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EXECUTIVE SUMMARY

A primary objective of the “Seismic Hazard Harmonisation in Europe (SHARE)” project is to provide a European seismic hazard analysis that is compatible with the engineering requirements of Eurocode (CEN, 2004), and to ensure that the pan-European seismic hazard results can be applied to improve and harmonise the Nationally Determined Parameters (NDPs) in the national code applications. A working consensus on the required output of this analysis, in terms of the demands for effective mitigation and risk modelling, is required. Eurocode will become the European building standard in 2010, with further revisions expected to follow in future. This report will outline the current demands of Eurocode 8 (EN 1998) in terms of the seismic action, and assess similar approaches taken in building codes around the globe. Three objectives are addressed within the SHARE project: 1) summary of the seismic input requirements in the current Eurocode, 2) assessment of the state-of-the-art requirements found in other building codes around the globe and 3) review of these requirements in the context of “best practice” in seismic hazard analysis and earthquake engineering. As the aim of this material is to assess the state-of-the-art in the definition of seismic action, we focus primarily on those provisions that represent the current standard in their respective regions of application. Amongst the codes considered are the current United States provisions (NEHRP, 2003; 2009), the New Zealand Standard (NZS 1170.5), the Building Standard Law of Japan (BSL-Japan, 2000), New Standard for Construction in Italy (NNTC-Italy, 2008) and the National Building Code of Canada (NBC, 2005). As a point of comparison, other existing codes including the 2002 seismic provisions for Indonesia and the 2007 provisions for Pakistan are also considered. The critical analysis of the definition of the seismic input will focus on several key areas where practice may vary across the globe. This includes requirements for performance-based seismic design and return period; definition of the response (or “design”) spectrum; identification of the controlling earthquake scenario; integration of epistemic uncertainty into the seismic action, and the use of acceleration time histories.

Performance-Based Seismic Design

The experience of the 1994 Northridge earthquake and the 1995 Great Hanshin [Kobe] earthquake demonstrated that both economic losses and human casualties could be considerable, even if the no-collapse objective had been met for many structures (Bommer & Pinho, 2006). Performance-based seismic design (PBSD) formalises the approach of citing multiple objectives for structures to withstand minor or more frequent levels of shaking with only non-structural damage, whilst also ensuring life-safety and no-collapse under severe shaking (ATC, 1978). These objectives define the limit states, which describe the maximum extent of damage expected to the structure for a given level of ground motion. Many seismic codes define limit states or performance objectives according to the return period (T_R) (or probability of exceedence, P_R) of a given level of ground motion. In EN 1998, two requirements are cited: 1) “no-collapse” (recommended $T_R = 475$ years) and 2) “damage limitation” ($T_R = 95$ years).

In many seismic design codes, performance-based requirements are implemented explicitly on the basis of return period. On others, serviceability and operational objectives may be assumed implicit, and are not necessarily afforded an explicit return period. Despite the diversity of definitions of different limit states, there is clear persistence in the adoption of 475 years as a basis for “life-safety”, although several codes have recently begun to adopt 2475 years as the return period for the no-collapse criterion, albeit subsequently rescaled to

incorporate an assumed inherent margin of safety against collapse (e.g., NEHRP, 2003; NBC, 2005).

Although the range of return periods recommended for performance requirements in EN 1998 is typical of those found in other codes, there is a complicating factor. The return period for each limit states is assigned as a Nationally Determined Parameter, thus allowing the value to be selected by each participating country's National Authority. It is therefore necessary for the SHARE output to define hazard at a range of return periods, or allow for hazard at a site, to be scaled to intermediate levels. EN 1998 suggests a convenient scaling relation for this approach; however, the recommended scaling power (k) is shown to vary significantly from that suggested in the code. If the scaling approximation is to be applied, particularly by virtue of modification of the design return period within a National Annex, it may be useful to identify the k -value appropriate to each site and to outline the limitations (in terms of upper and lower bound return periods) of the approximation.

The 2009 revision to the NEHRP *Provisions* introduces a new conceptual approach to the definition of the input seismic action (NEHRP, 2009). The seismic input (maximum considered earthquake) is modified by a risk coefficient (for both short and long periods). This coefficient is derived from a probabilistic formulation of the likelihood of collapse (Luco *et al.*, 2007). These modifications change the definition of seismic input to that which ensures a more uniform level of collapse prevention. This approach may be of interest for application in future modifications of Eurocode, where the collapse capacity of the structure can be defined and modelled within the National Annex. This would allow National Authorities autonomy to determine the level of seismic input needed for design in a manner that is appropriate to existing construction practice within the country, whilst also making the most effective use of the regionally harmonised seismic hazard output from SHARE.

The risk-targeted approach presented by Luco *et al* (2007) only considers the no-collapse limit state. It remains to be seen how this can be adapted to considered serviceability and damage limitation requirements. The degree of seismic detailing required to meet performance-based objectives is affected by the behaviour of the structure at lower intensities. A longer term objective for performance-based seismic design may be to consider the relative cost-benefit that a given level of detailing produces. Such an approach has been proposed by Bommer *et al.* (2005), who describe an iterative procedure to determine the cost versus benefit using displacement-based earthquake loss assessment. This approach would allow for designers to consider losses at other limit states besides collapse. The output of SHARE should provide sufficient information to begin to assess how this approach may be implemented within seismic code provisions.

Site Classification & Amplification

The classification of shallow geology in EN 1998 is largely consistent with practice in many other codes. Some provision is made to allow National Authorities to consider the influence of deeper geology on the hazard at a site. The influence of deep geology on the amplification of strong motion, along with basin and topographic amplification effects, is well-established but difficult to constrain in seismic hazard analysis. For most ground motion models, such effects contribute to the aleatory variability of the ground motion attenuation. The current practice by which hazard is defined for a reference rock site, and then scaled in accordance with site class, limits the extent to which more complex site effects can be incorporated into the analysis.

Few codes specify requirements to define the deeper geological characteristics of a site. In applications for ordinary structures, the cost and feasibility of obtaining a geotechnical profile to greater depths provides a limitation upon the extent to which deep geology can be defined. An alternative classification scheme, using both the horizontal-to-vertical spectral ratio fundamental period (T_0) and V_s for the full soil column, could be considered to assist in characterising the influence of deeper geology on the site effect. Such an example scheme is presented here, and comparison can be drawn with similar provisions in the New Zealand and Japan standards, which both use T_0 to assist in site classification.

Design Spectrum

One of the most significant areas where the seismic input definition in EN 1998 differs from that of other state-of-the-art codes is in the characterisation of the elastic response spectrum. Only one hazard parameter (PGA on a rock site) is currently used to anchor the full hazard spectrum, with corner periods and amplification factors fixed to specified values in the code provisions, according to whether the controlling earthquake magnitude is less than or greater than M_S 5.5. There is significant disparity between the code-specified design response spectrum, and both uniform hazard spectra and scenario spectra for typical European earthquakes (Bommer & Pinho, 2006; Bommer *et al.*, 2010). Whilst many of the parameters used to define the spectra may be altered by National Annex, this simplification may mask the influence of many different seismological factors upon seismic hazard at a site. It should also be recognised that the classification of the spectrum type on the basis of M_S is undesirable. M_W has become the standard magnitude parameter in empirical ground motion prediction models and for probabilistic seismic hazard analysis in general. Conversion between magnitudes introduces additional error, which is entirely unwarranted given that the arbitrariness of the M_S 5.5 boundary. It is strongly suggested that this particular requirement be amended in EN 1998.

In other seismic design codes constraint of the elastic response spectrum via two or more spectral ordinates is both practical and desirable. As more countries adopt approaches similar to those of the 2009 International Building Code it is likely that this will develop into a standard in future code revisions. NNTC-Italy (2008) also illustrates how the elastic response spectrum can be constrained by more ordinates to provide greater consistency with the uniform hazard spectrum (UHS), whilst still being practical to implement. With the development of a web portal for dissemination, however, there exists a practical means of allowing designers to select more spectral ordinates relevant to the site to better constrain the hazard spectrum. An alternative approach by Bommer *et al.* (2010) allows for the construction of a more realistic spectrum based on empirical relations between PGV/PGA ratio and key parameters of the elastic response spectrum (e.g., corner periods, decay factors and damping coefficients). This requires only three separate inputs (assuming T_D is constrained via the long-period controlling scenario) from the seismic hazard analysis. The use of empirical relations may also allow National Annexes to modify the coefficients of the equations to ensure a more appropriate fit to seismic hazard. Alternatively, the use of an open web portal for dissemination of seismic hazard output should provide a means of constructing a full (with the period limitations of the ground motion model) acceleration and displacement hazard spectrum. This may ultimately prove to be of more relevance to end-users of the hazard analysis.

Issues in Seismic Hazard

Seismic Zonation

EN 1998 prescribes the use of zones for which reference peak ground acceleration (PGA) hazard on a “rock” site (a_{gR}) is assumed uniform. Many seismic codes are moving away from this particular practice, opting instead to define the hazard directly for the site under consideration (e.g., NEHRP, 2003, 2009; NBCC, 2005; NNTC-Italy, 2008; MOC, 2008), or allowing for interpolation between contoured levels of uniform hazard (NZS 1170.5, 2004). Uniform zones effectively decouple the seismic input for design from the seismic hazard; often requiring a substantial degree of expert judgement in the delineation of the zones. This matter is complicated even further if new approaches to better constrain the response spectrum were to be adopted. Whilst zonation based exclusively on PGA may be simple to implement, modification or redevelopment of zones to incorporate the influence of longer period hazard, controlling earthquake scenarios etc., results in a substantial increase in complexity, with potential for bias. It is therefore suggested that in light of the SHARE output the use of zones be reconsidered in future Eurocode revisions.

Disaggregation

Disaggregation of the seismic hazard (McGuire, 1995; Bazzurro & Cornell, 1999), whilst rarely made explicit in seismic design codes, may be an inherent part of the seismic hazard process to meet other key provisions. In particular, specification of the “controlling [scenario] earthquake” is often necessary where provisions are made for the use of acceleration time histories in dynamic structural analysis. This is the case for Eurocode where controlling earthquake scenarios are required to define the type 1 ($M_S > 5$) or type 2 ($M_S \leq 5$) spectrum, and also to guide the selection of acceleration time histories in terms of compatible magnitude, distance, fault mechanism and site type.

The adoption of disaggregation output within seismic code provisions presents several challenges. The first challenge is that of disseminating the output. Hazard disaggregation typically presents the controlling earthquake in terms of a magnitude-distance- ϵ triple, where ϵ represents the number of standard deviations above the median ground motion. This information may be supplied by mapping each element (M , R and ϵ) separately, yet this approach is unwieldy and omits information about the probability distribution of each element, which may be of interest to engineers. A more transparent approach is the dissemination of the full disaggregation output using a web application, in the manner currently implemented via the scenario calculator compiled by United States Geological Survey. This allows the user to determine the disaggregation for the site, spectral period and return period of interest, even going so far as to disaggregate the epistemic uncertainty and identify controlling scenario for each ground-motion prediction model. Similar web-based disaggregation calculators have also been developed for Italy (INGV, 2008) and Canada (NBC, 2005).

If the full disaggregation is supplied for a location there remains some ambiguity as to how it should be interpreted. For any given set of M - R - ϵ bins, the controlling earthquake may be taken to be the mean, the median or the modal scenario bin; the latter being relevant for selection of time histories. There is little consensus as to which should be preferred. The situation is complicated further when disaggregations display several modes. In these circumstances the mean or median scenario will not necessarily correspond to the modal bin,

and it may be necessary to undertake dynamic analysis for separate scenarios corresponding to each of the modal bins identified by disaggregation.

Where the current EN 1998 provisions for the identification of the controlling earthquake scenario are most limiting is in the implicit consideration of a scenario earthquake based on PGA disaggregation. The simple categorisation of type 1 ($M_s > 5.5$) and type 2 ($M_s \leq 5.5$) spectra does not reflect the influence of larger magnitude earthquakes on ground motions at longer periods, which may be more relevant to the structure under consideration. For most structures it may be more appropriate to select the controlling earthquake scenario using disaggregation of the 1 s spectral acceleration or peak ground velocity (PGV). Ultimately this means that disaggregation should be possible for several ordinates of the uniform hazard spectrum, and/or PGV, and that the design spectrum should correspond more closely to the uniform hazard spectrum than is currently prescribed in EN 1998. The USGS Interactive Disaggregation illustrates how such information may be disseminated in such a manner that it is accessible to engineers. It may be necessary that where this information is supplied, additional code provisions are required to ensure that the controlling earthquake identified is that most relevant to the design of the structure under consideration.

Epistemic Uncertainty

Many seismic hazard analyses that form the basis for seismic input into design codes now consider epistemic uncertainty, typically in the form of a logic tree. Although seismic hazard may be determined for higher or lower fractiles, it is usually the mean or median hazard that actually forms the basis for seismic input into design codes. There are several issues to consider here. Firstly, the scenario earthquake (M - R - ϵ) will depend on the input model used. Scenario earthquakes defined for the overall mean or median hazard do not correspond exactly to those identified by each ground motion prediction model. It is possible, if not likely, that at higher fractiles of motion a different ground motion or seismogenic source model may contribute most significantly to the hazard than for the median, resulting in the identification of a different scenario earthquake

An alternative view may be to consider whether conservatism in design can be reconciled with the epistemic uncertainty calculated from the hazard analysis. For current design code provisions it is assumed that the seismic input is that which corresponds to the fixed return period for the given limit state. However, for structures that are considered more important or critical for post-earthquake operation a greater degree of conservatism is warranted. This is commonly via the importance coefficient (γ), which scales the elastic design spectrum to a higher level. In most codes γ is assigned a fixed value according to the structure importance, typically equal to unity for “ordinary buildings”, reaching as high as 1.5 or 2 for critical facilities. This factor effectively scales the hazard spectrum to levels consistent with that of a longer return period. However, in assigning fixed values of the importance coefficient irrespective of the hazard at the site, the actual margin of safety afforded to structure is spatially variable. A more consistent margin of safety may be achieved if the importance coefficient is correlated to the epistemic uncertainty at the site. One crude method of implementing this may be to require that higher importance classes should use the ground motion at a higher fractile, for the same probability of exceedence, as a basis for design. This may ensure a more uniform margin of safety for structures, and is based on the calculated uncertainty of the hazard rather than a fixed value.

Acceleration Time Histories

The use of dynamic analysis in seismic design has become widely adopted within many codes, especially those developed within the last decade. Yet provisions for the selection and scaling of acceleration records for use in dynamic analysis do not necessarily reflect the developments in understanding that have occurred in this area over the same period. The provisions for selection of time histories in EN 1998 are broadly consistent with those in other codes. This includes compatibility with the seismogenetic features of the sources and the conditions of the site; scaling of the acceleration history to the level of the UHS at the fundamental period of the structure, and provisions for the use of real, synthetic and artificial time histories. As with many codes, explicit requirements indicating how the histories should be scaled, how many histories are needed, the direction of application and the combination of components, do not reflect the current state of knowledge. In particular, the UHS is not considered an appropriate target spectrum for the selection and scaling of accelerograms. However, several of the preferred means of developing target spectra for selection of time histories require a more detailed specification of the earthquake scenarios contributing most significantly to hazard at a site than is currently mandated in many design codes worldwide. Whilst research and development of methods for selection and scaling of time histories is not an envisaged part of SHARE, the output of the project is relevant to this area and further discussion is therefore warranted in due course.

Summary

This report presents an overview of some of the areas in which seismic input defined in EN 1998 may be able to converge toward current best practice in seismic hazard. The SHARE project, and dissemination of the hazard results via the Internet, provides an opportunity to reassess some elements of seismic action in Eurocode and to consider new approaches to defining the seismic input. The definition of the UHS for given return periods and identification of the controlling earthquake scenarios for hazard at certain spectral periods may be viewed as achievable outcomes from SHARE. With the addition of more information, it may be necessary to define further provisions within Eurocode to ensure that this information is used appropriately. There are also areas, particularly relating to performance-based design and displacement-based design where the Eurocode approach may reflect more closely the state of the art in earthquake resistant design than is currently found in other codes. The topics that are the primary focus of discussion within this report are those for which the SHARE output may be most relevant in making recommendations for future code revisions. A balance may need to be struck between ensuring that the provisions reflect the state-of-the-art in seismic hazard practice, and the practicality of implementation.

The structure of EN 1998 with regard to the role of NDPs represents a novel challenge in defining seismic action. Performance-based seismic design provisions, and the autonomy afforded to National Authorities over the selection of design return periods, may limit the extent to which harmonisation of the seismic risk can be achieved, even if the harmonisation of seismic hazard is accomplished. The use of risk-targeted and/or cost-benefit guided seismic design may come to play a key role in reconciling the objective of a harmonised level of safety, with the local and regional variation in both seismic hazard and inherent earthquake resistance of existing structural design practice. The outcomes of SHARE will help determine if such approaches can be implemented, and how provisions for their use can be integrated into seismic design codes.

Table of Contents

List of Figures	ix
List of Tables	xi
1 Introduction	1
1.1 SHARE Objectives in the Context of Eurocode.....	1
1.2 About the Eurocodes.....	1
1.3 The Role of National Annexes.....	2
1.4 Other Design Codes Considered Within this Analysis.....	4
2 Performance-Based Seismic Design (PBSD)	6
2.1 Context and Theory.....	6
2.2 Design Requirements and Retrofit Assessment in Eurocode.....	7
2.3 PBSD and Return Period in the NEHRP and International Building Codes.....	12
2.4 Return Periods and Limit States in Other Codes.....	17
2.5 Issues in Defining Return Periods for PBSD.....	17
2.6 Summary of PBSD Requirements.....	23
3 Classification of Site Condition	24
3.1 Influence of Site Condition on Ground Motion and Classification Methods.....	24
3.2 Site Classification in Eurocode 8.....	26
3.3 Site Classification in Other Seismic Codes.....	27
3.4 Influences on Site Response.....	30
3.5 Comments on Site Classification in EN 1998-1.....	31
3.6 Defining Site Amplification in the Seismic Action.....	32
4 Characterising Seismic Action: Elastic Response Spectrum	42
4.1 Elastic Response Spectrum.....	42
4.2 EN 1998 “Elastic Response Spectra”.....	43
4.3 Elastic Response [Design] Spectra in Other Building Codes.....	48
4.4 Construction of the Elastic Response Spectrum.....	58
5 Inputs and Considerations for Seismic Design	71
5.1 Seismic Zoning.....	71
5.2 Disaggregation.....	74
5.3 Uniform Hazard Spectra.....	82
5.4 Damping.....	85
5.5 Epistemic Uncertainty.....	92

6	The Use of Time Histories	99
6.1	Background.....	99
6.2	Provisions in EN 1998.....	99
6.3	Time History Provisions in Other Codes.....	102
6.4	Selection of Time Histories.....	107
6.5	Scaling and Rotation of Time Histories.....	110
7	Discussion & Conclusions	112
	Acknowledgements	120
	References	121
	Appendix A: Nationally Determined Parameters in EN 1998	130
	Appendix B: Site Classifications in Other Listed Codes	132
	Appendix C: Construction of Design Spectra in Listed Codes	141

LIST OF FIGURES

2.1	Performance-Based Seismic Design Objectives (SEAOC, 1995; Bommer <i>et al.</i> , 2005).....	6
2.2	Linear approximations to the hazard curve for a site in California and L'Aquila (Italy).....	10
2.3	Variation in k -value across Italy	11
2.4	Example application of the collapse probability calculation for the SFBA and MMA sites.....	14
2.5	Example of rescaled ground motions for achieve same collapse probability in 50 years for the SFBA and MMA sites.....	15
3.1	Example of geotechnical profile from stations within K-Net database.....	25
3.2	Comparison of the site classification schemes listed in section 3.3, in terms of V_{s30} and N_{SPT}	29
3.3	Elastic acceleration response spectra and displacement response spectra, compared for the Boore & Atkinson (2008) and Akkar & Bommer (2010) ground motion prediction models for EN 1998 site conditions A, B and C...	36
3.4	As Figure 3.3, with the addition of the Cauzzi & Faccioli (2008) model and EN 1998 site class D.....	37
3.5	Ratio of spectral acceleration on soil sites (EN 1998 classes B, C and D) to spectral acceleration on rock (EN 1998 class A) sites.....	38
3.6	Comparison of the acceleration and displacement spectra with design spectra created according to EN 1998 for each site class.....	39
3.7	Comparison of spectral acceleration and displacement amplification factors for EN 1998 and the AB2010 (a, b), BA2008 (c, d) and CF2008 (e, f) GMPEs.....	40
3.8	Comparison of EN 1998 and nonlinear scaling of the EN 1998 response spectrum for a) Type 1 and b) Type 2 spectrum.....	41
4.1	Elastic response spectra from the 1992 Erzincan, Turkey, earthquake: acceleration spectra and displacement spectra, for different levels of damping (ξ).....	42
4.2	Idealised acceleration and displacement elastic response [design] spectrum.....	43
4.3	Type 1 and Type 2 elastic response spectra corresponding to the values in Table 4.1.....	45
4.4	EN 1998-1 vertical elastic response spectra.....	46
4.5	Extended EN 1998-1 displacement spectra.....	47
4.6	Design spectra for 39°N, 122°W using FEMA 450	50
4.7	Ratio of vertical to horizontal design spectra for different site classes using the EN 1998-1 formulation (Type 1, M_s 6.0) and NEHRP (2009) formulation.....	52
4.8	Design spectrum for an low-hazard (Winnipeg, Manitoba) and high-hazard (Vancouver, BC) site from NBC 2005.....	53
4.9	Design spectra for Numerical Integration Time History analysis according to NZS1170.5 for a) a low-hazard site (Christchurch) and b) a high-hazard site (Kaikoura).....	54
4.10	Example design acceleration and displacement spectra for Perugia, Italy for different site classes, according to NNTC-Italy (2008)	55

4.11	Example design spectra according to the Pakistan (2007) code.....	56
4.12	Comparison of code-defined elastic design spectra and uniform hazard spectra (475 year return period – shown in black), for four cities in Italy, a) L’Aquila, b) Messina, c) Naples and d) Turin.....	57
4.13	Formulation of the ERS for spectral acceleration and displacement.....	60
4.14	Pitilakis <i>et al.</i> (2004) design response spectra.....	61
4.15	Scaling of PGV/PGA with magnitude and Joyner-Boore distance using the ground motion prediction model of Akkar & Bommer (2010).....	62
4.16	Correlation of T_C with PGV/PGA.....	63
4.17	Ratios of PGV to Spectral Acceleration using the Akkar & Bommer (2010) GMPE, assuming a strike-slip earthquake at a Joyner-Boore distance of 20 km. a) variation with period for fixed magnitudes, b) variation with magnitude for fixed periods.....	63
4.18	Influence of decay parameter (α) on the EN 1998-1 elastic response spectrum.....	64
4.19	Ratio of largest S_a to PGA from Italian UHS. a) Peak S_a /PGA, b) 2 nd largest S_a /PGA and c) 3 rd largest S_a /PGA.....	67
4.20	Ratio of peak S_a (T) to PGA according to a) Akkar & Bommer (2007a,b) and b) Boore & Atkinson (2008).....	68
4.21	Amplification factor F_0 for California	68
5.1	Seismic zones defined in the a) Japan (2000) and b) Indonesia (2002) codes	72
5.2	Percentage increase in ground acceleration from 475 year to 2475 year return period for the United States.....	74
5.3	PGA disaggregation for Fresno, California, for an NEHRP Class B site, and 2475 year return period.....	76
5.4	PGA disaggregation for Seattle, Washington, on a NEHRP Class B site, with a return period of 2475 years.	77
5.5	PGA disaggregation for the 475 year return period for L’Aquila, Italy.....	78
5.6	Comparison for the PGA and S_a (2.0 s) disaggregations for Fresno, California.....	79
5.7	Maps of modal M, R and ϵ across southern Italy for PGA and S_a (1.0).....	80
5.8	Relative contribution of the second mode to the PGA and S_a (1.0 s) disaggregation for M, R and joint M-R.....	81
5.9	Comparison of the 2475-year UHS for western U.S. site calculated using BA2008, the spectrum for the scenario earthquake determined for the 1 s spectral acceleration and the conditional mean spectrum using BA2008.....	85
5.10	Ratio of 84 th to 50 th percentile ground motions for Italy.....	96
5.11	Epistemic uncertainty representations in hazard curves.....	97
C.1	Design spectra for a) Numerical Time History Analysis, b) General analysis, according to NZS 1170.5	143

LIST OF TABLES

2.1	Common Return Period and Probability Definitions.....	7
2.2	k -values for different spectral ordinates for the L'Aquila site.....	11
2.3	Summary of damage and limit state definitions for ordinary building codes considered in this report.....	17
3.1	Site classification scheme in EN 1998-1 3.1.2	26
3.2	Site classification in FEMA 450	28
3.3	Topographic site classification in EN 1998-5	30
3.4	Adapted Eurocode Site Classification Scheme (Pitilakis <i>et al.</i> , 2004).....	33
4.1	Site and corner periods for the EN 1998-1 ERS	44
4.2	Corner periods for the vertical elastic response spectrum (EN 1998-1).....	46
4.3	Long-period displacement corner periods from EN 1998-1 Annex A	47
4.4	Site coefficients for the FEMA 450 design spectrum.....	49
4.5	Site coefficients for the MCE Peak Ground Acceleration (NEHRP, 2009)..	51
4.6	C_V for short-period motion (NEHRP, 2009).....	52
4.7	Amplification and corner period parameters for alternative site classification scheme (Pitilakis <i>et al.</i> , 2004).....	61
4.8	Definition of T_L in FEMA 450	65
5.1	Available interactive web tools for disseminating national seismic hazard results.....	82
A.1	Nationally Determined Parameters in EN 1998	130
B.1	General site classification scheme in NZS1170.5 and soil type and description (for use in classes C or D).....	132
B.2	Site classification for buildings in BSL-2000.....	134
B.3	Site classification for railway structures from Japan (2000).....	135
B.4	Soil classification for Port and Harbour Facilities (Japan, 2000).....	135
B.5	Site classification for water supply facilities (Japan, 2000).....	136
B.6	Site classification for gas lines (Japan, 2000).....	136
B.7	Site classification in the Indonesian (2002) Seismic Code.....	137
B.8	Site classification in the 2007 Pakistan Code.....	137
B.9	Site classification in previous and current seismic design codes from Europe and the surrounding area.....	138
C.1	Site coefficients for NBC 2005	141
C.2	Definition of the design spectrum taken from NZS 1170.5 Commentary A	142
C.3	Definition of $N_{max}(T)$ according to NZS 1170.5	143
C.4	S_S and C_C site amplification factors from NNTC-Italy (2008)	144
C.5	Topographic amplification factors from NNTC-Italy (2008)	145
C.6	Corner periods for the vertical ERS according to NNTC-Italy (2008)	146
C.7	Long-period displacement corner periods from NNTC-Italy (2008)	147
C.8	Lateral force coefficient for highways defined by the Japan (2000) code...	148
C.9	Response spectra for dynamic analysis from Japan (2000).....	149
C.10	Acceleration for each zone according to the Indonesian (2002) code.....	150
C.11	Site coefficient according to the Indonesian (2002) code.....	150
C.12	Seismic coefficients according to the Pakistan (2007) code.....	151
C.13	Near Fault Factors according to the Pakistan (2007) code.....	151
C.14	Fault categorisation according to the Pakistan (2007) code.....	152

1. Introduction

1.1. SHARE Objectives in the context of Eurocode

A primary objective of the “Seismic Hazard Harmonisation in Europe (SHARE)” Project is to provide a European seismic hazard analysis that is compatible with the engineering requirements of Eurocode (CEN, 2004), and to ensure that the pan-European seismic hazard results can be applied to improve and harmonise the Nationally Determined Parameters in the national code applications (SHARE, 2009). A working consensus on the required output of this analysis, in terms of the demands for effective mitigation and risk modelling, is therefore required. Eurocode will become the European building standard in 2010, with further revisions expected to follow in future. It is therefore imperative to assess the demands of the code in the context of seismic action, and to compare this with similar approaches taken in building codes around the globe.

There are several key objectives to this report. The first is to briefly review the current approach to seismic hazard practice implemented in Eurocode, and to assess, critically, the standard in the context of the current understanding of best practice. This will consider many issues including objectives for performance-based seismic design, characterisation of site conditions, definition of the input for seismic motion and the use of acceleration time histories for dynamic analysis. The current provisions given for the definition of seismic action in Eurocode are outlined and illustrated with examples.

The second objective is to assess the state-of-the-art of seismic input required for current building codes in other regions across the globe. This too will be done in the context of current “best practice” in seismic hazard assessment, although some consideration will be given toward the provisions for seismic design within these codes and Eurocode 8. Seismic design codes selected for detailed analysis are mostly current (or very recent) standards for countries of high or moderate-high seismicity. As this objective focuses on current state-of-the-art provisions, detailed critical analysis is mostly directed towards seismic design codes outside of the European Union and European Free Trade Association. Some discussion of older or existing seismic design codes from within this region is also provided where relevant.

The final objective of this report is to provide preliminary recommendations for changes to the definition of seismic action in future revisions of Eurocode. These preliminary recommendations should, at the present stage, be recognised as suggested modifications given the current state of practice in seismic design codes worldwide. They may not necessarily reflect the final recommendations that are the intended output of the SHARE project. These will be made in future deliverables and may be revised according to the output from other areas of the SHARE project.

1.2. About the Eurocodes

The EN Eurocodes are now the current technical standards for structural design in Europe. Developed to contribute to the establishment and functioning of the internal market for construction products and engineering services, they apply to buildings and civil engineering works including: geotechnical aspects, structural fire design and situations including earthquakes, execution and temporary structures (EEC, 1993 – Construction Products

Directive). Eurocode is divided into 10 separate sections, indicated as EN 199#. These sections are:

- EN 1990: Basis of Structural Design
- EN 1991: Actions on Structures
- EN 1992: Design of Concrete Structures
- EN 1993: Design of Steel Structures
- EN 1994: Design of Composite Steel and Concrete Structures
- EN 1995: Design of Timber Structures
- EN 1996: Design of Masonry Structures
- EN 1997: Geotechnical Design
- EN 1998: Design of Structures for Earthquake Resistance
- EN 1999: Design of Aluminium Structures

The focus of this report is on **EN 1998** (often referred to as EC8), which is divided into six parts:

- EN 1998-1: General rules, seismic actions and rules for buildings
- EN 1998-2: Bridges
- EN 1998-3: Assessment and retrofitting of buildings
- EN 1998-4: Silos, tanks and pipelines
- EN 1998-5: Foundations, retaining structures and geotechnical aspects
- EN 1998-6: Towers, masts and chimneys

Throughout the duration of this report references given in bold correspond to the particular code, part, section or clause, whilst text given in italic indicates a verbatim quote from the referenced code. **EN 1998** refers to the standard approved on 23 April 2004, which supersedes previous Eurocode provisions for design against earthquake actions. In accordance with the full Eurocode, this standard was given the status of National Standard for all member states¹ in June 2005, and conflicting national standards are expected to be withdrawn by March 2010. Seismic input for design is mostly defined in **EN 1998-1**, although certain requirements for seismic actions may also be found in the other parts. These are referenced where appropriate.

Finally, it should be noted that special structures such as Nuclear Power Plants, offshore structures and large dams are beyond the scope of EN 1998. Many of these structures require site specific analysis and characterisation of seismic input that is beyond the scope of the provisions given in **EN 1998**. Seismic resistance of such structures is established via provisions specific to the structure under consideration, published by the relevant governing body (either national or international).

1.3. The Role of National Annexes

There are very few seismic design codes that have been created with the intention of superseding existing codes for many different countries, to be implemented at a single given time. It shall be seen that previous seismic design provisions adopted for the United States have developed into the Uniform Building Code (UBC, 1997) or the International Building Code (IBC, 2009). The notionally international codes have formed the basis for many national codes for countries across the globe, particularly in Latin America but also in places such as

¹ CEN Member States are: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom

Pakistan and Indonesia. Nevertheless, whereas the application of such codes may be international in scope, their adoption by other countries is entirely voluntary and adaptation of provisions readily undertaken by seismologists and engineers with local knowledge.

As adoption of Eurocode and retraction of existing national standards is mandatory for participating members, it has been necessary to make provision for many elements of the design standard, subject to alteration by each participating national body. Each of the 28 countries covered within the Eurocode scheme are required to produce a national standard. These should include *“the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex (informative)”* (CEN, 2004). The aim of the National Annex is to indicate how Eurocode standards are implemented in the context of existing national standards, and to provide values for the Nationally Determined Parameters (NDPs). These correspond to parameters indicated in the Eurocode as being subject to alteration by national standards authorities. The devolution to national bodies of many aspects of Eurocode is intended to:

- 1) take into account differences in geographical, geological or climatic conditions;
- 2) result from different design cultures and procedures for structural analysis;
- 3) arise from the requirement for safety levels in the relevant Member States.

A total of 1507 NDPs are given in the full Eurocode, of which 142 are relevant to Eurocode 8. The NDPs relevant to the input for seismic design are discussed in this report. For almost all NDPs, recommended values are provided within Eurocode itself. It may be an intention, both implicit within the code and explicit in guidelines for implementation² that NDPs should not diverge significantly from the recommended values in Eurocode.

The National Annexes and NDPs should ensure that transition from existing National Standards can be undertaken without requiring a substantial change in design practice. They should also allow for national standards bodies to maintain a degree of autonomy over national building practices, whilst ensuring that new designs are compatible with a baseline standard.

The use of NDPs and the devolution of many decisions to national authorities present a challenge in the definition of the seismic input that is unique to Eurocode. For other codes the seismic input, including return period and means for constructing the response spectrum, are fixed at given values, corresponding the definition of the limit states in the example of return period. Using either direct hazard results or seismic zones it is possible to supply seismic input that is consistent across the entire region (e.g., a map of seismic hazard with a given probability of being exceeded in an assumed time period). The input is then modified to the needs of the structure under consideration via parameters describing the importance or occupancy of said structure. Nationally Determined Parameters, of which return period is an important one, mean that although Europe is covered by the same legal standard, each country can opt to define seismic input in a different manner. Consequently a seismic hazard map of Europe for a given return period has limited practical application for Eurocode unless either: a) all national standards bodies assume the same return period, or b) a simple numerical formulation is introduced to allow designers to adjust from the hazard at the mapped return

² *“It is strongly recommended that Countries adopt the recommended values. However, when justified, a Country, may give a different value or may allow a range to be used for a project, from which the designer, owner or other relevant person can make a choice”* (EN Annex P 3.5(2)).

period to that at the return period of interest, without *a priori* knowledge of the hazard curve for a given location. Such a means is presented in Eurocode and will be discussed in the next section.

A crucial output of SHARE is to assist in the development and harmonisation of Nationally Determined Parameters. The creation of a Europe-wide seismic hazard assessment is a fundamental step towards achieving this goal. Currently, national seismic hazard assessments are created using different modelling assumptions and adopting different measures of the strength of ground motion (Solomos *et al.*, 2008). In producing a harmonized hazard map with the aim of standardising the parameters input into the seismic analysis (albeit allowing for variation in return period or response spectrum), design practice can be effectively compared across the European Union and European Free Trade Area.

1.4. Other Design Codes considered within this analysis

In addition to a critical analysis of seismic design provisions in Eurocode, it is the intention of this report to analyse the provisions for seismic action in seismic design codes from several other countries around the globe. The codes selected for this purpose apply to countries of generally high seismic hazard, or where regions within a country may be considered as being of high or moderate-high hazard. They may display some diversity in the approaches to effective seismic design. At the time of writing these codes are considered to be the current standard in their respective countries; hence detailed analysis is generally limited to codes published within the last 10 years. One notable exception to this is the 1997 Uniform Building Code. This particular code for the United States has been largely superseded by the 2003 National Earthquake Hazard Reduction Program (NEHRP) *Provisions* (2003; 2009) and the International Building Code (2009). However, the Uniform Building Code forms the basis for many other national seismic design codes in countries not considered in this report. As it retains importance in establishing seismic design standards worldwide, it is included in this analysis.

As critical analysis is directed at the provisions for seismic actions for ordinary buildings and structures, the codes considered are those that are generally applicable to ordinary or non-critical structures (except where specified otherwise). Separate provisions exist for the design of critical structures such as Nuclear Power Generation and Storage Facilities (e.g., USNRC 2007; IAEA, 2003), and for specific facilities such as ports (Port of Los Angeles, 2004). Where relevant to the discussion of seismic actions, such provisions shall be cited, but are not the focus of the analysis here.

The building codes considered, in detail, in this report are:

- 1) Eurocode 8 (CEN, 2004): Hereafter referred to as **EN 1998**. The definition of seismic action is mostly addressed in Part 1 of the report [**EN 1998-1**], although further provisions may be found in other parts.
- 2) NEHRP *Provisions* (NEHRP, 2003; 2009): Hereafter referred to as **FEMA 450**, this includes both the provisions and appropriate commentary (**FEMA 450c**). A revision to these *Provisions* has been published in 2009 (**FEMA 750p**), which contains some changes relevant to discussion within this report.
- 3) International Building Code (IBC, 2009): Hereafter referred to as **IBC 2009**. Structural design provisions, including those pertaining to seismic action, are given in section 1613 of the code.

- 4) Uniform Building Code (UBC, 1997): Hereafter referred to as **UBC 1997**. Earthquake design is addressed in Chapter 16 – Divisions 4 and 5.
- 5) National Building Code of Canada (NBCC, 2005): Hereafter referred to as **NBC 2005**. Earthquake design provisions are given in Division B – Part 4 of the code, with design parameters provided in Appendix C. Relevant insight into issues of seismic design is also provided in the Structural Commentaries to Division B.
- 6) Standards New Zealand (2004): Hereafter referred to as **NZS 1170.5**. Earthquake design is addressed in Part 5: Earthquake Actions, with accompanying commentary **NZS 1170.5 Supplement 1 (NZS 1150.5 S1)** hereafter).
- 7) Nuove norme tecniche per le costruzioni, Italy [*New technical standards for constructions*] – Ministero delle Infrastrutture e dei Trasporti, Italy (2008): Hereafter referred to as **NNTC-2008**. This code outlines provisions for seismic actions in chapter 3, and seismic design in chapter 6. Seismic parameters and guidance on their calculation are provided in appendices to this code.
- 8) Building Standard Law of Japan (2000): Hereafter referred to as **BSL-Japan**. Additional material pertaining to this code is derived from the report Earthquake Resistant Design Codes in Japan (Japan Society of Civil Engineers, 2000).
- 9) Standar Perencanaan Ketahanan Gempa Untuk Struktur Bangunan Gedung [*Earthquake Standard and Security Planning for Buildings and other Structures*] - Indonesian Seismic Code (2002): This is the current standard seismic design code for Indonesia, hereafter referred to as **SNI-1726**, although a major revision to this code is expected in 2010.
- 10) Building Code of Pakistan (Seismic Provisions) (2007): Hereafter referred to as **BC2007-Pakistan**.

Seismic codes for other countries are referenced where relevant to the discussion. The material for such codes is taken from the IAEE World list of Regulations for Seismic Design (IAEE, 1996 – Supplement, 2000; Web Archive, 2008). It should be emphasised, however, that in the cases of most existing European seismic design codes it is expected that they will be rescinded in favour of Eurocode by 2010. The full compendium of National Annexes has not yet been seen by the authors, and is therefore not addressed here.

2. Performance-Based Seismic Design (PBSD)

2.1. Context and Theory

Mitigation against seismic action leading to complete structural collapse has been a priority of many structural building codes for several decades. The experience of the 1994 Northridge earthquake and 1995 Kobe earthquake demonstrated that economic losses could still be considerable even if the no-collapse objective of earthquake engineering had been largely achieved for many structures (Bommer & Pinho, 2006). Recognising that substantial earthquake losses can result from such damage, much attention has been paid to the development and implementation of performance-based seismic sign (PBSD). This concept is, in many ways, a formalisation of the approach often taken by engineers when considering the lateral load to which a structure may be subjected within its lifetime.

Limit states have been used as a means of defining the fitness of a structure for its intended purpose (Fardis, 2004). A simple distinction can be in the definition of “ultimate limit state” and “serviceability limit state”. The former pertains to the safety of people or structures, whilst the latter pertains to the normal function and use of the structure, the comfort of the occupants or the damage to property (largely non-structural elements) (Fardis, 2004). Full definitions of these limit states, and any others considered, are usually given within a country’s design code.

Performance-based seismic design is the natural development of the limit state concept into the design provisions that form the objective of design codes. One of the clearest guides to the implementation of performance-based seismic design is given by SEAOC (1995) (Figure 2.1).

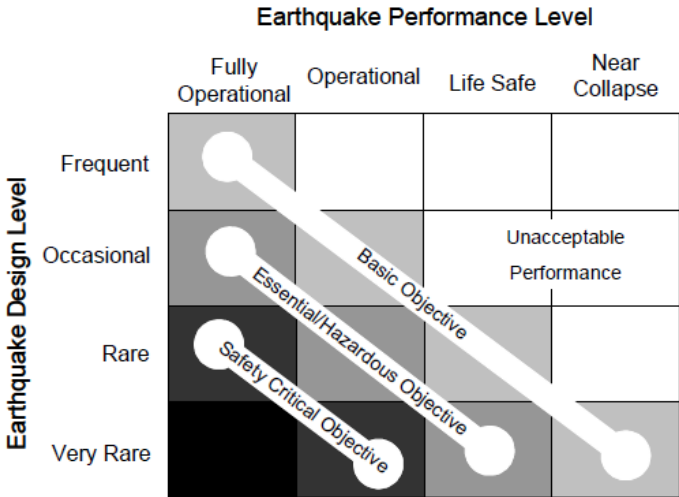


Figure 2.1: Performance-based Seismic Design Objectives (SEAOC, 1995; Bommer et al., 2005)

For most applications of seismic hazard analysis the design ground motion is usually expressed in a probabilistic sense. Most commonly this is the ground motion intensity with a P % probability of being exceeded in T years. For design applications, there has emerged a convention to consider the probability of a ground motion level within a given design life (T_L) of a structure in question. It should be made clear, however, that the design life assumed in

the definition of seismic hazard does not necessarily reflect the intended design life of the structure in question but that of an idealised design life, often arbitrary, for consideration. From the Poisson model, the fractional probability P_R that the ground motion will exceed level a_g one or more times within an assumed time period T_L is determined via:

$$P_R = 1 - e^{-\lambda T_L} = 1 - e^{-(T_L/T_R)} \quad (2.1)$$

Hence for a given probability and design life, the seismic hazard can also be expressed in terms of return period T_R given by:

$$T_R = \frac{-T_L}{\ln(1 - P_R)} \quad (2.2)$$

It is common to see seismic hazard defined in terms of P_R and T_L or in terms of T_R , or both. Some definitions of seismic hazard in common usage in design criteria across the globe are shown in Table 2.1 (Solomos *et al.*, 2008):

Table 2.1: Common Return Period and Probability Definitions

Probability (%) of Being Exceeded P_R	Time Period T_L (years)	Return Period T_R (years)
20	10	≈ 45
10	10	≈ 95
20	50	≈ 225
10	50	≈ 475
5	50	≈ 975
2	50	≈ 2475
10	100	≈ 950
5	100	≈ 1950
2	100	≈ 4950
1	100	≈ 9950

For some critical structures such as Nuclear Power Plants the design ground motion may be more commonly expressed as an annual probability or frequency of being exceeded (i.e., $T_L = 1$). Under International Atomic Energy Association regulations (IAEA, 2003) the typical design for critical elements of a nuclear power station is the ground motion with an annual probability of being exceeded of 10^{-4} . Using equations (1) and (2), this corresponds to approximately a 1 % probability of being exceeded in 100 years (or a 0.5 % probability of being exceeded in 50 years). Most practical applications of seismic design typically require return periods in the range suggested by Table 2.1. Longer return periods may be considered for critical structures, but such analysis would require careful treatment of uncertainties.

2.2. Design Requirements and Retrofit Assessment in Eurocode

Limit state design is well-established in every part of the Eurocode. The number of limit states to be considered may differ for a given structure. These generally fall into two categories: ultimate limit states (ULS) and serviceability limit states (SLS). The ultimate limit

states concerns the safety of people and/or the structure (**EN 1990 3.3**). The serviceability limit states concern the functioning of the structure or structural members, the comfort of people and the appearance of the construction works (**EN 1990 3.4**).

The two requirements of design and construction, as defined within the General Rules of Eurocode 8 are (**EN 1998-1 2.1 (1P)**):

- 1) No-collapse requirement: *“The structure shall be designed and constructed to withstand the design seismic action ... without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of: a) the reference seismic action associated with a reference probability of exceedance, P_{NCR} in 50 years or a reference return period, T_{NCR} , and b) the importance factor γ_I to take into account reliability differentiation”*.
- 2) Damage limitation requirement: *“The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the ‘damage limitation requirement’ has a probability of exceedance, P_{DLR} , in 10 years and a return period T_{DLR} . In the absence of more precise information, the reduction factor applied on the design seismic action ... may be used to obtain the seismic action for the verification of the damage limitation requirement”*.

The definition of three limit states in **EN 1998-3** (Assessment and Retrofitting of Buildings) expands upon the performance requirements specified with the general rules for seismic design (**EN 1998-1**). Within these provisions for the (**EN 1998-3**), three limit states are considered:

- 1) Near Collapse (NC) – *“The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity (**EN 1998-3 2.1 1(P)**)”*.
- 2) Significant Damage (SD) – *“The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although the partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain aftershocks of moderate intensity. The structure is likely to be uneconomic to repair (**EN 1998-3 2.1 1(P)**)”*.
- 3) Damage Limitation (DL) – *“The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measurements (**EN 1998-3 2.1 1(P)**)”*.

Return periods of seismic action are recommended for each of the limit states described (**EN 1998-3 2.1 3(P)**). These are:

- 1) NC: 2475 years, corresponding to a 2 % probability of being exceeded in 50 years.
- 2) SD: 475 years, corresponding to a 5 % probability of being exceeded in 50 years.
- 3) DL: 225 years, corresponding to a 20 % probability of being exceeded in 50 years.

2.2.1. Return Periods applied in Eurocode

Recommended values of P_{NCR} and P_{DLR} (and their corresponding return periods) are given in the notes provided in **EN 1998-1 2.1 (1)P**. These clearly indicate, however, that the values ascribed for a particular country should be found in its National Annex. The recommended values are $P_{NCR} = 10\%$ ($T_{NCR} = 475$ years) and $P_{DLR} = 50\%$ ($T_{DLR} = 95$ years). As noted in **EN 1998-3 2.1 (1)P**, *“the Limit State associated with the “no collapse requirement” ... is roughly equivalent to the one that is here defined as the Limit State of Significant Damage”*. Despite the use of the term “no-collapse” in the design requirements, it is recognised within the assessment and retrofit provisions that the limit state of near collapse corresponds more closely to the actual collapse of the building than that of the no-collapse design requirement, corresponding as it does to the *“fullest exploitation of the deformation capacity of the structural elements”*. Furthermore, the recommended return period for the damage limitation requirement (**EN 1998-1 2.1 (1)P**) does not correspond directly to the Limit State of Damage Limitation defined in **EN 1998-3 2.1 (1)P**.

Conceptually, in considering both the definition of limit states and performance requirements, four levels of hazard are needed in **EN 1998**. These correspond to return periods of 2475, 475, 225 and 95 years. All of these parameters may be subject to change by National Authorities as set out in **EN 1998-1 2.1 (2)P**: *“Target reliabilities for the no-collapse requirement and for the damage limitation requirement are established by the National Authorities for different types of buildings or civil engineering works on the basis of the consequences of failure”*.

The input of seismic action in Eurocode can be modified by the importance factor γ_I . This is a coefficient that can increase or reduce the level of the design seismic action, to account for the consequences of failure of the structure under consideration. A full definition of γ_I and recommended values can be found in **EN 1990** and structure-dependent values given throughout various part of Eurocode. This parameter is also subject to approval by the National Authorities, but usually assumes a value in the range 0.7 to 1.5. The reference value of γ_I is that for an “ordinary building”, which takes the value of $\gamma_I = 1.0$.

The devolution of return period to the National Standards Authorities presents a complication when undertaking a harmonised seismic hazard analysis across many countries. The extent to which the nationally determined return periods vary between countries will affect the degree to which these differences can be accommodated.

2.2.2. Scaling to different return periods

A possible solution, and one suggested in **EN 1998-1 2.1 (4) NOTE**, makes use of a convenient approximation to the hazard curve. The hazard curve indicates the change in strength of ground motion with decreasing annual probability of exceedence. It is observed that seismic hazard at a site $H(a_g)$ varies according to $H(a_{gR}) \approx k_0 (a_{gR})^{-k}$. The rate of change of expected ground motion with respect to annual frequency is defined by the parameter k , hereafter referred to as k -value. This indicates that in logarithmic space, the relation between

$H(a_{gR})$ and a_{gR} can be approximated by a linear relation, with slope k . For many hazard curves this approximation is not necessarily valid for very long ($T \geq 10,000$ years) or very short ($T \leq 30$ years) return periods, depending on the site in question. Two examples are shown in here; the first from an arbitrary site in California (40°N , 120°W) (USGS National Hazard Maps, 2008), and the second from a site near L'Aquila, Italy (INGV Italy Hazard Map, 2009).

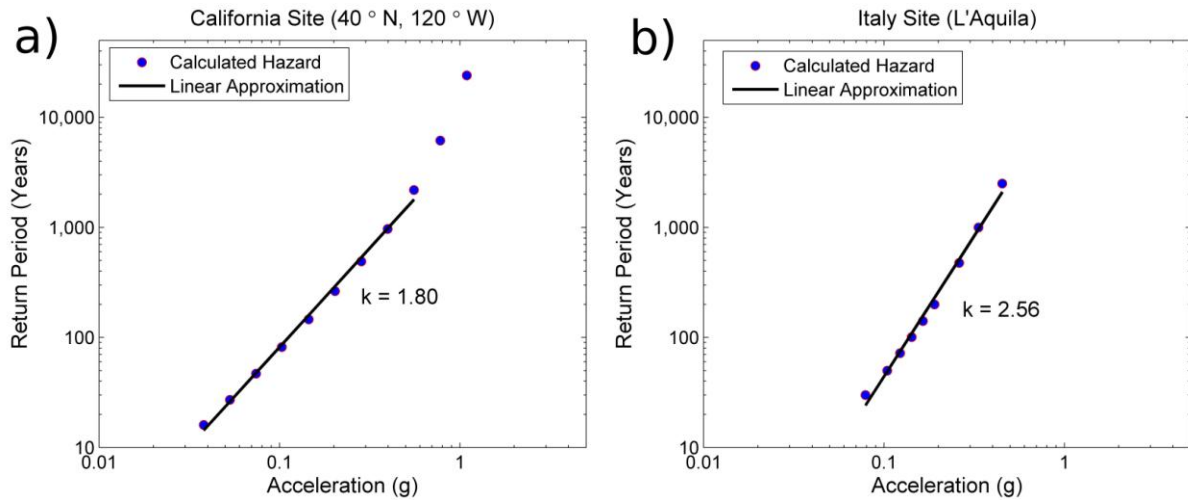


Figure 2.2: Linear approximations to the hazard curve for a) a site in California and b) L'Aquila (Italy)

These figures clearly indicate the range of return periods for which the linear approximation would appear to be reasonable. This range is approximately 50 years to 2,000 years. Since most of the recommended return periods for each limit state fall within this range the approximation via linearization may be appropriate. Extrapolation to return periods outside this range is clearly erroneous. It is reasonable to expect that for most National Annexes the design return periods should assume intermediate values within this range.

EN 1998-1 clearly defines a provision for the application of the linear approximation, as given in **EN 1998-1 2.1 (4) NOTE**: “if the seismic action is defined in terms of a_{gR} , the value of the importance factor γ_I multiplying the reference seismic action to achieve the same probability of exceedance in T_L years as in the T_{LR} years for which the reference seismic action is defined, may be computed as $\gamma_I \sim (T_{LR}/T_L)^{-1/k}$. Alternatively, the value of the importance factor γ_I that needs to multiply the reference seismic action to achieve a value of the probability of exceeding the seismic action P_L in T_L years other than the reference probability of exceedance P_{LR} , over the same T_L years, may be estimated as $\gamma_I \sim (P_L/P_{LR})^{-1/k}$.” Whilst this particular provision does not necessarily consider the return period and acceleration ranges over which this approximation is valid, it does allow for the linear approximation in the estimation of the design seismic action.

The remaining unknown in this approximation is the value of k . **EN 1998-1 2.1 (4)P** indicates that, “the value of the exponent k depend[s] on seismicity, but [is] generally of the order of 3”. The illustrations in Figure 2.2 from two randomly selected sites would suggest that the $k \sim 3$ approximation is not necessarily robust and may depend strongly on the seismic hazard of the site, as demonstrated in Figure 2.3. It is proposed therefore, that for each site the k -value be reported as output of the seismic hazard analysis.

