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EC8-BASED SELECTION AND SCALING OF ACCELEROGRAMS FOR ASSESSMENT OF THE RESPONSE OF A 5-STORY, IRREGULAR R/C BUILDING

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Abstract. *This study focuses on the assessment of the selection process of real records on the basis of EC8 provisions, through the performance of nonlinear dynamic analyses of a multi-storey R/C building, which was damaged during the Lefkada earthquake in 2003. The building was modelled and studied in the elastic and inelastic range with alternative finite element programs (i.e., Zeus-NL, ETABS and ANSYS) by considering R/C member nonlinearity and the presence of soil. The seismic response of the building is quantified through a displacement-based damage index, denoted as DCR (demand-to-capacity ratio). Five different sets of seven pairs of horizontal components of strong ground motion are selected from available databases from Europe, the Middle-East and the U.S., in compliance with corresponding EC8 (Part 1 and Part 2) guidelines. The results of the extensive parametric analyses performed permit quantification of intra - bin scatter of the building damage and also highlight the limitations of Eurocode 8 provisions. The paper concludes with specific recommendations that aim at eliminating the dispersion of the inelastic response results though appropriate modifications of the EC8-proposed selection parameters.*

1 INTRODUCTION

Among all types of dynamic loads that are imposed on conventional structures during their lifetime, earthquake-induced ground excitation seems to be the most complex and unpredictable [1]. Ground motions are random in space and time, mainly resulting from the enormous complexity of the path that seismic waves travel through, from the fault-plane source to the bedrock and then through soil layers (if present) to the foundation level of the structures. The latter local site effects cause also modifications to the seismic motion, both in terms of frequency and amplitude [2, 3]. Given the above uncertainties, it is always difficult to predict the maximum earthquake-induced loading that a structure of interest will have to withstand over its useful lifetime and hence modern seismic codes have undertaken the responsibility to quantify the seismic exposure of specific areas and provide smooth design spectra for response spectrum and linear elastic time history analysis.

Nowadays, the computational developments permit carrying out complex nonlinear dynamic (time history) analyses for the design and assessment of almost all kinds of regular or more complex structures; however, the problem of selecting and scaling an appropriate set of earthquake records that would lead to a stable mean of structural response is still a crucial problem. Equally, the number of records required to ensure the above requirement also cannot be easily assessed in advance [4].

Some state-of-the-art methods [5, 6, 7, 8] have been proposed along these lines in order to optimize the selection and scaling process of real records but it is unlikely that these methods can be used in common practice yet. On the other hand, seismic codes take advantage of the large databases and strong-motion arrays currently available and propose the use of earthquake accelerograms that comply with general pre-defined criteria while satisfying specific spectral matching requirements. Again though, whether a stable mean or some target percentile structural response is indeed achieved is neither ensured or even measured.

As a result, the relevant seismic codes guidelines are deemed as insufficient [9]. The study presented herein investigates the feasibility of selecting real records sets on the basis of the current EC8 provisions, for the seismic assessment of an existing building in the island of Lefkada in western Greece. This particular structure was adopted as the case study not only because it was heavily damaged by a severe seismic event (Ms=6.4, 14.08.2003) but also because both an earthquake record and an in-situ soil investigation where available at its vicinity.

Performing plethora of nonlinear dynamic analyses with the use of multiple sets of selected earthquake records, the scope of this paper is to:

(a) assess the feasibility and effectiveness of earthquake selection process prescribed in Eurocode 8.

(b) quantify the record-to-record variability of structural response for different EC8 compliant selection alternatives.

(c) investigate the implications and importance, in terms of structural response, of various individual earthquake record selection criteria such as the source-to-site distance and the seismotectonic environment.

(d) assess the relative importance of different earthquake record selection criteria.

(e) propose simple improvements that could potentially reduce the scatter in structural response when the selection is made according to Eurocode 8.

The process for selecting earthquake records according to EC8 and the inelastic response of the particular building are presented in the following.

2 SELECTION OF SEISMIC INPUT FOR NONLINEAR DYNAMIC ANALYSIS ACCORDING TO EUROCODE 8

2.1 Record selection on the basis of EC8,Part1

Eurocode 8, in Part 1 [10], prescribes that the earthquake loading, required for conducting nonlinear dynamic analyses of buildings, may be defined either generated artificial or simulated acceleration time histories that are compatible to target code spectra or appropriately selected, recorded seismic motions depending on the type of structural assessment and the data available at the location of the structure. It is notable that the use of artificial records is described in more detail in EC8 compared to both the real and simulated records for which it is generally outlined that: "*the use of recorded accelerograms - or of accelerograms generated through a physical simulation of source and travel path mechanisms - is allowed, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of* $a_g S$ *for the zone under consideration* (*§3.2.3.1.3.1*)."

The sets (bins) of accelerograms that can be selected by the designer, regardless whether they are real, simulated or artificial have to satisfy the following criteria:

a) the mean of the zero period spectral response acceleration values (calculated from the individual time histories selected) has to be higher than than the value of $a_g.S$ for the site in question, in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram is applied;

b) the mean of the 5% damping elastic spectrum that is calculated from all time histories should not be less than 90% of the corresponding value of the 5% damping EC8 elastic response spectrum (*§3.2.3.1.2.4*).

c) a minimum of 3 accelerograms has to be selected in each set. When three different accelerograms are used the structural demand is determined from to the most unfavorable value that occurs from the corresponding three dynamic analyses. On the other hand, in case that at least seven different (real, artificial or simulated) records are used, the design value of the action effect E_d (*§4.3.3.4.3*) can be derived from the average of the response quantities that result from all these analyses.

Seismic motion should consist of three simultaneously acting accelerograms representing the two horizontal and the vertical component of strong ground motion; however, the same record must not be used simultaneously along both horizontal directions. The vertical component of seismic motion should only be considered if the design vertical ground acceleration for type A ground, a_{vg}, is greater than 0.25g or in other cases (*§4.3.3*) primarily dealing with long structural members and base-isolation. As a result, typically, a set of excitation records is formed only for the two horizontal components.

2.2 Record selection on the basis of EC8,Part2

It is interesting to notice that, specifically for bridges, EC8-2 [11], provides more detailed provisions compared to EC8-1 for the selection of earthquake input for linear and non-linear dynamic analysis. More specifically, it is prescribed that simulated records can only be utilized in case that the required number of recorded ground motions cannot be found. Nevertheless, despite the fact that EC8-2 shares the same spectral shapes and site classification with those demonstrated in Part 1, additional criteria are provided regarding the spectral matching criteria are provided (*§3.2.3.3*):

a) matching should be satisfied for each earthquake record considering both horizontal components through their joint SRSS spectrum, which shall be created by taking the square root of the sum of squares of the 5%-damped spectra of each component;

b) based on the above, the spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step;

c) given the fact that the ensemble spectrum for each event is inevitably higher than its components, it is required that it is not lower than 1.3 times (compared to 0.9 prescribed in Part 1 for the individual components) the 5%- damped design seismic spectrum, in the period range between $0.2T_1$ and $1.5\ T_1$, where T_1 is the fundamental period of the mode of the (ductile) bridge, or the effective period (T_{eff}) of the isolation system in the case of a base-isolated bridge;

d) record scaling is permitted, but the scale factor required in the previous step shall be uniform for each pair of seismic motion components.

It is also notable that in case where near source effects are deemed significant (*§3.2.2.3*), moderate to long bridges sensitive to the spatial variation of seismic motion (*§3.3*, *Annex D*) and bridges where the vertical component of seismic motion is important (*§3.2.3, §4.1.7*), more specific provisions compared to Part 1 are also given.

3 CASE STUDY FOR EVALUATION OF EC8-BASED EARTHQUAKE RECORD SELECTION PROCEDURES

3.1 Overview of the Lefkada earthquake

The Lefkada earthquake took place on August 14, 2003, at 8:15am local time measuring 6.4 on the Richter scale being the most powerful event in the area since 1995 which is characterized by the highest seismicity in Greece, as also reflected on the Greek Seismic Code where the current peak ground acceleration is 0.36g. The epicenter, according to Athens Geodynamic Institute, was located 8.5 miles under the sea, approximately 20 miles north-west of Lefkada Island. Four strong aftershocks of magnitudes 5.3 to 5.5 followed the main shock in a time period of 24 hours. The shock caused severe damages to (primarily reinforced concrete) buildings, roads, quay walls, water and wastewater systems. Furthermore, extensive rock falls occurred all over the island, interrupting the road network and resulting to access disruption at several locations.

