2008

Anomalous effects of strong earthquakes

Michal Kekeňák

2008

Anomalous effects of strong earthquakes

BACHELOR THESIS

Michal Kekeňák

Comenius University
Faculty of Mathematics, Physics and Informatics
Department of Astronomy, Physics of the Earth and Meteorology

Supervisor: Prof. RNDr. Peter Moczo, Dr.Sc.

Bratislava 2008

Abstract

Michal Kekeňák

Anomalous effects of strong earthquakes

Comenius University
Faculty of Mathematics, Physics and Informatics
Department of Astronomy, Physics of the Earth and Meteorology

Supervisor: Prof. RNDr. Peter Moczo, Dr.Sc.

Bratislava 2008

This thesis is intended as an introduction to anomalous effects of strong earthquakes known as site effects. We start by introducing the reader in the first chapter to various elemental questions, starting with a simplified definition of earthquake source. Then we follow with a very brief summary of significant earthquakes, which provided scientists with valuable data and contributed to a better understanding of these phenomena. After that, our attention is turned towards the most important issues regarding the introduction to the earthquake related questions. First we give an overview of the ground motion characterization. Parameters are defined which characterize properties of the earthquake ground motion. We end the first chapter with a brief summary of the seismic hazard analyses. The second chapter introduces the basic site effects with the focus on the effects of the soft-soil deposits and topographical effects. The third chapter introduces the basic methods used to evaluate in a quantitative form the character of a particular site effect, ranging from experimental methods to numerical modeling. All chapters were written with the main idea to summarize the fundamental basics regarding information about site effects.

Abstract

Michal Kekeňák
Anomálne javy silných zemetrasení

Univerzita Komenského
Fakulta matematiky, fyziky a informatiky
Katedra astronómie, fyziky Zeme a meteorológie

Školiteľ: Prof. RNDr. Peter Moczo, Dr.Sc.

Bratislava 2008

Táto práca je myslená ako úvod do anomálnych efektov silných zemetrasení zvaných lokálne efekty. Začneme zoznámením čitateľa v prvej kapitole s rôznymi základnými problémami týkajúcich sa zemetrasní, pričom začíname zjednodušenou definíciou zdroja zemetrasenia. Nasleduje stručné zhrnutie význačných zemetrasení, ktoré umožnili vedcom z nich zistiť cenné dáta, ktoré prispeli k lepšiemu porozumeniu týchto fenoménov. Potom sme sa venovali najdôležitejšiemu aspektu úvodu do zemetrasení. Začali sme prezentovaním prehľadu charakterizácie pohybu zemského povrchu. Definujú sa parametre, ktoré charakterizujú vlastnosti pohybu zemského povrch spôsobeného zemetrasením. Prvá kapitola je ukončená stručným zhrnuťím analýz seismického ohrozenia. Druhá kapitola uvádza základy lokálnych efektov s hlavným zameraním na efekty spôsobené nánosmy mäkkej pôdy a topografiou. A v tretej kapitole sú uvedené základné metódy používané na vyhodnotenie kvantitatívnym spôsobom charakter daného lokálneho efektu využitím či už experimentálnych metód, alebo numerických metód. Všetky kapitoly boli písané s hlavnou myšlienkou zhrnúť absolútne základy týkajúce sa lokálnych efektov.

Contents

1 Elemental introduction to earthquakes

1.1 Earthquake source

Earthquakes exist because the Earth's interior is not a homogenous fluid substance. The interior is strongly heterogeneous and in a permanent motion. The Earth is covered by tectonic plates. These plates are in a permanent, yet slow, motion and are interacting with each other. If they are moving past each other or against each other they generate stress which can be accumulated and later, when it reaches its critical value, released in the form of an earthquake. The characteristics of plate boundaries influence the nature of earthquakes occurring on them.

Spreading ridge boundaries the plates move away from each other, letting fresh material from greater depths rise to the top. Subduction zone boundaries are the opposite of spreading ridge boundaries – plates move towards each other (which can result in the creation of mountain ranges). The intensity of force which propels these plates is not equal and that results in one plate being dominant. The dominating plate starts to subduct the other plate. Subduction zone boundaries are most often found near the edges of continents. Transform fault boundaries occur when plates move past each other. A new crust is neither consumed nor created.

An earthquake can be modeled as a spontaneous rupture on a fault, where enough stress is accumulated. As the earthquake starts, stress is gradually relieved and waves are generated. These waves generate additional displacement of material and travel until they are fully absorbed. When they reach the surface, ground shaking is observed which can result in damage especially to manmade constructions. This is the reason why earthquakes must be thoroughly studied to predict them (if possible) and mitigate the effects of future earthquakes for the economic loss and the loss of human life can be catastrophic during a large earthquake.

Here we closely follow Moczo et al. (2007) to provide a brief mathematical explanation of a simple model of earthquake source.

Let $\vec{n}(x_i)$ be a unit vector normal to the fault surface pointing from the "-"side to the "+" side. Then slip can be defined as a discontinuity in the displacement vector across the fault surface:

$$\Delta\vec{u}(x_i,t) = \vec{u}^{+}(x_i^{+},t) - \vec{u}^{-}(x_i^{-},t). \quad (1.1)$$

The time derivate of slip, slip rate (discontinuity in the particle-velocity vector across the fault surface) is defined as

$$\Delta\vec{v}(x_i,t) = \vec{v}^{+}(x_i^{+},t) - \vec{v}^{-}(x_i^{-},t). \quad (1.2)$$

The total traction on the fault is

$$\vec{T}(\vec{n};x_i,t) = \vec{T}^{0}(\vec{n};x_i) + \Delta\vec{T}(\vec{n};x_i,t), \quad (1.3)$$

where $\vec{T}^{0}(\vec{n};x_i)$ is the initial traction present before slip occurs and $\Delta\vec{T}(\vec{n};x_i,t)$ is the traction variation also called perturbation and is caused by the propagating rupture. At any point of the rupture the total traction is related to slip through the friction law

$$\vec{T} = \vec{T}^{f}(\Delta\vec{u},\Delta\vec{v},\theta), \quad (1.4)$$

where $\vec{T}^{f}$ is the frictional traction and θ represents a set of state variables. Equation (1.4) is called the fault constitutive law and means that the total dynamic traction on the fault is determined by the friction. It is the friction law which controls the initialization, propagation and healing (also called arrest) of the rupture.

1.2 Significant historical earthquakes

Not every earthquake is large and devastating. This is because it depends on how much stress can an active fault accumulate before slip occurs. In fact many earthquakes are recorded throughout the year. Over 500 000 earthquakes occur a year around the world which are practically unperceivable; around 1000 moderately damaging earthquakes and one great earthquake with very significant damage potential. Throughout the history several earthquakes were particularly significant due to either their intensity or the facts and insights engineers and scientists were able to uncover by studying the effects caused by them.

The 1960 Chile (9.5 magnitude) is probably the largest earthquake ever recorded. The number of human casualties was ~ 2300 and the earthquake caused an extensive damage to the surrounding areas.

In 1975 a successful prediction of an imminent earthquake saved thousands of lives in Haicheng, Liaoning Province, China.

Even though not the largest earthquake, in 1976 an earthquake literally destroyed the city of Tangshan, Habei Province, China with a death toll about 700 000 people.

Another significant earthquake was in 1985. The epicenter was off the Pacific coast, but the most damage occurred 220 miles (360 km) in Mexico city. This illustrated the importance of understanding the site effects on ground motion. Subsequent studies led to a better understanding of dynamic properties of fine-grained soils.

These are only a few of the most important earthquakes ever recorded. More detail can be found, e.g., in Kramer (1996).

What is particularly disturbing is that recent earthquakes cause more economic damage and greater loss of life even though they are smaller. This is because cities are becoming much more crowded and are generally built on high-risk places. This leaves us with only one possible option. Since we cannot stop earthquakes from occurring we have to minimize the damage they can cause by building “earthquake-resistant“ buildings.

1.3 Earthquake parameters

The following chapter contains several outtakes from Kramer (1996). One of the most important parameters of an earthquake is its “size”. It has been described in many different ways in the past as well as in the present. In the past descriptions were almost exclusively qualitative because no instruments were designed for measuring seismic activity. With the development of modern and precise seismographs new quantitative methods of determining the size of an earthquake could be used.

The oldest measure of an earthquake is its intensity. It is a qualitative description of an earthquake effects. Several scales have been used throughout the world to characterize the size of earthquakes and their impact on populated areas. The first scale was the Rossi-Forrel scale developed in 1880 describing intensities ranging from I to X, later being replaced by the modified Mercalli intensity scale (MMI) modified by Richter in 1958. Other examples of different scales are the Japanese Meteorological Agency used in Japan and the Medvedev-Spoonheuer-Karnik scale used in Central and Eastern Europe. Recently, a new European Macroseismic Scale EMS-98 has been introduced in Europe. The scale significantly improves characterization of the effects with respect to damages on buildings.

1.3.1 Earthquake magnitude

When the first reliable seismographs were constructed we were able to develop a modern way of measuring ground motion. Seismic instruments allow an objective and quantitative measurement of earthquake size called the earthquake magnitude.

In 1935 Charles Richter defined a magnitude scale for shallow, local (epicentral distances less than 600 km) earthquakes in southern California. His magnitude belongs to the local magnitudes. Over decades many other definitions have been developed. The only magnitude which does not suffer from the saturation is the magnitude defined using the scalar moment.

