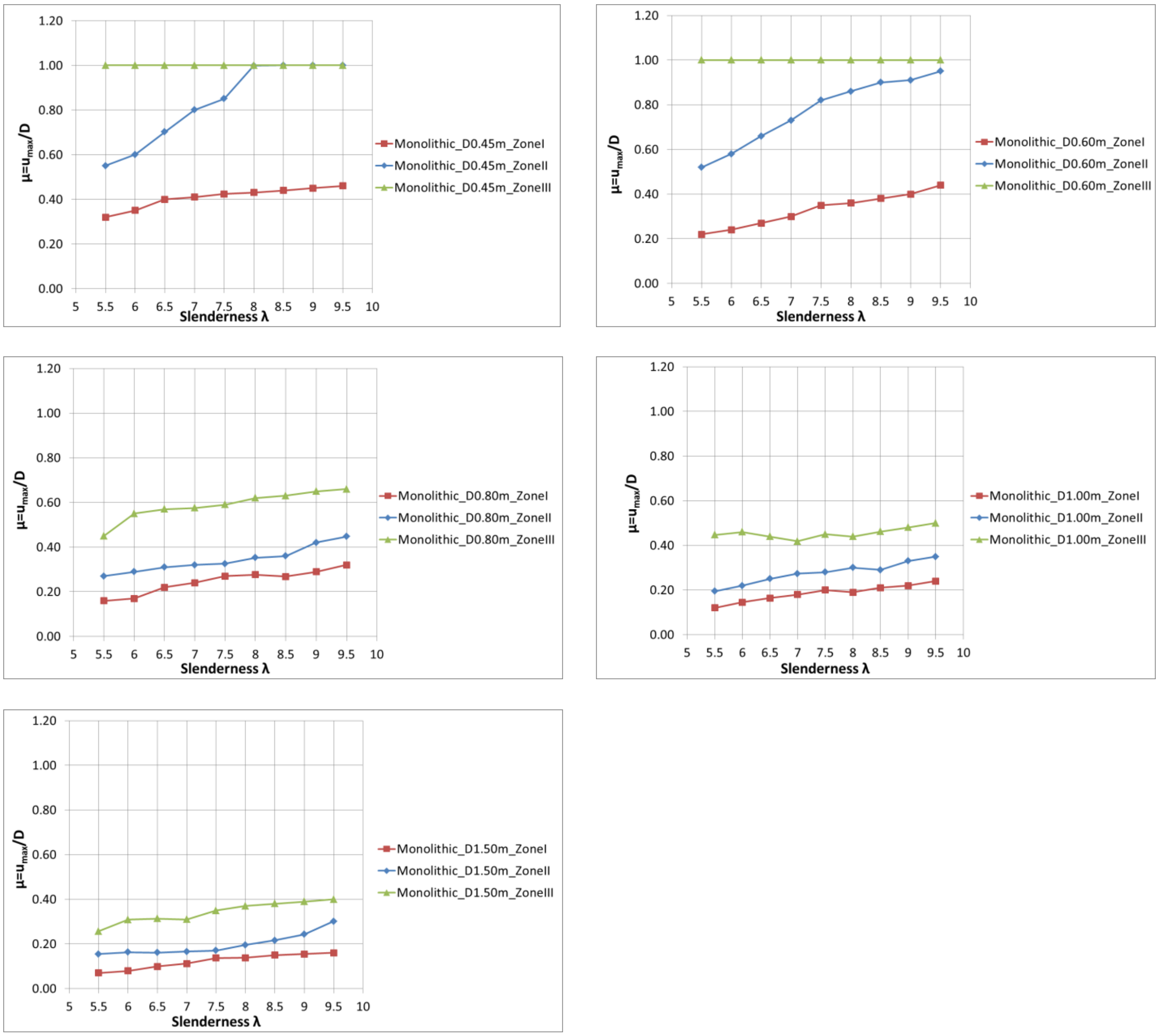



*Figure 2.7. Results from the analyses of the monolithic column with the seismic input motion of the Syntagma Earthquake*

