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A review of ground motion record selection strategies for dynamic structural analysis

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Abstract Non-linear dynamic analysis is recognized as the more accurate tool for seismic evaluation of structures in the case of both probabilistic assessment and design. The key issue in performing this kind of analysis is the selection of appropriate seismic input (e.g. ground motion signals), which should allow for a correct and accurate estimation of the seismic performance on the basis of the hazard at the site where the structure is located. To this aim several procedures have been proposed, they require specific characterization of real ground motion records via the so called ground motion intensity measures (mainly related with elastic spectral features of the record) proven to be generally efficient in the estimation of the structural performance. This kind of approach requires specific skills as well as detailed probabilistic evaluation of the seismic threat to be available to the engineers. For this and other reasons codes worldwide, in many cases, try to acknowledge these procedures in an approximate fashion.

In this paper recent and advanced literature on the topic is presented and discussed. The current best practice in record selection is reviewed for the case of probabilistic seismic risk analysis and for code- based seismic assessment and design with special attention to the prescriptions of Eurocode 8 for both buildings and bridges. Finally, some light is briefly shaded on the effects of time scaling of records and its use in shake-table structural testing.

1 Introduction

The issue of selecting the seismic input is seen to be one of the most critical in the seismic assessment of structures via non-linear dynamic analysis. It is sometimes considered more important even than structural modeling. Therefore, this problem has been the subject of large research recently. In general, the signals that can be used for the seismic structural analysis are of three types: (1) artificial waveforms; (2) simulated accelerograms; and (3) natural records (Bommer and Acevedo, 2004).

Spectrum-compatible signals of type (1) are obtained, for example, generating a power spectral density function from a code-specified response spectrum, and deriving signals compatible to that. However, this approach may lead to accelerograms not reflecting the real phasing of seismic waves and cycles of motion, and therefore energy. Simulation records (2) are obtained via modeling of the seismological source and may account for path and site effects. These methods range from stochastic simulation (Boore, 2003) of point or finite sources to dynamic models of rupture. However, they often require setting of some rupture parameters, such as the rise-time, which are hard to determine. Some state-of-the-art simulation methods seem to overcome these shortcomings, but they are not yet readily available to engineers.

Finally, of type (3) are ground-motion records from real events. The availability of on-line, user-friendly, databases of strong-motion recordings, and the rapid development of digital seismic networks worldwide, have increased the accessibility to recorded accelerograms, which, therefore, have become the most promising candidates for the seismic assessment of structures, both for code-related purposes and probabilistic risk analysis. However, due to the large record-to-record variability in records representing a specific scenario (i.e. a magnitude-distance pair), a number of points arise regarding the criteria for appropriate selection and manipulation of such records. The aim is to obtain a correct and accurate estimation of the structural performance in a way that it does not require a undesirably large number of non-linear dynamic analyses to run. Therefore, earthquake engineering research has focused lately on the selection of real ground-motions for non-linear structural analysis and effective procedures, with various degrees of simplicity in application, have been developed to link real ground motion records to the hazard at the site and to estimate probabilistically the structural seismic performance.

Although the record selection for probabilistic seismic risk assessment should be regarded as a reference case, codes only try to approximate such an approach for assessment and design purposes. This is because all the information needed to perform state-of-the art procedures for record selection are, nowadays, seldom available to engineers. Moreover, codes also try to warrant standard and conservative procedures, at least until probabilistic seismic hazard data are not made broadly available and/or certified by authorities.

In the case of code-based seismic structural assessment, another issue regarding the use of real recordings, whose spectra are generally non-smooth, is the selection of a set compatible with a code-specified spectrum. Several approaches have been developed to manipulate real records in order to match a target spectral shape, either by frequency-domain or by time-domain modification methods such as the wavelet transform. The wavelet transform basically consists of using modulating functions, selectively located in time to modify the spectrum of the signal, where and when it is needed, in order to match the target spectrum, see Hancock et al. (2006) for details. Although these methods produce records perfectly compatible with code's prescriptions, and have the additional advantage of reducing the dispersion in the response and hence the required sample size, some studies show that they may lead to a non-conservative estimation of the seismic response (Carballo and Cornell, 2000; Bazzurro and Luco, 2003), and therefore natural ground motion records still seem to be the most appropriate option as input signals.

In the following the recent issues in record selection for probabilistic assessment of structures are reviewed first, subsequently the code procedures are discussed with specific reference to the European regulation (Eurocode) in the case of both one-component and multi-dimensional record selection for bridges and building structures. Finally, the effects of time scaling of real records and its use in shake-table structural testing are briefly discussed.

2 Record Selection for probabilistic assessment of structures

The objective of the probabilistic assessment of structures is defined as the estimation of the probability of the structure reaching a limit state (e.g. collapse or failure) over a period of time (i.e. one year or the design life of the structure). The limit state is often formulated as a function of the required level of seismic performance for the structure, or the seismic demand, and the supply of such performance intrinsic to the structure, or the seismic capacity. Both capacity and demand have to be expressed in terms of a global structural performance indicator as, for example, the maximum interstory drift ratio (MIDR) as it is related to the rotation of elements in moment resisting frame structures, for which semi-empirical capacity model are available. Cornell (2004) discusses the problem of estimating such a probability via non-linear dynamic analysis. In such a case, in fact, the failure rate in one year for example, may be formulated separating the estimation of the structural response from the probabilistic characterization of the seismic threat it is subjected to because of the seismicity of the site where it is located. By the total probability theorem one can write:

$$\boldsymbol{P}_f = \int_M \int_R \boldsymbol{P[F|M,R]f(M,R)}\, dm\, dr \tag{2.1}$$

where $\boldsymbol{P[F|M,R]}$ is the probability of the structure getting the limit state conditioned to some earthquake characteristic, herein represented by magnitude (M) and source-to-site distance[1](R); while $\boldsymbol{f(M,R)}$ is the joint probability density function related, for example, to the annual occurrence of any $\boldsymbol{(M,R)}$ pair.

In principle, $\boldsymbol{P[F|M,R]}$ should be estimated by non-linear dynamic analyses using and adequate number of ground motion records consistent with the $\boldsymbol{(M,R)}$ domain of interest. In particular, if relationships expressing the likelihood of observing a certain magnitude and source-to-site distance (e.g. a Gutenberg-Richter relationship and a distribution of earthquake location to be provided by seismologists) are available, the engineer should get a sample of ground motion records for each $\boldsymbol{(M,R)}$ pair and run non-linear dynamic analysis for each record in the sample. The observed frequency of collapses over the size of the sample is an estimation of the failure probability conditional to that specific $\boldsymbol{(M,R)}$ pair.

The issue of establishing the adequate sample size for each $\boldsymbol{(M,R)}$ pair raises. It depends on the desired standard error in estimating the response quantity of interest (i.e. MIDR) and the dispersion of the variable to measure itself. Cornell (2004) clarifies that, arbitrarily assuming as 0.1 the acceptable value for the latter and 0.8 as the coefficient of variation of the failure-level MIDR for a moment-resisting frame, the necessary sample size is about $(0.8/0.1)^2$, or 64. Assuming that the domain of threatening magnitude and distance pairs can be partitioned in 20 bins, then the number of required records

[1] Other earthquakes characteristics components may be faulting style and others. It is currently under discussion which of these parameters is statistically significant for the structural response, however it is appropriate here to focus on M and R considering them exhaustive.

is in the order of 10^3 which is clearly impractical also because easily accessible record databases hardly would have, in each ***(M,R)*** bin, enough records which also match the appropriate faulting stile or, more importantly, soil conditions. Therefore alternative (more efficient) strategies based on a more refined selection of ground motion records have been developed and are reviewed in the following sections.

3 Smart strategies to estimate P_f: ground motion intensity measures

Smarter approaches aimed at reducing as much as possible the sample size required to correctly estimate the structural response have been developed. These approaches are based on the concept of ground motion intensity measure or IM, and are quite easy to apply at least in their basic representation, and therefore are starting to be a widespread practice.

An IM is a ground motion feature, which should be a proxy for the earthquake potential (e.g. it should be a relatively good predictor of the structural response), which therefore, allows to estimate the response with only "few" analyses given a level of accuracy. Typical ground motion IMs are the peaks of the acceleration, velocity, and displacement signals (PGA, PGV and PGD) respectively. This is also because the seismic hazard is often represented in terms of probability of exceedance of these quantities. Linear spectral ordinates, especially accelerations, at the fundamental period of the structure, $\boldsymbol{S}_a(\boldsymbol{T}_1)$, are also often used as IMs for probabilistic assessment of structures. This is mainly because $\boldsymbol{S}_a(\boldsymbol{T}_1)$ is the response of a linear single degree of freedom system (SDOF) and therefore it should be, in principle, more correlated with the structural global non-linear performance in respect to peaks of ground motion. Integral signal's parameters, i.e. the Arias intensity, are possible IMs, although they are considered more related to the energy dissipation rather than to displacement-related structural responses as MIDR (Iervolino et al., 2006). More sophisticated IMs as non-linear spectral ordinates or vector valued IMs are also the subject of current investigation and will be discussed in the following.

Regardless of its definition, introducing an IM implies to have an additional variable to condition $\boldsymbol{P}_f$ on, and Eq.(2.1) may be rewritten as follows:

$$\boldsymbol{P}_f = \int_{IM}\int_{M}\int_{R} \boldsymbol{P[F|IM,M,R]f(IM|M,R)f(M,R)}\, dm\, dr\, d(im) \qquad (3.1)$$

where **f(IM|M,R)** is the conditional distribution of the chosen IM to magnitude and distance (e.g. an attenuation law). $\boldsymbol{P[F|IM,M,R]}$, is the failure probability of the structure as a function of magnitude, distance and IM. If $\boldsymbol{P[F|IM,M,R]} = \boldsymbol{P[F|IM]}$, then the IM is said sufficient since its ability to predict the structural response given IM is the same as given the whole set of IM, M and R (e.g. the response is independent of M and R given IM).

