Design of Buildings and Structures in Low to Moderate Seismicity Regions

This Professional Guide represents CNERC's continued contribution towards the building of a resilient world community. The underlying objective is to maintain living standards and improve safety in the face of natural disasters such as earthquakes. The scope of the undertaking in its entirety is broad, drawing expertise from many allied disciplines. Managing natural disasters entails developing adequate mitigation measures and preparedness levels. This Professional Guide is concerned specifically with mitigating the impact of future disasters by improving the robustness of newly constructed buildings in regions of low to moderate seismicity through a better understanding of hazard assessment and design methodology. The targeted readers are designers of building structures and policy makers in these regions.

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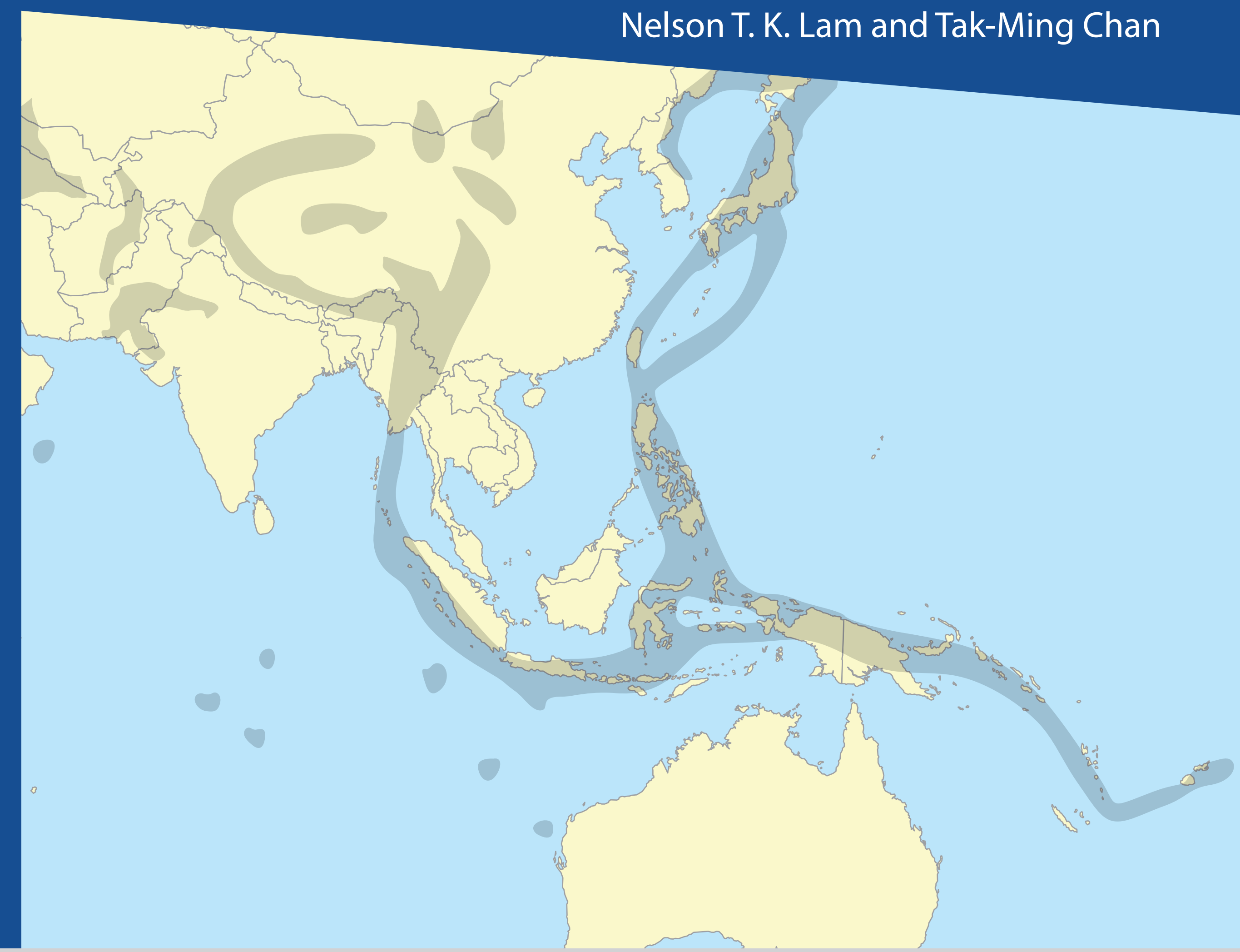
Professional Guide: PG-002

Nelson T. K. Lam and Tak-Ming Chan



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Earthquake Loading Characterisation for Regions of Low to Moderate Seismicity

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The earthquake loading model, the subject matter of this chapter, is to be decided first in the design process. No prior knowledge of earthquake engineering and structural dynamics is assumed of the reader. Basic principles and generic modelling techniques are explained early in the chapter for the benefit of those who are not already familiar with the fundamentals. Later sections are fully devoted to the modelling of earthquake actions on rock and flexible soil sites in regions of low to moderate seismicity. These are important key features which should be of interest to both regulators (code drafters) and designers.

Keywords: seismic design, loading characterization, low to moderate seismicity

1. Introduction

This chapter is primarily concerned with the presentation and interpretation of earthquake loads (seismic actions) and how challenges in the modelling of seismic hazards are tackled in a tectonically stable region where locally recorded data is typically lacking. Engineers who are not already familiar with the topic of seismic design are usually directed to world acclaimed textbooks including that of Chopra (2017) which places emphasis on the analytical aspects of the seismic design of the structure providing an in-depth treatment of structural dynamics phenomena and their mathematical modelling. Another highly recommended text is that of Fardis (2009) which places emphasis on the philosophies and considerations of seismic design and the assessment and retrofitting of RC buildings. Much of the contents in both textbooks concerns the application of a predefined earthquake loading model in analysis but neither explain how the loading model should be determined in the first place.

This chapter includes a range of basic topics including structural idealisation, dynamic equilibrium in natural vibration and base excited conditions and the presentation of their solutions as response spectra in different formats. This is followed by an introduction to probabilistic seismic hazard analysis and the uniform hazard spectrum (Section 2). A rundown of the earthquake loading models stipulated in major building codes of practice is then presented (Section 3). Textbooks by Naeim (2001) and Dowrick (2009) cover these topics. However, no detailed coverage exists for low to moderate seismicity conditions. In this chapter, a section is fully devoted to modelling considerations in regions of low to moderate seismicity (Section 4). Finally, a soil amplification model, recommended as a replacement for current code models for such regions, is introduced Section 5.

2. Basic principles

A thorough understanding of the derivation of a response spectrum model is essential for their correct, and effective, application in structural design practice. This section introduces the modelling of earthquake loads, step-by-step, starting from structural idealisation, followed by dynamic equilibrium analysis, the procedure for constructing a design spectrum, ground motion modelling, probabilistic seismic hazard analysis and the concept of the uniform hazard spectrum.

2.1 Structural idealisation

Analysis of the lateral resisting behaviour of a multi-storey building structure subject to seismic conditions can be simplified using a "stick model" (as depicted in Fig.1). Lumped masses are shown attached to the "stick" at regular intervals up the height of the building to emulate the masses of the building floors. The stick model is a reasonable, and commonly adopted, form of structural idealisation of the building, provided that the floor diaphragms can be assumed perfectly rigid in their own planes so that all masses at the same level in the building are subject to identical motion time-histories during an earthquake. This stick model can also be described as a multi-degree-of-freedom (MDOF) lumped mass system.

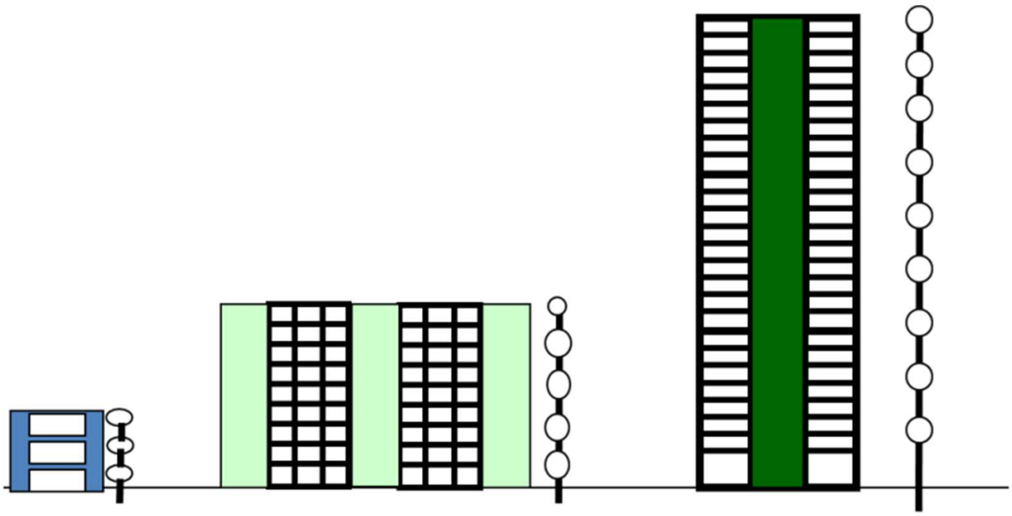


Figure 1. Idealisation of multi-storey buildings into stick models

The dynamic response behaviour of the MDOF lumped mass systems as depicted in Fig.1 can be resolved into a number of vibration modes each of which can be represented by an equivalent SDOF lumped mass system based on principles of modal superposition, forming part of the modal analysis methodology (Figure 2). Detailed descriptions of modal analysis can be found in most textbooks in the field of structural dynamics (e.g. Chopra 2017). The behaviour of a SDOF system in response to externally applied excitations is central to the response analysis of a building structure subject to seismic actions and other forms of dynamic actions.

