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1. INTRODUCTION

This state-of-the art report has been undertaken within the DRM-MERM Project to define the primary framework for the Seismic Zonation Manual, under preparation as the main output of this project, with the aim of improving the zonation methodology in Turkey. Thus the review of the literature has been conducted taking into consideration the state-of-the-practice in Turkey. Even though significant effort has been made to conduct a thorough review, due to the multi-disciplinary nature of the topic and very large number of papers, it would be unrealistic to claim that all the literature on seismic microzonation and related disciplines have been reviewed.

Seismic microzonation can be considered as the preliminary phase of earthquake risk mitigation studies. It requires multi-disciplinary contributions as well as comprehensive understanding of the effects of earthquake generated ground motions on man made structures (Ansal & Slejko, 2001). Seismic microzonation is evaluation and assessment of different inputs from different fields of earthquake engineering and engineering seismology. In most general terms, seismic microzonation is the process of estimating the response of soil layers under earthquake excitations and thus the variation of earthquake characteristics on the ground surface. However, it is also very important to select appropriate ground motion parameters for microzonation that correlate with the observed structural damage as well as that could be implemented in engineering design of the men made structures (Finn, 1991). The key issue in seismic zonation is how to combine contributions from different fields, what should be the weights of geological, seismological, geotechnical and structural information with respect to each other and how to issue recommendations suitable for application by local administrators and practitioners.

In seismic microzonation the variation of earthquake ground motion is studied taking into account the earthquake source and path characteristics as well as geological and geotechnical site conditions in a probabilistic manner. Due to the damage distributions observed during past earthquakes, it became evident that earthquake zonation maps prepared at small scales do not yield the necessary information for risk mitigation at a city level. Seismic zonation maps prepared at country level are based on predefined tectonic source zones and on the seismicity of these zones. The scale of these studies and related zonation are generally in the order of 1:1,000,000 or less. With the increase in the analytical, in-situ and laboratory investigation capabilities, there has been significant increase in the accumulated databases concerning the regional geological formations, earthquake source mechanisms, seismic activity and earthquake ground motion records. In the light of these scientific and technical advances, it became possible and feasible to conduct seismic zonation studies at regional and local levels with continuously increasing scales from 1:100,000 to 1:25,000 and in the case of microzonation 1:5,000. The main objective is to estimate more accurately what may happen during a possible earthquake in the future taking into account all the main controlling factors.

The response of men made structures during earthquakes are not only related with structural features but also are controlled by two main factors: earthquake source characteristics and local site conditions. One way of resolving this problem is to consider the interaction among all three components, the man made structures, site conditions and incoming seismic waves to this location. The characteristics of incoming earthquake waves may change during their propagation through soil layers. In addition soil characteristics may also change due to the induced cyclic excitations. Thus any seismic microzonation study neglecting the probable earthquake characteristics and the effect of the incoming seismic ways would be incomplete as outlined by Aki (1988) and Bolt (1996).

The structural vulnerability is an important factor in the mitigation of the earthquake risk. There may be deficiencies in the engineering design as well as in the construction that would increase the estimated vulnerability of the man made environment. The other issue is the possibility of forces generated by the earthquake to exceed the forces assumed during the design stage, either due to lack knowledge or due to lack of information or due to the probabilistic exceedance level selected for the particular structures (Seed et al, 1994; Naeim, 1995). Even though the main purpose of seismic microzonation is the mitigation of earthquake risk, it is also very important to realize that structural vulnerability is a very central issue and it is not very related to seismic microzonation.

In general, the national seismic zoning maps are prepared in small scales such as 1:1,000,000 or less. The scale of the map indicates the accuracy of the map. On the other hand, seismic microzonation for a city or a town requires 1:5,000 or even 1:1,000 scale studies. Earthquake codes need to utilize these national maps in specifying the minimum design requirements. And mostly these requirements are expressed in terms of peak or effective acceleration and spectral accelerations with respect to structural periods. According to these codes, the maximum ground accelerations are defined for different seismic regions. One of the objectives in seismic microzonation is to determine whether the calculated values are less or equal to the code specified values.

2. SCOPE AND METHODOLOGY

The purpose of this report is to review the literature to summarize the state-of-the-art and state-of-the practice in seismic microzonation that may be considered to be composed of five different phases as; site characterization, evaluation of the seismic hazard; estimation of the ground motion characteristics on the ground surface, assessment of liquefaction susceptibility, assessment of landslide hazard. Thus in reviewing the literature each phase will be treated separately and will be considered in sequential stages as; identification and explanation of the process, compilation of the relevant and needed databases, analysis and interpretation of the acquired databases. The investigations and suggestions encountered in the literature are reviewed with the perspective of mainly offering the scientist and engineers in Turkey information concerning all the stages of seismic microzonation. Therefore the necessary steps in each and all seismic microzonation components will be considered in terms of consecutive stages as; quantification of earthquake ground motion, determination of the necessary geological and geotechnical site conditions, analyzes of the available data, engineering interpretation of the results obtained. The findings and proposals related to all these stages will be reviewed briefly to give the reader a complete understanding of the whole process.

One of the most important issues in any zonation study is the selection of the appropriate scale to be adopted in the final output that would be the seismic zonation maps with respect to different parameters. The scale for the intended output maps in the light of the contemporary seismic microzonation methodologies is towards adopting the scale of 1:5000 with suggested mesh size of 0.005 x 0.005 degrees (roughly, 500 m x 500 m) in identifying, aggregating and analyzing the available data sets. The scales used in different applications will be considered in the light of the intended purpose of the zonation that has been conducted.

In the last section of the report some seismic microzonation case studies conducted in different parts of the world has been reviewed to give the readers a complete picture of the seismic microzonation process.

3. GEOLOGICAL AND GEOTECHNICAL SITE CHARACTERIZATION

Any seismic microzonation study begins with the site characterization based on the identification of the regional and local geological conditions in the investigated region. The primary aim of these studies is to identify the tectonic and geological formations in the selected map scale. This information is essential for determining the seismic source zones and for establishing a realistic earthquake hazard models for the investigated area. The regional neo-tectonic maps as well as surface geology maps and vertical geological profiles would be the essential ingredients of the seismic microzonation study to be carried out.

The characteristics and thickness of geological units and geotechnical site conditions need to be identified based on borings, in-situ geophysical and geotechnical tests for making realistic estimates of seismic wave propagation between the bedrock and the ground surface and for estimating the bedrock depth (Takimatsu, 1997; Su & Aki, 1995). The geological and geotechnical databases would be needed in the analysis to be performed for assessing the local site effects for site amplification as well as for liquefaction and landslide susceptibility.

3.1. Regional and Local Geology

The initial stage in a seismic microzonation is the geological investigation of the region to determine the geological settings. The first purpose is to identify the tectonic formations that may produce future earthquakes (Barka & Kadinsky-Cade, 1988; Barka, 1991, Barka, 1992; Jackson, 2001, Yeats, 1997). As observed in some recent major earthquakes (Kobe 1995, Spitak 1988), one of the reasons declared for the excessive damage was inadequate estimation of earthquake characteristics for the region. In evaluating the seismicity of a region two factors need to be considered. The first factor is the tectonic and geologic formations that can produce earthquakes in the region and the second factor is the seismic history. To understand the possible mechanisms that can generate earthquakes, detailed geological and seismological studies are necessary (Working Group on California Earthquake Probabilities, 1995; Bolt, 1999).

3.2. Geotechnical Site Conditions

The second need for detailed geological and geotechnical investigation is related with identification of local site conditions and their variation both in vertical and horizontal directions (Jongmans et al., 1998; Pitilakis, 1999; Mancuso, et al., 1997).

As shown by many researchers (Borcherdt & Gibbs, 1976, Iglesias, 1988; Gazetas et al., 1990; Seed et al., 1991; Ansal et al., 1993; Lekkas, 1996; Jennings, 1997; Ishihara, 1997; Geuguen et al., 1998; Pergalani et al., 1999; Tertulliani, 2000; Hartzell et al., 2001, Ozel et al., 2002) geotechnical site conditions could play a dominant role in damage distribution as well as in the recorded strong motion records (Aki, 1993, 1998; Bard, 1994; Reinoso & Ordaz, 1997; Shome et al., 1998). The determination of geotechnical site conditions requires identification of the soil stratification and properties of soil layers based on detailed in-situ tests, borings and sampling, and laboratory tests on recovered soil and rock samples (Kokusho, 1987; Atkinson & Sallfors, 1991; Pitilakis et al., 1992).

3.2.1 In-situ tests

The first group of in-situ tests generally conducted to identify the soil stratification and engineering properties of the soil layers are penetration tests. Two methods that have been widely used are the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). SPT is generally used to investigate cohesionless or relatively stiff soil deposits, whereas CPT is used to identify soil properties in soft soil deposits (Lunne et al., 1997).

The variability of the Standard Penetration Test equipment and procedures used have significant effects on the obtained blow counts (Seed et al., 1985; Skempton, 1986). The energy delivered to the split-spoon sampler, is strongly influenced by many factors such as: type of hammer release equipment, expertise of the operator, size of the cathead, diameter of the rope, number of wraps of the rope around the cathead, hammer type, borehole diameter, rod length, rod diameter, tightness of the rod joints, verticality of the rod string, and type of sampler. Therefore it is very important to have sufficient information to estimate the energy ratio correction for SPT blow counts before using these results for assessing the properties of soil layers.

Empirical relations have been proposed to correlate the penetration test results between CPT and SPT (Robertson et al., 1983) as well as with the shear-wave velocities (Ohta & Goto, 1978; İyisan, 1996; Mayne & Rix, 1995). A list of some of the relationships proposed to calculate shear wave velocity in terms of SPT and CPT results is given in Table 3.1.

Table 3.1 Proposed relationships to estimate shear wave velocity from in-situ penetration tests

Researchers	Relationship V_s in m/sec	Soil Type
Imai (1977)	$V_s = 91 N^{0.337}$	All
	$V_s = 102 N^{0.292}$	Clays
	$V_s = 80.6 N^{0.331}$	Sands
Ohta & Goto (1978)	$V_s = 85.35 N^{0.348}$	All
	$V_s = 67.79 N^{0.219} D^{0.230}$ D: m	Fine sands
	$V_s = 62.14 N^{0.219} D^{0.230}$ D: m	Clays
Barrow & Stokoe (1983)	$V_s = 154 + 0.64 q_c$ q_c :kg/cm ²	All
Sykora & Stokoe (1983)	$V_s = 100.5 N^{0.29}$	Sands
	$V_s = 134 + 0.52 q_c$ q_c :kg/cm ²	Sands
Jinan (1987)	$V_s = 116.1(N + 0.3185)^{0.202}$	Holocene
Lee (1990)	$V_s = 57.4 N^{0.49}$	Sands
	$V_s = 114.43 N^{0.31}$	Clays
	$V_s = 105.64 N^{0.32}$	Silts
İyisan (1996)	$V_s = 51.5 N^{0.516}$	All
	$V_s = 55.3 q_c^{0.377}$	All
Kiku et al., 2001	$V_s = 68.3 N^{0.292}$	Adapazari soils

The second group of tests is involved with the measurement of the shear-wave velocities of the soil layers using seismic exploratory or geophysical methods (Zen et al., 1978; Lefebvre et al., 1994; Brignoli et al., 1996; Stokoe et al., 1999, Stokoe & Santamarina, 2000). Widely used methods are PS logging, down-hole logging and cross-hole logging (Mancuso, 1994; Raptakis et al., 1994). In down-hole logging, the travel time used by vertically propagating shear-waves from a source on the surface to a subsurface receiver along a borehole is measured (Figure 3.1). Cross-hole logging is based on subsurface measurements in which the travel time is measured by horizontally propagating shear-waves from inside a borehole to neighboring boreholes (Stokoe & Woods, 1972; Hall & Bodare, 2000). Cross-hole logging also has the advantage of identifying the properties of the soil deposit between the boreholes. Furthermore, estimation of the shear-wave velocity from excitation of the surface waves, by determining the dispersion curve and applying an inverse analysis, has been also performed by many researchers (Nazarian & Stokoe, 1984; Tokimatsu et. al.1991; Stephenson et al., 2000; Dutta et al., 2001).

The shear wave velocity, V_s , is a soil property used to determine the shear modulus, G , of the soil:

$$G = \rho \cdot V_s^2 \tag{3.1}$$

where $\rho = \gamma_t/g_a =$ mass density, $\gamma_t =$ total unit weight, and $g_a =$ gravitational acceleration = 9.8 m/s^2 . At small strains, particle motion resulting from propagation of shear waves would give maximum small strain value, G_{max} but as the shear strain increases, the shear modulus will decrease. In-situ tests are assumed to produce small strains and the measurement of shear wave velocity will be directly related to the maximum shear modulus

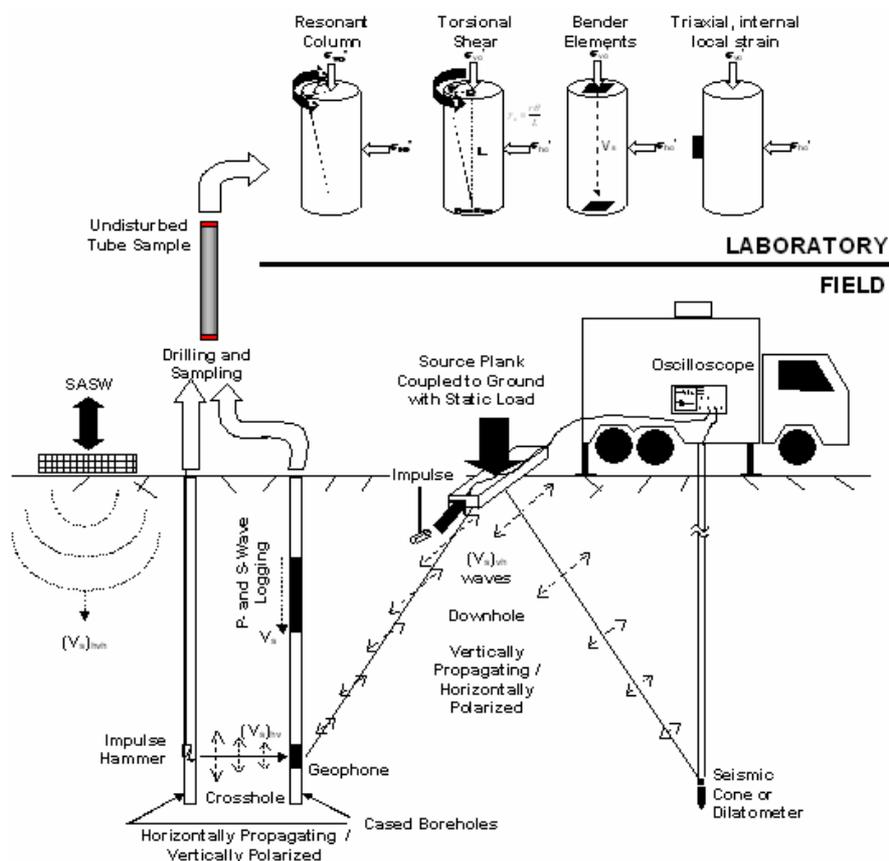


Figure 3.1 Field and Laboratory Methods for Determining Shear Modulus
(After Schneider et al., 1999)

In addition more conventional geophysical methods such as seismic refraction and reflection testing procedures are also widely used to determine the shear velocity profiles and equivalent shear wave velocities as well as soil stratification and bedrock depth (Williams et al., 1999, 2000).

More recently ambient noise array measurements were also utilized to estimate the variation of shear wave velocity profiles extending down to greater depths (Lu et al., 1992; Yamanaka et al., 1994; Yamanaka, 1998; Satoh et al., 2001a, 2001b; Seht & Wohlenberg, 1999; Louie, 2001; Milana et al., 1996).

On the other hand, Wills (1998) and Boore & Brown (1998) question the reliability of shear wave velocities measured using different techniques. Therefore it is important to adopt more conventional and reliable testing procedures to determine the shear wave velocities of the soil layers encountered in the soil profile.

3.2.2 Laboratory Tests

The laboratory tests conducted on soil and rock samples retrieved from boring operations could also be considered in two groups. The first group of tests (i.e. grain size distribution, water content, consistency limits) is needed to determine the soil classification, grain size characteristics and index properties of the soil and rock layers encountered in the soil profile. These tests would allow the classification of the encountered soil layers thus enabling to determine the site classifications according to different site classes proposed in different earthquake codes.

The second group of laboratory tests is conducted to obtain stress-strain-shear strength characteristics of soil specimens under cyclic excitations (Kokusho, 1980; Andersen et al., 1980; Ishihara, 1993). The three basic types of tests are resonant column, impulse wave velocity measurements, and low frequency cyclic loading tests (cyclic triaxial, cyclic simple shear, cyclic torsional triaxial) (Figure 3.1). It would be preferable to determine the dynamic shear modulus degradation curves based on these laboratory tests to be used for site response analyses.

3.2.3 Cyclic Behavior of Soils

Even though there are some similarities in the behavior of coarse grained and fine grained soils under cyclic stresses there are also important differences. In evaluating the behavior of soils under cyclic loads, it is preferable to consider stress-strain and shear strength properties separately. Dynamic shear modulus, damping ratio and their variation with shear strain may be regarded as the dynamic stress-strain properties of soils and would be used for site response analyses (Ishihara, 1982; Dobry & Vucetic, 1987). Cyclic stress amplitudes and number of cycles leading to failure or excessive deformations may be defined as dynamic shear strength properties and may be used to determine areas that would experience such shear strength degradation during earthquake excitations.

Dynamic response of soils under dynamic loads depends to cyclic stress-strain characteristics of the soil in shear. The small-strain shear modulus, G_{max} , the relation between shear modulus ratio, G/G_{max} and the cyclic shear strain amplitude are the basic characteristics of soil deformation and play an important role in dynamic response analysis (Sun et al., 1988; Seed et al., 1986). During earthquakes, soils are subjected to irregular dynamic loads that cause degradation of stiffness and shear strength with respect to number of cycles.

The behavior of soils subjected to cyclic loading has been studied by large number of researchers (Seed & Idriss, 1970; Hardin & Drnevich, 1972; Castro & Christian, 1976; Koutsoftas, 1978; Idriss et al., 1978; Sangrey & France, 1980; Koutsoftas & Fischer, 1980; Ishihara, 1980; Matsui et al., 1980; Wood, 1982; Kokusho et al. 1982; Yasuhara et al., 1982;

Ansal & Erken, 1982, 1989; Ishihara et al., 1983; Fujiwara et al., 1985; Vucetic, 1988,1994; Ohara & Matsuda, 1988; Hatanaka et al., 1988; Ansal & Yildirim, 1989; Kuebris & Vaid, 1997; Vucetic & Dobry, 1991; Georgiannou et al., 1991; Matasovic & Vucetic, 1992; Talesnick & Frydman, 1992; Vucetic et al., 1998; Viggiani & Atkinson, 1995; Ishihara, 1996; Lanzo et al., 1997; Vrettos & Savidis, 1999; Lanzo & Vucetic, 1999). Large numbers of studies were carried out to evaluate stress-strain behavior of soils utilizing both laboratory tests such as cyclic triaxial, cyclic shear, cyclic torsional triaxial, resonant column and in-situ seismic tests such as cross-hole, down-hole and surface waves. The comparison among laboratory and in-situ measured dynamic shear modulus and damping have indicated the importance of testing technique and soil type as well as disturbance and change of stress state during sampling, sample preparation and testing.

Relatively good agreement in the reported semi-empirical relationships such as normalized shear modulus versus shear strain justifies the use of simple models in analyzing soil behavior. However, inherent soil characteristics such as grain size distribution, fabric and soil structure for each soil lead to deviations from generalized and normalized relationships. The importance of nonlinearity of soil behavior under small and medium strain levels observed in laboratory and in-situ investigations demonstrate the necessity to carry out more comprehensive testing programs both in the laboratory and in-situ.

Various experimentally determined formulas, relationships and proposed design curves are available to obtain reasonable estimates of these stress-strain parameters (Sagaseta, et al., 1991; Vucetic and Dobry, 1991; Kagawa, 1992; Ishibashi and Zhang, 1993; Vrettos and Savidis, 1999; Kallioglou et al., 1999). Several expressions have been proposed for estimating the maximum shear modulus, G_{max} , as a function of void ratio, e , OCR, and confining pressure, σ'_c . The following normalized empirical equation to evaluate G_{max} at low-amplitude shear strains can be determined from the work of Hardin and Drnevich (1972):

$$\left(\frac{G_{max}}{p_a} \right) = 321 \frac{(2.97 - e)^2}{1 + e} OCR^M \left(\frac{\sigma'_c}{p_a} \right)^N \quad (3.2)$$

where, e = void ratio (≤ 2.0); M = exponent ranging between 0 and 0.5 depending on the PI; $N \approx 0.5$ for most sandy and clayey soils; σ'_c = mean effective stress; and p_a = reference atmospheric pressure.

An important interpretation of the observed cyclic soil behavior was the suggestion of two shear strain threshold levels, the first one as the beginning point of nonlinear behavior and the second one as the beginning point of inelastic behavior (Vucetic, 1994; Vucetic & Dobry, 1991; Ansal et al., 2001a). An important observation was the effect of soil plasticity expressed as plasticity index on the variation of dynamic shear modulus with respect to shear strain amplitude. For demonstrating the dependence of the dynamic shear modulus on plasticity index the modulus degradation curves obtained by using cyclic triaxial tests on undisturbed samples obtained from various locations in Turkey are shown in Figure 3.2 (Okur & Ansal, 2001).

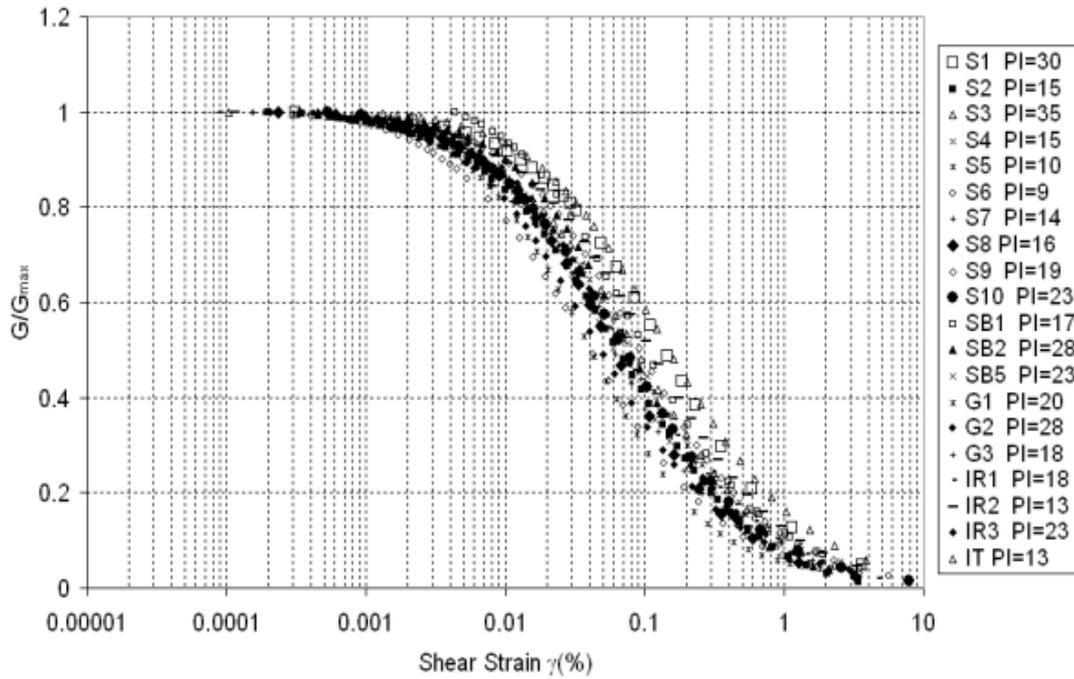


Figure 3.2 Modulus reduction curves obtained from dynamic triaxial test

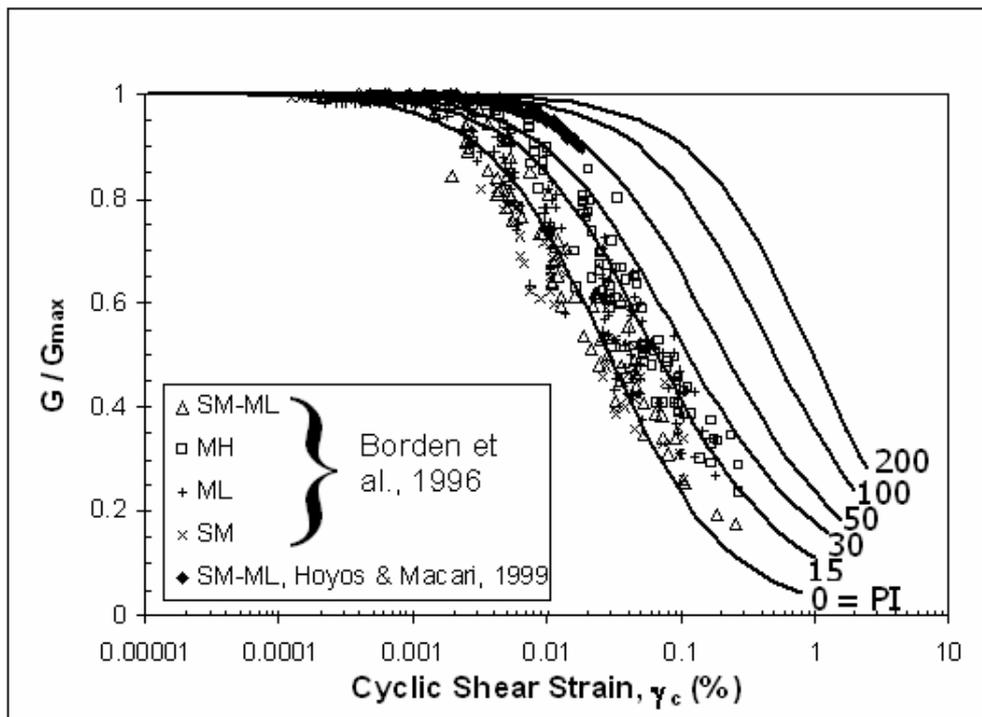


Figure 3.3 Laboratory Data for Piedmont Soils Compared to Normalized G / G_{max} Curves Based on Plasticity as Presented in Vucetic & Dobry (1991) (After Schneider et al., 1999)

As can be seen from Figure 3.2 and as reported by Vucetic & Dobry (1991) Figure 3.3, the modulus degradation is very much affected by the plasticity index of the cohesive soils.

Based on the tests conducted by Okur (2001) it was possible to develop a relationship:

$$\frac{G}{G_{\max}} = \frac{35.09}{\frac{\gamma_a}{1 - 0.99 \exp(-18.97 \cdot PI^{-1.27})} + 34.74} \quad (3.3)$$

to model the modulus degradation where PI is the soil plasticity expressed in percentage.

The other important parameter controlling the cyclic stress-strain characteristics of clays is the number of cycles. This parameter plays a crucial role especially in analyzing the behavior of soil layers under earthquake loads. It may be necessary to define a yield point for each stress-strain curve obtained for a specific number of cycles that can be considered as the "cyclic yield strength" for the corresponding number of cycles (Ishihara, 1980, 1996; Ansal & Erken, 1989).

A Case Study

Pitilakis, et al. (1999) conducted a detailed investigation to determine 2-D soil structure of the test valley known as Euroseistest site that is a relatively wide and deep sedimentary valley of about 5.5 km width, 200 m depth and located 30 km east of Thessalonica. The investigation was based on extensive seismic and geotechnical testing program. The obtained data was evaluated to determine the geological structure, geometry and the dynamic properties of main soil formations. The geophysical and geotechnical surveys performed at 15 locations separated 150 m to 300 m from each other were used to identify 8 main soil formations. Within the geophysical survey both SWI, P and SH refraction methods and cross-hole (CH) and down-hole (DH) seismic tests were used. 84 reversed profiles (lengths ranging from 44m to 60m to 970 m) were measured and 7 borehole seismic tests (3 CH and 4 DH of about 475 m length) were performed. Seismic measurements were followed by geotechnical in-situ and laboratory tests. In situ tests included 10 boreholes with the corresponding SPTs, 6 CPTs, 170 disturbed and 100 undisturbed samples, and 50 water table measurements. 10 additional boreholes were drilled, 3 to perform CH tests, 4 to install 3 down-hole accelerograms and 1 for pore pressure sensors and 3 for explosion tests. The explosion tests were used to investigate the depth of bedrock and to locate the faults. The laboratory tests included conventional tests, resonant column (RC) and cyclic triaxial (CT) tests. Geotechnical and geophysical data were then combined to construct representative 1-D soil profile for each of the 15 sites. Each profile was obtained with the same procedure: First an initial profile is designed based on V_s profile from refraction, surface wave inversion and cross-hole (or down-hole) tests. V_p velocities from refraction surveys were used as a supplementary index to distinguish main soil formations and the water table surface. Then borehole data, SPT values and the geotechnical classification of soil samples were used to adjust initial profile. The mean shear modulus degradation and material damping curves were obtained based on all available RC and CT tests for each soil formation. Once these 15 1-D profiles were obtained the synthesis of these were made in order to evaluate 2D soil model. At this stage, the comparisons and correlations for the main soil parameters were attempted in order to increase the accuracy of the 2-D model of the valley. Comparison of V_s values obtained from different values revealed that there is a good agreement between SWI and CH measurements whereas V_s values based on refraction tests were slightly higher with the difference increasing by soil stiffness. The authors concluded that SWI is an accurate and reliable method. V_s -SPT correlations for two general soil groups; silt-sands and clays were also attempted and two simple relationships were proposed (correlation factors of 0.70 and 0.75). Correlation of cone tip resistance and initial tangent shear modulus (from CH data using $G_o = pV_s^2$) were also performed and the data was found to be consistent with one of the proposed relationships in the literature. Final analysis of the geophysical and geotechnical data and the correlations between main in-situ soil parameters made it possible to determine the cross section of the valley. The final 2-D model of the valley was presented with the mean values of V_s , V_p and

quality factor Q_s and the unit weight for the 8 soil formations. Validation of this model was also made through an empirical study of site effects. Total of 495 accelerograms recorded on the permanent strong motion array that had been installed in the valley were used in site effect analysis. Two well known methods were used: SSR and HVSR. Both techniques showed similar spectral shapes with comparable, fundamental resonance frequencies, which were in good agreement with the obtained geotechnical-geophysical site conditions. The variation of the amplification characteristics was found to be consistent with the geometry and dynamic soil properties. Amplifications obtained from HVSR technique was observed to be systematically lower than those obtained from SSR technique. This phenomenon was attributed to the amplification of vertical motion due to the complex structure of the valley.

As explained above the geological and geotechnical site characterization requires detailed studies based in-situ and laboratory tests as conducted by Pitilakis et al., (1999) in order to have accurate database to be utilized to estimate site response characteristics (Abeki et al., 1995). In any seismic microzonation study the reliability of the site characterization plays a very crucial role. Most of the time and the effort are spent to acquire these data and thus the result of seismic microzonation study depends directly on how well the site characterization studies were conducted.

4. SEISMIC HAZARD FOR MICROZONATION

There are two general approaches in developing ground motions in professional practice: Deterministic and Probabilistic. The deterministic approach is based on selected scenario earthquakes and specified ground motion probability level, which is usually median ground motion or median-plus-one standard deviation. The probabilistic approach encompasses all possible earthquake scenarios, all ground motion probabilities and computes the probability of the ground motion to be experienced at the site exceeding a certain value in a given time period. Empirical attenuation relationships are generally employed in the quantification of seismic hazard in either deterministic or probabilistic approaches. These attenuation relationships exist for Intensity, PGA, PGV, Arias Intensity, duration related parameters and Spectral Accelerations.

4.1 Attenuation Relationships

Attenuation relationships are empirical descriptions providing the median and standard deviation of various intensity measures of the strong ground motion, assumed to be log-normally distributed, in terms of earthquake size, distance, source mechanism and site conditions. Assessment of the seismic hazard requires an appropriate strong-motion attenuation relationship, which depicts the propagation and modification of strong ground motion as a function of earthquake size (magnitude) and the distance between the source and the site of interest.

4.1.1 Factors Affecting Attenuation of Earthquake Strong Ground Motion

Findings (i.e. Somerville, 2000) indicate that while the average ground motions from one large earthquake are similar to those of another, there are conditions that cause the ground motions to vary significantly from one location to another at the same distance from a given event. This variability is related to earthquake source process, propagation and site response.

It has been well recognized that earthquake ground motions are affected by earthquake source conditions, source- to-site transmission path properties, and site conditions. The source conditions include the stress drop, source depth, size of the rupture area, slip distribution, rise time, type of faulting, and rupture directivity. The transmission path properties include the crustal structure and the shear-wave velocity and damping characteristics of the crustal rock. The site conditions include the rock properties beneath the site to depths of up to about few kilometers, the local soil conditions, and the topography of the site.

Recorded strong ground motion especially in the near field incorporates all the heterogeneities, complex arrivals and the high frequency content of the source process. Seismic Moment, Stress Drop, Effective Stress and the Corner frequency are the main parameters of the earthquake source that influences the strong ground motion characteristics. Seismic Moment, M_0 , is the most recognized measure of the earthquake size given by the multiplication of the shear modulus (Lame's constant) of the medium, the average total dislocation (i.e. mean fault offset or slip) and the area of the dislocation surface (i.e. fault rupture surface). Corner Frequency is the frequency where the high and low frequency trends of the Fourier Amplitude Spectrum. It is related to the inverse of the rise time (rate of growth in dislocation or roughly, time duration of rupture). Figure 4.1 through Figure 4.3 show the accelerograms and corresponding response and Fourier amplitude spectra for six records provided by Anderson and Quaa (1988) for a wide range of magnitudes (3.1 to 8.1). All records have epicentral distances of about 25 km and are obtained on rock. As it can be seen in Figure 4.1 with increasing magnitude the amplitudes of ground motion generally increase, and the duration of the accelerogram rapidly increases. Figure 4.2 shows that as the magnitude increase the corner frequencies decrease, the amplitudes of the low frequency waves increase dramatically and

the amplitudes of the high frequencies increase slowly. Similar trends can also be observed in the Fourier amplitude spectra of 1999 Kocaeli earthquake ground motion recorded in Sakarya station, provided in Figure 4.4.

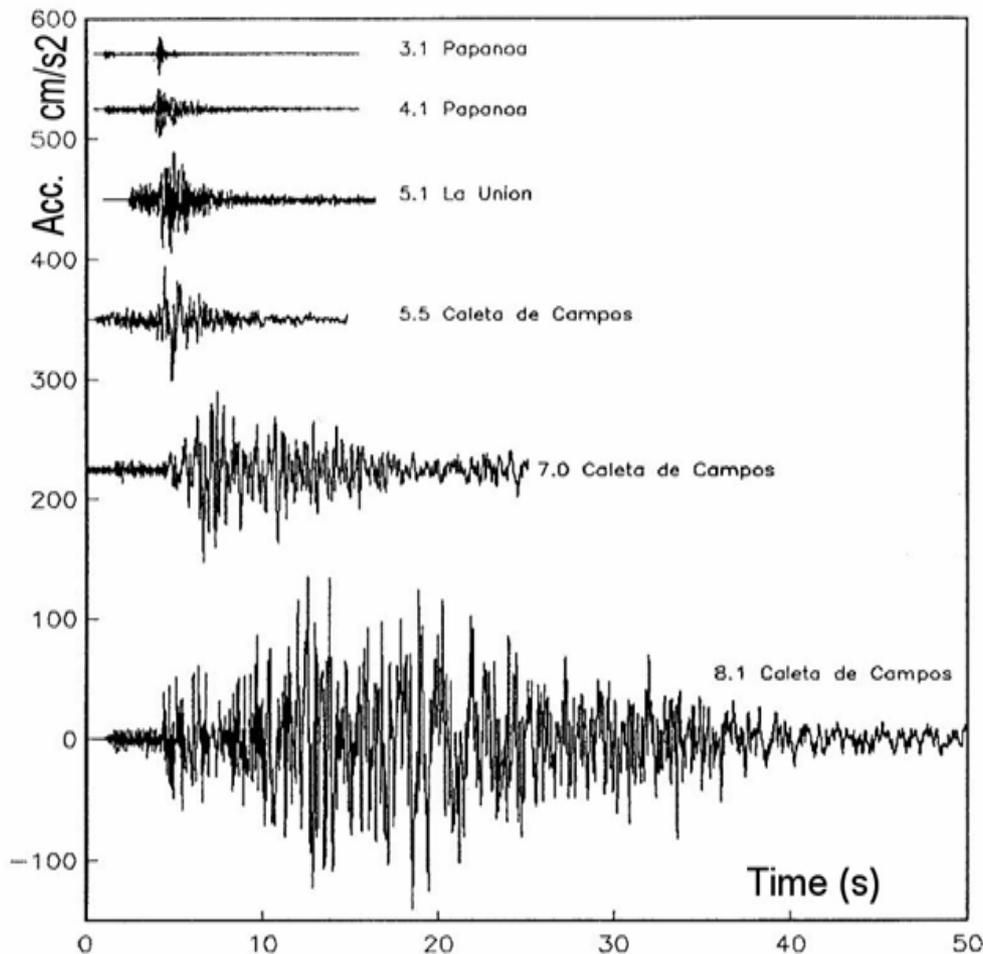


Figure 4.1 Accelerograms for six records of earthquakes with magnitudes between 3.1 and 8.1 provided by Anderson and Quas (1988)

Attenuation is strongly influenced by distance for both the geometric spreading and the material damping. Excluding material damping and considering only geometric attenuation it can be observed that the cylindrical body waves attenuate with inverse of the distance and spherical body waves attenuate with the inverse of the distance squared. The material attenuation is generally given by the following expression;

$$\exp [-(\pi f/Qc) x] \quad (4.1)$$

where f is the frequency, Q is the quality factor that accounts for the material damping, c is the shear wave propagation velocity and x is the distance to source. Assuming almost constant Q , it can be seen that the rate of attenuation increases exponentially with increasing frequency and distance. The spectral shape of the strong ground motion on competent soil sites indicates reduction in the high-frequency regions and increase in the low frequency regions with increasing distance. However, within distances of about 50 km, the effect of distance on spectral shape is much smaller than the effect of magnitude. The duration of the accelerogram tends to increase with increasing distance (e.g., Dobry, Idriss, and Ng, 1978).

Quantification of rupture directivity effects is an emerging component of attenuation studies. Directivity is the effect of rupture propagation along the fault on the ground motion. It

impacts both the high frequency and the low-frequency accelerograms. Let us imagine a rupture propagating at a certain velocity along a fault plane. Stations located in the direction of rupture propagation experience shorter duration ground motions than the ones located in the direction opposite to the direction of rupture. This is called directivity. Associated ground motion amplitudes are larger for stations in the forward directivity region than the ones in backward directivity region due to conservation of energy. At high frequencies, directivity shows up as a short, intense accelerogram at the far end of the fault, in contrast with a lower-amplitude, long-duration accelerogram near the origin of rupture. At high period ranges forward directivity effects at near-fault locations result in high amplitude velocity pulses. The fault normal component of the ground velocity will generally consist of a full cycle velocity pulse, which upon integration will not create a permanent displacement. Whereas, the fault parallel components will generally have half-cycle velocity pulse, which creates a permanent absolute displacement equal to the fault offset.