The sufficiency of the IM is a desirable property, which has an intuitive interpretation: since the IM is a ground motion feature which not only reflects the source features of the event, as magnitude, including also the path to the site and the site response, it represents a lager level of information about the shaking, which overwhelms the dependency of the structural response on M and R. In such a case, recognizing that

$$\boldsymbol{P(IM)} = \int_M \int_R \boldsymbol{f(IM|M,R)f(M,R)}\, dm\, dr \tag{3.2}$$

is the probability density function related to the frequency of occurrence of the IM in the time period of interest, Eq. (3.1) may be rewritten as:

$$\boldsymbol{P}_f = \int_{IM} \boldsymbol{P[F|IM]f(IM)}\, d(im) \tag{3.3}$$

If the IM is the PGA or another spectral quantity as the spectral acceleration at the fundamental period of the structure, $\boldsymbol{f(IM)}$ is simply the result of a common probabilistic seismic hazard analysis or PSHA (Cornell, 1968). This is why several studies focused on the estimation of structural response via IMs for which hazard curves are readily available.

Advantages of introducing an IM are easily recognized. In fact, if $\boldsymbol{S}_a(\boldsymbol{T}_1)$ is concerned, assuming that the COV of MIDR is about 0.4 (Shome et al., 1998), this will require about 15 records at each IM level. Therefore, if 10 levels are needed to cover the IM range of interest (i.e. from 0.1 g to 1.0 g) then the total number of runs is only about 150 which is an order of magnitude smaller than applying Eq. (2.1) to estimate P_f with the same accuracy. If only 15 records are selected and their intensity is progressively increased to cover the IM range of interest, then $\boldsymbol{P[F|IM]}$ (which is also called the fragility function of the structure) may be estimated with a very limited number of records via the so called incremental dynamic analysis, although this requires that such IM scaling do not bias the estimation of the seismic response (to follow).

4 Fundamentals of Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is a numerical simulation method to estimate the structural seismic performance. As extensively discussed by Vamvatsikos and Cornell (2002), it requires subjecting a structural model to a set of ground motion records, each of those scaled to multiple levels of intensity, represented by an IM. It is possible to recognize in IDA an analogy with the static pushover; in the IDA the seismic loading, in the form of ground motion record, is scaled rather than the horizontal forces distribution is incremented.

The main targets the IDA allows to reach are: (1) the description of the structural seismic response or "demand" versus the IM (e.g. a range of potential levels of a ground motion record); (2) the estimation of the seismic capacity of the structure; (3) the determination of a probabilistic characterization of the variability of capacity and demand from record-to-record, and therefore the fragility function for the structural model considered.

IDA requires that a single acceleration time-history is scaled (e.g. all its values are amplified by a common scaling factor, SF), to several values of the ground motion's destructive potential (e.g. the intensity). The scaling of the records by SF is basically the modification of the linear IM used (e.g., PGA, PGV or $\boldsymbol{S}_a(\boldsymbol{T}_1)$). SF is computed simply by Eq. (4.1), where $\boldsymbol{IM}_T$ is the target (desired) intensity level, and is the original value of the intensity, $\boldsymbol{IM}_U$ (e.g. of the record as unscaled). Multiplying all the values of the signal by the scaling factor, the resulting modified record, will have the IM coincident with the target value.

$$\boldsymbol{SF} = \frac{\boldsymbol{IM}_T}{\boldsymbol{IM}_U} \tag{4.1}$$

While the scaled IM monitors the level of the seismic input or loading on the structure, the parameter used to monitor the response of the structure to that ground motion (i.e. the MIDR) will be referred to as engineering demand parameter or EDP.

To cover a specific range of interest of IM, the record may be scaled several times to get, each time, a specific target level. If a nonlinear dynamic analysis for the structure is run for each of these IM levels and the EDP value is recorded, then the result of IDA for a single record is a plot of a response variable (EDP) versus the IM; this is called IDA curve. The IDA curve is different from record-to-record. In fact, since the IM is only a proxy for the potential of the ground motion, the structural response to two records scaled to the same IM, will be different as the response is also influenced by other features of the signal not captured by the IM the two accelerograms share. For example, if two records are scaled to have the same $\boldsymbol{S}_a(\boldsymbol{T}_1)$, the response may still differ if the two spectra are not similar in the range of periods beyond T_1, because the period lengthening during the shaking renders the structure sensitive to such portion of the records' frequency content.

In Figure 1 an example of IDA results for a reinforced concrete frame subjected to a set of about 20 real records is shown, in that case the EDP is MIDR and the IM is $\boldsymbol{S}_a(\boldsymbol{T}_1)$, where the fundamental period of the building is about 0.7 sec (De Risi, 2007). Each line illustrates the response of the structure to a specific accelerogram scaled several times to different intensities. Note that IDA curves are often represented with IM (the independent variable) on the vertical axis rather than on the horizontal one, this is to have a plot somewhat analogue to the push-over curves where the total base shear is the ordinate and the displacement is the abscissa.

If the structural model allows to account for some failure mode, which can be monitored via the EDP, a softening of the IDA curve may occur at high IM levels, signaling the onset of dynamic instability. The curve then goes flat at the maximum value of IM analogously to static instability, as it is the point where deformations increase indefinitely for small increments in the IM.

It is finally important to note that IDA curves are not necessarily monotonic (see the lower IDA curve in Figure 1 for example) because as the accelerogram is scaled up, weak response cycles in the early part of the response time-history become strong enough to "yield" the structure, the properties of the structure are altered for the subsequent stronger cycles (e.g. period elongation). Therefore, it may happen that a structure that showed high response at a given intensity level, may exhibit equal or even lower response when subjected to higher seismic intensities.

IDA obtained by a sample of ground motion records may be used to get a probabilistic representation of the seismic demand conditioned to IM. See for example Figure 1, where median, and 16 %, 84 % IDA curves are also plotted. For each (given) IM level those values read on the thick curves help to characterize the variability of the response.

IDA curves can also be used to determine the capacity of the structure. However, may be not obvious how to determine the capacity on an IDA curve. There are two possible

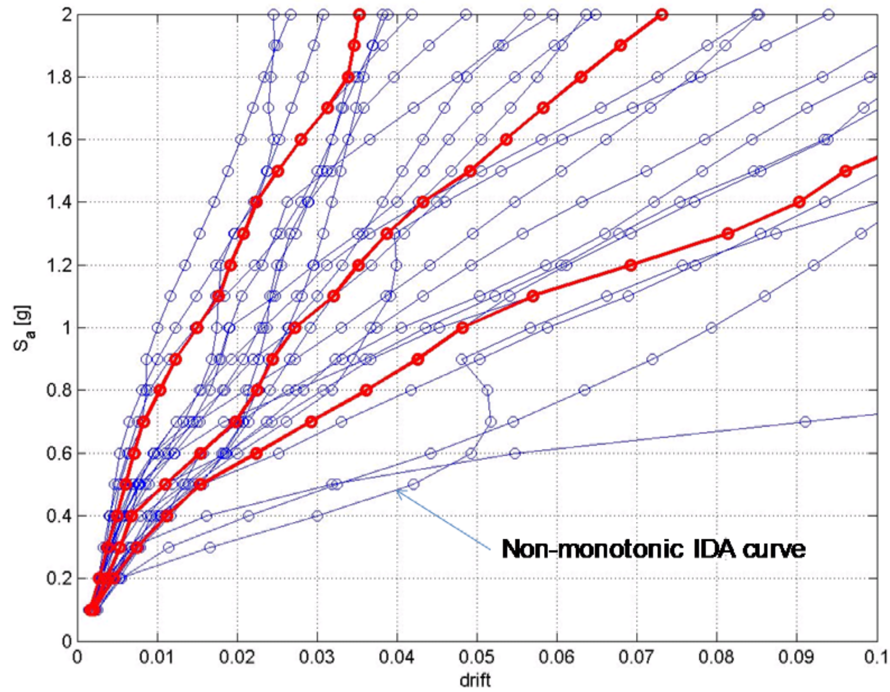

Figure 1. IDA curves in terms of MIDR vs. $\boldsymbol{S}_a(\boldsymbol{T}_1)$ for an underdesigned RC frame (De Risi, 2007). Thick curves represent medians and 16-84 percentiles of MIDR conditioned to IM.

approaches to solve this problem: (1) the capacity point is defined as that corresponding to $EPD \geq EDP_C$ where EDP_C is the response parameter's value that reflects the onset of the limit-state, this is consistent with static analysis methods and may be coincident with the reaching of limit rotations in an element, but it may ambiguous to find this capacity point on an IDA curve (see upper panel of Figure 2 for example); (2) the capacity may be defined in terms of a specific IM level (see bottom panel of Figure 2) that corresponds to the plateau in the curve, If $IM = IM_C$ then the limit-state is exceeded, a disadvantage of this latter method is that the IM_C level has to be determined curve by curve.

Clearly it is possible to have, therefore, a probabilistic characterization of the capacity when multiple IDA curves are available. In Figure 3 an example of the inferred distribution of collapse drift is given; each star in the plot represent the point where capacity is reached in each IDA curve (Jalayer, 2003).