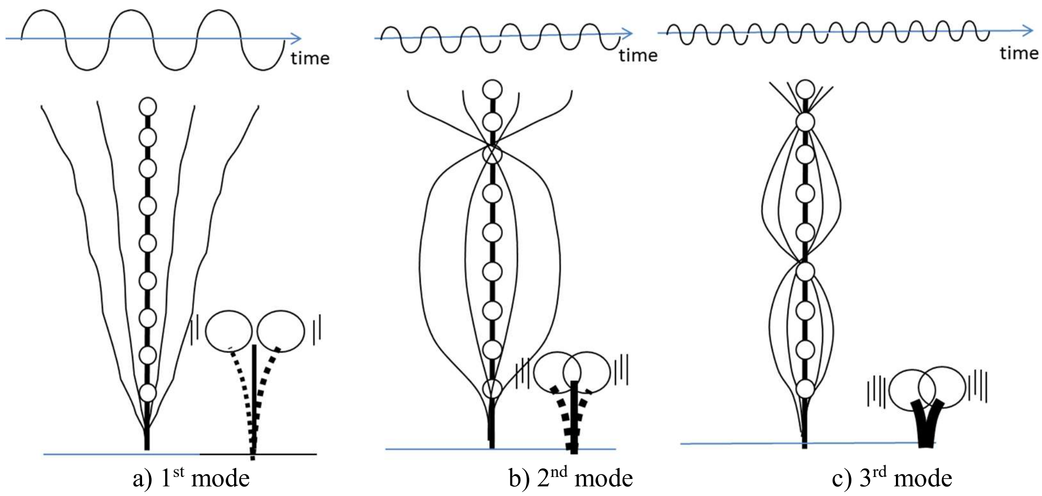


Figure 2. Use of single-degree-of-freedom lumped mass models in a modal analysis

The SDOF lumped mass model is depicted in different forms in the literature to illustrate the same principles. The "table" model of Figure 3(a), which can also be described as an idealized single-storey frame, is used below in this section to illustrate the motion behaviour of the building floor and the associated equilibrium of forces. The single lumped mass stick model of Figure 3(b) is commonly used in illustrations, as is the "trolley model" of Figure 3(c).

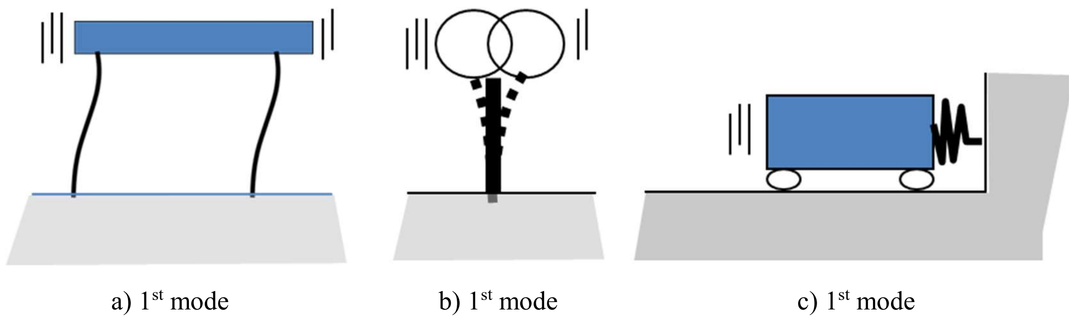


Figure 3. Different forms of single-degree-of-freedom lumped mass models

2.2 Horizontal dynamic equilibrium of forces

The dynamic equilibrium of an idealised single-storey frame induced into vibration by a horizontal ground displacement (Figure 4) is illustrated. For the sake of simplicity, all masses of the building frame are lumped at roof level as illustrated in Figure 4(a). The total (or absolute) displacement $u^t(t)$ of the mass has two components: (1) ground displacement induced by the earthquake $u_g(t)$; and (2) relative displacement between the mass and the ground $u(t)$ (i.e. equal to the deflection of the columns) resulting from the vibration of the structure.

$$u^t(t) = u(t) + u_g(t) \tag{1}$$

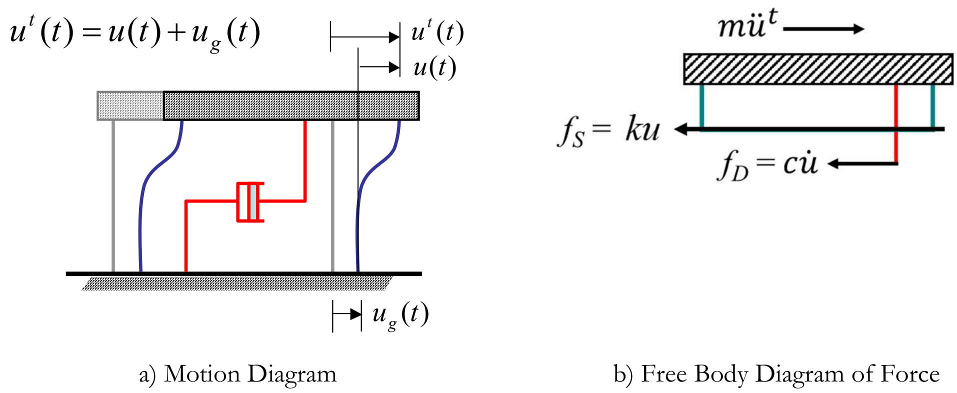


Figure 4. Dynamic behaviour of a single-degree-of-freedom lumped mass model

Consider the free body diagram of the "table top" of the frame (the lumped mass m) as shown in Figure 4(b). The internal force f_S and damping force f_D are in equilibrium with the inertial force. By Newton's Second Law of Motion, the following equation represents the equilibrium of forces at any snapshot of time.

$$-ku - c\dot{u} = m\ddot{u}^t \tag{2}$$

Rearranging Eq. (2), in combination with Eq. (1), leads to Eq. (3) which is the governing equation of dynamic equilibrium of the SDOF system depicted in Figure 4.

$$m\left(\ddot{u} + \ddot{u}_g\right) + c\dot{u} + ku = 0 \quad \text{or} \quad m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \tag{3}$$

The idealised structure can be subject to an externally applied dynamic (e.g. wind) force *p(t)* as shown by the diagram on the left of Figure 5, to applied base excitations (e.g. earthquake) as shown by the diagram on the right of the same figure. Comparing the two diagrams in the same figure reveals their equivalence. It is shown that an earthquake by itself does not impose any externally applied pressure (forces) on the structure, unlike a windstorm. The building is simply made to move (or displace) in an earthquake. Inertial forces are then resulted from accelerations that are associated with the ground motion.

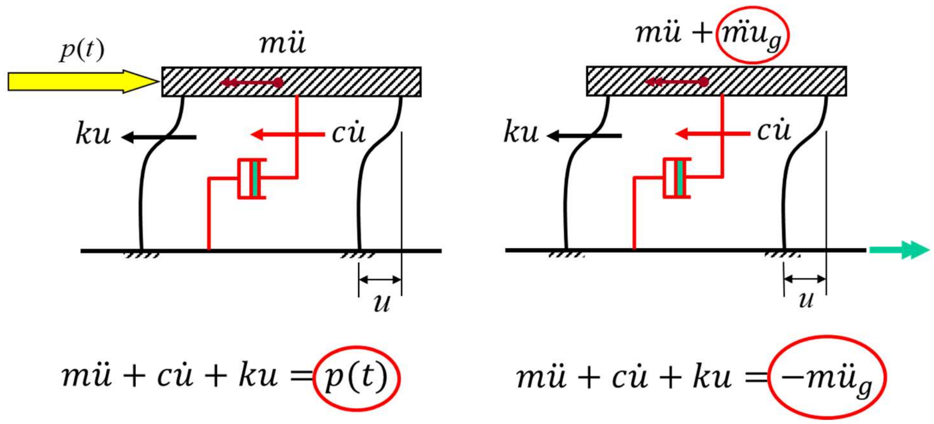


Figure 5. An analogy between external loads and earthquake-induced loads.

Dividing Eq. (3) by mass *m* gives Eq. (4).

$$\ddot{u} + 2\zeta\omega_n\dot{u} + \omega_n^{\ 2}u = -\ddot{u}_g \quad \text{where} \quad \omega_n = \sqrt{\frac{k}{m}} \quad \text{and} \quad \zeta = \frac{c}{2\sqrt{km}} \tag{4}$$

For a given ground acceleration time-history $\ddot{u}_g(t)$, the displacement time-history $u(t)$ of the responding structure depends only on the two parameters, the natural frequency ω_n, or natural period of vibration $T(=2\pi/\omega_n)$ of the system and its damping ratio ζ. Once the displacement time-history of the floor is known, its acceleration time-history response can be found by calculus. An example of the acceleration response of a SDOF system, defined by $T = 0.5$s and $\zeta = 0.05$, when subject to an idealised ground acceleration pulse is shown in Figure 6. Any two systems having the same values of T and ζ would respond in exactly the same way (i.e. the same $u(t)$) even though one system may be more massive than (or stiffer than) the other system.

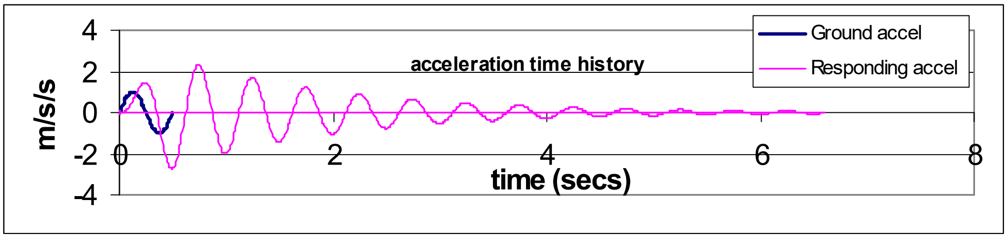


Figure 6. Example SDOF system (T = 0.5s, ζ = 0.05) responding to a ground pulse

Ground motions generated by an earthquake are non-stationary and random in nature, and so irregular that a closed-form analytical solution to the equation of motion is not feasible. Numerical methods, therefore, are used to model response behaviour. Once the deflection time-history has been evaluated using dynamic analysis, internal forces can be found by a quasi-static analysis of the structure at any snapshot of time.

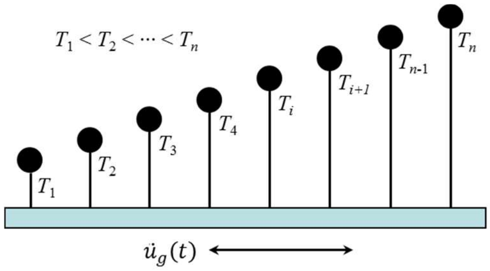
2.3 Response Spectral Acceleration (RSA) and Response Spectrum (RS)

In the design process, structural engineers are concerned mainly with peak values of the structural response, expressed in terms of displacements, forces and moments. The concept of an earthquake response spectrum (RS) serves this purpose well. The RS is a convenient means of presenting the estimated peak values for the whole range of linear elastic SDOF systems in response to earthquake ground motion, as shown in Figure 7(a). RS is a plot of the maximum absolute response values, expressed as a function of the system's natural vibration period T, (or a related parameter such as angular frequency ω_n, or cyclic frequency f_n). Each plot is based on a particular damping ratio ζ. Multiple plots can be shown in the same diagram to represent behaviour associated with the range of values of ζ typically encountered in real conditions, as shown in Figure 7(b).

