

**Microtremor Based Earthquake Disaster Risk Evaluation of the
Kathmandu Valley**

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June 2012

CERTIFICATE

This is to certify that this dissertation entitled “Microtremor analysis-based earthquake disaster risk evaluation of the Kathmandu Valley” submitted by Mr. Youb Raj Paudyal has been examined and accepted by the Evaluation Committee for partial fulfillment of the academic requirements towards completion of Doctor of Engineering Degree through Special Graduate Course on Disaster Mitigation Study for Asian Students at the Graduate School of Science and Engineering of Ehime University.

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Date: 2012.9.1

DECLARATION

I declare that this dissertation has not been previously accepted in substance or is not being concurrently submitted in candidature for any degree. I state that this dissertation is the result of my independent work/investigation. I hereby also give my consent for this dissertation, after acceptance by the Evaluation Committee, to be available for photocopying, and understand that any reference to or quotation from this dissertation will be duly acknowledged.

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SUMMARY

Nepal occupies the central one-third portion of the Himalayan Arc, which is formed by the collision of the Indian and the Eurasian plates that started about 55 million years ago. Many researchers report interseismic strain convergence between these plates in Nepal at a rate of 16-18 mm/year, and the release of this strain is considered the main reason for high earthquake risk in this region. Records show that Nepal and Kathmandu Valley have experienced recurring destructive earthquake in 1255, 1408, 1681, 1803, 1810, 1833, and 1866, while the latest strong quake was experienced in 1934, which had a magnitude of $M_w = 8.1$, destroyed about 19% and damaged about 38% of the buildings in the valley. A loss estimation study indicates that if an earthquake of the 1934 class recurs, massive damage to building structures and heavy loss of life can be expected. The trends of structural damage in the past earthquakes indicate that the valley is characterized by strong site effects, which was formerly a lake with sediment thickness as high as 500 m in the central part. Moreover, the thickness and properties of the valley sediments vary from place to place, which may cause trapping and focusing of seismic waves during an earthquake leading to an evident change of resonant frequency over a short distance.

A few studies attempt to address the problem of earthquake hazard in the Kathmandu Valley, but most of them focus more on a generalized approach to understand the earthquake hazard of the valley. Much of the risk reduction efforts have focused only on management and legal issues leaving the scientific aspects of potential damage in shadow. Particularly, the dynamic behavior of the sediment deposit must be considered while assessing the earthquake disaster risk in the Kathmandu Valley. Moreover, an in-depth study particularly in relation to the influence of sparsely distributed depth of sediment deposits and local variation in ground response to the structural damage is more important, considering the adoption of a structural design code and mitigation of expected disaster in the valley. One way to addressing this issue is ground response and building vulnerability analysis. There are different approaches for ground response analysis, such as from conventional method of numerical analysis using borehole data to advanced techniques like horizontal-to-vertical spectral ratio method using strong ground motion and microtremors. Evaluation of ground characteristics through numerical analysis from borehole drilling is expensive and slow, and installation of seismic observation stations at required sites is often not feasible in densely urbanized area like Kathmandu Valley.

Therefore, the horizontal-to-vertical spectral ratio technique using microtremor measurements is the only option in the the Kathmandu Valley.

In this study, an attempt has been made to characterize the Kathmandu Valley Basin in order to address the issue of earthquake hazard risk mitigation in the valley through analysis of microtremor data recorded at a total of 172 points using horizontal-to-vertical spectral ratio technique. This study is divided into four parts. In the first part, variation of dominant period in the study area is obtained, and then an estimated dominant period of shaking map is prepared. The obtained results and prepared map indicate that the central part of the valley that covers about 30 percent of the urban area of the valley falls under dominant period range from 1.0 s to 2.0 s, while the outer parts fall under dominant period range from 0.1 s to 1.0 s. The longer dominant period range in the central part is correlated with the extent of depth of Pliocene and Quaternary deposits, while the shorter dominant period in the outskirts corresponds to shallow stiffer deposits near the rocks flanking the valley sides.

In the second part, the characteristic of the Kathmandu Valley is evaluated by analyzing the obtained horizontal-to-vertical spectral ratio curves of 172 measurement points. The investigation results particularly reveal that the valley sediment is characterized by two amplified frequencies, which vary from 0.48 Hz to 8.89 Hz in first resonance and 3.1 Hz to 7.5 Hz in second resonance. These two amplified frequencies were detected at about 20% of the sites that are mainly distributed in the central and northern parts of the valley. These two resonance frequencies are characteristic to particular site condition, and describe the overall seismic site response in two scales, including the deep and surface soil layers. Depending upon the area, especially in the central and northern parts, it is found that the top 10-20 m depth of the sediment plays an important role in causing the second resonant effect in the valley.

In the third part, an analysis of the floor variation of the lacustrine sediments in the Kathmandu Valley is carried out. Due to lack of adequate and precise scientific studies on the floor variation of sediments in the valley, however, it is always difficult to ascertain their characteristics during earthquakes, which ultimately leads to erroneous and assumed data for ground modeling as well as analysis and design of the infrastructures. This particular study attempts to fulfill this gap by proposing an approximate basement topography of the valley, which is mainly based on the interpretation of the microtremor data and analysis results. According to the obtained results, the soft sediment depth can be expressed in terms of frequency by a non-linear regression equation $h=146.01f_r^{-1.2079}$,

which helps estimate the basement topography of the Kathmandu Valley.

In the final part, a rapid visual seismic vulnerability assessment of the buildings is done using a simple tool prepared specifically for the Kathmandu Valley's buildings, since other tool may not be suitable because of the difference in building types in the valley. The survey results in four typical locations in three cities of the valley indicate that more than twothird buildings in the urban center of the valley are vulnerable to earthquakes.

As an important conclusion, the sediment in the Kathmandu Valley is expected to shake the valley buildings during an earthquake in two ways: one due to shaking of the topmost layer and other due to the shaking of layer underneath. As higher fundamental frequencies are characteristic to the topmost layer, this mostly affects low- to medium-rise buildings, while the lower frequencies are characteristic to the layer underneath, so this may affect medium- to high-rise buildings during an earthquake. Additionally, because of the increasing population and development as a greater commercial hub, the central part has recently seen a sharp rise in the number of medium to tall buildings, which have been constructed without adequate geotechnical investigation. In order to reduce the disaster risk, therefore, the multilayer ground effect must be considered in seismic design of long period structures in the valley center. At the same time, it may also be necessary to consider enhanced seismic design criteria for medium- and short-period structures in the valley.

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CHAPTER 1

1 INTRODUCTION

1.1 GENERAL BACKGROUND

Natural disasters have always remained a resistance to sustainable socio-economic development. Natural events such as earthquakes, floods, landslides, tsunamis, hurricanes, volcanoes, tornados, etc. very frequently turn disastrous. Worldwide economic loss in natural catastrophes has increased from \$528 billion in 1981-990 to \$1213 billion in 2001-2010 (Munich, 2011). Therefore, management of natural disasters has still remained a big challenge for the scientists and researchers all over the world.

Some of the natural events, such as floods and typhoons, can be predicted in an advance, which often gives ample time to the authorities to minimize the loss. However, earthquakes are still unpredictable through the current understanding and technological development. Earthquakes have proved to be the most devastating natural disasters with high fatal rates and wide spread destruction (UN, 2009). On an annual basis, 70 to 75 damaging earthquakes occur throughout the globe (FEMA, 2007). Some of the recent damaging earthquakes are 1990 Manjil-Rudbar Earthquake, 1995 Kobe Earthquake, 2001 Bhuj Earthquake, 2004 Sumatra Earthquake, 2005 Kasmir Earthquake, 2008 Sichuan Earthquake, 2010 Haiti Earthquake, and 2011 Tohoku Earthquake. All this makes the earthquakes to be one of the important and immediate research themes for the protection of human life and environment. They can hit any place any time, but historical evidences show that they occur principally in three large zones of the earth, namely, the Circum-Pacific Seismic Belt, the Alpide Belt, and the Mid-Atlantic Ridge, in which the Alpide Belt extends from Java to Sumatra through the Himalayas, the Mediterranean, and out into the Atlantic.

Nepal lies in the central part of the Himalaya (Figure 1.1) and occupies central one-third of the Himalayan Arc (i.e., about 2400 km), which is formed by the collision of Indian and Eurasian plates that started about 55 million years ago (Molnar, 1986). Various research studies mention that the Himalayan Zone contracts due to the subduction of the Indian plate underneath the Eurasian plate at an annual rate of 16-18 mm (Bilham and

Szeliga, 2008). This particular process has increased the earthquake hazard in Nepal and the Himalayan Region. Over the past 100 years, the Himalayan Arc has experienced four major earthquakes; in 1897, 1905, 1934 and 1950 (Bilham et al., 1995; Pandey et al., 1995), and damaged many parts of Nepal, Pakistan, and India. The latest earthquake that extensively damaged many historic urban areas of Nepal, such as Kathmandu, Biratnagar, and Dharan, occurred some 80 years ago in 1934, which had a magnitude of $M_w = 8.1$ (Ambraseys and Douglas, 2004; Hough and Bilham, 2008). About 8519 people were killed and about 200,000 houses including temples and historic buildings were destroyed in Nepal in this earthquake disaster (Pandey and Molnar, 1988).

In this scenario, the study in this PhD research basically focuses on earthquake disaster risk in the Kathmandu Valley through the application of microtremor measurement and analysis technique. The following subsections shed light on the background information of the Kathmandu Valley and microtremor survey technique on ground response analysis.

1.1.1 Kathmandu Valley and earthquakes

Kathmandu Valley (Figure 1.1) has experienced many strong earthquakes in the past. Major earthquake damage in the Kathmandu Valley was reported to have taken place in 1255, 1408, 1681, 1803, 1810, 1833, and 1866 (Bilham et al., 1995; Pandey et al., 1995). The latest great earthquake of 1934 with a maximum intensity of X in Modified Mercally Intensity (MMI) Scale in the Kathmandu Valley induced reportedly killed about 4,296 people and destroyed about 19% and damaged about 38% of the buildings in the valley alone (Pandey and Molnar, 1988; Rana, 1935). Although the historical record is inadequate to predict accurately the recurrence period of great earthquakes in Nepal and the Kathmandu Valley, the available information indicates that a greater than M8.0 earthquake occurs at an interval of 100 years. As it is already about 80 years from the previous major earthquake of 1934, the threat of a major earthquake in the region, particularly the Nepal Himalaya, in near future (10-20 years?) is very high. The recent loss estimation study in the Kathmandu Valley predicts that the next major earthquake in Nepal might kill some 20,000 people, seriously injure some 59,000 people, and completely destroy about 59,000 houses and buildings (JICA, 2002).

Kathmandu Valley sits over an ancient lake deposit, which measures as deep as more than 500 m in the central part and is made up of thick layers of clay, silt, sand and gravel. Large variation in depositional environment of the sediments and their thicknesses

consequently exhibit extreme variation in geotechnical properties of soils in the lacustrine deposit, which may cause trapping and focusing of seismic waves during an earthquake leading to an evident change in the properties of seismic waves and their amplification value.

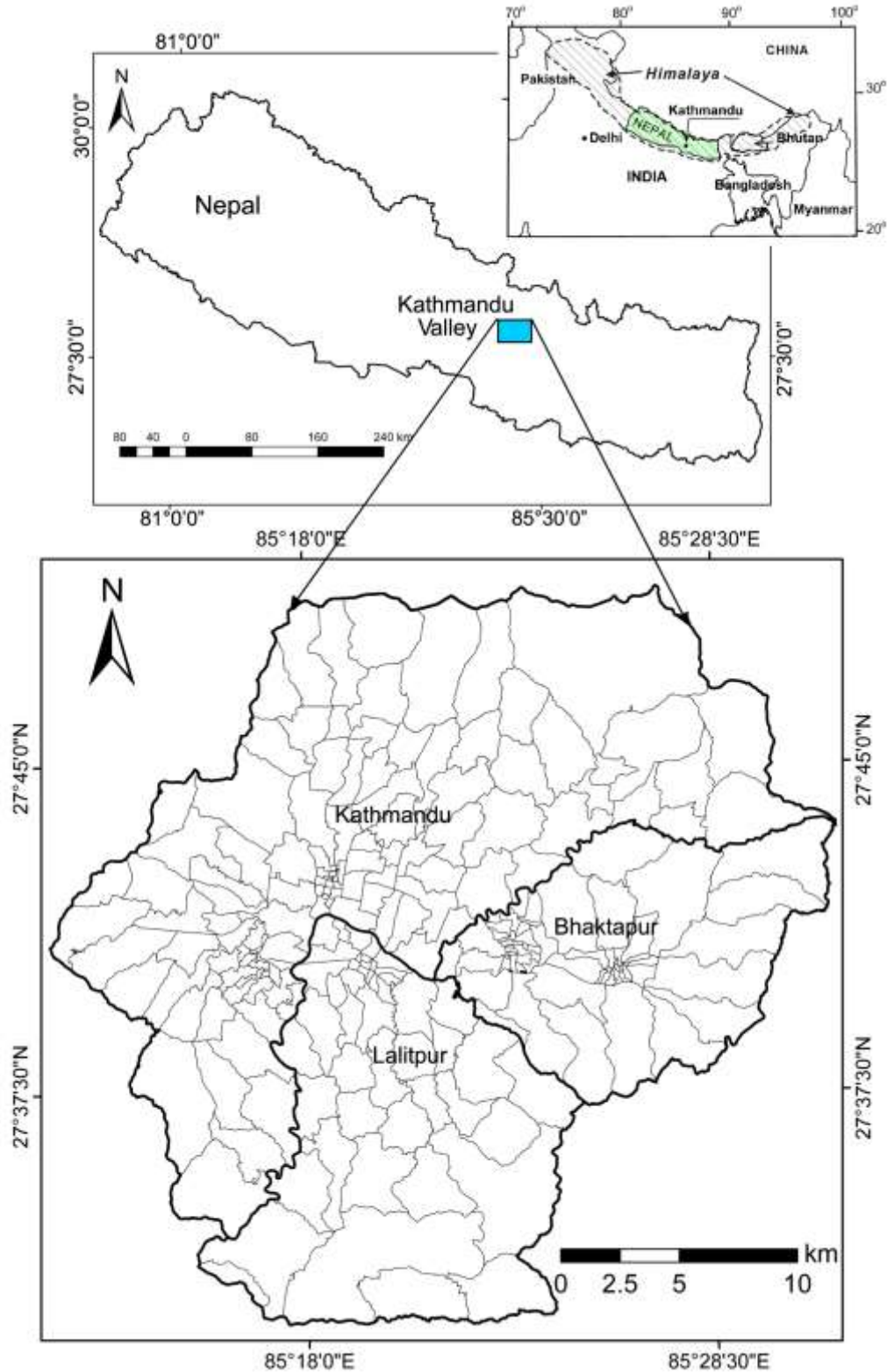


Figure 1.1: Map of Nepal and Kathmandu Valley.

So far, only a few studies have addressed the issues of ground response during an earthquake in the Kathmandu Valley (JICA, 2002; UNDP, 1994). Most of them have focused on a generalized approach to the earthquake hazard scenario in the valley, while an in-depth study in relation with the influence of the uneven distribution of sediment deposits to seismic ground motion can not be found in literature. In seismic risk studies, variation in the effect of ground response on the structural damage is always an important parameter in structural design and disaster mitigation. Past earthquake damage distributions are also evident that the Kathmandu Valley exhibits local variation of ground response (Hough and Bilham, 2008; Dixit et al., 1998; Mugnier et al., 2011). However, most risk reduction efforts in the valley have focused only on management and legal issues leaving the scientific aspect of potential earthquake damage in shadow. Particularly, the dynamic behavior of the sediment deposit must be considered while assessing the earthquake disaster risk in the valley. Therefore, in order to explore the earthquake hazard level as well as to estimate the risk of expected earthquake disaster in the Kathmandu Valley, the ground response analysis of the area needs to be carried out.

1.1.2 Microtremor in ground response analysis

A number of methods are available for ground response and microzonation studies. Most frequently used methods are boreholes with PS Logging, the use of strong ground motion observation, and microtremor measurement. The evaluation of ground characteristics based on the analysis of borehole including PS Logging is commonly practiced; however, this method is comparatively costly, time consuming, and generally not suitable for densely populated areas. The method based on strong ground motion observation is also very common (Borcherdt, 1970), but it is only possible in areas where earthquakes are frequent and the data can be recorded easily. In this situation, the microtremor analysis may be a good option for ground response analysis in the Kathmandu Valley.

Microtremor are small scale vibration of ground caused by multiple random sources such as sea water waves, turbulent wind, effects of wind on trees or buildings, industrial machinery, car and heavy vehicles or human activities and so on (Bonnefoy-Claudet et al., 2006a).

Kanai (1954) was probably the first to introduce a microtremor based method to analyze the ground response. Later on, Nakamura (1989) improved this method and it has

become widespread as a low-cost and effective tool to estimate the fundamental resonance frequency of sediments using horizontal-to-vertical (H/V) spectral ratio by measuring the microtremors in a single-station. According to Nakamura (1989), the horizontal-to-vertical ratio is the Quasi-Transfer Spectra (Transfer Function) of the soil strata over bedrock. In his paper, Nakamura (1989) brilliantly explains the use of this technique at a single station and gives detailed information and its subsequent assumptions.

Due to a close relationship between the nature of microtremors and the seismic response of soil layers, the practical use of microtremors in the field of engineering is increasing. In last two decades, microtremor analysis-based method has been widely used for various purposes, such as site effect evaluation, wave amplification estimate, liquefaction vulnerability assessment, sediment depth estimation, and microzonation studies in different geographical and geological regions of the world (Field and Jacob, 1993; Ibs-von Seht and Wohlenberg, 1999; Mucciarelli, 2011; and Tuladhar et al., 2004).

With the above background information, as the microtremor measurement-based method is quick, economical and effective, it is a quite appropriate technique in the Kathmandu Valley. So, microtremor survey and interpretation of the analytical results through horizontal-to-vertical (H/V) spectral ratio method have been used as the main tool for this study. Geological evaluation using borehole data, microtremor method of data collection and analysis, seismic microzonation, frequency response of top soil layer, soft sediment depth mapping and vulnerability assessment of buildings in the Kathmandu Valley were picked up as the main research themes for this research work.

1.2 LITERATURE REVIEW

At the beginning of the twentieth century, the relationship between structural damage and subsoil properties was established as “earthquake damage is great where the ground is poor” (Kanai, 1983). Now, however, the extent of earthquake damage is known to depend on different factors, including the earthquake source and path effects, local subsoil condition, and the condition of structure itself. The factors influencing the characteristics of earthquake ground motion (Seed and Idriss, 1982) at any site can be summarized as follows:

- ◆ Magnitude of the earthquake
- ◆ Source mechanism of the earthquake
- ◆ Wave interference effects related to the direction and speed of fault rupture
- ◆ Distance of the site from the source of energy release

- ◆ Geologic characteristics of the rocks along the wave transmission path from the source to the site
- ◆ Local soil conditions at the site

Seismic waves generated at the earthquake source (i.e. fault area) propagate through different geological formations of the earth until they reach the ground surface at a specific site. The travel paths of these seismic waves in the uppermost geological layers, which are also called the layers above the bedrock for a particular site, strongly affect their characteristics and produce different effects on the earthquake motion at the ground surface. Therefore, amount of damage to man-made structures during earthquakes depends on the nature of the subsoils where structure is constructed over it, as well as the nature of the arriving seismic waves. Gutenberg (1957) mentions that surface layer of soft soils overlying bedrock may considerably modify the propagating waves. He monitored several small earthquakes and compared the results with those obtained simultaneously on outcropping bedrock in the vicinity by using an array of seismographs stationed at varying depths of alluvium. His findings can be listed as follows:

- ◆ Amplitude of shaking increases with increasing thickness of the surface layer
- ◆ There is resonant period associated with soft site which increases with increasing thickness of surface layer
- ◆ Duration of shaking in deep alluviums may be several times greater than that experienced in exposed rocks
- ◆ Ground effects may result in appreciable difference in response at places that are only a fraction of a mile apart

Furthermore, observations of localized damage resulting from the earthquakes have led to a general acceptance of the fact that the degree of damage may be influenced by the characteristics of the soils in the affected area. The greatest impacts of damage-inducing earthquakes do not automatically appear in the epicentral area. The classic examples of damage resulting due to poor subsoil conditions were observed in Mexico City during 1985 Michoacan ($M_s = 8.1$) Earthquake (Kramer, 1996). The Mexico City sits over a deep, highly compressible alluvium, which formerly was bed of Lake Texcoco, which is about 350 km far from the epicenter of the earthquake. Almost all collapsed buildings were found to have been built in the lake zone. This indicates that when poorly consolidated critical sediment layers are present, stronger ground motion (high intensity shaking) can occur in larger epicentral distances. Therefore, determining site conditions for earthquake damage risk assessment is significantly important. This phenomenon of soil response

during earthquake excitation is known as seismic site effect, which can be divided into direct seismic site effects and indirect seismic site effects (Wang and Law, 1994) and described as follows.

1) Direct seismic site effects

When the site is weak and can not resist the earthquake excitation, a certain type of ground failure may occur, which is apparently visible and commonly recognized. The main patterns of such failure are soil liquefaction (resulting in settlements and tilting of structures), ground settlement, collapsible settlement (e.g. earthquake-induced collapse of underground cavity or presence of surface rupture), and seismically induced landslides provoked by slope instability.

2) Indirect seismic site effects

When the site is strong enough to resist earthquake forces, it remains stable and the chances of occurrence of ground failure are comparatively low. However, seismic waves transmitted through the ground material may be magnified either in amplitude, or in frequency content, or in time duration which ultimately affect the ground and structures. Examples of this type of site effect are amplification/filtering effects of soft ground, and resonance effects of earthquake ground motion, subsoil, and the structure.

In this context, it is understood that structural damage caused by an earthquake is due to the interaction between soil and structure. For this reason, the dynamic characteristics of the structure should also be considered during evaluation of earthquake damage risk of the structure. Therefore, the intensity and duration of earthquake-induced ground shaking and hence the structural damage at a site depend on: 1) earthquake magnitude, depth and location of the epicenter, which are known as source effects, 2) the medium of seismic waves travel which is known as medium effect, and 3) physical and geotechnical characteristics of the site known as site effects (Kramer, 1996), as illustrated in Figure 1.2. In order to identify these effects, instrumental recording of seismic ground motion at the site of interest is necessary. By the application of site response estimation techniques, an identification of local site effects may be carried out. From which, the characteristics of the ground as well as structure can be evaluated.

Recently, many instrumental field observations have been conducted during earthquakes for the evaluation of effects of local site conditions. As reported in various studies, such as Berilgen (2007), Borchardt (1970); Borchardt and Gibbs (1976), Gazetas

et al. (1990), Guéguen et al. (1998), Hartzell et al. (2001), Jennings (1997), Lekkas (1996, 2001), Mandal et al. (2005), Seed et al. (1990), Sitaram et al. (2006), and Walling and Mohanty (2009), local site conditions play a dominant role in seismic damage.

Geologic structures such as basins and sediment-filled valleys and topographical features may also have significant effects on the variation of the earthquake ground motions. Local site effects in basins and valleys and topographical effects have also been extensively investigated, by many researchers, such as Athanasopoulus et al. (1999), Bard and Bouchon, (1985), Bard and Tucker (1985), Bouckovalas and Papadimitriou (2005), Bouchon, M. (1973), Chávez-García et al. (1996), Dicapua et al. (2006), Faccioli et al. (2002), Finn (1991), Lee et al. (2009), Panzera et al. (2011), Paolucci (2002) Paolucci et al. (1999), Mucciarelli (2011).

Recently Several studies, such as Albarello and Lunedei (2010), Birgöen et al. (2009), Bonnefoy-Claudet et al. (2006b), Cadet et al. (2011), Chatelain et al. (2008), D'Amcio et al. (2004), Delgado et al. (2000), Dinesh et al. (2010), Fäh et al., (2001), Field et al. (1995), Fischer et al. (1995), Garcia-Jerez et al. (2006), Gosar (2007), Gosar and Martinec (2009), Gosar and Lenart (2010), Guéguen et al. (2007), Herak (2011), Hinzen et al. (2004), Huang (2002), Huang et al. (2002), Ibs-von Seht and Wohlenberg (1999), Konno and Ohmachi (1998), Lachet et al. (1996), Lermo and Chavez-Garcia (1993, 1994), Lunedei and Albarello (2009, 2010), Mucciarelli et al. (2003), Özalayboy et al. (2011), Panou et al. (2005), Parolai et al. (2002), Parolai et al. (2004), Sánchez-Sesma et al. (2011), Sukumaran et al. (2011), Theodulidis et al. (1996), Theodulidis et al. (2005), Teves-Costa et al. (1996), Tuan et al. (2011), Walling et al. (2009), and Zendieh and Pezeshk (2011) apply microtremor horizontal-to-vertical spectral ratio for site effect study and also mention that the frequency of the resonating waves in poorly consolidated sediments is related to shear wave velocity as well as thickness of the sediment.

Most of these studies are site specific and were conducted in a particular region based on the local requirements. Although many cities in developing countries are situated in soft sediment deposited areas with high risk of earthquake disaster, there is very few research work in relation to site effect, and hence, for the mitigation of earthquake disaster risk. Presumably, lack of resources (i.e. financial as well as technical) for the site effect study may be the main cause of less documentation in this area. Moreover, site specific research activities cannot be elaborated in different terrains. The problems and issues may differ according to the local geographical and geological conditions. In this context, to address the particular problem of site effect due to lacustrine deposit in the Kathmandu Valley, this

study focuses on ground response analysis using microtremor survey in the Kathmandu Valley.

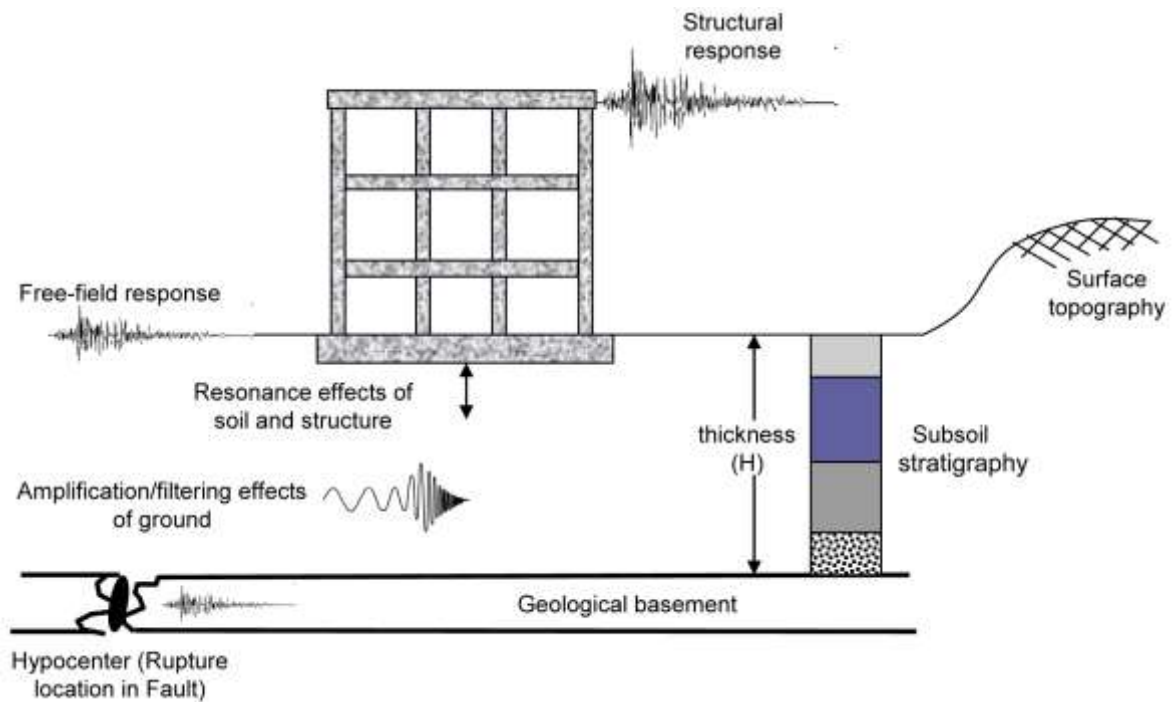


Figure 1.2: Factors responsible for the seismic ground shaking.

1.3 RESEARCH OBJECTIVES

The main objective of this study is to evaluate the earthquake disaster risk in the Kathmandu Valley of Nepal through the interpretation of microtremor data analysis. Generally in urban areas the impact of disastrous earthquakes is best portrayed and quantified through the understanding of “earthquake damage scenarios.” The first ingredient of such scenarios is hazard assessment, and seismic hazard in urban areas is usually quantified and portrayed by preparing microzonation maps, which are the most essential steps towards an effective planning of the infrastructures and hence mitigation of the urban earthquake risk. Therefore, evaluation of ground motion variation based on geological conditions through the application of seismic microzonation map technique is considered to help appropriately and quantitatively assess the earthquake hazard risk of the Kathmandu Valley. Moreover, in order to explore the earthquake hazard level and to assist in earthquake damage risk evaluation of the valley buildings, it is equally important to quantify the depth of sediment layer that is responsible for the amplification of seismic ground motion and building vulnerability level. So, to accomplish the main objective, the following specific research objectives are set.

- ◆ To study the geology and seismicity of the Kathmandu Valley
- ◆ To prepare a seismic microzonation map of the Kathmandu Valley
- ◆ To study the topsoil response during earthquake in the Kathmandu Valley
- ◆ To map the soft sediment depth of the Kathmandu Basin
- ◆ To generate the ground profile of the Kathmandu Basin
- ◆ To study a seismic vulnerability of buildings of the Kathmandu Valley

1.4 SCOPE OF THE STUDY

This thesis work addresses the issues of earthquake hazard risk in the Kathmandu Valley of Nepal through the interpretation of microtremor observations. Particular focus has gone to studying the behaviors of Kathmandu Valley sediments during earthquakes. The research flow is presented in Figure 1.3. The overall content of this thesis is divided into eight chapters including the conclusions. Latter, four chapters highlight the core content of this research, while former three chapters only present background information, literature review, study area details, and methodological description.

The first chapter briefly talks about the earthquake damage risk in the Kathmandu Valley and Nepal. It also briefly discusses microtremors and ground response analysis, literature review details on site effects study, research objectives and scope of the study. The second chapter elaborates the study area. Seismicity in Nepal Himalaya and the geotechnical aspects of the Kathmandu Valley are also presented in this chapter. The third chapter gives the detailed background information to strong ground motion, microtremors, and methods used in ground response analysis. The forth, fifth, sixth, and seventh chapters are the core chapters of this study. These chapters explain detailed analysis, results and major findings of this research work. Seismic microzonation of the Kathmandu Valley, response of the soil layers of the Kathmandu Basin, and depth of the responsible layers for the amplification of the ground motion in the valley are described in these chapters. Finally, the eighth chapter concludes the overall findings of this work. Figure 1.4 illustrates the thesis structure.

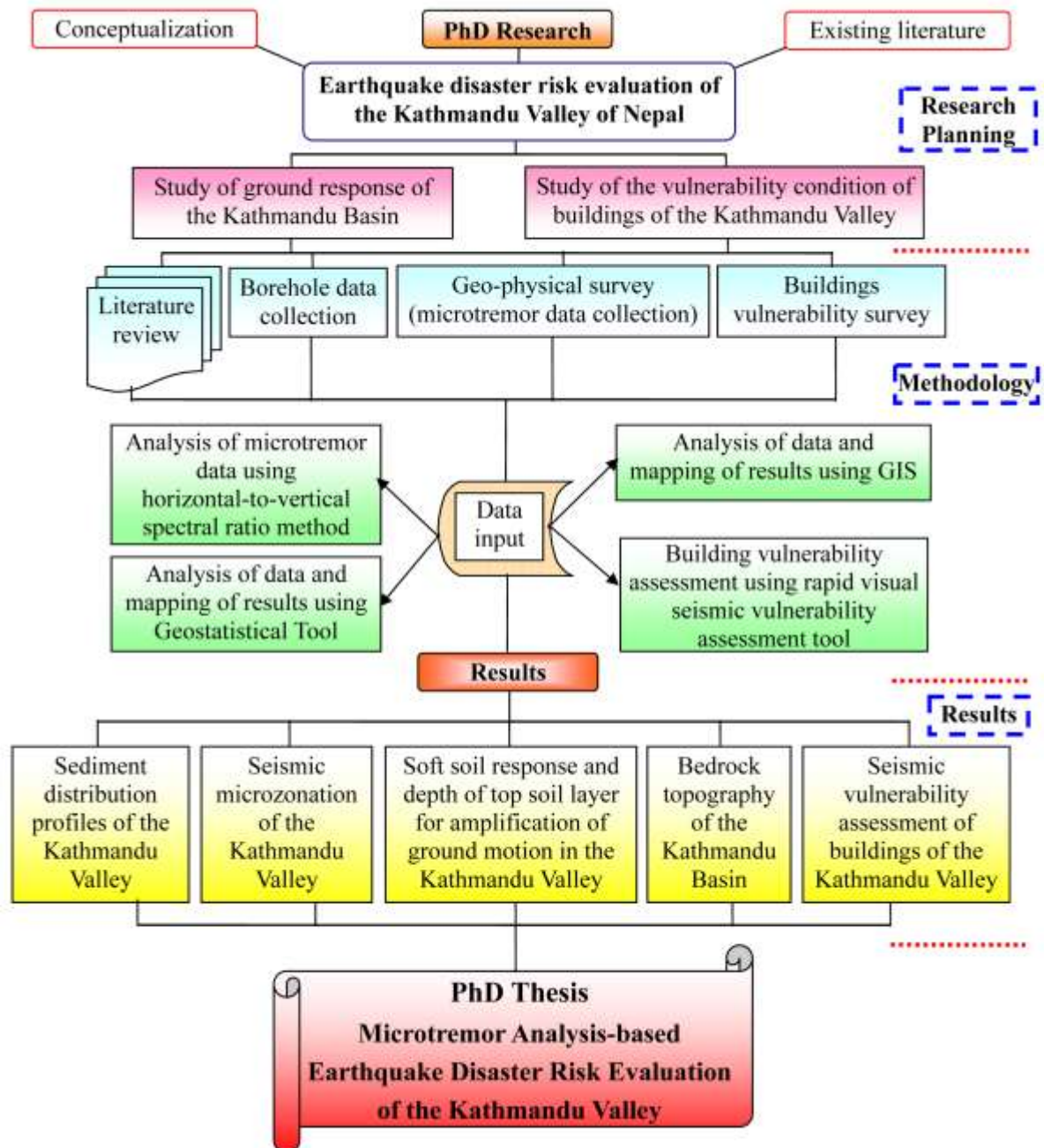


Figure 1.3: Research flow.

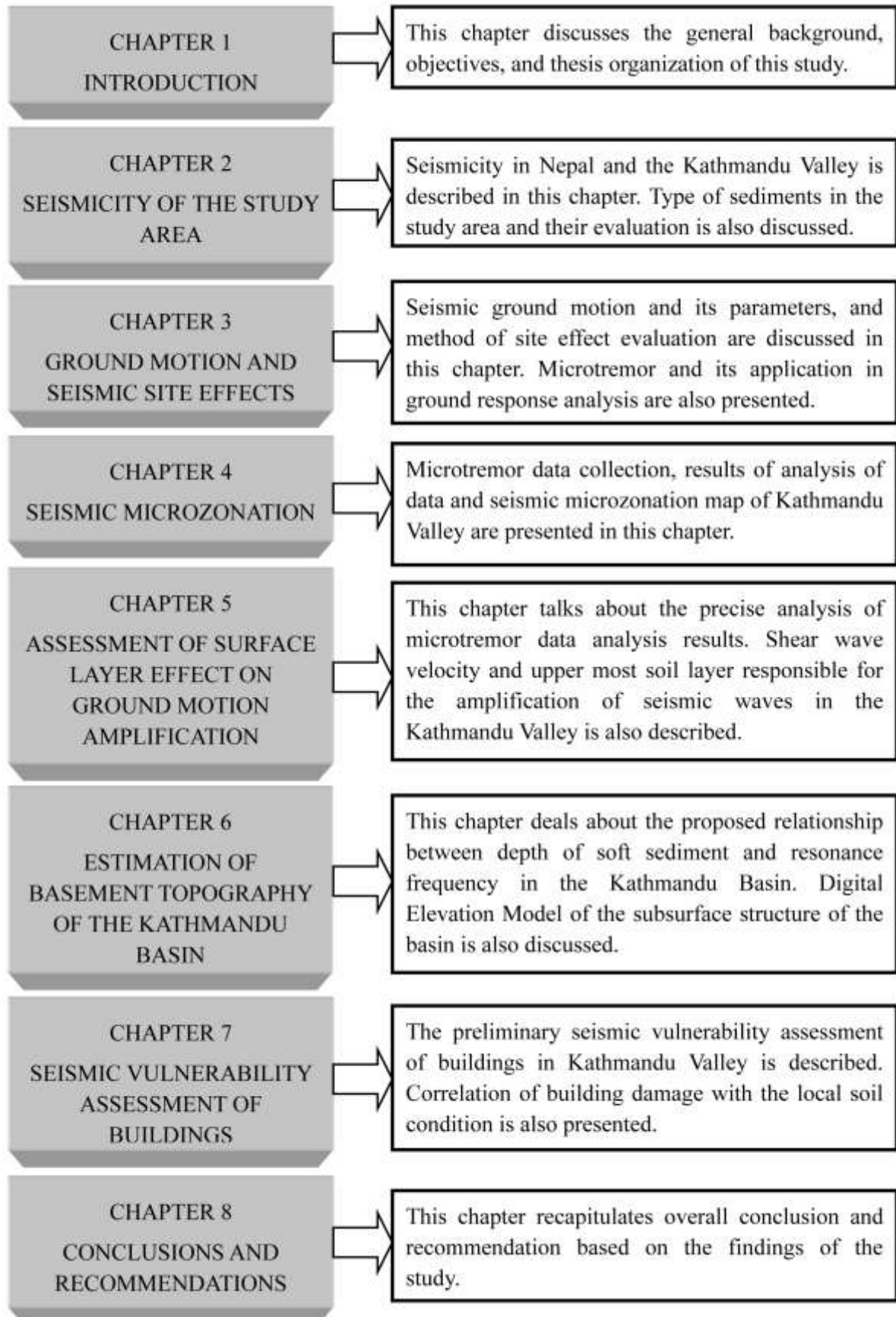


Figure 1.4: Thesis structure.

CHAPTER 2

2 SEISMICITY OF THE STUDY AREA

2.1 GEOGRAPHICAL SETTING AND NATURAL DISASTERS IN NEPAL

Nepal is a landlocked country and the world's youngest republic. It is bordered on the north by People's Republic of China and on the south, east, and west by the Republic of India. It is situated between the latitudes of 26°22' to 30°17' north and the longitudes of 80°4' to 88°12' east in the central part of the Himalayan belt as shown in Figure 2.1. The length from east to west of Nepal is 885 km and the width from north to south is minimum 145 km and maximum 241 km. It covers a land mass of 147,181 sq. km which is only about 0.01% of the total landmass of the earth and 4,000 sq. km inland water bodies such as lakes, reservoirs, and rivers. The elevation of the country rises from 60 m in terai (Kechanakalan, Jhapa) to 8,848 m Mount Everest in the north within a short distance of about 200 km. Nepal is characterized by a rugged topography, very high relief, variable climatic conditions, complex geological structure with active tectonic process and continued seismic activities.

Due to unfavorable natural conditions like tectonic setting, fragile geology and steep topography, Nepal is made as one of the 20 most disaster prone countries in the world (NRRC, 2012). Earthquake, flood, landslide, fire are the most frequent natural disaster in Nepal. An inventory of past disastrous events during 1971-2006 is shown in Figure 2.2, which illustrates that epidemics take the largest toll of life every year and that landslides, floods, fires, and earthquakes are the principal hazards in terms of their extent and frequency of occurrence (NSDM, 2008). These disasters occur almost every year in one part of the country and other causing loss of life and heavy damage to physical properties.

According to the recent study by UNDP/BCPR, Nepal ranks at 11th and 30th country with respect to relative vulnerability to earthquake and water induced disaster respectively (UNDP, 2004). Similarly, Nepal is one of the fourth most vulnerable countries in terms of climate change risk (HELVETAS, 2011).

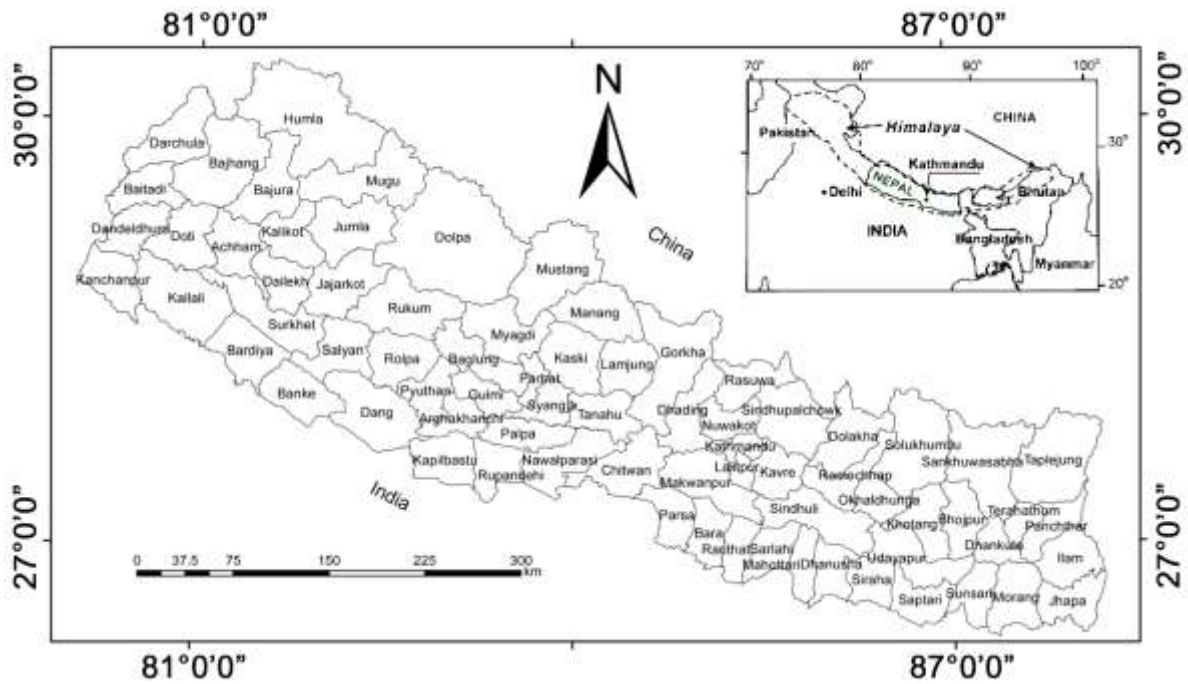


Figure 2.1: Map of Nepal showing the administrative division of Nepal (Inset: location of Nepal in the Himalaya).

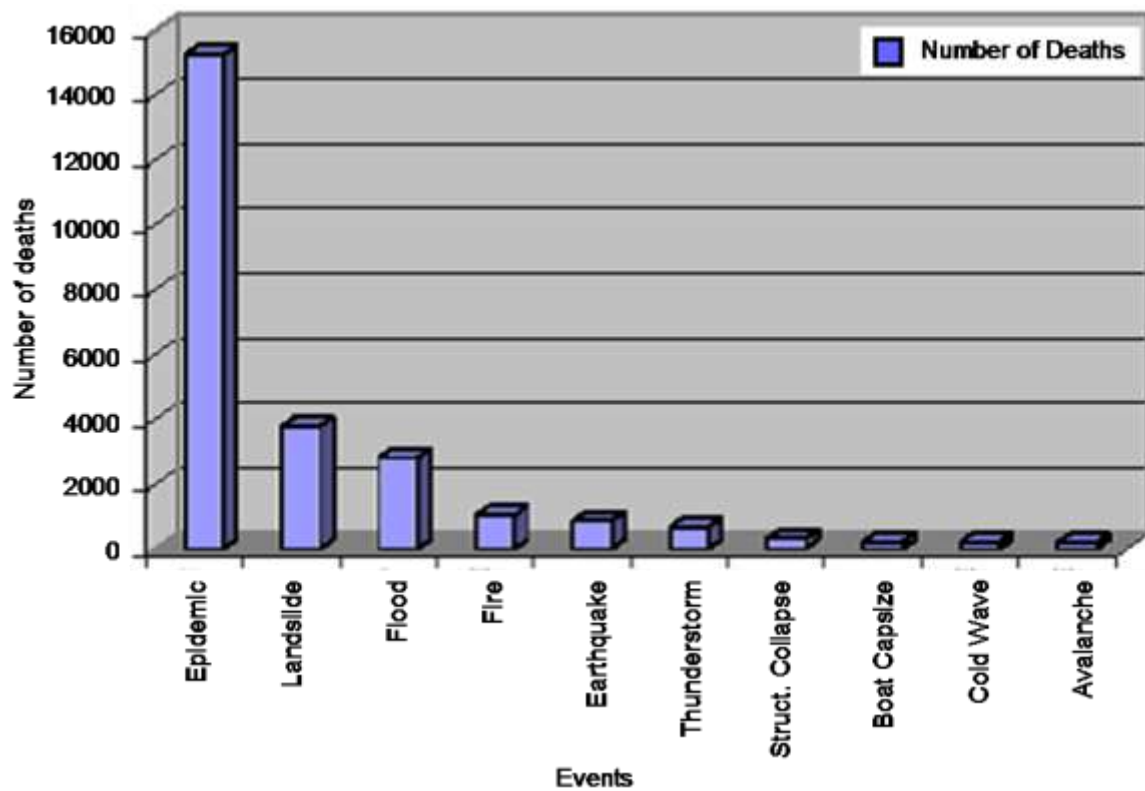


Figure 2.2: Ten Most fatal Hazards in Nepal (1971 - 2005).

2.2 TECTONIC SETTING OF NEPAL

The Himalayan Mountain, a concave chain, measures approximately 2,400 km in length and 200 km to 300 km in width, and is one of the most active and fragile mountain range on the earth (Hasegawa et al., 2009). The Himalayan Range is a broad continuous arc along the northern fringes of the Indian subcontinent from the bend of the Indus River in the northwest to the Brahmaputra River in the east and is a young mountain system in the earth (Figure 2.3), and it covers an area of approximately 600,000 sq. km in the south Asia. The Himalayan mountain chain extends in the east-west direction between the wide plains of the Indus and the Brahmaputra in the south and the vast expanse of the high Tibetan Plateau in the north. The Himalayan mountain system developed in a series of stage 30 to 50 million years ago, and they are still active and continue to rise today (USGS, 1999) due to the tectonic activities within the Himalayan Range (Figure 2.4).

Nepal Himalaya occupies the central one-third portion (i.e., about 800 km) of the Himalayan arc, which has been formed by the collision of Indian and Eurasian plates about 55 million years ago (Molnar, 1986). The mountain building (orogenic) process continues from this collision and this process is still on making process (Figure 2.4). This is noticeable by present day northward movement of India at the rate of 2 cm to 5 cm per year and the occurrence of frequent seismic activities along the Himalaya and its surroundings (Jackson and Bilham, 1994 a, b; Pandey et al., 1995; Bilham et al., 1997).



Figure 2.3: The Himalaya and its transverse division (modified after Zurick et al., 2005, and Dahal, 2009).

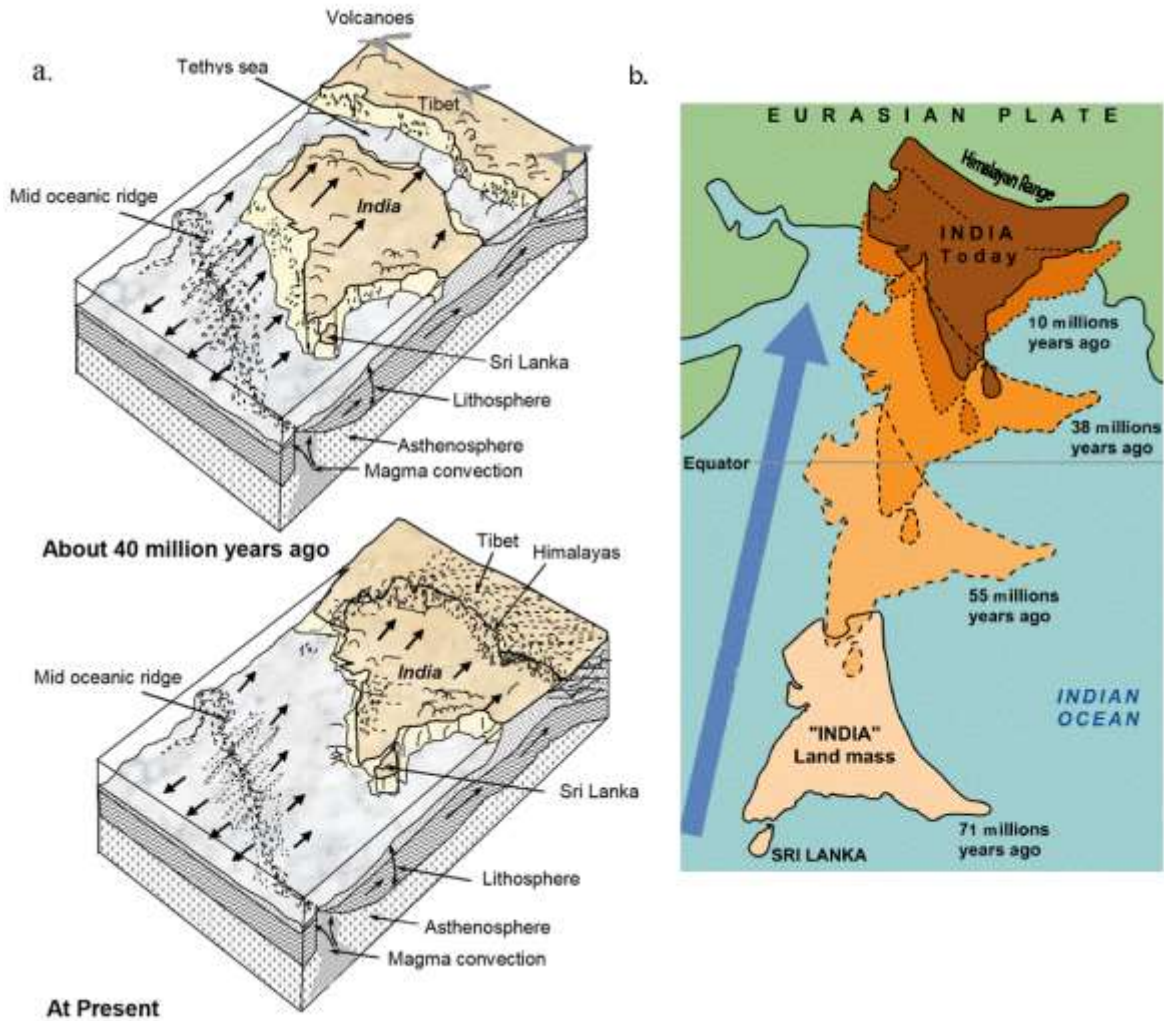


Figure 2.4: a) Collision of Indian plate with Eurasian plate and formation of the Himalaya, b) Shifting of Indian plate in geological past and collision with Eurasian plate (modified after Dahal, 2009 and USGS, 1999).

