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Structure-specific selection of earthquake ground motions for the reliable design and assessment of structures

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Abstract

A decision support process is presented to accommodate selecting and scaling of earthquake motions as required for the time domain analysis of structures. Code-compatible suites of seismic motions are provided being, at the same time, prequalified through a multi-criterion approach to induce response parameters with reduced variability. The latter is imperative to increase the reliability of the average response values, normally required for the code-prescribed design verification of structures. Structural attributes like the dynamic characteristics as well as criteria related to variability of seismic motions and their compliance with a target spectrum are quantified through a newly introduced index, $\delta_{sv\text{-}sc,}$ which aims to prioritize motions suites for response history analysis. To demonstrate the applicability of the procedure presented, the structural model of a multi-story building was subjected to numerous suites of motions that were highly ranked according to both the proposed approach ($\delta_{sv\text{-}sc}$) and the conventional one (δ_{conv}), that is commonly used for earthquake records selection and scaling. The findings from numerous linear response history analyses reveal the superiority of the proposed multi-criterion approach, as it extensively reduces the intra-suite structural response variability and consequently, increases the reliability of the design values. The relation between the target reliability in assessing structural response and the size of the suite of motions selected was also investigated, further demonstrating the efficiency of the proposed selection procedure to achieve higher response reliability levels with smaller samples of ground motion.

Keywords: selection of earthquake motions, response-history analysis, R/C multistory building, structural response variability, reliable design

1. 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, it constitutes a rigorous method that captures the hierarchy of failure mechanisms, the energy dissipation and force-redistribution phenomena as well as enables to control the level of structural and non-structural damage during the strong ground shaking. Such a time domain analysis requires as input the use of, at least, a suite of appropriately selected and scaled earthquake motions being consistent with a predefined earthquake scenario. Nevertheless, research has shown that among all possible uncertainty sources stemming from structural and soil material properties, the 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 McGlure 1996, Padgett and Desroches 2007). Inevitably, the above uncertainty propagates to structural demand and the selected seismic motions may ultimately govern the reliability of the seismic design or assessment outcome.

Since 1990's, various techniques have been developed to address the complex problem of selecting and scaling earthquake ground motions (PEER GMSM 2009, Katsanos *et al*. 2010). From the objective 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 its full probability distribution (i.e., median response and standard deviation). The rationale to calculate the central tendency of an appropriately chosen engineering demand parameter, *EDP*, (e.g., element forces and deformations, interstory drifts) 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 outcome (EN1998-Part 1 2004, ICC 2009, ASCE 7-10 2010, Hancock *et al*. 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 (FEMA P-58-1 2012) 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 (e.g., Malhotra 2003, Kwon and Elnashai 2006, Dhakal *et al*. 2006, Iervolino *et al*. 2006, Youngs *et al*. 2006, Kurama and Farrow 2003, Lee *et al*. 2000, Sorabella *et al*. 2006). However, the concurrent application of multiple selection criteria may significantly restrict the available number of earthquake records (Stewart *et al*. 2001, Bommer and Acevedo 2004). Thus, a balance has to be preserved between the extent of the selection criteria applied and the number of 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 structural engineers, while they can be readily obtained either by deterministic seismic hazard analysis, *SHA*, or by disaggregating the probabilistic SHA (Kramer 1996, Bazzurro and Cornell 1999).

Once the strong ground motions have been selected from an earthquake records archive, the most compatible records with a predefined target spectrum are primarily preferred for the structural analysis. The Conditional Mean Spectrum, CMS, (Baker 2011) and the related Conditional Spectrum, CS, (Jayaram *et al*. 2011) can be used as target spectra, being the most prevalent alternatives to the Uniform Hazard Spectra, UHS (Reiter 1990). The latter serves the basis to define the smooth code spectra and assumes equal probability of exceedance for the spectral accelerations along the entire period range of structural interest. Nevertheless, spectral values of high amplitude, computed for each period, are unlikely to occur simultaneously in a single ground motion (Bommer *et al*. 2000, Naeim and Lew 1995, McGuire 1995); hence, significant conservatism is related to the use of UHS especially for the rare levels of seismic hazard. On the contrary, CMS accounts for correlations among spectral accelerations at all periods, while it introduces the use of the epsilon parameter[1], ε, that has been found to be an efficient predictor of the spectral shape and thus the structural response (Baker and Cornell 2006). Recently, this approach was extended to consider conditional values of any ground motion properties (e.g., duration) rather than only spectral values (Bradley 2010). However, the application of those conditional spectra is relatively limited, since they are site-specific and advanced seismic hazard disaggregation information is required, while their spectral shape and amplitude are sensitive to the user-defined, conditioning period. Thus, inaccuracies in period estimation may adversely affect the structural analysis results (O'Donnel *et al*. 2013).

Independently on the target spectrum adopted, several methods have been developed to modify the ground motions and hence to 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 average earthquake records' response spectrum and the target one. Various metrics have been employed to quantify the spectral compatibility (Beyer and Bommer 2007, Buratti *et al.* 2011). 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 three to five or even higher - this issue is still controversial) are employed (e.g., Luco and Bazzurro 2007, Watson-Lamprey and Abrahamson 2006, Huang *et al*. 2011, Grigoriu 2011). For nonlinear RHA of both symmetric and asymmetric in-plan buildings, modal pushover-based scaling procedures have been recently introduced (Kalkan and Chopra 2011, Reyes and Quintero 2013, Reyes *et al*. 2015) and their performance was found to provide superior response results in terms of accuracy and efficiency than the strong motions scaling procedure prescribed by ASCE/SEI 7-10 standard. Alternatively, the frequency content of the recorded accelerograms can be modified using techniques from stochastic or random vibration theory (e.g., Naeim and Lew 1995, Hancock *et al*. 2006, Barenberg 1989, Carballo and Cornell 2000, Silva and Lee 1987, Boore 2000, Giaralis and Spanos 2009, Cacciola 2010, Lee and Han 2002). 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 (Bommer and Acevedo 2004). Finally, a systematic unconservative bias in the estimation of the mean structural response has been identified (Luco and Bazzurro 2007, Huang *et al.* 2011, Carballo and Cornell 2000).

[1] The epsilon parameter, ε, is defined as the number of standard deviations, by which an observed logarithmic spectral acceleration deviates from the mean logarithmic spectral acceleration of a ground-motion prediction equation.

2. Challenges and Objectives

Based on the discussion made above, substantial progress has been made for selecting and scaling strong motions. Nevertheless, the main findings of these evolutionary methods are still not reflected on the present state-of-the-practice and the seismic codes drafting. The current code provisions provide marginal and simplified guidance for such a critical issue. Thus, the practitioners take often subjective decisions that may lead to structural solutions of limited confidence. Along these lines, Sextos *et al.* (2011) showed that the nonlinear RHA of a multistory building using, different but fully legitimate, Eurocode 8-compatible suites of ground motions leaded to highly scattered response results, thus undermining the desired reliability for the structural analysis. The extensive variability in the predicted response is corroborated by relevant studies (e.g., Reyes and Kalkan 2012, Araújo *et al*. 2016), while a more accurate determination of the variation range for the M,R_s-based selection criterion is not expected to reduce the response variability, since structural behavior and M-R_s pairs were found only partially correlated (Shome *et al*. 1998, Baker and Cornell 2005, Krawinkler *et al*. 2003). Moreover, the absence in code drafting of explicit criteria to ensure the quality of the required compatibility between code spectrum and ground motions spectra was found to be responsible for overconservatism in the design outcome, being a-priori evident due to the uniform hazard-type (UHS) of the code spectrum (Sextos *et al*. 2011). The limited consensus of the designers for this complex issue of high, though, importance for the structural analysis reliability blurs further the application of the code-prescribed framework, already identified with deficiencies. Hence, the lack of a supportive background with reasonable rules, based on the state-of-the-art progress, often leads structural engineers to decide without the justifying appropriately their ground motion selection and scaling procedure.

To counteract this problem, innovative techniques, associated with robust algorithms and software, were recently made available to accommodate selecting and scaling of ground motions. Apart from the aforementioned advancements from Baker's research group (e.g., Baker 2011, Jayaram *et al*. 2011) and Bradley (2010,2012), Iervolino *et al*. (2010) developed a computational tool that enables the selection of suites of multi-component ground motions compatible with either code-based or user-defined pseudo-acceleration response spectra. Moreover, Smerzini *et al*. (2014) focused on selecting displacement-spectrum-compatible seismic ground motions, while the latter was also elaborated by Corigliano *et al*. (2012), who introduced an automated procedure to select seismic motions for RHA on the basis of a wealth database with good-quality seismic records. A web-based application, released by Dias *et al*. (2010), is capable of selecting ground motions accounting for geophysical and strong ground motion parameters, while spectral compatibility criteria have been also introduced therein. The presence of velocity pulses in near-fault time series was considered, among others, as a seismic records selection criterion via the Design Ground Motion Library (DGML, Wang *et al*. 2013), which utilizes the Next Generation Attenuation Strong-Motion Database, PEER-NGA (Chiou *et al*. 2008). A semi-automated algorithm was also proposed by Kottke and Rathje (2008) for selecting and scaling of strong motions that fit both to a target spectrum and a target standard deviation.