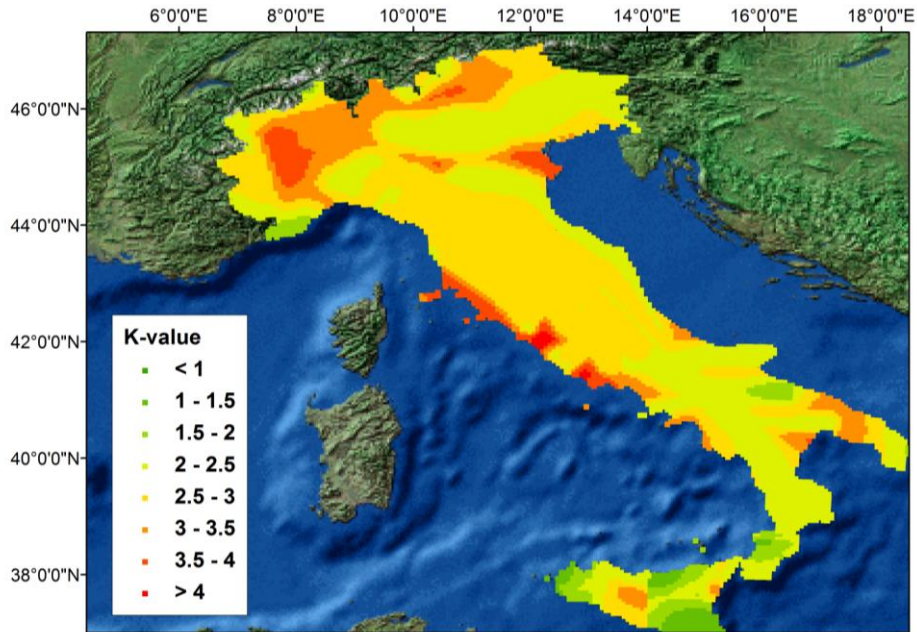


Figure 2.3: Variation in k -value across Italy. Hazard data from INGV National Seismic Hazard Map (2008)

It should be recognised that hazard curves for spectral ordinates will display different k -values for each spectral ordinate considered. For the L'Aquila example of Figure 2.2b, the k -values for each spectral ordinate, determined from the return period range of 10 – 2000 years, are given Table 2.2. This clearly indicates the variation in k with periods, and respective the scatter determined from the linear regression σ_k , which generally displays a decreasing trend at longer periods.

Table 2.2: k -values for different spectral ordinates for the L'Aquila site. Data from INGV National Seismic Hazard Maps (INGV, 2008)

T	0	0.1	0.15	0.2	0.3	0.4	0.5	0.75	1	1.5	2
k	2.56	2.61	2.65	2.41	2.42	2.13	2.10	1.99	1.93	1.81	1.67
σ_k	0.08	0.07	0.08	0.03	0.08	0.04	0.05	0.05	0.06	0.05	0.07

For compliance with Eurocode 8, where PGA is the only fixed input required for a design seismic action at a site, the single k -value is sufficient. In future codes it is likely that the elastic response spectrum may also be anchored to longer period spectral ordinates, which will require definition of seismic hazard across a range of spectra. Furthermore, the range of return periods over which the linear approximation may be valid also varies with the hazard at a site. It is therefore suggested that one of the following approaches be implemented:

- i) For each grid point at which hazard is determined, the k -value for each hazard curve be given for all spectral ordinates at which the hazard curve is determined. The fixed return period range over which the k -value is determined is 10 to 2500 years.
- ii) An application be included that allows for the user to specify the return period range and the k -value be calculated from the user-selected range.

- iii) For a user-specified return period the hazard is determined by linear interpolation (in logarithmic space) between the nearest two return periods for which hazard has been determined (Grases *et al.*, 1992; Bommer & Pinho, 2006).

2.3. PBSD and Return Period in the NEHRP and International Building Codes

In other building codes the definition of return period for different levels for seismic design also vary widely from country to country. Despite the considerable amount of research into PBSD undertaken in the United States, limit states of the type considered here are not given explicitly within the current building standards (**FEMA 450, IBC2009**), although performance objectives are stated **FEMA 450c**. In these codes, ground motion is given in terms of “Maximum Considered Earthquake” (MCE), with fixed parameters for the 0.2 and 1.0s spectral acceleration mapped accordingly (of which more discussion will follow in due course). Detailed discussion of this term is provided by Leyendecker *et al.* (2000), and supplied in the accompanying commentary (**FEMA 450c**). For most of the conterminous United States the “Maximum Considered Earthquake” corresponds to the ground motion with a 2 % probability of being exceeded in 50 years ($T_{LR} = 2475$ years). There are exceptions to this in regions close to major active faults (e.g., coastal California), where ground motions corresponding to these probabilities of being exceeded can produce structural demands much larger than have typically been recorded in previous earthquakes. In these regions MCE is taken from a “conservative” estimate of ground motion (the median value multiplied by 1.5) from the characteristic event of each fault (**FEMA 450c**).

The MCE described (**FEMA 450, IBC2009**) generally corresponds to the no-collapse level of seismic design. The “design ground motion” is prescribed as being two-thirds of the MCE. This provision is recognised as the margin of safety against collapse of conventionally designed structures (Leyendecker *et al.*, 2000). Hence a structure subjected to a ground motion 1.5 times the design level should have a low likelihood of collapse. It was initially suggested reduction by the 1.5 factor would correspond approximately to the 10 % in 50 years design level. Analysis of this adjustment indicates that this approximation is not necessarily valid across much of the United States, and was only acceptable in coastal California. As such, the design ground motion may correspond to different return periods depending on the regional seismicity within the United States (e.g. Frankel, 2004).

Elements of PBSD are implicit within the codes by virtue of the Seismic Design Category and Seismic Use Group [or Occupancy Category] defined previously. Suggestions can be found within the commentary to the code regarding the performance level (operational, immediate occupancy, life safety and collapse prevention) that a building within each occupancy category may be expected to achieve. Hence, although “the *Provisions* explicitly require design for only a single level of ground motion, it is expected that structures designed and constructed in accordance with these requirements will generally be able to meet a number of performance criteria, when subjected to earthquake ground motions of differing severity”. For Group I buildings (i.e., those with a lesser life risk due to fewer occupants) it is expected that they would achieve the life safety (or better) performance level when subject to the “design ground motion”. Similarly, for Group III structures (i.e., those essential for post-earthquake recovery) the immediate occupancy (or better) performance level is expected, and for Group II structures (i.e., many occupants or restricted exits) the life safety performance level is expected.

2.3.1. Risk Targeted Design in the 2009 NEHRP Provisions

During compilation of this report, the NEHRP *Provisions* have been revised for the 2009 publication. This revision sees a substantial shift in the way in which probabilistic seismic hazard is defined for input into seismic design. As has been discussed previously, **FEMA 450** adopted the use of ground acceleration with a 2 % probability of being exceeded in 50 years as the standard for the hazard input, except in coastal California. The new definition of the probability of the seismic action has changed to a “1% [probability] in 50 year collapse risk”. This change is intended to improve seismic design by achieving a more uniform level of collapse prevention.

The technical basis for this change in definition is given by Luco *et al.* (2007). It is motivated by the recognition that the lower bound estimator of the factor of safety inherent within structures, a factor assumed to be 1.5, is itself an uncertain and geographically variable parameter. The revised methodology in the 2009 NEHRP *Provisions* attempts to integrate the uncertainty of the probability of collapse for a given level of ground motion into the seismic design input. This results in greater uniformity in the risk of collapse.

In **FEMA 450**, the design ground motion is assumed to be two thirds of the maximum considered earthquake (MCE) ground motion. This 2/3 factor represents a lower bound estimate of the factor of safety against collapse for structures. This factor of safety, referred to from here on as collapse capacity, represents a deterministic judgement. In reality, the collapse capacity of a structure for a given level of seismic motion is uncertain. This uncertainty arises from the many different sources including characteristics of the strong-motion waveform and variability in the quality of construction detail.

The primary aim of risk-targeted design is to define the input ground motion that produces a more uniform probability of collapse within an assumed time period. This is in contrast to the previous input definition of ground motion with a probability of being exceeded within the time period. To make this transition, the hazard integral needs to incorporate the probability distribution for the collapse capacity of a structure. This probability is modelled via a lognormal distribution, parameterised by the logarithmic standard deviation (β) and a given percentile of the probability distribution. Typically the median (50th percentile) is used to characterise the lognormal distribution. In this application the 10th percentile is used. The use of the 10th percentile is intended to reflect the objective of achieving an “acceptably low probability of collapse”, interpreted as “less than a 10 % probability of collapse under MCE ground motions”. In using the 10th percentile, it is recognised that the probability of collapse under ground motions at the MCE is equal to 10 %. The 10th-percentile collapse capacity is hereafter referred to as $c_{10\%}$. This value is equivalent to the MCE ground motion at the fundamental period of vibration of a structure. The standard deviation (β) has been estimated based on nonlinear dynamic analyses of a selection of structures designed according to **FEMA 450** provisions. The best estimate of β is found to be 0.8 (Luco *et al.*, 2007).

The probability of density of the collapse capacity for a given ground motion (c) is defined via:

$$f_{Capacity}(c) = \phi \left[\frac{\ln c - (\ln c_{10\%} + 1.28\beta)}{\beta} \right] \cdot \frac{1}{c\beta} \quad (2.3)$$

where $\beta = 0.8$, $c_{10\%}$ is the maximum considered earthquake ground motion for the fundamental period of the structure, as determined from the hazard spectra, and $\phi[\cdot]$ the standard normal PDF.

The integral in equation 2.3 defines the probability of collapse, conditional on the level of ground motion. Were no uncertainty to exist in the collapse capacity ($\beta = 0$), the probability of collapse within a given time period (e.g. 50 years) would be equal to the probability of the ground motion exceeding the collapse capacity (within the same time period). In order to take into consideration the uncertainty in the collapse capacity (i.e. $\beta \neq 0$), the probability distribution of the collapse capacity must be coupled with the hazard curve at the location of the structure. The annual probability of collapse is therefore defined via:

$$P[\text{Collapse}] = \int_0^{\infty} P[S_a > c] f_{\text{Capacity}}(c) dc \quad (2.4)$$

where $P[S_a > c]$ indicates the annual probability of ground motion S_a exceeding some given level c . To obtain the probability of collapse with in a given time period Y years, the following equation is used:

$$P[\text{Collapse in } Y \text{ years}] = 1 - (1 - P[\text{Collapse}])^Y \quad (2.5)$$

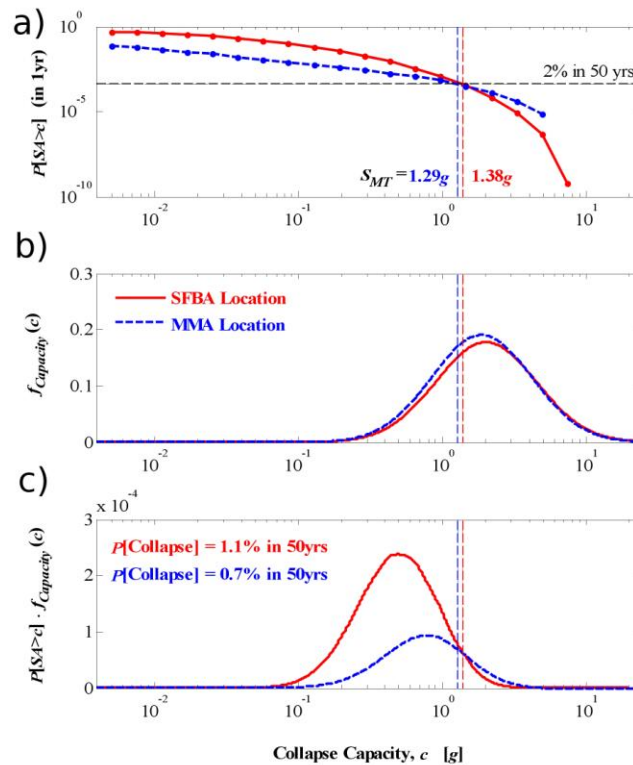


Figure 2.4: Example application of the collapse probability calculation for the SFBA and MMA sites (Luco et al, 2007). a) Hazard curves according to USGS National Seismic Hazard Maps, b) Collapse capacity probability distributions (equation 2.3), and c) the distributions of the probability of collapse (equation 2.4).

To illustrate this approach, the example sites given in Luco *et al.* (2007) are restated here. Two structures with fundamental periods of 0.2 s are considered, the first in the San Francisco Bay area (SFBA) and the second in the Memphis Metropolitan Area (MMA). Using the design ground motions given in **FEMA 450**, the MCE spectral acceleration (at 0.2 s) is 1.38 g for the SFBA site, and 1.29 g for the MMA site. These ground motions correspond to the spectral acceleration with a 2 % probability of being exceeded in 50 years. The hazard curves for the SFBA and MMA sites are shown in Figure 2.4a, in red and blue respectively. The corresponding collapse capacity distributions (equation 2.3) for the two sites are shown in Figure 2.4b. Given the similarity in the MCE spectral acceleration these distributions appear similar, albeit with the SFBA probability distribution shifted slightly towards higher ground motions. The resulting product of the two curves, i.e. the risk integral given in equation 2.4, is shown in Figure 2.4c. The corresponding probability of collapse in 50 years is 1.1 % for the SFBA site, and 0.7 % for the MMA site.

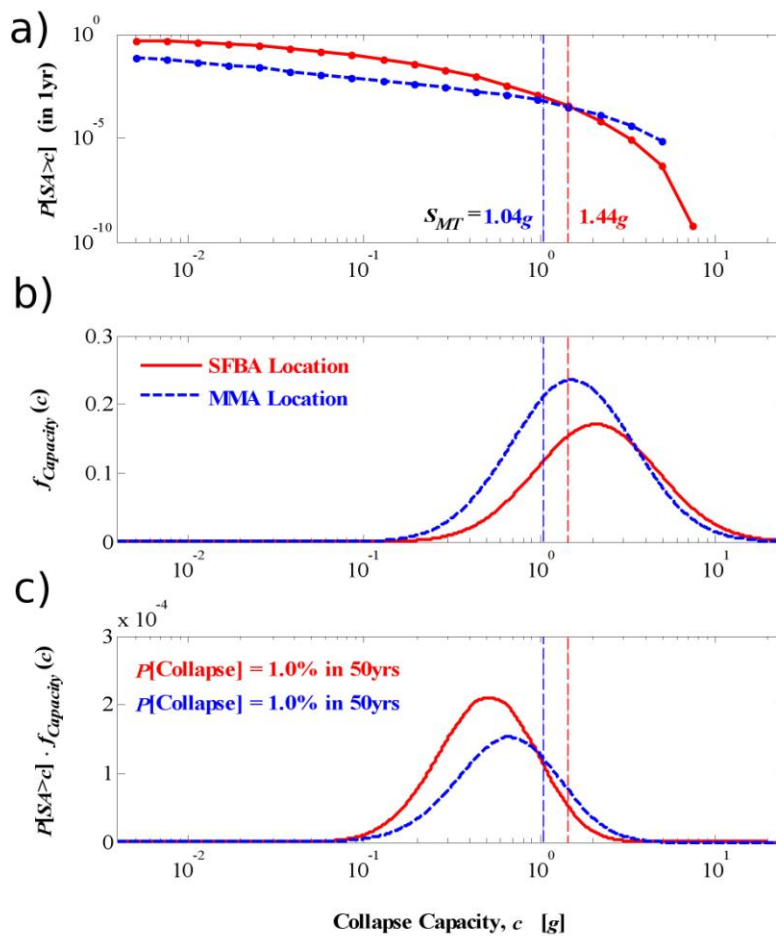


Figure 2.5: Example of rescaled ground motions for achieve same collapse probability in 50 years for the SFBA and MMA sites (Luco *et al.*, 2007). a) Rescaled hazard curves, b) corresponding collapse capacity probability distributions and c) corresponding distributions of the probability of collapse.

Using the same example sites and structures, the ground motions are adjusted by specific factor, so as to ensure the same probability of collapse in 50 years. Figure 2.5a shows adjusted hazard curves, this time with a slight increase in the ground motion for the SFBA site, and a

more significant decrease for the MMA site. To ensure a consistent level of seismic protection, an iterative approach is adopted to determine the degree of scaling of the maximum considered earthquake needed to achieve a 1 % in 50 year collapse probability for each site. The degree of scaling is presented as the “risk coefficient”, for which maps (for the short-period, 0.2 s, and long-period, 1.0 s) are presented in the NEHRP 2009 provisions accordingly. For much of the U.S. (especially in the central and eastern United States) this modification results in a reduction in the intensity of ground motion considered for each site, in some regions as much as 20 to 30 %. It is found that the modification factor for each site produces similar results for structures of different fundamental periods; hence the modification can be applied in the code without explicit consideration of the building type.

2.4 Return Periods and Limit States in Other Codes

The definition of limit states and their corresponding design return periods varies depending on the code. Table 2.3 provides a summary of the limit state definitions for the main codes considered within this report.

2.5 Issues in Defining Return Periods for PBSB

2.5.1. Upper bounds on the return periods considered in seismic codes

Within the design codes for ordinary structures shown in Table 2.3, the longest return period considered is 2475 years. This generally corresponds to the collapse (or near-collapse) limit state. For important buildings and structures the hazard input is modified by the importance coefficient. Effectively this scales the hazard to longer return periods, the extent of which depends on the hazard at the site. The margin of safety against collapse afforded by scaling the hazard according to a fixed coefficient will therefore vary spatially. This issue will be addressed further in section 5. To satisfy the requirements outlined in present codes, seismic hazard should be well-constrained for return periods up to 2475 years. It is anticipated that this may be extended further in seismic hazard analysis, possibly to as much as 5000 years to allow investigation into the margin of safety afforded to critical structures. Analysis of seismic hazard for annual probabilities of exceedance smaller than 10^{-3} to 10^{-4} introduces additional considerations that may be relevant for input to seismic design. For the prescription of return periods needed for seismic design of non-critical structures, to what extent can the seismic source and the site condition be characterised, and the respective uncertainties sufficiently constrained, so as to ensure that the hazard at a site is truly representative of the very small probabilities of exceedance at each location?

Table 2.3: Summary of Damage and Limit State definitions for Ordinary building codes considered in this report.

Code	Limit/Damage States	Description (<i>Not verbatim from codes</i>)	Return Period (Years) [Probability P in T years]
EN 1998	<ul style="list-style-type: none"> • Near Collapse • Significant Damage • Damage Limitation • Serviceability 	<ul style="list-style-type: none"> • Heavy damage, low residual lateral strength and stiffness • Some residual lateral strength, capable of sustaining vertical loads. Moderate permanent drifts. Uneconomic to repair. • Light damage, structural elements prevented from significant yielding. No repair needed. • No damage or limitation of use. 	2745 [2 % in 50 years] 475 [10 % in 50 years] 225 [20 % in 50 years] 95 [10 % in 10 years]
NEHRP (2003) / IBC (2009)	<ul style="list-style-type: none"> • Design Motion 	<ul style="list-style-type: none"> • Definitions not explicit in main code. Three performance levels are assumed: I (No-collapse), III (Immediate occupancy) , II (life safety) 	2475 [2% in 50 years]
NEHRP (2009)	<ul style="list-style-type: none"> • Probability of collapse 	<ul style="list-style-type: none"> • “Risk-targeted” Ground motion 	[1 % in 50 years] ¹
UBC (1997)	<ul style="list-style-type: none"> • Life Safety 		475 [5 % in 50 years]
NBC (2005)	<ul style="list-style-type: none"> • Ultimate • Serviceability² 	<ul style="list-style-type: none"> • Building ceases to fulfil design function – exceeds load-carrying capacity, overturning, sliding and fracture • Restricted occupancy – including deflection, permanent deformation and local structural damage 	2475 [2 % in 50 years]
NZS1170.5 ³	<ul style="list-style-type: none"> • Ultimate <ul style="list-style-type: none"> – Low risk structure – Normal building – Important structure – Critical structure • Serviceability (SLS1) • Serviceability (SLS2) 	<ul style="list-style-type: none"> • Avoid collapse of the structural system, collapse or loss of support to [life hazardous] parts and avoid damage to non-structural systems necessary for emergency building evacuation • Avoid damage to structure, and essential non-structural elements. • (For critical facilities) maintain operation or restore operation within a short period of time 	100 [≈ 10 % in 10 years] 500 [≈ 10 % in 50 years] 1000 [≈ 5 % in 50 years] 2500 [≈ 2 % in 50 years] 25 [≈ 50 % in 10 years] 500 [≈ 10 % in 50 years]

Table 2.3 (continued)

Code	Limit/Damage States	Description (<i>Not verbatim from codes</i>)	Return Period (Years) [Probability P in T years]
NNTC-Italy (2008)	• Operational	• Structural elements and non-structural equipment relevant to its function, should not suffer significant damage and disruption of use	30 [81 % in 50 years]
	• Damage	• Should not suffer damage that would put users at risk or jeopardise significantly the capacity of resistance and stiffness in horizontal or vertical action	50 [63 % in 50 years]
	• Life Safety	• Significant damage to structural and non-structural components, maintains safety margin against collapse under horizontal actions	475 [10 % in 50 years]
	• No-collapse	• Serious structural damage, but margin of safety against collapse	975 [5 % in 50 years]
BSL-Japan (2000)	• Level I	• Building remains within elastic limit – no residual deformation	75 [≈50 % in 50 years]
	• Level II ⁴	• Failure prevention (standard structures) and damage limitation (important structures)	
SNI-1726 (2002)	• No-collapse	• Ordinary structures in low seismicity locations	475 [10 % in 50 years]
	• Serviceability	• Serviceability of structure in high seismicity region (or essential facility)	≈72 [50 % in 50 years]
	• No-collapse	• No-collapse of structure in high seismicity (or essential facility) region	950 [10 % in 100 years]
	• No-collapse	• No-collapse of structure with hazardous material	2375 [10 % in 250 years]
Pakistan (2007)	• Life Safety		475 [10 % in 50 years]

¹ “Risk-targeted” motion refers to motion resulting in a given probability of collapse in T years, not directly comparable with other definitions.

² No explicit return period given for serviceability limit state, it is determined by re-scaling the ground motion for the ultimate limit state by a factor corresponding to occupancy class.

³ In **NZS1170.5**, the design hazard curve ($T_R = 500$ years) is rescaled by a factor R according to the desired return period. The scaling factors suggested in the code suggest an equivalent k -value of approximately 2.3

⁴ No explicit return period given – should be on the order of several hundred years.

This question has often been addressed, mainly in the application of seismic hazard analysis for critical structures such as Nuclear Power Plants (e.g., Abrahamson *et al.*, 2002) or radioactive waste storage (e.g., Stepp *et al.*, 2001). Such structures require extensive site specific analysis, for which a more detailed knowledge of the seismogenic source and the site response is required than would be considered feasible for regional hazard analysis. Design codes for critical structures mandate many more provisions for site investigation than for “ordinary” structures. This allows for the consideration of ground motions with annual probabilities of exceedance on the order of 10^{-4} to 10^{-5} , in some cases as low as 10^{-7} to 10^{-8} . Studies on hazard at return periods such as these often encounter issues that may not necessarily apply to hazard at return periods considered within design codes for “ordinary” structures. These include definition of upper bound ground motions (e.g. Bommer *et al.*, 2004), the maximum earthquake magnitude for the identified sources and the maximum number of standard deviations above the median (or expected) ground motions (Strasser *et al.*, 2008).

2.5.2. A role for time-dependent seismic hazard in design provisions?

Over the last twenty years time-dependent seismic hazard has become a growing area of earthquake research, with application in California (WGCEP, 2003; Petersen *et al.*, 2007), Italy (Akinci *et al.*, 2009), Turkey (Erdik *et al.*, 2004), Japan (Stein *et al.*, 2006) and many other regions. Time-dependent hazard analyses consider the probability of earthquake occurrence on major faults using renewal or time-dependent models. These describe the probability of a fault experiencing 1 or more events of $M \geq M_T$ (where M_T is a specified threshold magnitude) in a given time period, conditional upon the time elapsed since the previous earthquake. The function $f(t)$ defines the chance of failure occurring on a rupture segment within the time $t+\Delta t$, where t is the time since the most recent earthquake. The probability that at least time T will elapse between successive events is given by (WGCEP, 2003):

$$F(T) = \int_T^{\infty} f(t)dt \quad (2.6)$$

The conditional probability of an earthquake occurring along the segment within a given time period (e.g., ΔT years following a given starting date, T) is defined by:

$$P(T \leq t \leq T + \Delta T) = \frac{F(T) - F(T + \Delta T)}{F(T)} \quad (2.7)$$

The conditional probabilities of occurrence can be input into the seismic hazard model, giving the probability of exceeding a ground motion level within the period $T + \Delta T$.

There are many applications for which time-dependent seismic hazard can be useful in the insurance industry. In particular, they are widely applied when considering building contents loss and interruption of business activity. These impacts are particularly relevant when designing for compliance with the operational and serviceability criteria.

Whilst the consideration of time-dependent behaviour can be important in earthquake hazard analysis, there are limitations to its application within seismic design codes. It should be noted that whilst time-dependent earthquake hazard analysis has been an important focus of research in California for more than twenty years, there is still little consideration within

seismic design codes. The problems in time-dependent PSHA emerge in accurately characterising the time-dependent behaviour of active faults. For characterisation of the conditional probability of an earthquake on a rupture or segment, the average recurrence rate, its variability (either by standard deviation of inter-event time, or aperiodicity depending on the probability model used) and the date of the previous rupture are needed. For active shallow faults much of this information can be derived from paleoseismic investigation, but elsewhere this information is limited. For seismic hazard assessment on a European scale the variability in the quality of information available from country-to-country is a major hindrance to the implementation of time-dependent hazard assessment. Application may be limited only to certain regions such as the Apennines (e.g., Akinci *et al.*, 2009), Greece (Papaioannou & Papazachos, 2000) or northwestern Turkey (Erdik *et al.*, 2004).

Whilst there may, at present, be many difficulties in implementing time-dependent PSHA, it may still be important to consider how this information could be incorporated into code design, were it to become available. As has been discussed, it is most relevant when considering the serviceability and/or operational limit states, which may be exceeded several times within the design life of the structure. This could be integrated by means of a two tier system by which seismic input for the life-safety or collapse limit states are based upon input from time-independent analysis, whilst for operational limit states this should take time-dependence into account. This presents many problems, however. The seismic hazard will clearly change over very short time scales, making seismic hazard maps almost redundant by the time they reach publication. Hence a time-dependent map developed in one year, will not necessarily reflect the time-dependent hazard by the time the code is officially adopted. Provisions could be put in place to allow the designer to define the time-dependent hazard given the date upon which construction commences, but this requires a substantial understanding of seismological processes, which is not a practical requirement of an engineer or designer.

There is, perhaps, a greater underlying challenge in considering time-dependent hazard in seismic design, and that lays in the recognition that reassessment and retrofit of structures is simply impractical to implement at the frequency at which revisions to the hazard maps should be made. Arguably the only cost-effective modifications to existing structures would be for upgrade of non-structural elements (e.g., mechanical equipment, plumbing, gas, water and heating-ventilation and air-conditioning [HVAC] fixtures) of critical facilities in accordance with new time-dependent hazard maps issued at intervals of five or ten years for example. Application to ordinary structures or to structural elements of higher importance buildings would place an enormous burden on the property owner that would possibly limit effective implementation of such measures. It should also be recognised that the cost-benefit of implementing time-dependent provisions for serviceability of structures is inextricably linked to the implementation of building and contents insurance covering earthquake losses for a structure. Where such insurance schemes are mandated by law, time-dependent hazard has underpinned the fixing of premiums to cover such losses. This has become standard practice in many parts of the world, e.g. Japan (NLIRO, 2008).

Whilst it may be impractical to actually implement building design and retrofit in accordance with changes in hazard due to time-dependence, there may still be a role for time-dependent PSHA in seismic design. This role comes in identifying regions within which certain structures should become priorities for retrofit within a given time-period. Many codes outline clear provisions as to how structures should be assessed for retrofit (e.g. **EN 1998-3**), but few prescribe the time schedule over which this should be done. It can be reasonably argued that

the identification of priority structures for retrofit should be based on the probability of exceeding a particular limit state for the structure. It is foreseeable that this probability will change over time in accordance with seismic activity, and that the change in probability may be significant with respect to the expected annual probability of exceeding a given limit state. Decisions such as these are not necessarily made by the designers. Nevertheless, for ensuring the functionality of infrastructures (e.g., road networks, electrical power systems, water and gas supply), as would be envisaged by the serviceability requirements applied to bridges, pipelines etc., the inclusion of a clearer set of requirements for assessing retrofit priorities under time-dependent conditions could prove beneficial to regulatory bodies and local authorities. This actually reflects, at least to some extent, the emerging practice in regions where time-dependent PSHA is already well-established.

In addition to the definition of time-dependence that has been discussed so far, earthquakes display other non-Poissonian characteristics that may be of relevance to structural design. In particular, aftershocks and earthquake swarms (multiple events of a similar magnitude) can present a significant risk, especially to structures that have undergone inelastic deformation and have consequently lost lateral strength. These circumstances often lead to structures being subjected to significant horizontal loads several times before any remediation can be undertaken. The assumption of Poissonian earthquake occurrence discounts such events, although the retention of some resistance of the structure to horizontal motion is often envisaged within the definition of the life-safety limit state. The existing frameworks in which time-independent probabilistic seismic hazard assessment are implemented make the incorporation of aftershocks and earthquake swarms a difficult process. It may therefore be more prudent to accommodate such events by incorporating some conservatism into the design, which is already done in most codes via the importance coefficient.

There is clearly a substantial amount of progress that needs to be made before time-dependent seismic hazard can be considered within code design. Nevertheless, issues of time dependence should not be ignored if opting to consider seismic inputs for relatively higher annual probabilities of exceedance. It may be the case that for operational limit states the design input is not defined in terms of a design period, but simply via the application of a scaling coefficient applied to the design input for more damaging limit states. Such an approach is already implicit in some codes (e.g., NBC, 2005; BSL-Japan, 2000). This will, of course, result in a non-uniform margin between operational and damage limit states, which will vary according to hazard. Ultimately a different approach to defining the input for lower damage states may be needed for PBSA to be implemented consistently and effectively.

2.5.3. Selection of Return Periods

The emergence of PBSA as a basis for seismic design in building codes has clearly expanded the requirements placed upon engineers, although in some cases it may be a formalisation of the existing design practice. A change in the focus of anti-seismic design for the purposes of achieving multiple objectives is clearly welcome from the perspective of effective mitigation. It allows for the functionality of a building or system to be taken into consideration in the design process, which will improve the overall performance of building systems following moderate to large earthquakes. The performance objectives initially proposed in SEAOC (1995) have not all been implemented within design codes, the possible exception being the Italian code (NNTC-Italy, 2008). In many cases PBSA is implemented by an approach close to that in EN 1998, which defines a “no-collapse” and a “serviceability” requirement.

Despite the variety of approaches to seismic design and the inputs required from seismic hazard assessment, it can be clearly seen that design levels or limit states have tended to adopt the same values. For the codes considered here the return period for the life safety limit state (or similar definition) of ordinary buildings is usually 475 years, and 2475 years for the no-collapse criterion (10 % probability and 2 % probability of being exceeded in 50 years, respectively). The former (475 years) is also widely adopted as the design objective for many codes in other countries that are yet to implement PBSD style provisions (IAEE, 2008). This design level is first adopted in ATC 3-06 (ATC, 1978), and its origin is based on an arbitrary, albeit considered, judgement (Bommer, 2006; Bommer & Pinho, 2006). The implications of the selection of this return period upon structural performance have been assessed since its original formulation, leading to the adoption of 2,475 years as an appropriate return period for the collapse limit state in FEMA 450 and the International Building Code (2009).

The adaptation of design strength requirements in the context of the probability of structural collapse has, to a certain extent, been integrated into the new NEHRP *Provisions* (2009). In treating the likelihood of collapse as a probability distribution rather than a fixed value (1.5), and then integrating this distribution within the probabilistic hazard formulation, it has become possible to specify the seismic input in terms of a probability of structural collapse, rather than a probability of ground motion exceedence. For much of the United States this has reduced the degree of conservatism in the design code, ensuring that design is appropriate to both the seismic hazard and the inherent structural resistance. It remains to be seen whether this approach could be extended to other limit states for the basis of a fully performance-based design. It is possible to presume that such a method would determine serviceability and damage limit states on the basis of drift ratio (or some alternative demand parameter), which is described by a conditional probability upon the given strong motion level. Such an approach clearly integrates concepts of risk and loss into the design input. It is likely, however, that separate risk coefficients would need to be defined for each limit state, thus expanding the number of required inputs. With web-dissemination, such an approach may still be feasible.

The rationale for transition to the 2,475 year return period for the design earthquake may be based upon the consideration of the likelihood of structural collapse, but the implications of this move from a cost versus benefit perspective have not been evaluated (Bommer *et al.*, 2005). Likewise the return periods assumed for serviceability and damage limit states, whilst representative of considered judgement, are not based on cost-benefit decisions. The corollary to this line of thinking is the question: by whom should cost-benefit decisions be made? Are seismologists and engineers the most appropriate entities to make the decisions regarding acceptable loss?

One possible conceptual framework for calibrating codes according to cost-benefit design is outlined by Bommer *et al.* (2005). This presents an iterative scheme for assessing the required design level on the basis of the expected losses for a given portfolio, calculated using displacement based earthquake loss assessment (DBELA) (Pinho *et al.*, 2002; Glaister & Pinho, 2003; Crowley *et al.*, 2004; Crowley & Bommer, 2006). Such a scheme requires a computationally efficient method of assessing loss that is compatible with the current standards of output from seismic hazard analysis. DBELA is a particularly attractive option as it fulfils these criteria and is readily adapted to different structure types, e.g. reinforced concrete (Crowley *et al.*, 2004; 2008) or masonry (Crowley & Pinho, 2008; Borzi *et al.*, 2008). The use of loss models as a means for constraining design levels allows for explicit estimation of the total cost as the sum of the cost of seismic resistance and the average annual

loss. In expressing design levels as a function of total cost, the determination of the acceptable level of loss is no longer a decision for seismologists and engineers. However, it still ensures that such decisions are made on the basis of sound seismological and engineering models.

2.6 Summary of PBSD requirements

When evaluated in the context of seismic design codes from regions of high seismicity, it is clear that the PBSD provisions made in Eurocode are justified and consistent with standard practice. The extent to which the seismic input will need to be adjusted on the basis of National Annexes remains to be seen, and it is clear that the output of SHARE must be able to accommodate the differences that emerge. The use of k -value offers a convenient tool for computationally simple re-scaling of the hazard output to different return periods, but it is questionable whether this should persist in future. Provision of the seismic hazard curve for a given site, or a means of outputting the hazard parameters for a user-specified return period, is clearly an essential task of SHARE. It is clear, however, that the “recommended” approximation of $k \approx 3$, is not valid across all regions under consideration and is dependent on the hazard at a given site.

Over a longer timescale it is possible that many more seismic design codes may adopt the risk-targeted approach implemented in the 2009 NEHRP *Provisions*. This may present a new challenge for the development of Eurocode, where alterations to the seismic design input may take longer to implement due to the inertia that is inherent within the development of National Annexes. Integration of risk-targeted design into the seismic input may well be in keeping with the philosophical approach that underpins the Eurocode and its national annexes. Providing some quality control is implemented, the collapse capacity function (equation 2.3) may provide a more elegant approach to ensuring that the level of ground motion intensity is determined in such a manner as to take into consideration the seismic resistance that already exists within the structural design practice in each country. This may ensure a more homogenous level of conservatism within the seismic design than exists in the current approach of devolving the selection of return period to national authorities. The difficulty that arises, however, comes with ensuring consistency in the design requirements for damage limitation and serviceability. As a long term objective there is a good case for considering the cost-benefit approach to design outlined by Bommer *et al.* (2005), possibly with some modification subject to assessment in light of the SHARE output. There remains ample scope for national authorities to retain some autonomy in the design of structures appropriate to the given seismic hazard within the country, whilst converging towards a uniform level of safety. Thus the cost-benefit approach may be entirely consistent with the primary objective of Eurocode.

3. Classification of the Site Condition

3.1. Influence of Site Conditions on Ground Motion and Classification Methods

The influence of site geology on strong ground-motion has been known for many years, and its effects observed in many major earthquakes throughout history. Recorded ground motions show enormous variability, even over small distances owing to variations in geotechnical site properties, topography and basin response (Stewart & Choi, 2005). The incorporation of a parameter or parameters describing site condition has become a standard feature of most strong ground-motion prediction equations (GMPEs) developed in the last 15 to 20 years (Douglas, 2003).

As with much of our understanding of earthquake effects, the extent to which their complexity can be captured in models is largely dependent on the quantity and quality of strong motion data available. Strong motion recording networks have grown considerably in size and scale over the last twenty years. Nevertheless, global databases of high quality strong motion data, such as the PEER database (www.peer.berkeley.edu/nga), are strongly biased towards a small number of very well-recorded events. Of the 3552 strong motion records contained in the PEER database, more than 1800 originate from the 1999 Chi-Chi, Taiwan, earthquake and its aftershocks. Although many other seismic records exist for major earthquakes, detailed metadata describing the geotechnical properties of each recording site are not nearly as widely available. Consequently, despite the complex interaction between site condition and strong ground-motion, for many sites the geotechnical conditions are described by simple parametric approximations.

The simplest, and most widely applicable, method of analysing the influence of site condition on strong motion is to use multiplicative factors in the regression procedure for ground motion prediction models. This characterises the site according to a particular class, as determined via a recognised classification scheme. These schemes usually correspond to the site classifications described in seismic design codes, of which many will be compared shortly. The number of seismic records for use within each category can vary substantially. In most cases, those categories corresponding to moderate to well-consolidated soils contain the greatest number of records. The number of recordings taken on very hard rock ($V_s > 1100 \text{ m s}^{-1}$) or very soft soil ($V_s < 180 \text{ m s}^{-1}$) tend to be sparse in comparison, making analysis of the site amplification on such sites far less robust. When developing empirical ground motion prediction models records taken from very hard or very soft soil sites may be omitted, and hence the models are only applicable for a smaller number of well-represented classes.

As strong-motion recording arrays have expanded in use there have been accompanying improvements in the quality of geotechnical information for recording stations. This allows for the more direct representation of geotechnical site characteristics in strong-motion modelling. In most station records the site may be characterised via 30-m average shear wave velocity (see 3.1.1), whilst in some cases a detailed geotechnical profile of each station site may be available. An example of this type of information may be found from the K-Net database (www.k-net.bosai.go.jp) in Figure 3.1.

There are many measures from which the geotechnical site characteristics can be determined. Several common methods are considered within seismic design codes, and site classification based on the expected results. Common parameters to determine site condition are as follows.

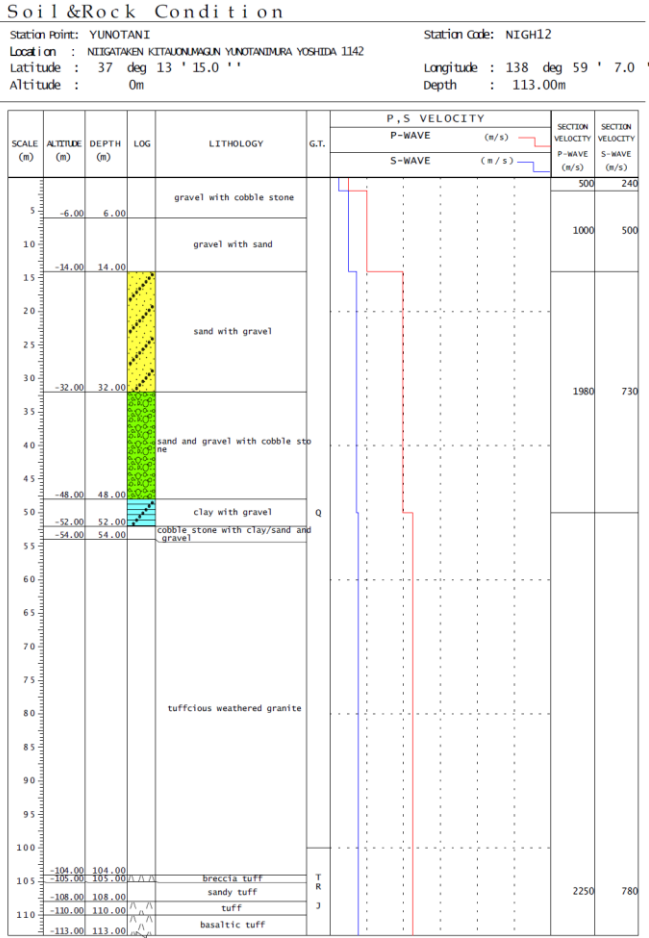


Figure 3.1: Example of geotechnical profile from station within K-Net database.

3.1.1. 30-m Average Shear Wave Velocity (Vs30)

This parameter characterises a site on the basis of the shear wave velocity in the upper 30 m of the soil. It is computed using the following equation (EN 1998-1 3.1.2(3)):

$$V_{s30} = \frac{30}{\sum_{i=1}^N \frac{h_i}{V_i}} \tag{3.1}$$

Where h_i and V_i are the thickness (in metres) and shear wave velocity (in $m\ s^{-1}$) of the i^{th} formation or layer, within a total of N layers in the top 30 m (EN 1998-1). The 30 m limit originates more from practical considerations than geophysical ones, as it corresponds to the economical depths of borehole drilling. It has, however, been shown to correlate well with geological properties, making it simple but effective proxy for site condition at short wavelengths motion. Furthermore, it has been shown to correlate well with topographic slope, allowing estimates of V_{s30} using digital elevation models, rather than field measurements. This property has been applied by Wald and Allen (2007) to provide a global V_{s30} database,

which produced V_{s30} estimates derived on the topographic slope calculated from the SRTM 30-arc second resolution database. Whilst these estimates are subject to error as a first order approximation of soil type they remain a useful tool, especially in regions where geotechnical information is limited.

3.1.2. Standard Penetration Test (N_{SPT})

The standard penetration test is one of the oldest techniques for in situ measurements of geotechnical soil properties. Standard penetration resistance is measured by the number of blows required to move the split-barrel sampler through 30 cm of soil material.

3.1.3. Undrained Shear Strength of Soil (C_u)

This refers to the maximum strength of saturated soil at which point significant plastic deformation or yielding occurs. This is measured in kPa.

3.1.4. Plasticity Index (PI)

This measure is usually only applied when considering high plasticity liquefiable clays. It indicates the range of water contents (measure in %) where the soil exhibits plasticity.

3.2 Site Classification in Eurocode 8 (Table 3.1)

Table 3.1: Site Classification Scheme in EN 1998-1 3.1.2 (Table 3.1)

Class	Stratigraphy	V_{s30} (m/s)	N_{SPT}	C_u
A	Rock or rock-like geological formation, including at most 5m of weaker material at the surface.	> 800	-	-
B	Deposits of very dense sand, gravel or very stiff clay, at least several tens of metres in thickness, characterised by gradual increase of mechanical properties with depth	360 - 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thicknesses several tens to many hundreds of metres	180 - 360	15- 50	70 – 250
D	Loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with V_s values of type C & D, and thicknesses varying between about 5m and 20m, underlain by stiffer material with $V_{s30} > 800$ m/s	-	-	-
S_1	Deposits consisting of, or containing a layer at least 10 m thick, of soft clays/silts with high plasticity index ($PI > 40$) and high water content	< 100	-	10 – 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1	-	-	-

Five ground conditions (A, B, C, D and E) are defined on the basis of stratigraphic profiles – including V_{S30} , N_{SPT} and C_u (Table 3.1). With the exception of buildings of importance class I, cone penetration tests, possibly with pore pressure measurements, should be included wherever feasible in the field investigation [EN1998-5 4.2.1(2)].

A more detailed description of the site investigation process is found in EN1998-5. Geotechnical or geological data for the construction site shall be available in sufficient quantity to allow the determination of an average ground type and/or the associated response spectrum [EN 1998-5 4.2.2 1(P)]. Shear wave velocity is regarded as the most reliable predictor of site dependent characteristics [EN 1998-5 4.2.2 4(P)], and in situ measurements of the V_s profile by in-hole geophysical methods should be used for important structures in high seismicity regions.

Whilst the shallow geological classification scheme is specified in EN 1998-1, the influence of deep geology is not. However, adaptation of the classification scheme to account for deep geology may be specified in the National Annex, providing revised estimates for the response spectrum are provided.

For very soft soil or highly liquefiable sites (S_1 and S_2 in Table 3.1) special investigations are required [EN 1998-1 3.1.2 4(P)]. For some structures this will require modelling the effects of soil-structure interaction.

3.3. Site Classification in Other Seismic Codes

The definition of site condition may vary depending on the country and on the predominant surface geology. As such, each national building code may prescribe the definitions of soil condition the development of the response spectra accordingly. A comparison of the definitions is shown here.

3.3.1. FEMA 450 & IBC 2009

The soils classification system obviously displays some similarity to that seen in EN 1998. The greatest difference is in the inclusion of a profile corresponding to hard rock (NEHRP Class A). This additional category is not found in EN 1998, where any profile with $V_{S30} \geq 800$ is simply classified as rock. The distinction may arise due to the presence of outcrops of hard crystalline rock in Eastern North America. Few such sites can be found in Europe. For most of the more consolidated soil categories (NEHRP Classes B, C, D and E) both the FEMA 450 and EN 1998 definitions (Classes B, C, D and E) are largely the same. Small differences can be found in the precise range prescribed by the undrained shear strength. This makes little practical difference to the soil classification as both codes suggest that V_{S30} and/or N_{SPT} are the preferred parameters used for classification.

When comparing many different codes, often the greatest distinction between the soil classification schemes is in the treatment of very soft and/or potentially liquefiable soils. This is clearly the case when comparing the FEMA 450 with EN 1998. The NEHRP class F describes soils requiring site specific evaluations. This includes liquefiable soils, peat and organic clays, high plasticity clays and thick soft/medium stiffness clays. Should any of these conditions arise the NEHRP guidelines mandate a site-specific investigation. EN 1998 makes a further division by separating out liquefiable and unclassifiable soils from very soft soils

and clays. However, it too mandates that “special studies for the definition of seismic action are required” in both of these circumstances.

Table 3.2: Site classification in FEMA 450

Site Class	Soil Profile Name	Average Properties to 30 m (100 ft)		
		V _{S30} (m/s)	N _{SPT}	C _u (kPa)
A	Hard Rock	1500	N/A	N/A
B	Rock	760 – 1500	N/A	N/A
C	V. Dense soil & soft rock	360 – 760	N > 50	C _u ≥ 100
D	Stiff soil profile	180 – 360	15 – 50	50 – 100
E	Soft soil profile	180		
E (b)	-	Any profile with the following: 1) N < 15 2) C _u ≤ 50 Or with more than 3 m of soft clay, defined as soil with: 1) Plasticity Index > 20 2) Moisture Content > 40 % 3) Undrained Shear-Strength C _u < 25 kPa		
F	Requires site-specific evaluations	Any profile with the following characteristics: 1) Soils vulnerable to failure under seismic loads (e.g., liquefiable soils) 2) Peats & Organic Clays (H > 3 m) 3) Very High Plasticity Clays (H > 25ft & PI > 75) 4) Very thick soft medium clays (H > 120 ft)		

The 2009 NEHRP Provisions mandate a greater range of circumstances for which site investigation is deemed necessary. Prior to construction of buildings and structures of seismic design category C – F it is necessary to provide a geotechnical investigation report. This should detail: slope instability, liquefaction, total and differential settlement and surface displacement due to faulting or seismic-induced lateral spreading or flow. Further requirements are specified for structures of design category D – F, which indicate that the geotechnical investigation should also include:

- i) Determination of dynamic lateral earth pressures on basement and retaining walls due to design earthquake ground motions.
- ii) Potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the maximum considered earthquake geometric mean peak ground accelerations (PGA_M)
- iii) Assessment of potential consequences of liquefaction and soil loss, including estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil bearing capacity and lateral soil reduction, soil downdrag and reduction in axial and lateral soil reaction for building pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
- iv) Discussion of mitigation measures such as selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated

displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.

It should be noted, however, that whilst the prescription for geotechnical site investigation is greater in the 2009 NEHRP *Provisions*, the site classification scheme itself has remained largely unchanged from the 2003 *Provisions*.

A broad comparison of site classification schemes for some of the seismic codes considered in this report is shown in Figure 3.2 (with the detailed classification schemes given in Appendix B). V_{s30} and N_{SPT} are the most common parameters used for soil classification, so their assumed range for each site class forms the basis for this comparison. Whilst the descriptions of the soil types may differ between codes, there is some general consistency in the boundaries for each site class. The most notable of these is the V_{s30} 360 m/s boundary that generally distinguishes between the stiff soils (e.g. NEHRP class D) and the highly consolidated soils and/or soft rock (e.g., NEHRP class C). To a certain extent, the site classifications may be guided by the geology of the region to which they are applied. This can be seen in the inclusion of a “hard rock” or “very strong rock” class in some codes, which correspond to V_{s30} in excess of 1500 m/s. Outcrops of such hard rocks may not necessarily be evident in some regions; hence the absence of this particular category in EN 1998 and SNI (2002) for example. There is perhaps greater disparity amongst the codes considered here in the classification of soft soils. A greater diversity of geotechnical quality may exist for soils such as these than is adequately described by the relatively simple classifications used at present.

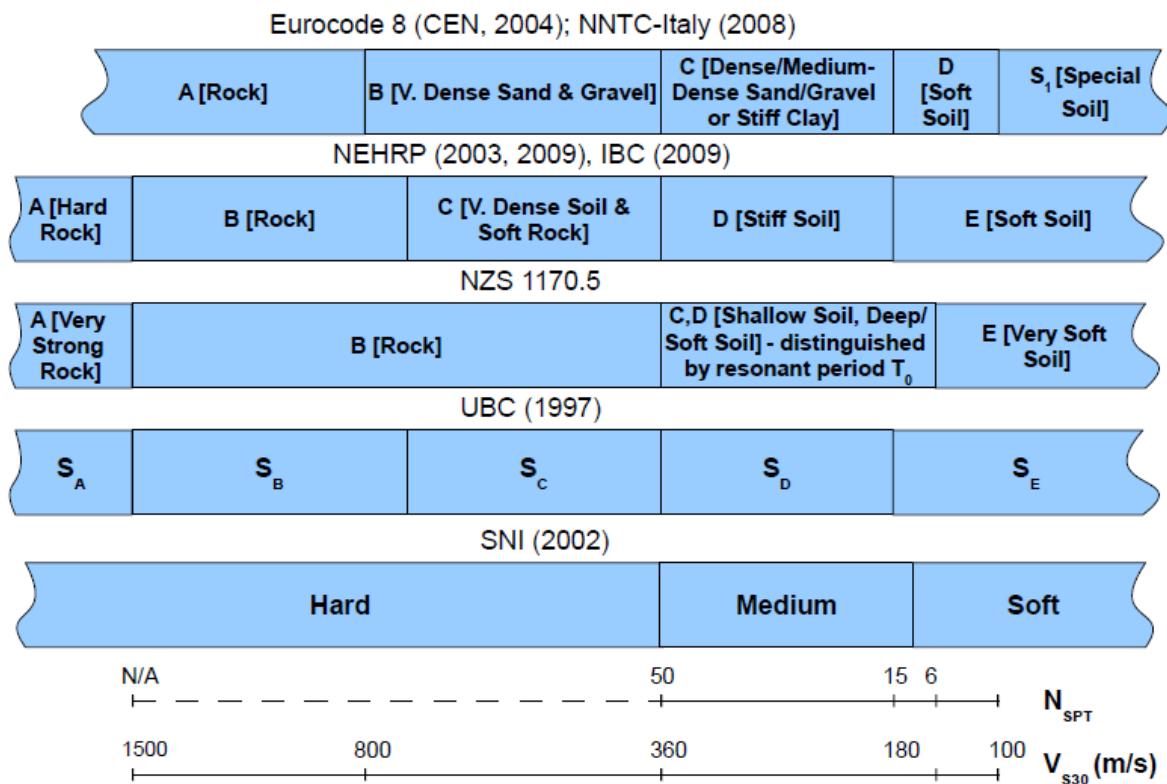


Figure 3.2: Comparison of the site classification schemes listed in section 3.3, in terms of V_{s30} and N_{SPT}

3.4. Influences on Site Response

Although soil type and shallow geology have the most substantial impact on the amplification of strong ground-motion at a site, other site-specific factors will also influence the strength and spectral shape of strong motion. These factors include topography (e.g., Boore, 1972; Geli *et al.*, 1988; Lee *et al.*, 2009) and basin characteristics (e.g., resonance, ridge effects etc.) (e.g., Stewart *et al.*, 2005; Choi & Stewart, 2005; Pacor *et al.*, 2007). The interaction of seismic waves with these geomorphological features is a complex process that may often be constrained by numerical modelling. It is because of these complexities that most building codes cannot necessarily prescribe their inclusion in the definition of seismic action. As such very few codes consider topography except in the cases of the most important structures.

Topographic amplification is given explicit consideration within EN-1998, albeit as part of the requirements for foundations, retaining structure and geotechnical aspects (and even then only in an Appendix). It is modelled as a single parameter (S_T), which assumes values between 1 and ≥ 1.4 . The classification is shown in Table 3.3.

Table 3.3: Topographic site classification in EN 1998-5

Topographic Condition*	S_T	S_T (in presence of loose surface layer)
Average slope of angles $< 15^\circ$	1.0	1.2
Isolated cliffs and slopes	≥ 1.2	≥ 1.44
Ridge crest width significantly less than base width with slopes $> 30^\circ$	≥ 1.4	≥ 1.68
Ridge crest width significantly less than base width with slopes $< 30^\circ$	≥ 1.2	≥ 1.44

*In the presence of a loose surface layer S_T should be increased by 20 %.