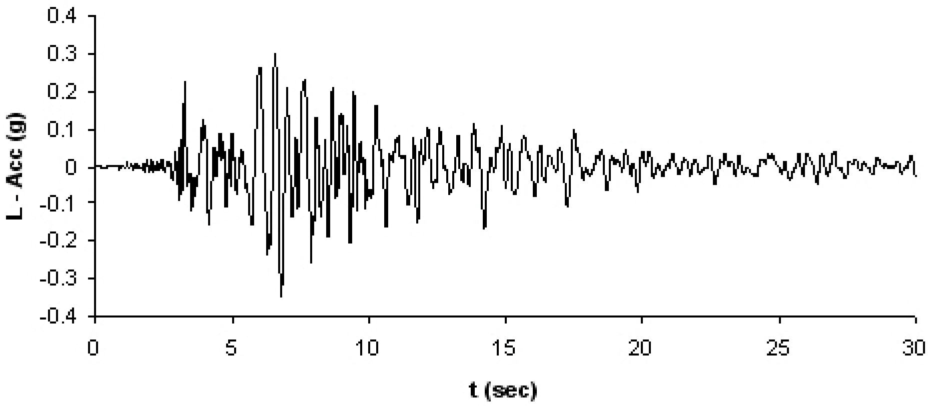
The acceleration time histories shown in Figure 1, were recorded by the permanent array of the Institute of Engineering Seismology and Earthquake Resistant Structures of Thessaloniki (ITSAK) [12] during the main shock of 14/08/2003 and clearly highlight the intensity of the earthquake a maximum horizontal ground acceleration of 0.4g was recorded.

3.2 Structural configuration and regional soil profile for the building under study.

The structure adopted to be examined in the present study constitutes a 5-storey RC building (including pilotis), located in the city of Lefkada and was heavily damaged during the particular event. This building is an interesting case study that has also been studied in the past [13] because not only was damaged but also all structural and foundation configuration plans, specific soil profile and earthquake records in its vicinity were reliably known. As a result, it offers the advantage that all numerical tools used and simulation assumptions made could be verified by first matching the numerical prediction with the observed inelastic response of the building. The structure was constructed in 1979 according to the seismic code of the time (i.e. earthquake forces corresponded to a seismic factor $\varepsilon = 0.16g$ and were applied uniformly with height as defined by the Greek Seismic Code of 1959 while member design was performed on the basis of the 1954 Reinforced Concrete Code). The building was also irregular in plan and height as can be seen in Figures 2 and 3 since the ground floor, with a height of 5.65m, was used as a super market and a loft was constructed at the back of the store at a height of 3.0m. Concrete class can be considered as equivalent to the current C16/20, while SIII steel bars were used for longitudinal reinforcement and SI for the transverse [14].

The soil conditions at the location of the structure but also in the overall bay area of the city of Lefkada is very soft (can be classified as category D to X according to EC8) [15]. In particular, based on in-situ geotechnical investigations, the superficial layer is consisting of debris to a depth of 3.5 m, followed by a layer of clay of medium to high density down to a depth of 4.6m. From 4.6m to 10.3m the soil is considered as loose, liquefiable, silty sand, followed by 1m of silt with varying percentage of loose sand and a deep layer of medium plasticity marl. Given the above conditions, the structure was designed to be supported on a set of small and dense pile groups (61 piles in total of diameter equal to $d=0.52m$ and length $L=18m$) connected with pile caps and tie beams (0.30x0.80m).

The damage observed during the 14th August 2003 earthquake [13] was mainly concentrated at the perimeter of the building and at the ground level where most columns failed in flexure, with the exception of the side short columns which exhibited shear failure as can be seen in Figure 2.



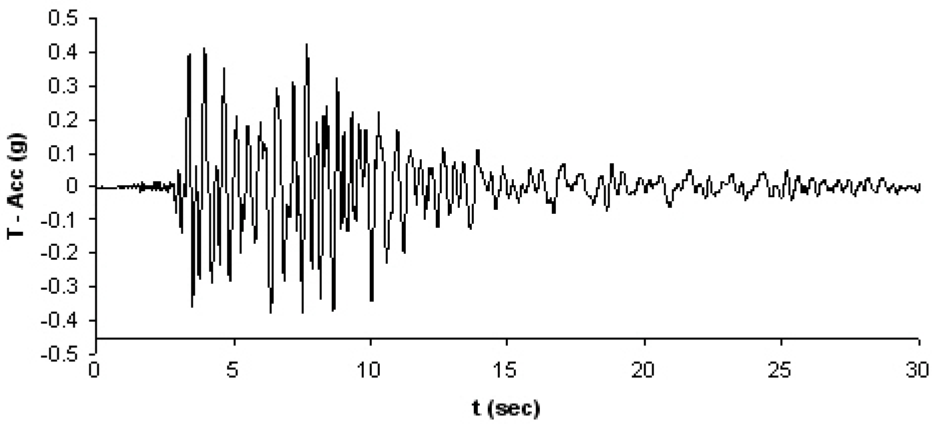


Figure 1: Longitudinal (top) and transverse (bottom) components of the recorded ground motion during Lefkada (M_s=6.4, 14/08/2003) earthquake [12].

Figure 2: The 5-storey RC building adopted as the case study.

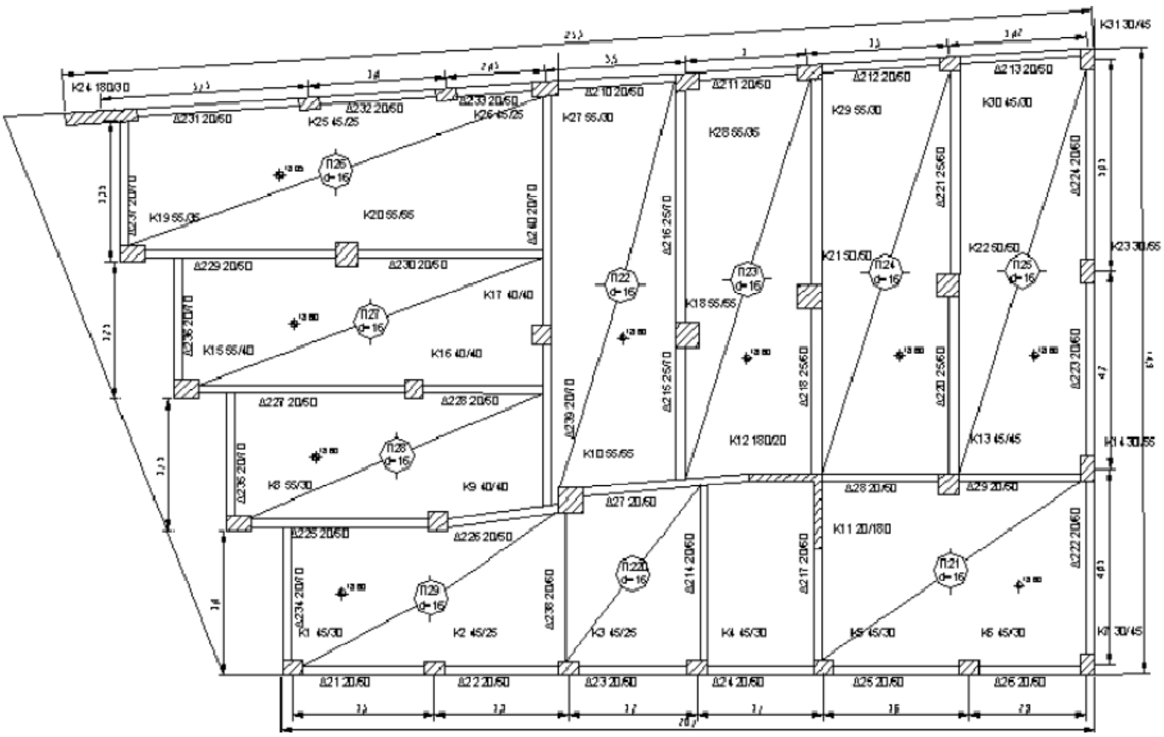


Figure 3: Plan view of a typical building storey

3.3 Numerical modeling aspects

For the assessment of the particular building for various sets (bins) of earthquake ground motion, a large number of nonlinear dynamic analyses were performed [16] using the Finite Element software, Zeus-NL [17]. As seen in Figure 4, all structural elements, such as beams, columns and walls, were modeled using the corresponding three-dimensional cubic frame elements, provided by the Zeus-NL FE library. Slabs were considered as external loads acting on the beams while the rigid diaphragm at each storey was achieved through appropriate strut connections. To obtain more accurate results from the analysis, and given the aforementioned observed damage concentration at the columns of the building ground floor, the corresponding elements where discretised into four sub-elements of unequal length (i.e. 15%, 35%, 35% and 15% respectively of the overall member length). The lumped mass element (Lmass) was used to define the lumped masses at the joints for the dynamic and eigenvalue analysis.

