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Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective

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ABSTRACT

This paper reviews alternative selection procedures based on established methods for incorporating strong ground motion records within the framework of seismic design of structures. Given the fact that time history signals recorded at a given site constitute a random process which is practically impossible to reproduce, considerable effort has been expended in recent years on processing actual records so as to become 'representative' of future input histories to existing as well as planned construction in earthquake-prone regions. Moreover, considerable effort has been expended to ensure that dispersion in the structural response due to usage of different earthquake records is minimized. Along these lines, the aim of this paper is to present the most recent methods developed for selecting an 'appropriate' set of records that can be used for dynamic analysis of structural systems in the context of performance-based design. A comparative evaluation of the various alternatives available indicates that the current seismic code framework is rather simplified compared to what has actually been observed, thus highlighting both the uncertainties and challenges related to the selection of earthquake records.

Contents

1. Introduction

It is now well established that elastic analyses of structures subjected to seismic actions, typically in the form of response spectra, do not always predict the hierarchy of failure mechanisms. It is also not possible to quantify the energy absorption and

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force redistribution phenomena that result from gradual plastic hinge formation in a structure. This information can only be obtained by studying the inelastic structural response in the time domain and is critical in the assessment of existing or new structures of major importance (i.e., high-rise buildings, storage tanks and nuclear power plants), complexity (coupled soil–structure systems, massive and irregular buildings), high degree of inelasticity (i.e., structures designed to exhibit large deformations) and geometrical nonlinearity (i.e., base-isolated structures). Linear elastic, dynamic time history analysis is very useful when dominant modes of vibration are closely spaced [1] or for multiply supported structures (i.e., bridges) where higher modes are excepted to be excited [2,3] due to the asynchronous nature of the incoming seismic waves.

During the last decade, elastic and inelastic dynamic analyses in the time domain have been made feasible for complex structures with thousands of degrees of freedom, thanks to rapidly increasing computational power and the evolution of engineering software. As a result, time-domain analysis is prescribed in the vast majority of modern seismic codes. On the other hand, recent work has shown that among all sources of uncertainty stemming from the material properties (concrete, steel and the supporting soil), the design assumptions and the earthquake-induced ground motion, the latter seems to be the most unpredictable [4] and has a significant impact on the variability observed in the structural response [5]. In fact, ground motions appear random in space and time, due to the inherent complexity of the path that seismically induced waves follow as they travel from the fault-plane source through bedrock [6] and finally through the soil layers to reach the foundation level of a structure [7].

Given the above uncertainties, and despite the relatively straightforward seismic code framework regarding transient dynamic analysis, it is still the designer's responsibility to find a 'reasonable' way for selecting a set of 'appropriate' earthquake records, a task that is technically easy, but at the same time difficult since any discrepancy in the computed structural response must be kept reasonably low. This is a complex task that cannot be accomplished on a 'trial and error' basis without an understanding of the basic concepts behind selection and scaling of earthquake records for use in dynamic analysis. In other words, the current code framework for ground motion record selection is considered rather simplified compared to the potential impact of the selection process on the dynamic analysis, thus giving the false impression that structural analysis results are as robust as the refined finite element model used allows them to be.

Thus, the aim of this paper is to present a state-of-the-art review on currently available methods for selecting and scaling ground motion records, in order to scrutinize the theoretical background of the problem and investigate its implications on analysis and design. This comparative evaluation also includes modern seismic design codes in an attempt to analyze the uncertainties inherent in the code provisions and highlight the necessity for more detailed and targeted guidelines regarding dynamic structural analysis in the time domain.

2. Selection of recorded ground accelerations for seismic design

As already mentioned, the structural design cycle requires earthquake loads to be represented either by a smooth response spectrum or by recorded acceleration time histories as input to a (linear or nonlinear) dynamic analysis. In the latter case, input motions have to be selected so as to represent regional seismicity and must conform to expected (or design) earthquakes. In other words, real records have to fulfill anticipated earthquake scenarios in order to be used in the framework of transient dynamic analysis.

Several studies have been published proposing methods for identifying earthquake scenarios that can be derived through seismic hazard analysis (SHA), which is linked to the estimation of some key measures of strong ground motion expected to occur during a pre-defined time interval at a given site [8]. There are two basic methodologies for SHA, namely the deterministic and probabilistic approaches. In the deterministic seismic hazard analysis approach (DSHA), strong-motion parameters are estimated for the maximum credible earthquake, which is assumed to occur at the closest possible distance from the site of interest, making allowance for the seismotectonic makeup of the surrounding area and for all available data on past earthquakes from this region [9,10]. The magnitude and distance combination, which yields the largest ground motion amplitude at the site of interest are the DSHA results that can be used in record selection.

On the other hand, if probabilistic seismic hazard analysis (PSHA) is employed to estimate earthquake loads, definition of the design earthquake scenario is no longer straightforward. More specifically, PSHA carries out an integration over the total expected seismicity during a given exposure period to provide an estimate of strong-motion parameters with a specified confidence level [11–15]. By performing PSHA in order to generate hazard curves, i.e., plots of spectral acceleration for various return periods, a site-specific uniform hazard response spectrum can be calculated [16] that is used for establishing spectral matching with a pre-selected family of records. Unfortunately, the physical meaning of an earthquake in terms of magnitude (M), source-to-site distance (R) as well as ground motion deviation (ε), is lost in a PSHA. In order to bring about a physical interpretation of the PSHA results and to accommodate engineering-type decisions, it is desirable to find a representative earthquake that is compatible with the results of the PSHA approach [17]. This can be achieved through de-aggregation (or disaggregation) of the computed probabilistic seismic hazard [18,19]. Thus, the M–R–ε sets that reflect dominant earthquakes for a particular site can be specified by this process.

In sum, the results of a SHA, performed by either deterministic or probabilistic approaches, are needed by engineers to tackle the problem of selecting and scaling recorded accelerograms.

2.1. Record selection based on earthquake magnitude (M) and distance (R)

Since earthquake magnitude (M) and distance (R) (in km) of the rupture zone from the site of interest are the most common parameters related to a seismic event, it is evident that the simplest selection procedure involves identifying these characteristic (M, R) pairs. Along these lines, Shome et al. [20] formed sets of real accelerograms (often denoted as 'a bin of records' in the literature) in order to assess the nonlinear seismic response of a five-story building. Record selection was made on the basis of four different magnitude–distance pairs, permitting a limited variation in the target values ($M\pm\Delta M$, $R\pm\Delta R$).

Furthermore, Youngs et al. [21] developed the Design Ground Motion Library (DGML), an electronic library of pre-selected recorded acceleration time histories considered suitable for use in the dynamic analyses of structures. These accelerograms were selected from the California Strong Motion Instrumentation Program (CSMIP) [22] and the PEER database [23]. Sets of records were formed for specific earthquake magnitudes and closest source-to-site distances and were derived from either a DSHA or a PSHA as dominant contributions to the site hazard through de-aggregation. The total number of the record sets produced was 26.

Bommer and Acevedo [24], Stewart et al. [25] and others also consider magnitude as an important earthquake record selection parameter, or at least as an initial criterion for use in the selection process, while the role of the closest source-to-site distance has not been established. Within this framework, band-widths for the magnitude of earthquake scenarios must be set; Stewart et al. [25] suggested a magnitude half-bin width of $\pm 0.25M_w$, while Bommer and Acevedo [24] recommended $\pm 0.20M_w$. Thus, if the magnitude search window can be kept as narrow as possible, the distance range can then be widened.