Geomorphologically, Nepal can be into eight geomorphological provinces (Hagen, 1969; Dahal, 2009) (refer, Figure 2.5). These provinces are E-W running and can be incorporated in the Indian Himalayan belt and represent all characteristic physiographic zones of Nepal.

From geological and seismic studies of the slip rate of faults in Eurasian Plate, it is believed that the present-day convergence rate between India and Eurasia is responsible for the shortening, uplift and seismicity of the Himalaya (Molnar and Tapponier, 1975). Various research studies have mentioned that the Himalaya zone contracts due to the subduction of India into Eurasia at a rate of 16-18 mm/year (e.g., Banerjee and Burgmann, 2002; Bettinelli et al., 2006; Bilham and Szeliga, 2008; Jouanne et al., 2004).

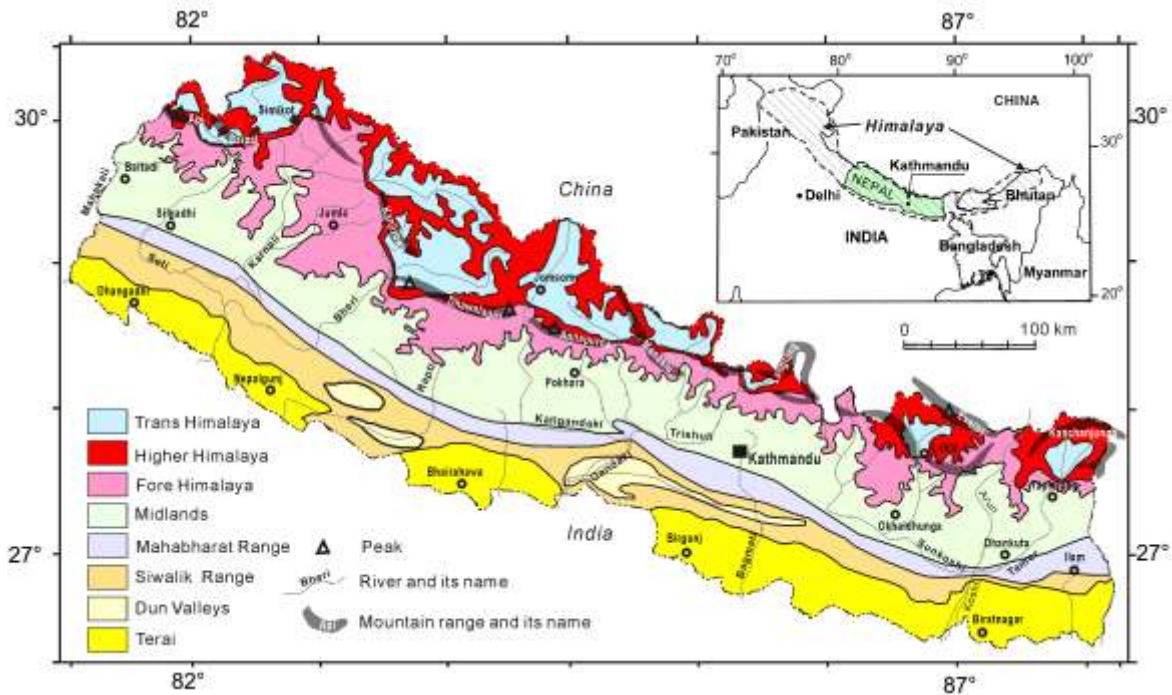


Figure 2.5: Digital elevation model (DEM) based regional geomorphic map of Nepal; inset illustrates local of Nepal in the Himalaya (taken from Dahal, 2009).

According to Yeats et al. (1992), the collision of Indian Plate and Eurasian Plate took place in Paleogene time, with the collision zone marked by the now-inactive Indo-Tsangpo Suture Zone Tibet and India, and the Main Mantle Thrust Zone of Pakistan. As a result, the rocks of the Himalaya are moving upwards as well as horizontally towards south along the major thrusts (Upreti, 1999). Geologically and tectonically, Nepal is divided into five major tectonic zones as follows (Ganser, 1964; Upreti, 1999).

- ◆ Terai (Gangetic Plain) Zone
- ◆ Sub-Himalayan (Siwaliks) Zone
- ◆ Lesser Himalayan Zone
- ◆ Higher Himalayan Zone and
- ◆ Tibetan-Tethys Himalayan Zone

These east-west extending zones run almost parallel to each other (Figure 2.6). These tectonic zones are separated by major thrusts and faults of the Himalaya, namely from north to south (Figure 2.7, Figure 2.8), South Tibetan Detachment System (STDS), Main Central Thrust (MCT), Main Boundary Thrust (MBT) and Main Frontal Thrust (MFT). The descriptions of each geological zones and the tectonic history of the Nepal Himalaya are given below.

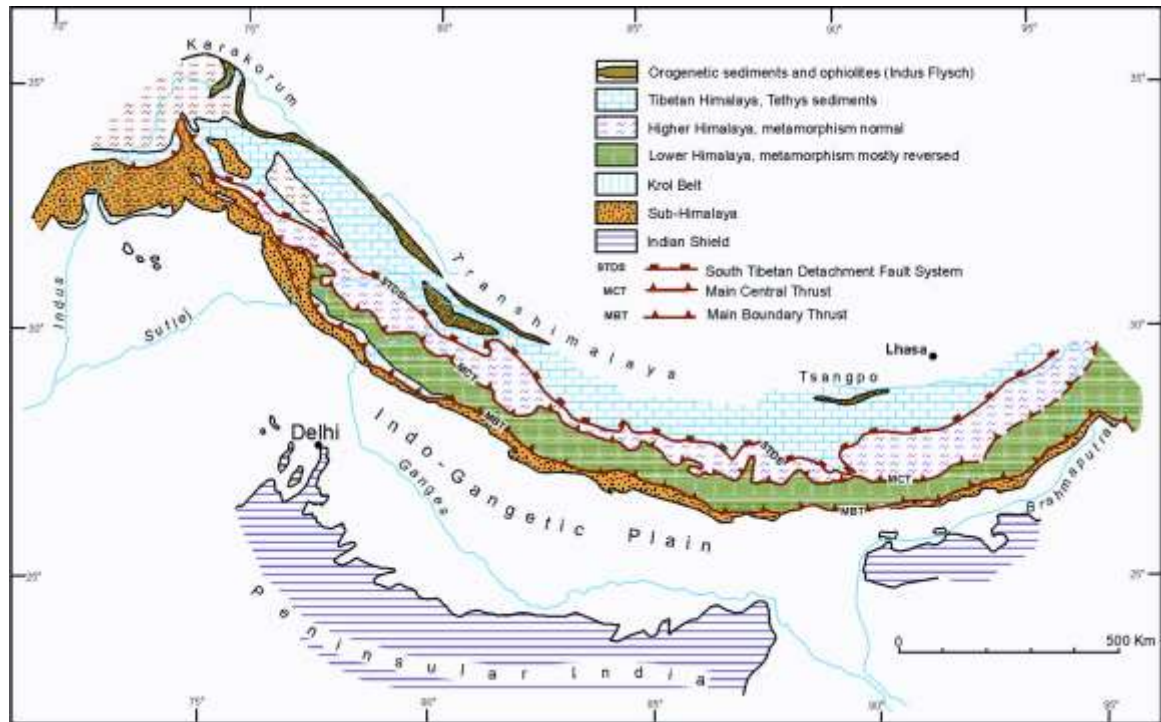


Figure 2.6: Longitudinal Subdivision of the Himalaya (After Gansser, 1964; Upreti and Yoshida, 2005; Dahal, 2009).

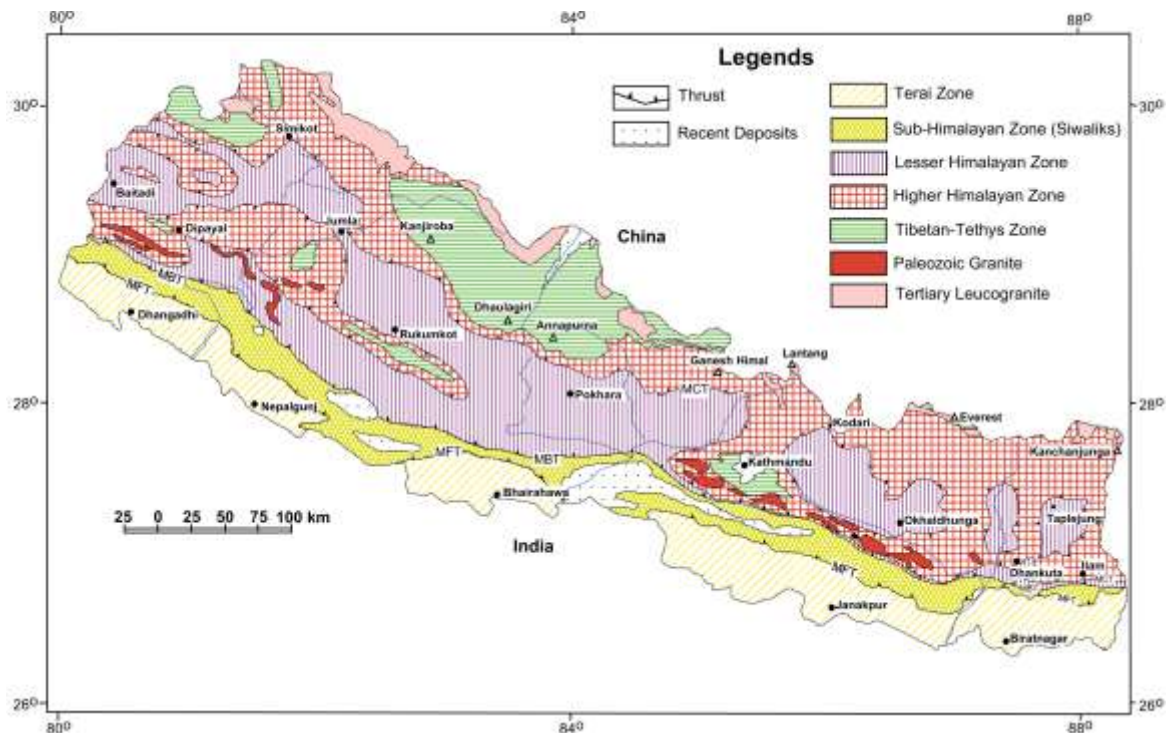


Figure 2.7: Tectonic zone of the Nepal Himalaya (after Amatya and Jnawali, 1998 and Dahal, 2006) (MCT: Main Central Thrust, MBT: Main Boundary Thrust and MFT: Main Frontal Thrust).

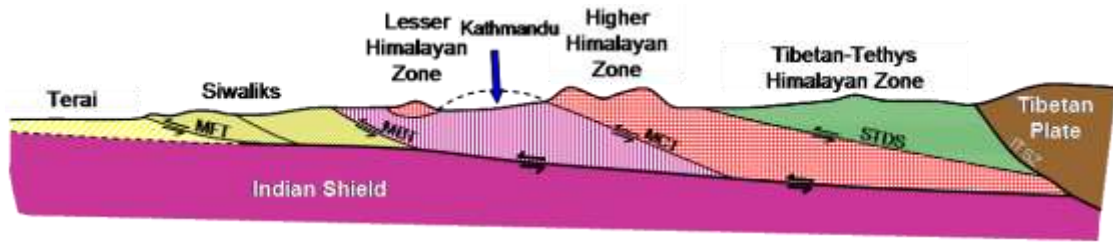


Figure 2.8: Generalized cross section of Himalaya (modified after Dahal, 2006) (MFT: Main Frontal Thrust, MBT: Main Boundary Thrust, MCT: Main Central Thrust, STDS: South Tibetan Detachment System, ITSZ-Indo-Tsangpo Suture Zone).

2.2.1 Terai (Gangetic Plain)

The Terai Zone also called as Gangetic plain. It extends from the Indian Shield in south to the Sub-Himalayan (Siwaliks) Zone to north. The Terai zone is a foreland basin and all the sediment in this zone has originated from peaks (hills and mountains) of northern part. The alluvial sediments contain mainly boulder, gravel, silt and clay. The subsurface condition of Terai Zone of Nepal is shown in Figure 2.9. This zone is separated with Sub-Himalaya (Siwaliks) by the Main Frontal Thrust (MFT) in north. At some places along MFT, the Siwaliks rocks are observed to rest over the recent sediments of the Terai. Many researches mentioned that the MFT is an active thrust system in the Himalayan region (Upreti, 1999).

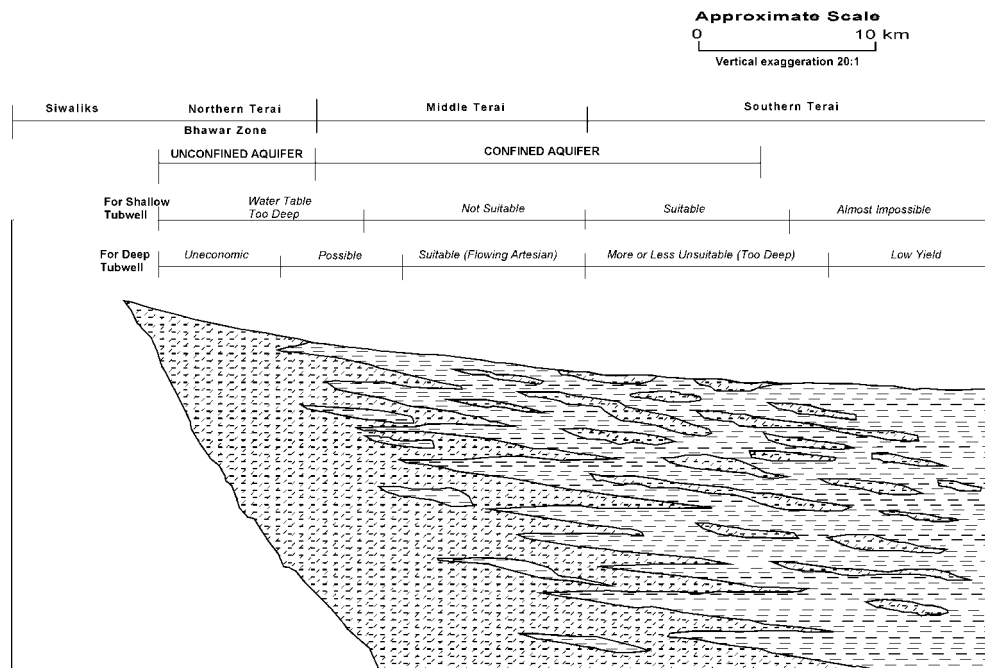


Figure 2.9: Subsurface condition of Terai Zone of Nepal (taken from Dahal, 2009, modified after GRDP, 1994)).

2.2.2 Sub-Himalayan (Siwaliks) Zone

The sub-Himalayan Zone is also known as the Siwaliks Zone and is bordered on the south by the Main Frontal Thrust (MFT) and on the north by the Main Boundary Thrust (MBT) (Figure 2.10). The MBT is now less active thrust system than MFT, and the frontal part of the Lesser Himalaya is comparatively rising fast even today. But the MBT could not accommodate all strains due to northward displacement of the Indian Plate, which resulted in the development of another thrust namely- Main Frontal Thrust (MFT) in the south at southern front of the Siwaliks. The Lesser Himalayan rocks thrust southward over the rocks of Siwaliks along the MBT. The Siwaliks Zone also has numbers of east-west running thrusts. Siwaliks consist of fluvial deposits of the Neogene age (23 millions years to 1.6 millions years old). Stratigraphically, Siwaliks can be classified into three groups, namely: Lower Siwaliks, Middle Siwaliks and Upper Siwaliks (Dahal, 2006).

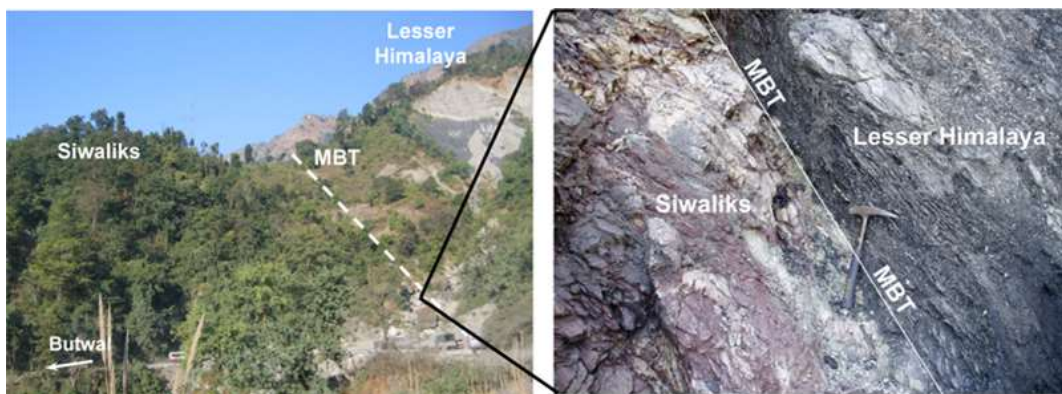


Figure 2.10: Map showing Siwaliks, MBT and Lesser Himalaya (Observed in Butwal-Tansen section of Siddhartha Highway (after Dahal, 2009).

2.2.3 Lesser Himalayan Zone

The Lesser Himalayan Zone is bounded to the north by the Main Central Thrust (MCT) and to the south by the Main Boundary Thrust (MBT). A thrust called MBT started in the south of the MCT about 10–11 million years ago, and all the horizontal movement that had been occurring along the MCT shifted to the MBT (Meigs et al., 1995). This movement along the MBT effectively created high mountain range in the Lesser Himalaya known as the Mahabharat Range. The MBT can be traced out in the whole Nepal Himalaya and it can be well observed even in aerial photographs (Figure 2.11). The Lesser Himalayan Zone varies in rock type, age, and structures. Arita et al. (1997) and Paudel and Arita (2000) have mentioned that the Lesser Himalaya is composed of mainly of slightly

metamorphosed shelf-type sedimentary rocks of late Proterozoic to early Paleozoic ages. The rock types in Lesser Himalaya Zone are found different depending upon the area. Eastern Nepal is characterized by the development of extensive thrust sheets of high-grade metamorphic rocks (gneiss and schist) which have moved southwards. In central Nepal, a large thrust sheet called the Kathmandu Nappe covers a wide area around the Kathmandu region (Hagen, 1969). In the west of Kathmandu covered by autochthonous sequence (between the Budhi Gandaki and Bheri rivers) and high-grade metamorphic rocks (west of the Bheri River mainly in Dadelhura-Baitadi).

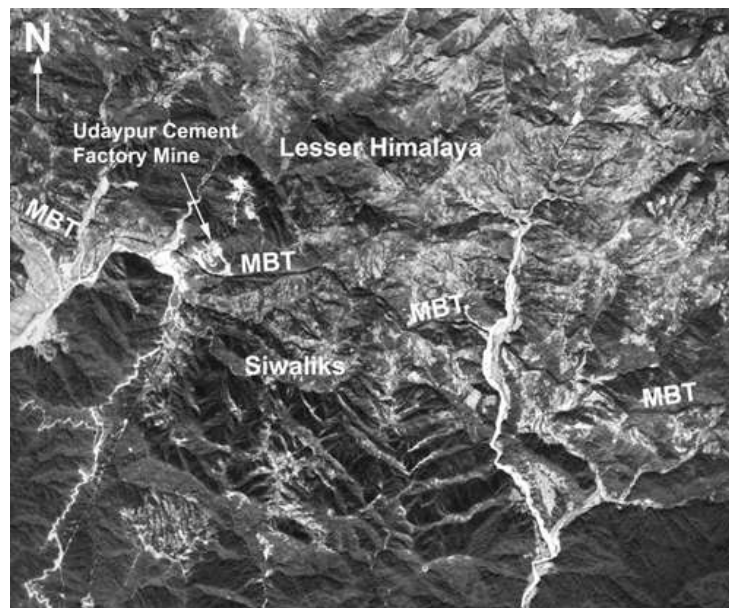


Figure 2.11: Main Boundary Thrust (MBT) observed in Udaypur district, (eastern Nepal) (Aerial photograph of Udaypur district, well marked the MBT is passing through middle of photograph) (after Dahal, 2009).

2.2.4 The Higher Himalayan Zone

The Higher Himalayan Zone is separated from the Tibetan-Tethys Zone in the north by normal fault system named as South Tibetan Detachment System (STDS) and from the Lesser Himalayan Zone in the south by the Main Central Thrust (MCT). The MCT was very active during the period between 25-19 million years ago, and the northern part of the Himalaya gained the present day height very quickly (Hasegawa, 2009). Steep slopes and deeply eroded river valleys are the outcomes of such movement in the Higher Himalaya. According to Arita et al. (1997), the Higher Himalaya composed of high-grade metamorphic rocks which have been thrust along the MCT zone over the rock of the lesser Himalaya. Many investigations have disclosed that movement along the MCT ceased by

around 18 Ma (Copeland et al., 1991; Hodges et al., 1996). Such movement caused Higher Himalayan rocks to slide horizontally towards south about 120 km over the young Lesser Himalayan rocks. Thus, it is believed that once upon the geological time much of the Lesser Himalaya must have been covered by the rocks of the Higher Himalayan crystalline rocks like a bedspread (Hasegawa, 2009). Evidences of such cover can be noticed very clearly in the geological map of Nepal. Most of the parts of Higher Himalayan crystalline rocks are already eroded in other area. Although it is presumed that MCT became inactive and the movement along this thrust practically ceased by 18 Ma, it has been found that MCT was reactivated during a period 6–8 million years ago (Harrison et al., 1999), and the Higher Himalaya again revitalized. Himalayas gain maximum topography around 17 Ma (Searle et al., 1997). As a result, normal fault system, STDS, developed as gravitational collapse structure, which is now considered as a tectonic boundary between the Tibetan-Tethys Zone and the Higher Himalayan Zone.

2.2.5 The Tibetan-Tethys Zone

The Tibetan-Tethys Zone begins from the top of the South Tibetan Detachment System (SDTS) (Figure 2.12) and extends to the north in Tibet. Most of the Great Himalayan peaks of Nepal such as Manaslu, Annapurna, and Dhaulagiri have rocks of Tibetan-Tethys Zone. This zone is composed of sedimentary rocks, such as shale, limestone, and sandstone ranging in age from Cambrian to Eocene.



Figure 2.12: South Tibetan Detachment System (STDS) separating Higher Himalayan Zone from Tibetan-Tethys Zone, Chhaktan Khola, north-west from Kokhethati, Mustang, Nepal (after Dahal, 2006).

2.3 SEISMICITY IN NEPAL AND THE HIMALAYA

Geologically, Nepal mountains and the Himalayan region were formed as a result of the collision of the Indian Plate with the Eurasian Plate about 55 million years ago (Molnar, 1986), which was then followed by subduction of the Indian Plate underneath the Eurasian Plate, which even continues today at an estimated rate of about 2 cm per year (Bilham et al., 1995). This has made Nepal and the entire Himalayan region one of the seismically active spots on the earth.

The major source of earthquakes in Nepal and the Himalayan Region is the subduction of the Indian plate underneath the Eurasian plate, resulting contraction effect, stress concentration. Seismicity is considered to be high in this region based on the frequency and strength of the past earthquakes (Figure 2.13). Seismicity of the Himalayan region has been studied in terms of its relationship with known geological faults and tectonic activities (Rai, 2004). During the past~120 years four great earthquakes were occurred along the Himalayan front (Seeber and Armbruster, 1981). From east to west, the sequence includes the 1905 Kangra earthquake ($M_w = 7.79$), the 1934 Bihar-Nepal earthquake ($M_w = 8.1$), the 1897 Shillong earthquake ($M_w = 8.03$), and the 1950 Assam-Tibet earthquake ($M_w = 8.44$) (Ambraseys and Bilham, 2000; Ambraseys and Douglas, 2004; Hough and Bilham, 2008; Pandey and Molnar, 1988) (Figure 2.14). Although none of the earthquakes were reported to have produced primary surface rupture (Seeber and Armbruster, 1981), it has generally been assumed on the basis of isoseismic map and location of damage that the earthquakes are the result of slip on the Himalayan Frontal Thrust (HFT). Lack of primary surface rupture during the major historical earthquakes has led previous workers to attribute their occurrence to a blind thrust, whereby strain release is expressed as anticline growth rather than primary surface rupture or coseismic surface rupture (Stein and Yeats, 1989; Yeats et al., 1992; Yeats and Thakur, 1998). However, Avouac et al. (2001) have mentioned that these events have ruptured 250-300 km long segments of the Himalayan arc with co-seismic slip estimated to around 5 km on average.

The rate of recurrence time for these great earthquakes is determined by the rate of slip of India beneath underneath Tibet. These slip rate is estimated based on global plate motions (DeMets et al., 1990), from seismic productivity (Bilham et al., 1997) and from the advance sediments on the northern Ganges plain (Bilham et al., 1997). Various Global Positioning System (GPS) geodetic survey were carried out in Nepal since 1991 for the measurement of these plate movements as a part of collaboration between the Department of Mines and Geology (DMG) of Nepal and various international organization/institutions,

such as IRES (Colorado University, USA), and French Laboratories under the Centre National de la Recherche Scientifique. Plate motion models and GPS measurements indicate that India-Eurasia convergence continues today at a rate of about 40-50 mm y⁻¹ (Demets et al., 1994; Paul et al., 2001), but the rate of contraction across the Himalaya is found to be 17.52±2 mm yr⁻¹ (Bilham et al., 1997). Armijo et al. (1986), Armijo et al. (1989), Avouac and Tapponnier (1993) have mentioned that the remaining motion is absorbed by a combination of thrusting, crystal extension and strike-slip motion within the Eurasian plate.

Himalayan foothills have long been known to be seismically active (Yeats et al., 1992). The major historical earthquakes, such as 1897, 1905, 1934, and 1950, are suggested to have originated as a result of slip on the MFT (Chander, 1989; Ambraseys and Bilham, 2000). Convergence south of the MFT in western Nepal and its absence in eastern Nepal reveals that deformation is not identical in both regions, indicating spatial and/or temporal variations in the convergence process (Bilham et al., 1997). In western Nepal, there have no recent great earthquakes for 200 years and perhaps for much longer. The pattern of crustal deformation across the eastern and central Himalayan implies that the MFT is locked over a distance of about 115 km from the surface to a depth of about 20 km beneath the front of the mountain range (Bettinelli et al., 2006). Across the western Nepal, the pattern of deformation and the shortening rate, given to be around 13.4±5 mm y⁻¹ and the width of the locked fault zone seems somewhat larger in this area. Apparently lower shortening rate in western Nepal than across eastern and central Nepal might be related to the even longer time span since the last large earthquake in this area. Since, there is an absence of historical earthquakes in several portions of the Himalayan arc, there is strain energy available to generate large earthquakes (Jouanne et al., 2004; Monsalve et al., 2006). Moreover, Bilham et al. (1997), Lavè and Avouac, (2000), and Molnar and Pandey (1989) have mentioned that the area particularly between the 1934 and 1905 seismic events appears as a long-standing seismic gap, long enough for two $M > 8$ faults segments, or may be an even larger magnitude event (Figure 2.14). Therefore, the area west of Kathmandu Valley thus stands as a potential location for the next large Himalayan earthquake. Figure 2.15 shows the distribution of Peak Horizontal Acceleration at bedrock that has a 10% probability of being exceeded over 50 years prepared by Department of Mines and Geology, Nepal (Pandey et al., 2002). The PGA distribution is varied from area to area and high PGA values can be seen along the western and central region of Nepal. This map neglects the effect of local geology on ground motion (i.e., site effects).

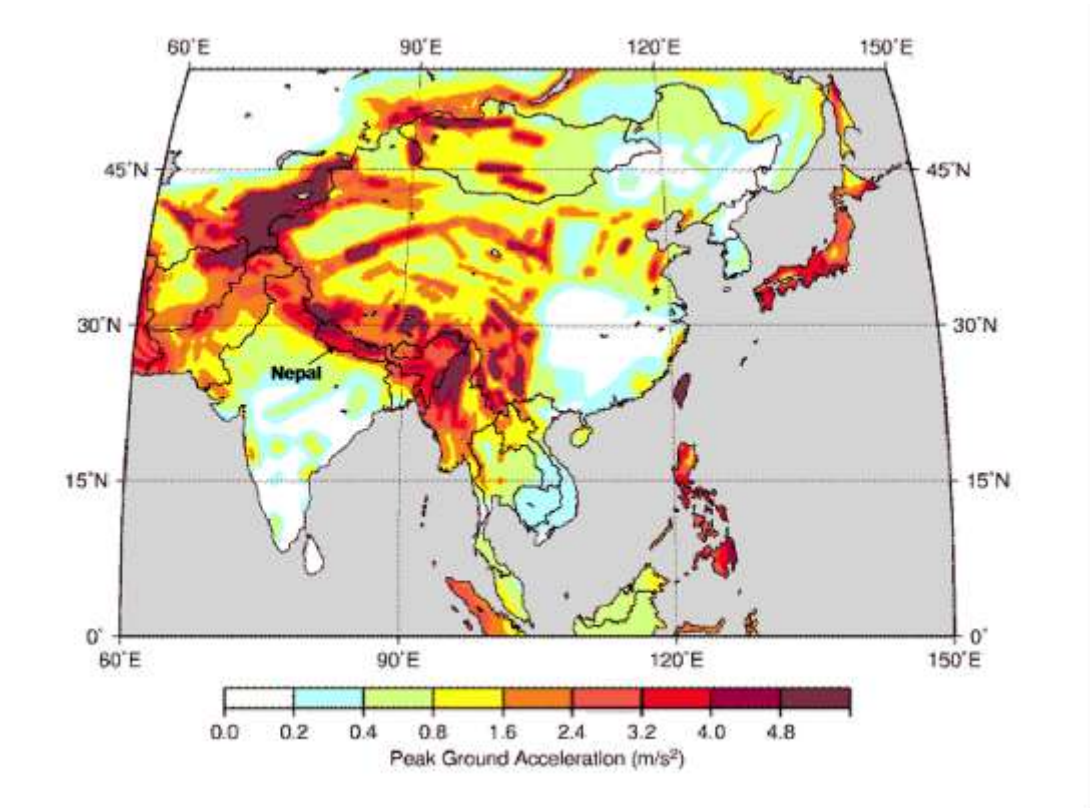


Figure 2.13: Location of Nepal showing its regional setting with respect to the seismic hazard zone along the Himalayan Arc. Peak ground acceleration (PGA) with a 10% chance of exceedance in 50 years (Zhang et al., 1999).

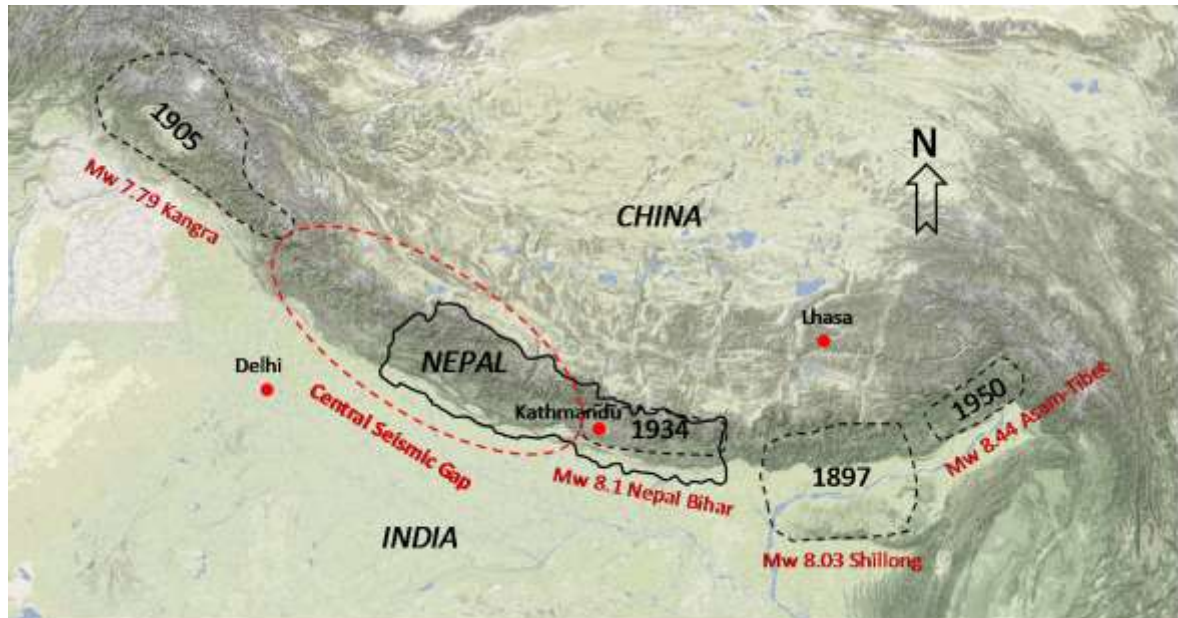


Figure 2.14: Distribution of probable rupture zones of the 1897, 1905, 1934 and 1950 earthquakes along the Himalayan arc. Modified after Yeats and Lillie (1991) and Yeats et al. (1992) and base map is taken from Google Terrain Map (2011).

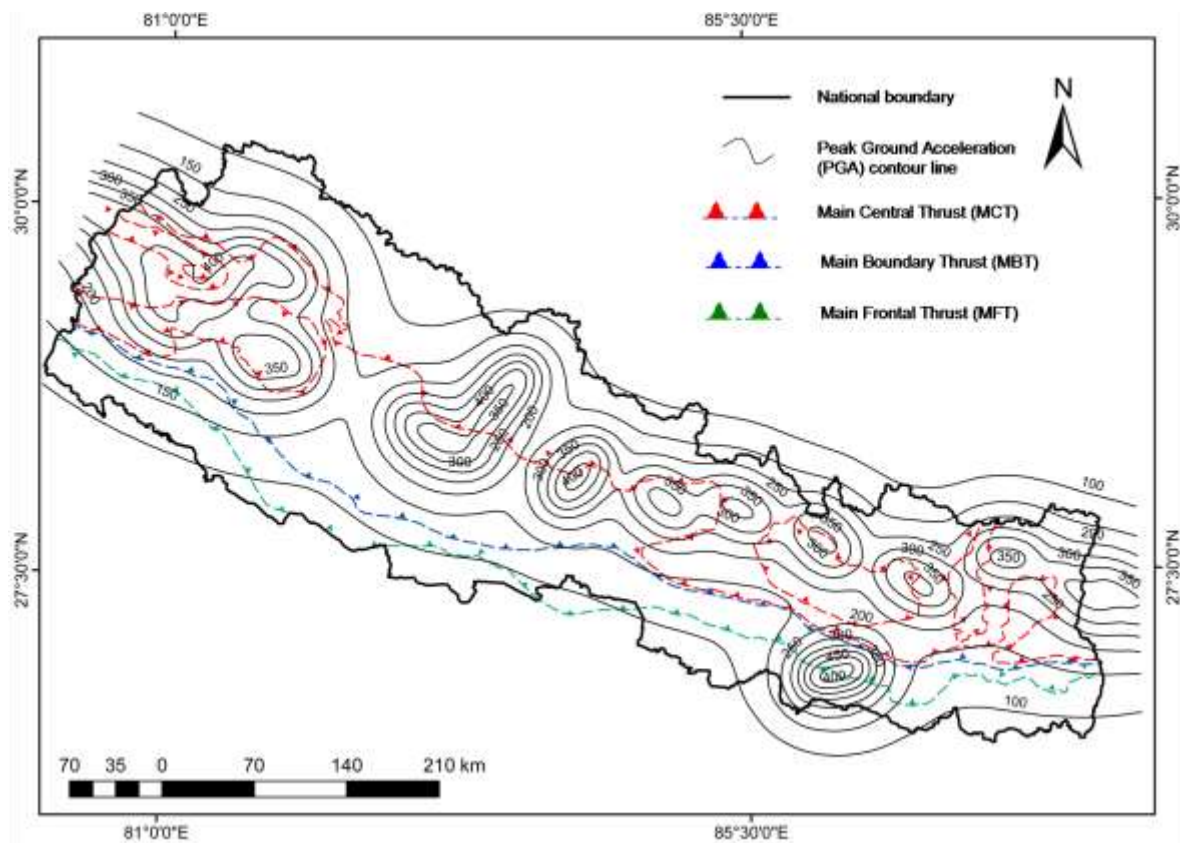


Figure 2.15: Distribution of Peak Ground Acceleration (PGA) with 500 year returns period (redrawn after Pandey et al., 2002).

2.4 KATHMANDU VALLEY

2.4.1 Physical and political features

Kathmandu Valley is located between latitudes 27°32'13" and 27°49'10" north and longitudes 85°11'31" and 85°31'38" east, which has an average elevation about 1,340 m from mean sea level. It encompasses three districts namely; Kathmandu, Lalitpur, and Bhaktapur as shown in Figure 2.16 and cover an area of 899 square kilometers, whereas the area of the valley as a whole is 665 square kilometers. The valley encloses the entire area of Bhaktapur district, 85% of Kathmandu district and 50% of Lalitpur district.

The valley is bowl shaped and surrounded by the Mahabharat Mountain range on all sides. There are four hills acting as forts of the valley, Phulchowki (2,762 m) in the south-east, Chandragiri (2,500 m) in the south-west, Shivapuri (2,732 m) in the north-west, and Nagarkot (2,195 m) in the north-east (refer Figure 2.16).

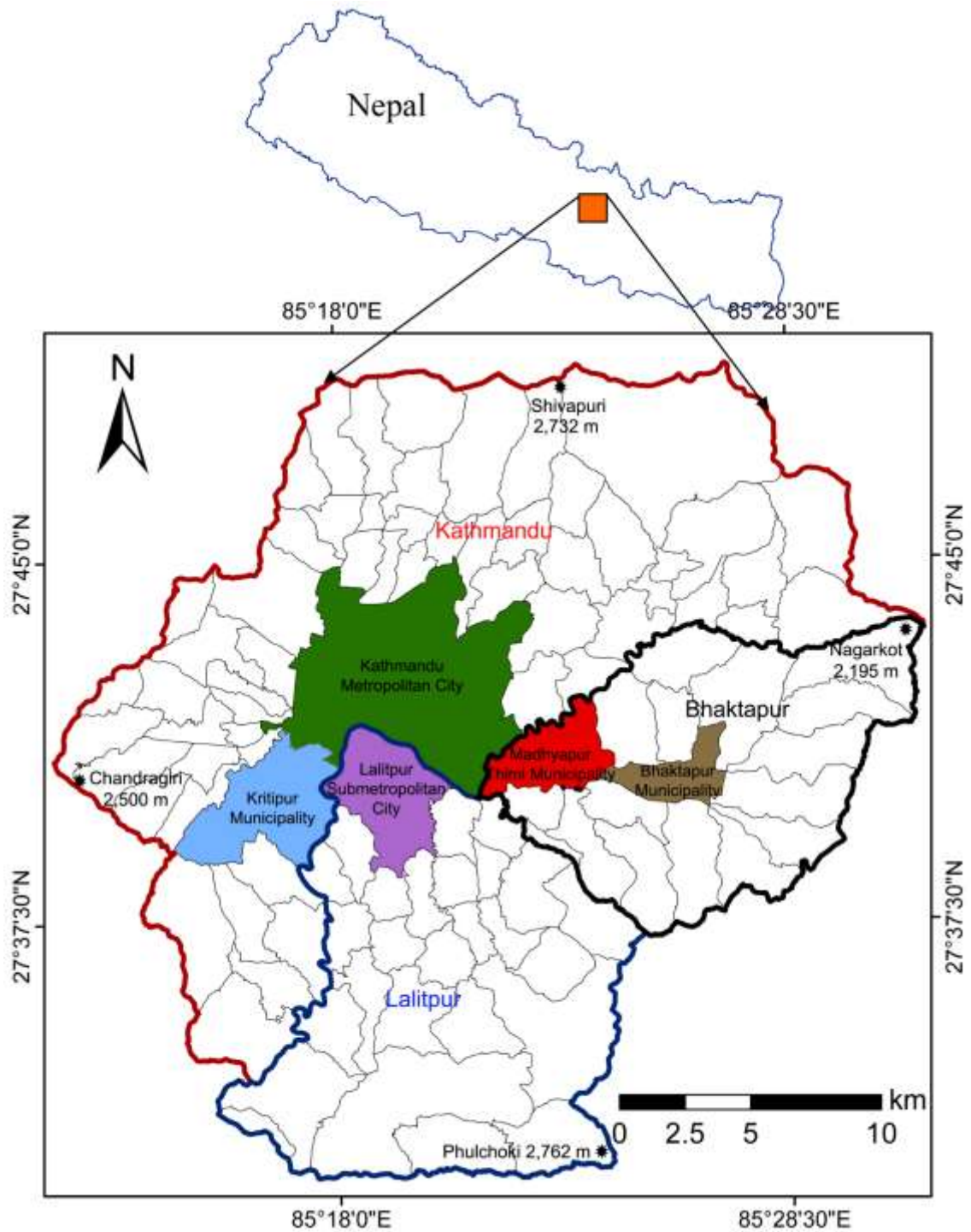


Figure 2.16: Detail map of Kathmandu Valley. Map showing the municipalities and village development committee in the three districts with in the valley.

The climate of Kathmandu is good, soil is fertile, and it is endowed with rich forests and scenic beauty. The three major river systems in the valley are the Bagmati, Bishnumati, and Manohara. There are small lakes and ponds in all three districts. In general, the annual maximum and minimum temperatures are between 29°C in June and 1°C in January. The

annual rainfall records for Kathmandu from 1995 to 2003 show fluctuations between 1,171 to 1,868 mm. Kathmandu Valley has five municipalities (i.e., Kathmandu Metropolitan City, Lalitpur Submetropolitan City, Kirtipur Municipality, Madhyapur Thimi Municipality, and Bhaktapur Municipality) and ninety-eight village development committees (VDCs), and 14 VDCs of the three districts fall outside the valley (Figure 2.16).

2.4.2 Cultural heritage site

Kathmandu Valley is known for its ancient art, culture, craftsmanship, and numerous monuments of historic and archaeological importance. UNESCO has described Kathmandu as a ‘living heritage site’. The valley has a number of temples, palaces, monasteries, and Buddhist stupas that are centuries’ old. There are many interesting sites within a radius of 20 km, and it used to be said that there were as many temples as houses and as many festivals as there are days on the calendar.

There are seven sites classified as world heritage sites by the UNESCO. The seven sites are as follows.

Kathmandu Durbar Square

Patan Durbar Square

Bhaktapur Durbar Square

Swayambhunath Stupa

Pashupatinath Temple

Boudhanath

Changunarayan Temple

Rapid urbanization and construction of concrete structures with little architectural merit threaten these sites. Besides the above the UNESCO world heritage sites, there are numerous important cultural sites in the valley. There are more than 360 ‘vihars’, ‘chaityas’, and ‘monasteries’. Even in rural areas, every village has religious and cultural monuments such as temples, vihars, stupas, or monasteries. There are also numerous wayside settlements and resting places, called ‘pati’, and stone water spouts.

2.4.3 Population and growth rate

The population census 1952/54, the first scientific census in Nepal, showed that the urban population of the Kathmandu Valley was 196,777 and in 2011, the population increases to 1,442,762, which is 21% of total urban population of Nepal. The increasing pattern of urban population of the Kathmandu Valley and Nepal is shown in Figure 2.17.

At present, Kathmandu Valley whole accommodates almost 40% of the total urban population of Nepal. If present population growth continues then the population of the Kathmandu Valley will be doubled in 17 years. Due to this rapid population growth, the urbanization has crossed the municipal boundaries of the cities of Kathmandu Valley. As per the local Self Governance Act, 1999 of Government of Nepal, there are three municipalities (Bhaktapur, Madhyapur, and Kritipur), one sub-metropolitan city Lalitpur, and one metropolitan city (Kathmandu) in the valley. The population of the three districts of Kathmandu Valley increased from 1,107,370 in 1991 to 1,647,092 in 2001. The population in designated urban areas of Kathmandu Valley has increased considerably. Most urbanized areas in the valley have contributed significantly to the overall urbanization status of the country. The urban population density of the Valley is 10,265 (the population is 995,966 and the area 97 sq. km) (CBS, 2003). On the other hand, the rural population is also increasing slowly in the valley. The average annual growth of the population is comparatively higher than for Nepal as a whole. Uneven allocation of resources for development and institutionalization in the valley has given rise to this movement of population.

As a city grows, the increasing population and economic activities demand more land be developed for public infrastructures (such as roads, water supply, and utilities), housing, industrial and commercial uses. Therefore, urbanization can be considered as gradual transformation of rural landscape into urban forms in which agricultural and forest land is gradually transformed into a built-up areas. The increase in built-up area of the valley from 1967 to 2000 is shown in Figure 2.18. These figures indicate that the built up area of the Kathmandu Valley was increased rapidly since 1978. This rapid transformation of rural landscape into urban landscape has caused various impacts on land use planning and infrastructure development. Data indicates that between 1984 and 1998, about 6,300 hectares (ha) of fertile and productive agricultural land were lost to urbanization, industrialization, and quarrying of sand, soil, and stone. Between 1984 and 1994, the valley's urban area increased from 3,096 ha to 8,378 ha, and 5,282 ha of fertile agricultural land were lost to urbanization (MoPE, 1999). It is estimated that more than half of the valley's 'A' grade land; i.e., 43% of the existing agricultural land will be lost to urban sprawl by 2010 (HFA, 1991; MoPE, 1999).

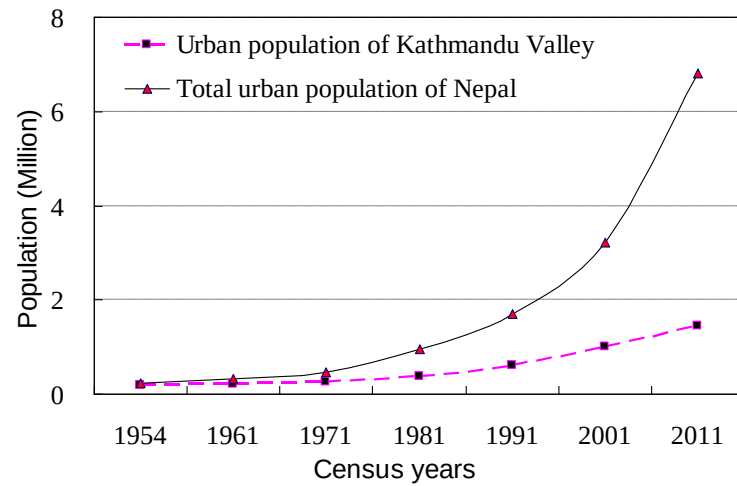


Figure 2.17: Urban population of Kathmandu Valley and Nepal.

