EARTHQUAKE DESIGN SPECTRA FOR PERFORMANCE-BASED DESIGN

M.D. Trifunac and M.I. Todorovska
Univ. of Southern California, Los Angeles, CA, 90089, USA

SUMMARY
The principles that guide the selection of earthquake design spectra and their performance-based objectives are examined through the procedures for adopting the design earthquake ground motions. The linear elastic design spectra are used to illustrate a contemporary scaling approach. The amplitudes and shapes of the Eurocode 8 (EC8) spectra are compared with what is known about strong ground motion. The estimation of seismic hazard is then discussed, and it is argued that hazard mapping in terms of one scaling parameter (peak ground acceleration) is not uniformly conservative at all frequencies and that it is in contradiction with the performance-based design objectives. It is suggested that seismic hazard mapping for use with national earthquake-resistant design procedures should be carried out via the Uniform Hazard Spectrum method.

Keywords: Earthquake design codes, elastic spectra in Eurocode 8, Uniform Hazard Spectra.

1. INTRODUCTION

Earthquake design code should be formulated based directly on an in-depth understanding of the intricacies of the nonlinear response of structures, in such a way that it wisely simplifies the complex phenomena and grasps the significant and dominant phenomena. This view is different from most of the current code-development approaches, which rarely start anew from the evolving knowledge about the physical nature of the problem, and which typically focus only on fine-tuning of the previously adopted parameters and on the correction procedures aimed at extending or correcting the contemporary codes for the observed discrepancies (Trifunac 2012). With the introduction of performance-based design principles into earthquake-resistant design, it is now necessary to work with realistic descriptions of the strong-ground-motion amplitudes. In terms of the traditional approach to specifying the design forces via response-spectrum amplitudes, it is now necessary to at least specify the amplitudes and shapes of the design spectra that are consistent with the performance-based design requirements. It is argued in the following that there is no point in performing the multiple levels of the performance-based design if inadequate and biased spectral shapes are used in the process.

The performance-based guidelines can be specified with respect of two different levels of excitation, which are termed the *no-collapse* and *damage limitation* requirements. The no-collapse requirement calls for the structure to withstand the design seismic action without local or global collapse and to retain its integrity and residual load-bearing capacity after the seismic event. This is associated with the largest credible level of ground shaking. What this means specifically is defined by the national committees of experts, and it could be described, for example, by a no-collapse requirement for seismic action with a probability of exceedance equal to P_{NCR} = 10% in 50 years (EC8). The damage limitation requirement calls for the design that will withstand the seismic action without the occurrence of damage and the associated limitations of use. EC8 guidelines recommended that this level of shaking be associated with the probability of exceedance of P_{DLR} = 10% in 10 years.

The structural design for *no-collapse* conditions will require nonlinear response analysis, while the *damage limitation* requirement will be associated with essentially linear response analysis. During the largest credible levels of shaking, the structure and its foundation soil will experience large nonlinear deformations, so that the *no-collapse* analysis will be associated with considerably longer system periods than the periods in the analysis of the same structure for the *damage limitation* requirements. In terms of the design ground motions, it is seen that the *no-collapse* and *damage limitation* analyses will have to be determined not only by the spectral amplitudes with different probabilities of being exceeded, but also at different system periods. Since the shapes of hazard spectra of strong ground motion depend on many factors, including the probability of being exceeded, it is seen that selection of the design spectra in terms of peak-acceleration and fixed-shape design spectra cannot satisfy the performance-based design objectives.

Contemporary design practices for *no-collapse* conditions add further complexities and increase the uncertainties in the final outcome by (1) providing only approximations associated with modifying the linear response spectrum to represent a nonlinear response spectrum, and (2) performing only what is at best an approximate nonlinear response analysis. The resulting uncertainties and errors can be larger than the errors and approximations in the selection of the linear response spectra for design. Analyses and discussion of these uncertainties is however beyond the scope of this paper. With a view that all stages in the design process should be performed as accurately as possible because the errors will propagate downstream, in this paper we focus only on the selection process for amplitudes of the linear response, which is equivalent to spectral characterization of strong ground motion.

