ICONHIC 2016
1st International Conference on Natural Hazards & Infrastructure
28-30 June, 2016, Chania, Greece

RELIABLE SELECTION OF EARTHQUAKE GROUND MOTIONS FOR PERFORMANCE-BASED DESIGN

E. I. Katsanos
Technical University of Denmark

A. G. Sextos[1]
University of Bristol & Aristotle University of Thessaloniki

ABSTRACT

A decision support process is presented to accommodate selecting and scaling of earthquake motions as required for the time domain analysis of structures. Prequalified code-compatible suites of seismic motions are provided through a multi-criterion approach to satisfy prescribed reduced variability of selected Engineering Demand Parameters. Such a procedure, even though typically overlooked, is imperative to increase the reliability of the average response values, as required for the code-prescribed design verification of structures. Structure-related attributes such as the dynamic characteristics, as well as criteria related to the seismic motions variability and their compliance with a target spectrum are quantified through a newly introduced index, $\delta_{sv\text{-}sc}$, tailored to prioritize motions suites for the response history analysis. An actual multi-story building is used to demonstrate the efficiency of the method, by being subjected to numerous suites of motions that were highly ranked according to both the proposed approach ($\delta_{sv\text{-}sc}$) and the conventional index (δ_{conv}), already used by most existing code-based earthquake records selection and scaling procedures. The findings reveal the superiority of the herein proposed multi-criterion approach, particularly in terms of extensively reducing the intra-suite response variability of ground motions, while at the same time increasing the reliability of the design values. They also demonstrate that the new index, greatly reduces the size of the suite of selected ground motions, for a given level of target reliability, with respect to the conventional methods.

Keywords: selection of earthquake motions, response-history analysis, R/C buildings, structural response variability.

INTRODUCTION

In contrast to the past when elastic static or response spectrum analyses were widely used for the seismic design and assessment of structures, response history analysis (RHA) is nowadays emerged as the most prevalent process for linear or nonlinear structural analysis. Particularly in the latter case, it constitutes a rigorous method that captures the hierarchy of failure mechanisms, the energy dissipation and force-redistribution phenomena. Such a time domain analysis requires as input, the use of a suite of appropriately selected and scaled earthquake motions being consistent with a predefined seismic scenario. Research has shown that among all possible uncertainty sources stemming from structural and soil material properties, modeling approximations, the design and analysis assumptions as well as the earthquake-induced ground motion, the latter yields the highest effect on structural response (Elnashai and McClure, 1996; Padgett and Desroches, 2007). Since 1990's, various techniques have been developed to address the complex problem of selecting and scaling earthquake ground motions (Katsanos, Sextos, and Manolis, 2010). From an objective

[1] Corresponding Author: A.G. Sextos, *Aristotle University of Thessaloniki & University of Bristol,* asextos@civil.auth.gr & a.sextos@bristol.ac.uk

point of view, most of the ground motion selection and scaling procedures aim to determine either the central estimate (i.e., mean or median) of the structural response or the full probability distribution (i.e., median response and standard deviation) of an appropriately chosen engineering demand parameter, *EDP*, (e.g., element forces and deformations, interstory drift). The rationale to calculate a central tendency is directly related to the code-based design verification of structures where stable estimates of the average structural response have to be achieved to ensure the reliability of the design values (Hancock, Bommer, and Stafford, 2008). On the other hand, when the seismic *performance* of existing structures is evaluated, knowledge of the central response estimate is unlikely to be adequate and the full response distribution is required to consider, for example, the damage associated with the entire range of the structural behavior. The probabilistic, risk-based assessment (Mahoney, 2010) requires also a comprehensive evaluation of structural behavior; hence, the use of the full response distribution is dictated. Numerous seismological (i.e., earthquake magnitude, distance between the seismic source and the site of interest, fault rupture mechanism and the directivity of seismic waves), strong-motion (i.e., duration and amplitude of seismic waves) as well as site parameters (i.e., the soil conditions at the structure's site) have been employed to select ground motions (Iunio Iervolino, Manfredi, and Cosenza, 2006; Kwon and Elnashai, 2006). However, the concurrent application of multiple selection criteria may significantly restrict the available number of earthquake records (Bommer and Acevedo, 2004). Thus, a balance has to be preserved between the selection criteria applied and the seismic motions required for the RHA. To compromise the above, most of the current state-of-the-art methods designate earthquake magnitude, M, and source-to-site distance, R_s, as the criteria for the preliminary selection of seismic motions. These seismological parameters are familiar to the structural engineers while they can be readily obtained either by deterministic seismic hazard analysis, *SHA*, or by disaggregating the probabilistic SHA (Bazzurro and Cornell, 1999).