The complex concrete behavior under cyclic loading, residual strength and stiffness degradation and the interaction between the flexural behavior and the axial force was taken into consideration by the inherent fiber (distributed plasticity) model of the program. Based on the (steel and concrete) material stress-strain relationships, moment-curvature analysis is conducted to predict the ductility and expected member non-linear behavior under varying loads. Along these lines, two material models were used in the ZEUS-NL model of the case study building from the various available: (a) the bilinear elasto-plastic model with kinematic strain-hardening model (stl1) that was used for the reinforcement and rigid connections, and (b) the uniaxial constant confinement concrete model (conc2) that was used for the concrete. The three parameters required for the stl1 model were taken as follows: Young's modulus ($E=200000N/mm^2$), yield strength ($\sigma_y=220N/mm^2$) and a strain-hardening parameter ($\mu=0.05$). For the conc2 model, four parameters were defined: compressive strength ($f'_c=16N/mm^2$), tensile strength ($f_t=3\ N/mm^2$), maximum strain ($\varepsilon_{co}=2‰$) corresponding to f'_c, and a confinement factor ($k=1.20$) based on the model of Mander et al. [18]. Time history analysis was conducted using the Newmark integration scheme with parameters equal to $\beta=0.25$ and $\gamma=0.5$.

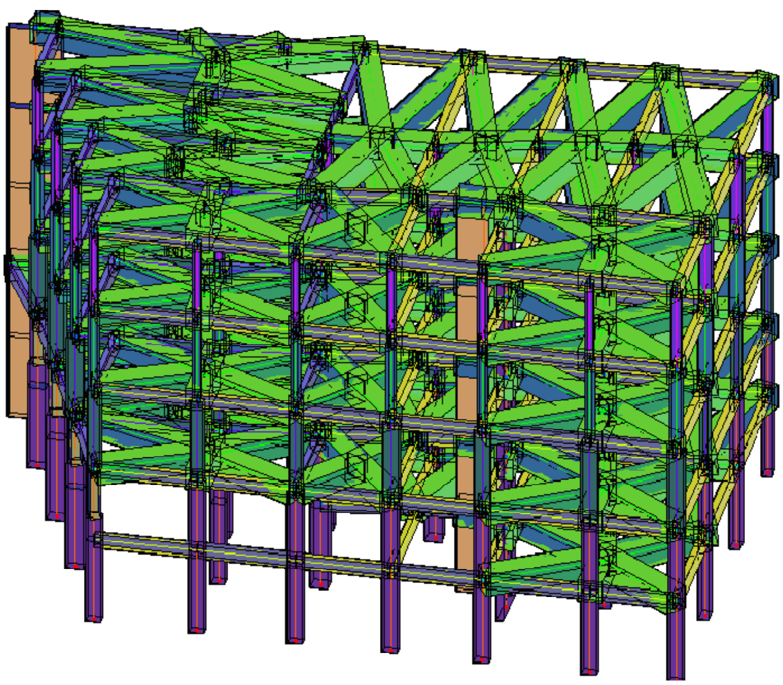


Figure 4: "Model A": 3-Dimentional finite element model of building under study (ZeusNL)

4 SOIL-STRUCTURE INTERACTION ASPECTS AND VALIDATION OF THE REFERENCE FINITE ELEMENT MODEL

Given the soft soil conditions at the location of the structure that were described previously but also being aware of the high computational cost associated with the non-linear time history analysis of the overall soil-structure system, alternative finite element models of increasing soil modeling refinement were developed. The aim was to decide, based on the predicted dynamic characteristics of the four models, whether it was indeed necessary to account for soil compliance in the reference finite element model whose inelastic response was to be assessed for various sets of accelerograms selected according to EC8 procedures outlined in section 2.

Along these lines, apart from “Model A”, the 3-D, fixed-base, frame model developed using Zeus-NL as described previously, three additional models were developed:

“Model B”: a fixed-base, spatial frame model using the finite element program ETABS [19], essentially identical to the first one (with the exception of the shear wall modeling using 2D shell elements and the representation of the short columns formed by the presence of masonry infill) and created solely for validation purposes,

	Period (sec)			
Mode	“Model A” ZeusNL fixed-base	“Model B” ETABS fixed-base	“Model C” ETABS spring- supported piles	“Model D” ANSYS 3Dsoil+piles
1st	0.539	0.527	0.584	0.693
2nd	0.439	0.433	0.505	0.624
3rd	0.401	0.395	0.455	0.573
4th	0.173	0.180	0.196	0.233
5th	0.134	0.141	0.164	0.197
6th	0.126	0.128	0.158	0.183

Table 1: Dynamic characteristics of the four alternative finite element models developed in order to identify the importance of soil compliance.

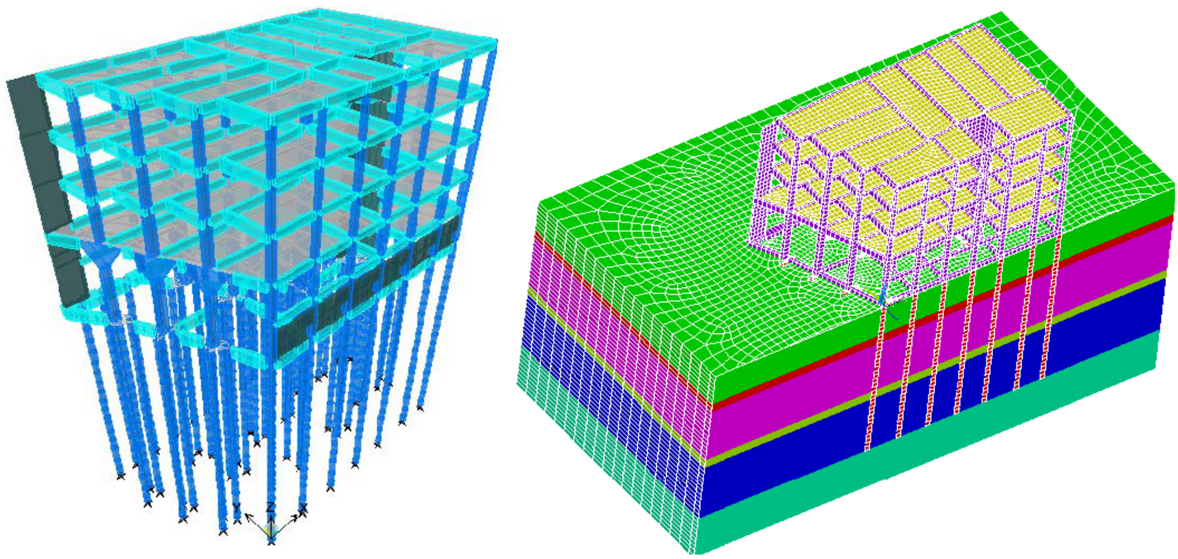


Figure 5: “Model C”: 3-Dimentional finite element model of building under study (developed in ETABS) considering both the superstructure and the spring-supported pile foundation [13] (left) and “Model D”: cross section of the 3-Dimentional finite element model of building (developed in ANSYS), considering the superstructure, the pile foundation and the layered subsoil [22] (right).

“Model C”: an extension of the latter model, where the pile group foundation is additionally modeled using length-dependent horizontal Winkler-type springs [20] in the two horizontal directions with properties considering for both stiffness reduction and damping increase at the layers exhibiting liquefaction [13] (see Figure 5, left) and

“Model D”: a refined 3-D model developed with the use of the general purpose finite element program ANSYS [21], considering the ‘exact’ soil stratification after appropriate modification of their geotechnical properties as a result of a separate site response analysis, again considering soil liquefaction at particular layers (see Figure 5, right).