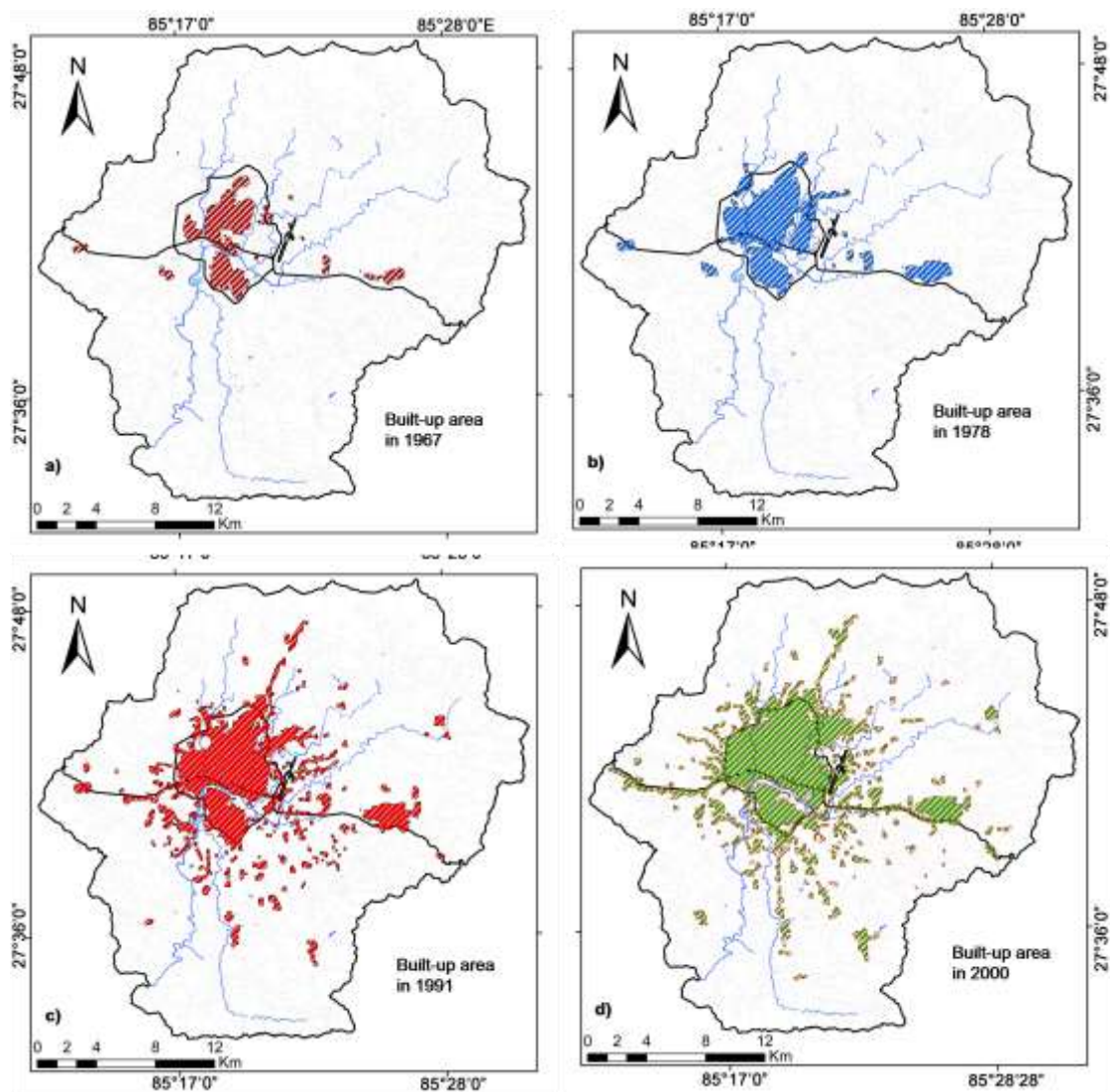


Figure 2.18: Increase of built-up area in Kathmandu Valley (redrawn after Thapa and Murayama, 2009).

One of the main reasons for the rapid rate of urbanization is migration from the villages. The recent data shows that when people migrate, they tend to migrate to large cities instead of small towns. Thus, the large cities will probably continue to grow more rapidly than small towns. Urbanization itself is not a problem, if it can be managed. In Nepal, however, institutional weakness from the perspective of managing urban development have resulted in haphazard growth, manifested in unplanned settlements, poor waste management, non-engineered building construction etc. Problem of unplanned growth is the development of incompatible urban forms, such as polluting industries in the middle of residential areas, haphazard planning of roads, haphazard planning of urban infrastructures etc.

2.4.4 Geological Features

2.4.4.1 Basement rock and faults

Geologically, the Kathmandu Basin (Figure 2.19) lies on the Kathmandu Nappe (Hagen, 1969), which is located along the southern slopes of the Himalaya. It is one of the several tectonic intermontane basins developed in the Lesser Himalayan belt (Sakai et al., 2002) as shown in Figure 2.19.

The Kathmandu Nappe (Hagan, 1969) is tectonically bounded by the MCT zone in its root zone, and extends southward near the MBT (Figure 2.19). According to Arita et al. (1973), and Stöcklin & Bhattarai (1981), the Kathmandu Complex has been regionally folded into a synclinorium, the axes of which tend in west-northwest-west-southwest direction. The Kathmandu Nappe is presently separated from the northern root zone by the Trisuli-Likhu fault which is the western continuation of the Sun Kosi Fault of eastern Nepal (Arita et al., 1973). The main fold axis lies on the line connecting the peaks of Phulchowki (2,762 m) and Chandragiri (2,500 m). There are many longitudinal faults which are run parallel to the fold axes. The northern and southern margins of the basins are bounded by the Kalphu Khola Fault and Chandragiri Fault respectively and they cut the late Pleistocene sediments (Figure 2.20). According to Saijo et al. (1995) and Yagi et al. (2000), there are also active faults in the southern part of Kathmandu Basin namely the Chobar Fault and Chandragiri Fault which are cutting the late Pleistocene sediments and have a vertical displacement rate of 1 mm y^{-1} . The fault systems in Kathmandu Basin are shown in Figure 2.20. The Chandragiri Fault is a thrust fault in which the rocks of the Lesser Himalayan in the south override the Quaternary sediments of the basin (Saijo et al.,

1995; Nakata et al., 1984). The Chandragiri Fault and Chobar Fault have played an important role for the basin development of the valley in the late Pliocene Period (Sakai et al., 2001 and Saijo et al., 1995). Sakai et al. (2001) have mentioned that Danuwargaon Fault in the southern margin of the Kathmandu Basin, which was very active in the late Pleistocene, might have been responsible for the draining of the Paleo lake of Kathmandu Basin.

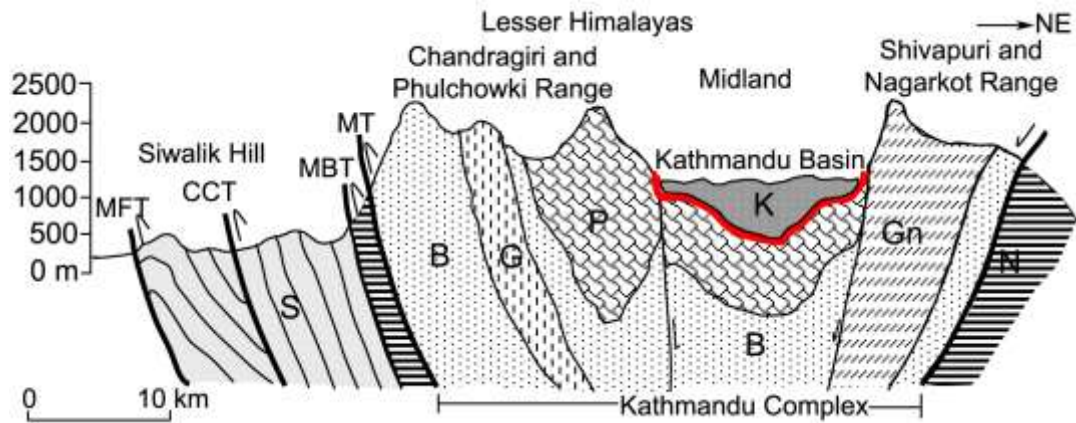


Figure 2.19: A schematic geological cross-section through Central Nepal (taken from Sakai et al., 2002, modified after Stocklin and Bhattarai, 1981). S: Shivalik Group, B: Bhimphedi Group, P: Phulchauki Group, N: Nuwakot Complexes, G: Granite, Gn: Gneiss complex, K: Kathmandu Complex, MFT: Main Frontal Thrust, CCT: Central Churia Thrust, MBT: Main Boundary Thrust, MT: Mahabharat Thrust. The thick red line represents the Kathmandu Basin.

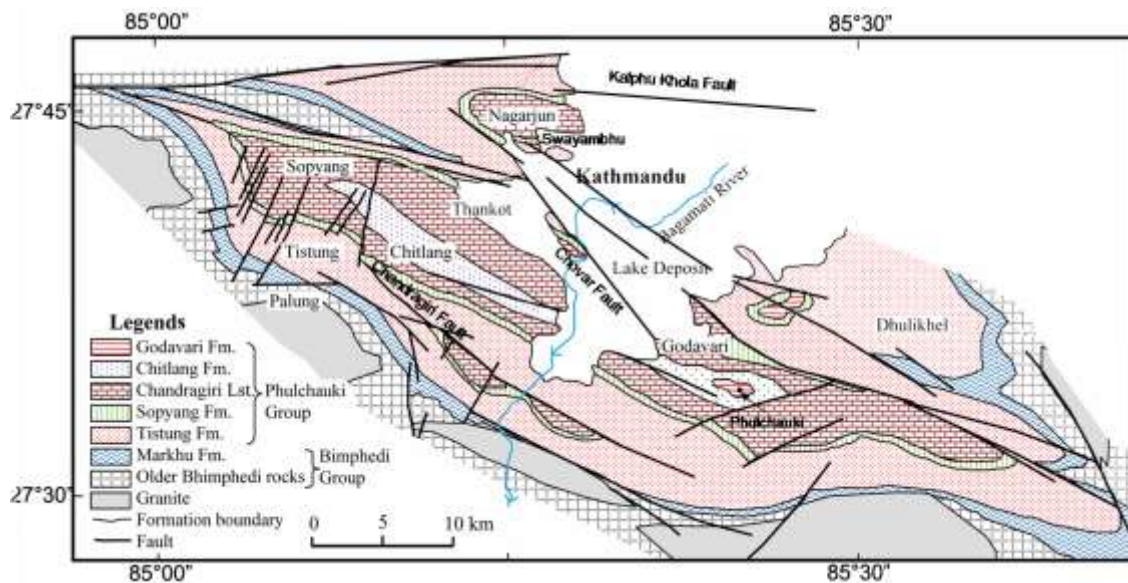


Figure 2.20: Simplified geological map of Kathmandu area (after Stöckline, 1980; Sakai, 2001; and Dahal, 2009).

The basement geology of the Kathmandu Basin has been described in detail by Stöcklin and Bhattarai (1977). The basement rock consists of Phulchauki Group and Bhimphedi Group of the Kathmandu Complex (Stockline and Bhattarai, 1977) (Figure 2.19) and is formed by Precambrian to Devonian rocks (See Table 2.1 and Table 2.2). The rocks of Phulchauki Group and Bhimphedi Group together form the so called Kathmandu Complex. As illustrated in Figure 2.19, the early Paleozoic Tethyan rocks, named as the Phulchauki Group, overlie the Bhimphedi Group in the Kathmandu region. The Total thickness of both groups attains 13 km (Stöcklin and Bhattarai, 1977). The northern slope of the Kathmandu Valley is mainly composed of gneiss, schist and granite, but the other slopes and the central part of the valley consists of weakly metamorphosed Phulchauki Group. The Nuwakot Complex is structurally located between the MCT and MBT, and underlies the Kathmandu Nappe as shown in Figure 2.19. According to Le Fort (1975), the Nuwakot Complex displays an inverted metamorphism as a result of the thrusting of a hot slab over cold sediments along the MCT zone.

Table 2.1: Stratigraphic sequences of basement rocks of Kathmandu Basin (Source: Stöcklin and Bhattarai, 1977).

Unit		Main lithology	Thickness approx. (m)	Age	
Kathmandu Complex	Phulchauki Group	e) Godabari limestone	Lime stone, dolomite	300	Devonian
		d) Chitlang Formation	Slate	1000	Silurian
		c) Chandragiri Limestone	Limestone	2000	Cambrian to Ordovician
		b) Sopyang Formation	Slate, calc-phyllite	200	? Cambrian or lower
		a) Tistung Formation	Metasadstone, phyllite	3000	Pre-Cambrian
	Transition				
	Bhimphedi Group	f) Markhu Formation	Marble, Schist	1000	Precambrian
		e) Kulekhani Formation	Quartzite, Schist	2000	Precambrian
		d) Chisapani Quartzite	Quartzite	400	Precambrian
		c) Kalitar Formation	Schist, Quartzite	2000	Precambrian
		b) Bhainsedobhan Marble	Marble	800	Precambrian
		a) Raduwa Formation	Garnet-Schist	1000	Precambrian

Table 2.2: Geological succession of the Kathmandu Valley (Source: Stöcklin and Bhattarai, 1977).

Cenozoic	Holocene		Fan Gravel, Soil, Talus, Fluvial deposits (gravel, sand, silt)
	Pleistocene		Lake deposits (gravel, sand, silt, clay, peat, lignite and diatomite)
	Late Pliocene-Early Pleistocene		Fluvial deposits (boulder, gravel, sand, silt)
Unconformity			
Lower Paleozoic	Devonian	Phulchauki Group	Godavari limestone-Limestone, dolomite
	Silurian		Chitlang Formation-Slate
	Cambrian-Ordovician		Chandragiri Limestone-Limestone, Phyllite
	Cambrian		Sopyang Formation-Slate, calcareous Phyllite
	Early Cambrian		Tistung Formation-meta sand stone, Phyllites
Unconformity			
Pre-Cambrian		Bhimphedi Group	Markhu Formation-marble, Schist
			Kulekhani Formation-Quartzite, Schist
In trusion			Metamorphic-sheopuri gneiss
			Igneous Rocks-pegmatite, granite, basic intrusive

2.4.4.2 Quaternary Sediments

The Kathmandu Basin is filled with upper Pliocene to Quaternary clay, silt, sand and gravel (Moriyabashi and Maruo, 1980; Yoshida and Gautam, 1988; Sakai, 2001) overlaying the Precambrian Bhimphedi Group and the lower Paleozoic Phulchauki Group (Stocklin and Bhattarai, 1981). Maribayashi and Maruo (1980) have done a study on the basement topography of the center part of the Kathmandu Basin based on the gravitational method. They estimated the maximum thickness of the basin-fill sediments to be 650 m. The main source of basin-fill sediments are the hills around the valley and these sediments are mainly derived from the hilly areas by an ancient drainage channel system in around the valley. The southern part sediments were derived from the southern hills such as Chandragiri and Phulchauki which consist of Argillaceous meta-sedimentary rocks where as the northern part sediments were derived from the Shivapuri hills which consist of intrusive gneiss and granites rock. Katel et al. (1996) and Sakai et al. (2002) mentioned

that more than 300 m thick muddy and sandy sediments of lacustrine origin are extensively distributed under the Kathmandu Basin.

Due to typical lake with its own kind and with containing a lot of seismo-tectonic information, Kathmandu Basin has attracted many researchers for the investigation of geology and stratigraphy of the sediment deposits (Dangol, 1985; Fujii and Sakai, 2002; Moribayashi and Maruo, 1980; Saijo and Kimura, 2007; Sakai, 2001; Sakai et al., 2002; Sakai et al., 2001; Tuladhar, 1982; Yoshida and Igarashi, 1984), lithological and mineralogical evaluation of soil (Fujii et al., 2001; Paudel, 2010; Rai et al., 2004), palynology investigation (Goddu et al., 2007; Kern, 2010; Paudyal, 2011; Paudyal and Ferguson, 2004; Yoshida and Gautam, 1988), and engineering geological and geotechnical properties of the Kathmandu lacustrine sediment (Dahal and Aryal, 2002; Katel et al., 1996).

Yoshida and Igarashi (1984) have classified the basin fill sediments into three groups and seven stratigraphic units (Table 2.3). Koirala et al. (1993) have prepared a geological map of the basin in which they have mentioned that the Lukundol Formation which includes the carbonaceous silty-sandy clay and sandy-clayey silt beds that overlie the gravel bed is extremely distributed in the central part of the valley. Shrestha et al. (1998) have prepared an engineering and environmental geological map of the Kathmandu Valley (Figure 2.21). This is the first detailed geological map of the valley which can be available from Department of Mines and Geology of Nepal. In the map, they have shown Lukundol Formation as narrowly distributed in the eastern bank of the Bagmati River and along the Nakhu Khola in the southern part of the valley. But they have shown same lithology and structure in the opposite bank of the Bagmati River by different name as Kobgaon Formation (Sakai et al., 2001). Similarly, they have given the name as Kalimati Formation for the central part of the valley which consists of thick muddy sequence. The detail of the formation name proposed by Shrestha et al. (1998) for the basin-fill sediments of the valley is given in Figure 2.21 and Table 2.3

From literature review, it is understood that different authors have given the name of the formation differently based on the name of the location and/or in own way. There is ambiguity in the naming of the formation. Different formation names are given to the similar type of deposits. The classification of sediments and stratigraphy type described by the different authors are mainly based on the field observation and with from some borehole information. However, the previous drilled boreholes have very limited information and the cores have never been recovered during these drillings (Sakai, 2001).

Sakai (2001) have mentioned that these ill-defined and inconsistencies in the usage of formation names have given rise to many confusions and misunderstanding on the stratigraphy of the Kathmandu Basin sediments.

To overcome the problems in the previous stratigraphic divisions of the basin-fill sediments of the Kathmandu Valley, Sakai (2001) has proposed a new classification system, which is based on the field observations and detail study of the boreholes lithology. For this purpose, Sakai et al. (2001) have conducted a core drilling of the basin-fill sediments during the Paleo Kathmandu Lake Project in 2000. This was the first-large scale drilling project in the valley with full-core recovery and solely dedicated to academic research purposes. They have drilled three boreholes in different location of the Kathmandu Basin namely, Tri-Chandra College (BH₁), Rabi Bhawan (BH₂), and Pulchowk (BH₃) as shown in Figure 2.24. Out of these three boreholes, Rabi Bhawan borehole was reached up to the bedrock level, and Sakai et al. (2001) were carried out detailed investigations. Based on this study, (Sakai et al., 2001) have divided the history of Palaeo-Kathmandu Lake into seven stages ranging from stage (1)- prior to the appearance of the lake to stage (7)- draining out of the lake-water (Figure 2.22) as follows:

Stage (1) – Deposition by Proto-Bagmati River

Stage (2) – Appearance of lake

Stage (3) – Expansion and deepening of lake water

Stage (4) – Lowering of lake water

Stage (5) – Gradual deepening of lake water

Stage (6) – Eutrophic condition

Stage (7) – Erosion and deposition by rivers

Subsequent details of the different stages of changes of lithology and sedimentary facies of the sediment is found in Sakai et al. (2001). Later, Sakai (2001) has proposed a different classification of basin fill sediment as per the study of drilled hole and field observations as mentioned in the following section (refer Table 2.3).

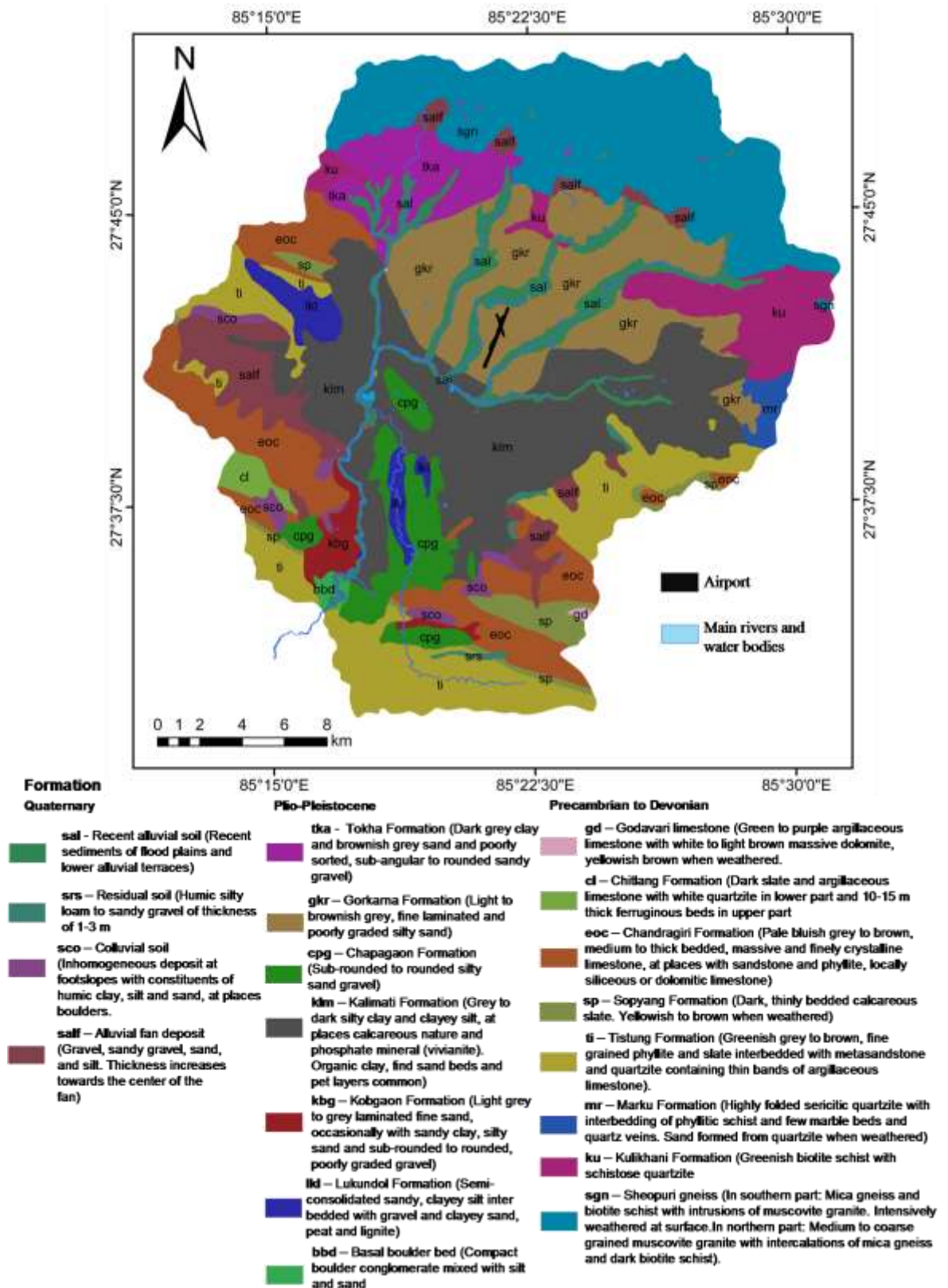


Figure 2.21: Engineering and environmental geological map of the Kathmandu Valley (redrawn after Shrestha et al., 1998, source: Department of Mines and Geology, Nepal).

Table 2.3: Correlation of stratigraphy of the Kathmandu basin sediments by different researchers.

Yoshida and Igarashi (1984) Yoshida and Gautam (1988)			Dangol (1985)	Shrestha et al. (1998)	Sakai et al. (2001)		Sakai, 2001			
							Southern part	Central part		
Patan Formation			Kalimati clay	Gokarna Fm			Itaiti Fm	Patan Fm		
Thimi Formation				Tokha Fm				Kalimati Fm		
Gokarna Formation				Kalimati Fm						
Boregaon terrace deposit			Champi-Itah ari gravel	Chapagaon Fm	Upper member			Basal lignite member		
Chapagaon terrace deposit				Lukundol Fm and Kobegaon Fm						
Pyangaon terrace deposit										
Lukundol Formation	Member	VII							Lukundol Formation	Middle member
		VI								
		V	Nakhu							
		IV	Khola							
		III	mudstone							
		II	and Keshari nayakhandi lignite							
		Tarebhir	Basal boulder		Lower member	Tarebhir Fm	Bagmati Fm			
		I	basal gravel bed							

1) Soil type classification

After review borehole data and available literatures of the Kathmandu Basin, it has been found that there is inconsistency in the classification of soil as well. Various names have been given for same type of soil. Moreover, the soil types have been named as per their own system of each worker. These previous workers did not follow any standard system of soil classification which causes the problems mainly during the analysis of engineering properties of soil, such as density, shear modulus, modulus of rigidity, shear wave velocity etc. This ultimately effect on the engineering design of the structures. To overcome these problems, in this study, we have classified the available soil types of each borehole based on the Unified Soil Classification System (USCS), as illustrated in Figure 2.23 (ASTM, 1996).

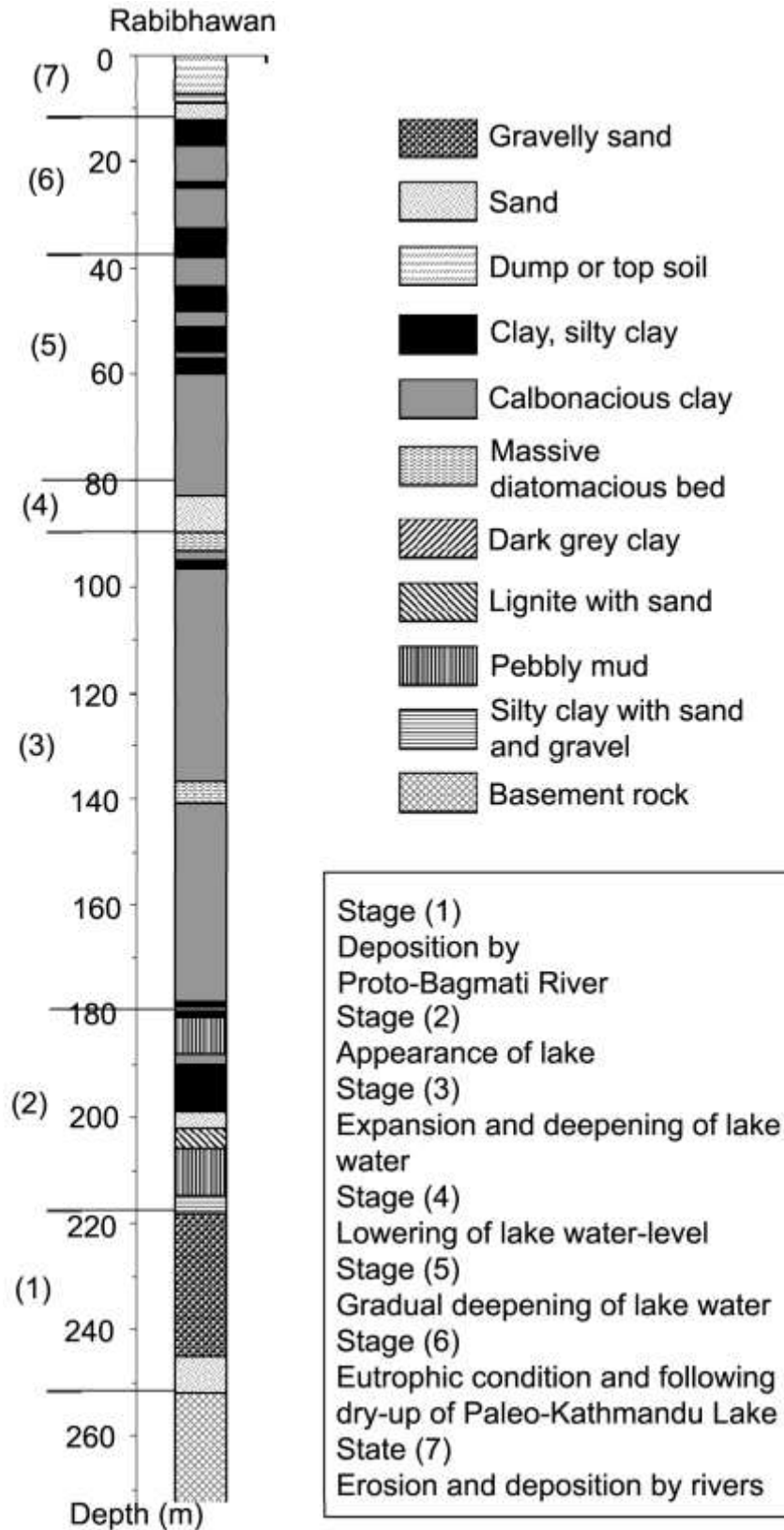


Figure 2.22: Litho-stratigraphic section in central part of the Kathmandu Basin, on the basis of drill-cores (BH₂) (refer Figure 2.24) collected by the Paleo-Kathmandu Lake Project in 2000.

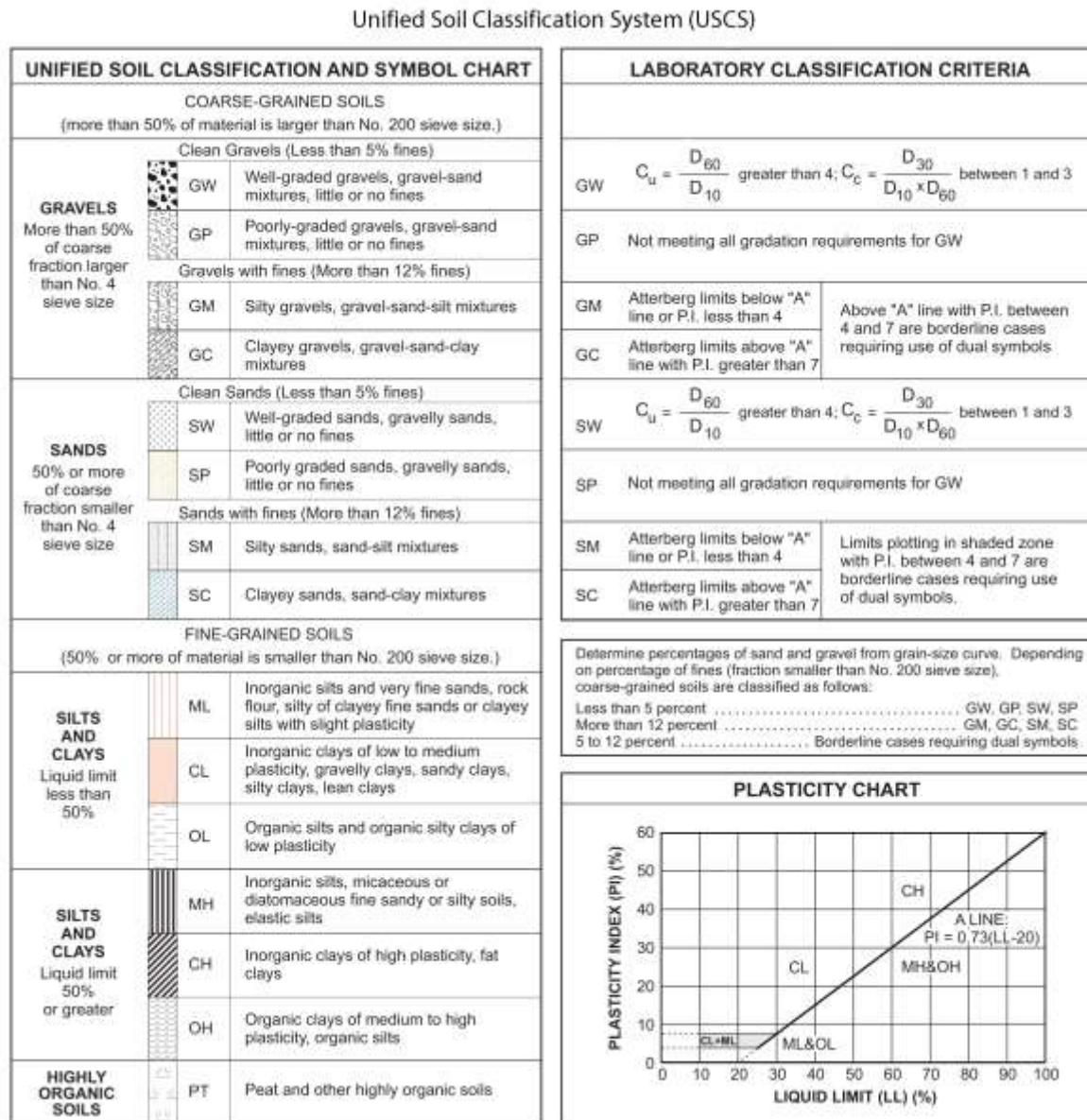


Figure 2.23: Unified Soil Classification System (ASTM, 1996).

However, the various soil parameters, such as percentage of fines, plasticity index (PI), liquid limit (LL), are still not available in the borehole data of the Kathmandu Valley, classification of soils are done based on the content of soil parameter, such as amount of soil type as well as other parameters mentioned in the borehole data, that matches with the USCS classification system.

2) Stratigraphic division of the Kathmandu Basin

Based on detailed study on three boreholes (i.e. BH₁, BH₂, and BH₃) (Figure 2.24) and available borehole data, Sakai (2001) has proposed stratigraphic schemes for the southern,

central and northern part of the valley. According to his study, the northern part of the valley mainly consists of marginal fluvio-deltaic facies (i.e., typical riverbed layered deposits of clays, silts, sand and gravels), while the southern part consists mainly of alluvial fan facies (i.e., lake deposits of clays and silts, usually sandwiched between thin layers of coarse sediments), and the central part of the basin consists of open lacustrine facies (i.e., a thick organic mud, the black clay) as shown in Figure 2.24. Figure 2.25 shows a schematic geological cross-section of the subsurface structure of the basin (Katel et al., 1996; Sakai et al., 2002), which indicates an extensive distribution of organic clay sediments of lacustrine origin. Sediment types and their ages in the Kathmandu Basin are tabulated in Table 2.4. The detail description in each part is given below.

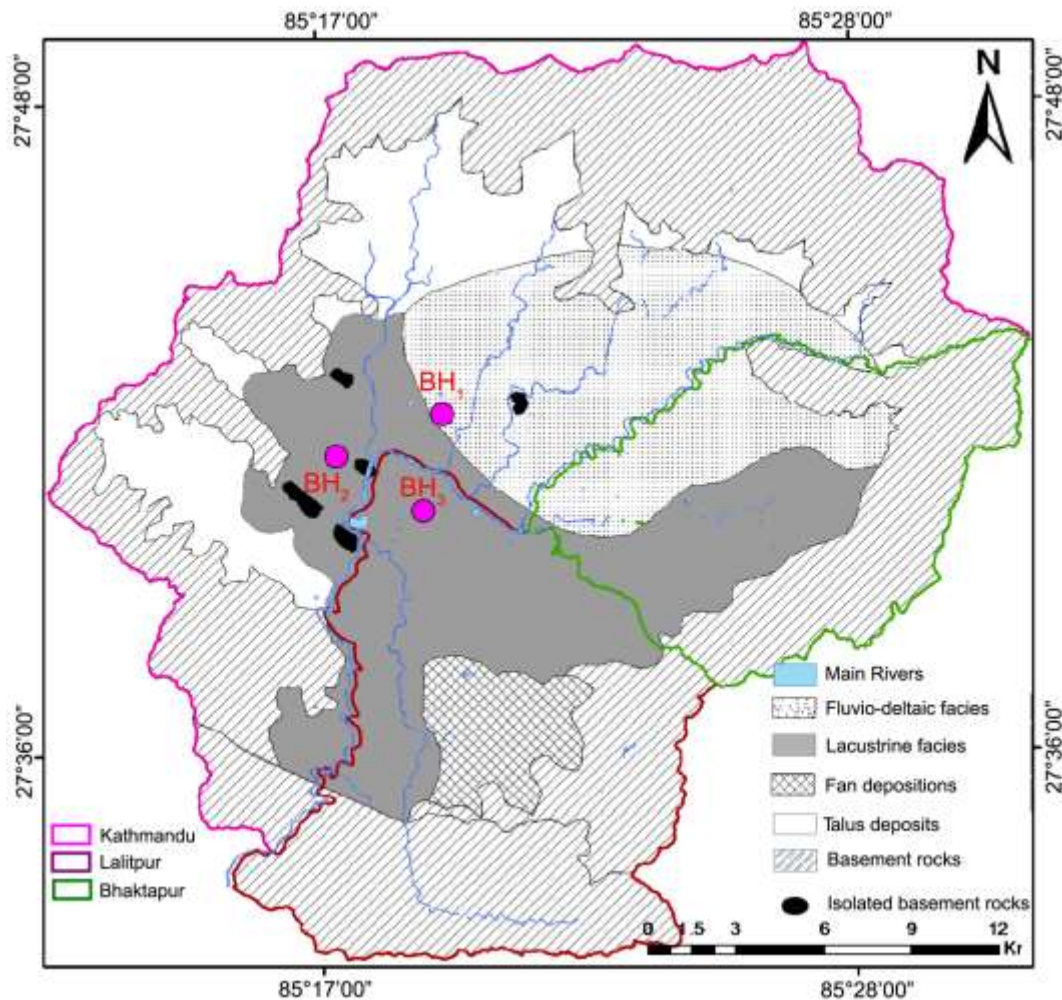


Figure 2.24: Sedimentary facies map showing the distribution of lacustrine, marginal lacustrine, and alluvial depositional environments of the Palaeo-Kathmandu Lake during the late Pleistocene. BH₁, BH₂ and BH₃ are the Borehole drilled during Paleo-Kathmandu Lake Project in 2000.

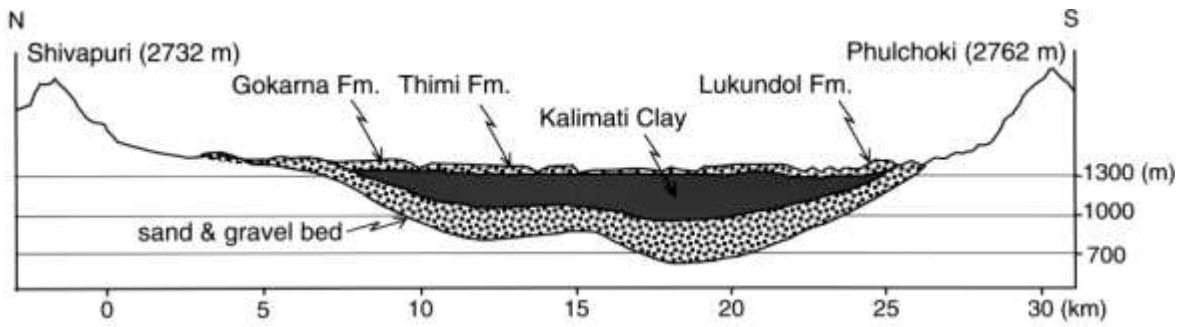


Figure 2.25: A schematic geological cross-section of the Kathmandu basin, showing north-south sediment distribution of the Kathmandu valley (after Sakai et al. 2002).

Table 2.4: Simplified stratigraphy and geology of the Kathmandu basin (modified after Sakai, 2001).

Region	Name of formations	Age of formations	Predominant sediments
Northern	Gokarna	1 myr -29 kyr	Gravel, medium to coarse grained sand, micaceous fine to medium-grained sand, mud-supported gravel bed, black and grey silt.
	Thimi	29 kyr – 19 kyr	Layers of clay, silt, silty sand and micaceous fine sand and gravels.
Central	Bagmati	Older than 2.8 myr	Fluvial gravel and sand deposits
	Kalimati	2.8 myr - 30 kyr	Massive black organic clay , sandy silt with gravel
	Patan	19 kyr – 11 kyr	Fluvial deposits (river channel deposits – sand silt and gravel)
Southern	Tarebhir	Older than 2.8 myr	Fluvial gravel and sand beds deposits
	Lukundol	2.8 myr – 1 myr	Carbonaceous muddy sediments
	Itaiti	1 myr - present	Boulder beds of alluvial fan deposit

3) The southern part

The southern part of the valley consists of mainly alluvial fan facies (i.e., lake deposits of clays and silts, usually sandwiched between thin layers of coarse sediments) (Sakai, 2001). These are occupied by many hilly terraces formed during late Plocene to middle

Pleistocene (Yoshida and Igarashi, 1984) and well exposed along the terraces and river beds. Sakai (2001) has subdivided the southern part sediments into three formations: 1) Tharebhir Formation, 2) Lukundol Formation, and 3) Itaiti Formation in ascending order (refer Table 2.3 and Figure 2.26).

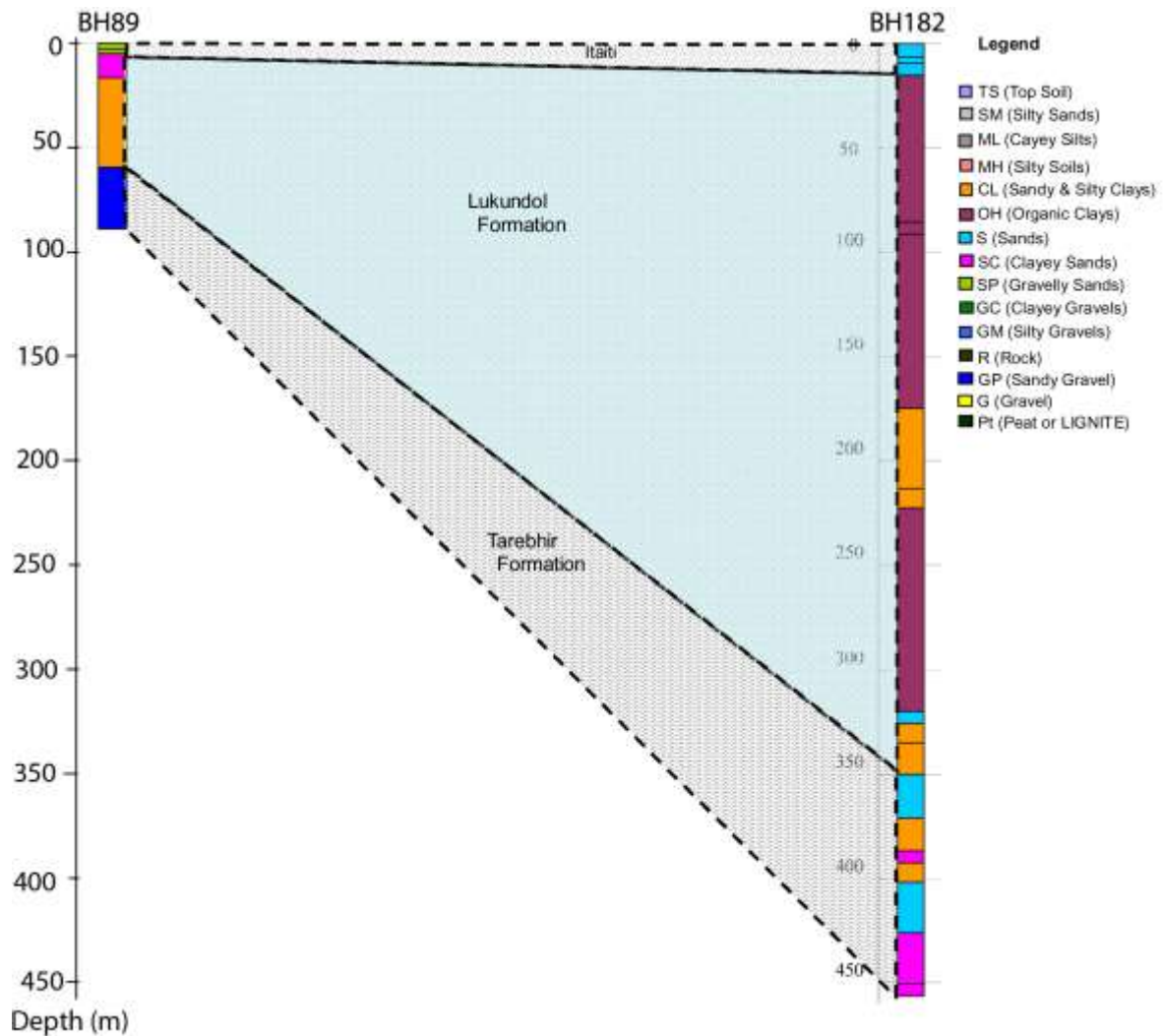


Figure 2.26: Stratigraphic division in southern part.

a) Tarebhir Formation

According to Sakai (2001), the Tarebhir Formation is the oldest basin fill sediments, and is unconformably overlying the Precambrian Tistung Formation. The formation is mainly composed of boulders and cobbles with minor amount of lenticular sand beds (Figure 2.26), which are fluvial in origin and were derived from the Chandragiri and Phulchauki hill in the south by the ancient river system. The thickness of this formation

varies from 1 m at Tokalmat area to 350 m at Dukuchap area (Sakai, 2001). The age of this formation is believed to be from late Pliocene to early Pleistocene (Yoshida and Igarashi, 1984).

b) Lukundol Formation

The Lukundol Formation is a mud dominant sequence of marginal lacustrine facies, which is conformably overlying the Tarebhir Formation (Figure 2.26). The name of the formation is named after the Lukundol village that lies in the southern part of the Kathmandu Basin. The formation consists of black to grey organic mud, rhythmite of sand and silt, coarse-grained sand and granule. Common occurrence of lignite beds and vertebrate fossils such as ancient elephant, crocodile, deer, and artiodactyl are characteristic features of this formation (Sakai, 2001). The fossils are believed to have been formed in marginal lacustrine facies deposited in swamp and lake of shallow water deposit, where many animals were living and aquatic plants also flourished. In the central part, the formation is correlated with the Kalimati Formation (Basal lignite member) of Sakai (2001). The age of this formation is believed to be from early to middle Pleistocene (Yoshida and Igarashi, 1984).

c) Itaiti Formation

Itaiti Formation is a cliff forming thick gravel dominant sequence resting on the Lukundol Formation. The formation is named after the Itaiti village. This formation comprises alternate sequence of gravel, fine sand and silty clay with carbonaceous mud in ascending order. The base of this formation consists of pebble and cobble beds of meta-sandstone derived from the southern hills and rests on the top of the Lukundol Formation (refer Figure 2.26). The Itaiti Formation is correlated with Terrace deposit of Yoshida and Igarashi (1984) (Table 2.3). The age of this formation is believed to be middle Pleistocene (Yoshida and Igarashi, 1984).

4) The Central part

Based on the compilation of large number of borehole data obtained from the Kathmandu Basin, the basin fills sediments in the central part can be roughly divided into three parts. They are: sandy lower part, muddy middle part and the sandy thin upper part (Katel et al., 1996; Kharel et al., 1998). However, the precise lithologic characters and stratigraphy were not well known, because the cores have never been recovered during

previous drilling. Based on the core drillings conducted in 2000, Sakai (2001) proposed the sediments in the central part of the basin namely, Bagmati Formation, Kalimati Formation and Patan Formation from bottom to top respectively (Table 2.3 and Figure 2.27).

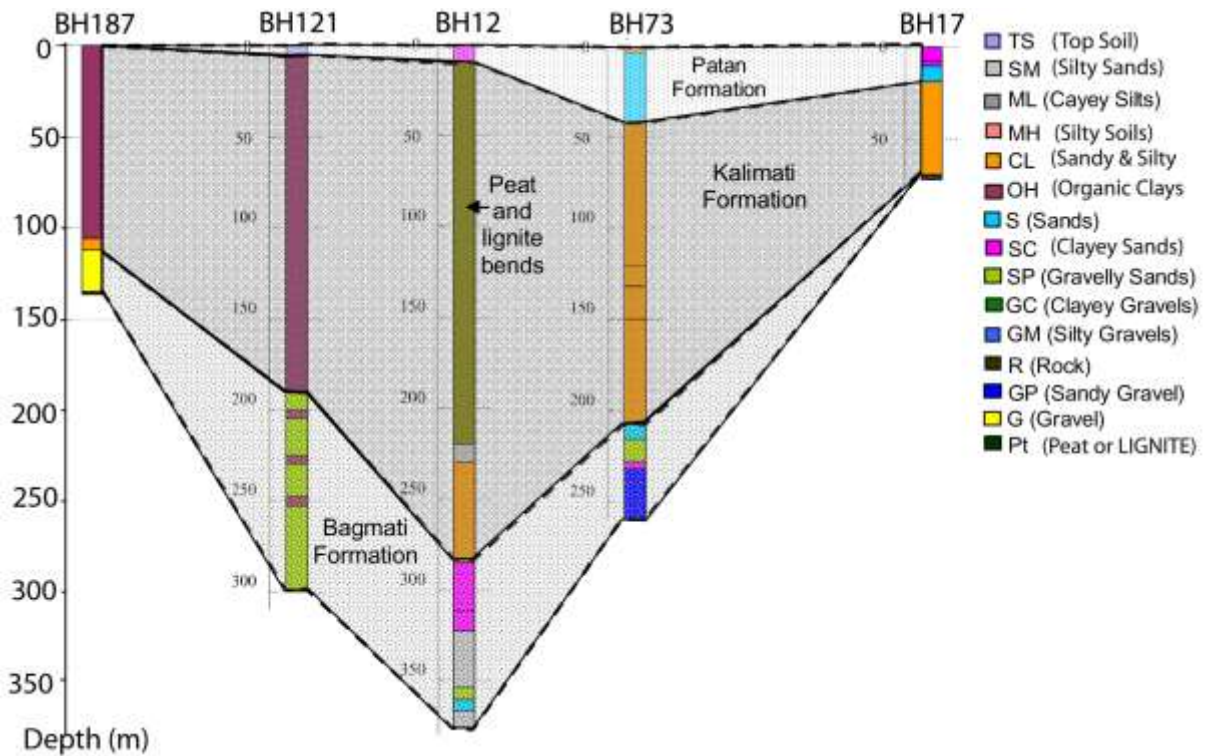


Figure 2.27: Stratigraphic division in central part.

a) Bagmati Formation

The Bagmati Formation was named after the Bagmati River (Sakai, 2001). The sediments in this formation is interpreted to have been deposited by the Proto-Bagmati River (Hagen, 1969) which was very active before the appearance of the Paleo-Kathmandu Lake, and Proto-Bagmati River was responsible for the deposition of the sediments in most part of the valley. Bagmati Formation consists of mainly medium to coarse sand, gravels and boulders that can be observed in the borehole log distributed in this part (Figure 2.27). The sediments of this formation were derived from the surrounding hills particularly from the Shivapuri hills in the northern part of the valley. The formation covers the basement rocks of the Kathmandu Complex. Figure 2.27 shows a typical Bagmati Formation at the central portion of the Basin.

b) Kalimati Formation

The formation name was given after a locality called Kalimati in the western part of the Kathmandu (Sakai, 2001). This formation mainly consists of dark gray carbonaceous laminated clay and diatomaceous beds of open lacustrine facies. A 4 to 6 m thick, medium- to coarse-grained sand layer is interbedded in the middle part of the formation. At some places thick lignite and bituminous pebbly mud can be seen (refer Figure 2.27) (Sakai, 2001). The thickness of this formation varies from place to place. The thickness of this formation varies from few meters upto more than 300 m, which is seen at borehole 182 (Harisiddhi) (Figure 2.26) and also described by Sakai (2001). The Kalimati sediment is extremely distributed beneath the central portion of the Kathmandu Basin (Figure 2.27). According to Yoshida and Igarashi (1984), this formation was formed during the period between 2,500,000 to 29,000 years B.P.

c) Patan Formation

The upper portion of the central part sediment is called Patan Formation (Sakai, 2001, and Yoshida and Igarashi, 1984), which is mainly distributed in an around the Kathmandu and Patan city. This formation consists of mainly fine to medium sand and silt intercalated with clays and fine gravels in some places (Figure 2.27). It is fluvial in origin. The sediments of this formation that overlie the Kalimati Formation may have been developed after the lake started drying up and fluvial process became active. The thickness of this formation varies from place to place but found maximum in the northern part than the other areas as shown in the Figure 2.27. According to Yoshida and Igarashi (1984), the age of this formation is about 19,000 to 10,000 BP.