Once the strong ground motions have been selected from an earthquake records archive, the records most compatible with a predefined target spectrum are used for structural analysis. The Conditional Mean Spectrum (CMS) (Baker, 2011) and the related Conditional Spectrum (CS) be used as target spectra, being the most prevalent alternatives to the Uniform Hazard Spectra (UHS). The latter that serves the basis to define the smooth code spectra and assumes equal probability of exceedance of spectral accelerations along the entire period range. Independently on the target spectrum adopted, several methods have been developed to modify the ground motions and achieve matching with the reference spectrum. A basic method is to scale the amplitude of ground motions in order to establish the required compatibility between the ground motion records' response spectrum and the target one. Various metrics have been employed to quantify this spectral compatibility (Beyer and Bommer, 2007). This type of amplitude scaling attempts preserving the inherent variability of the recorded ground motions as well as their frequency content and the spectral shape. Unbiased response results can be also derived unless extensive scaling factors (more than five or even higher - this issue is still controversial) are employed. Alternatively, the frequency content of the recorded accelerograms can be modified using techniques from stochastic or random vibration theory, e.g. (Spanos, Giaralis, and Li, 2009). In this way, artificial accelerograms are generated that match a given target spectrum for a specific period range. The reduced record-to-record variability, commonly identified for this category of spectrally matched accelerograms, enables calculating mildly-scattered response results. However, due to this artificially reduced variability, the artificial seismic records can be mainly used to determine mean (or median) response and not the full distribution. Moreover, these spectral matching techniques commonly result in accelerograms with excessive number of strong motion cycles and thus unreasonable high energy content. Finally, a systematic unconservative bias in the estimation of the mean structural response has been identified.

Given the above considerations, the scope of this study is to facilitate the RHA framework through the development of a decision support process, which provides suites of seismic motions prequalified that they will induce stable, and thus reliable, design (average[2]) response values. Along these lines, the proposed process

[2] In this manuscript, "average" is used in lieu of "arithmetic mean".

can be applied for the code-conformed design verification of buildings and bridges, since in this case stable central estimates of structural response are to be predicted. The process introduced herein may be also employed to evaluate the seismic performance under an arbitrary shaking intensity represented by a user-defined target spectrum (i.e., intensity-based assessment as defined by FEMA P-58-1). It is notable that the current decision support process is embedded into the already developed computational system called ISSARS (Katsanos and Sextos, 2013). These suites of motions are ranked by a complex system attempting to designate those that most probably lead to structural response results of limited variability, and thus, increased reliability. To achieve this target, the proposed ranking system quantifies both: (a) the spectral variability among the selected motions of each suite and, (b) the convergence between the suites average spectrum and the target one. Moreover, the dynamic characteristics of the structure studied, such as the elastic vibration periods and the inelastic ones due to the nonlinear structural behavior during the earthquake excitation as well as the modal mass participation factors are explicitly accounted within the current framework so as a structure-specific process for earthquake records selection and scaling is materialized.

Multi-criteria process for earthquake motions selection and scaling

ISSARS algorithm selects ground motions directly through the Next Generation Attenuation Strong-Motion Database, PEER-NGA. Based on preliminary selection criteria, including the earthquake magnitude, M, the epicentral distance, R, the soil conditions at the recording site and the peak ground acceleration, PGA, the eligible earthquake records are retrieved online and they are used to form alternative suites of motions that satisfy either the code-imposed or the used-defined requirements for compatibility with a target spectrum. The total number of suites, $N_{tot.suites}$, that consist of m seismic records and can be formed out of a larger group of k eligible motions, is calculated by the following factorial formula of the binomial coefficient:

$$N_{tot.\mathrm{suites}} = \binom{k}{m} = \frac{k!}{m!(k-m)!} \qquad (1)$$

It is worthwhile to mention that a popular design option for the number of records, m, per suite is seven, since this is usually the minimum required by most of the code provisions to permit the use of average response quantities as design values. For example, according to EN1998-Part 1 (CEN, 2004), when seven or more different records are selected and used for RHA, the average of the response values is considered as the design value. Otherwise, when the selected suite consists of three to six records, the design value is defined as the maximum response numerically derived. Even though larger samples of seismic motions favor, in principle, the reliability of the average (design) response estimates, the designers are often reluctant to select more than seven records due to the high computational cost. The preliminary earthquake records selection criteria precedes the amplitude scaling of seismic motions, through a scaling factor, sf_{avg}, which is employed to ensure that the average spectral values, $Sa_{avg}(T_i)$, of the scaled motions, which are included in a suite, will exceed, by a certain degree, the minimum allowable spectral ordinates of the target spectrum, $Sa_{target}(T_i)$, within a prescribed period range.

$$sf_{avg} = \left\{ \min\left(\frac{Sa_{avg}\left(T_i\right)}{a_{\min} Sa_{target}\left(T_i\right)} \right) \right\}^{-1}, \; i=1 \text{ to } N \qquad (2)$$

where α_{min} is the lower bound of the target spectrum that the suite's average spectrum has to exceed, T_i is the sample structural period and N is the size of the sample within which the prescribed period range is discretized. Normally, the quality of spectral compatibility constitutes a reasonable measure to rate the suites of motions and thus, to decide which suite(s) will be the most suitable to be used as the input motion for the RHA of the structure studied. Several indices have been proposed in order to quantify the spectral compatibility (I. Iervolino, Galasso, and Cosenza, 2010; Kottke and Rathje, 2008), most of them being similar to the one presented in eq. 3 that evaluates the convergence between the target spectrum and the average spectrum of motions suite for a specific range of periods.