The comments in this paper should apply to all modern earthquake design codes, but EC8 will be used as a vehicle to illustrate some specific inconsistencies with the stated objective of the performance-based design. The methods that lead to rational and region-specific amplitudes and shapes of the elastic design spectra will also be discussed. It is assumed that the reader is familiar with the details of at least one of the leading contemporary design codes, so that it is not necessary to outline all aspects of the code procedures. Our discussions will focus on the scaling of the response spectrum because it is the principal tool for selection of the design forces in engineering analyses of structures. Through its relation to the Fourier amplitude spectrum, it describes the frequency content of the design strong ground motion, and thus it is also a starting point for selection of strong-motion time histories for nonlinear response analyses. A crude and approximate approach to nonlinear design employs reduction factors to decrease the linear-response-spectrum amplitudes in order to determine design forces or design displacements. In spite of its many uncertainties, this approach is popular and common in simplified engineering design. In this paper, when discussing the consequences of the selected spectral amplitude for nonlinear response analyses the authors will think in terms of the method based on the time integration of nonlinear, dynamic differential equations.

2. DESIGN CODES

The modern work on developing building codes began in 1908, following the Messina earthquake in Italy; in Japan following the 1923 Tokyo earthquake; and in California after the Santa Barbara earthquake of 1925 (Freeman 1932; Suyehiro 1932). The “Provisions Against Earthquake Stresses,” contained in the Proposed U.S. Pacific Coast Uniform Building Code, was prepared by the Pacific Coast Building Officials Conference and adopted in 1927, but the provisions were not generally incorporated into municipal building laws (Freeman 1932). Following the 1933 Long Beach earthquake in California, the Field Act was implemented, and Los Angeles and many other cites in California adopted an 8% g for the design base coefficient. During the following four decades, earthquake design codes underwent many changes and revisions. Then, in 1978, the Applied

Technology Council (ATC) issued its ATC-3 report on the model seismic code for use in all parts of the United States. This report, written by 110 volunteers working in 22 committees, incorporated many new concepts, including more realistic ground-motion intensities. Much of the current Uniform Building Code is derived from the ATC-3 report.

Traditional code spectra were smaller than the spectra computed from recorded accelerograms. The code committees chose smaller amplitudes to reconcile the linear nature of the spectral method with the expected nonlinear response of structures. Thus, adopted spectra were termed "design spectra," and the design amplitudes were scaled using reduced (effective) peak ground "acceleration."

Eurocode 8

The leading modern seismic codes are EC8 (Eurocode 8 2005; Kappos 2010) and the International Building Code (IBC 2009); the latter has recently replaced the long-established previous codes, such as the *Uniform Building Code* (1997), in North and Central America. Most of these codes share essentially the same principles and design procedures. In Europe, the current design procedures are based on two standard shapes for the *linear* response spectra, Type 1 and Type 2. Type 1 spectra carry more energy in the long-period ground motions and are for use in regions with high seismic activity. Type 2 spectra are for areas that are experiencing low-to-moderate seismicity and that have less energy in the long-period motions and larger amplitudes at short periods than Type 1 spectra.

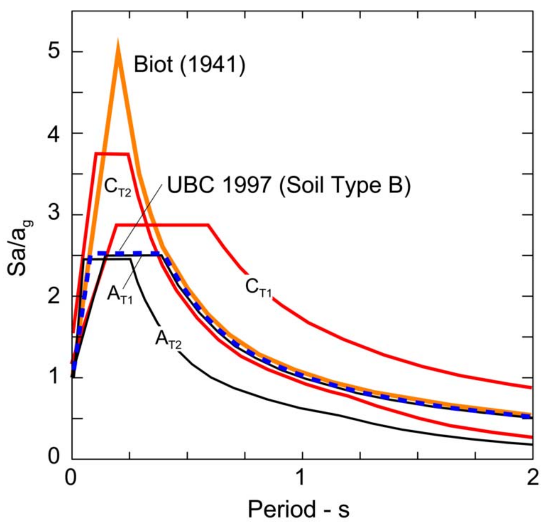

Figure 1. Comparison of spectral shapes of EC8 Type 1 (A_{T1} and C_{T1}) and Type 2 (A_{T2} and C_{T2}) at ground sites A and C, with UBC 1997 (soil type B) and Biot 1941 spectra (from Trifunac 2012).