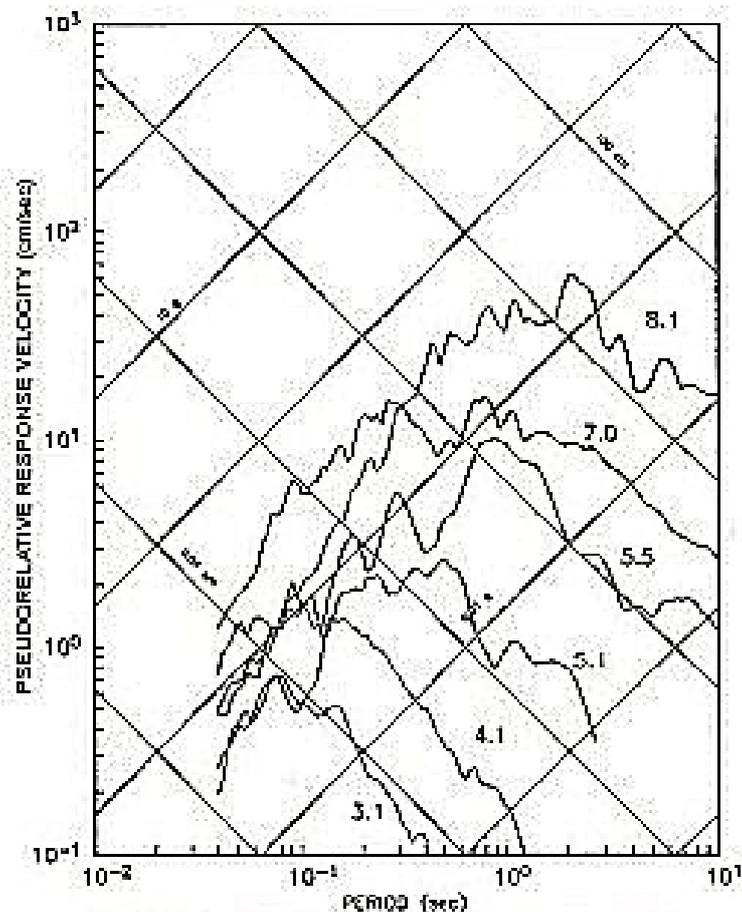


Figure 4.2 Response spectra corresponding to the accelerograms given in Figure 4.1 (provided by Anderson and Quaaas, 1988)

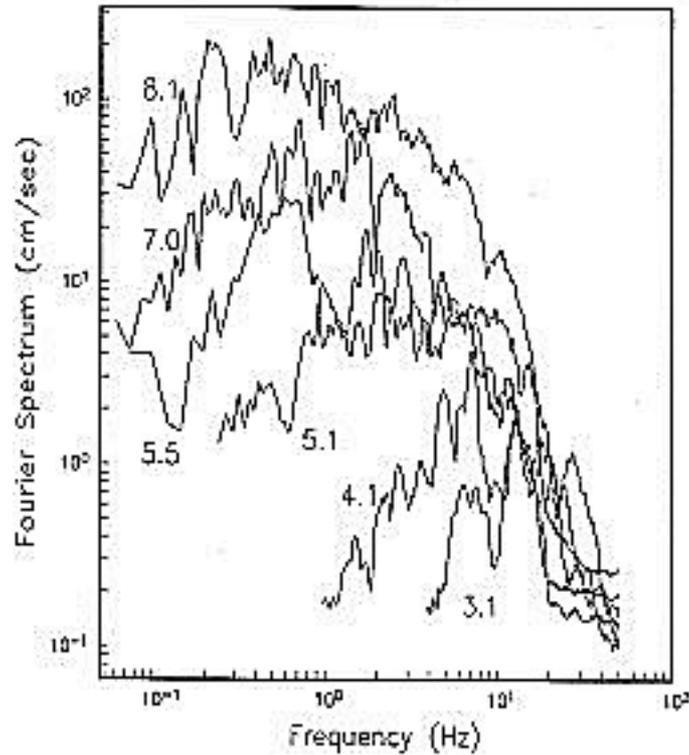


Figure 4.3 Fourier amplitude spectra (whole record) corresponding to the accelerograms in Figure 4.1 (provided by Anderson and Quaa, 1988)

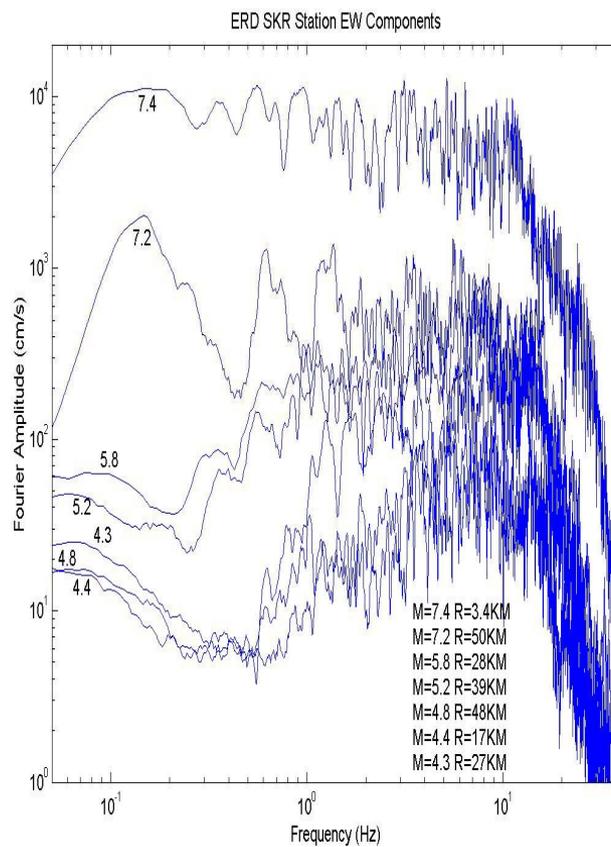


Figure 4.4 Fourier amplitude spectra of different magnitude earthquakes recorded at Sakarya station (data pertain to 1999 Kocaeli earthquake and its aftershocks, courtesy of GDDA)

Luco and Anderson (1983) have studied the near-fault ground motions using a simple theoretical model. The fault is modelled by a finite width, infinitely long vertical strike slip dislocation. The fault, buried in a homogenous half space, extends from a depth of $z_u=2\text{km}$ to $z_d=10\text{km}$. The p- and s- wave propagation velocities of the medium are 6 km/s and 3.464 km/s. A step-type dislocation of amplitude 100cm propagates horizontally with a rupture velocity of 3.184 km/s along the fault. The rupture front is vertical. Fault-parallel, fault-normal and vertical acceleration, velocity and displacement time histories at any observation point along the fault for various distances to the surface projection of the fault are shown in Figure 4.5 (Luco and Anderson, 1983). As it can be seen the peak amplitudes occur at distances (about 2km) comparable to the depth of the top of the fault. Figure 4.5 shows prominent velocity pulses in the fault-normal direction. The durations of the acceleration and velocity pulses increase, especially for fault-normal components, with distance from fault. Fault-normal displacement and velocities stay approximately constant for fault distances less than the vertical distance to the top of the fault. The pulse amplitudes are sensitive to the location of the top of the fault relative to the observation point and inclusion of softer layers at the top of the half-space can lead to substantial amplifications ranging from 2 to 8 (Bouchon, 1987).

Somerville et al (1997) have used empirical recordings from shallow crustal earthquakes in active tectonic regions to illustrate how directivity causes spatial variation in ground motion amplitude and duration around faults, leading to different strike-normal and strike-parallel components of horizontal ground motion and to develop modifications to ground motion attenuation relationships due to rupture directivity. These variations appear to be significant at a period of 0.6 second, and generally grow in size with increasing period. The spectral acceleration is larger for periods longer than 0.6 second, and the duration is smaller, when the rupture propagates toward a site. For sites located close to faults, the strike-normal spectral acceleration is larger than the strike-parallel spectral acceleration at periods longer than 0.6 second in a manner that depends on magnitude, distance, and angle. For design purposes, the variation of the average horizontal response spectra and the difference between the fault-normal and fault-parallel components of the response spectra in near-fault conditions becomes an important consideration.

Somerville et al. (1997) has presented a procedure for the modification of response spectra of near-fault strong ground motion to account for the rupture directivity. The modifications are based on an empirical analysis of limited near-fault data. The modification factor for the spatial variation of average horizontal response spectral acceleration is provided in Figure 4.6 for strikes slip and dip slip faulting. Definitions of the rupture directivity parameters of X and θ for strike slip faults and Y and ϕ for dip slip faults are indicated in the inset figures.

Figure 4.7 provides an empirical model of the ratio of the fault-normal spectral component amplitudes to average spectral amplitudes. Models of this ratio are given for different magnitudes and epicentral distances against period, and for different magnitudes and periods against distance. As it can be assessed, forward directivity caused larger spectral amplitudes at periods larger than 0.6s and the ratio of the fault spectral amplitudes to the average spectral amplitudes can be as high as 1.6 at periods in the vicinity of 6s under favourable forward directivity conditions.

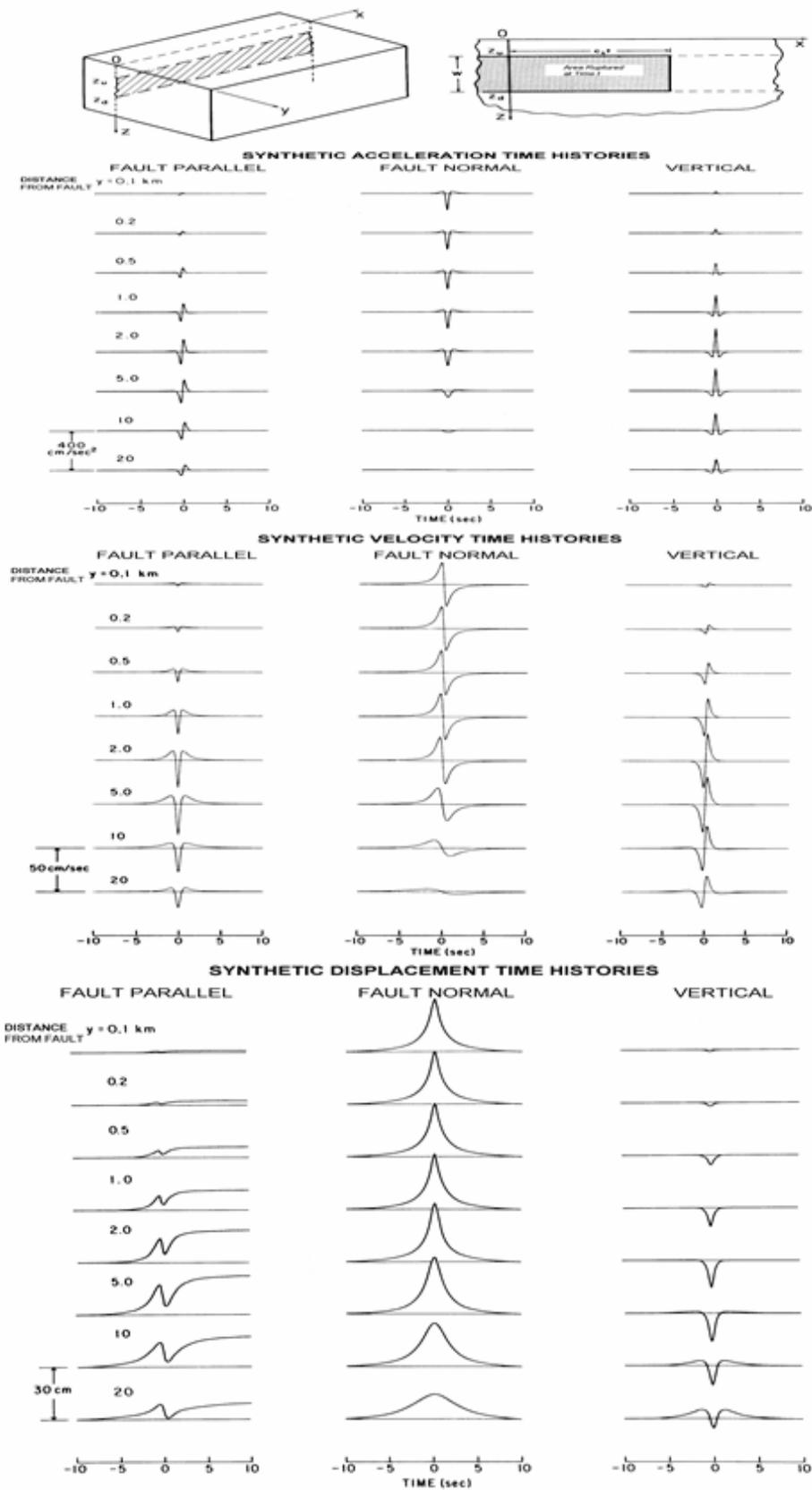


Figure 4.5 Characterization of near-fault ground motion (after Luco and Anderson, 1983)

Somerville et al. (1997)
**STRONG GROUND MOTION ATTENUATION
INCLUDING THE EFFECTS OF RUPTURE DIRECTIVITY**

**SPATIAL VARIATION IN AVERAGE HORIZONTAL
RESPONSE SPECTRA**

FOR $M \geq 6.5$
THE VARIATION IS A FUNCTION OF
 $X \cos \Theta$ (STRIKE SLIP)
 $Y \cos \Phi$ (DIP SLIP)

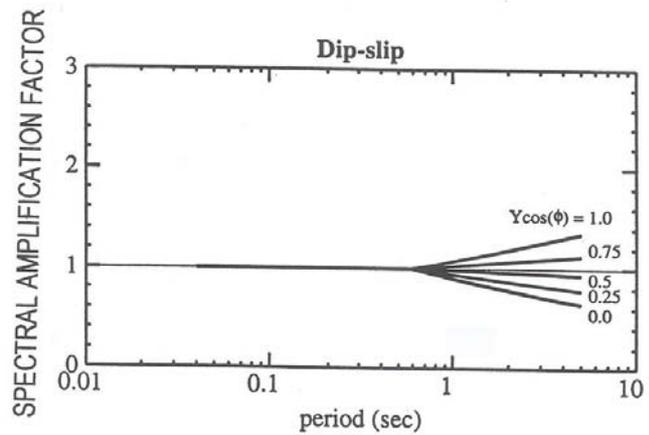
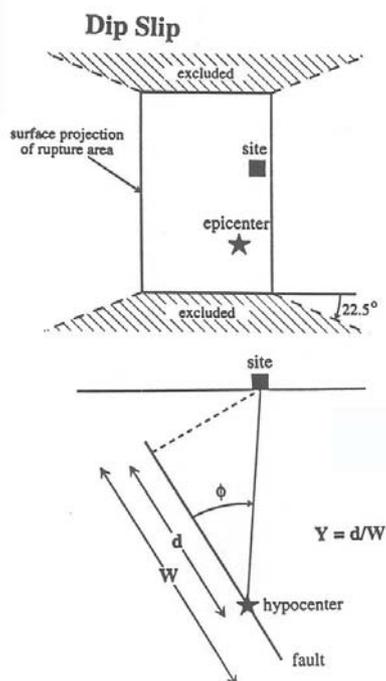
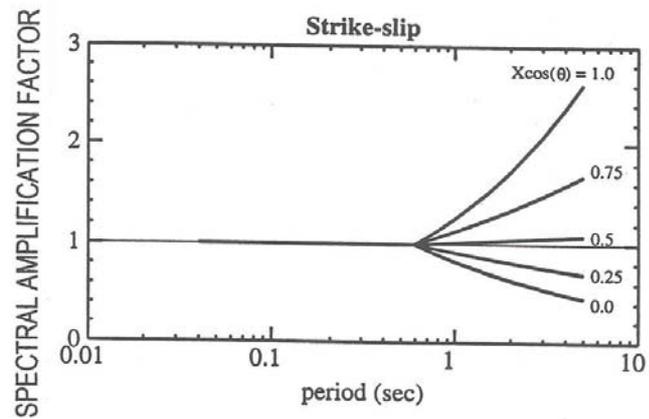
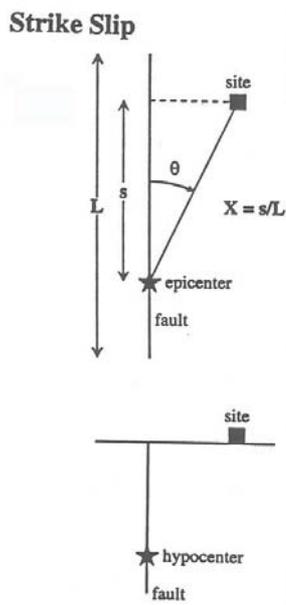


Figure 4.6 Spectral amplification factor given as a factor of period and distance parameter (Somerville, 1997)

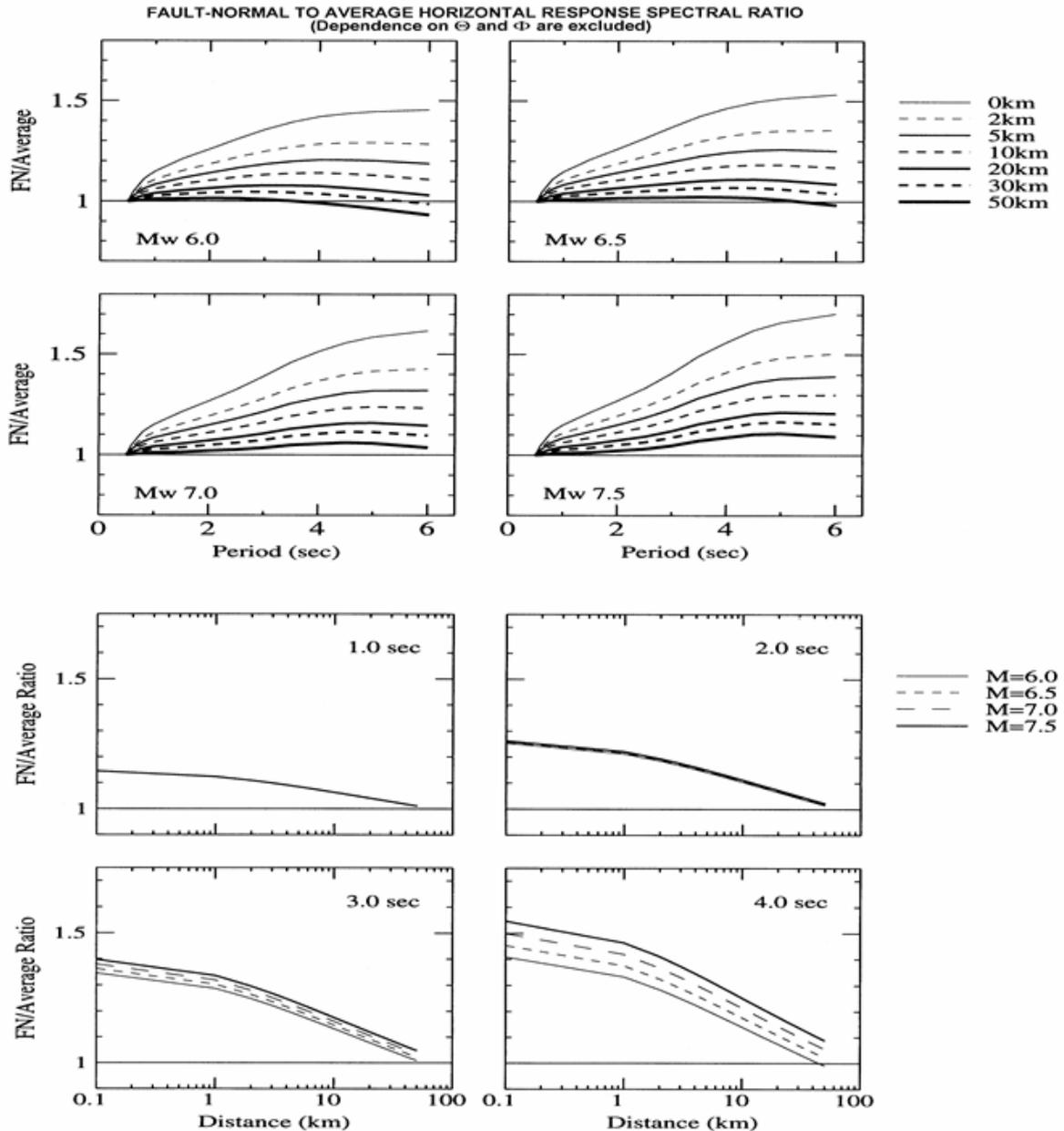


Figure 4.7 An empirical model of the ratio of the fault-normal spectral component to average spectral amplitude (from Somerville et al., 1997)

4.1.2 Appropriate Attenuation Relationships

Attenuation of ground motion has been differentiated with respect to three broad tectonic regimes. These are: Shallow crustal events in active tectonic regions (California, Turkey), Shallow crustal events in stable tectonic regions (Eastern North America) and Subduction Zone (Japan, Chile). For shallow crustal events in active tectonic regions, recent attenuation relationships distinguish between strike-slip, reverse and normal earthquake mechanisms. The ground motion from reverse faulting is about 30% larger than those associated with strike-slip faulting. The ground motion from normal faulting is smaller than those associated with strike-slip faulting.

The general form of the attenuation relationships used by the researchers has been of the following form:

$$Y = b_1 + f_1(m) + f_2(r) + f_3(M,r) + f_4(P) + E \tag{4.2}$$

Where: Y is the strong ground motion parameter to be predicted

$f_1(m)$ is a function of the earthquake size M, usually given by the form $f_1(m)=\exp(b_2 m)$

$f_2(r)$ is a function of the distance r, the most common form being

$f_2(r) = \exp(b_4 r) (r+b_5)^{b_3}$, where b_3 and b_4 represent respectively the geometric and inelastic attenuation rates.

$f_3(M,r)$ accounts for the possible variation of earthquake size measure with distance

$f_4(P)$ is the function accounting for the propagation path and site parameters

E is a random variable representing the uncertainty in Y.

The moment magnitude is currently the preferred scale for the size of the earthquake. Other magnitude scales suffer from the so-called “magnitude saturation” with increasing earthquake size as indicated in Figure 4.8. Most attenuation relationships use the geometric mean of the two horizontal component peaks, some use the peaks of both components of ground motion by treating them randomly, and some use the larger of the two horizontal component peaks. The difference between these approaches can be as much as 15%.

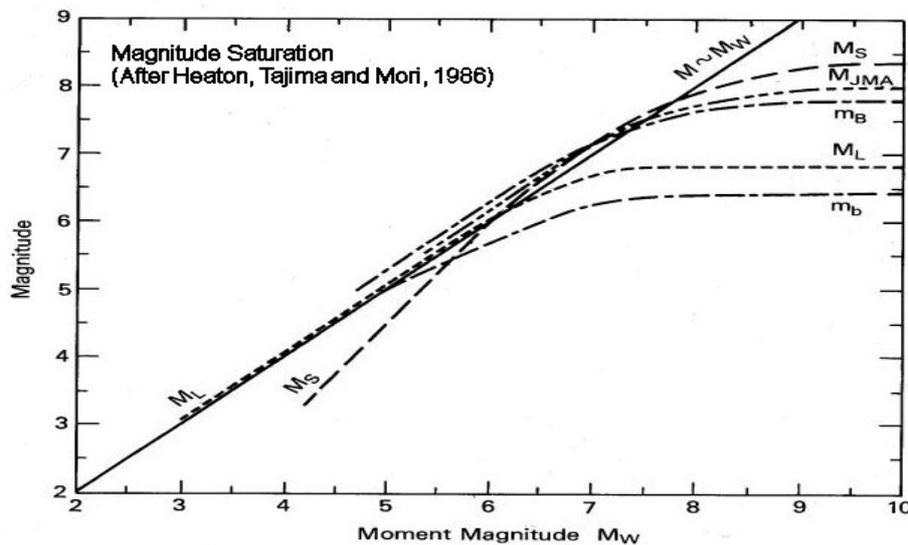


Figure 4.8 Relationship between different magnitude scales. Note the saturation effect on magnitude scales with increasing earthquake size

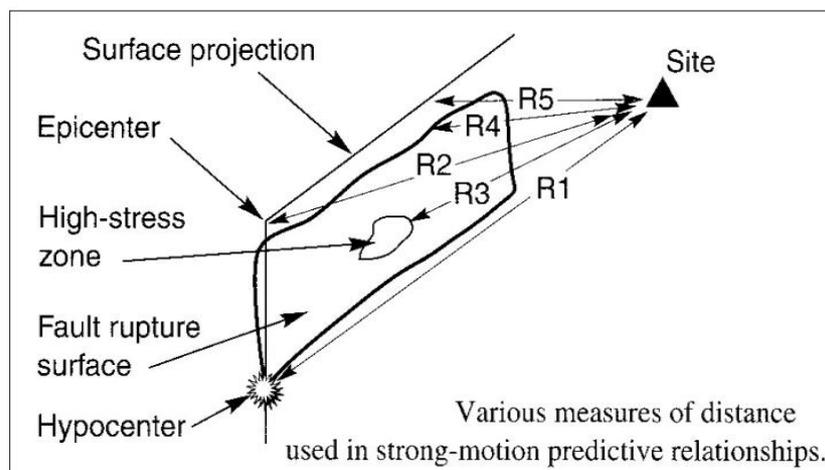


Figure 4.9 Measures of distance used in attenuation relationships

Especially for near field conditions, the definition of the site-to-source distance used gains importance. Commonly used distance measures are: Fault distance (closest horizontal distance to the vertical projection of the fault rupture); Rupture distance (closest slant distance to the rupture surface); Hypocentral distance and; Epicentral distance (Figure 4.9). For the distance parameter, distance to fault has gained importance for correlation with ground motion characteristics.

The deviation of the observed from the predicted strong ground motion (residual) generally fits to a lognormal distribution for up to two standard deviations. The standard deviation of the predictions is in the order of 0.5 natural logarithm units, corresponding to a multiplicative factor of 1.6 (times the mean value) to obtain the value, which exceeds 84 % of the data.

The limited strong motion data in Turkey and also in the Eastern Mediterranean region and ambiguities on the station site descriptions does not allow for the development of reliable region and site specific ground motion attenuation relationships. Owing to the geological and geo-tectonic similarity of Anatolia to the California and also on the basis of favourable predictive comparisons, the attenuation relationships, currently being used for the assessment of earthquake hazard for the Western US (Leyendecker et. al., 2000), can be used for applications in Turkey. Following is short description of these current attenuation relationships.

Boore et al. (1997) PGA and Spectral Acceleration attenuation relationship is based on the selected strong motion data from western North America. The equations predict the random horizontal component peak acceleration and 5% damped pseudo acceleration response spectra in terms of moment magnitude, distance and site conditions for strike-slip, reverse slip or unspecified faulting mechanism. Site conditions are represented by the shear wave velocity averaged over 30m. The smoothed coefficients in the equations for predicting ground motion were determined using a weighted, two-stage regression procedure. In the first stage, the distance and site condition dependence were determined along with a set of amplitude factors, one for each earthquake. In the second stage, the amplitude factors were regressed against magnitude to determine the magnitude dependence. The general form of the ground motion estimation equation used in the study is:

$$\ln(Y) = b_1 + b_2 (M-6) + b_3 (M-6)^2 + b_5 \ln r + b_V \ln (V_S / V_A) \quad (4.3)$$

where:

$$r = (r_{jb}^2 + h^2)^{1/2} \quad (4.4)$$

In this equation;

Y = peak ground motion measure

M= moment magnitude $M \geq 5.00$

r = closest distance from rupture to the station in km $r \geq 20\text{km}$.

r_{jb} = closest horizontal distance from the station to a point in km.

V_S = average shear-wave velocity (m/s) to a depth of 30 m

b_1 = parameter related to fault mechanism

b_{1SS} , b_{1RS} , b_{1ALL} , b_2 , b_3 , b_5 , b_V , V_A and h are regression coefficients provided in tabular form.

Figure 4.10 provides a comparison of PGA data in 1999 Kocaeli earthquake ($M_w = 7.4$) with the Boore et al. (1997) attenuation relationship.

Figure 4.11 compares the PSRV of Duzce record obtained in 1999 Kocaeli earthquake ($M_w = 7.4$) with the Boore et al. (1997) attenuation relationship.

Campbell (1997) study develops empirical attenuation relationships for horizontal and vertical PGA, PGV, and SA using accelerograms generated by western USA and other worldwide earthquakes of moment magnitude greater than 5 and sites with distances to seismogenic rupture within 60 km.

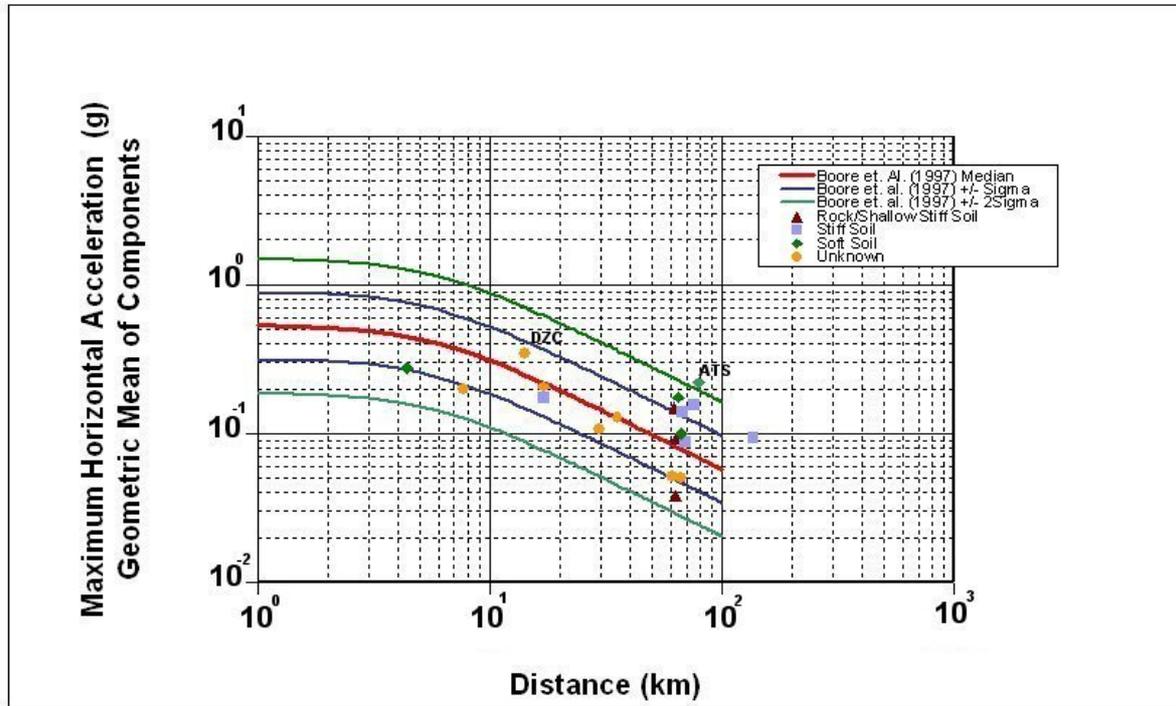


Figure 4.10 Comparison of 1999 Kocaeli earthquake (Mw = 7.4) PGA data with the Boore et al. (1997) attenuation relationship

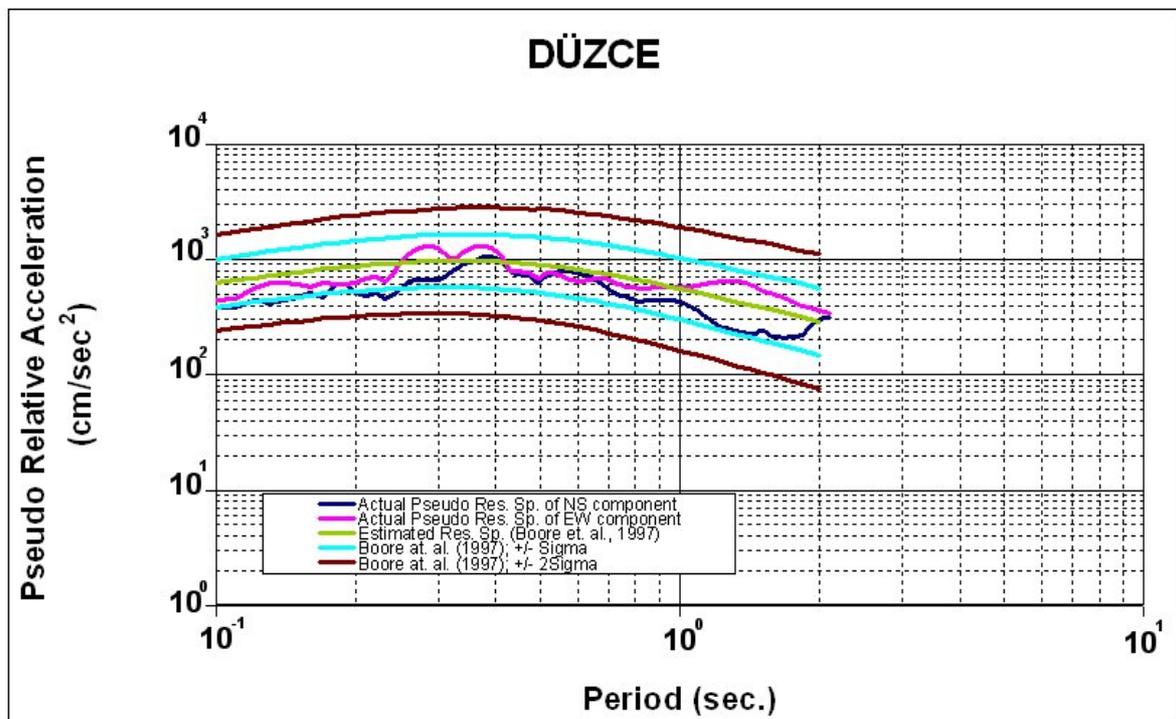


Figure 4.11 Comparison of the response spectra of the Duzce record (1999 Kocaeli earthquake, Mw=7.4) with the Boore et al. (1997) attenuation relationship

For the estimation of PGA values Campbell (1997) uses the following expression:

$$\begin{aligned} \ln(A_H) = & -3.512 + 0.904M - 1.328 \ln[R_{SEIS}^2 + (0.149e^{0.67M})^2]^{1/2} \\ & + [1.125 - 0.112 \ln(R_{SEIS}) - 0.0957M] F + [0.440 - 0.171 \ln(R_{SEIS})] S_{SR} \\ & + [0.405 - 0.222 \ln(R_{SEIS})] S_{HR} + \varepsilon \end{aligned} \quad (4.5)$$

where:

A_H = median of the geometric mean of the two horizontal PGA (g)

M = moment magnitude,

R_{SEIS} = the closest distance to seismogenic rupture on the fault (km),

$F = 0$ for strike-slip and normal faulting earthquakes and 1 for reverse, reverse-oblique, and thrust faulting earthquakes,

$S_{SR} = 1$ for soft-rock sites,

$S_{HR} = 1$ for hard-rock sites,

$S_{SR} = S_{HR} = 0$ for alluvial sites,

ε = random error term with mean of zero and a standard deviation equal to the standard error of estimate of $\ln(A_H)$.

Sadigh et. al (1997) present attenuation relationships for shallow crustal earthquakes based on strong motion data primarily from California earthquakes. Relationships are presented for the geometric mean of the two horizontal components, strike-slip and reverse-faulting earthquakes, rock and deep firm soil deposits, earthquakes of moment magnitude M between 4 and 8+ and distances up to 100km. The site conditions representative of rock attenuation models given here should be accepted as soft rock. The deep soil data are from sites with greater than 20m of soil over bedrock. Attenuation relationships of horizontal Response Spectral Acceleration (5% damping) are given in two separate equations according to the soil condition. Relationship for reverse/thrust faulting are obtained by multiplying the given strike-slip amplitudes by 1.2. The general form of the equation for rock sites is as follows:

$$\ln(y) = C_1 + C_2 M + C_3 (8.5 - M)^{2.5} + C_4 \ln[r_{rup} + \exp(C_5 + C_6 M)] + C_7 \ln(r_{rup} + 2) \quad (4.6)$$

y = PGA or SA (in g) represented by the geometric mean of the two horizontal components,

C_1 to C_7 = amplitudes given in tabular form

M = moment magnitude,

r_{rup} = Minimum distance to the fault rupture surface (km).

Ambraseys et al. (1996) attenuation relationship is based on 422 strong motion records from 157 earthquakes in Europe and adjacent areas. The equations use the larger horizontal acceleration response ordinate for 5 percent damping and give ground motion in terms of surface wave magnitude, distance and site conditions. Site conditions are represented by soil classes as rock, stiff soil and soft soil. The ground motion estimation equation used is of the form:

$$\log(Y) = C_1 + C_2 M + C_4 \log(r) + C_A S_A + C_S S_S \quad (4.7)$$

where $r = (d^2 + h_0^2)^{1/2}$

In this equation;

Y = peak horizontal accelerations in g.

M = surface wave magnitude $4 \leq M \leq 7.5$.

d = shortest distance to the surface projection of the fault in km.

h_0 = a constant determined with C_1 , C_2 , C_3 and C_4 .

$S_A = 1$ for stiff soils and 0 otherwise.

$S_S = 1$ for soft soils and 0 otherwise.

The period dependent coefficients C_1, C_2, C_4, C_A, C_S and h_0 and the error term σ are provided in tabular form.

Spudich et al. (1997) collected ground motions from extensional regimes throughout the world and derived attenuation relationship for PGA and SA in extensional tectonic regimes using globally obtained data. In general, their values suggest that most other attenuation models will significantly overestimate ground motions from normal faulting earthquakes.

Four different groups of earthquake data were processed including earthquakes occurred along extensional offsets in the Anatolian system. The data set was comprised of the earthquakes having moment magnitude $M > 5.0$ recorded at distances less than 105 km. Depending on the Joyner and Boore (1981) site classification the recording site was separated into two geologic categories, rock and soil.

The coefficient in the resulting equation for predicting ground motion were determined using a weighted, two stage regression procedure (Joyner and Boore, 1993). However, in derivation process, the equation was observed invalid when extrapolated to magnitudes 7.0 and larger. The main problem was that the magnitude range of data set used was not wide enough to determine the coefficients of magnitude dependent terms accurately. Thus, Boore et al. (1993, 1994) data set was utilized determining the magnitude dependence.

At the result of the regression analysis the general expression was obtained as;

$$\log_{10}Y = b_1 + b_2(M-6) + b_3(M-6)^2 + b_4 + b_5 \log_{10}R + b_6 \Gamma \quad (4.8)$$

where,

Y = Peak horizontal acceleration (g.) or pseudo velocity response (cm/s.) at 5 percent damping for the geometric mean horizontal component of motion.

$$R = (r_{jb}^2 + h^2)^{1/2}$$

$$\Gamma = 0 \quad \text{for rock}$$

$$\Gamma = 1 \quad \text{for soil}$$

M = Magnitude.

r_{jb} = Joyner-Boore distance.

h = Fictitious depth that is determined by the regression.

$b_1, b_2, b_3, b_4, b_5, b_6$ = Smoothed coefficients for regression relation, for geometric mean horizontal PGA and five percent damped Peak spectral velocity.

The standard deviation of $\log_{10}Y$ is given by:

$$\sigma_{\log Y} = (\sigma_1^2 + \sigma_2^2)^{1/2} \quad (4.9)$$

Values of $h, b_1, b_2, b_3, b_4, b_5, b_6, \sigma_1$ and σ_2 for PGA are given in tabular form. The resultant SEA96 relation may be used in the 5.0-7.7 magnitude range and 0-70 km. distance range for extensional regime earthquakes.