A near identical classification scheme for topography can be found in NNTC-Italy (2008), but otherwise topographic amplification is not explicitly accounted for in many other national codes. Similarly 2D and 3D site effects and basin responses are not explicitly accounted for in any of the codes considered here. These effects may be difficult to integrate into seismic code provisions as detailed basin models are needed for input into analysis (Stewart *et al.*, 2005). Basin models in the detail needed for numerical modelling of strong-motion are rare. In most cases such effects are only influential when using observed or synthetic time histories. Provisions for the use of such histories are discussed in section 6.

A different means of integrating the influence of site topography into the seismic action is outlined in the French (1990) seismic design provisions. These define a topographic site coefficient (τ), which is determined from the longitudinal section of the steepest gradient line going through the site (or the most unfavourable gradient line if composite sections are identified). For ridge ledges, if the gradient of the downhill slope is defined as \bar{I} , and the gradient of the uphill slope as ι , if the height of the ridge above the base relief exceeds 10 m and $\iota \leq \bar{I}/3$ then:

$$\begin{array}{ll}
\tau = 1 & \ddot{I} - \iota \leq 0.4 \\
\tau = 1 + 0.8(\ddot{I} - \iota - 0.4) & \text{for } 0.4 < \ddot{I} - \iota \leq 0.9 \\
\tau = 1.4 & \ddot{I} - \iota > 0.9
\end{array} \quad (3.3)$$

For watershed ledges $\ddot{I} - \iota$ is replaced by $\ddot{I} + \iota$ where \ddot{I} and ι are the gradients on the respective sides of the watershed. Such a definition is more complex than the general categories prescribed in Eurocode, although there is some similarity in the results. Nevertheless, the detailed description of topographic characterisation can be implemented within the scope of most site analysis, although judgement may still be required to define the conditions under which it is necessary to define the topography.

It should be emphasised that the topographic characterisation presented here is largely intended to take into account topographic amplification of strong shaking. Permanent slope displacements due to seismic actions are addressed in more detail in geotechnical provisions elsewhere within most codes.

3.5. Comments on Site Classification in EN 1998-1

The classification of site condition in Eurocode 8 is largely consistent with, and in many cases represents an improvement upon, classification schemes used in building codes elsewhere across the globe. As with many aspects of seismic code design, the representation of site conditions may be considered a compromise between the state of scientific knowledge and feasibility of implementation across the region for which the code is valid. For most structures assigned to all but the highest importance category, geotechnical investigation to the extent required for numerical analysis of site amplification comes at a prohibitive cost. In such circumstances rapid and inexpensive means of identifying the geological and geotechnical conditions of a site are necessary.

The prevalence of V_{s30} as a key indicator of site condition is likely to remain. It is the most viable parameter for this use as it can be determined, or well-estimated, using both invasive and non-invasive measures. Similarly, across many areas of geological homogeneity, V_{s30} can be interpolated from a few measurements without substantial loss of accuracy. It is understandable then that most classification schemes give V_{s30} ranges for each site condition.

Arguably the greatest criticism that may be levied at the **EN-1998** site classification scheme is emphasis placed on shallow geology (Bommer & Pinho, 2006). Provision is certainly made for the inclusion of deep geological profiles in the seismic design, but the inclusion of deep geological effects is optional and left to the National Annex. This is indicated in clause **EN 1998-1 3.1.2(1)** stating that “*Ground types A, B, C, D and E ... may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action*”. The point is further qualified in the subsequent note: “*The ground classification scheme accounting for deep geology for use in a country may be specified in its National Annex, including the values of the parameters S , T_B , T_C and T_D defining the horizontal and vertical elastic response spectra...*”. The decision not to include the influence of basement geology in the profiles may stem from several factors including the cost of obtaining such information and the current limitations on the understanding of the influence of deep geology on site amplification.

It may be possible to overcome the limitations of deep geology in **EN 1998-1** by considering some of the provisions set out in the New Zealand and Japanese code. This includes the interface depth term inferred from the peak of the horizontal-to-vertical (H/V) spectral ratio of ambient microtremor. The depth of sediment may be determined via the relation of Ibs-von Seht & Wohlenberg (1999): $h = 96f_r^{-1.388}$, or the relation of Parolai *et al* (2002): $h = 108f_r^{-1.551}$, where f_r is the frequency of the peak of the H/V spectral ratio. Whilst these relations are subject to error, and are heavily reliant on a sufficiently strong acoustic impedance between the sediment layers, the information is comparatively inexpensive to obtain, and could be feasibly implemented as part of the site investigation. As a first order approximation of sediment depth, this simple parameter for deep geology could be incorporated into other building codes at little additional cost.

An adaptation of the **EN 1998-1** site classification criteria that takes into account deeper geology is suggested by Pitilakis *et al.* (2004). In addition to the descriptive criteria, this classification considers both the H/V fundamental period (T_0) and the average shear-wave velocity of the whole soil column (as opposed to the upper 30 m). By considering the average V_s of the whole soil profile rather than the upper 30 m, this classification can utilise the following approximation to infer the depth to the bedrock (H):

$$T_0 \approx \bar{V}_s / 4H \quad (3.3)$$

The site classification criteria suggested by Pitilakis *et al.* (2004) considers 12 categories, largely subdividing each of the **EN 1998-1** classes (Table 3.4). The site amplification factors determined for this scheme are discussed in section 4.4.

Whilst the use of H/V spectral ratio to estimate sediment depth may be practical and inexpensive, there are some important caveats. There are many conditions, both geological and anthropogenic, under which an H/V peak may not be present or may be unrelated to the shallow geology (Bard *et al.*, 2004). These include the presence of noise of industrial origin, strong lateral variation of the soil type at depth, and a weak contrast in velocity at depth. In any of these conditions it may not be possible to use the H/V spectral ratio to estimate the depth of the soil profile. There may still be a significant impact on the amplification of strong motion at such sites. Array analysis and inversion of ambient microtremor may still be used as a means of deriving estimates of the shear-wave velocity profile at depths greater than 30 m (e.g., Scherbaum *et al.*, 2003; Di Guilo *et al.*, 2006).

3.6. Defining site amplification in the seismic action

The method of site characterisation is clearly an area that requires a substantial amount of research as it will define the measures by which site-specific analysis can be achieved. Adequate site characterisation, however, does not, by itself, define the actual influence of the site on strong shaking. Most of the building codes analysed here follow the same approach to incorporating site condition into the response spectra. Scaling using pre-determined factors for given site classes may be designed to capture the nonlinear amplification effects observed in strong-motion records on softer soil sites. This scaling controls not only the level of spectral acceleration, but also the shape of the spectrum by adjusting the constant velocity and constant displacement corner periods. **EN 1998-1** provides a clear illustration of this principle as the reference bedrock acceleration is defined as the input for hazard analysis. The

acceleration is then multiplied by a constant factor according to the site condition, and the corner periods adjusted.

Table 3.4: Adapted Eurocode Site Classification Scheme (Pitilakis et al., 2004)

Class	Description	T₀ (s)	Remarks
A1	Rock Formation	≤ 0.2	V _S ≥ 1500 m/s
A2	Slightly weathered / segmented rock formations (thickness of weathered layer <5.0m)	≤ 0.2	Surface weathered layer: V _S ≥ 300 m/sec Rock Formations: V _S ≥ 800 m/sec
	Geologic formations resembling rock formations in their mechanical properties and their composition (e.g., conglomerates)		V _S ≥ 800 m/sec
B1	Highly weathered rock formations whose weathered layer has a considerable thickness (> 5.0m - 30.0m)	≤ 0.4	Weathered layer, V _S ≥ 300 m/sec
	Soft rock formations of great thickness or formations which resemble these in their mechanical properties (e.g., stiff marls)		V _S : 400-800 m/sec N _{SPT} > 50 C _u > 200 kPa
	Soil formations of very dense sand – sand gravel and/or very stiff clay, of homogenous nature and small thickness (up to 30.0m)		V _S : 400-800 m/sec N _{SPT} > 50 C _u > 200 kPa
B2	Soil formations of very dense sand – sand gravel and/or very stiff clay, of homogenous nature and medium thickness (30.0 - 60.0m), whose mechanical properties increase with depth	≤ 0.8	V _S : 400-800 m/sec N _{SPT} > 50 C _u > 200 kPa
C1	Soil formations of dense to very dense sand – sand gravel and/or stiff to very stiff clay, of great thickness (> 60.0m), whose mechanical properties and strength are constant and/or increase with depth	≤ 1.2	V _S : 400-800 m/sec N _{SPT} > 50 C _u > 200 kPa
C2	Soil formations of medium dense sand – sand gravel and/or medium stiffness clay (PI > 15, fines percentage > 30%) of medium thickness (20.0 – 60.0m)	≤ 1.2	V _S : 200-400 m/sec N _{SPT} > 20 C _u > 70 kPa
C3	Category C2 soil formations of great thickness (>60.0 m), homogenous or stratified that are not interrupted by any other soil formation with a thickness of more than 5.0m and of lower strength and V _S velocity	≤ 1.4	V _S : 200-400 m/sec N _{SPT} > 20 C _u > 70 kPa
D1	Recent soil deposits of substantial thickness (up to 60m), with the prevailing formations being soft clays of high plasticity index (PI>40), high water content and low values of strength parameters	≤ 2.0	V _S ≤ 200 m/sec N _{SPT} < 20 C _u < 70 kPa

Table 3.4 (continued)

Class	Description	T_0 (s)	Remarks
D2	Recent soil deposits of substantial thickness (up to 60m), with prevailing fairly loose sandy to sandy-silty formations with a substantial fines percentage (so as not to be considered susceptible to liquefaction)	≤ 2.0	$V_S \leq 200$ m/sec $N_{SPT} < 20$
D3	Soil formations of great overall thickness (> 60.0m), interrupted by layers of category D1 or D2 soils of a small thickness (5 – 15m), up to the depth of ~40m, within soils (sandy and/or clayey, category C) of evidently greater strength, with $V_S \geq 300$ m/sec	≤ 1.2	$V_S \geq 300$ m/sec
E	Surface soil formations of small thickness (5 - 20m), small strength and stiffness, likely to be classified as category C and D according to its geotechnical properties, which overlie category A formations ($V_S \geq 800$ m/sec)	≤ 0.5	Surface soil layers, V_S : 150-300 m/sec
X	<ul style="list-style-type: none"> • Loose fine sandy-silty soils beneath the water table, susceptible to liquefaction (unless a special study proves no such danger, or if the soil's mechanical properties are improved). • Soils near obvious tectonic faults. • Steep slopes covered with loose lateral deposits. • Loose granular or soft silty-clayey soils, provided they have been proven to be hazardous in terms of dynamic compaction or loss of strength. • Recent loose landfills • Soils with a very high percentage of organic material 		

Whilst this characterisation of site modification is simple and easy to implement, it is increasingly at odds with new developments in the characterisation of site amplification in the ground motion models. Previous ground motion models developed for Europe and the Middle East have adopted linear site amplification factors. These are based on simple site classification schemes, usually the **EN 1998** or NEHRP classification schemes. This is partly done out of necessity owing to a paucity of geotechnical information for many of the strong-motion recordings available in Europe and the Middle East. Investigations of the statistical significance of site amplification factors in the European strong-motion data have been undertaken by the authors of several recent models. These would generally concur that the current characterisation of site amplification is appropriate given the limitations of the metadata for the current database, although improvements are ongoing.

3.6.1. Nonlinear Site Amplification

To determine whether the inclusion of nonlinear soil amplification effects should be considered within the characterisation of seismic actions in European design codes, it is helpful to address two questions. The first question is whether a nonlinear amplification response is evident in European strong-motion records? The second question is how to model the nonlinear response where it is evident?

The existence of nonlinearity in site amplification in strong-motion records has been the subject of several investigations on different databases: the PEER strong-motion database (Choi & Stewart, 2005), a global database constructed predominantly of Japanese K-Net strong-motion records (Cauzzi & Faccioli, 2008) and a European strong-motion database (Akkar & Bommer, 2007a, b). Tests for nonlinearity look for statistically significant negative trends in the residuals of the ground motion models, relative to an expected value, with respect to magnitude, distance and/or expected ground motion (typically PGA) on rock. Choi & Stewart (2005) demonstrate statistically significant negative trends for residuals on NEHRP category D and E soils. Similar analyses of residuals for European strong-motion records were undertaken by Akkar & Bommer (2007a) and Bommer *et al.* (2007). Their analyses did not reveal strong evidence for soil nonlinearity, although there are limitations to their analysis. These limitations include a paucity of records of high amplitude motion on soil sites (where it is expected that nonlinearity would be most evident), and the cruder categorisation of the site class for each record. They suggest that such evidence does not necessarily preclude the existence of nonlinearity in European strong-motion, but that it could not be identified in the data set used. Analysis of the Cauzzi & Faccioli (2008) dataset considers evidence for nonlinearity in displacement response spectra. They implement a similar analysis of the ground motion residuals for NEHRP categories B, C and D, and find statistically significant negative trends only for class D sites for displacements in the period range $0.45 \leq T \text{ (s)} \leq 1.3$. When modelling the site directly in terms of V_{s30} this range is limited only to the $0.45 \leq T \text{ (s)} \leq 0.6$ s, $T = 0.75$ s and $T = 0.95$ s spectral ordinates. They conclude therefore, that the records show only modest nonlinear effects for **EN 1998-1** class C and D sites.

Whilst the analyses of strong-motion records, and in particular European strong-motion records, do not necessarily imply strong nonlinearity in the site response, it cannot be assumed that soil nonlinearity should not be considered in the design spectra. Many ground motion prediction models now include nonlinear site amplification, and nonlinear amplification models used in their development could provide some guidance as to how such effects can be characterised within a seismic design code.

Of the five “Next Generation Attenuation” models, four include nonlinear terms for site amplification (Abrahamson & Silva, 2008; Boore & Atkinson, 2008; Campbell & Borzogna, 2008; Chiou & Youngs, 2008). For several of the models the nonlinear site amplification factor represents a development upon the nonlinear models in previous GMPEs (e.g. Abrahamson & Silva, 1997; Campbell, 1997). In most cases site amplification is defined as a function of V_{s30} and median acceleration on a reference rock surface. In most of these cases the aleatory variability of the model is also a function of site condition. This transition from linear to nonlinear site amplification is in no small part due to the work of Choi & Stewart (2005) and Stewart & Choi. (2005), whose nonlinear model for site amplification as a function of V_{s30} is frequently cited in the construction of these models. The Choi & Stewart (2005) model is expressed as:

$$\ln(F_{ij}) = c \ln\left(\frac{V_{s30}}{V_{ref}}\right) + b \ln\left(\frac{PHA_{r_{ij}}}{0.1}\right) + \eta_i + \sigma_i \quad (3.4)$$

Where $PHA_{r_{ij}}$ is the peak horizontal acceleration in a material of reference V_{s30} . η_i is the inter-event residual and σ_i the intra-event residual. For most of the NGA models the Choi & Stewart (2005) model forms only a starting point for characterisation of the site amplification term, all of which develop the nonlinear term further. In some cases (Abrahamson & Silva,

2008; Campbell & Bozorgnia, 2008) the site effect may also include a basin response factor (Day *et al.*, 2008). For many of the NGA relations the nonlinear site amplification factor is based on the model of Walling *et al.* (2008):

$$\ln(Amp) = \begin{cases} a \ln\left(\frac{V_{S30}}{V_{LIN}}\right) - b \ln(PGA_{ROCK} + c_1) + b \ln\left(PGA_{ROCK} + c\left(\frac{V_{S30}}{V_{LIN}}\right)^n\right) + d & \dots V_{S30} < V_{LIN} \\ (a + bn) \ln\left(\frac{V_{S30}}{V_{LIN}}\right) + d & \dots V_{S30} \geq V_{LIN} \end{cases} \quad (3.5)$$

Where, V_{LIN} is a reference shear-wave velocity, PGA_{ROCK} is the expected value of the ground acceleration on the reference bedrock, a , b , c_1 , c , d and n are parameters, some or all of which may be dependent on period. Whilst it may be possible to fix V_{LIN} , scaling of the spectrum to different values of V_{S30} is a more complex procedure.

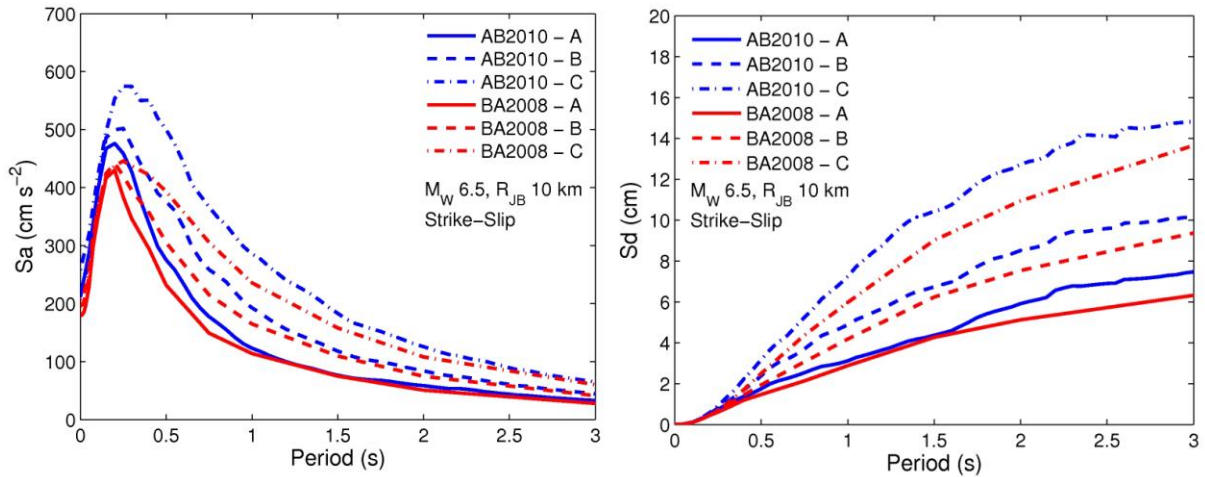


Figure 3.3: Elastic acceleration response spectra (left) and displacement response spectra (right), compared for the Boore & Atkinson (2008) [BA2008] and Akkar & Bommer (2010) [AB2010] ground motion prediction models for EN 1998 site conditions A, B and C.

The implications of this research, and the approach adopted within the NGA project, are that linear scaling of the elastic response spectrum is not necessarily appropriate for strong ground-motion models. Comparison can be made between two spectral acceleration GMPEs, one with nonlinear scaling of site response according to V_{S30} (Boore & Atkinson, 2008), the other with site response scaled according to EN 1998 site class (Akkar & Bommer, 2010). The examples shown in Figure 3.3 are the median 5 % damped spectra for a site at a Joyner-Boore distance of 10 km away from a M_W 6.5 strike-slip event. A further comparison is also made to the Cauzzi & Faccioli (2008) attenuation model (CF2008). This particular model is derived largely from strong-motion records from the Japanese K-net strong-motion database, which includes a larger number of records on soft soil (EN 1998 class D) sites. This particular model is derived for spectral displacements, which are also converted to acceleration pseudo-spectra. As indicated previously, soil nonlinearity in the Cauzzi & Faccioli (2008) database is only evident, at the 5 % level of significance, across a limited period range for class D sites.

The CF2008 model therefore considers only linear site amplification effects. As noted by Cauzzi & Faccioli (2008), for large magnitude events in the near field, such as the M_W 6.5 event at R_{JB} 10 km considered in Figure 3.3 (corresponding to an assumed focal distance of 14.1 km), the spectral displacements significantly exceed those other models. A comparison of all three models is shown in Figure 3.4, which demonstrates clearly the differences between the CF2008 spectra and those of AB2010 and BA2008. The three site classes correspond to **EN 1998** class A, B and C, with V_{S30} values assumed to be 850 m s^{-1} , 500 m s^{-1} and 300 m s^{-1} , respectively.

The comparison between linear and nonlinear scaling of the acceleration spectra with site condition reveals an interesting trend. The Boore & Atkinson (2008) model [BA2008] clearly shows greater amplification at longer periods on softer soil sites. The peak spectral acceleration shows relatively little variation. For the Akkar & Bommer (2010) model, there is greater amplification of strong-motion across short and long periods, with significantly higher peak spectral accelerations on soft soil sites. It should be recognised that not all of the differences between the site amplification of the two ground motion models can be attributed to nonlinear scaling. The depth of the soil deposits and the influence of basin effects may still be a contributing factor. The data used in the creation of the two models differs significantly, with a greater number of records from large earthquakes found in the NGA data set used to derive BA2008. For the CF2008 model, accelerations and displacements are significantly greater across the full period range shown; hence comparison between this model and the AB2010 and BA2008 models is more challenging. Aside from the higher spectral ground motions, the most significant different difference in the CF2008 model is the substantial amplification in the 0.5 to 1.5 s period range on class D sites. This is, of course, a direct manifestation of the nonlinear amplification effects identified previously in the database for this particular site class.

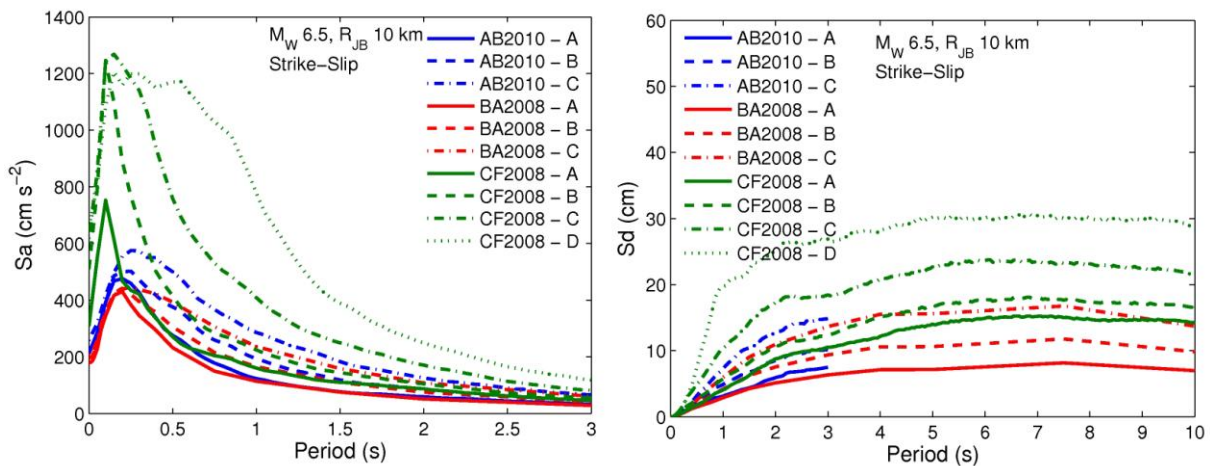


Figure 3.4: As Figure 3.3, with the addition of the CF2008 model and **EN 1998** site class D.

To obtain a clearer perspective as to the influence of site conditions on the different GMPEs considered in Figure 3.4, the models are plotted as ratios of spectral acceleration on soil sites to spectral acceleration on rock sites (Figure 3.5). The same scenario earthquake is assumed as for previous figures. For short-period acceleration ($T < 0.5 \text{ s}$), the scaling of ground motion with site class for the AB2010 and BA2008 model is nearly identical. The AB2010 model produces a slightly greater amplification in the 1 s to 2 s period range for class C sites, but

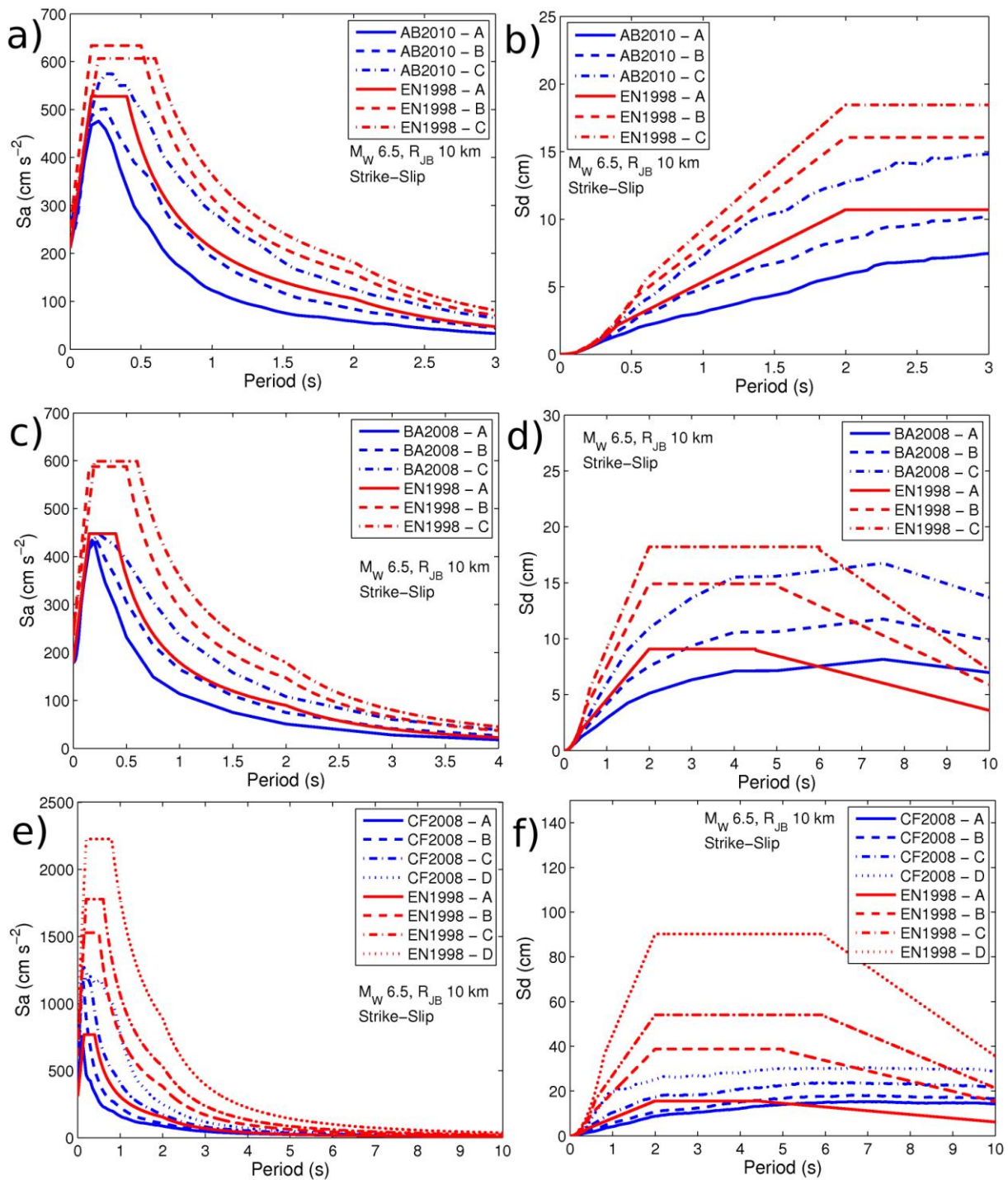


Figure 3.6: Comparison of the acceleration (left) and displacement (right) spectra with design spectra created according to **EN 1998** for each site class. The GMPEs used are: AB2010 (a, b), BA2008 (c, d) and CF2008 (e, f).

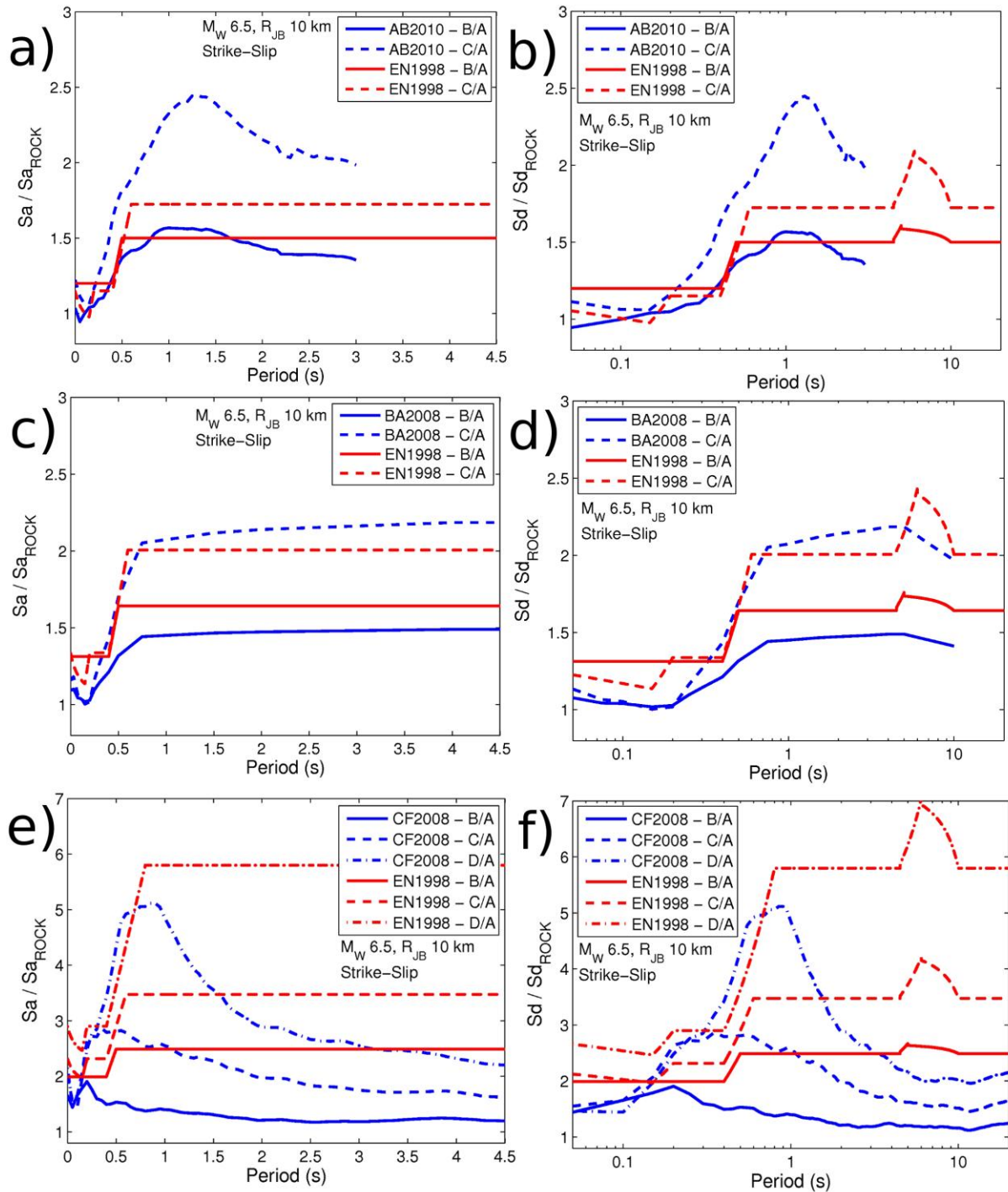


Figure 3.7: Comparison of spectral acceleration (left) and displacement (right) amplification factors for EN 1998 and the AB2010 (a, b), BA2008 (c, d) and CF2008 (e, f) GMPEs.

The comparison between NGA models and the EN 1998 spectrum provides only a limited insight into the issues of how to constrain nonlinear site amplification within a building code. In all of the NGA models the amplification factor varies to a greater or lesser degree with the period under consideration. Scaling of the spectrum based on only one or two anchor points (e.g., PGA) will not necessarily capture the impact of the site response on the hazard spectrum. The examples shown in Figure 3.4 are derived using only scenario spectra. A fairer

comparison may be attempted using the UHS constructed for a given ground motion prediction model.

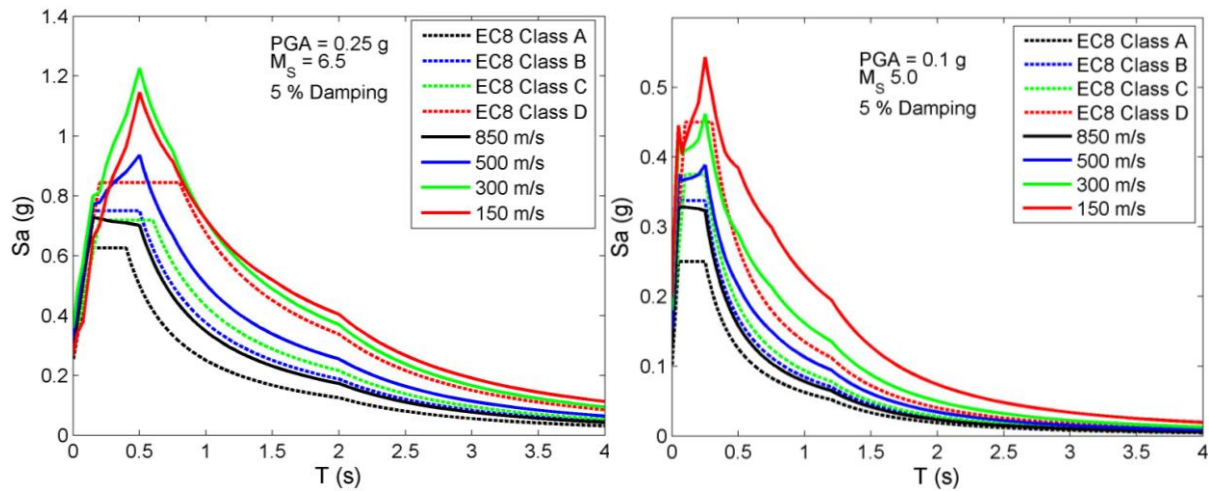


Figure 3.8: Comparison of EN 1998 and nonlinear scaling of the EN 1998 response spectrum for Type 1 (left) and Type 2 (right) spectrum.

The examples given in this discussion suggest that the linear approach to site scaling that is implemented within **EN 1998** is insufficient to characterise nonlinear site response. The question of whether to adopt a nonlinear scaling function in code design nevertheless remains open. Evidence of a nonlinear site response in European strong-motion records is not yet compelling, and provides only a limited amount of data from which to model such a response, especially on soft soil site. Decisions are needed to determine which nonlinear site amplification model is most appropriate, in addition to some agreement on the model parameters. Some amplification parameters are period dependent; hence constraint of the model would require explicit specification of many numbers. There is fortunate coincidence in the adoption of V_{S30} as the site coefficient within many GMPEs models, and the use of V_{S30} as the primary parameter for site classification. However, there are many geotechnical properties of the soil that may influence the site amplification, and fixed characterisation according to a single parameter risks overlooking such properties. Furthermore, whilst it may be anticipated that the use of NGA models will (and indeed has) become widespread, nonlinear site amplification is not yet widely adopted within GMPEs models from other regions. This situation may change in light of future research.

It is perhaps unrealistic to expect that seismic design codes should implement new models for site classification and amplification at the same rate as such models are adopted within strong ground-motion relations. This also applies to topographic amplification and basin response. To some extent the influence of more complex site response features are incorporated within the epistemic uncertainty of the ground motion models used in seismic hazard analysis. Given these challenges, there may be a case to be made for using the uniform hazard spectrum as a direct basis for seismic input rather than the Newmark & Hall (1983) design spectrum that is commonly adopted in codes. This allows the user to specify the site condition directly, as defined according to the input for the ground motion prediction model, thus providing a hazard spectrum that is consistent with the classification of the ground motion. A web-based application for definition the uniform hazard spectrum for a user-specified V_{S30} could prove a useful tool.

4. Characterising Seismic Action: Elastic Response Spectrum

4.1. Elastic Response Spectrum (ERS)

The elastic response spectrum, as it is defined within most seismic code formats, is used to represent the response of the ground motion in an idealised, or simplified, manner that is appropriate for the purposes of earthquake design. The response spectrum of an earthquake record represents the response of a set of single-degree-of-freedom (SDOF) oscillators with natural period, T , and damped at ξ fraction of critical damping. The response may be given in terms of displacement, pseudo-velocity and pseudo-acceleration, of which the latter remains most common for seismic design. An example of a pseudo-acceleration response spectrum and its corresponding spectral displacement, for different levels of damping, is given in Figure 4.1.

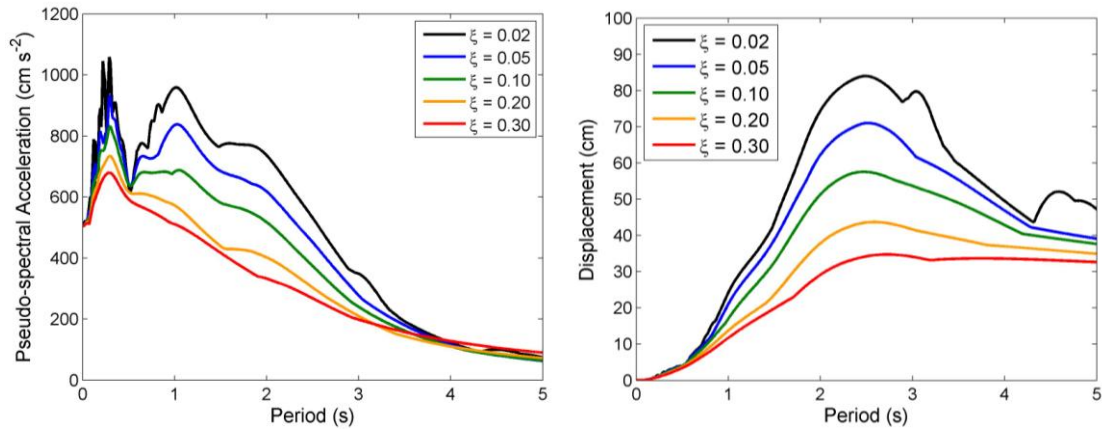


Figure 4.1: Elastic response spectra from the 1992 Erzincan, Turkey, earthquake: acceleration spectra (left) and displacement spectra (right), for different fractions of critical damping (ξ)

Observed earthquake response spectra provide important information about characteristics of ground motion to which a structure may be subjected. However, they also contain many features that are unique to each earthquake, or common to only a subset of earthquakes that may present a hazard to the structure in question. The spectral displacement (S_d), pseudo-spectral velocity (S_v) and the pseudo-spectral acceleration (S_a) are directly related via the following relations:

$$S_v(T) = \frac{2\pi}{T} S_d(T) \quad (4.1a)$$

$$S_a(T) = \left(\frac{2\pi}{T}\right)^2 S_d(T) \quad (4.1b)$$

To develop a spectrum that considers the response of many earthquakes other techniques may be used. For input into seismic design, the idealised response spectrum may correspond directly to the uniform hazard spectrum (UHS) developed for a site. This is an outcome of probabilistic seismic hazard analysis, whereby spectral attenuation relationships are employed and the spectrum illustrates the spectral acceleration (or displacement) with a given return

period; hence, each ordinate of the UHS gives the spectral response at the given period, irrespective of the originating earthquake. This will be discussed further in section 5.

For the purposes of seismic code design, Newmark & Hall (1973) introduce a method of approximating the hazard spectrum by a “design spectrum” (the terminology is distinct from that of EN-1998). The design spectrum assumes that for different period ranges of the spectrum, the response acceleration of a single-degree-of-freedom oscillator is controlled by acceleration (short-period), velocity (intermediate-period) and displacement (long-period). Originally represented on a tripartite plot, the resulting spectrum is controlled by the peak spectral acceleration, peak spectral velocity and peak spectral displacement, and the period of transition from each portion of the spectrum. The Newmark & Hall (1973) design spectrum is anchored to PGA for very high frequency motion, a practice that remains common in current codes. This is a convenient step, though no longer an essential one as seismic hazard analyses use spectral ground motion rather than peak ground motion. An idealised design response spectrum, given in terms of S_a -T and S_d -T, is shown in Figure 4.2.

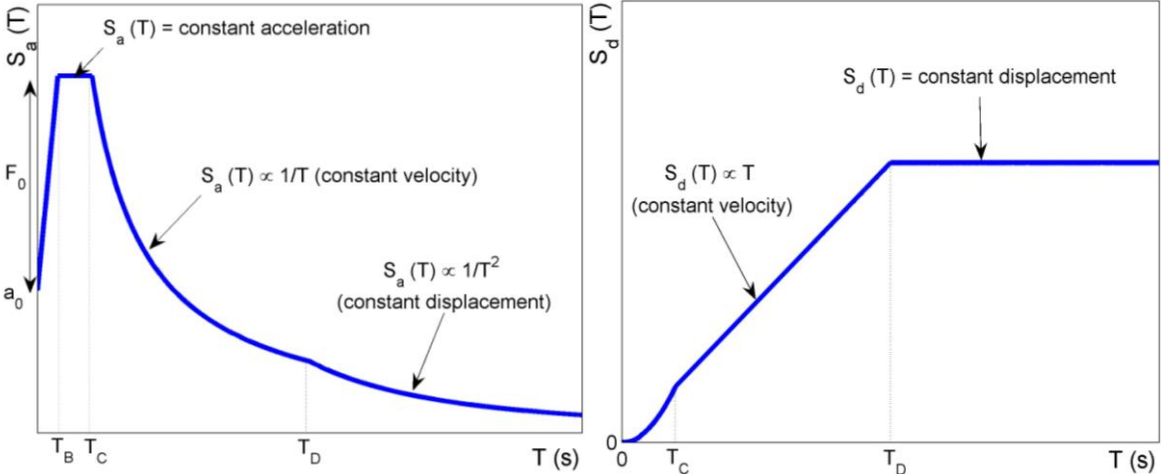


Figure 4.2: Idealised acceleration (left) and displacement (right) elastic response [design] spectrum.

The underlying necessity to consider code implementation of the response spectrum has made the Newmark & Hall (1973) design spectrum the most common method of characterising seismic input in earthquake building codes. There have been many adaptations to the original methods that can be seen in current design codes. In particular, the definition of the amplification factor (F_0) and the corner periods (T_B , T_C and T_D) has changed to allow the design spectrum to reflect, more accurately, the hazard at a site.

4.2. EN 1998 “Elastic Response Spectra”

The elastic response spectrum is clearly defined for the horizontal components of seismic action from the following equations (EN 1998-1 3.2.2.2 (1)P):

$$0 \leq T \leq T_B: S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1) \right] \quad (4.2a)$$

$$T_B \leq T \leq T_C: S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \quad (4.2b)$$

$$T_C \leq T \leq T_D: S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C}{T} \right] \quad (4.2c)$$

$$T_D \leq T \leq 4.0s: S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[\frac{T_C T_D}{T^2} \right] \quad (4.2d)$$

where:

- $S_e(T)$ is the elastic response spectrum (i.e., pseudo-spectral acceleration at vibration period T of a linear single-degree-of-freedom system)
- a_g is the design peak ground acceleration defined as the product of reference ground acceleration $a_{g,R}$ and importance factor γ_I .
- S is the soil factor
- η is the damping correction factor with a reference value of $\eta = 1$ for 5 % viscous damping

The shape of the curve is fixed by four points (a_g , T_B , T_C and T_D), and response acceleration is presented as the ratio of the pseudo-spectral acceleration to the site-adjusted peak ground acceleration, $a_{g,R} \cdot S$ (hence $S_e(0) = 1$). The lower limit of the constant acceleration part of the spectrum is given by T_B , and the upper limit by T_C . The third parameter is T_D , which marks the lower limit of the constant spectral displacement part of the spectrum.

Table 4.1: Site and corner periods for the **EN 1998-1 ERS**

Ground Type	Type 1 Spectrum ($M_S \geq 5.5$)				Type 2 Spectrum ($M_S < 5.5$)			
	S	T_B (s)	T_C (s)	T_D (s)	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.4	2.0	1.0	0.05	0.25	1.2
B	1.2	0.15	0.5	2.0	1.35	0.05	0.25	1.2
C	1.15	0.2	0.6	2.0	1.5	0.1	0.25	1.2
D	1.35	0.2	0.8	2.0	1.8	0.1	0.30	1.2
E	1.4	0.15	0.5	2.0	1.6	0.05	0.25	1.2

The three parameters T_B , T_C and T_D , as well as the soil parameter S are ascribed to each country by its National Annex. In the absence of the influence of deep geology, two different spectra are suggested for use [EN 1998-1 3.2.2.2 2(P) – NOTE 1]. These spectra are Type 1, if the earthquakes that contribute most to probabilistic seismic hazard at a site have surface-wave magnitudes $M_S \geq 5.5$, and Type 2, for all other events. Recommended values for the four parameters, for each of the two ERS types are given in EN 1998-1 3.2.2.2, as shown in Table 4.1, with the corresponding plots ($a_g = 0.15$ g on soil type A for a structure with 5 % damping) given in Figure 4.3. These values are based on the analysis of Rey *et al.* (2002) using data from the European Strong-Motion Database (Ambraseys *et al.*, 2004).

If the recommended values are adopted then the scaling of the ERS is dependent only on site-adjusted PGA. The values of the controlling periods (T_B , T_C and T_D) are fixed for each site

condition, irrespective of the actual PGA, subject to the selection of a controlling earthquake scenario.

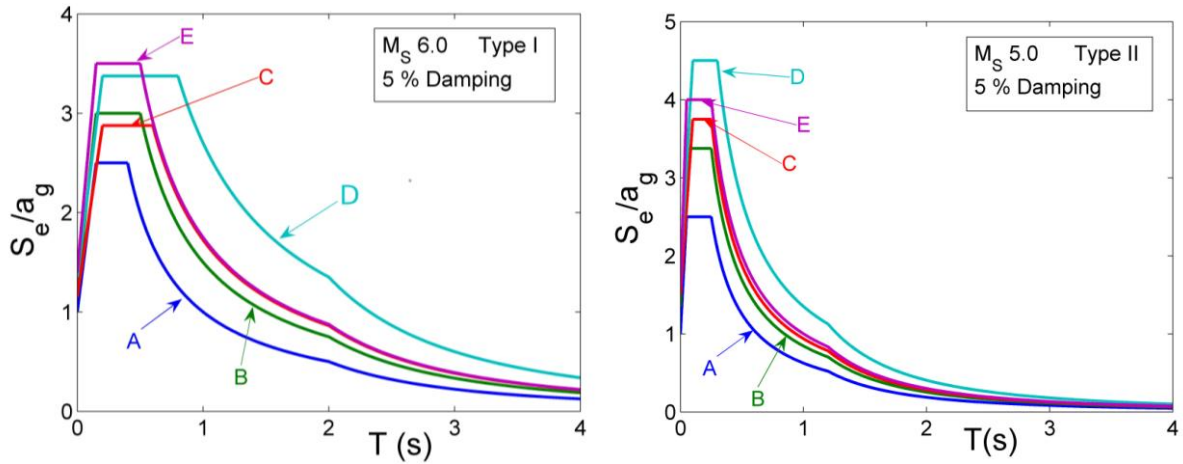


Figure 4.3: Type 1 (left) and Type 2 (right) elastic response spectra corresponding to the values in Table 4.1.

The damping correction factor (η) is commonly fixed at 5 % ($\eta = 1$), but can be scaled to other factors according to the expression given in EN 1998-1 3.2.2.2 (3), derived by Bommer *et al.* (2000):

$$\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0.55 \quad (4.3)$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage. Given the provisions set out in this particular clause, it is necessary to define the seismic action from the hazard analysis only for 5 % damping. Adjustment of the spectrum for different levels of damping should be made by scaling according to (4.3), or another formula should it be defined differently in future adaptations of the code, or in National Annexes. Since most ground motion prediction models only consider the 5 % damped motion, this is the approach that can be most widely applied. More discussion on this topic can be found in section 5.4.

EN 1998-1 also defines the elastic response spectrum for vertical seismic motion. This is provided by the following equations (EN 1998-1 3.2.2.3 (1P)):

$$0 \leq T \leq T_B : S_{ve}(T) = a_{vg} \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3.0 - 1) \right] \quad (4.4a)$$

$$T_B \leq T \leq T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \quad (4.4b)$$

$$T_C \leq T \leq T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \left[\frac{T_C}{T} \right] \quad (4.4c)$$

$$T_D \leq T \leq 4.0s : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \left[\frac{T_C T_D}{T^2} \right] \quad (4.4d)$$

Where $S_{ve}(T)$ is the vertical acceleration at period T , a_{vg} the vertical design peak ground acceleration and η the damping correction factor (as defined previously). No site condition is included in the definition of the vertical response spectrum. Values for T_B , T_C and T_D are ultimately devolved to the National Annexes. However, “recommended” values are given in Table 4.2.

In this definition of vertical action the vertical PGA (a_{vg}) is scaled from the horizontal PGA (a_g) and is therefore not calculated directly in the hazard analysis. The scaling factor is given for $M_s \geq 5.5$ earthquakes (Type 1) and $M_s < 5.5$ earthquake (Type 2). The difference in scaling due to the source-site distance is not taken into account.

Table 4.2: Corner periods for the vertical elastic response spectrum (EN 1998-1)

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0.9	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

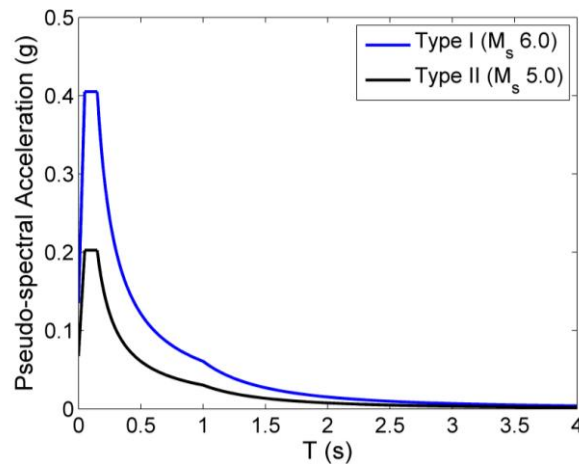


Figure 4.4: EN 1998-1 vertical elastic response spectra

In addition to the acceleration response spectrum, EN 1998 provides an explicit formulation of the displacement response spectrum. The elastic acceleration response spectrum is used to determine the elastic displacement response spectrum by virtue of the established expression [EN 1998-1 3.2.2.5(P)]:

$$S_{De}(T) = S_e(T) \left[\frac{T}{2\pi} \right]^2 \quad (4.5)$$

Which is a simple re-arrangement of equation 4.1. This relation is only considered for structures with vibration periods shorter than 4 s. For longer periods a more complete definition of the displacement response spectrum is mandated. A suggestion of such a spectrum can be found in the Informative Appendix A to Eurocode 8-1. This takes the form of the following relations:

$$T \leq T_E : S_{De}(T) = S_e(T) \left[\frac{T}{2\pi} \right]^2 \quad (4.6a)$$

$$T_E \leq T \leq T_F : S_{De}(T) = 0.025a_g \cdot S \cdot T_C \cdot T_D \left[2.5\eta + \left(\frac{T - T_E}{T_F - T_E} \right) (1 - 2.5\eta) \right] \quad (4.6b)$$

$$T \geq T_F : S_{De}(T) = d_g \quad (4.6c)$$

Where d_g is the peak ground displacement for very long-period ground-motion, defined by [EN 1998-1 3.2.2.4]: $d_g = 0.025 \cdot a_g \cdot S \cdot T_C \cdot T_D$. The parameters T_E and T_F are given in the informative Appendix A.1 of EN 1998 and Table 4.3 here.

Table 4.3: Long-period displacement corner periods from EN 1998-1 Annex A

Ground Type	T_E (s)	T_F (s)
A	4.5	10.0
B	5.0	10.0
C	6.0	10.0
D	6.0	10.0
E	6.0	10.0

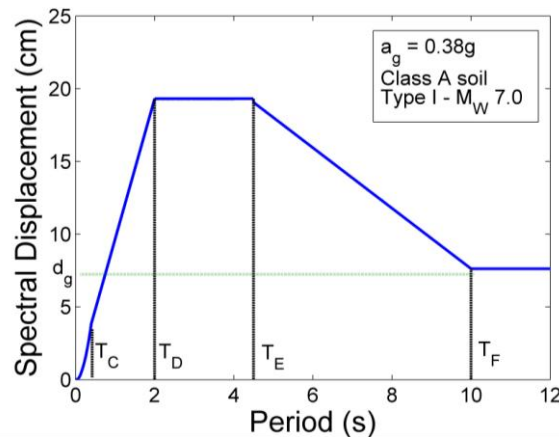


Figure 4.5: Extended EN 1998-1 displacement spectra

The extension of the displacement spectra to longer periods suggested from the EN 1998 Informative Annex A, is based on the work of Tolis & Faccioli (1999) and Bommer & Elnashai (1999). The generation of displacement spectra based on conversion from acceleration spectra using equation 4.1, was found to be inadequately characterised in most codes. Based on observation of long-period displacements from observed response spectra, Bommer & Elnashai (1999) suggested a six-segment displacement spectrum (adjusted to five segments in Eurocode), converging at peak ground displacement (PGD) for periods longer than 10 s. This extension was shown to be a more appropriate representation of earthquake motion for longer period structures. It should be noted that the displacement spectrum extension given in Annex A to EN 1998 is relevant only to the Type 1 spectrum.