At large epicentral distances body waves are almost completely scattered that the resulting ground motion is dominated by surface waves. The surface wave magnitude (Gutenberg and Richter, 1936) is a worldwide magnitude scale based on the amplitude of Rayleigh waves with a period of about 20 seconds and is most commonly used to describe the size of shallow (less than 70 km), distant (farther than 1000 km) moderate to large earthquakes. We can obtain it from

$$M_s = \log A + 1.66 \log \Delta + 2.0 , \qquad (1.5)$$

where A is the maximum ground displacement in micrometers and Δ is the epicentral distance of the seismometer measured in degrees.

When surface waves are too small to enable a reliable and precise evaluation, as is the case for deep-focus earthquakes, the body wave magnitude (Gutenberg, 1945) has been proposed and is based on the first few cycles of p-waves which are not strongly influenced by focal depth (Bolt, 1989) and we can express it as

$$m_b = \log A - \log T + 0.01\Delta + 5.9, \tag{1.6}$$

where A is the p-wave amplitude in micrometers and T is the period of the p-wave. Other scales have been proposed, for example, the coda magnitude (Aki, 1969) showing that certain characteristics are independent of travel path, the duration magnitude (Real and Teng, 1973) which is based on the total duration of an earthquake and only finds use when describing small earthquakes in which engineers have almost no interest.

The previously mentioned magnitude scales are all empirical methods based on observations from various instrumental measurements of ground-shaking. The problem arises with larger earthquakes where the level of ground-shaking does not necessarily increase at the same time. For strong earthquake, the measured ground-shaking characteristics become less sensitive to the size of the earthquake than for smaller earthquakes. This phenomenon is referred to as *saturation*. The body wave and Richter local magnitudes saturate at magnitudes 6 to 7 and the surface wave magnitudes saturate at about magnitude 8.

The only magnitude scale that is not subject to saturation is the moment magnitude (Kanamori, 1977; Hanks and Kanamori, 1979) since it is based on the seismic moment, which is a direct measure of the factors that produce rupture along the fault and we can obtain it by

$$M_w = \frac{2}{3} M_0 - 6.06 \tag{1.7}$$

where M_0 is the seismic moment in Newton-meters (Moczo and Labák, 2000). The seismic moment is given by

$$M_0 = \mu A \bar{D}, \tag{1.8}$$

where μ is the rupture strength of the material along the fault, A the rupture area and $\bar{D}$ the average amount of slip. Because of its units (force times length) it correlates well to the amount of "work" done by the earthquake and therefore to the amount of energy released (Bullen and Bolt, 1985).

1.3.2 Earthquake duration

The duration of an earthquake often has influence on how much damage the surrounding area sustains. This is caused by the fact that such constant change in stiffness and strength of many materials in certain types of structures are very sensitive to the number of stress reversals that occur during an earthquake. Just like a thin metal cable breaks after being constantly bent from one side to the other, structural materials are subject to the same destruction of their molecular cage. If the motion is not long enough it may not produce enough reversals for the building to collapse. Short duration motions with high amplitude produce less damage than long durations with moderate amplitude. The duration of ground motion is related to the time required for release of accumulated strain energy by rupture along the fault and therefore it increases with increasing earthquake magnitude. This relationship has been supported by empirical evidence. But advances in source mechanism modeling (Hanks and McGuire, 1981) indicated, that the duration should be proportional to the cube root of the seismic moment.

Different approaches have been taken to the problem of evaluating the duration of ground motion in an accelerogram. The bracketed duration (Bolt, 1969) is defined as the time between the first and last exceedances of a threshold acceleration, usually 0.05g. Another definition of duration (Trifunac and Brady, 1975) is based on the time interval between the points at which 5% and 95% of the total energy has been recorded. Boore (1983) has taken the duration to be equal to the corner period (the inverse of the corner frequency). McCann and Shah (1979) have taken as a reference point the rate of change of cumulative root-mean-square acceleration. Power spectral density concepts can also be used to define a ground motion duration (Vanmarcke and Lai, 1977) as well as other definitions of ground motion duration have been proposed (Perez, 1974; Trifunac and Westermo, 1977). The most commonly used duration definition in earthquake engineering is the bracketed duration for its implicit correlation to the level of ground shaking.

1.3.3 Earthquake energy

Earthquakes with a long duration generally release much more energy than earthquakes with shorter duration. This is because the longer an earthquake is in effect, more stress had to be previously accumulated at the source. And with increasing accumulated stress being released energy rises proportionally. A relationship was proposed that related the moment magnitude to earthquake energy (Kanamori, 1983). It implied that a unit change in magnitude corresponds to roughly a 32-fold increase in seismic energy. For example a magnitude 5 earthquake would release only about 0.001 times the energy of a magnitude 7 earthquake, thereby illustrating the ineffectiveness of small earthquake in relieving the buildup of strain energy that causes very large earthquakes. The amount of energy released by earthquakes is often difficult to comprehend. For example the nuclear blast from the Hiroshima nuclear bomb would correspond to a magnitude 6.0 earthquake, but the 1960 Chile earthquake (magnitude 9.5) released as much as 178 000 such bombs. The total amount of energy is often estimated from the relationship

$$\log E = 4.8 + 1.5 M_s , \qquad (1.9)$$

where E is expressed in Joules (Moczo and Labák, 2000). Kanamori (1983) has shown that this is also applicable to moment magnitude. A unit change in magnitude correlation corresponds to a $10^{1.5}$ (32-fold) increase in seismic energy.

1.4 Seismic hazard

There are many natural disasters capable of great destruction naming just a few like flood, tornados, hurricanes, wildfires and of course damage caused by earthquakes. The damage an earthquake can possibly cause is commonly referred to as seismic hazard. The hazard is carefully studied by earthquake engineers to produce best results, that means less economic loss and loss on human life, during an earthquake. The most important seismic hazards are listed and described in the following section.

1.4.1 Ground shaking

During an earthquake seismic waves are generated at the source and travel through the earth’s crust. This generates displacement of material the waves travel through and results in ground shaking which is present during every earthquake. The strength and duration of the shaking strongly depends on the location the site is placed and on the distance from the epicenter (shortest distance from the source to the surface). Ground shaking can cause tremendous damage when the distance is short and the characteristics of the site allow for great amplitudes and accelerations combined with a long duration of the earthquake. In fact ground shaking can be considered the most important seismic hazard because all other seismic hazards are caused by ground shaking. When ground shaking is low, other seismic hazards are either low or they do not even occur. Seismic waves travel their majority of distance through rock. Only just before they reach the surface do they travel through soil. But this short distance traveled through soil can greatly influence the level of ground shaking. The soil deposits tend to act as “filters”. At certain frequencies they greatly amplify the amplitude while diminishing it at other frequencies. This effect causes that even on a small area levels of ground shaking can vary dramatically. Therefore one of the most important aspects of geotechnical earthquake engineering is the evaluation of local soil conditions and effects.

1.4.2 Landslides

Earthquakes as small as magnitude 4.0 may dislodge landslides from susceptible slopes, and larger earthquakes can generate tens of thousands of landslides throughout areas of hundreds of thousands of square kilometers, producing billions of cubic meters of loose, surficial sediment. These landslides can have significant geomorphic effects that vary depending on the landslide characteristics and materials, and on the settings in which the landslides occur. In a number of unfortunate events landslides have buried entire towns underneath the ground. Although many landslides are caused due to liquefaction and the fact that the earthquakes occurred during or after heavy rainfall, it is not uncommon for landslides to occur, for simply many locations have poor stability and even a small earthquake can set in motion a chain of events that lead to a devastating landslide. The only way of preventing destructive landslides is by either upgrading the strength of the soil or simply avoiding high-risk areas.

1.4.3 Lifeline hazards

A network of facilities that are required for civilized life to even exist must be present in every at least partially developed area. These networks include electrical power, telecommunication, water and sewerage, transportation and are commonly known as lifelines. Damage to these lifelines can cause additional death or economic loss by simply not being able to relocate resources needed for saving those in need or by not being able to support heavily damaged building before they collapse. Additional damage can be caused by for example from an earthquake damaged water reservoir being contaminated. These facilities therefore have to have additional protection against earthquake and other natural hazards, for their operation is of crucial importance to everyone in the surrounding area. The damage to these facilities can even be so major, that it greatly exceeds the direct economic loss caused by the earthquake. Lifeline failures can hamper emergency response and rescue efforts, for example during the 1906 San Francisco earthquake most of the damage was caused by fire which could not be properly fought - the fire spread easily because of gas pipe failures and firefighters were hindered by broken water mains. The Loma Prieta earthquake also caused the collapse and near collapse of several elevated highways and the collapse of a portion of the San Francisco-Oakland Bay Bridge.

1.4.4 Structural hazards

During an earthquake the most economic loss is caused by structures that have not been constructed to withstand an earthquake up to a particular magnitude or by simply bad construction methods or design. It is also the number one reason for casualties because the chance of surviving a collapsing building falling directly on you is minimal. And even if you survive there is a great possibility you will not be rescued and even if you are rescued chances are you will suffer permanent disability. It is therefore crucial to construct buildings which are “earthquake-resistant” in areas where earthquakes are abundant and of particular high magnitude ratio. Even if the whole building is not destroyed in the earthquake there are still major hazards inside a building. Falling shelves, heavy pictures, electric currents are just some examples. Most structural damage can be seen in underdeveloped areas of civilization where barely standing barely reinforced buildings fall down like a house of cards even during a relatively weak earthquake. Even more disturbing is the fact that these underdeveloped areas have a high concentration of people living in these buildings.