Nevertheless, all these cases cannot be either easily applied in a design-office environment, unless either hazard disaggregation data is requested (note that for several earthquake-prone areas of the world this data is not readily available to the designers) or the reduced structural response variability is not considered as an objective. Thus, structural design solutions with limited confidence may be emerged. Along these lines, it is doubtful whether the designers are adequately supported to decide: (a) which records to be selected for the RHA and the consequent code-compatible design or assessment of structures, (b) how the selected records must be grouped into suites to match the spectral compatibility requirements, (c) which are the substantial structural properties that have to be considered when seismic records are to be selected and scaled for RHA of a given structure and (d) how the reliability of the response estimates can be ensured in order to achieve a design outcome with a predefined level of confidence.

Given the above considerations, the scope of this study is to improve the existing seismic design and assessment framework by introducing a decision support process, which provides prequalified suites of seismic motions that induce stable, and thus reliable, design (average[2]) response values. Along these lines, the proposed process 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 2012). It is notable that the current decision support process is implemented into a newer version of the computational system ISSARS (Katsanos and Sextos 2013), developed by the authors, which facilitates structure-dependent selection and scaling of earthquake ground motions rapidly formed into numerous suites that can be used for the RHA of structures. These suites of motions are ranked by a complex system that designates those motions that 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 vibrations 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 for within the current framework so as a structure-specific process for earthquake records selection and scaling is materialized.

In the following, the above response-oriented process is presented and its efficiency is evaluated through numerous response linear history analyses for the case of an existing multistory, reinforced concrete (R/C) building. The variability of the structural response induced by the conventional and the proposed procedure is then comparatively assessed. The number of earthquake motions required to obtain stable response estimates is also investigated.

3. Multi-criterion process for selecting and scaling of earthquake motions

Based on preliminary selection criteria, including the earthquake magnitude, M, the source-to-site distance, R_s, the soil conditions at the recording site and the peak ground acceleration, PGA, the eligible earthquake records are retrieved by the PEER-NGA Database (Chiou *et al*. 2008) and they are used to form alternative suites of motions that satisfy either the code-imposed or the used-defined requirements

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

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:

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

It is worthwhile to mention that a popular (and often code-prescribed) 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 (2004), when seven or more different records are selected and used for RHA, the average of the response values is taken as the design value. Otherwise, when the selected suite consists of three to six records only, the design value is defined as the maximum response resulted from the analysis. It is quite rare in design practice to use more than seven (pairs of) ground motions, first, on account of the high computational cost related to multiple response history analyses but also because suites with more than seven records require a disproportionally large number of eligible records, k, to achieve an acceptable matching between the average spectrum of the m individual records and the target spectrum (Kottke and Rathje 2008). However, larger samples of seismic motions favor, in principle, the reliability of the average (design) response estimates and thus, such a trade-off has to be reasonably handled.

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 included in a suite, will exceed the minimum allowable spectral ordinates of the target spectrum, $Sa_{target}(T_i)$, within a prescribed period range.

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

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 (Jayaram *et al.* 2011, Beyer and Bommer 2007, Buratti *et al.* 2011, Iervolino *et al.* 2010, Kottke and Rathje 2008), most of them, even though advanced, are in essence similar to the one presented in Eq. 3, which evaluates the convergence between the target spectrum and the average spectrum of motions suite for a specific range of periods:

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

However, the use of this conventional spectral compatibility measure, δ_{conv}, as a ranking index of seismic motions does not guarantee a stable enough average of the demand parameters. For this reason, the current study introduces a dual ranking measure for ground motion selection that leads in more stable structural response results. This index, $\delta_{spectral\ variability\ –\ spectral\ compatibility}$ (hereafter denoted as $\delta_{sv\text{-}sc}$) is composed by two secondary indices that consider: (a) the intra-suite variability of motions (i.e. 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 ground motions 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 - Upper bound for the period range used for spectral matching

Most of the code-based procedures for selection and scaling of 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's harder to obtain a large number eligible records but most importantly, it is unlikely that a low-to-moderate ductility structure will ever respond. Indeed, especially in case of EN1998-Part 1 (2004), the imposed upper bound, i.e., $2T_1$, (T_1 being the fundamental period), is deemed rather extensive for several structural configurations. For example, research has shown that R/C buildings, designed to modern seismic codes, were statistically found to experience significantly milder first-mode period lengthening, i.e., $1.2T_1$ up to $1.5T_1$, compared to the code-prescribed $2T_1$, even for twice the design earthquake (Katsanos *et al*. 2014). Moreover, spectral matching within such an long period range (up to $2T_1$) substantially increases the spectral ordinates of the selected records in other, more critical periods of vibration (close or lower than T_1), thus leading to overconservative design (Sextos *et al*. 2011). The latter can be partially waived by using individual factors to scale each of the strong motions included already into the records' suites that would be also related to lower intra-suite spectral variability (Kottke and Rathje 2008). However, such a scaling approach would excessively increase the population of the possible suites affecting, at the same time, adversely the efficiency of the entire process for selecting and scaling of strong motions unless a sophisticated, optimization-based method is followed.

Based on the above line of thought, the selection methodology presented herein makes use of the findings of a recent study to bound the upper spectral matching period range by the first-mode elongated (inelastic) period $T_{1,in}$ (Eq. 4), the latter being defined as a function of the corresponding elastic period, T_1, and the force reduction factor, R_y (or behavior factor, q, in EN1998), for which the building or bridge has been designed (Katsanos and Sextos 2015):

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

It should be noted that additional results from both experimental and numerical studies accounting for varying structural systems made of different materials are expected to further advance the calculation of first-mode elongated period on the basis of simplified analytical expressions.

Step 2 - Lower bound for the period range used for spectral matching

Similarly to the step described above, the lower bound for the period range of spectral matching is also adjusted so that spectral matching is imposed to low periods only in case where the participation of higher modes of vibration is significant for structural response. More precisely, the lower bound of $0.2T_1$, 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 ratios are higher than 80% for both main horizontal directions:

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

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

where $\sum_{i=1}^{n} \mathit{\Gamma}_{i,x}$ and $\sum_{i=1}^{n} \mathit{\Gamma}_{i,y}$ are the cumulative modal mass participation ratios calculated for the first n modes and 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 in the (shorter) period range $T{:}[T_{n,80},T_{1,in}]$, where the structure is expected to respond during the seismic excitation, deliberately neglecting unnecessarily low and excessively high vibration periods, which are ultimately irrelevant to the structure. It is notable that the redefined period range, $T{:}[T_{n,80},T_{1,in}]$ (resulting from Steps 1-2), is employed for suites of motions that have been already identified to comply with the basis of spectral matching requirements related either to a code or a user-defined framework respectively. In other words, the existing code provisions for earthquake records selection and scaling are by no means violated. What is affected is the relative ranking of the (otherwise code-compliant) suites of motions.

Step 3 – Period-dependent weighting array for spectral matching

Apart from the revisited period range, a weighting array is introduced to refine further the proposed ranking system. The weighting factors for each individual period used herein for spectral matching, are associated with their respective modal mass participation ratios. Given the epistemic uncertainty related to the calculation of the dynamic characteristics of the structure, the weighting of the different vibration periods is employed in appropriate period ranges. Along these lines, 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 as follows:

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

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 promotes ground motions, for which the quality of spectral matching is higher within period ranges that are most likely to be important for the overall seismic response of the structure.