In line with most seismic codes across the globe, the seismic action at a given point on the surface is represented by an elastic ground acceleration response spectrum (**EN 1998-1 3.2.2.1 (1)P**). Further provisions are made to comprehensively define the seismic action:

- *The spectrum shape is taken as being the same for both the no-collapse and the damage-limitation requirement (**EN 1998-1 3.2.2.1 (2)**).*
- *The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum (**EN 1998-1 3.2.2.1 (3)P**).*
- *For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquakes generated from them (**EN 1998-1 3.2.2.1 (4)**).*
- *For important structures ($\gamma_I > 1.0$) topographic amplification effects should be taken into account (**EN 1998-1 3.2.2.1 (6)**).*
- *Allowance for the variation of ground motion in space as well as time may be required for specific types of structures (see **EN 1998-2 [Bridges]**, **EN 1998-4 [Silos, Tanks and Pipelines]** and **EN 1998-6 [Towers, Masts and Chimneys]**).*

4.3. Elastic Response [Design] Spectra in Other Building Codes

Whilst some building codes have adopted different interpretations of return period and site classification, elastic response is almost always given in terms of the Newmark & Hall response spectrum. There are, however, many disparities in how the corner periods between the constant acceleration, displacement and velocity parts of the spectrum are defined. These differences indicate the degree to which the shape of the response spectrum is expected to change due to factors other than exclusively site class (Bommer, 2004).

Very little consideration is given to spectral displacement in most other codes. In many cases the elastic acceleration response spectra can be used to determine $S_{De}(T)$ via equation 4.1b, and this approach may be implicit within a given code. Without further constraint in the long-period spectral ordinates, the accuracy of this method when spectral displacement response is considered for long-period motion is questionable.

4.3.1. FEMA 450/IBC 2009

The clearest distinction between the **FEMA 450** approach to response spectrum development, and that of **EN 1998-1** is the use of two spectral acceleration parameters (0.2 s and 1.0 s) and a spatially variable long-period parameter T_L to define the curve. PGA is not used as an input parameter for the response spectrum, which is now anchored to a “short” and “long” period spectral acceleration. This approach incorporates the influence of larger earthquakes into the response spectrum, which will alter the relative amplitude of longer period motions to shorter period ones.

Seismic action in NEHRP/IBC earthquake design codes is defined from a suite of hazard maps indicating the Maximum Considered Earthquake for 0.2 s and 1.0 s spectral acceleration, in addition to a long-period parameter, T_L , for the United States and its territories (Puerto Rico, Guam and the U.S. Virgin Islands). Interpolation of these parameters from the contours presented in these maps is obviously an unsatisfactory approach. The errors arising from this process have been largely overcome by the dissemination of the hazard data online, accompanied by free software from the production of the ERS

(<http://earthquakes.usgs.gov/hazards>). The online database contains the relevant hazard parameters mapped at 0.05° resolution (approximately 5 km).

The design response spectrum proposed in the **FEMA 450** is created using the following relations:

$$0 \leq T \leq T_0: \quad S_a = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS} \quad (4.7a)$$

$$T_0 \leq T \leq T_S: \quad S_a = S_{DS} \quad (4.7b)$$

$$T_S \leq T \leq T_L: \quad S_a = \frac{S_{DL}}{T} \quad (4.7c)$$

$$T \geq T_L: \quad S_a = \frac{S_{DL} T_L}{T^2} \quad (4.7d)$$

Where T is the fundamental period of the structure, S_{DS} and S_{DL} are the design spectral response acceleration parameters at short (0.2 s) and long (1.0 s) periods, respectively. The parameter T₀ is determined by $T_0 = 0.2 S_{DL} / S_{DS}$ and T_S by $T_S = S_{DL} / S_{DS}$.

The design spectral response acceleration parameters S_{DS} and S_{DL} are derived from the Maximum Considered Earthquake (MCE) at 0.2 s (S_S) and 1.0 s (S_L) respectively. The parameters S_S and S_L are given in the hazard maps or database. These are adjusted by the site coefficients such that S_{MS} = F_aS_S and S_{ML} = F_vS_L, where F_a and F_v are the site coefficients, given in Table 4.4.

Table 4.4: Site coefficients for the **FEMA 450** design spectrum

F _a					
Site Class	Mapped MCE spectral acceleration @ 0.2s ^a				
	S _S ≤ 0.25	S _S = 0.50	S _S = 0.75	S _S = 1.0	S _S ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F ^b	-	-	-	-	-

F _v					
Site Class	Mapped MCE spectral acceleration @ 1.0s ^a				
	S _S ≤ 0.25	S _S = 0.50	S _S = 0.75	S _S = 1.0	S _S ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F ^b	-	-	-	-	-

^aLinear interpolation is used for intermediate values of S_S

^bSite-specific geotechnical investigation and dynamic site response analysis must be performed

Finally, the design response spectral acceleration parameters S_{DS} and S_{MS} are calculated by:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{4.8a}$$

$$S_{DL} = \frac{2}{3} S_{ML} \tag{4.8b}$$

This adjustment arises from the judgement that the conservatism of the actual design of a structure provides, as a minimum, a margin of about 1.5 times the design earthquake ground motions. As such, the appropriate level of design is approximately 2/3 of the maximum considered earthquake. This judgement is discussed in some detail in the Commentary Appendix to the NEHRP Provisions (FEMA 450c) and in Leyendecker *et al.* (2000). The application within a building code of a scaling term derived by expert judgement, tacitly illustrates the difficulty in representing hazard, and across a region that encompasses both high seismic activity at a major plate boundary and low seismicity in a stable continental interior, as well as a transition between the two regimes. An example design spectrum for a site in California, using the NEHRP (2003) Provisions, is shown in Figure 4.6.

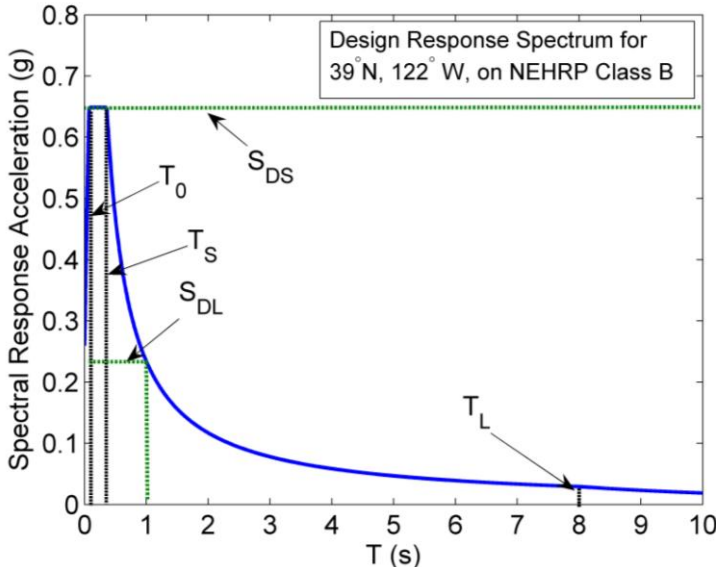


Figure 4.6: Design spectra for 39°N, 122°W using the NEHRP (2003) Provisions

As discussed in section 2.3.1, the 2009 NEHRP *Provisions* the definition of seismic input has changed from maximum considered earthquake motion (MCE) to risk-targeted maximum considered earthquake motion (MCE_R). The same two spectral ordinates are considered as for the **FEMA 450** (0.2 s – short-period; 1.0 s - long-period), albeit the definition of the horizontal seismic action now corresponds to the direction of maximum ground motion. This means that the ground motion inputs in those relevant maps are larger than those for the 2008 USGS National Seismic Hazard Maps, which used rotation-independent geometric mean acceleration to define horizontal motion, in line with the NGA models used in the construction of the maps. The 2009 NEHRP *Provisions* also make an explicit distinction between probabilistically determined seismic action and deterministic seismic action; a distinction that was found only in the commentary to the 2003 *Provisions*. The resulting seismic input is therefore defined as follows:

$$S_S = \min[C_{RS} S_{SUH}, S_{SD}] \quad (4.9a)$$

$$S_I = \min[C_{RI} S_{IUH}, S_{ID}] \quad (4.9b)$$

Where S_S and S_I are as defined for **FEMA 450**, S_{SUH} and S_{IUH} are the mapped uniform hazard (2 % probability of being exceeded in 50 years) for the 5 % damped spectral acceleration for $T = 0.2$ s and 1.0 s respectively. C_{RS} and C_{RI} are the risk-coefficients required to adjust the ground motions to the risk-targeted MCE (1 % probability of building collapse in 50 years). S_{SD} and S_{ID} are the mapped deterministic 5 % damped response spectral acceleration values. The deterministic motions are calculated from the 84th percentile of ground motion for the characteristic earthquake within a seismic source. These deterministic values have lower bounds of 1.5 g and 0.6 g for short and long-period acceleration, respectively, for Site Class B. Separate hazard maps are provided within the code for the six coefficients, as well as for T_L as defined in **FEMA 450**. The construction of the elastic response spectrum for horizontal motion is undertaken in exactly the same manner as for the 2003 *Provisions*.

In addition to the spectral acceleration parameters required to constrain the elastic response spectrum, the new site investigation requirements allow for the use of peak ground acceleration (PGA_M) to identify potential of liquefaction and soil strength loss:

$$PGA_M = F_{PGA} PGA \quad (4.10)$$

Where PGA is the maximum considered geometric mean peak ground acceleration, and F_{PGA} a site coefficient defined as in Table 4.5.

Table 4.5: Site coefficients for the MCE Peak Ground Acceleration (F_{PGA}) (NEHRP, 2009)

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration (g)				
	$PGA \leq 0.1$	$PGA = 0.2$	$PGA = 0.3$	$PGA = 0.4$	$PGA \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Special Investigation Required				

Whilst the definition of the geometric mean is consistent with the PGA hazard presented in the 2008 USGS National Seismic Hazard Maps, the input PGA given in within the 2009 NEHRP *Provisions* differs slightly. This is because the input PGA considered here is identified as the lesser of the uniform hazard (2 % in 50 years) and the deterministic PGA . The latter assumes a value of 180 % of the median ground motions for the characteristic event, and must exceed 0.6 g.

There is no mention of vertical elastic response in either **FEMA 450** or the International Building Code (2009), but detailed provisions for the definition are given in the 2009 NEHRP *Provisions*. The vertical design response spectrum is determined via:

$$0 \leq T \leq 0.025: S_{av} = 0.3C_V S_{DS} \quad (4.11a)$$

$$0.025 < T \leq 0.05: S_{av} = 20C_V S_{DS} (T - 0.025) + 0.3C_V S_{DS} \quad (4.11b)$$

$$0.05 < T \leq 0.15: S_{av} = 0.8C_V S_{DS} \quad (4.11c)$$

$$0.5 < T \leq 0.15: S_{av} = 0.8C_V S_{DS} \left(\frac{0.15}{T} \right)^{0.75} \quad (4.11d)$$

Where C_V is the vertical coefficient of seismic action, given as in Table 4.6, and S_{DS} is as defined previously. Several conditions are included within this definition here, and they are that S_{av} shall not be less than half of the corresponding S_a for horizontal components, and that for periods greater than 2.0 s the site specific procedure must be adopted. If the site specific procedure is adopted then S_{av} cannot be less than half of S_a for periods greater than 2.0 s, or for shorter periods less than 80 % of the values of S_{av} determined using the general procedure (equations 4.7a – 4.7d). This new, and comparatively complex definition of the vertical response spectrum, is based largely on the findings of Borzognia and Campbell (2004).

Table 4.6: C_V for short-period motion (NEHRP, 2009)

MCE _R for Short Periods (S_S)	Site Class A, B	Site Class C	Site Class D, E and F
$S_S \geq 2.0$	0.9	1.3	1.5
$S_S = 1.0$	0.9	1.1	1.3
$S_S = 0.6$	0.9	1.0	1.1
$S_S = 0.3$	0.8	0.8	0.9
$S_S \leq 0.2$	0.7	0.7	0.7

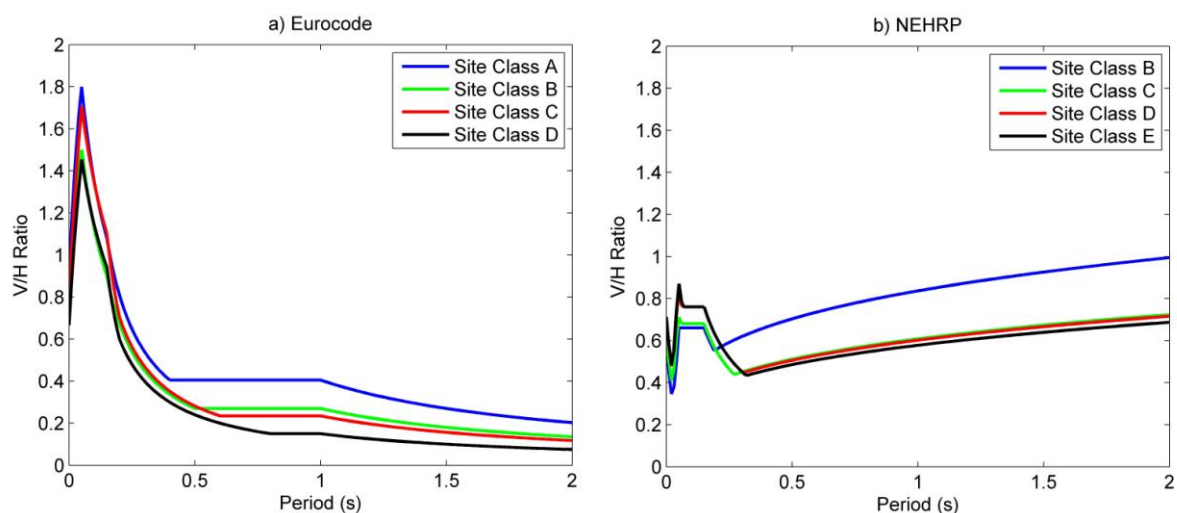


Figure 4.7: Ratio of vertical to horizontal design spectra for different site classes using the EN 1998-1 formulation (Type 1, M_s 6.0) (left) and NEHRP (2009) formulation (right)

An interesting comparison can be made between the ratios of vertical to horizontal design spectra using the EN 1998-1 formulation and the NEHRP (2009) formulation (Figure 4.7a

and 4.7b respectively). These ratios are constructed for a M_w 6.0 ($\approx M_S$ 6.0) event at a Joyner-Boore distance of 10 km, assuming a strike-slip event and using the GMPE of Akkar & Bommer (2010). It is evident that the respective ratios are very different according to the formulations.

4.3.2. NBC (2005)

Whilst many aspects of the 2005 National Building Code of Canada follow a similar approach to the NEHRP/IBC provisions (e.g., site classification, change of return period), the method for creating the design response spectrum is unique. Instead of a clear indication of the constant acceleration, velocity and displacement parts of the spectra (Newmark & Hall, 1973), the **NBC 2005** spectrum is derived from linear interpolation between given ordinates for the 0.2 s [$S_a(0.2)$], 0.5 s [$S_a(0.5)$], 1.0 s [$S_a(1.0)$], 2.0 s [$S_a(2.0)$] periods. Values of pseudo-spectral acceleration with a 2 % probability of being exceeded in 50 years are given in the Appendix to the code for many locations across Canada. Two examples of the response spectra, one for Winnipeg (a comparatively low hazard site) and one for Vancouver (a higher hazard site), are given in Figure 4.8. No explicit consideration is given to vertical acceleration in **NBC 2005**.

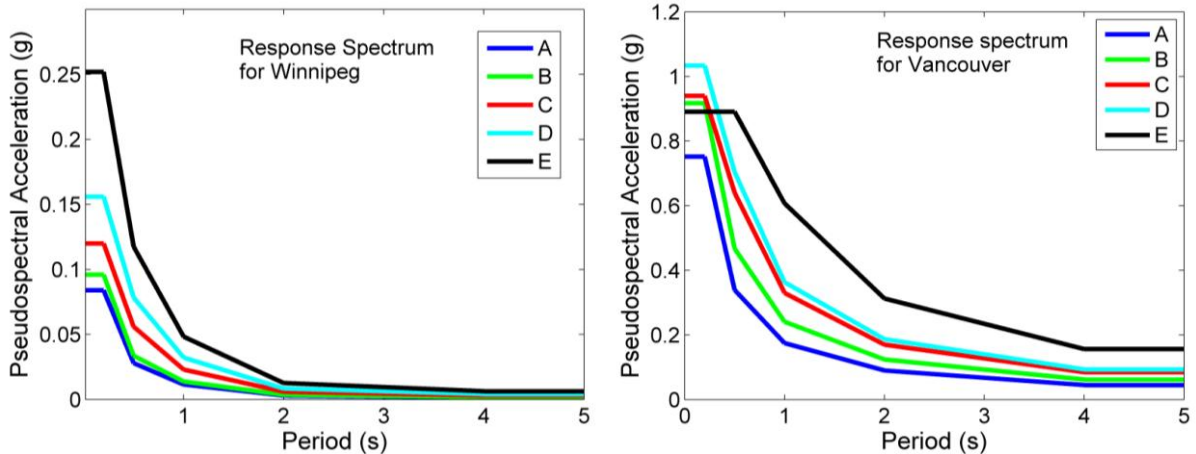


Figure 4.8: Design spectra for an a) low-hazard (Winnipeg, Manitoba) and b) high-hazard (Vancouver, BC) site from NBC 2005.

4.3.3. NZS 1170.5

The **NZS-1170.5** standard takes a different approach to modelling the response spectrum from either the **EN 1998** or NEHRP standards. The return periods for each limit state are given **NZS-1170.0** and are subject to the importance factor assigned to a structure (section 2.4.2). The elastic site spectra ($C(T)$) for horizontal loading are defined in **NZS-1170.5** as follows:

$$C(T) = C_h(T) \cdot Z \cdot R \cdot N(T, D) \quad (4.12)$$

Where $C_h(T)$ is the spectral shape factor, Z the hazard factor, R the return period factor for the ultimate (R_u) or serviceability (R_S) limit state (limited such that $ZR_u \leq 0.7$) and $N(T, D)$ the near-fault factor.

The spectral shape factor clearly defines the shape of the elastic site hazard spectrum. Two different types of spectral shape factors are presented within the code: spectra for the equivalent static method (ESM), and for the model response (MRS) or numerical integration time history (NITH). In the latter, $C_h(T)$ is defined in terms of smooth approximations to the shapes of the estimated hazard spectra for various site classes, whilst in the former the constant acceleration plateau is extended down to $T = 0$, to overcome problems estimating short fundamental periods accurately (NZS-1170.5 Commentary). The “Hazard Factor” Z corresponds to 0.5 times the magnitude-weighted 5 % damped response spectrum at a period of 0.5 s for site class C (shallow soil) that has a return period of 500 years. The return period factor is a simple parameter that represents the scaling required for return period according to limit state, assuming values between 0.2 (for the 20 year return period) to 1.8 (for the 2500 year return period). The near fault factor term is intended to account for the effects of rupture directivity and polarization, which are not included explicitly in the seismic hazard estimates. They apply only to source-site distances (D) less than 20 km and for return periods greater than 250 years. The return period corresponds to the importance of the structure as well as the selected limit state. Further details on these parameters can be found in Appendix C.

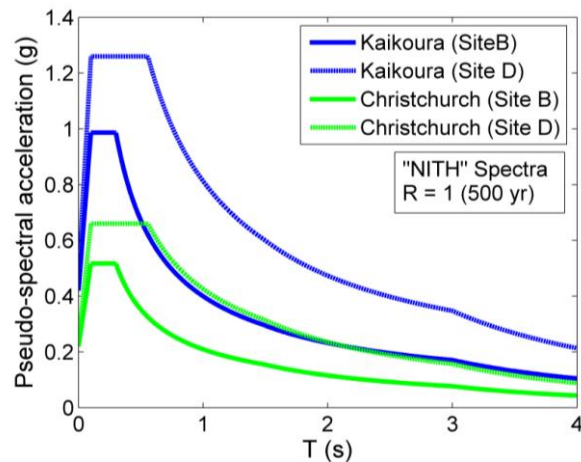


Figure 4.9: Design spectra for Numerical Integration Time History analysis according to NZS1170.5 for a) a low-hazard site (Christchurch) and b) a high-hazard site (Kaikoura).

4.3.4. NNTC-Italy (2008)

The elastic response spectra in the Italian code, whilst similar in underlying design, is constructed in a different manner to that found in EN 1998-1. There are two significant differences in the Italian definition of the response spectrum and they are the use of a location-variable scaling factor for the constant acceleration plateau (F_0), and a location-variable period fixing the constant velocity portion of the spectrum (T_C^*). The ground motion is defined according to the peak ground acceleration (a_g).

In contrast to Eurocode 8, the three parameters T_B , T_C and T_D are not fixed to constant values, but vary in accordance with the expected hazard at a site. The T_B and T_C term are both functions of T_C^* , which indicates the transition from constant acceleration to constant velocity in the response spectrum. The parameter T_C^* is location-dependent, and is supplied in the Appendix to the code alongside a_g and F_0 .

The use of location-dependent parameters to define the shape of the spectrum rather than simply the anchoring point, allows for a more realistic interpretation of seismic hazard. The differences in spectra when hazard is controlled by large earthquakes compared to small earthquakes are much more significant than is approximated by the two model spectra in Eurocode 8. They do, however, require the F_0 and T_C^* parameters to be known for each site. Hence, given with the code is a 211-page electronic appendix displaying the three parameters (including a_g) for 10,751 locations, spaced at approximately 5 km, across Italy (at 9 different return periods). To determine the parameters for a particular location it may be necessary to interpolate between the nearest points for which the parameters are given.

As with **EN 1998**, the **NNTC-Italy (2008)** gives explicit consideration to elastic displacement. The characterisation of spectral displacement is undertaken in the same manner as Eurocode. Both the **NNTC-Italy (2008)** acceleration and displacement response spectra for a site in Perugia, Italy, are shown in Figure 4.10.

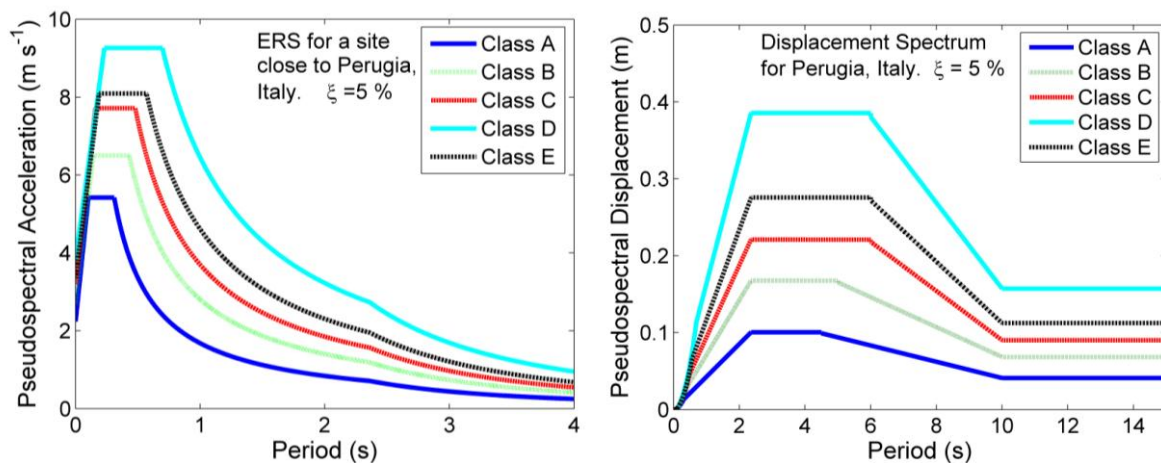


Figure 4.10: Example design acceleration (left) and displacement (right) spectra for Perugia, Italy for different site classes, according to **NNTC-Italy (2008)**

4.3.5. Japan

The complexity of the provisions for earthquake design in the Japanese Building Code means that definition of the elastic response spectrum is dependent on the structure under consideration. As illustrated in section 3.3.4, the definitions of the site condition can vary depending on the structure considered. A broader comparison of the Japanese and European codes can be found in Marino *et al.* (2005). A full discussion of the characterisation of the elastic response spectrum for each structure type is beyond the scope of this report. Descriptions of how the design spectra are calculated for buildings, highways and railways can be found in Appendix C.

4.3.4. Indonesia

The Indonesian Building Code illustrates a comparatively simple approach to developing an elastic response spectrum for a site. Here three spectra are considered for each zone, with each spectrum corresponding to either “hard”, “medium” or “soft” soil. Six zones are defined

for Indonesia with reference peak ground accelerations fixed for each zone. Analysis of the vertical elastic response spectrum is also included in the Indonesian Code. This analysis is intended for structures with elements that are sensitive to vertical loads (e.g., balconies, canopies etc).

4.3.5. Pakistan

The development of the ERS in the Pakistan Seismic Code is similar in concept to those found in the **FEMA 450**, which is to be expected given that the code is adapted from the 1997 Uniform Building Code. The spectrum is fixed by two parameters C_a and C_v , which depend on the zone and site category. The scaling factor of 2.5 is applied for the constant acceleration plateau, which is the same as that defined in Eurocode. Two important differences are evident, however. The first is that the spectrum does not extend as far as the constant displacement plateau. The second is the incorporation of near-fault scaling in the high seismicity zones. The absence of the constant displacement part of the spectrum clearly limits the applicable spectral range. The near-fault factor accounts for the impact of near-field pulse or directivity effects on the response spectrum. This results in a nonlinear scaling of the ERS with distance from the fault.

A plot of the ERS for the Pakistan code is given in Figure 4.11, for a scenario earthquake 10 km from a Type A fault (i.e. one that is capable of producing large magnitude earthquakes and has a high slip rate). The ERS for S_E sites clearly demonstrates the impact of soft soils by amplifying ground motion in the longer period range.

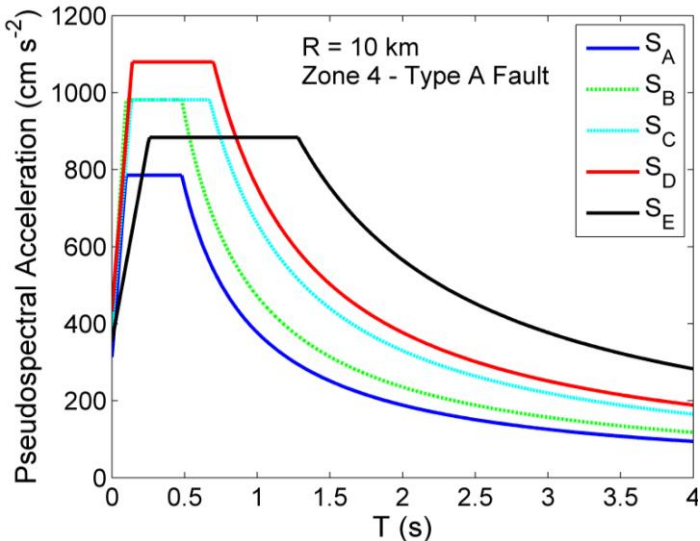


Figure 4.11: Example design spectra according to the Pakistan (2007) code

4.3.6. Comparison of Selected Code Spectra for Italian sites

To illustrate the impact that different approaches to construction of the ERS may have on the overall design spectrum, comparisons of three different approaches are considered here for four site in Italy: L’Aquila, Messina, Naples and Turin. The three code construction approaches compared are **EN 1998-1** (dark blue in Figure 4.12), **NNTC-Italy (2008)** (light green in Figure 4.12) and **FEMA 450** (red in Figure 4.12). For the **FEMA 450** spectra, the

input spectral acceleration values are for the 2 % probability of being exceeded in 50 years (which are then scaled down by a factor of 1.5). For a more appropriate comparison with **EN 1998-1** and **NNTC-Italy (2008)** an adjusted NEHRP method is also considered. This simply anchors the spectrum to the 0.2 s and 1.0 s accelerations for the 475 year return period, with no scaling applied. This variation is referred to as “NEHRP adjusted” (cyan in Figure 4.12). These design spectra are compared to the 475-year uniform hazard spectra (black in Figure 4.12), which is constructed via data from the 2008 Italian Seismic Hazard Map (INGV, 2008). The **EN 1998-1** type 1 spectrum is assumed for all sites except Turin, where both the type 1 and type 2 spectra are shown.

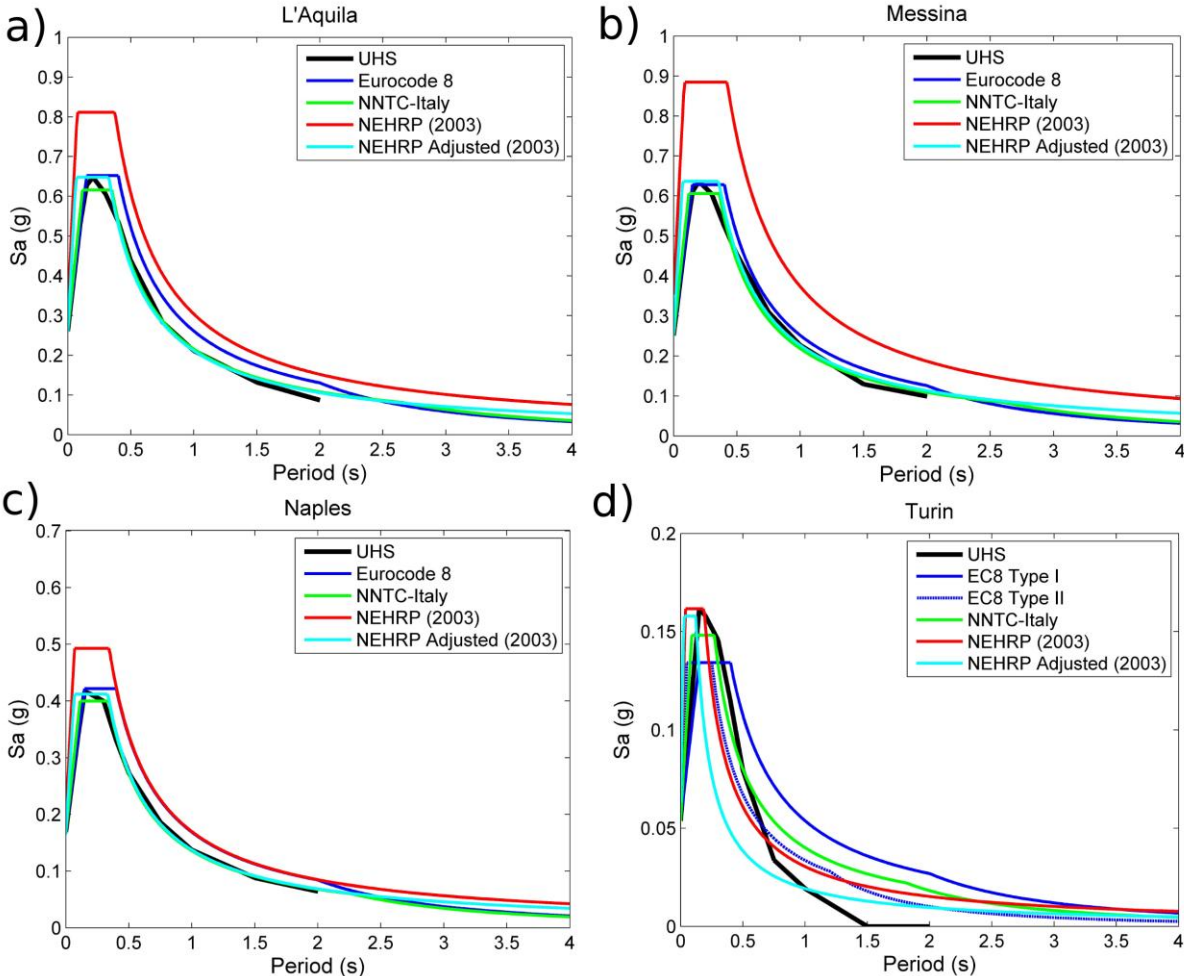


Figure 4.12: Comparison of code-defined elastic design spectra and uniform hazard spectra (475 year return period – shown in black), for four cities in Italy, a) L'Aquila, b) Messina, c) Naples and d) Turin.

For the L'Aquila, Messina and Naples sites, the same overall trend can be seen. Clearly the **FEMA 450** approach results in a design spectrum that is substantially greater across the 0 to 4 s range than the other code spectra. This is not surprising given that it is anchored to spectral accelerations with return periods of 2475 years, as opposed to 475 years for the other codes. The NEHRP adjusted curve is in much closer agreement to the UHS, which indicates that the 2/3 scaling factor in **FEMA 450** would result in a conservative design spectrum if applied to Italy. There is good agreement between the UHS and the design spectra for both the NEHRP

adjusted and the **NNTC-Italy (2008)** curves. The only difference is that the constant acceleration part of the spectrum is higher for the NEHRP adjusted curve than the **NNTC-Italy (2008)** curve. This is a consequence of being anchored to the 0.2 s acceleration, where the peak of the UHS is found in all four examples. This in itself may be a little conservative; hence the acceleration plateau of the **NNTC-Italy (2008)** curve is anchored to accelerations slightly lower than the peak of the UHS. The **EN 1998** type 1 spectrum envelopes the whole UHS (except for Turin), but clearly predicts higher accelerations in the constant velocity part of the spectrum than is found in the UHS. The situation is less clear for the Turin example, which is a low hazard site. The UHS suggests trivial spectral accelerations at periods greater than 1.5 s, but such an assumption would probably be inappropriate for a design spectrum. There is a degree of convergence between the **EN 1998** type 2 spectrum and **NNTC-Italy (2008)** curve for this site, although both slightly underestimate the short-period ($T < 0.5$ s) acceleration and possibly overestimate the long-period acceleration. In general, Figure 4.12 illustrates that greater agreement between the UHS and the design spectrum can be found if the curve is constrained by at least two parameters, rather than using a fixed spectral shape anchored to only one parameter.

4.4. Construction of the Elastic Response Spectrum

4.4.1. Definition of the horizontal ground motion

An important area of seismic hazard practice where there remains inconsistency is in the definition of the horizontal ground motion. Strong-motion is now routinely recorded on a set of triaxial sensors, with one vertical component and two horizontal components of motion. Analysis of the vertical strong-motion is not undertaken as extensively in most design application, although provisions for its use are given in many codes. Of more relevance, however, is the definition of the horizontal ground motion. **EN 1998** simply specifies that “*the horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum*” (**EN 1998-1 3.2.2.1 3(P)**). From the perspective of the seismic hazard analyst, this provides little information as to how the horizontal ground motion should be characterised in the seismic hazard analysis.

Empirical ground motion prediction equations can often characterise the horizontal ground motion in a variety of different ways. For derivation of the empirical models the horizontal ground motion may be determined from the two records in the following ways (Beyer & Bommer, 2006):

- 1) “Average” combination such as geometric (IM_{GM}), arithmetic (IM_{AM}), or vectorial (IM_{VEC}) mean of the two horizontal component spectra
- 2) Horizontal components treated independently (IM_{IND}) or a random component selected (IM_{RND})
- 3) Larger horizontal component for each period (IM_{ENV}) or consideration only of component with largest PGA (IM_{PGA}).
- 4) The ground motion in the most adverse direction for each ordinate (IM_{MAXD})
- 5) The 50th percentile geometric mean ground-motion from all non-redundant angles of rotation for each period ($IM_{GMRotD50}$), or the geometric-mean ground motion from horizontal components rotated to minimise IM_{GMRotD} across all periods ($IM_{GMRotI50}$).

In practice, many of these definitions are found rarely (IM_{PGA}), or have largely been phased out in recent models as more records have been made available (e.g., IM_{IND} , IM_{RND}). Elsewhere, the method chosen can be influenced by region, with regional and local European GMPEs (until recently) often defined in terms of IM_{ENV} . Within the last few years many ground motion models have adopted IM_{GM} as the preferred horizontal ground motion. However, several state-of-the-art models produced by the Next Generation Attenuation project use $IM_{GMRot150}$, although the scaling between this definition and IM_{GM} is generally small.

Where this particular issue has been subject to scrutiny is in the 2009 NEHRP *Provisions*. In the previous provisions (**FEMA 450**) the combination of the two horizontal components had not been defined, assumed implicitly to correspond to the geometric mean of the two components or as defined according to the hazard analysis. This definition has been changed in the 2009 *Provisions* to define the maximum considered motion as that corresponding to the maximum direction of motion (IM_{MAXD}). The latest national seismic hazard maps for the United States, currently give ground motion in terms of $IM_{GMRot150}$, in accordance with the definition in the ground motion models. It has therefore been necessary to implement a scaling factor (1.1 for short-period motion, 1.3 for long-period motion) between the two definitions to scale the values given in the national seismic hazard maps to those needed as input for design. However, as is shown in section 3.3, for the assessment of liquefaction potential the geometric mean PGA is used. This change is understood to have been put in place following judgement by engineers that the maximum direction of motion is more relevant for design using equivalent static methods as it reduces the probability of structural failure. There remains some debate as to whether this conservatism is warranted.

Within the current **EN 1998** provisions there is little consideration given to this debate. It is therefore likely that the horizontal ground motion input is that defined by the hazard analysis, which itself is likely to be controlled via the selected ground motion models. It is anticipated that the geometric mean or rotationally independent geometric mean ($IM_{GMRot150}$) will be preferred. Should the **EN 1998** requirements change, it is likely that scaling factors of the kind suggested in the 2009 NEHRP *Provisions* would be used, although this is largely speculative. There is some imperative, in the goal of hazard harmonisation, that consensus is reached regarding the horizontal component definition, and that this is made clear in future revisions to Eurocode.

4.4.2. Constructing the Spectrum

Whilst identification of the controlling earthquake scenario may present some challenges, an important concern is how to improve the representation of the elastic response spectrum to match more closely the uniform hazard spectrum (UHS). The most direct transition from the **EN 1998** fixed spectral shape approach to a UHS-compatible approach is to allow for the ERS to be defined by more parameters, all of which should be identified during the hazard analysis. To do this, it is necessary to consider an idealised Newmark-Hall response spectrum for both acceleration and displacement. This is shown in Figure 4.13, which follows the nomenclature of the current **EN 1998** response spectrum:

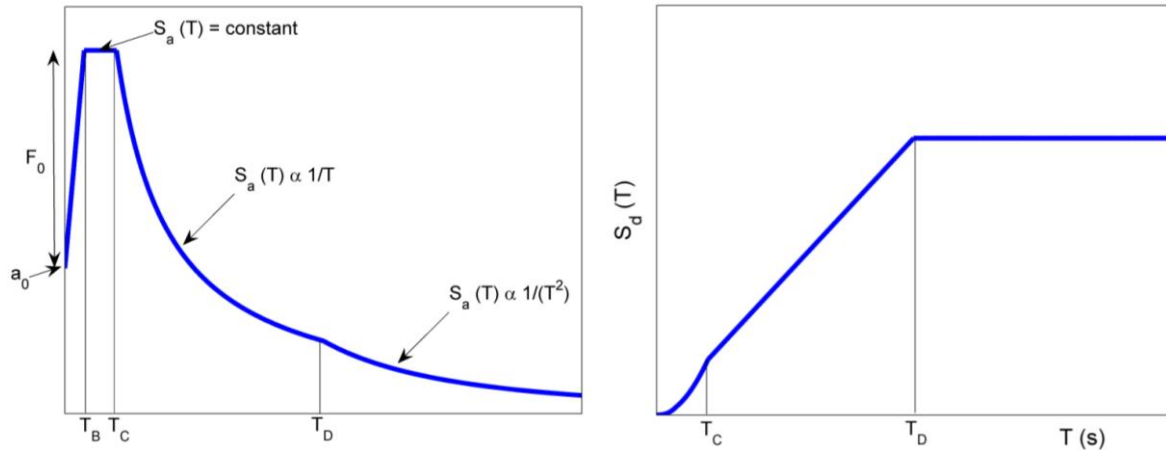


Figure 4.13: Formulation of the ERS for spectral acceleration (left) and displacement (right)

The NEHRP approach of constraining the seismic action via two spectral accelerations (0.2 s and 1.0 s) represents the simplest approach to adapt the spectrum to match a UHS. This approach assumes that the 0.2 s period lies within the constant acceleration part of the spectrum, and that the 1.0 s period lies within the constant velocity part of the spectrum. For application across a region encompassing both high and low seismicity it may be noted that this approach may not always yield satisfactory results (Bommer *et al.*, 2010).

More nuanced approaches to the constraint of parameters in the constant velocity portion of the response spectrum are visible in other codes. In **NNTC-Italy (2008)** the corner period T_C is derived from the hazard calculated at each location, modified by a site class factor. This clearly allows for variation in the breadth of the constant velocity part of the spectrum in accordance with the relative contribution of larger events to seismic hazard. This approach is made possible in Italy by provision of the three parameters in the Appendix to the code. Exactly how T_C^* is determined for each of the 10,751 sites supplied in the Appendix to the Italian code is unclear. Practical considerations emerge that limit the expansion of this approach across larger geographical scales. The PGA, T_C^* and F_0 (of which more consideration will be given later) parameters are given for each location for 9 return periods. Expansion of this across all of Europe, and allowing for scaling to other return periods in accordance with the National Annexes, is computationally difficult and risks compromising transparency to the engineer.

The **NZS 1170.5** and **NBC 2005** codes offer little further insight into this issue. This is because in both codes the spectral shapes are largely fixed according to the site class, only modified by a scaling factor that corresponds to the hazard at a given location. Similarly neither the Japanese, Indonesian or Pakistan codes address this issue.

The adapted site classification scheme suggested by Pitilakis *et al.* (2004), and shown in Table 3.4, also defines the amplification factors for each of the site classes. These amplification factors are determined via nonlinear site response analyses using both recorded acceleration spectra and simulated ground motions. In contrast to **EN 1998-1** the type 1 and type 2 spectral shape are defined according to the PGA on rock, with type 1 for $PGA_{rock} < 0.2$ g and type 2 otherwise. The amplification factors are given in Table 4.7 and example spectra shown in Figure 4.14.

Table 4.7: Amplification and corner period parameters for alternative site classification scheme (Pitilakis *et al.*, 2004)

Soil Category	Proposed Acceleration Response Spectra									
	$a_{gR} < 0.2 \text{ g}$					$a_{gR} \geq 0.2 \text{ g}$				
	T_B	T_C	T_D	S	F_0	T_B	T_C	T_D	S	F_0
A	0.05	0.25	1.2	1.0	2.5	0.15	0.4	2.0	1.0	2.5
B1	0.05	0.25	0.5	2.0	3.0	0.15	0.45	0.8	1.6	3.0
B2	0.05	0.35	0.7	2.0	3.0	0.20	0.55	1.0	1.3	3.0
C1	0.1	0.4	0.8	1.7	2.5	0.25	0.7	2.0	1.1	2.5
C2	0.1	0.5	0.8	2.0	2.5	0.25	0.8	2.0	1.1	2.5
C3	0.1	0.5	1.2	1.4	2.5	0.25	0.9	2.2	1.0	2.5
D1	0.1	0.7	1.2	1.8	2.5	0.25	1.0	2.0	1.2	2.5
D2	0.1	0.7	1.2	1.1	2.5	0.25	1.0	2.0	0.8	2.5
D3	0.1	0.7	1.2	1.1	2.5	0.25	1.2	2.0	1.0	2.5
E	0.05	0.25	0.4	3.0	3.0	0.1	0.4	0.7	2.0	3.0

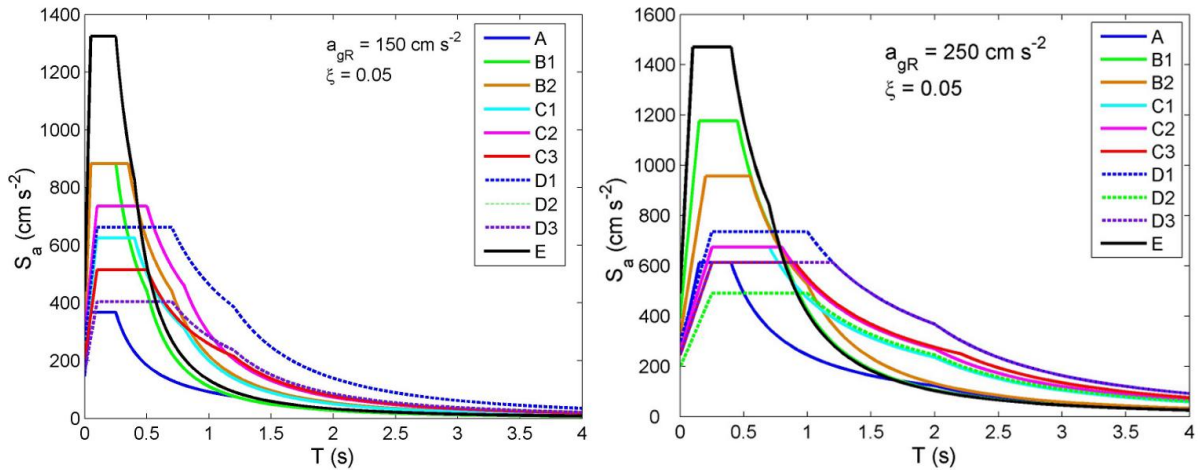


Figure 4.14 Pitilakis *et al.* (2004) design response spectra. a) Type 1, b) Type 2.

It is evident that the corner period T_C may depend on many factors, most of which influence the overall shape of the response spectrum. In particular the relative contribution of longer period motion to the spectrum will clearly influence the shape of the ERS within the constant velocity portion of the spectrum. In the Newmark-Hall formulation the intermediate and long-period motion, T_C is constrained by the peak ground velocity. The relative contribution of long-period to short-period motion may be quantified by the ratio of PGV to PGA. Dimensional analysis indicates that this ratio can be interpreted as the period of an equivalent harmonic wave, thus indicating the significant periods of motion (Kramer, 1996). It has also been shown that the ratio of PGV to PGA scales clearly with magnitude (Figure 4.15), and displays relative stability over distance (Bommer *et al.*, 2010).

Bommer *et al.* (2000) propose a linear scaling of T_C with PGV/PGA:

$$T_C = 5 \left(\frac{PGV}{PGA} \right) \quad (4.13)$$

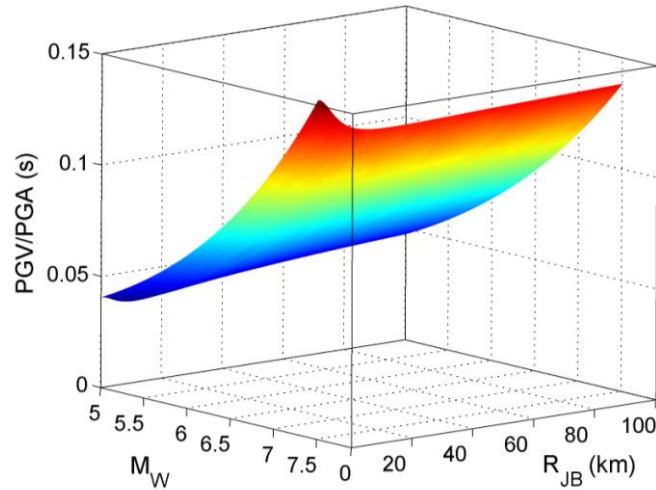


Figure 4.15: Scaling of PGV/PGA with magnitude and Joyner-Boore distance using the ground motion prediction model of Akkar & Bommer (2010)

This is further modified by Bommer *et al.* (2010), who express the relation as a 2nd-order polynomial:

$$T_C = a_1 + a_2 \left(\frac{PGV}{PGA} \right) + a_3 \left(\frac{PGV}{PGA} \right)^2 \quad (4.14)$$

Where a_1 , a_2 and a_3 are constants determined by nonlinear regression, and T_C , PGV and PGA calculated using the models of Akkar & Bommer (2007a,b). The estimation of T_C by this approach is may be dependent on the spectral acceleration (or velocity) model used in the hazard analysis.

An alternative approach to identifying T_C is adopted in HAZUS (FEMA, 2003), and has been implemented elsewhere (e.g., Crowley *et al.*, 2009). This uses the ratio of the 1.0 s (S_{A1}) and 0.2 s (S_{AS}) spectral acceleration as defined in the NEHRP *Provisions*:

$$T_C = \left(\frac{S_{A1}}{S_{AS}} \right) \left(\frac{F_{Vi}}{F_{Ai}} \right) \quad (4.15)$$

where F_{Vi} and F_{Ai} are the site coefficients for soil class i , for the 1.0 s and 0.2 s periods respectively. This approach is largely analogous to the PGV/PGA approach described previously, if it is assumed that direct scaling exists between PGV and the 1.0 s spectral acceleration (an assumption which is in itself questionable) and between PGA and 0.2 s acceleration. The F_{Vi} and F_{Ai} factors introduce a site-dependent modification to this relation that is not accounted for in the PGV/PGA formulation. A comparison of PGV/PGA and T_C , determined via the Akkar & Bommer (2010) GMPE, is shown in Figure 4.16.

A polynomial of the form in equation 4.36 is fit to the whole data set which contains estimates of PGA, PGV and T_C for records between magnitudes $5.0 \leq M_W \leq 7.5$ and distances $10 \leq R_{JB}$ (km) ≤ 100 (spaced at 1 km), determined via Akkar & Bommer (2010). The spread of the data at large PGV/PGA and T_C results from the nonlinearity of the PGV/PGA ratio at short

distances, which is exacerbated at greater magnitudes. These areas are where the models are least well constrained (Bommer *et al.*, 2010).

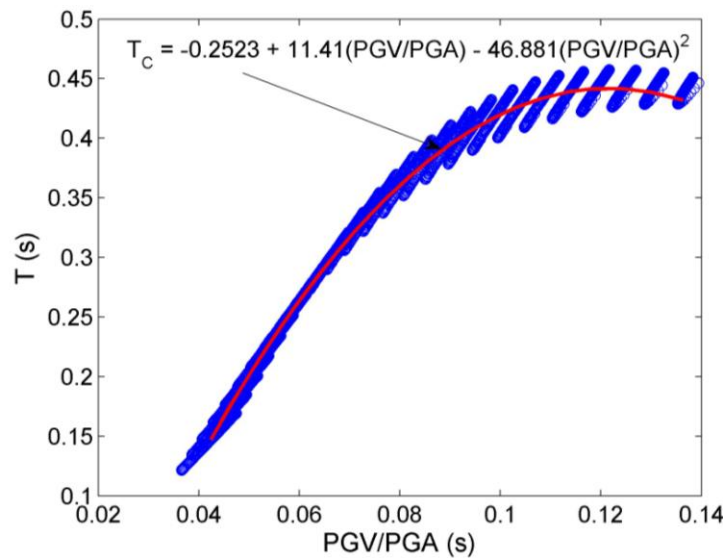


Figure 4.16: Correlation of T_C with PGV/PGA

To draw an analogy between the HAZUS method of estimating T_C and equation 4.14, is to make the assumption that 1.0 s spectral acceleration scales directly with PGV. The veracity of this assumption has been tested by Bommer & Alarcón (2006) who found that the 0.5 s acceleration provides an improvement in fit. The application of any scaling factor to relate PGV to the response spectrum may also be invalid under very soft soil conditions or other extreme conditions that may lead to unusual spectra. The relative stability (or insensitivity to earthquake magnitude) of the 0.5 s period spectral acceleration as a proxy for PGV can be illustrated using the Akkar & Bommer (2010) models, shown in Figure 4.17.

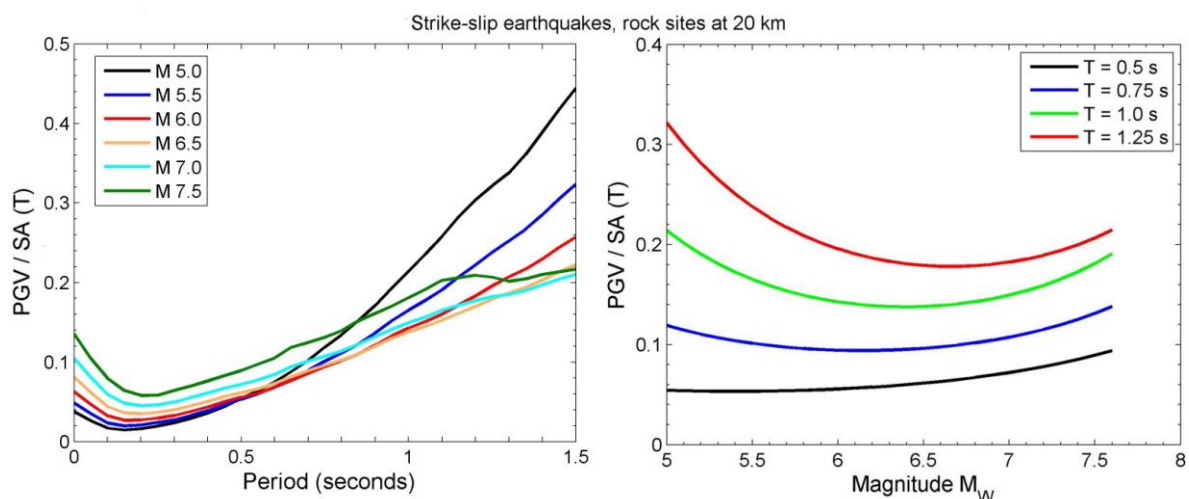


Figure 4.17: Ratios of PGV to Spectral Acceleration using the Akkar & Bommer (2010) GMPE, assuming a strike-slip earthquake at a Joyner-Boore distance of 20 km. a) variation with period for fixed magnitudes, b) variation with magnitude for fixed periods.

The above discussion has shown that whilst there may be some variability in the identification of T_C using PGV/PGA, this parameter does scale effectively with magnitude and is largely robust to the source-site distance, as demonstrated in Figure 4.15. Adoption of this method would require the production of hazard maps using PGV as a parameter. Further testing of this approach and comparison with observed strong-motion records and with other ground-motion prediction models is necessary before this should be implemented in Eurocode.