$$\delta_{conv} = \sqrt{\frac{1}{\mathrm{N}} \cdot \sum_{i=1}^{N} \left(\frac{sf_{avg} \cdot Sa_{avg}\left(T_i\right) - a_{\min} Sa_{target}\left(T_i\right)}{a_{\min} Sa_{target}\left(T_i\right)} \right)^2} \text{ , } i=1 \text{ to } N \tag{3}$$

However, the use of this conventional spectral compatibility measure, δ_{conv}, as a ranking index of seismic motions is rather insufficient, since there is no allowance to prioritize those suites that most probably lead to stable enough average demand parameters. For this reason, the current study introduces a novel ranking measure for the suites of motions that can be employed as a decision support mechanism to provide motions prequalified to result, in more stable response results. To achieve this requirement, the ranking measure (or index), $\delta_{spectral\ variability - spectral\ compatibility}$ (hereafter denoted as $\delta_{sv\text{-}sc}$) proposed herein is composed by two individual secondary indices that consider: (a) the intra-suite variability of motions (that is, the variability among the spectral ordinates of a motions suite), quantified through the ranking index, $\delta_{spectral\ variability}$ (hereafter denoted as δ_{sv}), and (b) the quality of the compatibility between target and suite's average spectrum respectively, quantified through the $\delta_{spectral\ compatibility}$ ranking index (hereafter denoted as δ_{sc}). Next, the steps to calculate the dual ranking index, $\delta_{sv\text{-}sc}$, are thoroughly described.

Step 1 - Definition of the upper bound for the period range

Most of the code-based methods, related to selecting and scaling earthquake records, prescribe a period range, within which compatibility between the target spectrum and the average spectrum of the selected suite of motions is enforced. The upper bound of this period range is associated with the elongation that periods experience due to the nonlinear performance induced during the earthquake strong ground shaking. The adoption, though, of a quite large upper bound forces spectral matching in the long period range, where it is unlikely that a low-to-moderate ductility structure will ever respond. Indeed, especially in case of EN1998-Part 1, the imposed upper bound, i.e., $2T_1$, is rather extensive for several structural configurations. For example, currently designed, code-compatible R/C buildings were found to experience significantly milder first-mode period lengthening, i.e., $1.2T_1$ up to $1.5T_1$, than the code-prescribed $2T_1$ even for twice the design earthquake (Katsanos, Sextos, and Elnashai, 2014). However, such an extensive period range imposes spectral compatibility at long periods that, in turn, substantially increase the spectral ordinates of the selected records in other, more periods which are more critical for structural performance (i.e., close or lower than T_1), thus leading to over-conservative design estimates (Sextos, Katsanos, and Manolis, 2010).

Given the importance of structural yielding, previous research has associated the first-mode inelastic period, $T_{1,in}$, with the corresponding elastic period, T_1, and the force reduction factor, R_y (or behavior factor, q, in EN1998), for which the building was designed (Katsanos et al., 2014):

$$\frac{T_{1,in}}{T_1} = \begin{cases} R_y = 2: 0.288T_1^6 - 2.404T_1^5 + 7.839T_1^4 - 12.646T_1^3 + 10.624T_1^2 - 4.542T_1 + 2.037 \\ R_y = 3: 0.118T_1^6 - 0.838T_1^5 + 2.235T_1^4 - 2.954T_1^3 + 2.529T_1^2 - 1.894T_1 + 2.151 \\ R_y = 4: \text{-}0.141T_1^6 + 1.402T_1^5 - 5.325T_1^4 + 9.441T_1^3 - 7.397T_1^2 + 1.347T_1 + 2.136 \\ R_y = 5: \text{-}0.144T_1^6 + 1.402T_1^5 - 5.301T_1^4 + 9.486T_1^3 - 7.601T_1^2 + 1.429T_1 + 2.303 \\ R_y = 6: \text{-}0.153T_1^6 + 1.511T_1^5 - 5.716T_1^4 + 10.205T_1^3 - 8.236T_1^2 + 1.730T_1 + 2.306 \end{cases} \tag{4}$$

Step 2 - Definition of the lower bound for the period range

Similarly to the step described above, the accurate revisit of the lower bound for the period range is also of high importance in order to account more precisely for the higher modes effect on the structural behavior under seismic loading. Particularly, the lower bound of $0.2T_1$ of the target spectral matching period range, which is imposed by most of the current seismic codes irrespectively of the dynamic characteristics of the structure studied, is replaced herein by the vibration period of the n^{th} mode (called hereafter $T_{n,80}$), for which the cumulative modal mass participation ratio is higher than 80% for both main horizontal directions:

$$\sum_{i=1}^{n}\Gamma_{i,x} \geq 80\%\,,\ \sum_{i=1}^{n}\Gamma_{i,y} \geq 80\% \tag{5}$$