5) The Northern part

According to Yoshida and Igarashi (1984), the northern part of the Kathmandu Basin mainly consists of terrace forming sands from fluivo-lacustrine origin which are extensively distributed in the northern and north eastern part of the Kathmandu Valley (see Figure 2.28). These sediments are represented by two formations namely, Thimi Formation and Gokarna Formation from top to bottom (Yoshida and Igarashi, 1984; Sakai, 2001). The age of the Gokarna Formation is determined as 28,000 to 30,000 years BP and the age of the Thimi Formation is determined as 23,000 to 28,000 years BP (Yoshida and Igarashi, 1984) and hence the Thimi Formation is younger than the Gorkarna Formation. The boreholes data in the northern part of the valley (refer Figure 2.11) indicate the distribution

of thick sequence of sandy and silty sediments without clay, which is regarded as the continuation of Thimi and Gokarna Formation (Sakai, 2001). From the lithological cross section drawn from south to north, it is evident that the Kalimati Formation diminishes its thickness and pinches out in the area to the north of Dhapasi (BH22 and BH21) which can be seen from Figure 2.28. Similar features are also evident in the eastern part in Mulpani village area, which is considered to be the northern, marginal line of the lake in which the basin fill sediments change their sedimentary facies to gravely braided river and fan deposits.

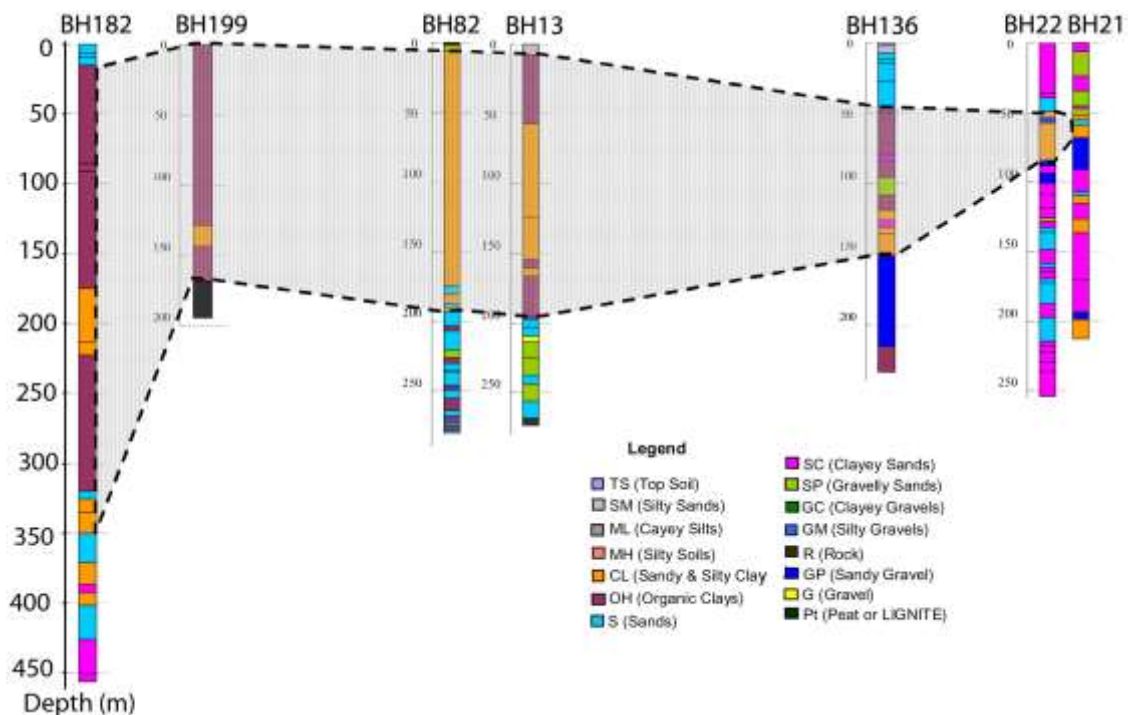


Figure 2.28: Sediment variation from south to north based on borehole.

2.4.4.3 Analysis of the sediments based on the borehole data

Kathmandu Valley lies in the Lesser Himalayan region and is a drained lake. A number of borehole data which have been drilled for the different purposes, especially for water supply, showed that the valley sediments are not uniform and are varied from place to place. Sakai (2001) has mentioned that the sediments of the valley consist of valuable information about the climate change, and uplift of the Himalaya. Analysis of the sediment can provide the valuable information about the stage of sedimentation in the basin and also about the engineering properties of the sediment.

Although many geo-science researchers have studied on the depositional environment and the stratigraphy of the basin fill sediments, the sedimentation process in the basin is

still not known in detail. Many previous studies expressed different views about the sedimentation process in the basin. According to Sakai (2001), there are two faults existing in the southern part of the Kathmandu Basin namely, Chandragiri Fault and Chovar Fault (Figure 2.20), which are active faults and played an active role for the basin development in the Kathmandu Valley. The main source of the sedimentation of the Kathmandu Basin, however is the surrounding mountains from where the sediments were carried by the ancient drainage system. Fujii and Sakai (2002) were estimated the average sedimentation rate of the clayey sediments to be about 104 mm/kyr on the basis of correlation of the palaeoclimatic curve of $\delta^{18}\text{O}$ from Ocean Drilling Program, site 722. Sakai (2001) mentioned that there were several confusions about the lithology and stratigraphy in the previous studies based on the borehole data which were drilled for various purposes especially for water supply project. Although not much is known about the distribution of sediments and bedrock topography, the following discussion has been made based on the study of the available borehole logs and the literatures.

Data acquisition of borehole particularly in Kathmandu Valley, where there is no any particular institution responsible for the borehole drilling and data management is a difficult task. It is time consuming and more efforts need to visit different organizations which are working in the field of planning and designing of infrastructures, and also the laboratory which are responsible for testing of materials. In Kathmandu Valley, many boreholes ranging from few meters to more than 550 m have been made by different organizations and private companies for various purposes. Most of the drillings of boreholes were done for drinking water supply purposes and investigation of soil parameters during design of buildings. But they have lack of many data and parameters such as coordinate, elevation, soil layers types and thickness, which are needed for preparation of geo-information data base. So present study, in total 340 boreholes data were collected from different sources (Figure 2.29). The main sources were various organizations and private companies and consultants, such as the Japan International Cooperation Agency (JICA, 1990) for ground water management Project in the Kathmandu Valley, the report of Geological Survey of India (GSI) for ground water resources of the Kathmandu Valley (1966) and the reports of the Environmental Geological Project (EGP) of the Department of Mines and Geology Project (DMG) for hydrological conditions and barrier potential sediments in the Kathmandu Valley (1998). Out of 340 boreholes, 175 boreholes are relatively shallow wells with drilling depth less than 100 m and the rest are greater than 100 m with maximum depth reaching up to 577 m

located in the central part of the valley. Thirty-four of them have reached up to bedrock level. The data of borehole contains information such as soil types, thickness of soil layers and Universal Transverse Mercator coordinates (UTM) etc. The Figure 2.29 shows borehole location in the study area. They are located unevenly throughout the Kathmandu Valley and most of them are concentrated in and around the center of the valley, where urbanization is very high. This is because of the fact that the most of the activities regarding geotechnical aspects are in the center part of the city.

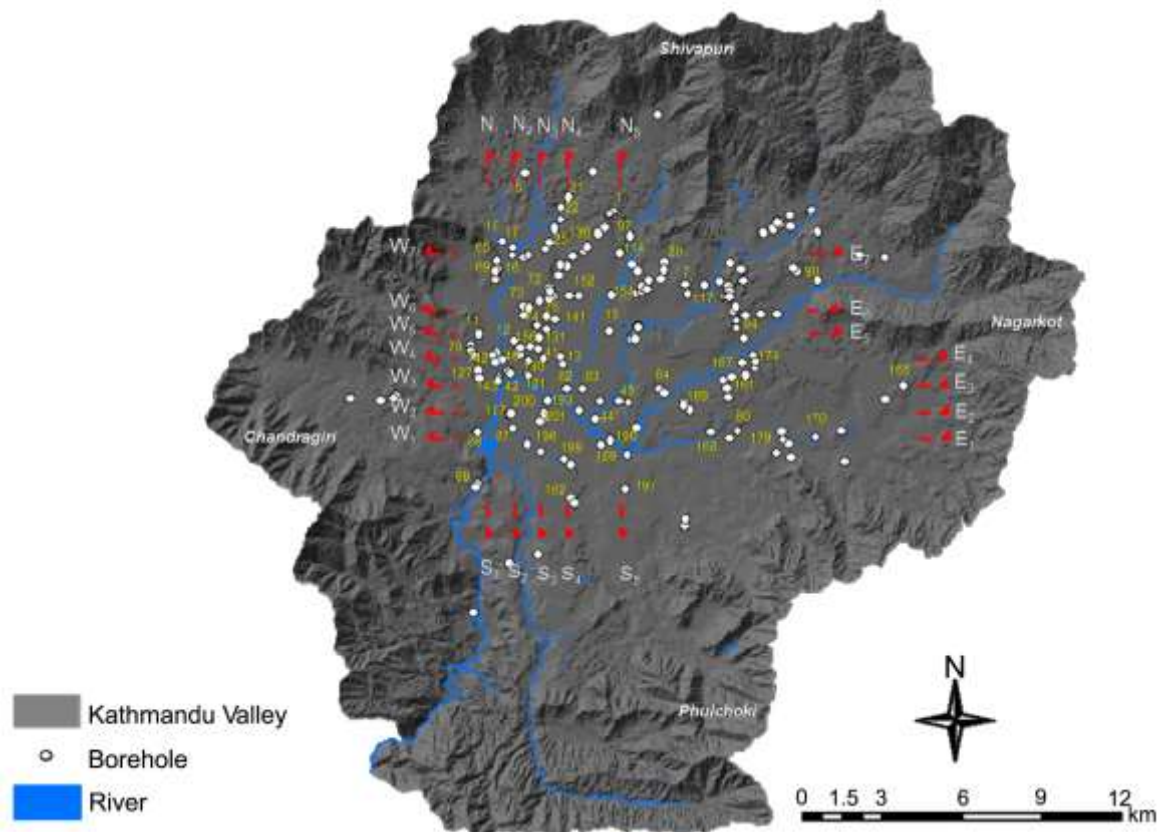


Figure 2.29: DEM of the Kathmandu Basin in which the location of the borehole, and propose sections along south-north and west-east are shown.

The collected borehole data were reviewed minutely and entered in the Excel sheets and tabulated as per the requirement for the analysis in the Geographic Information System (GIS). The table consists of attributes such as borehole identification number (BH_ID), coordinate (UTM), location of borehole, soil type and thickness of each soil layer.

However, the detail information about the subsurface geology of the Kathmandu Basin may not provide from the previous borehole data (Sakai, 2001), they are still useful for preliminary evaluation of the sediment type and its parameters for various purposes. As

mentioned in the Section 2.4.4.2.1, in this study the soil types in each borehole were classified according to the Unified Soil Classification System (USCS), and then the profile of each borehole were drawn. Finally approximate geological profiles of the Kathmandu Basin along north-south and west-east direction were drawn to see the variation of sediment types along these profiles. For generation of profiles, it is assumed that the boreholes which are nearest to the proposed sections represent the geological profile of that section. Figure 2.29 shows the borehole location in the Kathmandu Basin and location of section taken along south - north and west - east.

1) Profiles along south to north

In order to analyze the variation of the sediment distribution in the Kathmandu Basin, south-north profiles (S_1-N_1 , S_2-N_2 , S_3-N_3 , S_4-N_4 , and S_5-N_5), based on the available borehole data, were plotted as shown in Figure 2.29. The central shadow portion which is enclosed by the dotted line represents the soft soil (organic or in organic clay) layer (Figure 2.30 to Figure 2.34). In some areas, sandy layers are also interbedded within the clayey layer. We assume that these portions could represent a lake deposit section in the Kathmandu Basin.

In pre-lake formation, drainage system originating in the northern slope of the Shivapuri hill and termed as “Proto Bagmati River” by Hagen (1969) were very active. These river system were responsible for the deposition of the coarse-grained (gravel, sands) sediments below the lake deposits in the entire valley. Sakai (2001) named these sediments deposit as Bagmati Formation in central and northern part, and Tarebhir Formation in the southern part of the basin. This formation covers the basement rock of the Kathmandu Complex as defined by Stöckline and Bhattarai (1981). This formation also consists of sand and gravels along with detritus from weakly metamorphosed sedimentary rocks from the underlying Kathmandu Complex (Sakai et al., 2001).

From the observation of south-north profiles, it can be said that thickness of the sediment varies from area to area. It has been seen that the thickness of lake deposits at central part i.e. at Borehole 42 (Teku) (refer Figure 2.30, Profile S_1-N_1), Borehole 12 (Lagantol) (refer Figure 2.31, Profile S_2-N_2), Borehole 41 (Bhirkutimandap) (refer Figure 2.32, Profile S_3-N_3), Borehole 182 (Harisiddhi) (refer Figure 2.33, Profile S_4-N_4), is found higher than the other areas. Profile S_1-N_1 shows the increase in sediment depth in the central part and minimum in south at Borehole 89 (Chovar) and in north at Borehole 65 (Balaju). The abrupt change in basement depth can be seen in Profile S_2-N_2 (Figure 2.31),

in which the change in thickness is clearly seen from Borehole 16 to Borehole 17 within a very short distance. In profile S_3-N_3 , the abrupt change in lake deposits is found at Borehole 129 (DPTC Building). Profile S_4-N_4 gives some interesting information about the Kathmandu Basin in which the depth of lake deposit at Borehole 182 (Harisiddhi) is found about 350 m. However the number of boreholes in the southern portion of the valley is very low and much of the information of sediment deposit is still unknown, the depth of lake deposits indicates that the lake was deeper in the periphery of Borehole 182 (i.e. at Harisiddhi area). From section S_4-N_4 (Figure 2.33) and S_5-N_5 (Figure 2.34), it has been found that the thickness of lake deposits at Bansbari and Dhapashi area have a minimum value which indicates that the lake deposit nearly vanishes in those areas.

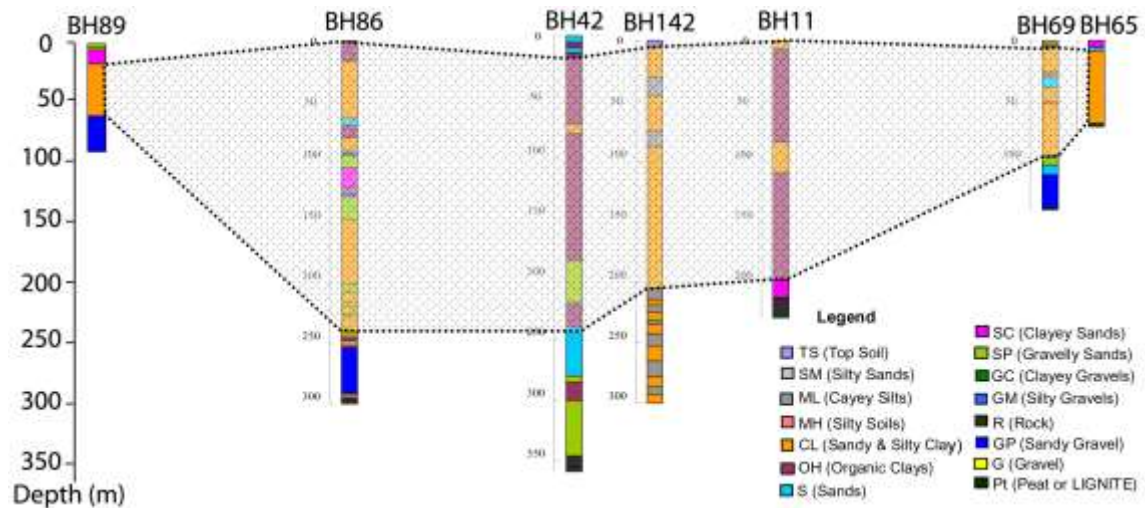


Figure 2.30: Profile along S_1-N_1 (refer Figure 2.29).

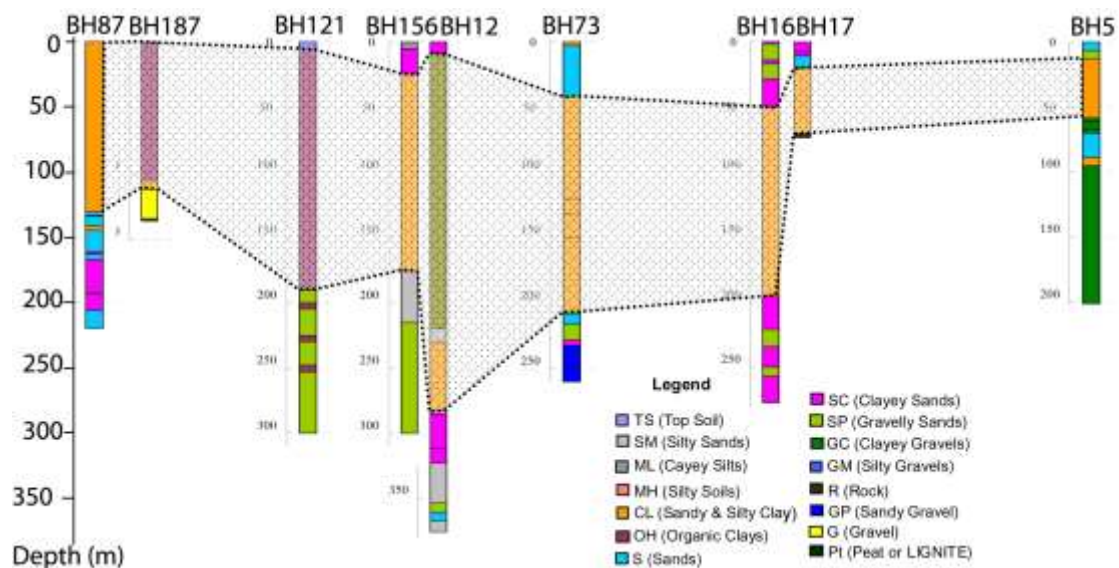


Figure 2.31: Profile along S_2-N_2 (refer Figure 2.29).

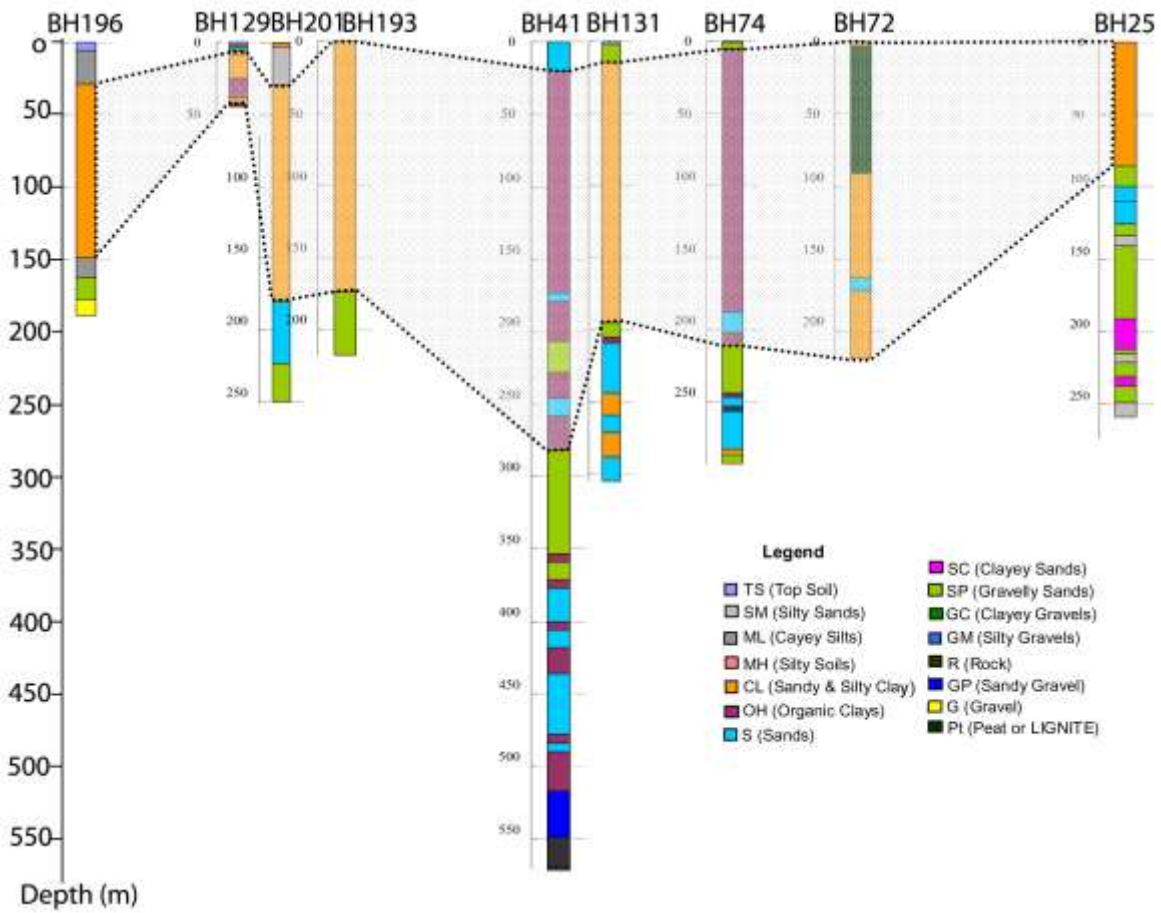


Figure 2.32: Profile along S₃-N₃ (refer Figure 2.29).

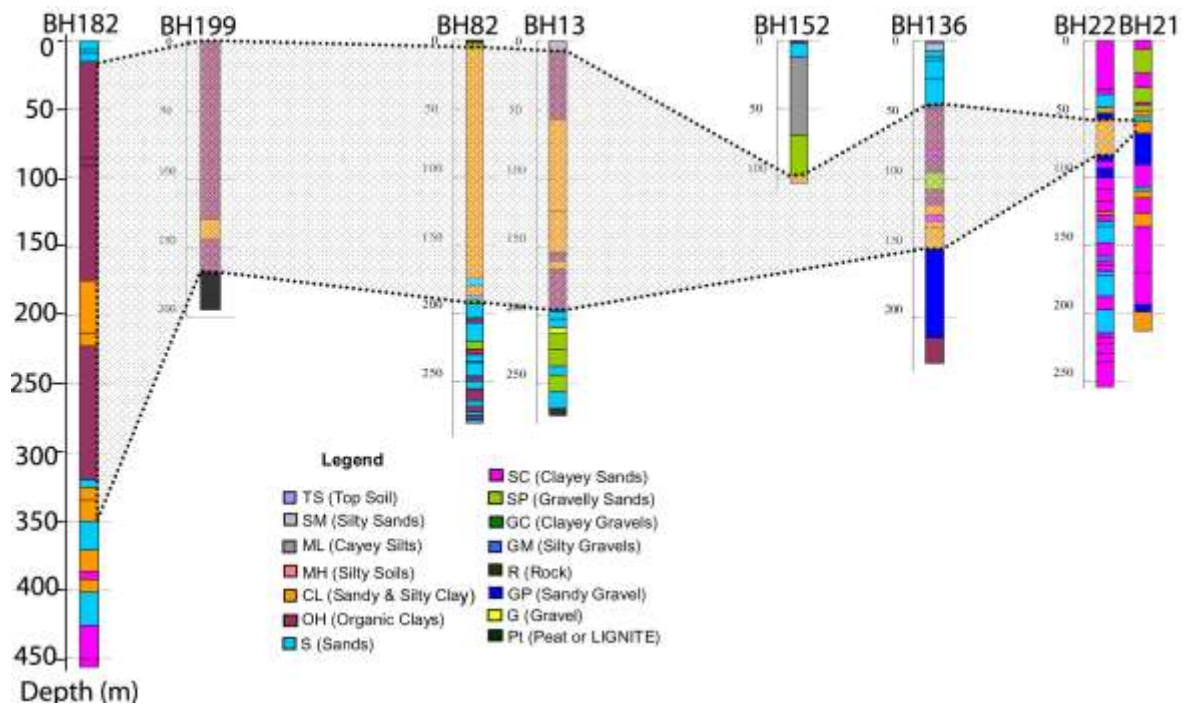


Figure 2.33: Profile along S₄-N₄ (refer Figure 2.29).

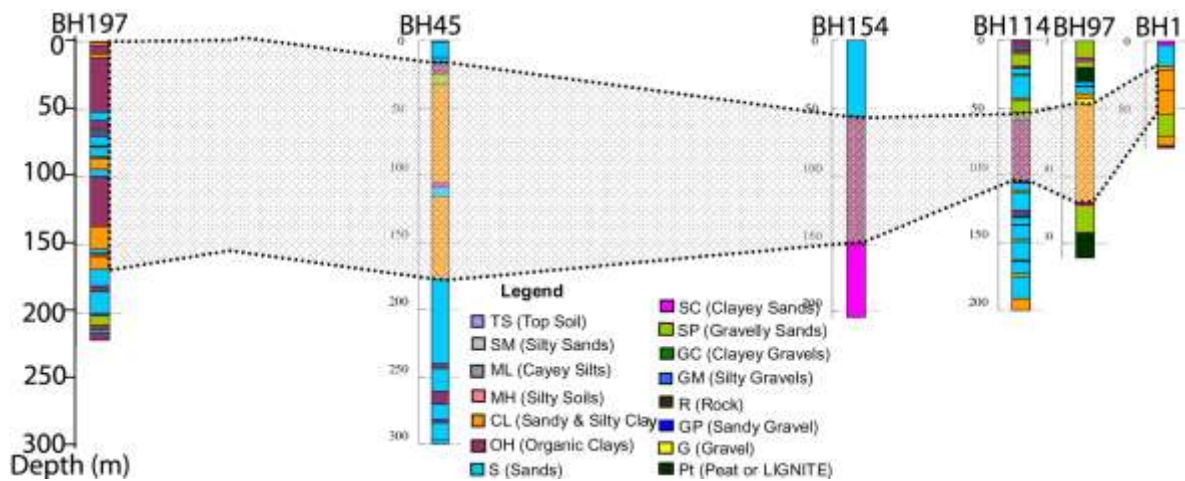


Figure 2.34: Profile along S₅-N₅ (refer Figure 2.29).

2) Profiles along west to east

Similarly, profiles along west-east (i.e., W₁-E₁, W₂-E₂, W₃-E₃, W₄-E₄, W₅-E₅, and W₆-E₆) (Figure 2.29) illustrate the variation of lake deposits along this direction (i.e. Figure 2.35 to Figure 2.41). Abrupt change of lake deposit is found at Borehole 187 and Borehole 200 (refer Figure 2.36, Profile W₂-E₂). From section W₄-E₄ (Figure 2.38) and W₅-E₅ (Figure 2.39), it is found that the lake deposit vanishes at Borehole 174 in Tingi area and at Borehole 94 in Mulpani area of the Kathmandu Valley. Similarly at Borehole 28 (Figure 2.41, Profile W₇ - E₇) in Phaika area, the lake deposit is not found.

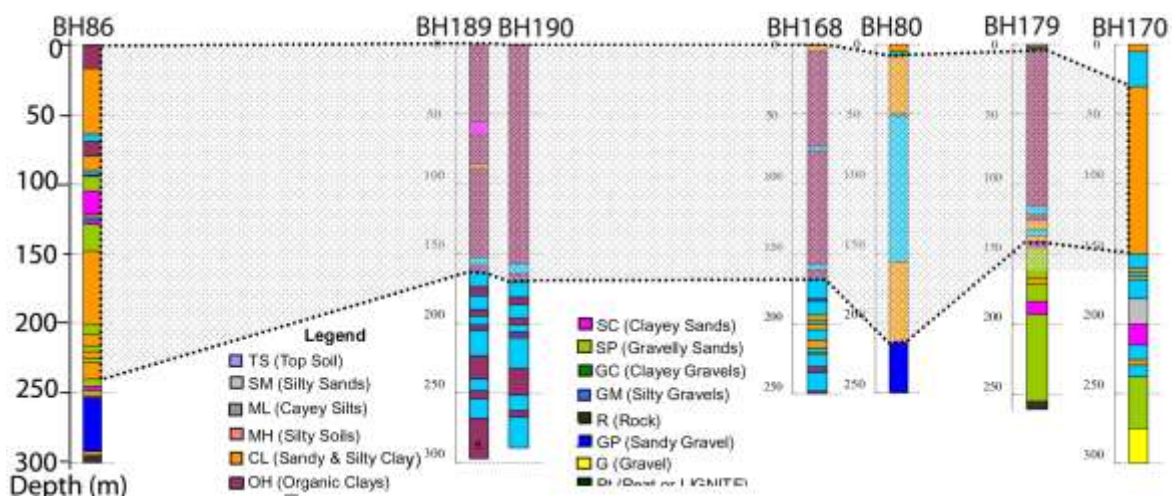


Figure 2.35: Profile along W₁-E₁ (refer Figure 2.29).

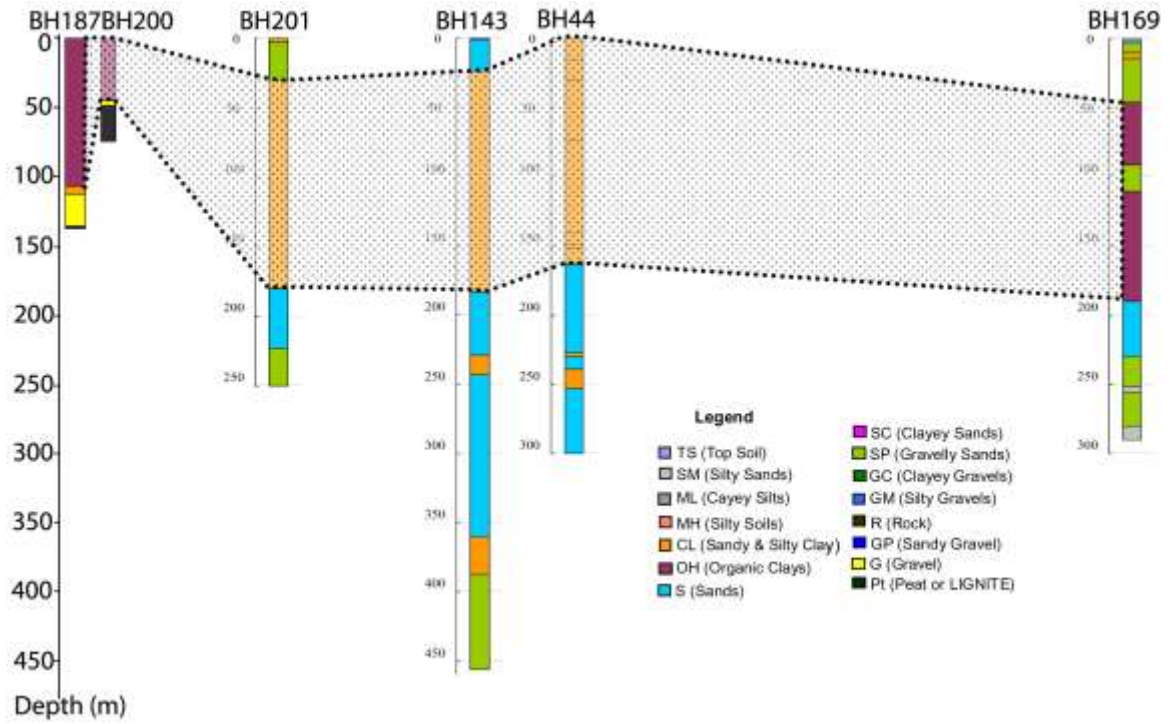


Figure 2.36: Profile along W₂-E₂ (refer Figure 2.29).

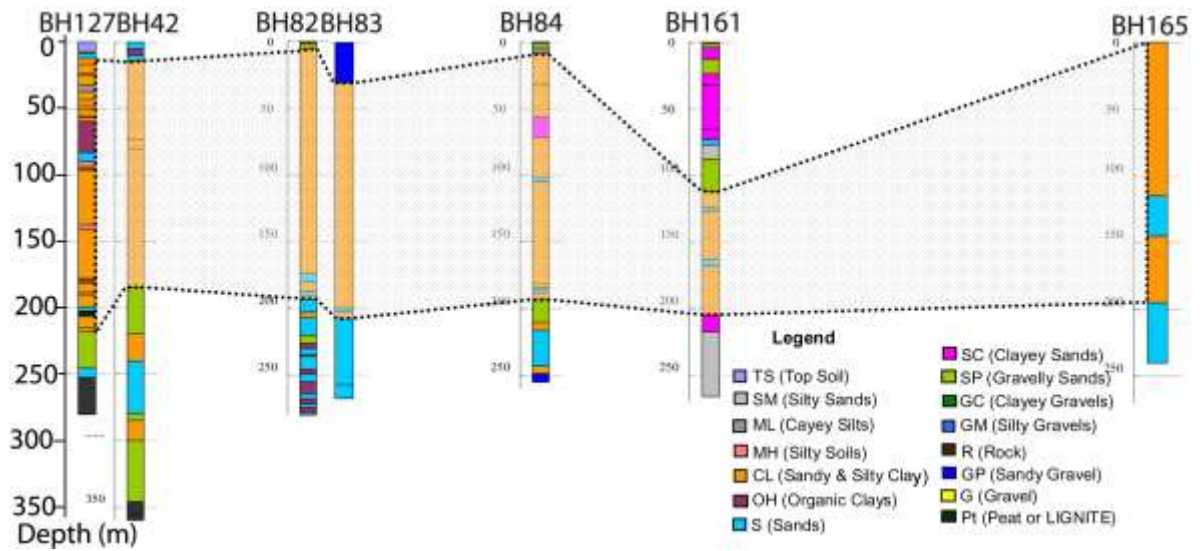


Figure 2.37: Profile along W₃-E₃ (refer Figure 2.29).

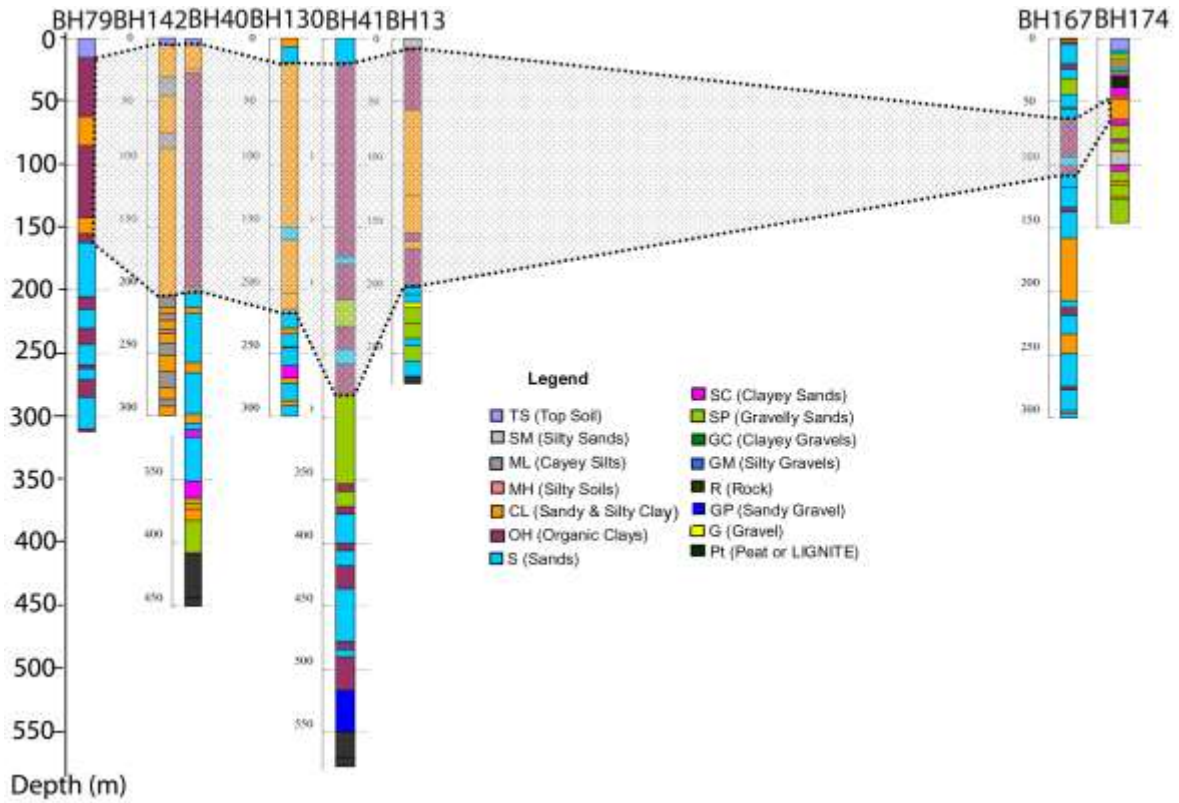


Figure 2.38: Profile along W₄-E₄ (refer Figure 2.29).

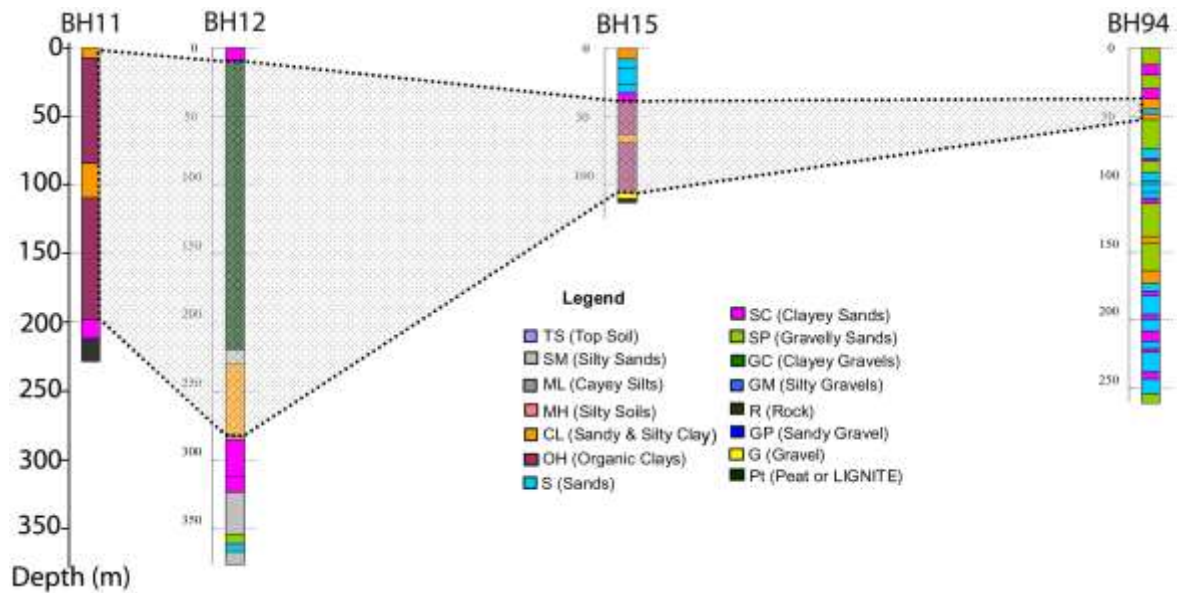


Figure 2.39: Profile along W₅-E₅ (refer Figure 2.29).

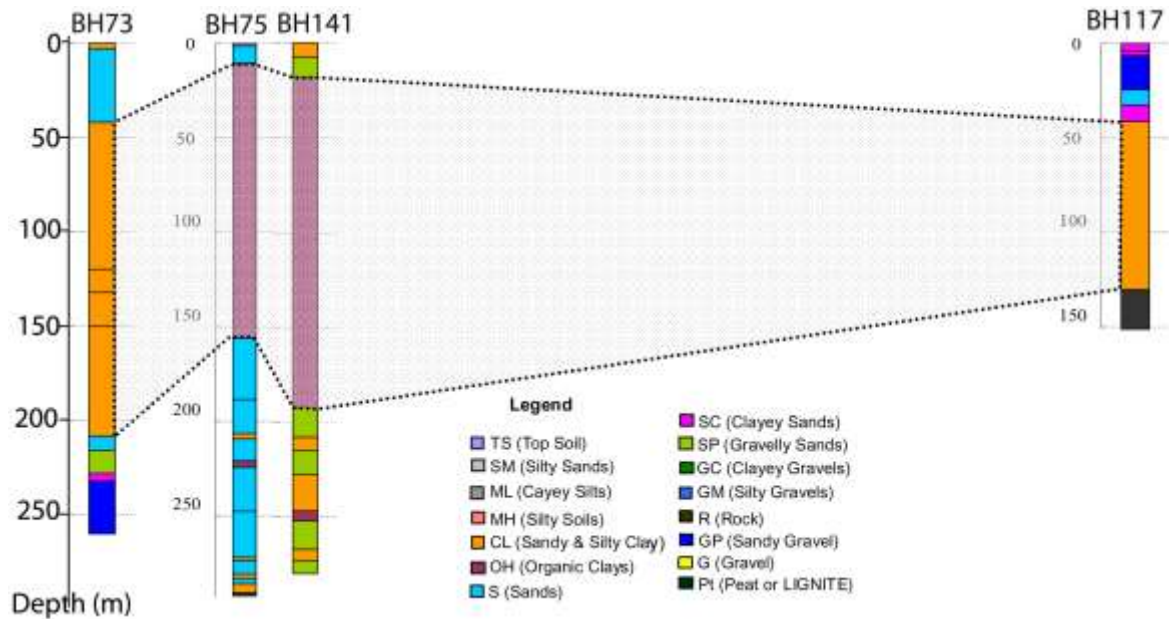


Figure 2.40: Profile along W₆-E₆ (refer Figure 2.29).

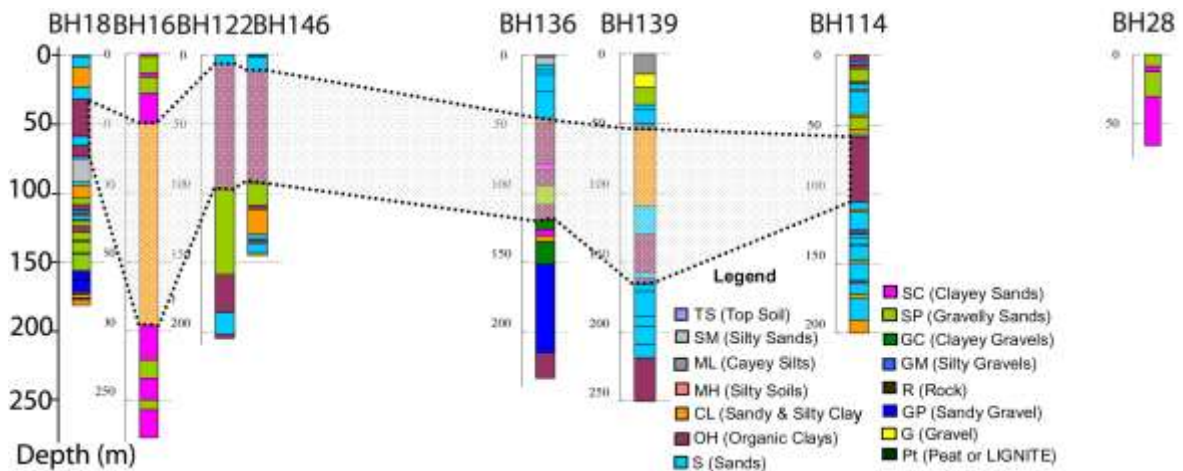


Figure 2.41: Profile along W₇-E₇ (refer Figure 2.29).

From the analysis of the above profiles and boreholes details, it has been found that the bedrock level is ranging in depth from 48 m from the surface in Borehole 200 (Nursing Campus Sanepa) (Figure 2.36) to 545 m in Borehole 41 (Bhirkutimandap) (Figure 2.32, and Figure 2.38). However, most of the boreholes in the Kathmandu Basin are still in soft deposits. These profiles show that the basement topography of the Kathmandu Basin is not uniform and depth of the sediment varies from place to place. The bottom layers (gravel and sandy layer) below the assumed lake deposits in above profiles are varied from place to place, however they are found maximum at the deepest part of the lake deposits. The

elevation of the borehole depth indicates that there are many buried ridges in the bottom of the basin as mentioned by Sharma and Singh (1966).

In summary, it can be said that the depositional environment of the sediments with in the Kathmandu Basin varies from place to place which was strongly controlled by the topographic situation of the valley. The sediments in the northern part of the basin are generally poorly sorted with highly micaceous coarse sands, gravels interbedded with clays layer in some places. These are mainly compacted sediments than other area. In the central and southern half part, the sediments mainly consist of a thick sequence of cohesive soil consisting of dark gray to black highly and medium plastic organic clay. The black plastic clay (locally called Kalimati) is exposed at surface in some places and is rich in organic matter. This clay is assumed Pliocene to Pleistocene in age by Yoshida and Igarashi (1984). Although, there is no much borehole exploration in the southern part of the basin, the available data show that this area consists mainly of alluvial fan facies (i.e., lake deposits of clays and silts, usually sandwiched between thin layers of coarse sediments).

2.4.5 Earthquakes disaster risk

2.4.5.1 Earthquake damage history

Earthquake is considered as one of the unpredictable deadly events, and each year causes tremendous damage throughout the world. Earthquakes have always been a serious threat for the population of Nepal. Historical evidence indicates that Nepal has experienced many destructive earthquakes in the past. Chitrakar and Pandey (1986), for example, mention that at least ten major earthquakes (i.e., greater than 8.0 on the Richter scale) have been recorded in the historical archives since the 13th century. Some of them with the corresponding damage records are given in Table 2.5. Kathmandu valley has experienced recurring destructive earthquakes in the past (Pandey et al., 1995). Major earthquake damage in Kathmandu valley was reported to have taken place in 1255, 1408, 1681, 1803, 1810, 1833, and 1866 (Bilham et al., 1995; Chitrakar and Pandey, 1986; Gupta, 1988; Pandey et al., 1995). The oldest recorded earthquake event, in 1255, was reported to have damaged Kathmandu valley badly, at an estimated intensity of at least X MMI (Rana et al., 2007). One third of the Kathmandu population (i.e., several thousands in number), including King Abhaya Malla, were killed in this earthquake (Rana et al., 2007). The last greatest earthquakes which significantly affected Nepal and Kathmandu Valley were the 1833 Kathmandu Earthquake and 1934 Bihar-Nepal Earthquake (Upreti, 2006). The 1980

Chainpur earthquake ($M_s = 6.6$) and 1988 eastern Nepal (Udyapur) earthquake ($M_s = 6.5$) also killed hundreds of people and damaged many infrastructures in those regions. The more recent Sikkim-Nepal earthquake ($M_s = 6.9$) which devastated, particularly in eastern part of Nepal, was occurred on September 18, 2011. About 6 people were killed and about 21,000 houses were damaged (CUEE, 2011).

The greatest earthquake of 1934, which had a magnitude of $M_w = 8.1$ (Ambraseys and Douglas, 2004; Hough and Bilham, 2008) and produced heavy damage with a maximum intensity of X in Modified Mercally Intensity (MMI) Scale in the Kathmandu Valley (Dunn et al., 1939; Pandey and Molnar, 1988). The total number of death in Nepal was about 8,519 which was more than the death in Bihar (India -7,188 nos). Total of 4,296 people died just within Valley (Rana, 1935; Pandey and Molnar, 1988) (refer Figure 2.43 and Table 2.7). This earthquake reportedly destroyed about 19% and damaged about 38% of the valley's buildings (Pandey and Molnar, 1988; Rana, 1935), and the level of destruction in Bhaktapur City in the eastern part of the valley and neighboring areas was found to be the highest (Pandey and Molnar, 1988; Rana, 1935) (refer Figure 2.42 and Table 2.6). Many historic temples were collapsed/damaged severely. Some of the typical picture of damaged buildings/temples during 1934 earthquake is shown in Figure 2.44.

Table 2.5: Past earthquakes and damage records.