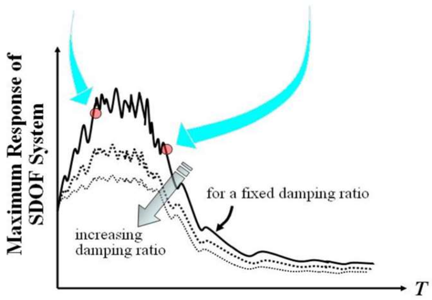
In structural design, the quantity of interest is the maximum (absolute) acceleration response of the lumped mass, $max|\ddot{u}^t(t)|$, to the earthquake load. This quantity is commonly referred to as the response spectral acceleration (*RSA*). A series of acceleration RS (for different structure damping ratios) is presented in Figure 8, based on the base excitations defined by the well known ground motion accelerogram recorded at El Centro in southern California in 1940.

2.4 Idealised Elastic Response Spectrum (ES)

Idealised elastic response spectrum, which can be abbreviated as "Elastic Spectrum" (ES), allows the analysis and design of a structure, as prescribed in a typical code of practice for seismic design of building structures. An ES should ideally be representative of earthquake ground motions that are likely to occur at the site of interest. The direct use of an RS calculated from actual accelerograms recorded from earthquakes is, however, inconvenient for such a purpose, because the spectral shape is highly irregular, featuring sharp fluctuations in spectral values over small changes in the structural period T. Thus, for practical design purposes, the ES should consist of a set of smooth curves and/or straight lines containing one curve for each level of damping. The idealisation of the response spectrum is therefore essential for practical application.



(a) Family of SDOF Damped Oscillators



(b) Response Spectrum (RS)

Figure 7. Concept of Response Spectrum (RS)

The mean RS obtained from an ensemble of earthquake ground motions is much smoother than the RS of an individual accelerogram, as shown by the solid line example in the schematic diagram of Figure 8. The mean RS is accordingly idealised as a "plateau" at low natural periods and as a "hyperbolic curve" at higher periods. At the zero natural period (representing an infinitely rigid structure), the acceleration of the structure, in theory, is identical to that of the ground. The peak acceleration at the centre of mass of the structure is accordingly equal to the peak ground acceleration (*PGA*) in which case the spectral ratio equals unity. As the natural period of the structure (*T*) increases slightly, the spectral value increases rapidly until the "plateau" is reached. The ratio of the spectral value at the plateau to the PGA value is typically of the order of 2.5. In practice, a conservative approach is recommended and the "plateau" is extended to the natural period zero axis. This is because of the unpredictable nature of the response spectrum at very low periods, coupled with the uncertainties associated with estimating building natural periods.

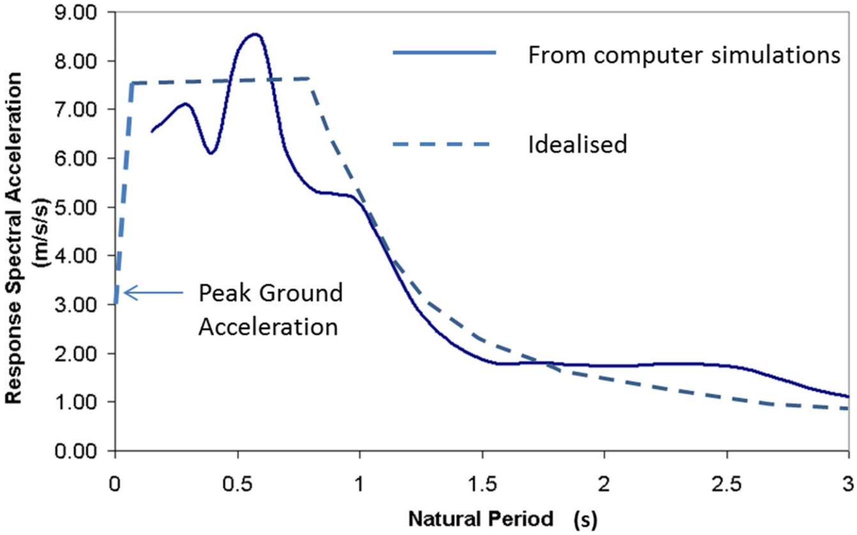


Figure 8. Example response spectrum of accelerogram recorded at 1940 El Centro, southern California, United States

Deriving an ES can be more complicated for sites subject to a variety of seismic sources, e.g. low-period (high-frequency) motions from nearby small earthquakes and high-period (low-frequency) motions from distant moderate to large earthquakes (Tsang 2015). An alternative way of determining an ES is to compute a uniform hazard spectrum (UHS), whereby each spectral ordinate is based on a probabilistic seismic hazard assessment (PSHA) procedure which takes account of contributions from all potential earthquake sources. Such a UHS has a uniform (or constant) probability of exceedance at all values of T. A UHS is effectively the ES enabling the production of structural designs all with the same probability of failure irrespective of the assumed value of T for the structure.

Using the method described above, parameterisation is required, if a consistent ES format is to be applied to a country or a region. In the following sections, parameterisation schemes incorporated in various major codes of practice around the globe are introduced.

From static analysis, RSA is related to peak base shear values $(V_b)_{max}$, i.e. the equivalent static force F_{max}, by:

$$(V_b)_{\max} = F_{\max} = mRSA = w\frac{RSA}{g} \tag{5}$$

where w is the seismic weight of the structure and g is the gravitational acceleration (9.81 m/s^2).

When written in this form, RSA/g may be interpreted as the base shear coefficient or lateral force coefficient (in units of g). The RSA (or RSA / g) parameter is used in building codes as the coefficient by which the structural weight is multiplied to obtain the design base shear force. A RS is therefore very useful in design because the manner in which structures of different natural periods of vibration will respond to a specific earthquake ground motion model can be found readily.

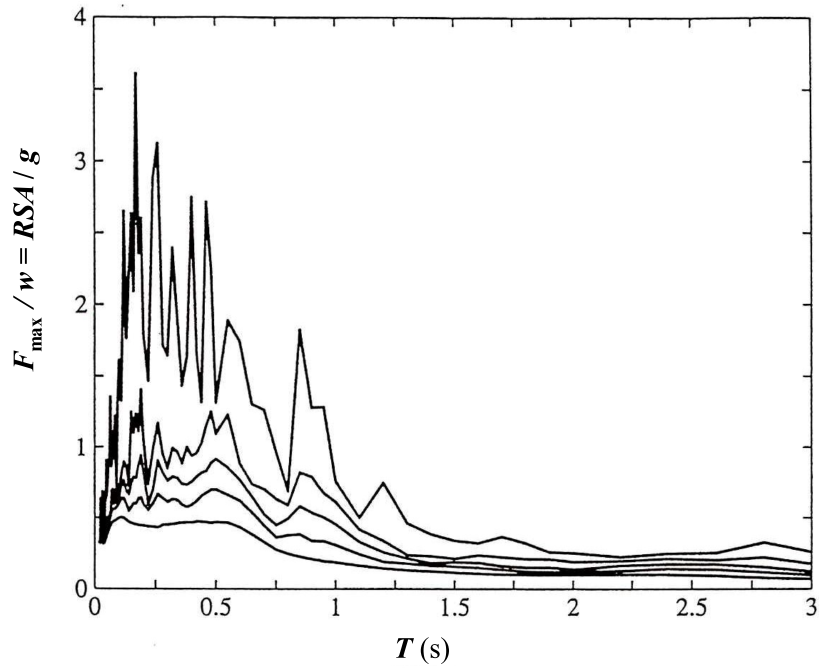


Figure 9. Normalised acceleration, or base shear coefficient, response spectrum for El Centro ground motion; ζ = 0, 2, 5, 10, and 20%. (reproduced from Chopra 2017)

2.5 Peak Ground Acceleration (PGA)

The ES specifies the design ground motions or equivalent static loadings for structural analysis and design. Thus, the seismic hazard level of the site should be reflected in the spectral parameters characterising the ground motion. In an acceleration RS, either the peak ground motion measurements (PGA or PGV) or spectral acceleration values at certain natural period(s) of vibration, *RSA(T)*, can be used as scaling parameter(s). The literal meaning of the term peak ground acceleration (PGA) refers to the highest (or lowest) point on one of the peaks (or troughs) in a recorded ground acceleration time-history, which can be ultra sensitive to high frequency signals (noise) in the record even though such noise only has a very minor influence on structural response behaviour. The implication is that the *PGA* value can be lowered considerably by artificially removing all the (tall and sharp) "spikes" from the record, knowing there can be little effect on the potential response behaviour of the structure. It is widely recognised that the maximum response spectral value (RSA_{max}), the highest point on the acceleration response spectrum, is much better correlated with the potential earthquake hazard than the *PGA* value. The term "effective PGA" (EPGA) which is defined herein as RSA_{max} divided by a dynamic amplification factor (typically 2.5 or 2.75) has been used by codes of practice to characterise the intensity of ground shaking. Similar terminology such as hazard factor (*Z*), or acceleration coefficient (a_g), which are similar in meaning to *EPGA*, have also been used to characterise city seismic hazard levels.

Each region, typically the size of a city, is assigned a set of parameters for ES construction purposes, stipulated in the relevant codes of practice. Each of the key parameters, e.g. PGA, is presented as a zonation map, or a table in which the parameter values are listed for the individual towns/cities concerned.

PGA has been used as the scaling parameter for constructing ES in the following major codes of practice:

- Chinese Code for Seismic Design of Buildings, GB 50011 (2010)
- Eurocode 8, EN 1998-1(2004) or CEN (2004),
- Australian Standard, AS 1170.4 (2007) or SA (2007) and
- New Zealand Standard, NZS 1170.5 (2004) or SNZ (2004).