where $\sum_{i=1}^{n}\Gamma_{i,x}$ and $\sum_{i=1}^{n}\Gamma_{i,y}$ are the cumulative modal mass participation ratios calculated for the first n modes along the two main horizontal directions (i.e., x and y) of the structure studied. Based on the definition described above, the quality of the spectral compatibility is evaluated solely in the period range, where the structure is expected to respond during the seismic excitation while the low period range, which has minor contribution to the dynamic response or even it is totally irrelevant to the structure, is excluded from this calculation. It is notable that the redefined period range, $T{:}[T_{n,80},T_{1,in}]$ (Steps 1-2), is employed within the currently proposed rating system for the suites of motions that have been already formed on the basis of spectral matching requirements related either to a code or a user-defined framework respectively. In other words, the existing code prescriptions for earthquake records selection and scaling are fully satisfied by the process introduced herein, whilst the latter improves the final ranking of the already eligible records.

Step 3 - Definition of the weighting factors array

Apart from the revisited period range, a weighting factor array is introduced to refine further the proposed ranking system. The weighting factors, which are directly associated with the modal mass participation ratios, are employed to credit the spectral compatibility in period zones, where the structure is anticipated to respond based on its elastic and post-elastic dynamic characteristics. Especially, the weighting factor, w_i, corresponding to the i^{th} vibration mode, for which the associated elastic period, T_i, appertains to the previously described periods range (Steps 1-2), is calculated with following expression:

$$w_i = \sqrt{\left(\Gamma_{i,x}\right)^2 + \left(\Gamma_{i,y}\right)^2 + \left(\Gamma_{i,Rz}\right)^2} \tag{6}$$

where Γ_i is the i^{th} mode mass participation ratio corresponding to the translational degrees of freedom along the main horizontal directions of the structure (i.e., u_x and u_y related to the horizontal x-x and y-y directions) and the rotational degree of freedom around the vertical direction of the structure (r_z around the z-z direction) respectively. As a result, the proposed rating system for the motions suites accounts for period zones (inside the entire period range) with different significance for the structural performance. It is recalled that the current state-of-the-art and code requirements for spectral matching effectively prescribe uniform weighting factors of unity for the entire period range of interest. As it is shown in Fig. 1, the weighting factor, w_1, calculated using eq. 6, is considered for the entire first mode-related period zone, i.e. $[T_1,T_{1,in}]$. In general, w_i, is assigned to the period zone $[T_i,T_{i,in}]$ that corresponds to the i^{th} vibration mode while linear interpolation is applied to determine the weighting factors for the intermediate period zones, i.e., $[T_{i+1,in},T_i]$. It is clear that the definition of the modes-related period zones (ranges) requires the quantification of the elongated vibration periods and, as described above (Step 1), the inelastic first-mode period can be estimated using eq. (4). Regarding the lower periods (i.e., higher modes), $T{:}[T_{n,80},T_{in}]$, shorter elongation is expected than the one corresponding to the fundamental period. For this study, the non-fundamental elastic periods, T_{nfp}, are assumed to exhibit uniform elongation, which is determined using eq. (7) as a function of the design peak ground acceleration, a_g, typically prescribed by code provisions, according to the literature (Katsanos et al., 2014).

$$\frac{T_{nfp,in}}{T_{nfp,el}} = -0.0622a_g^2 + 0.256a_g + 1.0765 \tag{7}$$

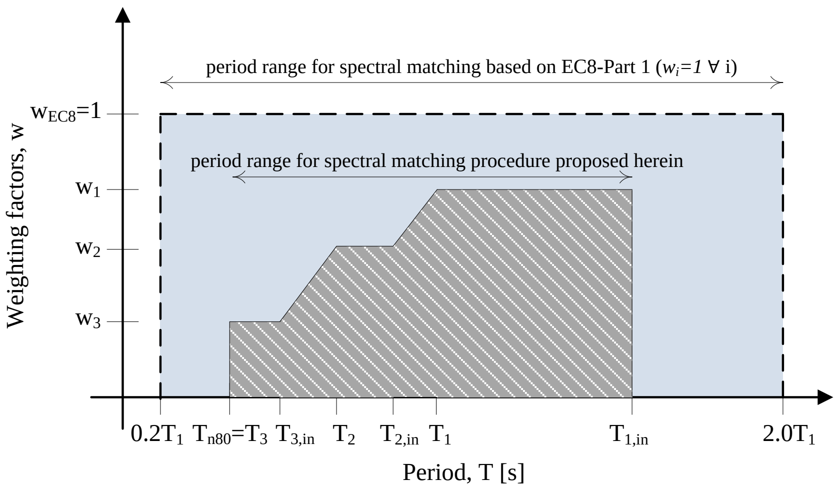

Figure 1. Schematic illustration of the proposed period range along with the assigned weighting factors.