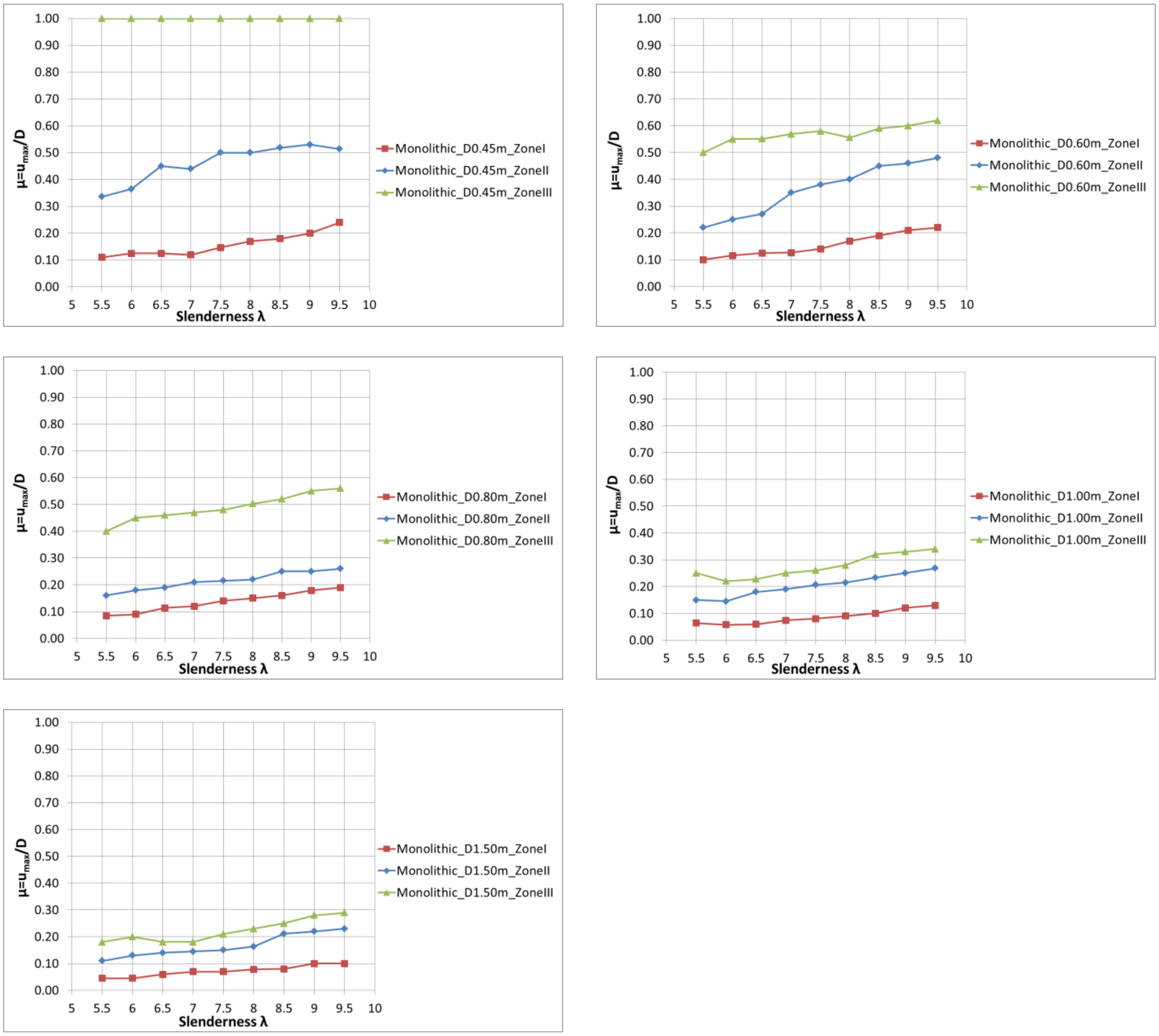



*Figure 2.8. Results from the analyses of the monolithic column with the seismic input motion of the Kozani Earthquake*

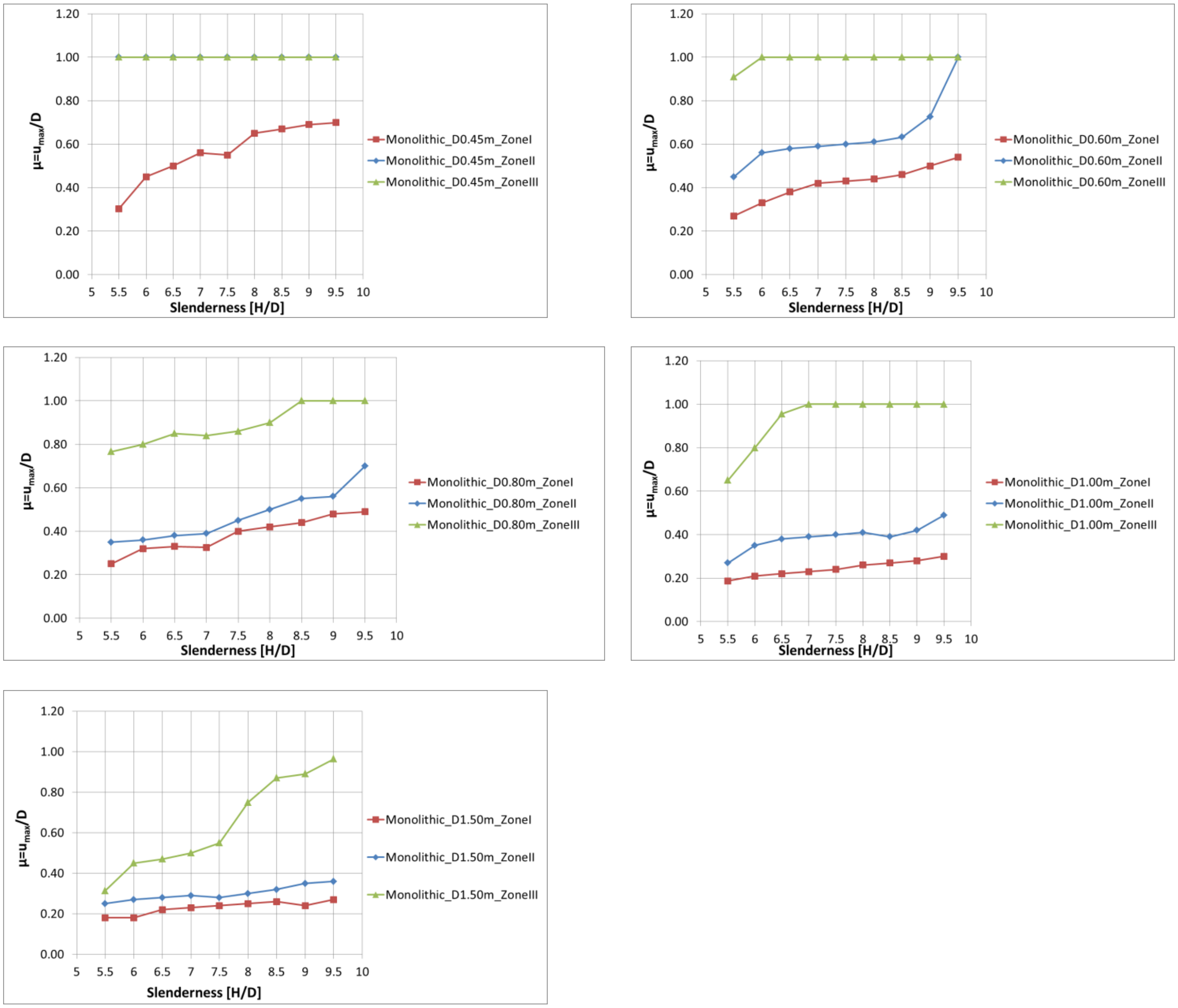



*Figure 2.9. Results from the analyses of the monolithic column with the seismic input motion of the Northridge Earthquake*

# 3 CALCULATION OF VULNERABILITY INDEX

## 3.1 GENERAL

The rating system is based and aligned to the proposed data sheet of the first part of the Project [1].