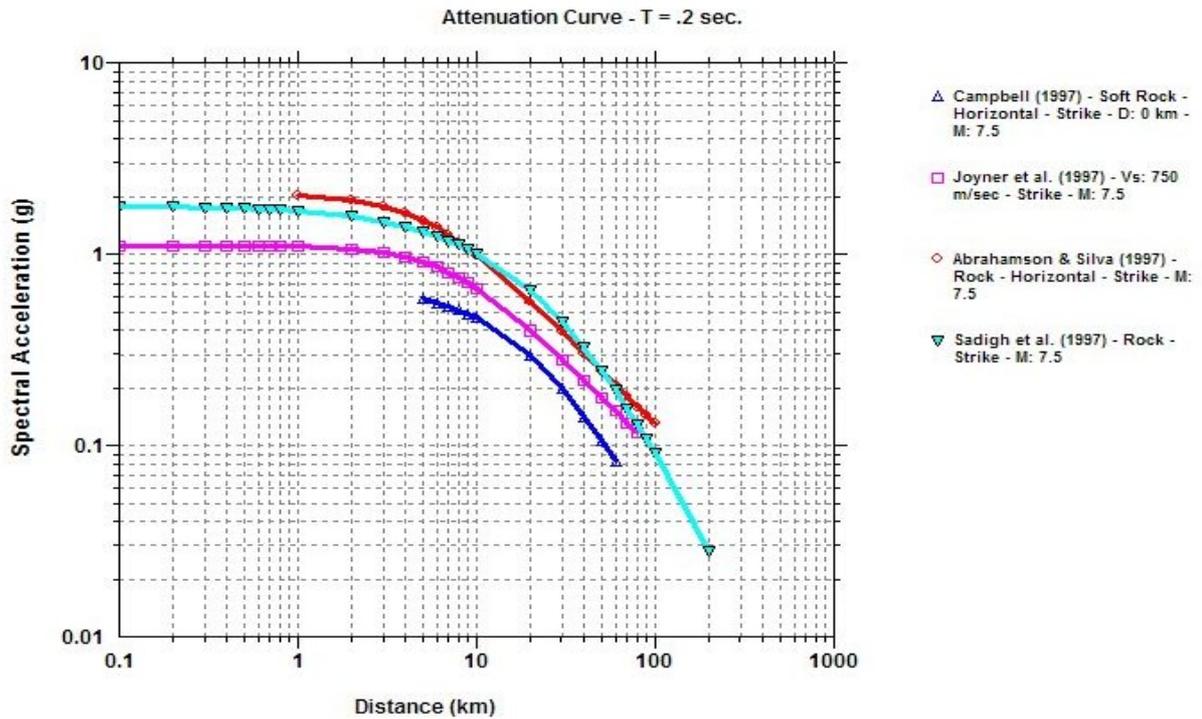


Figure 4.12 Comparison of SA (0.2 s) attenuation relationships for competent soil sites for Mw=7.5 earthquake

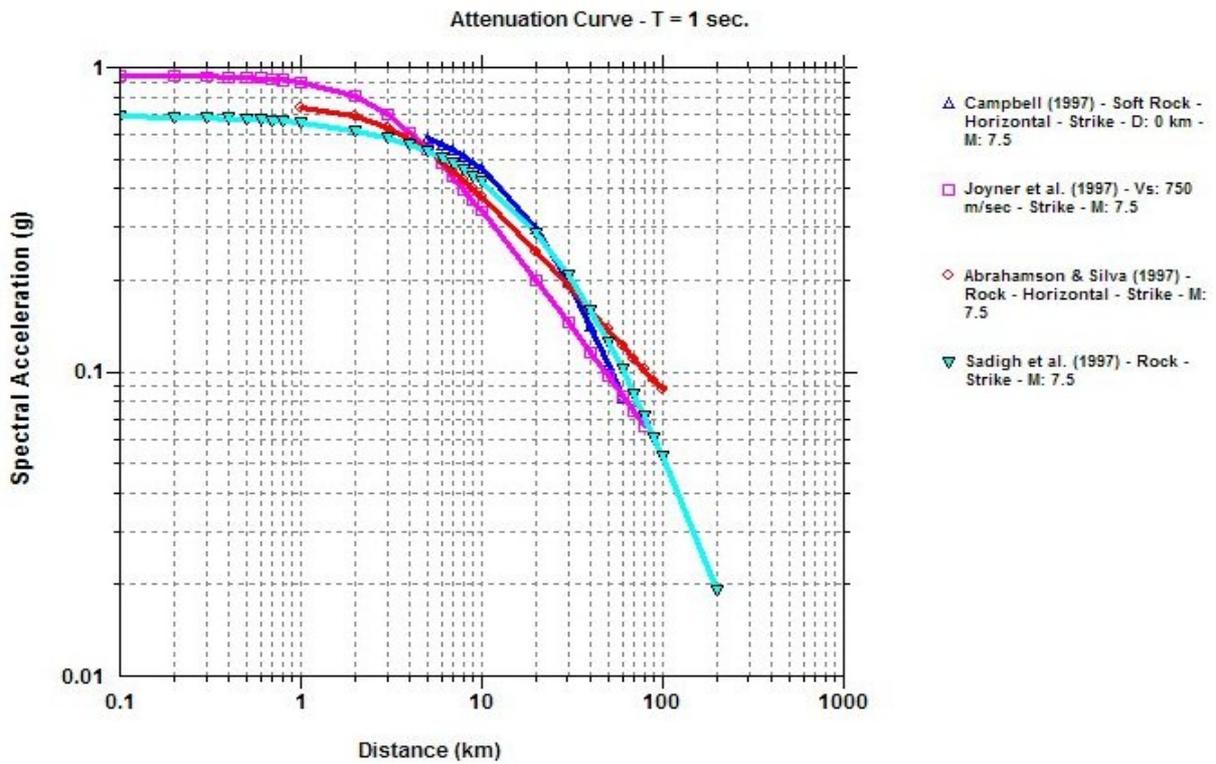


Figure 4.13 Comparison of SA (1.0 s) attenuation relationships for competent soil sites for Mw=7.5 earthquake

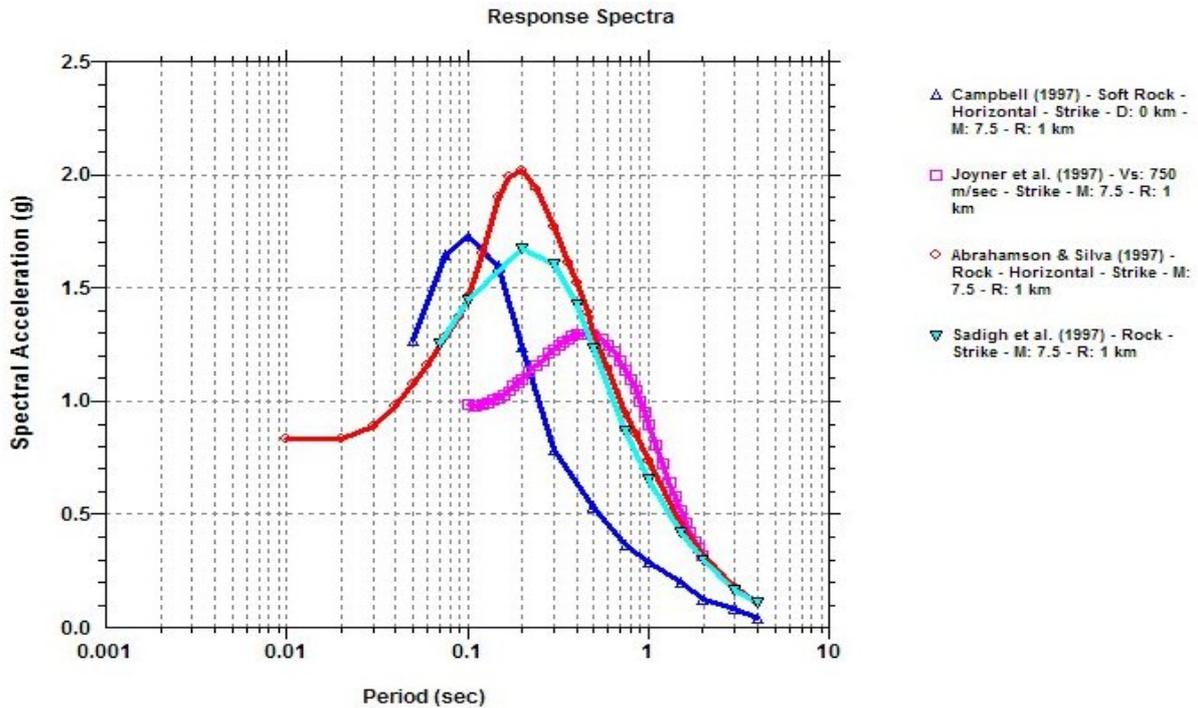


Figure 4.14 Comparison of response spectra on competent soil sites from different attenuation relationships for competent soil sites for Mw=7.5 at near fault conditions

Figure 4.12 and Figure 4.13 compare several SA (0.2s) and SA (1s) attenuation relationships for competent soil sites for an Mw = 7.5 earthquake. The variation between different spectral attenuation relationships for an Mw = 7.5 earthquake at near fault conditions is indicated in Figure 4.14.

Ozbey (2001) recently has developed an attenuation relationship for PGA applicable to Northwestern Turkey using a data set consisting of Kocaeli and Duzce earthquake main- and aftershock records. Figure 4.15 provides a comparison of this relationship with others.

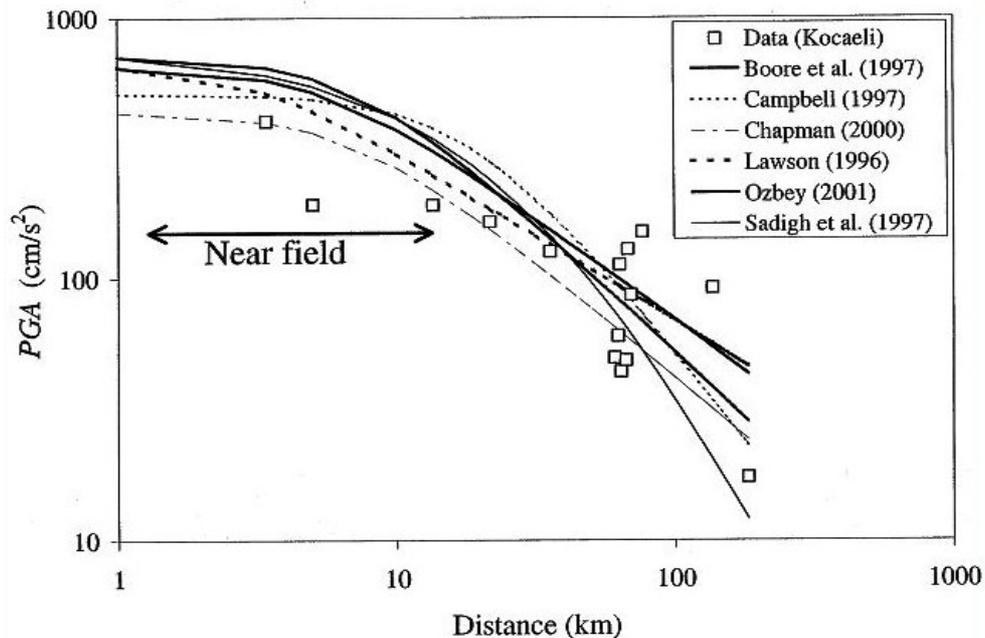


Figure 4.15 Comparison of Özbey (2001) attenuation relationship with other attenuation relationships

Among the attenuation relationships described above the usage of the average of the results obtained from Boore, Joyner and Fumal (1997), Sadigh et al. (1997) and Campbell (1997) for the computation of Peak Ground Acceleration and the average of Boore, Joyner and Fumal (1997) and Sadigh et. al. (1997) for Spectral Accelerations are suggested for applications in Turkey. However conditions necessitate the use of a single attenuation relationship, Boore, Joyner and Fumal (1997) is the most suitable attenuation relationship to be used for Turkey. For Peak Ground Acceleration (PGA) the regional attenuation relationship developed by Ozbey (2001) can also be considered.

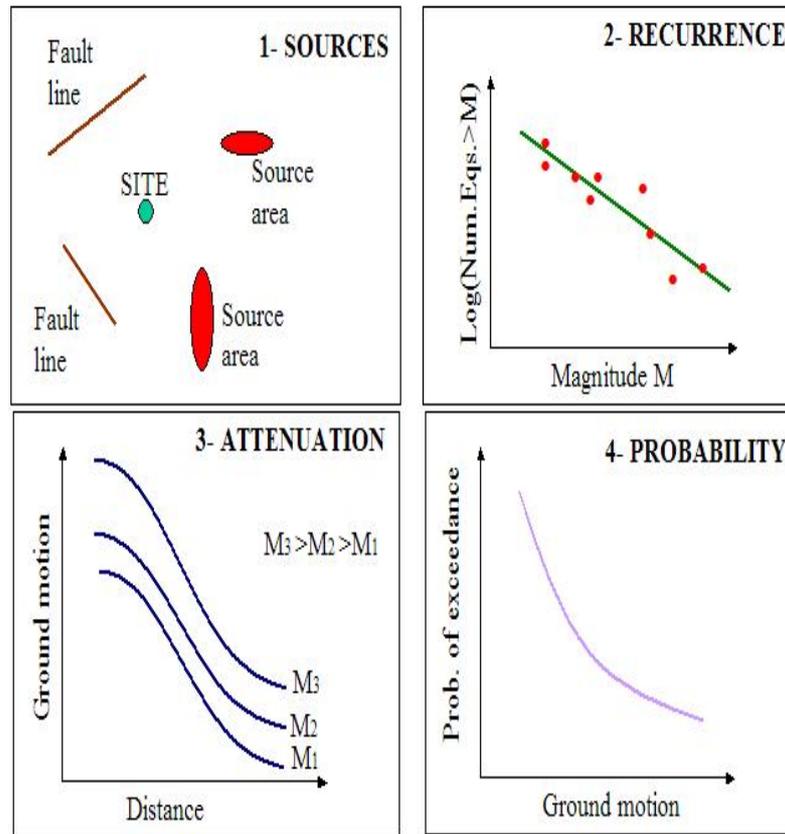
4.2. Probabilistic Earthquake Hazard Analysis (PSHA)

PSHA provides a site-specific assessment of the likelihood that ground shaking will occur during a specified time period based on seismic source modeling, distribution of earthquake occurrences and attenuation relationships. The foundation of the Probabilistic Seismic Hazard Analysis (PSHA) follows the classical study of Cornell (1968). Current practice is described in several publications, such as National Research Council (1988) and Earthquake Engineering Research Institute Committee on Seismic Risk (1989). Today there exist several methodologies that can account for ground motion variability, source boundary variability and planar (3-dimensional) source geometry. The randomness in frequency of occurrence of earthquakes, rupture geometry and the location of earthquakes for each source and in attenuation characteristics can also be considered.

The evolution of seismic hazard assessment can be traced in five generations of methodology (Muir-Wood, 1993). These are: Historical determinism; Historical probabilism; Seismotectonic probabilism; Non-Poissonian probabilism; and Earthquake prediction. The seismotectonic probabilism method of hazard assessment does not rely solely on historical seismicity records but combines it with geological knowledge that is the data of paleoseismic ground motions and data of neotectonic faulting and with the scientific seismotectonic understanding of earthquake causes. These data are combined through a seismic source model, but the uncertainty in the determination of the input parameters are incorporated in the form of a weighted range of values.

4.2.1 General Methodology

The general methodology in calculating seismic hazard is well established in literature (e.g. Cornell 1968). The method involves two separate models: a seismicity model describing a geographical distribution event sources and the distribution of magnitudes, and an attenuation model describing the effect at any site given as a function of magnitude and source-to-site-distance. The seismicity model may comprise a number of source regions, the seismicity of which should be expressed in terms of a recurrence relationship of events with magnitudes greater or equal to a certain value. The attenuation model relates the earthquake intensity (i.e. the effect of it, as a general term) at a site to magnitude, distance, source parameters and site conditions. The basic elements of PSHA (Source, Recurrence, Attenuation and Probability) are illustrated in Figure 4.16.



Steps of probabilistic seismic hazard analysis (1) definition of earthquake sources, (2) earthquake recurrence characteristics for each source, (3) attenuation of ground motions with magnitude and distance, and (4) ground motions for specified probability of exceedance levels (calculated by summing probabilities over all the sources, magnitudes, and distances).

Figure 4.16 Basic elements of probabilistic seismic hazard analysis

The PSHA computes the annual rate of events at which the specified levels ground motion at the site will be exceeded. The total annual rate is the sum of the annual rate from each source. The annual rate of earthquakes that produces ground motion amplitude at the site higher than the level (z), v_z , can be given by the following summation:

$$v_z = \sum_N \{ [\sum_M \lambda(m_i)] [\sum_R P(R=r_j|m_i)] [P(Z>z|m_i,r_j)] \} \quad (4.10)$$

where \sum_N denotes summation over all of the (N) seismic source zones.

For a given source $\lambda(m_i)$ is the annual rate of occurrence of earthquakes of magnitude m_i .

$P(R=r_j|m_i)$ is equal to the probability of an earthquake of magnitude $M=m_i$ occurring at a distance $R=r_j$.

$P(Z>z|m_i,r_j)$ is the probability that the ground motion parameter level (z) will be exceeded for given $M=m_i$ and $R=r_j$ combination.

In the simplest application of total probability theorem (McGuire, 1993) these summations are replaced by double integration over dr and dm and the probabilities are replaced by probability density function of magnitude ($f(m)$) and probability density function of distance given magnitude. For the most general case of planar sources this computation involves a six-fold integration that includes probability density functions for the magnitude, site distance, rupture width, rupture area, location of the center of rupture along the strike and the dip for each source, and the ground motion variability for the attenuation relationship. For the integration bounds the minimum and maximum magnitudes, minimum and maximum number

of standard deviations and geometric limits of each source are used. To convert the annual rate of events to a probability of exceedance of a specified ground motion level a certain stochastic occurrence model needs to be followed. The basic elements of a PSHA are illustrated in Figure 4.17 for the development of PGA based response spectra.

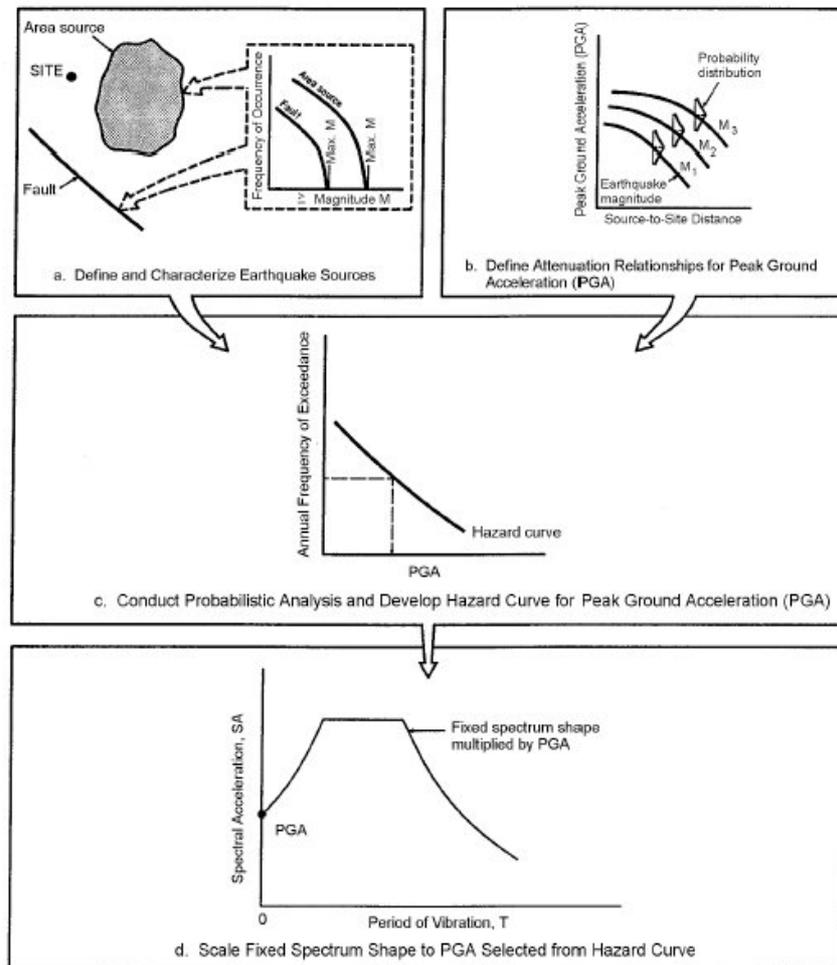


Figure 4.17 Development of PSHA for PGA-based spectrum (after EERI Committee on Seismic Risk)

4.2.2 Seismicity (Earthquake Catalogues)

The most important and fundamental input to a model of earthquake hazard is a collection of earthquake events that represent the temporal and spatial distribution of seismic activity in the region. In seismicity compilations attention is paid to the cross-correlation of instrumental and historical earthquake data. Uniformity in magnitude is implemented by converting all magnitudes to moment magnitude. In the traditional probabilistic seismic hazard assessment (Cornell, 1968) independent events must be treated. To satisfy this requirement, earthquakes in the study region needs to be de-clustered by removing foreshocks and aftershocks from the seismicity databases in order to obtain a Poissonian distribution. The seismicity data from different catalogues comes in different magnitude scales. For numerous earthquakes there exists more than one value, sometimes in different magnitude scales, taken from different sources. The moment magnitude M_w can be used as the general magnitude unit because, as suggested by various researchers, the use of moment magnitude avoids the "saturation" of the more traditional band-limited magnitude measures at large seismic moments and, therefore is a better measure of the true size of an earthquake.

4.2.3 Seismic Source Zonation

A seismic source zone is defined as a seismically homogenous area, in which every point within the source zone is assumed to have the same probability of being the epicenter of a future earthquake. An ideal delineation of seismic source zones requires a complete comprehension of the geology, tectonics, paleoseismology, historical and instrumental seismicity, and other neotectonic features of the region under study. The determination of the seismo-tectonic sources exhibiting coherent tectonic regimes and homogenous earthquake distribution is the first step in the probabilistic assessment of the seismic hazard. Seismic sources can be identified by using the macro-seismic locations of historic earthquakes and instrumental locations of the last 50 years' earthquakes. Delineation of the source boundaries is generally based on neo-tectonic elements and sudden variations in the homogeneity of the seismicity. In addition, background seismic zones may need to be defined to account for floating earthquakes not accounted by these sources and also to delineate zones where no significant earthquake has taken place for centuries.

Although seismic source zonation is a widely used methodology to determine earthquake hazard, it is not the only approach. Since delineation of the seismic source zones still remains rather subjective, at present researchers (e.g. Frankel, 1995) are suggesting other methods for evaluating seismic hazard, in order to eliminate the subjectivity of this procedure. This is particularly important in areas where the tectonic structure is very fragmented and the seismicity is diffuse.

4.2.4 Range of Magnitudes

Minimum magnitude delineates the minimum earthquake size that can have the capability to produce damage in built environment. It generally varies between 4.5 and 5.0. The expected maximum size of the earthquake that can occur on a seismic source is an important parameter, especially for high return period levels. Following methods are used in determining the maximum magnitude expected.

For very active faults with long seismic history and short recurrence intervals between large magnitudes, maximum historical earthquake procedure may provide reasonable assessments. In the standard practice, the maximum historical magnitude (recorded or inferred) is increased by half a magnitude unit to yield the maximum magnitude.

Using empirical relationships a magnitude can be estimated for a given fault rupture length, offset or rupture area. The estimated magnitude has usually a normal distribution and the relationship provides the mean magnitude with a standard deviation. The method involves the use of regression analysis of the surface rupture characteristics with the magnitudes. The worldwide data suggest that the maximum rupture amounts to about 1/3 to 1/2 of the fault rupture length. Widely used relationships are provided in Wells & Coppersmith (1994) as regressions among magnitude, rupture length, rupture width, rupture area and surface displacement (indicated by X in the following expression):

$$\log X = a + b M_w \quad (4.11)$$

where a and b are regression constants.

A relationship between the fault rupture (L, km) and magnitude (M_S) has been developed for Turkey by Erdik et al., (1985):

$$\ln L = 2.53 M_S - 14.04 \quad \text{for } M_S \geq 6.8 \quad (4.12)$$

with $\sigma_{\ln L} = 0.27$, $r^2 = 0.915$, where σ is the standard deviation and r^2 is the correlation coefficient. The relationship is based on data from earthquakes originating from the North Anatolian Fault.

The comparison of surface fault ruptures that occurred in the 20th century along the North Anatolian Fault, including the recent Kocaeli and Düzce events with the Wells & Coppersmith (1994) relationship is provided in Figure 4.18 and with the Erdik et al (1985) relationship in Figure 4.19.

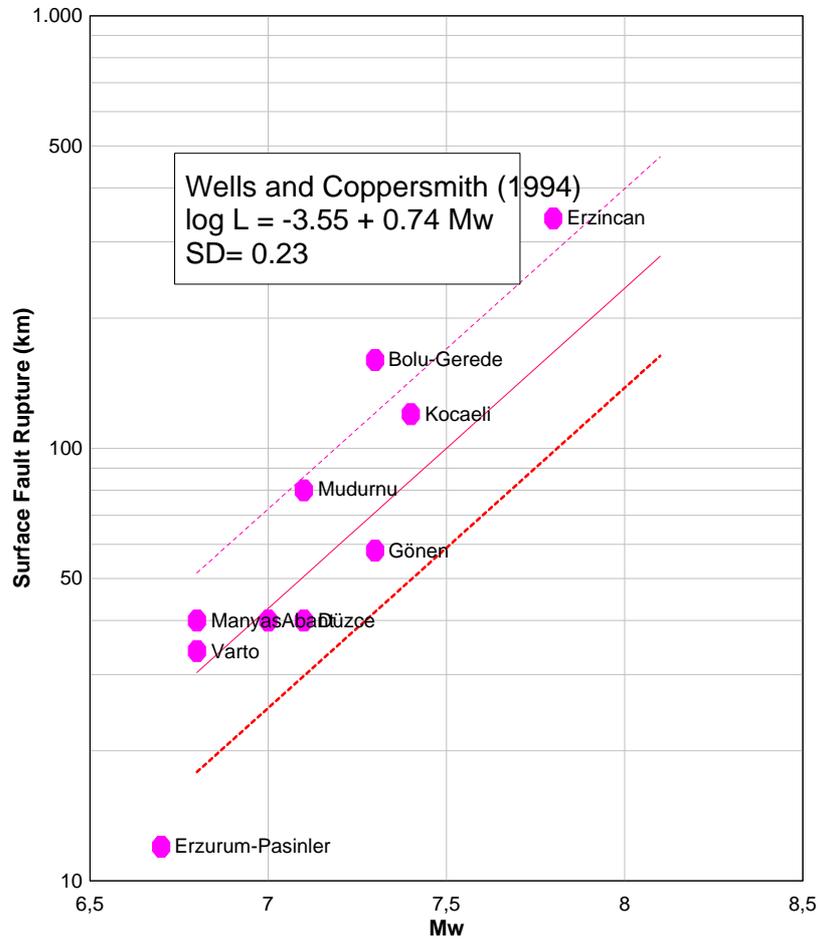


Figure 4.18 The comparison of surface fault ruptures that occurred in the 20th century along the North Anatolian Fault, including the recent Kocaeli and Düzce events with the Wells & Coppersmith (1994) relationship

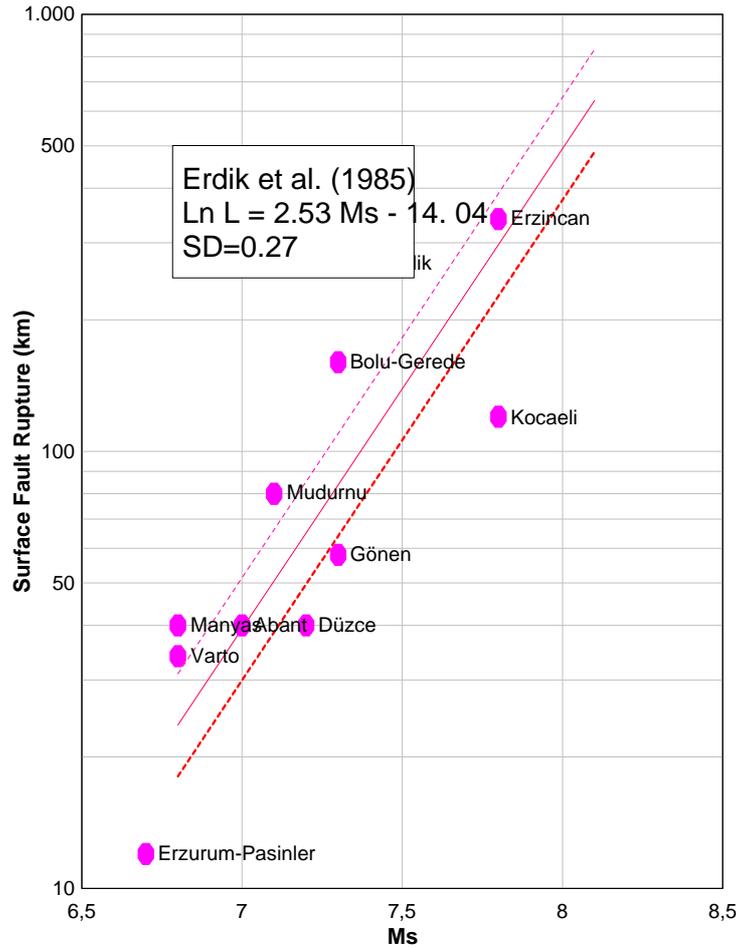


Figure 4.19 The comparison of surface fault ruptures that occurred in the 20th century along the North Anatolian Fault, including the recent Kocaeli and Düzce events with the Erdik (1985) relationship

4.2.5 Recurrence Relationships

The rate of occurrence of earthquakes is obtained based on earthquake recurrence assessments. Recurrence curves express the rate of occurrence of earthquakes equal to or greater than a certain magnitude.

For the assessment of the recurrence relationships in the source zones the rates of occurrence in different magnitude groups are adjusted by determination of the period over which the data in a given magnitude group are completely reported. The empirical recurrence relationship for earthquakes (Gutenberg and Richter, 1944) has been extensively used in many seismicity studies and has also been confirmed to hold for micro-earthquakes. The mean annual rate of occurrence of earthquakes with magnitudes greater than $m=M$ is given as follows:

$$\log N(M) = a - bM \tag{4.13}$$

or

$$\ln N(M) = \alpha - \beta M \tag{4.14}$$

where $\alpha=2.303a$ and $\beta=2.303b$

Parameters “a” and “b” are the regression constants. Parameter “a” is the activity rate and 10^a conveys the annual number of occurrence for earthquakes with magnitude above 0 in the source zone considered. Parameter “b” is related to the relative probability of occurrence

between small and large earthquakes. Generally values of “b” varies between $b=0.7-1.2$. There are no "typical" values of “a”. It is also possible to define a density function $f(M)=-dN(M)/dM$. With this density function, the incremental number of earthquakes in a magnitude interval of width ΔM is $f(M) \Delta M$.

The magnitude probability density function $f(m)$ for given lower and upper magnitudes m_0 and m_u are given as follows:

$$f(m) = \beta \exp[-\beta(m-m_0)] / [1-\exp[-\beta(m_u-m_0)]] \quad (4.15)$$

This is usually called the “truncated exponential Gutenberg-Richter model”. Figure 4.20 illustrates the frequency of earthquake occurrence and the truncated exponential probability density function for magnitude. To compute the mean annual rate of occurrence of earthquakes with magnitudes greater than $m=M$ one needs to integrate the magnitude probability density function $f(m)$ from right to left between the limits of $m= m_u$ and $m=M$.

$$N(M) = N(m_u) \int_{m_u}^M f(m) dm \quad (4.16)$$

The earthquake catalogues are often biased due to incomplete reporting for smaller magnitude earthquakes in earlier periods. Thus to fit the recurrence relationship to a region, one should choose among using (1) a short sample that is complete in small events or (2) a longer sample that is complete in larger events or (3) a combination of the two data sets to complete the deficient data thereby obtaining a homogeneous data set. A direct attempt to fit these data to a regression relationship may result in quadratic or higher order expressions to accommodate the inherent bias and inhomogeneity of the data. To overcome these difficulties an artificially homogeneous data set can be simulated through the determination of the period over which the data in a given magnitude group are completely reported (Stepp, 1973).

The activity rate can be obtained by fitting the truncated exponential Gutenberg-Richter model to the earthquake catalogue of the source zone. Maximum likelihood or least-square approaches are the generally used procedures for the assessment of the parameters “a” and “b”. The least square method places relatively greater weight on the rare, largest magnitude events. The maximum likelihood approach places more weight on the smaller events.

The coefficient "a" in the Gutenberg - Richter relation is directly related to the slip rate on the fault, or to the regional rate of deformation. The slip rate can be converted to the rate of earthquake occurrences by requiring that the seismic moment rates computed from the rate of slip and the by cumulative seismic moment of the earthquake occurrences. The Seismic Moment is defined by:

$$M_o = \mu A_R D \quad (4.17)$$

Where μ is the Lamé's constant (shear modulus) of the medium, A_R is the fault rupture area and D is the average fault offset. Seismic moment rate (annual rate of seismic moment built-up) on a fault becomes:

$$dM_o / dt = \mu A s \quad (4.18)$$

Where A is the area of the total fault plane and s is the annual rate of slip.

M_o and the moment magnitude M_w are related by the relationship given by Hanks and Kanamori (1979)

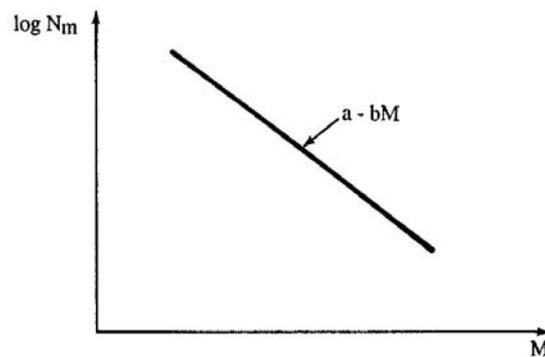
$$\log M_o = 1.5 M_w + 16.1 \quad \text{or} \quad M_o = 10^{(1.5 M_w + 16.1)} \quad (4.19)$$

The total annual seismic moment released by earthquake occurrences can be given by:

$$\frac{dM_o}{dt} = \mu \cdot A \cdot s = N(m_u) \int_{m_u}^M f(m) \cdot M_o \cdot dm \tag{4.20}$$

From this expression the annual activity rate $N(m_u)$ can be computed.

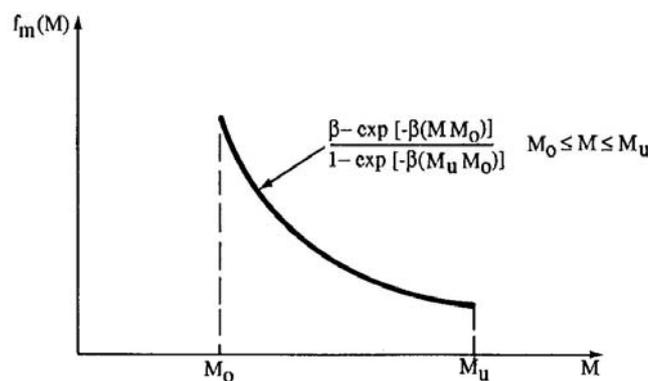
Evaluation of the contributions to the integral demonstrates that most of the seismic moment is released in magnitudes at or close to maximum magnitude earthquakes. Thus, small earthquakes do not release a significant fraction of the total moment that accumulates on a fault and they cannot act as a safety valve.



N_m = Number of Earthquakes with Magnitude $\geq M$ in Given Region and Time

a, b = Regional Seismicity Parameters

(a) Frequency of Earthquake Occurrence



$f_m(M)$ = Probability Density Function of Magnitude M

$\beta = 2.3b$ = Seismicity Parameter

M_0, M_u = Lower Bound and Upper Bound Magnitudes for each Source

(b) Probability Density Function of Magnitude

Figure 4.20 Frequency of earthquake occurrence and probability density function for magnitude (after EERI Committee on Seismic Risk)

4.2.6 Characteristic Earthquake

For a single fault, earthquakes may not obey the Gutenberg-Richter type of distribution. The recurrence relationships for several faults show a distinct lack of earthquakes about one magnitude unit smaller than the largest events observed on those faults. Studies done by Wesnousky et al. (1983) suggest that the b-value model is better as a statistical description of seismicity in a region consisting of several faults than it is for describing a single fault. The Gutenberg-Richter exponential model has been found to work well for non-fault generalized tectonic structures with distributed seismicity. This implies that the b-value relationship for a region is caused by a distribution of fault sizes.

Recent studies indicate that earthquake recurrence on individual faults may tend to rupture in what have been termed “characteristic” magnitude events at or near the maximum magnitude (Schwartz and Coppersmith 1984). Fault segments produce characteristic earthquakes within a magnitude range given by $m_u - \Delta m_c$ that has a higher likelihood of occurrence than other magnitude events. In other words in the characteristic model relatively more seismic energy is released by characteristic earthquakes compared to the truncated exponential model.

For recurrence modeling in such fault sources the maximum moment model (Wesnousky et al. 1983) and the characteristic magnitude recurrence model (Youngs and Coppersmith 1985a, 1985b) have been developed. In the characteristic earthquake probability density function model given by Youngs et al., (1985), $f(m)$ is exponentially distributed between magnitudes m_0 and $m_u - \Delta m_c$ and uniformly distributed (boxcar shape) in the magnitude range $m_u - \Delta m_c$ and m_u . The height of the boxcar (uniform distribution level) is generally taken equal to the value of the exponentially distributed region at $m_u - \Delta m_c - \Delta m_1$.

A comparison of Youngs and Coppersmith model with other magnitude probability density functions are given in Figure 4.21.

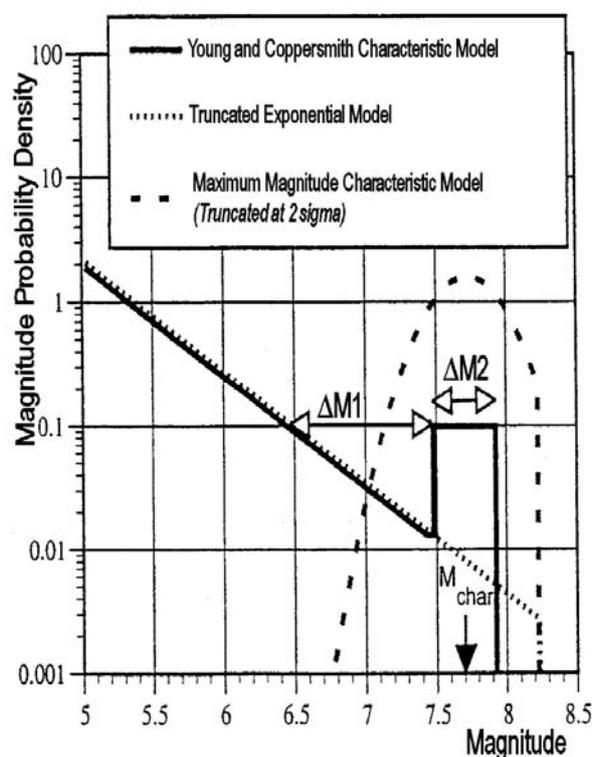


Figure 4.21 Comparison of Youngs and Coppersmith characteristic magnitude probability density function with other models (after Abrahamson, 2000)

The usual values of Δm_c and Δm_1 in the Youngs et al., (1985) model are respectively 0.5 and 1.0. This selection would associate 94% of the seismic moment rate with the occurrence of the characteristic earthquake.