Step 4 - Definition of the spectral variability ranking index (δ_{sv})

Consistent with intuition, suites of strong motions with limited variability among their spectral ordinates have been found to result in response estimates of low scatter (Tothong and Luco, 2007), thus enhancing the reliability of the average (design) response values. Along these lines, the process introduced herein aims to credit the selection of suites with naturally recorded ground motions of low (intra-suite) variability and the related ranking index is quantified as follows:

$$\delta_{sv} = \frac{\sum_{i=1}^{N} w(T_i) \cdot \sigma\left[Sa_1(T_i) + Sa_2(T_i) + \ldots + Sa_m(T_i) \right]}{\sum_{i=1}^{N} w(T_i)} \tag{8}$$

where $\sigma\left[Sa_1(T_i) + Sa_2(T_i) + \ldots + Sa_m(T_i) \right]$ is the standard deviation of the spectral acceleration values calculated for the m seismic motions included in each one of the already formed suites. The spectral acceleration values are calculated at sample periods, T_i, while N is the number of spectral ordinates within the previously described period range. Such a structure-specific index accounts for the vibration periods (elastic and inelastic) and the modal mass participation factors, thus enabling identifying suites of motions with limited spectral variability within the significant period zones for the structure studied.

Step 5 - Definition of the spectral compatibility ranking index (δ_{sc})

The quality of the compatibility between the average spectrum of the selected earthquake records and the target spectrum within the period range, defined in Steps 1-2, is quantified through an additional ranking index:

$$\delta_{sc} = \frac{\sum_{i=1}^{N} w(T_i) \cdot \left[\frac{sf_{avg} \cdot Sa_{avg} - a_{\min} Sa_{target}(T_i)}{a_{\min} Sa_{target}(T_i)} \right]^2}{\sum_{i=1}^{N} w(T_i)} \tag{9}$$

The index δ_{sc} considers the dynamic characteristics of the structure thus permitting spectral compatibility in those period ranges, which are particularly critical for the structural behavior under the earthquake loading.

Step 6 - Temporary ranking of the motions suites based on δ_{sv} and δ_{sc} rating indices

Two separate rankings of the already formed suites are materialized using the pair of the secondary indices δ_{sv} and δ_{sc} that has been described above. More precisely, each suite of motions is assigned with two unique integer coefficients, ID_{sv} and ID_{sc}, corresponding to the order that a suite has obtained according to the δ_{sv}- or δ_{sc}-based ranking system, respectively. It is notable that these two different ranking approaches are temporary and utilized only for the final ranking of the proposed decision support process.

Step 7 - Final ranking of motions suites based on the index δ_{sv-sc}

Based on the concept presented herein, the ideal suite of motions to be used for the time domain analysis of a structure would have both: (a) the smallest intra-suite variability possible among the spectral ordinates of the seismic motions (i.e., being ranked with ID_{sv}=1 according to the δ_{sv} ranking criterion) and (b) the highest quality of compatibility with the target spectrum (i.e., ID_{sc}=1 according to the δ_{sc} ranking criterion). Given the fact that it is rather impossible to find a suite of motions highly rated by both the aforementioned ranking systems, two additional weighting coefficients, f_{sv} and f_{sc}, to express the relative importance of the two criteria:

$$f_{sv}, f_{sc} \in R \text{ and } 0 \leq f_{sv}, f_{sc} \leq 1.0 \tag{10a}$$

$$f_{sv} + f_{sc} = 1.0 \tag{10b}$$

The pair of f_{sv}-f_{sc} coefficients enables quantifying the contribution of the two secondary (and temporary) ranking systems (δ_{sv} and δ_{sc}) to the final quality index, δ_{sv-sc}:

$$\delta_{sv-sc} = f_{sv} ID_{sv} + f_{sc} ID_{sc} \tag{11}$$

Based on the δ_{sv-sc} index, the highly ranked suites of motions are widely expected to result in the lowest structural response variability among the entire population of suites, which have been already formed on the basis of the preliminary earthquake records selection criteria and the adopted spectral matching requirements.

STRUCTURAL MODEL AND EARTHQUAKE SCENARIOS

An irregular, both in height and plan, multi-story RC building was adopted herein as the necessary testbed to evaluate the aforementioned multi-criteria procedure for selecting and scaling earthquake records. It is an existing four-story building of 14.60 m (including pilotis) located in Lefkada island, Greece and it has sustained severe damage after the 2013 earthquake of M_w=6.4. The soil profile comprises of very soft strata as described in detail elsewhere (Sextos et al., 2010).

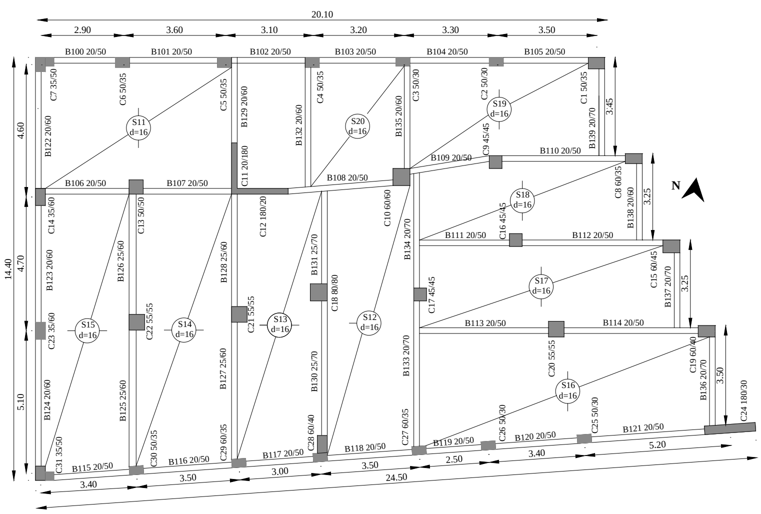

Figure 2. Plan view of the typical story of the four-story R/C building studied.