The generating shape for the linear response spectra in EC8 is defined by the scaling parameters S and η, while the shape is determined by the prescribed functions in the four period intervals: between 0 and T_B, T_B and T_C, T_C and T_D, and beyond T_D. Tables in EC8 define those scaling parameters (Trifunac 2012), but a comparison with the spectra in older codes shows that the EC8 spectra closely follow the previous experience (Fig. 1). The scaling parameter η in EC8 depends on the damping of the equivalent single-degree-of-freedom system and is given by $\eta = \left[10/(5+\varsigma)\right]^{1/2} \geq 0.55$, where ς is the fraction of critical damping expressed as a percentage. For ς = 5%, the damping scaling parameter η = 1. If the earthquakes that contribute most to the seismic hazard at the site have surface wave magnitude not greater than 5.5, EC8 recommends the use of the Type 2 elastic spectra, but when the local seismicity is expected to generate larger events, Type 1 elastic spectra should be used. In this respect, EC8 represents an improvement over the older codes, which typically ignored the changes in spectral shapes with earthquake magnitude. To account for the effects of local site conditions, EC8 distinguishes five ground types: A, B, C, D, and E (soil Types C or D over material with $v_{s,30}$ > 800 m/s), where $v_{s,30}$ represents the average shear-wave velocity in the top 30 m of soil at the site. While the EC8 code uses the term "ground types," it can be seen that those represent in fact only five ranges of soil stiffness near the surface, with no reference to the thickness of the soil layers or the geological deposits below (Lee and Trifunac 2010). The code allows more detailed consideration of the site effects in terms of "deep geology," which can be specified in the individual National Annexes, and which can also include the values of the parameters S, T_B, T_C, and T_D. In addition, it allows consideration of the associated resulting changes in the spectral shapes of both horizontal and vertical earthquake motions.

The EC8 acknowledges the need for more realistic procedures for selection of spectral amplitudes and spectral shapes. Unfortunately, thus far many committees responsible for the formulation of National Annexes have chosen not to address this need but instead have simply opted for the old approach of scaling the spectral amplitudes using peak acceleration, and also for the description of site conditions, which ignores the site geologic classification (Trifunac 2009).

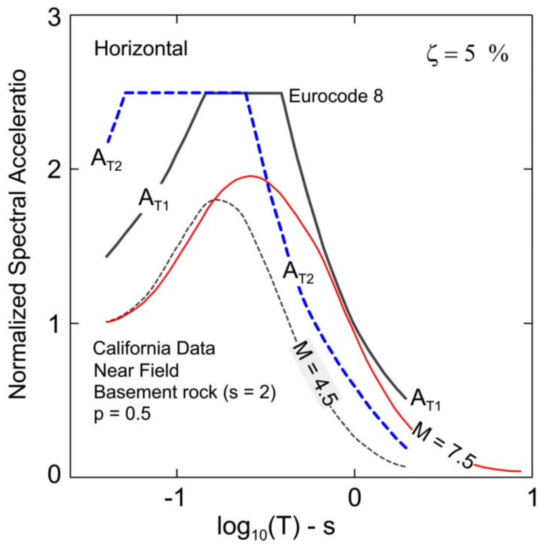

Figure 2. Comparison of elastic spectral shapes of EC8 (Types 1 and 2) at soil site A with the spectra based on the strong-motion data recorded at rock sites in California.