4.2.7 Homogeneous Poisson Model

For forecasting seismic occurrences numerous models have been developed. The simplest stochastic model for earthquake occurrences is the Homogeneous Poisson Model (Cornell 1968). For the earthquake events to follow that model the requirements of spatial and temporal independence are in order.

Using Poisson stochastic process the occurrence of earthquakes in a given seismic source is given by the expression

$$P_A(n|m,t) = [\exp(-\lambda_A(m)t) (\lambda_A(m)t)^n] / n! \quad (4.21)$$

Where $P_A(n|m,t)$ is the probability of having n occurrences of an earthquake of magnitude $M=m$ or larger in the time period t . The mean rate of occurrence is given by $\lambda_A(m)$. The expected number of occurrences of earthquakes of magnitude $M=m$ or larger in a time period t is given by:

$$E_A(n|m,t) = \lambda_A(m)t \quad (4.22)$$

Probability of having at least one event of magnitude $M=m$ or larger in time period t is given by:

$$P(n>0 | m,t) = 1.0 - \exp[-\lambda_A(m)t] \quad (4.23)$$

The average recurrence interval, $RI_A(m)$, of an event is the reciprocal of the mean rate of occurrence:

$$RI_A(m) = 1 / \lambda_A(m) \quad (4.24)$$

Using Simple Poisson Process the Probability that at least one occurrence of ground motion level above the level z in t years [$P_E(A>z, t) = P_E(z)$] is given by:

$$P_E(A>z, t) = 1 - \exp[-v(A>z)t] \quad (4.25)$$

or

$$P_E(z) = 1 - \exp[-v_z t] \quad (4.26)$$

Where $v_z = v(A>z)$ is the annual rate (frequency) of events with ground motions exceeding the level z at a given site. The average return period (RP) of the ground motion level is defined as the reciprocal of the v_z .

$$RP = 1 / v_z \quad (4.27)$$

In practice PSHA results are conveyed by one or more of the parameters $P_E(z)$, RP and v_z . The relationship between these parameters is provided in Figure 4.22.

4.2.8 Conditional Probability (Renewal) Model

A major fault is characterized as a series of segments that can rupture either individually or in series with several contiguous cascading possibilities. A fault segment is characterized by slip rate and a probability density function of earthquake occurrences.

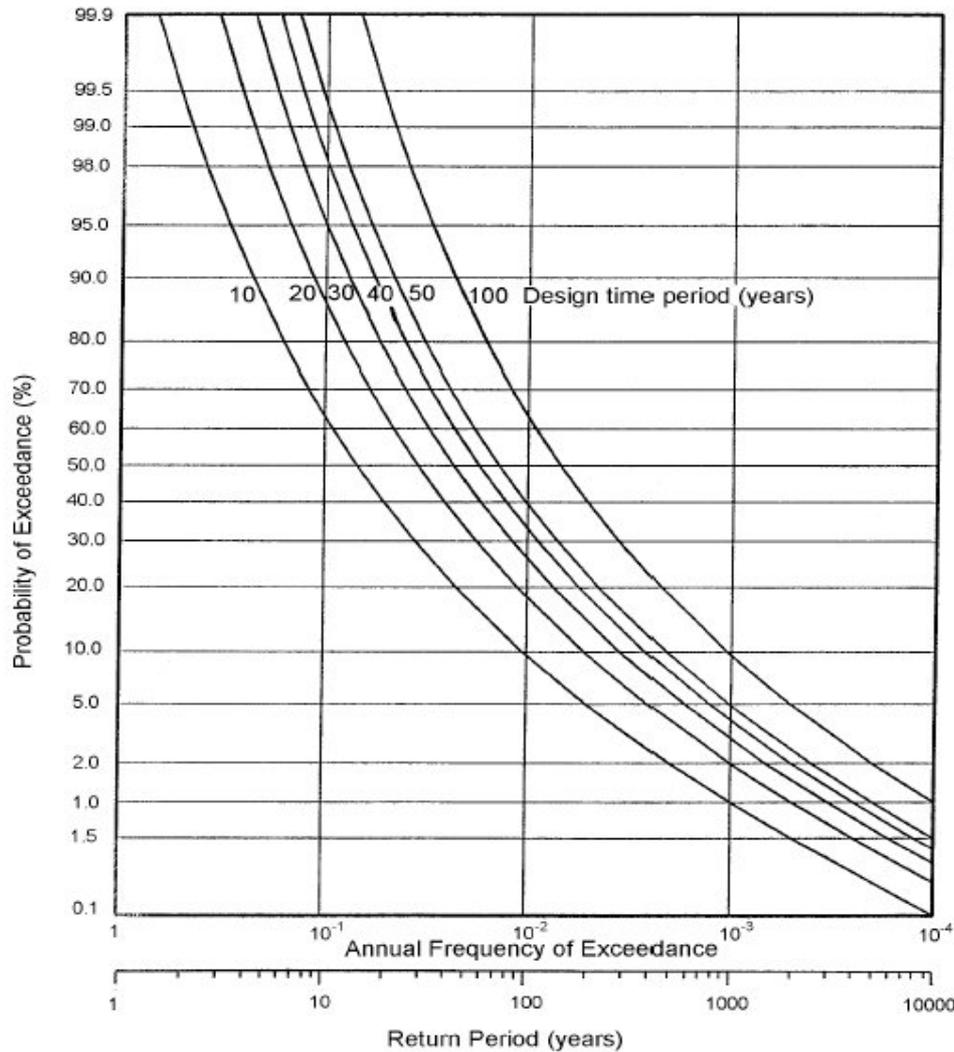


Figure 4.22 Relationship between various parameters for homogeneous Poisson Process

Probability of the rupture of the segment (i.e. characteristic earthquake) in the time interval t_e , $t_e + \Delta t$ is given by (WGCEP, 1990) the following expression:

$$P(t_e \leq T \leq t_e + \Delta t) = \int_{t_e}^{t_e + \Delta t} f_T(t) \cdot dt \tag{4.28}$$

Where t_e is the elapsed time since the last segment rupturing earthquake and $f_T(t)$ is the lognormal probability density function of the time of occurrence (T) of the segment rupturing (characteristic) earthquake.

$$f_T(t) = \frac{1}{t \cdot s \cdot \sqrt{2\pi}} \cdot e^{-\left[\frac{\ln(t/m)^2}{2s^2} \right]} \tag{4.29}$$

Where, T_m is the best median value of the recurrence interval T of the characteristic earthquake and s is the standard deviation of the natural logarithm of the time of occurrence (ln T). If S is not specifically known, it is generally assumed to be equal to one-third of ln (m). The median value of the recurrence interval, T_m , can be determined from the ratio of the displacement (D) of the previous segment rupturing earthquakes and the best median estimate of the regional slip rate (V).

$$T_m = D / V \quad (4.30)$$

The probability of occurrence of a segment-rupturing (characteristic) earthquake in the time interval t_e , $t_e + \Delta t$, given that the event did not happen prior to t_e , is given by the following expression:

$$P(t_e \leq T \leq t_e + \Delta t | T > t_e) = \frac{P(t_e \leq T \leq t_e + \Delta t)}{P(t_e \leq T \leq \infty)} \quad (4.31)$$

The nominator of this expression is equal to the hatched area, and the denominator is equal to the total blue colored area under the probability density function schematically drawn in Figure 4.23.

For modeling the probability density function for the characteristic earthquake recurrence intervals log-normal, Weibull and Gamma distributions have been used. The log-normal distribution, parameterized by the mean and the coefficient of variation, is commonly used. The coefficient of variation is a measure of the periodicity of the characteristic earthquake occurrences. The typical values range between 0.4 and 0.6. Small coefficient of variations indicates higher periodicity in the occurrence of characteristic earthquakes (Figure 4.24).

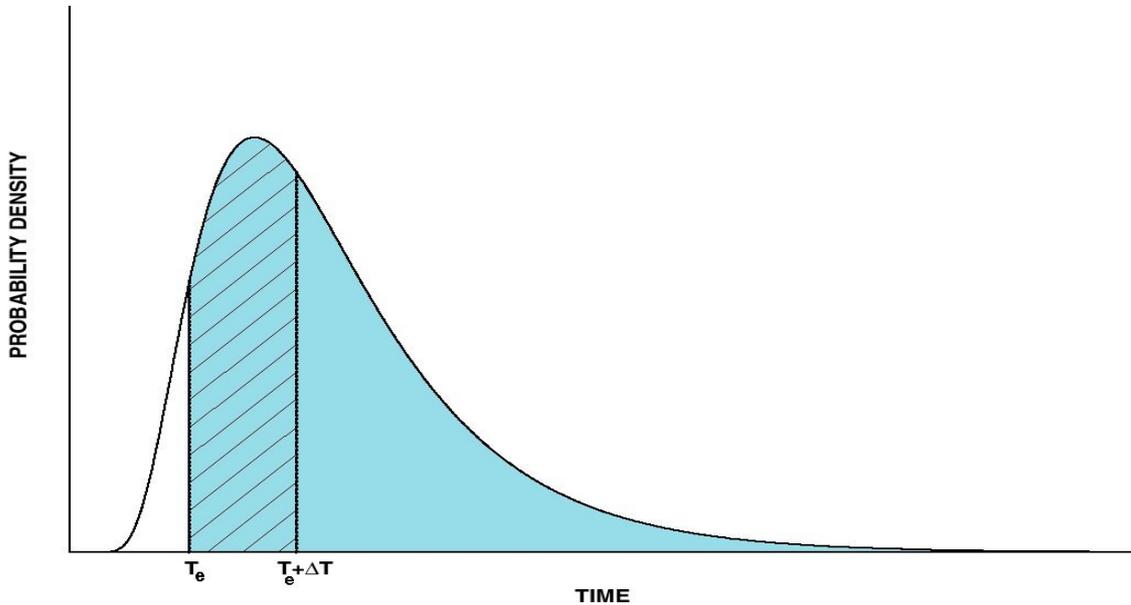


Figure 4.23 Diagram illustrating the calculation of conditional probability from a probability density function. The time interval of interest (the exposure time) is striped. The survivor function is equal to the shaded area. The conditional probability is the ratio of the two areas

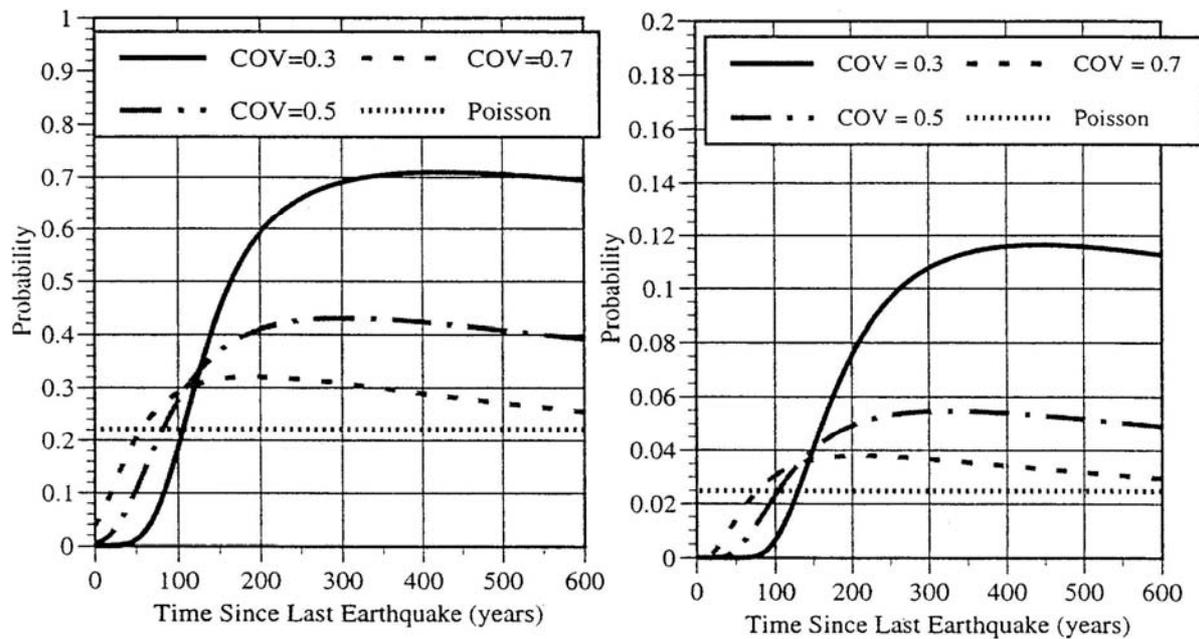


Figure 4.24 Sensitivity of the time dependent probabilities for a renewal model with 50 and 5 year exposure periods (after Abrahamson, 2000)

4.2.9 Treatment of Uncertainties

Aleatoric (randomness) uncertainty arises from the state of nature of the problem. Epistemic uncertainty arises due to limited knowledge about the problem. It can be controlled with increased data and better analytical tools. Uncertainty in hazard assessments is usually considered through the use of logic trees of alternate (but all credible) source zonation schemes, probability density functions, rates of occurrences and attenuation relationships. Each path through the tree to an end branch defines a set of parameters that are used to conduct a basic seismic hazard analysis for that path and end branch. Basic hazard analyses are carried out for each path. Each path also has an associated probability or weight that is determined by the product of the relative probabilities or weights assigned to the various models and parameters along the path. Usually, the mean estimate from a logic tree analysis is used as a basis for selecting project design criteria. A generic logic tree used in PSHA is provided in Figure 4.25.

4.2.10 Equal-hazard spectra

Equal hazard spectrum is developed by computing the probabilistic earthquake hazard in terms of spectral amplitudes at a suite of frequencies (or periods) (Figure 4.26). The computation of equal-hazard spectra directly accounts for the changes in contributions to the hazard from different magnitudes and source-to-site distances. Spectral amplitudes at different frequencies have the same probability of exceedance. The equal-hazard spectrum is the result of many possible earthquakes of different sizes and locations. As such, the equal hazard spectrum cannot be associated with a scenario earthquake and cannot be physically realized. Therefore, it is important to de-aggregate the results of a PSHA so that the primary contributors (earthquake sources, magnitude ranges, distance ranges) are known.

4.2.11 Hazard Deaggregation

Hazard de-aggregation is the process of breaking the hazard (quantified as spectral amplitudes) at a given site into contributions from magnitude-distance pairs at different frequencies. It facilitates the identification of the scenario earthquake, of the earthquake that most contributes to the hazard at the site at a given spectral frequency. Figure 4.27 illustrates the result of a typical deaggregation analysis.

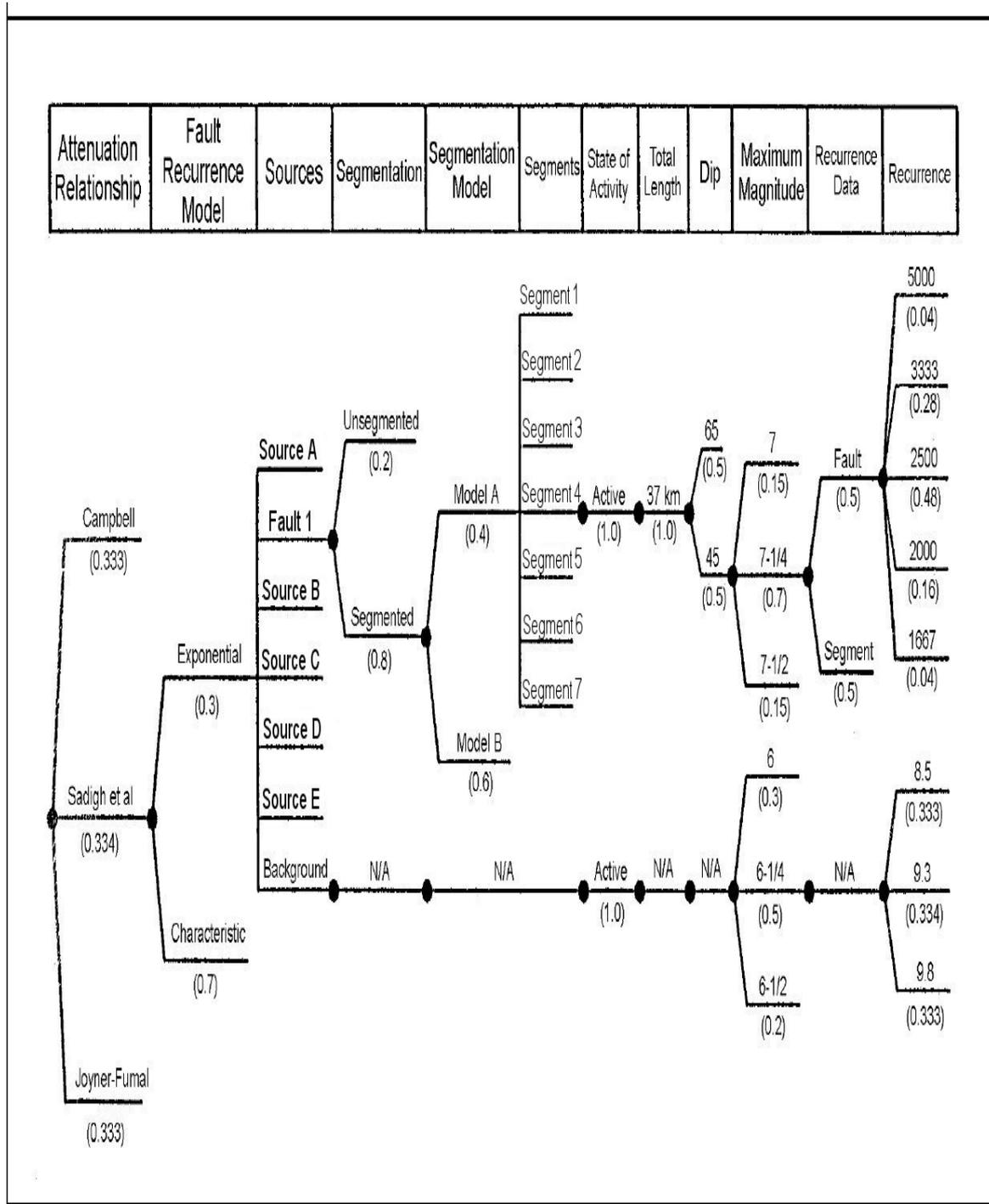


Figure 4.25 A generic logic tree model used in PSHA

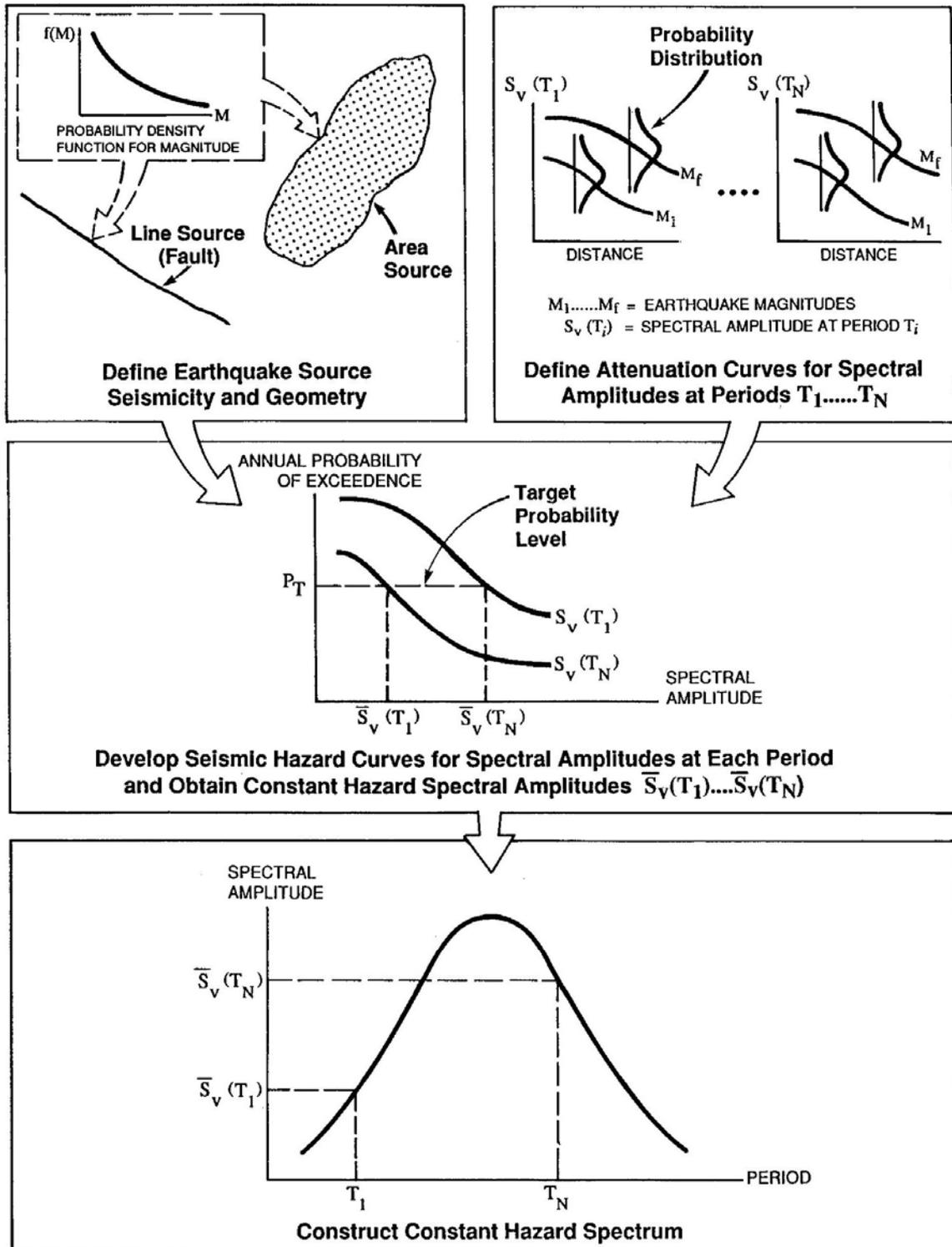


Figure 26 Development of Constant Hazard Spectra using Seismic Hazard Analysis (after EERI Committee on Seismic Risk)

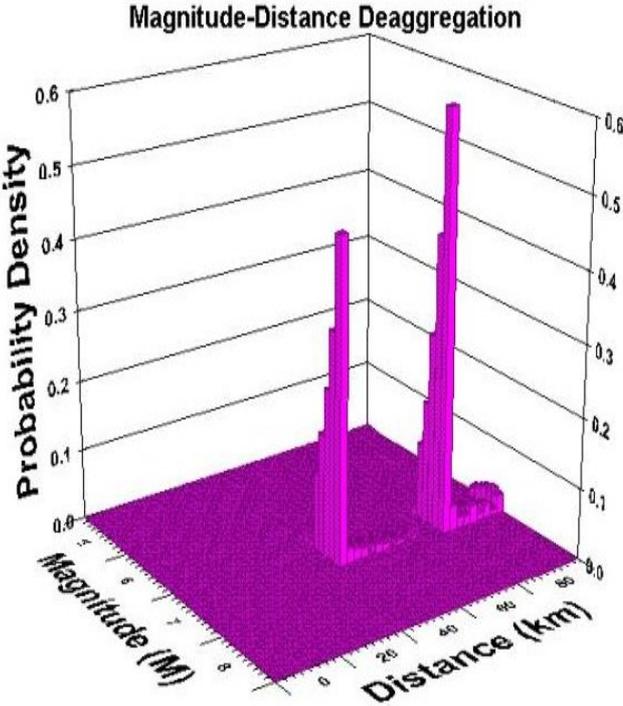


Figure 4.27 A typical deaggregation analysis

5. ZONATION WITH RESPECT TO GROUND MOTION CHARACTERISTICS

One of the objectives of seismic microzonation is to establish a seismic hazard map in the selected scale and thus estimate the earthquake induced forces on the ground surface and their variation in the investigated area (Aki & Irakura, 1991; Aguirre et al., 1994). This process should be based on the probabilistic seismic hazard analyzes as explained in detail in the previous section (Bender & Perkins, 1993; Allen, 1995; McGuire, 1995; Bommer, 2002).

Once the probable earthquake characteristics are determined the second step would be the evaluation of the ground motion characteristics on the ground surface in the investigate region taking into account local geological and geotechnical site conditions (Bouckovalas, 1997; Kudo, 1995; Marcellini, 1995). The complexity of this stage depends on the selected scale and the preferred accuracy of the results obtained. The analyzes that could be conducted would depend strongly on the available geological and geotechnical information.

For the purpose of seismic microzonation assuming that the preferred scale would be 1:5000, the possible approaches to estimate ground motion can be considered as; (1) empirical relationships based on contemporary attenuation relationships (2) various earthquake codes based on identified site parameters, (3) identification of geological/geotechnical units based on their properties such as equivalent shear wave velocity; (4) interpretation of strong and weak ground motion records as well as ambient noise measurements based spectral ratios (5) site response analyzes based on 1D or advanced numerical methods.

In the first part of this section an attempt will be made to review the literature for the purpose of identifying the effects of local site conditions and in the second part the above listed approaches will be summarised in more detail based on the literature available.

5.1. Site Effects

The second group of factors controlling ground motion characteristics during earthquakes could be considered as local geotechnical site conditions. The local site conditions could be very different due to variations in thickness and properties of soil layers, depth of bedrock and water table and could have significant effects on the characteristics of earthquake ground motions on the ground surface. There are extensive numbers of studies in the literature concerning the site effects based on damage distributions as well as on ground motion records (Idriss, 1990; Makra et al., 2001; Lachet, et al., 1996; Ansal, 1994; Beresnev et al., 1995 and many others cited in other parts of this report).

5.1.1 Topographical effects

The variation of damage as well as the strong motion records obtained during past earthquakes indicated that topographical effects can play important role on the ground motion characteristics (Geli et al., 1988; Faccioli, 1991; Athanasopoulos et al., 1999; Chávez-García et al., 1996, Reinoso et al., 1997). The approach adopted by AFPS (1995) was to introduce an additional empirical parameter as the topographical amplification factor in the definition of the design spectrum to account for the topographical effects;

$$\tau = 1 + 0.8(I - i - 0.4) \quad 1.0 \leq \tau \leq 1.4 \quad (5.1)$$

where I and i are the gradients of the lower and upper slopes respectively.

On the other hand geologic structures such as basins and sediment filled valleys have very important effects on the variation of the earthquake ground motion. Local site effects on basins and valleys have been investigated by many researchers (Murphy & Hewlett, 1975; Bard & Bouchon, 1980a, 1980b, 1985; Zhao, et al., 1993; Wen, et al., 1995; Rassem et al.,

1997, 1995; Gao, et al., 1996; Chin-Hsiung et al., 1998; Su et al., 1998; Wald & Graves, 1998; Kawase, 1998; Amirbekian & Bolt, 1998; Paolucci et al., 2000; Sokolov et al, 2000; Chávez-García & Faccioli, 2000).

Considering the suitability of basins and valleys for human settlements, large numbers of cities are located on basins and valleys. Therefore first special investigations are needed to determine the geological profiles by suitable geophysical methods such as in the approach adopted by Stephenson et al., (2000) when studying the basin profiles in Sherman Oaks after the 1994 Northridge earthquake. And secondly as summarized below based on some studies in the literature, microzonation conducted in these areas have to take into account the basin effects in order to be realistic and reliable.

Some Case Studies

Based on the records obtained from the dense strong motion array in Taipei basin for two earthquakes($M_L=6.5$ and 6.57) Chin-Hsiung et al., (1998) reported that there were significant differences among the peak ground accelerations, durations, and spectral accelerations in different parts of the basin as well as among the records for two earthquakes. The authors have concluded that for estimating ground motion variations for microzonation of the basin one must be very careful to draw conclusions by using only few seismic events and that it is necessary to collect more data from the array to perform a more detailed microzonation study.

Su et al., (1998) also observed similar effects where peak accelerations, peak velocity and peak displacements for the sediment sites are generally higher by a factor of 2-3 when compared with the records obtained on rock sites. The authors reported that there was significant increase in the duration of shaking in the sediment sites.

Sokolov et al, (2000) have observed that there were differences along the basin in terms of spectral ratios with respect to deep and shallow earthquakes where shallow events show low frequency amplification and deep events shows peaks in high frequency domain. The authors concluded that the site amplifications need to be determined by identifying the properties of multilayered and complex structure of the deposit and valley configuration.

Coutel & Mora (1998) compared the four site response estimation techniques, namely, sediment-to bedrock spectral ratio, sediment-to-bedrock noise ratio, horizontal-to-vertical noise ratio (Nakamura, 1989), horizontal-to-vertical spectral ratio. The authors concluded that 2D effects such as basin edge effects and focusing effects greatly influence site response and that estimation of site amplification spectra using observational methods yield unreliable or incorrect results when subsurface basin structure is present.

The strong motion records obtained by Kassem array during an earthquake clearly demonstrate the effect of surface irregularity (Kamiyama et al., 1999) as shown in Figure 5.1

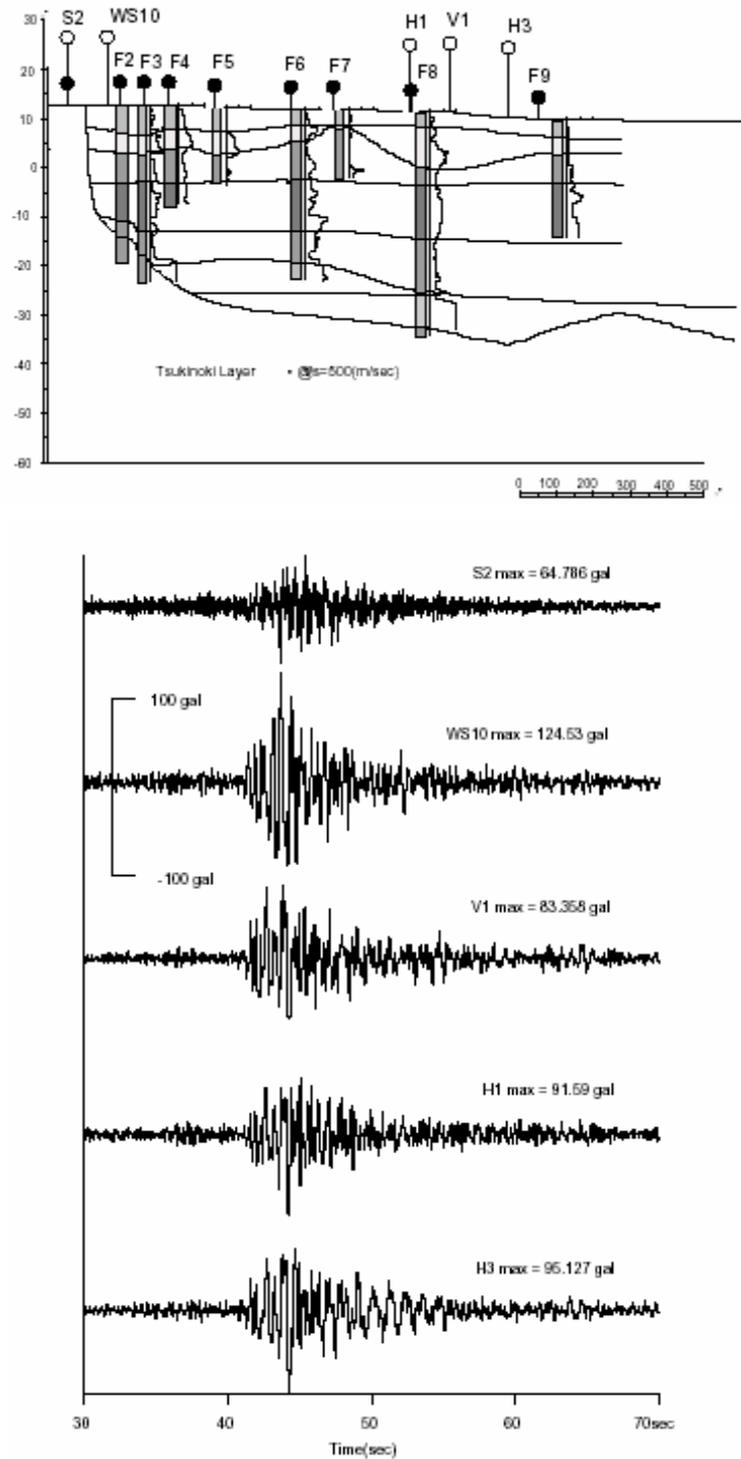


Figure 5.1 Soil profile and recorded ground motions by Kassem array (after Kamiyama et al., 1999)

From these conclusions as well as based on other investigations, it is evident that in conducting microzonation in regions including basins, it appears essential to determine the basin configurations and to adopt more advanced analysis and procedures such as 2D or 3D analyzes of the ground motion that could be induced by probable earthquakes.

5.1.2 Near field effects

The other important issue that was observed in the recent earthquakes is near field effects that modifies the earthquake ground motion characteristics depending on the directivity of the

fault rupture as well as prevailing site conditions (Vidale et al., 1991; Hall et al., 1995; Dimitriu, et al., 1999; Ansal, 1999). Numerical schemes were developed to analyze this situation (Suzuki & Asano, 2000; Pitarka, et al, 1998; Schneider et al., 1993). For practical design applications the empirical procedure suggested by Somerville et al., (1997) as explained in detail in the previous section offers one alternative.

5.2. Empirical correlations

One alternative as explained in detail in the previous section is to estimate the earthquake ground motion characteristics by using contemporary attenuation relationships that define ground motion in terms of peak ground acceleration or spectral acceleration depending on faulting and site classification (Ambraseys, 1995; Ambraseys et al., 1996; Abrahamson & Silva, 1997;. Boore et al., 1997; Campbell, 1993, 1997; Campbell & Bozorgnia, 1993; McVerry et al., 1993; Sadigh et al., 1997; Gregor & Bolt, 1997). However, due to the availability of the limited strong ground motion records obtained at different site conditions, the site classifications used in these types of contemporary attenuation relationships are very rough and in most cases may not be accurate enough to be used for seismic microzonation. One approach to overcome this limitation is to determine the earthquake ground motion characteristics on competent soil using a suitable attenuation relationship and then to modify this ground motion characteristics using site classifications given in one of the earthquake codes such NEHRP site classification and related amplification factors (Borcherdt, 1992, 1994).

Another approach which adopts a different methodology is to use seismic intensities. Seismic intensities are used to specify the shaking level and thus the extent of damage (Kagami et al., 1988). From this perspective one possibility is to perform microzonation with respect to the variation of the seismic intensity (Everden et al., 1973; Everden & Thomson, 1985; Bodel, 1992; Musson, 2000; Musson et al., 2001). In this approach attempts have been made to established relationships to estimate Modified Mercalli Intensity in terms of peak ground accelerations, peak ground velocities based on the obtained records (Wald et al., 1999; Atkinson & Sonley, 2000).

5.3. Site Classification according to Turkish Earthquake Code and NEHRP

One way of mitigating structural damage is to ensure that earthquake codes are in accordance with the expected seismic forces. Thus earthquake codes approach to this problem in slightly different ways inherently utilizing concepts that may also be used for microzonation purposes. Even though the purpose of an earthquake code is more related to site specific estimation of the induced earthquake forces in accordance with the selected exceedance probability for the structural design, there may be incompatibilities regarding to the differences among the map scales adopted for estimating the earthquake hazard and the site characterization. Thus one purpose of the microzonation may be considered as seismic zonation input for the structural design by replacing macro scale national zonation maps.

The 1998 Turkish Earthquake Code defines four local site classes to identify the site conditions. This method is one of the best methods used for site classification in earthquake codes. In fact, one can think that local soil conditions can be easily identified and reflected to design, if proper and appropriate experimental data are used for evaluating the soil conditions as specified in the code. However, strong motion records obtained in the near field during in past earthquakes display that earthquake characteristics can change significantly from one point to the other (Benuska, 1990; Chaves-Garcia et al., 1996; Field & Hough, 1997; Chin-Hsiung et al., 1998; Hartzell et al., 2000; Borcherdt, 2002). Thus, especially in case of high rise and high priority buildings, detailed investigation of local soil conditions and

determination of site specific design parameters must be obligatory in evaluating the seismic forces controlling the structural response. From this perspective, seismic microzonation studies may seem relatively inadequate because evaluating the design parameters for each lot in the selected area is not the objective. Moreover, converting the scale from 1:1,000,000 to 1:5,000 in earthquake hazard study as well as for the geological and geotechnical data will not provide enough resolution for a comprehensive assessment in such a scale.

Table 5.1. Soil Groups (1998 Turkish Earthquake Code)

Soil Group	Description of Soil Group	Standard Penetra. (N/30)	Relative Density (%)	Unconf. Compres. Strength (kPa)	Shear Wave Velocity (m/s)
(A)	1. Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks	—	—	> 1000	> 1000
	2. Very dense sand, gravel...	> 50	85–100	—	> 700
	3. Hard clay, silty lay.....	> 32	—	> 400	> 700
(B)	1. Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity.....	—	—	500–1000	700–1000
	2. Dense sand, gravel.....	30–50	65–85	—	400–700
	3. Very stiff clay, silty clay..	16–32	—	200–400	300–700
(C)	1. Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity	—	—	< 500	400–700
	2. Medium dense sand and gravel.....	10–30	35–65	—	200–400
	3. Stiff clay, silty clay.....	8–16	—	100–200	200–300
(D)	1. Soft, deep alluvial layers with high water table.....	—	—	—	< 200
	2. Loose sand.....	< 10	< 35	—	< 200
	3. Soft clay, silty clay.....	< 8	—	< 100	< 200

Table 5.2. Local Site Classes (1998 Turkish Earthquake Code)

Local Site Class	Soil Group according to Table 5.1 and Topmost Layer Thickness (h_1)
Z1	Group (A) soils Group (B) soils with $h_1 \leq 15$ m
Z2	Group (B) soils with $h_1 > 15$ m Group (C) soils with $h_1 \leq 15$ m
Z3	Group (C) soils with $15 \text{ m} < h_1 \leq 50$ m Group (D) soils with $h_1 \leq 10$ m
Z4	Group (C) soils with $h_1 > 50$ m Group (D) soils with $h_1 > 10$ m

One alternative to determine the effects of soil layers on design earthquake characteristics is to use the equivalent shear wave velocity that is defined as the weighted average of shear wave velocities of soil and rock layers in the upper 30 meters. Equivalent shear wave velocities may and are being used in earthquake codes for the purpose of evaluating the design earthquake characteristics on the ground surface. Based on the studies conducted in evaluating the site conditions at the strong motion stations, it was observed that the average shear wave velocity in upper 30 m can be used as one parameter to define the site conditions (Borcherdt, 1992; Anderson et al., 1996). Thus three criteria specified to distinguish the site classes are physical properties, mean shear-wave velocity to 30 m. and minimum thickness. These criteria define classes' apparent amplification characteristics that are useful for mapping purposes (Borcherdt, 1991; Borcherdt et al., 1991; Borcherdt 1994). The free field site specific response spectra was expressed as a function of the average short period (F_a) and mid-period (F_v) amplification factors with respect to the referenced ground condition used for determination of input ground motion spectral levels for the short period (acceleration, I_a) and mid-period (velocity, I_v) bands (Crouse & McGuire, 1996).