Year	Epice nter	Magn itude	Deaths	Houses Destroyed
1255	-	-	One third of the total population of Kathmandu, including King Abhaya Malla killed	A lot of damages to residential buildings and temples
1260	-	-	Many people died, Famine after the earthquake	A lot of damages to residential buildings and temples
1408	-	-	Many people died	A lot of damages to residential buildings and temples, fissures developed in the ground
1681	-	-	Many people died	A lot of damages to residential buildings

1767	-	-	No records of death	No records of damage
1810	-	-	Some people died and many lives were lost particularly in Bhaktapur	A lot of damages to residential buildings and temples
1823	-	-	No records of death	Some damage to houses
1833	-	7.7 (M_L)	414 people died in the vicinity of the Kathmandu valley	Nearly 4,040 houses destroyed in Kathmandu, Bhaktapur, and Patan in the valley and adjoining Banepa and a total of 18,000 buildings damaged in the whole country.
1834	-	-	No good record available	Many buildings collapsed
1837	-	-	No good record available	No damage in Nepal recorded but greatly affected Patna and other parts of Bihar, India.
1869	-	-	No good record available	No good record available
1897	-	-	No good record available	No good record available
1917 (1918?)	-	-	No good record available	No good record available
1934	East Nepal	8.1 (M_w)	8,519 people died out of which 4,296 died in Kathmandu Valley alone	Over 200,000 buildings and temples damaged out of which nearly 81,000 completely destroyed in the country
1936	Annapurna	7.0 (M_L)	No good record available	No good record available
1954	Kaski	6.4 (M_L)	No good record available	No good record available
1965	Taplejung	6.1 (M_L)	No good record available	No good record available
1966	Bajhang	6.0 (M_L)	24	6,544 houses damaged (1,300 collapsed)
1980	Chainpur	6.5 (M_L)	103	25,086 buildings damaged (12,817 completely destroyed)

1988	Udayapur	6.5 (M_L)	721	66,382 buildings damaged
2011	India/ Nepal	6.9 (M_L)	6 died and 30 injury	14, 544 damaged (6, 435 completely destroyed)

M_L - Richter Magnitude, M_w - Moment Magnitude

Source: *Earthquake Hazards in Nepal - Department of Mines and Geology; UNDP (1994), JICA (2002), Upreti (2006)*

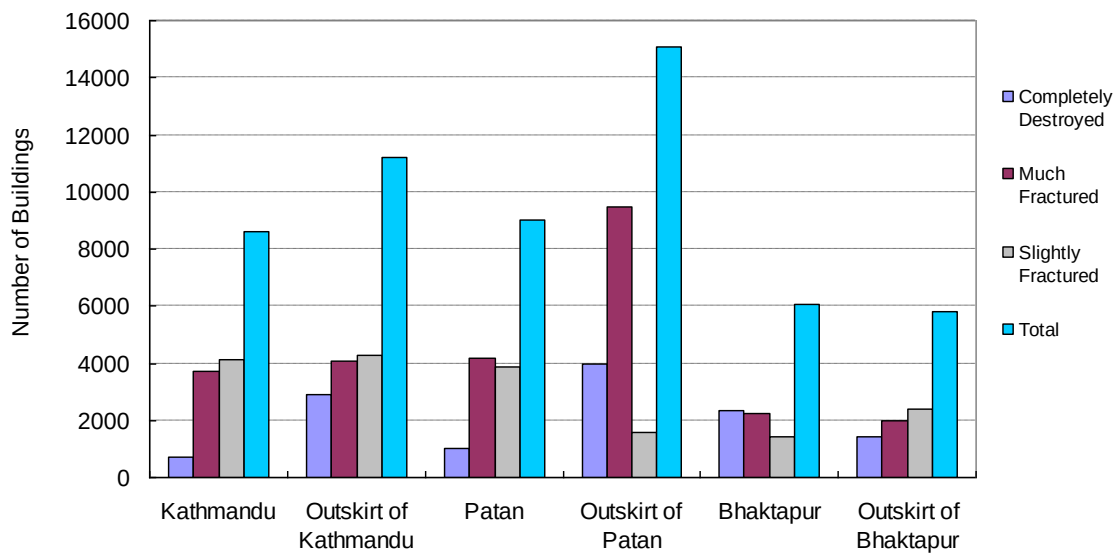


Figure 2.42: Houses destroyed within the Kathmandu Valley during 1934 Bihar-Nepal Earthquake.

Table 2.6: Houses destroyed within the valley due to 1934 Bihar-Nepal Earthquake.

Region	Completely Destroyed	Much Fractured	Slightly Fractured	Total
Kathmandu	725	3,735	4,146	8, 606
Outskirt of Kathmandu	2,892	4,062	4,267	11, 221
Patan	1,000	4,170	3,860	9, 030
Outskirt of Patan	3,977	9,492	1,598	15,067
Bhaktapur	2,359	2,263	1,425	6,047
Outskirt of Bhaktapur	1,444	1,986	2,388	5,5818
Total	12, 397	25, 708	17, 684	55, 789

Source: *Earthquake Hazards in Nepal – Department of Mines and Geology; UNDP(1994)*

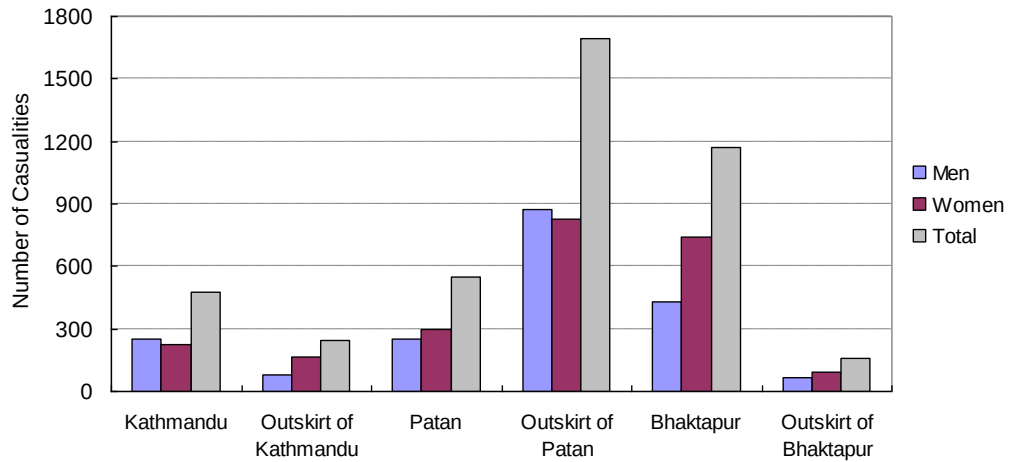


Figure 2.43: Number of casualties within the Kathmandu Valley during 1934 Bihar-Nepal Earthquake.

Table 2.7: Casualties with in the Kathmandu Valley.

Region	Men	Women	Total
Kathmandu	254	225	479
Outskirt of Kathmandu	79	166	245
Patan	250	297	547
Outskirt of Patan	871	826	1,697
Bhaktapur	433	739	1,172
Outskirt of Bhaktapur	65	91	156
Total	1,952	2,344	4,296

Source: *Earthquake Hazards in Nepal – Department of Mines and Geology; UNDP(1994)*



Figure 2.44: Typical example of a) buildings; and b) temples damage in Kathmandu Valley during 1934 earthquake.

2.4.5.2 Earthquake Scenario

In 2002, Japan International Cooperation Agency (JICA) has done the study for expected intensities for different probable earthquakes scenarios under the project name “The Study on Earthquake Disaster Mitigation in the Kathmandu Valley”. The damage by an earthquake will be different depending on the type and location of the earthquake. In the JICA study, based on the historical earthquake catalogue, recent seismicity, and tectonic aspects in and around Nepal and the Kathmandu Valley, the following four “Scenario Earthquakes” were chosen and expected earthquake intensity maps in Modified Mercalli Intensity have been prepared for the whole Kathmandu Valley. These model earthquakes have been defined in the following way (JICA, 2002)

1) Mid Nepal Earthquake

This earthquake with a magnitude of 8.0 has been included, considering the seismic gap in the middle of Nepal. Several researchers, such as Bilham et al. (1997), Pandey et al. (1995) have been pointed out that a great earthquake may occur in the near future in this gap. This is regarded as a great earthquake in this region, comparable to the 1934 earthquake. Except for mountainous areas, the valley would experience over 200 gal. Some areas would experience more than 300 gal (JICA, 2002).

2) North Bagmati Earthquake

This scenario of earthquake is based on the fact that relatively smaller earthquakes occur frequently just north of Kathmandu Valley. This is regarded as middle scale earthquake. The whole valley would experience less than 200 gal which is the smallest peak ground acceleration out of these four scenarios (JICA, 2002).

3) Kathmandu Valley Earthquake

This earthquake model has been included based on a local earthquake caused by an active fault in the southwest of the Kathmandu Valley. In the mountainous area of Kathmandu Valley experience less than 100 gal. The area along the fault would experience over 300 gal (JICA, 2002).

4) 1934 Bihar-Nepal Earthquake

This earthquake model has been set for effective comparison with the above three earthquakes, because most of the people in the Kathmandu Valley have an impression

and/or memory of this great earthquake. In this earthquake scenario, the valley would experience the largest peak ground acceleration in which most part of the valley would experience over 200 gal. Similarly, eastern part of the valley such as Bhaktapur would experience more than 300 gal. And some areas would suffer more than 400 gal (JICA, 2002).

2.4.5.3 Earthquake damage prediction

The historical record of earthquake generation points indicates that the strain energy in the central part of the Nepal Himalaya has not been released for a long time. This state of stress build-up in the area, which is often known as the seismic gap (Khattari, 1987), indicates it to be the most potential location for the next great earthquake in the Himalayas (Bilham et al., 1995; Pandey et al., 1995). Although the available historical record is inadequate to predict accurately the recurrence period of great earthquakes in Nepal, the available data indicates that an earthquake greater than M8.0 occurs at a 100-year interval. As it is already about 80 years since the previous major earthquake that hit Nepal and the Indian state of Bihar in 1934, the threat of a major earthquake in the region, particularly in the Nepal Himalaya, in the near future is very high.

Because of haphazard urbanization and poorly engineered buildings, if an earthquake of the 1934 class recurs, the damage scenario in the Kathmandu Valley will be unimaginable and the fatalities will be unexpectedly high. For such an earthquake scenario, the Japan International Cooperation Agency (JICA, 2002) has estimated some 59,000 houses to be destroyed, 20,000 deaths and 59,000 serious injuries. An another study carried out in the framework of the Kathmandu Valley Earthquake Risk Management Project (Dixit et al., 2000) estimates a total of 40, 000 deaths, 95,000 injuries and more than 600,000 homeless for the same scenario of earthquake in the valley.

2.5 CHAPTER SUMMARY

This chapter briefly describes the characteristics of the study area. The geographical as well as seismicity and geological condition of the Kathmandu Valley has been briefly presented.

Kathmandu, a capital city of Nepal, lies in southern slop of Himalaya and contains valuable information about the climate change, and seismo-tectonic information of the Himalaya. Due to lack of proper data management system and low awareness about the earthquake and its impact, there are a few historical records of earthquakes in Nepal.

Historical evidences showed that the 1255 AD earthquake was the oldest recorded earthquake event in Nepal which had few information regarding the infrastructure damage and casualties. The last greatest earthquakes which significantly affected Nepal and Kathmandu Valley was the 1934 Bihar-Nepal earthquake and produced heavy damage in Kathmandu Valley. Historical records as well as many geoscience researchers indicate that the central part of the Nepal Himalaya is the most potential location for the next great earthquakes in the Himalaya. According to damage prediction made by JICA in 2002, if an earthquake of the 1934 class recurs some 59,000 houses to be destroyed, 20,000 deaths and 59,000 serious injuries within the valley alone.

The evaluation of geological condition of sediment deposit through borehole data analysis shows the variation of sediment thickness and properties within the short distances of the Kathmandu Valley. Due to lack of enough geotechnical information in the boreholes which were mainly explored during water supply purposes, much of the subsurface structure of the Kathmandu Basin is still unknown. However, the compilation and analysis of borehole data in this chapter gives some information about the sediment distribution in the basin. From the analysis, it can be said that the depositional environment of the sediments within the valley varies from place to place which is strongly controlled by the topographic situation of the valley. The sediments in the northern part of the basin are poorly sorted with highly micaceous coarse sands, and gravels, while in the central and southern half part mainly consist of a thick sequence of cohesive soil consisting of dark gray to black highly and medium plastic organic clay, and southern part mainly consists of alluvial fan facies.

With this information, it can be said that the Kathmandu Valley is in extreme risk of experiencing an earthquake disaster, due to a combination of its seismicity, geological setting, ground conditions, building conditions, poorly managed urbanization, and little knowledge of earthquake effects among the residents. Therefore, it is an urgent need to evaluate risk of such expected earthquake disaster in the valley, so that an appropriate mitigation measure can be implemented in an advance. Planning well before onset of disaster is the best step in mitigating the earthquake disaster in the valley.

CHAPTER 3

3 GROUND MOTION AND SEISMIC SITE EFFECTS

3.1 EARTHQUAKE GROUND MOTION

3.1.1 Earthquakes and seismic waves

An earthquake is a shaking at the surface of the Earth caused by an underground movement along a fault plane or by volcanic activity. The length of the triggering fault plane determines the duration of the earthquake shaking. This sudden movement of fault releases a great deal of energy which is often estimated from the relationship given by Gutenberg and Richter (1956).

$$\log E = 11.8 + 1.5M_s \dots\dots\dots (3.1)$$

Where E is expressed in ergs and M_s is a surface wave magnitude. According to Kanamori (1983), this method can be applicable to moment magnitude as well. This release energy is then travels through the earth in the form of seismic waves. The size of the earthquakes is quantified in two ways, namely magnitude and intensity.

3.1.1.1 Magnitude

The magnitude (M) of an earthquake is a measure of earthquakes size and is a function of its energy release. Several scales have been defined for measuring the size of the earthquakes but most commonly used are:

- ◆ Local Magnitude (M_L) (Richter Magnitude)
- ◆ Surface-wave Magnitude (M_s)
- ◆ Body-wave Magnitude (M_b) and
- ◆ Moment Magnitude (M_w)

Local magnitude, surface-wave magnitude, and body-wave magnitude scale is based on the measurement of the amplitude of the seismic waves recorded by the seismograph. These magnitude scales are empirical quantities based on various instrumental measurements of ground-shaking characteristics (Kramer, 1996). They have limited range and applicability and do not satisfactorily measure the size of the largest earthquakes.

The moment magnitude (M_w) scale, based on the concept of seismic moment, is uniformly applicable to all sizes of earthquakes but is more difficult to compute than the other types. The moment magnitude is applicable to describe the size of very large earthquakes. It does not depend on ground-shaking levels, and consequently does not saturate (Kanamori, 1977; Hanks and Kanamori, 1979; Kramer, 1996). The moment magnitude of an earthquake is a measure of the amount of energy released which is an amount that can be estimated based on the surface area of the fault displaced, its average length of movement and the rigidity of the ground material, rather than from seismograph reading.

Table 3.1: Earthquake classes based on magnitude measurement.

Great	$M \geq 8$
Major	$7 \leq M < 7.9$
Strong	$6 \leq M < 6.9$
Moderate	$5 \leq M < 5.9$
Light	$4 \leq M < 4.9$
Minor	$3 \leq M < 3.9$
Micro	$M < 3$

An earthquake of 5.0 is a moderate event, 6.0 is a strong event, 7.0 is a major earthquake and a “great earthquake” exceeds 8.0 (refer Table 3.1). For unit increase in magnitude corresponds to a $10^{1.5}$ or 32-fold increase in seismic energy.

3.1.1.2 Intensity

The intensity is a qualitative description of the effects of earthquake at a particular location, as evidenced by observed damage and human reactions at that location (Kramer, 1996). Earthquake intensity is a measure of damage caused by an earthquake at a given place (UNDP, 1994). It describes the effect of an earthquake on the surface of the earth and depends upon the strength of the earthquake, distance of the location from the hypocenter and local subsoil conditions.

There is no instrument or mathematical basis to measure the earthquake intensity. Based on the observed damage and human reactions, intensity is assigned by the experts in the field. Various scales have been developed to measure and assign the earthquake intensity. These scales are divided into different range (degree) according to their application in different regions and location. In all of these scales, the definition of each

degree has been given with an intensity value which ranges from non-felt or no damage to total damage. Following are the main earthquake intensity scales used in different countries.

1) Modified Mercalli (MM) Intensity Scale

MM Intensity Scale was developed by the Italian seismologist Meralli and modified in 1931 to better represent conditions in California (Richter, 1958). The scale is divided into twelve degree and ranges from not felt (I) to total damage (XII). This is the most widely used intensity scale internationally.

2) Rossi-Forel (RF) Intensity Scale

RF Intensity Scale developed by Rossi Forel in 1880. It has ten degree scale ranging from micro-seismic shock (I): shock felt by experienced observer to shock of extreme intensity (X): great disaster.

3) Medvedev-Sponheuer-Karnik (MSK) Intensity Scale

MSK Intensity scale was developed by Medvedev, Sponheuer and Karnik in 1964. Like MM intensity scale, it also has twelve degree ranging from not noticeable (I) to total damage (XII).

4) European Micro-Seismic Scale of 1998

European Micro-Seismic Scale of 1998 (EMS-98) was developed by European Commission. Similar to MMI or MSK, EMS-98 also has twelve scales. The major difference between the EMS-98 and other intensity scale is the detail with which different terms used by this method are defined in particular building type, damage grades, and quantities.

5) Japanese Meteorological Agency (JMA) Intensity Scale

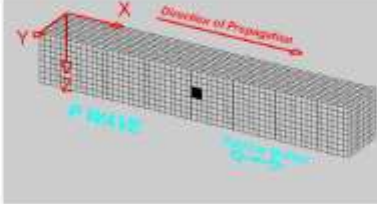
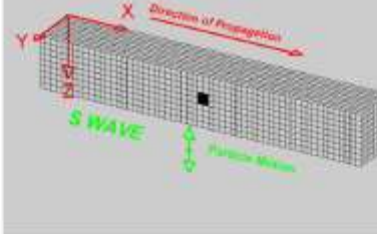
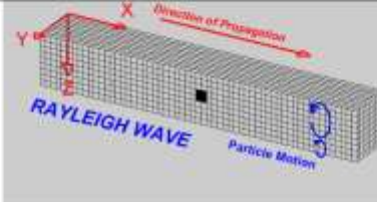
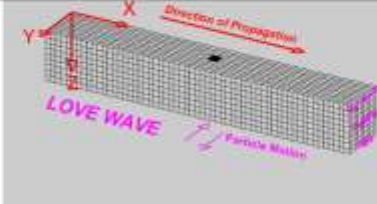
JMA Intensity scale was developed by Japanese Meteorological Agency in 1951. The scale is divided into seven degree.

3.1.1.3 Seismic waves

Seismic waves are generally two types, body waves and surface waves. Body waves, which can travel through the interior of the earth, consist of *P* (Primary, compressional, or

longitudinal) waves and *S* (secondary, shear, or transverse) waves. *P* waves are characterized by material particles moving along the direction of the wave propagation, while *S* waves cause the materials particles to move perpendicular to the wave propagation path, forming shear deformation. The direction of particles movement can be used to divide *S*-waves into two components, *SH* (horizontal plane movement) and *SV* (vertical plane movement). When the seismic wave energy in the form of *P* and *S* waves impinge on the free surface, surface waves are generated. The most important surface waves, for engineering purposes, are Rayleigh waves and Love waves. They travel along the earth's surface with amplitudes that decrease roughly exponentially with depth (Kramer, 1996). The description of seismic waves is summarized in Table 3.2.

Table 3.2: Properties of seismic waves (Kramer, 1996, and Lay and Wallace, 1995).

Type	Designation	Schematic particle motion	(Amplitude/frequency) characteristics	Frequencies range
Body waves	P waves		- normal stress waves - small amplitudes - high frequencies - mainly vertical component (longitudinal partially)	0.02 Hz - 100 Hz (0.01 s - 50 s)
	S waves		- shear stress waves - increase in amplitude - shorter frequencies - mainly horizontal component (longitudinal, transversal)	
Surface waves	Rayleigh waves		- tension-compression waves - greatest amplitude, diminishing with depth - regular waveform - vertical and longitudinal component only	0.003–0.1 Hz (10 – 350 s)
	Love waves		- shear waves, diminishing rapidly with depth - larger amplitudes - still shorter frequency - transversal component only	

As mentioned in the Table 3.2, the properties of these waves are different and propagate differently in the medium. They pose different wave propagation velocities, which strongly dependent on the stiffness or consistency of the propagation medium (subsoil). Figure 3.1 shows the relation between Poission's ratio (μ), velocities of propagation of primary (P), secondary (S), and Rayleigh (R) waves in a semi-infinite medium.

Due to their high propagation velocity, P waves travel most rapidly through the subsoil materials and therefore, they are designated as Primary waves. The S waves have a slower propagation velocity and come after the P waves at any proposed recording point, and are designated as Secondary waves. Because both types of surface waves (Rayleigh waves and Love waves) and their slower propagation velocities come from the body waves near the earth surface, their arrival times are accordingly late.

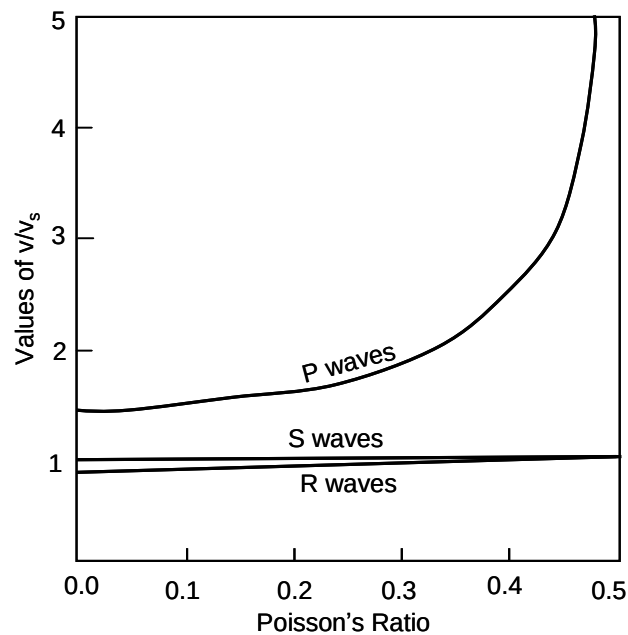


Figure 3.1: Relation between Poission's ratio and velocity of propagation of Primary (P), Secondary (S) and Rayleigh (R) waves in a semi-infinite medium (taken from Lay and Wallace, 1995).

3.1.2 Ground motion parameters

When an earthquake occurs, seismic waves radiate away from the source (epicenter) and travel rapidly through the earth's crust. The shaking produce by these waves in the ground surface is called earthquake ground motion that may last from seconds to minutes.

Ground motion is divided into two general levels; weak motion or small amplitude motion from distance or small earthquakes, and strong motion, or large amplitude motion from nearby or large earthquakes. Seismologists usually defined strong shaking that occurs close to (less than about 50 km from) a causative fault as a strong ground motion. The strength of shaking involved in strong ground motion usually overwhelms a seismometer, forcing the use of accelerograph (or strong ground motion accelerometer) for recording. The level of shaking intensity at a particular site mainly depends on the size of the earthquake itself, the distance to the source, and the local site conditions. Intense ground shaking during earthquakes can damage or even cause failure of engineered structures such as buildings, bridges, highways, and dams, and sustained strong shaking can also trigger ground failures, such as rock falls, landslides, earth flows and liquefaction.

In seismic hazard analysis, estimates of ground motion are defined in the context of strong ground motion, and engineers as well as those concerned with the effects of earthquakes on society are more interested to this kind of motion. Actually, ground shaking can be considered to be the most important of all seismic hazards because all the other seismic hazards are caused by ground shaking.

The instrumental record of earthquake events can be described in terms of acceleration, velocity, or displacement time-histories. The application of site response estimation techniques, and especially the frequency content, and the parameters influencing the spectral characteristics of earthquake ground motion, are of particular interest. These ground motion parameters are essential for describing the important characteristics of a ground motion in compact and quantitative form. Earthquake ground motion is a very complex; single parameter is not enough to describe all important ground motion characteristics (Kramer, 1996). Many parameters have been proposed by different researchers to characterize the amplitude, frequency content, and duration of strong motions. However, usually the ground motion parameters can be defined in three categories

- ◆ Amplitude parameters
- ◆ Frequency content parameters
- ◆ Duration parameters

3.1.2.1 Amplitude parameters

Time history of ground motion (i.e., acceleration, velocity and displacement) is the most common way of describing the strong ground motion amplitude parameters.

Acceleration time history shows a significant proportion of relatively high frequencies, and velocity time history and displacement time history is dominated by relatively low frequency motion. Typically, only one of these quantities is measured directly, while the others computed from it by integration and/or differentiation.

1) Peak horizontal acceleration (PHA)

Peak horizontal acceleration (Figure 3.2) is the largest (absolute) value of horizontal acceleration obtained from an accelerogram. Horizontal accelerations have commonly been used to describe ground motions as it has natural relationship to internal forces. For stiff structures, the largest induced dynamic force is related to peak horizontal acceleration. However, it does not provide any information on the frequency content or duration of the motion.

It is often assumed that the ratio of peak vertical acceleration (PVA) to peak horizontal acceleration (PHA) is $2/3$ for engineering purposes, although the ratio is quite variable. Generally, PVA/PHA is greater than $2/3$ near the source and less than $2/3$ at large distance (Kramer, 1996).

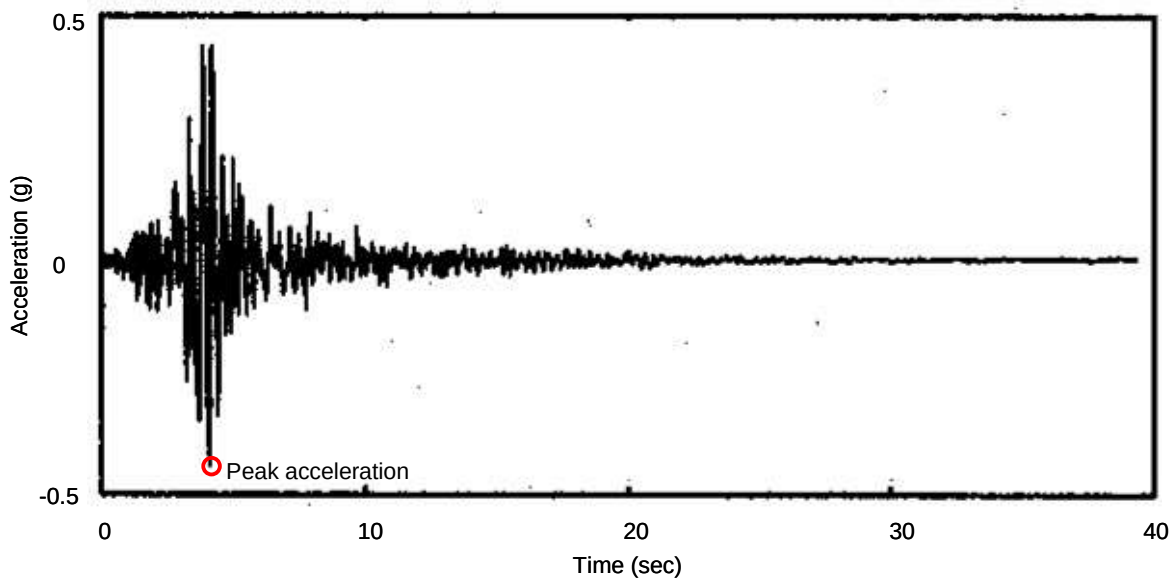


Figure 3.2: Typical acceleration time history graph showing peak acceleration 0.442 g
(After, Kramer, 1996).

The peak horizontal acceleration can also be correlated to earthquake intensity. Trifunace and Brady (1975) have developed the relationship between peak horizontal acceleration and intensity (refer Figure 3.3) due to earthquake as follows.

$$MMI = \frac{1}{0.3}(\log(PGA \times 980) - 0.014) \dots\dots\dots (3.2)$$

Where, MMI- Modified Mercally Intensity, and PGA – Peak Ground Acceleration

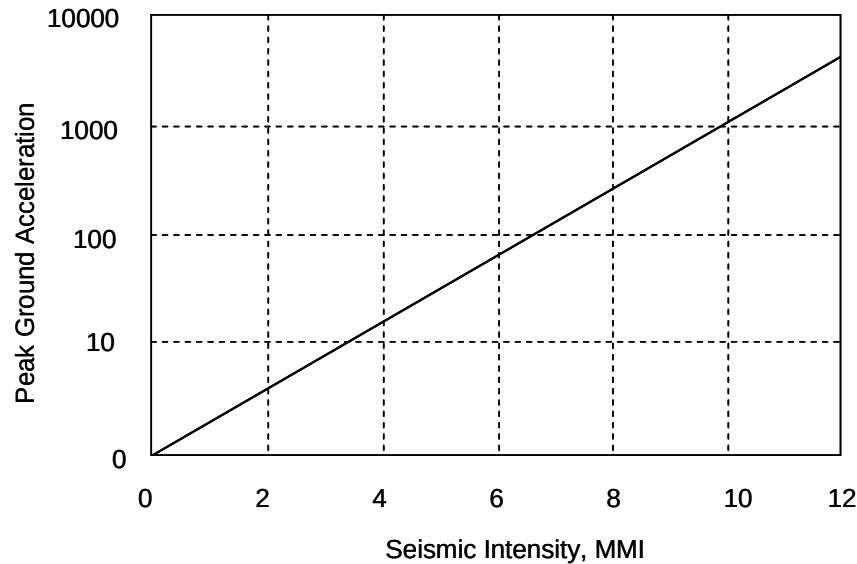


Figure 3.3: Relation between PGA and MMI by Trifunac and Brady (1975) (taken from JICA, 2002).

Ground motions with highest peak accelerations are usually, but not always, more destructive than motions with lower peak accelerations. Very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures. If we know the magnitude of the earthquake and the distance of the fault rupture, the peak horizontal acceleration can be obtained. Based on worldwide data, Campbell (1981) proposed the following attenuation relationships for the estimation of peak horizontal acceleration.

$$\ln PHA(g) = -4.141 + 0.868M - 1.09 \ln[R + 0.606 \exp(0.7M)] \dots\dots\dots (3.3)$$

$$\sigma_{\ln PHA} = 0.37$$

Where M is the local magnitude or surface wave magnitude (≥ 6), R is the closest distance to the fault rupture in km (≤ 50).

Similarly, Campbell and Bozorgnia (1994) used worldwide accelerograms from earthquakes of moment magnitude ranging from 4.7 to 8.1 to develop the following attenuation relationship (Kramer, 1996).

$$\ln PHA(gals) = 3.512 + 0.904M_w - 1.328 \ln \sqrt{R^2 + [0.149 \exp(0.647M_w)]^2} + (1.125 - 0.112 \ln R - 0.0957M_w)F + (0.904 - 0.171 \ln R)S_{SR} + (0.405 - 0.222 \ln R)S_{HR} \quad (3.4)$$

$$\sigma_{\ln PHA} = \begin{cases} 0.889 - 0.0691M & M \leq 7.4 \\ 0.38 & M > 7.4 \end{cases} \quad (3.5)$$

where M_w is the moment magnitude, R is the closest distance to seismic rupture in km (≤ 60 , with minimum values of 7.3, 5.8, 3.5, and 3.0 km for magnitudes of 5.0, 5.5, 6.0, and 6.5, respectively), F is the source term (0 for strike-slip and normal faulting, and 1 for reverse faulting), $SSR = 1$ for soft-rock sites (sedimentary deposits of Tertiary age), $SHR = 1$ for hard-rock sites, and $SSR = SHR = 0$ for alluvium sites.

2) Peak horizontal velocity (PHV)

Peak horizontal velocity (refer Figure 3.4) is another useful parameter for characterization of ground motion amplitude which is less sensitive to higher-frequency components of the ground motion. Peak horizontal velocity may provide a more accurate indication of structural damage than peak horizontal acceleration at intermediate frequencies for structures that are sensitive to loading in the intermediate-frequency range (i.e. tall or flexible buildings, bridge etc). Peak horizontal velocity has also been correlated to earthquake intensity (e.g. Trifunace and Brady, 1975).

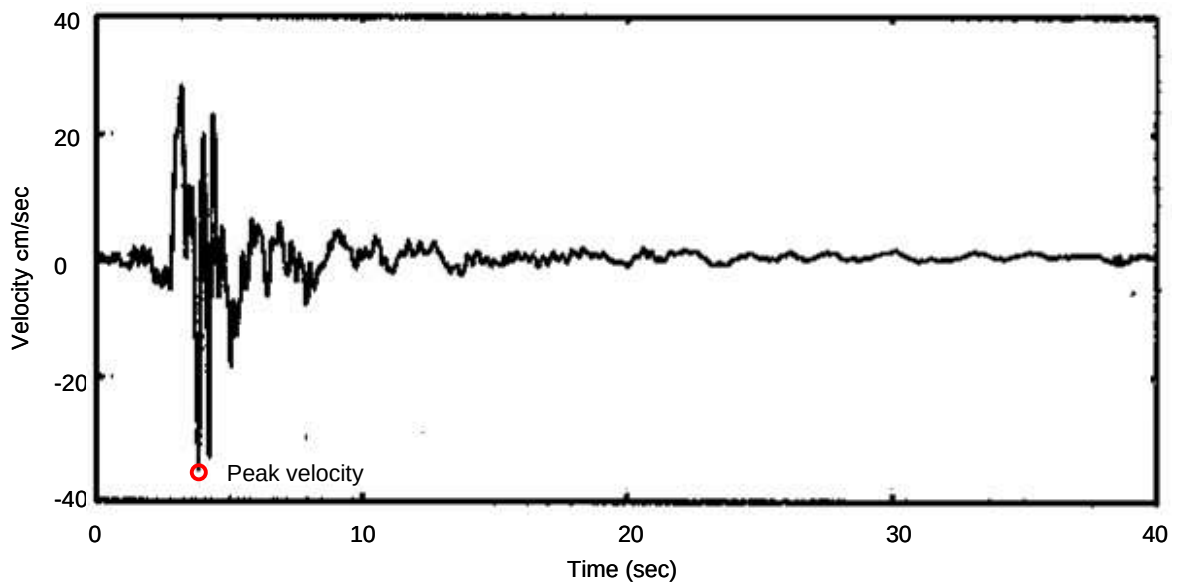


Figure 3.4: Typical velocity time history graph showing peak velocity 33.7 cm/sec (Kramer, 1996).

Joyner and Boore (1988) have developed the attenuation relationship, based on the strong-motion records from earthquakes of moment magnitude between 5.0 and 7.7, to obtain the peak velocity

$$\log PHV(cm/sec) = j_1 + j_2(M - 6) + j_3(M - 6)^2 + j_4 \log R + j_5 R + j_6 \dots (3.6)$$

where $R = \sqrt{r_0^2 + j_7^2}$, and r_0 is the shortest distance in km from the site to the vertical projection of the earthquake fault rupture on the surface of the earth, $j_1 - j_7$ are coefficients.

3) Peak horizontal displacement (PHD)

Peak horizontal displacement (refer Figure 3.5) is the largest absolute value of horizontal displacement obtained from an accelerogram. Peak horizontal displacement is generally associated with lower-frequency components of an earthquake motion. They are, however, often very difficult to describe accurately due to signal processing errors in the filtering and integration of accelerograms and due to long-period noise. Peak horizontal displacement may not common tool for ground motion measurement.

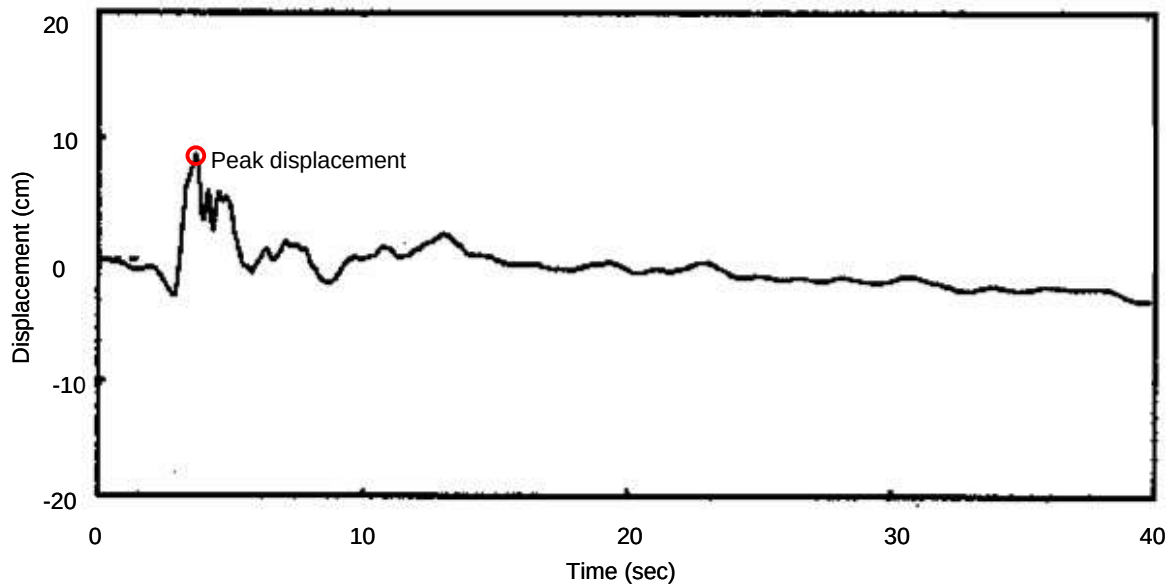


Figure 3.5: Typical displacement time history graph showing peak displacement 8.5 cm (Kramer, 1996).

3.1.2.2 Frequency content parameters

Earthquake produces very complicated loading with various component of motion that span abroad range of frequencies. The dynamic response of structural systems and soil is very sensitive to the frequency content of the ground motions. The frequency content

strongly influences the effects of the motion. Thus the characterization of the ground motion can not be complete without considering its frequency content.

1) Ground motion spectra (Fourier spectra)

Any periodic function (i.e. any functions that repeats itself exactly at a constant interval) can be expressed using Fourier analysis which is a sum of a series of simple harmonic terms of different frequency, amplitude, and phase. Using the Fourier series, a periodic function, $x(t)$ can be written as

$$x(t) = c_0 + \sum_{n=1}^{\infty} c_n \sin(w_n t + \phi_n) \dots\dots\dots (3.7)$$

where, c_n – amplitude

ϕ_n – phase angle

The Fourier series provides a complete description of the ground motion since the motion can be completely recorded by the inverse Fourier transform.

2) Fourier spectrum

For any given time series, $g(t)$, the Fourier spectrum is

$$G(w) = \int_{-\infty}^{\infty} g(t)e^{iwt} dt \dots\dots\dots (3.8)$$

For finite duration, t_d

$$G(w) = \int_0^{t_d} g(t)e^{iwt} dt \dots\dots\dots (3.9)$$

Then the Fourier amplitude can be written as

$$|G(iw)| = \left[\left\{ \int_0^{t_d} g(t) \cos wtdt \right\}^2 + \left\{ \int_0^{t_d} g(t) \sin wtdt \right\}^2 \right]^{1/2} \dots\dots\dots (3.10)$$

and the Fourier phase angle is

$$\phi(iw) = \left[\int_0^{t_d} g(t) \sin wtdt \right] / \left[\int_0^{t_d} g(t) \cos wtdt \right] \dots\dots\dots (3.11)$$

The plot of Fourier amplitude versus frequency is known as a Fourier amplitude spectrum. The Fourier amplitude spectrum of a strong ground motion shows how the amplitude of the motion is distributed with respect to frequency (period). It expresses the frequency content of a motion very clearly. Large earthquakes produce greater low

frequency motions than smaller earthquakes. Fourier phase spectrum is a plot of Fourier phase angle versus frequency. It describes the relative variation between the constituent harmonic signals in the motion time history.

The Fourier amplitude spectrum may be narrow or broad. A narrow spectrum implies that the motion has a dominant frequency (or period), which can produce, a smooth, almost sinusoidal time history. A broad spectrum corresponds to a motion that contains a variety of frequencies that produce a jagged, irregular time history.

The frequency characteristics of earthquake ground motion are directly influenced by the following:

- ◆ Magnitude of the earthquake (M)
- ◆ Distance to the seismic source (epicentral distance) (d)
- ◆ Subsoil conditions of the recording site

3) The impact of earthquake magnitude

The amount of energy released during earthquake shaking is strongly related to its magnitude, and hence the characteristics of the seismic waves will also be related to magnitude of the earthquakes. Ambraseys et al. (2000) have done the statistical analysis of a set of European strong-ground records. Figure 3.6 shows the normalized response spectra (=5%) of different earthquake magnitude range in three different subsoil conditions. All earthquakes were recorded in epicentral distance (R_e) < 20 km. These figures clearly show that the shape of response spectra varies according to the magnitude range of earthquakes for the same soil condition and a clear shift of the spectra to longer periods as well as slight decrease of the highest amplification can be observed when magnitude (M_s) increases.

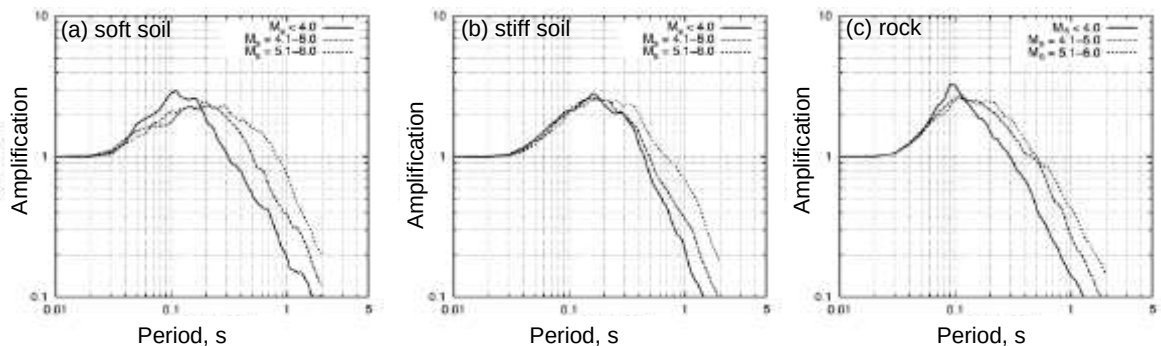


Figure 3.6: Statistical analysis of near-field strong-motion data ($R_e < 20$ km) (Ambraseys et al., 2000); normalized response spectra dependent on magnitude (M_s) for different subsoil classes.

Figure 3.7 shows the idealized shape of smoothed Fourier amplitude spectra versus frequency plot (Kramer, 1996). This shows that a log log scale smooth and Fourier amplitude spectra tends to be largest over an intermediate range of frequencies bounded by the corner frequency “ f_c ” on the low side and the cut off frequency f_{max} on the high side.

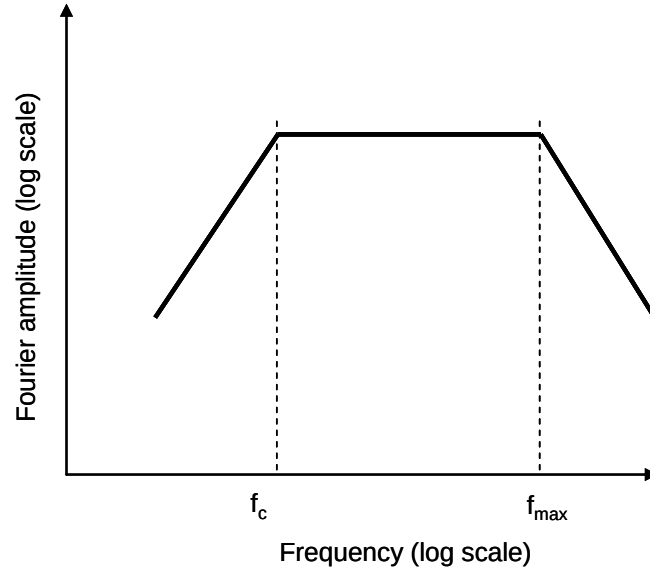


Figure 3.7: Idealized shape of smoothed Fourier amplitude spectra versus frequency plot
(After, Kramer, 1996).

Brune’s (1970, 1971) solution for instantaneous slip of a circular rupture surface is given in the Equation 3.12 in which the Fourier amplitudes for a far-field event at distance R can be expressed (Boore, 1983; Kramer, 1996) as

$$|A(f)| = \left[CM_0 \frac{f^2}{1 - \left(\frac{f}{f_c}\right)^2} \frac{1}{\sqrt{1 + \left(\frac{f}{f_{max}}\right)^8}} \right] \frac{e^{-\pi f R/Q(f) v_s}}{R} \dots\dots\dots(3.12)$$

where, f_c and f_{max} are the corner and cutoff frequencies of the Fourier amplitude spectrum in log scale. $Q(f)$ is the frequency-dependent quality factor which is inversely proportional to the damping ratio of the rock, and C is a constant give by

$$C = \frac{R_{\theta\phi} F V}{4 \pi \rho v_s^3} \dots\dots\dots(3.13)$$

where $R_{\theta\phi}$ ($=0.55$) accounts for the radiation pattern, F ($=2$) accounts for the free-surface effect, V ($=\sqrt{2}/2$) accounts for partitioning the energy into two horizontal

components, ρ is the density of the rock along the rupture surface, and v_s is the shear wave velocity of the rock.

If f_{max} is assumed constant for a given geographic region, the spectra for different earthquakes are functions of the seismic moment, M_0 , and f_c , which can be related by (Brune, 1970)

$$f_c = 4.9 \times 10^6 v_s \left(\frac{\Delta\sigma}{M_0} \right)^{1/3} \dots\dots\dots (3.14)$$

Where v_s is in km/sec, M_0 is in dyne-cm, and $\Delta\sigma$ is referred the stress parameter or stress drop in bars which is depend upon the earthquake sources.

Figure 3.8 shows how the Fourier amplitude spectra obtained using Equation 3.12 vary with magnitude, which indicates that large earthquakes produce greater low-frequency motions that do smaller earthquakes.

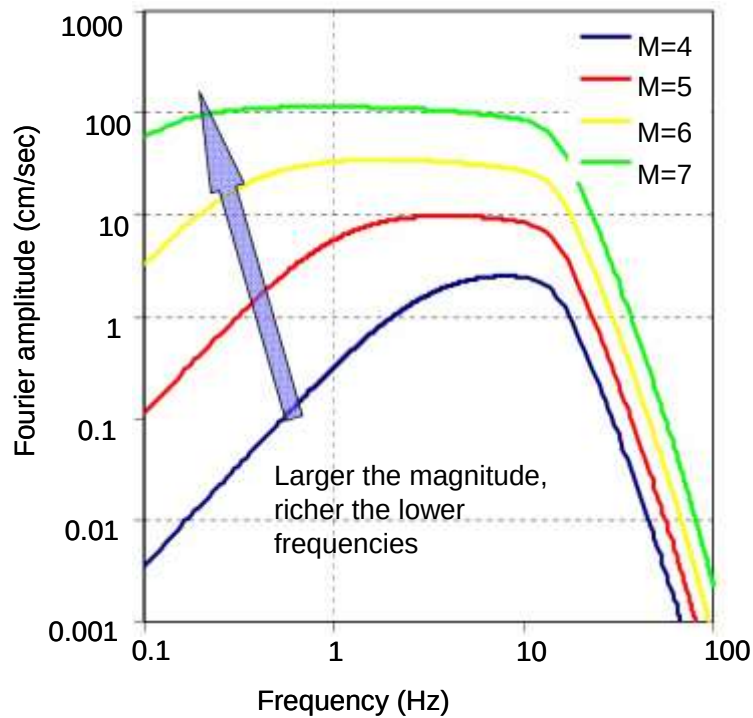


Figure 3.8: Variation of Fourier amplitude spectra at $R = 10$ km for different magnitudes ($\Delta\sigma = 100$ bars and $f_{max} = 15$ Hz) (After Boore, 1983).

Apart from the subsoil conditions and distance to the earthquake source, the frequency characteristics of the earthquake ground motion depend on the magnitude of the earthquake. The spectral amplification factors are also function of earthquake magnitude (Gh.marmureanu et al., 1996). The large magnitude earthquakes have their highest

amplification at longer periods whereas highest amplification of the of smaller magnitude earthquakes is shifted into the short-period range.

The ground acceleration tends to decrease as earthquake magnitude increases. Large earthquakes obviously produce larger and longer-period ground motions on the surface than to smaller earthquakes. This can be explained by nonlinear effects occurring in the near-surface soil materials during stronger shaking. It is also obvious that the peaks of the displacement seismograms in the linear and nonlinear cases are controlled by frequencies that are de-amplified due to the nonlinearity effects.

4) Impact of seismic source distance

Earthquakes generate the seismic waves at the source (epicenter), and they travel through the different geological formation until they reach to the surface of the earth. As a result their higher-frequency components are scattered and absorbed more rapidly than their lower-frequency components (Kramer, 1996). This indicates that the frequency contents of seismic waves also changes with distance. One characteristic of the change in frequency content with distance involves the shifting of the peak of the Fourier amplitude spectrum to lower frequencies (or higher periods) which result the predominant periods increase with increase in distance as shown in Figure 3.9 (Kramer, 1996; Seed and Idriss, 1969). Therefore, the frequency characteristics of seismic ground motion at the same site vary with respect to the seismic source distance. Moreover, regardless of possible amplification effects of local subsoil, amplitude level of seismic motion decreases with the distance from the seismic source increases. These illustrations indicate that the relationship between frequency characteristics and distance to the seismic source is more complex.

Wang and Law (1994) have mentioned that response spectra of the ground motion become increasingly flat as the epicentral distance increases. Similarly, the peak period corresponding to the maximum acceleration may also changes slightly. In order to check the relationship between the predominant period of earthquake motion and the distance to the seismic source, Seed and Idriss (1969) analyzed a number of earthquake records at surface rock stations. Figure 3.9 shows the variation of predominant period at rock outcrops with magnitude and distance.