Nevertheless, recent studies have questioned the effectiveness of the (M, R) based selection method, primarily because of unstable results in the structural response observed after direct use of earthquake record sets based on this particular criterion as input to nonlinear dynamic analyses. More specifically, Iervolino and Cornell [26] studied the dependence of structural response on common earthquake parameters such as the (M, R) pair. In their work, real accelerograms were arranged in two main classes. The first comprised six sets of ten accelerograms formed from available records characterized by comparatively high magnitudes and small distances (defined as the closest distance to a rupture greater than 15 km), thus simulating the case of a carefully chosen scenario. The records in the second class comprising ten 'arbitrary' sets of ten accelerograms were chosen randomly from a large catalog of real records without limitation, other than being scaled to the median, first-mode spectral acceleration period of the first class of 'target' sets. In order to generalize their conclusions, Iervolino and Cornell [26] chose a series of structures with different (a) fundamental periods, (b) hysteresis relationships, (c) target ductility, (d) number of degrees of freedom (i.e., single (SDOF) and multiple (MDOF)) and (e) structural type (i.e., concrete or steel skeletons). The key response parameter considered was the maximum drift and inter-story drift of the SDOF and MDOF systems, respectively. Conclusions based on investigating the nonlinear response of these model structures to sets of records selected by matching a specific moderate magnitude and distance scenario and other moderate-magnitude records selected arbitrarily, showed no evidence that the first, site-specific (M, R) record pair selection process was superior in terms of measured structural response discrepancy.

Moreover, Shome et al. [20] pointed out the insensitivity of some post-elastic damage indices (i.e., three local and three global measures) to the (M, R) parameter pair of a four-story steel structure subjected to real records. In particular, the choice of the (M, R) pair in record selection was shown to be of no consequence in the estimation of deformation-based damage indices. This conclusion, however, is not quite valid in the case of cumulative damage measures (i.e., energy-based indices), since these damage models show some form of dependency on record duration. The exact influence that record duration exerts in predicting nonlinear response and consequently in the selection process is clarified in the next section.

Shome and Cornell [27] also confirmed the weak dependence of structural response on the (M, R) pair by focusing on MDOF systems that are characterized by a low first natural period. Furthermore, other studies [28–33] verified the general independence of nonlinear structural displacements to distance R, while at the same time estimated the sensitivity to magnitude M as low. The latter insensitivity may not be valid in the case of tall buildings with important higher mode effects [20]. By quantifying the effects of seismological parameters (M, R) on the structural response through regression analyses, Baker and Cornell [34] corroborated the above conclusions. In particular, their study confirmed that the closest source-to-site distance is statistically insignificant to the structural response, while the earthquake magnitude indeed shows some significance.

In sum, since earthquake magnitude exerts an influence on various response quantities, it can be considered as an accepted criterion in the selection process of the real records for performing an initial refinement of the selected accelerograms. In contrast, (source-to-site) distance derived from earthquake scenarios has been proven an inadequate predictor of structural response and is therefore considered as a supplementary criterion in the selection procedure. Nevertheless, both parameters are still used in practice for the arrangement of the records into sets, since the (M, R) seismological pair is a concept familiar to structural engineers.

2.2. Additional record selection criteria

2.2.1. Soil profile

A selection criterion complementing both earthquake magnitude and distance in the search window is the actual soil profile S at the site of interest, leading to (M, R, S) record sets [21,35]. The geotechnical profile is known to influence seismic motions by modifying both their amplitude and the computed response spectra [24]. In order to introduce the soil profile into the selection process, site classification and strong-motions recording sites must be known with a high degree of confidence. Generally speaking, shear-wave velocity at the uppermost 30 m ($V_{S,30}$) can be used as a suitable metric for site classification, although there are cases where deeper soil structure can also exert a strong influence [36]. Alternatively, site classification can be done according to seismic codes provisions and well-established soil categorization schemes may be utilized for the record selection process. For instance, in selecting earthquake ground motions for tall building design, Lee et al. [36] used soil categories such as S_1 (rock), S_2 (stiff soil) and S_3 (soft soil) in accordance with the United States Geological Survey (USGS) database [37].

By compiling a strong-motion databank with nearly 1600 accelerograms recorded between 1933 and 1995, Bommer and Scott [38] quantitatively assessed the influence of soil profile to the selection procedure. More specifically, they noticed that the (M, R, S) selection process drastically reduces the number of records eligible for selection when compared to the simpler (M, R) parameter pair. Based on this observation, they suggested that there may be cases where it would be advisable (or even inevitable) to relax the matching criteria for site classification in order to ensure a reasonable number of records will be made available [24]. Notwithstanding the necessity to adopt this pragmatic attitude, consideration of local soil profile remains an important selection parameter.

Finally, in order to clarify the role played by liquefaction-susceptible soil profiles, Kwon et al. [39] introduced magnitude and distance criteria to select appropriate records at bedrock level in the analysis of the Meloland Road Overcrossing (MRO) bridge. Next, surface motions were derived through a separate nonlinear site response analysis, making allowance for soil liquefaction and thus bypassing the lack of earthquake records in liquefiable soils. This alternative approach, however, has some inherent limitations that are associated with the relative paucity of available records from rock sites, the more elaborate manipulations required in site response analysis, and the sensitivity of analytically derived surface motions on the assumptions made regarding the mechanical properties of the liquefiable soils.

2.2.2. Strong motion duration

Apart from the soil profile, strong-motion duration constitutes a complementary criterion for the selection of real records. As indicated in Ref. [38], duration of ground shaking is typically controlled by duration of the fault rupture, which is in turn a function of magnitude that has already been taken into consideration. Hancock and Bommer [40] have stated that "duration is a secondary predictive parameter and to explore direct correlations between duration and damage would be futile". Indeed, strong motion duration affects the various types of damage indices in different ways. More specifically, damage measures based on peak response do not depend on duration, while damage measures such as absorbed hysteretic energy and fatigue damage are correlated with this particular parameter. Iervolino et al. [41] have confirmed this last statement by examining the nonlinear response of single DOF systems. By selecting real accelerograms (three sets comprising 20 records characterized by short, moderate and large duration) as representative of specific duration scenarios, they concluded that duration is insignificant for displacement-based demand indices, while it influences energy-based models such as hysteretic ductility and equivalent number of cycles [41]. Shome et al. [20] as well as Hancock and Bommer [40] drew the same conclusions by, respectively, investigating the nonlinear response of a five-DOF steel structure and an eight-storey RC wall-frame building.

In a review paper on the influence of duration on structural damage, Hancock and Bommer [42] pointed out that this phenomenon is also dependent on the structural model itself. Indeed, structures exhibiting stiffness or/and strength degradation under cyclic loading will be more sensitive to the number of cycles of motion and hence to the duration of shaking. It is interesting to note here that despite a plethora of more than 40 different definitions for strong-motion duration that have been proposed [43], engineers are deterred from using them as their main selection parameter, a fact that can be also attributed to their reluctance to decide on a specific duration definition among the numerous ones available. In closing, some modern seismic codes such as the ASCE Standards 4-98 [44] propose that duration of selected records shall be representative of the expected ground-motion at a site for a given level of seismic hazard. Furthermore, Malhotra [45] proposed that the selection process should be based on both spectral shape and significant duration. In any case, adoption of strong-motion duration in the record selection process must always be carefully justified.