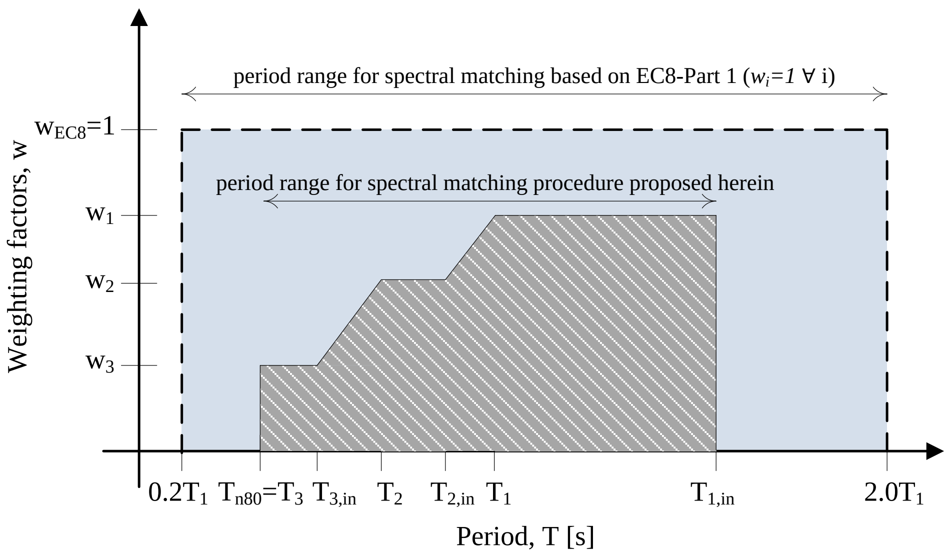

Fig. 1 Schematic illustration of the currently proposed period range along with the assigned weighting factors applied for a structure, in which the sum of the modal masses for the first three vibration modes exceeds the 80% of the total mass

It is recalled that the current state-of-the-art and code requirements for spectral matching prescribe, most of the times, implicitly uniform (and equal to unity) weighting factors for the entire period range of interest. The weighting scheme adopted is presented in Fig. 1, where the weighting factor, w_1, calculated using Eq. 6, is applied for the entire first mode-related period zone, i.e. [T_1,$T_{1,in}$]. In general, w_i, is assigned to the period range [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 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) that participate into to revisited period range, 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, $T_{nfp,el}$, are assumed to exhibit standard elongation, which is determined using Eq. (7) as a function of the design peak ground acceleration, a_g, typically prescribed by code provisions.

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

The equation above was derived from regression analysis of inelastic time-history analysis response results for five multistory, code-conformed R/C buildings that have been thoroughly studied in Katsanos *et al*. (2014).

Step 4 - 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 lower scatter (Tothong and Luco 2007), thus enhancing the reliability of the average (design) response values. Along these lines, the process introduced herein credits the selection of suites with ground motions records of low (intra-suite) variability. The related ranking index is quantified as follows:

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

where *σ(…)* 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, accounting for the vibration periods (elastic and inelastic) and the modal mass participation factors through the already defined period range and the weighting factors, enables identifying the suites of motions with limited spectral variability within the significant period zones for the structure studied.

Step 5 - Spectral compatibility ranking index (δ_{sc})

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

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

Likewise to the previous ranking index (Step 4), the index δ_{sc} considers the dynamic characteristics of the structure permitting, in such a way, the spectral compatibility in those period zones, which are critical for the structural behavior under the earthquake loading. Note that it is probable a suite of ground motions to present nearly perfect matching (i.e. very low value of δ_{sc}) of its average spectrum to the target one but undesirably high spectral variability among its individual records (i.e., very high value of δ_{sv}). This dual criterion is not currently prescribed in modern seismic codes.

Step 6 - Temporary ranking of the motion 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 have been described above. Then, each suite of ground 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} (only) and δ_{sc}-based (only) 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 ground motion suites based on the index $\delta_{sv\text{-}sc}$

Based on the framework presented herein, the ideal suite of motions to be used for the time domain analysis of a structure would have both: (a) the lowest possible intra-suite variability among the spectral ordinates of the seismic motions, hence being naturally ranked first with the δ_{sv} criterion (i.e., ID_{sv}=1) and (b) the highest quality of compatibility of its average spectrum with the target spectrum (i.e., ID_{sc}=1 based on δ_{sc} ranking criterion). Given the fact that it is rather impossible to find a suite of motions to fully satisfy both criteria but also because the relative importance of each criterion is rather subjective, two additional weighting coefficients, f_{sv} and f_{sc}, were introduced, to balance the impact of the two indices, δ_{sv} and δ_{sc}, respectively:

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

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

Then, 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 system, which is defined by introducing the following ranking index, $\delta_{sv\text{-}sc}$:

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

The highly ranked suites of motions according to the structure- and record-dependent $\delta_{sv\text{-}sc}$ index are 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. Therefore, the suites of motions with the lowest values of $\delta_{sv\text{-}sc}$ are prioritized by the decision support system to be used for the RHA of the structure studied.

4. Validation study: Structural model and earthquake scenarios

Having defined the dual index criterion to select earthquake ground motions for the purposes of response history analysis, a real, extensively investigated building is adopted for further study, as a means to comparatively assess the variability of structural response under the conventional and the proposed selection procedure. Especially, an irregular, both in height and elevation, multi-story R/C building was adopted herein as the necessary testbed to evaluate the aforementioned multi-criterion 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 suffered severe damages after the 14.08.2003 strong earthquake of M_w=6.4 occurred in the Ionian Sea area. Very soft soil strata were found underneath the foundation of the building, which was designed according to an out-of-date seismic code dating 1959. The lack of sufficient number of shear walls (Fig. 2) and the discontinuous distribution of the stiffness with elevation due to a 3.0 m high loft constructed at the back of the ground floor increased further the vulnerability of that building.

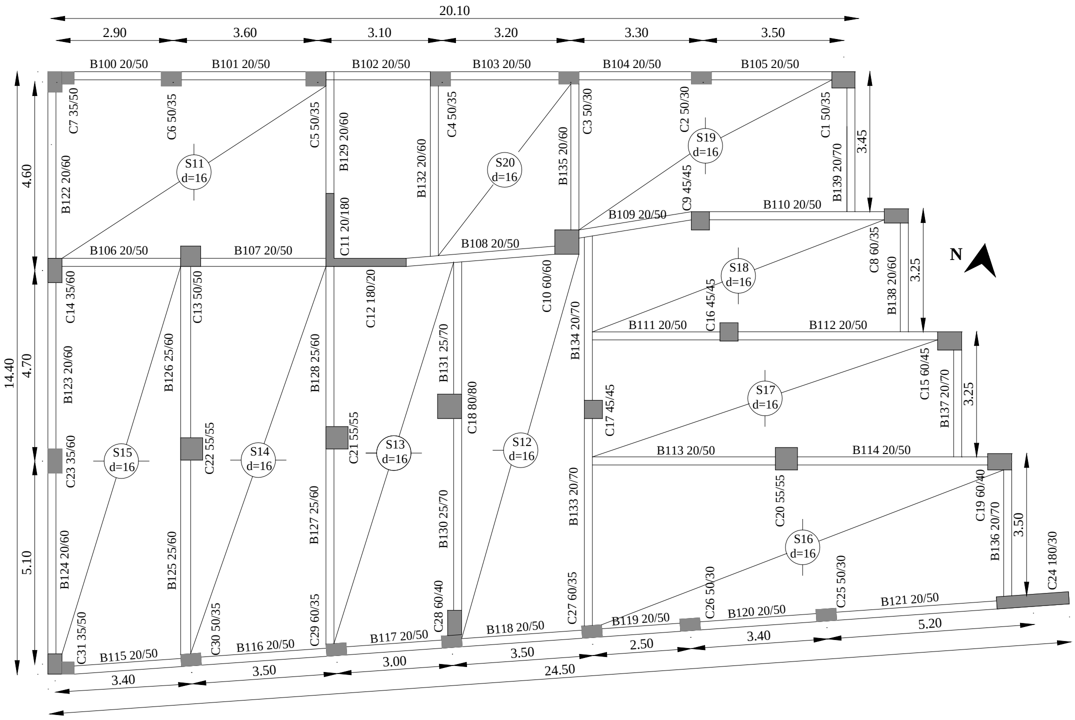

Fig. 2 Plan view of the typical story of the four-story R/C building

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 2011). A three-dimensional (3D), fixed-base model was developed to capture efficiently the complex dynamic behavior of such an irregular structural system 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. An existing authors' publication (Sextos *et al.* 2011) provides extensive details for the case-study building while its dynamic characteristics, derived from eigenvalue analysis, are presented in Table 1. It is notable that the advancements of the proposed method were chosen herein to be illustrated via the performance of linear RHA, since its application is the common case for the code-conformed, response history-based design verification of structures, the latter being a final and detailed check of a designed structure.

A pair of seismological parameters, consisting of the moment earthquake magnitude, M_w, and the epicentral distance, R, was employed to create two alternative seismic scenarios, being representative for several earthquake-prone areas (e.g., Wester US and Southern Europe). More precisely, 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 (codified as SSA) while the second scenario (SSB) encloses far-field seismic motions related to earthquakes of high moment magnitude, i.e., $6.5 \leq M_w < 8$ and $30 < R \leq 80$ km. The soft soil conditions of the site of interest, classified as *C* soil category according the EC8 classification (EN1998-Part1 2004) 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), was also used as an additional preliminary criterion for the selection of the earthquake ground motions.