Table 1 summarizes the first six periods, derived by modal analysis for each one of the aforementioned models. The results indicate, as anticipated, absolute agreement between the fixed-base models (“A” and “B”), thus establishing a first level of confidence at least with respect to the simulation of the elastic response of the building for the case of fixed-base conditions. From the first two models, it is clearly observed that the fundamental mode of the structure is primarily torsional due to the lack of adequate shear walls, the irregularity in plan (i.e. presence of the loft) and the distance between the center of stiffness and mass.

Comparing the fixed-base models (“A” and “B”) with the two flexibly-based ones (“C” and “D”), it is seen that consideration of soil compliance leads to a fundamental period elongation of the order of 10% to 25% for the case of spring-supported piles and 3D soil modeling respectively. A first comment that can be made is that the 3-Dimentional representation of the subsoil volume diverges from the Winkler-type solution, a fact that can be attributed to the well known, inherent difficulty to obtain compatibility between the modulus of elasticity of the soil solids and the spring parameters considered in the case of laterally supported piles, as also discussed elsewhere [23]. Secondly, it can be stated that if one considers that the 3-D soil-structure system is the most refined, the effect of soil compliance is non-negligible compared to the fixed-base case, at least in terms of its dynamic characteristics; this is also anticipated given the soft soil profile and especially the reduction of soil stiffness at particular layers due to liquefaction (also introduced in the finite element model based on the information of stand alone, liquefaction-dependent site response analysis).

On the other hand, consideration of soil does not affect the sequence of vibration modes of the fixed-base system (i.e. the torsional vibration mode remains fundamental and dominant, while the order of the higher modes also remains unaffected). Moreover, “Model D” is related to significantly higher computational cost compared to “Model A”, without providing equal refinement with regard to modeling of the reinforced concrete behavior under cyclic loading (i.e. use of the built-in concrete material and element Solid65 would require 3D modeling of the building as well while its numerical stability in transient analysis is rather subjective).

Independently from the above and for all practical purposes of the present paper, which aims at investigating the (relative) sensitivity of the inelastic response quantities of the building for various sets of selected accelerograms, it is deemed that “Model A” not only can be used as the reference model but this decision has the additional advantage that the dynamic characteristics of the building will be explicitly affected by the concrete sections yielding only and not by the flexibility of soil. As such, the potential scatter in structural demand that can result from the earthquake records selection and scaling process can be isolated from the coupling effect of soil-structure interaction and be studied more efficiently.

5 QUANTIFICATION OF DAMAGE

The performance of a building under earthquake loading and its related damage cannot be assessed solely on the basis of structural demand. For this reason, numerous (local) Damage Indices have been proposed in the literature, essentially relating demand with member capacity. These Damage Indices may be generally sub-divided into three groups: non-cumulative, cumulative, and combined [24] depending on the response parameters that are used, i.e. the maximum deformation, the hysteric behavior or fatigue, and the deformation and energy absorption respectively, each one presenting its own advantages in terms of robustness and computational simplicity. Due to the significant torsional sensitivity of the case study building though, conventional Damage Indices were deemed as rather insufficient to reflect 3D structural behavior and bi-directional damage. Along these lines and in order to provide a more reliable and robust damage measure for the particular case, the following demand-to-capacity ratio (DCR), proposed by Jeong and Elnashai [24] was calculated for all columns of the ground level:

$$DCR = \sqrt{\left(\frac{\Delta_x}{\Delta_{u,x}}\right)^2 + \left(\frac{\Delta_y}{\Delta_{u,y}}\right)^2} \quad (1)$$

where Δ_x and Δ_y are the inter-story drift in the x and y direction respectively, while the subscript u denotes the ultimate condition of inter-story drift which is computed individually for each column, as the drift where the column curvature reaches its ultimate value under an average value of axial force. In the case that there is yielding at both ends i, j of the column, and assuming that the inflection point of the member can be taken approximately at the middle, the inter-story drift Δ_x in the x direction can be derived as:

$$\Delta_x = {}^{h}\!/_{2} \tan(\theta_{y,i} + \theta_{pl,i}) + {}^{h}\!/_{2} \tan(\theta_{y,j} + \theta_{pl,j}) \quad (2)$$

where θ_y and θ_{pl} is the yield and plastic member rotation at the corresponding member end, while h is the height of the column. Similarly, for the extreme case that both ends reach the ultimate condition simultaneously, the ultimate interstorey drift $\Delta_{u,x}$ in the x direction is equal to:

$$\Delta_{u,x} = {}^{h}\!/_{2} \tan(\theta_{y,i} + \theta_{pu,i}) + {}^{h}\!/_{2} \tan(\theta_{y,j} + \theta_{pu,j}) \quad (3)$$

where $\theta_{p,u}$ is the plastic rotation at the corresponding end. Given that the section geometry, material properties and amount of reinforcement are typically identical at both column ends i, j it can be also considered that:

$$\theta_{y,i} = \theta_{y,j} = \theta_y \quad (4a)$$

$$\theta_{pu,i} = \theta_{pu,j} = \theta_{pu} \quad (4b)$$

hence, equation (3) can be written as:

$$\Delta_{u,x} = h \tan(\theta_y + \theta_{pu}) \quad (5)$$

Assuming linear variation of the bending moment, both the above terms θ_y and θ_{pu} can be calculated by the following expressions, provided in Annex E of EC8-2 [11]:

$$\theta_y = \frac{\varphi_y L}{3} \quad (6)$$

$$\theta_{pu} = (\varphi_u - \varphi_y) L_p \left(1 - \frac{L_p}{2L}\right) \quad (7)$$

where L is the distance from the end section of the plastic hinge to the point of counterflexure, L_p is the plastic hinge length, φ_y is the yield curvature and φ_u is the ultimate curvature at each plastic hinge of the member. It s noted that both φ_y and φ_u are determined from the moment-curvature relationship of each column using the fiber analysis computer program RCCOLA [25] for an 'average' axial load corresponds to the dead and 30% of the live load. The length of the plastic hinge, L_p, is calculated according to the expression provided in EC8 [11]:

$$L_p = 0.1L + 0.015 f_{yl} d_{bl} \tag{8}$$

where f_{yl} is the characteristic yield stress (in MPa) of the longitudinal reinforcement of the cross-section and d_{bl} is the reinforcing bar diameter. It is also noted that alternatively, the ultimate curvature can also be taken as the curvature where the strain of the confined concrete core is equal to 3‰ or the rupture strain of tension steel is 0.1 [24].

6 SELECTION OF EARTHQUAKE RECORD SETS

6.1 General criteria and spectral matching

Currently, numerous resources are available for obtaining earthquake records. A review of the available (on-line and off-line) strong-motion databases may be found in Bommer and Acevedo (2004) [26]. For the purposes of the current study records were sought in the European Strong-Motion Database (ESD) [27, 28] (http://www.isesd.cv.ic.ac.uk) and the database of the Pacific Earthquake Engineering Research Center (PEER) (http://peer.berkeley.edu/smcat/) [29] in order to be grouped in appropriate sets (bins) according to the criteria set by Eurocode 8.

Along these lines, four different sets of accelerograms (denoted hereafter as A, B, C and D), were selected. An alterative fifth set was also considered (denoted as set E), consisting of accelerograms recorded in California, U.S. Each set consisted of seven earthquake records (of two horizontal components each) from seven different seismic events.

An effort was made the records selected to correspond to the specific conditions of the building studied, that is, to (a) similar seismotechtonic conditions (typical of the shallow depth earthquakes that occur in the south-eastern Mediterranean area), (b) high peak ground acceleration of the order of 0.36g (corresponding to Zone III of the Greek Seismic Code where the island of Lefkada belongs to) as well as (3) similar very soft soil conditions.

It was found that for the particular case studied, the above criteria could not be easily satisfied simultaneously, since only few records have been recorded in the Balkans or in Italy on soft to loose soil formations and with a peak ground acceleration exceeding 0.2g. As a result, it was decided that no further specifications should be sought regarding particular source parameters (e.g. hanging/foot wall, rupture mechanism), path characteristics or strong-motion duration limitations, the latter being already a controversial criterion given the almost 40 different definitions provided in the [30]. Furthermore, the aforementioned three selection criteria were relaxed and accelerograms from all over Europe and the Middle East were considered as eligible while the restriction to match the exact soil profile was also removed.