2.2.3. Seismotectonic environment and other geophysical/seismological parameters

An important issue in the literature is consideration of regional seismotectonic features and the degree to which these are reflected on the ground motions recorded during a seismic event. These effects comprise rupture mechanism, source environment, type of faulting, source path and directivity of seismic waves, and their influence on strong motions has been observed in a plethora of studies. For example, Kawaga et al. [46] mentioned that ground motions generated by buried rupture-type earthquakes in the period range around 1.0 s are significantly stronger than ground motions caused by surface rupture events. Campbell [6] also demonstrated some differences in the predicted response spectra derived from strong motions in shallow stable regions (Europe, Eastern USA) and in subduction zones (Japan, New Zealand, Taiwan), while Bolt and Abrahamson [47] as well as Lin and Lee [48] showed that peak ground motions from subduction zone earthquakes generally attenuate more slowly than those from shallow crust earthquakes in tectonically active regions. Also, Kappos et al. [49,50] explored the effect of different seismotectonic environments on earthquake strong motions by studying elastic and inelastic spectra for structural strength and displacements. Moreover, information generated from the predictive equations developed in the NGA project [51] demonstrated that earthquakes with reverse-oblique mechanism have similar characteristics to those from reverse faulting events. Investigating the extent to which new ground motion prediction equations from the NGA could be applied to seismic hazard analyses in Europe and the Middle East, Stafford et al. [52] concluded that there are no systematic differences between ground motions in western North America versus those in Europe and Middle East. As a result, it seems logical to infer that it could be beneficial to combine these two datasets so as to increase the pool of available records for dynamic analyses.

The impact that various features of the seismotectonic environment exert on strong motions led many researchers to use these parameters in the earthquake record selection process. Along these lines, Sorabella et al. [53] assessed the nonlinear dynamic response of structures in Boston, Massachusetts (Eastern North America—ENA), which is an area characterized by low values of seismic hazard. For this, they had to enrich the limited number of records from ENA with records from Western North America (WNA) or from a similar tectonic environment. Dhakal et al. [54] also proposed some specific criteria, such as earthquake magnitude, closest site-to-source distance, seismotectonic region characteristics and local soil conditions for record selection purposes. However, a balance needs to be reached [24] in considering the 'overall' selection process, because inclusion of criteria using seismotectonic environment features may significantly reduce the acceptable number of records required in nonlinear dynamic analyses. Apart from that, further refinement of the search window may be ineffective because the impact of the seismotectonic environment on the structural response quantities is still not well understood.

2.2.4. Acceleration to velocity ratio (a/v)

By considering the seismotectonic features and site characteristics of the location where strong motion has been recorded, the maximum acceleration over maximum velocity (a/v) ratio has been proposed as a complementary measure for the selection process. In particular, Tso et al. [55] and Sawada et al. [56] studied the (a/v) ratio for strong ground motions and concluded that this parameter is related to earthquake magnitude, distance from source plus frequency content of accelerograms. Consequently, selecting accelerograms with ratios that can be grouped as 'low' ($a/v < 0.80$ g/m/s), 'intermediate' ($0.80 < a/v < 1.20$ g/m/s) and 'high' ($a/v > 1.20$ g/m/s), is believed to cover a wide range of earthquakes [57] that are independent of the particular site in question and yield a conservative upper bound of expected ground motion. However, it is noted that this particular index cannot be considered as the sole criterion for a selection process, but rather as a conservative upper bound of the expected ground motion.

2.3. Record selection based on spectral matching

The selection of real accelerograms is often performed on the basis of compatibility between their response spectra and a corresponding 'target' spectrum as defined by code provisions or computed directly through a PSHA. Spectral matching is the most commonly proposed earthquake record selection method by seismic codes and, as such, can be utilized in the framework of both force-based and performance-based design. Spectral matching should not be confused with the generation of spectrum-compatible artificial accelerograms, as the latter require specialized software; furthermore, they do not constitute real seismic waves and as a result carry unreasonably high energy content. Spectral matching is usually considered a second-level selection criterion, following an initial selection based on magnitude and distance. It must be mentioned here that the term 'spectral matching' is also utilized in some recent studies to describe special techniques (e.g., wavelets) for establishing spectral convergence [58]. However, the terminology used herein corresponds to the widely used method for selecting real records based on shape compliance between the response and target spectra.

When spectral matching is sought within a seismic code framework, the target spectrum is the elastic (or design) spectrum. Along these lines, Ambraseys et al. [59] through the European strong-motion databank proposed Eq. (1) as a means to verify spectral compatibility of a given record with the target one:

$$D_{rms} = \frac{1}{\mathrm{N}}\sqrt{\sum_{i=1}^{N}\left(\frac{S\alpha_0(T_i)}{PGA_0} - \frac{S\alpha_s(T_i)}{PGA_s}\right)^2} \tag{1}$$

In the above equation, N is the number of periods at which the spectral shape is specified, $S\alpha_0(T_i)$ is the spectral acceleration of the record at period T_i, $S\alpha_s(T_i)$ is the target spectral acceleration at the same period value, while PGA_0 and PGA_s are the peak ground acceleration of the record and the zero-period anchor point of the target spectrum, respectively. A small value of D_{rms} indicates closer matching between the shapes of the record and target spectra. In general, the value of D_{rms} depends on the size of the earthquake record databank and the number of records required. It is also dependent on the period range of interest that must be specified for spectral matching, with a shorter range being preferable to a longer one. For instance, Bommer et al. [60] noticed that when spectral matching was sought in the range of moderate to long periods (0.4–0.8 s), the values of D_{rms} required for returning 30 accelerograms were of the order of 0.15. In contrast, when matching was done for shorter periods (0.1–0.3 s), D_{rms} was as low as 0.06, which implies spectral compatibility was easier to be achieved.

In order to avoid using peak ground acceleration as the normalizing parameter in Eq. (1), since it may not be relevant to longer period spectral ordinates, Iervolino et al. [61] proposed an alternative expression for the average spectrum deviation δ (i.e., a quantitative measure of how far the mean spectrum of a record set deviates from the code spectrum) as

$$\delta_i = \sqrt{\frac{1}{\mathrm{N}}\sum_{i=1}^{N}\left(\frac{S\alpha_j(T_i) - S\alpha_{REF}(T_i)}{S\alpha_{REF}(T_i)}\right)^2} \tag{2}$$

In the above equation, $S\alpha_j(T_i)$ represents the pseudo-acceleration ordinate of the real spectrum j at period T_i, $S\alpha_{REF}(T_i)$ is the value of the spectral ordinate of the code spectrum at the same period, and N is the number of values used within a pre-defined range of periods.

Furthermore, the need to efficiently match the target spectrum over the longer period range, which is of primary interest in many structural engineering problems, led Beyer and Bommer [62] to modify Eq. (1) by proposing a scale factor (α) for each record that minimizes the root-mean-square difference D_{rms} between the scaled geometric mean spectrum of the real record and the target spectrum:

$$D_{rms} = \sqrt{\frac{1}{k-j+1}\sum_{i=j}^{k}(aS\alpha_R(T_i) - S\alpha_T(T_i))^2} \tag{3}$$

In the above equation, T_j and T_k define the period range for which spectral matching is sought.

Along similar lines, Malhotra [45] presented a procedure to select and scale real records on the basis of spectra matching with the site response spectrum as proposed by the International Building Code [63]. The 'smooth' spectrum, originally proposed by Newmark and Hall [64], is defined by nine parameters, i.e., peak ground acceleration (PGA), maximum spectral acceleration ($S\alpha_{max}$), corner periods (T_1-T_6) and peak ground displacement (PGD). Malhotra [45] recommended a simplified smooth spectrum, defined by just three parameters: $S\alpha_{max}$ and corner periods (T_3-T_4). These parameters are adequate to describe the amplitude and frequency content of strong motion, i.e., $S\alpha_{max}$ characterizes high-frequency content while T_3-T_4 capture the contribution of medium to low frequencies. To these was added the significant duration of strong motion parameter (D_s) proposed by Trifunac and Brandy [65], since structures may fail in low cycle fatigue [66]. The four parameters were computed for numerous ground motions, and accelerograms were selected so as to comply with corresponding target values indicative of the dominating seismic events, derived by a site-specific de-aggregation of PSHA.