In addition to the identification of T_C further consideration should also be given to the decay of the elastic response spectrum within the constant velocity range. The decay of the spectrum in this range is determined via the parameter α , where:

$$T_C \leq T(s) \leq T_D: S_a(T) = a_g \cdot \eta \cdot 2.5 \cdot \left(\frac{T_C}{T}\right)^\alpha \tag{4.16}$$

In EN1998-1 α is assumed to be equal to unity, but the influence of this parameter is seen in Figure 4.18

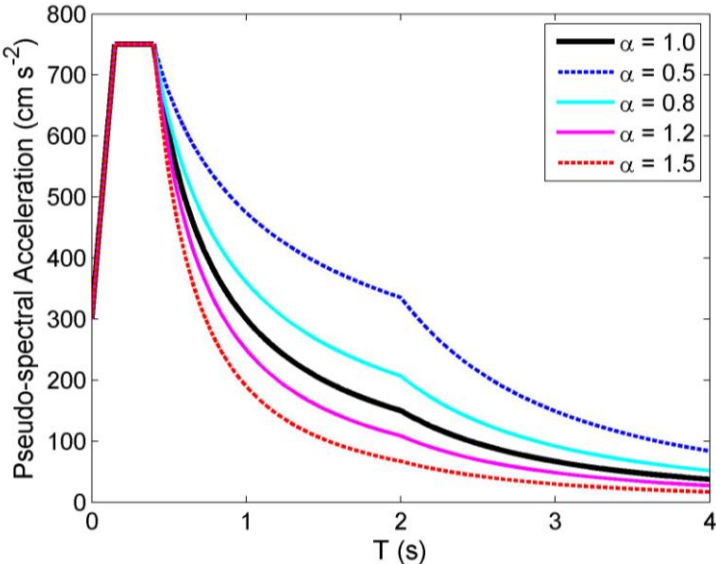


Figure 4.18: Influence of decay parameter (α) on the EN 1998-1 elastic response spectrum

The designation of $\alpha = 1$ is common in many codes (**EN-1998**, **FEMA 450**, **NNTC-Italy**). Notable exceptions are the **NZS 1170.5**, where it is equal to 0.5, and some conditions of the Japanese code, where it ranges from 2/3 to 5/3. A degree of conservatism in the decay of this branch of the spectrum is warranted. Estimation of the fundamental period of oscillation of mid-rise buildings may be subject to error, and static analyses may not take into account the increase in fundamental period as inelastic deformation occurs. These decisions were taken into account in the development of the current **EN-1998** formulation.

A third parameter T_D is needed to constrain the constant displacement part of the spectrum. This parameter characterises the strength of ground motion at long periods, and is therefore a challenge to adequately characterise. It is clear from the selection of codes presented here that constraint of the constant displacement spectrum is not addressed in all definitions of seismic action. **EN 1998** presents T_D as a fixed parameter that is dependent only on the

magnitude of the controlling earthquake scenario ($T_D = 2.0$ s for $M_S \geq 5.5$, 1.2 s otherwise). In NNTC-Italy (2008), T_D scales linearly with PGA up to 4 s, whilst in **NZS1170.5** it is fixed at 3.0 s for all scenarios and site classes. The **FEMA 450** provides a more flexible approach to T_D that is dependent on the controlling earthquake scenario. However, the actual values of T_D are taken from maps presented in the provisions, rather than via direct determination of the controlling scenario. The values of T_D given in the **FEMA 450** are related to the magnitude of the controlling scenario by a two-step procedure using seismic source theory followed by examination of the corner period of accelerograms from moderate and large magnitude earthquakes. This analysis resulted in the following formulation:

$$T_L \approx 10^{(0.3M_w - 1.25)} \quad (4.17)$$

T_L is equivalent to T_D for the purposes of this analysis, but we retain the separate nomenclature to reflect the derivation of this parameter by the means suggested in **FEMA 450**. Equation 4.17 is an approximation that represented the best fit between the corner periods expected from source theory and those on the observed accelerograms. To arrive at the values of T_L given in the code the controlling earthquake magnitude was taken as the modal magnitude identified by disaggregation of the 2 s spectral acceleration with a 2 % probability of being exceeded in 50 years. Table 4.8 was used to relate T_L to the modal magnitude bin, albeit adjustments were made in certain areas.

*Table 4.8: Definition of T_L in the **FEMA 450***

M	T_L	T_L Range (s) (equation 4.17)	T_D Range (s) (equation 4.18)
6.0 – 6.5	4	3.5 – 5	3.1 – 5.6
6.5 – 7.0	6	5 – 7.1	5.6 – 10
7.0 – 7.5	8	7.1 – 10	10 – 17.8
7.5 – 8.0	12	10 – 14.1	17.8 – 31.6
8.0 – 8.5	16	14.1 – 20	31.6 – 56.2
≥ 8.5	20	≥ 20	≥ 56.2

The HAZUS documentation (FEMA, 2003) outlines a slightly different relationship between controlling earthquake scenario and T_D :

$$T_D = 10^{\left(\frac{(M_w - 5)}{2}\right)} \quad (4.18)$$

As is seen in Table 4.8, however, this formulation produces much larger corner periods for large magnitude earthquakes. The remaining codes considered here do not define a constant displacement part of the spectrum and hence no upper limit on displacement can be inferred from the elastic response spectrum.

The existing **EN 1998** requirement that controlling earthquakes be defined for selection of the ERS type means that either equation 4.17 or 4.18 provides a feasible estimate of T_D for little additional cost, although estimates of T_D using equation 4.18 would appear to be unrealistic for larger events. However, the method adopted in the **FEMA 450** would imply that disaggregation using PGA or a short-period acceleration would not be sufficient to determine

T_D from the controlling earthquakes. It is therefore necessary to perform a disaggregation at a longer spectral period. The exact choice of period may still be a matter of some debate. The current fixed value of T_D in the **EN 1998-1** type 1 spectrum is 2.0 s, a choice that does not appear to be well-supported, even by the ground motion prediction models available at the time of its original formulation. This value may appear to be the most obvious choice given the typical maximum magnitudes across Europe. Some reconsideration may be needed in the Hellenic Arc where the maximum earthquake magnitude may be in excess of M_W 8. It is also likely that some adjustment of T_D may be necessary in certain regions, which should be taken into account in future zonation maps.

An alternative formulation of T_D , and one that is closer to the displacement-based design philosophy that is already accommodated to some extent in the **EN 1998-1** provisions, is that implemented by Faccioli and Villani (2009). Using displacement spectra as a means of defining seismic hazard, via the GMPE of Cauzzi & Faccioli (2008), it is demonstrated that the hazard spectrum can be defined via a bilinear representation:

$$0 < T \leq T_D: S_d(T) = \frac{D_{10}}{T_D} T \quad (4.19a)$$

$$T > T_D: S_d(T) = D_{10} \quad (4.19b)$$

Where D_{10} is the spectral displacement at a period of 10 s, and the corner period T_D is defined according to:

$$T_D = \frac{2\pi D_{10}}{\max_T S_v(T)} \quad (4.20)$$

Where $S_v(T)$ is the pseudo-velocity response spectrum, which may be estimated from the displacement response spectrum via equation 4.1a. The comparative fit of the bilinear approximation to the actual displacement hazard spectrum for locations across Italy is demonstrated by Faccioli and Villani (2009), and it is shown to be good estimator of the corner period T_D . It should be emphasised, however, that bilinear approximation of the displacement response spectrum cannot, in turn, be used to estimate pseudo-spectral acceleration at short periods. Instead, it may be more appropriate to extend the bilinear approach only as far as T_C , thus ensuring that short-period motion is constrained by the acceleration spectrum.

The situation may also be more complicated in regions where two or more contrasting scenarios influence the UHS. An example of this may be a region where a location may be affected by moderate earthquakes in the nearby locality, and larger, possibly offshore, earthquakes at greater distance. This circumstance is accounted for in the present Eurocode, and it is likely that separate response spectra may need to be defined from which different values of T_D are estimated. The approach by Faccioli and Villani (2009), may accommodate such circumstances, using as it does, a long-period spectral displacement (D_{10}) to constrain the longer period part of the design spectrum.

An additional parameter that requires consideration is the scaling factor that controls the value of the constant acceleration plateau relative to the PGA, which is referred to here nominally as F_0 . This parameter is fixed at 2.5 in **EN 1998**, a value that is also implicit in the *NEHRP Provisions*. A similar value is defined for the Japanese Building Code as well as the

Indonesian and Pakistan codes. PGA itself is not used in the **NBC-2005** code as the constant acceleration is extrapolated to $T = 0$ s. The scaling factor is dependent on site class in **NZS1170.5**, varying between about 2.5 and 3.0. The **NNTC-Italy (2008)** is the only code to allow F_0 to be entirely dependent on the seismic hazard for a given location as a free parameter. The values of F_0 across Italy seem to vary between 2.3 and 3, and are derived from the Uniform Hazard Spectra (INGV, 2008). This can be seen in Figure 4.19, which considers the ratio of the 1) peak of the UHS to PGA, 2) the second highest S_a to PGA and 3) the third highest S_a to PGA.

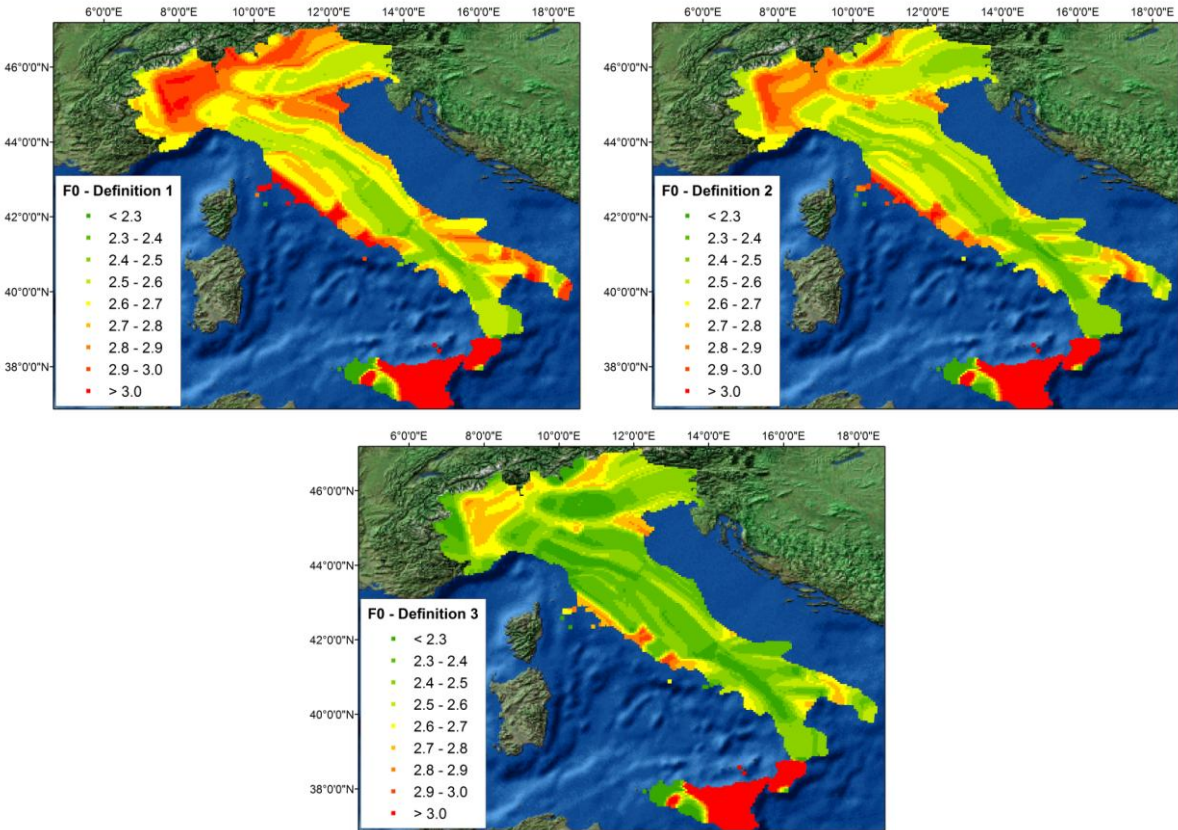


Figure 4.19: Ratio of largest S_a to PGA from Italian UHS. a) Peak S_a/PGA , b) 2nd largest S_a/PGA and c) 3rd largest S_a/PGA

The value commonly ascribed to F_0 of 2.5 emerges as an approximation from both predictive ground motion models and uniform hazard spectra that are developed from them. The actual value of F_0 varies with both magnitude and distance in both the models. This can be seen in Figure 4.20, which compares the value of F_0 as derived using 50th percentile PGA and spectral acceleration from Akkar & Bommer (2007a,b) and Boore & Atkinson (2008). It should be recognised that the design spectrum shouldn't necessary fully envelop the hazard spectrum. It is therefore of little benefit to fix F_0 according to the very peak of the UHS. The ratio of 2nd or 3rd largest S_a to PGA (Figures 4.19b and 4.19c respectively) is perhaps more akin to the likely approach for implementation in design codes. From these maps it is clear that the deviation of F_0 from the fixed value 2.5 is much less than if the peak spectral acceleration is used. Nevertheless, the geographic variability of this parameter is quite evident and clear contrasts can be seen for lower hazard (e.g., northeast Italy) and higher hazard (e.g., central Italy)

regions. These contrasts will likely be even greater on a pan-European scale, so reconsideration of F_0 may still be necessary in future codes.

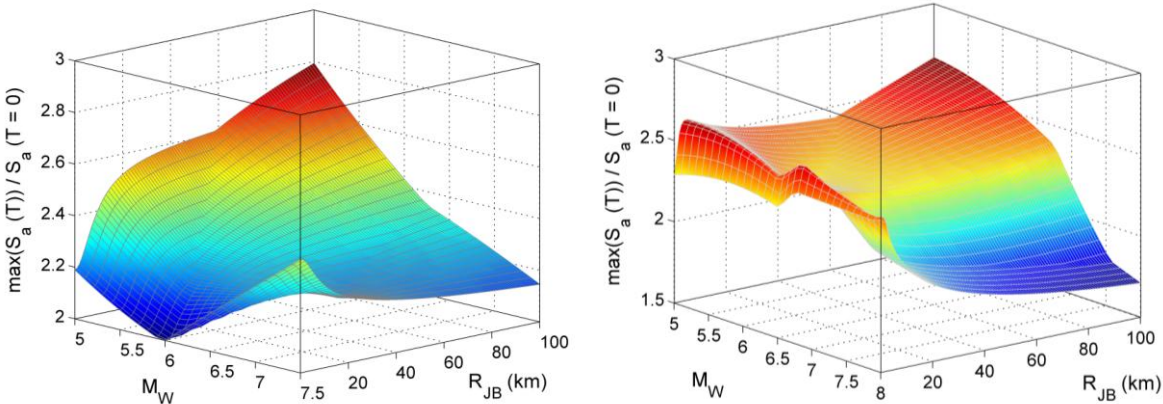


Figure 4.20: Ratio of peak $S_a(T)$ to PGA according to Akkar & Bommer (2007b) [left] and Boore & Atkinson (2008) [right].

This comparison illustrates that the actual value of F_0 is highly dependent on the GMPE used, but shows little coherent magnitude or distance correlation. Whilst both these models support the selection of 2.5 as a representative median value, it is clear that variability in this term exists. Similarly, a plot of the spatial distribution of F_0 across California and Nevada (from the USGS 1996 Hazard Maps) is shown in Figure 4.21. This too demonstrates the spatial variation in F_0 which deviates substantially from the value of 2.5 implied in the NEHRP Provisions.

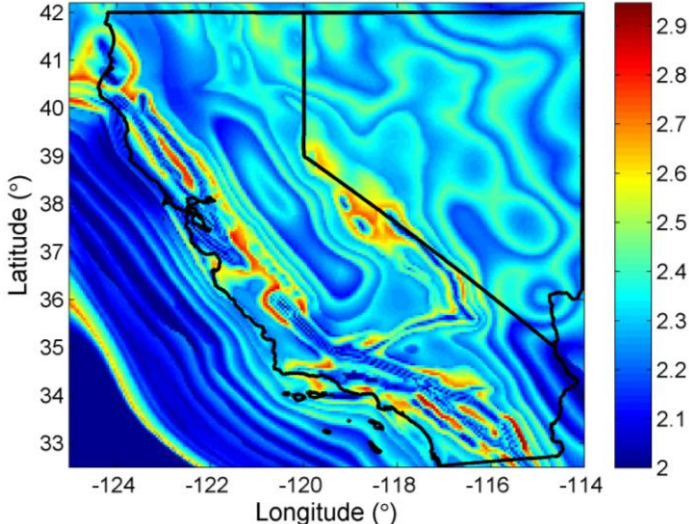


Figure 4.21: Amplification factor F_0 for California (data from USGS National Seismic Hazard Maps, 2008)

The analysis presented here clearly indicates that constraint of the elastic response spectrum requires some reconsideration of parameters that have been fixed in Eurocode, and in many other codes. An important objective of the SHARE project is to provide guidance and data to

allow for both the implementation of the current Eurocode regulations, but also for revisions to regulation that may occur in the short- to medium-term future. A balance must be struck, however, between the definition of the ERS that seismologists may consider most desirable, and that which is feasible to implement in the near term.

For a homogenised zonation of hazard across Europe, it will be necessary to produce maps of the key parameters required for design. In the current implementation of Eurocode a homogenous map of PGA in addition to the magnitude (M_S) of the controlling earthquake scenario would be an important step towards achieving this objective. Revisions to the definition of the ERS will almost certainly require the wide distribution of spatial analyses of other ground motions metrics that can be directly related to many of the parameters (T_C , T_D , F_0 and α) addressed here.

The adaptation proposed by Bommer *et al.* (2010) offers one possible route to producing UHS compatible response spectra, whilst maintaining dependence only on parameters that can be readily incorporated into current hazard analysis techniques. In their formulation, three control periods (T_B , T_C and T_D) are defined via:

$$T_B = 0.2T_C \quad (4.21a)$$

$$T_C = a_1 + a_2 \left(\frac{PGV}{PGA} \right) + a_3 \left(\frac{PGV}{PGA} \right)^2 \quad (4.21b)$$

$$T_D = 10^{\left(\frac{M-5}{2} \right)} \quad (4.21c)$$

Where M is the magnitude of the controlling earthquake scenario, and a_1 , a_2 and a_3 are constants. The calculation $S_a(T)$ can be performed using the current **EN 1998-1** formulation, with $F_0 = 2.5$ and $\alpha = 1$. An alternative formulation is also suggested for the case of $\alpha \neq 1$. Instead α is determined via:

$$\alpha = b_1 + b_2 \left(\frac{PGV}{PGA} \right) + b_3 \left(\frac{PGV}{PGA} \right)^2 \quad (4.22)$$

As with parameters a_1 , a_2 and a_3 , the parameters b_1 , b_2 and b_3 can be determined from regression. It remains an open question as to how these parameters should be determined in a seismic code. It may be the case that they can be fixed at given values in the code, most likely subject to alteration within National Annexes. The application shown in Bommer *et al.* (2010) determines these parameters from the empirical GMPEs given in Akkar & Bommer (2007a, b), which has subsequently been revisited using the Akkar & Bommer (2010) GMPEs (Bommer, 2011). It is therefore possible that use of different GMPEs, as they emerge, may give rise to different values of the parameters. Bommer *et al.* (2010) indicate that the full formulation needs to be tested against the UHS, rather than just scenarios or the GMPEs present in their paper. Also of importance in determining the parameters $a_{1,2,3}$ and $b_{1,2,3}$ is the impact of the site class. As noted in Bommer *et al.* (2010), in the current Eurocode formulation the corner periods are determined by site class, whereas in their paper they only consider the rock site case.

The use of the alternative formulation of the acceleration response spectrum will obviously affect the representation of the displacement spectrum. In particular, if α differs significantly from 1 then the displacement spectrum may no longer be approximated as a bilinear shape.

Furthermore, the constant displacement plateau must be determined by an alternative means from that given in the present **EN 1998-1**. The modification is also suggested by Bommer *et al.* (2010):

$$S_e(T) = a_g \cdot \eta \cdot F_0 \cdot \left(\frac{T_C^\alpha T_D^{2-\alpha}}{T^2} \right) \quad (4.23)$$

Where F_0 is modified from 2.5 (as in the present **EN 1998**) to 2.265. A further suggestion is given regarding modification of the damping factor, which is addressed in section 5.4.

Whilst there is clearly further investigation of this method required before even provisional adoption within a seismic design code, the method of Bommer *et al.* (2010) does have clear advantages. In particular it can constrain the response spectrum on the basis of only a few parameters, most of which can be readily implemented in a seismic hazard analysis. For any given site, or selection of sites, PGA and PGV must be determined in addition to the magnitude of the controlling earthquake scenario. This requires only one additional parameter (PGV), to be constrained, providing that the aforementioned constants are known *a priori*.

A more fundamental question, from the perspective of seismic design, is whether the aim of providing a closer match between the uniform hazard spectrum and the code design spectrum should be a primary motivation? As will be addressed in due course, the UHS is not representative of a spectrum from a single event (or even likely events) but represents a range of spectral accelerations (or displacements) with an equal probability of being exceeded within a given time period. For the selection and scaling of time histories for dynamic analysis it is important to be able to define the hazard at the fundamental period of the structure under consideration, for which the UHS is invaluable (albeit the UHS itself is not a good target spectrum for matching the time histories). However, where analysis is undertaken using static analysis or modal dynamic analysis, as is more common for ordinary structures, there is perhaps a need for design spectra that may correspond more closely to, or at the very least a more consistent with, the spectra of scenario events rather than the UHS. Furthermore, in regions of low-to-moderate seismicity, the hazard at long periods may prove harder to constrain because of the paucity of quality strong-motion records for smaller events. This may result in trivial long-period hazard (such as the Turin example in Figure 4.12d), which may not be appropriate for design, particularly if large earthquakes exist in the historical record. In such circumstances scenario spectra may be more appropriate as a basis for design than the UHS.

Identification of the controlling scenario may prove a more challenging task, as discussed in section 5.2. This is largely because the scenario earthquake is needed to constrain the period of initiation of the constant displacement part of the spectrum. Disaggregation of PGA, as is implied presently in **EN 1998**, would likely identify controlling scenarios that are inconsistent with the UHS in the longer period part of the spectrum. The adoption of the HAZUS relation for estimating T_D from the controlling earthquake magnitude is more consistent with the NEHRP approach than the current **EN 1998** approach. In **FEMA 450** the controlling earthquake is estimated by disaggregation of the 2 s spectral acceleration at the given return period (2475 years), except where it is modified by expert judgement. Whilst the 2 s period may be debatable in Europe, the notion that T_D should be constrained by a scenario earthquake that relates to the longer period part of the spectrum is less so. It is suggested, therefore, that zonation maps be presented for the controlling earthquake scenario, in the same manner as that of the NEHRP *Provisions*, rather than devolving the decision to the end-user.

5. Inputs & Considerations for Seismic Design

5.1. Seismic Zoning

5.1.1. Seismic Zones in EN 1998

Contrary to the approach that is becoming widely adopted in other building codes (**NEHRP, 2003; IBC 2009; NBC, 2005; NNTC-Italy, 2008**), **EN 1998-1** requires the definition of seismic zones. This is outlined in **EN 1998-1 3.2.1 (1)P**: “*For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant*”. This clearly indicates that the final decision on the zonation of a national territory resides with the respective National Authorities. Further advice is therefore given in the respective National Annexes. An additional provision indicating the variable to be mapped is given in **EN 1998-1 3.2.1 (2)P**: “*For most of the applications of EN-1998, the hazard is described in terms of a single parameter, i.e. the reference peak ground acceleration on type A ground, a_{gR} . Additional parameters required for specific types of structures are given in the relevant parts of EN 1998*”. The reference ground motion a_{gR} corresponds to the reference return period of seismic action for the no-collapse requirement (T_{NCR}) (**EN 1998-1 3.2.1 (3)**). This is assigned an importance factor γ_I of 1.0.

Two further provisions are added for the cases of low and very low seismicity. In the case of the former “*reduced or simplified seismic design procedures for certain types of categories of structures may be used (EN 1998-1 3.2.1 (4))*”. In the case of the latter “*the provisions of EN 1998 need not be observed*”. The determination of whether a region is low or very low seismicity may be found in the National Annex. A classification scheme is recommended, but may be subject to alteration by National Standards authorities. A low seismicity region is one where a_{gR} is not greater than 0.08 g (0.78 m s⁻²) or where $a_{gR} \cdot S$ is not greater than 0.1 g (0.98 m s⁻²). A very low seismicity region is suggested to be one where a_{gR} is not greater than 0.04 g (0.39 m s⁻²) or where $a_{gR} \cdot S$ is not greater than 0.05 g (0.49 m s⁻²).

One objective of the SHARE project is to define a reference zonation for the whole of Europe for the application of Eurocode 8. This will be implemented following the regional seismic hazard analysis, and requires little further discussion here. A detailed analysis of the current zonations for each country is given in Solomos *et al.* (2008).

5.1.2. Seismic Zones in other codes

The use of zones to define regions of uniform seismic action is a factor that distinguishes the practice of many different codes. It also raises the issue of the role that hazard maps play when providing information necessary for seismic design. Seismic zones of the nature described in **EN 1998-1 3.2.1** are not found in the International Building Code, **FEMA 450**, the National Building Code of Canada or the Italian Seismic Code. In each of these cases, however, designers can determine the seismic action at any site of known latitude, longitude and site condition. For the IBC/NEHRP provisions, the exact value of the seismic action can be calculated via free software supplied by the United States Geological Survey (<http://earthquakes.usgs.gov/hazards>). A similar web-based application is supplied by Natural Resources Canada, for use with the National Building Code of Canada (<http://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-eng.php>). A web-based

application is also available to derive seismic hazard parameters for **NNTC-Italy (2008)** (<http://esse1.mi.ingv.it>). However, this Web-GIS application only plots the hazard values for the points given in the code itself. Guidance on interpolation of hazard at a site from its nearest points is given in the Appendix to the **NNTC-Italy (2008)**.

The approach to hazard mapping in the New Zealand code (**NZS 1170.5**) represents something of a compromise between a zoned and non-zoned approach. The level of ground motion is defined according to the hazard factor *Z*. This value is given explicitly in the code for 129 major towns and cities in New Zealand. Elsewhere the hazard factor *Z* should be interpolated from the contours of maps of *Z* supplied in the code. In effect, the *Z* value is detached from the actual seismic hazard at a site in two ways. Firstly, to produce a contour map, hazard calculated at specific sites is interpolated and smoothed. Interpolating between contours also assumes that hazard scales in an approximately linear manner between them, in the absence of specification of the interpolation method. A similar approach had also been implemented in the Costa Rica (1986) seismic code, which also used interpolation between contours of acceleration to determine the input ground motion (although the more recent Costa Rica (2002) code now implements seismic zones).

There has been a steady transition from the use of seismic zones towards explicit seismic hazard parameters. Many of the standards developed between the publication of the 1997 Uniform Building Code (1997) and the NEHRP 2003 *Provisions* demonstrate this trend. The latest update of the Mexican Code (MOC, 2008) is a good example of such a transition. The previous publication of the code (MOC, 2003) had designated four seismic zones for Mexico (Tena-Colunga, 1999). The latest update provides the hazard values explicitly, in addition to the identification of known earthquake sources and their maximum credible earthquake scenarios (Tena-Colunga, 2009). This approach bears a much closer resemblance to the **FEMA 450**. The Japanese code and the Indonesian code both use seismic zones, however, as a basis for seismic design. For Japan, the zones and their respective seismic coefficients are shown in Figure 5.1a, and for Indonesia in Figure 5.1b. For the Japanese code, the seismic coefficients for each zone assume a number between 0.7 and 1. These coefficients are intended to modify the elastic response spectrum to account for changes in seismic hazard across the country. In Indonesia, each zone is assigned a reference PGA (on hard rock), which is used to anchor the elastic response spectrum.

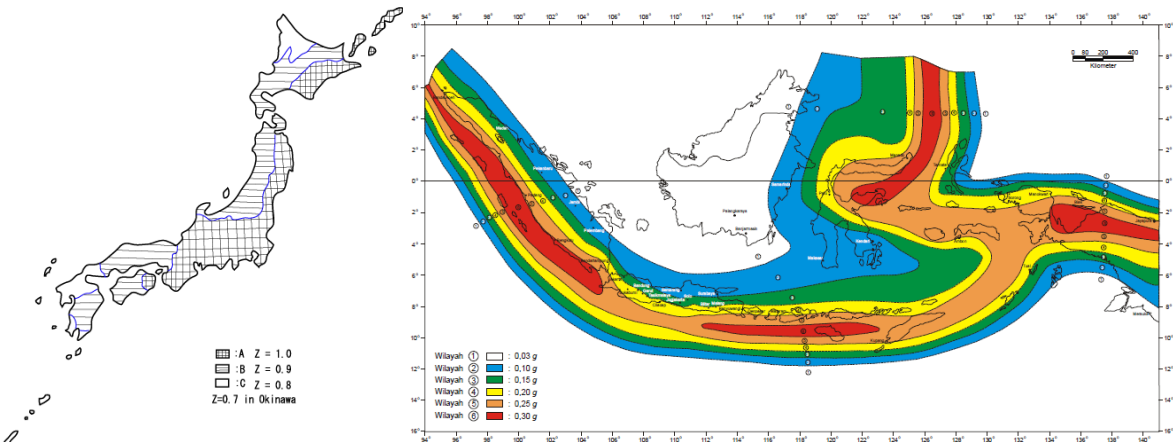


Figure 5.1: Seismic zones defined in the a) Japan (2000) and b) Indonesia (2002) codes

5.1.3. The use of seismic zones in building codes

The assignment of seismic zones across a region has serious implications for the definition of seismic hazard for design purposes. Systematic errors and bias are introduced into the hazard analysis, which as engineering requirements become better defined, will have a detrimental effect on the information needed for seismic design. The most obvious bias arises from the development of zones on the basis of hazard maps, or more specifically hazard contours. Whilst there are many important applications of hazard maps, particularly in the context of GIS-based hazard and risk scenarios, many assumptions are made as to the nature of the information represented. The translation of seismic hazard contours into zones requires the attribution of a level of seismic load that is uniform between the respective contours. This will, of course, increase the hazard estimate in some areas and reduce it in others.

There are circumstances when this designation of a single hazard value across a region may be appropriate and that is when representing hazard in terms of macroseismic intensity. A discussion of the merits and shortcomings of using macroseismic intensity is beyond the scope of this research as it is not considered as a hazard parameter in Eurocode and is of no use as an input for engineering design. It is evident from some older codes, however, that macroseismic intensity played a significant role in delineating the seismic zones (Garcia-Mayordomo *et al.*, 2004; Solomos *et al.* 2008).

The process of interpolating and contouring seismic hazard, followed by the arbitrary assignment of a fixed level of ground motion, begins to disconnect the design seismic load from the hazard input. In this respect the **EN 1998-1** definition of seismic action is internally inconsistent. In clause **3.2.1 1(P)** “*seismic hazard within each zone is assumed to be constant*”. The attribution of a single hazard value for a spatial region is not easily reconcilable with the controlling earthquake scenario needed to determine whether the Type 1 or Type 2 spectrum is applied at a specific location within the region (**EN 1998-1 3.2.2.2 (2)P – NOTE**). The controlling scenario determined for a site via disaggregation is no longer related to the design seismic load at a site, given that a_{gR} has been altered by the zonation.

Seismic zonation also impacts the flexibility of the hazard analysis to consider different return periods of ground motion. The change in design ground motion when moving to shorter or longer return periods is not spatially uniform. This is clearly demonstrated by Leyendecker *et al.* (2000), when considering the transition from the 475 to 2475 year return period (Figure 5.2). The scaling of hazard is different for high and low hazard sites. A consequence of this for performance-based seismic design is that, as different return periods may be used for the different damage states, to ensure the same level of seismic protection, a different zonation will be needed for each return period.

Furthermore, the devolution of both the practice of zonation and the designation of the “low seismicity” and “very low” seismicity criteria into the National Annexes introduces additional complication. Different zonations produced by each country may result in changes in seismic design load across political boundaries. This can result in contrasting design standards over small geographical areas due to political boundaries.

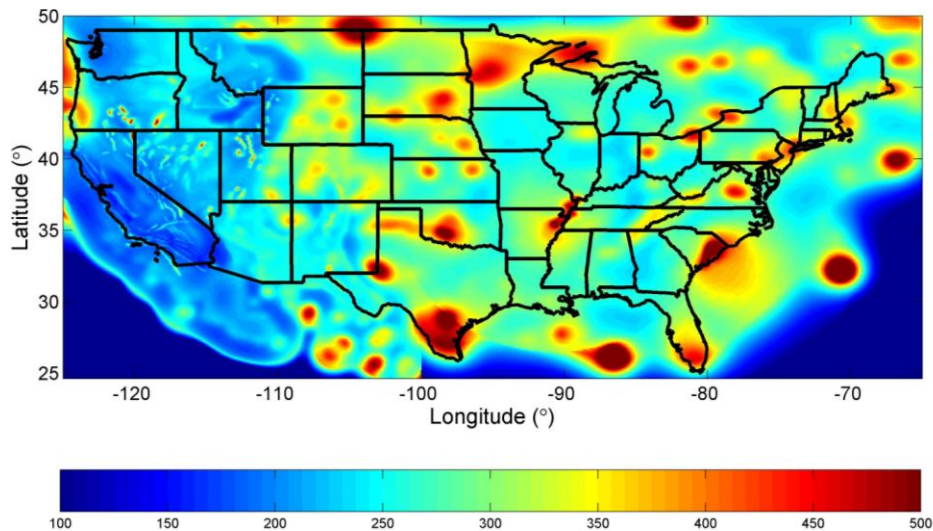


Figure 5.2: Percentage increase in ground acceleration from 475 year to 2475 year return period for the United States (data from USGS (2008) National Seismic Hazard Map)

5.1.4. Recommendations for Future Practice

The effective transition from a zone-based definition of seismic action and a “hazard” based one depends on the ability to disseminate the necessary information in the most accessible manner possible. For those regions where the seismic code has made this transition, web applications are available to allow engineers to access data from the region’s current hazard analysis (see previous). A similar deliverable is a recognised objective of the SHARE project, which could allow designers and engineers to directly determine the design ground motion at the location of the site under consideration from the hazard analysis. The precedent for the use of web applications in the implementation of seismic design provisions in other codes would suggest that a practical framework transition from zones to site specific hazard analysis is feasible. A further objective of SHARE is the production of a uniform hazard spectrum (UHS) for each site, and for this information to be made available alongside the hazard data currently required for EN 1998. On the assumption that this data is made available for design, then it becomes harder to justify continued use of seismic zones in further revisions to Eurocode.

5.2. Disaggregation

An important provision for the objectives of SHARE is that given in EN 1998-1 3.2.21 (5), which relates to the controlling earthquake scenario for a given hazard level at a site. *“When the earthquakes affecting the shape of the site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to enable the design action to be adequately represented. In such circumstances, different values of a_g will normally be required for each type of spectrum and earthquake”* (EN 1998-1 3.2.2.1 (5)). Compliance with this provision clearly indicates the need for a means of identifying the controlling earthquake scenario or scenarios for a site. The most common means of achieving this is via disaggregation, possibly implemented as an online web database in the manner of USGS (2008) and INGV (2009).

There is certainly a degree of criticism that can be levelled at the **EN 1998-1** representation of the elastic response spectrum, and the anchoring to PGA. Whilst the introduction of a two-spectrum system does, in a crude manner, allow the code to accommodate differences between smaller and large magnitude events, this introduces another problem. Selection of the Type 1 or Type 2 spectrum is made on the basis of M_S , where M_S is the surface-wave magnitude of the controlling earthquake scenario. As such, for the code to be effectively implemented it is necessary to know the controlling earthquake scenario. Whilst simple in concept, this information may not be easy to constrain when mapping hazard over a region as large as Europe.

The absence of clear provisions for obtaining the controlling earthquake scenario is common to all design codes. This may, in part, be a deliberate approach to allow a designer scope to use their judgement as to how an appropriate controlling scenario is defined. There may be precedent for defining the controlling earthquake as the maximum earthquake considered possible within a source region, the characteristic earthquake or perhaps a large historical event. An alternative approach, and one that has become increasingly common amongst seismic hazard analyses, is the use of disaggregation. This methodology, described in more detail shortly, closely reconciles the scenario earthquake with the probabilistic hazard formulation. It does so by considering the contribution of each event (or collection of events within a pre-defined magnitude and distance bin) to the probability of the ground motion at a given site being exceeded within the specified periods. It should be noted, that although widespread in seismic hazard analysis, disaggregation is not necessarily the most common method by which the controlling earthquake is obtained in engineering practice. As more codes come to define the seismic input in terms of a probabilistic ground motion, however, it is foreseeable that the use of disaggregation may become more widespread. In any case, it remains arguably the most practical means by which a seismic hazard analyst can identify a controlling earthquake scenario that is consistent with the PSHA being undertaken for a given site.

Disaggregation is an important tool in earthquake hazard analysis for the very purpose of identifying controlling earthquake scenarios. As part of the PSHA procedure, when implemented by either the Cornell (1968) – McGuire (1976) method or the Monte Carlo approach, the contribution of a particular magnitude-distance-variability (M-R- ϵ) scenario to the rate of exceedence of a given level of ground motion (A_0) at a site is calculated within the integral (McGuire, 1995; Bazzurro & Cornell, 1999):

$$\gamma(A \geq A_0) = \sum_i v_i \int_M \int_R \int_\epsilon H[A \geq A_0 | m, r, \epsilon] f(m) f(r) f(\epsilon) dm dr d\epsilon \quad (5.1)$$

Where v_i is the activity rate for point source i , $H[.]$ is the Heaviside Step function, $f(m)$, $f(r)$ and $f(\epsilon)$ are the probability distribution functions of M , R and ϵ respectively. Equation 5.1 assumes a point source, if two and/or three dimensional source geometries are considered the distribution of $f(r)$ may be conditional upon magnitude, or the magnitude and distance are described by a joint probability distribution function $f(m, r)$. Assuming the formulation in equation 5.1, disaggregation can be viewed as the joint PDF:

$$f(m, r, \epsilon | A \geq A_0) = \frac{\sum_{i=1}^N v_i H[A \geq A_0 | m, r, \epsilon] f(m) f(r) f(\epsilon)}{\sum_{i=1}^N \gamma_i(A \geq A_0)} \quad (5.2)$$

Where γ_i is the mean rate of exceedence of ground motion A_0 arising from source i . From this PDF, marginal PDFs can be derived that describe the univariate distribution of each of the disaggregation variables M , R and ε :

$$f(m | A > A_0) = \int_R \int_{\varepsilon} f(m, r, \varepsilon | A > A_0) dr d\varepsilon \quad (5.3a)$$

$$f(r | A > A_0) = \int_M \int_{\varepsilon} f(m, r, \varepsilon | A > A_0) dm d\varepsilon \quad (5.3b)$$

$$f(\varepsilon | A > A_0) = \int_M \int_R f(m, r, \varepsilon | A > A_0) dm dr \quad (5.3c)$$

It is a relatively simple procedure to identify M-R- ε combinations that produce the required level of ground motion, and to produce histograms indicating the contribution of specific M-R combinations. This is usually done by separating the contributions into separate bins (as in the examples shown here), although plots of the full marginal PDF for each variate may reveal more information about the features of the distribution (McGuire, 1995).

From the output of the disaggregation it is not always a simple matter to identify the controlling earthquake scenario. In order to meet the requirements of **EN 1998-1**, this information is clearly necessary. Two examples are given in Figures 5.3 and 5.4. These are outputs from the disaggregation calculator for the United States, available online from the United States Geological Survey (<http://eqint.cr.usgs.gov/deaggint/2008/index.php>). One example is a disaggregation for a site in the town of Fresno, California, the other a location north of Seattle, Washington. The disaggregations are split into 10 km – 0.2 M - 1 σ bins, with the mean value and the modal values, both with and without aleatory variability, defined.

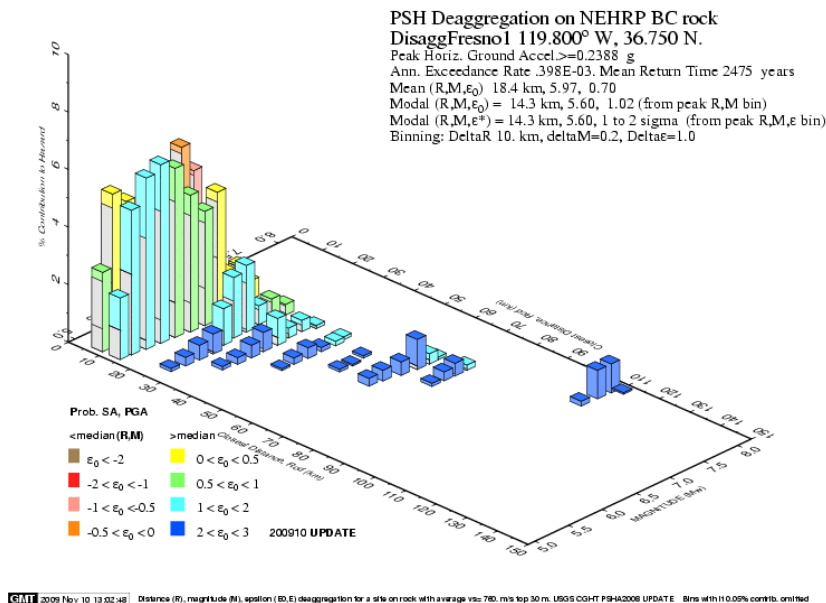


Figure 5.3: PGA disaggregation for Fresno, California, for an NEHRP Class B site, and 2475 year return period. (USGS Interactive Disaggregation, 2008)

The disaggregation for Fresno would appear to be simple to interpret. The overall distribution of events would seem to indicate that the controlling earthquake is likely to be between M 5.6

– M 6.0, at a distance of 14 – 19 km (typically with aleatory variability within 1σ above the median). The extent to which aleatory variability may influence the identification of events with magnitude greater or less than M_S 5.5 in Europe is not made clear in **EN 1998-1**. Were this a disaggregation for a site in Europe, it could be assumed, with a reasonable degree of confidence, that the controlling earthquake was greater than M_S 5.5, thus the Type 1 spectrum would be appropriate. Nevertheless, a significant proportion of earthquake scenarios contributing to the given level of hazard at a site (incidentally the 2475 year return period) would be smaller than the M_S 5.5 threshold.

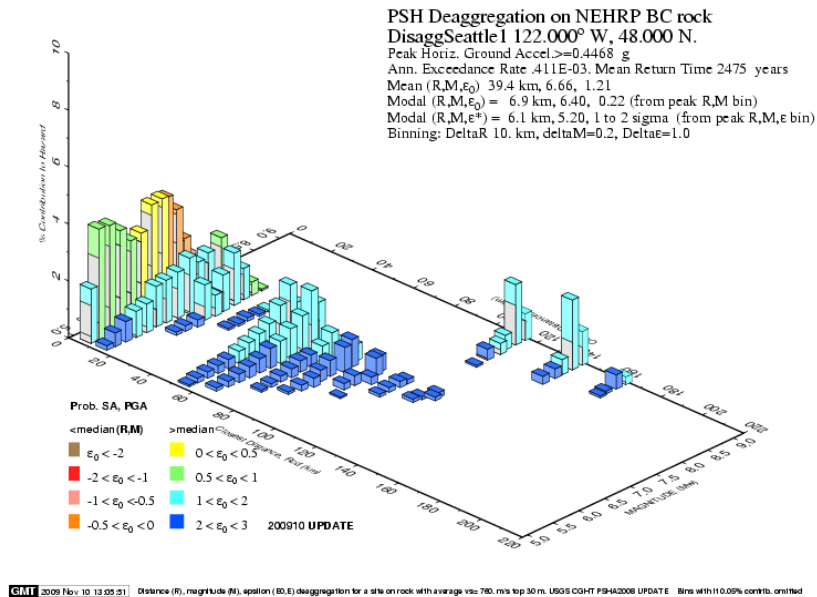


Figure 5.4: PGA disaggregation for Seattle, Washington, on a NEHRP Class B site, with a return period of 2475 years. (USGS Interactive Disaggregation, 2008)

The situation is more complicated for Seattle, where the disaggregation displays more multi-modal properties. In this example the mean and modal magnitude bin differ significantly when aleatory variability is included, with the former being M 6.4 and the latter M 5.2. There is also a clear distinction between the contribution to hazard at the site arising from moderate magnitude near-field events ($5 \leq M \leq 6.5$, $R < 10$ km, $|\epsilon| \leq 1$), moderate-magnitude mid-distance events ($6 \leq M \leq 6.5$, $50 \leq R$ (km) ≤ 90 , $1 \leq \epsilon \leq 2$) and great far-field events ($8.5 \leq M \leq 9$, $110 \leq R$ (km) ≤ 160 , $1 \leq \epsilon \leq 2$).

Whilst the seismotectonic setting of Seattle is not a direct analogue of a European site, it clearly illustrates the difficulty that can arise in using the disaggregations to identify controlling earthquake scenarios. A more relevant European example comes from the L’Aquila site in Italy, for which PGA disaggregations are produced using the INGV data set (INGV, 2009).

The L’Aquila site illustrates more clearly the difficulty in selecting the controlling earthquake scenario from the disaggregation. In this particular example the modal magnitude bin is that of $5.0 \leq M_W \leq 5.5$, although contribution to hazard from the $4.5 \leq M_W \leq 5.0$ bin is only slightly lower. There is general agreement that the scenario distance is less than 10 km.

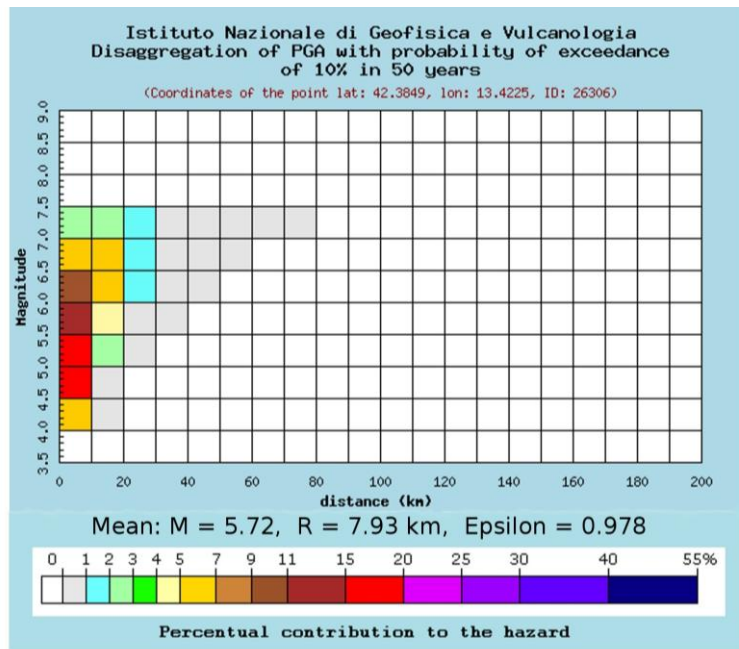


Figure 5.5: PGA disaggregation for the 475 year return period for L'Aquila, Italy. (<http://esse1.mi.ingv.it/>)

The mean values given for this particular scenario are $M_w = 5.72$, $R = 7.93$ km and $\epsilon = 0.978$. Allowing for conversion between moment magnitude, the preferred magnitude for hazard analyses, and M_S the required magnitude in the code the mean magnitude is approximately $M_S 5.5$. It should be noted that this magnitude conversion may also introduce error, which further complicates situations such as these. Furthermore, the mean scenario is determined from statistical analysis and may not necessarily represent a scenario that is realistic given the seismic sources. This may be particularly relevant if using the mean scenario of a multi-modal distribution, where the mean scenario may lie in between the modal ranges (Abrahamson, 2006). If a disaggregation results in a situation such as this, it is clear that some judgement and interpretation of the disaggregation is needed. It does, however, raise the question as to how to allow for this in a building code.

Returning to the L'Aquila example, the results of this particular disaggregation, if no further knowledge or judgement of the earthquake source is assumed, illustrate the ambiguity is presented for the designer. Using the current Eurocode formulation, selection of the mean scenario would result in $M_S 5.5$, which would correspond to the type 1 spectrum. Judgement based on the use of modal bins, however, would suggest scenarios of $M_S < 5.5$, corresponding to the type 2 spectrum. Conservatism, and in this particular example observations from recent earthquakes in the area, would most likely push the designer towards the use of the type 1 spectrum, which is again a judgement based on existing knowledge and not necessarily a direct interpretation of the building code.

In the examples given previously, only disaggregation for PGA hazard has been considered. Whilst the choice of PGA as the basis for estimating the controlling earthquake scenario is not explicit in **EN1998-1**, the dependence of the elastic response spectra on PGA would, at the very least, make this an implicit assumption. The examples from Italy illustrate how disaggregations can produce small scenario magnitudes in regions of moderate and even high seismic hazard. To scale time histories to match scenario earthquakes derived from PGA or short-period motion, will result in bias at longer periods, particularly those relevant to design.

Disaggregation may therefore be needed for longer period ground motions, or possibly PGV. A comparison of the PGA disaggregation and the 2 s spectral accelerations for the Fresno, California, considered previously is shown in Figure 5.6. Here the change from PGA to 2 s spectral acceleration has drastically altered the scenario earthquake, from a low magnitude near-field event to a large far-field one.

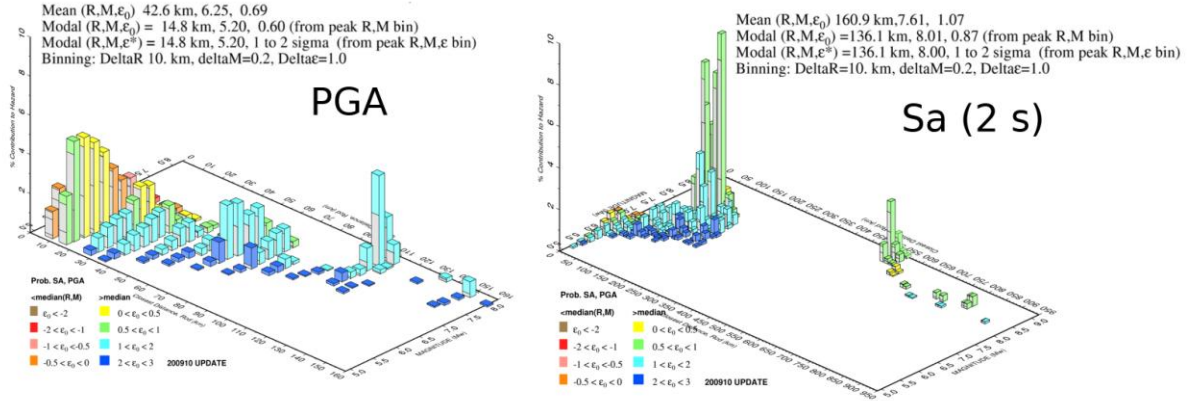


Figure 5.6: Comparison for the PGA and S_a (2.0 s) disaggregations for Fresno, California. (USGS Interactive Disaggregation, 2008)

These examples illustrate the difficulties in extracting controlling scenario events from disaggregations at given return periods. The absence of detail in all codes as how to determine the scenario event is designed to allow for some flexibility regarding methodology. In practice, however, there is a convergence toward the use of disaggregation for this purpose. A decision should there be made as to whether guidelines for the interpretation of disaggregation data should be developed. These should address the use of ϵ , errors and bias resulting from bin width, choice and interpretation of mean and modal scenarios, and the selection of scenario events from multiple disaggregations at different spectral periods.

The suggestion of presenting maps of M-R pairs (Bommer and Pinho, 2006), although clearly illustrative of the nature of the source influencing hazard at a site, presents a challenge in the interpretation of the disaggregation results. An example of how to maximise the disaggregation information within maps is illustrated in Figures 5.7 and 5.8. For a given ordinate of ground motion, bivariate PDFs of two variables (M-R, M- ϵ , and R- ϵ) can be obtained by integration of equation 5.2 with respect to the third variable:

$$f(m, r | A > A_0) = \int_{\epsilon} f(m, r, \epsilon | A > A_0) d\epsilon \quad (5.4a)$$

$$f(m, \epsilon | A > A_0) = \int_R f(m, r, \epsilon | A > A_0) dr \quad (5.4b)$$

$$f(r, \epsilon | A > A_0) = \int_M f(m, r, \epsilon | A > A_0) dm \quad (5.4c)$$

Modal values of M, R and ϵ are determined from the maxima of each PDF. The PDF can also reveal the presence of multiple modes in the disaggregation. The modal values of M, R and ϵ can then be plotted in separate maps for a region (e.g., Figure 5.7, Convertito *et al.*, 2009), which can consider each modal set separately. The relative contribution of different modes for each variate can also be mapped (Figure 5.8, Convertito *et al.*, 2009). These maps can be particularly useful in identifying where bimodal disaggregations exist, which can provide

important information for designers as to the different types of earthquake that may be relevant to the response of a structure.