Over the years considerable advances have been made in the design of structure to make them capable to sustain a great deal of stress and strain which it is subject to during earthquakes. It was required to be able to perform very accurate predictions of ground motion in high-risk areas. In current design practice a geotechnical earthquake engineer provides the structural engineer with appropriate design ground motions and provides guidance for the development of site-specific design ground motions.

However this is not always the case. It is not uncommon that certain commercial buildings in many countries are being constructed hastily without a proper ground response analysis. This can have terrible consequences in case an earthquake occurs and creates just the kind of ground motion that this building is susceptible to. The risk is even greater as most of commercial buildings are designed to be able to accommodate great numbers of residents or visitors and it is more often observed that buildings are constructed to be taller not thicker, which is even more dangerous.

1.4.5 Liquefaction and other site effects

Some of the most spectacular examples of earthquake damage have occurred when soil deposits have lost their strength and appeared to flow as fluids. In this phenomenon, termed liquefaction, the strength of soils is reduced to a point where it is not able to support structures and remain stable. This occurs only on saturated soils and therefore can be observed near rivers, bays and other bodies of water.

Liquefaction encompasses several related phenomena. Flow failures generally occur when the strength of soils drops below a needed threshold level to maintain stability under certain static conditions. Therefore this effect is driven by gravitational forces and can produce very large movements in the downward direction. Flow failures have caused the collapse of earth dams and other slopes, and the failures of foundations of buildings. The 1971 San Francisco earthquake caused a flow failure in the upstream slope of the Lower San Francisco Dam that nearly breached the dam.

Another related phenomenon is called lateral spreading. It is characterized by incremental displacements during earthquake shaking. Depending on the number and strength of the stress pulses that exceed the strength of the soil, lateral spreading can produce displacements ranging from negligible to dramatically large. It can be commonly observed near bridges and the displacements it produces can damage foundations and superstructures of bridges as well as the surrounding area. The phenomenon of level-ground liquefaction does not involve large lateral displacements but is easily identified by the presence of sand broils produced by groundwater forming a circle on leveled surfaces. Although not particularly damaging by themselves they are a great indicator for very high groundwater pressure which can damage the surrounding area when the pressure reaches a critical level.

The effects of local sites are a complicated matter and are explained in greater detail in section 2 of this thesis. It presents the concepts of conditions for triggering and understanding as well as practical procedures on liquefaction and other site effects.

1.4.6 Possible mitigation of damage

Ultimately it is in everyone's best interest to minimize the damage caused by future earthquakes. For this purpose seismic hazard mitigation should be embedded in every new building being constructed. However unless it is enforced by law it is up to the people in charge who ordered the construction of the building in question, whether or not and what kind of earthquake countermeasures they want to install. Earthquake resistant design is a relatively complicated issue and therefore it costs often more than the owners are willing to pay. This act is irresponsible especially if the building is located in an earthquake prone area.

Earthquake damage can be assessed by thorough investigation of the surrounding area. The type of soil or rock it is placed on has to be determined to predict ground motion caused by earthquakes of a particular magnitude. While earthquakes of great magnitude cause often tremendous damage and their effects are harder to counter it is often more important to ensure that earthquakes of moderate magnitude do not cause damage due to bad construction. Determining local effects is also a very important aspect of prevention for they can amplify incoming seismic waves and create damage greater than the normal scope of damage an earthquake with a particular magnitude can possibly cause. A good example of the importance is the damage in Mexico City caused by site effects during the 1985 earthquake with epicenter close to the pacific coast.

This is a big issue for older cities because earthquake resistant design is relatively new. For best results the majority of the buildings have to be attuned to predicted ground motion. In that case the buildings actively reduce ground motion and thus the damage. This scheme can be applied in new cities that are being constructed. For instance the area around the city of Istanbul is literally waiting for a big earthquake. And since it has many old buildings, which have little or no modern earthquake resistant design, it would be very expensive to reconstruct practically the whole city. Therefore it was proposed to construct a satellite city near Istanbul with an earthquake resistant design (Source: Purdue University, West Lafayette, USA)

1.5 Ground motion parameters

This chapter closely follows Kramer (1996). Ground motion parameters are essential for describing all the important aspects occurring during an earthquake in a quantitative way. Many parameters have been proposed. Some characterize only one parameter, while others characterize two or three at the same time. This has proven to be quite complex and it is believed, that it is almost impossible to have one ground motion parameter which describes accurately all important ground motion characteristics (Jennings, 1985; Joyner and Boore, 1988).

1.5.1 Peak acceleration

One of the most common ground motion parameters is the peak acceleration. It is divided into two categories, one being peak horizontal acceleration (PHA) and the other peak vertical acceleration (PVA). PVA is considered not so important for THE engineering purposes because of gravity and engineers consider PVA being 2/3 PHA, although more recent observations have proven that PVA is quite variable but generally it is higher than 2/3 PHA when in close proximity to moderate to large earthquakes and less than 2/3 PHA at greater epicentral distances (Campbell, 1985;

Abrahamson and Litehiser, 1989). PHA for a given component of motion is simply the largest absolute value obtained from a seismogram of that component.

Ground motion with very high peak acceleration is considered to be more destructive than ground motion with low peak acceleration. Very high peak accelerations that do not last long deal only little damage to many structures. However, the largest dynamic forces produced in stiff buildings originate from peak acceleration. Although peak acceleration is a very important ground motion parameter it provides no information on the frequency content or duration and therefore other ground motion parameters have to be defined to accurately measure ground motion characteristics.

1.5.2 Peak velocity

Peak horizontal velocity (PHV) is another useful parameter which describes ground motion. PHV is less sensitive at higher frequency ranges and is therefore a much more reliable source of information for damage prediction to buildings sensitive to intermediate frequencies.

1.5.3 Peak displacement

Peak displacement is generally associated with lower frequency components of an earthquake motion. However, the peak displacement is often quite difficult to determine accurately (Campbell, 1985; Joyner and Boore, 1988), due to the signal processing errors and background noise. Therefore the peak displacement is a less commonly used ground motion parameter than PHA or PHV.

1.5.4 Frequency content parameters

Earthquakes produce motions with a broad range of frequencies and it is easy to show that loaded objects are sensitive to certain frequencies and it is therefore important to determine what frequency a given earthquake produces. Characterization of motion cannot be complete without it.

1.5.4.1 Fourier spectra

The Fourier amplitude spectrum shows how the amplitude of the motion is distributed with respect to frequency or period during strong ground motion. The spectrum provides a clear picture of the frequency content of motion. It can be narrow or broad. If the spectrum is narrow it implies that the motion has a dominant frequency. This can produce a smooth almost sinusoidal time history on a seismogram. A broad spectrum on the other hand produces very irregular patterns on a seismogram and implies that this motion contains a large number of frequencies. The Fourier spectra of ground motion are usually plotted on logarithmic scales. This procedure ensures that the characteristic shape can be more easily identified. On this scale it is easily observable that the Fourier acceleration amplitudes are largest over an intermediate area bound by the corner frequency f_c on the low side and the cutoff frequency $f_{\max}$ on the high side. The corner frequency is inversely proportional to the cube root of the seismic moment (Brune, 1970, 1971). This result indicates that large earthquakes produce higher amplitudes in the low frequency ranges. The cutoff frequency is not well understood. It has been characterized as a near-site effect (Hanks, 1982) as well as a source effect (Papageorgiou and Aki, 1983), and is usually characterized as a constant in a given geographic region.

1.5.4.2 Power spectra

The total intensity of a ground motion of duration T_d is given by the area under the time history of squared acceleration in the time domain:

$$I_0 = \int_0^{T_d} [a(t)]^2 \, dt . \tag{1.10}$$

Using Parseval's theorem, the total intensity can also be expressed in the frequency domain as

$$I_0 = \frac{1}{\pi} \int_0^{\omega_N} c_n^{\ 2} \, d\omega , \tag{1.11}$$

where $\omega_n = \pi / \Delta t$ is the Nyquist frequency. The average intensity λ_0 can be obtained by dividing equations (1.10) and (1.11) by the duration T_d.

$$\lambda_0 = \frac{1}{T_d} \int_0^{T_d} [a(t)]^2 \, dt = \frac{1}{\pi T_d} \int_0^{\omega_n} c_n^{\,2} \, d\omega . \qquad (1.12)$$

The average intensity λ_0 is equal to the mean-squared acceleration. The power spectral density $G(\omega)$ is defined as

$$\lambda_0 = \frac{1}{\pi T_d} \int_0^{\omega_N} G(\omega) \, d\omega \qquad (1.13)$$

and by comparing equation (1.13) and (1.14) we get

$$G(\omega) = \frac{1}{\pi T_d} c_n^{\,2} . \qquad (1.14)$$

The power spectral density function is useful in characterizing the earthquake as a random process namely as a stationary random process whose statistical parameters do not vary over time.

1.5.5 Spectral parameters

The Fourier amplitude spectrum and power spectral density, which is closely related, are very useful because they can characterize ground motion completely. However these are really complicated functions and a great deal of data has to be aquired to describe them completely. Therefore a number of spectral parameters have been proposed to extract pieces of information from these spectra.

1.5.5.1 Predominant period

This parameter represents the frequency content. It is defined as the period which corresponds to the maximum amplitude in the Fourier amplitude spectrum.