The strong ground motions that correspond to *no-collapse* and to *damage limitation* requirements are caused by different earthquakes in time and space, and consequently their spectral amplitudes and spectral shapes will be different. Thus, these two requirements cannot be satisfied by one fixed spectral shape (Type 1 or 2) and by scaling of spectral amplitudes by their corresponding peak acceleration. The only way to meet these two requirements is to work with spectra, which have variable shape and are not scaled by the corresponding peak accelerations. In view of this, the Eurocode 8 guidelines, which recommend that the national territories should be subdivided by national authorities into seismic zones, depending on the local hazard, and that the hazard within each zone can be assumed to be constant, are in contradiction with the stated performance-based requirements. The procedure for describing this hazard in terms of a single parameter, the reference peak acceleration on Type A ground—which is chosen by the national authorities for each seismic zone and corresponds to the reference return period T_{NCR} for the no-collapse requirement, or to the reference probability of exceedance P_{NCR} in 50 years—is likewise in contradiction with the expected performance-based design objectives.

According to EC8 the earthquake motion at a given site on the ground surface is represented by the elastic, absolute acceleration spectrum, or simply the *elastic response spectrum.* The shape of this spectrum is taken as a Type 1 or Type 2 spectrum of seismic action, and it is the *same* for the no-collapse and damage-limitation requirements. The two orthogonal horizontal components of motion are described by the same spectrum. The vertical spectrum is also defined by the same shape functions as the horizontal elastic spectrum in the four intervals between 0 and T_B, T_B and T_C, T_C and T_D, and beyond T_D, except that the scaling factor $2.5a_g S\eta$ used for horizontal motions is replaced with $3.0a_{vg}\eta$, where a_{vg} is the reference vertical-peak ground acceleration. Selection of other elastic acceleration spectral shapes can be specified in the National Annex of the EC8. In this process, consideration should be given to the magnitude of earthquakes that contribute most to the seismic hazard and not to the conservative upper limits like the most credible earthquake, for example. For important structures, topographic and other local site-amplification effects should be taken into account, and for long structures the variation of ground motion in space and time should be considered. Research has shown that the additional effects of differential motions can be particularly important when such structures are relatively rigid and long (Jalali and Trifunac 2007).

Kappos (2010) describes the application aspects of EC8, and presents comparisons with the IBC (2009), which generally adopts the ASCE 7 standard (MDLBS 2006). He states that the EC8 is a *performance-based* code, in line with current trends for this code format. However, due to its nature (a code that should be accepted by and implemented in countries with very different seismic hazards, as well as seismic design "cultures"), Kappos notes that EC8 does not go all the way toward a multiple-performance-objective check. Rather, it focuses on a single performance objective (limit state), the one related to *protection of human life*, while serviceability (or damage limitation) is checked in a rather simplified way. A review of all parts of EC8 can be found in the published literature, which includes books (Faccioli et al. 2005) and chapters (e.g., Chapter 4 in Kappos 2001), which describe in detail the provisions as well as the background of this Eurocode. Kappos (2010) discusses in a critical way whether current code procedures are adequate and whether the new generation of seismic codes should be based on the currently adopted "philosophy" or should switch toward new proposals that are based on response quantities such as displacements and/or deformations. The paper provides a step-by-step summary of the EC8 procedure for seismic design of buildings.