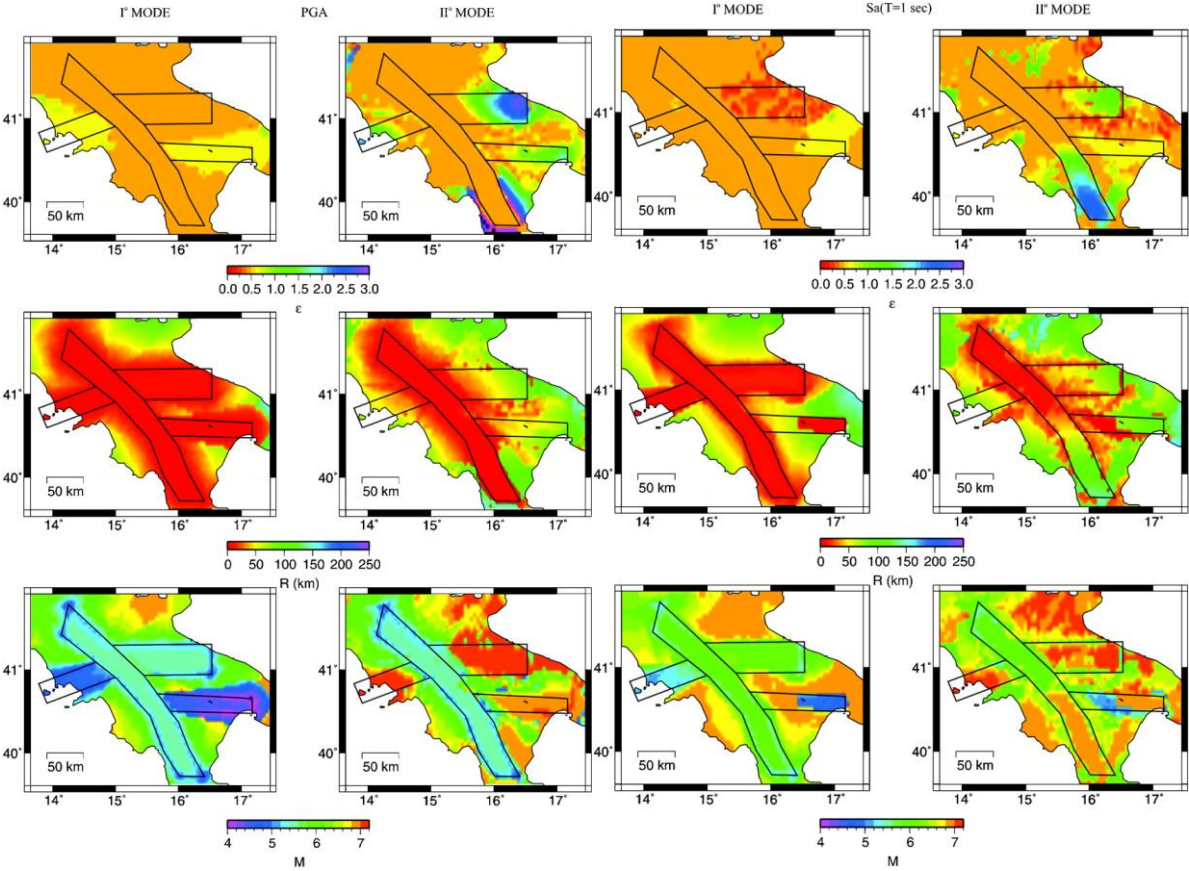


Figure 5.7: Maps of modal M , R and ϵ across southern Italy for PGA (left) and S_a (1.0s) (right). Images taken from Convertito *et al.* (2009).

The examples illustrated by Convertito *et al.* (2009) demonstrate how scenario earthquakes can be mapped in such a manner as is relevant for design. Maps such as these could be integrated within a design code, although separate modes of the disaggregation are considered, in addition to the relative contribution of modes, then the number of maps is considerable even for just one or two intensity measures. EN 1998-1 does have a provision in place for locations where different controlling earthquake scenarios may be defined. This is clause EN-1998-1 3.2.2(5), as discussed in section 6. There is no further guidance as to how the controlling earthquake scenario should be determined or how aleatory variability should be taken into consideration.

In view of the shortcomings of disaggregation, what features of other building codes could be considered as “best practice” for potential adoption in future Eurocodes? Focus will be placed on four of the codes considered here: NEHRP/IBC, National Building Code of Canada, the Italian Standard and the New Zealand Standard. In all of four of these codes the fundamental input data for a location is derived either from appendices or tables printed within the code itself (Canada, Italy and New Zealand), or from material published by the national body responsible for hazard mapping (i.e the USGS in the NEHRP provisions). For most of these bodies, web-based applications are available from the respective national agencies to produce

the required hazard parameters (in some cases including disaggregations and uniform spectra) for a site specified by the user. These websites are listed in Table 5.1

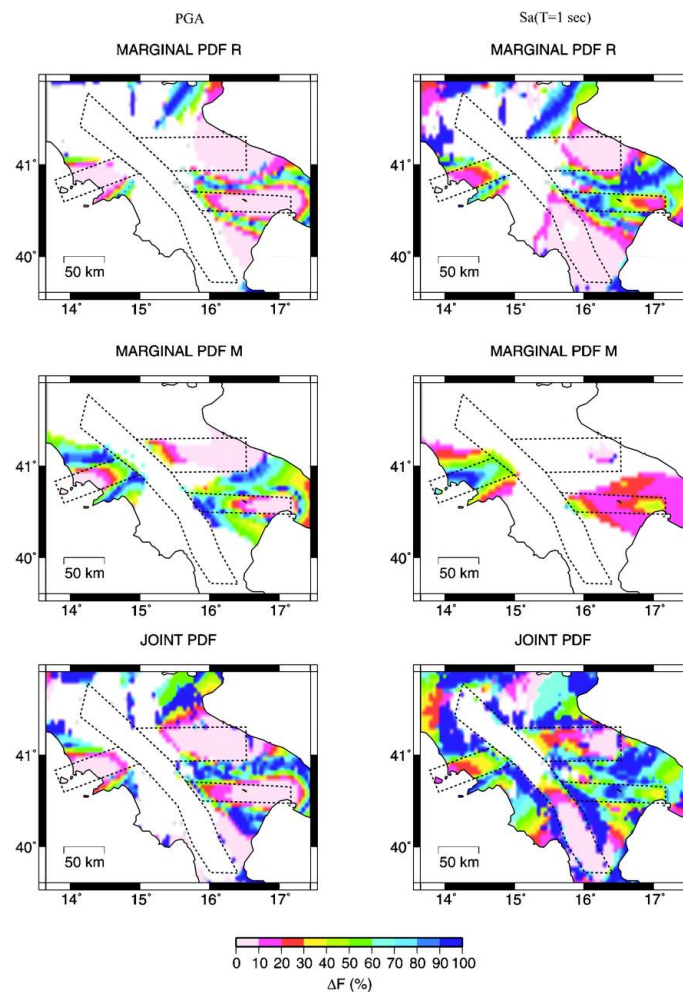


Figure 5.8: Relative contribution of the second mode to the PGA (left) and Sa (1.0 s) (right) disaggregation for M, R and joint M-R. Image taken from Convertito et al. (2009).

This is an effective method of dissemination that ensures that end users of the building code can rapidly access the necessary input parameters for seismic building design. In order for hazard to be estimated at any given site, these programs interpolate between the nearest grid points for which the hazard has been calculated. In most cases spatial linear interpolation is used (USGS Website). The extent to which interpolation introduces error into crucial parameters of the hazard analysis is unclear. This may depend on the resolution of the hazard grid points, the number of nearby points used for interpolation and the method applied. The matter is complicated further by the manner in which aleatory variability is assimilated into the hazard calculations. For many regions of moderate to high seismicity (of which areas of both Italy and Greece could be considered) the disaggregation may often indicate that the modal M-R combination may be in the region of $5 \leq M \leq 6$ at distances of $R \leq 10$ km, typically with $\epsilon \geq 0.5$. If seismic hazard is rendered onto a grid of approximately 5 km resolution (as is the case for the **NNTC-Italy (2008)** parameters) then interpolation may bias the actual change in modal M-R. As the modal M for many sites in these regions is close to the M_s 5.5 threshold it is not clear whether the controlling earthquake has crossed the threshold.

Table 5.1: Available interactive web tools for disseminating national seismic hazard results

Building Code	Website
NEHRP/IBC	http://earthquakes.usgs.gov/hazards/design/buildings.php http://earthquakes.usgs.gov/hazards/design/bridges.php
NBC 2005	http://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-eng.php (French Version also available)
Italy	http://esse1.mi.ingv.it

It should be noted that despite the vast quantity of data available within the web-based hazard mapping applications produced by the USGS, the values of the long-period transition parameter (T_L) do not appear to be among them. Spectra produced by the Java-based applet for buildings only span the range $0 \leq T \text{ (s)} \leq 2$. The minimum value of T_L within the whole United States, indicated in the **FEMA 450**, is 4.0 s. This would suggest that for large structures sensitive to longer periods, the software is not sufficient to develop the response spectra over the required range.

The framework for web dissemination of hazard data required for many building codes is obviously integral to the effective implementation of such codes. However, **EN 1998-1** is constructed under different conditions to those of the four codes discussed in this section. In each of those countries the definition of seismic action is uniform across the country. Whilst the derivation of MCE may be different in the western United States when compared to the eastern United States, the definition of MCE in the context of the design spectra is the same. Similarly the design specifications are clearly specified in the New Zealand, Canadian and Italian standards. This is not the case for Eurocode 8, where the definition of return period is subject to revision within each country's national annex. Consequently, a map of maximum considered earthquake ground motion, such as those given in **FEMA 450**, may not have any significance when preparing the ERS for the design of a structure according to Eurocode.

5.3. Uniform Hazard Spectra

Whilst the design response spectrum is considered an idealised approximation to constrain the spectral response in a manner that is consistent with the hazard at a site, the Uniform Hazard Spectrum is a more direct connection between hazard and spectral response. The UHS is constructed from probabilistic seismic hazard analyses undertaken for a number of ordinates that span the spectrum range of interest – usually limited by the period range of the spectral GMPEs. The spectral acceleration at each ordinate has an equal probability of being exceeded in the given time period (e.g., 50 years), irrespective of the probability of the earthquake scenarios that contribute most strongly to hazard. The UHS is therefore an important tool that allows for the identification of the probabilistic hazard at the fundamental periods of structures under consideration.

The UHS cannot necessarily be considered as a direct representation of the design elastic response spectrum. There is no explicit assumption as to which parts of the UHS are controlled by acceleration, velocity or displacement, as each ordinate is independent. The shape of the hazard spectra, and the relative contribution of longer period or shorter period motion at a site, is a reflection of the nature of the seismicity giving rise to seismic hazard at a site. For example, a site at which hazard originates from larger earthquakes over greater source-site distances will likely show greater hazard contribution from longer period

earthquakes resulting in a “flatter” UHS. Conversely, a site for which hazard originates from small near-field earthquakes may show a sharp peak at shorter periods, decaying to near trivial values for longer periods. In reality, hazard may originate from different sources over a range of source site distances, so the resulting UHS may reflect different source contributions over different periods. It should be recognised, therefore, that the UHS is not representative of a single given earthquake scenario.

This last point is an often applied criticism of the UHS in seismic design, and is equally valid for the use of the ERS in seismic design. If the ERS is anchored via spectral accelerations for specific periods (as is the case for the NEHRP *Provisions*) then the earthquake scenarios controlling each of the periods will most likely differ. Where the UHS offers an important advantage is that the hazard is more closely constrained at intermediate periods, rather than assumed using approximation by simple analytical functions of period. Another important feature of the UHS is the prevalence of spectral accelerations greater than the median dominating the hazard at each ordinate (a phenomenon easily confirmed by disaggregation). Whilst a ground motion may match the hazard value at the period of interest for a given number of standard deviations above the mean, it is highly unlikely to do so across the spectrum. A ground motion that matches or exceeds the UHS across the full spectral range will likely arise from a less probable event than that producing the hazard at any single given period.

It should be recalled that the elastic response spectrum defined for design in Eurocode is not developed from the UHS but from scenario spectra from European, and in the case of long-period displacements non-European, strong-motion records. Consequently the corner periods T_B , T_C and T_D are not necessarily intended to reflect the shape of the UHS in the manner in which they might for other design codes such as **NNTC-Italy (2008)**. For older design codes hazard could not be constrained for spectral ordinates owing to the lack of appropriate spectral ground motion models for use within a given region. Similarly, direct use of the UHS would require knowledge of the parameters for every spectral ordinate for the site of interest. Where seismic input was defined using zones (e.g., UBC 1997) there was no simple means by which the zone definition could accommodate the differences in UHS. Similarly, spatial representation of the seismic input would be complicated, requiring separate hazard maps for each ordinate. Constraint of the UHS using even a small number of ordinates would require a very large number of maps to be presented within the code. The Newmark-Hall ERS, when fixed by a small number of spectral acceleration parameters, represents a compromise in this respect.

The difficulties previously assumed in defining the UHS within a building code can now be overcome via open web-dissemination of hazard data, either via an online archive or a stand-alone application (e.g., Table 5.1). As with many other parts of the code, this provides a simple interface to allow end users to define the hazard spectrum for a given site of interest. There is no limitation on the number of spectral ordinates that can be used to derive the UHS within the valid period range.

The use of a UHS via a web application also has a significant advantage in that disaggregations can be provided for the relevant period of interest. This will allow for identification of controlling earthquakes that are most relevant for the structure under consideration, which may also assist in the selection of appropriate time histories for dynamic analysis. The conservatism of the UHS may be undesirable as a means of selecting time histories for dynamic analysis. The persistence of the use of the UHS, or more accurately a

representative design spectrum constructed under similar assumptions, for the purposes of time history selection is a fundamental shortcoming of most current design codes. In such circumstances appropriate disaggregation for the spectral period of interest may allow for the development of conditional mean spectra, which can be used as a reference for spectral matching without the conservatism associated with the UHS.

5.3.2. Conditional Mean Spectra

For many engineering applications, and in particular for the selection of time histories for dynamic analysis (Chapter 6), the conservatism of the UHS (and the ERS) may result in undesirable bias. When considering a structure of fundamental elastic period T_0 the spectral acceleration at T_0 may be determined from the UHS. Disaggregation at T_0 will produce a scenario event M-R- ε particular to the given period. For certain structures it may be necessary to consider higher modes, with periods typically on the order of $1/3$ (2nd mode) to $1/5$ (3rd mode) of that of the fundamental mode, and period elongation due to stiffness degradation. For a given M-R combination, the values of the UHS at periods other than T_0 correspond to values of ε different from, and usually higher than, that of T_0 . The correlation between $\varepsilon(T_0)$ and $\varepsilon(T_i)$ will naturally decrease with increasing difference between T_0 and T_i . However, the correlation between ε at two given periods (T_0 and T_i) can be used to define a response spectrum that corresponds more closely to those the observed strong-motion spectra.. The resulting distribution is given as the conditional mean spectrum (Baker & Cornell, 2006; Baker, 2010), so called because it consists of the mean values of the spectrum at all periods, conditional on the spectral acceleration at a single period.

The conditional mean spectrum is relatively simple to construct, providing that both the UHS and the M-R- ε combination relevant to the fundamental period of the structure are available. These are common outputs of seismic hazard analysis, as has been discussed previously, although there are few examples of regions where this information is made so widely available as to be practical to implement in a design code (the USGS National Seismic Hazard Maps being the notable exception). The CMS is constructed as follows:

- 1) For a structure of fundamental period T_0 , identify target spectral acceleration from the UHS, and the M-R- ε combination for that same period via disaggregation.
- 2) Using an appropriate ground motion prediction equation, of the sort referred to widely in this report, calculate the mean, $\mu_{\ln Sa}(M,R,T)$, and standard deviation, $\sigma_{\ln Sa}(T)$, of the logarithm of spectral acceleration corresponding to the magnitude and distance of the controlling earthquake scenario identified in step 1. The range of periods considered may depend on the application. As shall be seen in chapter 6 in many design codes, the period range may be typically on the order of $0.2T_0$ to $1.5T_0$ or $2T_0$.
- 3) The conditional mean ε at other periods is determined on the basis of correlations between ε values at different periods. One such model suggested by Baker (2010) is the following, originally formulated by Baker & Cornell (2006a):

$$\rho(\varepsilon(T_{\min}), \varepsilon(T_{\max})) = 1 - \cos\left(\frac{\pi}{2} - \left(0.359 + 0.163I_{(T_{\min} < 0.189)} \ln \frac{T_{\min}}{0.189}\right) \ln \frac{T_{\max}}{T_{\min}}\right) \quad (5.5)$$

Where $I_{(T_{\min} < 0.189)}$ is equal to 1 if $T_{\min} < 0.189$ and 0 otherwise. This is one of the simpler correlation models, and is believed valid over the range 0.01 s to 5 s. Other

models such as those of Jayaram & Baker (2008) may be valid for a larger period range, but with increased complexity.

4) The conditional mean spectrum is then determined from:

$$\mu_{\ln Sa(T_i) | \ln Sa(T_0)} = \mu_{\ln Sa}(M, R, T_i) + \rho(\varepsilon(T_i), \varepsilon(T_0)) \varepsilon(T_0) \sigma_{\ln Sa(T_i)} \quad (5.6)$$

An example of the conditional mean spectrum (CMS) for a Western U.S. site is shown in Figure 5.9.

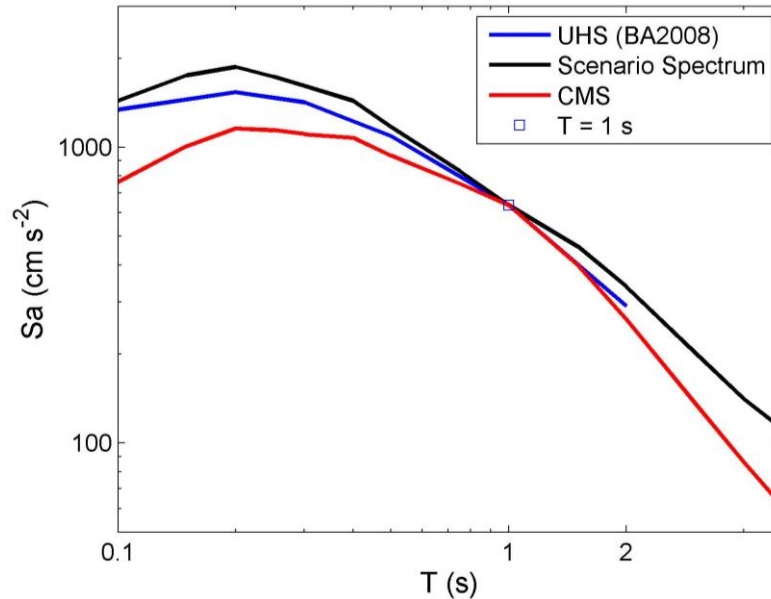


Figure 5.9: Comparison of the 2475-year UHS for western U.S. site calculated using the Boore & Atkinson (2008) GMPE [BA2008] (blue), the spectrum for the scenario earthquake determined for the 1 s spectral acceleration (black) and the conditional mean spectrum using BA2008 (red).

As can be seen from Figure 5.9, the spectral acceleration of the CMS at the fundamental period of a given structure is the same as the UHS (for the same ground motion model). Therefore the response of a SDOF oscillator at that period will be the same. For multi-degree-of-freedom structures with nonlinear response the impact of the CMS is greater due to the consideration of a wider range of periods. Further discussion on the use of the CMS as a means of selecting time histories for dynamic analysis is given in Chapter 6. Construction of the CMS is dependent on the characteristics of the structure under consideration, which need to be defined *a priori*. Nevertheless, the CMS is an important development upon the scenario spectrum that is more relevant for structural analysis. Whilst the CMS cannot necessarily be explicitly defined within seismic design provisions, there may be scope to allow certain aspects of its construction to be made transparent. These include identification of the controlling earthquake scenario and definition of the correlation function between the strong ground-motion residuals at different periods. Again, the use of a web interface may allow for construction of the CMS for a given site at user-specified periods.

5.4. Damping

For all building codes there is an established convention that the “standard” damping factor is 5 % of critical damping ($\xi = 0.05$, if expressed as a proportion of critical damping). There are many circumstances under which it is necessary to consider other levels of damping, particularly in the case of base isolated buildings or structures with passive energy dissipation devices. Similarly, the use of capacity spectrum and direct-displacement based loss estimation methodologies require a means of modifying the seismic input to different levels of damping.

The constraint of elastic response spectra for different levels of damping can be undertaken in different ways. Several empirical ground motion prediction models are available that give spectral ordinates of acceleration or displacement for different levels of damping (Akkar & Bommer, 2007b; Cauzzi & Faccioli, 2008). Direct calculation of the damped acceleration and displacement from the GMPE has a greater degree of accuracy for damping factors for which the ordinates are given. This has important uses for design of higher damped structures. Equivalent linearization procedures, however, require iterative calculation of the effective period and equivalent viscous damping of the structure. Damping at values other than those given in the GMPEs are needed.

The most common approach to address this is to calculate the spectrum for the new damping level using a damping modification factor (η). The damping modification factor may be determined using analytical equations given in existing building codes or published literature. The determination of η in **EN1998-1** and other codes is the focus of this section. Also introduced are other relations to determine η that have been developed by various authors.

5.4.1. Damping modification in EN 1998

The elastic response spectra presented in **EN 1998** refers to the acceleration (and displacement) at 5 % damping ($\eta = 1$). The modification of η is achieved via:

$$\eta = \sqrt{\frac{10}{(5 + \xi)}} \geq 0.55 \quad (5.7)$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage. This equation is originally presented in Bommer *et al.*, (2000). It replaces a similar formula presented in earlier drafts of **EN 1998**:

$$\eta = \sqrt{\frac{7}{(2 + \xi)}} \quad (5.8)$$

The origins of this relation do not appear to be well documented (Bommer & Mendis, 2005). Two other notable adjustments to equation 5.7 have been suggested by other studies. Tolis & Faccioli (1999) compared the damping modification factor in 1 to displacement response spectra obtained from the 1995 Kobe earthquake. The motivation for this approach is the analysis of the digital strong-motion records from this event, which constrain ground motions at substantially longer periods than for previous European records. They suggest the following revision:

$$\eta = \sqrt{\frac{15}{(10 + \xi)}} \quad \text{for } \xi > 5 \% \quad (5.9)$$

Similarly, Priestley (2003) suggested that for near-source ground motions the forward directivity has an influence on the damping modification. For records demonstrating a forward directivity pulse the following modification factor is suggested:

$$\frac{S_d(\xi)}{S_d(5\%)} = \left(\frac{10}{5 + \xi} \right)^{0.25} \quad (5.10)$$

It should be noted, however, that the basis for this modification is not well-documented. An example comparison of this model and the displacement spectra of a record with a clear directivity pulse shows that in the part of the spectrum where the pulse is most evident, the damping correction is closer to that of equation 5.7, and that the pulse itself is less evident for spectra at higher levels of damping. Comparison between equation 5.10 and observed damping correction factors for near-fault records ($R < 10$ km, $M_w > 6$) by Cameron & Green (2007) suggest that the damping correction for such records is further from unity than is implied by 5.10. Although, it is again important to note that not all records within the magnitude and distance range selected by Cameron & Green (2007) will necessarily demonstrate a clear velocity pulse. Further analysis of the damping response for records with a directivity pulse is clearly needed.

5.4.2. Damping Modification in Other Codes

There are very few codes for which a damping modification factor, of the sort given in Eurocode, is made explicit. For all the codes the “standard” seismic input is given for 5 % critical damping. The NEHRP *Provisions* define damping modification for the design displacement separately for seismically isolated structures and structures with damping devices. Modification parameters (B) are given for fixed levels of damping, expressed as a percentage of critical damping, where:

$$\frac{S_d(T, \xi)}{S_d(T, 5\%)} = B \quad (5.11)$$

The HAZUS technical manual (FEMA, 2003) details a more explicit formulation for modifying damping that is implemented within the capacity spectrum method. This formulation modifies the 5 % damped spectral acceleration at the short (0.2 s) and long (1.0 s) period ordinates respectively:

$$R_A = \frac{2.12}{3.21 - 0.68 \ln(B_{eff})} \quad (5.12a)$$

$$R_V = \frac{1.65}{2.31 - 0.41 \ln(B_{eff})} \quad (5.12b)$$

Where B_{eff} is the effective damping, given by:

$$B_{eff} = B_E + \kappa \left(\frac{Area}{2\pi \cdot D \cdot A} \right) \quad (5.13)$$

B_E is the elastic (pre-yield damping), A the area enclosed by the hysteresis loop, D the peak displacement response of the push-over curve, A the peak acceleration response at peak displacement, and κ the degradation factor that defines the effective amount of hysteretic damping as a function of earthquake duration (structure-specific). This approach is designed for the use of the capacity spectrum method within the loss model and does not form a direct analogue of the modification factor given in Eurocode. The earlier Uniform Building Code (1997) and previous versions of the NEHRP *Provisions* do give modification coefficients for particular levels of damping to be used for design of base isolated structures. It should be emphasised that the modification factors suggested by the NEHRP *Provisions* correspond to those for pseudo-accelerations, which derived are from the displacement response (equation 4.1) rather than the acceleration response itself (Lin & Chang, 2003).

As noted by, Bommer & Mendis (2005) few other codes make any use of a damping modification factor. This is borne out by the studies here, for which none of the current codes for Japan, New Zealand, Canada, Indonesia and Pakistan contain any such modification factor. One notable exception, besides **NNTC-Italy (2008)**, whose design is based heavily on Eurocode, is the latest draft of the Mexican code (MOC, 2008). Here the damping modification factor is calculated via:

$$\eta = \left(\frac{0.05}{\zeta_e} \right)^\lambda \quad \text{where } \lambda = \begin{cases} 0.35 & T < T_c \\ 0.35(T/T_c)^{if} & T \geq T_c \end{cases} \quad (5.14)$$

Where ζ_e is the effective damping of interest for the structural system as a proportion of critical damping, and T_c is the corner period between the constant velocity and constant displacement parts of the spectrum. This damping modification resembles, albeit with adjustment for long-period motion, the modification factor found in the 1990 French Code and the 1994 Spanish code, expressed here as a percentage of critical damping:

$$\frac{SA(\xi)}{SA(5\%)} = \left(\frac{5}{\xi} \right)^{0.4} \quad (5.15)$$

The current seismic design criteria of the California Department of Transportation (Caltrans, 2006) include a modification factor to allow for damping to be increased from 5 % to 10 % for bridges that are heavily influenced by energy dissipation at the abutments. The following equation is used to scale the spectral displacements:

$$\frac{SD(\xi)}{SD(5\%)} = \left(\frac{1.5}{0.4\xi + 1} \right) + 0.5 \quad (5.16)$$

where ξ is expressed as a percentage of critical damping. Whilst there is clearly precedent for the inclusion of a damping modification factor within a seismic design code its use is not widespread.

5.4.3. Considerations in development and use of damping modification

The development of damping modification factors requires some analysis of how their adjustment of the response spectra to higher or lower levels of damping compares with observed response spectra at such levels. Furthermore, it is necessary to determine whether the modification factors are robust across the period range under consideration and to the earthquake scenario. Such investigation has been undertaken via several studies, many of which have led to the development of damping modification schemes, either of the direct analytical form shown here or via published ordinates for a range of scenarios.

A detailed account of the origin of damping modification found in earlier publications is given by Lin *et al.* (2005), and need not be recited in its entirety here. Newmark & Hall (1982) developed an early approach to damping modification based on their elastic response spectrum. This required three separate modification factors corresponding to the constant acceleration, velocity and displacement ranges of the response spectrum. Statistical analysis of a wider set of observed response spectra by Naeim & Kircher (2001) finds that whilst damping modification is greater in the longer period (constant displacement) part of the spectrum, the trend is difficult to detect given the dispersion of results. Cameron & Green (2007) develop this further, comparing western US and central and eastern US records, which show greater disagreement between the Newmark & Hall (1982) modification factors and those developed from CEUS records. Ling & Chang (2003) give modification factors for periods in the range 0.1 s to 10 s for damping ratios between 2 % and 50 %. These are derived from statistical analysis of North American records. Similarly, Atkinson & Pierre (2004) derived modification factors for a narrower period and damping range ($0.05 \text{ s} \leq T \leq 2 \text{ s}$, $1 \% \leq \xi \leq 15 \%$) based on stochastic simulation of ground motions for eastern North America. This study indicated general stability of the damping modification factors over the narrow spectral range. Subsequent analysis by Cameron & Green (2007) suggests that the Atkinson & Pierre (2004) correction factors may be too high in the short-period range for low damping, and too low in the short-period range for higher damping levels.

A stronger theoretical basis for damping modification is given by Bommer & Mendis (2005). They illustrate how the damping responds over many cycles of oscillation, reaching a steady state after a number of cycles of motion. The degree of damping affects how soon this steady state is reached. They conclude, therefore, that damping is controlled by the number of cycles of motion. This theoretical and observational basis for dependence of damping modification on the number of cycles of motion is expanded upon by Cameron & Green (2007). They demonstrate the response of a SDOF oscillator under different numbers of cycles of motion, and for different oscillator periods and damping values. Their results illustrate a clear dependence of the damping modification factor on both the number of cycles and the ratio of oscillator period to ground motion period. Empirical models relating the damping modification factor, for spectral displacements, to strong-motion duration (defined by significant duration of 5 – 75 % and 5 – 95 % Arias Intensity) and effective number of cycles on the damping, have been developed by Stafford *et al.* (2008b).

$$\frac{S_d(\xi)}{S_d(5\%)} = 1 - \frac{\beta_1 + \beta_2 \ln(\xi) + \beta_3 \ln(\xi)^2}{1 + \exp\left(\frac{-[\ln(x) + \beta_4]}{\beta_5}\right)} \quad (5.17)$$

Where x is the duration (significant duration or number of cycles of motion), and β_i for $i = 1 \dots 5$ are coefficients for each target damping (2 % to 55 %, as given in the paper). The model given by Stafford *et al.* (2008b) is also shown to be a good fit to the observed data across the range of durations and damping modifications considered.

Further damping modification coefficients for SDOF systems are also presented by Hatzigeorgiou (2010), who compare spectral displacement damping factors for strong-motion records. Separate coefficients are given according to site class, near-fault/far fault and real/artificial accelerograms. For near-fault motions, however, there appears to be no explicit dependence on site class. The following functional form is suggested:

$$\beta(T, \xi) = 1 + (\xi - 5) \cdot \left[1 + c_1 \ln(\xi) + c_2 \ln(\xi)^2 \right] \cdot \left[c_3 + c_4 \ln(T) + c_5 \ln(T)^2 \right] \quad (5.18)$$

where β is the modification factor and T the period of the SDOF system. This function allows for direct scaling across a broader range of periods ($0 \leq T$ (s) ≤ 5) and damping ratios ($0.5 \leq \xi \leq 50$). This particular model may allow for damping modification to be applied in a practical way, avoiding the need to interpolate between coefficients for fixed damping values. It should be recognised, however, that the functional form is derived via selection of an optimal model in the statistical sense, and not from a physical interpretation of energy dissipation within a structure. Nevertheless, the strong dependence of the damping modification factor upon the period of the ground motion is consistent with the observations of Cameron & Green (2007).

A further suggestion for estimating the damping modification factor can be found in Bommer *et al.* (2010), who suggest constraining the factor using PGV and PGA. A more detailed discussion of the seismic design input methodology for Bommer *et al.* (2010) is presented in section 4.4, so further detail of the justification for this approach shall not be expanded upon here. Based on the comparison of spectral ground motion at different damping ratios, created using the Akkar & Bommer (2007b) ground motion model, it is found that the damping modification factor can be determined via:

$$\eta = c_1 + c_2 \ln(\xi) + \left[c_3 + c_4 \ln(\xi) \right] \left(\frac{PGA^{c_5}}{PGV} \right) + c_6 \frac{PGV}{PGA} \quad (5.19)$$

Where PGV and PGA are the design hazard values output from the hazard analysis, and $c_1 - c_6$ are coefficients to be determined within the code. This formulation offers a simple advantage in that it is based upon parameters that are common, or at least achievable, outputs of a seismic hazard analysis, and are independent of period and structure. A formulation such as that shown in equation 5.19 is entirely consistent with the theoretical basis that damping modification is dependent on the number of cycles within (or duration of) a time history. Both are dependent upon earthquake magnitude, for which PGV/PGA may be considered an effective surrogate. However, further testing of this assumption on the basis of hazard spectra is needed before recommending any of these methods for revision to Eurocode.

The dependence of the damping modification factor upon site class has been investigated by Lin & Chang (2004) and Bommer & Mendis (2005). In the former, site conditions are shown to have a small, but not negligible, impact on the damping modification factor, but may be neglected for systems with damping ratio less than 20 %, if a 5 % error is tolerable. A

comparison of response spectra derived by predictive ground motion models is implemented within Bommer & Mendis (2005), which finds a discernible trend harder to detect, in part due to the uncertainties in classification of the site condition for some European records.

5.4.3. Implementing Damping Modification in Building Codes

The brief analysis of damping modification factors given here would suggest that current implementation within building codes may not be sufficient to adequately characterise the observed influence of damping on response spectra. There are clearly many methods of modifying the damping coefficient, yet care is needed in identifying any one method or procedure as being suitable for building codes. Furthermore, the implementation of capacity spectrum and direct design-based procedures for PBSD means that computational accuracy and efficiency are necessary objectives.

The current **EN 1998** modification factor (or one of its adaptations) may represent the simplest approach, but not necessarily the most accurate. The use of empirical ordinates for the damping modification factor (e.g., Atkinson & Pierre, 2004; Cameron & Green, 2007) may be closest to the approach in the current NEHRP *Provisions*, but have limited application within the iterative requirements of direct displacement-based design. This is because damping is needed at intermediate levels, and for intermediate periods, rather than exclusively those described by the ordinates. To determine the damping modification coefficients for these intermediate values it is likely that interpolation will be needed. This is computationally inefficient and reduces the overall accuracy.

The approaches offered by Hatzigeorgiou (2010), Stafford *et al.* (2008b) and Bommer *et al.* (2010) may arguably represent the compromise between practicality and accuracy. The Hatzigeorgiou (2010) formulation is consistent with Eurocode site characterisation. It is also practical for iterative analysis as it describes the damping modification factor as a function of target damping and period. Some uncertainty exists in the distinction between near-fault and far-fault motion. This may be elucidated via identification of the controlling earthquake scenario, as required elsewhere in the record. It should be noted that the coefficients given by Hatzigeorgiou (2010) for artificial records should not be used, as the artificial records considered an arbitrarily long duration of motion.

The Stafford *et al.* (2008b) formulation has the advantage of a basis upon engineering dynamics rather than statistics, as well as derivation from real records. It is also generally efficient, as the damping modification is determined via a continuous function and is independent of period. In these respects this formulation may actually represent the least drastic adaptation of the current implementation. It is, however, dependent on identification of the duration of strong-motion (or the number of effective cycles). This issue may be harder to resolve as ground motion duration is not a common output from seismic hazard analysis. Since development of the Stafford *et al.* (2008b) model, and presumably motivated in part by its implications, empirical ground motion models have been developed for number of cycles (Stafford & Bommer, 2009) and for strong-motion duration (Bommer *et al.*, 2009). Implementation of these GMPEs in seismic hazard analysis, whilst not an immediate objective, is a feasible extension of ongoing seismic hazard work.

Seismic hazard maps or zonations on the basis of duration can be produced, but are not recommended for this purpose, as disaggregation of these maps will produce controlling scenarios inconsistent with those for the hazard spectra (Bommer & Mendis, 2005). Instead

the duration could be forward modelled using the controlling earthquake scenario identified by disaggregation of the 5 % damped spectrum at the fundamental elastic period of the structure (Stafford *et al.*, 2008b). This may be the most accurate approach, although it is still not clear how the aleatory variability in the duration ground motion model is incorporated into the analysis. However, the Bommer *et al.* (2010) formulation defines the damping modification factor in terms of PGA and PGV. This convenient formulation arises from the use of PGV to PGA ratio as a surrogate for magnitude, recognising that duration of motion depends strongly on magnitude. PGA and PGV can both be derived directly from the hazard analysis, thus avoiding the need to identify a controlling earthquake scenario for the purposes of defining the damping modification.

5.5. Epistemic Uncertainty

The nature of randomness and uncertainty in seismic hazard analysis has been the subject of interesting and insightful debate in recent years (Abrahamson, 2000; 2006; Bommer & Abrahamson, 2006). To date, however, the inclusion of matters relating to the treatment of uncertainty in the input for seismic analysis has rarely been made explicit in code provisions. More discussion is often found in commentaries to the code or in other supplementary material, much of which may not be scrutinised by the end-users of the code. It should be noted at this point that the uncertainty discussed here is that arising from the definition of the seismic input and not the structural response. The uncertainty in the latter is usually addressed in many other provisions of the code.

Clarification of the term epistemic uncertainty, and its meaning in the context of the input for seismic design, is needed here. The conceptual framework in wide use is that of a distinction between *aleatory variability* and *epistemic uncertainty*. The former describes the natural randomness in a process, whilst the latter describes the scientific uncertainty in the simplified model of the process (Abrahamson, 2006). Within PSHA, many aspects of the earthquake process are included as aleatory variability. These include earthquake magnitude and distance, as well as the variability of the predicted ground motion for a given magnitude and distance scenario. Current practice in seismic hazard analysis requires the use of empirical predictive ground motion equations, regardless of whether it is the Cornell (1968) – McGuire (1976) approach or Monte Carlo (Musson, 1999) approach that is used. These functions typically assume the following form:

$$\log(Y) = f(M, R) + g(\theta) + \varepsilon\sigma \quad (5.14)$$

Where, σ is a lognormally distributed variable that accounts for both inter- and intra-event scatter in the model of ground motions. Aleatory variability in the earthquake process is incorporated into the seismic hazard analysis by virtue of the probabilistic approach, which should include integration of σ in the ground motion modelling. It may be assumed that in best practice, aleatory variability is accounted for in the definition of the seismic action at the point at which it meets the requirements of the current code.

Epistemic uncertainty in the definition of the seismic action can originate at many points in the seismic hazard analysis: in the definition of the seismic source, in the characterisation of earthquake behaviour for a given source, and in the selection of the ground motion model. Logic tree analyses have emerged as a widely-used tool to characterise epistemic uncertainty in the seismic hazard analysis. A full explanation of the formulation and interpretation of a

logic tree is beyond the scope of this report and the reader is referred to Scherbaum *et al.* (2005) and Bommer & Scherbaum (2008) for further details, although some of their discussion shall be assessed here.

Seismic hazard curves may be developed for a given combination of input models for each part of the PSHA process (e.g., seismic source, earthquake recurrence, ground motion attenuation etc.). It may be recognised, however, that each model may represent a simplification of the process it is describing, or may represent only part of the true variability in the process modelled. Logic trees combine all the alternative models into a branch and node structure, where different models are placed on the branches for each part of the earthquake process (nodes) where it is recognised that epistemic uncertainty exists. This last statement gives a brief sample of the conceptual and practical problems that occur in real application of logic trees. Consequently, real logic trees can display enormous disparity in both the number of branches and the number of nodes.

For a given node, each branch of the logic tree may be ascribed a weight. This weight may reflect the degree of confidence the seismologist or engineer may have in the accuracy of the model or its ability to capture adequately the aleatory variability of the process modelled. The combination of models and the products of the respective weights will result in a suite of hazard curves, with a weight ascribed to each curve. The suite of curves can be considered a numerical representation of the epistemic uncertainty range. This may be viewed as the uncertainty in ground motion for a given return period, or alternatively, the range of return periods for a given design ground motion. The former is more consistent with current definitions of seismic input in code design, which consider fixed probabilities of exceedence (Bommer & Scherbaum, 2008). It is often expected that design ground motion may be selected from the “average” curve or, where conservatism may dictate, from a higher fractile (e.g., 84th percentile). The definition of this average has been the topic of some debate, with some advocating the use of a given fractile of the hazard curves. The fractile may reflect the desired degree of confidence that the safety level implied by the return period is being achieved in light of the uncertainty in the hazard analysis (Abrahamson & Bommer, 2005). Alternatively (McGuire *et al.* 2005; Musson, 2005) defend the use of the mean curve on the basis of consistency with the probability theory underpinning the epistemic uncertainty analysis.

Whilst decisions regarding the definition of the “average” curve remain unresolved, there are other considerations in the formulation of the logic tree. Inherent within the probabilistic framework invoked by use of the logic tree is the mutually exclusive and collectively exhaustive criterion (MECE). Mutual exclusivity requires that a model on a branch of any given node is applicable, but not in combination with models on other branches. The collectively exhaustive criterion assumes that one model in the set is fully applicable for each scenario (Bommer & Scherbaum, 2008), noting that the same model may not necessarily be applicable for all scenarios.. In practical application it is not uncommon for both criteria to be violated. For ground motion models it is common that two or more different empirical models, with different parameterisations, may be developed from common data. This results in some model redundancy. To achieve the collectively exhaustive criterion it is tempting to simply increase the number of models considered; the assumption being that with a greater number of models there is a higher probability of including a fully applicable model. With more models, however, the probability of model redundancy is increased. An additional consequence is that an exhaustive number of branches may result in the inclusion of models with very low levels of expert confidence. These models may be those that produce extreme

hazard curves, whose influence on the mean hazard is more pronounced at longer return periods (Abrahamson & Bommer, 2005), and whose inclusion could therefore be questioned (Musson, 2005).

One issue that is emerging as epistemic uncertainty analysis becomes more widespread is the contradiction of apparent increasing epistemic uncertainty in regions where knowledge of the seismogenic source and ground motion is increasing. This situation emerges as regions with more data will inevitably give rise to a broader set of models, which in turn may result in more branches of the logic tree and a more complete characterisation of epistemic uncertainty. Although this should be interpreted in terms of a significant underestimation of epistemic uncertainty in previous hazard analyses, which is gradually being rectified as more information becomes available. Conversely, seismic hazard in regions with little data may only be characterised by a small number of alternative models. Formulation of a logic tree in these circumstances would lead to the assumption that epistemic uncertainty in such regions is small, or even absent. Where situations such as this arise the epistemic uncertainty represented in the logic tree may only be considered a minimum uncertainty.

Despite the widespread use of logic trees in seismic hazard analysis, there are many issues that arise in the formulation and interpretation of the trees. An important question to ask is how this should be addressed in the development of building codes? To date, no national building code includes an explicit formulation of epistemic uncertainty analysis within the definition of seismic action. Any prescriptive measures to include epistemic uncertainty into the code must be sufficiently robust as to allow for difference in practice between site-specific analysis and regional scale analysis, be it urban or national. In the case of the former it can be expected that some areas of uncertainty within the hazard analysis are well-defined, such as the geotechnical characterisation of the site and the rupture geometry of potential seismogenic sources. In the latter case the characterisation of sites may require the use of geological or topographic proxy. Issues such as this affect the manner in which epistemic uncertainty and aleatory variability are treated, and how the logic tree is formulated.

The main focus of this discussion is on the use of logic trees for estimating epistemic uncertainty. The possibility that other methods for epistemic uncertainty analysis (Helton *et al.*, 2004) may gain more widespread use in seismic hazard applications should not necessarily be excluded from seismic code provisions. Given the widespread adoption of logic trees for this purpose, these are the main focus of discussion here.

Analysis of the use of epistemic uncertainty in existing seismic design codes generally indicates that such provisions are not considered within the seismic input. For highly critical structures the guidelines for the use of expert judgement (SSHAC, 1997) have become informally adopted as a standard for epistemic uncertainty analysis. A critical review of applications of the SSHAC methodology addresses some issues in the use of logic trees (Hanks *et al.*, 2009). These criticisms may assist in the development of a code-based approach to epistemic uncertainty estimation.

To clarify the ideas needed to define provisions for the use of logic trees in seismic design codes, it is helpful to consider application to an idealised “rock” site in an unspecified moderate to high seismic hazard location. For convenience the logic tree nodes can be divided into separate categories rather than specify particular nodes: i) source I (physical location, e.g. tectonic zones, uniform hazard zones), ii) source II (source properties, e.g. fault type, rupture characteristics), iii) behaviour (e.g., magnitude frequency distribution, M_{MAX} [although this

may instead be considered within source II]) and iv) ground motion. It is not necessarily assumed that both the mutual exclusivity and the collective exhaustiveness criteria are met for all branches, in fact it is perhaps more realistic to assume that they are not.

Arguably the first aim of any provisions guiding the use of logic trees should be one of quality control. This should require that, prior to the application of the MECE criterion, it can be demonstrated that the model is appropriate given the observed seismic behaviour (earthquake behaviour and/or ground motion) at a site. Or, conversely, that it can be demonstrated the observed seismic behaviour can be reproduced (within the given uncertainties) by the model. This can be a difficult step as it cannot always be assumed that observed data is sufficient to validate the model, particularly in regions with few strong-motion observations. Furthermore, issues of objective quality control in seismic hazard analysis using observed data are not necessarily resolved and for some aspects of the earthquake process there is no established methodology to do so. For the seismic source some attempts to objectively validate the model using Monte Carlo methods have been suggested (Musson, 2004), but these are not yet widespread. Similarly, models of earthquake recurrence can be validated using the historical earthquake catalogue and fit via an information criterion (e.g., Utsu, 1999). Such techniques may not necessarily be considered validations but rather as tests of internal consistency. They are only testing the models against historical events, which themselves represent only a limited sample of the long term distribution and recurrence of seismicity. Where possibly the most research has been developed is in the fit of the strong-motion model to observed data, where maximum likelihood (Scherbaum *et al.*, 2004; Stafford *et al.*, 2008a) and information-theoretic (Scherbaum *et al.*, 2009) approaches could be applied. It is not clear that seismic design codes should necessarily specify the methods adopted for quality control, but it may be reasonable to require that quality control be undertaken whilst leaving the method to the designer.

Where seismic code provisions can be more explicit is in how to interpret and adopt the logic tree output for practical use in seismic design. For the UHS this may require that separate spectra be defined for the mean/median and for a higher fractile (e.g., 84th percentile). From a computational perspective, however, it may not be feasible to expect engineers to undertake separate analysis for two different levels of motion. It may be the case that the 50th percentile is sufficient for analysis of ordinary structures, but the conservatism required for critical structures may warrant the use of the 84th percentile. It could be argued therefore that the importance category coefficient should no longer remain fixed but should be a site specific parameter whose value is determined via the ratio of fractiles (e.g., 84th/50th percentile for importance category III, 95th/50th percentile for importance category IV). An example of the spatial distribution of the importance category III coefficient for Italy (for PGA) is shown in Figure 5.10. Of course, the actual value of the coefficient is much lower using this formulation than the importance factor currently given in the code for importance class III (1.5), but the basis for this modification comes from the hazard analysis. The selection of the 84th percentile is arbitrary here, but a higher fractile ratio could be chosen if a more conservative value is warranted (e.g., 95th/50th percentile for category III, 99th/50th percentile for category IV).

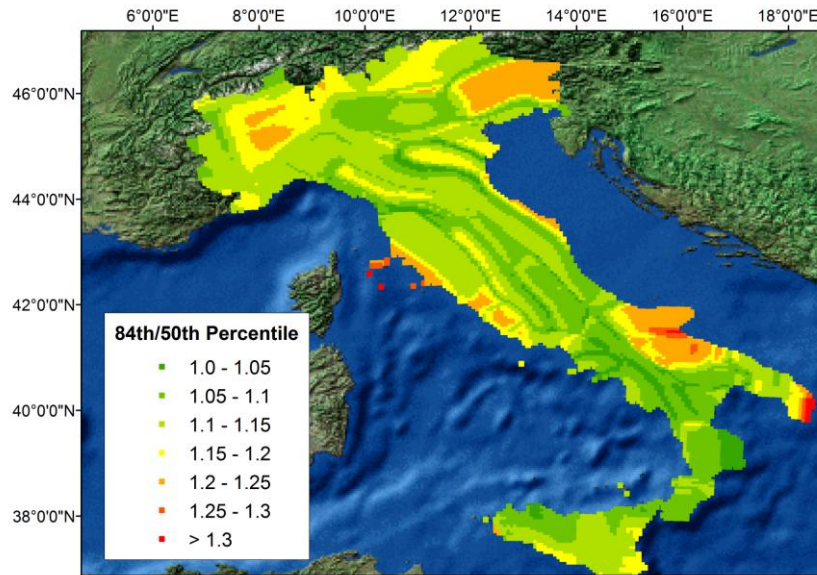


Figure 5.10: Ratio of 84th to 50th percentile ground motions for Italy (data from INGV, 2008)

The approach shown here is not dissimilar to one of the observations in Bommer & Scherbaum (2008). In the current Italian code (2008) the importance category coefficient modifies the return period. If, instead of selecting the mean and fractile levels for a given return period (e.g., Figure 5.11a), different return periods are selected for a given ground motion level (e.g., Figure 5.11b), then the approach shown here for Italy becomes directly analogous to the importance category modification in the current code. The conservatism in design level currently adopted on an arbitrary basis would then be appropriate to the hazard at a site, and is based on quantitative estimates of the epistemic uncertainty. In effect, this forms an initial step towards a risk based design approach. In **EN 1998-1**, the importance coefficient (importance factor) is a nationally determined parameter. It is therefore envisaged that it may be adjusted according to the seismic hazard and/or the degree of anti-seismic detailing within the existing building stock. Consequently, adjustment of the importance factor according to the epistemic uncertainty in hazard at a site may be a natural extension of this provision. Furthermore, whilst the ERS is anchored to PGA in **EN 1998**, it is simple to characterise the importance factor according to the epistemic uncertainty in PGA, rather than adjustment of the shape for different spectral ordinates.

Whilst the role of epistemic uncertainty in defining the importance factor may have some appeal from the perspective of the seismic hazard analyst, the adoption of such a measure should also require awareness of how uncertainty is treated in the design code. It may be necessary to consider how uncertainties in other actions (e.g. wind load, extreme snow loads) are characterised in the code to ensure compatibility in the reliability analysis of a structure. This is addressed in Eurocode 0 (“Basis of Structural Design”), which presents the guidelines, and informative annex, for verification of structural response by the partial factor method. A full treatise of the EN 1990 provisions is well beyond the scope of this report, but it is recognised that the uncertainty in the action effect does have a role in reliability analysis. The consideration of such effects within the Eurocode design philosophy may allow for the epistemic uncertainty to play a more transparent role, at least in the reliability analysis if not the design itself.

Another area where the effects of epistemic uncertainty may need to be addressed in seismic design codes is in the identification of the controlling scenario. The controlling earthquake scenario is dependent on the input to the seismic hazard analysis, which includes the source model and the GMPE. A recent modification to the USGS Interactive Disaggregation web application (<http://eqint.cr.usgs.gov/deaggint/2008/index.php>) illustrates this point. By default, the disaggregation corresponds to that for the mean hazard curve. If requested, however, the user can be given the disaggregation results for each individual GMPE used in the analysis. These results identify different, though not vastly dissimilar, mean and modal scenarios (M , R , ϵ), as well as indicating the percentage contribution of each GMPE to the mean disaggregation scenario. This degree of detail for any given site is only possible because of the web dissemination implemented by the USGS. Yet this information is particularly useful for identifying the controlling earthquake scenarios, not just for static analysis procedures but also for the synthesis and selection of acceleration histories for dynamic analysis.

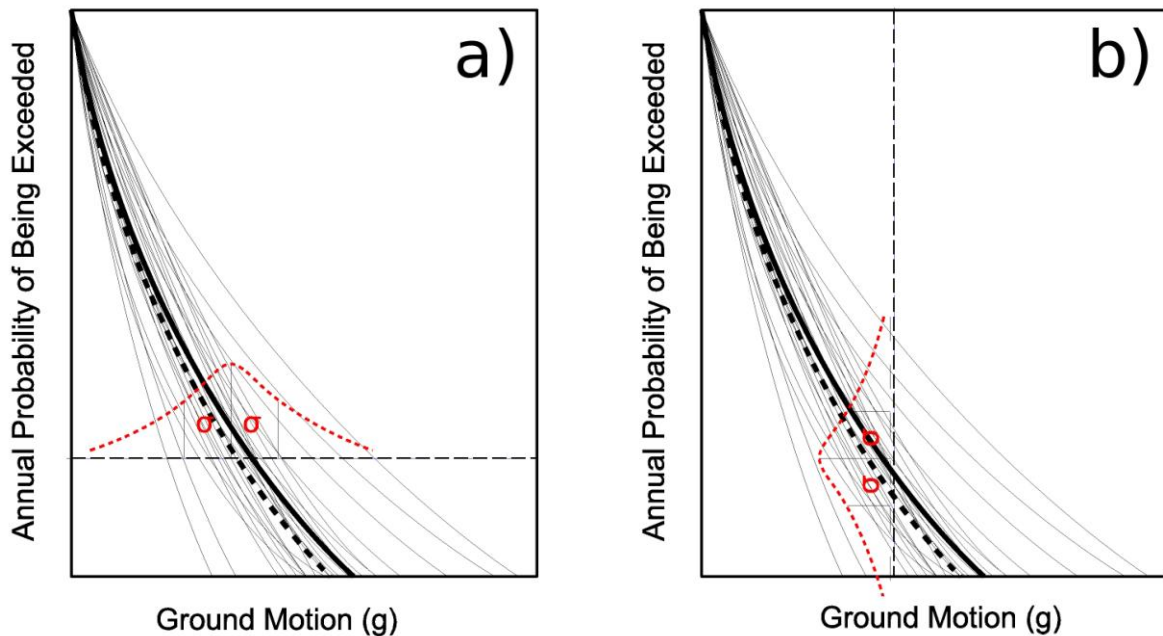


Figure 5.11: Epistemic uncertainty representations in hazard curves (thick solid line = mean ground motion, thick dashed line = mean hazard). a) uncertainty in ground motion for a fixed return period, b) uncertainty in return period for a fixed fractile of ground motion.