Table 1 Dynamic characteristics of the structural system studied considered herein

Mode	Period T [s]	Modal mass participation ratio					Cumulative modal mass participation ratio[†]	
		Γ_x	Γ_y	Γ_{Rx}	Γ_{Ry}	Γ_{Rz}	$\sum \Gamma_x$	$\sum \Gamma_y$
1	0.692	0.074	0.774	0.780	0.071	0.090	0.074	0.774
2	0.619	0.797	0.072	0.072	0.753	0.366	0.871	0.846
3	0.136	0.001	0.001	0.078	0.072	0.044	0.872	0.847
4	0.109	0.001	0.020	0.005	0.009	0.327	0.873	0.867
5	0.065	0.002	0.001	0.006	0.007	0.001	0.875	0.868
6	0.044	0.001	0.004	0.002	0.001	0.004	0.876	0.872

[†] The cumulative modal mass participation ratios are deliberately presented only for the two main horizontal directions of the structure under study (i.e., x and y) along with the multi-criterion approach proposed herein.

Table 2 Seismic scenarios adopted for the time-domain analysis of the case-study building

Seismic scenario	Selection criteria of seismic motions			EC8 elastic spectrum		Algorithm output (ISSARS)		
						Pairs of seismic motions		Number of suites used for RHA
	Magnitude M_w	Epicentral distance R (km)	Soil conditions based on $v_{s,30}$ (m/s)	Peak ground acceleration a_g (g)	Soil type	Compatible with criteria	Chosen for the RHA	
A	5.5 – 6.5	10 – 30	180 – 360	0.36	C	100	20	77,520
B	6.5 – 8	30 – 80	180 – 360	0.36	C	184	20	77,520

For both SSA and SSB, 20 pairs of horizontal components of seismic motions were selected out of 100 and 184 pairs of earthquake records that were initially eligible for the two scenarios. Based on Eq. 1, 77,520 alternative suites that consist of seven pairs of seismic motions were formed in line with the EC8 provisions. The target (elastic) spectrum was defined for reference peak ground acceleration, a_{gR}, equal to 0.36g (Zone III of the EC8 national Annex), while the importance factor and the damping ratio were set to 1.0 and 5%, respectively. Table 2 summarizes the code spectrum parameters adopted herein along with the initial earthquake records selection criteria. Table 3 (Annex A) lists the earthquake events and the related strong ground motions that were used for the RHA of the case-study building.

5. Response history analysis results

As already mentioned, the main objective of the current analysis scheme is to evaluate the efficiency of the proposed multi-criterion process in terms of prioritizing code-compatible suites of motions that induce, via the RHA of structures, stable, and hence reliable, design values. Along these lines, a sensitivity analysis is firstly required to identify the impact of the weighting coefficients, f_{sv} and f_{sc}, on the ranking index $\delta_{sv\text{-}sc}$ (§5.1), which is then comparatively assessed with the conventional index, δ_{conv}, concerning their effect on both structural response variability (§5.2) and the design values (§5.3). Finally, the number of seismic motions, required to form a suite, is examined in relation with the two ranking systems (§5.4).

5.1 Sensitivity of the structural response in the weighting coefficients f_{sv} - f_{sc}

According to the Eq. 11, the definition of the introduced ranking index, $\delta_{sv\text{-}sc}$, involves the use of two weighting coefficients, f_{sv} and f_{sc}, related to the spectral variability (δ_{sv}) and spectral compatibility (δ_{sc}) indices. It is recalled that the coefficients pair favors the temporary prioritization of the motions' suites before calculating the final index, $\delta_{sv\text{-}sc}$ (§3, Steps 4-7). Following the Eqs. (10a) and (10b), three main cases are detected regarding the values that can be assigned to the f_{sv}-f_{sc} coefficients: (a) $f_{sv}<f_{sc}$: suites of high quality in terms of the compatibility with the target spectrum are prioritized by the final ranking system, (b) $f_{sv}>f_{sc}$: the dominant criterion for ranking suites is the low intra-suite variability among the spectral acceleration values, and (c) $f_{sv}=f_{sc}$: both the secondary ranking criteria (i.e. δ_{sv} and δ_{sc} temporary indices) are evenly contributing to the final $\delta_{sv\text{-}sc}$ index. To visualize the impact of the f_{sv}-f_{sc} coefficients on the final ranking system and the prioritized suites of motions, two characteristic cases including f_{sv}=0 - f_{sc}=1 and f_{sv}=1 - f_{sc}=0 were examined. In particular, for the SSB, Fig. 3 depicts the spectra of the top suite ranked according to the $\delta_{sv\text{-}sc}$ index that was defined with f_{sv}=0 and f_{sc}=1. The dominance of the spectral convergence criterion (f_{sc}=1) led to almost perfect matching between the suite's mean spectrum, which was obtained by averaging the response spectra of all individual seismic motions, and the EC8 elastic spectrum. The spectral matching was established within the structure-specific period range defined on the basis the dynamic characteristics of the multi-story R/C building (Table 1). On the contrary, the spectral acceleration ordinates of the motions included in this suite are significantly diverged due to the absence of the spectral variability criterion (f_{sv}=0) from the definition of the $\delta_{sv\text{-}sc}$ index. The inverse case is depicted in Fig. 4, where the response spectra are plotted for the top motions suite ranked according to the $\delta_{sv\text{-}sc}$ index with f_{sv}=1 and f_{sc}=0 (i.e., dominance of the spectral variability criterion).

To quantify the sensitivity that the f_{sv} and f_{sc} coefficients induce to the final suites ranking and the associated structural response, the proposed multi-criterion process for selecting and scaling earthquake records was performed considering five different pairs for the weighting coefficients. Accounting for the SSA, characteristic *EDPs* were calculated via RHA of the R/C building subjected to the top motions suites, which were ranked using the $\delta_{sv\text{-}sc}$ index along with: (a) f_{sv}=0 & f_{sc}=1 : dominance of the spectral compatibility criterion, (b) f_{sv}=0.25 & f_{sc}=0.75, (c) f_{sv}=0.5 & f_{sc}=0.5: even contribution of the temporary ranking criteria, δ_{sv} and δ_{sc}, (d) f_{sv}=0.75 & f_{sc}=0.25, and (e) f_{sv}=1 & f_{sc}=0 : dominance of the spectral variability criterion. In total, 35 time domain analyses were performed (i.e., one case-study building, five top suites according to the (a) - (e) cases for the $\delta_{sv\text{-}sc}$-based ranking system, seven pairs of seismic motions per suite) while, for comparative purposes, the structural response was also calculated via RHA of the R/C building with the top suite of motions ranked according to the conventional system, δ_{conv}. The latter evaluates only the convergence of the motions with the target spectrum within a predefined period range (e.g., $0.2T_1$ - $2T_1$ based on EC8 prescriptions) disregarding any further criterion. Thus, such a δ_{conv}-based rating is considered to match better the aforementioned case (a) considered for the evaluation of the currently proposed ranking system (i.e., dominance of the spectral convergence criterion).

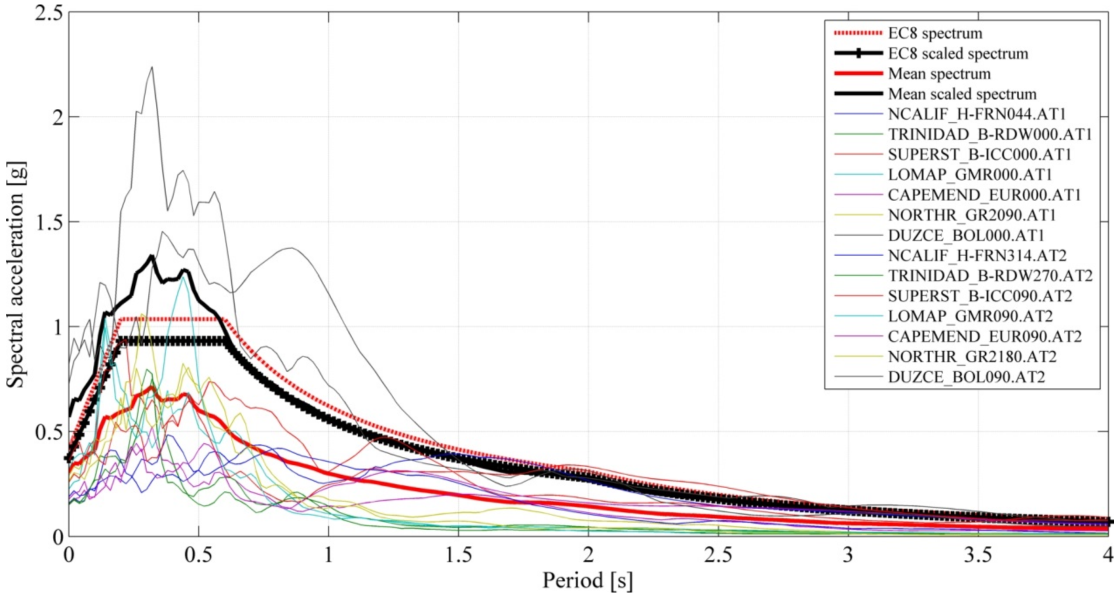

Fig. 3 Convergence between the EC8 spectrum and the response spectra of the top motions suite $\delta_{sv\text{-}sc}$-ranked with f_{sv}=0 and f_{sc}=1