1.5.5.2 Bandwidth

Two completely different spectra can have the same predominant period therefore the area around the predominant period has to be taken into account to have a more objective value. Bandwidth is usually measured between points when the power of the spectrum drops to a level of ½ its maximum value, which corresponds to $1/\sqrt{2}$ times the maximum Fourier amplitude. Bandwidth is usually obtained from smoothed spectra, as it makes them easier to determine where to start and end the measurement.

1.5.5.3 Central frequency and shape factor

The *n*th spectral moment of ground motion is defined as

$$\lambda_n = \int_0^{\omega_N} \omega^n \, G(\omega) \, d\omega \, . \tag{1.15}$$

The central frequency Ω (Vanmarcke, 1976) is given by

$$\Omega = \sqrt{\frac{\lambda_2}{\lambda_0}} \, . \tag{1.16}$$

It is a measure of power spectral density and indicates where it is concentrated. To indicate the dispersion about the central frequency Vanmarcke (1976) used the shape factor

$$\delta = \sqrt{1 - \frac{\lambda_1^{\,2}}{\lambda_0 \, \lambda_2}} \, , \tag{1.17}$$

which always lies between 0 and 1. Higher values correspond to larger bandwidths.

1.5.6 Combined ground motion parameters

Ground motion parameters which were stated above characterized only a single parameter in the frequency or time domain. Several parameters have been proposed which characterize at least two ground motion parameters. The root-mean-square acceleration characterizes the effects of amplitude as well as the frequency content of motion and is defined as

$$a_{rms} = \sqrt{\lambda_0} = \sqrt{\frac{1}{T_d}\int_0^{T_d} [a(t)]^2 \, dt} \tag{1.18}$$

Since it is dependent on T_d its value is dependant of the definition of time duration of an earthquake. A parameter closely related to the root-mean-squared acceleration is the Arias intensity (Arias, 1970), which is defined as

$$I_a = \frac{\pi}{2g}\int_0^{\infty} [a(t)]^2 \, dt \,. \tag{1.19}$$

Seismic hazard analyses contain a variety of information and therefore it is only rarely appropriate to characterize ground motion by a single parameter. Different engineering problem require the use of different parameters to effectively solve them.

1.6 Seismic hazard analysis

This section contains several outtakes from Kramer (1996). The threat from earthquakes to manmade constructions cannot be ignored. Therefore earthquake resistant design has a goal to reduce the damage caused by earthquakes with a specific amount of ground motion called the design ground motion. Design ground motion can be characterized by design ground motion parameters. Determining these parameters is one of the most difficult and most important problems in geotechnical engineering.

Much of its difficulty comes from having to make decisions based on uncertain and subjective decisions. For instance a decision has to be made how much damage is still acceptable. The exact location, time and magnitude of a future earthquake also cannot be determined with adequate accuracy. If very little damage is acceptable then the level of predicted shaking is greater to minimize damage. However buildings constructed to withstand a large earthquake have proven to be quite expensive. Therefore a greater level of damage is often acceptable for minimizing the costs required to construct the building. These trade-offs often present a hard decision to be made.

Seismic hazard analyses involve the quantitative estimation of ground shaking hazards at a particular site. They can be analyzed deterministically when a particular earthquake scenario is assumed. Probabilistic scenarios are used to demonstrate uncertainties in earthquake size, location and time of occurrence.

1.6.1 Identification of earthquake sources

With the availability of modern seismographs identification of seismic sources has become much more easier than in the past. The occurrence of a significant earthquake is recorded by several seismographs and scientists can evaluate in mere hours their magnitude, locate the source rupture surface and even evaluate source parameters.

The fact that no strong motion has been recorded in a particular area does not guarantee that they have not occurred in the past or that they will not do so in the future. In the absence of reliable instrumental data other sources of evidence that implicate seismicity have to be taken into account.

The theory of plate tectonics indicates that the occurrence of earthquakes is written in the geologic record, primarily in the form of offsets, or relative displacements, of various strata. In some parts of the world this geologic record is relatively easy to find. In other parts of the world, however, this record is very complex in its nature or hidden. The search for geologic evidence of earthquake sources centers around identification of faults. Criteria for identification of faults are described in numerous textbooks on structural geology, field geology and geomorphology (Adair, 1979).

Even after locating and identifying a fault it is still important to determine whether the fault is active or not. Active faults represent an ongoing earthquake threat while inactive faults are deemed unlikely to produce an earthquake. There were several definitions concerning the term active fault, most of them revolving around how much time has passed since the last earthquake ranging from 10 000 years to 100 000 years (Idriss, 1985). This specification of fault activity is in fact not realistic for faults do not become inactive on a particular anniversary. Cluff and Cluff (1984) suggested six classes of fault activity based on characteristics as slip rate, slip per event, rupture length, earthquake size and recurrence interval. This categorization offers more satisfying framework for characterization of fault activity but can be a little difficult to implement in political and economic environment in which many seismic hazard analyses are conducted.

Studies of worldwide earthquakes have shown that faults do not rupture over their entire length but instead individual segments rupture repeatedly (Schwartz and Coppersmith, 1986; Schwartz, 1988). Rupture length can be evaluated after an earthquake by field geological instruments and by processing the data that was collected, the magnitude of that earthquake can be estimated, although this estimate can, and often is, quite uncertain and this fact has to be taken into account. Rupture length methods are best suited to cases in which the rupture surface is fairly narrow, typically less than 20 km (Bonilla et al., 1984).

1.6.2 Deterministic seismic hazard analysis

The deterministic seismic hazard analysis revolves around creating a hypothetical seismic scenario and predicting its effects. The scenario consists of postulated occurrence of an earthquake of a specified size occurring at a specified location and can be typically described in 4 steps (Reiter, 1990):

1. Identification and characterization of all earthquake sources capable of producing significant ground motion at the site.
2. Selection of source-to-site distance parameter, usually the shortest distance.

3. Selection of the “controlling earthquake” (the earthquake that is expected to produce the strongest level of ground shaking). The controlling earthquake is described usually in terms of its size (magnitude) and distance from the site.

4. Formal definition of the hazard at the site usually in terms of ground motion produced at the site by the controlling earthquake.

The deterministic seismic hazard analysis provides a decent framework for the worst case scenario at the site. However it does not take into account the likelihood of such an occurrence, level of shaking expected during a finite period of time nor the various uncertainties in the various steps required to compute the resulting ground motion characteristics.

The most important issue is that it involves subjective decisions that require the combined opinions and expertise of seismologists, engineers and economists as well as government officials. The broad range of backgrounds and often divergent goals of such professionals can cause difficulty in reaching a consensus regarding the earthquake’s potential and has led to delay and even cancellation of a number of large construction projects.

1.6.3 Probabilistic seismic hazard analysis

Probabilistic seismic hazard analysis provides a framework in which uncertainties can be identified, quantified and combined in a rational manner to provide a more complete picture of the seismic hazard. It can also be described as a procedure of 4 steps (Reiter, 1990), each of which bear some degree of similarity to the steps of deterministic seismic hazard analysis procedure:

1. The first step is identical to the first step of the deterministic seismic hazard analysis, except that the probability distribution of potential rupture locations within the source is also characterized. In most cases, uniform probability distributions are assigned to each source zone, implying that earthquakes are equally likely to occur at any point in the source zone. The deterministic seismic hazard analysis implicitly assumes that the probability of occurrence on all sources is 1 at the points closest to the site and 0 elsewhere.

2. A recurrence relationship, which specifies the average rate at which an earthquake of some size will be exceeded, is used to characterize the seismicity of each source zone. This may be used in accommodation of the maximum possible

earthquake but it leaves some degree of consideration to that earthquake, which deterministic seismic hazard analyses do not.

3. With the uncertainty in mind, ground motion produced at the site by earthquakes of all possible magnitudes started at any possible point in a source location must be predicted with the use of predictive relationships.

4. Finally all uncertainties of earthquake location, size and ground motion are put together and form the probability that the ground motion will be exceeded during a particular time period.

2 Site effects

In the last few decades the importance of thorough study of site effects has been discovered. During observations scientists have spotted a recurrence that implies that buildings having foundations on solid rock suffer less damage than buildings situated on soft soils. This has proven to be an important discovery, because it has become clear that site effects are the number one cause of damage during earthquakes. The availability of strong-motion instruments has enabled the quantitative study of the site effects. Correct accounting for the site effects leads to a decent earthquake design in the building codes.

Local site conditions influence some or all of the important ground motion parameters – amplitudes, frequency content, duration. The extent of influence is geometry and material dependent. There is also theoretical evidence which implies that ground motion should be influenced by site effects. At most sites the density and S-wave velocity of materials is smaller than in greater depths. By neglecting the effects of scattering and material damping, the conservation of elastic energy requires the flow of energy ($\rho\, v_s\, \dot{u}^2$) to be constant. Therefore since the density (ρ) and S-wave velocity (v_s) decrease with decreasing depth, particle velocity ($\dot{u}$) must increase. The characteristics of local soil deposits can also have influence in ground motion amplification.