The building was designed according to first (and quite simplified) national seismic code of 1959. The lack of sufficient number of shear walls (Fig. 2) and the discontinuous distribution of stiffness in height due to a loft constructed at the back of the ground floor further increased its vulnerability. Based on site investigation, the concrete class can be considered equivalent to the current C16/20 (i.e., compressive strength $f_c' = 16$ N/mm^2), while the yield strength (f_y) for the longitudinal and the transverse steel reinforcing bars is equal to 400 N/mm^2 and 220 N/mm^2 respectively. The numerical modeling of the structure was facilitated using SAP2000 finite element code (CSI, 2014). A three-dimensional (3D), fixed-base model was created subjected to bi-directional earthquake-induced excitations. Linear frame elements were used to model both the beams and the columns while the shear walls and the slabs were modeled by shell elements. The dynamic characteristics of the building are presented in Table 2. A pair of seismological parameters, consisting of the moment earthquake magnitude, M_w, and the source-to-site distance, R, was employed in order to create two alternative earthquake scenarios, being representative for several earthquake-prone areas worldwide (e.g., Wester US and Southern Europe). The soft soil conditions of the site of interest, classified as *C* soil category according the EC8 classification on the basis of the average shear wave velocity, $v_{s,30}$, of the upper 30 m of the soil profile (i.e., $180<v_{s,30}<360$ m/s), refined further the definition of the earthquake scenarios and hence the selection of the seismic motions. Strong ground motions recorded close to the seismic source, i.e., $10\leq R\leq 30$ km, during earthquake events of moderate-to-high magnitude, i.e., $5.5\leq M_w<6.5$, are selected for the first seismic scenario A (codified as SSA), while the second scenario B (SSB) involves far-field seismic motions from earthquakes with high moment magnitude, i.e., $6.5\leq M_w<8$ and $30<R\leq 80$ km. For each scenario SSA and SSB, 20 pairs of horizontal components of seismic motions were selected out of 100 and 184 pairs of eligible earthquake records. Based on eq. 1, a total 77,520 alternative suites of seven pairs of seismic motions were formed in line with EC8. The target (elastic) spectrum was defined for reference peak ground acceleration, a_{gR}, equal to 0.36g (Zone III of the national Annex), while the importance factor and the damping ratio were set to 1.0 and 5%, respectively.

RESPONSE HISTORY ANALYSIS RESULTS

Effect of ranking ($\delta_{sv\text{-}sc}$ and δ_{conv}) on the reliability of the design values

The already achieved reduction in the intra-suite response variability using suites, which are prioritized via the proposed multi-criteria process and the related $\delta_{sv\text{-}sc}$ index, is highly expected to increase the design values reliability, quantified herein by the standard error, *SE*, of the estimated average response, $\overline{x}_s$. As a matter of fact, based on eq. 12, the standard error is proportional to the response variability calculated for a specific *EDP* after the RHA of the structural model using a selected suite of motions.

$$SE = \frac{s}{\sqrt{n}} t(\mathrm{CL}, \mathrm{df}) \tag{12}$$

where *s* is the standard deviation of the sample of the response values and *n* is the sample size (in this case the sample size *n* coincides with *m*, which has been already defined as the number of motions pairs that are included into the suites). The *t-factor* depends on the confidence level, *CL*, typically assigned to predict the central response estimate, and *df* represents the degrees of freedom for the two-sided *Student's t* probability distribution function. From a practical point of view, assuming a suite of motions with seven records, the *t-factor* is equal to 1.943 for *df* =6 and *CL*=90%, hence, the SSE value, which can be defined as percentage of the estimated average base moment with *s*=0.30, is equal to 22%. The interpretation of such a result reveals that if one were to form several response samples of common size drawn from the same population, 90% of the times the true, though unknown, average (design) response will be included within the $\pm 0.22 \cdot \overline{x}_s$ confidence interval. Thus, calculating quite narrow confidence intervals for the central estimates of the *EDPs* lead to increased reliability for the design values. It is also interesting to see that the central tendency of the log-normally distributed *EDPs*, being already a mature consideration, is more rational to be represented by the geometric mean rather than the arithmetic mean. However, the consequence of using one of the aforementioned moments is not significant when a consistent definition of the central tendency is made for both the seismic records scaling (through the spectral matching procedure) and the structural response measuring. Moreover, the arithmetic mean has been extensively specified by codes drafting (e.g., Eurocode 8, ASCE/SEI-7) to be used for the central response (design) values if, at least, seven seismic motions are considered for the time domain analysis. Thus, the engineers are more familiar to the arithmetic mean for the response parameters, which is also adopted in the current to study.