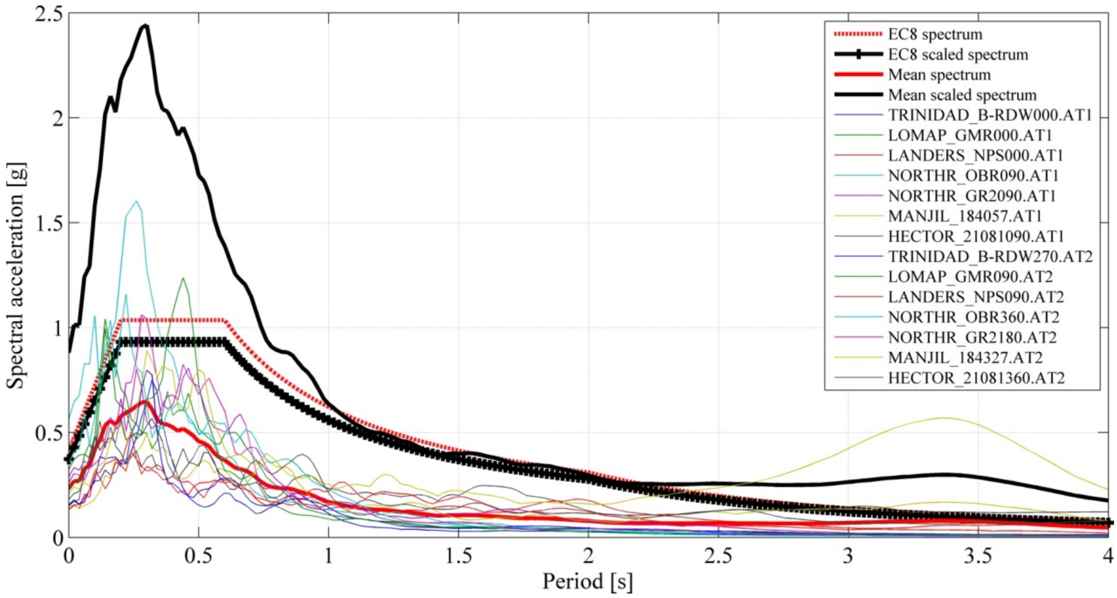

Fig. 4 Convergence between the EC8 spectrum and the response spectra of the top motions suite $\delta_{sv\text{-}sc}$-ranked with f_{sv}=1 and f_{sc}=0

Along these lines, Fig. 5 (left) demonstrates the effect of the five f_{sv}-f_{sc} pairs to the intra-suite variability of characteristic *EDPs* (i.e., bending moments of the ground floor columns C1 and C8 as well as lateral displacements at the top of the building), which is quantified using the coefficient of variation *COV* ($COV = \mu/\sigma$, where μ and σ is the arithmetic mean and the standard deviation of a sample of values). The *COV* is also calculated for the *EDPs* associated with the δ_{conv}-ranked top suite (i.e., four hollow symbols at f_{sv}=0 & f_{sc}=1). It is seen that the top suite, which was $\delta_{sv\text{-}sc}$-ranked with f_{sv}=1 & f_{sc}=0, corresponds to the lowest *COV* independently on the demand parameter considered. The latter was expected as the lower the spectral variability among the motions is (f_{sv}=1), the lower the response variability is expected to be. Monotonic trend is also observed between the response variability and the f_{sv}-f_{sc} coefficients, since increasing the contribution of the f_{sv} coefficient into the proposed ranking system (and thus, decreasing proportionally the f_{sc} contribution) the response variability keeps constantly reducing. Moreover, even in the extreme case of f_{sv}=0 & f_{sc}=1, where the spectral variability criterion is waived from the $\delta_{sv\text{-}sc}$-based rating system (and hence, the response variability is not directly controlled), the *COV* was found significantly lower (almost 50%) than the one corresponding to the conventional ranking system, δ_{conv}. The latter is attributed to the extensive involvement of the structure's dynamic characteristics (i.e., T_i, $T_{i,in}$ and Γ_i) into the current multi-criterion process. Especially, the structure-specific and weighted period range (§3, Steps 1-3), within which the spectral convergence is to be achieved, enables reduction of the resulting response variability independently on the additional reduction that may be induced by the spectral variability criterion (δ_{sv}).

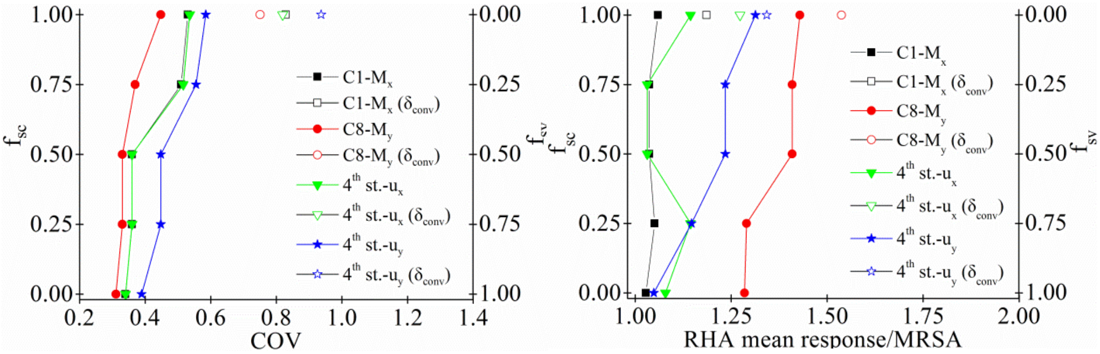

Fig. 5 Impact of the f_{sv} and f_{sc} coefficients on the response variability (left) and the average (design) values (right) for characteristic *EDPs* of the R/C building studied

Besides the response variability, the average (design) response values corresponding to the aforementioned suites of motions were presented for the five pairs of the f_{sv}-f_{sc} coefficients described above (Fig. 5, right). Response values resulted from the performance of the EC8 modal response spectrum analysis (MRSA) were used to normalize the RHA-driven design values. It was found that independently of the f_{sv}-f_{sc} pairs adopted herein, the use of the $\delta_{sv\text{-}sc}$-prioritized motions suites disfavored either underdesign (normalized design values<*1*) or significant overdesign (normalized design values>>*1*) of the structural system studied. A deeper insight into Fig. 5 reveals also that high contribution of the spectral variability criterion (f_{sv}>0.50) into the proposed rating system deterred both widely scattered response results (*COV*<0.40) and highly conservative, hence costly, design values. Nevertheless, the discussion made above concerns only a single structural system subjected to several suites of motions that are selected through the proposed multi-criterion process following a specific seismic scenario. Wider investigation is still needed to verify the response measures sensitivity, observed herein, into f_{sv}-f_{sc} weighting coefficients. For that reason, their use is currently released by the decision-support system allowing the engineer either to adopt the recommended values, i.e., $0.75 \leq f_{sv}^{recom} \leq 0.90$ and $0.10 \leq f_{sc}^{recom} \leq 0.25$, or to choose a different f_{sv}-f_{sc} pair that can assure structural response values with reduced variability (thus, increased reliability) and disfavor significantly conservative design values. In the current study, the weighting coefficients were taken equal to f_{sv} = 0.80 and f_{sc} = 0.20.

5.2 Effect of the ranking systems ($\delta_{sv\text{-}sc}$ and δ_{conv}) on the response variability

As described above (§ 4), two seismic scenarios, SSA and SSB, were considered and 77,520 EC8-compatible suites of motions were formed for each scenario via the use of the ISSARS algorithm (Katsanos and Sextos 2013). Both the conventional rating index (δ_{conv}) and the one proposed herein ($\delta_{sv\text{-}sc}$) were employed to rank these motions suites. The top 20 suites were used for the linear RHA of the multi-story building and the intra-suite variability for an extensive set of *EDPs* (i.e., bending moments of several ground floor columns and lateral displacements at each story level) was quantified via the *COV*. To summarize, 560 bi-directional linear time domain analyses were performed (i.e., one case-study building, two seismic scenarios, two rating indices, 20 suites of motions per rating system, seven pairs of seismic motions per suite) and appropriate functions of the SAP200 Application Programming Interface (API) (CSI 2011) were utilized to hasten the calculation of the demand parameters and their intra-suite variability. It should be also noted that the relevance of varying strong motions rotation angle for the RHA results, already highlighted elsewhere both for symmetric and asymmetric structures (Reyes and Kalkan 2015, Kalkan and Reyes 2015, Athanatopoulou 2005), was not considered herein.