Here we closely follow Kramer (1996). We consider the wave equation of damped soil

$$\rho \frac{\partial^2 u}{\partial t^2} \;=\; G\frac{\partial^2 u}{\partial z^2} \;+\; \eta \frac{\partial^3 u}{\partial z^2\, \partial t}, \tag{2.0}$$

where ρ is the density of the material, u is the particle displacement, G is the shear modulus and η is viscosity of the material. The solution to this wave equation is

$$u(z,t) \;=\; A e^{i(\omega t \,+\, k^* z)} \;+\; B e^{i(\omega t \,-\, k^* z)}, \tag{2.1}$$

where *k** is the complex wave number with real part k_1 and imaginary part k_2. The transfer function for the case of the damped soil over a rigid halfspace can be expressed as

$$F_2(\omega) \;=\; \frac{1}{\cos(k^* H)} \;=\; \frac{1}{\cos\left(\dfrac{\omega H}{v_s{}^*}\right)} \qquad \left(k^* \;=\; \omega\sqrt{\frac{\rho}{G^*}}\right), \tag{2.2}$$

where *H* is the thickness, $v_s{}^*$ complex shear wave velocity, ρ density and *G** the complex shear modulus of the soil deposit. The complex shear modulus *G** can be expressed as

$$G^* \;+\; G(1 \;+\; i\,2\,\xi), \tag{2.3}$$

where ξ is the damping ratio. Therefore, for small ξ, the complex shear velocity is

$$v_s{}^* \;=\; \sqrt{\frac{G^*}{\rho}} \;=\; \sqrt{\frac{G(1 \;+\; i\,2\,\xi}{\rho}} \;\approx\; \sqrt{\frac{G}{\rho}}\left(1 \;+\; i\,\xi\right) \;=\; v_s\,(1 \;+\; i\,\xi) \tag{2.4}$$

and the complex wave number *k** is

$$k^* \;=\; \frac{\omega}{v_s{}^*} \;=\; \frac{\omega}{v_s\,(1+i\,\xi)} \;\approx\; \frac{\omega}{v_s}(1 \;-\; i\,\xi) \;=\; k(1 \;-\; i\,\xi) \tag{2.5}$$

and implementing (2.4) and (2.5) into the transfer function (2.2) we get

$$F_2(\omega) \;=\; \frac{1}{\cos[k\,(1 \;-\; i\,\xi)\,H} \;=\; \frac{1}{\cos\left[\dfrac{\omega H}{v_s\,(1 \;+\; i\,\xi)}\right]}. \tag{2.6}$$

Then using the identity $|\cos(x \;+\; i\,y)| \;=\; \sqrt{\cos^2 x \;+\; \sinh^2 y}$ we get

$$|F_2(\omega)| \;=\; \frac{1}{\sqrt{\cos^2(k\,H) \;+\; \sinh^2(\xi\,k\,H)}} \tag{2.7}$$

and since $\sinh^2 x \;\approx\; x^2$ for small x, the amplification function takes the form of

$$|F_2(\omega)| \approx \frac{1}{\sqrt{\cos^2(k\,H) + (\xi\,k\,H)^2}} = \frac{1}{\sqrt{\cos^2\left(\frac{\omega H}{v_s}\right) + \left[\xi\left(\frac{\omega H}{v_s}\right)\right]^2}}. \tag{2.8}$$

This amplification equation (2.8) indicates that amplification on damped soils depends on the frequency. A local maximum of the amplification is reached whenever the argument of cosine is zero, but will never reach infinity because the denominator will always be greater than zero. That is because there is not a single material which has a damping ratio $\xi = 0$. The frequencies that correspond to the local maxima of the amplification function (2.8) are called natural frequencies of the soil deposit. The *n*th natural frequency is given by

$$\omega_n \approx \frac{v_s}{H}\left(\frac{\pi}{2} + n\,\pi\right) \qquad n \in N_0. \tag{2.9}$$

Because of the nature of the amplification function, the highest amplification will occur at the lowest frequency ω_0. This frequency is called the fundamental frequency and is given by

$$\omega_0 = \frac{\pi\,v_s}{2\,H}. \tag{2.10}$$

The corresponding period T_s to the fundamental frequency ω_0 is called the characteristic site period and is given by

$$T_s = \frac{2\pi}{\omega_0} = \frac{4\,H}{v_s}. \tag{2.11}$$

It is important to note that the characteristic site period is only influenced by the thickness and the shear wave velocity of the soil. This period provides a very useful indication of the period on which the most important resonance effects will occur. It is also important to note that at each natural frequency ω_n a standing wave will be formed. Also the soil is in phase at the fundamental frequency but may not be at higher natural frequencies. At higher frequencies parts of the soil may be moving in one direction while other parts may be moving in a different direction. All this phenomena have to be accounted for in site effect studies, because they contribute to them.

Now we consider two soil deposits with identical geometries but one is significantly stiffer than the other. And by assuming that these soils are linearly elastic it is now clear that the softer site will produce low-frequency amplification while the stiffer side produces high-frequency amplification.

2.1 Topography

Surface topography is another aspect, which has to be accounted for. The best example how topography affects spectral amplification was apparently the 1971 San Fernando ($M_L = 6.4$) earthquake which produced about 1.25g peak horizontal acceleration, more than was expected from an earthquake of this size. The acceleration occurred due to the effect of the narrow ridge adjacent to a dam (Trifunac and Hudson, 1971). The effects caused by simple irregularities in the free-surface topography can be estimated from exact solutions to idealized problems (Aki, 1988). Mostly, the peak of the ridge can be roughly compared to a triangular wedge. After this simplification we consider propagation of SH-waves. Displacements have shown to be amplified by a factor of $2\pi/\theta$ where θ is the vertex angle of the wedge. This method provides a simple yet effective estimation of peak amplifications at ridges and can be used in most cases. Jibson (1987) states that during an earthquake on a mountain ridge at Matsuzaki, Japan, five different points were under observation from the crest to the peak. Peak amplification has proven to be 2.5 times greater at the peak, compared to the peak acceleration at the crest. Another similar pattern was observed by the damage pattern in some of the earthquakes in Italy and Chile (Finn, 1991). The recent Kozani earthquake, Greece, of May 1995 brought again evidence of severe damage in villages built on hilltops (Bard and Riepl, 2000). The analysis of topographic effects is a complicated problem depending on a variety of parameters like angle of incidence of the waves, their frequency and the type of geometrical irregularity.

2.2 Basins

Softer alluvial soil deposits surrounded by thick bedrock are referred to as basins in geotechnical engineering. It is of great importance to predict ground motion on basins for many cities have been built on basins or near them. Basins can create amplifications of peak ground motion due to the fact that they can trap incoming waves and reflect them at the edge of the basin, possibly causing them to propagate to the surface as surface waves. This can produce stronger ground motion as well as an increase in duration than would be predicted by simple one-dimensional (1D) analysis which considers only vertically propagating S-waves

They measured ground motions along the Chusal Valley near the Afghanistan border. Interpretation of response from small earthquakes suggested that 1D prediction would predict only the average response of the sediments at the center. At the edge of the basin, the amplification function was different and not predicted by simple 1D analysis.

Bard and Gariel (1986) generalized the studies from the years 1980 to 1985 for purposes of evaluating the gradient of velocity in sedimentary basins. They compared the response of a shallow and deep alluvial valley using 1D and 2D analyses. Their results were that for the shallow valley, the amplification function was very similar both in 1D and 2D analyses in the center of the valley. This was the case also in the deep valley but the similarity of those functions was not as good as in the shallow valley. However, at the edges of both basins the 1D and 2D results were quite different and have proven that simple 1D analysis can only be used to predict ground motion amplifications at the center of a shallow alluvial valley.

Also the potential for significant differential motion has an important impact on the design of long-span structures like bridges which are often constructed across valleys. The differential motion, which can result in quite complex motion in valleys of irregular shape, can cause heavy load of stress to these types of structures.

Evaluation of topographic and subsurface irregularities requires 2D and in some cases 3D analyses, which are often complicated, time consuming and require a detailed site characterization. These effects are difficult to predict but their existence is non-questionable. Silva (1988) summarized the effects of topography and subsurface irregularities in Table 2-1. Comments on their quantitative predictability are also present.

Structure	Conditions	Type	Size	Quantitative Predictability
Surface topography	Sensitive to shape ratio, largest for ratio between 0.2 and 0.6; most pronounced when λ = mountain width	Amplification at top of structure, amplification and deamplification at base, rapid changes in amplitude phase slopes	Ranges up to a factor of 30 but generally from about 2 to 10	Poor: generally unpredict size; may be due to ridge-ridge interaction and 3D effects
Sediment-filled valleys	Local changes in shallow sediment thickness	Increased duration	Duration of significant motions could be doubled	Fair
	Generation of long-period surface waves from body waves at shallow incidence angles	Increased amplification and duration due to trapped surface waves	Duration and amplification of significant motions may be increased over 1D projections	Good at periods exceeding 1 second
Shallow and wide (depth/width < 0.25) sediment-filled valleys	Effects most pronounced near edges; largely vertically propagating shear waves away from edges	Broadband amplification near edges due to generation of surface waves	1D models may under-predict at higher frequencies by about 2 near edges	Good: away from edges 1D works well, near edges extent 1D to higher frequencies
Deep and narrow (depth/width > 0.25) sediment-filled valleys	Effects throughout valley width	Broadband amplification across valley due to whole valley modes	1D models may under-predict for a wide bandwidth by about 2 to 4; resonant frequencies shifted from 1D	Fair: given detailed description of vertical lateral changes in material properties

Table 2-1 Effects of Topographic and Subsurface Irregularities (after Silva; 1988)

2.3 The Loma Prieta earthquake

On October 19th a M_s = 7.1 earthquake occurred at Mt. Loma Prieta located about 100 km south of San Francisco and Oakland, California, USA. It produced a MMI VIII shaking in the epicentral region, but was actually more intense in portions of San Francisco and Oakland (MMI IX). This earthquake caused severe damage in certain areas while only minimal damage was caused to other areas. This suggested that local effects had a great deal on damage distribution throughout the area.