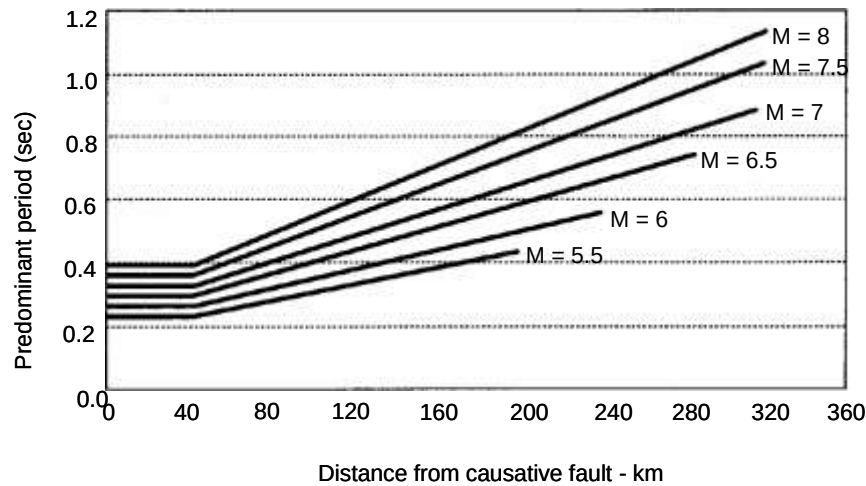


Figure 3.9: Variation of predominant period of ground motion with magnitude and distance (from Seed and Idriss, 1969).

5) Predominant period

Since the end of the 19th century and then after, the seismic instruments were put into practice to use for obtaining the correlations between earthquake ground motion and subsoil condition. Kanai (1983) has done a detailed investigation of earthquake records in Tokyo and find out that, for every recording site, there exist a certain frequency/period where the amplitude reaches a maximum. Figure 3.10 illustrates the relationship between the level of earthquake magnitudes, and the predominant period of earthquake ground motions for earthquakes recording at different stations in Japanese prefectures. Based on the comments made by Kanai (1983), the following points can be summarized:

- ◆ The predominant period of a site decreases slightly for a magnitude of earthquakes smaller than the certain value.
- ◆ The predominant period of a site is seen nearly constant for a magnitude of earthquakes greater than certain value which give important information about the independence of predominant time period with the level of earthquake magnitude.

This observation and findings by Kanai (1983) is important in applying different types of earthquake data for site response analysis. Therefore, predominant period is a very useful strong ground motion parameter for evaluation of subsoil structure of ground. It is a single parameter that provides a useful, although somewhat crude representation of the frequency content of a ground motion. In broad definition, the predominant period is defined as the period of vibration corresponding to the maximum value of the Fourier amplitude spectrum (SESAME, 2004). During analysis of the ground motion, we generally obtained jagged Fourier spectrum which makes inconvenience to obtained accurate value

of predominant period. In such a case, Fourier amplitude spectrum is smoothed using different smoothing function.

Important remarks about predominant period

- ◆ While computing the predominant period, the smoothing should be done carefully.
- ◆ For near-fault records, the pulse period strongly dominates (influences) the predominant period. This affects the nonlinear response of the structures.
- ◆ A clear dominant period indicates that the ground motion response is harmonic.
- ◆ The predominant period should be considered in caution. In some cases, it may cause misleading results about the frequency content of a ground motion.

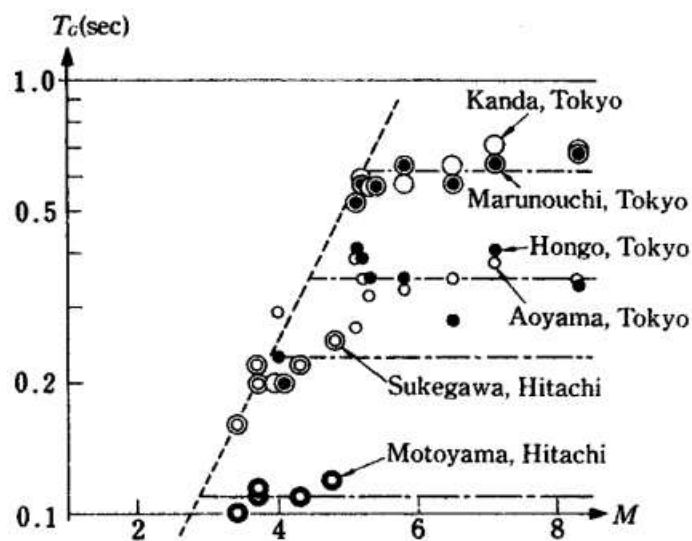


Figure 3.10: Relation between magnitude, and predominant period of earthquake ground motion for different recording sites in Japan (Kanai, 1983)

3.1.2.3 Duration parameters

Duration of strong ground motion plays an important role similar to amplitude and frequency content parameters in seismic hazard assessment. The duration of ground motion can have strong influence on earthquake damage. Ground motion duration is important for the response of foundation materials because the building up of pore water pressure in the underneath subsoil and essentially the liquefaction is strongly dependent on duration of ground motion. Duration of ground motion is particularly important for relatively weak and short period structures because their inelastic deformations are strongly dependent on duration. Moreover, evaluation of duration parameter of ground motion is important for any structures which have stiffness and strength degrading characteristics.

Ambrasseys (1988) has mentioned that if peak ground acceleration of two records is same, the ground motion with larger duration is more likely to induce liquefaction than smaller duration.

3.1.3 Application of Earthquake ground motion data

The seismic ground motion data has been applying in wider field, such as geotechnical earthquake engineering, seismology and so on. It has wider application in building code development. The earthquake ground motion data has been applying by the seismologists to study the interior of the earth. Nowadays, it has wider application in the ground response analysis of the particular areas. The following are the major application of seismic ground motion data

- ◆ It has widely used for seismic hazard assessment of the area
- ◆ It can be used for the development of ground motion distribution (such as peak ground acceleration) map of a particular area.
- ◆ It has wider application in the development of zoning map (risk map) of a particular area
- ◆ It has wider application for the development of attenuation relations for strong ground motion which has a wider application in the earthquake hazard risk assessment for the particular area.

3.2 MICROTREMORS

3.2.1 Terms and descriptions

In any ordinary place of the earth, a very sensitive seismograph shows the ground motion data continuously even in the absence of earthquakes. This indicates that the earth is never truly at rest. This ground motion is termed as microtremor. Japanese seismologist Kiyoshi Kanai primarily investigated the microtremor. Kanai et al. (1954) was probably the first paper to use microtremor in order to study site effects. Prior to his systematic measurement of microtremor little attention had been given to the study about microtremor and its application. According to Kanni and Tanaka (1961), amplitudes of the microtremor are 0.1 to 1 microns and the frequency of motion may be 0.5 to 20 Hz or more. They have mentioned that microtremor are caused by artificial disturbances such as traffic, industrial machines, and pointed out that larger the city greater will be the amplitudes of microtremors. Therefore any seismograph with a magnification exceeding 1000 times will capable of recording microtremors.

According to Lachet and Bard (1994), ambient seismic noise at any place is caused by a set of (surface) sources randomly arranged and with varying amplitude. Classification of noise sources, clearly, is a difficult task and it, generally, depends upon the frequency and their amplitude. Different authors have classified the noise sources in different ways. Gutenberg (1958) established a list of the different types of noise sources according to their frequency. Asten (1978), and Asten and Henstridge (1984) also classified the noise sources according to their frequency as shown in Table 3.3.

Table 3.3: Frequency of different types of noise sources (Gutenberg (1958), Asten (1978), and Asten and Henstridge (1984) (taken from Bonnefoy-Claudet et al. (2006a)).

SN	Description	Gutenberg (1958)	Asten (1978, 1984)
1	Oceanic waves striking along the coasts	0.05-0.1 Hz	0.5-1.2 Hz
2	Monsoon/Large scale meteorological perturbations	0.1-0.25 Hz	0.16-0.5 Hz
3	Cyclones over the oceans	0.3-1 Hz	0.5 - 3 Hz
4	Local scale meteorological conditions	1.4-5 Hz	
5	Volcanic tremor	2-10 Hz	
6	Urban	1-100 Hz	1.4 - 30 Hz

According to Frantti et al. (1962) and Frantti (1963), the changes in noise behaviors seem to be at around 1 Hz. They measured the seismic noise at 48 sites in the United States and other countries having different geological sites (rock or sediment), different geographical locations (close to ocean, mountains or cities), and in different times (varying hours and seasons). For processing purposes, a two minute recording interval was selected, which was free from any known disturbances nearby. A plot of vertical component of noise particles velocities observed on 8 different sites in Arizona is shown in Figure 3.11 (Frantti, 1963), in which a consistent decrease in seismic noise level from 0.6 Hz to 1 Hz can be seen. At higher frequency, the noise levels show relatively stationary trends over frequencies. While the absolute noise level exhibits large changes from site to site, the general trend showed in Figure 3.11 is similar for all sites presented in Frantti et al. (1962) and Frantti (1963).

The results from the different authors (Toksöz and Lacoss, 1968 and Satoh et al., 2001) show that the sources of long-period noise are mainly oceanic waves and those of

short-period noise are due to human activities such as traffic or machine vibrations. Depending on the origin the noise behavior is different (in time and spectral domains). Microtremor amplitudes have daily and weekly variations, while natural noise exhibits amplitude variations linked with natural phenomena, most often oceanic activity for long period noise. Thus spectral boundary between microseisms and microtremors is around 1 Hz.

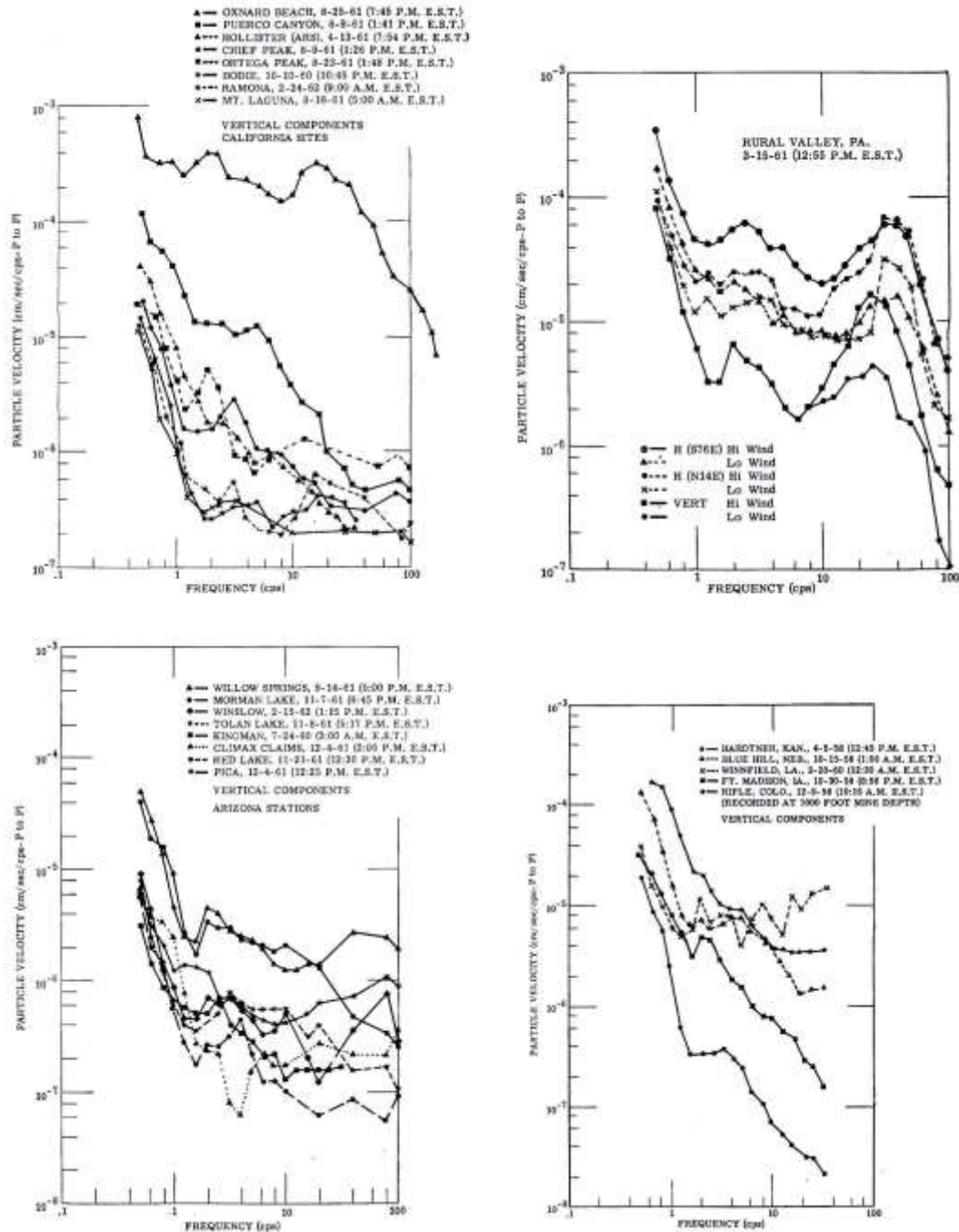


Figure 3.11: The vertical component of noise particle velocities recorded at 8 distinct sites located in Arizona after Frantti (1963), show the typical seismic noise level between 0.6 and 100 Hz.

Based on the results obtained of noise measurement in different area having different geological conditions, Bonnefoy-Claudet et al. (2006a) summarized that there are two distinct origins of noise: natural and cultural, and presented the following classification for the origin of noise: 1) seismic noise below 0.5 Hz is generated by oceanic and large-scale meteorological conditions; 2) noise at frequencies around 1 Hz is due to wind effects and local meteorological conditions; 3) above 1 Hz, seismic noise is due to human activities. The 1 Hz boundary, however, is not a universal limit. According to Seo (1997), depending on geology, the limit between microseisms and microtremors can be shifted to a lower frequency. In deep soft basin, such as the Mexico basin, microtremor behavior (i.e., daily variation) can be observed at frequencies lower than 1 Hz.

Therefore, based on the review of the literature, microtremor, also known as ambient noise, are defined as a small scale vibrations of ground caused by multiple random sources such as sea water waves, turbulent wind, effects of wind on trees or buildings, industrial machinery, car and heavy vehicles or human activities, etc. The following Table 3.4 summarizes the distinction between long-period microseisms and short-period microseisms according to the literature review of different authors.

Table 3.4: Distinction between long-period and short-period noise according to different authors.

Author, Reference	Marginal frequency	Microseisms: long-period noise	Microtremors: short-period noise
Aki, 1957	1.0 Hz		> 1.0 Hz: traffic
Bard, 1998	1.0 Hz	< 0.3/0.5 Hz: ocean waves at long distances (0.3/0.5)-1.0 Hz: close coastal sea waves and wind	> 1.0 Hz: human activity, reflecting human cycles
Field et al., 1990	1.0 Hz	0.2-1.0 Hz: oceanic disturbances	
Lermo and Chávez-García, 1994	0.5 Hz	< 0.5 Hz: atmospheric perturbations over the oceans	> 0.5 Hz: Rayleigh waves excited locally from traffic disturbances
Nakamura, 1989	0.3-0.5 Hz	< 0.3-0.5 Hz: sea waves	> 0.3-0.5 Hz: storm and artificial forces

Kanai (1957, 1961), and Kanai et al. (1958) were described the microtremor recording equipment and analysis method. From their investigation; they were drawn three important conclusions.

- ◆ The time period distribution curves obtained from microtremor analysis may be categorized according to the nature of the surface layer and the thickness of the soft soil to bedrock. They have proposed a representative period distribution curves as shown in Figure 3.12. As shown in the figures, on firm soils the peak value of the curve lies in the period range 0.1-0.3 s, Type I (Figure 3.12 a). On softer soils the peak value of the curve lies in the range 0.4-0.6 s, Type II, and Type III (Figure 3.12 b and c) while on thick deposits of particularly soft soils the peak value lies in the range of 0.4-1.0 s, Type IV (Figure 3.12 d).
- ◆ The period distribution curves of microtremors show similar results to those of earthquake motion recorded at the same site.
- ◆ The motion of microtremor are said to result from a disturbance at bedrock level being propagated vertically through the deposit as a body waves.

Kanai and his team have done a series of microtremor measurements in free-field as well as in buildings in different Japanese cities. They have published a series of paper about microtremor, and its application in ground and building response analysis. The detailed of the microtremor, measurement instrument and its applicability can be found in Kanai et al. (1955), Kanai (1957), Kanai et al. (1958), Kanai (1961), Kanai and Tanaka (1961), Kanai (1962), Tanaka (1962), Kanai and Morishita (1963), Kanai et al. (1965), and Kanai et al. (1966).

After detailed study about microtremor, Kanni and Tanaka (1961) have presented their findings of microtremor as follows:

- ◆ The relative hardness and thickness of the soft soil layers may affect the period distribution curves of microtremors.
- ◆ There exists a diurnal variation in amplitude of microtremors which is based on the measurements of microtremor in daytime and at night.
- ◆ The shape of the period distribution curves change with depth which may be due to the presence of resonance effect of the surface layers.
- ◆ Single peak of microtremor analysis result shows the predominant period of earthquake motion at that locality, but where the curve is multi-peaked; the predominant period of earthquake shaking may take any peak value.

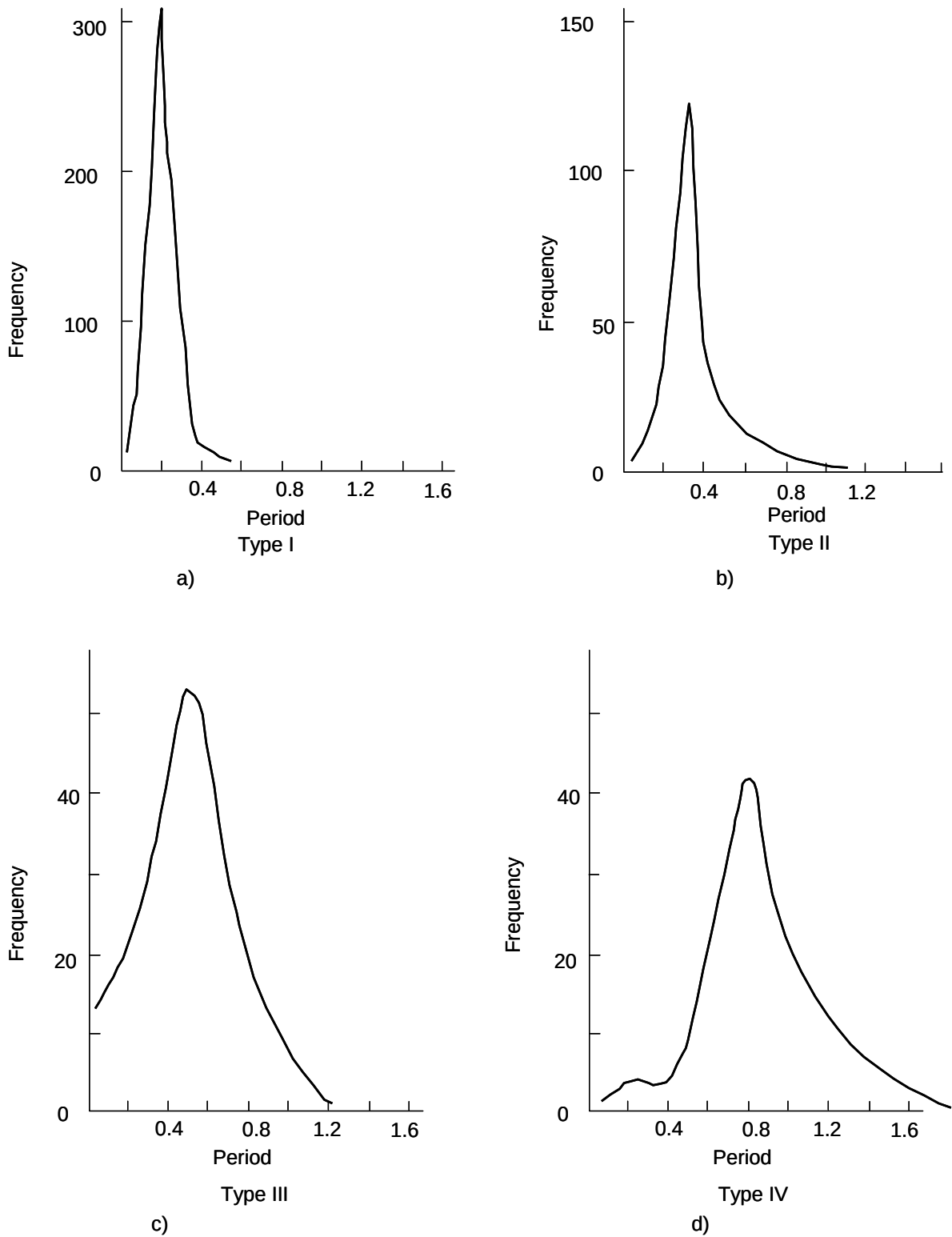


Figure 3.12: Representative period distribution curves of microtremors (After Kanai, 1954).

3.2.2 Influencing parameters

The characteristics of microtremors are varied both temporally and spatially. Temporal or spatial variations of microtremors can generally be attributed to human activities or meteorological action. Human activities are mainly responsible for the variation of microtremor temporally (Kanai, 1957). The difference in characteristics of microtremor can be seen clearly in day time data and night time data. In day time, the cultural noises are generally high than in night time, which may be due to the human and industrial activities in the day time. Meteorological action can be short term or long term. The short term variation in microtremor may be due to rainfall, wind, storms, snow etc, while the long-term variation may be due to seasonal variations such as stronger oceanic sea disturbance. Several studies have reported that meteorological conditions can disturb long-term ambient noise (Friedrich et al., 1998). It has been found that human activities mainly affect the frequency range above 1 Hz, where as meteorological variations mainly influence the short-frequency range below 1 Hz.

Besides these temporal and meteorological influencing parameters, microtremors can be affected by a variety of other factors, such as water movements, in general, may influence the short frequency range of ground motion. Lay and Wallace (1995) mentioned that wave surf and standing waves in the ocean are some of the primary source of seismic noise, mostly affecting frequencies between 0.15 and 0.2 Hz. Kanai and Tanaka (1961) were analyzed the microtremors which was observed continuously for 24-hour as shown in Figure 3.13a. This figure demonstrates that the mean period of microtremors does not vary much with time, but their amplitude during daytime is considerably larger than that at night. They further observed the microtremor at several dozen locations and compared the period and amplitude of microtremors for day and night as shown in Figure 3.13b. Based on his observation, the following empirical formula was obtained.

$$[Amplitude(nighttime)] = 0.3 \times [Amplitude(daytime)]^{1.5} \dots\dots\dots (3.15)$$

Borcherdt (1970) has mentioned that the cultural noise sources which are very near from the observation site should be avoided in the analysis as they have not traveled through the underlying soil. These kinds of effect can be observed close to running machines, turbines, water pumps, and generator.

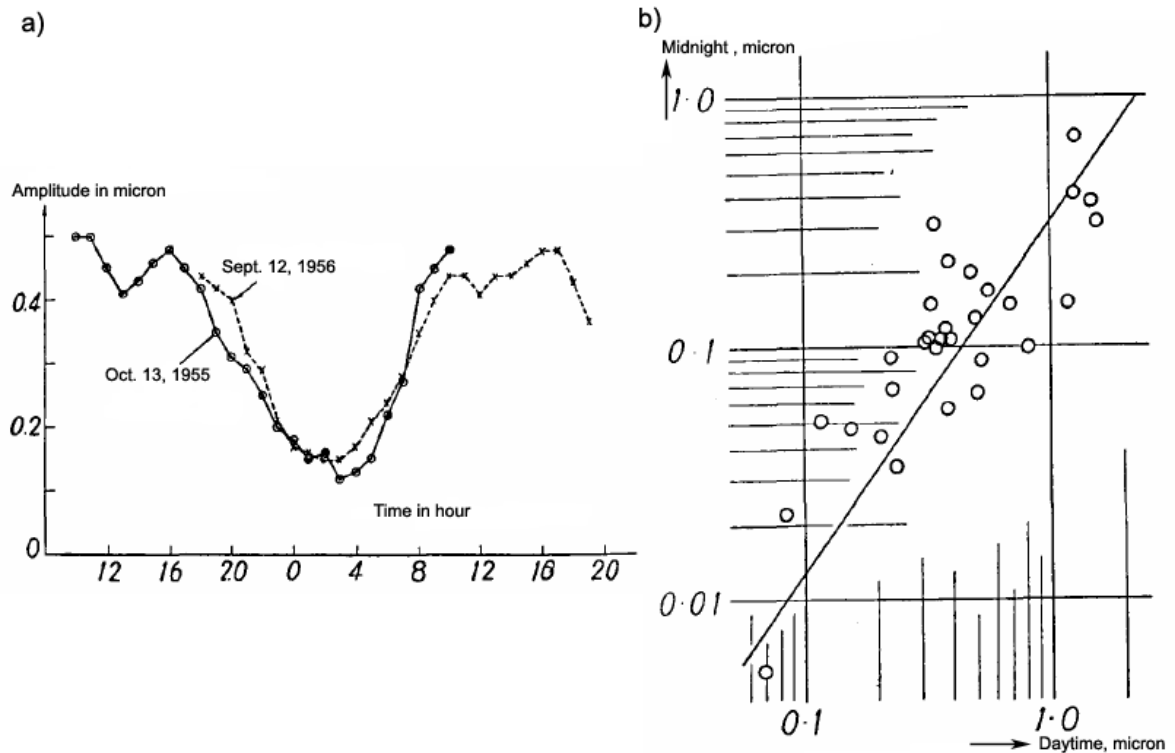


Figure 3.13: a) Results largest amplitudes of microtremor measurements for a 24 hour period at the Earthquake Research Institute, b) Amplitude relation of microtremor between midnight and daytime (after Kanai and Tanaka, 1961).

3.2.3 Practical advantage and application of microtremor

Due to close relationship between the nature of ambient vibrations and the fundamental dynamic behavior of soil layer, the practical use of microtremor in engineering purpose is increasing. In comparison to use of earthquake data in site effect study, it has following advantage and application.

- ◆ Microtremor can be available anywhere and at any place and it does not depend on the seismic event. Therefore it can be measured at any required place and time (Kanai, 1962) and can be applied for evaluation of ground characteristics even in areas of low seismicity (Field and Jacob, 1993).
- ◆ Measurement of microtremor is very easy and economical, and data acquisition also comparatively convenient than earthquake data.
- ◆ In urban areas where population density is high, ground response analysis by using seismic data is quite impossible. Therefore, microtremor data can be an alternate for site effects study.
- ◆ Compared to other method, such as borehole data analysis, this method is

economical, quick for ground as well as structural response analysis.

- ◆ It has wider application, such as site effect study, ground amplification, shear waver velocity computation, liquefaction study, and bedrock depth estimation.
- ◆ No environmental impairment in comparison to other method such as borehole drilling.
- ◆ Microtremor can be used in the analysis of building vulnerability with economical way which became one of the convenient techniques in all geographical regions.

3.2.4 Limitations in use of microtremor data

- ◆ Only weak-motion site investigation is provided by the observation of microtremors, and hence nonlinear site effects are not covered by the investigation of microtremors (Field and Jacob, 1993).
- ◆ The characteristics of ambient noise are varied from area to area and the theoretical basis of these waves is unknown. Some time the results may mislead the actual condition of the site.
- ◆ There is a large uncertainty in the true amplification value of the seismic waves by using microtremor data (Bard, 1998).
- ◆ It is not possible to separate source-path effects of ambient noise from the site effects.
- ◆ Significant variation in the noise level during the day- and nighttime is present (Kanai, 1983). For precise investigation, repeated measurement and analysis of data need to be done which makes this method less attractive and more expensive.
- ◆ In the case of thick sedimentary structures which have low resonance frequencies ($f < 0.5$ Hz), some times microtremor may not excite the soil structure at such low frequencies (Lachet and Bard, 1994)

Table 3.5: Comparative description of ambient noise and earthquake recordings.

Description	Recordings of	
	Ambient seismic noise (microtremor)	Earthquakes (weak- and strong-motion data)
Complexity of measurement	No	Yes
Applicability in low-seismicity regions	Yes (highly applicable)	(comparatively less applicable)
applicability in noisy areas	Yes	No
Cost of instrumental maintenance	No (very less)	Yes (comparatively high)
Extent of environmental impairment	No (very less)	No
Identification of source and path effects	No	Yes
Reflecting the nonlinear soil response	No	Yes (Partially)

3.3 SEISMIC SITE EFFECTS

3.3.1 Site effects

Seismic waves generated at the earthquake source propagate through different geological formations until they reach the surface at a specific site. The travel paths of these seismic waves in the sediment layers which are above the bedrock strongly affect their characteristics, producing different effects on the earthquake motion at the ground surface. The fundamental phenomenon responsible for the amplification of ground motion over the soft sediments is the trapping of seismic waves due to impedance contrast between sediments and the underlying bedrock. When the structure is horizontally layered (which will be referred to in the following as 1-D structures), this trapping affects only body waves traveling up and down in the surface layers. When the surface sediments form a 2-D or 3-D structure, i.e., when lateral heterogeneities such as thickness variations are present, this trapping also affects the surface waves which develop on these heterogeneities, and thus reverberate back and forth. The interference between these trapped waves leads to

resonance patterns, the shape and the frequency of which are related with the geometrical and mechanical characteristics of the structure. These complex physical phenomena are known as site effects.

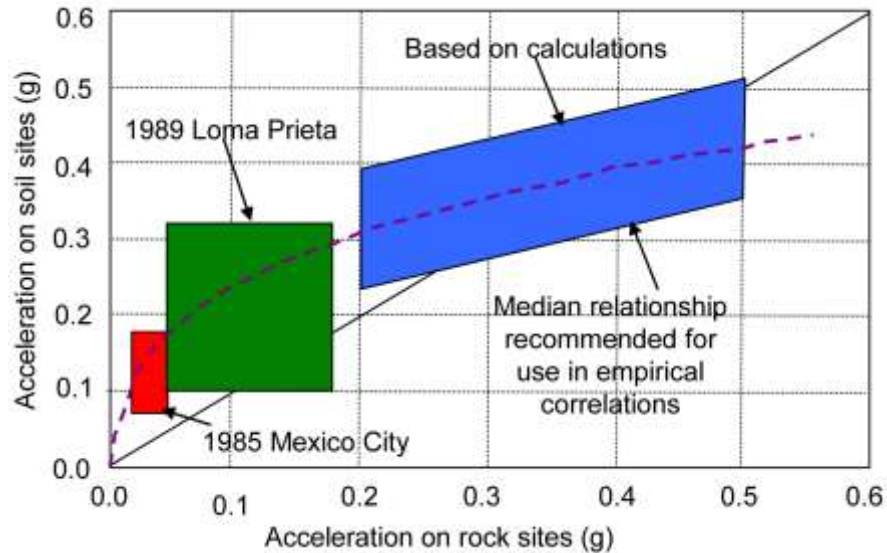


Figure 3.14: Approximate relationship between peak acceleration on rock and on soft soil sites (redrawn after Idriss, 1990).

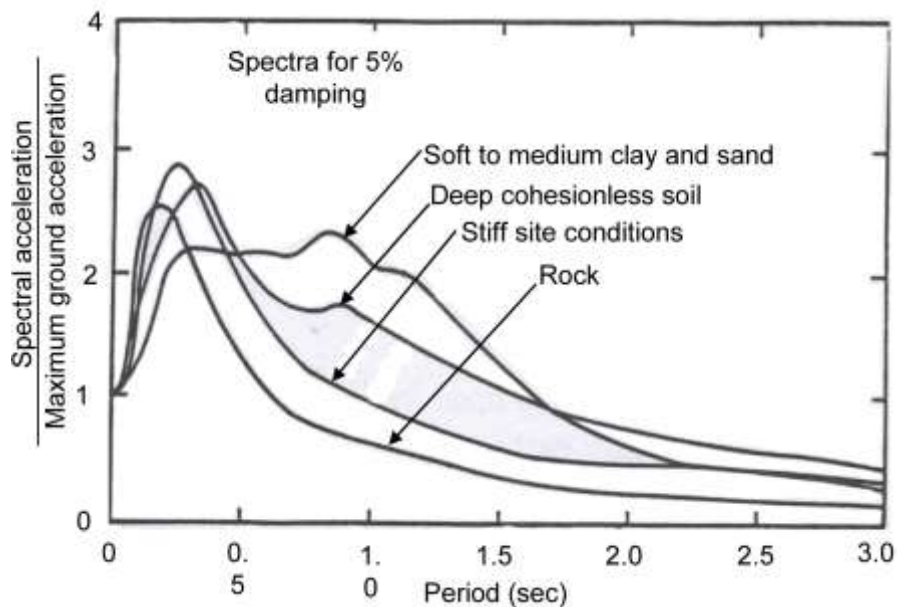


Figure 3.15: Average normalized response spectra for 5% damping for different local site conditions (after Seed et al., 1976).

The evaluation of local site effects is particularly important because most of the densely populated urban settlements, such as Izmit, Lima, Lisbon, Kobe, Kathmandu,

Manila, Mexico, Osaka, San Francisco, San Salvador, Thessaloniki, Tokyo, are situated over sediment deposit and are under earthquake threat. In past and recent earthquakes, such as in 1988 Armenia ($M_w=6.9$) (Armenian et al., 1997), 1990 Manjil-Rudbar ($M_s=7.7$) in Iran (Yegian et al., 1995), 1994 Northridge ($M_s = 6.8$) in USA (Todd et al., 1994), 1995 Kobe ($M_w= 6.9$) in Japan (Kitagawa and Hiraishi, 2004), 2001 Bhuj ($M_w=7.7$) in India (Kayal and Mukhopandhey, 2006), 2001 Kocaeli ($M_b=7.4$) in Turkey (Saatcioglu et al., 2001), 2004 Sumatra ($M_w=9.3$) in Indonesia (Saatcioglu et al., 2005), 2005 Kashmir ($M_L =7.6$) in Pakistan (Magsood and Schwarz, 2008), 2008 Shichuwan ($M_L= 6.98$) in China (Lieping, 2008), 2010 Haiti ($M_L=7$) in Haiti (RMS, 2010), and 2011 Tohoku ($M_w = 9.0$) in Japan (Koketsu et al., 2011), it has been observed that the local site conditions have a great influence on the damage distribution. It is, therefore, very important to take into account and predict these possible local site effects when assessing the earthquake hazard at regional and local scale.

Many researches (such as Berilgen, 2007; Borchardt, 1970; Borchardt and Gibbs, 1976; Gazetas et al., 1990; Guéguen et al., 1998; Hartzell et al., 2001; Jennings, 1997; Lekkas, 1996, 2001; Mucciarelli, 2011; Panzera et al., 2011; Seed et al., 1990) have mentioned that local site condition could play a dominant role in damage distribution during strong ground shaking. Effect of local site conditions can be found by comparing ground surface motions measured at different sites. Local site conditions strongly influence peak acceleration amplitudes (refer Figure 3.14), frequency content of surface motion, and the shapes of response spectra (refer Figure 3.15) (Kramer, 1996). An example about the local site effects can be illustrated from the recorded ground motion and damage distribution during 1985 Michoacan ($M_s = 8.1$) earthquake. Figure 3.16b) shows the response spectra computed from recorded motion at the Universidad Nacional Autonoma de Mexico (UNAM) and the Secretary of Communications and Transportation (SCT) sites (Figure 3.16). The UNAM site was located in the Foothill zone on 2 to 5 m of basaltic rock underlain by softer strata of unknown thickness, and the SCT site was located on the soft soils of the lake zone. Figure 3.16a and b illustrate the pronounced effects of the lake zone soils at periods of approximately 2 sec and also shows the accelerations at the SCT site were about 10 times greater than those at the UNAM site. The damage distribution in the Mexico City during this earthquake also showed negligible damage in the Foothill zone and the greatest damage in the area of thick soft sediments. These all are due to the site effects phenomenon in those areas.

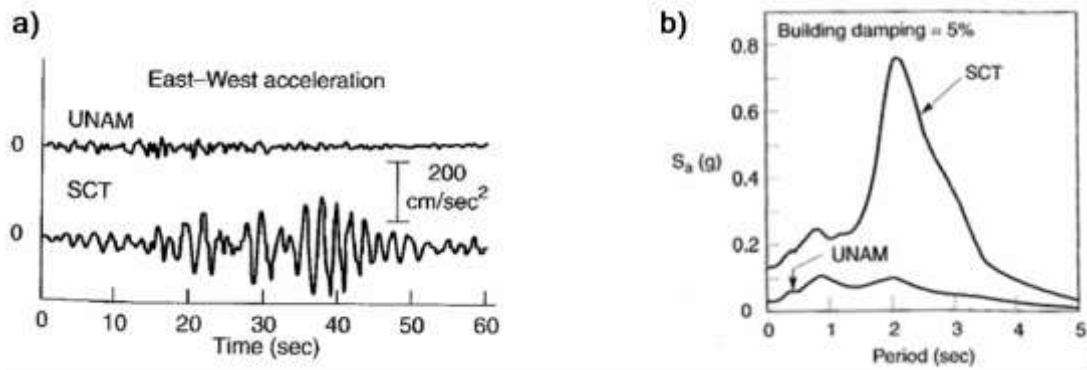


Figure 3.16: a) Time histories of acceleration, b) Response spectra for UNAM and SCT sites obtained from strong ground motion records from Mexico City during 1985 Michoacan Earthquake (After Kramer (1996), and Stone et al. (1987)).

3.3.2 Topographic effects

The modification and amplification of seismic ground motion in the vicinity of topographic features such as hillsides, ridges, and canyons is called topographic effects. Damage distribution during past earthquakes reported that buildings located at hill tops suffered much more intensive damage than those located at the base. Géli et al. (1988), Faccioli (1991), and Finn (1991) have shown that surface topography considerably affects the amplitude and frequency contents of ground motion. According to Bard (1982), amplification of ground motion in a topographic irregularity is generally related to the focalization of seismic waves at its topmost part due to the existence of diffraction, reflection, and conversion of the incident waves. Panzera et al. (2011) have mentioned that resonance phenomenon at the top part of the hill occur when the wavelength of the incident wave is comparable to the horizontal dimension of the hill. Jibson (1987) also mentioned that increase amplification near the crest of a ridge was measured in five earthquakes in Matsuzaki, Japan.

Analysis of topographic irregularities is a complicated problem. The interaction of waves can produce complex patterns of amplification and deamplification, depending on the geometry of the irregularity and on the types, frequencies, and angles of incidence of the incoming waves (Sanchez-Sesma and Campillo, 1993). This well documented phenomenon has yet to be addressed in design codes.

For a triangular infinite wedge subjected to vertically propagating SH-waves, apex displacements are amplified by a factor $2\pi/\Phi$, where Φ is the vertex angle of the wedge (Figure 3.17a) (Kramer, 1996). This approach can be used to approximate topographic effects for certain cases of ridge-valley terrain (Figure 3.17b). Soil and topographic effects

are considered under the general denomination of local site effects. Beyond these effects and under certain circumstances, induced effects may occur for large amplitude incoming waves, among which are slope instabilities (landslides) and liquefaction. Topographical effects play an important role for making resonance between ground and the structures and hence its effects need to be considered during seismic hazard assessment.

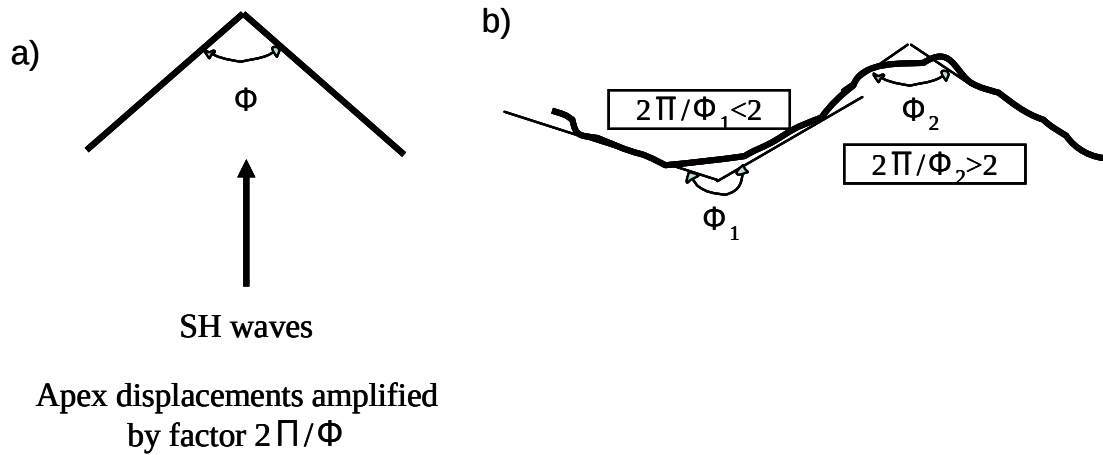


Figure 3.17: Characterization of simple topographical irregularities (a) triangular wedge and b) trough and crest (redraw after Kramer, 1996).

3.3.3 Basin effects

Basin edge effect is an effect in which the seismic waves are amplified strongly and produce longer duration of shaking in short distance away from the basin edge. This type of effect can be attributed to the areas which are situated on a deep basin. The curvature of a basin (refer Figure 3.18) in which softer alluvial soils have been deposited can trap body waves and cause some incident body waves to propagate through the alluvium as surface waves (Vidale and Helmberger, 1988; Kramer, 1996). Kawase (1996) hypothesized that amplification causing the damage belt in Kobe in 1995 earthquake was due to constructive interference between horizontally propagating Rayleigh waves and the direct shear wave propagating vertically through the sediments a short distance away from the basin edge, a phenomenon he subsequently termed the “basin-edge effect”.

Bard and Gariel (1986) have demonstrated that the one-dimensional and two-dimensional amplification function are quite similar at the center of the shallow, flat valley, which indicates that one-dimensional analysis can be appropriate in that area. Closer to the edge of the valley, however, the amplification functions are considerably different. They again pointed out that for the deep valley agreement between the one-and two-dimensional amplification functions was much better at the center of the valley than near the edges, but

was not as good as for the shallow valley. The potential for significant differential motion across such alluvial valleys has important implications for the design of long-span structures such as bridge and pipelines that often cross the valleys.

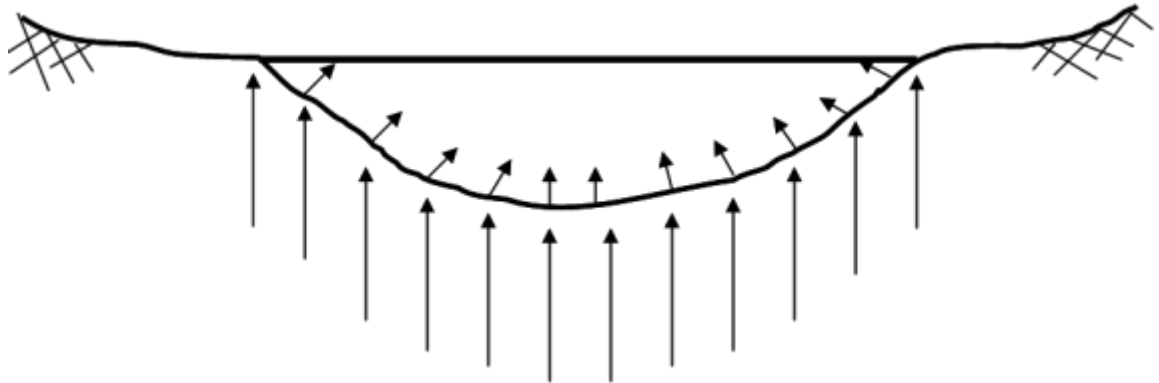


Figure 3.18: Focusing of seismic waves due to basin edge effects.

3.4 EVALUATION OF SITE EFFECTS

The main objective of the site response estimation is the evaluation of the geotechnical characteristics of a site. Numerous observations in the past earthquakes have clearly shown that the damage distribution is not uniform for the particular area and mostly varied according to the local soil condition and depth of the soft soil over the bedrock respectively. So, it is necessary to understand how characteristics of the ground motion are changed from bedrock to the surface. The difference between earthquake motion at the ground surface and geological basement (bedrock) are known as the transfer characteristics of the local subsoil, which is expressed by the transfer function. Thus, by knowing the transfer function for a particular area, it can easily understand the level of ground motion and its characteristics at the surface.

The variation of ground motion (i.e. amplification or deamplification) due to subsoil characteristics can be investigated by different techniques. These techniques often divided into two groups, namely:

- ◆ Analytical technique
- ◆ Instrumental technique

Analytical technique requires detailed information about the sub soil such as geotechnical parameters of each soil layers while the instrumental technique requires seismic data measured at the required site. The detail description of the above two methods is presented in the following section.

3.4.1 Analytical method: Theoretical transfer function of one-dimensional sub soil layers

If detail information about the subsoil parameters, such as modules of elasticity, shear modulus, density, Poisson's ratio, shear wave velocity of each soil layer is known, an analytical method can be used for site response analysis. The input parameters play an important role to obtain the reliable results of the particular site from site response analysis. Some of these parameters that are used in analytical approach are described as follows:

1) Modulus of elasticity (young's modulus) E

The modulus of elasticity E is a measure of elongation with respect to the total length of the burdened soil specimen under unidirectional impact. For any soil materials it can be calculated as

$$E = (2.0 \dots \dots \dots 3.0)G \dots \dots \dots (3.16)$$

2) Shear modulus G

The shear modulus G is a measure for the material resistance of a test piece under shear deformation. The shear modulus can also be defined as the ratio of shear stress τ and shear strain γ .

3) P-wave and S-wave velocity, V_p and V_s

These are the body waves and propagate within the elastic solid. They depend on the stiffness of the soil material with respect to the types of deformation induced by each wave. For soil materials the P wave velocity exceeds the S wave velocity by an amount that depends on the compressibility (reflected in Poisson's ratio, ν) of the solid (Kramer, 1996). The ratio between V_p and V_s is given by:

$$\frac{V_p}{V_s} = \sqrt{\frac{(2 - 2\nu)}{(1 - 2\nu)}} \dots \dots \dots (3.17)$$

4) Poisson's ratio ν

Poisson's ratio ν represents the load-dependent ratio between the specimen alternations in transversal and longitudinal direction. The value of Poisson's ratio for soil materials varies between 0.2 (rock) and ≈ 0.5 (water-saturated loose sediments).

5) Theoretical transfer function of one-dimensional subsoil layers

Depending upon the level of accuracy needed in the results, type of the subsoil characteristics and geomorphology of the site, the method of evaluation will be different. Moreover, depending upon the data available of the local site, the characterization of the site can be performed in terms of a one-, two- or three-dimensional subsoil model. Generation of two- or three- dimensional subsoil models requires sufficient boreholes data and also enough computation time and hence generally one-dimensional modeling is performed. However, the quality of results obtained from the analytical site response analysis methods depend on the detailed information about the subsurface stratigraphy, including the layer thickness, depth of the basement from surface, geotechnical (mechanical and dynamic) properties of the soil materials.

The transfer function describes the resonance characteristics of the subsoil profile, which in turn is described by its geological stratigraphy (thickness of the soil layers), total sedimentary thickness, and mechanical (ρ , ν) or dynamic parameters (E , G , ν_p , ν_s) of the subsoil materials. Waves passing through the sedimentary layers are modified in frequency and amplitude depending upon the conditions at the layer boundaries and transfer processes of the different soil materials. The transfer function provides information about the factor with which amplitudes of the harmonic motions are amplified due to the local soil conditions.

The one-dimensional transfer functions determined based on the following assumptions (Mayer-rosa and Jimenez, 1999; Kramer, 1996):

- ◆ The structural soil model consists of horizontally stratified soil layers, each being homogeneous and isotropic, and extended to infinity,
- ◆ The ground surface is level, which means that no effects coming from surface topography can occur,
- ◆ Only plane horizontal shear wave motions are addressed, as vertically propagating SH waves have the greatest influence on the dynamic site period and soil behavior,
- ◆ Each soil layer is described by its respective thickness h , shear modulus G , mass density ρ , and damping factor ζ .

This technique has been used into the different programs (e.g. ProShake, WakaShake, SIRESCO) to calculate the transfer functions. Theoretical method for obtaining frequency-dependent transfer function of one-dimensional soil models is described in detail below.

6) Evaluation of Transfer function of damped layered soil

Considering a soil deposit having N numbers of horizontal layer where the N^{th} layer is bed rock as shown in the Figure 3.19 below. Assuming that each layer of soil behaves as a Kelvin – Voigt solid, the wave equation is of the form

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^3 u}{\partial z^2 \partial t} \dots \dots \dots (3.18)$$

The solution to this wave equation can be expressed in the form

$$u(z, t) = Ae^{i(\omega t + k^* z)} + Be^{i(\omega t - k^* z)} \dots \dots \dots (3.19)$$

Where A and B represent the amplitudes of waves traveling in the $-z$ (upward) and $+z$ (downward) directions, and ρ , G , η represents density, shear modulus and viscosity of the soil respectively.

$$\text{Complex wave number } K^* = \frac{\omega}{v_s^*} = \frac{\omega}{v_s(1+i\xi)} \approx \frac{\omega}{v_s}(1-i\xi) = k(1-i\xi) \dots (3.20)$$

Similarly the complex shear wave velocity

$$v_s^* = \sqrt{\frac{G^*}{\rho}} = \sqrt{\frac{G(1+i2\xi)}{\rho}} \approx \sqrt{\frac{G}{\rho}}(1+i\xi) = v_s(1+i\xi) \dots \dots \dots (3.21)$$

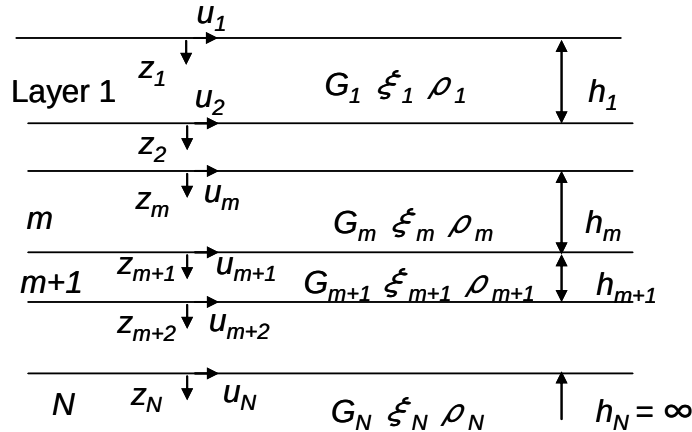


Figure 3.19: Layered soil deposit on elastic bedrock.