Figure 6 plots the variability (*COV* values) for the chosen *EDPs* that were calculated via the RHA of the case-study building when subjected to the first and the 20th suites according to both the δ_{conv} and $\delta_{sv\text{-}sc}$ ranking indices. Irrespectively of the seismic scenario and the *EDP* considered, a consistent trend was revealed regarding the substantially lower variability induced by the two $\delta_{sv\text{-}sc}$-ranked suites than the corresponding δ_{conv}-ranked suites. Accounting for the entire set of the *EDPs* considered herein and the pair of seismic scenarios, the first and 20th $\delta_{sv\text{-}sc}$-ranked suites led, on average, to *COV* of 0.35 and 0.38 respectively, being almost the half of the average *COV* values that correspond to the first and 20th δ_{conv}-ranked suites (0.76 and 0.72). It is also highlighted the excessive response intra-suite variability, i.e., the *COV* is close to unity for some response parameters, driven by the conventional rating system. The latter is corroborated elsewhere (Sextos *et al*. 2011). Moreover, the lowest *COV* is associated, most of the times, with the top $\delta_{sv\text{-}sc}$-ranked suite, while there are few cases that both the first and the 20th $\delta_{sv\text{-}sc}$-ranked suites led to quite similar intra-suite variability. Hence, the proposed multi-criterion process may efficiently prioritize the motions suites that induce response results with minimum variability.

The higher efficiency of the $\delta_{sv\text{-}sc}$-based ranking system compared with the conventional one in terms of reducing the response variability was further evaluated through the calculation of the variability ratios, $COV_{\delta_{\mathrm{mod}}}^{EDP}/COV_{\delta_{sv-sc}}^{EDP}$, for the top 20 motions suites ranked using both the ranking indices. For instance, the variability of the third story lateral displacement induced by the 10th δ_{conv}-ranked suite (out of the 77,520 for the SSA) was divided with the *COV* value, being related to the 10th suite of motions according to the $\delta_{sv\text{-}sc}$-based ranking system. Along these lines, Fig. 7 plots such $COV_{\delta_{\mathrm{mod}}}^{EDP}/COV_{\delta_{sv-sc}}^{EDP}$ ratios, which highlight the significantly lower response variability that the $\delta_{sv\text{-}sc}$-ranked top 20 suites induced (instead of the δ_{convv}-ranked top 20 suites) when they were employed as the required input for the seismic analysis of the multi-story R/C building. The latter was found to be valid independently on the seismic scenario and the *EDPs* adopted, while the average $\delta_{sv\text{-}sc}$-driven decrease (1-$COV_{\delta_{\mathrm{mod}}}^{EDP}/COV_{\delta_{sv-sc}}^{EDP}$) in the response variability was almost 50% for both the seismic scenarios (i.e., 46.25 % and 48.80 % for the SSA and SSB respectively).

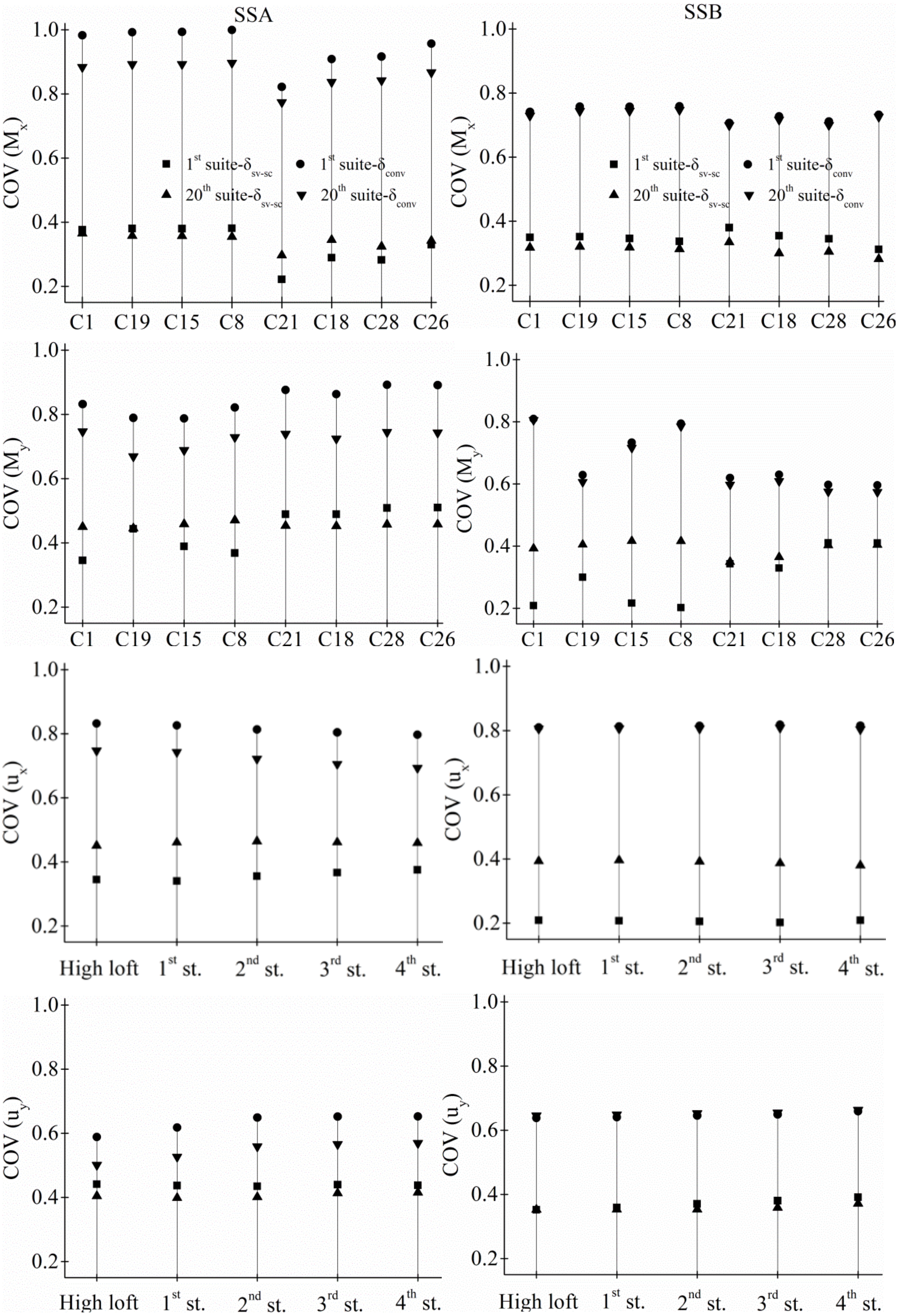

Fig. 6 Response variability corresponding to the first and 20th suites after employing the conventional (δ_{conv}) and the currently proposed ($\delta_{sv\text{-}sc}$) ranking system respectively