The San Francisco Bay is largely filled with alluvial deposits of clays and some layers of sandy and gravelly soils. This material is called the San Francisco Bay Mud and is highly compressible with strength grades from soft near the ground to medium stiff at depth. The San Francisco Bay Mud is usually found in direct vicinity of the San Francisco Bay. Its thickness ranges from zero up to several tens of feet. For purposes of seismic zonation the Bay area can be divided into three zones – Rock/Shallow Residual Soil Zone, Alluvium Zone and Bay Mud Zone. Both the epicentral region as well as the San Francisco Bay area were well instrumented with seismograms. Peak horizontal accelerations were quite high but dissipated with distance from their respective sources. However the dissipation has proven to be more rapid in the Rock/Shallow Residual Soil Zone than in the remaining zones. The response data from two instruments has proven to be particularly useful – these instruments were located on Yerba Buena Island and Treasure Island. The Yerba Buena Island is a 400-acre man-made hydraulic fill with a rock outcrop while the Treasure Island is underlain by loose sandy soil over the San Francisco Bay Mud. Both of these reference sites were located at practically the same distance from the source but recorded dramatically different results of ground surface motion. Peak accelerations at Yerba Buena Island were 0.06g in the E-W direction and 0.03g in the N-S direction, while at Treasure Island these values were 0.16g and 0.11g.

The amplification at Treasure Island clearly occurred due to the presence of soft soils. Evidence on how selective was the damage provides the Cypress Viaduct which had only its northern portion collapsed. The northern part was situated on the San Francisco Bay Mud while the southern part was situated on the rigid bedrock.

2.4 The Michoacan earthquake

On September the 19th in 1985, a $M_s = 8.1$ earthquake caused only moderate damage in the vicinity of its epicenter near the Pacific coast of Mexico. However, it caused significant damage roughly 360km away in Mexico City. Several reference sites in Mexico City recorded and illustrated the significant relationship between local soil conditions and structural damage.

For seismic purposes, Mexico City can be divided into three zones with different soil conditions – the Foothill Zone located west of downtown, the Lake Zone, and, in between, the Transition Zone. The Foothill Zone contains mainly shallow, compact deposits of mostly granular soil, basalt or volcanic tuff. The Lake Zone contains thick deposits of quite soft soils. These soils consist generally of two soft clay layers called Mexico City Clay separated by a thin (~5 m) compact sandy layer called the "capa dura". In the Lake Zone, groundwater can be usually found in a depth of about 2 m. In the Transition Zone soft soil deposits are randomly intersected with alluvial deposits. Prior to the earthquake, Mexico City had strong-motion instruments installed which provided thorough data of this event. The Universidad Nacional Autonoma de Mexico (UNAM) was a site located at the Foothill Zone while the Secretary of Communications and Transportation (SCT) site was located on the soft soils of the Lake Zone.

At the UNAM site, the Michoacan event produced only peak accelerations from 0.03g to 0.04g. In the Transition Zone the accelerations were slightly higher but still not really high. In the Lake Zone however, peak accelerations at the SCT were up to five times greater than those at the UNAM site. The frequency content was also quite different. At the SCT site, the predominant period was about 2 sec. Strong shaking which caused severe damage in certain areas was also amplified by longer duration at the SCT site. The spectral accelerations were about 10 times higher at the SCT site than at the UNAM site.

The damage was also very selective and corresponded well with the seismic zones. Damage in the Foothill zone was negligible and minimal in the Transition Zone. However, in the Lake Zone, extensive damage was caused mainly to parts that were underlain by 38 to 50m of soft soil. These parts had a characteristic period ranging from 1.9 to 2.8 sec and this corresponds with the data that was recorded. Even then, most buildings with less than 5 stories or modern design buildings greater than 30 stories suffered only slight damage. The buildings in this area in the 5 to 20 range either collapsed or suffered extensive damage. Using a crude rule of thumb in determining the fundamental frequency of an N-story building as $N/10$ sec, most of the damaged buildings had a fundamental frequency somewhere around or equal to the fundamental frequency of the site. This resonance effect caused buildings to shake at their fundamental frequency for a period of time which led to a great buildup of large dynamic forces. Combining this with a poor to none seismic design leads to a locally devastating damage (Kramer, 1996).

3 Methods for estimating site effects

Some critical facilities require a thorough study of the site in question while other projects require only minimal knowledge. There are several methods available for site characterization – experimental, numerical and empirical approach. These approaches can be either quite expensive or totally inexpensive.

3.1 Experimental methods

3.1.1 Macroseismic observations

It can happen, especially in very seismo-active areas, that a particular area in question has undergone a destructive earthquake and detailed macroseismic observations are available. Then a detailed analysis of data from geotechnical and topographical maps can uncover a qualitative estimate of the most hazardous zones. This approach was first used in Tokyo as early as 1913, the data from a previous earthquake in 1854 enabled a division of the city into 3 separate zones each with their own hazard level (Bard and Riepl, 2000). Detailed macroseismic observations right after destructive earthquakes are of prime importance for microzonation efforts. Also a careful survey of insurance files after a damaging earthquake often proves well correlated with subsurface conditions (Jongmans and Campillo, 1990).

3.1.2 Microtremors

Microseisms and microtremors are terms used to define seismic activity caused by natural, ambient sources, also called background noise. They are caused by wind, sea, traffic and other sources and are recorded using highly sensitive seismometers. Since the early work by Kanai (1983) evidence was found that correlates the data obtained from microtremors to a site's geological condition. However principles based on microtremor observations are used almost exclusively in Japan. In other countries it is believed that several questions are not yet satisfactory answered as, e.g., large uncertainties in relative spectral amplitudes which represent not only site characterization but also site and path effects. There is also quite a difference between day and night data.

Even though it has its limitations, efforts for widespread use are being taken because microtremor data acquisition is one of the least expensive methods available for site characterization.

3.1.2.1 Microtremor spectra

The crudest way is simply determining the peak frequencies obtained from average spectra. Short dominant microtremor periods (< 0.2 sec) is an indicator of rather stiff rock while larger periods indicate for a correspondingly softer and thicker deposits and is a good qualitative estimation of a site (Kanai, 1983). Peak spectral amplitudes in the long period range ($T > 1$ sec) are also reported to be correspondent to fundamental frequencies at the sites in question. However, in the short period range, recent data provides controversial results (Bard and Riepl, 2000).

3.1.2.2 Spectral ratios

Spectral ratios for noise recordings are analogous to earthquake recordings. However, such recordings are only appropriate in the long period range where the source of the noise is the same for all studied locations including the site itself. At short period range, Guttierez and Singh (1992) obtained a very good qualitative agreement with strong motion ratios, but a significant quantitative mismatch. It is believed that this technique may be used when the reference site is very close to the site under investigation (Seo, 1998).

3.1.2.3 H/V ratio (Nogoshi-Nakamura's technique)

The H/V ratio is the ratio between the horizontal and vertical components of the Fourier spectra obtained at the same station recording microtremors. This approach was introduced in the early seventies by several Japanese scientists (Bard and Riepl, 2000). The physical importance of this ratio was being assessed. It later showed that this ratio is related to the elliptical trajectory of Rayleigh's waves (a type of surface wave). They also concluded that this ratio can be used to identify the fundamental frequencies of soft soils. This has been deduced from the fact that the vertical motion of Rayleigh waves almost vanishes in the vicinity of the fundamental S wave resonant frequency. Due to the fact that their publications were only available in Japanese, this fact had not been conveyed to the outside of Japan until Kudo's (1995) review.

Meanwhile Nakamura (1989) proposed on the basis of qualitative arguments, that a reliable source of information on the site response to the S waves is the H/V spectral ratio. It provides information not only on resonant frequencies but also about the corresponding amplification. But some scientists believe that no straightforward relation exists between the H/V peak amplitude and the site peak amplification. But this view is not shared unanimously. A thorough comparison between observed amplifications derived from earthquake records and derived from H/V peak amplitudes at more than 30 sites demonstrates that the H/V peak amplitude is almost always lower than the actual earthquake produces. This information, if proven correct, could provide with a lower estimate of actual peak amplification. This view needs to be confirmed by a larger set of experimental data. Another aspect worth noting is that Nakamura's technique is a very simple and cheap way to determine a site's fundamental frequency, which is a very important parameter when considering site effects.

3.1.2.4 Array recordings

Aki (1957) showed that noise recordings on small aperture arrays could be used as a way to measure phase velocities of surface waves. It is based on analyzing the spatial correlation of microtremors and obtaining by inversion the surface velocity. From here it is possible to determine the site's response. This approach relies heavily on computing power. By combining this approach with Nakamura's technique (array measurements at a few sites and H/V measurements at many sites) they believe, that they may have found a reliable way to map 2D and 3D subsurface conditions (Gitterman et al., 1996).

3.1.3 Weak-motion data

Weak-motion data records are from small to moderate seismic events of both natural as well as artificial origin. The greatest challenge in the estimation of the site response using this data is removing the source and path effects. The several methods proposed can be divided into two main groups. One needs a reference site, the second one does not need it.