The shear stress is then given by the product of the complex shear modulus, G^* , and the shear strain, so

$$\tau(z, t) = G^* \frac{\partial u}{\partial z} = (G + i\omega\eta) \frac{\partial u}{\partial z} = G(1 + 2i\xi) \frac{\partial u}{\partial z} \dots \dots \dots (3.22)$$

Introducing a local coordinate system, Z , for each layer, the displacement at the top and bottom of layer m will be

$$u_m(Z_m = 0, t) = (A_m + B_m)e^{i\omega t} \dots\dots\dots (3.23)$$

$$u_m(Z_m = h_m, t) = (A_me^{ik_m^*h_m} + B_me^{-ik_m^*h_m})e^{i\omega t} \dots\dots\dots (3.24)$$

Displacement at layer boundaries must be compatible (i.e., the displacement at the top of a particular layer must be equal to the displacement at the bottom of the overlying layer). Applying the compatibility requirement to the boundary between layer m and layer $m+1$, that is,

$$u_m(Z_m = h_m, t) = u_{m+1}(Z_{m+1} = 0, t) \dots\dots\dots (3.25)$$

Yields

$$A_{m+1} + B_{m+1} = A_me^{ik_m^*h_m} + B_me^{-ik_m^*h_m} \dots\dots\dots (3.26)$$

The shear stresses at the top and bottom of layer m are

$$\tau_m(Z_m = 0, t) = ik_m^*G_m^*(A_m - B_m)e^{i\omega t} \dots\dots\dots (3.27)$$

$$\tau_m(Z_m = h_m, t) = ik_m^*G_m^*(A_me^{ik_m^*h_m} - B_me^{-ik_m^*h_m})e^{i\omega t} \dots\dots\dots (3.28)$$

Since stresses must be continuous at layer boundaries,

$$\tau_m(Z_m = h_m, t) = \tau_{m+1}(Z_{m+1} = 0, t) \dots\dots\dots (3.29)$$

So

$$A_{m+1} - B_{m+1} = \frac{k_m^*G_m^*}{k_{m+1}^*G_{m+1}^*}(A_me^{ik_m^*h_m} - B_me^{-ik_m^*h_m}) \dots\dots\dots (3.30)$$

Adding Equation 3.26 and Equation 3.30 and subtracting Equation 3.26 from Equation 3.30 gives the recursion formulas

$$A_{m+1} = \frac{1}{2}A_m(1 + \alpha_m^*)e^{ik_m^*h_m} + \frac{1}{2}B_m(1 - \alpha_m^*)e^{-ik_m^*h_m} \dots\dots\dots (3.31)$$

$$B_{m+1} = \frac{1}{2}A_m(1 - \alpha_m^*)e^{ik_m^*h_m} + \frac{1}{2}B_m(1 + \alpha_m^*)e^{-ik_m^*h_m} \dots\dots\dots (3.32)$$

where α_m^* is the complex impedance ratio at the boundary between layers m and $m+1$:

$$\alpha_m^* = \frac{k_m^*G_m^*}{k_{m+1}^*G_{m+1}^*} = \frac{\rho_m(v_s^*)_m}{\rho_{m+1}(v_s^*)_{m+1}} \dots\dots\dots (3.33)$$

For each soil material m , the product of mass density, ρ_m , and shear wave velocity, v_s , is defined as the impedance ($\rho_m v_s$) due to the impedance contrast between the layers, a trapping of the seismic waves occurs, leading to an amplification of the seismic wave amplitudes.

At the ground surface, the shear stress must be equal to zero, which requires (from

Equation 3.27) that $A_l = B_l$. If the recursion formulas of equation of equation (3.31 & 3.32) are applied repeatedly for all layers from 1 to m , functions relating the amplitudes in layer m to those in layer 1 can be expressed by

$$A_m = a_m(\omega)A_1 \dots\dots\dots (3.34)$$

$$B_m = b_m(\omega)B_1 \dots\dots\dots (3.35)$$

The transfer function relating the displacement amplitude at layer i to that at layer j is given by

$$F_{ij}(\omega) = \frac{|u_i|}{|u_j|} = \frac{a_i(\omega) + b_i(\omega)}{a_j(\omega) + b_j(\omega)} \dots\dots\dots (3.36)$$

Because $|\ddot{u}| = \omega|\dot{u}| = \omega^2|u|$ for harmonic motion, Equation (3.36) also describes the amplification of accelerations and velocities from layer i to layer j . Equation (3.36) indicates that the motion in any layer can be determined from the motion in any other layer. Hence if the motion at any one point in the soil profile is known, the motion at any other point can be contribute.

In the case of uniform damped soil on rigid rock as shown in Figure 3.20, the transfer function can be written as (Kramar, 1996).

$$|F(\omega)| \approx \frac{1}{\sqrt{\cos^2(\omega H/v_s) + [\xi(\omega H/v_s)]^2}} \dots\dots\dots (3.37)$$



Figure 3.20: Soil deposit of thickness H underlain by rigid bedrock.

For small damping ratios, Equation (3.37) indicates that amplification by a damped soil layers also varies with frequency. The amplification will reach a local maximum whenever $kH \approx \pi/2 + n\pi$ but will never reach a value of infinity, since (for $\xi > 0$) the denominator will always be greater than zero. The frequencies that correspond to the local maxima are the natural frequencies of the soil deposit. The amplification factor is equal to

the ratio of the free surface motion amplitude to the bedrock motion amplitude.

The n th natural frequency of the soil deposit is given by

$$\omega_n \approx \frac{v_s}{H}(\Pi/2 + n\Pi) \quad n=0, 1, 2, \dots, \infty \dots\dots\dots (3.38)$$

Since the peak amplification factor decreases with increasing natural frequency, the greatest amplification factor will occur approximately at the lowest natural frequency, also known as fundamental frequency

$$\omega_0 = \frac{\Pi v_s}{2H} \dots\dots\dots (3.39)$$

The period of vibration corresponding to the fundamental frequency is called the characteristic site period

$$T_s = \frac{2\Pi}{\omega_0} = \frac{4H}{v_s} \dots\dots\dots (3.40)$$

The characteristic site period, which depends only on the thickness and shear wave velocity of the soil, provides a very useful indication of the period of vibration at which the most significant amplification can be expected.

3.4.2 Instrumental methods

Ground response analysis based on the numerical method is time consuming, and required large number of input data. Numeral method of analysis is comparatively uneconomical and really not possible for the evaluation in the large area. The outcome from the numerical method depends primarily on the quality and accuracy of the input data. In fact, generation of input data for the large area is difficult and time consuming. Therefore, instrumental method of site response evaluation is extremely used in different part of the world. Basically two kinds of input data – seismic (strong and weak) data and microtremor (i.e. ambient vibration) are used in the instrumental method. Therefore, instrumental method is become fast, economical and widely used method for ground response analysis. Some of the mostly used instrumental techniques for the site effect study are described below.

3.4.2.1 Spectral amplification function method

The amplification characteristics of a site can be obtained by analyzing the recorded earthquake data using the spectral amplification functions of earthquake data. The standard amplification function (S_a/a) of earthquake data basically relies on the calculation of response spectra normalized with respect to the peak value of acceleration $S_a(T)/a$ (peak

ground acceleration PGA), velocity $S_v(T)/v$, or displacement $S_d(T)/d$ of the ground motion. Since strong ground motion data is available in terms of ground acceleration, the calculation of normalized response spectra with respect to peak ground acceleration $S_a(T)/a$ is most commonly used.

In earthquake engineering practice, the direct estimation of the expected structural demand from earthquake excitation is usually done by calculating response spectra. A response spectrum describes the maximum response induced by the ground motion in oscillators with a single-degree-of-freedom (SDOF) which is a function of the natural frequency (or natural period) and damping ratio of the SDOF system. The response spectra represent only the maximum response of a number of different structures.

In order to eliminate the absolute value of earthquake ground motion and to establish a common basis that can compare the amplifications of different events, a normalization of the response spectrum has to be carried out. Figure 3.15 shows the normalization of an acceleration response spectra ($\zeta=5\%$) by its value of peak ground acceleration. In the past, it was assumed that the shapes of the response spectra for a given class of soil were identical and the level of peak ground acceleration was only distinguishing criteria. Later, due to the availability of many earthquakes data, it is understood that the spectral shape's is also depend on both the magnitude as well as the distance of recorded site from source (Silva et al., 1999). According to this observation, it can be said that spectral investigations of earthquake data do not necessarily reflect the local site conditions only, but also the type of the earthquake record itself. Dependencies on the magnitude level and/or the distance to the seismic source exist, leading to certain ranges of variation with respect to the frequency band of maximum amplification (predominant frequency) and to the amplification level itself. Past investigations at single station where a sufficient amount of earthquake records are available confirm the sharp discrepancy between the spectral shapes of small-to medium-magnitude and large-magnitude earthquakes (Kramer, 1996; Silva et al., 1999).

Most of the international earthquake code uses the normalized response spectra for making soil-site dependent design spectra. Shapes of normalized response spectra for each site class are simplified by using average curve. Response spectra from the earthquake are highly irregular and their shapes reflect the details of their specific frequency contents and phasing. Design spectra are generally quite smooth; they are usually determined by smoothing, averaging the response spectra of multiple motions.

3.4.2.2 Fourier amplitude spectrum method

Kannai (1954), preliminarily introduce a use of Fourier amplitude spectra method on microtremor data for ground response analysis. Kanai (1962) did a lot of pioneer works for using microtremor data to study the underground geological structures. One his conclusion is that the amplitudes of certain period waves exist in the alluvium of weathered sediment sites are larger than those at rock sites. This is because the multiple reflections and resonance of seismic signals in the sediment layers. After that Kanai and Tanaka (1961) have discussed the relation between predominant period and geological layer structures using the distribution curves of seismic wave periods.

There are few assumptions during applying Fourier amplitude spectra as follows:

- ◆ Ambient seismic noise solely is composed of vertically incident shear waves (S-waves)
- ◆ The source spectrum of microtremors is characterized by white noise, which means that its power spectral density is constant with frequency

Lermo et al. (1988), Nakamura (1989), Ohta et al. (1978) have mentioned that there are various limitation of this technique. They pointed out that microtremors are mainly composed of surface waves and each spectrum based on free field noise data is not white, being influenced by band limited perturbing signals and temporal variations. Although, there are many short coming of Kanni's assumption, the Fourier amplitude spectra method on microtremor data sometimes lead to satisfactory results. According to the above researchers, the reliability of the result from this method is strongly related to the impedance contrast at the boundary of the basement rock. When the impedance contrast ratio is high, surface and/or body waves are trapped and there is clear spectral peak at the resonance frequency, whatever the origin of the noise waves. When this ratio is low, the trapping is not enough, and noise spectra reflect mainly the source spectra.

Tokimatsu (1995) investigated the variation of Fourier spectra of the horizontal and vertical motion on microtremors at two different sites (*A* and *B*) at 3-hour intervals over a 24-hour period as shown in Figure 3.21. The Fourier amplitude spectra at site *A* show a stable predominant peak at around 0.35 sec in both direction (horizontal, vertical), but the Fourier amplitude spectra for the horizontal and vertical component at site *B* are not satisfactory. The two site subsoil characteristics are described by the V_s -and V_p -function with depth H , the fundamental site periods are 1.39 sec for site *A* and 0.21 for site *B*.

It showed that the fundamental periods at both sites can not be estimated either from the horizontal or from the vertical Fourier amplitude spectra. Lermo and Chavez-Garcia

(1994), Tokimatsu (1995) have mentioned that spectral amplitude period obtained from Fourier amplitude spectra mislead the actual site characteristic. Therefore, the results from the Fourier amplitude spectra sometime may not give the accurate result of the site. This method must be carried out in a very cautions manner. It is therefore, the results obtained from this method should be compared or validated with the other site response analysis method.

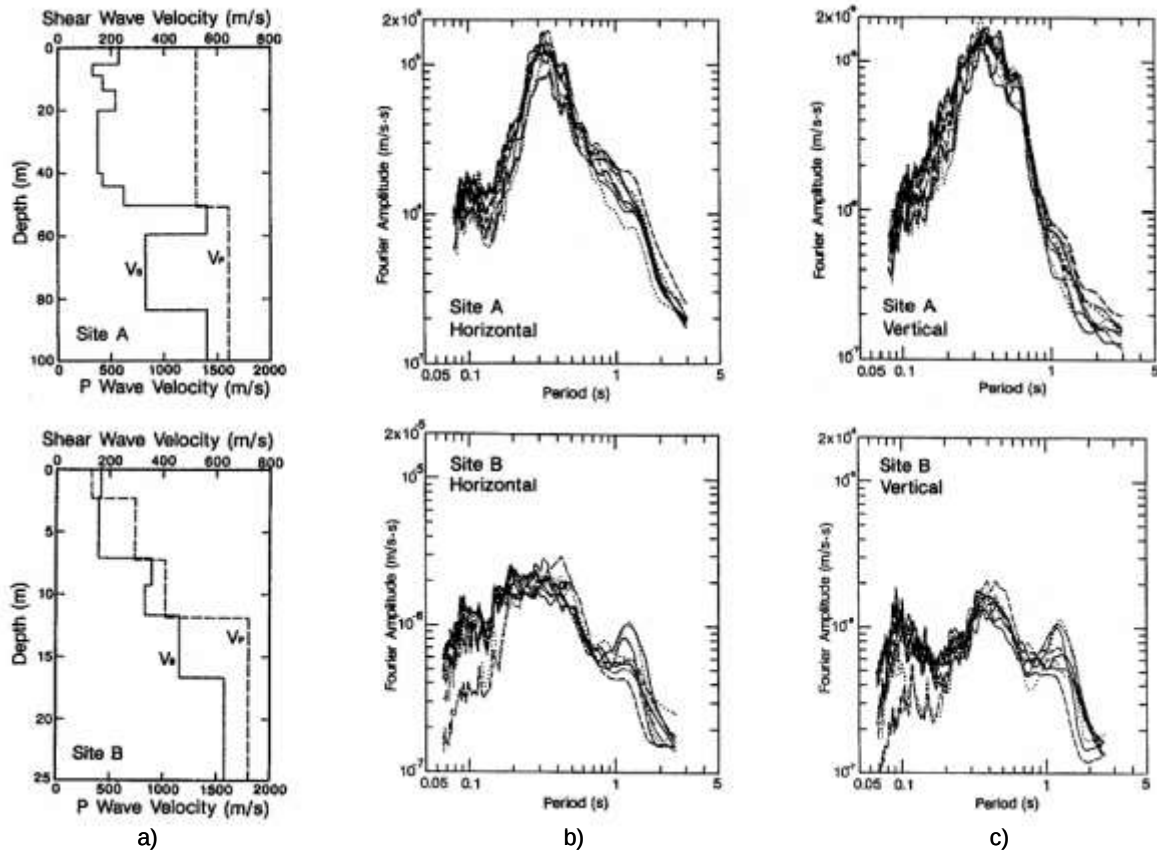


Figure 3.21: a) Seismic velocity models with depth, b) horizontal, and c) vertical Fourier spectra of microtremors at sites A and B) (Tokimatsu, 1995).

3.4.2.3 Standard spectral ratio method

The standard spectral-ratio method (SSR) technique is primarily introduced by Borchardt (1970). It is the most used method in studying the response of the sub-soil layer (Borchardt, 1970; Chavez-Garcia et al., 1990; Field et al., 1992). This method is a simple and effective to eliminate the source and propagating path effects for the regions with many earthquakes happen in the vicinity (Bonilla et al., 1997; Field and Jacob, 1995; Lermo and Chavez-Garcia, 1993). It is obtained by dividing the Fourier spectrum obtained on soft ground point by that obtained on a nearby reference point right on the bedrock.

There are two basic assumptions in this technique

- ◆ The source and path effects must be same for both measurement points
- ◆ The reference site or point must also have negligible site effects (i.e. negligible site response)

The first assumption requires that the source should be located near enough to the examined site in order to have a similar wave field (with similar incidence angles and azimuths) arriving at the two sites. The second assumption requires that the record on the reference site (i.e. bedrock) should be unaffected by any kind of site effect which is the case when the reference site is located on an unweathered, horizontal bedrock. This is an important point for using SSR method. If the reference site is on the out crop of bedrock which is not too far way from the soil site, the spectrum ratio can really eliminate the source and path effects for the strong ground motion data from the same earthquakes and or microtremor from any random noise sources. These two conditions prove to be rather restrictive in using this method. If the bedrock site exhibits a site response i.e. shows some value, the comparative site response evaluation of the nearby area can be done. For such case, Field et al. (1992) have suggested considering the estimate as a “relative site response.”

According to Lacave et al. (1999) the principal of SSR method may be described as follows: for a network of i site having recorded j events, the amplitude spectrum of the ground motion $R_{ij}(f)$ recorded at site i during the event j can be written as:

$$R_{ij}(f) = E_j(f) \cdot P_{ij}(f) \cdot S_i(f) \quad (3.41)$$

Where, $E_j(f)$ – source function, $P_{ij}(f)$ - path contribution between the source and the local site; and $S_i(f)$ - local site contribution written in logarithmic form, it leads to a simple linear equation as

$$\ln(R_{ij}(f)) = \ln(E_j(f)) + \ln(P_{ij}(f)) + \ln(S_i(f)) \quad (3.42)$$

Apart from the theoretical background, one of the major problems during application of the standard spectral ratio method is identification of a pure bedrock site near the measurement point or at required site.

According to Tucker et al. (1984), the topography of a rock site plays an important role for the reliability of the results from this method. The SSR method was applied by different researchers (Field et al., 1990; Field and Jacob, 1993; Hough et al., 1992; Lermo et al., 1988; Morales et al., 1991; Ohta et al., 1978) for ground response analysis and they investigated its application for different purpose such as site effects estimation, microzonation and so on. According to their findings, the quality of results may depend

upon the following factors:

- ◆ The type of recording instruments (quality of recording instruments)
- ◆ The recorded data types (acceleration, velocity and displacement), duration of recording
- ◆ Type of source (for earthquake source-magnitude and distance)
- ◆ Site condition of recording sites (i.e., reference site and soil sites)

Rovelli et al (1995) showed that the topographic effects at the reference stations situated on an elevation have a greatest influence in the result in spectral ratios. Similarly Chavez-Garcia et al. (1990) mentioned that spectral ratios on data coming from distance earthquakes are only valid in the frequency band 1 to 10 Hz. In contrast Gutierrez and Singh (1992) stated that the results do not depend on epicentral distance, focal depth, and or magnitude of the earthquakes.

In summary, it can be said that there are three important factors which affect the ground motion are the source, path and site characteristics. The main challenge in evaluating the site response is the identification and removal of these effects. If the recording in the rock is at a close distance to the soil site, then three governing factors, which will affect the ground motion, will be the same for both the soil site and the rock. However, when the reference site and the soil site are not near by, then the influence of source, path and the site characteristics will be different for these two sites. Borchardt and Glassomyer (1992), Hartzell et al. (2001) have mentioned that the geometric spreading of the seismic waves also affect the standard spectral ratio results.

Several researchers such as Chavez-Garcia and Cuenca (1998), Field et al. (1992), Lermo and Chavez-Garcia (1993, 1994), Lermo et al. (1988), Raptakis et al. (1998), Rodriguez and Midorikawa (2002), and Satoh et al. (2001) were applied the standard spectral ratio method and found good approximation when compared with the other site response estimating techniques. In the other hand, many researchers such as Field et al. (1990), Field et al. (1995), Hough et al. (1992), Lermo and Chavez-Garcia (1993), and Rovelli et al. (1995) have discuss about its applicability and reliability in the site response analysis.

3.4.2.4 Horizontal-to-vertical spectral (H/V) ratio method

The horizontal-to-vertical spectral ratio, i.e. the ratio between the Fourier spectra of the horizontal and vertical components of ambient vibrations (often called “microtremors” or “ambient noise”), was introduced in the early seventies by the Japanese scientists

Nagoshi and Igarashi (1971). Later, Nakamura (1989) improved this method, and it became widespread as a low-cost and effective tool to estimate the fundamental resonant frequency of sediments by measuring the microtremors in a single-station. According to Nakamura (1989), the horizontal-to-vertical spectral (H/V) ratio is the Quasi-Transfer Spectra (Transfer Function) of the soil strata over bedrock.

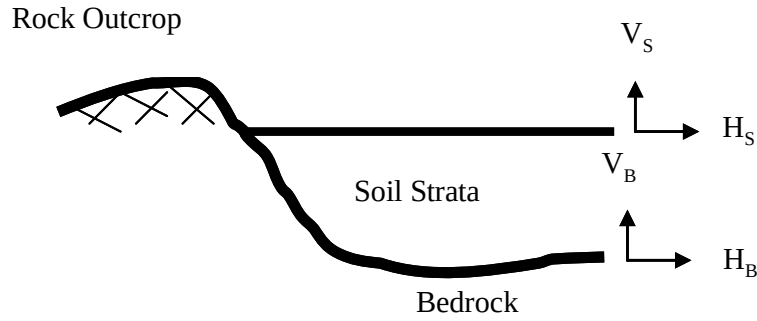


Figure 3.22: Typical geological structure of a sedimentary basin. H_S and H_B represent the horizontal Fast Fourier Transform (FFT) amplitude spectrums at the ground surface and bedrock and V_S and V_B represent the vertical Fast Fourier Transform (FFT) amplitude spectrums at the ground surface and bedrock.

The principle of the Nakamura method can be explained more precisely with the help of Figure 3.22. As indicated in this figure, the standard spectral ratio, denoted by S_E , is calculated by dividing the horizontal Fourier spectrum of the ground motions on a site, H_S , by that recorded on a nearby reference site (bedrock) H_B as follows.

$$S_E = \frac{H_S}{H_B} \dots\dots\dots (3.43)$$

Based on the work of Nakamura (1989), Lermo and Chavez-Garcia (1993) was estimated amplitude effect of the source by using the following spectral ratio, A_S .

$$A_S = \frac{V_S}{V_B} \dots\dots\dots (3.44)$$

Here, V_S and V_B are the Fourier spectra of the vertical motions on the surface and the reference site (bedrock), respectively. Originally, Nakamura (1989) assumed that the vertical component of the microtremor spectrum was not amplified by the low-velocity surface waves, and he also mentioned that the effect of Rayleigh waves should be in the vertical component of the tremor. As per his assumption, if the effect of Rayleigh waves was the same on the vertical and horizontal components, then A_S could be used to eliminate

the effect of Rayleigh waves on the transfer function. By applying this assumption to compensate the source effect (A_S), Lermo and Chavez-Garcia (1993) later introduced a modified site effect function denoted by S_M as follows.

$$S_M = \frac{S_E}{A_S} = \frac{[H_S/H_B]}{[V_S/V_B]} = \frac{[H_S/V_S]}{[H_B/V_B]} \dots\dots\dots (3.45)$$

Nakamura (1989) also pointed out that the ratio H_B/V_B was nearly equal to unity, and moreover, Huang and Teng (1999) examined the ratio using microtremors and earthquake recordings at a bedrock in Chiawan, Taiwan, and found it to be nearly equal to unity. So, Equation (3.45) can be rewritten as follows.

$$S_M = \frac{H_S}{V_S} = \frac{H}{V} \dots\dots\dots (3.46)$$

This expression is particularly true for the case of a simple, one-dimensional geological condition. This demonstrates that the H/V ratio, as defined by this transfer function, can be obtained solely from the motion of microtremors at the surface, which obviously makes it easier to estimate the characteristics of ground motion. This method overcomes the difficulty in searching a good reference site in standard spectral ratio method.

In the beginning, the H/V spectral ratio method is applied in studying site response for using microtremor data. Later, Lermo and Chavez-Garcia (1993, 1994) applied this technique to strong ground motion data of Mexico and they have compared the results obtained from microtremor measurements with standard spectral ratios of the intense, S -wave part of weak or strong motion records obtained at the same sites. Their results showed that the H/V -ratios of microtremors are able to provide a robust estimate of frequency in the first resonance mode, and a rough assessment of wave amplification.

Field and Jacob (1993) collected microtremor data in Fushing Meadows, New York and they applied the H/V method for analyzing the data. The results suggest that the H/V ratio is an effective and reliable tool to calculate the fundamental frequency of a layered sedimentary basin. Later on, Field and Jacob (1995) have compared the three various site-response study techniques that are independent of reference site to estimate the source and path effects using aftershock events of the 1989 Loma Prieta earthquake in California. They concluded that these techniques were successful in identifying the fundamental resonant frequency of the study area.

Suzuki et al. (1995) collected a microtremor data and strong ground motion data in Hokkaido, Japan. They mentioned that the peak frequency of the H/V ratio corresponded

well to the predominant frequency estimated from the thickness of a sedimentary unit. Toshinawa et al. (1997) showed that H/V spectral ratio technique can be a useful tool for the assessment of ground motion characteristics. He correlated the weak-motion amplification factors for peak ground acceleration with respect to a rock with the peak values of H/V spectral ratio of microtremors. Bonilla et al. (1997) performed a detail study of 38 aftershocks of the 1994 Northridge earthquake. They concluded that predominant frequency peaks from the H/V method correlated well with those of the ground motion data, but not the amplification values. Ibs-von Seht and Wohalenberg (1999) have applied the H/V technique to relate fundamental frequency with sediment thickness at sites in the Rhine Embayment, Germany. They collected microtremor data from 102 sites and mentioned that the main peak in the spectral ratios correlated well with the sediment thickness at the site.

Application of the H/V spectral ratio technique using microtremor measurements is very important especially in the region where big earthquake present a long return period but which exhibits a high seismic risk, according to the historical records/reports, population distribution and its socio-economic importance (Teves-Costa et al., 2001). Hardesty et al. (2010) have analyzed the microtremor data that was collected in the New Madrid Seismic Zone (NMSZ) using H/V spectral ratio method. They concluded that the longer periods corresponded to the thicker area (interface between sediment and Paleozoic basement rocks) and the shorter period with a shallow area. Huang and Tseng (2002) applied Nakamura's method in the Yuan-Lin area, Taiwan. They have mentioned that the microtremor method is suitable to identify the sites susceptible to strong ground shaking. Huang (2002) also analyzed the spectral relationship between microtremors and earthquake ground motions using the 1999 Chi Chi earthquake in central Taiwan. Mucciarelli and Gallipoli (2004) have analyzed 608 earthquakes time histories and 1280 triggered nose signals recorded at four different stations in Southern Apennines, Italy, the have obtained very similar H/V spectral ratios for these two data sets. Similarly several researchers, such as Albarello and Lunedei (2010), Bonnefoy-Claudet et al. (2006b), Bour et al. (1998), Chatelain et al. (2008), Gosar, (2007), Herak (2011), Hasancebi and Ulinsay (2006), Lachet et al. (1996), Langston et al. (2009), Lunedei and Albarello (2009, 2010), Nakamura, (2000), Parolai et al. (2004), S  nchez-Sesma et al. (2011), Teves-Costa et al. (1996), Theodulidis et al. (2005), and Tuan et al. (2011) have discussed its application in site effect study and mentioned that the H/V method reliably estimates the fundamental resonant frequency of a site.

1) Limitation of H/V method of using microtremor

Accepted as one of the most convenient methods for estimating the fundamental frequency of soft soil deposits, as mentioned by Fäh et al. (2008), the H/V method however has certain limitations in the theoretical explanation, as pointed out by Nogoshi and Igarashi (1971), Bour et al. (1998), Delgado et al. (2000), Fäh et al. (2001), Field and Jacob (1993), SESAM, (2004).

Nogoshi and Igarashi (1971) elucidated that H/V-ratios on microtremors are directly related to the ellipticity curve of Rayleigh waves because of the predominance of Rayleigh waves in the vertical component. Later they assumed that the frequency-dependent ellipticity leads to conspicuous peaks and troughs in the H/V-ratio, the first being generated by the vanishing of the vertical component around the fundamental S wave resonance frequency. According to their explanations, the H/V-ratio results are also valid in the case of Love waves, since Love waves do not impact the vertical component.

Several researchers, such as Asten (2004), Bonnefoy-Claudet (2006b), Chavez-Garcia and Rodriguez, (2007), Konno and Ohmachi (1998), Lermo and Chávez-García (1994), Tokimatsu et al. (1992) have mentioned that the microtremors are composed of a fundamental mode of Rayleigh waves. Bard (1998) pointed out that there is no ultimate correlation between the H/V peak amplitude and the maximum spectral amplification of the site and he expressed that there might be some local relationships for limited area. Similarly Horike et al. (2001) mentioned that the H/V spectral ratios partly reflect site amplification factors.

In contrast to the above explanation, Nakamura (1989, 2000) proposed that the H/V-ratio can be esteemed as the site response function to S waves as explained in the above paragraphs during derivation of H/V-ratio. According to his explanation, dividing the horizontal components of surface ground motion by the vertical, a removal of the source as well as the Rayleigh wave effects can be obtained. He assumed that the H/V-ratio is mainly determined by SH waves and ruled out the possibility of Rayleigh waves being responsible for the generation of the spectral peak, since Rayleigh wave energy is very small for this particular frequency range and that the H/V curve provides a consistent estimate of the site amplification value

Another limit of application of this method, which is reported by many researchers, is that the H/V technique only provides a reliable estimate of the site characteristics if a significant impedance contrast between overlying stratum and half space exists. This indicates that H/V-ratios on rock sites usually will not exhibit a clear frequency peak, if

any, in the high-frequency range. Although a clear threshold of impedance contrast is in most cases not given, there is still an obvious correlation between the generation of a clear H/V peak and the stiffness of overlying sedimentary layers. Applicability of the H/V technique or reliability of results also depends on several criteria; some of them are discussed below.

a) The type of the seismic sensor and digitizer

Guillier et al. (2007) was done a detailed study on the applicability of different instrumental equipment for the horizontal-to-vertical spectral ratio technique. They mentioned that reliability of H/V-spectral ratios does not depend on the type of the acquisition system (digitizer), but strongly on the type of seismic sensor. They pointed out that accelerometers in general and velocity sensors with natural frequencies higher than 1 Hz should be avoided.

b) Topographical site conditions

The topographical features should be avoided, since they unfavorably alter the H/V spectrum. The topographical effect in the results of microtremor analysis is also discussed in Section 3.3.2. With respect to the shape of spectral H/V-ratios on microtremors recorded on a steep slope or on a small hilltop, an unusual generation of distinct peaks and troughs can occur. Lermo and Chavez-Garcia (1993) observed a systematic peak of amplification (at 5 Hz) on the H/V spectral ratios of ambient noise. In which the recording instrument was installed at the top of a hill, indicating that the peak was caused by the topographical effects.

c) The local base of the sensor

The shape of H/V-spectral ratios is affected by the material of the local base. As compared to natural soil or concrete foundation, results on paved or tarmac roads and grass provide different result (Guillier et al., 2007). This may be due to the probable damping effects of the local material that may modify the seismic signals.

d) The environmental and the meteorological condition

The results of the nearby noise sources such as signals coming from the nearby machinery can change the H/V spectral ratios. Wind gusts and rainfall have an observable influence on the stability of H/V-ratios, while no changes could be observed due to

variations in temperature or atmospheric pressure conditions (Mucciarelli, 1998). Cara et al. (2003) compared the amplitude of the H/V ratio in the 0.1-3 Hz frequency band with the trends in three meteorological parameters (i.e., wind speed, atmospheric pressure, and precipitation amount). They noticed that among the meteorological parameters, wind speed had a direct link with the H/V ratio variations in the low-frequency band below 1 Hz. Mucciarelli et al. (2005) analyzed the influence of wind on seismic noise using observations, experiments, modeling. They concluded that wind increased the amplitude of the microtremor wave field without affecting the horizontal-to-vertical spectral ratio.

Similarly, Kang and Shin (2011) have studied the stability of H/V ratio using 12 days of continuous microtremor measurements. No variation was found in the frequency and amplitude of the peak spectral ratio over time. They mentioned that the amplitude patterns varied slightly for each station however the peak frequencies were consistent. They concluded that the H/V ratio estimate had little or no dependency on variation in meteorological parameters, namely atmospheric pressure, temperature, wind speed, rainfall, and humidity because energy produced on the ground surface by meteorological parameters is sheared equally among all the energy propagation directions in the subsurface.

The other factors which influence the H/V-spectral ratios are the parameters related to the instrumental settings and to the procedure of analysis. Averaging the H/V-spectral ratios on microtremors is strongly recommended to allow a clear identification of a single predominant peak while minimizing large amplitude variations. In addition to conventional averaging, spectral resolution can also be influenced by a variety of filter techniques, tapering the time-history, or smoothing the spectrum.

2) Application of H/V method

Horizontal to vertical spectral ratio method is one of the convenient, economical, efficient and widely used techniques for the following application:

- ◆ Investigation of structure vibrations
- ◆ Characteristics of geological structures
- ◆ Site selection for important facilities
- ◆ Sediment thickness measurements
- ◆ Velocity structure of sediment
- ◆ Amplification effect of soft soil
- ◆ Liquefaction vulnerability assessment

3.5 SITE CLASSIFICATION

Past earthquakes (e.g., 1985 Mexico City, 1995 Kobe earthquake, 1934 Nepal/Bihar earthquake) have demonstrated that structural damage and loss of human life have directly attributed to the effect of local site conditions. There are other factors such as, topography, basin effects, liquefaction and structural deficiencies that can also be contributed for the damage and loss of life; however the amplification of ground motion due to local site condition plays an important role in increasing seismic damage. The correlation between site effects and building damage is illustrated in Figure 3.23 by the work of Seed and Alonso (1974) for the 1967 Caracas earthquake. Damage of buildings is found significantly higher when the natural period of the building matched that of the soil deposit (Rodriguez-Marek et al., 2000). Therefore the classification of site in terms of amplification of ground motion is very important from which an accurate seismic design load can be proposed during seismic design of infrastructures.

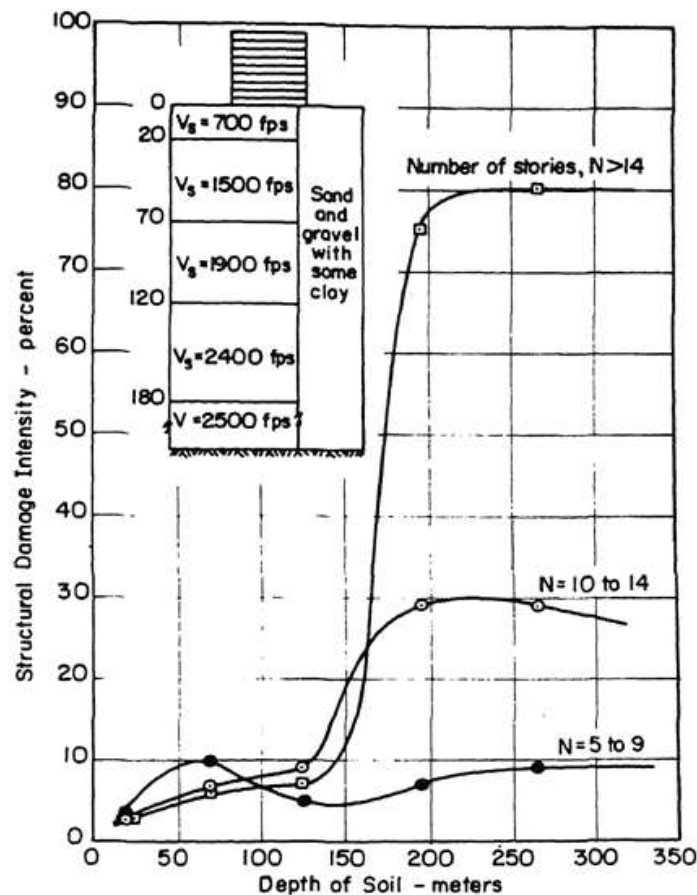


Figure 3.23: Relationship between structural damage intensity and soil depth in Caracas earthquake, 1967 (after Seed and Alonso (1974)).

There are various site classification schemes which are proposed by different researchers and are also given in the different international building codes. Many of site classification system are based on the soil characteristics of the upper 30 m of the soil profile and most of them are classified the site by rock, stiff and soft soils. Based on the elastic design spectra for each soil classes, their influences on amplification and frequency characteristics are considered.

The site characterization is very important not only for defining the seismic loads in the design of infrastructures but also in different scientific fields such as geotechnical earthquake engineering and engineering seismology. Since the local soil conditions may have a considerable influence on the characteristics of seismic waves, the importance of a reliable seismic site assessment is increasing. Various agencies purpose different site classification system. Some of important site classification system is presented in the following section.

Based on the review of the different literatures, the site classification system can be divided into three groups.

- ◆ Classification based on stiffness of soil layers
- ◆ Classification based on stiffness and sediment depth
- ◆ Hybrid classification schemes

The stiffness related classification schemes are based on the soil parameters, i.e. stiffness parameters of the uppermost 20 or 30 m from the surface. Stiffness and depth related classification is based on the similar soil layer stiffness but in different depths. Both classification systems are based primarily on the available information from borehole data and laboratory test of soils sample. The hybrid system of site classification is based on the principal of stiffness-and depth-related classification endorsed by the results of experimental site investigations. Example of this system includes horizontal-to-vertical spectral ratio from the microtremor data recorded at the ground surface. The main characteristics of the different classification schemes and their limits of application are summarized in Table 3.6

Table 3.6: Characteristics of the different types of site classification schemes.

S N	Description	Classification scheme			
		Stiffness-related		Stiffness-and-depth-related	Hybrid
		N_{SPT}	V_{s30} , G_{dyn}	V_s and H	V_s , H , f_s
1	Identification of dynamic soil properties	No	Yes	Yes	Yes
2	Identification of predominant periods of site	No	No	Yes	Yes
3	Identification of relative site amplification	No	No	To some extent	Yes
4	Identification of depth of soil layer	No	No	Yes	Yes
5	Identification of liquefaction susceptibility	Yes	No	No	To some extent

3.5.1 Stiffness-related classification schemes

Stiffness-related soil classification schemes are based on the properties of the soil layer to the uppermost 30 m from the surface (Ambraseys et al., 1996; ICBO, 1997). Value of Standard Penetration resistance of the soil (N_{SPT}) or dynamic soil parameters (e.g., average shear wave velocity (V_{s30}), shear modulus (G_{dyn}) of the 30 m soil profile) act as a main criteria for the site classification. Ambraseys et al. (1996) have classified the site based on the average shear wave velocity, V_{s30} of the uppermost 30 m of the soil profile (Table 3.7).

Table 3.7: Site classification system based on shear wave velocity (V_{s30}) of upper 30 m soil layer (Ambraseys et al., 1996).

Grade of soil stiffness	Site class	Shear wave velocity ($V_{s,30}$) (m/s)
Very low	Very soft	$V_{s,30} < 180$
Low	Soft	$180 < V_{s,30} < 360$
Medium	Stiff	$360 < V_{s,30} < 760$ (800)
high	rock	$V_{s,30} > 760$ (800)

The stiffness-related site classification is adopted by United States. The classification of the 1997-ICBO of the United States (ICBO, 1997) is done either regarding the average shear wave velocity, $V_{s,30}$ or the average standard penetration resistance, N_{SPT} , of the uppermost 30 m soil layer (Table 3.8).

Table 3.8: Site classification according to the 1997 Uniform Building Code (ICBO, 1997).

Site Class	Site Class description	Average soil properties for upper most 30 m of soil profile		
		Shear wave velocity, $V_{s,30}$ (m/s)	Standard penetration test, N (blows/ft)	Undrained shear strength, u_s (kps)
A	Hard rock, eastern United States sites only	> 1500	not applicable	not applicable
B	Rock	760 – 1500	not applicable	not applicable
C	Very dense soil and soft rock	360 – 760	$N \geq 50$	$u_s \geq 100$
D	Stiff soils	180-360	$15 \leq N \leq 50$	$50 \leq u_s \leq 100$
E	Soft soil, profile with more than 3 m of soft clay defined as soil with plasticity index $PI > 20$, moisture content $w > 40\%$	<180	$N < 15$	$u_s < 50$
F	Soils requiring site specific evaluations: 1. Soil vulnerable to potential failure or collapse under seismic loading, e.g. liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays: layers 3 m thick or more. 3. Very high plasticity clays: layers 8 m thick or more, with $PI > 75$. 4. Very thick soft/medium stiff clays: layers 36 m thick or more.	not specified	not specified	not specified

3.5.2 Stiffness-related-and-depth-related classification schemes

Stiffness-related site classification system is limited up to the uppermost soil layers, i.e. up to 30 m. The influence of the total soil thickness to the amplification characteristics of the site is not covered in stiffness-related site classification system. It is understood from the many years that the amplification of ground motion is depend on the predominant period or corresponding predominant frequency of the site. The predominant period of a site depends on the dynamic characteristics of the soil materials (e.g., shear wave velocity (V_s), and material density) as well as total thickness of the soil. Table 3.9 describes the site classification system in Turkish Earthquake Code (MPWS, 1998) which is based on the definition of different soil groups A-D. These soil groups further sub-divided into different group based on the total thickness of soil layer

Table 3.9: Site classification according to the Turkish Earthquake Code (MPWS, 1998).

Soil group	Site class description	Average soil properties		
		Shear wave velocity, v_s (m/s)	Standard penetration test, N (blows/ft)	Unconfined compression strength (kPa)
A	1. Rock (massive, unweathered, metamorphic, stiff cemented)	> 1000	-	> 1000
	2. Very dense sand, gravel	> 700	> 50	-
	3. Hard clay, silty clay	> 700	> 32	> 400
B	1. Soft/weathered rocks	700-1000	-	500-1000
	2. Dense sand, gravel	400-700	30-50	-
	3. Very stiff clay, silty clay	300-700	16-32	200-400
C	1. Highly weathered soft rocks	400-700	-	< 500
	2. Medium dense sand and gravel	200-400	10-30	-
	3. Stiff clay, silty clay	200-300	8-16	100-200
D	1. Soft, deep alluvial layers with high water table	< 200	-	-
	2. Loose sand	< 200	< 10	-
	3. Soft clay, silty clay	< 200	< 8	< 100

3.5.3 Hybrid classification schemes

The above soil classification systems mainly based on the geotechnical information about the soil materials. These required accurate information of depth of soil and other dynamic parameters such as shear wave velocity and modulus of rigidity of the soil layers in each site. These parameters can be acquired from geotechnical investigations, such as borehole exploration or seismic exploration method (i.e. seismic reflection/refraction method), which usually required high computational cost and time. Therefore, reliable seismic site assessment on the basis of these conventional procedures can only be performed if sufficient information about the local subsoil conditions is available. Therefore the hybrid site classifications system has been developed which consider the predominant frequency of site, f_r , as an additional differentiating factor. The predominant frequency of the site obtained from the analysis of seismic data or microtremor recorded at the ground surface can be used as an alternative site classification even when information on geotechnical sub soil are not sufficient to evaluate the site.

Bray and Rodriguez-Marek (1997), and Rodriguez-Marek et al. (2001) have illustrated a hybrid site classification scheme that meets both the characteristics and demands of a stiffness-and-depth-related classification system, as well as the specification of ranges for predominant frequency of the sites, f_r . Table 3.10 summarized the classification primarily based on the 1997-ICBO site classes into different soil depth. A further sub-classification of the soil classes is made by accounting for the depositional age (Holocene or Pleistocene) and the soils cohesive level.

Lee et al. (2001) have described the practical application of a hybrid site classification. They used response spectra and horizontal-to-vertical spectral ratio of earthquake records for classification of strong ground-motion sites in Taiwan. The normal response spectra were categorized based on mean acceleration spectral shapes from Seed et al. (1976) (Figure 3.15), while the results from HVSr were classified according to the regulation of the Japanese Code (Japan Road Association (JRA), 1980) (Figure 3.24).

Table 3.10: Classification of site based on 1997-ICBO site classes system.

Site	Site class description	frequency, f_r (Hz)	Comments
A	Hard rock	≥ 10.0	Hard, strong, intact rock: $v_s > 1500$ m/s.
B	Rock	≥ 5.0	Most “unweathered” California rock cases: $v_s \geq 760$ m/s or $H < 6$ m of soil.
C-1	Weathered/soft rock	≥ 2.5	Weathered zone: $H = 6 - 30$ m; $v_s > 360$ (700) m/s.
C-2	Shallow stiff soil	≥ 2.0	Soil depth: $H = 6 - 30$ m.
C-3	Intermediate depth stiff soil	≥ 1.25	Soil depth: $H = 30 - 60$ m.
D-1	Deep stiff holocene soil, either sand or clay	≥ 0.7	Soil depth: $H = 60 - 200$ m. Sand has low fines content ($< 15\%$) or non-plastic fines ($PI < 5$). Clay has high fines content ($> 15\%$) and plastic fines ($PI > 5$).
D-2	Deep stiff pleistocene soil, sand or clay	≥ 0.7	Soil depth: $H = 60 - 200$ m. See D-1 for sand or clay sub-categorization.
D-3	Very deep stiff soil	≥ 0.5	Soil depth: $H > 200$ m.
E-1	Medium depth soft clay	≥ 1.4	Thickness of soft clay layer: $H = 3 - 12$ m.
E-2	Deep soft clay layer	≥ 0.7	Thickness of soft clay layer: $H > 12$ m.
F	Special, e.g. potentially liquefiable sand or peat	≈ 1.0	Holocene loose sand with high water table ($z_w < 6$ m) or organic peat.

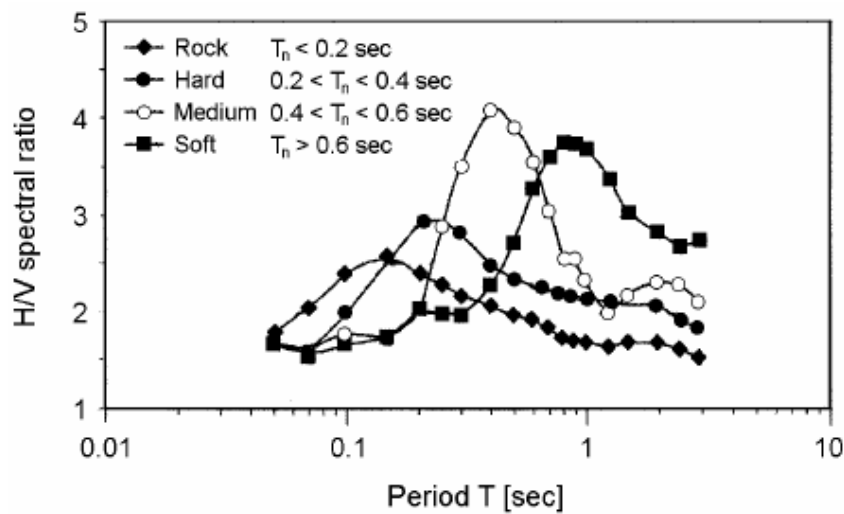


Figure 3.24: H/V spectral ratio for hybrid site classification as proposed by Lee et al., 2001 (figure taken from Lee et al., 2001).

3.6 CHAPTER SUMMARY

This chapter describes the strong ground motion, weak motion and microtremor, and their application in ground response analysis. Earthquake ground motion is a very complex. Single parameter is not enough to describe all important ground motion characteristics and hence different parameters, such as amplitude parameters, frequency content parameters, and duration parameters are essential for describing the important characteristics of a ground motion in compact and quantitative form.

Variation of ground motion due to subsoil characteristics can be investigated by different techniques and can be divided into two groups: 1) analytical technique and 2) instrumental technique. Analytical technique requires detailed information of subsoil, such as geotechnical parameters of each soil layers, while the instrumental technique requires measurement of data at the required site. The analytical techniques is good and can give comparatively accurate information of the site response, however it is time consuming, and required large number of input parameters which may not be possible while covering wider area. Therefore, instrumental method of site response evaluation is extremely used in different part of the world. Basically, two kinds of input data- seismic (strong and weak) data and microtremor are used in the instrumental method. Some of the widely used instrumental methods for site effect study are: spectral amplification function method, Fourier amplitude spectrum method, standard spectral ratio method, and horizontal-to-vertical spectral ratio method. In the absence of weak and strong motion records, the site response study has been successfully estimated with the microtremor survey.

The horizontal-to-vertical (H/V) spectral ratio method was introduced by Nagoshi and Igarashi (1971), and later improved by Nakamura (1989), and it became widespread as a low-cost and effective tool to estimate the fundamental frequency of sediments by measuring the microtremor in a single-station. Application of the H/V method using microtremor measurements is very important especially in the region like Nepal where big earthquake present a long return period but which exhibits a high seismic risk. Many experimental results have supported that the H/V spectral ratio exhibits a clear peak which is well correlated with the fundamental resonant frequency of the site. In last two decades, the H/V method has been widely used for various purposes, such as site effect evaluation, wave amplification estimation, liquefaction vulnerability assessment, sediment depth estimation, and microzonation studies in the different geographical and geological regions of the world.

The main objective of the site response evaluation is to classify the sites according to their geotechnical as well as geophysical parameters. The site characterization is very important not only for defining the seismic loads in the design of the infrastructures but also in different field, such as in seismic hazard assessment etc. The site classification system can be divided into three groups: 1) classification based on stiffness of soil layers, 2) classification based on stiffness and sediment depth, and 3) Hybrid classification schemes. The stiffness-related classification schemes are based on the soil parameters, i.e. stiffness parameters (N_{SPT} , V_s , G_{dyn}) of the uppermost 20 or 30 m from the surface. Stiffness and depth related classification is based on the similar soil layer stiffness but in different depths. The hybrid system of site classification is based on the principal of stiffness-and depth-related classification endorsed by the results of experimental site investigations. Example of this system includes horizontal-to-vertical spectral ratio from the microtremor data recorded at the ground surface.