Having tackled these eligibility aspects, 4x7=28 earthquake records of two horizontal components each, were selected appropriately (the records of set A have also been scaled) in order to match the EC8 (compulsory) quantitative criteria (a) to (c) described in detail in section 2.1. It is recalled that criteria (a) and (b) impose spectral matching between the average response spectrum of the earthquake records selected and the code prescribed spectrum. However, for the particular, irregular and torsionally sensitive building for which simultaneous bi-directional excitation was deemed necessary, it was decided that the, more detailed, matching requirements prescribed in EC8 - Part 2 [11] were used.

As a result, the SRSS response spectra of each pair of horizontal components of the selected strong-motions were derived and then, the mean spectrum of the seven SRSS-combined spectra of each set was obtained. This spectrum was finally compared with the reference 5%-damped elastic code spectrum, until the spectral acceleration of their mean SRSS-derived spectrum exceeded 1.3 times the corresponding values of the target spectrum, in the period range between $0.20T_1$ and $2.0\ T_1$, where T_1=0.539sec is the fundamental period of the reference model.

Given the high level of target peak ground acceleration (equal to 0.36g) though, and the wide range where spectral matching was required (i.e. 0.108sec $< T_1 <$ 1.08sec), only few of the earthquake records were eventually found to satisfy the above criteria, a fact that has also been pointed out by other researchers for areas of high seismicity [31]. As a result, an also in agreement with the aforementioned studies [31] the criteria were further relaxed and the target peak ground acceleration was set to 0.24g as if the structure was located in an area belonging to the seismic zone II (instead of III) according to the Greek Seismic Code. Apparently this lack of earthquake record availability for criteria for areas characterized by high seismicity is an issue that questions the applicability of the EC8 prescribed record selection process and requires further discussion. On the other hand, the use of properly scaled earthquake records of lower (compared to the target one) initial peak ground acceleration seems to be the only currently feasible solution for such cases.

6.2 Sets of selected records and mean spectra

Based on the above criteria, decisions and assumptions, four sets (A-D) of earthquake records from European seismic events and an additional set (E) from the U.S. were formed as summarized in Tables 2-6. In particular, Set A consists of 14 accelerograms (two records per seismic event), recorded mainly on soft soils, recorded on South Europe and Middle East and generally characterized by high values of PGA, a selection that is closer to the above criteria and possibly reflects the first choice of a designer for the assessment of the particular building.

In order to match the target spectrum the records were scaled down uniformly, using a common factor equal to 0.69. Sets B, C and D consist of seven pairs of horizontal components of recorded strong ground motions which were selected based on their respective epicentral (source-to-site) distance R, a criterion that is not explicitly imposed by the Eurocode 8 but is commonly adopted in many relevant studies. In particular, the records selected in Sets B, C and D are characterized by epicentral distances R≥35 km, 15≤R≤35 km and R≤15 km respectively. This distinction was deemed necessary in order to investigate the effect of distance criterion (and in turn of the seismic scenario that could possibly adopted) on the final inelastic response of the building.

The alternative Set E consists of seven pairs of horizontal components recorded in the near-field (R≤15 km) and on soft soils in the region of California, recorded on soft soil that were retrieved by the PEER database. The reason for developing such a set is to investigate the potential implications of selecting records from a completely different seismotectonic environment.

Event – Country	Date	Station Name	Magnitude	Soil	File Code
Gazli – Uzbekistan	17.05.1976	Gazli	7.04	very soft	000074
Montenegro – Montenegro	15.04.1979	Petrovac-Hotel Oliva	7.04	stiff	000196
Tabas – Iran	16.09.1978	Tabas	7.33	stiff	000187
Erzincan – Turkey	13.03.1992	Erzincan-Meteorologij	6.75	stiff	000535
Kocaeli – Turkey	17.08.1999	Duzce- Meteorologij	7.80	unknown	001226
Duzce – Turkey	12.11.1999	Bolu-Bayindirilik ve Iskan Mudurlugu	7.30	unknown	001560
Ionian – Greece	11.04.1973	Lefkada-OTE Building	5.73	soft	000042

Table 2: Selected records for the set A (ESD)

Event – Country	Date	Station Name	Magnitude	Soil	File Code
Friuli – Italy	06.05.1976	Barcis	6.50	soft	000047
Campano Lucano – Italy	23.11.1980	Mercato San Severino	6.87	soft	000289
Manjil – Iran	20.06.1990	Abhar	7.32	soft	000475
Tabas – Iran	16.09.1978	Tabas	7.33	stiff	000187
Kocaeli – Turkey	17.08.1999	Duzce- Meteorologij	7.80	unknown	001226
Duzce – Turkey	12.11.1999	Bolu-Bayindirilik ve	7.30	unknown	001560
Spitak – Armenia	07.12.1988	Gukasian	6.76	soft	000439

Table3: Selected records for the set B (ESD)

Event – Country	Date	Station Name	Magnitude	Soil	File Code
Gazli – Uzbekistan	17.05.1976	Gazli	7.04	very soft	000074
Ionian – Greece	11.04.1973	Lefkada-OTE Building	5.73	soft	000042
Alkion – Greece	24.02.1981	Korinthos-OTE	6.69	soft	000333
Campano Lucano – Italy	23.11.1980	Sturno	6.87	rock	000290
Kocaeli – Turkey	17.08.1999	Yarimca-Petkim	7.80	unknown	001257
Friuli – Italy	06.05.1976	Tolmazo-Diga	6.50	rock	000055
Montenegro - Montenegro	15.04.1979	Petrovac-Hotel Oliva	7.04	stiff	000196

Table4: Selected records for the set C (ESD)

Event - Country	Date	Station Name	Magnitude	Soil	File Code
Umbro-Marchigiano – Italy	26.09.1997	Colfiorito	5.50	stiff	000591
Dinar – Turkey	10.01.1995	Dinar-Meteorologij	6.07	soft	000879
Kocaeli – Turkey	17.08.1999	Izmit- Meteorologij Istasyonu	7.80	unknown	001231
Kalamata – Greece	13.09.1986	Kalamata-OTE Building	5.75	stiff	000414
Duzce – Turkey	12.11.1999	Duzce-Meteorologij	7.30	unknown	001703
Erzincan – Turkey	13.03.1992	Erzincan-Meteorologij	6.75	stiff	000535
Ionian - Greece	11.04.1973	Lefkada-OTE Building	5.73	soft	000042

Table5: Selected records for the set D (ESD)

Event – Country	Date	Station Name	Magnitude	Soil
Coyote Lake - California, U.S.A.	06.08.1979	47380 Gilroy Array #2	5.70	soft
Imperial Valley - California, U.S.A.	15.10.1979	955 El Centro Array #4	6.50	soft
Loma Prieta - California, U.S.A.	18.10.1989	47125 Capitola	6.00	soft
Superstitn Hills - California, U.S.A.	24.11.1987	01335 El Centro Imp. Co. Cent	6.70	soft
Westmorland - California, U.S.A.	26.04.1981	5169 Westmorland Fire Station	5.80	soft
Northridge - California, U.S.A.	17.01.1994	0655 Jensen Filter Plant	6.70	soft
Morgan Hill - California, U.S.A.	24.04.1984	57382 Gilroy Array #4	6.20	soft

Table6: Selected records for the set E (PEER Database)

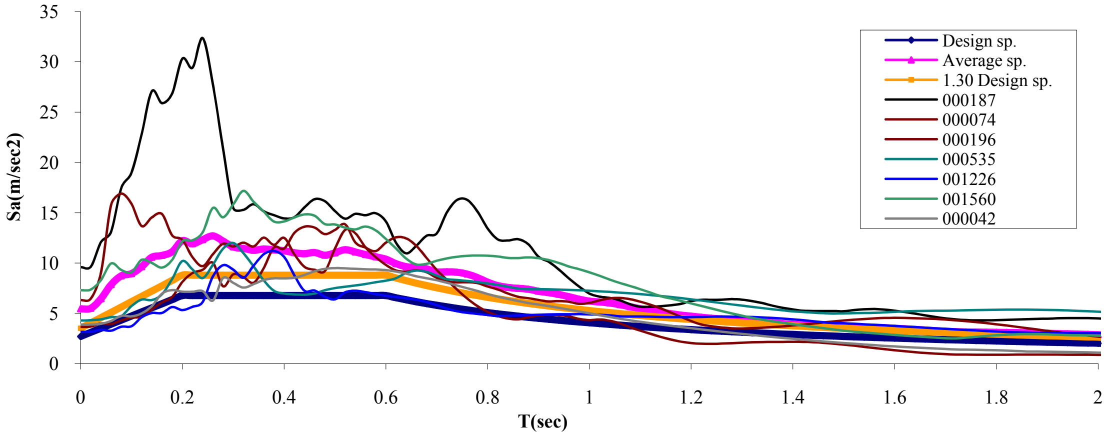


Figure 6: Response, average and design spectra, calculated for soil type C and a_g=0.24 g, for set A records.