2.6 Ground Motion Models (GMMs)

A RS based on averaging RSA accelerogram values can be used to represent the potential hazard of a particular earthquake scenario, defined in terms of a magnitude-distance (*M*-*R*) combination along with the site class. For example, a RS can represent the effects of ground shaking by a magnitude 6 earthquake at a distance of 20 km on stiff soil sites (i.e. M6 at R = 20 km for stiff soil sites). The RS changes with the magnitude and/or distance. Mathematical expressions providing predictions of the RS as a function of *M*, *R* and site class (and faulting mechanism with certain GMMs) are called *Ground Motion Prediction Equations* (GMPEs). To develop a GMM, or GMPE, statistical analysis of the RS values based on an ensemble of earthquake ground motions are commonly used to develop the earthquake loading model. Earthquake records are then sorted in accordance with the design parameters: earthquake magnitude, site-source distance, faulting mechanism and site soil conditions. This approach to the development of a ground motion model is only feasible for heavily instrumented, tectonically active, regions where strong motion records of shallow earthquakes are in abundance. Thus, conventional GMMs have been developed mainly from strong motion accelerogram data collected in tectonically active regions in western North America. For a reference site class, an ES can be based on a range of possible earthquake scenarios (i.e. possible *M*-*R* combinations) related to the various identified fault sources surrounding the city. A reference site class is the most common class of site existing in a city, enabling the maximum number of common site condition accelerograms to be included in the database for statistical analyses. Empirical GMPE models developed in western North America typically relate to "stiff soil sites" as most data collected applies to such site conditions (Boore et al. 2014; Abrahamson et al. 2014; Campbell and Bozorgnia 2014; Idriss 2014; Chiou and Youngs 2014). The most reliable model predictions relate to this reference site class.

In other regions, such as tectonically stable regions of low to moderate seismicity, the process of developing a GMM is hampered by the lack of local earthquake records. An alternative approach, that of stochastic simulations of the seismological model, which includes the generation of artificial earthquake accelerograms has become the commonly-adopted approach in stable continental regions such as Central and Eastern North America. The seismological model itself is partly based on theoretical principles and partly on the analysis of seismogram data recorded during small magnitude earthquakes and tremors originating at long distances. (see literature review of articles on stochastic simulations of the seismological model (e.g. Lam et al. 2000)). Rock site seismological models, for areas of low to moderate seismicity, are more reliable than soil site models. The program GENQKE implements rock site stochastic simulations (Lam 2002). More detailed descriptions of ground motion modelling in tectonically stable regions of low-to-moderate seismicity are given in Section 4, below.

2.7 Probabilistic Seismic Hazard Analysis (PSHA)

To develop an ES, it is necessary to integrate an RS associated with a range of earthquake scenarios, for the reference site class concerned, as predicted by a GMPE. The integration procedure involves numerical simulations based on the established principles of conditional probability and is known as probabilistic seismic hazard analysis (PSHA). Most PGA or hazard factor values stipulated as standards for seismic actions are based on PSHA results. Thus, an ES cannot be associated with a particular earthquake scenario nor with any particular accelerogram. Consequently, it is inappropriate to simulate earthquake ground motions based on an ES. The PSHA procedure which is typically undertaken by seismologists and risks analysts, using specialist software, comprises the following calculation steps in deriving the Design PGA and ES for a city:

1) Identifying potential fault sources around the city
2) Modelling the recurrence behaviour of earthquakes relating to each identified fault source
3) Modelling the GMPE for different ranges of M-R combinations
4) Integrating contributions from one or more sources having influence on the hazard of the city

PSHA results based on the use of GMMs derived for tectonically active regions cannot automatically be taken as representative of places in tectonically stable regions such as Central North America, Australia, and Southern India. Thus, alternative GMMs developed for intra-plate seismo-tectonic environment need to be sought for realistic hazard predictions in areas of low to moderate seismicity.

2.8 Uniform Hazard Spectrum (UHS)

ES is normally constructed based on one to two spectral values (RSA(T) or PGA) according to the standard spectral shape as specified in codes of practice. The single-parameter or dual-parameter scaling approach has been adopted because PSHA was typically conducted for PGA (and sometimes PGV) only, partly because GMPEs were only available for PGA (and PGV) in the 1960's and 1970's. The resulting ES, adopted in many current codes of practice, does not have a uniform probability of exceedance (PE) at all natural periods of vibration.

Importantly, UHS takes account of seismic hazards from all potential seismic sources surrounding the site. Normally, low-period spectral values of the UHS are attributable to near-source (moderate) earthquakes, whereas the high-period end reflects the potential hazard from more distant (larger) earthquakes. The spectral shape of the UHS is therefore usually not consistent with that of an RS from any recorded accelerogram. Since the 1990's, the concept of the uniform hazard spectrum (UHS) has become commonly accepted, for the construction of site-specific ES when GMPEs are normally used for predicting RSA values across the entire range of natural periods of the building. The response spectral ordinates for a range of periods of oscillation are computed using the same PSHA procedure. The advantage over the scaled-spectrum approach is the uniform (and constant) PE at all structural periods, hence representing region-specific, and site-specific, frequency content more accurately.

The International Building Code (2015) (B 2015) adopted in the United States (abbreviated herein as IBC), gives RSA values at natural periods of 0.2 s and 1.0 s as scaling parameters for constructing the ES. Such a dual-parameter approach represents an attempt to mimic the shape of the UHS. In the 2015 version of the National Building Code of Canada (NRC 2015) abbreviated herein as NBCC, PGA and RSA values, at natural periods of 0.2, 0.5, 1.0 and 2.0 s which make up a total of five parameters for constructing an ES closely matching the shape of the UHS. Contour maps for Canada, for each of the five parameters, have been published.

3. Elastic response spectrum (ES) models in codes of practice

3.1 International building codes of the united states (IBC 2015)

In IBC (2015), the ES is specified as in Figure 10. Apart from the RSA values at natural periods 0.2 s (S_s) and 1.0 s (S_1), the site coefficients F_a and F_v, are two other scaling parameters which take site effects into account for the low period and high period ranges respectively. Further information on site coefficients is provided in the next section. The 'corner' period T_1 is determined on the basis of the four scaling parameters.

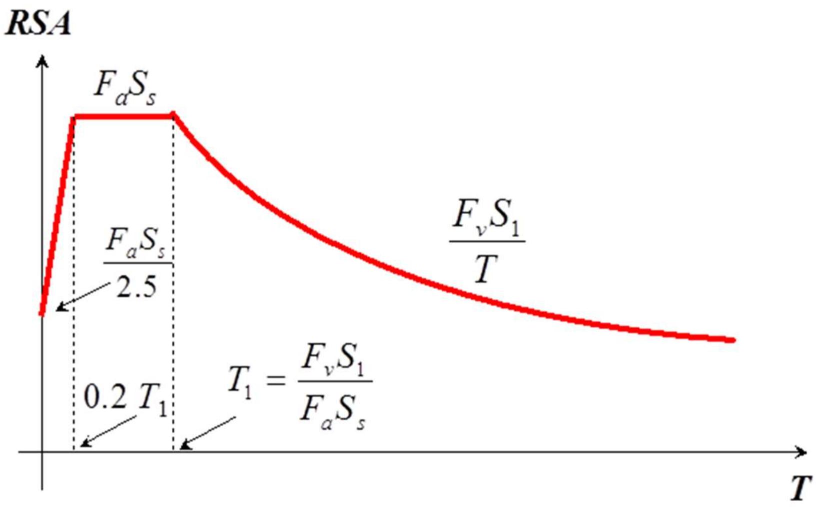


Figure 10. The format of ES adopted in IBC

The ES in the IBC comprises three parts which closely follow the spectral shape of the idealised RS as shown in Figure 8:

1) A linearly increasing line from PGA at T = 0 to RSA_{max} at $0.2T_1$.
2) A flat part at RSA_{max} from T = 0.2 T_1 to $T = T_1$.
3) A decreasing curve, as a function of $1/T$.

As stated in the ASCE/SEI Standard 7-16 Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 2016), another corner period T_2 ($\geq$ 4.0 s) is specified in the ES, beyond which, the values of RSA follow a curve which features the rate of decrease of $1/T^2$ in response spectral values.

3.2 The Chinese code (GB 50011-2010)

In Chapter 5: Earthquake Action and Seismic Checking for Structures – Section 5.1 of the Chinese Code (which is abbreviated herein as "GB"), the ES is specified with a detailed description and the required equations. Unless otherwise specified, the structural damping ratio is set at 5% of critical damping.

The ES in GB comprises four parts as follows:

1) A linearly increasing line from PGA at T = 0 to RSA_{max} at 0.1 s.
2) A flat part at RSA_{max} from T = 0.1s to $T = T_1$. The characteristic period T_1 is discussed below.
3) A decreasing curve as a function of $1/T^{0.9}$ is adopted between $T = T_1$ and $T = 5\ T_1$.
4) A decreasing linear function is adopted between $T = 5\ T_1$ and T = 6.0 s.

The spectral shape of the ES in GB is broadly consistent with the idealised ES shown in Figure 8. There are two major differences:

a) the hyperbolic curve of decay between $T_1 < T < 5\ T_1$ has a smaller exponent (=0.9) than the unity (=1.0) of the idealised ES;
b) for $T > 5\ T_1$, the linear rate of decrease of spectral values is more gradual than the rate of decay defined by $1/T^{0.9}$.

Both modifications result in higher spectral values in the long-period range, mainly taking account of the more conservative estimates of design loadings, as well as the larger uncertainties relating to ground motion characteristics at high natural periods of vibration exceeding 1.0 s.

The characteristic (corner) period T_1 is another scaling parameter used in constructing the ES based on GB. The ES in GB is regionally dependent as well as site class dependent. The characteristic (corner) period T_1 is primarily based on the design earthquake group (reference site class II). Based on this classification scheme, each grid is specified by the value of T_1 shown on the zonation map.

3.3 Other major codes of practice

In the Australian Standard (AS1170.4 2007) and in Eurocode (EN1998-1 2004), abbreviated herein as "AS" and "EC" respectively, the spectral shape of the ES follows the ASCE/SEI Standard exactly. The second corner period T_2 is set at 1.5 s and 2.0 s (Type 1) respectively.

In the New Zealand Standard (NZS1170.5 2004), the ES hyperbolic curve of decay is divided into three parts. The function of $1/T^{0.75}$ up to 1.5s is followed by a function of $1/T$ up to 3.0s and a function of $1/T^2$ beyond 3.0s. That is, the second corner period T_2 is set at 3s.