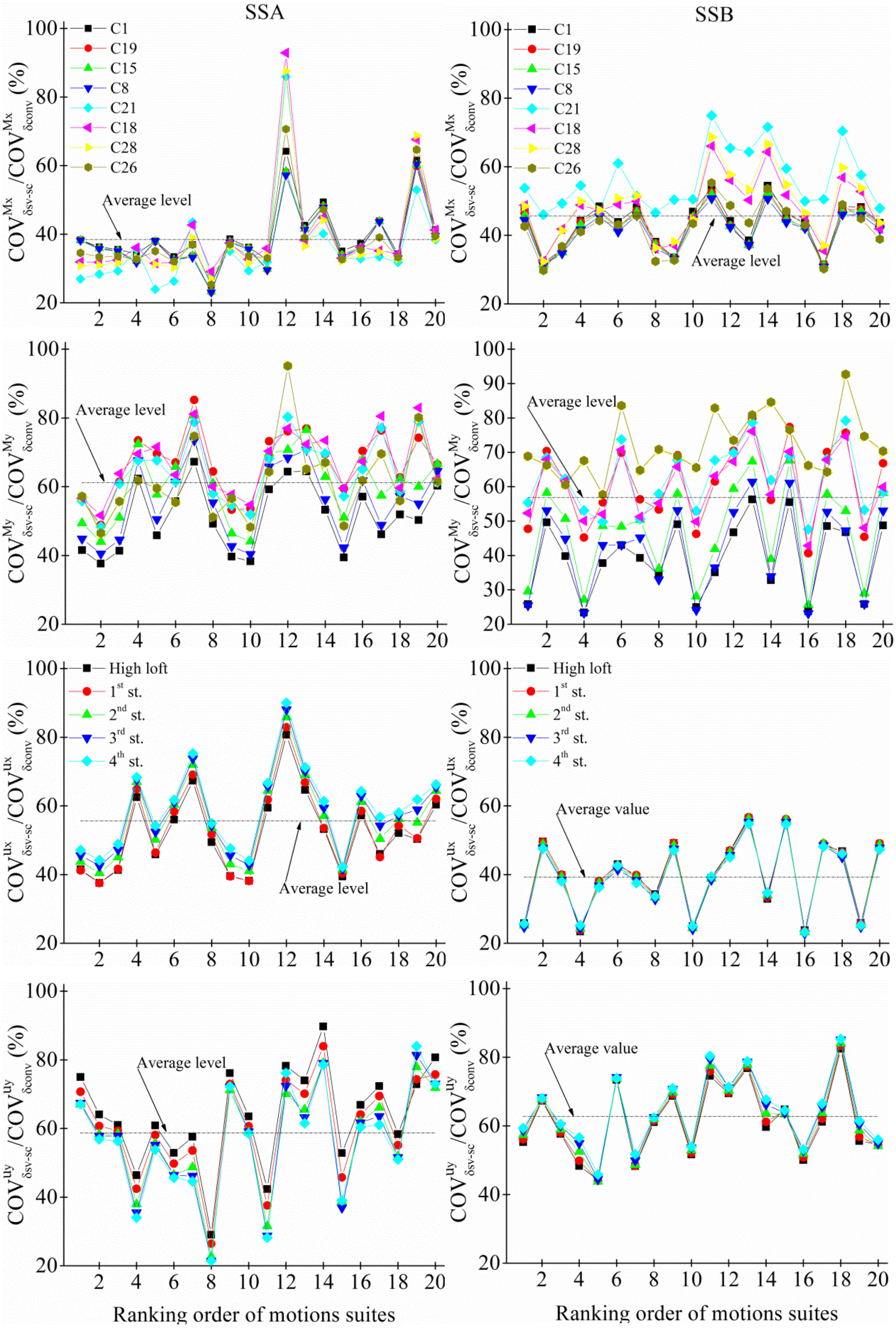

Fig. 7 Effect of the motions suites rating systems (δ_{conv} and $\delta_{sv\text{-}sc}$) on the response variability calculated for characteristic *EDPs* of the multi-story, R/C building.

5.3 Effect of the ranking systems ($\delta_{sv\text{-}sc}$ 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-criterion process and the related $\delta_{sv\text{-}sc}$ index, is 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 *SE* value 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 (Benjamin and Cornell 1970).

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 - *CL*=90% and hence, the SE 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 (Benjamin and Cornell 1970). 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 (Hancock *et al*. 2008). 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.

Along these lines, the *SE* values for the estimated average *EDPs* were calculated considering the pair of seismic scenarios and the associated top 20 suites 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 (2012) as well as recent studies (Huang *et al*. 2011, Reyes and Kalkan 2012) prescribe similar confidence level, *CL*, and SE_t to be adopted for the average (design) values. Figure 8 visualizes 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 revealing, in such a way, the superiority of the currently proposed multi-criterion process 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.

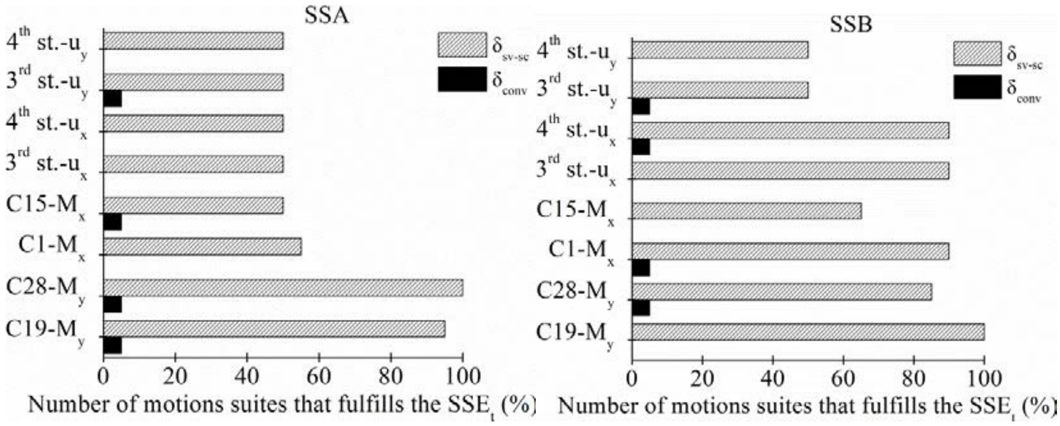

Fig. 8 Effect of the motions suites rating systems (δ_{conv} and $\delta_{sv\text{-}sc}$) on the average (design) values reliability for the multi-story, R/C building

5.4 Effect of the rating systems ($\delta_{sv\text{-}sc}$ and δ_{conv}) on the number of motions required to achieve reliable design values

Based on Eq. 12, the *SE* of an average response estimate is inversely proportional to the square root of the number of motions, *m*, included into a suite. This means that the richer motions dataset is employed for the RHA of a structure, the higher reliability is anticipated for the average (design) values. Nonetheless, the computational burden along with the time cost is also increasing with *m*; hence, such a trade-off renders the number of motions to be used for RHA a controversial issue in the earthquake engineering community (NIST 2011). Several researchers (e.g., Reyes and Kalkan 2012, Shome *et al*. 1998, Cimellaro *et al*. 2011) have already focused on using more than seven seismic records, being the typical, code-based prescription for *m*, in order to increase the reliability of the response results while, most of the times, 15 up to 20 motions were found necessary to meet reliability threshold similar to the one introduced above (Hancock *et al*. 2008, Shome and Cornell 1998). Similar findings were observed by Araújo *et al*. (2016), while they further highlighted that the number of motions required to calculate stable enough response results is dependent on the seismic demand to be calculated (e.g., global or local deformation demands) and the performance limit state (e.g., damage limitation or near collapse) adopted. As a result, it is expected that the reduction in the response variability, already seen with the use of the proposed ranking index, $\delta_{sv\text{-}sc}$, may heal the demand of using suites with numerous motions as an imperative to obtain design values with increased reliability.

To this end, a further comparative assessment was conducted between the number of earthquake motions, included into a suite, and the reliability level that they induce for the estimated average response values. Along these lines, standard error estimations were performed for the average (design) *EDPs* values, which were calculated after the RHA of the case-study building subjected to: (a) the top 20 $\delta_{sv\text{-}sc}$-ranked suites consisting of seven pairs of horizontal strong motions, i.e., m=7, and (b) the top 20 δ_{conv}-ranked suites consisting of 15 pairs of earthquake motions. i.e., m=15. Regarding the former "excitation case", the required *SE* estimations have been already calculated for the purposes of identifying the effect of the 20 $\delta_{sv\text{-}sc}$-based ranking system on the design values (§ 5.3). However, an additional analysis scheme was required to be carried out for the latter "excitation case". Especially, for each seismic scenario considered, the already mentioned 20 pairs of earthquake motions (§ 4, see also Table 3 in Annex A) were re-utilized by the ISSARS algorithm to form and then select the top 20 δ_{conv}-ranked suites out of the 15,504 ones consisting of 15 seismic motions pairs. Afterwards, 600 additional linear time domain analyses (i.e., one case-study building, two seismic scenarios, one rating index, 20 suites of motions, 15 pairs of seismic motions per suite) were performed enabling the the evaluation of the required *SE* estimations.

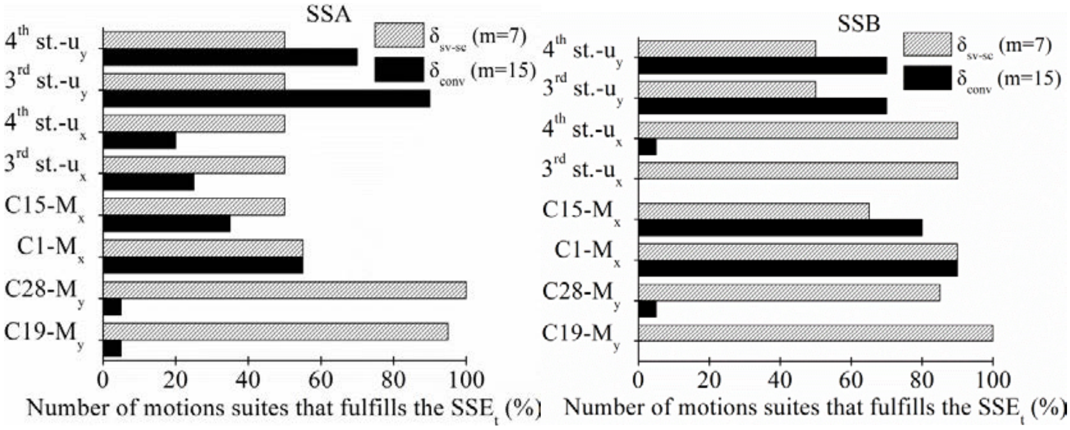

Fig. 9 Effect of the rating systems (δ_{conv} and $\delta_{sv\text{-}sc}$) on the number of motions included into the suites in terms of achieving reliable (design) results for the multi-story, R/C building