The rating scale of the various parameters is set within the margins from 1 to 3, with 1 corresponding to the most unfavorable situation, meaning the parameter undermines the stability of the column.

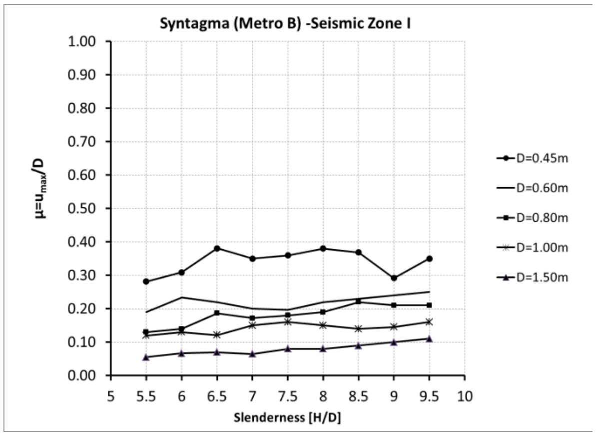
The vulnerability index is equal to the sum of the rates of each parameter multiplied to its importance factor. In the following paragraphs the rating and the importance coefficients of the parameters evaluated via numerical analyses are presented.

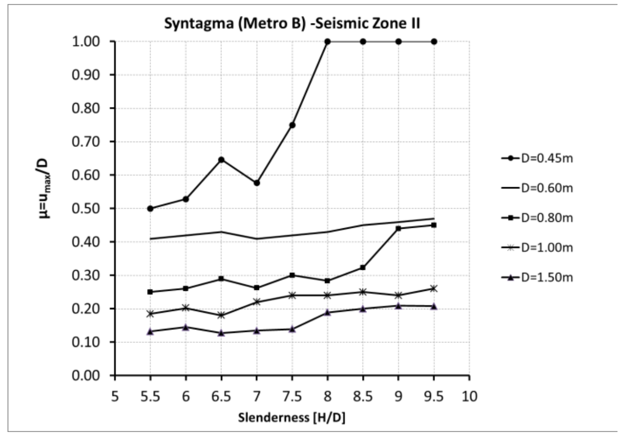
## 3.2 PARAMETER RATING

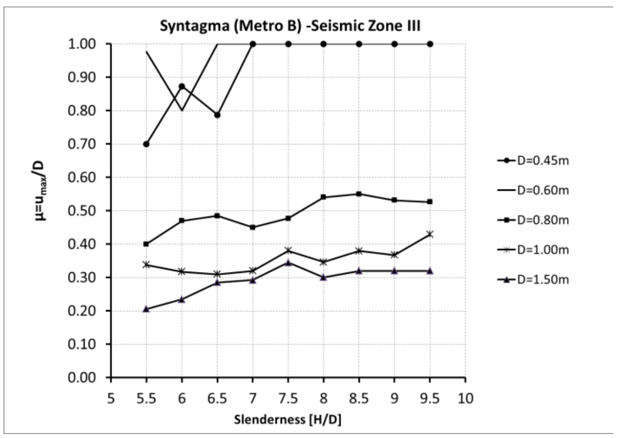
### 3.2.1 'Size effect'

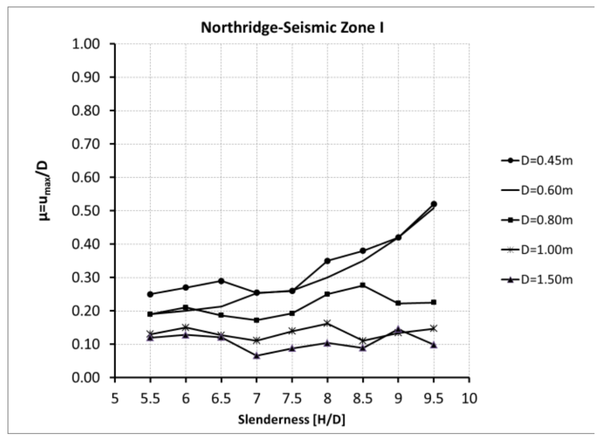
The size effect is incorporated in the calculation of the vulnerability index through the dimension of the diameter of the lowest drum or the base of the column. From the results of the numerical analysis it is proven (Figure 3.1) that in all cases the column with the smallest base diameter (D=0.45m) exhibits the largest relative displacements (large values of μ ratio) and the column with the largest base diameter (D=1.50m) exhibits the smallest relative displacements (smaller values of μ ratio).

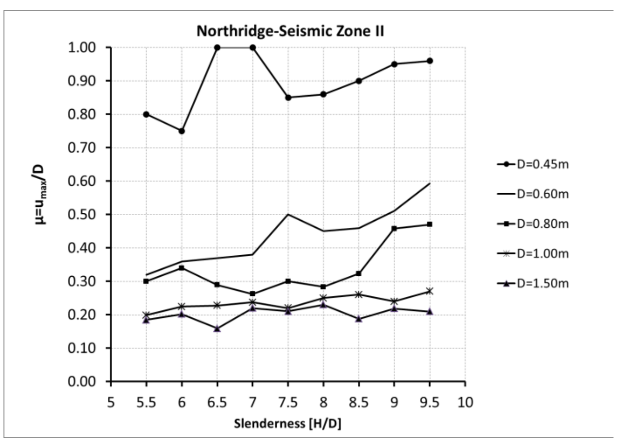
In Figure 3.1 the comparison of the results for the different diameter sizes of the multi-drum columns are presented.