In NBCC (2015), the exact UHS spectral values have been adopted in constructing the reference site ES (firm soil) condition, with all spectral ordinates having a uniform (or constant) probability of exceedance. As the spectral ordinates are determined directly for each geographical location, the spectral shape is controlled by the seismicity pattern at the specific location. There is no spectral shape consistent across the whole of Canada.

4. Concepts of particular relevance to low to moderate seismicity regions

4.1 Classical response spectrum model

The classical response spectrum model developed originally by Newmark and Hall (1982) was intended to be generic in nature but most codes of practice have not adopted the classical model in which the ES is divided into the three zones of acceleration, velocity and displacement control (Figure 11a). Displacement controlled behaviour is typically not featured in code models. But in regions of low to moderate seismicity, characterised by moderate intensity ground shaking, it is particularly important to feature displacement controlled behaviour in the ES. This is because of the limited displacement of the ground that is generated by small and medium-sized earthquakes. The drift demand of a high period structure can be predicted directly by reference to the peak response spectral displacement (RSD_{max}) and the step of estimating seismic forces as is done conventionally can be bypassed. By similar reasoning, the drift demand of a low period structure can be predicted directly by reference to the peak response spectral acceleration (RSD_{max}) and peak response spectral velocity (RSD_{max}) (Figure 11b).

The physical meaning of the classical model can be easily understood if the ES is presented in multiple formats namely the acceleration, velocity and displacement format. Standards and Codes of practice for structural design typically present an ES in the acceleration format only in which the RSA value (variable on the vertical axis) is presented as a function of the natural period (T) of the structural system (variable on the horizontal axis). In a similar manner, the velocity format has the response spectral velocity (RSV) presented as a function of T where RSV is indicative of the maximum relative velocity developed in the system during the course of ground shaking. Similarly, the displacement ES format features the use of the response spectral displacement (RSD), presented as a function of T. Eqs. (6a) – (6c), defining the relationships between RSA, RSV and RSD, can be approximated by applying the basic principles of mechanics.

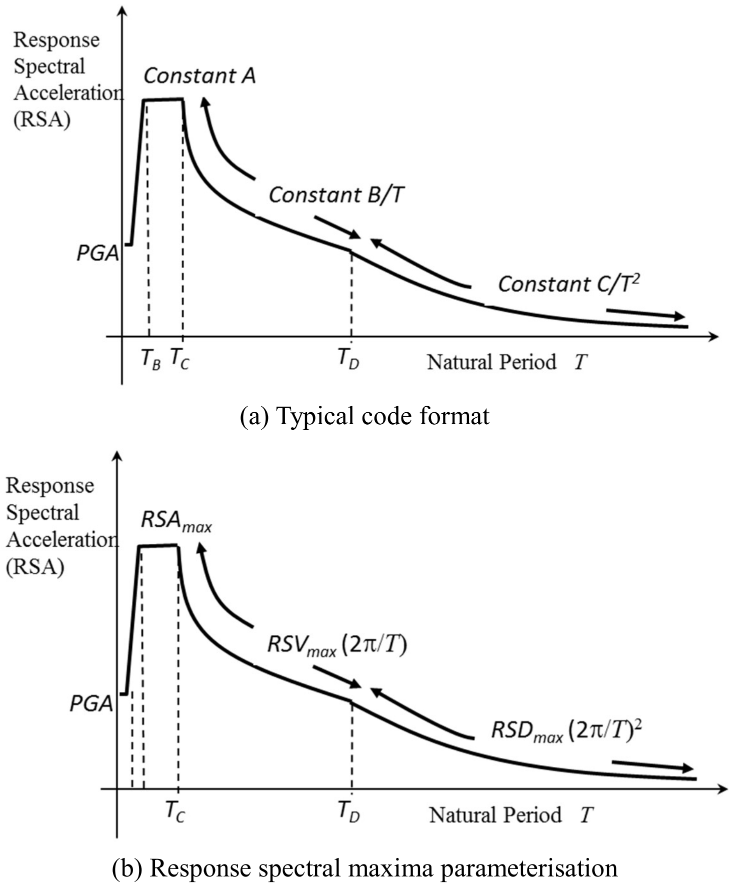


Figure 11. The three zones of an elastic response spectrum (ES)

$$RSV = RSA \times \frac{T}{2\pi} \tag{6a}$$

$$RSD = RSV \times \frac{T}{2\pi} \tag{6b}$$

$$RSD = RSA \times \left(\frac{T}{2\pi}\right)^2 \tag{6c}$$

Eqs. (6a) & (6b) can be derived by equating kinetic energy with the energy of absorption of a linear elastic system (Eq. 7a & 7b). The derivation of these equations makes use of the well-known relationship, Eq. 7c, from which the natural period of vibration (*T*) can be found when the mass (*m*) and stiffness (*k*) of the system are known.

$$\frac{1}{2} mRSV^2 = \frac{1}{2}\frac{F^2}{k} \tag{7a}$$

$$\frac{1}{2} mRSV^2 = \frac{1}{2} kRSD^2 \tag{7b}$$

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{7c}$$

where the term on the RHS of Eqs. (7a) and (7b) represents the area enclosed by the triangular force-displacement diagram.

It is shown in Figure 12(a) that the hyperbola in the acceleration (RSA) format is a flat line in the velocity (RSV) format of Figure 12(b). The constant velocity demand shown in the figure correlates directly with the peak ground velocity (PGV). Similarly, the hyperbola in the RSV format is a flat line in the displacement (RSD) format of Figure 12(c). The constant displacement demand shown in the figure relates directly to the peak ground displacement (PGD). Thus, any part of the ES has the response spectral behaviour characterised by some form of period insensitivity, which is a very useful observation from the design practice perspective, given the difficulties in predicting the natural period of vibration of real buildings. In other words, the constant (and maximum) values of RSA, RSV and RSD are functions of the PGA, PGV, and PGD of the ground shaking as shown by Eqs. 8(a) – 8(c).

$$RSA_{\max} = \alpha_A \cdot PGA \tag{8a}$$

$$RSV_{\max} = \alpha_V \cdot PGV \tag{8b}$$

$$RSD_{\max} = \alpha_D \cdot PGD \tag{8c}$$

where the value of α_A and α_V is typically in the range: 2.5 – 3.0 and 1.8 – 2.0 respectively; and α_D has been recommended a value of 1.4 by Newmark & Hall (1982).

The fourth format correlating *RSD* (in the horizontal axis) with the RSA (in the vertical axis) is called the acceleration-displacement response spectrum (ADRS) diagram. The construction of this diagram is illustrated in Figure 13.

Ideally, the three ground motion parameters: PGA, PGV and PGD would be better obtained independently in order to provide an accurate estimate of the ES, which is characterised by values of RSA_{max}, RSV_{max} and RSD_{max}. When sufficient information is not available the following rules of the thumb (Eqs. 9(a) – 9(c)) may be employed to help construct the ES on rock sites, say, to model the effects of a local earthquake not more than 50 km from the epicentre.

$$RSA_{\max} = RSV_{\max} \times \frac{2\pi}{T_1} \quad \text{where} \quad T_1 = 0.3s \tag{9a}$$

$$RSD_{\max} = RSV_{\max} \times \frac{T_2}{2\pi} \quad \text{where} \quad T_2 = 0.5 + \frac{M-5}{2} \tag{9b}$$

M is the moment magnitude of the earthquake

A T_2 value of 2.0 s is a reasonable, and conservative, assumption to make for local earthquakes occurring in intra-plate regions of low to moderate seismicity. The listed rules-of-thumb infer RSA_{max} (g's) as 4 times PGV (mm/s) divided by 1000; RSV_{max} (mm/s) as 1.8 times PGV (mm/s); and RSD_{max} (mm) as 0.6 times PGV (mm/s). Thus, the RS can be constructed in the different formats described above once the value of PGV is known. If only the PGA value is known, the rule-of-thumb conversion of PGV (mm/s) = 750 times PGA (g's) may be applied.

In the following example, a classical ES model is to be constructed for a city with a PGA value of 0.08g, say. The following inferences can be drawn: PGV = 60 mm/s, RSA_{max} = 0.24g, RSV_{max} = 108 mm/s and RSD_{max} = 36 mm based on T_2 = 2s (only 25 mm approximately in the case of Australia as T_2 = 1.5s is assumed).

Intuitively, the *RSA* format is convenient to use when designing low rise (low period) buildings whereas the RSV, and RSD formats, are convenient when designing medium rise, and high rise, buildings respectively, Period insensitivity in the ES is a useful phenomenon that can be utilised for circumventing challenges associated with uncertainties over the predicted natural period of vibration of a building structure.

Although structural engineers would typically ignore contributions by non-structural components (NSC) such as facades and partitions to the potential response behaviour of the building, there are concerns that NSC can result in a lowering of the natural period of the building thereby increasing the seismic forces applied. But to include the effects of NSC in the analysis can be time-consuming and would also introduce many uncertainties in the analysis of building behaviour. If the building is acceleration (force) controlled, the influence of the NSC needs not be considered when estimating the seismic forces given the insensitivity of seismic force applied to changes in the value of *T*.

By the same argument, if the building is sufficiently slender and tall (in order that its behaviour in seismic conditions is displacement controlled) the influence of the NSC need not be considered when modelling the building drift demand. If the building is velocity (energy) controlled the influence of the NSC on the kinetic energy demand also need not be considered, where estimates of energy demand can be made using the left-hand side of Eqs. (7a) and (7b) along with a constant *RSV* value. Given that the kinetic energy is known, the seismic force demand, and drift demand, of the building can be estimated accordingly, for the given mass and stiffness properties of the building in the structural model when contributions by NSC are ignored. In conclusion, the notion that the NSC increases the force, and drift, demand of the building in seismic conditions is a myth. Engineers with the skills to employ displacement, or energy, principles in the seismic analysis of a building should be able to circumvent this issue. However, the influence of NSC on the potential failure mechanism of the adjoining structural element can be real.