A site classification procedure was adopted in the NEHRP Provisions (Dobry et al., 1999, 2000) where two amplitude dependent site amplification factors were introduced; (F_a) for short periods and (F_v) for longer periods. Site classification is based on average shear wave velocity in the upper 30 m. Five site categories were specified as shown in Table 5.3.

Table 5.3 Site Categories according to the NEHRP 2000 Provisions

Site Class	Description	Shear wave velocity V_s Top 30 m (m/sec)	Stand.Pen. Res. N or Nch (blows/30cm)	Undrained Shear Strgth S_u (kPa)
A	Hard rock	> 1500		
B	Rock	760 - 1500		
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	1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.			
	2. Peats and/or highly organic clays ($H > 3$ m of peat and/or highly organic clay where $H =$ thickness of soil)			
	3. Very high plasticity clays ($H > 8$ m with $PI > 75$)			
	4. Very thick soft/medium stiff clays ($H > 36$ m)			

Where V_s and N are calculated average values for top 30 m, Nch and S_u are average values for cohesionless and cohesive soil layers in top 30 m.

For the purpose of taking into consideration the intensity of the expected ground shaking and related nonlinearity of soil layers, the short period factor (F_a) is defined with respect to spectral acceleration at short periods as given in Table 5.4 and the long period factor (F_v) is defined with respect to spectral acceleration at 1 second periods as given in Table 5.5.

TABLE 5.4 Values of F_a as a Function of Site Class and Spectral Acceleration

Site Class	Maximum Considered Earthquake Spectral Response Acceleration at Short Periods				
	$SS \leq 0.25$	$SS = 0.50$	$SS = 0.75$	$SS = 1.00$	$SS \geq 1.25$
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	By site-specific geotechnical investigation and dynamic site response analyzes				

TABLE 5.5 Values of F_v as a Function of Site Class and Spectral Acceleration

Site Class	Maximum Considered Earthquake Spectral Response Acceleration at 1 Second Periods				
	$S1 \leq 0.1$	$S1 = 0.2$	$S1 = 0.3$	$S1 = 0.4$	$S1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	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	By site-specific geotechnical investigation and dynamic site response analyzes				

As pointed out by Rodriguez-Marek, Bray & Abrahamson (1999, 2001) the average shear wave velocity neglects the effects of bedrock depth in site response. The authors list three important factors that influence the ground motion as; the natural period of the site, the impedance ratio between surficial and underlying deposits and the intensity of the ground motion. The suggested site classification system is based on two primary and two complementary parameters. The primary parameters are (1) the generic description deposit type such as hard rock, competent rock, weathered rock, stiff soil, soft soil, potentially liquefiable soil; (2) depth to bedrock or to a significant impedance contrast. The secondary parameters are; (1) the depositional age as Holocene or Pleistocene; (2) soil type as cohesionless or cohesive.

5.4. Identification of geological/geotechnical units based on their properties

5.4.1 Zonation with respect to geological units

One option is to adopt site classification based on geologic units. It is important as pointed out by Wills & Silva (1998) and Willis et al. (2000) to base this classification on accumulated data concerning the characteristics of each geologic unit (Nath et al., 1997). However, when conducting seismic microzonation study at a scale of 1:5000, it is also essential to realize the variations in each geologic unit. The deviations from the mean values obtained for each geologic unit can exceed the permissible limits to justify its use for assessing the effects of local soil conditions. Wills & Silva (1998) suggested and utilized the average shear wave velocity in the upper 30 m as one parameter to characterize the geologic units admitting the importance of other factors such as impedance contrast, 3-dimensional basin and topographical effects, and source effects such as rupture directivity on ground motion characteristics. The authors have used geological maps (generally 1:24 000 scale) to identify the geologic units. In the compiled database they have encountered significant variations in the equivalent shear wave velocities especially in the case of alluvium deposits. These variations were deduced to be mainly due to the age and grain size characteristics which are not always indicated in the geologic maps. It was concluded that it would not be possible to distinguish among different alluvium units based on shear wave velocity. The authors suggest

using shear wave velocity for classifying site conditions rather than geological units even though the determination of shear wave velocities requires extensive field investigations.

In this perspective Park & Elrick (1998) made an effort to correlated measured average shear wave velocities with the geological units determined from more detailed geologic maps. They concluded that incorporation of more detailed geological maps in the exploration process lowered standard deviations of the average shear wave velocities substantially over the values for the generalized geologic map.

5.4.2 Zonation with respect to equivalent shear wave velocity

The influence of local soil stratification on earthquake characteristics is one of the major factors in evaluating the earthquake forces and thus the structural response. Local soil conditions may amplify or deamplify the earthquake forces in different regions. However, sometimes an appropriate simplification may be needed to reflect the complex stratification characteristics in describing the soil amplification phenomena. As summarized above the upper 30 meters of the soil profile plays an important role in soil amplification and equivalent shear wave velocity can be used as one parameter for estimating the site amplification (Joyner & Fumal, 1984; Beresnev, & Atkinson, 1997; Ansal et al., 2001b).

5.5. Interpretation of earthquake records and ambient noise measurements

One possibility is to use weak and strong ground motion records as well as ambient noise measurements to estimate the site response. With the increase in the number of strong motion stations and especially with the installation of dense arrays, there has been a significant increase in the earthquake ground motion database (Frankel et al., 2001). Thus it is becoming possible and feasible to use these databases to estimate ground motion characteristics for seismic microzonation purposes.

5.5.1 Use of weak and strong ground motion records

The earthquake records obtained from broad spectrum of earthquakes can be used basically in two ways; by determining the spectral ratios of the recorded motions with respect to the spectrum obtained in the same event on the rock site SSR, or by determining the spectral ratio of the horizontal to vertical components for each record HVSR (Reinoso & Ordaz, 1999; Hartzell, et al., 1997; Raptakis et al., 1998a; Martirosyan et al., 2002). Both of these parameters are often very dependent on the magnitude and distance of the earthquake thus few sets of records would not yield very reliable estimates especially if strong ground motion records are limited. The weak to strong motion spectral ratios were observed to be variable from site to site most likely due to the differences in the site conditions as well as due to the differences in the frequency contents among the weak and strong ground motion records (Hartzell, 1998; Higashi & Sasatani, 2000). However, with the increase in the accumulated data, it is also appears possible to determine soil amplification and site response maps based on ground motion records (Boore et al., 1993; Hartzell et al., 1997; Atkinson & Cassidy, 2000; Beresnev et al., 1995; Theodulidis & Bard, 1995).

The other alternative is to use the obtained records as input design motion for more detailed engineering analysis (Bommer et al., 1998).

Some Case Studies

Raptakis, D. et al. (1998a) described a study in which the effectiveness of SSR and HVSR techniques on site response estimation were evaluated. For this purpose both techniques were applied to an acceleration data set recorded at the Euroseistest array during the period of 1994-1996. The data set consists of 495 time history records triggered by 36 events ($2.7 \leq M \leq 6.6$ and $12 \leq R \leq 160$ km). It was observed that both techniques show similar spectral

ratio shapes with comparable fundamental resonance frequencies, which were also well correlated with the well known geotechnical-geological conditions. A good stability of the HVSR in comparison with that of the SSR was observed. The results from these empirical analyzes were also found to be consistent with theoretical 1D equivalent linear response analyzes (SHAKE) and 1D elastic response analysis performed at the centre of the Euroseistest valley using the detailed geophysical and geotechnical data collected. It was observed that the amplitudes obtained from HVSR were systematically lower than those of the SSR. This was attributed to the relative enrichment of the vertical component. The authors suggest that the difference in amplitude between two methods could also be useful as an indication of 2D or 3D site effects due to near surface geology.

Riepl, et al. (1988) presented the results derived from the study of amplification effects at Euroseistest valley using records from a very dense network installed in the valley. The network was consisted of 31 stations installed perpendicular and parallel to the valley axis with minimum intervals of 250m. The main data set was consists of 13 local events recorded at least 15 stations. In addition 4 teleseismic events were used in the study. In the first part of the study variation of the mean amplifications due to the choice of data were tested by applying standard spectral ratio (SSR) technique to different data subsets recorded by the same network. Data up to 89 local events were used at this stage. It was observed that mean amplification values calculated from about 10 local events were as representative as the results obtained from a larger number of events and that the teleseismic events significantly improve the low frequency resolution. In the second part of the study, using the data set of 13 previously mentioned local events, results from SSR technique were compared with results from a generalized inversion technique (GIT), the H/V ratio technique, a coda wave technique and Nakamura's technique. It was concluded that SSR and GIT techniques reveal very similar results, and that the GIT is the most rapid and efficient method when processing data sets of several events and stations. The coda wave method, on the other hand, was the most time consuming and showed higher amplifications than the other methods, especially at higher frequencies ($>2\text{Hz}$). Spectral ratios from the HVSR and Nakamura's technique were observed to fail in estimating the amplification level. This pattern was attributed to the complex near surface geology. The authors suggested that the similar shapes of the site spectra estimated from coda wave method, the HVSR and Nakamura's techniques offer some information about significant frequencies particularly in the low frequency range. They concluded that a general application of HVSR and Nakamura's technique was unreliable and the use of these methods could be more useful as an indicator for 2D or 3D effects, if used together with the SSR or the GIT methods, rather than as a mean to evaluate ground motion amplification.

Field, E. and Jacob, K.H (1995) using the data from 18 aftershock events of Loma Prieta Earthquake recorded at 4 sediment sites and 1 bedrock site, the paper compares 2 reference-site dependent methods: standard spectral ratios (SSR) method, generalized inversion method (GIT), parameterized source and path effects inversion method and 2 non-reference-site dependent techniques: receiver function technique and Nakamura's technique. The study pays particular attention to the uncertainties involved in each method. It was found that site amplification estimates are very similar between SSR and GIT approaches. The uncertainty estimates can be significantly different depending on the data weighting procedure. If all data were weighted equally the uncertainties obtained from GIT can be up to a factor of 0.7 less than those of SSR. All of the three non-reference-site-dependent techniques are found to reveal useful information regarding the site response estimate. The parameterized inversion and receiver function techniques were found to be successful in identifying the frequency dependence of site response and the Nakamura's technique was found to be successful in identifying the fundamental resonant frequency. However it was observed that there are

discrepancies exist among all of the site response estimation methods when the frequency independent amplitude values are considered. If the SSR estimates are taken as the most reliable then all others under predict the site response (at this stage the results from an earlier work of one-dimensional model predictions are also compared and found to be lower by a factor of 2). The authors conclude that the frequency dependence of site response estimated from any of these methods is promising and if corroborated with other studies these estimations will be useful in areas that lack an adequate reference site.

5.5.2 Microtremor measurements

In the past decades, microtremor measurements became one of the popular alternatives in earthquake engineering for the purpose of identifying the local site conditions. With the development in technology, many improvements were made in test equipment, which increased the effectiveness and feasibility of microtremor measurements and opened the way for new methods. As a result, many new microtremor techniques are introduced to determine different site properties such as spectral amplification, predominant soil period, depth of bedrock, equivalent shear wave velocity profiles, etc. in order to obtain information of local site response characteristics. However despite the large number of studies made on microtremors, the discussion on the limitations and the use of microtremor measurements in earthquake engineering are still unresolved (Lachet & Bard, 1994).

Microtremors are defined as very little oscillations, which have amplitudes varying between 0.01~0.001 mm and periods of 0.01-20 seconds. The sources of microtremors can be natural and/or artificial, like wind, little magnitude earthquakes, ocean waves and/or industrial noise, traffic, to name a few. The classification of microtremors is (generally) made in terms of their period characteristics. Excitations with periods greater than 1 sec. are considered as long period microtremors, while excitations in the period range below 1 sec. is considered as short period microtremors. Although some researchers referred long period microtremors as microseisms in their studies (Dravinski et. al., 1991; Finn, 1991; Yamanaka et. al., 1994; Trifunac & Todorovska, 2000), microtremors with periods greater than 2 sec. are generally referred as microseisms (Akamatsu et. al., 1991; Lermo & Chavez-Garcia, 1994b).

Many studies were conducted to define the nature of microtremor motion (Aki, 1957; Santo, 1959; Kanai and Tanaka, 1961; Lermo et. al., 1988; Yamanaka et. al., 1993; 1994) since their first observation by Omori (1908). The results of these studies showed that the classification of microtremors according to the short and long period range approximately fits the boundaries in characterization of source and the nature of microtremors. The explanation of microtremor source characteristics and important points in regarding these features were discussed and summarised by Seo et al., (1990). Main criteria concerning the use of microtremors in geotechnical earthquake engineering and classification of microtremors are presented in Table 5.6. In this table, long period microtremors are regarded as microseisms originated from natural sources like sea waves, which display surface wave characteristics; where short period microtremors are defined as excitations originating from human activities traffic, machinery noise, etc. with constant predominant characteristics and both S-wave and Rayleigh wave like particle motion.

As seen from the table, the source of microtremors (thus the environmental factors in close neighborhood) plays an important role on the character of microtremor motion and the spectral characteristics. These effects were also observed in different studies for both short period (Nakamura, 1989) and long period microtremors (Yamanaka et. al., 1993).

From the engineering perspective, this becomes a significant issue as the vibration features of the source affects the propagation characteristics and the type of the waves dominant in motion, which are both important in spectral analysis stages. At this point, however, there is

no agreement among the researchers. Some researchers think that microtremors are made up of surface waves or Rayleigh waves (Frantti, 1963; Toksoz, 1964; Konno and Ohmachi, 1998; Bard, 1998), while the others consider that microtremors are made up of waves of different types including surface and s-waves (Nakamura, 1996; Kind et. al., 2000).

Table 5.6 Classification of microtremors (Seo, et. al., 1990)

Microtremors	
What is the source of microtremors?	
Human Activities Traffic and/or machinery noise	Natural Forces Sea waves under weather conditions
Microtremors S-Wave, Raleigh Wave	Microseisms Raleigh Wave, Love Wave
Stable Characteristics Constant predominant period Daily variation of amplitude	Unstable Characteristics Variation of period and amplitude due to weather conditions
Can we understand the site condition from measured microtremors?	
Are they applicable for predicting seismic motions?	

As a result of different approaches and assumptions, the studies on microtremors also show a wide variation in terms of the targeted parameters since the pioneering work made by Kanai and his colleagues (Kanai, et. al., 1954). In the following years, Kanai and Tanaka (1961) have applied short microtremor measurements to estimate the local site effects with respect to Japanese Code. This approach, which can be also accepted as one of the primary studies towards microzonation approach, included the evaluation of spectral analysis of microtremors by two different techniques in terms of predominant period-maximum amplitude and maximum period-average period.

Many different researchers used microtremor measurements in geotechnical earthquake engineering after these preliminary studies by Kanai and his colleagues in the 1950-1960s. In 1960s microtremor measurements were evaluated with the results of direct spectral analyzes of microtremor components (Kanai et al., 1966). New techniques were introduced after these years. Most popular two of these methods are the reference station and the single station (Nakamura's technique) methods, which are still used today. Reference station method was proposed by Borchardt (1970) for analyzing the earthquake records to obtain site effects. Reference station method requires records to be taken in the same time interval at least at two stations, one of which should be located on a rock site. The spectral ratio with respect to the rock site would represent the transfer function at each location. The main idea underlying this approach was to eliminate the source and path characteristics from earthquake records and to estimate the local site effects. Reference station method gives satisfying results, especially in case of far (distant) earthquakes, however, due to variation in local source characteristics and their nature, microtremors observed in the reference station and other stations may change significantly, which may lead to erroneous results. This was observed by different researchers in the literature (Seo, 1992; Iyisan et. al., 1997a), where there are many others who have successfully applied the reference station method (Kagami et. al., 1986, 1991; Iyisan et. al., 1997b; Taber, 2000).

The single station method is the most practical and feasible alternative among other microtremor methods. This method is simply based on calculation of the ratio of horizontal to vertical microtremor spectra to estimate the site amplification and predominant soil period. Although there were other studies on the H/V spectrum of microtremors (Shiono et. al., 1979; Kobayashi, 1980) before Nakamura's work (1989), this method is recognized as Nakamura's technique. There are still many arguments about the theory of Nakamura's technique among

the researchers (Konno & Ohmachi, 1998; Bard, 1998; Taber, 2000; Tsuboi et al., 2001), but many studies (Ansal et al., 1997, 2001c; Chavez-Garcia et al., 1996; Lermo & Chavez Garcia, 1994a; Bodin ve Horton, 1999; Teves-Costa et al., 1996) showed that it provides fair results, especially in terms of predominant soil periods, if velocity contrast is present and two or three dimensional discontinuities are negligible in the geological setting of the investigated area.

Regardless of the aim and the used technique; microtremor measurements became an inseparable part of microzonation studies as a result of their feasibility. In Manual for Zonation on Seismic Geotechnical Hazards (1993, 1999), ISSMGE included microtremor measurements besides the other methods in second level microzonation studies. Many different researchers used microtremor measurements in microzonation project to determine different characteristics and their geographical variation (Norio et al., 1996; Lachet et al., 1996; Gaull et al., 1995; Delgado et al., 2000; Nakamura & Saita, 1994).

Some Case Studies

Chavez-Garcia et al. (1996) presented a complementary study to microzonation of Mexico City where the authors studied the site effects at two counties that are located at the north of the city and that have been recently incorporated into the Mexico City conurbation. Measurements of microtremors at 67 points and weak motion recordings from a temporary digital seismograph network were analyzed according to HVSR technique to extend the existing predominant period map of Mexico City. Additionally a map of maximum relative amplification was proposed. Both maps were found to be consistent with local geology and suggested to be useful for a detailed microzonation study of the region.

Regnier, M. et al. (2000) presented a microzonation study conducted to evaluate possible site effects across Port Vila, Vanuatu, located very near to New Hebrid subduction zone. In this study the resulting maps were obtained in terms of predominant frequency and approximated amplification of the ground motion. The microzonation was based on microtremor measurements obtained at 100 sites across Port Vila. Nakamura's technique was used to estimate the site response. Using these results together with the topographic map and available borehole information four zones were identified. The resulting map showed that except for a few sites, for most of the city there is no significant site effect. These findings were in agreement with the damage observed at previous earthquakes.

Chavez-Garcia & Cuenca (1998) presented a study to improve microzonation of Acapulco, Mexico. Two maps that reflect the fundamental characteristics of the site effects in the area were proposed: dominant period and maximum relative amplification. The microzonation was based on three basic sources of data: strong motion records obtained from 7 permanent stations, weak motion data obtained from a temporary digital seismograph network of 6 stations, and measurements of microtremors at 35 sites. The strong motion data included recordings from 24 events with magnitudes of 3.5 to 6.9. The data was analyzed using both SSR and HVSR techniques. The weak motion data included recordings from 8 events in more than 4 stations, 3 events in more than 3 stations and a large number of events in 1 or 2 stations. They were analyzed using three techniques: SSR, HVSR and the computation of response spectral ratios (RSR). In the third method the velocity response spectra ratios at 5% damping were computed. Finally, microtremor recordings obtained at 35 sites were analyzed by Nakamura's technique. The resulting transfer functions obtained from weak and strong motion data using HVSR method are very similar to those obtained using SSR technique. This good agreement was observed both for dominant period and for the average value of maximum relative amplification. RSR technique applied to weak motion data also revealed similar values of dominant period. Amplification factors obtained from this method were in

general half of those determined from SSR. Comparing the results with those obtained from microtremor data showed that microtremors could provide a good estimation of dominant period. The maximum relative amplification factors obtained with this method were also good within an uncertainty factor of 2. Relying on the results obtained at this study the authors concluded that microtremors were useful tools to interpolate sparse earthquake data. The resulting microzonation maps of dominant period and maximum relative amplification factor were presented as based on the integration of all data obtained in the study.

Gutierrez, C. et al. (1992) described a site effect study carried out in Acapulco based on strong motion data from eight events ($4.2 \leq M \leq 6.9$) recorded by 6 accelerographs in the city. 4 of these instruments were located at soft sites while the other 2 were located at firm sites. One of the hard sites was selected as the reference site. Spectral amplification ratios for both S waves and S-wave coda were computed. The spectral amplifications of S wave coda were found to be very similar to those of S waves. Microtremor measurements at the 6 strong motion sites were also performed. Fourier amplitude spectra of the microtremor recordings were interpreted to obtain average spectral ratio curves for each site. The comparison of results evaluated from strong motion and microtremor data showed that the agreement was not good. Thus, authors conclude that microtremors can only give rough estimate of site response at the area of interest and caution is needed for its use at elsewhere.

Ochiai, et al. (2000) conducted a microzonation study based on microtremor measurements carried out for Sagami Plain, located in Kanto district of Japan. The microtremor measurements included fix-point continuous measurements at 3 sites and high density moving measurements at 932 sites dividing the area of interest into 500mx500m grids. HVSR technique was used to evaluate the predominant period for each of the grids. Fix-point continuous measurements were used to check the stability of HVSR technique. The resulting map of predominant periods was found to be closely related with the depth of alluvial deposit that was known from available borehole data as shown successively in Figures 5.2, and 5.3.

Figure 5.3 shows the result of microzonation in the Sagami Plain based on the high density moving measurements shown in Fig. 5.2. The zone A, where located near the mouth of the Sagami River, predominant periods are distributed between 1.0s and 1.5s, and these values were the longest periods. In the zone B located at east of the Sagami River, the predominant periods were between 0.8s and 1.2s, in the zone C, located in the basin of the Sagami River between 0.5s and 1.0s, in the zone D located in the east end and west end in the Sagami Plain between 0.3s and 0.5s, and finally in the zone E located plateau area between 0.1s and 0.4s. The result was closely related with depth of alluvial deposit which was indicated Fig. 5.2.

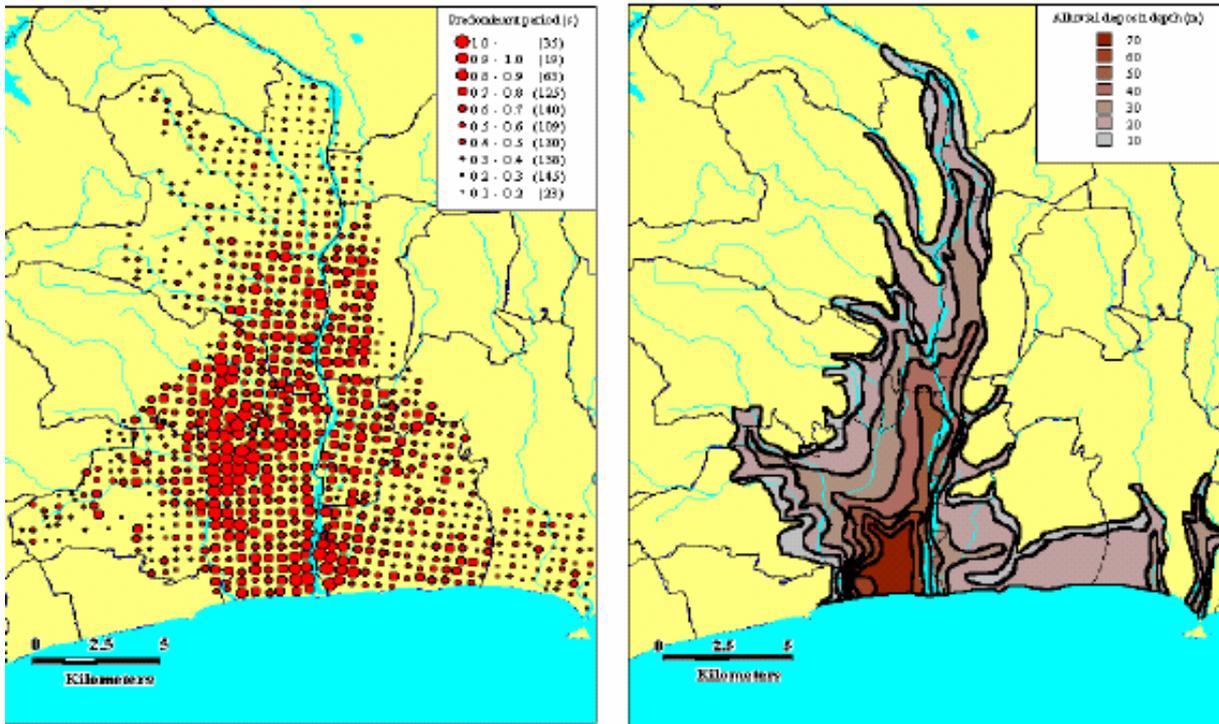


Figure 5.2 The result of high density moving measurements and alluvial plain depth with the depth contours (after Ochiai, et al., 2000)

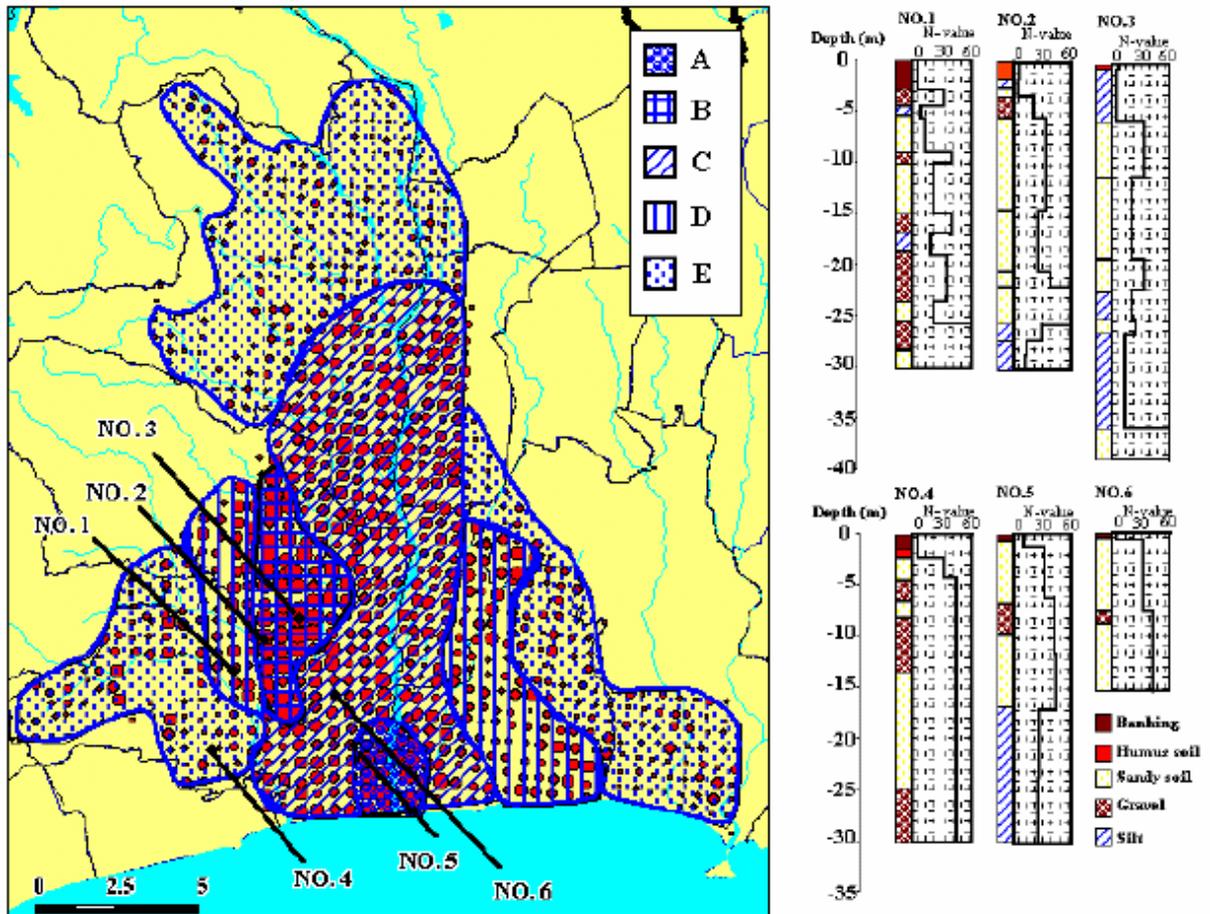


Figure 5.3 Seismic microzonation and typical soil profiles for each zone (after Ochia et al., 2000)

Lermo & Chavez Garcia (1994a) described a site effect study carried out for Mexico City. The study resulted in maps of dominant period and maximum amplification relative to a firm site within the city. The data used was a compilation of strong motion data and microtremor measurements. Strong motion data consisted of acceleration time histories from 9 events recorded at 81 locations in Mexico City. Microtremor data consisted of recordings at 409 points compiled from measurements of different groups during 1985-1992. Analysis of strong motion data was done using SSR technique. Empirical transfer functions were calculated by considering a hypothetical reference station which resulted from the average of all Fourier spectra recorded on firm soil in Mexico City for each earthquake. A map of maximum relative amplification was produced at this stage of the study. In the next step, information from microtremor analyzes performed by different groups with different techniques was compiled into a common format database. It was observed that different techniques by different groups lead to the same results. At 13 locations microtremor and strong motion data were both available. A very good agreement was observed between the results obtained from two types of data. Integration of all strong motion and microtremor data resulted in a map of dominant period for Mexico City. Based on this study some modifications were proposed by the authors to improve the current microzonation of the city.

Bour et al. (1998) presented an experimental study carried out on a plain of 60 km² near South France. Microtremor measurements at 137 points were obtained. The recordings were analyzed according to Nakamura's technique. H/V spectral ratios calculated by this method were then compared with those obtained from 1D numerical simulation (SHAKE91). The fundamental frequencies were in good agreement. The amplification levels were sometimes different. H/V spectral ratios were used to produce microzonation maps: a map of resonance frequency and maps of amplification levels as a function of frequency range. The conclusion was that the microtremors provide a very useful complement to the numerical methods for determining the fundamental frequencies of the surficial formations in the linear domain and both methods should be combined so as to better constrain the microzonation of a region.

Lermo & Chavez-Garcia (1994b) reviewed the applicability of microtremor measurements to evaluate site response of soft soils. Microtremor measurements obtained in Mexico City, Oaxaca and Acapulco were used to compare three most common methods of analysis: interpretation of Fourier amplitude spectra, computation of spectral ratios relative to a reference site and Nakamura's technique. In order to evaluate the results obtained from microtremor measurements they were compared to those obtained from standard spectral ratios method applied to weak and strong motion data recorded at the sites where microtremors were performed. The comparisons showed that microtremors allow for determination of the dominant periods with good reliability in the range of 0.3 to 5Hz. The best results were obtained from Nakamura's technique, which also could give rough estimation of the amplification level where the local geology was relatively simple. The authors recommend the use of this technique since they believe that it compensates for the source effects in microtremor measurements. They conclude that the microtremors are very useful in complementing other site effect studies.

Konno & Ohmachi (1998) assuming that microtremors consist of surface waves, the H/V has been formulated in terms of Rayleigh and Love waves in layered soils. The authors have concluded that peak periods in H/V ratios of fundamental mode Rayleigh waves agree well with fundamental periods of transfer functions for vertically incident S waves, and the H/V ratio tends to show a trough at half of the peak period when site has soft surface soil. Based on these, the authors recommend using the periods of not the peaks but also the troughs in the H/V ratios of microtremors in view of estimating the fundamental periods. The amplification factor has been correlated with H/V peak ratio of fundamental mode Rayleigh waves. In the

process, when smoothed with logarithmic function, the H/V peak ratio was found to be directly related with the amplification factor, especially when proportion of Rayleigh waves in microtremors is 0.4, the peak ratio becomes nearly equal to the amplification factor.

5.6. Site response analyzes

5.6.1 One dimensional analyzes

In one-dimensional analyzes, the most widely used analytical method is the multiple reflection models for the propagation of S waves in a one-dimensional column (Haskell, 1953; Iida et al., 1978). The data from borings or measurements of shear-wave velocity are used to construct the soil medium model as a mesh. When such data are not available, generic ground conditions can be used (Shima & Imai, 1982).

Since all soils have highly nonlinear properties, nonlinearity in site characterization and analysis has to be taken under serious consideration. Moreover nonlinear behavior can also be observed in the earthquake ground motion records (Tokimatsu & Midorikawa, 1982; Chang et al., 1991). In order to account for the nonlinear behavior of soil layers, a widely used computer program Shake was developed by Schnabel et al. in (1972). Later Shake was modified in accordance with the advances in computational capabilities by Idriss & Sun (1992). The program utilizes an equivalent linear analysis to approach the nonlinear solution and computes the response in a horizontally layered soil deposit subjected to transient, vertical traveling shear waves.

5.6.2 Advanced Methods

In order to include the non-homogeneous nature of natural soil deposits, analytical methods incorporating two or three-dimensional analysis have been developed (Olsen, 2000). Aki & Irikura (1991) have reported an extensive investigation on the recent advances in numerical simulation. After the classification by Horike et al. in 1990, these can be summarized as: the wave function expansion method (Trifunac, 1971; Sanchez-Sesma, 1985); finite element method (Lysmer & Drake, 1971); finite difference method (Boore, 1972; Virieux, 1984); the discrete wave number method (Aki and Larner, 1970; Bouchon and Aki, 1977; Bard and Bouchon, 1980a, 1980b, 1985); boundary integral method (Wong and Jennings, 1975; Sánchez-Sesma & Esquivel, 1979); discrete wave number boundary element method (Bouchon, 1985; Kawase, 1988); ray tracing and beam methods (Hong and Helmberger, 1978; Lee and Langston, 1983; Nowack and Aki, 1984).

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Raptakis, D. et al (2000) re-interpreted the available geotechnical and geophysical data in order to define subsoil structure of Euroseistest in terms of the most important parameters needed to model site response. The resulting 2D model of the valley differs from that previously published. In this reinterpretation V_s values, mainly determined from SWI, played the most important role for constructing the 1D and 2D profiles of the valley. In the second part of the paper, the geometry and the dynamic properties of this recent 2D model were correlated with the results of site response determined from the analysis of two representative events in frequency and time domains and 1D numerical modeling. The records of the events were obtained from two different arrays-a temporary seismograph and a permanent accelerograph network- that were deployed in 1994. (39 digital seismographs operated for more than two months during 1994, while 7 surface accelerographs plus 2 borehole accelerographs were in operation since that date) Empirical transfer functions were estimated using the SSR technique. The analysis of the seismograph and accelerograph data in both frequency and time domains revealed that the amplification of ground motion was due not only to the resonance of vertically propagated shear waves but also to locally generated

surface waves. 1D theoretical estimates of site response were also computed at sites where seismographs and accelerographs had been installed, using Kennet's reflectivity coefficient method. In comparing both the theoretical and the empirical transfer functions, agreement on the resonance frequency was found while it was observed that theoretical fundamental peaks were lower than empirical ones. This was taken as the evidence that 1D theoretical transfer functions cannot estimate the amplification where 2D phenomena are included. Synthetic time histories were also obtained by the convolution of 1D transfer functions with records at the reference site. A large disparity was observed between recordings and the synthetics concerning their duration. In summary, the study of the site response by the methods mentioned above showed the important effect of the lateral variations on the ground motion and suggested that the contribution of locally generated surface waves to the resonant peak might be important. It was concluded that the use of 1D transfer function to model ground motion under complex soil conditions like Euroseistest valley is not efficient and that underestimated duration of shaking may lead to erroneous predictions of ground motion.

Chavez-Garcia, F.J. et al. (2000) presented the results of numerical modeling of site response at Euroseistest. 2D model proposed by Raptakis et al.(2000) was used for the study. Response of this 2D structure to vertically incident SH waves was computed by finite difference method of Moczo (1989). The results confirm that the site response at this valley was dominated by laterally propagating surface waves generated at the edge of the valley. Evidence of these surface waves were also found in recorded earthquake data and the observations made suggested that locally generated waves contribute significantly to ground motion and that they appear at the same frequencies as resonance of vertically propagating shear waves. Finally it was decided that empirical transfer functions give only partial information (the amplitude) of site response and thus are misleading where 2D site effects are important.

Makra et al. (2001) taking advantage of the previous studies of site effects done at Euroseistest site evaluated the differences between site effects obtained by simple 1D computations and those obtained by using a detailed 2D model of the subsoil structure. The study indicated that 2D model produced an additional amplification in response spectra that cannot be accounted for without reference to the lateral heterogeneity of the valley structure. Comparison of numerical results with observations in time, frequency and response spectra domain showed that this additional amplification computed from 2D model was real and that it affected the response spectra by a significant factor. Thus, the authors suggested that the definition of an aggravation factor due to the complexity of local geology is worthy of consideration in microzonation studies and seismic codes. For the case of Euroseistest such a factor attains a value of 5 for periods longer than 0.3s. Despite the fact that the results were only for Euroseistest and that all numerical computations were based on linear behavior of soil material, the authors think that similar studies within Europe may confirm this aggravation factor and if so, it can be used to incorporate the effects of complex geology in seismic design codes and microzonation codes.

Teves-Costa et al. (2001) utilized geological maps with 1:10,000 scale complemented with available geotechnical borehole data to construct 22 geological profiles along east-west direction approximately 500 meters apart, together with 2 oriented in the north-south direction. The site response analyzes were conducted using 1-D Thomson-Haskell method to 314 profiles selected from these geological profiles according to a regular grid 500 meters long. The transfer functions for vertical and horizontal movements were obtained with the vertical incidence of P waves and the SH waves respectively. The physical parameters needed for Thomas Haskell method (seismic velocity, thickness and density) were estimated taking into account the laboratory data and seismic experiments from previous studies. The results were

plotted as contour maps for the peak frequencies and for the corresponding amplification factors. A good agreement was found when these resulting maps were compared with the previously published microzonation map obtained by microtremor analysis and with the damage distribution observed at past earthquakes. The authors concluded that the theoretical 1D analysis used in this study and the microtremor analysis could be used as complementary methods for microzonation purposes as a first approximation.

Harris, J.B. (1999) presents a site response study carried out in Paducah, Kentucky. In the first part of the study seismic refraction and reflection data were collected at 37 locations. Soil column models based primarily on shear wave seismic data and existing geotechnical information were developed for these 37 sites. A one dimensional site-response modeling computer program WAVES designed to simulate propagation of vertically incident shear waves from bedrock through a column of horizontally layered soil formations was used to compute the response of these soil columns. Due to the lack of strong motion data a suite of five accelerograms from Canadian earthquakes was used as input motion at the bedrock level. The soil parameters needed for the WAVE analysis were G_{\max} values, thickness and damping ratios of soil layers. In this study shear wave velocity and layer thickness data were collected using seismic refraction and reflection methods, densities were estimated based on geotechnical reports and damping ratio estimates were made from relationships between damping and strain, and using attenuation measurements from the seismic data. 5% damped acceleration response spectra were calculated at both the bedrock and surface levels. Spectral ratio values and the periods at which the spectral ratio makes a peak (dynamic site period) were obtained. Based on the seismic field investigation, the spectral ratio data and the dynamic site period data a generalized microzonation map of Paducah was developed. The map suggested that the area could be separated into 3 zones that appeared to correlate with the near surface geology. Maps of depth to bedrock, average shear wave velocity were also presented in the study. In the second part of the paper synthetic seismic records were generated through stochastic modeling of earthquake source parameters. Three soil columns were designed to represent typical conditions for each of the 3 zones. The WAVES program was used to simulate the propagation of hypothetical earthquake through the typical soil columns. 5% damped acceleration spectra were computed and spectral ratios curve for each zone was plotted. In third part of the paper shear wave velocities obtained from seismic data, SPT values from drillings at representative sites and laboratory resonant column tests were compared. Values estimated from SPT values generally agreed with those obtained from SH wave seismic data while values from resonant column tests were consistently on the order of 30% lower. The paper concludes that surface seismic techniques are reliable and cost effective and they can be used for generalized evaluation of site conditions. A practical site characterization procedure might be an integrated surface seismic and drilling program in which the seismic data are used to direct the placement of boreholes and insure that representative conditions within the area are adequately sampled.