IDA may also directly help to compute the seismic risk, by Eq. (3.3). In fact, the fraction of records causing collapse at each IM level provides an estimation of the $\boldsymbol{P[F|IM]}$ term, which is the seismic fragility of the structure. In Figure 4 the determination of the fragility is represented for a case when the capacity of the structure is deterministic;

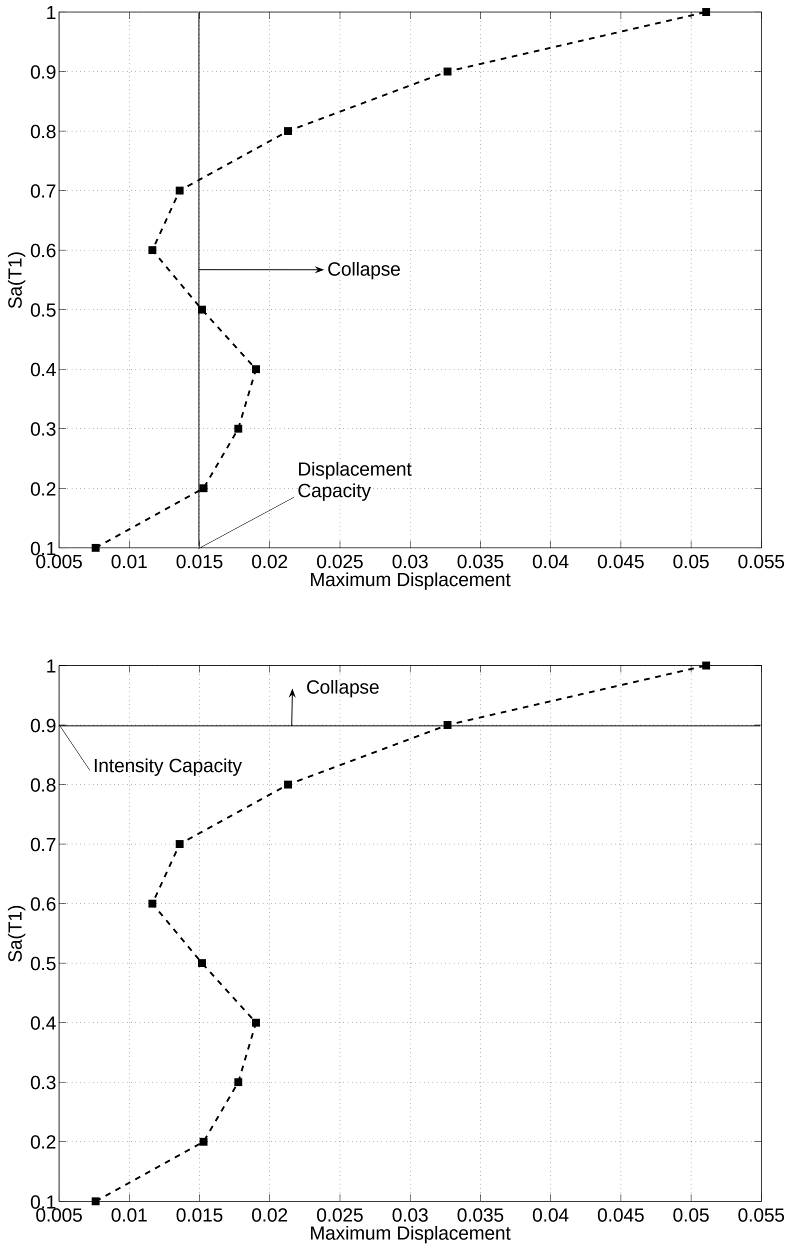

Figure 2. Measuring capacity on IDA curves.

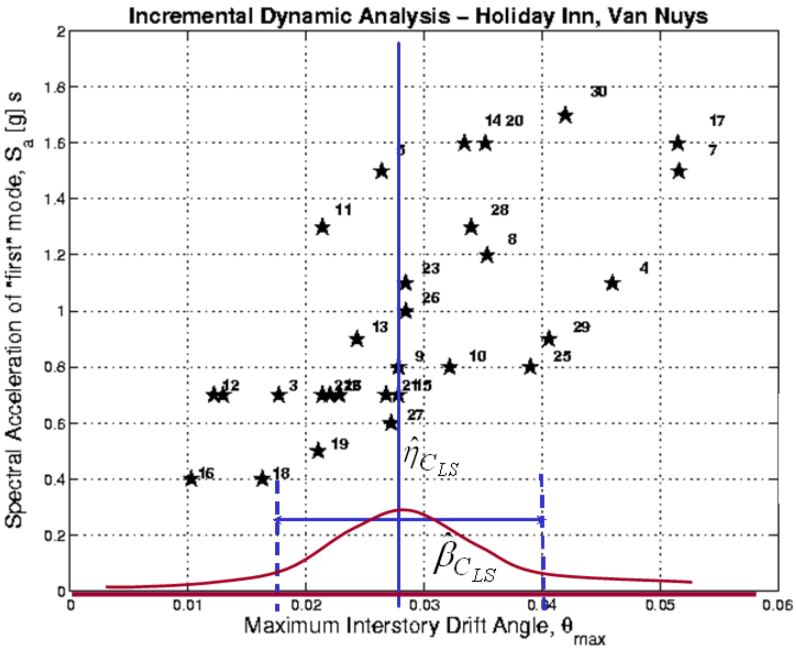

Figure 3. Probability density function of capacity from multiple IDA curves. Courtesy of Dr. Fatemeh Jalayer (2003).

the solid and dashed curves in the left panel represent the median and 18-84 percentiles of the distribution of the demand at each specific IM level. The probability that the capacity is exceeded according that distribution, is the failure probability for that IM.

5 The use of first-mode spectral acceleration as an IM

In describing the fundamentals of IDA it was assumed that the IM is the spectral acceleration at the fundamental mode of the structure, this is because not only hazard curves are readily available for $\boldsymbol{S}_a(\boldsymbol{T}_1)$, but also because it has been shown to be a sufficient and, to some extent, an efficient IM in many seismic risk engineering applications. It is to recall here that a sufficient IM makes the structural response variable conditionally independent of earthquake magnitude and distance, e.g. allow to use Eq. (3.3) to compute the seismic risk. At the same time a certain IM is defined efficient if the structural response, given IM, has a relatively small dispersion showing explanatory power of the seismic demand (e.g. the 16%, 84% IDA curves are "close" to the median curve).

Several studies have demonstrated that the displacement-based response measures, given $\boldsymbol{S}_a(\boldsymbol{T}_1)$, are independent of magnitude and distance (i.e. Iervolino and Cornell, 2005) and also that for drift response $\boldsymbol{S}_a(\boldsymbol{T}_1)$ is better (efficiency-wise) than PGA. Suffi-

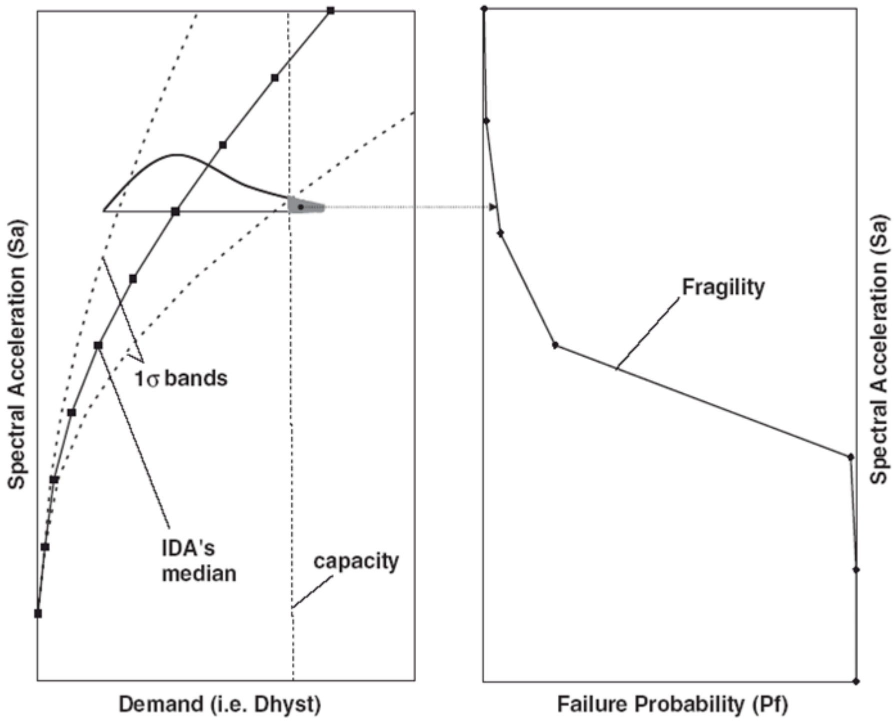

Figure 4. Example of determination of fragility form IDA curves (Iervolino et al., 2006).

ciency of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ has an important reflection on the record selection for seismic structural assessment. In fact, when real recordings are concerned, the current state of best practice is based on the determination of magnitude and distance most contributing to determine the $\boldsymbol{S}_a(\boldsymbol{T}_1)$ value corresponding to a specified probability[2] (i.e. a 10% chance of exceedance in 50 years) in the hazard curve. This provides a scenario earthquake and therefore, the records are then chosen to match within tolerable limits these values of M and R. Sufficiency of the IM significantly simplifies this procedure as all this care about the selected records' may be avoided, and records may be, at least in principle, randomly chosen and then modified to have the $\boldsymbol{S}_a(\boldsymbol{T}_1)$ value of interest just by the linear amplitude scaling described above.