Likewise to Fig. 8, two sets of horizontal bars are presented in Fig. 9 for the chosen set of *EDPs* investigated herein. The grey bars are identical to the ones of the same color plotted in Fig. 8 (i.e., they refer to top 20 $\delta_{sv\text{-}sc}$-ranked suites with m=7). Moreover, the black horizontal bars (Fig. 9) demonstrate the number of δ_{conv}-ranked suites with m=15 that are associated with *SE* estimations for the average *EDPs* lower than the SE_t=0.30 for confidence level equal to 90%. Based on Fig. 9, it is seen that the use of the conventional ranking system for suites with m=15 led, on average, to lower reliability-oriented performance than the $\delta_{sv\text{-}sc}$-based prioritization system even when the latter was applied for suites containing less than the half of motions (m=7, motions' decrease equal to 114,3%). Particularly for the SSB, the imposed SE_t=0.30 for *CL* of 90% was fulfilled, on average, by the 40% of the top 20 δ_{conv}-ranked suites with 15 pairs of motions. On the contrary, the reliability criterion successful rate was calculated almost twice, i.e., 77.50%, for the top 20 suites with seven pairs of motions that were prioritized according to $\delta_{sv\text{-}sc}$-based ranking system. Similar findings are observed when the first seismic scenario is considered (i.e., 38.13% for the δ_{conv}-ranked suites with m=15 *vs* 62.50% for the $\delta_{sv\text{-}sc}$-ranked suites with m=7). Hence, the use of the currently proposed multi-criterion process for selecting earthquake motions can lead, via the RHA of structures, to reliable design values releasing, at the same time, the time and computational costly requirement of employing an increased number of motions per suite formed.

6. Conclusions

A decision support system is presented herein in order to facilitate the intricate task of selecting and scaling earthquake ground motions as required for the time domain seismic analysis. Earthquake ground motions suites are provided that fully conform to the current normative framework while, at the same time, induce, via the RHA of structural models, 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 process described herein, which may also be used to evaluate the seismic performance of structures under a target spectrum, incorporates a multi-criterion 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, dual 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 proposed ranking index, $\delta_{sv\text{-}sc}$, was quantified through its comparative assessment with the conventional index, δ_{conv}, which is widely used by existing frameworks related to earthquake motions selecting and scaling procedures. It is noted that the proposed method is quite generic, since it can be essentially applied for any target spectrum prescribed either from codes or derived by site-specific seismic hazard analysis, while it is applicable for any structure as long as its dynamic characteristics are known in advance.

The main conclusions from this comparison scheme, based on RHA response results of a multi-story, R/C building, are briefly summarized below:

(a) Significantly lower (almost 50%) intra-suite response variability was calculated when the case-study building was subjected to the top $\delta_{sv\text{-}sc}$-ranked motions suites instead of using the ones prioritized by the conventional index.

(b) The latter was observed independently on the seismic scenarios and the *EDPs* adopted in the analysis increasing, in such a way, the efficiency and validity of the currently proposed process. Wider analysis scheme is, though, required to verify the findings mentioned above for different structural configurations.

(c) Over than 62% (on average) of the top $\delta_{sv\text{-}sc}$-ranked suites fulfilled the reliability criterion imposed herein that was marginally met by the highly prioritized suites using the conventional approach.

(d) Even in case of using suites with 15 pairs of earthquake motions, the conventional approach performed significantly worse in reliability terms than the proposed multi-criterion process, which was used to prioritize suites consisting of seven pairs of earthquake records. Hence, the reliability-driven, though burdensome, requirement for richer suites of motions may be released applying the current approach for selecting and scaling seismic motions.

It should be, finally, mentioned that the efficiency of the proposed method to credit motions suites that induce stable enough response results should be further investigated for nonlinear RHA, since in this case, the spectral variability control for the motions suites along with their convergence with the target spectrum cannot de facto lead to inelastic response results with limited variability. Indeed, more strong ground motion and structural-based proxies than the spectral shape are necessary to control efficiently the nonlinear response and hence, the relevant variability.

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Annex A

Table 3 Earthquake events and the associated strong ground motions used in the study

	Earthquake event (Date)	Earthquake magnitude M_w	Recording station	Epicentral distance R (km)	Soil type[†]	PGA[††] (g)
Seismic Scenario A (SSA)	Hollister (09.04.1961)	5.60	Hollister City Hall	20.61	D	0.121
	North California (10.12.1967)	5.60	Hollister City Hall	29.73	D	0.190
	Coyote Lake (06.08.1979)	5.74	Gilroy Array #2	10.94	D	0.294
	Livermore (24.01.1980)	5.80	Del Valle Dam (Toe)	26.79	D	0.173
	Mammoth Lakes (25.05.1980)	6.06	Long Valley Dam (Upr L)	12.65	D	0.340
	Mammoth Lakes (25.05.1980)	5.91	Long Valley Dam (Upr L)	11.51	D	0.329
	Westmorland (26.04.1981)	5.90	Parachute Test Site	20.47	D	0.219
	Coalinga (22.07.1983)	5.77	Coalinga-14th & Elm	11.21	D	0.454
	Coalinga (22.07.1983)	5.77	Pleasant Valley P.P. - Yard	16.17	D	0.427
	Mountain Lewis (31.03.1986)	5.60	Halls Valley	15.86	D	0.161
	N. Palm Springs (08.07.1986)	6.06	North Palm Springs	10.57	D	0.590
	Chalfant Valley (21.07.1986)	6.19	Zack Brothers Ranch	14.33	D	0.425
	Whittier Narrows (01.10.1987)	5.99	Downey - Birchdale	15.29	D	0.298
	Whittier Narrows (01.10.1987)	5.99	Santa Fe Springs - E. Joslin	11.73	D	0.433
	Big Bear (28.06.1992)	6.46	Big Bear Lake - Civic Center	10.15	D	0.503
	Upland (28.02.1990)	5.63	Pomona - 4th & Locust FF	10.82	D	0.201
	Sierra Madre (28.06.1991)	5.61	LA - Obregon Park	29.61	D	0.224
	Northwest China (11.04.1997)	6.10	Jiashi	19.11	D	0.293
	Chi-Chi, Taiwan (20.09.1999)	5.90	TCU073	10.30	D	0.136
	Chi-Chi, Taiwan (20.09.1999)	6.20	CHY101	27.97	D	0.152
Seismic Scenario B (SSB)	Northern California (21.12.1954)	5.60	Ferndale City Hall	30.79	D	0.186
	San Fernando (09.02.1971)	5.60	LA - Hollywood Stor FF	39.49	D	0.210
	Imperial Valley (15.10.1979)	5.74	Delta	33.73	D	0.285
	Trinidad & Tobaco (08.11.1980)	5.80	Rio Dell Overpass	76.75	D	0.152
	Superstition Hills (24.11.1987)	6.06	El Centro Imp. Co. Cent	35.83	D	0.293
	Spitak, Armenia (07.12.1988)	5.91	Gukasian	36.19	D	0.207
	Loma Prieta (18.10.1989)	5.90	Gilroy Array #3	31.40	D	0.462
	Loma Prieta (18.10.1989)	5.77	Gilroy Array #7	39.88	D	0.312
	Cape Mendocino (25.04.1992)	5.77	Eureka - Myrtle & West	53.34	D	0.167
	Landers (28.06.1992)	5.60	North Palm Springs	32.26	D	0.131

	Earthquake event (Date)	Earthquake magnitude M_w	Recording station	Epicentral distance R (km)	Soil type[†]	PGA[††] (g)
Seismic Scenario B (SSB)	Northridge (17.01.1994)	6.06	LA - Obregon Park	39.39	D	0.467
	Northridge (17.01.1994)	6.19	LA - S grand Ave	33.77	D	0.273
	Kobe, Japan (16.01.1995)	5.99	Shin-Osaka	45.97	D	0.229
	Kobe, Japan (16.01.1995)	5.99	Takarazuka	38.60	D	0.707
	Kocaeli, Turkey (17.08.1999)	6.46	Iznik	39.82	D	0.111
	Chi-Chi, Taiwan (20.09.1999)	5.63	CHY101	31.96	D	0.382
	Chi-Chi, Taiwan (20.09.1999)	5.61	WGK	31.96	D	0.387
	Duzce, Turkey (12.11.1999)	6.10	Bolu	41.27	D	0.766
	Manjil, Iran (20.06.1990)	5.90	Abhar	77.84	D	0.170
	Hector Mine (16.10.1999)	6.20	Amboy	47.97	D	0.194

[†] According to the NEHRP site classification (FEMA 450, 2003): Site class A ($v_{s,30}$≥1500 m/s), B (760 m/s<$v_{s,30}$<1500 m/s), C (360 m/s<$v_{s,30}$≤760 m/s), D (180 m/s<$v_{s,30}$≤360 m/s) and E ($v_{s,30}$≤180 m/s).
[††] The peak ground acceleration (PGA) is the geometric mean derived from the two orthogonal horizontal components orientated randomly.

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