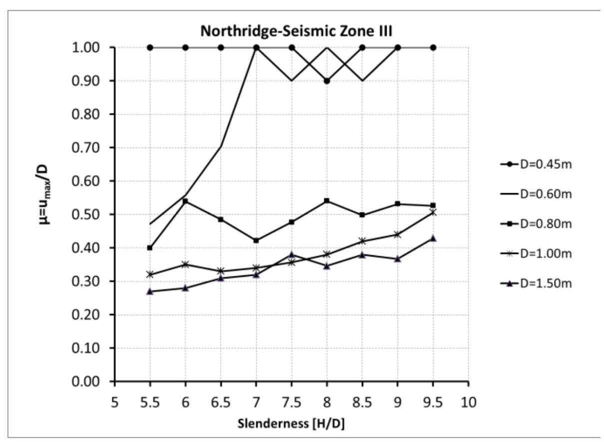

Syntagma (Metro B) -Seismic Zone I
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m


Syntagma (Metro B) -Seismic Zone II
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m


Syntagma (Metro B) -Seismic Zone III
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m


Northridge-Seismic Zone I
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m


Northridge-Seismic Zone II
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m


Northridge-Seismic Zone III
μ=u_max/D
Slenderness [H/D]
D=0.45m
D=0.60m
D=0.80m
D=1.00m
D=1.50m

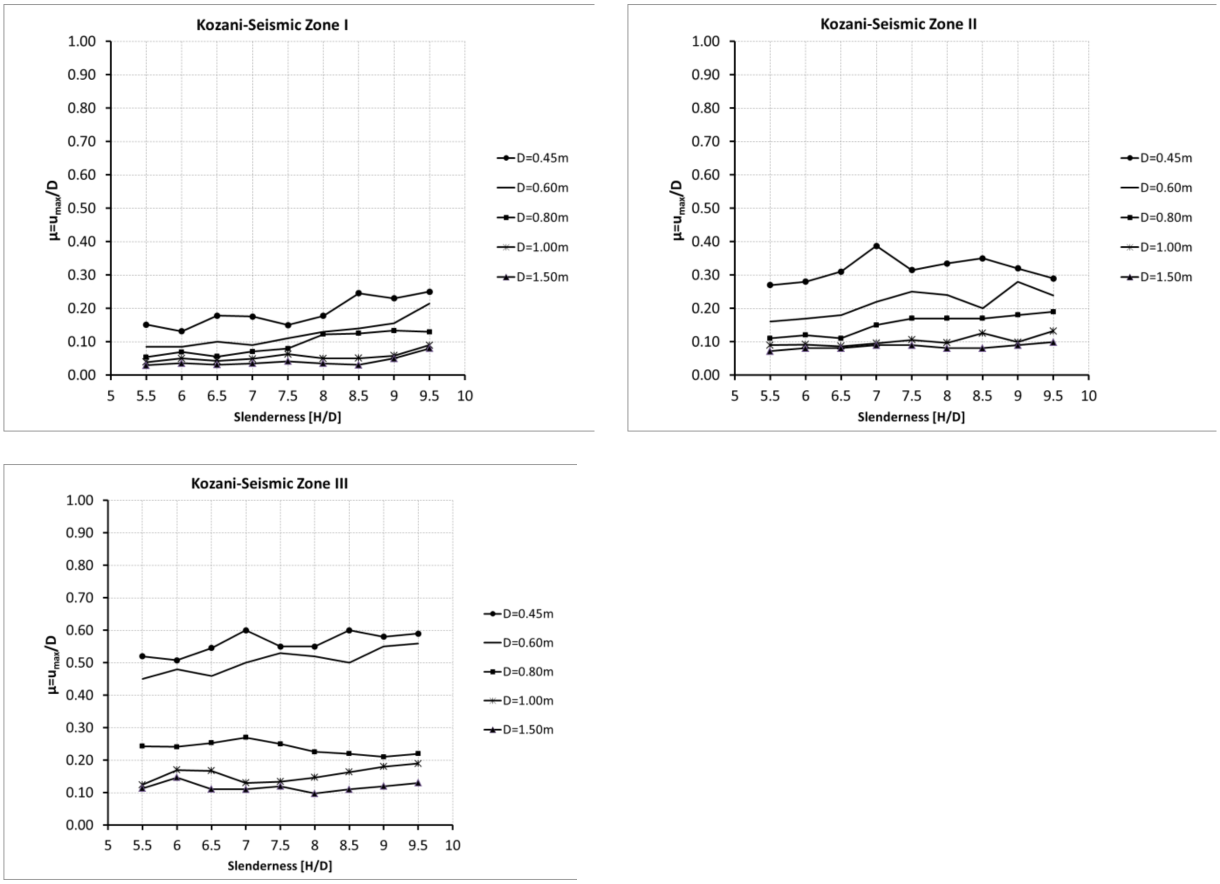



*Figure 3.1. Comparison of the results for the different diameter sizes of the multi-drum columns*

In order to calculate the rating of each diameter category and to align with the rating system proposed in in the Pre-Earthquake assessment data sheet [1] following assumptions and calculation is made:

1. The rating of the columns with base diameter of 1.50m (and larger) are set with a rating equal to 3.
2. The results of the column with the largest base diameter (D=1.50m) are set as the base to normalize the results of the rest of the columns by using the following expression:

$$r_D = 3 \cdot (\mu_{D=1.50m} / \mu_D) \qquad \text{[Ex.3.1]}$$

This calculation was performed for all slenderness ratios and all seismic inputs (three earthquakes and three seismic zones) and the following average values were extracted:

*Table 3.1 Average value of parameter $r_D$ for each examined base diameter*

| Diameter [m] | $r_D = 3 \cdot (\mu_{D=1.50m} / \mu_D)$ |
|---|---|
| 0.45 | 0.80 |
| 0.60 | 1.10 |
| 0.80 | 1.65 |
| 1.00 | 2.30 |
| 1.50 | 3.00 |

Thus based on the values in Table 3.1 the following rating for the 'size' parameter is proposed:

*Table 3.2 Rating for the 'size' parameter*

| Diameter | Rating |
|---|---|
| $D_{Base}$≤0.60m | 0.80 |
| 0.60m≤$D_{Base}$<0.80m | 1.10 |
| 0.80m≤$D_{Base}$<1.00m | 1.65 |
| 1.00m≤$D_{Base}$<1.50m | 2.30 |
| 1.50m≤$D_{Base}$ | 3.00 |

### 3.2.2 Slenderness ratio

From the results of the numerical analyses of the multi-drum and the monolithic columns no safe conclusions can be drawn regarding the slenderness ratio. In general the increase of the column's slenderness leads to increase of the maximum displacement and the possibility of overturning, but there are no clear margins that can be set in order to determine the rating values. In order to evaluate the effect of the slenderness ratio to the stability of a freestanding column further research and numerical analyses is needed.

### 3.2.3 Number of Drums

Based on the numerical results, the number of drums affects slightly the dynamic behavior of a free-standing column. By increasing the number of drums (and simultaneously decreasing their slenderness) the stability increases.