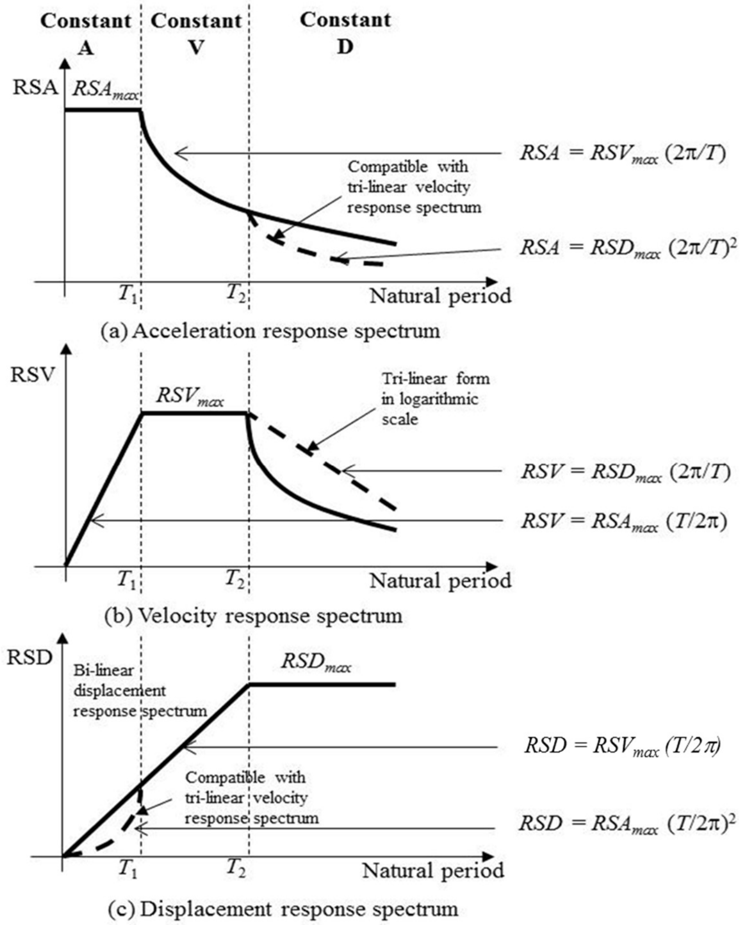


Figure 12. Elastic response spectrum (ES) model in acceleration, velocity and displacement formats

4.2 Minimum design seismic hazard

In regions of low to moderate seismicity, the number of historical earthquake events with a magnitude, M > 4, exceeding the structural damage threshold, are few in number. The main concern with probabilistic seismic hazard modelling analysis is an underestimation of seismic hazard in areas where no local seismic activity appears in the historical record and no fault structure has been found. Consequently, such areas can be assigned a nominal level of seismic hazard. Examples of such areas can be found in many parts of Australia, Peninsular Malaysia and the island of Sri Lanka. Intra-plate seismicity, by definition, exists in all areas distant from tectonic plate margins. Thus, earthquake events are assumed to be possible at virtually any place on earth (Bird 2010; Okal & Sweet 2007). Some of these areas may show little sign of activity because the period of observation is not sufficiently long or because the catchment area is too small. To define areas in an intra-plate region where earthquake tremors have never been recorded as "earthquake free", significantly discounts the actual underlying hazard, and risk. For example, for Peninsular Malaysia, where no $M > 5$ event has been recorded in the past 50 years the level of seismicity cannot be discounted to zero. The same argument applies to the island states of Singapore and Sri Lanka. Unfortunately, there is no general consensus over the minimum threshold (baseline) hazard to adopt for structural design purposes in these areas, to allow for intra-plate seismicity.

A new form of investigation involving a global survey of the frequency of local earthquakes in intra-plate regions found that 5 - 10 earthquakes exceeding magnitude 5 occur in a normalised land area of one million square kilometres in a 50 year period. In view of the uncertainties it is considered prudent to use the larger count of 10 for calculation of background seismic hazard. The normalised count of events exceeding M5.5 was accordingly taken as 3-4. Going by this trend, only one event exceeding M6 is predicted. Clearly, the background hazard is mostly contributed to by small events of up to M5.5. Earthquakes of this order of magnitude can be generated by the rupturing of faults over a length not exceeding 5km. Thus, a geological survey of known faults, as a means of modelling background seismic hazard is not viable, given the difficulties of detecting potential fault sources which are small. The relevant period for this counting scheme could not have been longer than 50 years because of the paucity of historical data in regions which are not active. Thus, a large normalised area (of 1 million square kilometer) was used to compensate for the short time span of the survey (Lam 2016).

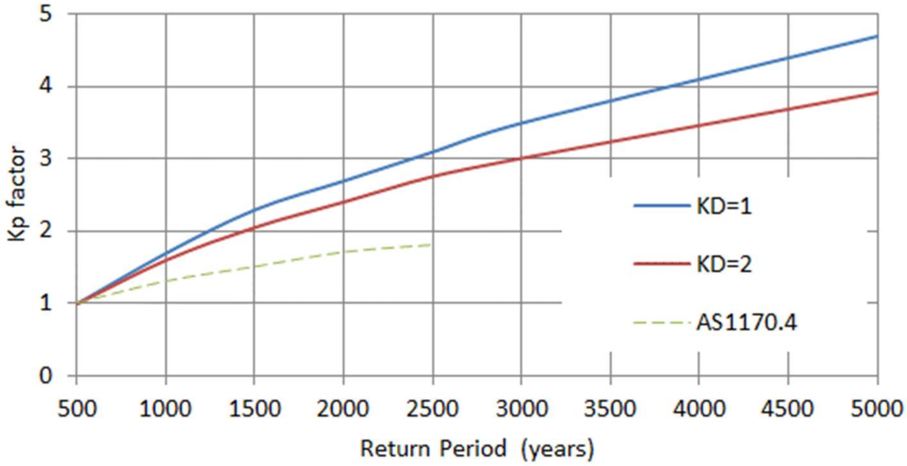
The rate of occurrence has been found to translate to an *EPGA* value of 0.1g for a RP of 2500 years, or 0.07g for a notional RP of 500 years which is taken typically as the hazard factor in earthquake loading standards. The design ground motion intensity of ordinary buildings is based on this hazard factor. The associated ES is then modified in accordance with the site classification to take account of soil amplification effects. Recommendations from the literature around the globe supporting this level of minimum hazard are summarised in Table 1.

4.3 Behaviour of ground motion intensity with increasing return period

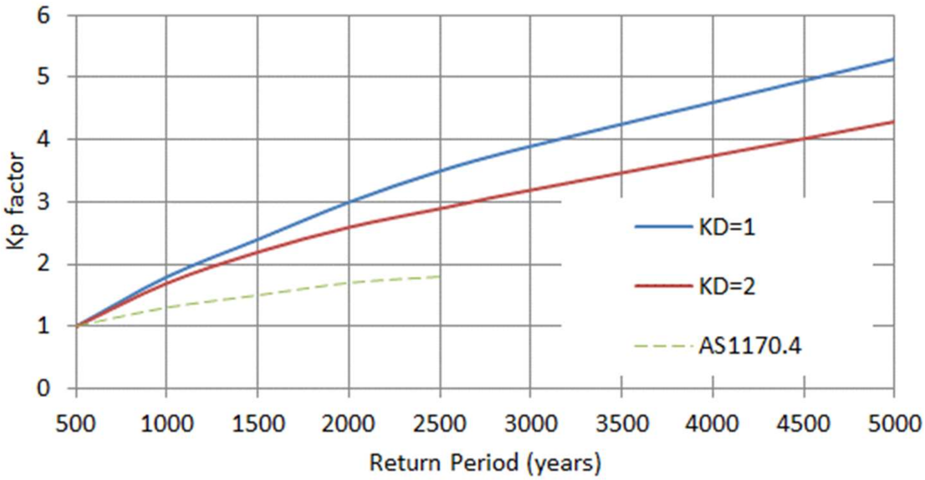
Calculations for the EPGA values were then repeated for return periods up to 5000 years. *RSA* values that have been normalised with respect to the 500 year reference RP value are presented in Figures 14a and 14b in the form of return period (K_p) factors for comparison with recommendations in codes of practice (e.g., AS1170.4 2007). It is shown that the value of K_p for a return period of 2500 years is of the order of 3.0, to be compared with the code specified value of 1.8. K_p values calculated for 0.3s and 1.0s periods show similar comparisons.

It is stated in the footnote attached to Clause 2.1 in Eurocode 8 – Part 1 that ground motion intensity in a rare earthquake event, consistent with a 10% chance of exceedance for a design life of 50 years (i.e. return period of 475 years) is recommended for the design seismic action. It is noted that this recommendation was drafted in the late 1990’s at a time when it was still the norm not to consider return periods exceeding 500 years in the design of structures supporting ordinary buildings. Implicit in the no collapse (NC) performance criterion is that the building is expected to have sufficient additional reserve capacity to sustain a very rare, and extreme, earthquake event without experiencing wholesale collapse (Fardis 2009).

Seismic actions to be considered for design purposes are based on a return period of 2500 years when fulfilling the performance criterion of collapse prevention, and then scaled down by a factor of 2/3 (reciprocal of 1.5) for fulfilling the NC, or life safe, performance criterion (Fardis 2009). In a high seismicity region, the scaled-down intensity of ground shaking would be close to the intensity corresponding to a RP of 475 years. It is shown in Figures 14a and 14b that the scaled-down intensity could be a great deal higher than the intensity corresponding to a RP of 475 years. Thus, the term "notional return period" is introduced herein to refer to the scaled down intensity which may or may not be consistent with the intensity corresponding to a RP of 475 years.



(a) 0.3s period



(b) 1.0s period

Figure 14. Return Period Factor M_{min} = 4 and M_{max} = 7 (response spectral values are based on 5% viscous damping)

Table 1. Recommendations derived from the literature on minimum seismic hazard

Ref.	Literature References	Comments / Recommendations
1	Mulargia, Stark & Geller (2017)	"The insufficiency of the available seismicity data means that it is highly questionable that PSHA can provide reliable and accurate hazard analyses".
2	Lam, Tsang, Lumantarna & Wilson (2016)	Minimum design PGA value of 0.07g is recommended for stable regions.
3	Wilson (2015)	Minimum design PGA value of 0.08g is recommended for the stable region of Australia.
4	Tongkul (2016)	Design PGA value of 0.05g – 0.1g is recommended for Peninsular Malaysia and Sarawak.
5	Pinto (2000)	Minimum design PGA value of 0.07g – 0.1g.