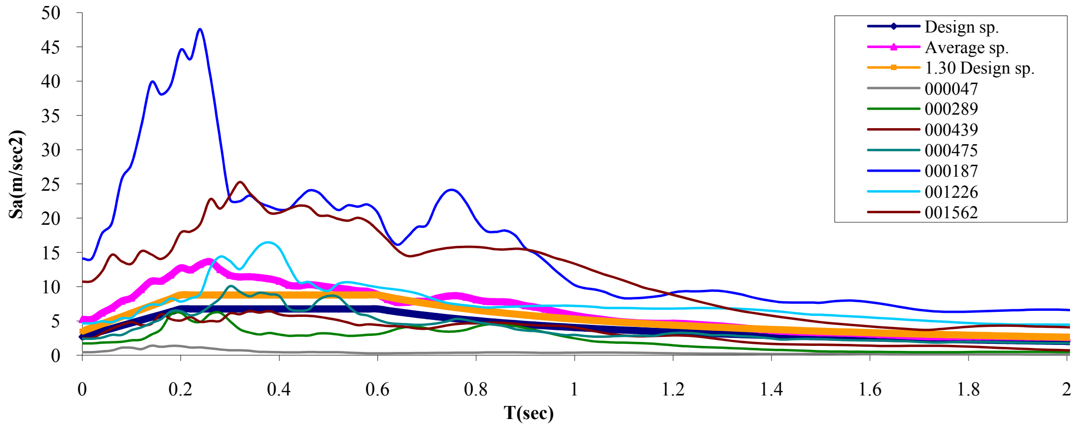


Figure 7: Response, average and design spectra, calculated for soil type C and a_g=0.24 g, for set B records.

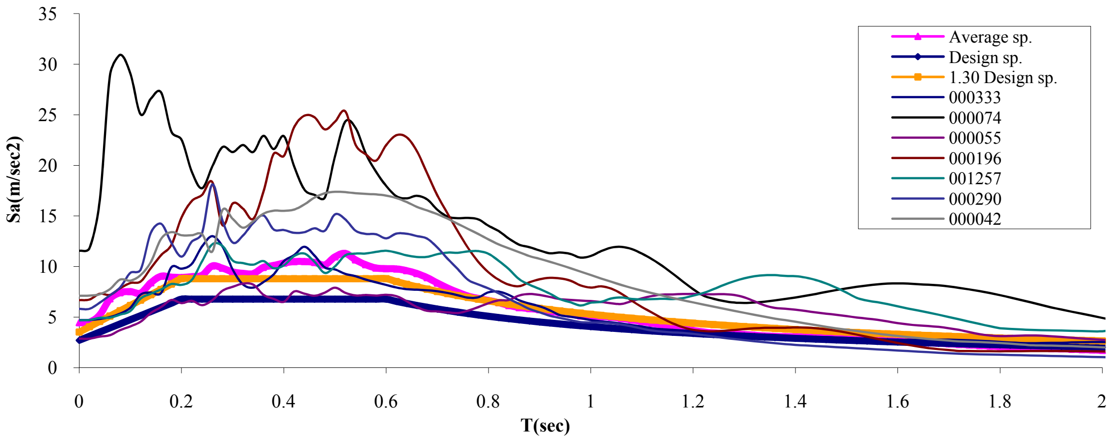


Figure 8: Response, average and design spectra, calculated for soil type C and a_g=0.24 g, for set C records.

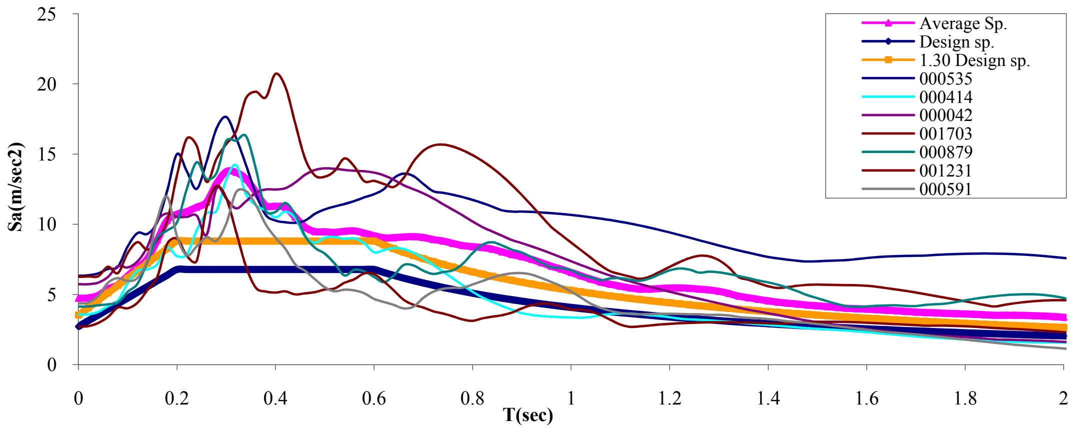


Figure 9: Response, average and design spectra, calculated for soil type C and a_g=0.24 g, for set D records.

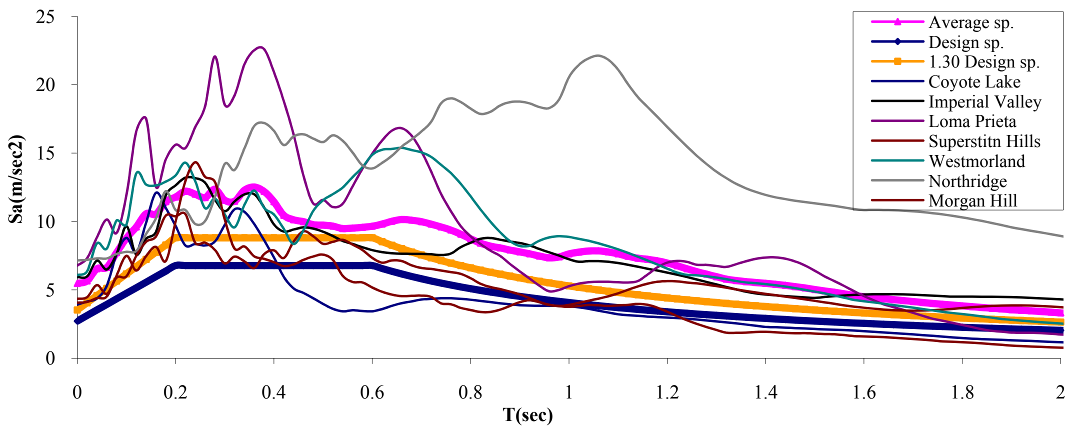


Figure 10: Response, average and design spectra, calculated for soil type C and a_g=0.24 g, for set E records.

In particular, surface faults are common in western United States thus leading to large amplitude surface waves and an increased frequency content of earthquake motions in the long-period range, unlike the case of Mediterranean regions [32]. Figures 6-10 illustrate the SRSS spectra of the seven pairs of seismic records,their corresponding mean spectrum in comparison to the target Eurocode 8 spectrum for Sets A, B, C, D and E.

As can be seen from Figures 6-10, the mean spectra of all Sets indeed satisfy the EC8 provisions about spectral matching as they exceed 1.3times the target spectrum at all periods in the range 0.108sec < T_1 < 1.08sec. It is interesting to notice though, that in case that the target PGA criterion was not matched (i.e. seismic zone was retained as III and a_g=0.36g), none of the above mean spectra would meet this requirement. It is also notable that with the exception of Set A, no scaling is performed in order to avoid the bias possibly induced to the structural seismic response [33]. By studying Figures 6-10 more thoroughly it can be also commented that almost in all cases, it was necessary to include an earthquake record of significant spectral accelerations, primarily in order to meet the spectral matching requirement at longer periods (close to $2.0 \cdot T_1$).