It is also worth to note that it has also been recently demonstrated that $\boldsymbol{S}_a(\boldsymbol{T}_1)$ may not be particularly efficient, nor "sufficient", for some structures. If long periods of oscillation are called into question (i.e. tall structures), the higher modes typically play a larger role into the seismic response and $\boldsymbol{S}_a(\boldsymbol{T}_1)$ has less prediction power than

[2]This procedure is called disaggregation of seismic hazard and allows one to determine the M and R scenario causative for the acceleration corresponding to the probability of interest in the hazard curve, see Bazzurro and Cornell (1999) for details.

for first-mode dominated structures; it may be also insufficient because it is not able to capture the spectral shape in a range of frequencies where it depends on the magnitude. For soft-soil $\boldsymbol{S}_a(\boldsymbol{T}_1)$ may also be insufficient. There are also cases for which PGA may be a more suitable IM than $\boldsymbol{S}_a(\boldsymbol{T}_1)$, this happens for example, when the EDP to estimate is the peak floor acceleration, which is an important response variable correlated to inertia forces and therefore with non-structural damage (Taghavi and Miranda, 2003).

5.1 Using consistent Sa

In performing seismic assessment of structures via dynamic analysis it is important to bear in mind that structural engineers and seismologists sometimes intend Sa differently. This mismatch is due to the decomposition of ground-motion along two directions (Baker and Cornell, 2006a). For the aims of non-linear seismic assessment of structures, $\boldsymbol{S}_a(\boldsymbol{T}_1)$ is considered as the one along a single axis. Conversely seismologists may compute ground-motion prediction equations using the geometric mean of the spectral accelerations in the two horizontal directions; this is because using one arbitrary component would lead to a larger dispersion of hazard curves. Both uses of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ are legitimate, but inconsistent if combined for the probabilistic seismic assessment of structures. Therefore, it is preferable to define the same Sa in both the hazard and response. This means either that in the seismic risk analysis of structures one should use hazard curves that use one-component $\boldsymbol{S}_a(\boldsymbol{T}_1)$, or estimating structural response using the geometric mean of the two components as an IM. This latter method has the advantage of not requiring new ground-motion prediction equations for hazard analysis. However, it will introduce additional dispersion into the response prediction and Sa will result less efficient. Alternatively, if the structural response is estimated using a single axis $\boldsymbol{S}_a(\boldsymbol{T}_1)$, while hazard refers to the mean of the two components, the dispersion of the response may be inflated, as proposed by the cited authors, to reflect that which would have been seen if the mean $\boldsymbol{S}_a(\boldsymbol{T}_1)$ had been used as the intensity measure.

5.2 Sa and records' duration

First-mode spectral acceleration has also been proven to be sufficient in respect of duration, at least for SDOF structures and if displacement-related response measures are of concern (Iervolino et al., 2006). It was shown, in the mentioned study, choosing three different record samples representing different duration scenarios. For each of these samples the non-linear response has been evaluated via incremental dynamic analysis, considering several EPDs, from kinematic ductility to equivalent number of cycles. This allowed to compare median responses to different duration bins.

In this framework duration has been found to be statistically insignificant to displacement ductility demand assessment, regardless of oscillation period and backbone curve. Conversely duration has been found to strongly affect, as expected, other demand parameters accounting for cyclic behavior such as hysteretic ductility or equivalent number of cycles (Cosenza et al., 1993). This is shown in Figure 5 where median IDA curves are given, for a bilinear SDOF, for the three record sets representing the duration scenarios (rhombuses, squares and triangles for short, moderate and long duration respectively).

It may be seen that curves are perfectly superimposed in the case of kinematic duc-

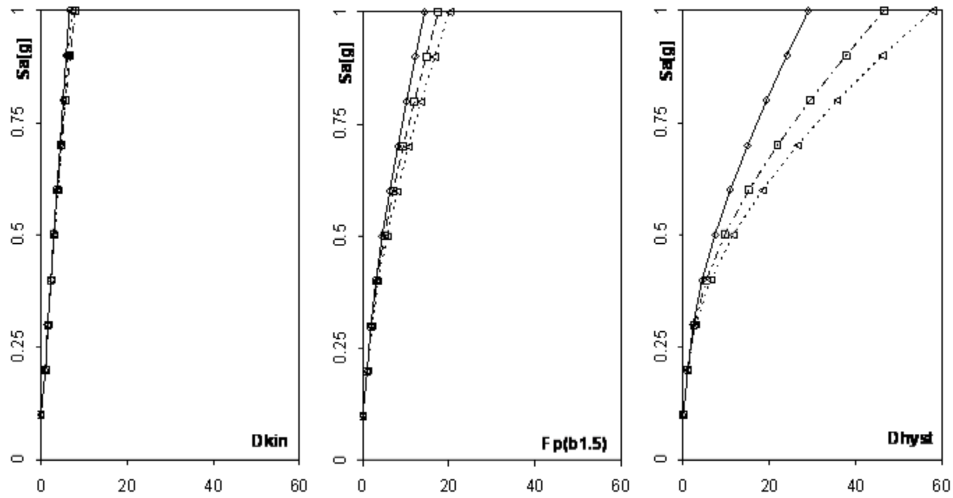

Figure 5. Comparison of median IDA curves, for an elastic-perfectly-plastic SDOF with a 1 sec oscillation period, in terms of kinematic ductility (left), plastic fatigue (center) and hysteretic ductility (right), (Iervolino et al., 2006).

tility, while for plastic fatigue the energy content of ground motion starts to play a role and median demand is proportional to the duration of the set that generated the curve. This effect, as expected is magnified in the case of hysteretic ductility.

Therefore, at least for the purposes of displacement-related demand assessment, it seems that one should also not take too much care in selecting records to match a specific duration value given they have (or are scaled to) a common spectral acceleration level.

5.3 Sa-based scaling of records and epsilon

Amplitude scaling of different records to get a target (common) intensity is useful to estimate the fragility of the structure; this do not bias the estimation of the response and also reduces its variability, achieving larger efficiency of the IM (Iervolino and Cornell, 2005). In fact, Figure 6 shows a record set used for non-linear dynamic analysis of a bilinear SDOF. The records are first used as unscaled (circles), and this is called cloud analysis; after the records in the set are scaled to a common Sa (which is the median of the set unscaled) and the response is computed. Results shows that the median response is similar in the two cases indicating no bias induced by scaling (this property is called robustness, e.g. the structural response estimated with scaled records is virtually the same as that from unscaled records featuring the same IM level). Moreover, the dispersion of the response (proportional to the horizontal bars in the figure) is significantly reduced with scaling.

However, there are cases in which scaling of the spectral acceleration of the record

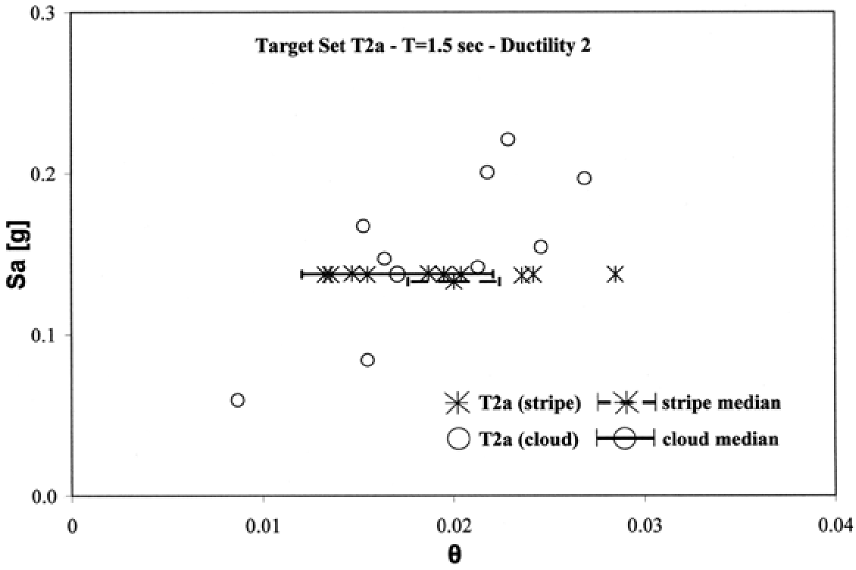

Figure 6. Variability of drift before and after scaling a set of records to the same IM level for a bilinear SDOF system with a 1.5 sec oscillation period (Iervolino and Cornell, 2005).

would introduce a bias in the estimation of the structural response. It may be the case when the record shows a deviation of a record's Sa from the value predicted by the ground-motion prediction equation (Baker and Cornell, 2006b). This deviation is called *epsilon*[3] or "normalized residual". High (positive) epsilon values are associated with peaks in the spectrum at the fundamental mode of the structure, and hence with more benign nonlinear structural behavior. In fact, during the shaking the effective period of the structure lengthens descending the peak toward a less energetic portion of the frequency content. Therefore, scaling down a positive epsilon record (to match a specific spectral acceleration value for example) would introduce an un-conservative bias in the demand estimation because, due to the lengthening of the period during the shaking, the structure will be sensitive to a part of the spectrum which is away from the peak; conversely scaling up a negative epsilon record could lead to an overestimation of the seismic response (Figure 7).

Even though some researchers believe epsilon is not an intrinsic ground-motion feature, PSHA disaggregation of seismic hazard for epsilon often shows that high IM levels, contributing directly to rare maximum interstory drift ratio levels, are associated with

[3]Epsilon (ε) is defined as the difference between the log of the spectral acceleration, at a given period, of a record and that predicted by an ordinary ground-motion prediction equation divided by the standard deviation of the residuals.