Spectrum compatibility as a means for selecting earthquake records was also investigated by numerous other researchers who focused on the nonlinear dynamic response of buildings [53,62,67]. Hancock et al. [68] implemented comprehensive analyses for evaluating the efficiency of different linear scaling and spectral matching procedures related to the required number of accelerograms in order to predict the expected response within a specified period interval at a given confidence level. They also evaluated bias in the inelastic response estimate, and used spectral matching as an initial way to select the necessary accelerograms. The spectral match was assessed using the RMS difference proposed by Ambraseys et al. [59]. Furthermore, the feasibility of spectral matching within the framework of Eurocode 8-Part 2 [69] provisions for bridges was also studied [70].

An alternative to spectral matching for use in nonlinear dynamic analysis was proposed by Shantz [71]. This selection procedure is based on matching the surface defined by the maximum displacement response ($D_{max-rec}$) of inelastic single DOF oscillators over a range of periods (T) and ductilities (μ) as estimated for a particular record, with the target displacement surface, defined over a specific region in the $T-\mu$ space. The target (or design) displacement surface is calculated as

$$D_{design}(T,\mu) = \mu_{NIDD(T,\mu)}S_{d-A.R.}(T) \tag{4}$$

$$NIDD(T,\mu) = \frac{D_{\max-rec.}(T,\mu)}{S_{d-A.R.}(T)} = \frac{D_{\max-rec.}(T,\mu)}{S_{d-rec.}(T)}\frac{S_{d-rec.}(T)}{S_{d-A.R.}(T)} = C_\mu(\mathrm{T},\mu)C_a(T) \tag{5}$$

where $S_{d\text{-}A.R.}(T)$ is the mean spectral displacement at the specified structural period estimated by an attenuation relationship using parameters such as magnitude and distance that are consistent with the seismic scenario. The median values of the normalized inelastic displacement demand $NIDD(T,\mu)$ are determined by using regression models for an extended area in the $T-\mu$ space ($0.5 \le T \le 5$ and $2 \le \mu \le 8$), and for 1827 earthquake records retrieved from the Next Generation Attenuation (NGA) dataset [51,72]. Having defined both a target displacement surface and a displacement surface for an unscaled record, Shantz [72] then recommended the use of a scale factor for better fit between these two surfaces.

When this method was used to select records for the dynamic loading in the transverse direction of the LADWP East Aqueduct Bridge [73], there was significant reduction in the dispersion of results obtained by two other selection methods that were based on magnitude–distance criteria. Therefore, the number of records required to achieve stable estimates of mean response values was also lower. However, this particular method is appropriate for structures where response is dominated by the first mode and furthermore depends on the engineer's judgment to determine optimal size, shape and location of the target displacement surface.

2.4. Record selection based on ground motion intensity measures

At the core of recent developments in performance-based seismic design lies the computation of the mean probability of exceedance of a particular value for a chosen demand measure or a specified limit state, denoted as $\lambda_{DM}(x)$. Estimation of this probabilistic measure for a particular return period of a specific seismic event controls the accuracy of the desired response variable estimate. Thus, selection of recorded earthquake ground motions for dynamic excitation of a particular structure must be performed in such a way that dispersion in the resulting response quantities will remain relatively low.

Performance assessment can be de-convolved by introducing two intermediate variables, namely the structural demand measure (i.e., the drift response) and an earthquake parameter (such as magnitude or distance). Depending on the seismic hazard at a particular site, the resulting mean annual frequency of exceeding the demand measure $\lambda_{DM}(x)$ can be expressed as

$$\lambda_{DM}(x) = \sum_{i=1}^{n} G_{DM|M,R}(x|M_i,R_i)\lambda(M_i,R_i) \tag{6}$$

In the above, $G_{DM|M,R}(x|M_i,R_i)$ denotes the probability of exceeding the demand measure over a given value of x, conditional on each one of the n-specific (M, R) pairs of interest that account for all seismic scenarios that may contribute to overall seismic hazard. This particular term is determined using the results of nonlinear dynamic analyses of the structure of interest under a set of earthquake ground motions. Furthermore, $\lambda(M_i,R_i)$ denotes the mean annual frequency of occurrence of each one of these seismic events that are characterized by specific values of earthquake magnitude and distance. Typically, $\lambda(M_i,R_i)$ is estimated via a PSHA.

If a rational number for different seismic events that may contribute to defining seismic hazard at a particular site is between five to ten, and if acceptable values of $\lambda_{DM}(x)$ are in the range $10^{-3}-10^{-4}$, Cornell [74] estimated that the number of records required and the subsequent number of analyses that have to be performed may approach 10^3, which is prohibitive for practical purposes. Moreover, when additional parameters are to be considered, such as duration of strong motion, rupture mechanism, directivity of seismic waves or the general seismotectonic environment [24,38,75], the number of accelerograms required for an accurate estimate of $\lambda_{DM}(x)$ increases even further.

Selection of real records based on strong motion parameters that are strongly related to structural response constitutes an efficient way to address the problem of excessive number of accelerograms required for reliable dynamic analyses. These types of parameters, that quantify the effect of records on structure, are usually known as intensity measures (IM). The ground motion IM serves as link between seismic hazard analysis, typically provided by seismologists, and structural analysis conducted by engineers. Typical examples of IM include peak ground acceleration (PGA) and the first mode period spectral acceleration (SA). Along these lines, Giovenale et al. [76] proposed that the choice of an appropriate IM should be based on its sufficiency and efficiency [77], as well as on its 'predictability' through a seismic hazard analysis. An efficient IM can be simply defined as one that results in relatively small variability in a demand measure (DM) for a structure excited with ground motions compatible with that particular IM. Efficiency reduces the number of nonlinear dynamic analyses and of earthquake records necessary to estimate the conditional distribution $G_{DM|IM}$ of the DM for a given IM with adequately small uncertainty [78]. On the other hand, a sufficient IM is defined as one that renders the DM conditionally independent, given IM, of the earthquake magnitude and source-to-site distance. Sufficiency is desirable because it ensures an accurate estimate of $G_{DM|IM}$, independently of the magnitudes and distances that characterize earthquake records that were used in assessing structural performance. It is important to note that both efficiency and sufficiency of an IM depend not only on the type of ground motions considered (e.g., near-source, far-source, etc.), but also on the characteristics of the structure under study.

By eliminating the terms that are related to earthquake magnitude and source-to-site distance and simultaneously introducing the IM, Eq. (6) may be expressed as follows:

$$\begin{aligned}\lambda_{DM}(x) &= \int_x P(DM > x|IM = y)|d\lambda_{IM}(y)| \\ &\cong \sum_{\text{all } x_i} P(DM > x|IM = y_i)\Delta\lambda_{IM}(y_i)\end{aligned} \tag{7}$$

In the above, the term $P(DM > x|IM = y_i)$ represents the probability of exceeding a specified DM level x, given an IM=y_i. The term $d\lambda_{IM}(y)$ is the mean annual frequency of exceeding a given IM value y_i and $\Delta\lambda_{IM}(y_i) = \lambda_{IM}(y_i) - \lambda_{IM}(y_{i+1})$ is approximately the annual frequency of IM=y_i. Assuming that efficiency and sufficiency are the sole criteria considered in selecting an appropriate IM, it is interesting to observe that the most reliable ground motion IM will be the structural demand measure itself. A direct computation of the above λ_{DM} (mean annual frequency of exceeding a given value of demand measure) via a PSHA requires either a structure-specific attenuation relationship for DM or extensive ground motion simulations [79,80]. Both approaches require an excessive number of nonlinear dynamic analyses for a given structure and different (M, R) pairs. To that end, Hancock [81] conducted regression analyses to produce predictive equations for a significant number of DM. These structure-specific models, determined by a ten-parameter functional form, were derived for a particular eight-storey, regular-type R/C wall-frame, excited by a large set of accelerograms from the NGA dataset.