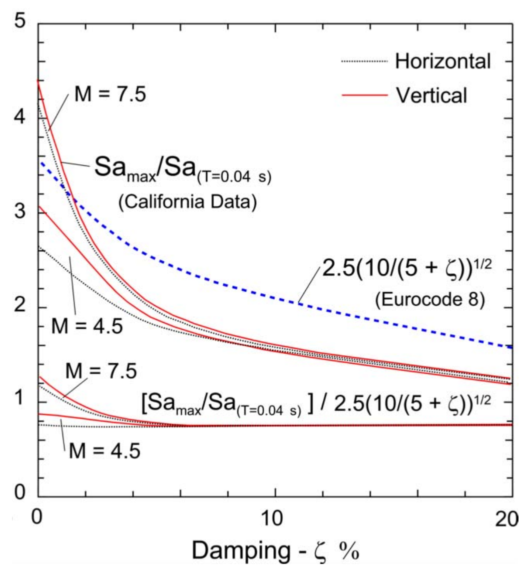

Figure 3. Comparison of the dependence of the ratio $Sa_{\max} / Sa_{(T=0.04s)}$ (maximum amplitude divided by the spectral amplitude at T = 0.04 s) on the fraction of critical damping ς (in percent), with 2.5η, where $\eta = \left[10/(5+\varsigma)\right]^{1/2}$.

3. SOME LIMITATIONS IN SCALING OF EUROCODE 8 SPECTRA

In the following, we discuss some limitations of the EC8 spectral shapes and examine the alternatives that can reduce or eliminate them.

The spectra used in most design codes are fixed and represent the average trends for all epicentral distances and for all earthquakes. The EC8 is an exception. It approximates the dependence of spectral shape with respect to magnitude by two spectra, Type 1 and Type 2, for seismicity characterized by earthquakes with surface-wave magnitude less than 5.5 (Type 2) and for seismicity that includes larger events (Type 1). Dependence of the normalized (to unit peak acceleration) spectral shapes shows that larger earthquakes lead to a larger presence of the long-period strong-motion energy (Fig. 2). This dependence is well understood, and a description of the factors that govern it can be found in the papers that deal with direct empirical scaling of strong-motion spectral amplitudes (e.g., Lee 2007). Here, we illustrate only one additional dependence—on the fraction of critical damping in the SDOF oscillator—which is used in the computation of the response spectral amplitudes. The difference between the peak of the spectral accelerations $Sa_{\max}$ and the spectral amplitude $Sa_{(T=0.04s)}$ at the anchoring short period at T = 0.04 s increases when the fraction of critical damping ς decreases.

Figure 3 shows $Sa_{\max}/Sa_{(T=0.04s)}$ vs. the fraction of critical damping ς expressed as a percentage, computed from the empirical scaling equations of Trifunac and Anderson (Trifunac 2012) for strong-motion data in California. The dependence of this trend on the regional differences should be minimal because it is governed mainly by the physical nature of oscillator response and less by the regional differences in spectral shapes and the duration of strong ground motion. This figure also shows 2.5η, where $\eta = \left[10/(5+\varsigma)\right]^{1/2}$, which, according to EC8, describes the amplitudes of the normalized spectral maxima by $Sa_{\max}/Sa_{(T=0.04s)} = 2.5\eta$. It can be seen that for damping, ς greater than ~4%, $\left[Sa_{\max}/Sa_{(T=0.04s)}\right]/2.5\eta$ is approximately equal to 0.75, indicating that 2.5η over-estimates the spectral peaks by ~25%. However, for ς less than 1%, 2.5η is not conservative and under-estimates the spectral peaks by as much as ~15%. Since most examples and discussions of the ratio $Sa_{\max}/Sa_{(T=0.04s)}$ revolve around ς equal to 5%, for this value of damping the EC8 spectra will be associated with peak amplitudes too large by ~25%. This applies also to all examples considered in this paper.

The scaling factor S in EC8 describes the spectral amplitudes at "zero-period" and is equal to 1.0 for both Type 1 and Type 2 spectra, and for the ground site type A. This leads to distortions of high-frequency spectral amplitudes defined by EC8 (Fig. 2) since all empirical scaling equations for peak acceleration are based on the recorded data, which does not contain the frequencies higher than 25 Hz.

Recorded strong-motion data show a significant reduction of peak accelerations at progressively "softer" geological and soil formations underlying the site, which is mainly associated with nonlinear site response. At sites that also have soft soil layers, this nonlinearity can begin even at strain levels as small as 10^{-4} (Trifunac and Todorovska 1996). With reference to the recommended spectral scaling coefficients in EC8, this would imply that S should progressively decrease for sites from A to D. We find just the opposite trend in EC8 tables. Why this is so in EC8 is not clear, and the code commentary does not provide an explanation.