The result of this almost inevitable decision is to have at least one pair of horizontal components (one earthquake record) that could possibly lead to strong inelastic behavior of the building, at least compared to the structural response that result from the application of the other six records, thus questioning the overall rational of 'averaging' the action effects of the building obtained partially obtained from elastic and partially from inelastic structural response under the seven records of a given set. It is therefore deemed necessary to examine further the required range of spectral matching especially towards longer periods and the threshold value of $2.0 \cdot T_1$ bearing in mind that in most cases, the fundamental period T_1 of the structure is not expected to be doubled unless the latter is subjected to very high seismic forces and suffers subsequent structural damage. It has to be noted herein, that the presence of soft soil and foundation compliance should not be confused with period elongation *during* seismic excitation since the flexibility of the soil-structure system inherently affects the *initial* fundamental period of the structure T_1, prior to and independently of earthquake loading.

7 NONLINEAR DYNAMIC ANALYSIS RESULTS

Utilizing the 5x7=35 selected earthquake records, bi-directional non-linear dynamic analysis of the building under study was performed, with the finite element program Zeus-NL and the damage was assessed through the *DCR* index given in eq.1. Figures 11-15 illustrate the demand-to-capacity ratio (DCR) values for some characteristic columns and the shear walls of the ground floor for all analyses with the above sets of records. It is recalled the damage initiates since the interstory drift is greater than the interstory drift which corresponds to yield conditions in either x or y direction. A first general observation is that in all Sets (A to E) the intra-bin scatter (that is, the coefficient of variation of the DCR values for a given column and for the seven earthquake records of the same set) is certainly non-negligible (see Table 7). This scatter is more profound in Set B (near filed motions from European earthquake events, Figure 12) where for all ground floor columns studied, the coefficient of variation of the DCR values exceeds 0.623. On the contrary, selection based on the most commonly adopted criteria, such as those of Set A lead to lower, but still noticeable, intra-bin scatter (the maximum COV among all columns is 0.394). This overall scatter, is believed to be attributed to the adverse effect of the aforementioned very strict criterion impose by EC8 to obtain structural matching at long periods up to $2.0 \cdot T_1$ and the implication of specific strong records that are used in order to match this criterion.

Set	min COV	max COV
A	0.159	0.394
B	0.623	0.879
C	0.106	0.491
D	0.204	0.489
E	0.243	0.899

Table 7: Maximum and minimum values of *COV* of all examined columns for the seven records of each earthquake record sets

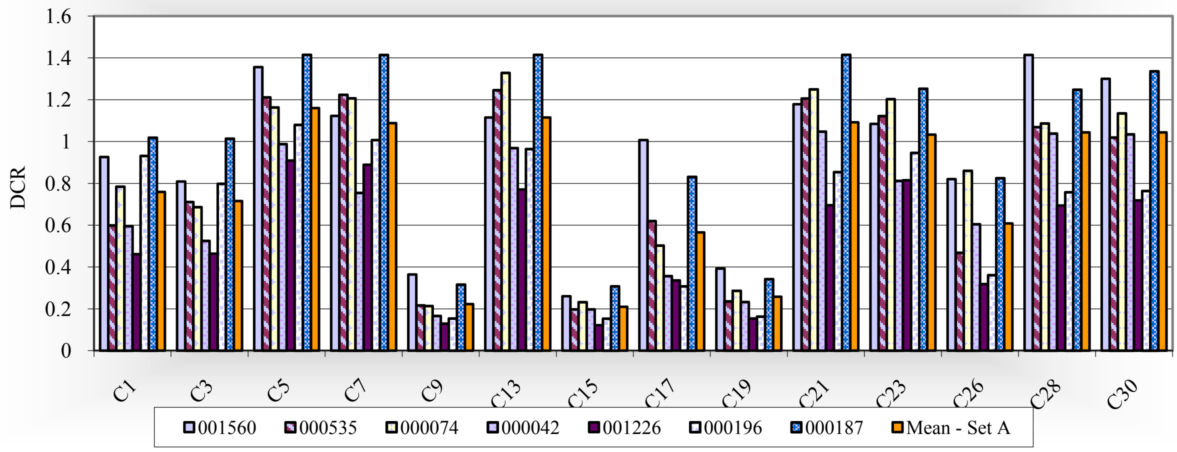


Figure 11: DCR values of characteristic structural members at the ground level, computed for excitations with the use of the earthquake records of Set A.

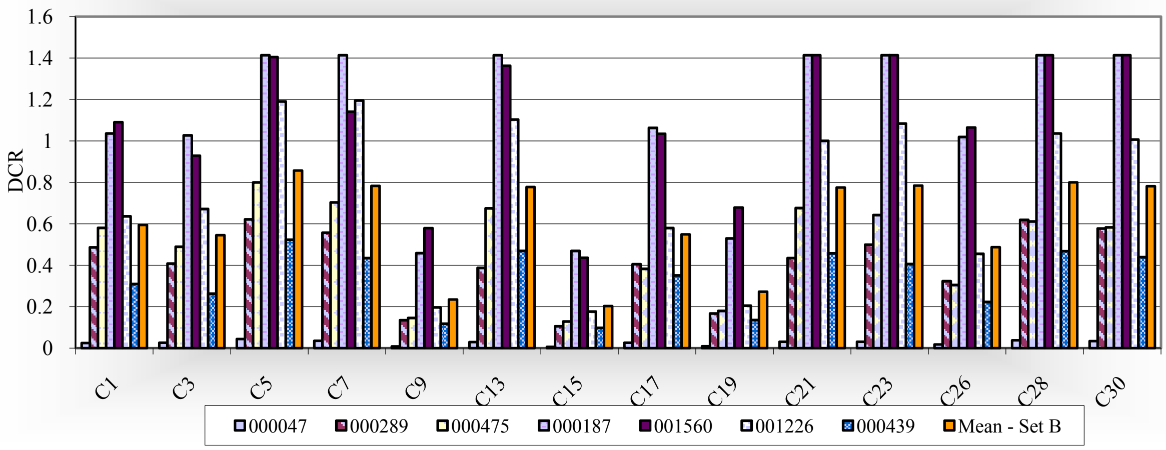


Figure 12: DCR values of characteristic structural members at the ground level, computed for excitations with the use of the earthquake records of Set B.

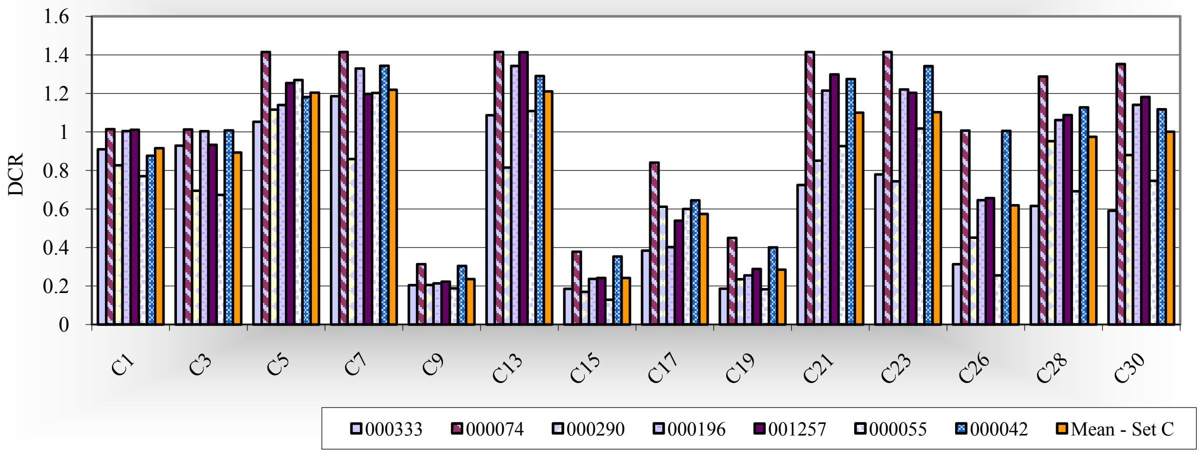


Figure 13: DCR values of characteristic structural members at the ground level, computed for excitations with the use of the earthquake records of Set C.