2.5. Investigation of scalar ground motion intensity measures

For an assessment of structural performance, the ground motion IM customarily adopted is spectral acceleration near the fundamental period of the structure with a damping ratio of 5%. Firstly, this choice is driven by convenience, since seismic hazard curves in terms of $S\alpha(T_1)$ are either readily available or easily computed. It is clear that SA is related to both structural and seismic motion characteristics, while the PGA of a record, which constituted a commonly used IM in the past, accounts only for strong motion features. In principle, $S\alpha(T_1)$ is an attractive intensity measure for record selection, but among records with similar values of SA, there is significant variability in the level of structural response in a MDOF nonlinear model.

When performing nonlinear dynamic analyses in order to study the seismic response of a five-story, four-bay steel moment-resisting frame, Shome et al. [20] selected real records, arranged in four different sets, and afterwards normalized them to PGA as well as $S\alpha(T_1)$ values. The results demonstrated that SA at the fundamental period is closely related to inelastic demands (e.g., drift) for moderate-period structures (e.g., around 1.0 s); consequently, relatively few nonlinear dynamic analyses for different earthquake records are necessary to estimate the conditional distribution of the demand measure given by $S\alpha(T_1)$. On the other hand, the use of PGA was not encouraged. The superiority of $S\alpha(T_1)$ compared to PGA was also observed in Ref. [76], which confirmed the ability of the former measure to capture the damaging power of an earthquake on a specific structure. Nonetheless, some studies have demonstrated that the spectral acceleration at the fundamental period may not necessarily be efficient, nor sufficient, for certain kinds of structures such as tall, long-period buildings [78,82] or for near-field ground motions [77,83,84] used in the inelastic analysis of both SDOF and MDOF systems. Furthermore, since spectral acceleration calculation requires estimation of just the fundamental period of the structure, bias may be introduced in the overall procedure.

In order to address the above shortcomings of the $S\alpha(T_1)$ measure, several alternative IMs considered as multiplicative adjustments of spectral acceleration, have been proposed. In order to allow for the contribution of higher modes and period lengthening of the structure of interest, Mehanny [83] and Cordova et al. [85] proposed use of the following two-parameter scalar IM:

$$S\alpha C = S\alpha(T_1)\left(\frac{S\alpha(cT_1)}{S\alpha(T_1)}\right)^{a} \tag{8}$$

where c and a are two parameters estimated by proper calibration. A noticeable advantage of $S\alpha C$ is the ease in obtaining the attenuation law directly, as a linear combination of the attenuation functions for the spectral acceleration values $S\alpha(T_1)$ and $S\alpha(cT_1)$. This requires estimation of the statistical correlation coefficients between these two variables that has been implemented by Inoue and Cornell [86].

Moreover, Shome and Cornell [78] proved that reduction in the dispersion of the predicted structural response is possible by averaging $S\alpha(T)$ values over a narrow range of periods centered around the fundamental period, and by using higher damping values in order to calculate the spectral accelerations. Along the same lines, Shome et al. [20] showed that the reduction effect of higher damping values on the structural response dispersion depends on the demand measure. By considering a deformation-based DM, the dispersion reduced as damping values increased, while adopting both deformation and energy-related indices, such as Park and Ang index [87,88], this trend was not followed. Use of higher damping values was also recommended in Ref. [89]. Furthermore, Hutchinson et al. [90] suggested an alternative IM, based on the spectral displacement as a means to reduce uncertainty in predicting inelastic displacement demands for ductile bridge structures excited by near-fault ground motions. This intensity measure (Δ_{mean}) uses the mean elastic spectral displacement between two characteristic periods of the structure, namely elastic period (T_{el}) and secant period at the peak displacement demand (T_{sec}).

Since the period-specific $S\alpha(T_1)$ requires only an accurate estimate of the fundamental period of the structure under study and an SDOF time-history analysis, Luco and Cornell [77] proposed some more structure-specific intensity measures. The IMs introduced here were meant to be related to peak drift demands. In particular, the six alternative intensity measures that were proposed as an attempt to reduce dispersion in the structural response because of ineffective record selection, are all recoverable pieces of structural information (e.g., modal vibration properties and nonlinear static pushover curve for the structure of interest) and ground motion characteristics (elastic or inelastic spectral displacements). By appropriate adjustment of the $S\alpha(T_1)$ in order to reflect the contributions of higher modes and the effects of inelasticity on structural demands, the first four IMs involve elastic, inelastic, equivalent (as developed by Iwan [91]) and effective [83,85] first mode vibration properties, while the remaining two make allowance for the second mode (for an exact formulations of the IMs see Ref. [31]).

Using DM results from nonlinear dynamic analyses of three steel moment-resisting frame buildings (moderate-to-long period structures) [92] under both 'ordinary' and near-source earthquake records, Luco and Cornell [77] performed one-parameter linear regression analyses in order to quantify the efficiency as well as the sufficiency of the proposed IMs. The ground motion intensity measure ($\mathrm{IM}_{1I\&2E}$) that reflects both first and second mode frequency content plus inelasticity was demonstrated to be the best comparing to the remaining measures. Also $\mathrm{IM}_{1I\&2E}$ was proved to be more suitable for higher-mode sensitive structures. An approximate attenuation relationship for the $\mathrm{IM}_{1I\&2E}$, in order to estimate the corresponding hazard curves, has been proposed by Tothong and Cornell [93]. It should be noted here that structure-specific intensity measures require more detailed calculations in comparison with the standard IMs (e.g., spectral acceleration), since they are dependent on some detailed structural properties. Moreover, their use is relatively limited because they have to be always straight related to the SA otherwise the computability of the ground-motion hazard in terms of the particular IM, which is needed for the probabilistic structural performance-assessment scheme, becomes quite involved. The outcome of the study [77] verified also the sensitivity of the basic two IM proxies (efficiency and sufficiency) to the type of ground motions considered as well as to the properties of the structure under investigation.

One of the benefits of using a carefully chosen intensity measure is that more accurate evaluations of seismic performance are achieved, without the need to perform detailed ground motion record selection for the nonlinear dynamic structural analyses as required by a PSHA. Along these lines, Tothong and Luco [94] suggested that use of the inelastic spectral displacement (Sd_i) as an advanced IM, can be advantageous relative to the conventionally used elastic spectral accelerations ($S\alpha$) for structural demands that are dominated by the first mode of vibration. Use of Sd_i, which fulfills the criteria for efficiency, sufficiency, hazard computability (the required attenuation relationship has been determined in Ref. [95]) and scaling robustness (i.e., scaling records to an IM value resulted in unbiased structural response compared to the analogous response obtained by unscaled ground motions), can significantly reduce response variation for $T>T_1$ for either ordinary or near-source pulse-like ground motions. Indeed, the inelastic spectral displacement better captures the spectral shape at periods longer than T_1, because those spectral values directly affect Sd_i due to period elongation. Therefore, records scaled to a common Sd_i value can be expected to produce less record-to-record variability in the response spectra at the period range $T>T_1$. Finally, use of intensity measure $\mathrm{IM}_{1I\&2E}$ was also recommended for the higher mode sensitive structures in Ref. [94].