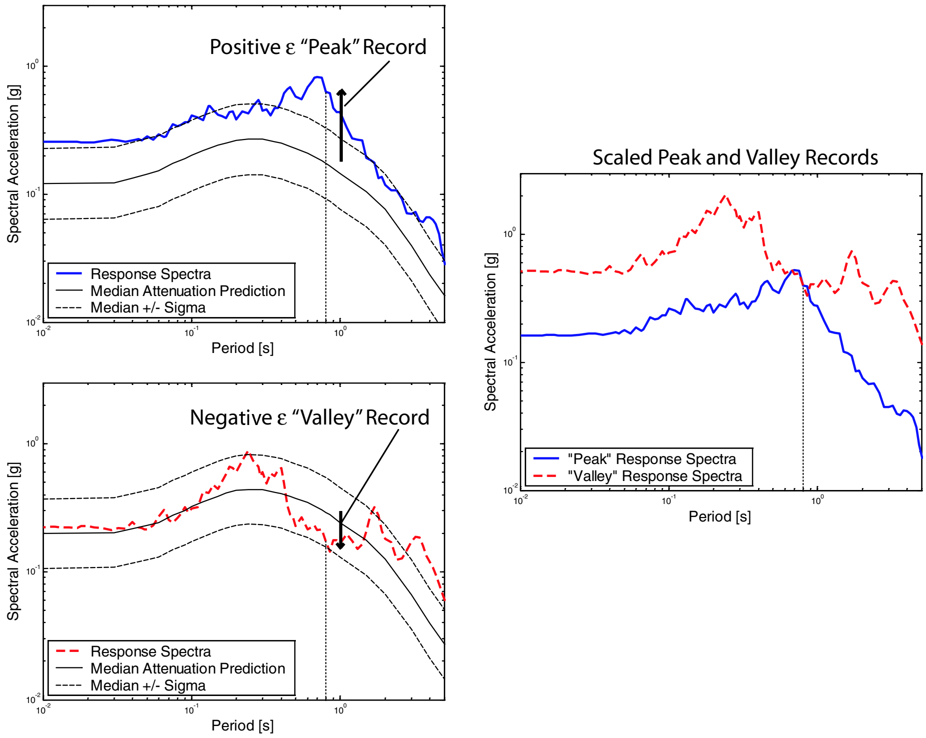

Figure 7. Scaling a negative ε record and a positive ε record to the same spectral acceleration at the period of 0.8 s. Courtesy of Jack W. Baker, see Baker and Cornell (2005) for details.

high values of epsilon. Therefore, when selecting records for analyses at these high IM levels, one should consider to choose them among those having the right epsilon, in order to have the correctly deviating spectral shape around the period of interest, for a more efficient and unbiased estimation of structural response. This is more important than matching records with scenario M and R values.

A method has also been proposed (Baker and Cornell, 2006b) for developing a target spectrum which accounts for the effect of magnitude, distance and epsilon. This spectrum, discussed in the following of the paper, allows the selection of records that only have a spectral shape that matches the mean spectrum from the causal event, without taking care of appropriate magnitude, distance and specific epsilon.

5.4 Near-source

It has been briefly reviewed above why in seeking for characteristics to mirror in the record selection one should look to any systematic effect on spectral shape as epsilon is. Therefore, it is prudent to avoid selecting records from soft soil sites or from near-source records showing directivity effects. In fact, it should briefly be mentioned that a site located close to the source of a seismic event may be in a geometrical configuration, in respect to the propagating rupture, which may favor the constructive interference of waves (synchronism of phases causing building up of energy) traveling to it, which may result in a large velocity pulse, Figure 8 (Tothong et al., 2007). This situation, for dip-slip faults, requires the rupture going toward the site and the alignment of the latter with the dip of the fault, whereas for strike-slip faults the site must be aligned with the strike; if these conditions are met the ground-motion at the site may show forward directivity effects (Somerville et al., 1997).

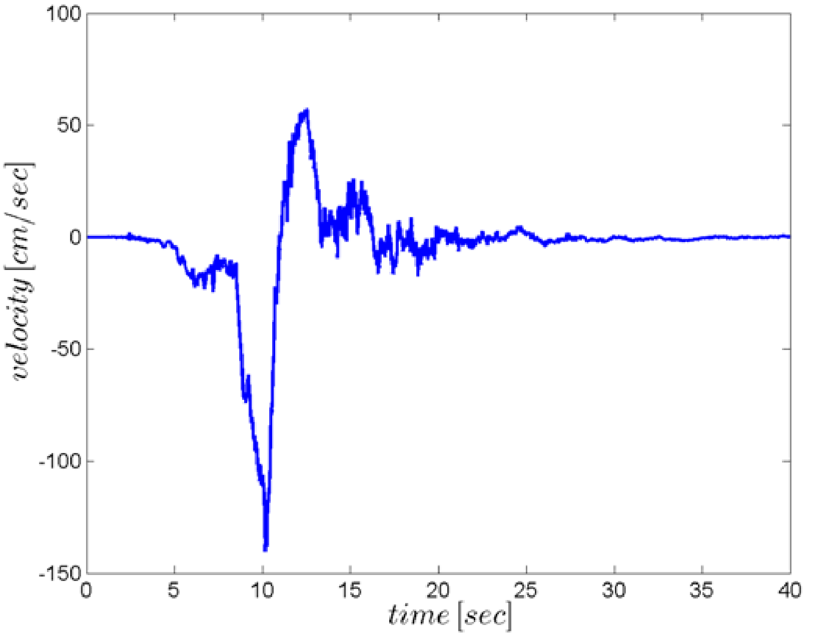

Figure 8. A directivity-related velocity pulse in the Lucerne record of the 1992 Landers earthquake (Tothong et al., 2007).

Parameters driving the amplitude of the pulses are related to the above-discussed rupture-to-site geometry, while empirical models positively correlating the earthquake's magnitude to the period of the pulse have been proposed by seismologists (Somerville, 2003). Pulse-type records are of interest for structural engineers because they: (1) may induce unexpected demand into structures having the fundamental period equal to a certain fraction of the pulse period; (2) such demand may not be adequately captured by ground-motion intensity measures such as first mode spectral acceleration (Tothong and Cornell, 2006). Therefore, common record selection practice and classical PSHA do not apply in the near-source. In fact, the latter requires ground-motion prediction relationships able to capture the peculiar spectral shape driven by the pulses and an estimation of the likelihood of pulse occurrence, while the former should produce record sets reflecting the pulse features compatible with the near-source PSHA (Iervolino and Cornell, 2007).

6 Beyond first mode-spectral acceleration: advanced IMs

Several recent studies propose IMs alternative to the first-mode spectral acceleration with the main purpose of having synthetic parameters able to capture the spectral shape in a period range related to the non-linear behavior of the structure, and to overcome some of the shortcomings of $\boldsymbol{S}_a(\boldsymbol{T}_1)$. They include both scalar and vectors, linear and non-linear quantities. A detailed review is given by Tohong and Luco (2007); in their paper the authors compare the use of different scaling methods for vector-valued and advanced scalar ground motion IMs.

An example of vector-valued IM is given by $\boldsymbol{S}_a(\boldsymbol{T}_1)$ and ε, which for the reasons discussed in section 5 has been found to be an effective predictor of structural response because it is an implicit measure of spectral shape. Using the vector $\boldsymbol{S}_a(\boldsymbol{T}_1)$ and ε allows to properly take into account the effect of peak and valleys in the spectrum of the record for which $\boldsymbol{S}_a(\boldsymbol{T}_1)$ alone is insufficient (Baker, 2007). In general the main benefit provided by vector-valued IMs is increasing efficiency, which means that fewer nonlinear dynamic analyses will be needed to characterize the relationship between structural response and the IM. However, vector-valued IMs have the disadvantage that they are more complicated to use in respect to scalar IMs. Therefore, advanced scalar IMs are worth to be investigated. One successful attempt in this direction is that of Cordova et al. (2001) which propose an IM, called $\mathbf{S}^{*}$, which is the product of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ and a non-dimensional parameter power function of the ratio of $\boldsymbol{S}_a(\boldsymbol{T}_f)$ over $\boldsymbol{S}_a(\boldsymbol{T}_1)$, Eq. (6.1), where $\boldsymbol{T}_f$ is a period accounting for the reduced stiffness of the structure because of non-linear behavior. This IM has proven to be more efficient than $\boldsymbol{S}_a(\boldsymbol{T}_1)$ as it somehow accounts for the spectral shape in a broader range.

$$\boldsymbol{S}^{*} = \boldsymbol{S}_a(\boldsymbol{T}_1) \cdot \left(\frac{\boldsymbol{S}_a(\boldsymbol{T}_f)}{\boldsymbol{S}_a(\boldsymbol{T}_1)} \right)^{\alpha} \tag{6.1}$$

An extension of this concept of capturing the spectral shape in a range is the so called Sa average, Eq. (6.2), which is the geometric mean of the spectral acceleration in an interval from T_1 until a final period of interest. Clearly, this IM is able to provide improved information about Sa values at a range of periods.

$$S_{a,avg}(T_1, \cdots, T_n) = \left(\prod_{i=1}^{n} \boldsymbol{S}_a(\boldsymbol{T}_i) \right)^{1/n} \tag{6.2}$$

Another advanced scalar IM for structural demands that are dominated by a first mode of vibration, is that based on inelastic spectral displacement, $\boldsymbol{S}_{di}$, of a bilinear SDOF. This IM which can be advantageous if compared to both elastic spectral acceleration and the vector consisting of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ and ε. It has been show to be sufficient, 50% more efficient (50% reduction in variability of response) than $\boldsymbol{S}_a(\boldsymbol{T}_1)$, and robust in respect to scaling. Tohong and Luco (2007) shows, via IDA, for the cases investigated that, using $\boldsymbol{S}_{di}$, the dependence of the structural responses on ε is substantially reduced, which therefore avoids the need to include ε in a vector IM. $\boldsymbol{S}_{di}$ also allows to overcome the problems with near-source pulse-like records, this is because that IM *can distinguish the amount of period elongation induced by the record*.