3.1.3.1 Techniques requiring a reference site

The most common procedure is comparing the response from two stations located near each other. For these stations, the source and path effects are considered the same. This proves a reliable source of information if the sites in question are free of site effects. Therefore it has to fulfill these conditions. It should be located near the site it compares its spectral ratios to. This condition is deemed fulfilled when the epicentral distance is greater than 5 times the array aperture. And then it has to lie on unweathered horizontal bedrock which is free of site effects. Fulfilling these conditions has proven to be quite restrictive (Bard and Riepl, 2000). The principle of these methods can be described as follows. Here we closely follow Bard and Riepl (2000). For a network of i sites having recorded j events, the amplitude spectrum of the recorded ground motion $R_{ij}(f)$ can be written as

$$R_{ij}(f) = E_j(f) P_{ij}(f) S_i(f), \tag{3.1}$$

where $E_j(f)$ is the source function, $P_{ij}(f)$ is the path contribution between the site and the source and $S_i(f)$ is the contribution of the local site. This can be expressed in the logarithmic form as

$$\ln[R_{ij}(f)] = \ln[E_j(f)] + \ln[P_{ij}(f)] + \ln[S_i(f)]. \tag{3.2}$$

The traditional spectral ratio technique corresponds well with the case where the path $P_{ij}(f)$ is considered to be site independent. This is the case when the distance to the reference site is small compared to the source-to-site distance. Then, $i+j$ terms

$E_j(f)$ and $S_i(f)$ are estimated from *i*j* observations, provided that all *i* stations recorded all *j* events. Since both series of terms are determined only by their product it is only needed to determine one term, as the other, generally $S_i(f)$ (which is usually taken by default as 1 at a reference site i_0).

It has proven to be quite preferable to follow this generalized inversion technique, for it allows reliable estimate of site effects even when only a few recordings at the site at hand are available (Bard and Riepl, 2000). However, the traditional spectral ratio technique is better suited for sites, where noise level varies between stations or when the response at certain sites is exceptionally more variable than it is at other sites.

3.1.3.2 Techniques that do not require a reference site

In the traditional spectral ratio method and in the generalized inversion approach, site and source effects are estimated from observations at a reference site. In practice, adequate reference sites are not always available. For this reason, different methods not needing reference sites have been developed. The general form of the source and path terms may be assumed through formulae providing the spectral shape as a function of a few parameters. These can be the corner frequency, the seismic moment, the quality factor *Q*, the near-site attenuation term or the dominant frequency $f_{\max}$. Although these methods have been first proposed to eliminate site effects and improve estimates of source and path characteristics, they may also be used for the purpose of estimating site effects (Bard and Riepl, 2000)

Although the parameterization of source and path effects does remove the need for a reference site, there still exists in this procedure an unconstrained, frequency-independent degree of freedom. However, the resulting unavoidable uncertainties in those parameters may easily double the value of the scaling factor. The inversion scheme is generally more complicated than in the general inversion approach since the dependence on some parameters (such as the corner frequency) is nonlinear (Bard and Riepl, 2000).

3.1.4 Strong-motion data

The development of strong-motion arrays makes it now possible to apply the previous weak-motion method on strong-motion data. In thes cases, there is no longer any question as to the reliability and applicability of the results. Non-linear effects are also included in the recordings. In urban areas such as Mexico City where the strong-motion network is triggered, on average, at least once a year, several specific techniques have been developed in recent years that allow deriving both reliable and detailed enough empirical microzonation rules. Recent methodological studies show that there is a fairly good agreement between old and new techniques. Except for those based on microtremor recordings, all reveal with comparable accuracy the frequency-dependent character of site amplification, at least for soft sites. The recently proposed techniques based on the H/V spectral ratio (using either microtremor or earthquake recordings) provides very simple and reliable estimates of site fundamental frequency, but further investigations are needed concerning its ability to measure the site amplification factor (Bard and Riepl, 2000).

3.2 Numerical methods

When the geotechnical characteristics of the area are known, site effects can, in principle, be estimated through numerical analysis. Such an approach, however, requires an in-depth understanding both of the analytical models and of the numerical schemes being used. When the required expertise is lacking, it may occur that sophisticated numerical analyses lead to less reliable results than simpler and cruder, but more robust, approximations. This section only provides a brief insight on the methods in use.

3.2.1 Simple hand calculations

Simple methods are available only for the estimation of the amplification on soft soils. As already stated in Chapter 2, amplification in soft soils is related to the resonance effects. Because the strongest effects generally occur at the fundamental frequency, the most simple numerical methods aim at estimating the fundamental period of the soil, T_0, and the corresponding amplification. Such a simple simultaneous estimation of these two parameters is indeed possible only for sites that can be approximated as a one layer over bedrock structure, for which the formulae (2.11) given in Chapter 2 may be applied. These formulae show that the estimation of T_0 is relatively easy, since only the S-wave velocity and the thickness of the surface layer are needed, while the estimation of the corresponding amplification requires the additional knowledge of the bedrock velocity and of the sediment damping as is explained in equation (2.8).

For sites with a tabular, multi-layer structure, hand calculations can provide satisfactory estimates of the fundamental period T_0, using for instance the formulae given in Dobry et al. (1976) and summarized in Table 3-1. According to this table, method No.3, which is relatively often used and consists in simply summing up the natural periods of each individual layer considered alone, is a wrong one, since it greatly overestimates the actual periods. The best of these is method No.5, based on the Rayleigh procedure to estimate the shape of the fundamental mode. However, methods using weighted averages of velocities (No.1) or rigidities and densities (No.2) also provide very simple and satisfactory estimates in usual conditions. Nevertheless, they fail completely in case of a thin soft layer at depth, embedded between much stiffer and thicker layers. There do not seem to exist any approximate formulae providing reliable estimates for the fundamental amplification in horizontally layered sites (Bard and Riepl, 2000).

3.2.2 Advanced methods

Since this is beyond the scope of this thesis we will only outline the basic principles used in advanced numerical methods.

Although all numerical methods are based on the wave equation, many different models have been proposed to investigate the several of the various aspects of site effects which involve complex phenomena. For example, one must consider various types of wave fields (near-field, far-field, body waves, surface waves); the structure geometry may be 1-D, 2-D or 3-D; or the mechanical behavior of the earth materials (i.e. the rheology) can be indeed very complicated (viscoelasticity, nonlinear soil behavior, water-saturated media, etc.). Advanced methods may be classified into five groups:

Table 3-1: Approximate methods for estimating the fundamental period T_0 of a horizontally layered soil (after Dobry et al.; 1976)

Method	Description	Mathemathical formulation	Comments
1	Weighted average of S wave velocities	$\overline{V} = \frac{\sum_{i=1}^{i=n} V_i H_i}{H}$ $T_0 \approx T_1 = \frac{4H}{\overline{V}}$	Slight mean overestimation: 10% to 15% Precision: about 30% Limitation: no important velocity jump between two contiguous layers ($V_i / V_{i-1} \in (0.5;1.5)$)
2	Weighted average of shear moduli and densities	$\overline{G} = \frac{\sum_{i=1}^{i=n} G_i H_i}{H}$ $\overline{\rho} = \frac{\sum_{i=1}^{i=n} \rho_i H_i}{H}$ $T_0 \approx T_2 = \frac{4H}{\sqrt{\overline{G} / \overline{\rho}}}$	Very slight mean overestimation: 5% Precision: about 30%
3	Sum of natural periods of each layer	$T_0 \approx T_3 = \sum_{i=1}^{i=n} \frac{4H_i}{V_i}$	Large mean overestimation: 25% to 30% Precision: about 40%
4	Linear approximation of the fundamental modal shape	${\omega_4}^2 = \frac{3\sum_{i=1}^{i=n} {V_i}^2 H_i}{H^3}$ $T_0 \approx T_4 = \frac{2\pi}{\omega_4}$	Very slight underestimation: 5% Precision: 25% to 30%
5	Simplified version of Rayleigh approach	$X_{i=1} = X_i + \frac{z_i + z_{i-1}}{{V_i}^2} H_i ; X_n = 0$ ${\omega_5}^2 = \frac{4\sum_{i=1}^{i=n} \frac{(z_i + z_{i-1})^2}{{V_i}^2} H_i}{\sum_{i=1}^{i=n} (X_i + X_{i-1)}{}^2 H_i}$ $T_0 \approx T_5 = \frac{2\pi}{\omega_5}$	No bias Precision: 5% No limitation

- Analytical methods can be used only for a very limited number of simple geometries.
- Ray methods are basically high frequency techniques and are difficult to use when wavelengths are comparable to the size of the heterogeneities.
- Boundary element techniques (including all kinds of boundary integral techniques, or those based on wave function expansions) are efficient if the site of interest consists of a limited number of homogeneous geological units.
- Domain-based techniques (such as finite-difference or finite-element methods) allow accounting for very complex structures and rheologies.
- Hybrid methods combine two or more individual methods in order to overcome limitations of the individual methods.

The main advantage of the advanced numerical methods rests in their flexibility and versatility which have lead to significant breakthroughs in the understanding of site effects during the last two decades. They allow not only carrying out phenomenological and parametric studies, but can also be used to assess the uncertainty in a site seismic response to the imperfectly known site conditions and its mechanical parameters (Bard and Riepl, 2000).

4 Conclusions

In this thesis we have summarized the most elementary viewpoints and occurrences during an earthquake as well as provided some information on how an earthquake motion can be predicted using deterministic or probabilistic approach. Then we have introduced several (but certainly not all) ground motion parameters which are essential in a quantitative measurement of earthquake effects and subsequently earthquake-resistant design.

In the second chapter we have introduced the site effects with the main preference of soft soil deposits and free-surface topography. Other site effects as liquefaction and landslides were only briefly mentioned.

In the third chapter a brief summary of quantitative measurements and predictions of site effects has been presented.

References

Abrahamson, N. A., Litehiser, J. J., 1989. Attenuation of vertical peak acceleration, *Bulletin of the Seismological Society of America* vol. 9, no. 1, pp. 31-34.