2.6. *Investigation of vector-valued ground motion intensity measures*

As previously discussed, spectral acceleration at the fundamental period of the structure has been proven to be a common and relatively efficient intensity measure. However, among records with the same value of $S\alpha(T_1)$, there is still significant variability in the response of MDOF nonlinear structural systems. In order to reduce this remaining record-to-record variability so as to increase the accuracy and efficiency of the structural response calculations, Baker and Cornell [34] proposed an improved IM, which besides $S\alpha(T_1)$ includes the ε ('epsilon') parameter associated with ground motions. This particular IM is termed vector-valued because it comprises two parameters, as opposed to the traditional scalar IMs. These intensity measures have been proposed by other researchers as well as a means to enhance structural predictions [96,97].

While magnitude and distance are familiar to structural engineers, the aforementioned ε parameter is less so. It was first defined by engineering seismologists as the number of standard deviations by which a given spectral acceleration, expressed in logarithmic terms, differs from the mean logarithmic spectral acceleration provided by a ground motion prediction (attenuation) equation. In other words, ε is derived by subtracting the predicted mean logarithmic spectral acceleration at given period $T_1(\ln\{S\alpha(T_1)\})$, from the corresponding value $(\ln\{S\alpha(T_1)\})$ of the record under examination and then dividing by the logarithmic standard deviation estimated by the attenuation relationship. Furthermore, ε is determined with respect to the unscaled record and does not change in value in case of record scaling. It is also noticeable that for a given ground motion record, ε is a function of the period of interest and depends on the particular ground motion prediction model used, since different attenuation relationships lead to different mean and standard deviation of $\ln\{S\alpha(T)\}$. Therefore, it is important to ensure that the ground motion prediction model used to compute ε is the same model used in the ground motion hazard assessment. This dependence of ε to attenuation relationships seems to be a drawback in the use of this parameter.

The inclusion of ε in the vector-valued intensity measure [34] is derived mainly by its proxy as a reliable indicator of the spectral shape. More specifically, Baker [98] investigated the influence that ε exerts on the spectral shape and it was inferred that records with positive ε values (e.g., records that have larger than the expected spectral acceleration at a particular period) tend to have peaks in the response spectrum at the specified period, while records with negative ε values tend to present valleys. As a result, ε can be considered as an indicator of spectral shape, and this is the reason it is effective in predicting the response of nonlinear MDOF systems. Indeed, given $S\alpha(T_1)$, additional information on spectral acceleration values at periods different than the fundamental one is required to determine the nonlinear structural response, since a MDOF structure is affected by the excitation of its higher modes at periods shorter than T_1[99] and the effective period of its first mode increases to a value larger than T_1 in case of nonlinear behavior [100].

Similarly to the definition in Eq. (7) which incorporates a single (or scalar) parameter intensity measure, the mean annual frequency of exceeding a specific value of a demand measure (in PEER practice, the response of a structure is termed as Engineering Demand Parameter, EDP) on the basis of a vector-valued intensity measure comprising $S\alpha(T_1)$ and ε may be expressed as

$$\lambda_{EDP}(x) = \int_{x_1}\int_{x_2} P(EDP > x | Sa(T_1) = x_1, \varepsilon = x_2) \left| \frac{\partial^2 \lambda_{\mathrm{IM}}(x_1, x_2)}{\partial x_1 \partial x_2} \right| dx_1 dx_2$$

$$\cong \sum_{all\ x_{1,i}} \sum_{all\ x_{2,j}} P(EDP > x | Sa(T_1) = x_{1,i}, \varepsilon = x_{2,j}) \Delta\lambda_{IM}(x_{1,i}, x_{2,j}) \quad (9)$$

In the above, $\lambda_{EDP}(x)$ is the mean annual frequency of exceeding a given EDP value x, $P(EDP > x|Sa(T_1) = x_1, \varepsilon = x_2)$ represents the probability of exceeding a specified EDP level x given specific values for $S\alpha(T_1)$ and ε, $\lambda_{IM}(x_i)$ is the mean annual frequency of IM exceeding a given value x_i and $\Delta\lambda_{IM}(x_{1,i}, x_{2,j})$ is approximately the annual frequency of the vector-valued IM. In reference to Eq. (9), Baker [101] reviewed the current methods used and proposed some improved procedures to accurately estimate the probability of exceeding a specified EDP level, making allowance for both scalar and vector-valued intensity measures.

In re-defining the term $\Delta\lambda_{IM}(x_{1,i}, x_{2,j})$ as $\lambda_{Sa \in [x_{1,i}, x_{1,i+1}], \varepsilon \in [x_{2,j}, x_{2,j+1}]}$, it is feasible to express it as the marginal rate density of $S\alpha(T_1)$ and the conditional probability distribution of ε at a specific value of $S\alpha(T_1)$, namely:

$$\Delta\lambda_{\mathrm{IM}}(x_{1,i}, x_{2,j}) = P(x_{2,j} < \varepsilon < x_{2,j+1} | Sa(T_1) = x_{1,i}) \Delta\lambda_{Sa(T_1)}(x_{1,i}) \quad (10)$$

Eq. (9) can then be re-written as

$$\lambda_{EDP}(x) = \sum_{all\ x_{1i,}} \sum_{all\ x_{2j,}} P(EDP > x | Sa(T_1) = x_{1,i}, \varepsilon = x_{2,j})$$
$$\times P(x_{2,j} < \varepsilon < x_{2,j+1} | Sa(T_1) = x_{1,i}) \Delta\lambda_{Sa(T_1)}(x_{1,i}) \quad (11)$$

It is best to use Eq. (11), because the term $\Delta\lambda_{S\alpha(T1)}(x_{1,i})$ can be derived from a standard PSHA hazard curve, and also $P(x_{2,j} < \varepsilon < x_{2,j+1} | S\alpha(T_1) = x_{1,i})$ is a typical de-aggregation result. A convenient way for estimating the term $P(EDP > x\ |S\alpha(T_1) = x_1, \varepsilon = x_2)$ is first to scale all records to $S\alpha(T_1)$ and then to apply regression analysis for the estimation of the desired EDP as a function of ε[102]. Consequently, $S\alpha(T_1)$ is treated as in the scalar case, but information is incorporated from the regression analysis on ε.

In using the vector-valued intensity measure presented above, Baker and Cornell [103] evaluated the efficiency of particular record selection methods and their resulting structural response output in terms of the maximum inter-story drift ratio. The records were selected from the PEER database [23] and used as input for nonlinear dynamic analyses of a 2D model of the transverse frame of a seven-storey reinforced concrete building [33]. Four selection strategies were considered: (a) randomly (AR), (b) using site-representative M, R values (MR-BR), (c) using ε values representative of site hazard without any criterion for M, R (ε-BR) and (d) using spectral shapes that matched a conditional mean spectral shape based on specific values of $\overline{M}, \overline{R}$ and $\overline{\boldsymbol{\varepsilon}}$ (CMS-$\boldsymbol{\varepsilon}$).