On the other hand, $\boldsymbol{S}_{di}$ is less efficient, sufficient and robust for higher-mode-sensitive structures (i.e. long-period buildings). These shortcomings are largely due to the fact that using $\boldsymbol{S}_{di}$ alone does not capture the ground motion frequency content at higher-mode periods. However, also $\boldsymbol{S}_{di}$ can incorporate second-mode effects, for example, via a scalar modification function of the elastic spectral displacement of the second mode and the elastic participation factors of the first two modes of the structure of interest. Authors show that this advanced IM leads to an increase in the efficiency, because it reduces the conditional dispersion of the response spectra.

It is finally worth to mention that the choice of an IM should not be unrelated to the possibility of computing the probabilistic seismic hazard for that IM. In fact, computing the seismic risk via Eq. (3.3) requires the distribution of the IM over a period of time at the site of interest. Therefore, it may be said that the best candidates to be ground motion intensity measures are those for which attenuation relationships are easy to compute.

7 Code-based record selection

It has been described, so far, the main issues in real records selection and scaling for non-linear dynamic analysis with the purpose of seismic risk assessment, which requires integration of fragility and seismic hazard. To discuss the code-based procedures to select the seismic input for nonlinear time-history analysis of structures, the concept of Uniform Hazard Spectrum (UHS) has to be clarified first. The UHS is an elastic spectrum defined on the basis of the seismic hazard at the site where the structure is supposed to be located. In particular, if for such a site, curves representing the probability of exceedance of elastic spectral acceleration, for example in 50 years, are available for several oscillation periods, $\boldsymbol{T}$, in a range of engineering interest, it is possible to enter all this curves with a the same specified probability (i.e. 10%) and get the corresponding spectral ordinate, one for each $\boldsymbol{T}$ value. These uniform probability ordinates determine a spectrum if plotted versus the corresponding $\boldsymbol{T}$ values. Such a spectrum is called Uniform Hazard Spectrum and it is often used as a reference for structural assessment and design via non-linear dynamic analysis.

As, discussed above, advanced practice today (i.e. in U.S.) would find a seismologist responsible for providing input to an engineer who has to do a nonlinear dynamic assessment for design. The seismologist would provide (1) a probabilistic seismic hazard analysis, (2) for one probability level, a uniform hazard spectrum, and (3) for such level, a suite of n accelerograms for use in nonlinear dynamic analyses. Typically the seismograms have been selected to reflect the likely magnitudes, distances, and other earthquake parameters thought to dominate the hazard at the site; this choice is guided disaggregation of hazard (Bazzurro and Cornell, 1999). Finally, the records are usually scaled to match the UHS level at the period corresponding to the first mode of the structure.

In Figure 9 (top), the hazard curves for a site in southern Italy are reported for several oscillation periods in the range of 0-2sec (data provided by the Istituto Nazionale di Geofisica e Vulcanologia, http://esse1.mi.ingv.it/, accessed November 2007). These curves are all entered at 10% probability and the corresponding acceleration values are plotted versus each period to form the UHS, which is displayed in the bottom panel along a set of records scaled, as an example, to the level of the UHS corresponding to T = 0.6 sec.

The engineer will subsequently run time history analyses for each of the n accelerograms in the chosen set, and observe for each one or more measures of structural performance, for example, MIDR. If the average of MIDR of the n records exceeds 7% (in a steel moment resisting frame) he may conclude that frame failure is likely given that ground motions.

Code-based procedures (i.e. in Europe) apparently try to approximate this procedure. In fact, to design using an UHS corresponding to a small probability of exceedance is, in principle, analogous to choose a conservative value of the action in the load-resistance factor design and therefore it is consistent with the common design philosophy of codes worldwide, which allows the practitioner to check the seismic structural performance in semi-deterministic conditions where the action are amplified and the capacity is reduced on a probabilistic basis.

However, the use of UHS requires the seismic hazard at the site provided for all the national territory, national agencies often take care of this. In the U.S., for example, hazard curves and UHS for design may be downloaded by the USGS website (http://earthquake.usgs.gov/research/hazmaps/design/); recently also Italy has such a service by the mentioned website and it is likely that the next generation of Italian codes will allow for the use of site-specific UHS for design and assessment of structures. This is not the case for many other countries where seldom engineers are able to easily obtain PSHA data for the site of interest, therefore the record selection procedure described above is often only approximated as happens in the case of Eurocode 8 (EC8), record selection prescriptions of which are discussed below.

7.1 EC8 prescriptions for buildings and bridges

In EC8 the seismic input for time-history analysis is defined after the elastic response spectrum. In Part 1 (CEN, 2003), which applies for buildings, the spectral shapes are given for both horizontal and vertical components of motion. In Section 3.2.2 two spectral shapes, Type 1 and Type 2, are defined. The latter applies if the earthquake contributing most to the seismic hazard has surface waves magnitude not grater that 5.5, otherwise the former should be used. In Figure 10 the 5% damped elastic spectra for the five main soil classes are given as normalized in respect to a_g, which is the anchoring value of the spectral shape.

a_g is to be determined depending on the seismicity of the site of interest. In Italy, for example, the seismic territory is divided into four zones representing different hazard levels, where seismic resistant design is mandatory only in the upper three zones. The ag values for the Zone 3, 2 and 1 are 0.15g, 0.25g and 0.35g respectively. These values are related to the probabilistic seismic hazard analysis (PSHA) for the site of interest.

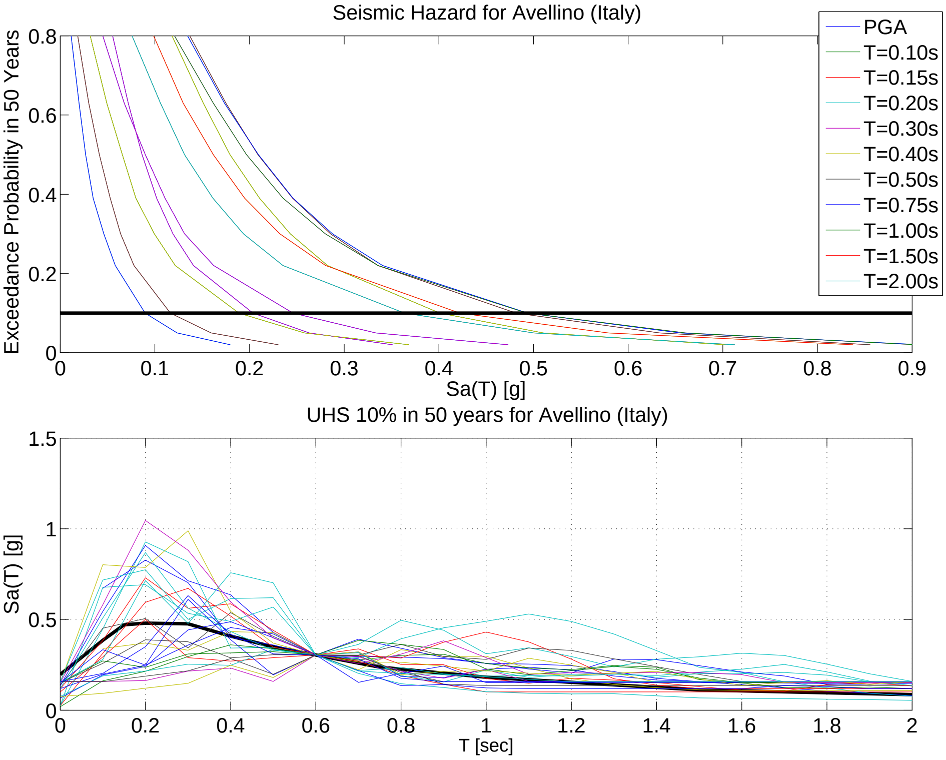

Figure 9. 10% in 50 years exceedance probability Uniform Hazard Spectrum (bottom) for the site of Avellino (Italy), from the INGV hazard data (top) available at http://esse1.mi.ingv.it/.

In fact, if the PGA (on rock) with a 10% exceeding probability in 50 years falls in one of the intervals]0.25g, 0.35g],]0.15g, 0.25g], or]0.05g, 0.15g], then the site is classified as Zone 1, 2 or 3 respectively (OPCM n.3519, 2006). It is clear, therefore, the indirect relationship between seismic hazard and the code spectrum, which may be considered a crude approximation of the UHS.

Eurocode 8 part 1 allows the use of any form of accelerograms for structural assessment. The main criterion the set of accelerograms should satisfy, regardless they are natural, artificial or simulated, is that in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum. The upper limit accounts for the lengthening of period due to the non-linear structural behavior, while the lower considers the contribution of higher modes to structural response.