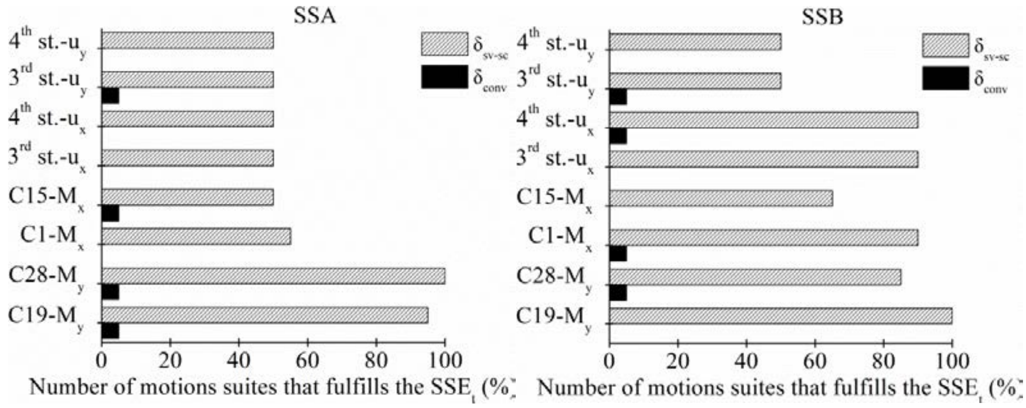

Figure 3. Effect of the motions suites rating systems (δ_{conv} and $\delta_{sv\text{-}sc}$) on the average (design) values reliability for the multi-story, RC building

Along these lines, the standard error for the estimated average *EDPs* was calculated considering the associated top 20 suites for each scenario, ranked according to the δ_{conv} and $\delta_{sv\text{-}sc}$ indices. A reliability criterion was also formulated by setting a target threshold (lower bound) for the standard error, SE_t=30%, of the estimated average response measures while the confidence level was taken equal to 90%. Next, each suite of motions investigated herein was considered to fulfil the reliability criterion once the related *SE* was found to be lower than the target threshold, SE_t. It is notable that FEMA P-58-1 as well as recent studies (Huang, Whittaker, Luco, and Hamburger, 2011) prescribe similar confidence level criteria, *CL*, and SE_t for estimating the average (design) values. Figure 3 illustrates the outcome of such a comparative assessment performed on the basis of the aforementioned reliability criterion. For each one of the *EDPs* considered herein, the grey bar reflects the number (as a percentage) of the top 20 $\delta_{sv\text{-}sc}$-ranked suites of motions that induced average (design) values with standard error estimate lower than the target one (for common *CL* equal to 90%). Likewise, the black bars show the reliability criterion success rate for the top 20 δ_{conv}-ranked suites. The superiority of the currently proposed multi-criteria process is evident in terms of fulfilling the specific target reliability level for the design values. The latter was found to be independent on the *EDP* and the seismic scenario considered. Furthermore, it is worth noting that only a considerably low fraction of the top 20 δ_{conv}-ranked suites met the specific reliability criterion while, on the other hand, 62.50% and 77.50% of the top 20 $\delta_{sv\text{-}sc}$-ranked suites, corresponding to the SSA and SSB respectively, met, on average, the chosen reliability requirements.

CONCLUSIONS

A decision support system is presented herein to facilitate the intricate task of selecting and scaling earthquake ground motions as required for response history analysis. Motions suites are provided to fully conform to the current normative framework while, at the same time, induce, response parameters with highly reduced intra-suite variability. The latter is prerequisite to achieve increased reliability levels for the average (design) response estimates, normally predicted during the code-prescribed design verification of structural systems. The structure-specific ground motion selection process described herein, which may also be used to evaluate the seismic performance of structures under a target spectrum, incorporates a multi-criteria framework considering: (a) the spectral variability among the selected motions of the suites, (b) the compliance between the suites average spectrum and the target one and, (c) the dynamic characteristics (elastic - inelastic vibrations periods, modal mass participation factors) of the structure studied. A novel ranking index ($\delta_{sv\text{-}sc}$) is introduced to materialize the aforementioned criteria by prioritizing suites of motions that have been implicitly prequalified to induce design values of increased reliability. The efficiency of the ranking index, $\delta_{sv\text{-}sc}$, was quantified through its comparative assessment with the conventional index, δ_{conv}. The main conclusions from this study, based on RHA response results of a multi-story, RC building, is that significantly lower (almost 50%) intra-suite response variability was calculated when the case-study building was subjected to the most highly ranked suites according to the new index $\delta_{sv\text{-}sc}$ compared to the ones prioritized by the conventional index δ_{conv}, independently of the *EDPs* examined. Additionally, more than 62% (on average) of the top $\delta_{sv\text{-}sc}$-ranked suites fulfilled the reliability criterion imposed herein, which was only marginally met by the highly prioritized suites by the conventional approach (δ_{conv}). Overall, the efficiency and greater reliability of the proposed ground motion selection and prioritization index was demonstrated.