The peaks of elastic spectral amplitudes, $Sa_{\max}$, in EC8 are determined by $2.5\eta S$, where S determines the overall spectral amplitudes. This leads to overestimates of elastic spectral amplitudes by about 25% when the fraction of critical damping is equal to or larger than 5%. For small damping and large earthquakes, $\eta = \left[10/(5+\varsigma)\right]^{1/2}$ leads to spectral amplitudes that are not conservative.

The linear (transfer-function) representation of strong ground motion can be viewed in the frequency domain as $O(f) = E(f)P(f)S(f)$, where *f* is frequency; *O*(*f*) and *E*(*f*) are, respectively, the Fourier spectra of the motion at a site and at the earthquake source; and *P*(*f*) and *S*(*f*) are the transfer functions of the propagation path and of the local site effects, respectively. This representation is meaningful for epicentral distances that are large relative to the source dimensions, when the earthquake source can be approximated by a point source. However, in the near field, the small distance between the site and the large area of the rupturing fault results in geometrical nonlinearities, which are caused by the spatial distribution of wave arrivals from different segments of the fault surface. Thus, in the near field, *O*(*f*) ceases to be valid because *E*(*f*), *P*(*f*), and *S*(*f*) become complex, geometrically nonlinear functions of the space coordinates.

For two sites having different site conditions and a separation distance that is small relative to a large epicentral distance, it is reasonable to assume that their motions will differ mainly due to the differences in *S*(*f*), while their *P*(*f*) can be assumed to be nearly the same. This reasoning has evolved into a framework for most theoretical and empirical studies of the effects of site conditions on the amplitudes of strong ground motion (Trifunac, 2009). In many previous studies that analyzed the nature of the role of $S(f)$, it has been associated with the geological site effects, soil site effects, or both of those together. While using this approach, it is important to define precisely and a priori what is included in $S(f)$ to avoid ambiguity in interpreting the end results. It is remarkable how many papers, even some written by very experienced researchers, use imprecise site descriptions (e.g., by mixing the geological and soil-site conditions), only to arrive at wrong conclusions. In this respect, most modern earthquake design codes make the same fundamental mistake and present the variations of spectral shapes only in terms of $v_{s,30}$, which represents only the average shear-wave velocity in the top 30 m of soil at the site (Lee and Trifunac 2010; Trifunac 2009). The EC8 uses the term "ground types" to characterize the site characteristics for use in spectral-amplification analyses, but those are in fact *only* five incomplete descriptions of *soil* types, within the 30 m below the ground surface. The code does allow more detailed consideration of the site effects in terms of "deep geology," and it states that those can be specified in the individual National Annexes, which can also include the values of the parameters S, T_B, T_C, and T_D. Thus, the EC8 leaves it to the national code committees to refine the description of the local soil and geological site conditions. Unfortunately, this refinement rarely takes place, and the spectral shapes and amplitudes end up being used in engineering practice only in terms of the soil-site classification, which leads to biased and in some instances erroneous design (Trifunac 2009). In view of the above comments, the seismic hazard maps should not be presented in terms of peak accelerations expected at the "basement rock sites," with expectation that the local site response (amplification) can be calculated by vertically propagating shear waves through "soft" soil layers. In spite of the fact that this approach is popular in the geotechnical engineering profession, it leads to unnecessary additional biases in the prediction of ground-motion amplitudes, and it is contradicted by the analyses of recorded motions. Seismic waves *do not arrive at the building site vertically*, and their amplification *cannot and should not* be modeled by one-dimensional models (Trifunac 2009).