Some duration prescriptions are given for artificial accelerograms, while recorded or

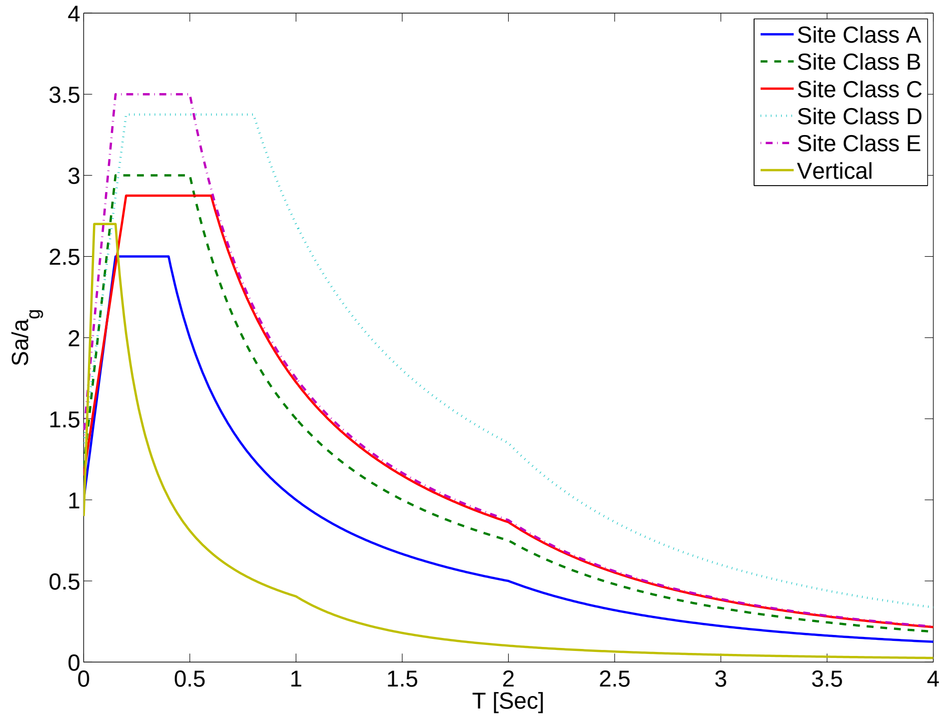

Figure 10. EC8 horizontal and vertical type 1 spectral shapes.

simulated records should be adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site.

The set has to be made of at least 3 accelerograms, however, the code allows the consideration of the mean effect on the structure, rather than the maximum, if at least seven non-linear time-history analyses are performed. In the case of spatial structures, the seismic motion shall consist of three simultaneously acting accelerograms representing the three spatial components of the shaking; therefore 3 is the minimum number of triplets to be used. However, the vertical component of the seismic action should be taken into account only in special cases, as long span elements, not applying to most common structures. Therefore, sets for analysis of common spatial structures are made up of 14 records. The code also specifies that the same accelerogram may not be used simultaneously along both horizontal directions.

EC8 Part 2 (CEN, 2005) refers to the same spectral shapes of Part 1 in order to define the seismic input for time-history analysis of bridges. The requirements for the horizontal seismic input for dynamic analysis are somehow similar to those for building but not coincident. The main differences are: a) for each earthquake consisting of a pair of horizontal motions, the SRSS spectrum shall be established by taking the square root of the sum of squares of the 5%-damped spectra of each component; b) the spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step; c) the ensemble spectrum shall be scaled so that it is not lower than 1.3 times the 5% damped elastic response spectrum of the design seismic action, in the period range between $0.2T_1$ and $1.5T_1$, where T1 is the natural period of the fundamental mode of the structure in the case of a ductile bridge. The prescriptions for the vertical component of motion in part 2 are the same as part 1, except the fundamental vertical mode has to be considered in place of T_1.

Note that EC8 Part 1 specifies that the chosen set should not underestimate the code spectrum more than 10%. The prescription of EC8 Part 2 seems to be equivalent because the SRSS, for equal spectra in the two directions, is exactly one of the two spectra times 1.4. Part 2 also allows the consideration of the mean effects on the structure, rather than the maximum, when non-linear dynamic analysis is performed for at least seven independent ground motions. The effects of the vertical seismic component on the piers may be omitted in cases of low and moderate seismicity, while in zones of high seismicity these effects need only be taken into account special cases. EC8 Part 2 has also specific prescriptions for near-source conditions. The code also prescribes when the spatial variability of ground motion has to be considered.

The EC8 criteria clearly try to relate the assigned spectrum with the hazard at the site (the code spectrum is related to the hazard for the site of interest only through the anchoring value). Moreover, prescribing that records chosen for time-history analysis should reflect magnitude and other scenario parameters, the code tries to reproduce record selection based on disaggregation of seismic hazard.

In Figure 11 an example of a horizontal natural records set matching the EC8 Part 1 spectrum for a site with ag = 0.25g in the range 0-2 sec, is given (Iervolino et al., 2008). In Figure 12 a set reflecting spectral matching criteria of EC8 Part 2 in terms of horizontal and vertical component of motion is given (Iervolino et al., 2007).

It emerges from the cited studies that the code prescriptions do not allow to control the record-to-record variability (important to estimate the response with a limited number of analyses) and also that may be unfeasible to satisfy the matching to any specific source parameter and at the same time having the required average spectral compatibility if real records are concerned. Therefore, spectrum matched or synthetic records seems to be favored by EC8 and by codes that prescribe the matching with a specific (smooth) spectral shape in general.

7.2 Beyond the uniform hazard spectrum: conditional mean spectrum including epsilon

It has been discussed that the current best practice (not yet taken in by many codes worldwide) in record selection for seismic structural assessment relies on the concept of uniform hazard spectrum. However the UHS has some limitations that should be bared in mind when using it. The UHS is built considering hazard curves independently, which do not allow to properly account for the actual probability of joint occurrence of spectral ordinates at different periods. Moreover, the low-period range of the UHS is often dominated by small earthquakes "close" to the site, while the high-period branch is dominated by far and high magnitude events (Baker and Cornell, 2006b). For these reasons the UHS hardly may represent the spectrum from a single threatening event, and

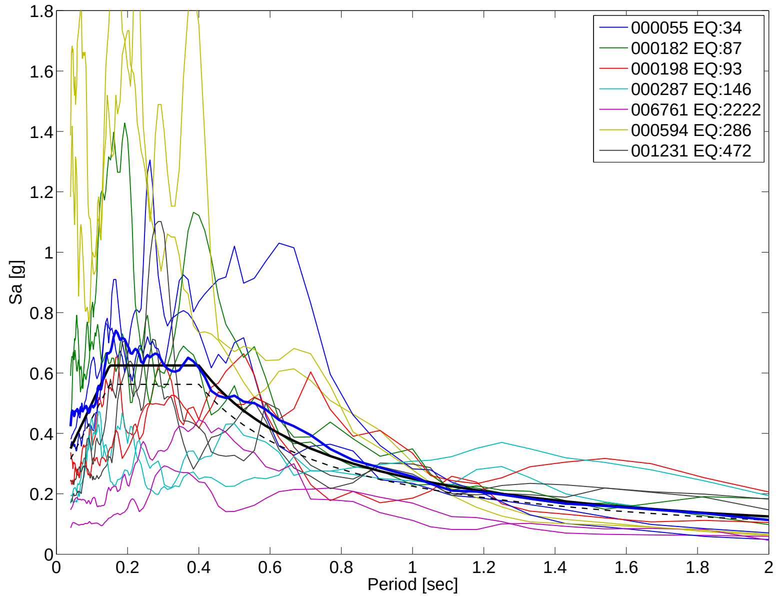

Figure 11. Example of a record set matching, in the average, the EC8 part 1 criteria for a site with ag = 0.25g (Iervolino et al., 2008).

the UHS is often quite conservative in probabilistic terms. In fact, even when a record has a spectral acceleration value as large as the UHS at a given period, it is unlikely to be as high as the uniform hazard spectrum at other periods.

A method has been proposed by (Baker and Cornell, 2006b) to develop a target spectrum which overcomes the limitations of the uniform hazard spectrum. Such a spectrum is an extension, to include epsilon, of that used in the U.S. nuclear industry and accounts for the effect of magnitude and distance from disaggregation. This spectrum is called conditional mean spectrum including epsilon (CMS-ε); the fundamental steps of the procedure to built the CMS-ε are:

1. Evaluate, from the spectral acceleration hazard curve corresponding to the fundamental mode of the structure, the value of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ corresponding to the exceeding probability of interest;
2. Disaggregate the PSHA curve at the determined value of $\boldsymbol{S}_a(\boldsymbol{T}_1)$ and obtain the design (scenario) M, R, ε_{T_1} values;
3. Compute, for any period T_2 different from the fundamental period of the structure, the approximate median spectral ordinate under the assumption of jointly Gaussian distribution of the logarithms, e.g. via Eq. (7.1).