If the inclusion of epistemic uncertainty is to be considered as a possible objective of future revisions of Eurocode, then this analysis should be, and is, included in the SHARE project. This is an important first step that, in conjunction with web dissemination, will allow engineers to begin to integrate considerations of epistemic uncertainty in seismic input into seismic design in Europe. The USGS interactive disaggregation may provide a good example of the necessary output, but if computational memory allows it may be possible to develop this further. This could include presentation of the full suite of hazard curves for each model, identification of the mean, median and significant fractiles (e.g., 16th, 84th, 95th percentiles), disaggregations for the fractile curve and for individual curves. The amount of information required may be ambitious, but could prove useful in improving the transparency of the hazard process for the end user. Code provisions should be more limited in scope, at least

initially. These provisions may suggest consideration of only the mean/median and/or one or two higher fractiles, with their respective disaggregations.

Characterising epistemic uncertainty within seismic design provisions may present a challenge. Transparency in the process of defining the seismic input has to be weighed against the cost in terms of additional analysis for the designer. Some of the simpler approaches suggested here may be feasible to implement in such a way that is both meaningful and practical. The general absence of epistemic uncertainty provisions in the seismic input of design codes would imply that formulation of the means of assessment (e.g., logic tree), and its interpretation, is the responsibility of the seismologist or seismic hazard analyst, and not necessarily that of the engineer or designer. In that respect, it is reasonable to question whether guidelines for the use of logic trees should fall within the remit of seismic design codes for ordinary structures. Ultimately, the adoption of a set of “best practice” guidelines for characterisation of epistemic uncertainty should be undertaken by seismic hazard analysts, and only minimal requirements implemented in design codes.

6. The Use of Time Histories

6.1. Background

The use of dynamic procedures for seismic analysis (both linear and nonlinear) of structures has become increasingly widespread over the last two decades. For many applications, dynamic analysis is necessary where the structure may exhibit pronounced inelastic response in addition to higher mode effects. As computational processing power has grown rapidly over the last 15 to 20 years, the number of applications, and the complexity of structural models have grown accordingly. As this field of engineering has developed, seismic design codes have needed to add provisions to allow for the use of acceleration histories in structural analysis.

As shall be seen in this section, most current codes permit the use of acceleration time histories as a means of structural analysis. Their application may often be limited to complex structures such as those with base isolation, high damping and significant higher mode effects. However, in many provisions, a choice of dynamic or equivalent static procedures can be made. It should be noted that there is no consistent definition among codes as to whether provisions for the use of accelerograms are specified within the seismic input (as in **EN 1998**), or within the structural analysis procedures (as in **NZS 1170.5**; **FEMA 450**; **FEMA 750p**). It could be argued that the definition of seismic input may extend only as far as defining the hazard spectrum, and that issues arising from the use of acceleration time histories are pertinent only to the structural analysis. That view is not adopted here. **EN 1998-1** (and **NNTC-Italy, 2008**) defines the selection of time histories within the “Ground Conditions and Seismic Action” provisions. This suggests that time histories constitute an input for seismic action, thus clearly placing the onus upon the hazard analyst to contribute, to a certain degree, to the selection of suitable time histories. It is also recognised that selection and use of time histories is a process that is not independent of the full scope of inputs required to define the seismic action. In particular, identification of the controlling earthquake scenario, and other scenario events relevant to the hazard at a given location, is a central tenet that connects the seismological inputs and the engineering applications.

In this section the provisions given for the selection and use of time histories in structural analysis procedures found in many state-of-the-art codes will be analysed. This will include descriptions of the different procedures given by each code, as well as discussion of some of the scientific issues that have been raised by research in this area over the last decade.

6.2. Provisions in EN 1998

The provision for the use of time history representations of earthquake motion is clearly set out in **EN 1998-1 3.2.2.1(7)**, which states that “*Time-history representations of earthquake motion may be used*”. This is reiterated in clause **EN 1998-1 3.2.3.1.1 (1)P**, indicating that “*The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement)*”. Initial discussion of the use of time histories is set out in **EN 1998-1**. However, further details that are specific to certain types of structures are also given in the relevant parts of Eurocode 8. Of particular note is **EN 1998-2**, which specifies the design requirements for bridges, and shall be considered in further detail later within this section.

The second provision put in place within the discussion of time-history representation of seismic action regards spatial models of structures. **EN 1998-1 3.2.3.1.1 (2) P** states that, “When a spatial model of the structure is required, the seismic motion shall consist of three simultaneously acting seismograms”. This is qualified further in that “The same accelerogram may not be used simultaneously along both horizontal directions”, and “Simplifications are possible in accordance with the relevant Parts of EN 1998”. Finally the code allows for the use of both artificial accelerograms as well as real time histories, as set out in clause **EN 1998-1 3.2.1.1 (3)P**: “Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms ... and recorded or simulated accelerograms”. Separate clauses govern the use of the two types of accelerograms.

Both the artificial and recorded/simulated accelerograms must be subject to the rules specified in clause **EN 1998-1 3.2.3.1.2 (4)**. These state that:

- a) *A minimum of 3 accelerograms should be used*
- b) *The mean zero-period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of $a_g S$ for the site in question [where $a_g S$ is as specified in EN 1998-1 3.2.2.2].*
- c) *In the range of periods between $0.2T_1$ and $2T_b$, 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 the time histories, should be less than 90 % of the corresponding value of the 5 % damping elastic response spectrum.*

Beyond these three rules, separate guidelines are specified for artificial accelerograms and for recorded/simulated accelerograms.

6.2.1. Artificial Accelerograms

Artificial accelerograms can be generated using a variety of techniques, the advantages and disadvantages of which can be found in Douglas & Aochi (2008). In regions where observed strong-motion data is limited (or at least a paucity of high quality records exists for the particular tectonic environment) and time-history response is required for the seismic design of a structure, artificial accelerograms can be of some use. Guidance for their application varies for each building code, and little consensus has emerged as to how to process, interpret and validate such records.

In addition to the three rules specified, three further rules must be observed for artificial accelerograms to be used in seismic design. These are:

- 1) *Artificial accelerograms shall be generated so as to match the elastic response spectra given in 3.2.2.2 and 3.2.2.3 for 5 % viscous damping [EN 1998-1 3.2.3.1.2 (1)P].*
- 2) *The duration of the accelerograms shall be consistent with the magnitude and other relevant features of the seismic event underling the establishment of a_g [EN 1998-1 3.2.3.1.2 (2)P].*
- 3) *When site-specific data are not available, the minimum duration T_s of the stationary part of the accelerograms should be equal to 10 s [EN 1998-1 3.2.3.1.2 (3)].*

These guidelines indicate that it is paramount that the artificial accelerograms must be entirely reconcilable with the elastic response spectra defined previously. No further guidance is given as to how the accelerograms should be produced.

6.2.2. Recorded or simulated accelerograms

In addition to compliance with the rules specified for artificial accelerograms, recorded accelerograms must also comply with the following condition: *“Recorded accelerograms, or accelerograms generated through a physical simulation of source and travel path mechanisms [simulated], may be used, provided that the samples are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values scaled to the value of $a_g S$ for the zone under consideration” [EN 1998-1 3.2.3.1.3 (1)P]*. This clause is intended to ensure that selected accelerograms are consistent with the controlling scenario required by the spectrum. Specific details regarding tolerance of the selection criteria are not given, and this is discussed in section 5.4.

A smaller provision is also presented in Eurocode 8, which relates to the dynamic slope stability. In EN 1998-1 3.2.3.1.3 (2)P the following clause is inserted: *“For soil amplification analyses and for dynamic slope stability verifications see EN 1998-5:2004 2.2”*. This refers to the Part of the code pertaining to time-history representations for seismic design of foundations, retaining structures and geotechnical aspects. The particular clause of interest in EN 1998-5 is clause 2.2 (2): *“In verifications of dynamic stability involving calculations of permanent ground deformations the excitation should preferably consist of accelerograms recorded on soil sites in real earthquakes, as they possess realistic low frequency content and proper time correlation between horizontal and vertical components of motion. The strong-motion duration should be selected in a manner consistent with EN 1998-1 3.2.3.1”*.

Further guidance as to how the time histories should be applied in structural analysis is found in EN 1998-1 4.3.3.4.3. Amongst the provisions given, there is clear regulation as to how the response should be defined. *“If the response is obtained from at least 7 nonlinear time-history analyses with ground motions in accordance with [EN 1998-1] 3.2.3.1, the average of the response quantities from all of these analyses should be used as the design value of the action effect E_d ... Otherwise, the most unfavourable value of the response quantity among the analyses should be used as E_d ” [EN 1998-1 4.3.3.4.3 (3)]*. Regulations regarding the direction of horizontal loading are less clear. Clause EN 1998-1 4.3.3.4.1 (7)P indicates that *“the seismic action shall be applied in both positive and negative directions and the maximum seismic effects as a result of this shall be used”*. As noted by Beyer & Bommer (2007), no specifications are made regarding the original orientation. This would imply that the components are applied with an arbitrary orientation in the first analysis, before switching the polarities for the second analysis. This effectively represents a rotation by 180°. However, further qualification is provided in subclause EN 1998-1 4.3.3.5.1 (7)P, which states that *“the sign of each component in the above combination shall be taken as being the most unfavourable for the particular action effect under consideration”*.

Clear recommendations are made as to how the horizontal components of motion should be combined. In clause EN 1998-1 4.3.3.5.1 (1)P they *“shall be taken as acting simultaneously”*, whilst clause EN 1998-1 4.3.3.5.1 (2)b indicates that *“the maximum value of each action effect on the structure due to the two horizontal components of seismic action may then be estimated by the square root of the sum of the squared values of the action effect due to each horizontal component”*. This second statement is qualified further in EN 1998-1

4.3.3.5.1 (2)c, which states that this rule “generally gives a safe side estimate of the probable values of other action effects simultaneous with the maximum value as obtained in b)”. A possible alternative representation is also suggested in **EN 1998-1 4.3.3.5.1 (3)**, where “the action effects due to the combination of horizontal components of the seismic action may be computed using both of the two following combinations:

$$a) E_{Edx} "+" 0.30E_{Edy}$$

$$b) 0.30E_{Edx} "+" E_{Edy}$$

where “+” implies “to be combined with”; E_{Edx} represents the action effects due to the application of seismic action along the chosen horizontal axis x of the structure; E_{Edy} represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

In addition to the guidance provided for the horizontal components, Eurocode also provides regulation for the use of the vertical component of seismic action [**EN 1998-1 4.3.3.5.2**]. The vertical component of seismic action must be considered if $a_{vg} \geq 0.25$ g and the following cases apply [**EN 1998-1 4.3.3.5.2 (1)**]:

- i) for horizontal or nearly horizontal structural members spanning 20 m or more;
- ii) for horizontal or nearly horizontal cantilever components longer than 5 m;
- iii) for horizontal or nearly horizontal pre-stressed components;
- iv) for beams supporting columns;
- v) in base-isolated structures

Consideration of the effects of vertical motion may only be limited to “the elements under consideration [as listed above] and their directly associated supporting elements or substructures” [**EN 1998-1 4.3.3.5.2 (3)**]. An adaptation of subclause **EN 1998-1 4.3.3.5.1 (3)** is given for vertical analysis, whereby the actions effects may be computed by:

$$a) E_{Edx} "+" 0.30E_{Edy} "+" 0.30E_{Edz}$$

$$b) 0.30E_{Edx} "+" E_{Edy} "+" 0.30E_{Edz}$$

$$c) 0.30E_{Edx} "+" 0.30E_{Edy} "+" E_{Edz}$$

Where E_{Edz} “represents the action effects due to the application of the vertical component of the design seismic action...”.

6.3. Time history provisions in other codes

6.3.1. FEMA 450; FEMA 750p

The provisions for use of time-histories are presented within the “Structural Analysis Procedures” of FEMA-450, rather than within the section addressing the definition of seismic motion. There is clear scope for both two dimensional and three dimensional analysis in the NEHRP provisions.

For both linear and nonlinear response history analysis “a suite of not fewer than three appropriate ground motions shall be used” [FEMA 450 – 5.4.2]. As with the EN-1998 provisions, “if at least seven ground motions are analyzed, the design member forces, Q_E , used in the load combinations...and the design interstorey drift, Δ , ... shall be permitted to be taken, respectively, as the average of the scaled Q_{EI} [member forces] and δ_i [interstorey drift] values determined from the analysis”. If fewer than seven ground motions are analysed these parameters will be taken as the maximum value determined from the analysis.

Requirements for compatibility between the selected time histories and the controlling maximum considered earthquakes are: magnitude, fault distance and source mechanism. Site class is not included within these requirements. Provision is made for the use of simulated ground motion records to supplement real time histories “where the required number of appropriate ground motion records are not available”. Little guidance is given regarding the direction of the bi-directional motion, although this is now adjusted in the 2009 NEHRP Provisions.

“Ground motions should be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the structure in the fundamental mode for the direction of response being analyzed) the average of the 5 % damped response spectra for the suite of motions is not less than the corresponding ordinate of the design response spectrum”. For three dimensional analysis and for each pair of horizontal components “an SRSS spectrum shall be constructed by taking the square root of the sum of squares of the 5 % damped response spectra for the components”. It is made clear, however, that the same scale factor is applied to both components. There is further distinction for three dimensional analysis where “the average SRSS spectral from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the design response spectrum”. No direct requirements are given for vertical time histories, which is in contrast to the Eurocode provisions.

It should be noted that whilst the NEHRP guidelines do give adequate scope for response history analysis, the commentary to these provisions is more cautious about the extent to which these analyses can be applied. “The extra complexity and cost inherent in the use of response history analysis rather than the modal response spectrum is seldom justified”. One example is cited where they may appropriate and that is in the design of structures with energy dissipation systems comprising linear viscous dampers. Such caution in recommending the use of time-histories in structural analyses may explain why the NEHRP provisions are perhaps a little less prescriptive in their use than EN 1998-1.

There is some modification of these requirements within the 2009 NEHRP Provisions, corresponding to the changes in the definition of seismic action. The input seismic action is now explicitly defined as maximum direction of the two horizontal components, so time histories should be scaled to the new values accordingly. Further requirements for sites within 5 km of an active fault require that the pair of components be rotated in correspondence to the fault-normal and fault parallel directions. They should then be scaled such that the fault normal component is not less than the MCE_R response spectrum in the period range $0.2 T$ and $1.5 T$.

6.3.2. NZS 1170.5

The New Zealand code is a more prescriptive in the use of time histories than many other seismic design codes with regards to scaling of the strong-motion records. There are many

more guidelines pertaining to the scaling of records than in either the Eurocode or the NEHRP provisions. Similarly **NZS 1170.5** takes into consideration directivity, both within the elastic response spectrum and the time histories.

Two horizontal components of motion for each record are required, with the vertical component only necessary when considering the response of structures, or their parts, that are sensitive to vertical accelerations. The two horizontal components shall be applied simultaneously to the structure, as should the vertical component when considered. As with Eurocode and NEHRP, at least three time histories are required, with simulated ground motions only being used to make up the family of records. It is noted that the target spectrum is determined from ground motion prediction models that use the larger of the two horizontal components (**NZS 1170.5c**)

Time histories should be selected from records with a seismological signature that is the same as (or reasonably consistent with) the controlling earthquake scenarios of the target spectra at the site over the period range of interest. The term seismological signature is defined as magnitude, source characteristic (including fault mechanism) and source-site distance. Equally, the site conditions should be the same for the selected time histories as the target spectra. These conditions are the same as those specified in Eurocode 8. In accordance with the elastic response spectrum, for near fault sites one record in three in each family shall have a forward directivity component.

NZS 1170.5 presents a more complex approach to scaling time histories to match the target spectra. Initially each record is scaled by a factor, k_1 , to match the target spectra over the period range of interest. Then each record within the family of records is scaled by a factor, k_2 , which is applied to ensure that at least one record in the family exceeds that of the design spectrum over the target range. It is noted that the scale factors k_1 and k_2 will be different for different directions. The direction of application of the components is that which produces the most adverse response of the parameter under consideration.

Calculation of the k_1 and k_2 factors is made clear in the code. A summary of the method is given here:

- i) For the site of interest, determine the target spectrum from the elastic response spectrum. Calculate 5 % damped spectrum for each component of ground motion within each family of records.
- ii) Determine structure orientation relative to the selected direction, then calculate the largest translational period (T_1) of the response mode in the direction of interest.
- iii) Calculate period range of interest such that $0.4 T_1 \leq T \leq 1.3 T_1$ where $T_1 \geq 0.4$ s.
- iv) Select records with the same seismological signature (i.e., magnitude, fault mechanism and distance) as the site or “reasonably consistent” with the seismological signature of the site.
- v) Determine scale factor k_1 for each of the horizontal components, where k_1 minimises least square fit to target spectrum. Reject record if $k_1 < 0.33$ or $k_1 > 3.0$. Also reject if root mean square fit difference between the logs of the scaled primary component and the target spectra are greater than $\log(1.5)$.
- vi) Principal component of the family is that which minimises k_1
- vii) Determine the record family scale factor k_2 , which is required to ensure that for every period within the range of interest, the principal component of at least one record scaled by its scale factor k_1 exceeds the target spectrum.

- viii) If $k_2 > 1.3$ then continue, select different records as one of the family to better cover target spectrum then reassess k_2 , or if record scale factors are within 20 % of each other at period T, swap the principal and secondary component and reassess k_2 .
- ix) Repeat analysis for other directions of interest.

This complex algorithm for record selection is a clear departure from the selection process outlined in other models. A large suite of strong-motion records is widely available for New Zealand, although it is quite possible to expect that under these guidelines comparatively few of the records may be considered a reasonable fit. New Zealand strong ground-motions are widely disseminated via the GeoNet website (www.geonet.org.nz/resources/basic-data/strong-motion-data/index.html). No restrictions are placed on using records from outside New Zealand, so there is a clear opportunity to use other records from appropriate tectonic environments.

6.3.3. NNTC-Italy (2008)

Many provisions outlining the use of time histories in the Italian code, mirror those presented in Eurocode. The code recommends that “*limit states, ultimate and serviceability, can be verified through the use of simulated, artificial or natural accelerograms*”. It should be recognised, however, that seismic hazard parameters and disaggregation data are disseminated via the Istituto Nazionale di Geofisica e Vulcanologia. This allows for the controlling earthquake scenario (for PGA hazard) to be determined for a site with relative ease. Real accelerograms should be consistent “*with the characteristics of the seismogenic source, site conditions of the record, the magnitude, the distance from the source and the maximum expected horizontal acceleration at the site*”. The same conditions apply for the use of simulated accelerograms, which must be consistent with the source mechanism and path of strong motion. Selected records should be scaled to approximate the elastic response spectrum within the spectral range of interest for the structure, though no further guidance is given on scaling. Artificial records are subject to further conditions. They cannot be used, however, for the dynamic analysis of geotechnical systems.

The duration of the accelerograms must be determined by the controlling earthquake scenario that determines the values of a_g and S_S in the elastic response spectrum. In the absence of further information, the pseudo-stationary part of the accelerograms must be at least 10 s, and be preceded and succeeded by stretches where amplitude increases from and decreases to zero. The total duration of the artificial accelerogram must be at least 25 s.

Additional conditions from Eurocode are also implemented in the provisions for the use of artificial accelerograms. For the ultimate limit state artificial accelerograms must not underestimate the elastic response spectrum by more than 10 % in the period ranges 0.15 - 2.0 s and $0.15 - 2T_L$, where T_L is the fundamental period of the structure in the elastic range. For the operational limit state the corresponding ranges are 0.15 - 2.0 s and $0.15 - 1.5T_L$. For seismically isolated structures the range is reduced to $0.15 - 1.2T_S$, where T_S is the equivalent period of the isolated structure.

6.3.4. Japan (2000)

The use of time histories is integral to many parts of the Japanese seismic design code, although the use does depend on the nature of the structure under consideration. Many of the

provisions for the use of time histories relate to the level II ground motion, which corresponds to the no-collapse requirement.

A time-history assessment procedure is outlined for application to ordinary structures under certain conditions. These include the presence of an active fault in the near vicinity, the location of the building site on a deep sedimentary basin and the structural configuration of the building. The time histories should be prepared so as to ensure: i) conformance with the acceleration response spectrum, ii) continuous period of at least 60 seconds, iii) time intervals that allow proper evaluation of the behaviour of the natural periods range that influence the response of the structure to be properly evaluated, iv) required number of items to enable the variation in values to be checked. Although these measures are prescriptive in a qualitative sense, the description of the application method is limited, thus affording engineers a large amount of discretion in the selection and manipulation of time histories.

For highway bridges, records can be selected so that they correspond to the response spectrum outlined in the dynamic response analysis requirement. It is suggested that at least three records be used, with the average taken as the design ground motion. No explicit provision is given for scaling or selection of the records.

For railway structures the use of real acceleration time-histories is permissible depending on the site. Where no active fault is identifiable, the acceleration time history corresponds to that of a scenario event, modified by a risk factor for the site. Such time histories are also used as comparison models against which numerical (synthetic) or attenuation-modified time histories are compared.

6.3.5. Indonesia (2002)

The absence of good quality strong-motion records from Indonesia severely limits the use of appropriate local time histories from local earthquakes in nonlinear analysis; hence records from global databases are used. There is some provision for the use of time histories in dynamic analysis, obviously requiring records from other regions. Real time histories should be selected from locations with similar geological conditions and topography, and from events of a similar seismotectonic structure. To reduce the uncertainty arising from the site conditions at least four accelerograms from four different events should also be used. This must include the NS orientated accelerogram from the El Centro (15 May 1940) records. Artificial accelerograms may also be used, providing they are scaled appropriately to the elastic response spectrum.

6.3.6. Pakistan (2007)

More explicit guidance in the use of time histories is given in clause **5.31.6** of the Pakistan seismic code. This indicates that “*time history analysis shall be performed with pairs of appropriate horizontal ground motion time histories that shall be selected and scaled from not less than three recorded events*”. The time histories should have magnitudes, fault distances and source mechanisms consistent with the design earthquake. As with the NEHRP and **EN 1998** codes, where three time histories are used the maximum response parameter of interest shall be used for design. For seven or more time histories the average value may be taken. Simulated ground motion history pairs may be used to make up the number of records when fewer than three are available. Records should be scaled such that the SRSS of the 5 % damped spectrum does not fall below 1.4 times the 5 % damped spectrum of the design-basis

earthquake within the periods $0.2 T_0$ to $1.5 T_0$ (where T_0 is the fundamental period of the structure). This guidance is taken verbatim from the Uniform Building Code (1997).

6.4. Selection of Time Histories

In almost all the codes considered here, and in many other current codes (Bommer & Ruggeri, 2002), there is some provision for the use of time histories in dynamic analysis of structures. There are, however, many issues and uncertainties that arise from the use of acceleration time histories. These are not necessarily well-scrutinised in existing seismic design codes. The limited detail for the selection and use of time histories provided by design codes may be understood when considering the period of time for which computational means of implementing dynamic analysis have been available on the scale needed here. As noted by Katsanos *et al.* (2010), the lack of specific detail for selection and use of records may be due to: a) the recent emergence of time-history analysis in engineering practice and insufficiency of expertise developed on the subject; b) research on the topic still largely in development, hence recent innovations may require several years before they may be adopted in design codes; c) the lack of full agreement on establishment of selection criteria for earthquake records. The onus therefore remains on the designer to determine the best approach for a given analysis, within the limited provisions of the code.

There is a considerable body of literature developing that addresses many pertinent issues in the use of time histories within seismic design codes, and in particular the use and selection of synthetic (Naeim & Law, 1995) and real accelerograms (Bommer & Ruggeri, 2002; Bommer & Acevedo, 2004; Iervolino *et al.*, 2008). These issues include criteria for the selection of real time histories (or generation of synthetic/artificial time histories), the method and extent of scaling of histories to match code spectra and the combination and direction of horizontal accelerograms.

The first issue to consider is whether the current EN 1998 specification enabling the use of real, synthetic and/or artificial accelerograms is appropriate. Where dynamic analysis is specified by the code, in the vast majority of current provisions real accelerograms are not only permitted but explicitly preferred. Guidelines permitting the use of artificial and synthetic time histories are less consistent. All three types are permitted in **EN 1998**, whilst **NNTC-Italy (2008)** limits the circumstances under which artificial accelerograms can be applied. For the *NEHRP Provisions* and **NZS 1170.5**, selection is limited exclusively to real records. The Japanese code does not explicitly prohibit the use of artificial motions, whilst the Indonesian code clearly allows for their use.

The use of real/synthetic motions is generally preferred in most codes. Artificial motions designed to match the response spectrum are inconsistent with observed time histories, usually producing excessive durations or cycles of motion. In intra-plate regions of low-to-moderate seismicity, however, there may be too few, if any, real records available to the designer. Furthermore, the real accelerograms, even when scaled to the desired level of ground motion, represent the earthquake characteristics that give rise to them. They may not necessarily be representative of the future ground motion to which a structure is subjected. Synthetic motions may be used where the source and path effects can clearly be defined, but these too may not be adequately constrained to accurately represent the likely earthquake motions to which structures may be subjected. For regions such as Europe, which include areas of high and low-moderate seismicity, it seems inevitable that artificial accelerograms

may be needed. Since the original development of **EN 1998**, some new procedures have been developed that may be useful for synthesis of time histories. These include neural network based approaches that use real accelerograms to tune the stochastic simulation (Ghaboussi & Lin, 1998; Lin & Ghaboussi, 2001) or application of energy-based envelope functions to improve the duration and energy characterisation of the stochastic ground motions (Stafford *et al.*, 2009).

Where time history analysis is permitted, and accelerogram selection based primarily, though not exclusively, on real records, the next issue to address is the quantity of record groups (two horizontal components and one vertical component, if required) needed for analysis. This is where code provisions begin to become less prescriptive. As was seen for Eurocode, the number of groups needed for dynamic analyses must be at least three. If more than seven analyses are used then the average response can be used for design, otherwise the maximum response should be used. Similar provisions can be found in **FEMA 450** as well as the Pakistan code. The Indonesian Code specifies a total of four accelerograms, not necessarily three-component groups, from four different events. **NZS1170.5** adopts a different selection procedure altogether, which enables the use of three accelerogram pairs. It has been well-illustrated by Beyer & Bommer (2007) that use of only three component pairs is insufficient for structural analysis. This assumes real records are used, however, and it is possible that where artificial histories are used to supplement the data set the sufficiency of the three-group approach is even poorer. It is generally accepted in most provisions that the same time history cannot be used for both horizontal components of a single group.

The selection criteria for real records is usually intended to ensure that the selected time histories are consistent with the controlling earthquake scenario for hazard at a given site. The merit of this objective will be discussed shortly. Provisions for bi-directional analysis are inconsistent across codes, although it is clear that such analyses will require two horizontal components to be defined. The extent to which the record parameters and the controlling scenarios agree differs between codes. **EN 1998** defines the “controlling scenario” in terms of magnitude, distance, rupture mechanism and site class, whilst the Italian code also mandate consistency with the maximum expected horizontal acceleration at a site. **FEMA 450** mandates only magnitude, distance and source mechanism, thereby relaxing the site classification condition. Other codes range from generally qualitative (suggesting consistency with the scenario but not prescribing matching parameters) to moderately prescriptive (requiring magnitude and distance matching, but inconsistent elsewhere).

Many codes require that selected time histories should originate from earthquakes whose parameters match, within a given degree of tolerance, those of the controlling earthquake scenario. In the current Eurocode formulation the controlling scenario must be identified in terms of magnitude and site condition. Via the application of disaggregation it can also be assumed that source-site distance and $\varepsilon(T_1)$ are feasible outputs (where T_1 is the fundamental period of the structure under consideration). Exact matching of the scenario earthquake will, even for extensive strong-motion datasets, likely yield too few records for analysis. Instead, bin widths should be adopted. Recommended bin widths for magnitude are typically on the order of $\pm 0.2 M_w$ (Bommer & Acevedo, 2004) or $\pm 0.25 M_w$ (Stewart *et al.*, 2001). For source-site distance there is some greater disparity in the recommended bin widths, with windows of several tens of kilometres used in analyses (Bommer & Acevedo, 2004; Beyer & Bommer, 2007).

Some of the requirements for compatibility with the controlling earthquake scenario are not necessarily justified. In addition to the recommendation of wider distance bins (Bommer & Acevedo, 2004; Beyer & Bommer, 2007) several investigations have demonstrated that structural response has no significant dependence on the source to site distance (Shome *et al.*, 1998; Baker & Cornell, 2005; Iervolino & Cornell, 2005). When considering the first elastic mode of a structure there is some magnitude dependence, although this depends on the definition of the engineering demand parameter. The magnitude dependence may be attributed to the dependence of the record duration on the duration of fault rupture (Hancock & Bommer, 2006). Many structural analyses now aim for the objective of magnitude and distance sufficiency, i.e. conditional independence of both these quantities. This would make the scenario earthquake a secondary consideration to the selection of records, with the primary criteria relating to the record properties themselves.

The selection of strong-motion records whose properties match those of the hazard spectrum (either UHS or ERS) is also widely prescribed in codes. Most require that the response spectra of the selected time histories should not be less than some proportion (often 100 %) of the target spectrum for a given period range of interest. **EN 1998-1** specifies that the mean 5 % damped elastic spectrum of all the time histories should be scaled so that they are not smaller than site-adjusted PGA for $T = 0$, and no less than 90 % of the corresponding ERS in the period range $0.2T_1$ to $2T_1$. For bridges, where pairs of components are used, the requirements are strengthened such that the ensemble spectrum (from square-root sum of squares) should be scaled such that it is not less than 1.3 times the 5 % ERS in the period range $0.2T_1$ to $1.5T_1$, and the scaling factor should be applied to all individual time histories. Similar requirements are found in **FEMA 450**, albeit with the $0.2T_1$ to $1.5T_1$ range applied for all structures. There is no prescribed method for how to determine the match of the target spectrum in the considered codes, although matching criteria have been suggested by Beyer & Bommer (2007) and Iervolino *et al.* (2009). It should be noted that when the matching criteria are set for the records as an ensemble there can be enormous variability of ordinates within the set. This leads to instability in the estimates of inelastic response.

The use of the UHS (or the Newmark & Hall design spectrum) as the target for ground motion selection has come under some scrutiny. For a given return period, disaggregation of the controlling earthquake scenario can be applied to the UHS to determine the scenario most relevant for the structure under consideration (see section 5.2). The disaggregation should give the scenario event in terms of $M-R-\epsilon$, where ϵ represents the number of standard deviations by which the logarithm of the ground motion at T s differs from the median logarithm of ground motion at T s. Baker (2010) shows that for a ground motion approximately matching the UHS at period T_0 , the value of ϵ at other periods of motion differs considerably, albeit that the correlation between $\epsilon(T_0)$ and $\epsilon(T_i)$ increases as T_i tends towards T_0 . It is therefore highly unlikely to find a ground motion that approximately corresponds to the UHS across the period range of interest assuming that $\epsilon(T_0)$ remains unchanged. The UHS is a conservative approximation across the range of periods, thus making it a poor target for matching of time histories.

The conditional mean spectrum presented in section 5.3.2 may offer a more appropriate alternative as a basis for spectral matching than the UHS (Baker, 2010). This requires definition of the controlling earthquake scenario, in addition to identification of a GMPE appropriate to the hazard at the site under consideration and a correlation model for the spectral ordinates at different periods. The CMS will define lower spectral ground motion values than the UHS at higher and lower periods than T_0 . A detailed analysis of the impact of

time history record selection procedures on structural response (interstory drift ratio) has shown spectral matching to the CMS to be more accurate and precise, in terms of prediction of structural response, than other selection methods (Haselton *et al.*, 2009). That includes selection procedures currently implemented in design codes for ordinary buildings, UHS targeted scaling and scenario binning record selection resulting in a substantial overestimation of the structural response, by as much as 30 to 40 %.

For most ordinary structures the CMS is relatively simple to construct, in theory. As with the UHS selection procedure, the period of the structure under consideration should be known *a priori*. A spectral range around this period should be defined, ideally sufficiently widely as to allow for constraint of period elongation and/or higher mode effects. $0.2 T_0$ to $2 T_0$ may be sufficient for this purpose. The ε correlation model used for construction of a spectrum may be prescribed explicitly in a code, although allowance for correlation models corresponding to state-of-the-art strong-motion prediction models may be preferable. Also required are the M-R- ε properties of the controlling earthquake scenario (see section 5.3.2) relevant to the period of the structure under consideration. Where there is perhaps a greater challenge in reconciling the CMS with the hazard at a site, is in the use of the GMPE. The model selected for the construction of the CMS should be prominent within the suite of models used within the construction of the UHS. The approach implemented in the USGS Interactive Disaggregation may be suitable for this purpose as it is able to deconvolve the disaggregation for each ground motion model used. This allows for the identification of scenario events that are relevant to hazard at a site taking into consideration the GMPEs preferred in that region. Spectral matching of acceleration time histories to the target CMS is undertaken in the same manner, and therefore subject to the same considerations, as that of the UHS.

6.5. Scaling and Rotation of Time Histories

Scaling of records to match the target spectrum is required in most codes, yet with the exception of **NZS1170.5**, there is very little guidance as to how to approach this. Generally it is assumed that linear scaling is implemented, usually at the period relevant to analysis of the structure. Alternative methods such as wavelet transformation (Hancock *et al.*, 2006) may also be considered, but their use in such applications is not yet widespread. The scaling approach also depends on whether the analysis is uni- or bi-directional, although **EN 1998-2** specifications for bridges a clearer in describing the scaling needed for bi-directional motion. Where detail is provided it is common to see all records scaled by the same factor, usually determined from the SRSS matched spectrum. The approach listed in **NZS1170.5** may provide an alternative to this, requiring as it does a two step process for scaling the component pairs. Similarly the approach suggested by Beyer & Bommer (2007) may also be considered. This assumes that the geometric mean of the component pairs is scaled to the target level. The two individual records are then scaled by a factor of β and $1/\beta$ respectively to ensure a constant geometric mean.

Of the codes considered here, most base the record selection on the scenario earthquake of the spectral matching properties discussed here. Some may add additional requirements regarding the duration of strong-motion. Mostly these specifications apply to artificial time histories (e.g., **EN 1998**), although some may apply to real records (e.g., France, 1990; Turkey, 2007). Duration provisions are usually intended to ensure that the record is sufficiently long to accurately describe motion at the fundamental period of the structure under consideration. The impact of strong-motion duration on structural response is investigated by Hancock &

Bommer (2006), who suggest the influence is strongest when energy-based models of structural response are considered. There is little to suggest that additional characterisation of duration is explicitly needed in design codes. Current selection criteria for magnitude and requirements for spectral matching at T_1 (or up to $2T_1$) both ensure that longer periods are adequately represented for the scenario and structure under consideration.

Although there are still many issues that require further resolution in the use of time histories for structural analysis, one limiting factor has been the availability of strong-motion data and appropriate tools to implement the selection criteria required. Many strong-motion databases are widely disseminated via the Internet, a tool that has developed significantly since most code provisions were created. This may provide necessary information, yet the selection process may not be as adequate. A standard computational process has yet to be developed for artificial time history simulation and different methodologies may still produce inappropriate time histories, even if they conform to the standards required by a code. A more detailed quality assurance process may be required if artificial time histories are to continue their use as input for dynamic analysis. For real record selection, some code compatible selection programs have been developed such as REXEL (Iervolino *et al.*, 2010) and the Design Ground Motion Library (Power *et al.*, 2004). REXEL utilises records from the European Strong-Motion Database (Ambraseys *et al.*, 2004) as well as the Italian Strong-Motion Database (Montaldo *et al.*, 2007) as a basis for matching to **EN 1998** and **NNTC-Italy (2008)** hazard spectra. Whilst the program is directed towards selection of time histories for Eurocode or Italian code analysis, it may serve as a potential blue print for an open source system where additional modules and databases can be added to incorporate more seismic design codes. The Design Ground Motion Library selects accelerograms from the NGA database, and allows for some flexibility in the definition of the target spectrum.

The application of dynamic analysis has clearly become a recognised practice in most seismic design codes. There is a growing disparity, however, between the current state of knowledge regarding best-practice and the requirements of even the most recent codes. Discussion of time history analysis has been included in this report so that the issue remains pertinent throughout the SHARE project, even if improvements to the process are not an expected deliverable. Many objectives of SHARE are relevant to the selection process, and it may logically follow that where recommendations are made to revise **EN 1998** in light of SHARE outcomes, revisions to the dynamic analysis provisions should not be far behind. This discussion illustrates some of the factors that should be taken into consideration for future revisions of Eurocode to represent best practice.

7. Discussion & Conclusions

The facets of seismic input for building design considered within this report represent some of the most critical areas in translating state-of-the-art seismic hazard analysis into effective seismic design. There are many areas in which different seismic codes diverge in their requirements for seismic input, and also many more areas where similarities exist to the extent that they could be considered “standard practice”. Within each of the codes, however, there lies a balance between implementation of new approaches to characterising seismic input, and the practical requirements for these approaches to be widely adopted.

It is clear that the advent of web dissemination tools may allow for greater scope in the information it is possible to provide within a design code. For many current codes, such tools were not available at the initial development stages so most are limited in the full scope of seismic information made available to the designer, with the notable exception of the **FEMA 450** and **NNTC-Italy (2008)**. A similar web dissemination tool is an expected output of the SHARE project. It is with this in mind that some suggestions can be made as to how to approach future revisions to Eurocode 8.

The suggestions that follow should not be interpreted as expert recommendations for the revision of Eurocode. These recommendations will follow the completion of several other tasks within the SHARE project. Instead they summarise some of the key points of the topics discussed here. Some may be interpreted as actions that could be taken within the next revision of the code, whilst others may be seen as potential objectives for the basis of intermediate and long term code development. The time frame for potential implementation of the suggestions given is therefore not specified.

1) Return Periods

The growing integration of performance-based design requirements into seismic design codes is a positive step towards further reducing losses in both moderate and major earthquakes. This is generally well-reflected in the “no-collapse” and “damage limitation” requirements in Eurocode. For the “no-collapse” and/or “life safety” requirements, it can be seen from Table 2.3 that the 475 year return period forms the basis for seismic design for many codes. However, several countries (US, Mexico and Canada) have already moved to the 2475 year return period as a basis for seismic design in accordance with the approach taken in **FEMA 450** and the International Building Code (2009), with more expected to do so in forthcoming code revisions (e.g., Indonesia).

As the return period for each of the two design requirements is a nationally determined parameter, there is little merit in suggesting that **EN 1998** consider adopting the 2475 year return period as a basis for design. Instead this decision will be left to the respective national authorities. As in the United States, it is expected that in areas of lower seismicity in Europe the risk may be controlled by very low probability large earthquakes. In the United Kingdom, for example, it is suggested that design for ground motions with a return period of more than 2000 years would be needed to give the same reliability at the ultimate limit state as the current requirements for gravity and wind loads (Booth & Baker, 1990; Booth *et al.*, 2008). This situation will be reflected to varying degrees in the provisions for other low and low- to moderate seismicity countries.

Arguably the most rational approach to ensure reliability levels consistent with those of other hazards, is to allow countries to select their design return periods according to the integrated seismic risk. It is therefore necessary for the SHARE output to be readily adapted to the design levels required by the end user. Whilst provision is made for interpolating between fixed return periods, the basis for the Eurocode recommendation is possibly spurious. As has been shown, interpolation via the scaling factor k is a convenient approximation, but adopting a fixed value of $k \sim 3$ is erroneous. The k -value depends on the overall shape of the hazard curves at a site, and the period under consideration. The figures in section 2 also indicate that the k -value approximation is clearly limited over a range of return periods. There are several options to pursue to ensure that the output of SHARE can be easily adopted in accordance with the NDPs. Within the expected web portal output it should be possible for the user to determine acceleration and/or displacement, both PGA and spectral ordinates, at fixed return periods. Should the user demand an intermediate return period it is perhaps more appropriate that interpolation between the fixed returns is implemented within the program rather than by the user. For transparency, however, there may be some merit in indicating which fixed return periods have been used for interpolation and the accompanying k -values for each ordinate of the hazard spectrum.

2) Risk-Based Design

The integration of structural performance into the seismic input demands found in the 2009 NEHRP *Provisions* may provide a useful springboard from which risk-based design is incorporated into building codes. Clearly defining the input motion in terms of the probability associated with structural collapse rather than the probability of exceedence may be more useful to engineers and decision makers. If the risk-targeted coefficient is correctly determined, possibly as an NDP, then this will allow for risk to be evaluated in the context of local or existing design practice. This would allow national authorities greater autonomy over their design standards, whilst still fulfilling the Eurocode objective of standardising the internal market for engineering services. As is seen in the NEHRP *Provisions* (2009), the adoption of a risk-targeted earthquake motion reduces the seismic demand in almost all low-to-moderate seismicity regions. Where there is difficulty in adopting the risk-targeted approach is in the consideration of other limit states. It remains to be established how well the risk-targeted approach can be implemented within the damage limitation and/or serviceability limit states.

This is a clear distinction between the implementation of performance-based seismic design in American codes compared to those of Europe or New Zealand. The demands of the Eurocode approach, and the need to consider the structural performance at lower limit states, may require the use of a fully risk-targeted approach. The iterative loss estimation method suggested by Bommer *et al.* (2005) may have some potential here. There is a cost in terms of computational complexity, and possibly a lack of transparency to decision makers unfamiliar with the process of loss estimation, in adopting such an approach. It therefore needs to be determined, in the context of the SHARE output, how efficiently this approach can be implemented and the likely implications in terms of accuracy and increased or decreased design cost for ordinary building stock.

3) Site Condition

The analyses of provisions for determining site condition clearly show a degree of convergence in the classification of soils. Most adopt a four- or five-class scheme whose

categorisation is based on V_{s30} , N_{SPT} or C_u . These generally make a distinction between rock (or in the case of the NEHRP *Provisions* rock and hard rock), stiff soils, soft soils and special or liquefiable soils. Although the geotechnical definitions may vary between different codes, there may be little difference in the practical classification for a given soil profile. It can be seen clearly that the NEHRP and Eurocode categorisations will gradually replace the three-class (rock, stiff soil and soft soil) categorisations found in earlier codes. In terms of shallow site classification there is very little in seismic design practice around the globe to suggest that substantial revision of the soil categorisation scheme is needed in the near future. Increasing the amount of geotechnical information required to determine the site class may require a degree of site investigation that may be unnecessarily costly for ordinary structures.

Within the **EN 1998** provisions, there is clear scope for the adaptation of site response and classification to allow for the inclusion of the effects of deeper geology on the seismic waveform. This presents a greater challenge from the perspective of code design, which needs to take into consideration the relative cost versus benefit of constraining deeper geological profiles, and the uncertainty of the effects on the seismic motion at a site. Arguably the most cost effective approach to characterising the geology at greater depths is via array analysis of ambient shear-wave velocity (e.g., Bonnefoy-Claudet *et al.*, 2006; Hagshenas *et al.*, 2008, and references within). These techniques may determine, with uncertainty, the depths of sediment layers and their accompanying shear-wave velocity characteristics. It has also been demonstrated that the H/V ratios of ambient microtremor may reveal characteristics of the 2D and 3D profile of the site (Guiller *et al.*, 2006; Tokimatsu *et al.*, 2004), as well as basin resonance (Roten *et al.*, 2006; Roten *et al.*, 2009). It should be recognised, however, that whilst ambient vibration analysis may be useful in providing low cost, first order estimates of the soil profile, their ability to constrain the actual site amplification is limited. Nevertheless, it is not unreasonable to expect that H/V spectral ratio analysis of ambient microtremor may play a significant role in the characterisation of the deep geological profile of a site. Given the influence of experimental (Chatelain *et al.*, 2008) and instrumental conditions (Guiller *et al.*, 2008), in addition to the uncertainties and limitations of the method (SESAME WP12, 2004), it is likely that additional provisions may be needed for the use of ambient microtremor in classifying deep geology.

An additional difficulty in the consideration of deep geology and 2D/3D effects is in their influence on dynamic analyses of structures. Further constraints on the selection of time histories due to deeper site characteristics may further limit the number of real accelerograms available for analysis. This is made additionally complicated in Europe where site classification is not always consistent or robust for many of the available strong-motion records. Most are classified according to general soil type, whilst a fraction may have reliable V_{s30} estimates. The vast majority of records lack any metadata regarding the deeper geology of the site, so time-history selection via matching of the site classification factors not be practical, or may drastically limit the number of available records. Similarly, the influence of 2D/3D effects and basin resonance on the real accelerograms may result in bias when scaled to higher or lower intensity levels. Where basin resonance may alter spectra in such a way as to adversely affect the structure, it may be more appropriate for acceleration histories to be generated via numerical simulation.

Whilst site classification may be an important issue, the modelling of the effect of site amplification on the hazard spectrum remains an area of considerable uncertainty. Both **EN 1998** and **NZS1170.5** allow both the shape and the intensity of the spectrum to change according to the site condition. This clearly results in a relative increase in long-period

amplitude for softer soil sites. The approach to spectral modification in **NNTC-Italy (2008)** code may offer some potential as a means of characterising the site influence in terms of a continuous function, rather than fixed shapes for discrete classes. This also requires that the corner period of the constant velocity portion of the UHS (T_C) is specified *a priori*.

4) Hazard Spectrum

One of the most significant areas where the seismic input definition in **EN 1998** differs from that of other state-of-the-art codes is in the characterisation of the elastic response spectrum. In anchoring the full acceleration spectrum to PGA and allowing for the influence of magnitude only via the selection of one of two spectral shapes, there is significant disparity between the code response spectrum and both uniform hazard spectra and scenario spectra (Bommer & Pinho, 2006; Bommer *et al.*, 2010). Whilst many of the parameters used to define the spectra may be altered by National Annex, this simplification may mask the influence of many different seismological factors upon seismic hazard at a site.

It can be clearly seen in other seismic design codes that constraint of the elastic response spectrum via two or more spectral ordinates is both practical and desirable. As more countries adopt approaches similar to those of the 2009 International Building Code it is likely that this will develop into a standard in future code revisions. **NNTC-Italy (2008)** also illustrates how the elastic response spectrum can be constrained by more ordinates to provide greater consistency with the UHS, whilst still being practical to implement. The provision of parameters for the ERS for all permitted return periods in the form of a 211 page portable document file may not necessarily be the most elegant approach. With the development of a web portal for dissemination, however, there exists a practical means of allowing designers to select more ordinates relevant to the site to better constrain the hazard spectrum. The approach suggested by Bommer *et al.* (2010) allows for the construction of a more realistic spectrum based on empirical relations between PGV/PGA ratio and key parameters of the elastic response spectrum. This requires only three separate inputs (assuming T_D is constrained via the long-period controlling scenario) from the seismic hazard analysis. The use of empirical relations may also allow National Annexes to modify the coefficients of the equations to ensure a more appropriate fit to seismic hazard. Alternatively, the use of an open web portal for dissemination of seismic hazard output should provide a means of constructing a full (with the period limitations of the ground motion model) acceleration and displacement hazard spectrum.

A more open question regarding improvements to the characterisation of the response spectrum, is how to incorporate nonlinear factors such as site effects. As noted previously, **NNTC-Italy (2008)** code offers one potential route as it relates the corner periods of the spectrum closely to the hazard spectrum for a given soil type. Alternatively, a web portal may allow for the UHS to be calculated directly for a given site class, removing the need for modification by a single site coefficient. This would also allow for nonlinear amplification effects to be modelled according to the best practice within the ground motion prediction models, which will likely evolve more rapidly than revisions to the code provisions. Similarly, site specific features such as directivity and footwall/hanging wall effects may be incorporated directly into the hazard spectrum. This should result in a more realistic characterisation of hazard at a site, whilst still being feasible to implement given the SHARE output.

In terms of the displacement spectrum, **EN 1998** is ahead of many other codes in defining the displacement explicitly and providing additional constraints for very long-period motion. The challenge, therefore, is how best to determine the corner periods for low-frequency spectral displacements. These may be estimated using the magnitude of the controlling earthquake for long-period displacements (e.g., *NEHRP Provisions*), or via characteristics of the displacement response spectrum at long periods (e.g. Faccioli & Villani, 2009). As the number of long-period displacement records from large earthquakes increases it is hoped that long-period motions will be better constrained within ground motion prediction models. This should result in reliable estimates of acceleration and displacement hazard spectra to periods in excess of 10 s. Direct use of the UHS will, again, improve the characterisation of long-period response as ground motion models improve in these areas. Until this occurs it is likely that the corner periods T_D , T_E and T_F may be constrained using fixed ordinates of the form presently shown in Eurocode, or via proxies such as the ratio of long-period displacement to PGV (Faccioli & Villani, 2009).

5) Zoning

Comparison of the seismic design provisions published before 2000, and those published after the **FEMA 450** clearly shows a shift away from seismic zones of fixed acceleration and towards direct representation of the seismic hazard. Whilst the use of zones may provide a simple means of characterising the seismic hazard, it decouples the seismic input for engineering design from the actual hazard at a site. If a more scenario specific approach to defining the seismic input is required then seismic zones become increasingly incompatible with the design requirements. They also limit the ability to constrain the hazard spectrum as zonation can only be done on the basis of a small number of parameters. Even then, the process by which zones are defined becomes increasingly opaque and more reliant on judgement. The relative role that short-period and long-period hazard play in defining the extent of a zone is subjective, and in some cases may require separate zonations. Such a solution is inelegant and there are no precedents for separate zonations in existing design codes (IAEE, 2000; 2008), although a modification factor is present in the 1992 Spanish code to allow for the influence of large Atlantic earthquakes on the hazard spectrum. It is therefore suggested that in light of the SHARE output the use of zones be reconsidered in future Eurocode revisions.

6) Disaggregation

The need to identify the scenario(s) that contribute most significantly to seismic hazard at a site is a common facet of many codes. In some cases this provision only exists for the selection of accelerograms, whilst in others the controlling scenario may form an important basis for defining the elastic response spectrum. Both requirements are present in **EN 1998**, with the response spectrum type being selected on the basis of surface wave magnitude, and the time histories selected. Whilst the use of disaggregation (McGuire, 1995; Bazurro & Cornell, 1999) is not explicit in **EN 1998**, it is the most likely tool to identify controlling earthquake scenarios. There are several issues arising from the use of disaggregations that may need greater control if it is to become the primary tool for identifying design earthquake scenarios in codes. Firstly, it is now standard practice to define the controlling earthquake in terms of the M - R - ϵ triple. If the controlling magnitude is to be selected on this basis it is necessary to take into consideration the conditionality upon ϵ , possibly adding explicit provision that the type 2 spectrum should not be adopted if the controlling magnitude is not significantly smaller than M_S 5.5 and ϵ exceeds 2 (for example). Adoption of the probabilistic

disaggregation methodology outlined by Pagani & Marcellini (2007) may assist in defining an appropriate probability level for this criterion. The second consideration is the specification of the M-R- ϵ bin width. As noted by Bommer & Acevedo (2004), for the selection of time histories, a narrower magnitude bin width is preferred, and can be compensated for by a wider distance bin width. Finally, it is necessary to consider how the output of the disaggregation should be interpreted and used as a basis for selecting scenario earthquakes. It may be the case that to develop a standardised approach to interpreting and using disaggregation output it may be necessary to produce an additional informative annex. The USGS disaggregation web tool clearly provides a useful interface by which controlling scenario can be identified, and is accessible to engineers. The output from this program does not resolve the issues discussed here, but it does produce greater transparency.

The use of disaggregation provides further impetus to abandon the use of seismic zones in future Eurocode revisions. The degree of smoothing within the zonation means that for many sites the controlling earthquake scenarios based on disaggregation of the fixed a_0 level has no connection to the actual controlling scenario for a site. Instead it may be preferable to allow for disaggregation of PGV and/or longer period spectral accelerations, so as to ensure that the controlling scenarios are more relevant to the response of the structure under consideration. The USGS disaggregation portal allows for this, whilst the INGV portal only provides disaggregations for PGA. For long-period motions where the corner period of the displacement spectra are closely related to magnitude it is important that the disaggregation reflect the controlling magnitude for long-period motion rather than that for PGA.

7) Damping

Current **EN 1998** provisions for damping modification allow for a more efficient and flexible approach than provisions in many other codes. Damping modification is dependent on the duration of strong-motion, possibly via the effective number of cycles (Bommer & Mendis, 2005; Stafford *et al.*, 2008b) and on the period of motion (Cameron & Green, 2007; Hatzigeorgiou, 2010). Several new damping modification models have been suggested (e.g., Stafford *et al.*, 2008b; Hatzigeorgiou, 2010) as feasible alternatives. These would, however, require the definition of the effective number of cycles (or duration) of ground motion within the hazard analysis. Whilst this is feasible due to new GMPEs developed for this purpose (Bommer *et al.*, 2009; Stafford & Bommer, 2009), the additional hazard values may be inconsistent with the scenario earthquake identified via the disaggregation of the response spectrum. Bommer *et al.* (2010) suggest that the damping modification may be expressed as an empirical function of PGV/PGA ratio, which may make it feasible to implement within code requirements, albeit subject to constraint of the coefficients of the model. There are clearly several options available, but it remains to be seen how these alternatives perform when applied to Europe-wide hazard spectra.