4.4 Ground motion models for different tectonic and crustal classifications

The Next Generation Attenuation of eastern North American (NGA-East) database comprises 29000 records from 81 intra-plate earthquake events recorded at 1379 stations (PEER 2015). Ground Motion Models (GMMs) derived from this database, which is currently the most elaborate intra-plate earthquake database, should be taken as indicative of the intrinsic source characteristics of earthquakes generated in an intra-plate environment. A literature review of seismological ground motion model studies in *Eastern North America* (ENA) identified some 40 models developed in the period 1983-2014 (PEER 2015). Many of these models are seismological models requiring stochastic simulations to transform them into GMMs for engineering applications. A subset of 22 models was selected based on the quality and age of the data. Further screening managed to reduce the 22 models to 6 representative models (PEER 2015). The point source simulation models of Atkinson and Boore (1995), Boatwright and Seekins (2011) and Boore (2010), abbreviated herein as AB95, BS11 and BCA10d respectively have been found to generate ground motions that are the most consistent with recorded data.

An independently developed GMM by Darragh et al. (2015), abbreviated herein as *DASG15*, as introduced in PEER (2015), has also been constructed from a seismological model, derived more recently from the broadband inversion of the *NGA-East* database. Good consistencies have been observed with predictions by DASG15 and AB95 whereas other GMMs such as those developed by Shajouei and Pezeshk (2015) and Pezeshk et al. (2015), abbreviated as *SP15* and *PZCT15* respectively, are not as consistent.

The crustal conditions of tectonically stable regions can either be shield (or cratonic) or non-cratonic for a range of regions around the world. It is shown that two tectonically stable regions can have different crustal classifications. For example, the crustal conditions in Southeastern Australia and Peninsular Malaysia are much closer to those of California than of Central North America. Most ground motion data used to develop ground motion models of NGA-East were collected from the shield crustal region of Central North America (i.e., region 2 as defined in chapter 1 of PEER2015/04). However, it is a mistake to consider NGA-East models to be automatically suited for application across all intra-plate regions. Adapting GMMs for use in low-to-moderate seismicity countries must take into account factors controlling (a) the wave generation behaviour at the source of the earthquake in a given tectonic setting and (b) the wave modification behaviour of the earth's (basement rock) crust. These are not to be confused with the modification behaviour due to near-surface sediments. Most of the GMMs of NGA-East are implicitly representative of shield, or cratonic, conditions, which are identified with negligible modifications to seismic waves as they propagate up the crustal layers. In contrast, a GMM model adopted for use in non-cratonic regions requires substantial modification to take account of the wave amplification and attenuation within the upper 3 – 4 km of the earth's crust.

The authors have experience of combining the source model of AB95 with the (non-cratonic) crustal model of generic rock (Boore and Joyner, 1997, abbreviated herein as BJ97) for predicting ground motions generated by intra-plate earthquakes in what has been described as the Component Attenuation Model framework (Lam et al. 2000; 2010). The crustal model of BJ97 has since been made more versatile by parameterising the V_s (30 m) value as an input parameter (Chandler et al. 2005; Boore 2016) in order that any crustal velocity profiles that are intermediate between the classical generic rock and generic hard rock limits can be incorporated into an existing ground motion simulation framework. An alternative crustal velocity profile modelling approach has also been developed for various crustal conditions (Chandler et al. 2005; 2006a). Simulated RSA values for the non-cratonic version of AB95 based on the classical generic rock class of Boore and Joyner (1997) are representative of non-cratonic regions. The credibility of those simulations under the framework of the Component Attenuation Model has been established by demonstrating agreement with field observations from different countries (expressed in the form of Intensity data) as shown in some earlier journal publications by the authors and their co-workers over the years (e.g., Chandler and Lam 2002; Chandler et al. 2006b; Lam et al. 2003 & 2006; Tsang and Lam 2010; Yaghmaei-Sabegh and Lam 2010). Predictions by the (non-cratonic) model can be significantly higher than the upper limit of predictions by the NGA-West2 models. Thus, the use of NGA-West2 models or NGA-East models in PSHA could result in the seismic hazard being understated in tectonically stable, and non-cratonic regions, such as Southeastern Australia, Malaysia and a large part of China.

5. Site classification and soil response spectra

Zoning maps of certain parameters, as shown in the previous section, specify only one value for each region of the grid. Such a value represents the hazard level of an average (or reference) site in the region. The site-to-site variability of design ground motions has to be taken into account by scaling parameter(s). In codes of practice, a site can be classified into a number of pre-defined site classes, and site effects are commonly related to a reference site class.

Generally, site effects characterise the filtering mechanisms and superposition of reflected waves within the soil sedimentary layers overlying the bedrock. Ground motions can be significantly modified, in terms of amplitude and frequency content, as seismic waves propagate through the near-surface sedimentary layer. The degree of site effects mainly depends on the level of shaking, the thickness of the soil layer, and the properties of the soil (e.g. shear modulus and plasticity) as well as the bedrock materials underneath (shear modulus mainly).

Site effects can be conveniently observed on response spectra. Figure 15 shows the acceleration RS recorded on rock and soil sites, respectively, at Oakland Outer Harbour in the 1989 earthquake at Loma Prieta, California, United States (Dickenson et al. 1991). It is clear that the spectral acceleration values are a few times larger on a soil site than on a rock site. The amplification ratio is of the order of four times for the peaks at 0.7 s. Such significant effects have to be properly taken into account in constructing ESs based on codes of practice.

In situations where a distinct soil-rock interface exists, the amplification ratio usually has its maximum value at the site natural period of the soil layer (T_S), which can be estimated using Eq. (10) if the thickness (d_i) and shear wave velocity (SWV) ($V_{s,i}$) of the individual soil layers are known. Alternatively, the value of T_S can be expressed in terms of the total thickness of the soil layers (H_S) and their weighted average SWV (V_S) as shown in Eq. (10).

$$T_S = \sum_{i=1}^{n} \frac{d_i}{V_{s,i}} \times 4 = \frac{4H_S}{V_S} \tag{10}$$

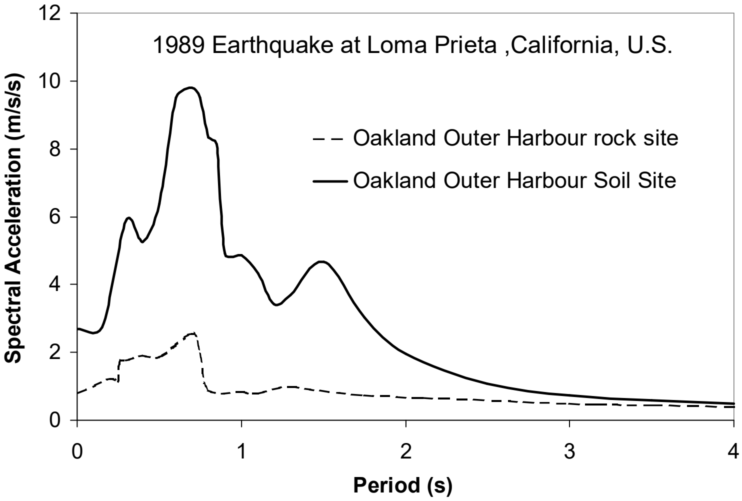


Figure 15. Acceleration RS recorded on rock and soil sites, respectively, at Oakland Outer Harbour in the 1989 earthquake at Loma Prieta, California, United States (Tsang *et al.* 2017a)

5.1 IBC (2015), EC (2004) and NBCC (2015)

Although the importance of the total soil layer thickness has been well recognised, site classification nowadays is based solely on the properties, to a certain depth, of the near-surface materials, since incorporated into the 1994 NEHRP Provisions (BSSC 1995). This definition permits sites to be classified unambiguously, as pointed out in Dobry et al. (2000).

In IBC, EC and NBCC, a site is classified according to the value of the average SWV ($V_{s,30}$), or the value of Standard Penetration Resistance Test (SPT-N) (for cohesionless soil), or the value of undrained shear strength (for cohesive soil), over the upper 30 m (or 100 feet). The IBC classification scheme is described in more detail in this sub-section. NBCC essentially adopted the same classification scheme as in IBC, whilst that in EC is slightly different. In IBC, a site is classified as either Site Class A, B, C, D, E or F based on the site soil properties. Profiles containing distinctly different soil and/or rock layers are subdivided into layers designated by a number from 1 to *n* at the bottom for the total of *n* distinct layers in the upper 30 m. The symbol i refers to any one of the layers between 1 and n. The average shear wave velocity $V_{s,30}$ can be calculated by applying Eq. (11).

$$V_{s,30} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{s,i}}} \tag{11}$$

where $V_{s,i}$ = The shear wave velocity in m/s;

d_i = The thickness of any layer between 0 and 30 m.

Eq. (11) also applies to the computation of the values of SPT–N and undrained shear strength. The classification scheme is shown in Table 2. Note that IBC contains more detailed descriptive criteria (e.g. based on plasticity index, PI) and steps for classifying a site into Site Classes E and F. Refer to the code for details.

As mentioned in Section 5.1, the site-dependent ES as shown in Figure 9 is scaled by the site coefficients F_a and F_v, to take into account the site effects for short-period and long-period ranges respectively. The derived short-period site coefficients are based on recorded motions over the period range 0.1s to 0.5s, whilst the long-period site coefficients are based on recorded motions over the period range of 0.4s to 2.0s.

Table 2. Site classification scheme adopted in IBC

Site Class	Soil Profile Name	$V_{s,30}$ (m/s)	SPT–N	Undrained Shear Strength (kPa)
A	Hard rock	> 1500	N.A.*	N.A.*
B	Rock	760 – 1500	N.A.*	N.A.*
C	Very dense soil and soft rock	360 – 760	> 50	> 100
D	Stiff soil	180 – 360	15 – 50	50 – 100
E	Soft soil	< 180	< 15	< 50
F	Special soils requiring site-specific evaluation			

* N.A. = Not applicable

The site-dependent ES in NBCC is also scaled by the site coefficients F_a and F_v, as modified based on a reference Site Class C. EC applies a uniform soil factor S across the whole ES for each site class (ground type), whilst varying the corner period T_1 between 0.4s to 0.8s (for Type 1 spectrum model). Larger values of T_1 essentially translate to a higher demand at the long-period range.

5.2 AS (2007) and NZS (2004)

In AS and NZS, the site classification schemes are very similar. Sites are classified into five classes using methods recommended in the standard. For rock sites, compressive strength or average SWV over the top 30 m is used. For soil sites, the site natural period T_S (refer to Eq. (10)), depths of soils H_S, undrained shear strength and SPT–N values are used.

As stated in the Commentary to AS (AEES, 2007) and NZS (SNZ, 2004), the basic parameter for site classification in the standard is the site natural period. The site-period approach recognises that deep deposits of stiff or dense soils exhibit long-period site response characteristics not shown by deposits of only a few tens of metres of the same material. The approach adopted in IBC of placing deep stiff soil sites in Site Class D – stiff soil, with relatively short-period spectral characteristics may be non-conservative in the medium to long period spectral range.