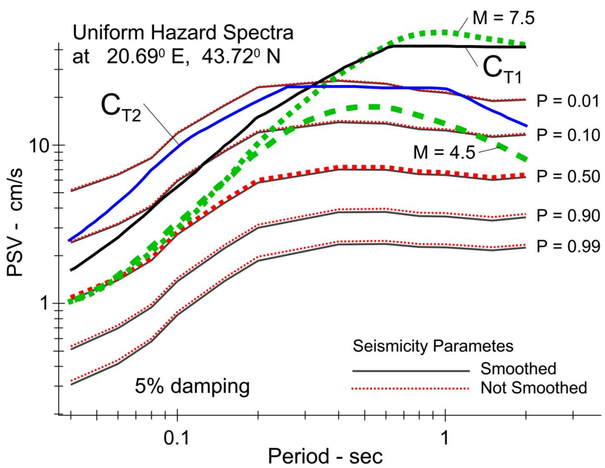

Figure 4. Uniform Hazard Spectra of PSV for 5% damping, at geological site condition s = 0 (sediments), local soil site condition S_L = 1 (stiff soil), for Y = 50 years of exposure and probabilities of being exceeded 0.01, 0.10, 0.50, 0.90, and 0.99. Continuous and dotted lines show differences in using smoothed or not smoothed representation of seismicity parameters (see Lee at al. 2011). EC8 spectral shapes at C soil sites for Type 1 and Type 2 spectra (C_{T1} and C_{T2}) and normalized spectral shapes for California (for M = 4.5 and 7.5) anchored to peak acceleration for UHS amplitude at T = 0.04 s and p = 0.50 are also shown.

4. UNIFORM HAZARD METHOD

A balanced and realistically formulated design spectrum must include all relevant factors that contribute to its amplitudes. It must also successfully pass the tests that compare its predictions with subsequent earthquake outcomes. The methodology for estimating the uniform-hazard spectra (UHS) at a site (Anderson and Trifunac 1978) offers a general approach that meets those requirements. The

UHS is the spectrum that has the same probability, at all frequencies, of spectral amplitudes being exceeded by any event, which can affect the site during its exposure to earthquake shaking in Y years. The UHS method requires: (1) description of the area surrounding the site in terms of all seismic sources, their activity, and geometrical extent; (2) site characteristics in terms of local soil conditions and the depth of sedimentary deposits or the site geological classification; and (3) description of attenuation of strong-motion amplitudes with distance from the earthquake source.

To see how the UHS approach can be used, the reader can peruse, for instance, the example from the Los Angeles Metropolitan area (Lee and Trifunac 1987). We note that all of the earthquakes, including the Northridge earthquake of 1994, that have occurred during the past 25 years since this report was published in 1987 have so far not contradicted the maps of UHS amplitudes in the report, and thus this constitutes a fair test of this method. Lee and Trifunac (1987) show how different the UH spectral shapes and amplitudes can be, even for two sites that are relatively close to each other (e.g., less than 30 km), when the sites are located on different geological structures and when their distances to the active faults are different. The remarkably different spectral shapes and amplitudes illustrated in their work for the Los Angeles metropolitan area show the futility of evaluating the seismic hazard by one amplitude parameter only (peak acceleration) and using the fixed-shape design spectra (specified by the design codes).

In the following example, we show the UHS computed for the city of Kraljevo, in Serbia, which experienced a moderate earthquake in 2010. Figure 4 shows the UHS for PSV amplitudes in Kraljevo for probabilities of exceedance between 0.01 and 0.99, with 5% damping, for exposure of 50 years, at sedimentary sites (s = 0), and for stiff soil (S_L = 1). Added to these curves are (1) the normalized spectral shapes for M = 4.5 and 7.5 (from Fig. 2), scaled by peak acceleration determined by hazard analysis (heavy dashed and dotted lines); and (2) Type 1 (C_{T1}) and Type 2 (C_{T2}) spectral shapes defined by EC8 for sites with soil Type C (180 < $v_{s,30}$ < 360 m/s), also scaled by peak acceleration determined by hazard analysis.