The conditional mean spectral shape accounts for the relation between ε and the spectral shape [98]. To develop this new target spectrum, PSHA can be used to determine the spectral acceleration $S\alpha(T_1)$ that corresponds to the target probability of exceedance at the site of interest, denoted as $S\alpha(T_1)^*$. De-aggregation can then be used to estimate the mean values $(\overline{M}, \overline{R}, \overline{\varepsilon})$ for the seismic parameters that lead to an acceleration equal to $S\alpha(T_1)^*$. More specifically, using $\overline{M}$ and $\overline{R}$ in conjunction with ground motion prediction models, yield the mean and standard deviation of the response spectral values for the entire period interval, while $\overline{\boldsymbol{\varepsilon}}$ specifies the number of standard deviations away from the mean $\overline{S\boldsymbol{\alpha}}(T_1)$, as provided by the attenuation relation. Since the mean value $\overline{\varepsilon}(T_1)$ is known, Harmsen [104] proposed that the conditional distribution of SA values at other periods can be calculated using only the de-aggregation data and the known correlations of ε values at a particular period range. Thus, both conditional mean value and standard deviation of the target response spectrum can be computed using the following equations:

$$\mu_{\ln Sa(T_2)|\ln Sa(T_1) = \ln Sa(T_1)^*} = \mu_{\ln Sa}(\overline{M}, \overline{R}, T_2) + \sigma_{\ln Sa}(\overline{M}, T_2) p_{\ln Sa(T_1), \ln Sa(T_2)} \overline{\varepsilon}(\mathrm{T}_1) \quad (12)$$

$$\sigma_{\ln Sa(T_2)|\ln Sa(T_1) = x} = \sigma_{\ln Sa}(\overline{M}, T_2) \sqrt{1 - p^2_{\ln Sa(T_1), \ln Sa(T_2)}} \quad (13)$$

As previously mentioned, $\overline{M}, \overline{R}, \overline{\varepsilon}(T_1)$ are all derived by de-aggregation given that $S\alpha(T_1) = S\alpha(T_1)^*$. The terms $\mu_{\ln Sa}(\overline{M}, \overline{R}, T_2)$ and $\sigma_{\ln Sa}(\overline{M}, T_2)$ respectively are the marginal mean and standard deviation of the $\ln(S\alpha)$ at T_2 obtained from a ground motion prediction relation [105], while $p_{\ln Sa(T_2), \ln Sa(T_1)}$ represents the correlation between $\ln(S\alpha)$ values at two values of periods. An empirically determined relationship for the last term is given in Ref. [86] as

$$p_{\ln Sa(T_1), \ln Sa(T_2)} = 1 - 0.33 \left| ln\left(\frac{T_1}{T_2}\right) \right|, \quad 0.1\,\mathrm{s} \le T_1, T_2 \le 4.0\,\mathrm{s} \quad (14)$$

Spectral matching with the conditional mean spectrum by utilizing ε (CMS-ε) may help widen the range of acceptable records for nonlinear dynamic analysis, because selected accelerograms may no longer have the appropriate (M, R, ε) values, but only posses a spectral shape that matches the mean spectrum with the casual event. Furthermore, the proposed CMS-ε measure seems better suited for use in design and probabilistic assessment of structures in comparison with the Uniform Hazard Spectrum (UHS), which is nowadays the most frequently used target spectrum in seismic structural analysis. It should be noted that many researchers [13,106,107] doubt that the UHS can be considered as a spectrum of a single earthquake event rather than an envelope of the spectra corresponding to different seismic sources. Therefore, use of UHS may result in designing for an unjustifiably conservative scenario of earthquakes occurring due to different seismic sources acting simultaneously [98]. In sum, the CMS-ε measure helps eliminate this conservatism.

When estimating the mean annual frequency of exceeding a given structural response level (sometimes referred to as 'drift hazard curve') derived by nonlinear dynamic analyses for the selected records with the four different methods proposed above, it was inferred that the AR as and MR–BR method produced higher estimated probabilities of exceedance compared to the other two methods, which yielded nearly identical results. However, when using a vector-valued intensity measure comprising $S\alpha(T_1)$ and ε in the first two selection methods, the resulting drift hazard curves were in much closer with the those derived from the last two methods, which make allowance for the ε parameter. This observation seems to verify that the variation among the results in the drift hazards curves comes from variation in the spectral shape, which reflects the influence of epsilon. Furthermore, Baker [98] attempted to assess the response-predicting effectiveness of the (M, R, ε) parameters by incorporating them in a vector-valued intensity measure, conditioning on $S\alpha(T_1)$, and then performing linear and logistic regression analyses on the response results. Now, an effective predictor should show a trend in these regressions analyses and the slope of the regression has to be statistically significant. When distance was considered as a possible IM parameter, no statistical significance was derived. In contrast, both (M, ε) demonstrated some statistical significance, with the latter parameter having more influence than the former M. Finally, this trend is consistent with results from past work (e.g., [20]).

3. Seismic code provisions for selection of real records

National seismic codes prescribe general guidelines but do not provide specifics for selecting the type of earthquake records required for nonlinear dynamic analysis purposes. This is for three main reasons: (a) time–history analyses are rather recent in engineering practice and expertise developed to date is not considered sufficient; (b) research on this topic is still under development and regulations to include the recent innovations require at least a few years time; (c) full agreement has not yet been reached regarding the establishment of commonly accepted selection criteria for earthquake records. What seems to be current consensus is simply the requirement that acceptable acceleration time histories, whether recorded, artificial, or synthetic, should be compatible with the code-prescribed smooth design spectrum; some allowance is also made at the same time for a few other minor requirements. A comprehensive review of can be found in Bommer and Ruggeri [108] and Beyer and Bommer [62].

Most contemporary seismic codes, such as Eurocode 8 [69,109], ASCE standards 7-05 [110] and 4-98 [44], FEMA regulations [111] as well as various national norms (New Zealand Standards [112], Italian Code [113] and Greek Seismic Code [114]), describe relatively similar procedures for the simulation of seismic actions to be used as dynamic loading in structures. Most frequently, seismic motions can be represented by real, artificial or even simulated records, while some important seismological parameters, such as earthquake magnitude, distance, the seismotectonic environment and the local soil conditions, should reflect in the local seismic scenarios. Nevertheless, some differences between the codes on strong motion representation remain. For example, the New Zealand Standards [112] allow use of real records only, while EC8 [69,109] leaves this choice to the structural engineer. It is also noted that some code-based selection strategies require inclusion of additional parameters; ASCE 4-98 [44] specifies that duration of the selected accelerograms has to be representative of expected ground motion at the site of interest, something that is also required by the Greek seismic code [114].

Furthermore, when defining seismic hazard in terms of the uniform hazard spectrum, spectral matching between the design spectrum and the response spectrum of a selected record is required in most codes. However, the period range for spectral matching varies among code provisions. In addition, the majority of the codes do not distinguish between record selection for uni-directional and bi-directional dynamic analysis, while Beyer and Bommer [62] noted that use of the same component for the two horizontal components is not recommended by most aforementioned codes. Moreover, the minimum number of records required for structural analysis is three in all cases, the exception being ASCE 4-98 [44] which specifies that at least one record should be used, unless the structure is sensitive to long-period motion. It is finally noted that when a set of at least seven ground-motions is used, the structural engineer is allowed to compute the mean structural response. Otherwise, only a maximum response value is computed if three to six recordings are used.

3.1. Evaluation of code-based selection processes

In order to evaluate the code-based selection process, many researchers have looked into efficiency and feasibility issues of the techniques proposed for selecting appropriate acceleration time histories that can serve as input for dynamic analysis of structural systems. Along these lines, Iervolino et al. [67] investigated the possibility to find unscaled record sets that fulfill the requirements of EC8 with respect to the seismic input for 2D and 3D structural models. To this end, accelerograms corresponding to moderate-high magnitude events from the European Strong-Motion Database (ESD) [115] were selected for two different classes: (1) sets comprising seven single-components of seismic motion for analysis of plane structures and (2) sets comprising seven groups of two horizontal components from the same recording station for analysis of spatial structures. These sets were also ranked in terms of additional criteria [24,41,62,116,117] that addressed similarity issues of the average spectrum with the reference one, record-to-record variability of the spectral ordinates, prevention of single-event domination, and the range of magnitudes within a given set.