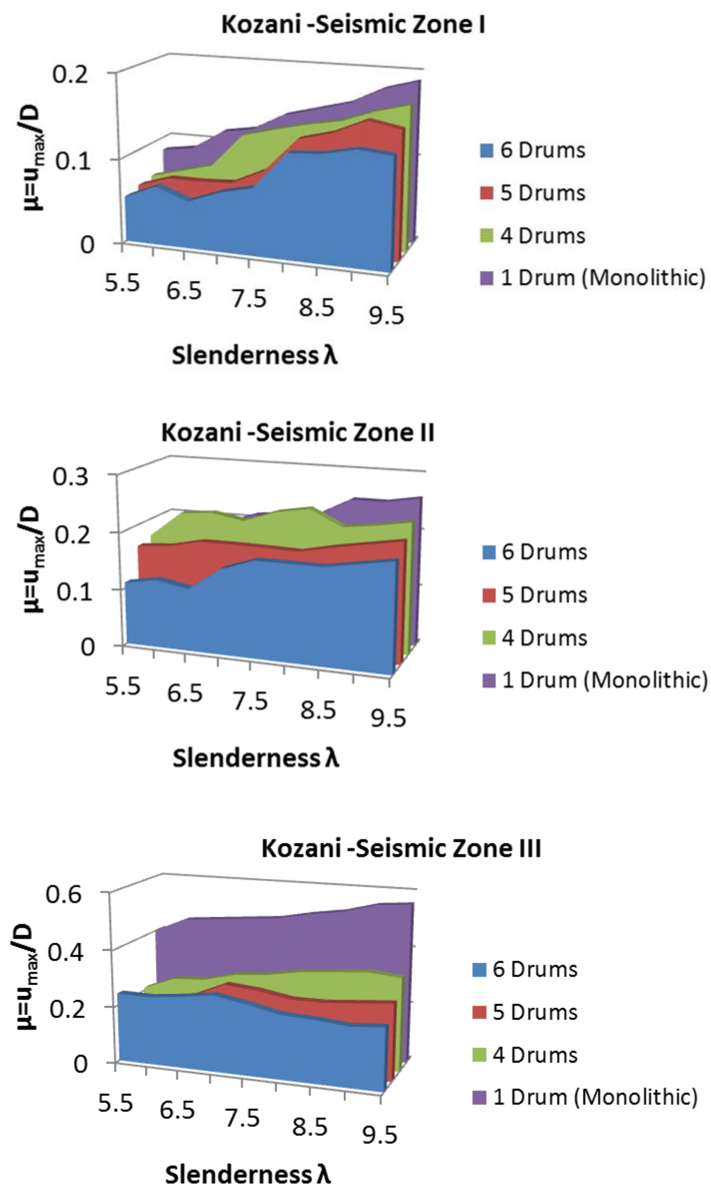



*Figure 3.2. Comparison of the results for the different number of drums*

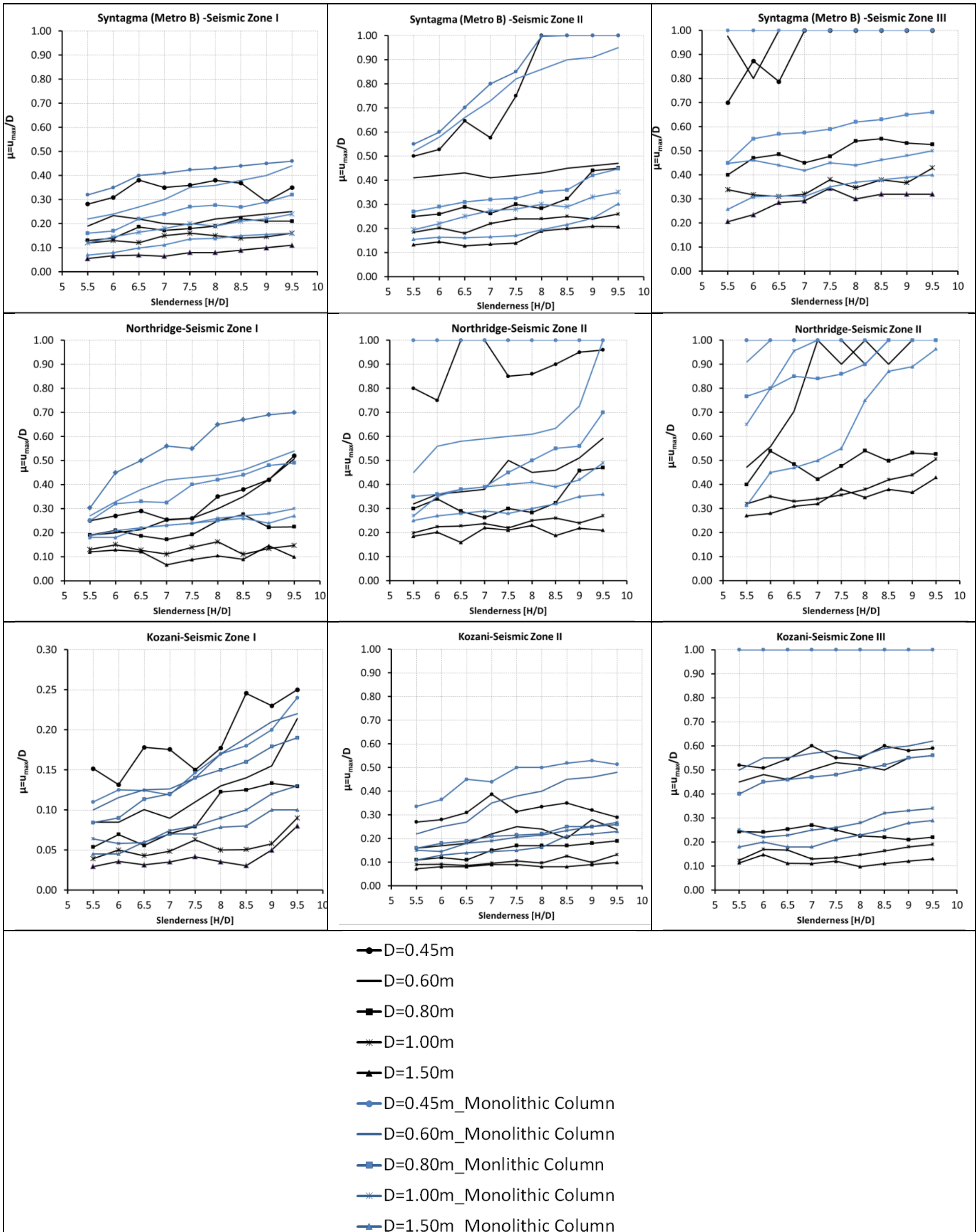



*Figure 3.3. Comparison between multi-drum and monolithic columns.*

In order to calculate the rating of each category for the parameter of the number of drums composing a column and to align with the rating system proposed in in the Pre-Earthquake assessment data sheet, the following assumptions and calculations are made:

1. The rating of the columns with 6 Drums are set with a rating equal to 3.
2. The results of the column with the 6 Drums are set as the base to normalize the results of the rest of the columns by using the following expression:

$$r_{NDrums} = 3 \cdot (\mu_{NDrums=6} / \mu_{NDrums}) \quad \text{[Ex.3.1]}$$

This calculation was performed for all slenderness ratios and all seismic inputs (three earthquakes and three seismic zones) and the following average values were extracted:

*Table 3.3 Average value of parameter $r_{NDrums}$ for each examined number of drums composing a column*

| Number of drums | $r_{NDrums} = 3 \cdot (\mu_{NDrums=6} / \mu_{NDrums})$ |
|---|---|
| 1 *(Monolithic Column)* | 1.10 |
| 4 | 2.05 |
| 5 | 2.51 |
| 6 | 3.00 |

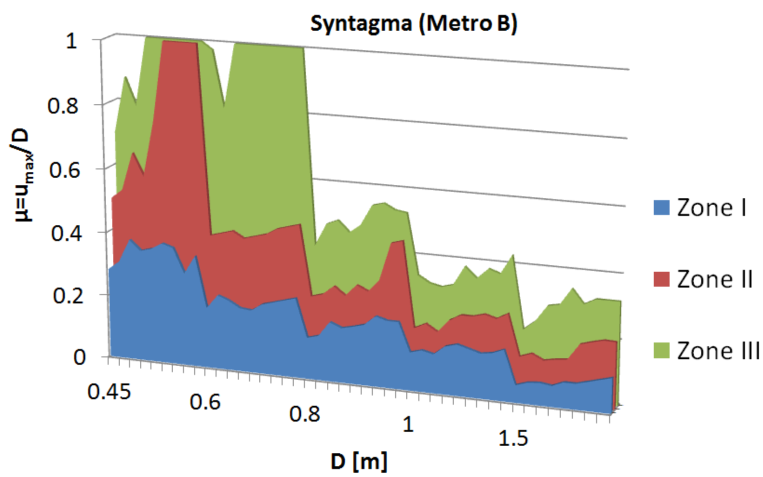
Thus based on the values in Table 3.3 the following rating for the ‘number of drums’ parameter is proposed:

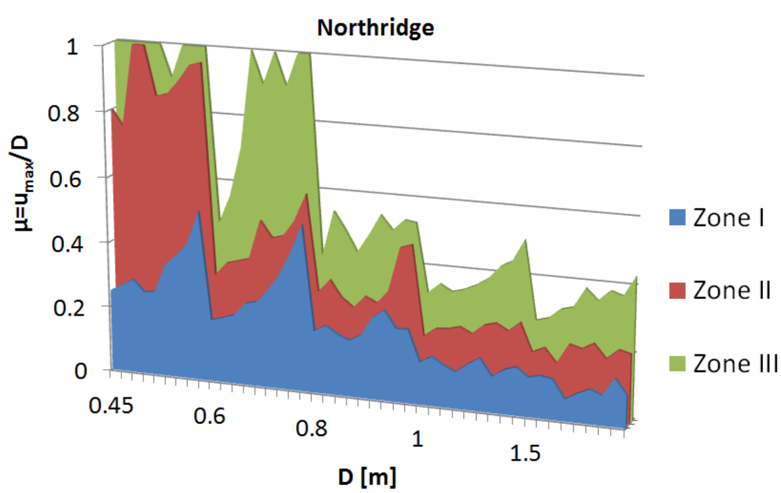
*Table 3.4 Rating for the ‘number of drums’ parameter*

| Number of drums | Rating |
|---|---|
| $N_{Drums}$=1 (Monolithic Column) | 1.00 |
| 1<$N_{Drums}$≤4 | 2.00 |
| 4< $N_{Drums}$≤6 | 2.50 |
| 6<$N_{Drums}$ | 3.00 |