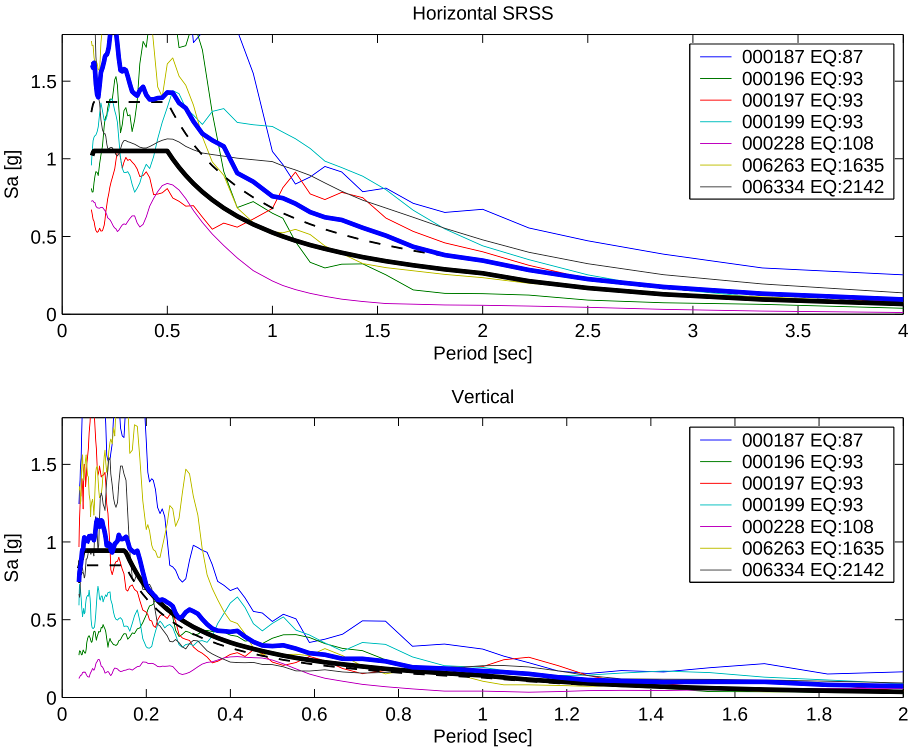

Figure 12. Example of a record set matching, in the average, the EC8 part 2 criteria for horizontal (top) and vertical (bottom) components of ground motion (Iervolino et al., 2007).

$$\mu_{ln\mathbf{Sa}(T_2)|ln\mathbf{Sa}(T_1)} = \mu_{ln\mathbf{Sa}(T_2)|\mathbf{M},\mathbf{R}} + \sigma_{ln\mathbf{Sa}(T_2)|\mathbf{M}}\rho_{ln\mathbf{Sa}(T_1),ln\mathbf{Sa}(T_2)}\varepsilon_{T_1} \tag{7.1}$$

where $\mu_{ln\mathbf{Sa}(T_2)|\mathbf{M},\mathbf{R}}$ is the mean of the log of Sa for the disaggregated magnitude and distance (from an attenuation relationship, which is usually expressed in logarithmic terms), $\sigma_{ln\mathbf{Sa}(T_2)|\mathbf{M}}$ is the standard deviation of the residuals of the attenuation at the same period and conditioned to the disaggregated magnitude; $\rho_{ln\mathbf{Sa}(T_1),ln\mathbf{Sa}(T_2)}$ is the correlation coefficient of the log of the spectral ordinates at the two periods (i.e. from Baker and Cornell, 2006c), and ε_{T_1} is the disaggregated epsilon value;

4. Repeat step 3 for all values of T_2 in the interval of interest and get the conditional mean spectrum accounting for epsilon.

In Figure 13 examples of conditional mean spectra are compared to the UHS. It is clear how the CMS-e is more probabilistically consistent than the UHS, and therefore allows to control more directly the likelihood of the performance one is designing for. Moreover, it has been shown that searching records with a shape similar to that of the conditional mean spectrum including epsilon allows to not take care in selection of appropriate magnitude, distance and specific epsilon coming from disaggregation. In fact, records with a spectral shape similar to that of the CMS-ε induce on the structure the same unbiased response than records chosen to reflect scenario M, R and ε_{T_1}. Therefore, CMS-ε seems to be a good candidate to overcome the shortcomings related to the UHS in respect of which is also less conservative.

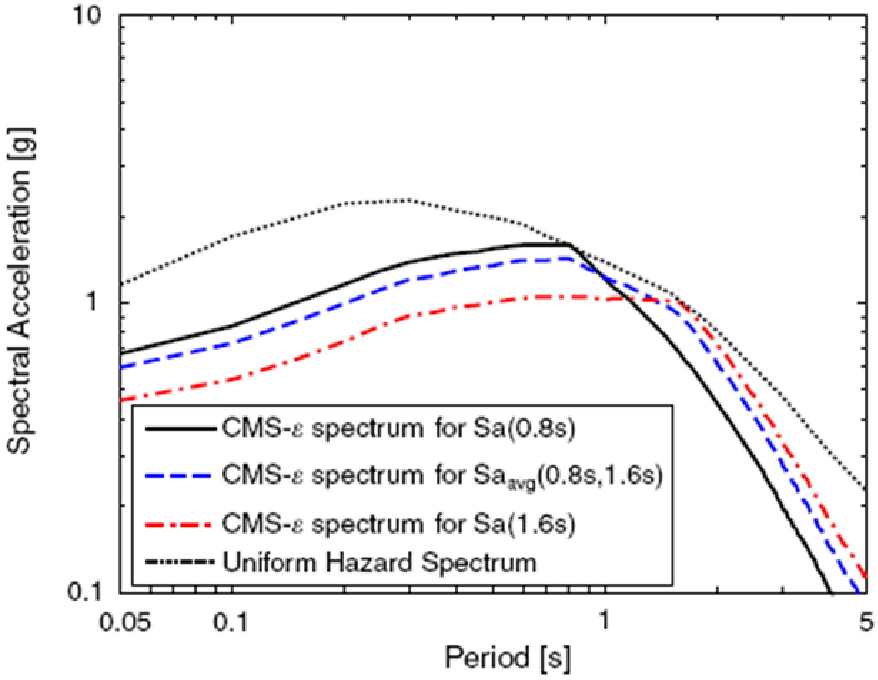

Figure 13. Examples of conditional mean spectra compared to the corresponding UHS. Courtesy of Jack W. Baker see Baker and Cornell (2005) for details.

7.3 Time scaling of records and shake table testing

It is finally to mention a manipulation procedure of real ground motion records that is nowadays not used frequently, if at all, for the computer-aided simulation of structures, while it is still popular in the shake table testing of structures if the specimen is reduced-scale. This procedure is the time-scaling of records; it consists of multiplying the time scale by a factor, similarly to what is done in amplitude scaling of records via the IM discussed above. However, while the records subjected to amplitude scaling show response spectra amplified by the scaling factor and keep the same shape of the unscaled record, the time scaling changes all properties involving time. In particular, scaling a record in time by a scaling factor $\boldsymbol{SF}$ implies that the ordinates of the original acceleration, velocity and displacement spectra have to be multiplied by 1, $\boldsymbol{SF}$, and $\boldsymbol{SF}^2$, while the abscissas have to be divided by to obtain the spectra of the modified signal (Kaldjian and Fan, 1968). When both time and amplitude scaling are performed the effects of the two procedures on the spectra should be superimposed.

Because time scaling changes the frequency content of the signal it was used when there was lack of recordings, since the earthquake engineering community has gotten more records (e.g. in the last 20 years) and some spectral matching scheme accomplishes the frequency content changes more intelligently this procedure has been almost abandoned. However, in the case of shake-table testing of structures time scaling of records may still be useful. In fact, usually shake-table infrastructures are limited in dimensions and load capacity so that it is not possible to test full size structures and reduced-scale models have to be employed instead. In this case real ground motion records used as an input for tests are compressed in time by a factor equal to the square root of the scaling factor of the specimen to be tested in respect to the full-scale case. In this way the frequency content of the compressed record at the natural frequency of the undamaged reduced-scale test structure is the same as that of the un-compressed record at the natural frequency of the test structure without scaling (Hashemi and Mosalam, 2006).

8 Summary

Both the increase in easy available computing power and in the accessibility to natural ground motion recordings have enhanced the attractiveness of non-linear dynamic analysis for seismic structural assessment. In the paper the main issues and recent literature regarding seismic input preparation, when real ground motion records are concerned, have been reviewed. It emerges that, in the probabilistic structural assessment, the record selection is driven by the purpose of achieving small computational effort to estimate the response correctly and accurately. In the code-based design, in principle, the hazard is disaggregated to determine the design scenario and records are chosen to mach it.

In the probabilistic assessment, the number of records required to estimate the seismic risk may be extremely large and may be driven by many parameters. The introduction of a ground-motion intensity measure simplifies the assessment of seismic risk. To reduce the number of records to estimate the structural performance one should use a ground motion intensity measure able to capture the spectral shape in a range the structure is sensitive to, rather than try to match any event feature as magnitude and distance or faulting style. (This, procedure also takes advantage of amplitude scaling of records, while time scaling is usually used in experimental testing of reduced size specimens.)

If displacement response is of interest, it is widely accepted that first-mode spectral acceleration is a more efficient IM in respect to PGA, as it allows to select records without taking care of magnitude, distance and related features as duration. Moreover, it allows to estimate the response with a smaller uncertainty. However, $\boldsymbol{S}_a(\boldsymbol{T}_1)$ has also shown some limitations when employed in incremental dynamic analysis which may lead to an incorrect estimation of the performance in respect to unscaled records, especially for structures for which the response is dominated by a wider portion of the spectral shape, or if the hazard is contributed by near-source pulse-like events. Therefore, advanced vector-valued and scalar IMs have been proposed, an example of that is an IM which accounts for the spectral ordinates at the first and lengthened periods. Also the inelastic spectral displacement is an efficient scalar measure representing an sufficient and robust alternative to vectorial IMs. However, it requires a specifically modified hazard analysis as it happens for sites dominated by near source effects.

The more advanced code procedures for record selection and structural assessment are based on the concept of uniform hazard spectrum which has been shown to produce conservative estimation of seismic performances. A possible improvement is represented by the conditional mean spectrum which has a more sound probabilistic basis. However, often codes do not allow even the use of UHS for a number of reasons. First of all, the discussed procedures requires a large amount of information especially regarding the hazard at the site seldom available to engineers. Moreover, as they are not fully standardized, the record selection procedures involve a series of semi-arbitrary choices of the practitioner which is reasonable to implicitly control. The result is that, for example in Europe, the code-based record selection prescription are based on standard spectral shapes and are not easy to apply to real records favoring the use of spectrum matching records; although they have shown some, even significant, limitations in the estimation of the structural performance.

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