Continuing with the code evaluation, Iervolino et al. [67] concluded that selection of accelerograms for categories A, B, C of the EC8 soil classification was possible, while not so for very soft soil sites (D, E), a conclusion also confirmed by Sextos et al. [118]. The lack of accelerograms recorded for the latter soil categories in strong motion databases constitutes one reason for this finding. Also, for the highest seismic zone in the Italian code [113], it was impossible to find a set of records compatible with the EC8 spectra, although linear scaling of a small number of records within this set may restore compatibility. Moreover, it was inferred that unscaled record sets strictly matching EC8 spectra resulted in a large record-to-record variability in the spectral ordinates within the same set [67]. It is interesting to note that similar conclusions have been also drawn by Iervolino et al. [70] for EC8-based selection of real records in the design of bridges following both linear and nonlinear dynamic analysis.

On the basis of the above, it is clear that EC8 provisions still lack the necessary completeness for effective selection of real accelerograms from current strong motion databases in order to be used as input to structural dynamic analyses, especially in case of sites with high seismicity and soft soil profiles. Application of additional criteria proposed by some researchers, as well as use of computer codes and supporting software [61], are all deemed necessary to improve the selection procedure. Nevertheless, even if such a selection process is indeed feasible, it does not ensure that from the ensuing structural analysis dispersion in the response quantities will remain reasonably low. Sextos et al. [118] attempted to assess the efficiency of EC8 requirements regarding the record selection process by checking for minimal scatter in the structural response, as derived by the nonlinear dynamic analysis of a real multi-storey building located in Greece struck by a severe seismic event in 2003. More specifically, five different sets of strong motion accelerograms comprising seven pairs of horizontal components were built in compliance with EC8 guidelines. By trying to establish spectral matching between the code spectrum and the mean spectrum of each record set in the recommended period range ($0.2T_1-2.0T_1$, T_1 is the fundamental period of the structure) without any scaling, it was necessary to use at least one earthquake record with high spectral accelerations in order to achieve spectral matching at higher periods close to $2.0T_1$ (see Fig. 1). The requirement for inclusion of records with high spectral values is most pressing when designing or assessing an existing structure for input from the highest seismic zone. This means using at least one earthquake record that would lead to strong inelastic behavior in the structure, while the remaining 'normal' records in the set may have little influence on the structural response. As a result, the overall rational for 'averaging' partially elastic and partially inelastic structural response coming from records within a given set seems questionable. Furthermore, concurrent use of severe enough (or 'correcting') accelerograms from the entire EC8-based record selection process results in non-negligible values of scatter in the structural response quantities among the seven pair comprising a set. Table 1 lists values for the coefficients of variation (C.O.V.) in the response of columns located at the left-side of the R/C building under study [118]. The structural response was quantified through the damage index proposed by Jeong and Elnashai [119].

The conclusion here is that it is necessary to modify the required range of spectral matching, especially for longer periods. In most cases, the upper bound of $2.0T_1$ seems to be excessive, since it is unlikely that doubling the first-mode period of the structure in question is rational, unless the structure is subjected to extremely high seismic forces and suffers much structural damage. One must not confuse the presence of soft soil and foundation compliance with period elongation in the structure during seismic excitation, since flexibility in the combined soil–structure system affects the fundamental period of the structure $T1$, prior to and independently of any earthquake loading. Similarly, dependence of the upper period limit on the expected displacement demand in the structure has been proposed by Beyer and Bommer [62]. In there, a period range $[T_m, \sqrt{\boldsymbol{\mu}_\Delta}T_1]$ was proposed when matching record with target spectra, where T_m is the period of the highest mode that contributes significantly to the elastic response of the structure and $\sqrt{\boldsymbol{\mu}_\Delta}$ is the displacement demand in the structure. Nevertheless, when designing a typical structure, the displacement demand is not specified in advance. Thus, the upper bound of the period interval may be related to behavior factor q that expresses the necessary level of inelastic response for which the structure has been designed. It is believed that for structures designed for low ductility (i.e., not corresponding to ductility class 'high' of Eurocode 8), there is no reason for establishing spectral matching at twice as long periods that are no longer related to the expected structural response. Similarly, the lower bound of the period range for which spectral matching is desired could be considered as a function of higher mode contributions. This is not necessarily equal to $0.2T_1$, but rather equal to the (lower) period T_L of the highest mode of vibration of the structure for which the activated mass is about 90% of total.

Table 1
Maximum and minimum COV values of of the response of columns in the building under study, estimated for the seven pairs of horizontal components of the selected strong motions comprising five sets (after Sextos et al. [118]).

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

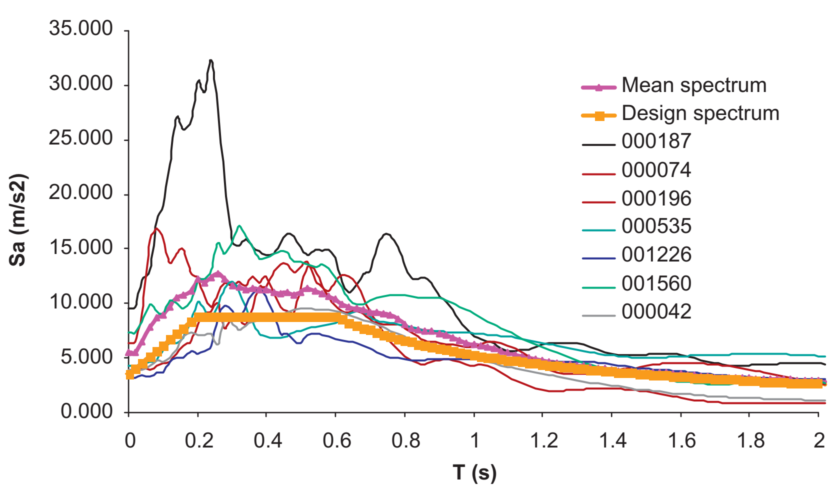

Fig. 1. . Indicative overview of individual and mean response spectra of records selected according to EC8 for the assessment of the inelastic dynamic response of a multi-story RC building (after Sextos et al. [118]).

Following the above line of thought, it is proposed to relax the prescribed period range from ($0.2T_1-2.0T_1$) to ($T_L-1.5T_1$), where T_L defined as previously, at least for structures designed for moderate ductility, in order to increase the number of records available for dynamic analyses and lessen the dominance of severe strong motion records on inelastic response and on the subsequent dispersion in the response quantities. Further investigation is certainly required until reaching a balance between earthquake record selection efficiency and design reliability.

4. Conclusions

This review presented various methodologies by which rational decisions can be made regarding the time-dependent earthquake input to be used for transient dynamic analysis of a structural system built in seismically prone regions. It can be concluded that there quite a few ways to achieve record selection, but it is still not possible to limit the bounds of the ensuing structural response dispersion uniformly. Moreover, despite much progress made, these record selection techniques have not yet been included in contemporary seismic code provisions. Because of that, seismic design codes used nowadays present a rather simplified version of the full picture when it comes to assessing seismically induced loads, which may or may not be commensurate with the detailed numerical modeling effort often expended in representing the structural system. In sum, seismic loading code provisions are adequate for a large class of conventional structures. This, however, may not be true for more complex situations which require sound engineering judgment, in addition to competence in setting up an adequate structural model, determining the seismic input and interpreting the response output.

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