### 3.2.4 Seismic Zones

The intensity of the expected seismic event affects the stability of the column severely (Figure 3.4). Thus, location of the examined monument and the corresponding seismic hazard zone are crucial for the evaluation of the vulnerability index.



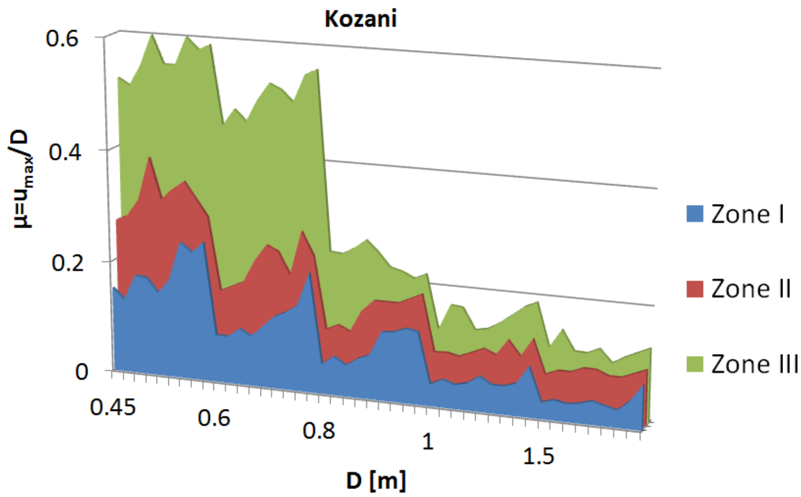



*Figure 3.4. Comparison of the results for the different seismic zones.*

In order to calculate the rating of seismic zone category and to align with the rating system, proposed in in the Pre-Earthquake assessment data sheet, the following assumptions and calculations are made:

1. The rating of the columns located in seismic zone areas 'I' is set equal to 3.
2. The results of the columns located in seismic zone areas I are set as the base to normalize the results of the rest of the columns by using the following expression:

$$r_{SeismZones} = 3 \cdot (\mu_{SeismZone=I} / \mu_{SeismZone}) \quad [Ex.3.1]$$

This calculation was performed for all slenderness ratios, column diameters and the three seismic input motions and the following average values were extracted:

*Table 3.5 Average value of parameter $r_{SeismZone}$*

| Seismic Zone | $r_{SeismZones} = 3 \cdot (\mu_{SeismZone=I} / \mu_{SeismZone})$ |
|---|---|
| ZI | 3.00 |
| ZII | 1.65 |
| ZIII | 1.03 |

Thus based on the values in Table 3.5 the following rating for the 'number of drums' parameter is proposed:

*Table 3.6 Rating for the 'seismic zone' parameter*

| Seismic Zone | Rating |
|---|---|
| ZI | 3.00 |
| ZII | 1.65 |
| ZIII | 1.00 |

## 3.3 PARAMETER PARTICIPATION/IMPORTANCE FACTORS

For the calculation of the vulnerability index, each parameter rate is multiplied with the corresponding participation factor. This factor represents the importance of the specific parameter on the stability of a single freestanding column compared to the other parameters.

For the calculation and determination of the participation factor of the examined parameters, the following methodology is applied:

1. A model with middle sized geometrical characteristics, commonly found in Greece according to [1], is used to create a 'reference' model. Thus, the height of the columns is taken equal to 6.00m, the base diameter 1.00m and the number of drums 5. It is also assumed that the reference model is located in an area of seismic zone II.
2. Each parameter of the model is altered twice. In the first alteration, the column falls into a lower rating category and in the second alteration the column changes to a higher category. For each alteration the relative displacement and the factor μ is calculated. Thus, the importance factor is determined by calculating the percentage of change in the μ factor. In Table 3.7 the data of the reference model and it's alterations are presented.

For the numerical analysis of the reference models and the alternative models, the input motion of the Syntagma Earthquake, presented in Chapter 2, is used.

It should be noted that the importance factors determined herein are only for the parameters examined numerical and presented in the previous chapters. For the final calculation of the vulnerability index of column further research is needed in order to include the rest of the parameters in the calculation.

*Table 3.7 Data of the numerical models for the determination of the participation factors*

| No | Model | Size effect / Base diameter [m] | Number of drums | Seismic Zone |
|---|---|---|---|---|
| 1 | Reference model | 1.00 | 5 | ZII |
| 2 | 1st Alteration – Smaller Base diameter | 0.80 | | |
| 3 | 1st Alteration – Larger Base diameter | 1.50 | | |
| 4 | 2nd Alteration – Fewer number of drums | 1.00 | 1 | |
| 5 | 2nd Alteration – Increased number of drums | | 10 | |
| 6 | 3rd Alteration - Lower seismic zone | | 5 | ZI |
| 7 | 3rd Alteration - Higher seismic zone | | 5 | ZIII |