Adair, M. J., 1979. Geologic evaluation of a site for a nuclear power plant, *Geology in the Siting of Nuclear Power Plants,* Geological Society of America Reviews in Engineering Geology, Boulder, Colorado, vol. 4, pp. 27-39.

Aki, K., 1957. Space and time spectra of stationary stochastic waves with special reference to microtremors, *Bull. Earthq. Res. Inst.* vol. 35, pp. 415-456.

Aki, K., 1969. Analysis of the seismic coda of local earthquakes as scattered waves, *Journal of Geotechnique* vol. 37, pp. 615-631.

Arias, A., 1970. A measure of earthquake intensity, *Seismic Design for Nuclear Power Plants,* MIT Press, Cambridge, Massachusetts, pp. 438-483

Bard, P. Y., Gariel, J. C., 1986. The seismic response of two-dimensional sedimentary deposits with large vertical velocity gradientsm *Bulletin of the Seismological Society of America* vol. 76, pp. 343-356.

Bard, P. Y., Riepl, J., 2000. Wave propagation in complex geological structures and their effects on strong ground motion, Chp. 2, *Wave Motion in Earthquake Engineering,* Kausel & Manolis Edts, WIT Press.

Bolt, B. A., 1969. Duration of strong motion, *Proceedings of the 4th World Conference on Earthquake Engineering,* Santiago, Chile, pp. 1304-1315

Bolt, B. A., 1989. The nature of earthquake ground motion in F. Naeim, ed., *Thr Seismic Design Handbook,* Van Nostrand Reinhold, New York.

Bonilla, M. G., Mark, R. K., Lienkaemper, J. J., 1984. Statistical relations among earthquake magnitude, surface rupture length, and surface fault displacement, *Bulletin of the Seismological Society of America* vol. 74, pp. 2379-2411.

Boore, D. M., 1983. Stochastic simulation of high-frequency ground motions based on seismological models of the radiated spectra, *Bulletin of the Seismological Society of America* vol. 73, pp. 1865-1884.

Brune, J. N., 1970. Tectonic stress and the spectra of seismic shear waves from earthquakes, *Journal of Geophysical Research* vol. 75, pp. 4997-5009

Brune, J. N., 1971. Correction, *Journal of Geophysical Research* vol. 76, p. 5002.

Bullen, K. E., Bolt, B. A., 1985. *An Introduction to the Theory of Seismology,* Cambridge University Press, Cambridge.

Campbell, K. W., 1985. Strong ground motion attenuation relations: A ten-year perspective, *Earthquake Spectra* vol. 1, no. 4, pp. 759-804.

Cluff, L. S., Cluff, J. L., 1984. Importance of assessing degrees of fault activity for engineering decisions, *Proceedings, First International, Conference on Microzonation,*Seattle, Washington, vol. 2, pp. 629-636.

Dobry, R., Oweis, I., Urzua, A., 1976. Simplified procedures for estimating the fundamental period of a soil profile, *Bulletin of Seismological Societies of America,* vol. 66, pp. 1293-1321.

Finn, W. D. L., 1991. Geotechnical engineering aspects of microzonation, *Proceedings, 4th International Conference on Microzonation,* Earthquake Engineering Research Institute, Stanford University, Palo Alto, California, vol. 1, pp. 199-259.

Gitterman, Y., Zaslavsky, Y., Shapira, A., Shtivelman, V., 1996. Empirical site response evaluations: case studies in Israel. *Soil Dyn. Earthq. Eng.* vol.15, pp. 447−463.

Gutenberg, B., 1945. Magnitude determination for deep-focus earthquakes, *Bulletin of the Seismological Society of America* vol. 35, pp. 117-130.

Gutenberg, B., Richter. C. F., 1936. On Seismic Waves, *Gerlands Beitrage zur Geophysik* vol 47, pp. 73-131.

Guttierez, C., Singh, S. K., 1992. A site effect study in Acapulco, Guerrero, Mexico: comparison of results of strong-motion and microtremor data, *Bulletin of Seismological Society of America* vol. 82, pp. 642−659.

Hanks, T. C., 1982. f(max), *Bulletin of the Seismological Society of America* vol. 72.

Hanks, T. C., Kanamori, H., 1979. A moment magnitude scale, *Journal of Geophysical Research* vol. 84, pp. 2348-2350.

Hanks, T. C., McGuire, R. K., 1981. The character of high frequency strong ground motion, *Bulletin of the Seismological Society of America* vol. 71, pp. 2071-2095.

Idriss, I. M., 1985. Evaluating seismic risk in engineering practice, *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering,* San Francisco, vol. 1, pp. 255-320.

Jennings, P. C., 1985. Ground motion Parameters that influence structural damage, in R. E. Scholl and J. L. King, eds. *Strong Ground Motion Simulation and*

Engineering Applications, EERI Publication 85-02, Earthquake Engineering Research Institute, Berkeley, California.

Jibson, R., 1987. Summary of research on the effects of topographic amplification of earthquake shaking on slope stability, *Open-File, Report 87-268,* U.S. Geological Survey, Menlo Park, California.

Jongmans, D., Campillo, M., 1990. The 1983 Liège earthquake: damage distribution and site effects. *Earthquake Spectra* vol. 6, no. 4., pp. 713−737.

Joyner, W. B., Boore, D. M., 1988. Measurement, characterization, and prediction of strong ground motion, *Earthquake Engineering and Soil Dynamics II - Recent Advances in Ground-Motion Evaluation,* Geotechnical Special Publication 20, ASCE, New York, pp. 43-102.

Kanai, K., 1983. *Engineering seismology,* University of Tokyo Press, Tokyo, pp. 251.

Kanamori, H., 1977. The Energy Release in Great Earthquakes, *Journal of Geophysical Research* vol. 82.

Kanamori, H., 1983. Magnitude scale and quantifiaction of earthquakes, *Tectonophysics* vol. 93, pp. 185-199.

Kramer, S. L., 1996. Geotechnical earthquake engineering, *Prentice Hall, First Edition,* pp. 1-345.

Kudo, K., 1995. Practical estimates of site response, *Proceedings of the Fifth International Conference on Seismic Zonation,* France, Ouest Editions Nantes vol. 3, pp. 1878-1907.

McCann, K., Shah, H. C., 1979. Determining strong motion durations of earthquakes, *Bulletin of the Seismological Society of America* vol. 52, no. 4, pp. 1253-1265.

Moczo, P., J. Kristek, M. Galis, P. Pazak, and M. Balazovjech, 2007. The finite-difference and finite-element modeling of seismic wave propagation and and earthquake motion, *Acta Physica Slovaca* vol. 57, no. 2, pp. 177-406.

Nakamura, Y., 1989. A method for dynamic characteristics estimation of subsurface using microtremor on the ground surface, *Q.R. of R.T.R.I.*vol 30, no. 1, pp. 25-33.

Papageorgiou, A. S., Aki, K., 1983. A specific barrier for the quantitative description of inhomogeneous faulting and the prediction of strong ground motion. II. Applications of the model, *Bulletin of the Seismological Societies of America* vol. 73, pp. 953-978.

Perez, V., 1974. Time dependant spectral analysis of thirty-one strong motion earthquake records, *Open File Report 74-48,* U.S. Geological Survey, Reston, Virginia.

Real, C. R., Teng, T., 1973. Local Richter magnitude and total signal duration in southern California, *Bulletin of the Seismological Society of America* vol. 63, no. 5.

Reiter, L., 1990. *Earthquake Hazard Analysis - Issueas and Insights,* Columbia University Press, New York, 254 pp.

Seo, K., 1998. A joint microtremor measurement in the Fukui basin to discuss the effects of surface geology on seismic motion during the 1948 Fukui, Japan, earthquake, *Proceedings of the Eleventh European Conference on Earthquake Engineering,* Paris.

Schwartz, D. P., 1988. Geology and seismic hazards: Moving into the 1990's, *Proceedings, Earthquake Engineering and Soil Dynamics II: Recent Advances in Ground Motion Evaluation,* Geotechnical Special Publication 20, ASCE, New York.

Schwartz, D. P., Coppersmith, K. J., 1986. Seismic hazards: New trends in analysis using geologic data, *Active Tectonics,* Academic Press, Orlando, Florida, pp. 215-230.

Silva, W. J., 1988. Soil response to earthquake ground motion, *EPRI Report NP-5747,* Electric Power Research Institute, Palo Alto, California.

Trifunac, M. D., Brady, A. G., 1975. A study of the duration of strong earthquake ground motion, *Bulletin of the Seismological Society of America* vol. 65, pp. 581-626.

Trifunac, M. D., Hudson, D. E., 1971. Analysis of the Pacoima Dam accelerograms - San Fernando earthquake of 1971, *Bulletin of the Seismological Societis of America* vol. 61, no. 5, pp. 1393-1411.

Trifunac, M. D., Westermo, B., 1977. A note on correlation of frequency-dependent duration of strong earthquake ground motion with the MMI and geologic condition at the recording stations, *Bulletin of the Seismological Societies of America* vol. 67, no. 3, pp. 917-927.

Vanmarcke, E. H., 1976. Structural response to earthquakes, *Seismic Risk and Engineering Decisions,* Elsevier, Amsterdam, pp. 287-338.

Vanmarcke, E. H., Lai, S. P., 1977. Strong motion duration of earthquakes, *Report R77-16,* Massachusetts Institute of Technology, Cambridge, Massachusetts.