Martin, C. et al. (1995) discusses a method developed under a geographic information system (GIS), which helps to identify areas with similar geomechanical behavior within a microzonation study. The method results in the creation of a 3D geotechnical model of the subsurface. 8 successive steps were followed: 1. creation of a topographical digital elevation model at 10m intervals, 2. analysis and input of data of point data from boreholes and digitizing of surface data, 3. identification of geotechnical units characterized by geomechanical values that are defined by an average value and standard deviation, 4. drawing and digitizing of geotechnical cross-sections, 5. creation for each geotechnical unit of a data file X,Y,Z, Z being the elevation of the base of the unit of interest, 6. discretization of the study area into a finite number of meshes, 7. for each unit interpolation of these data and

creation of a digital elevation model corresponding to the elevation of its base, 8. calculation of a final geotechnical model through the differential of all numerical models. Once the geotechnical model is constructed, it can be exploited according to the method of increasing level of complexity for the definition of seismic reference motions. Three levels of complexity is defined by AFPS: A, B and C. For level A the microzonation is restricted to the mapping of site categories S0, S1, S2 and S3 defined in the French code. In this case geotechnical model is used to attribute a site category for each mesh. For the intermediate level B, maps of fundamental frequencies F0 and their corresponding amplifications A0 are drawn up. This is done with either simplified methods or empirical laws. In this case, the parameters at each mesh are applied to the simplified formulae, and then the obtained values for F0 and A0 are plotted on maps. Comparison of two maps enables the calculation of a simplified transfer function for each site. For level C, the transfer function of each site is estimated by numerical modeling. In this study SHAKE was used for sites with a simple geometry, whereas a finite element program GEFDYN was used for sites with more complex 2D geometry. In the second part of the paper an application of this method to Lamentin (about 30 km² in size). 3D geotechnical model was obtained as based on data from 300 boreholes and corresponding tests, 12 seismic refraction and electric tests, 2 crosshole tests, 22 geotechnical sections spaced at 250 m, several reports of geotechnical investigations. Statistical analysis of these data identified 6 geological units whose main characteristics were quantified. For each of these units the data were interpolated with the minimum curvature method to create a DEM of each unit base. The model is then discretized into meshes of 25 meters, resulting in 40 000 meshes for each DEM. Statistical processing based on this model led to the recognition of 44 different geotechnical configurations. Then using the seismic hazard parameters from an earlier work site response of the area was evaluated for levels A and C. For level C 44 response spectra were compared to identify 6 seismic zones each characterized with a specific response. The study resulted in two maps: a map of microzonation for level A which maps sites S0 to S3 and a map for level C which maps 6 zones R1 to R6.

6. ZONATION WITH RESPECT TO LIQUEFACTION SUSCEPTIBILITY

Liquefaction resistance can be estimated by in situ test or laboratory test. Standard penetration and cone penetration tests are the most used for the estimation of liquefaction susceptibility. Methods based on the SPT were developed by Seed and Idriss (1971), Seed et al. (1985), Iwasaki et al. (1978), Tokimatsu and Yoshimi (1983), Iai et al. (1989) and the Japan Road Association (1991). Methods by using the CPT include those developed by Seed and Alba (1986), Ishihara (1985), Shibata and Teeparaksa (1988) and Robertson and Campanella (1985). For engineering purposes, data obtained from site investigation including boring, laboratory test need to be used besides methods based on SPT and CPT (Finn, 1993; Ansal, 1991).

There are large numbers of zonation with respect to liquefaction (i.e., Hitchcock et al., 2000). As an example in the study conducted by Erdik et al. (2000) liquefaction susceptibility was determined for the city of Izmir based on SPT test results obtained from large number of borings using Seed et al. (1985) method as shown in Figure 6.1. The liquefaction susceptibility was classified as high, medium and low with respect to the safety factors.

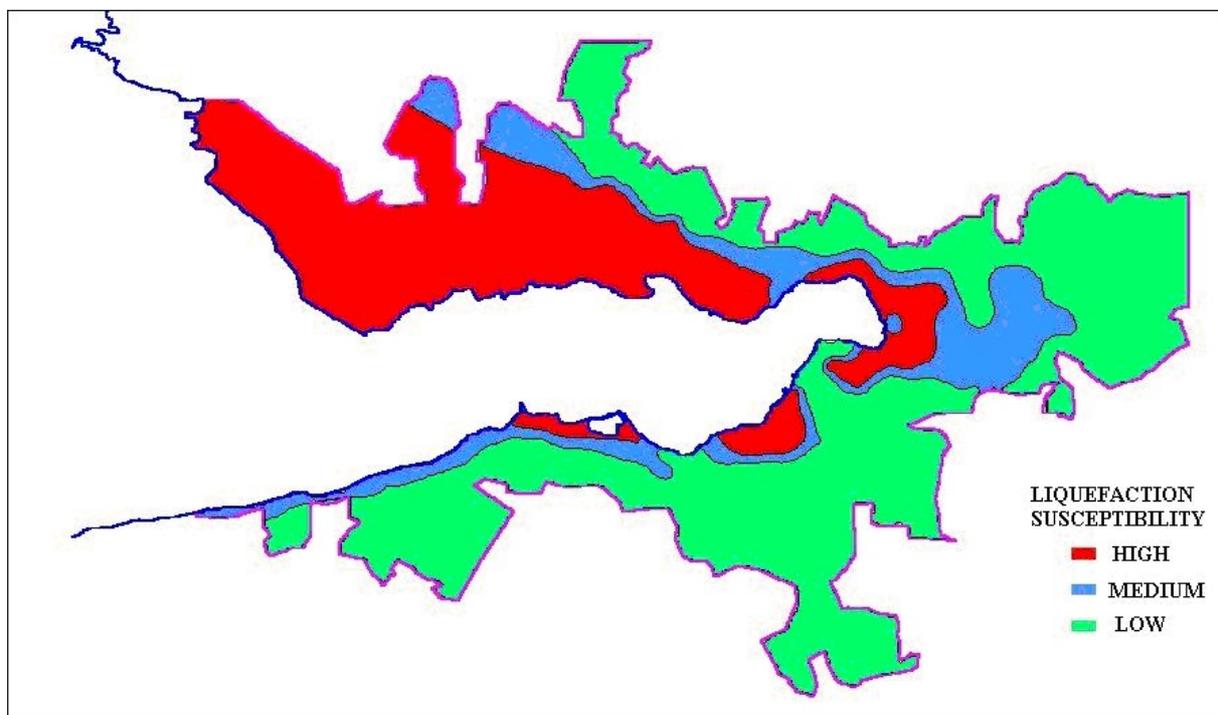


Figure 6.1 Liquefaction susceptibility for the city of Izmir

6.1. Liquefaction phenomenon, factors affecting liquefaction

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress (Martin et al., 1975; Marcuson 1978, Castro & Poulos, 1977). Increased pore water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils, such as silty sands and sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs soil stratum softens, allowing large cyclic deformations to occur. In loose materials softening also accompanied by a loss of shear strength that may lead to a large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or a sloping ground. In moderately dense to dense materials, liquefaction leads

to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressure dissipates. (Youd et al., 2001)

More investigations on factors causing and phenomena observed during and after earthquake-induced liquefaction are studied, primarily by means of geotechnical centrifuge modeling. The principles and the advantages of centrifuge modeling in investigating earthquake related problems are given in Chapter 8.3.1. These studies can be found in the regular conference proceedings from the Centrifuge and physical modeling community (Corte 1988, Ko & McLean 1991, Leung et al. 1994, Kimura et al. 1998 or Phillips et al. 2002). Some of the most recent findings are given here.

Explanation of liquefaction processes has been based on the use of more advanced measuring options as well as physical and numerical modeling techniques, which were developed in a framework called VELACS (VERification of Liquefaction Analysis using Centrifuge Studies, Arulanandan et al. 1994). Subsequently, Kutter & Balakrishnan (1998) described the development of dynamic physical modeling techniques required for investigation of liquefaction potential. Using more recent developments, like waterproof shear wave transducers or harmonic wavelet techniques it is possible to identify the liquefied area inside the soil mass (Teymur & Madabhushi 2002, Imamura & Fujii 2002). These techniques can be used in the future to analyze the interaction of the soil with any kind of structure in more detail during earthquake events. More specific studies with respect to liquefaction are dealing e.g. with the comparison and thus the reliability of laboratory investigations by comparing the results of laboratory tests with data from centrifuge model tests. The data from centrifuge model tests under defined boundary conditions are used here as a replacement for field measurements (Dief & Figueira 2002).

Some studies on liquefaction are related to other methods of physical modeling or analysis of full-scale events. Vallejo & Scovazzo (2002), for example, studied the development of sand boils developing from sand lenses by means of a shaking table for specific p-wave conditions. A recent masters thesis (Jacka 2002), addressed the question of the collapse of a bridge abutment in the post-earthquake period due to the influence of particle redistribution and void ratio increase during liquefaction (see also Berrill et al. 2001), whereas Malvick et al. (2002) observed the change of soil conditions during the earthquake and showed the development of shear strains in the soil strata for a slope focusing on the influence of input motion, relative density and shaking sequence.

6.2. Modified Chinese Criteria (Finn, et al. 1994)

According to these criteria as shown in Figure 6.2 soils are considered to be potentially liquefiable if (after Seed et al., 2001):

- (1) clay fines (based on Chinese definition of clay sizes as less than 0.005 mm) are less than 15%,
- (2) Liquid Limit is equal or less than 35%,
- (3) current in-situ water content is equal or greater than 90% of the Liquid Limit.

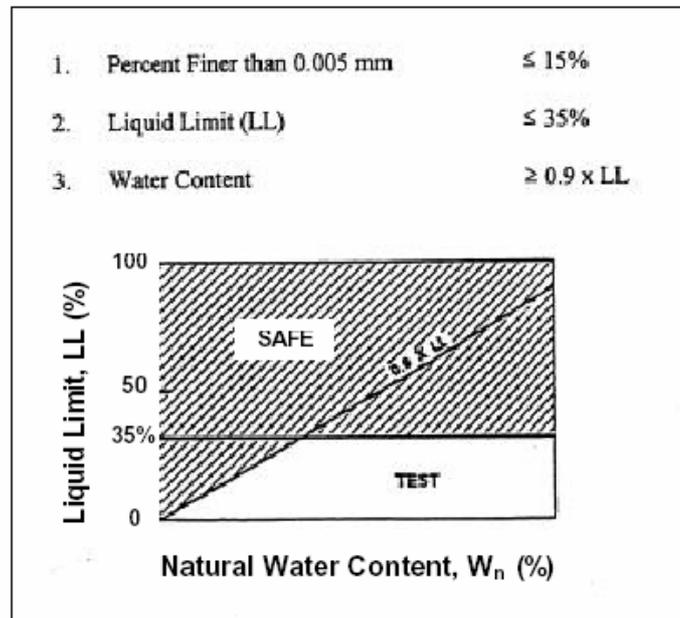


Figure 6.2 Modified Chinese Criteria (after Finn et al., 1994)

6.2.1 Re-evaluation of Modified Chinese Criteria (Andrews & Martin, 2000)

Andrews & Martin (2000) have analyzed the liquefaction field case histories from the Seed et al. (1984, 1985) database and applied Modified Chinese Criteria to US conventions (with clay sizes defined as those less than about 0.002 mm). Andrew and Martin recommend that soils with less than about 10% clay fines and a Liquid Limit (LL) in the minus #40 sieve fraction of less than 32% be considered potentially liquefiable, that soils with more than about 10% clay fines and $LL \geq 32\%$ are unlikely to be susceptible to classic cyclically-induced liquefaction, and that soils intermediate between this criteria should be sampled and tested to assess whether or not they are potentially liquefiable (Seed et al., 2001). Figure 6.3 provides a summary of the criteria.

	Liquid Limit ¹ < 32	Liquid Limit ≥ 32
Clay Content ² < 10%	Susceptible	Further Studies Required <i>(Considering plastic non-clay sized grains – such as Mica)</i>
Clay Content ² ≥ 10%	Further Studies Required <i>(Considering non-plastic clay sized grains – such as mine and quarry tailings)</i>	Not Susceptible

Notes:

1. Liquid limit determined by Casagrande-type percussion apparatus.
2. Clay defined as grains finer than 0.002 mm.

Figure 6.3 Liquefaction susceptibility of silty and clayey sands (after Andrews & Martin, 2000)

6.3. Use of Laboratory Testing of Undisturbed Samples

The use of laboratory testing of undisturbed samples is complicated by difficulties associated with sample disturbance during both sampling and reconsolidation. It is also difficult and expensive to perform high-quality cyclic simple shear testing, and cyclic triaxial testing poorly represents the loading conditions of principal interest for most seismic problems. Use of appropriate “frozen” sampling techniques and subsequent testing in a high quality cyclic simple shear or torsional shear apparatus can be helpful to some extent. However, the difficulty and the cost of these techniques, place their use beyond scope of most engineering studies. (Seed et al., 2001)

6.4. Use of Empirical Relationships based on Correlations with In-Situ Index Tests

The use of in-situ index testing is the dominant approach in common engineering practice for quantitative assessment of liquefaction potential. Calculation or estimation of two variables is required for evaluation of liquefaction resistance of soils (Seed et al., 2001; Youd et al., 2001):

- (1) the seismic demand on a soil layer, expressed in terms of *CSR*,
- (2) the capacity of the soil to resist liquefaction, expressed in terms of *CRR*.

6.5. Criteria based on SPT (Standard Penetration Test) data

Criteria for evaluation liquefaction resistance based on SPT data are largely embodied in the *CSR* versus $N_{l,60}$ plot (Youd, et al. 2001).

6.5.1 Simplified Procedure (Seed et al., 1984, 1985)

This procedure is based on the relationship of SPT *N*- values, corrected for both effective overburden stress and energy, equipment and procedural factors affecting SPT testing (to $N_{l,60}$ -values) versus intensity of cyclic loading, expressed as magnitude-weighted equivalent uniform cyclic stress ratio (CSR_{eq}). The correlation between corrected $N_{l,60}$ -values and the intensity of cycling required to trigger liquefaction is also a function of fines content as shown in Figure 6.4 (Seed et al., 2001).

CSR_{eq} is estimated from the simplified method of Seed and Idriss (1971) as,

$$CSR_{Peak} = \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_v} r_d \quad (6.1)$$

$$CSR_{eq} = 0.65 CSR_{Peak} \quad (6.2)$$

where a_{max} = peak horizontal ground surface acceleration
 g = acceleration of gravity
 σ_v = total vertical overburden stress
 σ'_v = effective vertical overburden stress
 r_d = stress reduction factor

The r_d values can be estimated from,

$$r_d = 1 - 0.015z \quad (6.3)$$

where, z is the depth below ground surface in meters

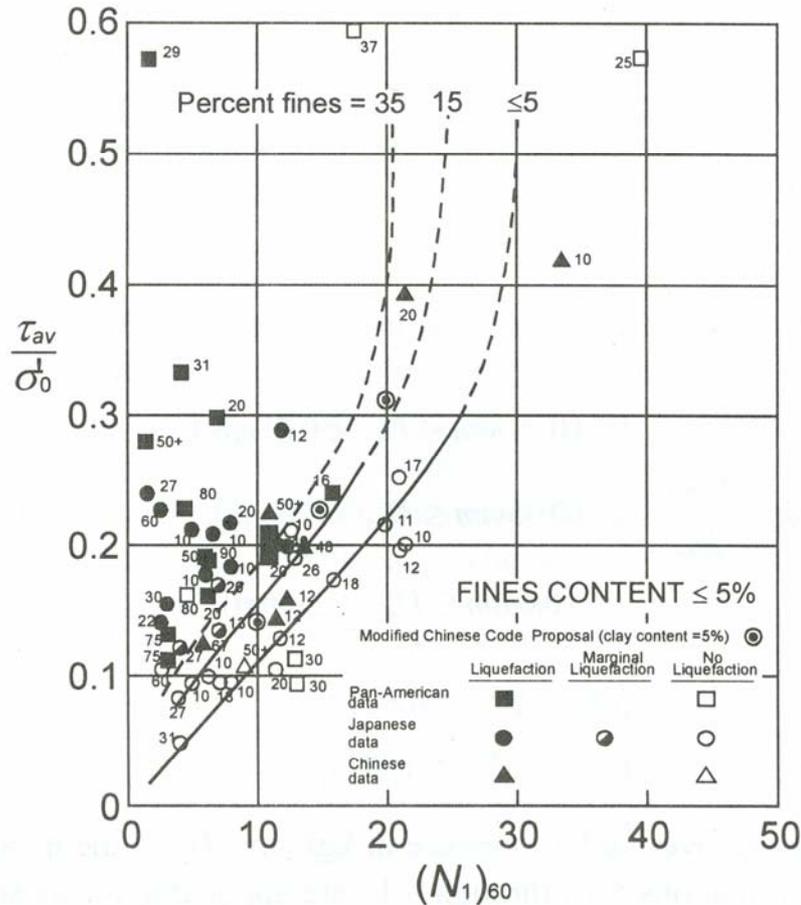


Figure 6.4 Correlation between equivalent uniform cyclic stress ratio and SPT $N_{1,60}$ value for events of magnitude $M_w = 7.5$ for varying fines contents (Aftee Seed et al., 1984, 1985)

6.5.2 Revised Simplified Procedure (NCEER, 1997, Youd et al., 2001)

The procedure for this revised version of Simplified Method is summarized from Youd et al. (2001).

Step 1. CSR is calculated from Seed and Idriss (1971)

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_v} r_d \tag{6.4}$$

where a_{max} = peak horizontal ground surface acceleration

g = acceleration of gravity

σ_v = total vertical overburden stress

σ'_v = effective vertical overburden stress

r_d = stress reduction factor.

The average values r_d is estimated from Liao & Whitman (1986b) as,

$$r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15m \tag{6.5a}$$

$$r_d = 1.174 - 0.0267z \text{ for } 9.15m < z \leq 23m \tag{6.5b}$$

where, z is the depth below ground surface in meters.

Step 2. Corrected $N_{l,60}$ values are calculated as,

$$N_{1,60} = NC_N C_R C_S C_B C_E \quad (6.6)$$

where N = measured standard penetration resistance,

C_N = factor to normalize N to a common reference effective overburden stress

C_R = correction for rod length,

C_S = correction for non-standardized sampler configuration,

C_B = correction for borehole diameter,

C_E = correction for hammer energy ratio.

C_N is calculated from Liao & Whitman (1986a) as,

$$C_N = \left(\frac{P_a}{\sigma'_v} \right)^{0.5} \quad (6.7)$$

where C_N normalizes N to an effective overburden pressure σ'_v of approximately 100 kPa (1atm) P_A . C_N should not exceed a value of 1.7. C_N can also be calculated from Kayen et al. (1992), which limits its maximum value to 1.7,

$$C_N = \frac{2.2}{(1.2 + \sigma'_v / P_a)} \quad (6.8)$$

Table 6.1 summarizes the values of the other correction factors used in Eq. (6.6).

Table 6.1 Corrections to SPT as listed by Robertson & Wride (1998)(after Youd et al., 2001).

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_A/\sigma'_v)^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5-1.0
Energy ratio	Safety hammer	C_E	0.7-1.2
Energy ratio	Automatic-trip Donut type-hammer	C_E	0.8-1.3
Borehole diameter	65-115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3-4 m	C_R	0.8
Rod length	4-6 m	C_R	0.85
Rod length	6-10 m	C_R	0.95
Rod length	10-30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1-1.3

Step 3. $N_{l,60}$ values are further corrected to account for the influence of fines content using,

$$N_{1,60,CS} = \alpha + \beta N_{1,60} \quad (6.9)$$

where α and β coefficients determined from the following relationships:

$$\alpha = 0, \beta = 1.0 \quad \text{for } FC \leq 5\% \quad (6.10a)$$

$$\alpha = \exp[1.76 - (190/FC^2)], \beta = [0.99 + (FC^{1.5}/1000)] \quad \text{for } 5\% < FC < 35\% \quad (6.10b)$$

$$\alpha = 5.0, \beta = 1.2 \quad \text{for } FC \geq 35\% \quad (6.10c)$$

Step 4. The resulting $N_{1,60,CS}$ is used with modified 5% or less fines content curve of Seed et al. (1985) to evaluate liquefaction resistance CRR (Figure 6.5).

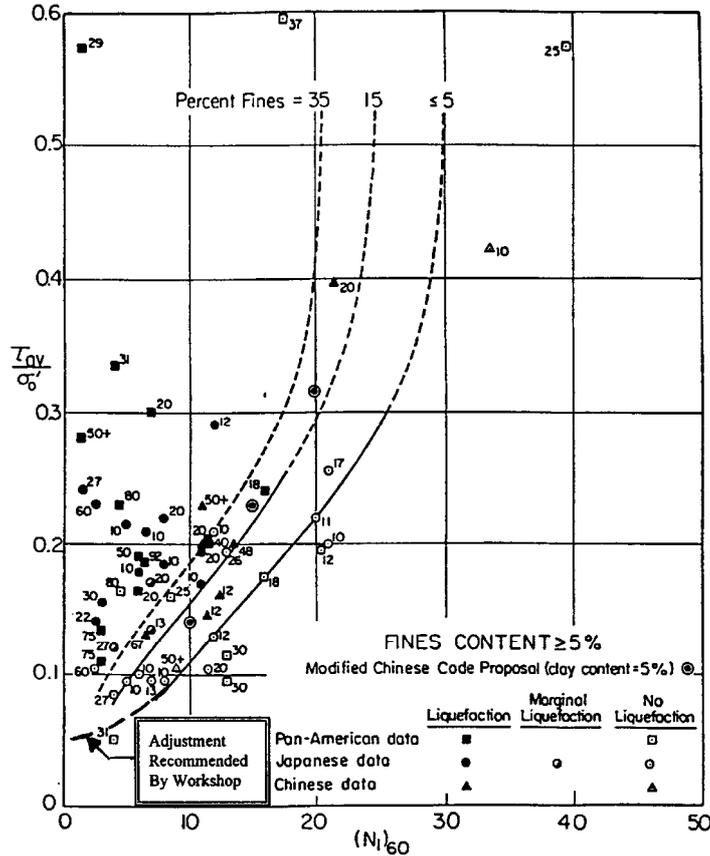


Figure 6.5 Correlation between equivalent uniform cyclic stress ratio and SPT $N_{1,60}$ value for events of magnitude $M_w = 7.5$ for varying fines contents, with adjustment at low cyclic stress ratio as recommended by NCEER Working group (Modified from Seed et al., 1986)

The equation of this curve is approximated by,

$$CRR_{7.5} = \frac{1}{34 - N_{1,60}} + \frac{N_{1,60}}{135} + \frac{50}{(10N_{1,60} + 45)^2} - \frac{1}{200} \quad (6.11)$$

This equation is valid for $N_{1,60} < 30$. For $N_{1,60} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable (Youd et al., 2001).

Step 5. Since the curves in Figure 6.5 are valid only for magnitude 7.5 earthquakes magnitude scaling factor MSF is applied to adjust to the other magnitudes and the factor of safety is calculated from,

$$FS = (CRR_{7.5} / CSR)MSF \quad (6.12)$$

where MSF will be chosen from a range of recommended values (Youd et al., 2001) as,

$$10^{2.24} / M_w^{2.56} (\text{Idriss,1995}) \leq MSF \leq (M_w / 7.5)^{-2.56} (\text{Andrus \& Stokoe,1997}) \quad \text{for } M_w < 7.5 \quad (6.13)$$

$$MSF = 10^{2.24} / M_w^{2.56} (\text{Idriss,1995}) \quad \text{for } M_w > 7.5 \quad (6.14)$$

6.5.3 New Procedures developed by Cetin et al. (2000) and Seed et al. (2001)

The procedure for these new relationships is summarized from Seed et al. (2001).

Step 1. Measured N values are corrected to $N_{l,60}$ values using Eq.(6.15), where C_N is taken (after Liao & Whitman, 1986) as,

$$C_N = \left(\frac{1}{\sigma_v'} \right)^{0.5} \quad (6.15)$$

Table 6.1 summarizes the values of the correction factors that are recommended for use in Eq.(6.16).

Step 2. The resulting $N_{l,60}$ values are then further corrected for fines content to $N_{l,60,CS}$ values using,

$$N_{1,60,CS} = N_{1,60} C_{FINES} \quad (6.16)$$

where C_{FINES} is obtained from a regression relationship as,

$$C_{FINES} = (1 + 0.004FC) + 0.005 \left(\frac{FC}{N_{1,60}} \right), \quad \text{lim: } FC \geq 5\% \text{ and } FC < 35\% \quad (6.17)$$

where FC = percent fines content (percent by dry weight finer than 0.0074 mm), expressed as an integer (e.g. 15% is expressed as 15) and $N_{l,60}$ is in units blows/ft.

Step 3. In-situ equivalent cyclic stress ratio CSR_{eq} is evaluated from Seed and Idriss (1971) using Eq.(6.4) with r_d values estimated from,

$d < 65$ ft:

$$r_d(d, M_w, a_{max}, V_{s,40}^*) = \left[\frac{1 + \frac{-23.013 - 2.949a_{max} + 0.999M_w + 0.016V_{s,40}^*}{16.258 + 0.201e^{0.104(-d+0.0785V_{s,40}^*+24.888)}}}{1 + \frac{-23.013 - 2.949a_{max} + 0.999M_w + 0.016V_{s,40}^*}{16.258 + 0.201e^{0.104(0.0785V_{s,40}^*+24.888)}}} \right] \pm \sigma_{\varepsilon r_d} \quad (6.18a)$$

$d \geq 65$ ft:

$$r_d(d, M_w, a_{max}, V_{s,40}^*) = \left[\frac{1 + \frac{-23.013 - 2.949a_{max} + 0.999M_w + 0.016V_{s,40}^*}{16.258 + 0.201e^{0.104(-65+0.0785V_{s,40}^*+24.888)}}}{1 + \frac{-23.013 - 2.949a_{max} + 0.999M_w + 0.016V_{s,40}^*}{16.258 + 0.201e^{0.104(0.0785V_{s,40}^*+24.888)}}} \right] - 0.0014(d - 65) \pm \sigma_{\varepsilon r_d} \quad (6.18b)$$

where $\sigma_{\varepsilon r_d}(d) = d^{0.850} 0.0072$ for $d < 40$ ft, and

$$\sigma_{\varepsilon r_d}(d) = 40^{0.850} 0.0072 \quad \text{for } d \geq 40 \text{ ft,}$$

and where d = depth in ft

a_{max} = peak ground surface acceleration

M_w = earthquake magnitude

$V_{s,40}$ = site stiffness, ft/sec (average shear wave velocity over the top 40 ft, which will be calculated from 40 ft divided by shear wave travel time in traversing this distance)

Step 4. The obtained CSR_{eq} value is then adjusted to CSR_N by the magnitude-correlated Duration Weighting Factor (DWF_M) using,

$$CSR_N = CSR_{eq,M=7.5} = CSR_{eq} / DWF_M \tag{6.19}$$

where CSR_N is the CSR_{eq} corrected for duration (or number of equivalent cycles), representing the equivalent CSR for a duration typical of an average event of $M_w=7.5$. Figure 6.6 shows DWF_M as a function of $N_{1,60}$ values.

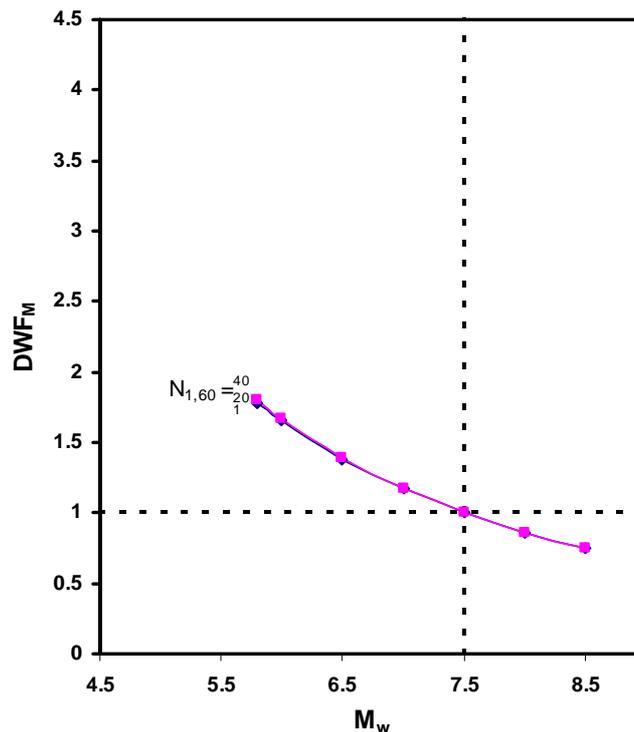


Figure 6.6 Recommended Magnitude-Correlated Duration Weighting Factor as a function of $N_{1,60}$ (after Seed et al., 2001)

Step 5. The new CSR_N is then further adjusted for effective overburden pressure by,

$$CSR^* = CSR_{eq,M=7.5,1atm} = CSR_N / K_\sigma \tag{6.20}$$

where K_σ can be estimated from

$$K_\sigma = (\sigma'_v)^{f-1} \tag{6.21}$$

where $f \approx 0.6-0.8$ (as $N_{1,60,CS}$ varies from 1 to 40 blows/ft). For $\sigma'_v > 2atm$ use of Figure 6.7 is recommended (Seed et al., 2001).

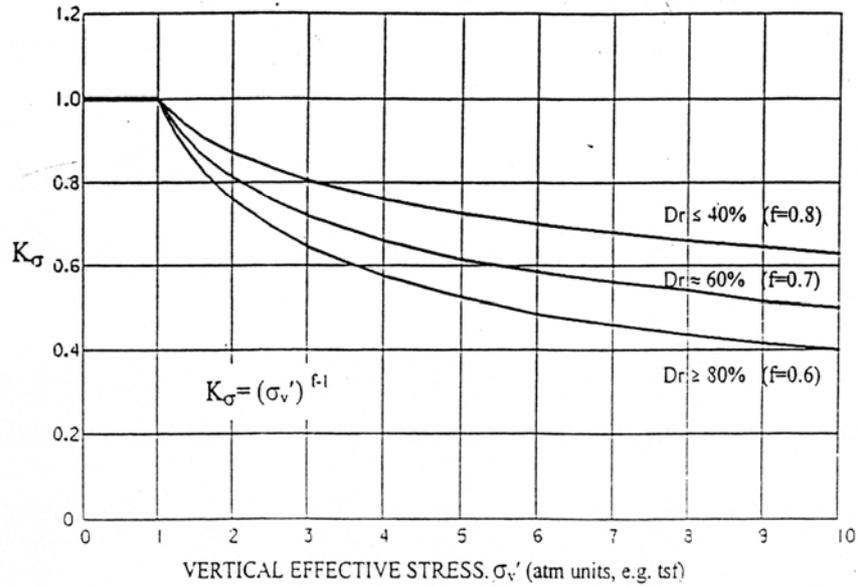


Figure 6.7 Recommended K_{σ} values for $\sigma'_v > 2$ atm (Seed et al., 2001)

Step 6. The resulting fully adjusted and normalized values of $N_{1,60,CS}$ and $CSR_{eq,M=7.5,1atm}$ values are then used with Figure 6.8 to assess probability of initiation of liquefaction (Seed et al., 2001).

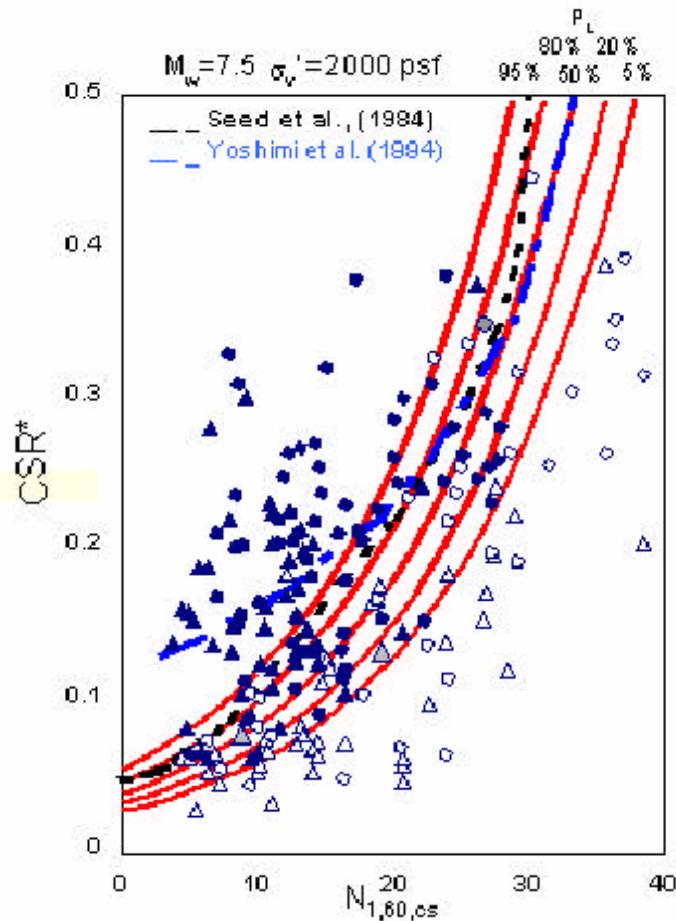


Figure 6.8 Recommended SPT based liquefaction triggering correlation (for $M_w=7.5$ and $\sigma'_v=1$ atm)

6.6. Criteria based on CPT (Cone Penetration Test) data

The procedure below is summarized from (Youd et al., 2001).

6.6.1 Robertson and Wride (1998) Procedure

Step 1. CSR is calculated from Seed and Idriss (1971) as in Revised Simplified Procedure.

Step 2. Tip resistance q_c is normalized to approximately 100 kPa (1 atm) using,

$$q_{c1N} = C_Q(q_c / P_a)$$

where

$$C_Q = (P_a / \sigma'_v)^n \quad (6.23)$$

and where C_Q = normalizing factor for cone penetration resistance,

P_a = 1 atm of pressure in the same units used for σ'_v ,

n = exponent that varies with soil type,

q_c = field cone penetration resistance measured at the tip.

At shallow depths C_Q becomes large because of low overburden pressure, however values >1.7 should not be applied (Youd et al., 2001). The value of exponent n varies from 0.5 to 1.0 depending on the grain characteristics of the soil (Olsen, 1997). Exponent n of 1.0 is the appropriate value for clayey type soils. For clean sands, an exponent value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and sandy silts (Youd et al., 2001).

Step 3. The normalized penetration resistance q_{c1N} is further corrected to $q_{c1N,CS}$ to account for the influence of fines content using,

$$q_{c1N,CS} = K_c q_{c1N} \quad (6.24)$$

where K_c , the correction factor for grain characteristics is defined by (Robertson & Wride, 1998) as,

$$K_c = 1.0 \quad \text{for } I_c \leq 1.64$$

$$K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \quad \text{for } I_c > 1.64 \quad (6.25b)$$

where I_c , the soil behavior type index is calculated from (Robertson & Wride, 1998) as,

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F)^2 \right]^{0.5} \quad (6.26)$$

where

$$Q = \left[(q_c - \sigma_v) / P_a \right] \left[(P_a / \sigma'_v)^n \right] \quad (6.27)$$

and

$$F = [f_s(q_c - \sigma_v)] \times 100\% \quad (6.28)$$

where f_s is the sleeve resistance. Robertson & Wride (1998) recommend the following procedure for calculating soil behavior type index I_c :

- I. If I_c calculated from $Q = \left[(q_c - \sigma_v) / P_a \right] \left[(P_a / \sigma'_v)^n \right]^{1.0}$ is < 2.6 the soil is classified as clayey and considered nonliquefiable and the analysis is complete.

- II. If I_c calculated from $Q = [(q_c - \sigma_v) / P_a] [P_a / \sigma'_v]^{1.0}$ is > 2.6 the soil is classified as granular and C_Q and Q is recalculated using exponent n of 0.5. I_c is then recalculated using Eq.(6.26).
- If recalculated I_c is < 2.6 the soil is classified as nonplastic and granular. This I_c is used in liquefaction analysis.
 - If recalculated I_c is > 2.6 the soil is classified as silty and I_c is once more calculated using an exponent n of 0.7. This I_c is used in liquefaction analysis.

It is recommended that all soils with an I_c of 2.4 or greater should be sampled and tested to confirm the soil type and to test the liquefiability with other criteria (Youd et al., 2001)

Step 4. The resulting $q_{c1N,CS}$ is used with curve developed by Robertson & Wride (1998), to estimate liquefaction resistance CRR (Figure 6.9).

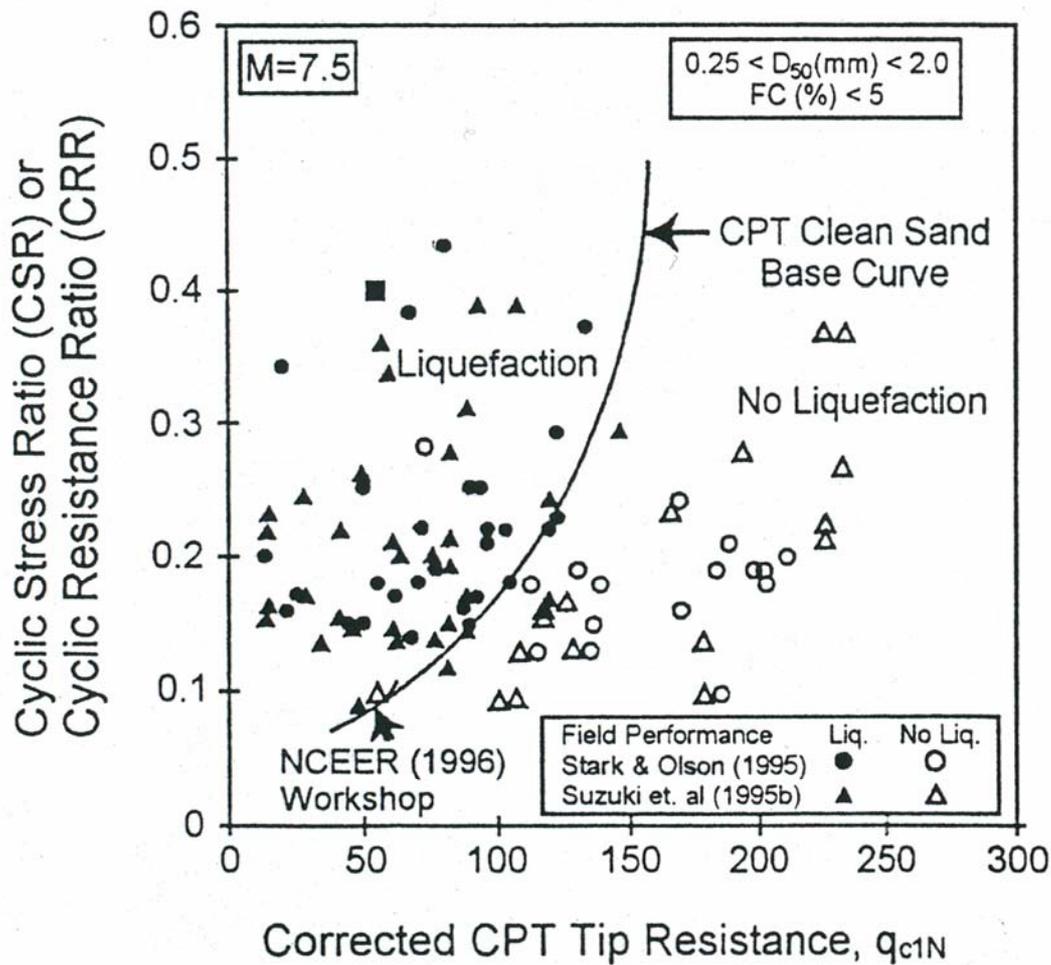


Figure 6.9 Recommended curve for calculating liquefaction resistance from CPT data (after Robertson & Wride, 1998)

The equation of this curve is approximated by,

$$CRR_{7.5} = 0.833 \frac{q_{c1N,CS}}{1000} + 0.05 \quad \text{if } q_{c1N,CS} < 50, \tag{6.29a}$$

$$CRR_{7.5} = 93 \left(\frac{q_{c1N,CS}}{1000} \right)^3 + 0.08 \quad \text{if } 50 \leq q_{c1N,CS} < 160 \tag{6.29b}$$

Step 5. Step 5 of Revised Simplified Procedure is followed to adjust *CRR* to magnitudes other than 7.5 and to calculate the factor of safety against liquefaction.