ACKNOWLEDGEMENTS

Financial support was provided to the first author by the People Programme (Marie Curie Actions) of the EU FP7 Programme (FP7/2007-2013) under REA grant agreement n° 609405 (COFUNDPostdocDTU).

REFERENCES

Baker, J. W. (2011), "The Conditional Mean Spectrum: A Tool for Ground Motion Selection", *Journal of Structural Engineering*, Vol. 137, No. 3, pp.322–331.

Bazzurro, P., Cornell, C. A. (1999), "Disaggregation of seismic hazard", *Bulletin of the Seismological Society of America*, Vol. 89, No. 2, pp.501–520.

Beyer, K., Bommer, J. J. (2007), "Selection and Scaling of Real Accelerograms for Bi-Directional Loading: A Review of Current Practice and Code Provisions", *Journal of Earthquake Engineering*, Vol. 11, pp.13–45.

Bommer, J. J., Acevedo, A. B. (2004), "the Use of Real Earthquake Accelerograms As Input To Dynamic Analysis", *Journal of Earthquake Engineering*, Vol. 8, No. 1, pp.43–91. doi:10.1080/13632460409350521

CEN (2004), *European Standard EN 1998-1. Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings", Committee for Standarization*. 3 DesignVol.3. European Committee for Standardization, Brussels, Belgium

CSI (2014), *SAP2000: integrated building design software, v.17—user's manual. Berkeley, California, USA*.

Elnashai, A. S., McClure, D. C. (1996), "Effect of modelling assumptions and input motion characteristics on seismic design parameters of RC bridge piers", *Earthquake Eng. & Structural Dynamics*, Vol. 25, No. 5, pp.435–463.

Hancock, J., Bommer, J. J., Stafford, P. J. (2008), "Numbers of scaled and matched accelerograms required for inelastic dynamic analyses", *Earthquake Engineering & Structural Dynamics*, Vol. 37, pp.1585–1607.

Huang, Y.-N., Whittaker, A. S., Luco, N., Hamburger, R. O. (2011), "Scaling earthquake ground motions for performance-based assessment of buildings", *Journal of Structural Engineering*, Vol. 137, No. 3, pp.311–321.

Iervolino, I., Galasso, C., Cosenza, E. (2010), "REXEL: Computer aided record selection for code-based seismic structural analysis", *Bulletin of Earthquake Engineering*, Vol. 8, pp.339–362. doi:10.1007/s10518-009-9146-1

Iervolino, I., Manfredi, G., Cosenza, E. (2006), "Ground motion duration effects on nonlinear seismic response", *Earthquake Engineering & Structural Dynamics*, Vol. 35, No. 1, pp.21–38. doi:10.1002/eqe.529

Katsanos, E. I., Sextos, a. G., Elnashai, A. S. (2014), "Prediction of inelastic response periods of buildings based on intensity measures and analytical model parameters", *Engineering Structures*, Vol. 71, pp.161–177.

Katsanos, E. I., Sextos, A. G. (2013), "ISSARS : An intergrated software environment for structure specific earthquake ground motion selection", *Advances in Engineering Software*, Vol. 58, pp.70–85.

Katsanos, E. I., Sextos, A. G., Manolis, G. D. (2010), "Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective", *Soil Dynamics and Earthq. Eng.*, Vol. 30, No. 4, pp.157–169.

Kottke, A. R., Rathje, E. M. (2008), "A semi-Automated procedure for selecting and scaling recorded earthquake motions for dynamic analysis", *Earthquake Spectra*, Vol. 24, No. 4, pp.911–932.

Kwon, O.-S., Elnashai, A. S. (2006), "Fragility Analysis of RC Bridge Pier Considering Soil-Structure Interaction", *Procced. of the Structures Congress: Structural Engineering and Public Safety* , St. Louis, Missouri: ASCE.

Padgett, J. E., Desroches, R. (2007), "Sensitivity of Seismic Response and Fragility to Parameter Uncertainty", *Journal of Structural Engineering*, Vol. 133, No. 12, pp.1710–1718.

Sextos, A. G., Katsanos, E. I., Manolis, G. D. (2010), "EC8-Based earthquake record selection procedure evaluation: validation study based on observed damage of an irregular R/C building", *Soil Dynamics and Earthquake Engineering*, pp.1–37.

Spanos, P. D., Giaralis, A., Li, J. (2009), "Synthesis of accelerograms compatible with the Chinese GB 50011-2001 design spectrum via harmonic wavelets: artificial and historic records", *Earthquake Engineering and Engineering Vibration*, Vol. 8, No. 2, pp.189–206.

Tothong, P., Luco, N. (2007), "Probabilistic seismic demand analysis using advanced ground motion intensity measures", *Earthquake Engineering and Structural Dynamics*, Vol. 36, pp.1813–1835.