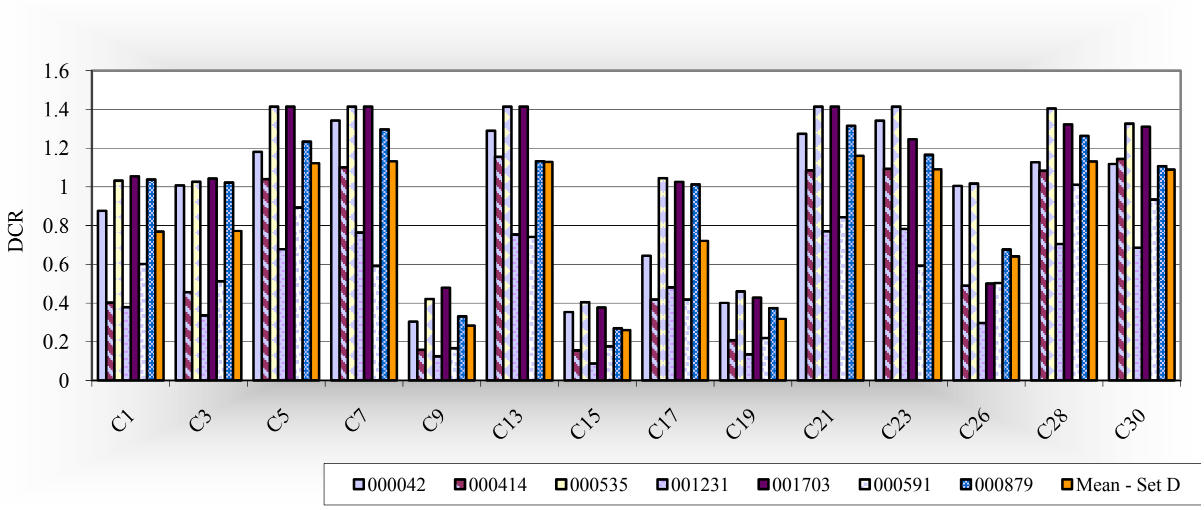


Figure 14: DCR values of characteristic structural members at the ground level, computed for excitations with the use of the earthquake records of Set D.

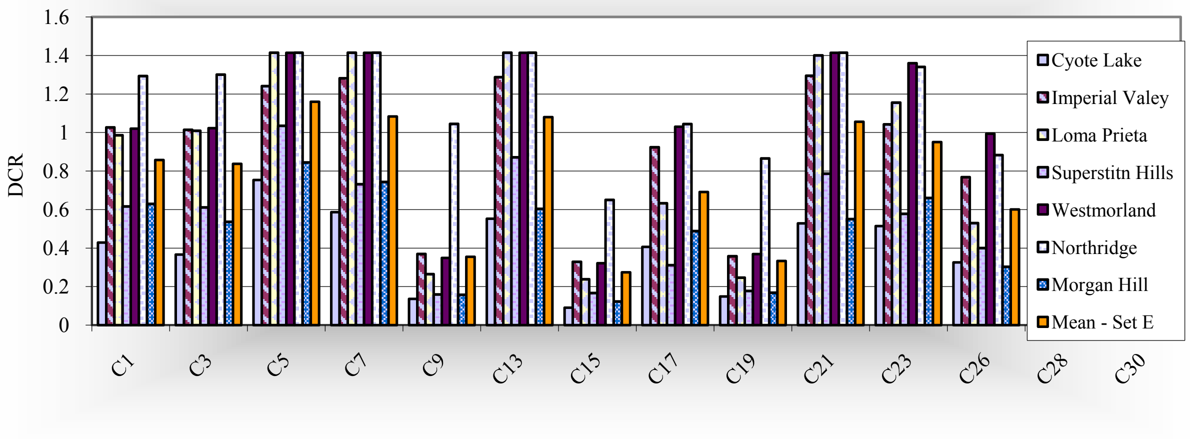


Figure 15: DCR values of characteristic structural members at the ground level, computed for excitations with the use of the earthquake records of Set E.

The particular observation also implies that the main purpose of earthquake record selection, which is to form a set of ground motions that would ideally lead to the same inelastic seismic response is not met using the EC8 selection procedure at least for the case of irregular buildings, founded on soft soils and located in areas of high seismicity.

It is also notable that if the designer had decided to form a set consisting of only three earthquake records and sought to obtain the maximum structural response from these three analyses (a decision that is also permissible according to EC8), then the significantly stronger earthquake record required to achieve spectral matching along such a wide period range would not only affect the intra-bin scatter but essentially dominate the maximum structural response, thus leading to unrealistically predicted member forces and displacements.

Furthermore, by focusing on the both the DCR coefficients of variations of Sets B, C and D and the absolute values of DCR, it is seen that the differences in terms of inelastic response of columns for the different sets of records are significant (for instance, for the case of column C13, the DCR value, calculated for records of set B, is equal to 0.77, while for records of set C it is equal to 1.21. The corresponding value, estimated for records of set D is found equal to 1.11. This implies that the response, as derived by one set is inelastic but when computed by another set is purely elastic). As a result, it is deemed that the decisions made with regard to the seismic scenario adopted (as expressed by the anticipated source-to-site distance R) play an important role and have to be made carefully.

Finally, the comparison of Sets D and E (essentially reflecting near field earthquake motions on soft soil in Europe and the U.S. respectively) also yields to important differences in terms of final inelastic response of the building. It can be therefore stated that using accelerograms recorded in a different seismotectonic environment has to be made with appropriate caution and only in cases when no other records are available; an observation also made by other researchers [32].

8 CONCLUSIONS

This study aims to quantify the effect of earthquake record selection strategy on structural response, according to Eurocode 8 provisions, on the structural performance of an existing, 5-storey irregular building, damaged during the Lefkada, Greece earthquake of 2003. The main conclusions are as follows:

- The number of records that can be retrieved from current strong-motion databases to fulfill the selection requirements imposed by EC8 (general criteria and spectral matching) in case of structures founded on soft soils and located in areas of high seismicity is very limited and more detailed guidelines should be provided to aid the designer with this particular problem.
- Even for low levels of seismicity (i.e., PGA=0.24g) the intra-bin scatter of the inelastic structural response of a building can be significant. As a result, at least for the particular case studied, the main objective of selecting and scaling real accelerograms in a way that they can form a set of ground motions which not only satisfy the expected seismic scenario but at the same time they infer the same inelastic response (in terms of mean or some target percentile response), which would be estimated if the structure was analyzed with a large set of "suitable" ground motions, cannot be met [4].
- It is thought that this scatter cannot be attributed to the selection process proposed by EC8 as a whole, but rather to the wide period range for which spectral matching is imposed (i.e., $0.2T_1 < T_1 < 2.0T_1$). This particular requirement leads to a selection of at least one record with high spectral accelerations at long periods to 'correct' the mean spectrum of the selected earthquake record set with respect to the target one, but in turn this yields unrealistic structural response. It is noted that the above conclusions are generally observed even if the upper bound for spectral matching was set to 1.5T1 according to EC8-2.
- The use of a dominating 'correcting' earthquake record also results in higher scatter of the structural response quantities among the seven records used and questions whether an *average* value can indeed be used in design, or since some records correspond to elastic and others to inelastic response.
- Based on the above, the range of spectral matching of target and mean spectrum of the seven individual earthquake records should be limited to $0.2T_1 < T_1 < 1.3T_1$. It is also noted that this is the proposed matching range for bridges according to EC8-2. Ideally, the upper bound of this range could be a function of seismic zone, since period elongation is directly related to structural yielding and on the level of seismic forces.
- By comparing the structural response obtained from different sets of records, it was shown that, in case of applying the EC8 selection strategy, the decisions made in regard to the seismic scenario adopted (as expressed by the anticipated source-to-site distance R) or to the use of accelerograms from various parts of the world have to be made with caution.

The above conclusions cannot be generalized since they have been drawn from a limited set of nonlinear dynamic analyses of a single building in a given area. Further study has to be made for various seismic zones, seismic scenarios, soil conditions and other types of structures in order to be able to confirm the conclusions reported here.

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