6.7. Criteria based on V_s (Shear-Wave Velocity) data

The procedure below is summarized from Youd et al (2001).

Andrus and Stokoe (1997, 2000) Procedure

Step 1. CSR is calculated from Seed and Idriss (1971) as in revised simplified procedure.

Step 2. Measured V_s values are corrected to V_{s1} using (Kayen et al., 1992; Robertson et al., 1992),

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_v} \right)^{0.25} \tag{6.30}$$

where V_{s1} = shear wave velocity corrected for overburden-stress
 P_a = 1 atm of pressure in the same units used for σ'_v
 σ'_v = effective vertical stress

Step 3. The resulting V_{s1} is used with curve developed by Andrus & Stokoe (1998), to estimate liquefaction resistance *CRR* (Figure 6.10).

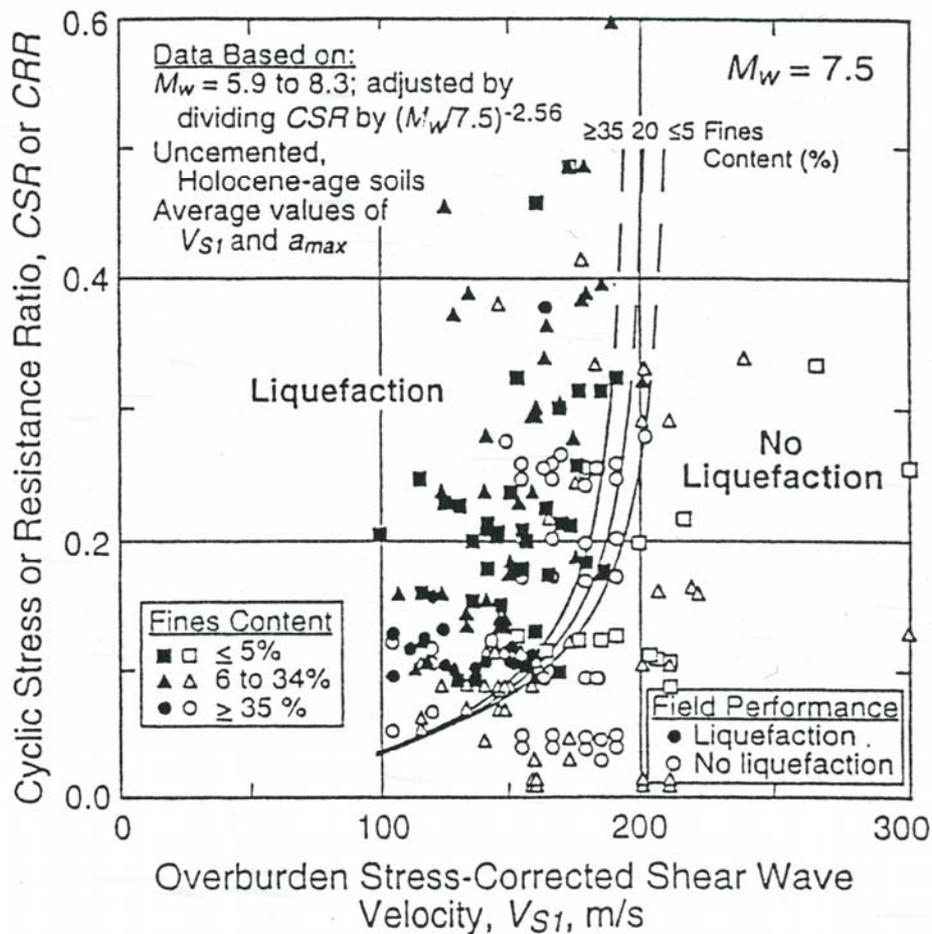


Figure 6.10 Recommended curve for calculating liquefaction resistance from V_s data (after Andrus & Stokoe, 2000)

The equation of this curve is approximated by,

$$CRR_{7.5} = 0.022 \left(\frac{V_{SI}}{100} \right)^2 + 2.8 \left(\frac{1}{V_{SI}^* - V_{SI}} - \frac{1}{V_{SI}^*} \right) \quad (6.31)$$

where V_{SI}^* = limiting upper value of V_{SI} for liquefaction occurrence. Values of V_{SI}^* are assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less.

Step 4. Step 5 of Revised Simplified Procedure is followed to adjust CRR to magnitudes other than 7.5 and to calculate the factor of safety against liquefaction.

6.8. Ground Deformations

6.8.1 Analytical Methods

Several models based on analytical methods are proposed in order to predict the liquefaction induced ground displacements (e.g. Baziar et al., 1992; Jibson, 1994; Towhata et al., 1996, 1997; France et al., 2000; Shamoto et al., 1998). The principle difficulty in using these models is to evaluate numerous parameters required for the analyzes and to assess the sensitivity of these parameters.

6.8.2 Empirical Methods

An alternative approach to analytical methods is the use of empirical methods that are based on statistical analysis of liquefaction-induced ground deformation case histories. Proposed methods include the models by Hamada et al. (1986), Youd & Perkins (1987), Bartlett & Youd (1992) and Bardet et al. (2002). The procedure for each of these models is summarized from Bardet et al. (2002).

Hamada et al. (1986)

The model predicts the horizontal ground displacement in terms of topographic and geotechnical parameters,

$$D = 0.75H^{0.5}\theta^{0.33} \quad (6.32)$$

where D = horizontal displacement (m)
 θ = slope of ground surface or base of liquefying soil (%)
 H = thickness of liquefying soil

Youd & Perkins (1987)

The model predicts maximum horizontal ground failure displacement in terms of seismic parameters,

$$\log LSI = -3.49 - 1.86 \log R + 0.98M \quad (6.33)$$

where LSI = general maximum amplitude of ground failure displacement (inch),
 R = epicentral distance (km)
 M = earthquake moment magnitude.

LSI estimated by this model should not exceed 2.5 meters (Youd & Perkins, 1987).

Bartlett & Youd (1992)

The model predicts the horizontal ground displacement in terms of seismic, topographical, geological and geotechnical parameters using the following relation,

$$\log(D + 0.01) = b_0 + b_{off} + b_1M + b_2 \log(R) + b_3R + b_4 \log(W) + b_5 \log(S) + b_6 \log(T_{15}) + b_7 \log(100 - F_{15}) + b_8 D50_{15} \quad (6.34)$$

where D = horizontal displacement, (m),
 M = earthquake moment magnitude,
 R = epicentral distance, (km),
 S = slope of ground surface, (%),
 W = free-face ratio, (%),
 T_{15} = thickness of saturated cohesionless soils with $N_{L,60} < 15$, (m),
 F_{15} = average fines content, (%),
 $D50_{15}$ = average D_{50} grain size in T_{15} , (mm).

The values of coefficients (b_0 , b_{off} and b_1 to b_8) are given in Table 2. The model is applicable for both free-face cases and ground-slope cases. For free-face cases, b_5 equals to zero and value b_{off} will be taken from Table 3. For sloping ground cases, b_4 and b_{off} will both be taken as zero.

Bardet et al. (2002)

The model predicts the horizontal ground displacement by the following relation,

$$\log(D + 0.01) = b_0 + b_{off} + b_1M + b_2 \log(R) + b_3R + b_4 \log(W) + b_5 \log(S) + b_6 \log(T_{15}) \quad (6.35)$$

where variables are defined as in Eq. (6.34) and the values of coefficients are given in Table 6.2. Similar to Bartlett & Youd (1995) model this model is applicable for both free-face and ground-slope cases ($b_5=0$ for free-face; $b_4=0$ and $b_{off}=0$ for ground-slope).

Table 6.2 Values of coefficients for the Bartlett & Youd (1992) and Bardet et al. (2002) models.

Coefficients	Bartlett & Youd (1995)	Bardet et al. (2002)
b_0	-15.787	-6.815
b_{off}	-0.579	-0.465
b_1	1.178	1.017
b_2	-0.927	-0.278
b_3	-0.013	-0.026
b_4	0.657	0.497
b_5	0.429	0.454
b_6	0.348	0.558
b_7	4.527	-
b_8	-0.922	-
r^2	82.60%	64.25%
Number of data	467	467

When the predicted values from each of the above mentioned models are compared to the measured displacements (data points from Bartlett and Youd (1992) database) following results are observed:

(1). There is a poor agreement between the measured and predicted values when Youd & Perkins, (1987) model is used, (2). Both Bartlett and Youd (1992) and Bardet et al. (2002)

models have scattered predictions within 2:1 and 1:2 slope, where 1:1 slope line indicates perfect prediction. (3). 6 parameter model of Bartlett and Youd (1992) is more accurate than four-parameter Bardet et al. (2002) model. The choice of a particular model depends on the available geotechnical data. Model of Bartlett and Youd (1992) is recommended if information on the grain size distribution of soils is available (Bardet et al., 2002).

6.9. Evaluation of Liquefaction Effects

After the estimation of liquefiable layer, it is necessary to evaluate the effect of liquefaction on structures such as buildings. The following two methods have been proposed and used to evaluate the effect roughly (Manual for Zonation on Seismic Geotechnical Hazards, 1999).

(1) Method proposed by Ishihara (1985)

To decide whether will or will not exert damage on the ground surface, the thickness of the liquefiable layer can be compared with the thickness of the surface crust using criteria such as that shown in Figure 6.11. If the thickness of the surface layer, H_1 , is larger than that of the underlying liquefiable layer, resulting damage on the ground surface may be insignificant. If the water table is below the ground surface, the definition of H_l depends on the nature of the superficial deposits, as shown in Figure 6.12. For a deposit of sandy soils, the thickness H_l can be taken to be equal to the depth of the water table.

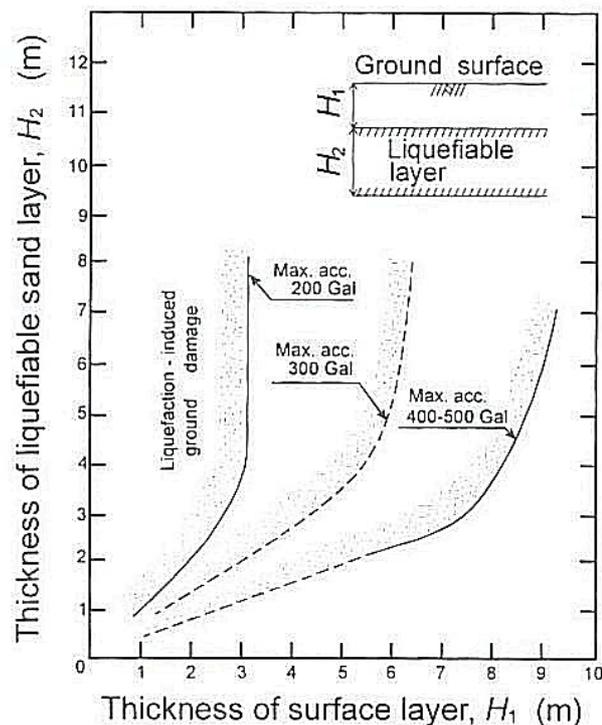


Figure 6.11 Proposed boundary curves for surface manifestation of liquefaction-induced damage (Ishihara, 1985)

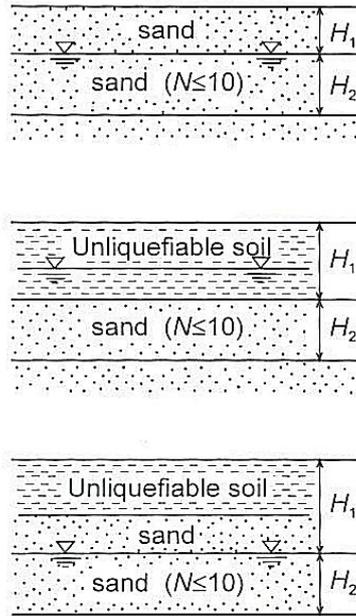


Figure 6.12 Definitions of the surface unliquefiable layer and the underlying l liquefiable sand layer (Ishihara, 1985)

(2) Method proposed by Iwasaki et al. (1982).

Iwasaki et al. (1982) quantified the severity of possible liquefaction at any site by introducing a factor called the liquefaction potential index, P_L , defined as

$$P_L = \int F(z)w(z)dz \tag{6.36}$$

where z is the below the ground water surface, measured in meters; $F(z)$ is a function of the liquefaction resistance factor, F_L , where $F(z)=1- F_L$ but if $F_L>1.0$, $F(z)=0$; and $w(z)=10-0.5z$. Eq.(6.36) gives values of P_L ranging from 0 to 100. By calculating this index for 63 liquefied and 22 noliqefied sites in Japan, Iwasaki et al. concluded that sited with P_L values greater than about 15 suffer severe liquefaction effects whereas effects are minor at sites with a value of P_L less than about 5 (ISSMFE/TC4, 1999).

7. ZONATION WITH RESPECT TO LANDSLIDE HAZARD

Large numbers of landslides that have taken place during recent earthquakes have demonstrated that instability of natural and man-made slopes is one of the major causes of damage. In order to estimate susceptible zones of slope failures during earthquakes, various zonation methods were developed in the literature. These methods may be considered in three levels as Grade 1, 2 and 3 depending on the comprehensiveness of the approach (TC4 Committee of ISSMGE, 1997).

Grade-1 Methods were developed based on number of landslides observed in earthquakes with respect to earthquake magnitude and epicenter or fault distance. Due to approximate nature of these methods, geotechnical and topographical conditions were not taken into account. The methods proposed by Tamura (1978), Yasuda and Sugitani (1988), Keefer et al. (1978), and Ishihara and Nakamura (1987) are some of the procedures in this category.

Grade-2 Methods requires more detailed evaluation since geological characteristics as well as the topographical aspects are taken into consideration along with earthquake magnitude and epicentral distance. Methods adopted by Kanagawa Prefectural Government (1986) and developed by Mora and Vahrson (1991) are the two alternatives in this category.

Grade-3 Methods are the most comprehensive microzonation procedures to identify areas with different degrees of susceptibility of slope instabilities. In this category the slope failure potential is evaluated using a suitable analysis scheme that requires geotechnical properties of soil and rock layers, slope geometry as well as peak ground acceleration representing the effects of earthquake.

7.1. Landslides during earthquakes

Earthquakes are known to be one of the potential sources for slope instability and landslide hazards. Earthquake induced landslides have been extensively documented in the literature (Okimura, 1995; Sassa et al. 1996; Harp & Jibson, 1996; Jibson & Crone, 2001; Esposito et al., 2000; Keefer, 1984; Ishihara, 1985; Mizuta & Seo, 2000 (Figure 7.1)).

Some of these studies were conducted based on air photos complemented with field observations such as Okimura (1995) and Sassa et al. (1996), while some are more related to specific landslides observed during earthquakes. Huang et al. (2001) based on the observations of two massive rockslides caused by the earthquake investigated the effects of surface-normal acceleration (currently not a parameter used in the analyzes) on the initiation of the landslides and concluded that including the effects of seismic accelerations normal to the slide surface doubles the displacements estimated from the Newmark's (1965) sliding block model. Jibson & Crone (2001) gave a detailed account of the landslides triggered by the earthquake and the related geological conditions.

A lot of studies have been reported on the influence of earthquakes on different ground conditions and the stability of slopes by means of physical modeling. Focus was given in the earlier studies to the observation of the overall behavior of dams and slopes under different kind of earthquake conditions as well as on differences in the subsoil. As an example for these studies the work of Habibian (1987) should be mentioned, who studied the influence of inhomogeneous subsoil conditions below a dam in terms of the likelihood of cracking and loss of stability. Another more recent study (Taboada et al. 1998) deals with the effect of frequency of the input shaking movement on slopes. The behavior of non-liquefiable soil layers has been studied for example by Yu & Lee (2002), who focused on the seismic response of soft clay strata.

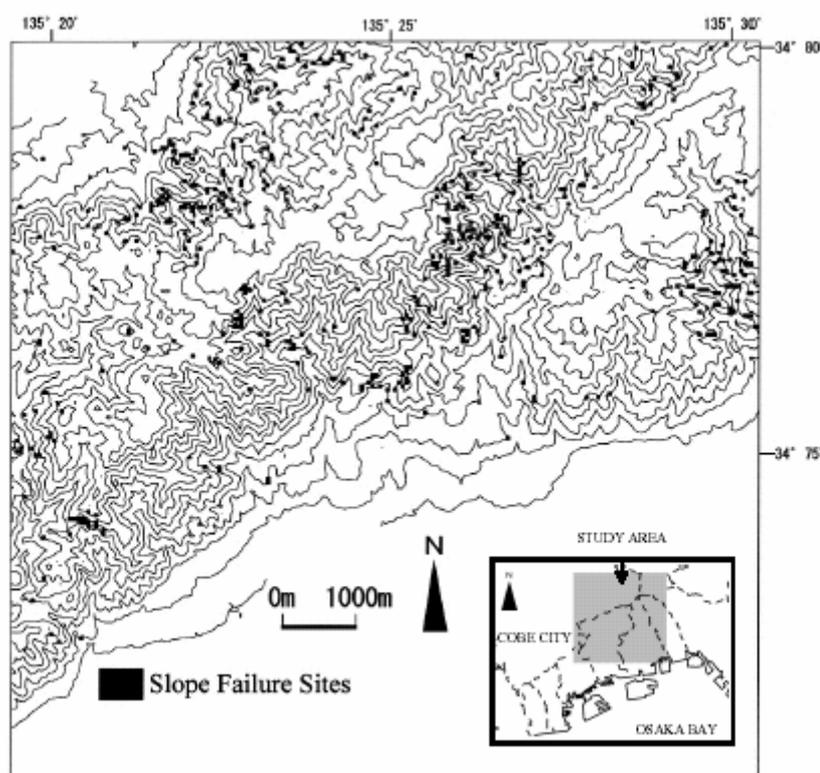


Figure 7.1 Distribution of slope failures in the study area during the 1995 Hyogoken-Nanbu Earthquake and topographic map (50m interval) using 50-m grid DEM (After Mizuta & Seo, 2000)

Most of the studies deal with the combined effect of liquefaction (compare Chapter 6.1) and slopes. For the description of the behavior of slopes during an earthquake, the most recent studies should be mentioned here. The behavior of slopes under earthquake conditions has been discussed by Nagase et al. (1994) for slopes up to 10° or Madabhushi et al. (2002) for much steeper slopes from 30° to 45° . The applicability of these test results have been studied for example on the case of the loss of the ancient cities of Sodom and Gomorrah, which were built on a slope with an angle of 3° , including two small silt layers (Haigh & Madabhushi 2002a). The additional influence of seepage on slopes or embankments have been analyzed by Pilgrim and Zeng (1994) and Hayashi et al. (2002). The effect of the combination of liquefiable soils, different layers and slopes has been reported by Imamura et al. (2002a).

7.2. Slope Stability Analyzes

Zonation for seismically induced landslides, as zonation for other hazards, mainly aims to support the settlement planning activities and to reduce possible casualties and property losses due to landslides. Slope instability hazard zonation is defined as the mapping of areas with an equal probability of occurrence within a specified period of time (Varnes, 1984). The degree of accuracy in landslide hazard zonation depends on many interrelated factors:

1. *Scale*; the scale of the study to be conducted is determined by the purpose for which the maps are executed and should be in conformity with the quality and availability of data and the methodology to be used.

- a. Regional scale studies: (1:1,000,000 – 1:50,000) this type of studies mainly aims to help the regional planners by identifying large landslide problem areas. These areas can later be subjected to more detailed studies.
 - b. Medium scale studies: (1:100,000 – 1:10,000) these studies can be used for regional planning, for local engineering works, for planning of infrastructures, housing and industrial projects.
 - c. Detailed scale studies: (> 1:5,000) these studies are mainly done for companies or municipal agencies dealing with hazards of individual sites.
2. *Data availability*; there are five main sources of information: literature collection, existing maps, remote sensing data, aerial photographs and laboratory test results.
 3. *Methodologies used*: three basic approaches exist for conducting seismic landslide hazard analysis. These consist of statistical, pseudo-static, and permanent-displacement approaches. These approaches will be elaborated in detail in the following sections.

7.2.1 Statistical Approaches

Statistical approaches assess the hazard by assuming that the past predicts the future. Hazard is assessed through correlation of past landslides with several influential factors. Results of such analyzes can range from an estimated probability of failure to some index indicating degrees of hazard.

The statistical approaches with minimum data requirement are the ones that try to establish a simple relationship between the magnitude or intensity of earthquakes and the distances at which various possibilities of landslides will occur (Tamura, 1978; Ishihara & Nakamura, 1987; Yasuda & Sugitani, 1988; Keefer & Wilson, 1989; Papadopoulos & Plessa, 2000; Prestininzi & Romeo, 2000).

Due to the lack of information on material properties, the studies mentioned above can only be used for most general purposes and are not adequate for site specific evaluations nor for seismic microzonation purposes. However, with addition of some information on local geology, soil and ground water level conditions, the degree of accuracy of the hazard assessment may be considerably improved. These kinds of information may be obtained from published topographic, geologic and hydrologic maps. The following approaches use these data in defining the hazard zones.

The method proposed Kanagawa Prefectural Government (1986), is based on slope failures during three large recent earthquakes in Japan where seven factors were identified as factors governing the slope instability:

- a) maximum surface acceleration
- b) length of the contour line at the average elevation
- c) maximum difference in elevation
- d) hardness of rock in a typical slope
- e) total length of faults
- f) total length of artificial cut or filled slopes
- g) topography of typical slopes in each mesh

A weighting for each factor is evaluated in the study and by summing up the weighted factors, a total susceptibility factor (W) is determined for each mesh. Based on the overall susceptibility W , the number of likely slope failures in each mesh is estimated. Maps of 1:50,000 and 1:25,000 scale and mesh sizes of 500m x 500m were used in this study.

Mora, S. & Vahrson, W. (1993) identified three factors, relative relief, lithologic conditions and soil moisture as factors affecting susceptibility based on case studies of slope failures

induced by earthquakes and also by heavy rainfall in Central America. Seismic intensity and rainfall intensity are identified as the triggering factors. By combining these factors, a landslide hazard index and consequently a degree of slope failure hazard is determined. The obtained degree of susceptibility ranges from negligible to very high.

Keefer, D.K. (2000) conducted a statistical analysis of landslides triggered by the 1989 Loma Prieta earthquake and analyzed the dependence of landslide concentration (number of landslides per square km) on distance from epicenter, slope steepness, lithology and degree of induration.

Some Case Studies

Coe et al. (2000) conducted statistical analysis of historical landslides where the slopes shown on the map were computed using elevations from USGS 10 m (100 m² cell area) digital elevation models (DEMs). A slope value for each elevation cell was derived based on the maximum change in elevation from surrounding cells. Roads and railroads were extracted from the USGS 1:24,000 scale digital line graph (dlg) database. Landslide densities shown on the map were determined based on the number of landslides occurring within a moving count circle (see Campbell, 1973). Count-circle software developed by the authors was used to determine landslide densities as follows. First, the City of Seattle was digitally overlain with a grid of 25 x 25 m (625 m²) cells. Next, a count circle covering an area of 40,000 m² (4 hectares, equivalent in area to a 200 x 200 m cell) was digitally placed at the center of each cell and the number of landslides occurring within the circle were counted. The landslide density or number of landslides within each 4 hectare count circle, was then assigned to the 625 m² cell at the center of each circle. The grid of 625 m² cells was then saved for later contouring and calculation of mean recurrence intervals and exceedance probabilities. The 625 m² cell size was used because it is roughly equivalent in area to an average sized city lot. The 4 hectare count circle was used because it is roughly equivalent in area to the largest landslides that have occurred in the Seattle area.

Wasowski & Del Gaudio (2000) developed landside hazard assessment methodology based on a statistical analysis of historic and pre-historic landslides and the regional seismicity.

Parise & Wasowski (1999) produced landslide activity maps based on historical data on slope instability combined with aerial photographs for three selected sites in Italy. These maps represent a short-cut in the assessment of mass movement hazard, because they focus on the effects of slope instability rather than on the causative conditions and processes. The quantification of landslide activity in terms of aerial frequency is a useful step towards landslide hazard zonation.

Parise & Jibson (2000) have proposed a susceptibility rating to be conducted for various geological units based on the statistical analysis of landslides triggered by the 1994 Northridge earthquake where landslide morphologies are described with the help of various parameters (area, length, width, aspect ratio, slope angle). The relative susceptibility of each geologic unit to seismic landslide is ranked with the help of two parameters: 1) Susceptibility index (ratio of the area covered by landslide sources within a geologic unit to the total outcrop area of the unit), 2) frequency index (number of landslides per square km of that unit).

7.2.2 Pseudo-Static Approaches

The aim of the analysis in pseudo-static conditions is the evaluation of the factor of safety (Fs) and the coefficient of critical acceleration for landslides and landslide free areas. The critical acceleration is defined as that acceleration which when applied to the slope produces a state of incipient failure. This acceleration is assumed to be constant over the slope as if the slope is a rigid body and usually, the term refers the horizontal component of the acceleration. The

factor of safety, on the other hand, is defined as the factor by how much the available strength should be reduced so that a state of incipient failure is reached. The state of incipient failure is termed the limiting equilibrium condition. Therefore the two terms, the critical acceleration and the factor of safety are both representative of the available strength in some sense. The critical acceleration related to the load factor while the factor of safety related to the strength factor. In his landmark paper, Newmark (1965) proposed modeling a slope subjected to earthquake-induced landslides as a friction block resting on an inclined plane subjected to the same accelerations as the modelled slope. Therefore, in each instance when the sum of the static and dynamic forces exceeds the shear resistance of the sliding interface the block will displace. Newmark defined the following relationship to calculate the critical acceleration in case of planar slip:

$$a_c = (FS - 1) \sin \alpha \quad (7.1)$$

where FS is the static factor of safety of the slope and α is the thrust angle of the landslide block which can be approximated by the slope angle.

By combining data from geotechnical surveys with data mentioned in studies above, and by using pseudo-static approaches a much finer slope stability zonation can be obtained. The results obtained are on a site specific basis with scales up to 1:5,000.

A second approach was proposed by Wilson et al., (1979) by equating sliding and resisting forces in a thin sliding layer, the following equation is obtained:

$$a_c = g \left[\frac{c}{\gamma h} + (\cos \theta \tan \phi - \sin \theta) \right] \quad (7.2)$$

where a_c : critical acceleration inducing the slide,

- g : acceleration of gravity
- c : cohesion of soil
- ϕ : internal friction angle of the layer
- γ : unit weight of the soil
- θ : angle of slope
- h : thickness of sliding layer.

When a geological map, the above mentioned soil properties for each formation and a slope map are available for the region to be studied, by the superposition of the maps in a GIS environment critical accelerations can be calculated and they can also be superposed with an expected earthquake acceleration map to identify the areas prone to seismically induced landslides.

Siyahi & Ansal (1999) proposed a method that defines the factor of safety in terms of the shear strength angle ϕ and the stability number N_l such that:

$$Fs = \tan \phi N_l \quad (7.3)$$

N_l , representing the configuration of the slope and failure surface (Figure 7.2). The variation of minimum N_l is represented as a function of slope angle and the earthquake acceleration. Thus, if the topography (slope angle), the shear strength angle of the soil, and the expected earthquake accelerations are known for each mesh, the resulting factor of safety can be calculated, a factor of safety less than 1, between 1 and 1.5, and larger than 1.5 indicating high, moderate and low risk respectively.

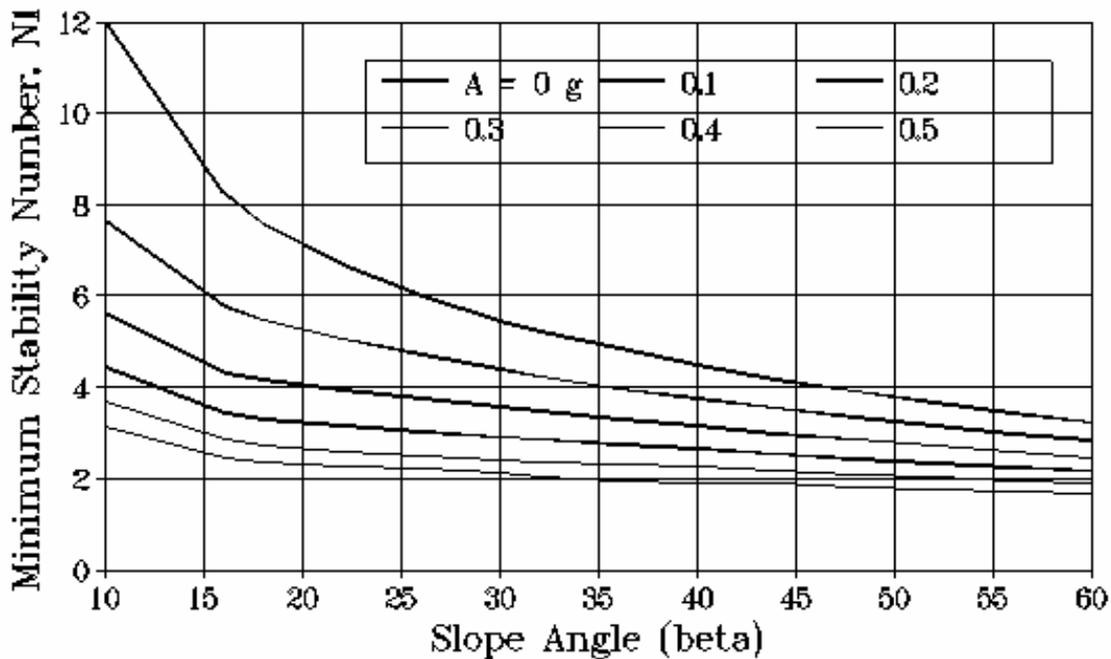


Figure 7.2 Variation of Stability number with respect to slope angle and peak ground acceleration

Del Gaudio et al. (2000) used pseudo-static approach to identify the effects of low magnitude shocks repeated at short intervals on the Acquara-Vadoncello landslide in 1995. The study points out the complexity of interaction between different factors affecting the slope stability in a seismic area where the effects of the seismic shocks could be very different during the evolution of the landslide both due the different shaking characteristics and as a consequence of the physical-geometric modification of the slope. This can change the vulnerability of the slope and the triggering threshold of the sliding phenomena thus it is necessity to consider all these factors and their possible inter-relationship when attempting to predict mass movement in a similar physical environment.

Nagarajan (2002) has developed a procedure for the rapid assessment of landslide susceptibility induced by earthquake and/or rainfall, taking into account the potential liquefaction of the soil. The procedure is then evaluated for a part of northeast India vulnerable to earthquake and the uncertainties of a monsoon climate. It is suggested that this procedure could be used in other tropical/monsoon climatic regions which experience seismic events or are situated on a plate boundary. The procedure defines a total landslide susceptibility hazard index as the sum of terrain vulnerability, rainfall and earthquake indexes, which are in their turn defined in terms of several parameters in connection with the land cover, surface drainage, morphology, slope material, rock type, lineament/joint discontinuities, geological structure, hydrology, rainfall and seismicity conditions. Although not demonstrated in the present paper, the authors indicate that the method could also be used in conjunction with the geographical information system (GIS).

Luzi et al. (2000) calculated the susceptibility of slopes to failure during earthquakes, in terms of critical horizontal acceleration, on a subregional scale (1:10 000) for the upper part of the Serchio River basin (Tuscany, Italy). The infinite slope analysis is chosen as the appropriate method, but particular attention was devoted to the error evaluation due to spatial variability of the geotechnical, geometric, and hydrologic parameters. Geologic, geomorphologic and hydrologic data are obtained from field surveys and geotechnical parameters are collected at local administrations. A Monte Carlo procedure and a first-order second-moment method

were applied and compared as error estimators in assessing the slope susceptibility to failure and differences between the two methods are discussed, two maps showing, respectively, the critical horizontal acceleration and the probability of failure associated with each slope are also provided.

Mizuta & Seo (2000) included the ground water level in the calculation of the factor of safety in the infinite slope model. Two case studies (based on the observations of the 1995 Kobe earthquake) are conducted to indicate the change in the factor of safety in cases of earthquake occurrence before and after a rainfall.

7.2.3 Permanent Displacement Approaches

The field of earthquake-induced landslide hazard mapping has matured to the point where a well-accepted framework exists for conducting zonation investigations. This framework is founded on Newmark's displacement method for a sliding block (Newmark, 1965). When the applied ground acceleration is bigger than the critical, the factor of safety becomes less than one temporarily and the mass slides downhill. Since the accelerations last for a very short duration, the motion will stop after some time. The safety of the slope is then assessed in terms of this displacement. Newmark suggested the use of the equivalent sliding block model to determine the displacement of the sliding mass. In this model, the mass of the sliding body (moving on a curved surface) is placed on an equivalent plane surface so that the same critical acceleration is obtained. Then assuming that there is no change in the critical acceleration during the movement, the displacement is computed by double integrating the equation of motion of the sliding block. Newmark's model was extended by Sarma (1975) to include the effect of cyclic pore water pressure. Wilson and Keefer (1983) showed that using Newmark's method to model the dynamic behavior of landslides on natural slopes yields reasonable and useful results. Wieczorek and others (1985) subsequently produced an experimental map showing seismic landslide susceptibility in San Mateo County, California, using classification criteria based on Newmark's method. Wilson and Keefer (1985) also used Newmark's method as a basis for a broad regional assessment of seismic slope stability in the Los Angeles, California, area. However, predicted Newmark displacements do not necessarily correspond directly to measurable slope movements in the field; rather, modeled displacements provide an index to correlate with field performance. For the Newmark method to be useful in a predictive sense, modelled displacements must be quantitatively correlated with field performance (Jibson et al., 1998, 2000).

Some case studies

Jibson et al., (1998) assessed the dynamic performance of slopes using the permanent-displacement analysis developed by Newmark (1965). The authors identify the Northridge earthquake as the first earthquake for which all the data sets needed to conduct a detailed regional analysis of factors related to triggered landsliding are available. These data sets include (1) a comprehensive inventory of triggered landslides (Harp and Jibson, 1996), (2) about 200 strong-motion records of the mainshock recorded throughout the region of landsliding, (3) detailed (1:24,000-scale) geologic mapping of the region, (4) extensive data on engineering properties of geologic units, and (5) high-resolution digital elevation models of the topography. A relationship is defined between Arias Intensity and Newmark displacements. A comparison is made between predicted Newmark displacements and the actual inventory of landslides triggered by the Northridge earthquake to find out whether larger predicted displacements relate to greater incidence of slope failure. The paper provides an equation to predict probability of slope failure as a function of predicted Newmark displacement.

In contrast to the studies trying to establish a simplified relationship between Newmark displacement and earthquake parameters such as Arias Intensity or PGA, Miles & Ho (1999) applied the full Newmark method and calculated the displacements using acceleration time histories. The ideal input ground motion for regional instability zonation using Newmark's displacement method is an acceleration time history recorded during an event that occurred on the fault of interest, having a magnitude of adequate size. As obtaining such records would be difficult or even in some cases impossible, artificial acceleration time stories are used. A prototype vector based GIS, applying rigorous Newmark analysis on simulated accelerograms is developed. The results are compared with those obtained from simplified approaches.

Miles, et al. (2000) compared four permanent displacement models based on Newmark's sliding-block analogy for assessing regional seismic slope-performance. The models vary primarily by the ground motion descriptor used to correlate with Newmark displacement. The first uses peak ground-acceleration (PGA). The second uses PGA but normalizes displacements by predominant period and equivalent cycles. The third uses Arias intensity. The fourth calculates cumulative displacements from double-integrating simulated earthquake accelerograms. The models are implemented in a GIS to characterize seismic slope-performance for the Oakland East quadrangle near San Francisco, California. The resulting slope-performance maps are compared visually and through statistical analysis to expose potential differences and assess the effects of using a particular approach within a decision-making context. These maps were created for the purpose of comparison and are not suitable for use as critical decision-making tools. The models forecast notably different levels of slope-performance, with the PGA-based models predicting the greatest Newmark displacement on average. Thus, considering the variety of slope-performance models, it is suggested that practitioners avoid reliance on a single model. Instead, multiple models can be implemented in a GIS framework to gain a better perspective of the potential hazard and make a more informed decision.

Miles et al. (1999) established the double integration method of simulated accelerograms in classical Newmark's displacement method mentioned as the fourth model in the paper above. In extending Newmark's method to regional analysis, simplified approaches are commonly employed to avoid the rigors associated with conventional Newmark's analysis (e.g. earthquake record selection and computer processing time). With the flexibility of current geographic information systems and increasing computer processing speeds, it is feasible to implement a conventional Newmark analysis at regional scales, thus retaining the method's flexibility in characterizing ground-motions. Conventional Newmark's can be implemented using real or artificial time-histories. A popular stochastic time-history simulation that incorporates physical parameters of the source and site, which facilitates application to a variety of seismic zones, is well-suited for regional analysis. Results of conventional Newmark's exhibit similar areal patterns as those from three simplified Newmark models. The first model incorporates Arias intensity to characterize ground-motion. The others use peak ground-acceleration (PGA). Each of the four models, conventional and simplified, produce significantly different magnitudes of Newmark displacement with the simplified models predicting generally greater Newmark displacements (i.e., more conservative). Although moderately conservative, the areal distribution predicted by the Arias-based model is most comparable to that from the conventional analysis. Arias intensity correlates well with earthquake-induced landslide potential. The PGA-based models are highly over-conservative and overly sensitive to PGA attenuation and should be used with caution. The difference among the conventional and simplified models with respect to input, processing time, and results suggests different roles for each. The simplified models are best suited for very large areas or for applications such as Monte Carlo simulation or probabilistic hazard analysis.

Conventional Newmark's method can be applied to moderate-sized areas and is well suited to use of actual earthquake records or ground-motion characterization using time-history simulations.