It should be clear from Fig. 4 that design ground motions cannot be determined accurately by scaling the fixed-shape spectra by peak ground acceleration. The errors can be too large, not only because the shapes of Type-1 and Type-2 (or some other fixed-shape) spectra may not be acceptable for a given region, but also because any fixed-shape spectra cannot represent the balanced contributions from local seismicity for the purposes of the performance-based design at all spectral periods.

In this regard, EC8 states (§3.2.2.1) that: "When the earthquakes affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to enable the design seismic action to be adequately represented. In such circumstances, different values of peak ground acceleration will normally be required for each type of spectrum and earthquake." In the limit, considering all such events and describing the outcome by a distribution of computed spectral amplitudes would converge to a result that is equivalent to UHS for the site. However, this limit is rarely approached because in most projects the design spectra are chosen based only on the five to seven largest earthquake events.

5. DISCUSSION AND CONCLUSIONS

It can be seen that the basic approach for selection of the earthquake design spectra has not changed since the introduction of the response spectrum concept in the 1930s. Modern design codes have incorporated many improvements, but several basic problems still remain, including: (1) scaling the spectra in terms of only one variable, such as a_g; (2) using the spectral shape that is fixed (i.e., that does not depend continuously and simultaneously on magnitude, epicentral distance, soil and geologic site conditions, and probability of exceedance; and (3) using the spectral shape that does not depend on the spatial distribution of the contributing earthquake sources. The consequence of all these simplifications is that the final outcome of the design process is not representative of the shaking that can be expected at the building site, may be seriously biased, and is not uniformly conservative over the frequency band covered by the buildings in the population of any given urban environment. The economic consequence of these simplifications is the waste, especially the biased and wrong distribution of the construction and rehabilitation costs, which is a burden that no society should have to bear. This problem can be fixed only by using the variable-shape design spectra, which include all relevant frequency-dependent factors contributing to the variability of spectral amplitudes.

This paper shows how the UHS method can be used to determine the elastic spectral amplitudes, which have frequency-independent probabilities of being exceeded. All necessary scaling models and seismicity data are available for implementation of such an approach in the countries where strong-motion data have been recorded, and these models can be used for region-specific description of attenuation of strong-motion amplitudes. Construction of UH elastic acceleration spectra can be performed from tables computed for the regional models of seismicity and distribution of active faults (as in Lee and Trifunac 1987 and Lee et al. 2011), or they can be approximated from tables computed for uniform seismicity.

The process, which leads to the selection of the design spectra, includes steps that are each associated with some aleatoric uncertainties that accrue toward overall uncertainties in the final result, which is the performance-based design. While we cannot eliminate all of these uncertainties, we can and must eliminate or reduce those that result from inadequate regional scaling of strong ground motion and from incomplete and erroneous modeling of the processes involved. Examples presented in this paper describe such errors, omissions, and inconsistencies in the scaling of strong-motion amplitudes, which can all be easily and immediately corrected. While the design of important structures like nuclear power plants, for example, will always require detailed studies of three-dimensional seismic activity surrounding the site, for most engineering applications UHS computed from a continuous representation of seismicity in terms of parameters a, b, and $M_{\max}$, as in Lee et al. (2011), for example, can be readily done for almost any country.

The anachronic scaling of design spectra for performance-based design in earthquake engineering, in terms of peak acceleration, will soon become a subject of past history. This is because all modern and economically rational societies recognize the necessity for wise use of their resources, and so they require sound and realistically formulated insurance strategies against natural disasters.

The methods we considered in this paper are suitable for implementation in intermediate and far fields of large, shallow earthquakes, where linear and almost-linear mechanics can provide reasonably accurate representations of wave motion in sediments and soil, as well as of structural response. The chaotic phenomena that accompany large, nonlinear response in the near field are beyond the scope of this paper and will be dealt with separately in our future studies. The multitude and the complexity of the factors that govern the large, nonlinear response of soils and structures need both broader and more numerous scaling distributions that go well beyond the classical response spectrum method.

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