8) Epistemic Uncertainty

The output of epistemic uncertainty analyses is rarely considered in design codes, yet it is an inherent part of seismic hazard analysis. It is not envisaged that explicit provisions for the construction of epistemic uncertainty analysis should be incorporated into codes, although some quality control guidelines may be of benefit to seismic hazard practitioners as well as designers. As epistemic uncertainty analyses become standard practice, some consideration should be given as to how they can influence design. The simplest approach may be to apply separate structural analysis for the median and a higher fractile of motion. The cost of

additional analysis, as well as the implications of designing to a higher level of ground motion, may be prohibitive, however. A slightly different approach has been considered here, one that uses the distribution of the epistemic uncertainty as a basis for modification of the design level according to importance category. This is an alternative to the use of importance coefficients that remain fixed to specific values, regardless of the implications of modification of the return period. A site-specific or region-specific importance coefficient may ensure that the degree of conservatism in the design is appropriate to the seismic hazard within the region.

The incorporation of epistemic uncertainty into design codes is clearly a matter that will require some debate before any revision can be made to the building code. As with many of the issues raised in this report, a more sound judgement may be made once the seismic hazard output from SHARE is available. From the perspective of dissemination there is clear scope for providing higher fractiles of seismic hazard from the logic tree analysis within the web portal. This has already been achieved for the Italian national seismic hazard maps (<http://esse1.mi.ingv.it/>). If such output is envisaged within SHARE, then further guidance may be needed to ensure that it is appropriately used within design practice.

9) Time Histories

The use of linear and nonlinear dynamic procedures for structural analysis is now widespread. Whilst most seismic codes contain provisions for the use of acceleration histories (real, artificial or synthetic), they rarely reflect the current state of knowledge in seismological and engineering practice. Some of the most crucial issues relating to selection and scaling of real accelerograms have been discussed here, and more information is available in the references cited. Similarly, the use of artificial and/or synthetic accelerograms has also been debated.

A full treatment of the use of time histories is beyond the scope of this report. However, the SHARE project should provide a basis for recommendations when revising provisions for the use of time histories. The definition of the hazard spectrum directly influences the approach taken to scale the time histories. Further refining the ERS to match the uniform hazard spectrum, whilst necessary in many ways, does not necessarily make selection and scaling of time histories a more accurate process as the UHS itself is an inappropriate target spectrum for this purpose. However, the definition of scenario events relevant to the fundamental period of the structure may help guide the selection of recordings, especially when used to define the conditional mean spectrum. To implement this approach it is necessary to constrain the seismic hazard across a range of spectral periods relevant to the types of structures considered within the code. This clearly illustrates the need for wider scope in the disaggregation procedure and the hazard spectrum to allow for identification of scenario events relevant to different parts of the spectra.

Whilst it is recognised that development of dynamic analysis procedures is not an objective of SHARE, any revisions to the definition of seismic input in Eurocode should take this into account. We suggest, therefore, that discussion of provisions for the selection and application of time histories continue throughout the course of the project.

Conclusions

The review of the seismic input provisions for different design codes shows that there are some aspects of practice that may reflect, more closely, the state of the art, than those

provisions given in Eurocode. There are also areas, particularly relating to performance- and displacement-based design where the Eurocode approach may be preferable. The topics that have been the primary focus of discussion here are those for which the SHARE output may be most relevant in making recommendations for future revision. A balance may need to be struck between ensuring that the provisions reflect the state-of-the-art in seismic hazard practice, and the practicality of implementation.

Many issues relating to practicality may depend on the means of dissemination for the SHARE output, and in particular the web portal. Some of the suggestions made here would require that more information is made available to the engineer than can be effectively represented by a map or suite of maps. The use of web output, be it via an online database or a web application, provides practical tools for allowing engineers access to more relevant hazard information, without necessarily requiring expertise in the calculation of seismic hazard. The greatest danger when providing such information is the potential for misinterpretation or misuse. The role of the seismic design code in these circumstances is to ensure that the available hazard output is used effectively. Consequently it is expected that an interface is needed between code regulations and data provision. Such an interface is becoming established in the NEHRP *Provisions*, and it is expected that Eurocode should develop in a similar manner. If the implementation of performance-based design provisions eventually leads to risk targeted or cost-benefit design, the need for closer interaction between design and hazard analysis will grow. If Eurocode is to integrate the progress made in seismic design, then these issues should be among the many considered.

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APPENDIX A: Nationally Determined Parameters in EN 1998

Table A.1: Nationally Determined Parameters in EN 1998

EN 1998-Reference	Item
Part 1: General Rules, Seismic Actions and Rules for Buildings	
1 2.1(1)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
1 2.1(1)P	Reference return period T_{DLR} of seismic action for the no-collapse requirement (or, equivalently, reference probability of exceedance in 50 years, P_{DLR}).
1 3.1.1(4)	Conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classification may be used.
1 3.1.2(1)	Ground classification scheme accounting for deep geology, including values of parameters S , T_B , T_C and T_D defining horizontal and vertical elastic response spectra in accordance with 3.2.2.2 and 3.2.2.3.
1 3.2.1(1), (2), (3)	Seismic zone maps and reference ground accelerations therein.
1 3.2.1(4)	Governing parameter (identification and value) for threshold of low seismicity.
1 3.2.1(5)	Governing parameter (identification and value) for threshold of very low seismicity.
1 3.2.2.1(4), 1 3.2.2.2(1)P	Parameters S , T_B , T_C and T_D defining shape of horizontal elastic response spectra.
1 3.2.2.3(1)P	Parameters a_{vg} , T_B , T_C and T_D defining shape of vertical elastic response spectra.
1 3.2.2.5(4)P	Lower bound factor β on design spectral values
1 4.2.5(5)P	Importance factor γ_I for buildings.
1 4.3.3.1(4)	Decision on whether nonlinear methods of analysis may be applied for the design of non-base-isolated buildings. Reference to information on member deformation capacities and the associated partial factors for the Ultimate Limit State for design or evaluation on the basis of nonlinear analysis methods
1 4.4.3.2(2)	Reduction factor v for displacements at damage limitation limit state.
Part 2: Bridges	
2 2.1.3(P)	Reference return period T_{NCR} of seismic action for the no-collapse requirement of the bridge (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).

Table A.1 (continued)

2 2.3.7(1)	Identification of the case of low seismicity
2 3.2.2.3	Definition of active fault
2 3.3(6)	Distance beyond which the seismic ground motions may be considered as completely uncorrelated.
Part 3: Assessment and Retrofitting of Buildings	
3 2.1(2)P	Number of limit states to be considered
3 2.1(3)P	Return periods of seismic actions under which the Limit States should not be exceeded.
Part 4: Silos, tanks and pipelines	
4 2.1.2(4)P	Reference return period T_{NCR} of seismic action for the ultimate limit state (or, equivalently, reference probability of exceedance in 10 years, P_{NCR})
4 2.1.3(5)P	Reference return period T_{DLR} of seismic action for the ultimate limit state (or, equivalently, reference probability of exceedance in 10 years, P_{DLR})

APPENDIX B: Site Classifications in other Listed Codes

B.1. NBC (2005)

The site classification scheme in the Canadian building code [NBC 2005] is identical to that of the FEMA 450 and International Building Code shown in Table 3.2. The code indicates that the site should be determined using V_{S30} where available. If V_{S30} is not available then N_{SPT} or C_u shall be used to determine site class, where N_{SPT} and C_u are calculated based on rational analysis. As with the FEMA 450, site specific special investigation and dynamic response analysis must be performed for class F sites.

B.2. NZS1170.5

The NZS 1170.5 code is something of a departure from the other codes considered here, as it aims to incorporate information about the characteristics of the site at depths greater than 30 m. It does this by defining soil classes on the basis of low-amplitude site period (T_0). This is determined from the quarter-wavelength approximation: $T_0 = 4h/V_s$, where h is the depth to the surface of the interface of seismic impedance. Where V_s is not known directly T_0 can be estimated from the resonant period identified in the horizontal-to-vertical spectral ratio (H/V) of ambient vibration (Nakamura, 1989). The adoption of the quarter wavelength approximation allows identification of the long-period response that is observed from deep deposits of stiff or dense soils or gravels, which are markedly different from those of deposits that are only a few tens of metres in depth.

Table B.1a: General site classification scheme in NZS 1170.5

Class A	Strong to extremely strong rock	Unconfined compressive strength (C_u) > 50 MPa $V_{S30} > 1500$ m/s Not underlain by material with $C_u < 18$ MPa or $V_{S30} < 600$ m/s
Class B	Rock	$1 \leq C_u$ (MPa) ≤ 50 ; $V_{S30} \geq 360$ m/s Not underlain by material with $C_u < 0.8$ MPa or $V_{S30} < 300$ m/s Surface layer of ≤ 3 m of highly-weathered rock ($C_u < 1$ MPa) may be present
Class C	Shallow Soil	Not class A, B or E Low Amplitude natural period ≤ 0.6 s Soil Depths to not exceed those in suggested Table 3.3b
Class D	Deep/Soft Soil	Not Class A, B, or E Low Amplitude natural period > 0.6 s Soil Depths to not exceed those in suggested Table 3.3b Underlain by < 10 m of soils (Undrained shear strength (S_{TRU}) < 12.5 kPa; $N_{SPT} < 6$)
Class E	Very Soft Soil	> 10 m very soft soils ($S_{TRU} < 12.5$ kPa) OR > 10 m soils with $N_{SPT} < 6$ OR > 10 m soils with $V_{S30} < 150$ m/s OR > 10 m combined depth of soils with any of the described properties above

A description of the site classification hierarchy is provided in the code. The shear-wave travel time is the preferred method, followed then by borehole logs and geotechnical measurements or H/V spectral ratio or recorded earthquake motions. In the absence of these measurements, classification may be made on the basis of descriptive borehole information. Knowledge of surface geology and estimates to bedrock are the least preferred method. Two rock classes and three soil classes are defined according to the criteria shown in Table B.1a and B.1b.

Table B.1b: Soil Type and Description (for use in classes C or D)

Cohesive Soil	Representative undrained shear strengths (kPa)	Maximum Depth of Soil (m)
Very Soft	< 12.5	0
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very stiff or hard	100 – 200	60
Cohesive Soil	Representative N_{SPT} Values	
Very loose	< 6	0
Loose dry	6 – 10	40
Medium dense	10 – 30	45
Dense	30 – 50	55
Very Dense	> 50	60
Gravels	> 30	100

For layered sites the natural period of the site can be estimated by summing the contributions from each layer. This is done by calculating the ratio of the layer’s thickness to the corresponding maximum depth of soil for the given class (Table B.1b column 3), then multiplying by 0.6.

The code also makes provision for piled foundations, indicating that the shallow classification be used for the site even if the piles penetrate into deeper, more consolidated rock or soil. This is because the shallow geology may still influence the site response, except in situations of sleeved piles or specifically designed separation of the structure from the surrounding soil, whereby the basement motions will clearly affect the structure.

Despite the classification of two different rock types, the same spectral shape factor presented in the code is the same for classes A and B. A distinction between the two is made to allow for the existence of a weathered layer.

The treatment of saturated loose sands, combinations of sand and silt, uncontrolled fill and other liquefiable soils is not specifically addressed in the code. As with the NEHRP and EN1998 guidelines, “special studies” are necessary to determine the spectra for such sites.

B.3. NNTC-Italy (2008)

The soil classification in the Italian building code is almost identical to the classification in **EN 1998-1**. Only minor differences arise from some of the definitions of layer thickness, e.g. a weathered layer of only 3 m is permissible for a Class A site, compared to 5 m in **EN 1998-1**. In all cases the V_{S30} , N_{SPT} and C_u criteria are identical.

B.4. BSL-Japan (2000)

As the Japanese code is separated into specific areas according to structure type, so too are the definitions of ground condition. For many of the classification schemes it is natural period (T_0) of the surface layer that forms the primary basis for delineating a soil type. For other structures a simpler three-tier classification system is applied, with allocation made on the basis of geology. For some structures, most notably high-pressure gas pipelines and port facilities, the influence of ground motion is incorporated directly into the calculations rather than via parameterisation into separate categories. A summary of the classifications follows:

B.4.1. Ordinary Buildings

Three ground types are considered (Type I, II and III), whose definition is based on descriptive properties of the soil (Table B.2). Each ground type is assigned a value of the parameter T_c , which adjusts the response spectrum (section 4.3.5)

Table B.2: Site classification for buildings in BSL-2000

Ground Type	Ground Definition	T_c (s)
Type I	Ground consisting of mainly rock mass or hardened gravel beds from the Tertiary era or earlier, or ground that has been shown on the basis of surveys or studies concerning ground periods, etc., to have a ground period that is equivalent to these.	0.4
Type II	Ground types other than type I or type III	0.6
Type III	Alluvial layers consisting mainly of humus, mud or similar materials (including embankments, if these are present), to a depth of approximately 30 m or more, marsh or mud sea, etc., filled to a depth of approximately 3 m or more within a period of approximately 30 years, or ground that has been shown on the basis of surveys or studies concerning ground periods, etc., to have a ground period that is equivalent to these.	0.8

B.4.2. Highway Bridges

Three separate design spectra are presented corresponding to three soil conditions: Group I (stiff site), Group II (moderate site) and Group III (soft site). The definitions are presumed consistent with those for ordinary buildings.

B.4.3. Railway Structures

Explicit design spectra are provided for eight different ground conditions (Table B.3), determined by resonant period T_0 .

Table B.3: Site classification for railway structures from Japan (2000)

Soil Profile Type	Period (T_0) (s)	Profile Name/Description
G0	-	Hard Rock
G1	-	Bedrock
G2	≤ 0.25	Diluvium
G3	0.25 – 0.5	Dense Soil
G4	0.5 – 0.75	Dense to Soft Soil
G5	0.75 – 1.0	Soft Soil
G6	1.0 – 1.5	Very Soft Soil
G7	≥ 1.5	Extremely Soft Soil

As with the New Zealand code, the direct use of resonant period will take into account the depth of the soil layer.

B.4.4. Port Facilities

In the determination of the design seismic coefficient a subsoil factor is used, based on one of three types of ground condition. This is classified according to the thickness of quaternary deposit.

Table B.4: Soil classification for Port and Harbour Facilities (Japan, 2000)

Thickness of Quaternary Deposit	Gravel	Sand or Clay	Soft Ground
< 5 m	1 st	1 st	2 nd
5 – 25 m	1 st	2 nd	3 rd
> 25 m	2 nd	3 rd	3 rd
Classification	1st	2nd	3rd
Factor	0.8	1.0	1.2

However, given the high risk of liquefaction at such facilities, more explicit detail is given as to how to estimate the liquefaction potential. This is done using measurements of N_{SPT} and/or Undrained Cyclic Triaxial Tests.

B.4.5. Water Supply Facilities

As with the highway bridges, three categories of ground condition are considered. These are based on surface geology and resonant period (Table B.5).

Table B.5: Site classification for water supply facilities (Japan, 2000)

Ground Type	Period	Description
Type I	≤ 0.2 s	Ideal diluvial ground and “proper” rock bed
Type II	0.2 – 0.6 s	Either alluvial or diluvial categories
Type III	> 0.6 s	Poor ground and alluvial layers including sedimentary layers created by landslides, landfills and other weak ground

B.4.6. Gas Lines

Separate consideration is given to high pressure gas lines, compared with low and medium pressure lines. In the latter case, a three-tier site classification scheme is used, which is largely consistent with three-tier schemes used elsewhere in the code (Table B.6).

Table B.6: Site classification for gas lines (Japan, 2000)

Ground Type	Description
I	Soil Layer from Triassic era or earlier Diluvium Layer Alluvium layer ≤ 10 m thick, with soft layer ≤ 5 m thick
II	Alluvium > 10 m thick, or soft layer > 5 m thick
III	a) Mixture of layer equivalent to Type I and layer equivalent to Type II, or mixture of two types. b) Border between soil layer and sturdy structure built upon foundation equivalent to condition II, and other locations where displacement is evidently discontinuous.

For high pressure gas lines, however, the site amplification is calculated as a direct function of resonant period (T_0):

$$U = \frac{2}{\pi^2} \cdot T_0 \cdot v \cdot S_v(T) \cdot \cos\left(\frac{\pi z}{2H}\right) \quad (\text{B.1})$$

T_0 is calculated from the relation $T_0 = \frac{4H}{V_s}$, where H is the thickness of the surface layer(s),

$S_v(T)$ is the bedrock velocity spectrum, v is the seismic zone coefficient and z the depth of the buried pipeline.

B.5. Indonesia

A simpler soil classification scheme is implemented in the Indonesian seismic code (Table B.7). Here only three site conditions are considered, plus an addition category of “special soil”. As with many of the previous codes, the classification is largely based on V_{s30} , N_{SPT} and Shear Strength of the upper 30 m of the ground.

Table B.7: Site classification in the Indonesian (2002) Seismic Code

Soil Types	V_{s30} (m/s)	N_{SPT}	Shear Strength (S_u) (kPa)
Hard Ground	≥ 350	≥ 50	≥ 100
Medium Ground	175 – 350	15 – 50	50 – 100
Soft Ground	< 175	< 15	< 50
	Any soft profile where the total thickness is more than 3 m with $PI > 20$, soil moisture content $w_n \geq 40\%$ and $S_u < 25$ kPa.		
Special Soil	Special Investigation required for the site – includes soils with more than 3 m of soil with: More than 10 m of soft clay ($PI > 75$) More than 30 m of clay with $25 \text{ kPa} < S_u < 50 \text{ kPa}$		

B.6. Pakistan

The classification scheme found in the Pakistan Seismic Code largely follows that found in the NEHRP *Provisions*. Minor differences in the defined ranges of V_{s30} can be seen for Type B, C and D soils (Table B.8), but these are very minor and usually well within the typical error in the estimates of V_{s30} for a profile.

Table B.8: Site classification in the 2007 Pakistan Code

Soil Profile Type	Soil Profile Description	Average Properties for top 30 m of soil		
		Shear Wave Velocity (V_{s30} m s ⁻¹)	N_{SPT} (blows/foot)	C_u (kPa)
S_A	Hard Rock	> 1500	-	-
S_B	Rock	750 – 1500	-	-
S_C	Very Dense Soil & Soft Rock	350 – 750	> 50	> 100
S_D	Stiff Soil	175 – 350	15 – 50	50 - 100
S_E^1	Soft Soil	< 175	< 15	< 50
S_E^2	Soft Soil	Any profile with more than 3 m of soft clay, defined as a soil with: v) $PI > 20$ vi) Water Content $> 40\%$ vii) $C_u < 25$ kPa		
S_F	Special Investigation Required	i) Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick or highly sensitive clays, collapsible weakly cemented soils ii) Peats and/or highly organic clays (Thickness (H) > 3 m) iii) Very high plasticity clays (H > 7.5 m, with $PI > 75$) iv) Very thick soft/medium clays (H > 37 m)		

B.7. Site Classification in Other European codes

To illustrate the different ways in which the site characteristics have been classified in older European codes, a summary comparison is given in Table B.9. Many of these codes, particularly the older ones, classify the site according to descriptive criteria. A few of the codes use the classification parameters seen previously, which more closely resembles the standard classification scheme for many seismic codes.

Table B.9: Site classification in previous and current seismic design codes from Europe and the surrounding area (IAEE, 1996; 2008).

Code	Classes (Descriptions <u>NOT</u> taken verbatim from code)	Categorisation
Albania (1989)	<p>I: Rock (excluding weathered rock), compact gravel and unweathered marl</p> <p>II: Weathered rocks and marls; coarse gravels and medium-grained sands and gravels; compact fine-grained sand; clay-sand (stiff, semi-stiff & plastic)</p> <p>III: Fine grained sand (semi-compact); dusty sand; clay-sands (soft plastic)</p> <p>U: Unfavourable (liquefiable, steep gradient, karst development potential, rockfall, active/passive landslides, proximity to active faults)</p>	Descriptive
Algeria (2003)	<p>S1: Rock or other geological formation with $V_s \geq 800 \text{ m s}^{-1}$</p> <p>S2: Very dense gravel or sand and/or overconsolidated clay deposits</p> <p>S3: Thick deposits of moderately dense gravel and sand or moderately stiff clay</p> <p>S4: Loose sand deposits with or without soft clay, or soft to moderately stiff clay with $v_s < 200 \text{ m/s}$</p>	<ul style="list-style-type: none"> • V_s • N_{CPT} • N_{SPT} • Pressiometer • Compression strength • I_p
Bulgaria (2007)	EN 1998-1 Classification	
France (1990)	<p>S1: Bedrock sites; group A (good to very good mechanical resistance) soils $\leq 15 \text{ m}$ thick layer.</p> <p>S2: Group A soils $> 15 \text{ m}$ thick layer; Group B (average mechanical resistance) soils $\leq 15 \text{ m}$ thick layer.</p> <p>S3: Group B soils $15 - 50 \text{ m}$ thick layer; Group C (low mechanical resistance) $\leq 10 \text{ m}$ thick layer.</p> <p>S4: Group B soils $> 50 \text{ m}$ thick; Group C soils in $10 - 100 \text{ m}$ thick layer.</p> <p>U: Group C soils $> 100 \text{ m}$ thick require special investigation</p>	<ul style="list-style-type: none"> • Static penetration • N_{SPT} • Pressiometer • Unconfined compressive strength • Relative density • Compression index • V_s, V_p

Table B.9 (continued)		
Germany (1992)	<p>A: Areas of solid bedrock with velocities of typical bedrock V_s: A1 (solid-medium strength), A2 (unconsolidated coarse-grained and gravel) and A3 (fine grained unconsolidated aquifers)</p> <p>B: Transition areas between classes A and C: B2 (unconsolidated coarse-grained and gravel) and B3 (fine grained unconsolidated aquifers).</p> <p>C: Deep basement structures with sediment infill (C3)</p> <p>S: Separate consideration if $v_s < 150 \text{ m s}^{-1}$</p>	<ul style="list-style-type: none"> • Descriptive • Shear-wave velocity in the case of rock and special soils
Greece (2000)	<p>A: Unweathered rock; dense granular material; over-consolidated clay (< 70 m)</p> <p>B: Strongly weathered rocks & soils; medium density granular material (> 5 m thick); high density material (> 70 m); over-consolidated clay > 70 m.</p> <p>C: Granular material low relative density (thickness > 5 m) or medium density (> 70 m). Silty-clay soils < 5 m thick</p> <p>D: Soft clays with high plasticity index ($I_p > 60$) with total thickness > 12 m</p> <p>E: Loose sand; liquefiable soils; steep slopes; close proximity to faults;</p>	<ul style="list-style-type: none"> • Mostly descriptive • Plasticity index for some clays.
Hungary (1981)	<p>1: Rocky soils, coarse gravel.</p> <p>2: Medium quality soils (dry)</p> <p>3: Moist, less load-bearing soils.</p> <p>4: Soft, saturated soils.</p>	Descriptive
Macedonia (1981)	<p>I: Rock or rock-like ground and similar very dense or hard soils of thickness < 60 m, with stable layers of gravel, sand or stiff clay on top of a firm geological foundation</p> <p>II: Dense and medium dense soils; very dense soils > 60 m thick;</p> <p>III: Low density and soft soils > 10 m thick, consisting of loose gravel, medium-dense sand and clay, with layers of sand and other cohesionless soils</p>	Descriptive
Norway (1998)	<p>Nationally Determined Adaptation of 1998 Eurocode provisions:</p> <p>A: Rock or rock like formation ($V_s > 800 \text{ m s}^{-1}$); stiff deposits sand, gravel or over-consolidated clay (several 10s m thick) with gradual increase of mechanical properties at depth.</p> <p>B: Medium dense sand, gravel or medium stiff clays with thicknesses of several tens to hundreds of m. $V_s > 200 \text{ m s}^{-1}$ at a depth of 100m, increasing to > 350 m s^{-1} at 50 m.</p> <p>C: Loose cohesionless soil deposits with $V_s < 200 \text{ m s}^{-1}$ in upper 20 m Special definition of “alluvium”, may required site-specific studies</p>	<ul style="list-style-type: none"> • Descriptive • V_s

Table B.9 (continued)		
Portugal (1983)	I: Rock and stiff cohesive soils II: Very hard; hard and medium cohesive soils; hard cohesionless soil. III: Very soft and soft cohesive soil; loose cohesionless soils	Descriptive.
Romania (2007)	A: Rock-type ground ($V_{s30} \geq 760 \text{ m s}^{-1}$) B: Hard Ground ($360 < V_{s30} (\text{m s}^{-1}) < 760$) C: Intermediary ground ($180 < V_{s30} (\text{m s}^{-1}) \leq 360$) D: Soft ground ($V_{s30} \leq 180 \text{ m s}^{-1}$)	<ul style="list-style-type: none"> • V_{s30} • Determined from H/V ratio of ambient vibration
Serbia (1981)	As Macedonia (1981)	
Spain (1992)	I: Compact rock, cemented or very dense deposits ($V_{s30} \geq 750 \text{ m s}^{-1}$) II: Compact or cohesive granular deposits situated above the water table ($400 < V_{s30} (\text{m s}^{-1}) < 750$) III: Soft granular soil or soft and medium cohesive soils ($V_{s30} < 400 \text{ m s}^{-1}$)	<ul style="list-style-type: none"> • Descriptive • V_{s30}
Switzerland (2003)	A: Firm rock or soft rock beneath a maximum soil cover or 5 m B: Extensive cemented gravel and sand and/or overconsolidated soils $\leq 30 \text{ m}$ C: Normal consolidated and uncemented gravel, sand or moraine $> 30 \text{ m}$ D: Unconsolidated fine sand, silt or clay ($> 30 \text{ m}$) E: Alluvial surface layer (C or D) with a thickness of 5 – 30 m lying above ground classes A or B. F: Structurally sensitive or organic deposits with thickness $> 10 \text{ m}$	<ul style="list-style-type: none"> • V_{s30} • N_{SPT} • C_U • Some alteration from EN 1998-1 definition.
Turkey (2007)	S1: Group A soils (unweathered rock, very dense sand/gravel, hard clay) or Group B (soft/weathered rock, dense sand/gravel, stiff clay) $\leq 15 \text{ m}$ thickness S2: Group B soils $> 15 \text{ m}$ thick or Group C soils (highly weathered soft rocks, medium-dense sand/gravel, stiff/silty clay) $\leq 15 \text{ m}$ thick. S3: Group C soils 15 – 50 m thick, or Group D soils (deep alluvium, high water table, loose sand, soft/silty clay) $\leq 10 \text{ m}$ thick. S4: Group C soils $> 50 \text{ m}$ thick, or group D soils $> 10 \text{ m}$ thick	<ul style="list-style-type: none"> • Relative Density • N_{SPT} • Unconfined Compressive Strength • V_{s30}

APPENDIX C. Construction of the Design Spectra in Listed Codes

C.1. NBC (2005)

The elastic response spectrum is fixed to spectral acceleration at four periods: 0.2 s [$S_a(0.2)$], 0.5 s [$S_a(0.5)$], 1.0 s [$S_a(1.0)$], 2.0 s [$S_a(2.0)$]:

$$T(s) \leq 0.2: \quad S(T) = F_a S_a(0.2) \quad (C.1a)$$

$$T(s) = 0.5: \quad S(T) = \begin{cases} F_v S_a(0.5) \\ F_a S_a(0.2) \end{cases} \quad \text{if} \quad \begin{cases} F_v S_a(0.5) \leq F_a S_a(0.5) \\ \text{Otherwise} \end{cases} \quad (C.1b)$$

$$T(s) = 1.0: \quad S(T) = F_v S_a(1.0) \quad (C.1c)$$

$$T(s) = 2.0: \quad S(T) = F_v S_a(2.0) \quad (C.1d)$$

$$T(s) \geq 4.0: \quad S(T) = F_v S_a(2.0)/2 \quad (C.1e)$$

F_v and F_a are acceleration-dependent site coefficients, given in Table C.1. These are given for site classes A- E (special investigation is required for class F). For intermediate values of $S_a(0.2)$ and $S_a(1.0)$ between the ordinates given, F_a and F_v are calculated via linear interpolation.

Table C.1: Site coefficients for NBC 2005

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.5$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.0$	$S_a(0.2) \geq 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	-	-	-	-	-
Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	-	-	-	-	-

C.2. NZS 1170.5

For each method four spectral shape factors are presented in the code. These correspond to the five soil types, with categories A and B (strong rock and rock) combined in one shape factor. In the code only key shape factors for regular spectral ordinates are presented; the full equations are given in the commentary. For MRS and NITH methods, the spectral shapes take the form $C_h(T)_{\Delta T} = f(T)_{\Delta T}$, where $f(T)$ is the given function of T in the spectral range ΔT . These functions and their relevant spectral ranges are given in Table C.2 for each site class, and illustrated in Figure C.1.

Table C.2: Definition of the design spectrum taken from NZS 1170.5 Commentary A

Class A & B	Strong rock & class B rock					
ΔT (s)	0	0 – 0.1	0.1 – 0.3	0.3 – 1.5	1.5 - 3	> 3
$f(T)_{MRS/NITH}$	1.0	$1.0 + 1.35(T/0.1)$	2.35	$1.60(0.5/T)^{0.75}$	$1.05/T$	$3.15/T^2$
ΔT (s)	0 – 0.4		0.4 – 1.5		1.5 - 3	> 3
$f(T)_{ESM}$	1.89		$1.60(0.5/T)^{0.75}$		$1.05/T$	$3.15/T^2$
Class C	Shallow Soil					
ΔT (s)	0	0 – 0.1	0.1 – 0.3	0.3 – 1.5	1.5 - 3	> 3
$f(T)_{MRS/NITH}$	1.33	$1.33 + 1.6(T/0.1)$	2.93	$2.0(0.5/T)^{0.75}$	$1.32/T$	$3.96/T^2$
ΔT (s)	0 – 0.4		0.4 – 1.5		1.5 - 3	> 3
$f(T)_{ESM}$	2.36		$2.0(0.5/T)^{0.75}$		$1.32/T$	$3.96/T^2$
Class D	Deep or soft soil					
ΔT (s)	0	0 – 0.1	0.1 – 0.56	0.56 – 1.5	1.5 - 3	> 3
$f(T)_{MRS/NITH}$	1.12	$1.12 + 1.88(T/0.1)$	3.0	$2.4(0.75/T)^{0.75}$	$2.14/T$	$6.42/T^2$
ΔT (s)	0 – 0.56			0.4 – 1.5	1.5 - 3	> 3
$f(T)_{ESM}$	3.0			$2.4(0.75/T)^{0.75}$	$2.14/T$	$6.42/T^2$
Class E	Very Soft Soil					
ΔT (s)	0	0 – 0.1	0.1 – 1	1 – 1.5	1.5 - 3	> 3
$f(T)_{MRS/NITH}$	1.12	$1.12 + 1.88(T/0.1)$	3.0	$(3.0/T)^{0.75}$	$3.32/T$	$9.96/T^2$
ΔT (s)	0 – 1			1 – 1.5	1.5 - 3	> 3
$f(T)_{ESM}$	3.0			$(3.0/T)^{0.75}$	$3.32/T$	$9.96/T^2$

This also corresponds to the value (in g) of the peak ground acceleration for site classes A and B for the same return period (**NZS-1170.5 commentary**). However, defined upper and lower limits are given to the value of Z that can be taken in New Zealand. The minimum bound is $Z = 0.13$ g (for $R = 1$). This value is derived from the 84th percentile ground motion from a normal faulting earthquake $M 6.5$, 20 km from the site, allowing for the margin against collapse (assumed to be 1.5, as in the International Building Code). This event corresponds to the largest magnitude earthquake likely to occur in low seismicity regions of New Zealand

(McVerry, 2003). The maximum bound on Z is that which produces $ZR_u = 0.7$. This particular value corresponds to the 84th percentile ground motion likely to be experienced at a site adjacent to the largest New Zealand fault (the Alpine fault), in the event of a M_w 8.1 earthquake (McVerry, 2003). Values of Z are given presented in the building code for 129 locations, with maps produced accordingly. For any site in New Zealand not explicitly listed, the values of Z should be interpolated from these maps (**NZS 1170.5**).

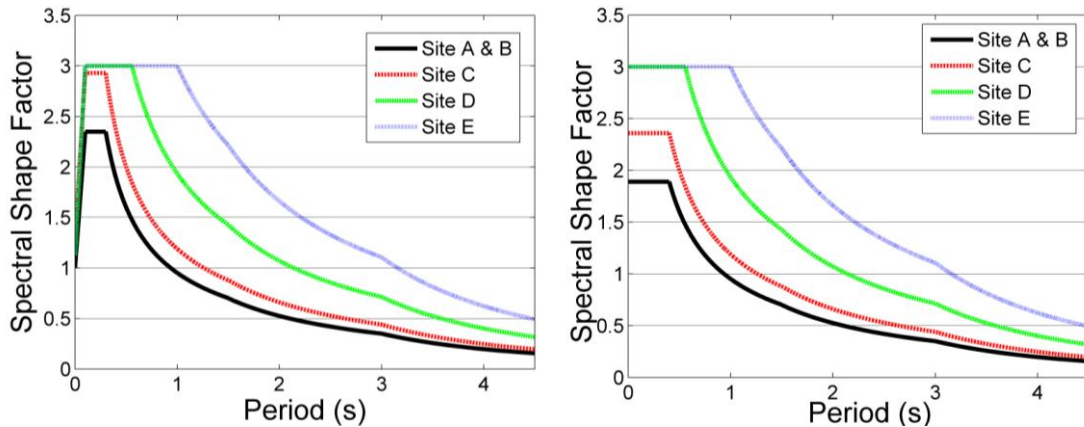


Figure C.1: Design spectra for a) Numerical Time History Analysis, b) General analysis, according to **NZS 1170.5**.

Scaling for near fault effects is only applied to longer period motion ($T \geq 1.5$ s). **NZS-1170** clearly indicates which faults should be considered for this term (a total of eleven faults). The source-site distances are given for each of the 129 localities listed in the code. Elsewhere, D is defined as the distance to the fault plane, which is determined from geological investigation. $N(T, D)$ is calculated by:

$$N(T, D) = \begin{cases} 1 & R \geq 1/250 \\ N_{\max}(T) & D \leq 2km \\ 1 + (N_{\max}(T) - 1) \left(\frac{20 - D}{18} \right) & 2 < D(km) < 20 \\ 1 & D \geq 20km \end{cases} \quad \text{for} \quad \begin{matrix} R \geq 1/250 \\ D \leq 2km \\ 2 < D(km) < 20 \\ D \geq 20km \end{matrix} \quad (C.2)$$

Where $N_{\max}(T)$ is defined according to Table C.3, where intermediate values of $N_{\max}(T)$ should be linearly interpolated.

Table C.3: Definition of $N_{\max}(T)$ according to **NZS 1170.5**

Period $T(s)$	$N_{\max}(T)$
≤ 1.5	1.0
2	1.12
3	1.36
4	1.60
≥ 5	1.72

The vertical response spectrum is also given consideration in **NZS-1170**. In the code itself it is simply defined as $C_v(T) = 0.7C_h(T)$. This is derived from the approximation factor of 2/3 to convert horizontal acceleration to vertical acceleration (Newmark & Hall, 1973). The influence of short source-site distances on V/H ratio is recognised in the commentary, where it is suggested that at distances of less than 10 km, vertical motion may equal horizontal motion for short-period motion ($T \leq 0.3$ s). This is not made explicit in the code, however.

C.3. NNTC-Italy (2008)

The elastic design spectrum defined in **NNTC-Italy (2008)** is constructed via:

$$0 \leq T \leq T_B: S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_0} \cdot \left(1 - \frac{T}{T_B} \right) \right] \quad (\text{C.3a})$$

$$T_B \leq T \leq T_C: S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \quad (\text{C.3b})$$

$$T_C \leq T \leq T_D: S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T_C}{T} \right] \quad (\text{C.3c})$$

$$T_D \leq T \leq 4.0s: S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T_C T_D}{T^2} \right] \quad (\text{C.3d})$$

where a_g is the peak horizontal acceleration at a site for the specified return period, and η a factor to account for the damping ($\eta = 1$ for 5 % damping). F_0 is the maximum amplification factor of horizontal acceleration, a parameter which is required for each location. S is the site amplification factor to take into account both soil (S_S) and topographic (S_T) amplification $S = S_S C_C$, where S_S and S_T are defined as in Table C.4 and C.5 respectively.

Table C.4: Calculation of S_S and C_C site amplification factors from the NNTC-Italy (2008)

Subsoil Category	S_S	C_C
A	1.0	1.0
B	$1.0 \leq 1.4 - 0.4 \cdot F_0 \cdot \frac{a_g}{g} \leq 1.2$	$1.1 \cdot (T_C^*)^{-0.20}$
C	$1.0 \leq 1.7 - 0.6 \cdot F_0 \cdot \frac{a_g}{g} \leq 1.5$	$1.05 \cdot (T_C^*)^{-0.33}$
D	$0.9 \leq 2.4 - 1.5 \cdot F_0 \cdot \frac{a_g}{g} \leq 1.8$	$1.25 \cdot (T_C^*)^{-0.50}$
E	$1.0 \leq 2.0 - 1.1 \cdot F_0 \cdot \frac{a_g}{g} \leq 1.6$	$1.15 \cdot (T_C^*)^{-0.40}$

T_B , T_C and T_D are determined as follows:

$$T_C = C_C T_C^* \quad (\text{C.4a})$$

$$T_B = T_C / 3 \quad (C.4b)$$

$$T_D = 4.0(a_g / g) + 1.6 \quad (C.4c)$$

Where g is the acceleration due to gravity and C_C a site-dependent factor given in Table 4.10.

Table C.5: Topographic amplification factors from the NNTC-Italy (2008)

Topographic Category	S_T
T1	1.0
T2	1.2
T3	1.2
T4	1.4

Vertical acceleration is also given explicit consideration in the **NNTC-Italy (2008)**. This is done using the formula:

$$F_V = 1.35 \cdot F_0 \cdot \left(\frac{a_g}{g} \right)^{0.5} \quad (C.5)$$

Where F_0 , a_g and g are as given before. For vertical acceleration, however, the parameter S_S , T_B , T_C and T_D are fixed to site-independent constants. These are given in Table C.6.

Table C.6: Corner periods for the vertical ERS according to NNTC-Italy (2008)

Subsoil Category	S_S	T_B	T_C	T_D
A, B, C, D & E	1.0	0.05 s	0.15 s	1.0 s

As with **EN 1998**, the **NNTC-Italy (2008)** gives explicit consideration to elastic displacement. The characterisation of spectral displacement is undertaken in the same manner as Eurocode, with displacement over the shorter period range ($0 \leq T (s) \leq 4$) defined by equation 4.1b. For longer period displacement, two parameters T_E and T_F are defined as in Table C.7.

Table C.7: Long-period displacement corner periods from NNTC-Italy (2008)

Subsoil Category	T_E (s)	T_F (s)
A	4.5	10.0
B	5.0	10.0
C, D, E	6.0	10.0

The corresponding displacements over this spectral range are determined from:

$$T_E \leq T \leq T_F : S_{De}(T) = 0.025a_g \cdot S \cdot T_C \cdot T_D \left[F_0 \cdot \eta + \left(\frac{T - T_E}{T_F - T_E} \right) (1 - F_0 \cdot \eta) \right] \quad (C.6a)$$

$$T \geq T_E : S_{De}(T) = d_g \quad (C.6b)$$

Where d_g is the peak horizontal displacement defined by:

$$d_g = 0.025a_g \cdot S \cdot T_C \cdot T_D \quad (C.7)$$

For the ultimate limit state design, the factor η can be replaced with $(1/q)$, where q is the behaviour factor of the structure.

C.4. Japan

The general provisions for three common systems (buildings, highways and railways) are illustrated as follows:

C.4.1. Buildings

The design acceleration spectrum (in $m\ s^{-2}$) for an engineering bedrock site at the life-safety limit state is given by:

$$T(s) \leq 0.16 : S_0(T) = (3.2 + 30T) \quad (C.8a)$$

$$0.16 < T(s) < 0.64 : S_0(T) = 8.0 \quad (C.8b)$$

$$T(s) \geq 0.64 : S_0(T) = \frac{5.12}{T} \quad (C.8c)$$

For the damage-limitation state these values are reduced to one fifth of the values given for the life-safety limit state. The design response spectrum at the ground surface is determined by:

$$S_d(T) = G_s(T) \cdot Z \cdot S_0(T) \quad (C.9)$$

Where Z is the seismic zone factor (a geographically variable factor between 0.7 and 1.0) and $G_s(T)$ the surface soil layer amplification factor, given by:

$$T(s) \leq 0.8T_2 : G_s = G_{s2} \frac{T}{0.8T_2} \quad (C.10a)$$

$$0.8T_2 < T(s) \leq 1.2T_1 : G_s = G_{s2} + \frac{G_{s1} - G_{s2}}{0.8(T_1 - T_2)} (T - 0.8T_2) \quad (C.10b)$$

$$0.8T_1 < T(s) \leq 1.2T_1 : G_s = G_{s1} \quad (C.10c)$$

$$T > 1.2T_1 : G_{s1} + \frac{G_{s1} - 1.0}{\frac{1}{1.2T_1} - 0.1} \left(\frac{1}{T} - \frac{1}{1.2T_1} \right) \quad (C.10d)$$

Where T_1 and T_2 are the predominant periods of surface soils layers for the first and second modes respectively, and G_{s1} and G_{s2} the amplification factors at the periods T_1 and T_2 respectively.

The site amplification factor is determined from geotechnical data at a site, including density (ρ_i), shear-wave velocity (V_{si}) and viscous damping ratio (h_i) of each soil layer i . If H is the total thickness of i soil layers, each with thickness d_i , above the engineering bedrock then:

$$H = \sum d_i \quad (\text{C.11})$$

the equivalent surface wave velocity of surface soil layers (m s^{-1}) is given by:

$$V_{se} = \frac{H}{\sum \left(\frac{d_i}{V_{si}} \right)} \quad (\text{C.12})$$

and the equivalent mass density of the surface soil layer:

$$\rho_e = \frac{\sum \rho_i d_i}{H} \quad (\text{C.13})$$

T_1 and T_2 are determined by $T_1 = \frac{4H}{V_{se}}$ and $T_2 = \frac{T_1}{3}$ respectively. If the wave impedance ratio (α) between the single model surface layer and the bedrock are calculated by:

$$\alpha = \frac{\rho_e V_{se}}{\rho_b V_{sb}} \quad (\text{C.14})$$

Where ρ_b and V_{sb} are the mass density and shear wave velocity of the engineering bedrock respectively. Finally if the elastic potential energy of each soil layer (W_{si}) is known then the viscous damping ratio of the whole soil column is given by:

$$h_{seq} = 0.8 \frac{\sum h_i W_{si}}{\sum W_{si}} \quad (\text{C.15})$$

then the parameters G_{s1} and G_{s2} are calculated from:

$$G_{s1} = \frac{1}{1.57h_{seq} + \alpha} \quad (\text{C.16a})$$

$$G_{s1} = \frac{1}{4.71h_{seq} + \alpha} \quad (\text{C.16b})$$

For the damage limitation state G_s has a minimum value of 1.5 for $T \leq 1.2T_1$ and 1.35 for $T > T_1$. For the life-safety state these values are 1.2 and 1.0 for the same spectral ranges respectively.

C.4.2. Highways

The application of performance-based seismic design to highway bridges in Japan influences the definition of elastic response spectra. The spectra used depend on the method applied for

seismic design and the level of protection required for the structure. For moderate [high probability] ground motions the “Seismic Coefficient Method” is used as the basis for seismic design. For low probability events, however, the ductility design method is applied. This requires that the lateral capacity of a pier (P_a) satisfies the condition: $P_a > k_{he} \cdot W$, where

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu_a - 1}} \text{ and } W = W_u + c_p \cdot W_p. \text{ In these equations } \mu_a \text{ is the allowable displacement}$$

ductility factor of the pier, W_p the weight of a pier, W_u the weight of part of a superstructure supported by the pier and c_p a coefficient depending on the failure mode. k_{hc} is the lateral force coefficient. This is a product of the standard modification coefficient k_{hc0} (given in Table C.8) and a modification coefficient, c_z , which assumes a value of 0.7, 0.85 and 1.0 depending on the seismic zone. The lateral force coefficient assumes the shape of the elastic response spectrum and is given in Tables C.8.

Table C.8: Lateral force coefficient for highways defined by the Japan (2000) code

Type I			
Soil Cond.	Lateral Force coefficient k_{hc0}		
Group I (Stiff)	$k_{hc0}=0.7$ for $T \leq 1.4s$	$k_{hc0} = 0.876T^{2/3}$ for $T > 1.4$	
Group II (Moderate)	$k_{hc0} = 1.51T^{1/3}$ ($k_{hc0} \geq 0.7$) for $T < 0.18$	$k_{hc0} = 0.85$ for $0.18 \leq T \leq 1.6$	$k_{hc0} = 1.16T^{2/3}$ for $T > 1.6$
Group III (Soft)	$k_{hc0} = 1.51T^{1/3}$ ($k_{hc0} \geq 0.7$) for $T < 0.29$	$k_{hc0} = 1.0$ for $0.29 \leq T \leq 2.0$	$k_{hc0} = 1.59T^{2/3}$ for $T > 2.0$

Type II			
Soil Cond.	Lateral Force coefficient k_{hc0}		
Group I (Stiff)	$k_{hc0}=4.46T^{2/3}$ for $T \leq 0.3$	$k_{hc0}=2.00$ for $0.3 \leq T \leq 0.7$	$k_{hc0} = 1.24T^{4/3}$ for $T > 0.7$
Group II (Moderate)	$k_{hc0} = 3.22T^{2/3}$ for $T < 0.4$	$k_{hc0} = 1.75$ for $0.4 \leq T \leq 1.2$	$k_{hc0} = 2.23T^{4/3}$ for $T > 1.2$
Group III (Soft)	$k_{hc0} = 2.38T^{2/3}$ for $T < 0.5$	$k_{hc0} = 1.50$ for $0.5 \leq T \leq 1.5$	$k_{hc0} = 2.57T^{4/3}$ for $T > 1.5$

The highway bridge provisions also indicate requirements for a dynamic response analysis. Here, ground motions are fitted to the following response spectra defined in the Table C.9. The input response spectra (S_{I0} and S_{II0} for Type I and Type II earthquakes respectively) are modified by the modification coefficient for a hazard zone (c_z) and a modification factor for damping ratio (c_D).

C.4.3. Railway Structures

The design requirements for railway structures follow similar guidelines to those for highway bridges. Two levels of design are considered: L1 (high probability motion), and L2 (very low probability motion). Three types of spectra are presented:

Spectrum I – Acceleration corresponding to near-land inter-plate events of magnitude 8.0 at a distance of 30 to 40 km (Type II previously),

Spectrum II - Acceleration spectrum based on static analysis of the earthquake data recorded in past inland earthquakes caused by active faults.

Spectrum III – As **Spectrum II** but based on analysis of active faults, if model of the active fault is available.

Table C.9: Response spectra for dynamic analysis for highway structures from Japan (2000)

Type I Response Spectra			
Soil Cond.	Response Acceleration S_{I0} (cm s^{-2})		
Group I (Stiff)	$S_{I0} = 700$ for $T_i \leq 1.4$		$S_{I0} = 980/T_i$ for $T_i > 1.4$
Group II (Moderate)	$S_{I0} = 1,505T_i^{1/3}$ ($S_{I0} \geq 700$) for $T_i < 0.18$	$S_{I0} = 850$ for $0.18 \leq T_i \leq 1.6$	$S_{I0} = 1,360/T_i$ for $T_i > 1.6$
Group III (Soft)	$S_{I0} = 1,511T_i^{1/3}$ ($S_{I0} \geq 700$) for $T_i < 0.29$	$S_{I0} = 1,000$ for $0.29 \leq T_i \leq 2.0$	$S_{I0} = 2,000/T_i$ for $T_i > 2.0$
Type II Response Spectra			
Soil Cond.	Response Acceleration S_{II0} (cm s^{-2})		
Group I (Stiff)	$S_{II0} = 4,463T_i^{2/3}$ for $T_i \leq 0.3$	$S_{II0} = 2,000$ for $0.3 \leq T_i \leq 0.7$	$S_{II0} = 1,104T_i^{5/3}$ for $T_i > 0.7$
Group II (Moderate)	$S_{II0} = 3,224T_i^{2/3}$ for $T_i \leq 0.4$	$S_{II0} = 1,750$ for $0.4 \leq T_i \leq 1.2$	$S_{II0} = 1,104T_i^{5/3}$ for $T_i > 1.2$
Group III (Soft)	$S_{II0} = 2,381T_i^{2/3}$ for $T_i \leq 0.5$	$S_{II0} = 1,500$ for $0.5 \leq T_i \leq 1.5$	$k_{hc0} = 1.59T_i^{2/3}$ for $T_i > 1.5$

For **Spectrum III** events real or artificial time-histories are used. For **Spectrum II** events real time-histories may be selected that satisfy given conditions for a site. These can be attenuated to compensate for source to site distance. An upper bound spectrum for inland events is defined, which can also be attenuated to compensate for distance. This upper bound spectrum corresponds to the design motion directly above an inland fault:

$$T(s) = 0.1: S_a(T) = 1100 \text{ cm s}^{-2} \quad (\text{C.17a})$$

$$0.2 \leq T(s) \leq 0.7: S_a(T) = 1700 \text{ cm s}^{-2} \quad (\text{C.17b})$$

$$T(s) = 5.0: S_a(T) = 154 \text{ cm s}^{-2} \quad (\text{C.17c})$$

Intermediate values of T are determined by linear interpolation. For inter-plate events the design spectrum is defined by:

$$0.1 \leq T(s) \leq 1.0: S_a(T) = 1100 \text{ cm s}^{-2} \quad (\text{C.18a})$$

$$T(s) = 5.0: S_a(T) = 154 \text{ cm s}^{-2} \quad (\text{C.18b})$$

C.5. Indonesia (2002)

The elastic response spectrum is developed by:

$$T(s) \leq 0.2: S_a(T) = a_0 + T \left(\frac{a_m - a_0}{0.2} \right) \quad (\text{C.19a})$$

$$0.2 < T(s) < T_c: S_a(T) = a_m \quad (\text{C.19b})$$

$$T(s) \geq T_C: S_a(T) = \frac{a_r}{T} \quad (\text{C.19c})$$

Where a_0 is the peak ground acceleration (g) at $T = 0$, T_C the corner period (taking the value of 0.5 s on “hard” sites, 0.6 s on “medium” sites and 1.0 s on “rock” site), $a_m = 2.5 a_g$, and $a_r = a_m T_C$. Whilst values of a_0 are given for the “bedrock” condition, the parameter T_C is not made explicit, although it could be assumed equal to 0.5 s. The values of a_0 for each zone and site condition are given in Table C.10.

Table C.10: Acceleration for each zone according to the Indonesian (2002) code

Zone	a_0 (g)			
	Bedrock	“Hard” Soil	“Medium” Soil	“Soft” Soil
1	0.03	0.04	0.05	0.08
2	0.10	0.12	0.15	0.20
3	0.15	0.18	0.23	0.30
4	0.20	0.24	0.28	0.34
5	0.25	0.28	0.32	0.36
6	0.30	0.33	0.36	0.38

vertical acceleration is calculated from the relation:

$$S_v(T) = \psi \cdot a_0 \cdot I \quad (\text{C.20})$$

Where a_0 is the horizontal acceleration for a given zone and soil type as indicated in Table 4.17, ψ is the vertical to horizontal scaling factor given in Table C.11, and I the importance category of the structure.

Table C.11: Site coefficient according to the Indonesian (2002) code

Zone	ψ
1	0.5
2	0.5
3	0.5
4	0.6
5	0.7
6	0.8

C.6. Pakistan (2007)

The ERS is defined according to parameters fixed for each seismic zone, with five zonation levels considered. For each of the site categories listed in section 3.3.6, the ERS is determined from:

$$T \leq T_0: S_a(T) = C_a + T \left(\frac{2.5C_a}{T_0} \right) \quad (C.21a)$$

$$T_0 \leq T \leq T_S: S_a(T) = 2.5C_a \quad (C.21b)$$

$$T > T_S: S_a(T) = \frac{C_v}{T} \quad (C.21c)$$

Where $T_0 = 0.2 T_S$ and $T_S = C_v / 2.5C_a$, and C_v and C_a are zone- and site-dependent parameters indicated in Table C.12.

Table C.12: Seismic coefficients according to the Pakistan (2007) code.

Seismic Coefficients C_a					
Soil Type	Seismic Zone Factor Z (Zone Name)				
	$Z = 0.075$ (I)	$Z = 0.15$ (2A)	$Z = 0.2$ (2B)	$Z = 0.3$ (3)	$Z = 0.4$ (4)
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	Special Investigation				

Seismic Coefficients C_v					
Soil Type	Seismic Zone Factor Z (Zone Name)				
	$Z = 0.075$ (I)	$Z = 0.15$ (2A)	$Z = 0.2$ (2B)	$Z = 0.3$ (3)	$Z = 0.4$ (4)
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	Special Investigation				

The parameters N_a and N_v are near fault factors defined according to Table C.13. The definition of the faulting categories A, B and C are given in Table C.14.

Table C.13: Near Fault Factors according to the Pakistan (2007) code

N_a			
Seismic Source Type	Closest Distance to Known Source		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

N_v				
Seismic Source Type	Closest Distance to Known Source			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

Table C.14: Fault categorisation according to the Pakistan (2007) code (originally taken from UBC, 1997)

Seismic Source Type	Seismic Source Description	Seismic Source Definition	
		Maximum Moment Magnitude (M_w)	Slip Rate (mm/yr)
A	Faults that are capable of producing large magnitude events that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All other fault types	$M \geq 7.0$	$SR < 5$
		$M < 7.0$	$SR > 2$
		$M \geq 6.5$	$SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$