Romeo (2000) used Newmark's model for predicting earthquake induced landslide displacements to predict the coseismic displacements affecting a sliding mass subject to earthquake loading. In this model, seismic slope stability is measured in terms of critical acceleration, which depends on the mechanical soil properties, pore-pressure distribution, and slope geometry. The triggering seismic forces are investigated in terms of energy radiation from the source, propagation, and site effects, based on recorded accelerations. The method is based on the calibration of relations having the general form of an attenuation law that relates the energy of the seismic forces to the dynamic shear resistances of the sliding mass to propagate the expected landslide displacements as an inverse function of the distance from the fault rupture; the amount of displacement computed through these relations provides a criterion to predict the occurrence of slope failures. Finally, maps showing, in a deterministic and a probabilistic way, the potential of seismically induced landslide displacements are displayed as a tool to provide seismic landslide scenarios and earthquake-induced landslide hazard maps, respectively.

Luzi & Pergalani (2000) proposed two empirical relations based on Italian data that relates the displacement evaluated by the application of Newmark's model to several landslide samples, to Arias Intensity, and the other to Spectral Intensity.

Luzi & Pergalani (1996) evaluated the vulnerability of landslides in static, pseudo-static and dynamic conditions are evaluated and slope instability zonation maps are produced for the Fabriano region. For the dynamic analysis the Newmark's model is applied and two recorded accelerograms considered as representative of the study area are used as inputs to the Newmark's displacement analysis.

Biondi et al., (2000) analyzed the effects of pore water pressure based on Newmark's sliding block analysis. In cohesionless saturated slopes the seismic loading may produce an increase in pore pressures, which in turn, reduces significantly the effective state of stress. Depending on the seismic intensity and on the effective stress conditions before the seismic loading, the induced excess pore pressures may produce the slope failure. Slope displacements may occur even for seismic accelerations lower than the initial critical acceleration, because of pore pressure build-up. The paper describes a simplified procedure for the seismic response of infinite slopes, and points out the role played by relevant parameters such as soil relative density and initial pore pressure ratio.

Silva et al., (2000) extended the Newmark analysis to saturated slope conditions, constituting an important step towards a more comprehensive hazard zone mapping considering earthquake occurrences in wet seasons.

Some comprehensive studies on landslide hazard zonation do not specifically concentrate on the triggering mechanisms such as earthquakes, but the ideas could anyhow be used for seismic landslide hazard zonation (Carrara et al., 1991; van Westen et al., 1997; Aleotti & Chowdhury, 1999)

As described above, several methodologies exist for the assessment of seismic landslide hazard and for the zonation of areas with similar susceptibility levels. Taking into account the necessary scale, the availability and accessibility of required data and the effort required for the computations, the researcher or the engineer can choose an adequate methodology to assess the landslide hazards in a given area. The development of computer tools especially of the GIS technologies have considerably reduced the computing efforts and allowed the easy

usage of more complicated methods. However Carrara et al. (1999) draw attention to the following points: Computer generated maps are considered to be more accurate and credible than similar hand-drafted maps. Results obtained from systematic manipulation of data through GIS based programs are assumed to be more objective, and data manipulation can also be done by inexpert people. Carrera et al. state that these assumptions may lead to some errors, such as random or systematic errors introduced during digitization of data or the usage of the GIS tools by people who are not experts in the hazard assessment and in the specific hazardous process.

The majority of methodologies described in this study concentrate on the spatial distribution of the landslide hazard. An ideal map of slope instability hazard however should provide information on the spatial distribution as well as the temporal conditions (Varnes, 1984). The time concept could be expressed in terms of probabilities of occurrence in a given time period. For the studies considering the seismically induced landslides this is rather easy since the probability of occurrence of the triggering earthquakes could be related with the probability of occurrence of landslides for a given time period.

8. SYNTHESIS OF THE FINDINGS FOR LAND USE MANAGEMENT

8.1. Identification of different zones

It is possible as done in large number of seismic microzonation studies to delineate different zones based on different parameters such as geologic units, site classifications as given in earthquake codes, average shear wave velocity, peak ground acceleration, spectral acceleration, Arias intensity, Modified Mercalli Intensity and many others. However, as pointed out by Finn (1991) the important issue that should control the selection of the microzonation parameter is the purpose or the intended use of the seismic microzonation maps produced as the end product. One possibility is to use the seismic microzonation maps to replace the small scale seismic hazard maps that are used in conjunction with the national earthquake codes. In this case the zonation parameters should be in accordance with the code design procedures such as design spectrum. However, the applicability of this modification would be limited to the area investigated and in the remaining part of the region national seismic hazard maps need to be valid. There is a possibility that this may create confusion in the design process especially in the border regions.

The main purpose of seismic microzonation is its interpretation for long term city planning. Taking into consideration the seismic microzonation with respect to ground shaking intensity, liquefaction susceptibility and landslide hazard, the city plans can be revised to change the population and building densities and characteristics. In most cases seismic microzonation studies, as those summarised in the next section, are conducted for earthquake prone cities with significant history and large number of building stock. It would be very difficult and unrealistic to expect short term impact on the city planning. However, with the additional study on the vulnerability of the building stock and with earthquake scenario studies, short to midterm plans can be made to determine the priorities for strengthening the building stock based on their importance and to make post earthquake preparations for mitigation of the earthquake effects (Bendimeraad, 2002; Hays et al., 1998; King et al., 1997).

8.2. Recommendations for the individual zones

As suggested by Cardona & Yamin (1997) after the identification of different zones depending on the microzonation studies, design spectra could be developed for analyzes and design of structures to be adopted in the earthquake code.

8.3. Recent studies on Soil Structure Interaction during earthquakes and the effectiveness of countermeasures mainly against liquefaction

8.3.1 Advantages of physical modeling procedures for the study of the interaction of soil structure interaction problems during earthquakes.

Field observations very seldom allow systematic studies of soil structure interaction problems under earthquake shaking conditions. Thus investigations by means of physical modeling at the correct stress levels are the only possibility to conduct the necessary studies under defined boundary conditions. For all physical modeling the fulfilment of the correct boundary conditions is crucial. The stress state in the subsoil as well as input motions and load conditions must be appropriately represented.

There are only two possibilities of simulating the stress-state in the subsoil correctly by physical model tests. A full-scale test to be placed on a shaking table needs, in general, at least a height of several metres of the soil strata, as well as a significant plan area, to be able to represent the real stress states. Those tests also require huge shaking tables to apply the correct loading. An easier solution is to use the advantages of geotechnical centrifuges, where

a small-scale model can be subjected to a higher acceleration, which allows building up the stress state in the soil to the same values than in the prototype situations. This interaction is shown in Figure 8.1 in a comparison with 1g model tests (small scale models in the laboratory).

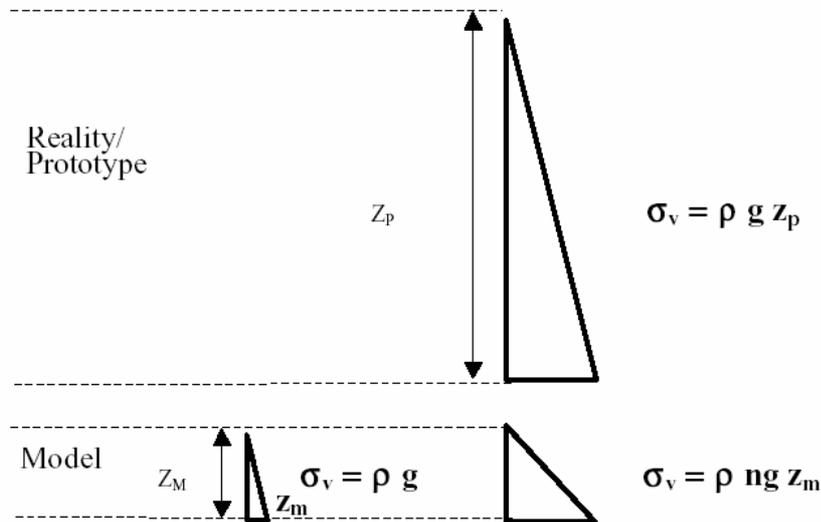


Figure 8.1: Vertical stresses (σ_v) in a soil model (index m) comparing reality (index p) (top) with 1 g small-scale model (bottom left) and centrifuge model (bottom right) (Laue 2002)

Based on the effect of increased gravity on the soil strata in a small scale model, which is scaled down by a factor of n and tested in an n -times increased level of gravity, scaling laws can be derived (Table 8.1 and 8.2).

Table 8.1. Scaling law relationship for time effects (based on Schofield 1980)

Parameter	Unit	Scale (model/prototype)
Time: diffusion	s	n^2
Time: dynamic	s	n
Frequency	1/s	n

Table 8.2. Scaling law relationships (based on Schofield 1980) following $\sigma_v = \rho_m (ng)_m (z/n)_m = \rho_p g_p Z_p$, where subscripts m & p refer to model and prototype

Parameter	Unit	Scale (model/prototype)
Acceleration	m/s^2	n
Linear dimension	m	$1/n$
Stress	kPa	1
Strain	-	1
Density	kg/m^3	1
Mass or Volume	kg or m^3	$1/n^3$
Unit weight	N/m^3	n
Force	N	$1/n^2$
Bending Moment	Nm	$1/n^3$
Bending moment /unit width	Nm/m	$1/n^2$
Flexural Stiffness/ unit width	Nm^2/m	$1/n^3$

As well as the possibilities of simulating the stress state of a prototype situation correctly, the main advantage of using a geotechnical centrifuge is the scaling of time for problems that include consolidation (diffusion) processes. However, for some problems like e.g. the study of

earthquakes, the scaling of time is crucial since different processes scale by different amounts, as shown in Table 2. In these cases, the quicker scaling of the diffusion processes conflicts with the time scaling of the application of load (load frequency or velocity). Thus measures must be taken to slow down the flow in the diffusion process – for example to increase the viscosity of the fluid by, for instance, adding glycerine or other fluids, or to reduce the permeability of the soil. Both of these options have been examined, e.g. by Laue (1997), who found good consistency of both techniques in a series of modeling of models tests. Ellis et al. (1998) describe the necessities of additional laboratory testing in the respect of minimising the influence of these measures on the behavior of the soil.

With respect to the reliability of earthquake tests in geotechnical centrifuges, effort has also been spent on building suitable containers (laminar model boxes) to be able not to effect the result of the tests by the boundaries (e.g. Arulanandan et al. 1994) and to the development of suitable shaking tables which can withstand the additional acceleration acting on them as well as to be able to simulate a wider range of earthquakes, as the time acceleration input has to be n-times faster in the physical model than in reality (comp. Table 2).

More details on the concepts of physical modeling can be found in Schofield (1980). Ko (1994) summarised the advantages of centrifuge modeling of seismic events. An overview of the potential of centrifuge modeling is given by Taylor (1995), including a more detailed description of dynamic modeling given by Steedman & Zeng (1995). Laue (2002) summarized the development of centrifuge technology by 2001.

8.3.2 Recent studies on soil structure interaction and countermeasures

Since the start of the use of centrifuge modeling in respect of earthquakes, many soil structure interaction problems have been investigated. In the earlier phases, most of these studies were related to the observation and understanding of the failure mechanism like e.g. the work of Habibian (1987), who studied the behavior of costal dykes on layered sand foundations due to seismic loadings. Nowadays alot of studies deal in more detail with the countermeasures against the consequences of liquefaction, while the applicability of centrifuge model testing for developing and studying different countermeasures was already discussed by Koga et al. 1991. Most of the studies can be found in the following conference proceedings (Corte 1988, Ko et al. 1991, Leung et al. 1994, Kimura et al. 1998 or Phillips et al. 2002). Others might be found in the database of centrifuge model tests (Thorel et al. 2000) at <http://necer.lpc.fr/cleo>.

Some of the recent studies are given for typical interaction problems in the following tables as well as for the studies on different procedures for countermeasures focusing on liquefaction.

Table 8.3: Centrifuge model studies on the soil structure interaction during earthquakes

Structural member	References
Walls (general)	Matsuda et al. 1998, Matsuo et al. 2002, Sato & Zhang 1998, Watabe et al. 2002
Caissons	Kamon et al. 1998, Kawai et al. 2002, Lee et al. 2002
Piles, including piles in slopes	Chen et al. 2002, Higuchi & Matsuda 2002, Haigh & Madabhushi, 2002b, Taji et al. 1998, Takahashi et al. 1998
Tunnel and buried cylinder	Onoue et al. 1998, Takahashi et al. 2002,
Embankments	Hayashi et al. 1998, Hayashi et al. 2002, Peiris et al. 1998, Sakemi 1998
Foundations	Haigh & Madabhushi 2002a, Kawasaki et al. 1998

Table 4: Centrifuge model studies for different countermeasures against liquefaction

Countermeasure	<i>References</i>
Densification	Elgamal et al. 2002, Koseki et al. 1998, Uno et al. 2002
Artificial blocks/ sheet piles	Okamura & Matsuo 2002, Sasaki & Matsuo 2002
Grouting	Imamura et al. 2002b, Kitazume et al. 2002
Drainage	Brennan & Madabhushi 2002, Yang & Ko 1998
Geofabrics	Adalier & Elgamal 1998

9. CASE STUDIES

Large numbers of seismic microzonation studies were conducted in all earthquake prone areas of the World (Marcellini et al., 1982, 1998; Astroza & Monge, 1991; Lasterico & Monge, 1972; Faccioli et al., 1991; Chavez-Garcia & Cuenca, 1998; Lungu et al., 2000; Faccioli & Pessina, 2001; Fäh et al., 1997, 2001). However, few of these studies are well documented in the literature. An attempt will be made in this section to review the available literature briefly and give more detailed explanation about four studies conducted in Benevento, Italy (Marcellini et al., 1991, 1995a, 1995b; Barcelona, Spain (Cid et al., 2001, Jimenez, et al., 2000;), Thessaloniki, Greece (Lachet et al., 1996; Raptakis et al., 1998b); Bagcilar-Istanbul, Turkey (Ansal et al., 2001).

Abeki, N. et al. (1995) presented a microzonation map for a small city in Japan based on the soil dynamic properties presented in this conference paper. At the first step the subsurface ground conditions were studied and the upper stratum was classified by type based on borehole data. Data from 200 drillings were used to classify 34 soil types within the studied area. In the second step, soil transmission coefficients for each soil type were obtained using the multiple reflection theory. In the third step the studied area was divided in a check pattern of 200m by 200m and microtremor measurements were made at its intersections. Predominant periods and spectral ratios were calculated with respect to a fixed measurement point. In addition, the results of seismic intensity survey obtained after a small earthquake were used to produce a isoseismal map of the city. Finally, 34 soil types were reduced to 5 according to the similarity of the soil transmission coefficients, the amplification factor and the spectra obtained in the second step and the results were presented as a microzonation map.

Marinos, et al., (2001) presented a zonation study performed shortly after 1999 Athens earthquake to categorize ground conditions in the damaged parts of the city with regard to seismic risk. In the absence of systematic geotechnical investigations the ground was classified by assessing mainly geological data supported by the available geotechnical data according to the criteria provided by the Greek seismic code. A ground zoning map of 1:25,000 scale was compiled which divides the area of interest into 4 zones. Each zone was defined in terms of basic engineering geological characteristics and associated with a possible range of ground categories. This proposed zoning was then checked against two kinds of field evidence: local distribution of damage and seismic ground response analyzes performed at 8 selected sites. The comparison with observed damage showed that the boundaries between the zones often coincide with the boundaries between differently damaged areas. For the site response analyzes, borehole geotechnical investigations supplemented by cross-hole and down-hole Vs measurements were used to construct soil profiles for selected sites. Each of these profiles was subjected to 2 different seismic excitations: a synthetic accelerogram simulating 1999 earthquake and an actual recording from 17/01/1983 earthquake. Site response analyzes was conducted using Shake program to determine the ground motion on the ground surface by applying the above mentioned excitations as outcropping bedrock motions. The results of these numerical analyzes were found to be in agreement with the geology-based classification of site conditions.

Anastasiadis et al., (2001) with the intention of providing a basis for a detailed site response analysis oriented to a microzonation study, geotechnical and geophysical nature of soil materials and formations under the urban area of Thessaloniki was investigated and the results are presented as 1-D profiles, 2-D cross sections and 3-D thematic maps. The first step was to organise a large database comprising all available geotechnical information. At this stage data from 440 boreholes, 4000 samples, 171 CPT and corresponding laboratory tests were used to classify 60 different soil categories into 7 main soil formations. Then the mean values

of the physical and mechanical properties for each soil formation were determined and a geotechnical map was designed. This map showed the variation of soil conditions in the urban area. The second stage was to perform a detailed geophysical and geotechnical testing program in order to quantify the most important parameters of soil categories and to determine their dynamic soil properties. In the geotechnical survey 14 drillings with more than 400 samples, 280 SPT and 40 resonant column tests were performed and the results provided the description of the surficial formations up to 40-50 m depth for several sites. Resonant column tests were the main tests for determining degradation and damping curves of the soil formations. The geophysical survey included 14 crosshole and 3 downhole tests and seismic surficial methods such as surface wave inversion (SWI), P and SH refractions (REF) techniques. Vs profiles are mainly obtained from CH tests. The correlations of data from different methods were also made in order to define uniform average values. At the third stage all data is combined to define vertical profiles. To design 2-D cross section and 3-D maps all parameters were correlated in spatial distribution. The result of this mapping showed the topographical irregularities, lateral variations, possible discontinuities and the variation of soil types. Comparison of results with macroseismic observations relative to maps of intensities, damage and microtremor measurements, indicated important relationships with the factors mapped.

Lachet et al., (1996) presented the experimental work which aims to make a contribution to the microzonation study of Thessaloniki and to provide data for a methodological study about three different site effect evaluation methods. With this intention ten seismological stations were installed for 3 months in the city of Thessaloniki. About 40 local earthquakes were recorded in more than two stations. The records were then analyzed with three different techniques: a) classical spectral ratios to b) receiver functions, c) (H/V) spectral ratios on noise recordings. The results were compared in terms of predominant frequencies and amplification levels. The three methods are found equally able to reveal the fundamental frequency. However regarding site amplification, they provide significantly different results with a general trend of the two H/V ratio methods to underestimate the amplification levels compared to that inferred from the classical spectral ratio technique. The results were then used in relation with the microzonation of the city. A good correlation is found between the local site amplification and the type and age of the local geological formation as well as with the depth of bedrock. Moreover, amplification levels from spectral ratios are found to be in good agreement with the damage distribution observed in 1978 earthquake. The paper concluded that, for the moment, amplification information gathered from H/V techniques should be considered as a qualitative measure of site amplification. However, the authors suggests that it is suitable to use of H/V techniques applied on noise recordings for rough urban microzonation studies in areas of moderate to low seismic activity.

9.1. Seismic microzonation of Suihua city

Suihua is a small city in the north-eastern part of China. The microzonation project of the city is an example for the conventional procedure of the Chinese code.

9.1.1 Regional seismicity and neotectonics

According to the code all destructive earthquakes within 150 km and all earthquakes recorded within 25 km around the city with magnitude not less than 2.0 should be collected and analyzed. 2 reported strong events (May 5, 1941 and Sep. 2, 1942) and 8 recorded events (during 1973-1993 and $2.2 \leq M \leq 4.3$) were considered in this study.

Known features of the regional neotectonics were reviewed. According to code, a field investigation in the area within 25 km of the city was carried out for active faults. 10

geophysical and geochemical survey sections were carried out. 4 of the studied sections showed the evidence of faults.

Based on the information regarding the seismicity and neotectonics of the region, potential source areas were evaluated and the upper bound magnitude for the Suihua area was estimated as 6.5.

9.1.2 Seismic hazard assessment

Cornell type probabilistic seismic hazard computation was used to evaluate the seismic hazard. Potential source areas were defined with seismicity parameters such as annual occurrence rate of an event magnitude greater than a given lower bound and b-value.

Attenuation relationship was defined by adopting an elliptical attenuation model. From a set of attenuation relationships, the design intensity of the city was calculated as VII for exceeding probability of 10% in 50 years. A set of uniform hazard spectra was calculated for exceeding probabilities of %63, %10 and %3 in 50 years and considered as the design response spectra at rock surface.

Then the design ground motion was simulated at free rock surface. Total of 9 acceleration time histories were generated three for each probability level. Superposing harmonic wave procedure was used for the generation of these artificial time histories. The total duration was taken as 20 sec. with a strong shaking phase of 10 sec. for all three exceeding probabilities.

9.1.3 Microzonation

In order to define local site conditions in the city boring data at 291 sites, SPT data at 150 points and VS logging data at 10 points with experimental results of 1082 soil samples were collected. After ground survey and preliminary analysis, 15 CPT tests, 4 boreholes with sampling and VS loggings at 23 sites (including 4 wells) were carried out. Cyclic triaxial tests for 26 undisturbed soil samples were performed to derive a set of nonlinear relationships.

Seismic response of the ground was calculated through 1D wave propagation analysis. The site profile was modelled by horizontal layered homogenous media with varying densities, wave velocities and thicknesses. The design ground motions at rock surface were reduced by half and used as input motions. The equivalent linearization procedure was applied for accounting soil nonlinearity. 56 sites, which describe the variation of soil condition in the city, were selected and ground motions at these sites with the above probability levels were calculated.

The ground acceleration response spectrum at each site was represented by four segment curve with parameters T_1 , T_2 , T_3 and S_{max} . From the minimum maximum and mean values of the four parameters the city area was divided into two subareas. Design ground response spectra for each subarea were defined by T_1 , T_2 , T_3 and S_{max} parameters for each of the three probability levels.

Evaluation of ground failure hazard on the basis of available information led to the conclusion that liquefaction, surface rupture or seismic settlement will not occur under ground shaking with the design intensity.

9.2. The Benevento seismic risk project (Marcellini et al., 1991, 1995a, 1995b)

Benevento is a town of 62000 inhabitants with approximately 13 sq kilometers area. It is located in Sannio region, one of the most seismic zones of the Southern Italy. The town has been a European test site for hazard and microzonation investigations.

9.2.1 Geology and Geotechnical characterization

The succession of the various geological units underlying the Benevento area was described in detail by reviewing the previous studies and existing data. A geological map of 1:25 000 scale was obtained based on this information.

The existing geotechnical data was used to evaluate the geotechnical aspects of the studied area. The data consisted of approximately 200 boreholes, unequally distributed in the investigated area, and limited in depth (less than 30m). Physical and mechanical properties of the soils were determined by standard laboratory tests carried out on 50 undisturbed samples, several penetration tests (SPT and CPT), and few seismic surveys (DH). Through the main alignment of available boreholes three sections were drawn and characterized by suitable properties for the seismic response analysis. On the basis of available information Benevento was found to be only marginally susceptible to instability phenomena (landslides and liquefaction). Thus, only the response of soil layers to cyclic loading was considered in the microzonation study. Seismic ground response was evaluated using the linear equivalent approach. Small strain shear modulus and damping ratio for most of the soils were evaluated by engineering judgment and the well known correlations with index properties. Only for few sites, where seismic loggings were performed, G_0 was directly measured. For some of the materials, nonlinear curves of shear modulus and damping ratio were obtained by assuming the Ramberg-Osgood model with Masing criteria. For the others, results from cyclic laboratory tests carried out on soils with similar physical and mechanical properties were used. Preliminary analysis of all geotechnical data led to the identification of 3 main geotechnical zones within the city.

9.2.2 Seismic hazard

Seismic hazard was evaluated in terms of response spectra. The input parameters were chosen as follows:

Seismic source zones obtained from recent research studies were considered. The seismicity was considered homogeneously distributed within the zones.

Two options of earthquake catalogues were selected. a) The Postpischl 1985 catalogue updated by 1990 and 1991 Potenza earthquakes, b) a new version of the Italian catalogue.

Three different intensity-magnitude conversion relationships were applied to convert intensity based events. Two of them were specifically designed for hazard analysis and the third one ad hoc determined for the purposes of this study.

Recurrence laws were computed according to Gutenberg-Richter relation. An upper bound for magnitude of 3.5 was estimated for each seismic source zone on the basis of the seismotectonic and seismological information.

Two attenuation relations, both derived from regressions of Italian data, were used. Peak horizontal acceleration, velocity and spectral ordinates were attenuated as ground motion parameters. Both mean and mean plus one standard deviation was considered.

Uniform hazard spectra was computed by using SEISRISK III, computer code with earthquake catalogue option a, and using a Bayesian approach with earthquake catalogue option b.

The input accelerograms were derived from pseudo acceleration response spectra after 1D deconvolution at a selected reference site in Benevento. The procedure was to select 8 real accelerogram that showed good fitting with the expected amplification spectra. An additional accelerogram selected as reference motion was included. 9 selected accelerograms were then

deconvoluted using a representative soil column to obtain the corresponding S wave motion as input motion to 1D model.

Only accelerograms derived from uniform probability response spectrum computed using data from catalogue b were selected for site effects evaluation.

9.2.3 Microzonation

Geological and geotechnical investigations led to the conclusion that liquefaction hazard could be neglected and the likelihood of slope instability phenomena was very limited. Thus, the microzonation study was carried out in terms of ground motion amplification. Two complementary approaches were applied to estimate site response in the Benevento area. These were instrumental measurements for investigating weak motion (linear) amplification in different frequency ranges and numerical simulations to obtain strong motion spectra at selected sites.

Instrumental measurements were consisted of 3 different shots recorded at an array of 40 stations deployed in the city. The shots were located at epicentral distances ranging between 7km and 19 km. The basic processing was to compare the recordings with those recorded at a reference station. Ambient noise recordings before and after each shot were also performed at these stations and analyzed by Nakamura's technique. The results obtained using recordings of shots were not found to be reliable enough to map the resulting spectral ratios throughout the city. Instead, the spectral ratio values were used as a check to Nakamura's technique. Results from 42 ambient noise recordings over an area of 9 km² were used to develop H/V amplification maps for 6 different frequency ranges.

For the numerical simulations few sites in each of three main geotechnical zones were selected as representative profiles. Total of 15 profiles were analyzed using 1D linear equivalent approach, and the input spectra derived from the probabilistic seismic hazard study. 9 accelerograms were used as input motions. Both linear and linear-equivalent analyzes were performed by SHAKE. The results showed that there were no remarkable differences between the different parts of the city and the microzonation results could be reduced to an overall response spectrum which accounts for the amplification effects of the whole area. Thus, 5% damping PSA obtained at 15 sites were used to suggest a single spectrum for Benevento.

9.3. Seismic zonation of Barcelona (Alfaro et al., 2001)

Barcelona is a 2 000 000 inhabitants city, located north-east of the Iberian Peninsula. The area is considered to be of a moderate seismic activity.

9.3.1 Local Geology and Geotechnical Data

All related data to local geology (age and lithology) were compiled and a map showing different geological units were generated.

Existing data relevant to geotechnical characteristics of the subsoil of the city was collected. Most of the geotechnical information was obtained from civil construction work related boreholes for which SPT data exists. Based on this data base 70 representative soil columns were selected. Dynamic properties of the soil layers were estimated through a series of empirical correlations with SPT values. Estimation of the depth of basement formation was obtained through an inversion of a detailed gravity survey. The curves for shear modulus degradation and damping ratio were taken from the recent experimental data obtained at Euroseistest, selecting the more appropriate to local conditions of Barcelona.

9.3.2 Seismic hazard

Recent studies of seismic hazard were reviewed. According to these studies expected intensity in Barcelona is VI-VII MSK for a return period of 500 years. This corresponds to a value of peak ground acceleration of 0.054g according to the relation proposed in the Spanish Seismic Code.

The design ground motion was defined as a series of Ricker pulses with predominant frequencies 2-7 Hz. Acceleration level of this motion was 0.054g as defined by the seismic hazard study.

9.3.3 Site response evaluation

Seismic responses of the selected sites in the city were estimated through 1D numerical analysis. Shake 91 was used to compute transfer functions at 70 selected soil columns. The input motion for SHAKE analysis was obtained after deconvoluting the motion defined at seismic hazard assessment to the basement rock formation. The deconvoluted signals were introduced at the base of each soil column. 5 functions were computed for each column: signal (acceleration-time history) at the surface, transfer function between the basement formation and the surface, pseudo-acceleration elastic response spectrum for 5% damping, pseudo-acceleration elastic response spectrum for 5% damping normalized to peak ground acceleration, pseudo-velocity elastic response spectrum for 5% damping. In order to analyze the uncertainties associated with input data a Monte Carlo's simulation process was applied. In summary, all five above mentioned functions were computed considering 6 different input signals at 70 sites and using a Monte-Carlo's simulation process with 500 computations in order to take into account the uncertainties of dynamic parameters.

9.3.4 Seismic Zonation

The transfer functions of 70 sites were grouped into 3 zones taking into account the analogies. Each zone corresponded roughly to the geological units defined for the city. An additional zone corresponding to rock outcrops was introduced. Finally a microzonation map consisted of four zones was proposed. In this map each zone was characterized by a transfer function together with one-standard deviation curves and by an amplification factor for the peak ground acceleration level relative to a reference site. The resulting map was compared with that previously obtained through microtremor measurements. A good agreement was obtained for two of the zones. However, for the other zone Nakamura's technique was observed to estimate the frequency corresponding to the first peak of the amplification which was different from the predominant frequency for this zone.

9.4. Seismic microzonation of Bağcılar, Istanbul (Ansal et al., 2001)

A microzonation case study conducted for the town of Bağcılar in Istanbul. Bağcılar town lies on 22 sq km with a population of 600,000. In the first stage of the study, probabilistic analyzes were conducted using this seismic database for evaluating the seismic hazard and design earthquake magnitude using single circular aerial source zone with 100 km radius. The magnitude of design earthquake was calculated as $M=7.3$ corresponding to the exceedance probability of 10% for a period of 50 years, as adopted for determining the design earthquake in most earthquake codes. A statistical analysis was conducted to estimate the possible epicentral location assuming that all epicentres of past earthquakes are possible epicentres for future earthquakes (Kafka & Walcott, 1998). By this probabilistic approach, the epicentral distance for a $M=7.3$ earthquake corresponding to 10% exceedance probability around Bağcılar is determined as 32 km, with a 10% probability of being less. The maximum acceleration at the bedrock level was estimated by using a revised version (Ansal, 1997) of the attenuation relationship proposed for Europe by Ambraseys (1995) as:

$$\text{Log } A_p = 0.33M - 0.00327R - 0.79\text{Log}R + 1.177 \quad (9.1)$$

where A_p is the maximum horizontal acceleration, M is the design earthquake magnitude. R is the hypocentral distance that can be calculated from $R = \sqrt{(R_{\text{epc}})^2 + h^2}$ where R_{epc} is epicentral distance and h is focal depth. The standard deviation of this relationship was calculated as 0.42 indicating the margins of scatter of the relationship between the observed peak acceleration and hypocentral distance. Thus adopting a probabilistic approach the maximum acceleration on the bedrock was computed as 0.28 g with respect to 10% exceedance probability.

9.4.1 Geology and Local Soil Conditions

A detailed geological study (1:1000 scale) was conducted to determine the variation of the geological units within the city boundaries supported with borings, in-situ testing and laboratory tests to identify the properties of the soil layers encountered in the soil profile. It was observed that the geological units in Bağcılar were not homogenous and significant changes in soil properties were observed from one point to another, even in the same geological unit. As a result, the geological units were not considered as the basis in seismic microzonation. Borehole samples and geological studies were used to classify the soils in Bağcılar town according to the *1998 Turkish Earthquake Code*. These studies showed that using a two class definition would be the best way to identify the soil conditions in Bağcılar town as a result of limited borehole data and uncertainties in the site classification boundaries. In the end, three zones (Z1-Z2), (Z2-Z3) and (Z3-Z4) were identified in accordance with this approach. This is an approximate zonation in terms of site classification obtained from limited number of boreholes. It is clear that more detailed soil investigations are required for determination of local site classes for design purposes.

9.4.2 Earthquake Characteristics on the Ground Surface

Equivalent shear wave velocity were calculated by conducting in situ seismic tests or by using the correlations developed in terms of SPT-standard penetration tests (Iyisan, 1996). Both methods were applied in Bağcılar to obtain the shear wave velocities and seismic refraction tests were conducted at 40 locations, where the correlations were used on the SPT data from 40 different boreholes. The equivalent shear wave velocities calculated and measured range between 200 m/sec and 700 m/sec in Bağcılar town, which clearly displays the variability of local conditions from one point to another.

9.4.3 Soil Amplification and Predominant Soil Period

The variation of predominant site periods in Bağcılar was determined from 250 microtremor measurements conducted with approximately 500 meters spacing. The variation of predominant periods indicated a large scatter between 0.1 to 0.9 seconds. The Turkish Earthquake Code defines 0.4 and 0.6 seconds for local site classes of Z2 and Z3, respectively. The measurements in Bağcılar also showed that predominant soil periods may change significantly even in the same local site category.

Spectral amplification (defined as spectral acceleration coefficient in the 1998 Turkish Earthquake Code) is another important parameter to evaluate the seismic forces in earthquake resistant design. The upper limit for this parameter is given as $S=2.5$ in the Code. The spectral acceleration coefficient obtained from microtremor measurements in Bağcılar has values up to $S=4.5$. Even though these results may be considered approximate, they indicate the importance of site specific determination of spectral accelerations in design process, especially in the case of important structures.

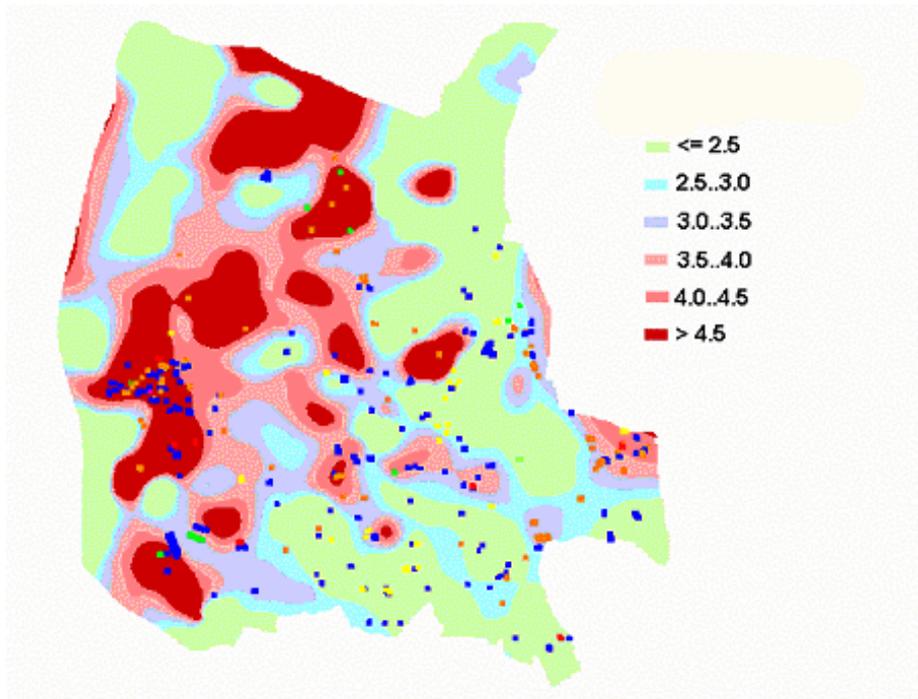


Figure 9.1. Variation of spectral acceleration coefficient in Bağcılar determined based on microtremor records

The damaged buildings in Bağcılar due to the Kocaeli Earthquake are also marked in Figure 9.1. An intense distribution of damage can be observed around the regions, where high spectral amplifications (spectral acceleration coefficients) are computed. The increase of spectral coefficients would lead to higher seismic forces on structures, thus playing a major role in structural response, especially in the case of structural weaknesses.

9.4.4 Seismic Microzonation

Depending on the scale chosen, the parameters obtained from different locations in seismic microzonation studies are not always precise and may not reflect the existing conditions for structural design. Therefore, site specific investigations need to be performed in each site for more accurate and safer design. A classification in terms of priority levels, can be used to overcome this situation and parameters determined from seismic zonation studies may be used in the design process of the selected structures, i.e. one or two storey residential buildings. On the other hand, site specific studies, including in situ and laboratory tests, must be obligatory in assessment of the required parameters for the structures with higher importance levels.

In structural seismic design three parameters can be considered as the main controlling factors. In the 1998 Turkish Earthquake Code these parameters are specified as effective acceleration, spectral acceleration coefficient and corner periods. Spectral accelerations (the product of spectral acceleration and effective acceleration coefficients) may be used for determining the seismic forces affecting the structures. Therefore it is appropriate to select spectral accelerations as one of the parameters for seismic microzonation.

The spectral acceleration coefficients are assessed based on microtremor measurements and equivalent shear wave velocities and the effective accelerations were calculated using site response analysis and by considering the 70% of the calculated peak ground acceleration as the effective acceleration in the light of different earthquake codes. The spectral accelerations are calculated as the product of these two parameters. In structural seismic design the other parameter needed is the predominant soil or spectral corner period. The predominant soil

periods were obtained from microtremor measurements in the light of the site response analysis and are considered as the spectral corner period, T_B .

The final seismic microzonation map is obtained by superposition of these two variations as shown in Figure 9.1. The advantage of this assessment is the capability to consider both spectral accelerations as well as predominant soil periods jointly in suggesting some guidelines with respect to structural requirements for city planning purposes.

Six zones (A1 to A6) were defined with respect to spectral acceleration and predominant periods (Table 1) to provide city planners with some guidelines for specifying population density or in other terms building characteristics such as height, and percentage of the build area in the lot.

Accordingly the zones classified as A1 is observed to be the most suitable areas for high population density, high rise construction and important buildings such as hospitals, fire stations. This suitability decreases with increasing zone number. However, as mentioned previously, these suggestions are based on the analysis of the data obtained mostly at a scale of 1/5000, therefore they should be considered as general guidelines and not as definite findings. Small scale geotechnical and site specific studies need to be conducted to have more accurate assessment for structural design purpose.

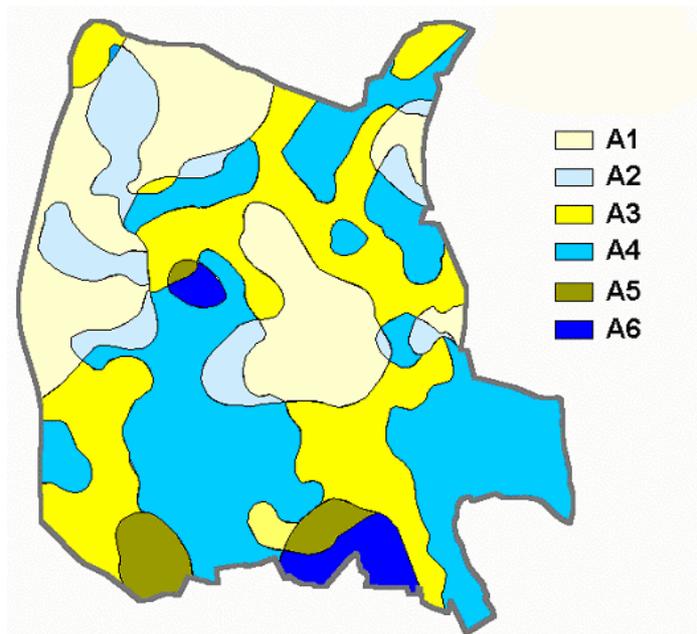


Table 9.2 Estimated Zone characteristics

Zone Name	Spectral Acceleration (g)	Predominant Period (sec)
A1	$S_a \leq 1$	$T_B \leq 0.5$
A2	$S_a \leq 1$	$T_B > 0.5$
A3	$1 < S_a < 1.5$	$T_B \leq 0.5$
A4	$1 < S_a < 1.5$	$T_B > 0.5$
A5	$S_a \geq 1.5$	$T_B \leq 0.5$
A6	$S_a \geq 1.5$	$T_B > 0.5$

Figure 9.2. Proposed seismic microzonation

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