*Table 3.8 Calculated μ-Factors per model*

| No | Model | $\mu = \frac{u_{\max}}{D_{base}}$ |
|---|---|---|
| 1 | Reference model | 0.19 |
| 2 | 1st Alteration – Smaller Base diameter | 0.27 |
| 3 | 1st Alteration – Larger Base diameter | 0.10 |
| 4 | 2nd Alteration – Fewer number of drums | 0.30 |
| 5 | 2nd Alteration – Increased number of drums | 0.18 |
| 6 | 3rd Alteration - Lower seismic zone | 0.16 |
| 7 | 3rd Alteration - Higher seismic zone | 0.30 |

In Table 3.8 the calculated μ-factor is given. From the results of the reference model and model 2 and 3 the following change (ρ) in the rate can be calculated:

1. Base diameter parameter

$$\rho_{Model2} = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{Model2} - \mu_{ReferenceModel}}{\mu_{ReferenceModel}} = \frac{0.27 - 0.19}{0.19} = 0.42$$

$$\rho_{Model3} = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{ReferenceModel} - \mu_{Model3}}{\mu_{ReferenceModel}} = \frac{0.19 - 0.10}{0.19} = 0.47$$

$$\rho = \frac{\rho_{Max} + \rho_{Model3}}{2} = \frac{0.27 + 0.47}{2} = 0.45$$

2. Number of drums parameter

$$\rho_{Model4} = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{Model4} - \mu_{ReferenceModel}}{\mu_{ReferenceModel}} = \frac{0.30 - 0.19}{0.19} = 0.58$$

$$\rho_{Model5} = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{Model5} - \mu_{ReferenceModel}}{\mu_{ReferenceModel}} = \frac{0.19 - 0.18}{0.19} = 0.05$$

$$\rho_{Average} = \frac{\rho_{Model4} + \rho_{Model5}}{2} = \frac{0.58 + 0.05}{2} = 0.32$$

3. Seismic zone parameter

$$\rho = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{Model6} - \mu_{ReferenceModel}}{\mu_{ReferenceModel}} = \frac{0.19 - 0.16}{0.19} = 0.15$$

$$\rho_{Model7} = \frac{\Delta\mu}{\mu_{ReferenceModel}} = \frac{\mu_{Model7} - \mu_{ReferenceModel}}{\mu_{ReferenceModel}} = \frac{0.30 - 0.19}{0.19} = 0.58$$

$$\rho_{Average} = \frac{\rho_{Model6} + \rho_{Model7}}{2} = \frac{0.15 + 0.58}{2} = 0.37$$

In order to unify all the importance factors (IF), the maximum change ρ, which corresponds to the ρ-value of the base diameter parameter, is used as a reference and is set as 1.0. Thus all importance factors can be written as:

1. Base diameter parameter

$$IF = \frac{\rho_{Average-Parameter1}}{\rho_{Average-Parameter1}} = \frac{0.45}{0.45} = 1.00$$

2. Number of drums parameter

$$IF = \frac{\rho_{Average-Parameter2}}{\rho_{Average-Parameter1}} = \frac{0.32}{0.45} = 0.71$$

3. Seismic zone parameter

$$IF = \frac{\rho_{Average-Parameter3}}{\rho_{Average-Parameter1}} = \frac{0.37}{0.45} = 0.82$$

It should be noted that the importance factor determination is implemented only on the parameters examined numerically. Further research is also needed to refine the calculation of the importance/participation factor of each parameter.

## 3.4 VULNERABILITY INDEX CALCUALTION - EXAMPLES

As example of the rating methodology the vulnerability indexes for the reference model and models 2 to 7 are calculated in Table 3.9. The calculation is based on the following expression:

$$VI_{total} = r_{Parameter1} * IF_{Parameter1} + r_{Parameter2} * IF_{Parameter2} + r_{Parameter3} * IF_{Parameter3}$$

,

,where

- $VI_{total}$ is the vulnerability index of the column
- $r_{Parameter1}$ and $IF_{Parameter1}$ is the rating and the importance factor of the first parameter (size effect)
- $r_{Parameter2}$ and $IF_{Parameter2}$ is the rating and the importance factor of the first parameter (number of drums)
- $r_{Parameter3}$ and $IF_{Parameter3}$ is the rating and the importance factor of the first parameter (seismic zone)

| No | Model | Size effect (IF=1.00) | | Number of drums (IF=0.71) | | Seismic Zone (IF=0.82) | | $VI_{total}$ |
|---|---|---|---|---|---|---|---|---|
| | | Base diameter [m] | Rating | Number | Rating | Category | Rating | |
| 1 | Reference model | 1.00 | 2.30 | 5 | 2.50 | ZII | 1.65 | 2.30*1.00+2.50*0.71+1.65*0.82=**5.43** |
| 2 | 1st Alteration – Smaller Base diameter | 0.80 | 1.65 | 5 | 2.50 | ZII | 1.65 | 1.65*1.00+2.5*0.71+1.65*0.82=**3.13** |
| 3 | 1st Alteration – Larger Base diameter | 1.50 | 3.00 | 5 | 2.50 | ZII | 1.65 | 3.00*1.00+2.5*0.71+1.65*0.82=**6.13** |
| 4 | 2nd Alteration – Fewer number of drums | 1.00 | 2.30 | 1 | 1.00 | ZII | 1.65 | 2.30*1.00+1.00*0.71+1.65*0.82=**4.36** |
| 5 | 2nd Alteration – Increased number of drums | 1.00 | 2.30 | 10 | 3.00 | ZII | 1.65 | 2.30*1.00+3.00*0.71+1.65*0.82=**5.78** |
| 6 | 3rd Alteration - Lower seismic zone | 1.00 | 2.30 | 5 | 2.50 | ZI | 3.00 | 2.30*1.00+2.50*0.71+3.00*0.82=**4.23** |
| 7 | 3rd Alteration - Higher seismic zone | 1.00 | 2.30 | 5 | 2.50 | ZIII | 1.03 | 2.30*1.00+2.50*0.71+1.03*0.82=**4.91** |

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