A hierarchy of measurement methods, listed in order of preference, is provided. The most preferred method to determine site periods is based on four times the shear-wave travel-time from the surface to the underlying rock. Using bore logs, and measurement of geotechnical properties is the second preferred method.

Upon classifying a site, an ES can be drawn up according to the *spectral shape factor* C_h (as a function of T) corresponding to the site class specified in the standard.

5.3 GB (2010)

In GB, a site is initially classified according to the value of equivalent SWV (V_s) which can be calculated from Eq. (12).

$$V_S = \frac{H_S}{\sum_{i=1}^{n} \frac{d_i}{V_{s,i}}} \tag{12}$$

where H_S is the total thickness of soil sediment, or 20 m, whichever is the smaller, d_i is the thickness of any layer within H_S.

Based on the equivalent SWV (V_S), the site is classified into a specific soil type. A site is then assigned to one of the classes I_0, I_1, II, III, IV, according to both the equivalent SWV (soil type) and the total thickness (in units of m) of the overlying soil.

Upon assigning the site to a class, together with the design earthquake group as determined in Section 6.2, the characteristic (corner) period T_1 can be obtained. Larger values of T_1 essentially translate to a higher demand at the long-period range for softer or deeper soil sites. It is noted that the value of T_1 has to be increased by 0.05 s when constructing an ES for a low-probability of exceedance design level.

5.4 Elastic response spectrum (ES) for flexible soil sites

A heuristic site-specific elastic response spectrum (ES) model for flexible soil sites (T_i > 0.5 s) which takes into account resonant-like amplification behaviour of soil sediments has been developed. The construction of the idealised soil ES model, with parameters of site natural period (T_s) and soil amplification factor (S), as depicted in Figure 16, was first proposed by Lam et al. (2001). Analytical investigations undertaken by Tsang et al. (2006a; 2006b; 2012) employing fundamental principles in relation to the reflection and propagation of shear waves in a homogenous soil medium, elaborated the idea much further, resulting in the development of an algorithm for determining the values of these two parameters, as an alternative approach to that of the classical 1D site response analysis using program SHAKE (Schnabel et al. 1972). Predictions by the alternative calculation methodology, which is much simpler and transparent to the user, has been well validated against results obtained from dynamic analyses of soil column models derived from real borehole records, as well as from strong motion data recorded during the 1994 Northridge earthquake.

This alternative soil response modelling methodology has been refined and simplified further into a model of potential utility in structural design practice (Tsang et al, 2017a; 2017b). In the proposed procedure, the initial small-strain site natural period (T_i) is first calculated from information on the shear wave velocity (SWV) values (inferred from

SPT-N values) and the thicknesses of the individual soil layers. The effects of period-shift resulting from shear straining (non-linearity) of the soil materials are taken into account in the estimation of the (shifted/final) large-strain site natural period (T_S). Design charts have also been developed for T_S and S as functions of the SWV of the soil, the intensity of ground shaking and the rigidity of the bedrock medium. These factors are represented by parameters $V_{s,i}$, RSV_{Ti} and V_R respectively. See Figs.17, 18a & 18b for example design charts used to construct the ES model for a flexible soil site. The proposed ES model, presented in the displacement format and the acceleration-displacement response spectrum (ADRS) format (Figure 13) can be compared with results from higher-tier site response analysis (using program STRATA) and code specified ES models to see the significant differences. Some example comparisons are in Tsang et al. (2017a), and an example for a flexible soil site is reproduced in Figure 19.

The approach to site classification and modelling of an ES for a flexible soil site as described in this section, featuring the site natural period as the controlling parameter, is consistent with recommendations by Pitilakis et al. (2012 & 2013).

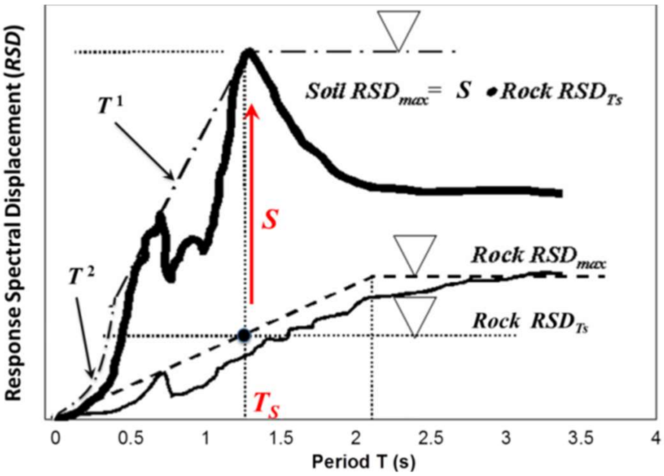


Figure 16. Schematic diagram of the proposed site-specific ES model (in RSD format) (Tsang et al. 2017a)

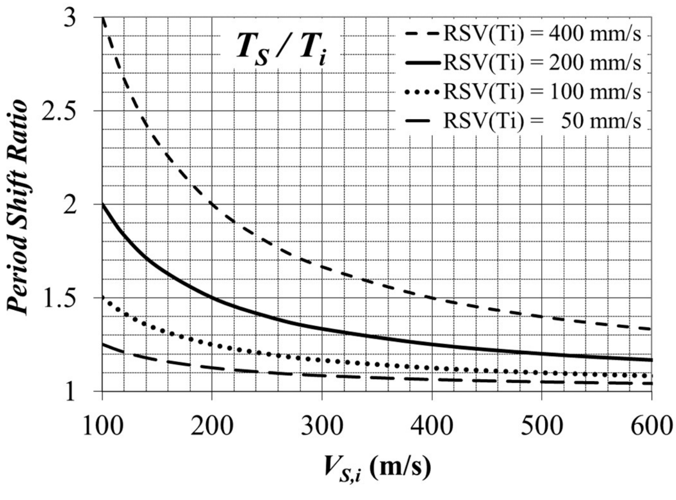
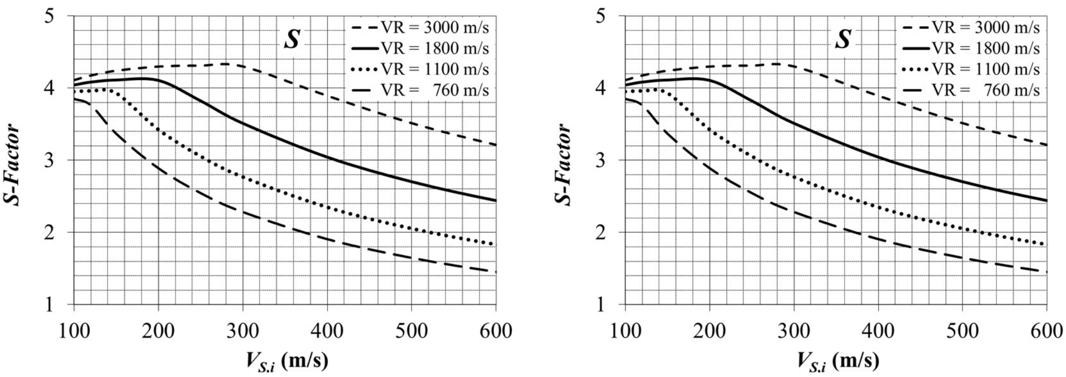


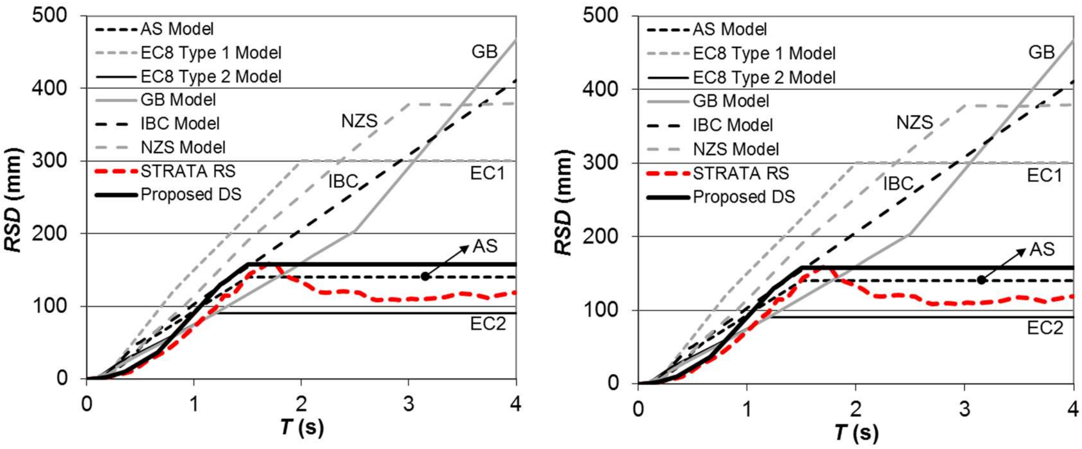
Figure 17. Design chart for the period-shift ratio TS/Ti as functions of intensity of shaking (RSVTi) and initial soil SWV (VS,i) (Tsang et al. 2017a)



(a) function of initial SWV on soil and rock ($V_{S,i}, V_R$) fixed at RSV_{Ti}= 200 mm/s

(b) function of intensity and initial SWV on soil ($RSV_{Ti}, V_{S,i}$) fixed at V_R = 3000 m/s

Figure 18. Design charts for site amplification factor (*S-Factor*) (Tsang et al. 2017a)



(a) RSD format

(b) ADRS format

Figure 19. Example comparison of the proposed ES model with results from higher-tier site response analysis (using program STRATA) and code models (Tsang et al. 2017a)

6. Conclusions

This chapter introduces contemporary methodologies concerned with the modelling of seismic actions and the multitude of formats used to present key information. There is no intention of using the chapter to waive away the need for the reader to systematically plough through the literature to gain the requisite working knowledge across so many topics. The earlier sections of the chapter, however, may well help to speed up this learning process. Seismic hazard modelling in regions of low to moderate seismicity is a controversial topic presenting many factors for code drafters and users to consider. It is hoped that readers of this chapter will gain a clear perspective on the factors concerned. Code drafters, in the future, may also apply the guiding principles, when developing realistic earthquake loading models, as appropriate, for the range of site classes.

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