CHAPTER 4

4 SEISMIC MICROZONATION

4.1 SEISMIC MICROZONATION: PRINCIPLES AND PRACTICES

A natural hazard is defined as the probability of the occurrence, within a specified period of time and a given area, of a particular, potentially damaging phenomenon of a given severity/intensity (DMTP, 1994). Hazard is a source for any kind of disaster. A hazard becomes disaster when it turns into injuries, loss of life and damage to the infrastructures and properties. In this context, seismic hazards represent the probable occurrence of earthquakes and seismically induced processes, which include ground motions, liquefaction and land-sliding. Geotechnical hazards are described as the influence of natural hazards on engineering objects. Earthquake hazard maps may include one or more of the aforementioned seismic hazards (Mihalić et al., 2011).

According to Mihalić et al. (2011), Seismic macrozonation includes delineation of the zones that are homogenous in seismological and geological characteristics, and a description of zone characteristics by associating dynamic parameters (peak ground acceleration- PGA, peak ground velocity- PGV, or spectral acceleration- SA) with the specified probability of occurrence. Therefore, seismic zonation is the first step for all further assessments of seismic hazards (Markušić and Herak, 1999). These parameters are mapped at a national scale for a standard ground condition, which are usually rock or stiff soil. Preparation of map at a national scale is called macrozonation (Finn et al., 2004). Building code utilizes national seismic macrozonation maps in specifying the minimum design requirements (DRM, 2004c).

There are two aspects of earthquake hazard safety: i) safety of the structure due to earthquake dynamic forces, and ii) safety of a site related to geotechnical phenomena, such as amplification, landslides, and liquefaction. Dynamic effects have been considered in building codes worldwide to ensure the safety of structures under earthquake loading. However, little attention has been paid to the safety assessment of individual sites in the form of land use regulation (ISSMGE, 1999).

Mapping of seismic hazards (local sources or distant sources) at local scales to incorporate the effects of local soil conditions is called seismic microzonation (Finn et al., 2004). Microzonation is the most accepted tool for seismic hazard assessment and risk evaluation (ISSMGE, 1999; Roca et al., 2008). According to Finn (1991) and Finn et al. (2004), seismic microzonation is a process of subdividing an area into zones of different hazard or risk level depending on the geological characteristics of the site so as to specifically assess or quantify the seismic hazard at different locations within an urban area or other area of interest. Microzonation map is a detailed map of a particular location which does not necessarily imply a scale of mapping. Seismic microzoning, including approaches for assessing local ground response, slope instability and liquefaction, has become a useful tool for cost effective earthquake risk mitigation.

ISSMGE (1999) has described few approaches to the evaluation of local site effects, which according to Mihalić et al. (2011) are considered the most important factor in the ground motion based zonation method. ISSMGE (1999) has suggested three grades of microzonation depending upon the method and data used.

Grade I method of zonation involves evaluating the local site effects using existing information that is readily available such as published reports (e.g. distribution of damage induced during past destructive earthquakes). Using this approach, the isoseismal maps of past destructive earthquakes can be prepared. The intensity increment map is then established based on the seismic intensity distribution. Geological condition of the site has often been used to interpret the observed intensity increment at each site.

Grade II methods of zonation require additional investigations such as geotechnical investigations, geophysical testing and soil sampling from boreholes for laboratory tests. If site investigation data is not available, indirect information on the site period may be obtained by means of microtremor measurements. Based on the results (i.e., predominant frequency and corresponding characteristic period) of the microtremor data analysis, the site is delineated into different zones.

In grade III methods of zonation require the conducting of ground response analysis (including the one-dimensional equivalent-linear and nonlinear analyses, and 2D and 3D analyses). Additional laboratory tests are necessary if equivalent-linear and/or nonlinear analyses are performed. SHAKE 2000 (Ordóñez, 2005) is the most-widely used computer program based on equivalent linear analysis. However, this method requires high computational cost and time.

4.1.1 Need of seismic microzonation

Seismic microzonation is the first step in earthquake risk mitigation study. It involves a very detailed field investigation to evaluate the hazard. The national seismic zonation map (Figure 4.1) which is prepared in small scales neglecting the factors, such as near field effects, focusing, topographical, and basin effect etc. which are important in assessing ground motion characteristics. The national hazard map does not consider the geological and geotechnical site conditions.

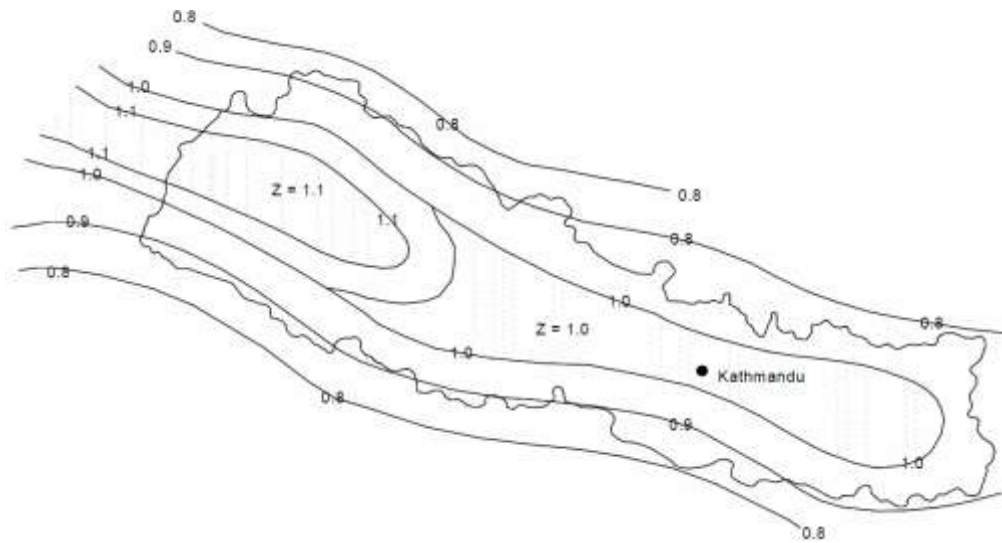


Figure 4.1: Seismic zonation map of Nepal, z represents the zoning factor (taken from NBC, 105)

Seismic microzonation map is very effective in delineating the spatial variations in the seismic hazard. More importantly, it is useful to evaluate the risk scenarios in the study area. The detailed seismic microzonation map may include different information about the area, such as seismic response of different surface geological formations, liquefaction potential, topographical amplification etc. Seismic microzonation map has a wider application in geotechnical earthquake engineering and urban planning. It can also be used as a tool for land use planning. The map helps to predict the impact of future earthquakes and also useful to plan the key facilities, such as hospitals, fire stations, emergency operation centers etc.

Nowadays, the importance of the seismic microzonation map has been increased significantly, especially for the developing cities of the developing countries like Nepal. Due to insufficient resources, like financial and technical, detailed site specific geotechnical assessment (such as borehole exploration) is difficult, time consuming and

uneconomical. Therefore, preparation of microzonation map using available economical tool is only the option for the mitigation of risk due to earthquake. Then, the prepared map can apply for the planning of new infrastructures and developing strategy for the improvement of the existing structures as well.

Some general examples of applications of microzonation map in land use planning can be summarized as below

- ◆ Identification of areas where lifeline system (i.e. water, sewer, gas, electricity) are most vulnerable to seismic hazards.
- ◆ Setting priorities for seismic upgrading or remediation of work.
- ◆ Selection of sites for essential facilities (e.g., hospitals, fire halls) or for high hazard structures (e.g. toxic waste containment facilities).
- ◆ Providing a basis for restrictions on development in high seismic hazard areas.
- ◆ Identification of areas requiring special study before development is allowed to proceed.
- ◆ Assessment of the relative seismic hazard-related risks of a project to be insured or financed.
- ◆ Providing information on site effects to be accounted for in design of new structures.
- ◆ Establishing more stringent design requirements for certain types of structures on site with high seismic hazard levels.

4.1.2 Seismic Microzonation in the Kathmandu Valley

As mentioned in Chapter 2, Kathmandu Valley lies in an active seismic region in the Lesser Himalayan zone which is built over soft soil layers of lacustrine origin, and has a repeated history of frequent earthquake damage. Fujii and Sakai (2002), Moribayashi and Maruo (1980), Saijo and Kimura (2007), Sakai (2001), Sakai et al. (2002), Sakai et al. (2001), and Yoshida and Igarashi (1984) have studied the geology and stratigraphy of the sediment deposits. Some other researchers such as Dahal and Aryal (2002), Katel et al. (1996) have studied the engineering geological and geotechnical properties of the Kathmandu lacustrine sediment

Although a few studies have attempted to address the problem of earthquake hazard, such as Pandey (2000), Dixit et al. (1998, 2000), and JICA (2002), most studies focus more on a generalized approach to the earthquake hazard scenario in the Kathmandu Valley, while an in-depth study in relation to the influence of the sparsely distributed depth of

sediment deposits and the local variation in ground response to the structural damage is considered more important, keeping in mind the adoption of a structural design code and mitigation of expected disaster. One of the ways to address this issue is to develop a seismic microzonation map for the valley. Owing to the inadequate or sparsely distributed subsurface geophysical exploration points in the valley, which has led to a lack of geotechnical information for a ground response analysis, such a map of the whole valley has yet to be prepared. The study of the variation in ground motion according to the geological conditions using the seismic microzonation method is expected to assess appropriately and quantitatively the earthquake hazard risk in the valley.

Although there were many subsurface borehole explorations in the valley as mentioned in Chapter 2, they have lack of geotechnical data for ground response analysis. Moreover, borehole exploration based ground response analysis is uneconomical for the densely populated urban area like Kathmandu Valley and required much time for data analysis. This method also has environmental problem as borehole need to be drilled for data taken. Due to infrequent earthquakes, it is rarely possible to collect the data on the required sites and hence the ground response analysis, based on the analysis of earthquake data is not possible for the Kathmandu Valley. Therefore, the only alternative method using microtremor measurements and its analysis is an appropriate for the seismic microzonation of the Kathmandu Valley.

Recently, many researchers have applied H/V spectral ratio method and recordings of ambient noise in seismic microzonation of various urban areas in the world situated over soft soil deposits like the Kathmandu Valley, for instance, microzonation of south east Tehran (Jafari et al., 1996); microzonation of Thessaloniki Greece (Lachet et al., 1996); microzonation of Bogota, Columbia (Cardona and Yamin, 1997); seismic microzonation of Rhone Delta (South of France) (Bour et al., 1998); microzonation of Acapulco (Chavez-Garcia and Cuenca, 1998); microzonation of Almeria (Southern Spain) (Navarro et al., 1998); microzonation of Odawara, Japan (Midorikawa (1998); microzonation the city of Tunis (Bouden-Romdhane et al., 2000); microzonation of Delhi, India (Mukhopadhyay et al., 2002); microzonation of Shin-Yokohama area , Japan (Mirzaoglu and Dykmen, 2003); Seismic microzonation in Pointe-a-Pitra (French West Indies) (Lebrun et al., 2004); seismic microzonation of the Bangkok, Thailand (Tuladhar et al., 2004); microzonation of the city of Catania, Italy (Grasso and Maugeri, 2009); seismic zonation and microzonation studies in India (Walling and Mohanty, 2009); and microzonation of Barcelona, Spain (Cadet et al., 2011).

4.2 MATERIALS AND METHOD

4.2.1 Area covered

The study area covers the densely populated core urban areas of the three main cities in the Kathmandu Valley, i.e. Kathmandu, Lalitpur, and Bhaktapur, as shown in Figure 4.2, and measures about 210 square kilometers. With an average ground elevation of 1340 m from mean sea level, the Kathmandu Valley measures about 25 km east to west and 20 km north to south, and is the most prominent human settlement basin in the mid-Himalayan Mountains. The population density in the urban parts of the valley is about 13,225 per square kilometers, which makes it the largest urban agglomerate in Nepal (KMC, 2011). The location of the microtremor observation points are shown in Figure 4.2. The details of the study area (Kathmandu Valley) are given in Chapter 2.

4.2.2 Microtremor measurement device

Table 4.1 and Figure 4.3 show the set of the instruments used for the measurements of microtremor in the Kathmandu Valley. Table 4.1 also shows the parameters of recording and data processing for the horizontal-to-vertical (H/V) spectral ratio method on microtremor.

Table 4.1: Detailed equipment, parameters of recording and data processing for horizontal-to-vertical spectral ratio method on microtremor data.

Parameter, setting	Type of noise recording
Type of seismic sensor	Velocity sensor NewPIC
Size of sensor	9cm × 9 cm × 6 cm
Sample frequency	100 Hz
Recording duration	~5 min (300 seconds)
Windowing (length of FFT-window)	2048 values (20.48 sec)
Smoothing (of spectral data)	Parzon window
Calculation of H/V-ratio	H(EW)/V(UD) H(NS)/V(UD) (H(EW)+H(NS))/V(UD)
Averaging of H/V-ratios	Mean value of several H/V spectral ratios
Data acquisition system	
Global Positioning System (GPS)	

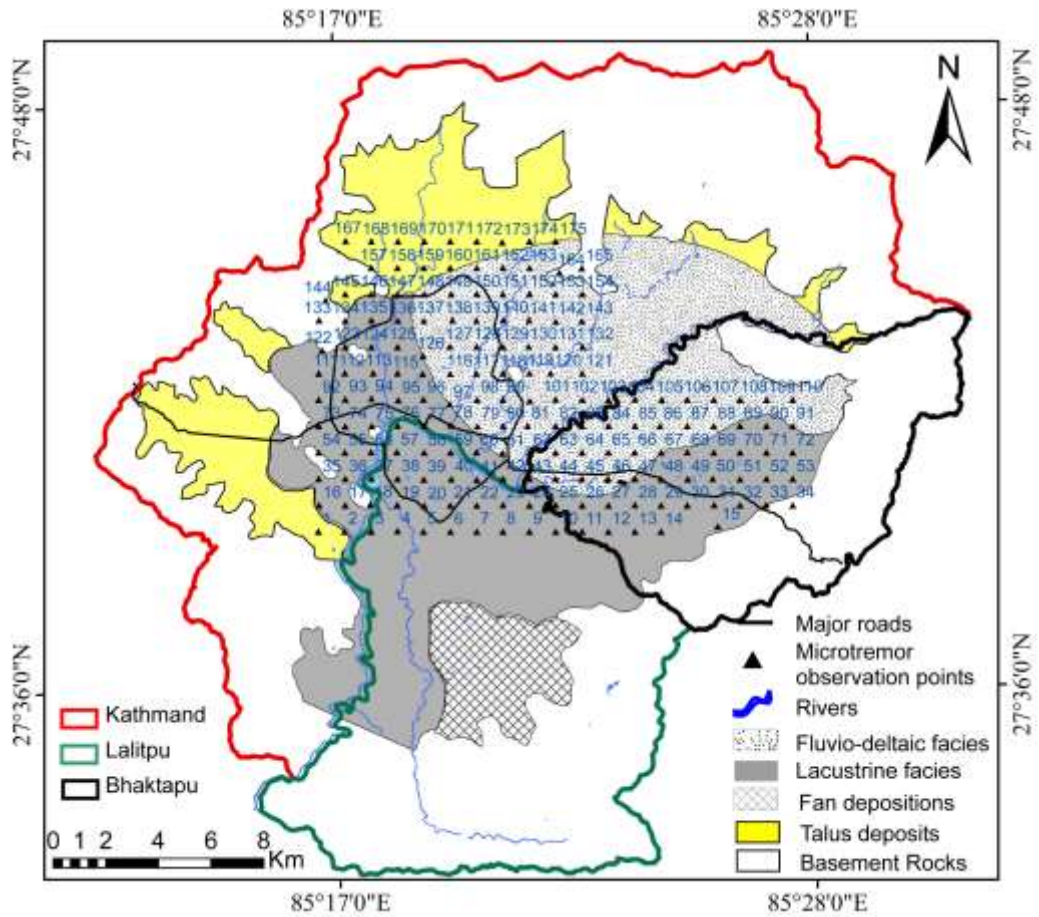


Figure 4.2: Map of study area and microtremor observation points.



Figure 4.3: Instruments used for the microtremor survey in Kathmandu Valley.

4.2.3 Data acquisition and analysis

4.2.3.1 Sensor location

For microtremor data acquisition, approximately 1 km square grid points in the study area were first plotted over a topographic base map (scale 1:25,000) of the Kathmandu valley in a GIS environment so that each grid point could serve as a microtremor observation site, as shown in Figure 4.2. The grid point data layer was then exported as a point overlay from the GIS environment to Google Earth (2010) as shown Figure 4.4 so as to pinpoint the exact locations of the microtremor observation sites in the study area with reference to road crossings, school buildings, houses, private boundary walls, playgrounds, and open spaces. The Google Earth information was then used in preparing a location map (refer Figure 4.5) of each observation point along with the predetermined geographic coordinates and elevation. The map-to-ground technique employed in this study enabled us to locate the grid points precisely.

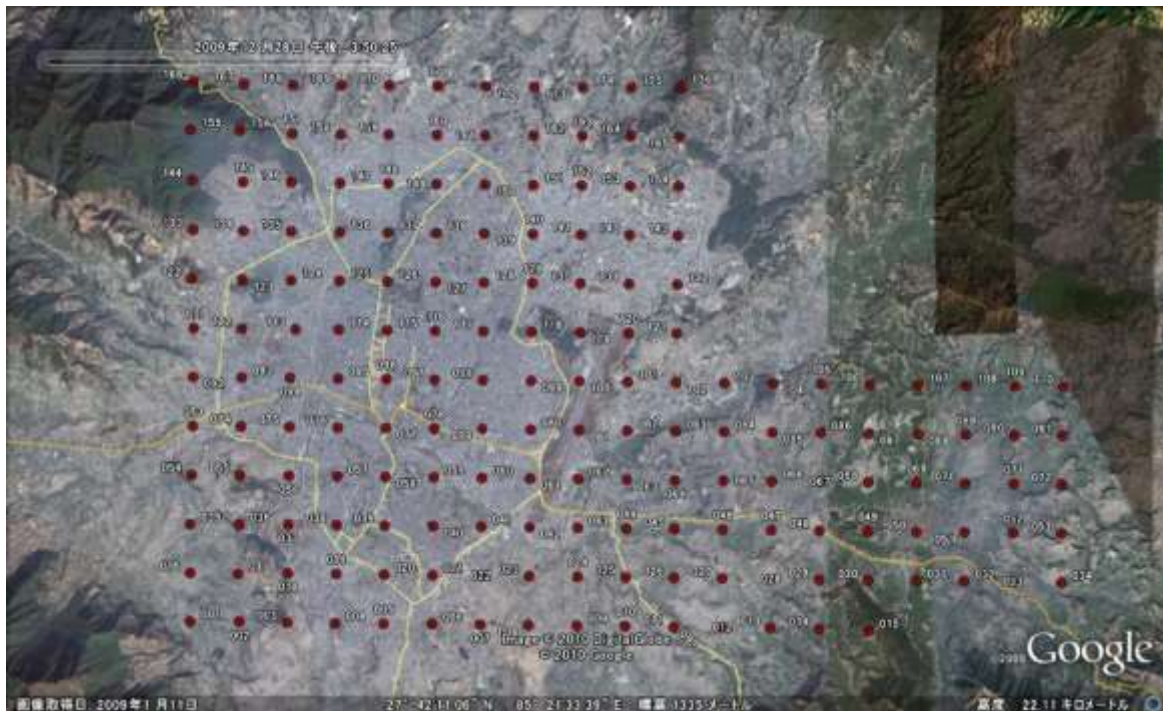


Figure 4.4: Microtremor observation points on study area (Kathmandu Valley) on Google Earth (2010) Image.



Figure 4.5: Typical site maps for microtremor observation in the study area, red points represent the microtremor survey stations.

Therefore, by using site map and pre-determined GPS coordinates, microtremor observation sensor was placed in the exact location as shown in the Figure 4.4 as far as possible. The observation points should be far from any influential surrounding structures such as tall buildings, electric pole, trees, etc. The existing underground engineering structures such as drainage, sewer line, electric storage etc. were avoided as far possible during selecting location of sensor since they could influence the microtremors and hence ground response.

4.2.3.2 Microtremor observation

Microtremor observation was done in the pre-determined location as far as possible using a portable velocity sensor NewPIC, manufactured by System and Data Research (SDR) Co. Ltd., Japan, as mentioned in Section 4.2.1. This device is capable of recording three components of vibration: two horizontally, i.e., east-west and north-south and one vertically. Two orthogonal horizontal components were setup in north-south and east-west directions at an observation point. After placing sensor at specified point, the sensor was oriented and leveled in specified direction with the help of leveling screws of sensor and compass. The connectivity and sensitivity of sensors were checked before starting the recording of ground motions. At each point, the microtremor data were recorded for 300 seconds at a sampling frequency of 100 Hz (i.e., 30,000 samples at each point). A single recording might be enough if there were not any influences on recorded data by traffic, or nearby artificial noise sources, but more than one time recordings were conducted when the recorded data was unsatisfactory. A global positioning system (GPS) was used for recording the coordinates of observation points. The velocity sensor is very sensitive instrument, so the vibration from traffic especially large vehicles even from the long distances from the measurement point can be easily recorded. Therefore, the condition of

vehicle movement and near walking pedestrian during measurements time was monitored and recorded in a memo book. The flow chart of the microtremor observations in the study area is shown in Figure 4.6. Few pictures of microtremor survey in the study area (Kathmandu Valley) are shown in Figure 4.7.

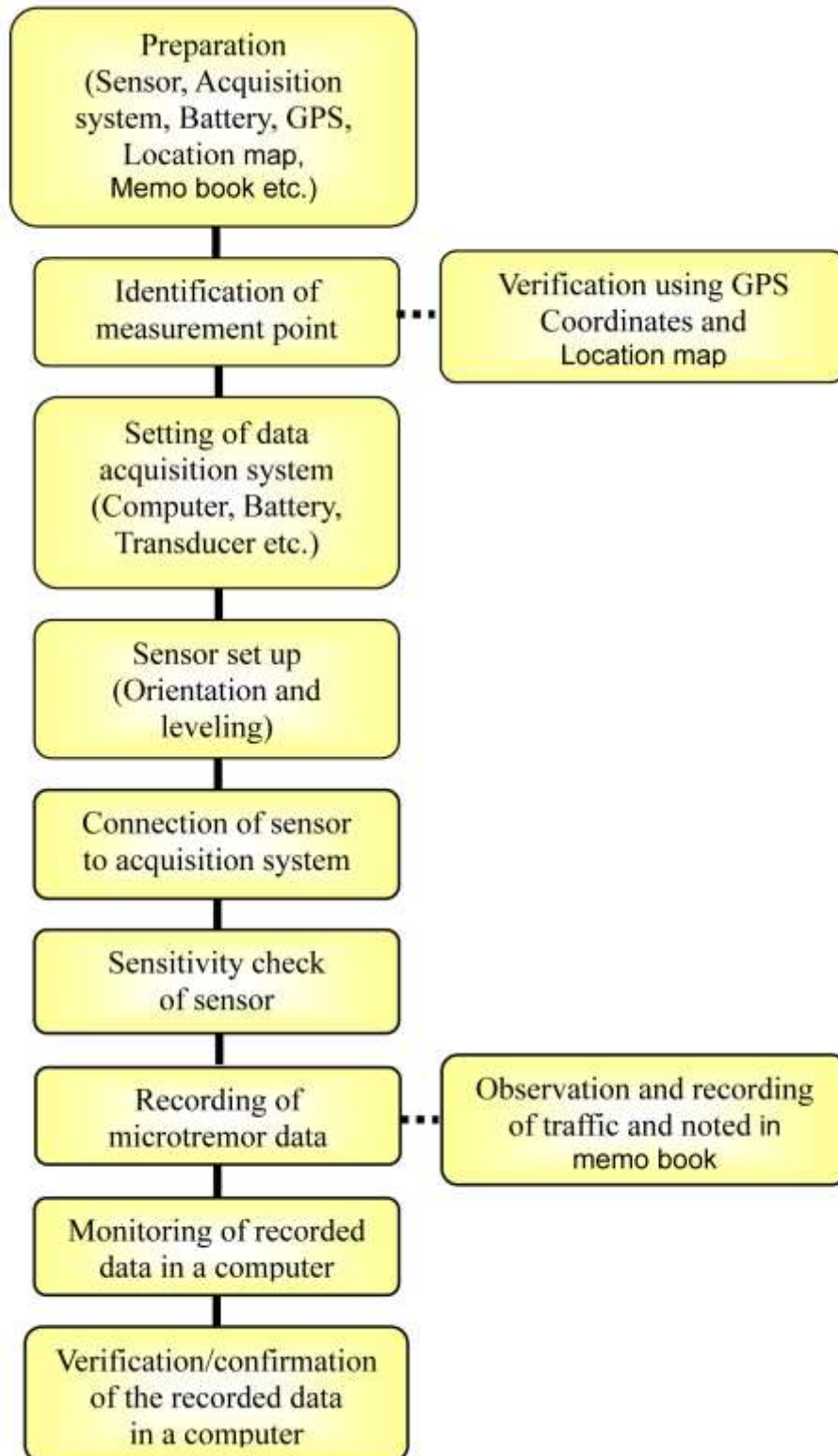


Figure 4.6: Flow chart of the microtremor measurement in the field.



Figure 4.7: a) Microtremor measurement in open field; b) measurement inside PM Quarter c) measurement during night; d) interaction with local people during microtremor survey.

4.2.3.3 Analysis of data

Except for inaccessible grid points, the microtremor observation was possible at a total of 172 grid points in the demarcated area as shown in Figure 4.2. All the recorded data were then plotted in terms of velocity-time histories, and each component of the recorded signal was corrected by the base line and divided into 15 windows of 2048 samples (i.e., equivalent to 20.48 s) in each of them, as illustrated in Figure 4.8. After reviewing the plotted time history data, and field memo book, the segments that were influenced by transient signals (i.e., nearby heavy traffic) (refer Figure 4.8) were removed. The selected data were combined to get the analysis data. In this process, we tried to make as many segments as possible for the analysis data. Figure 4.8a, b and c show the velocity time history of north-south, east-west and up-down components of microtremors respectively. Here, the time history graphs were divided into windows of 20.48 seconds. In the measured data only one segment was influenced by transient singles and hence in total

14 segments that were relatively noise less segments was used for Fourier analysis. In general, in this study, 10-14 windows with little or no noise record were considered in the Fourier analysis, which was carried out separately for each window using the Fast Fourier Transform (FFT) computer program provided by Associate Professor S. Mori, Ehime University to obtain Fourier spectra. The Fourier spectrum of two horizontal microtremor motion and one vertical motion were obtained and H/V spectral ratio was obtained after dividing a horizontal by vertical component of the obtained spectrum. Then, the obtained spectra were smoothed by the Parzen window of bandwidth 0.5 Hz. Figure 4.9a, b, and c show the Fourier spectra of measured microtremor in north-south, east-west, and up-down direction before smoothening respectively. Similarly, Figure 4.9d, e, and f shows the Fourier spectra of measured microtremor after smoothing. After getting Fourier spectrum and H/V spectral ratio, the ensemble averaging was conducted to get average of all segments using Equation (4.1) (Delgado et. al., 2000).

$$H/V = \sqrt{(F_{NS}^2 + F_{EW}^2)/(2F_{UD}^2)} \dots\dots\dots (4.1)$$

Here, F_{NS} , F_{EW} and F_{UD} are the Fourier amplitude spectra in the north-south (NS), east-west (EW) and vertical (UD) directions, respectively.

Figure 4.10a and b show the horizontal-to-vertical spectral (H/V) ratio versus frequency graph in north-south and east-west direction before smoothing and Figure 4.10c, and d show the same after smoothing. The flow chart of the data acquisition is shown in Figure 4.11.

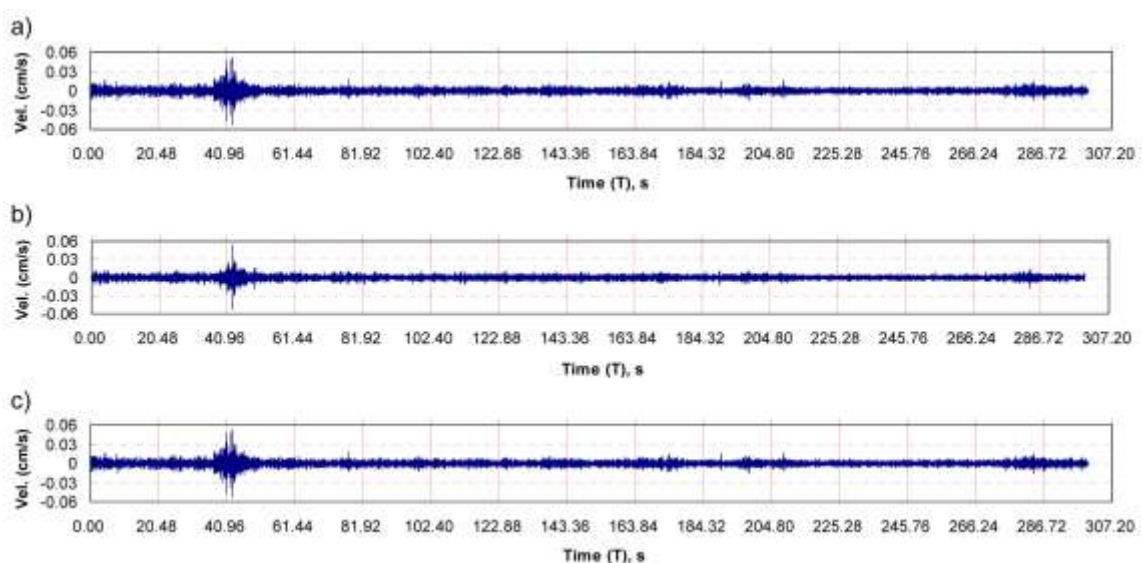


Figure 4.8: Typical pattern of measured microtremor data; a) in east-west direction (X-axis); b) in north-south direction (Y-axis); c) in up-down direction (Z-axis).

Based on the fact that the frequency corresponding to the first peak of the H/V spectrum plot represents fundamental frequency of the site (Bonney-claudet et al., 2006b; Field and Jacob, 1993; SESAME, 2004) which is also a minimum frequency in the H/V spectrum, the site specific fundamental frequency and the corresponding predominant period for each measurement point were obtained. Sometimes, the averaged H/V spectral ratio has apparent peak due to magnification by extraordinary magnitude of pin pointed noise at specified frequency with frequency shift from true peak. To avoid such kind of apparent miss judgment of shifted peak, the smoothened H/V spectrum ratio was compared with raw H/V spectrum. Figure 4.10 shows the H/V spectrum before smoothing and after smoothing. We can see the graph is rough, and is exhibiting many unclear peaks, which may make confusion to obtain the exact value of predominant frequency. In order to overcome this type of miss judgment, the obtained results were smoothened using Parzen window of bandwidth 0.5 Hz.

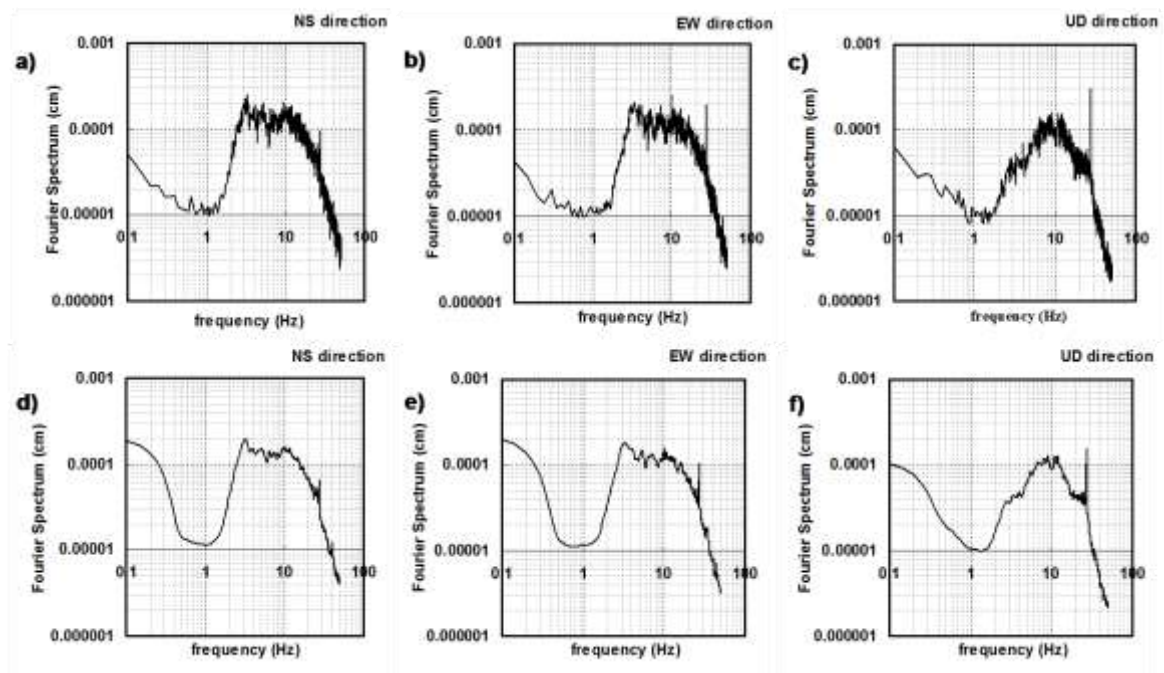


Figure 4.9: A typical Fourier spectrum graph of microtremor measurements in Kathmandu Valley, a) - c) Fourier spectrum before smoothing, d) – f) Fourier spectrum after smoothing using Parzen window of bandwidth 0.5 Hz.

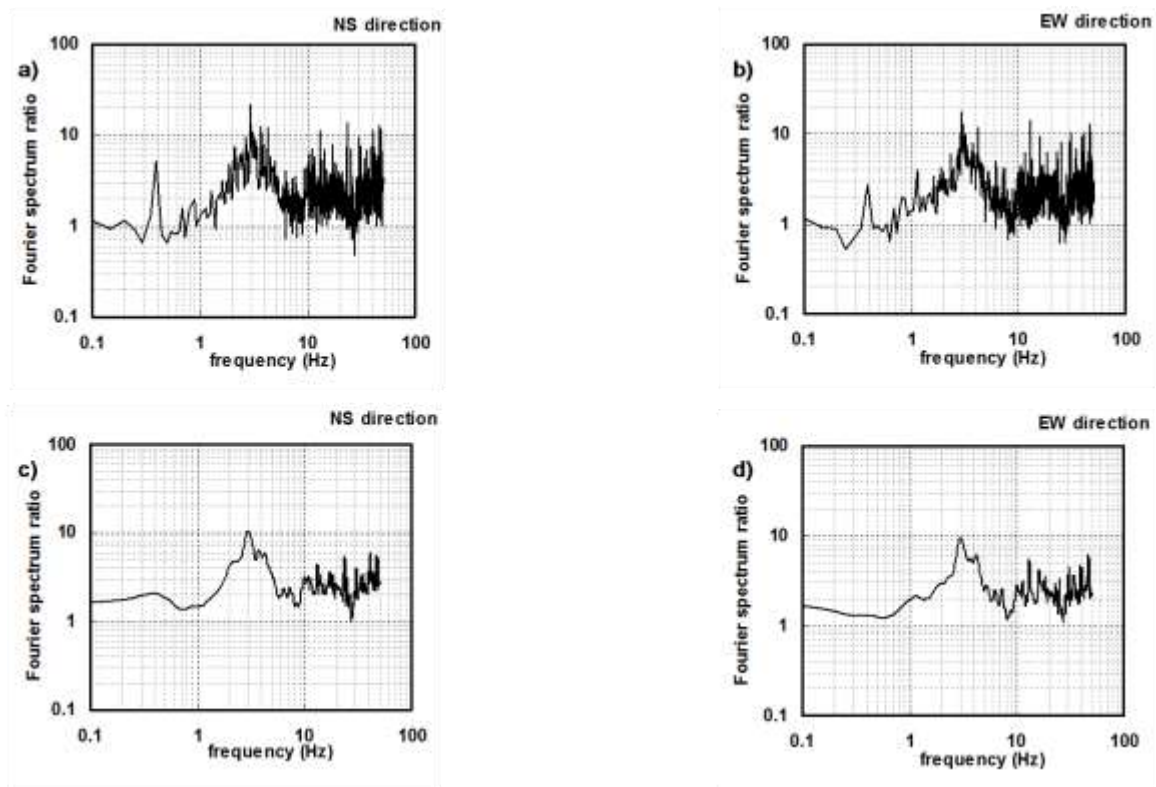


Figure 4.10: Typical horizontal-to-vertical spectral ratio in the Kathmandu Valley, a) and b) Horizontal-to-Vertical spectral ratio before smoothing, and c) and d) horizontal-to-vertical spectral ratio after smoothing using Parzen window of bandwidth 0.5 Hz.

Then, these point-specific data were used in GIS environment in order to see the spatial distribution of the dominant periods over the Kathmandu Valley. For this, the Inverse Distance Weighted (IDW) method of interpolation was employed owing to its simplicity and used in similar kind of works in Tuladhar et al. (2004). After that, based on the variation of the dominant periods, the whole survey area was divided into different period ranges in order to prepare an approximate microzonation map out of the estimated dominant periods in the GIS platform.

Finally, in order to understand the variation of the dominant period and to obtain a clear picture of the subsurface structure in the study area, ground profiles through different grid point were plotted. The obtained profiles were then compared for verification purpose with the sediment distribution pattern of the Kathmandu Basin interpreted by Sakai et al. (2002).

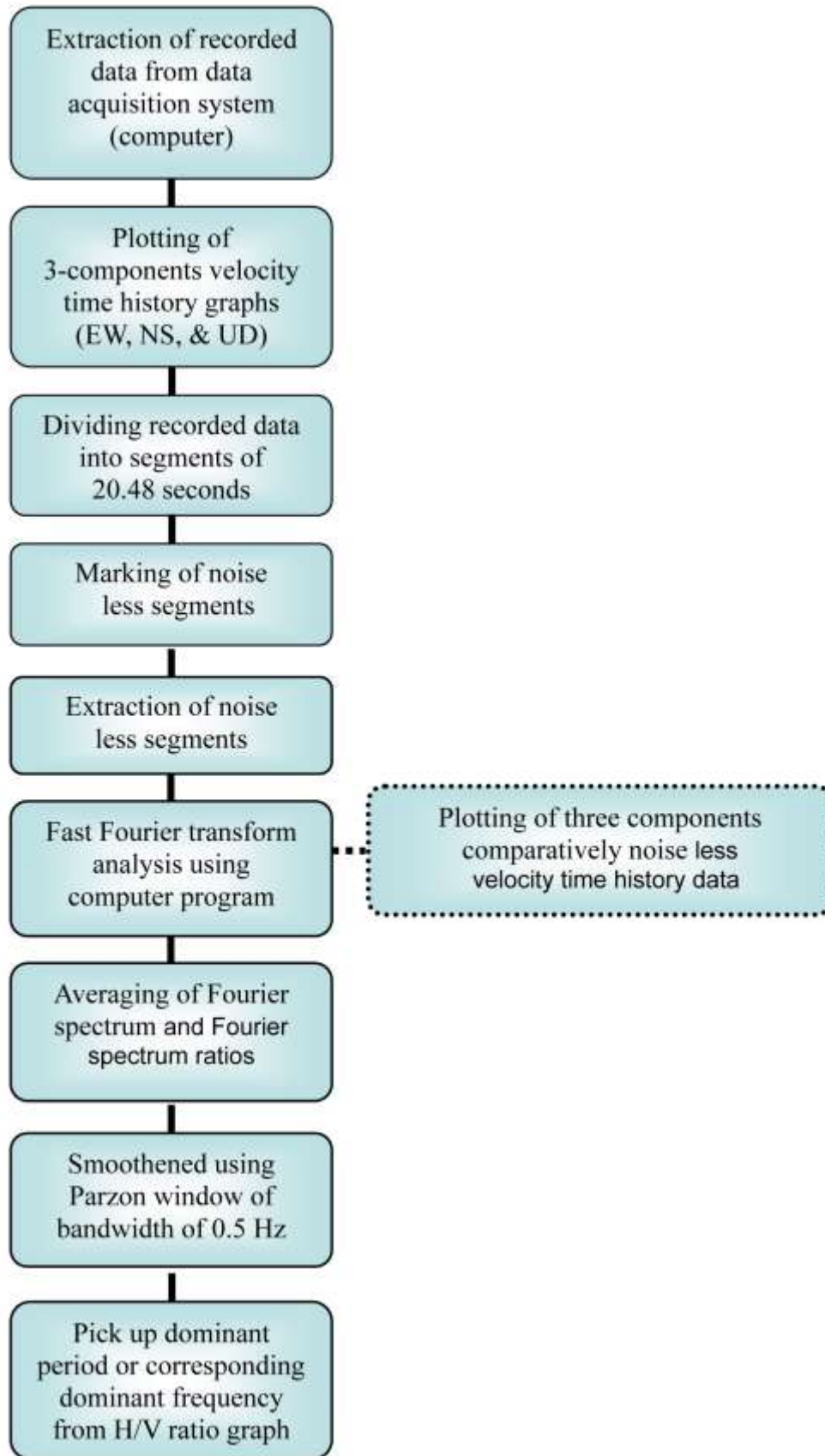


Figure 4.11: Flow chart of the microtremor data analysis.

4.3 RESULTS AND DISCUSSION

4.3.1 Seismic microzonation

The results of the microtremor survey are expressed in terms of resonance frequency and corresponding dominant period of the ground at each survey point, obtained from the H/V spectral ratio analysis, as typically shown in Figure 4.12a and b. Although there is noticeable scatter these curves, the dominant period can usually be estimated uniquely from the maximum smoothed peak (inverted black triangles in Figure 4.12a and b. The dominant periods of all measurement points obtained in this way are listed in Table 4.2. The data in the table indicate that the dominant period in the surveyed area varies from 0.11 s to 2.05 s. Based on this variation, the study area is divided into five different ranges grouped by dominant period (i.e., Period range A to Period range E), as listed in Table 4.3, using the natural breaks technique, which is used for this purpose because this method regroups similar values together and represents the distribution appropriately. Then, as shown in Figure 4.13, the H/V spectral ratios for all measurement points in each range of dominant period were separated on the basis of above division. Finally, an approximate seismic microzonation map based on the dominant period of shaking was prepared as in Figure 4.14 and Figure 4.15. These two dominant period distribution maps indicate that the central part of the Kathmandu valley falls under Period range D and Period range E, which cover about 30 percent of the urban area of the valley while the outer parts fall under Period range A to C.

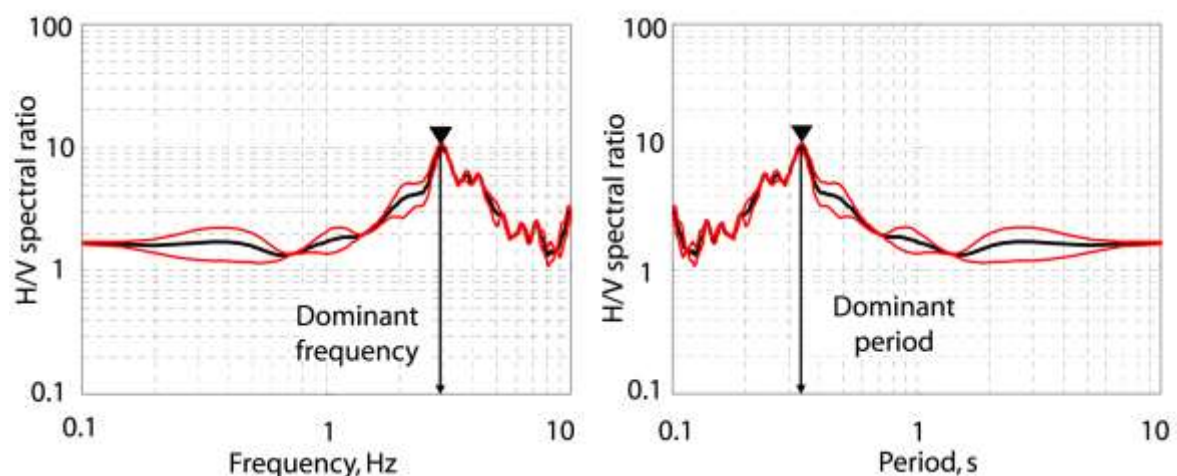


Figure 4.12: A typical horizontal to vertical spectral ratio graphs in the study area; a) frequency graph, b) period graph. The black line represents the mean H/V ratio and the red lines represent the mean $\pm$ standard deviation.

Table 4.2: Dominant periods in the study area.

Microtremor observation point	Dominant Period (s)	Microtremor observation point	Dominant Period (s)	Microtremor observation point	Dominant Period (s)	Microtremor observation point	Dominant Period (s)
1	0.91	44	0.77	87	0.98	130	1.00
2	0.50	45	1.08	88	0.67	131	0.83
3	0.17	46	0.76	89	0.60	132	1.11
4	0.77	47	0.59	90	1.11	133	0.11
5	0.79	48	0.26	91	1.11	134	1.11
6	0.83	49	0.77	92	0.91	135	0.83
7	1.09	50	1.20	93	1.08	136	1.00
8	1.05	51	1.20	94	1.43	137	1.33
9	0.82	52	0.79	95	1.28	138	0.50
10	0.82	53	1.25	96	1.47	139	1.43
11	1.00	54	0.51	97	1.20	140	0.82
12	1.28	55	0.91	98	1.08	141	1.11
13	0.68	56	0.50	99	1.20	142	0.93
14	0.89	57	1.00	100	1.37	143	0.83
15	0.33	58	0.80	101	0.60	144	0.33
16	1.20	59	1.20	102	0.77	145	1.00
17	0.24	60	1.15	103	1.11	146	0.33
18	1.25	61	0.67	104	1.15	147	1.28
19	0.64	62	1.10	105	1.15	148	1.25
20	0.82	63	0.83	106	0.71	149	1.37
21	0.90	64	0.50	107	0.69	150	1.25
22	1.00	65	0.89	108	0.70	151	1.05
23	0.83	66	0.77	109	0.50	152	0.66
24	1.00	67	1.11	110	0.83	153	1.00
25	0.91	68	0.91	111	0.70	154	1.02
26	0.76	69	0.83	112	0.33	157	0.74
27	1.28	70	0.71	113	1.11	158	0.83
28	0.50	71	0.71	114	1.58	159	1.00
29	1.11	72	0.98	115	1.71	160	1.25
30	0.43	73	0.53	116	2.05	161	0.73
31	1.6	74	0.79	117	1.71	162	0.77
32	0.66	75	0.83	118	0.64	163	0.35
33	0.54	76	0.82	119	0.62	164	0.60
34	0.64	77	1.0	120	0.83	165	0.33
35	0.77	78	1.11	121	0.82	167	0.66
36	0.40	79	1.14	122	0.33	168	0.23
37	1.43	80	1.20	123	0.36	169	0.27
38	0.91	81	0.98	124	0.91	170	0.85
39	0.76	82	0.68	125	1.86	171	1.00
40	1.25	83	0.33	126	1.28	172	0.13
41	1.11	84	0.30	127	1.20	173	0.33
42	0.76	85	0.82	128	1.47	174	0.71
43	0.91	86	1.20	129	0.33	175	0.82

As seen also in Figure 4.14 and Figure 4.15, the dominant period for the central core and its peripheral areas in the Kathmandu basin ranges from 1.0 s to 2.0 s; then, it gradually decreases outward and reaches minimum in the outskirts areas of the valley, which infers that the Kathmandu core city area has a considerably long dominant period in comparison to other parts. The higher period range in the central part can be related with the extent of depth of Pliocene and Quaternary deposits while the lower period in the outskirts to shallow stiffer deposits or a rocky area on the outer side. An abrupt decrease in the period can be found at a few points (Figure 4.14), such as microtremor observation point 138 (period: 0.5 s, 2 km northeast of the central part) and point 129 (period: 0.3 s, 3 km east of the central part), which may be due to the presence of rock strata at a shallow depth. The lowest dominant period of 0.11 s is located at point 133 outside of the basin, about 6 km northwest from the center of the valley.

4.3.2 Analysis of period variation

In order to analyze the variation in dominant periods across the Kathmandu Basin, the dominant period profiles through each microtremor observation points in west-to-east and south-to-north directions are plotted as shown in Figure 4.16 and Figure 4.17 respectively. This can be related with the higher variability in sediment thickness and its properties in the Kathmandu basin. The dominant period of a ground is a function of shear wave velocity and thickness of the sediments. Due to lack of enough and reliable borehole or geophysical exploration data in the Kathmandu valley, the geotechnical characteristics of the soft sediments are not adequately known in this area. The shear wave velocity profile up to the basement rock is also not available. As a result, the correlation of dominant period with sediment depth is also not possible. However, a few available borehole data indicate that the basin soil is complex in terms of thickness and soil types, and the sediment thickness has been recorded as deep as 550 m in the central part of the basin (Sakai et al., 2002; Fujii and Sakai, 2002). A longer period is usually obtained over a thick soil deposit area, whereas a short period over a shallow deposit area. Our observations in the Kathmandu valley are in well acceptable agreement with the theoretical consideration as well as the observations made in different parts of the world that the dominant period increases with the increase in sediment thickness, as described by Al-Qaryouti and Al-Tarazi (2007), Dinesh et al. (2010), Ibs-von Seht and Wohlenberg (1999), Özalaybey et al. (2011), Parolai et al. (2004), SESAM (2004).

In the next step, in order to analyze the variation of dominant period in the central part of the study area, four linear surface profiles were prepared from the observations (Figure 4.14). Profile S4 – N4 (Figure 4.18) passes through Pulchowk (P 58), Thapathali (P 77), the Tri-Chandra Campus (TC) of Tribhuvan University (P 115), and Ministry of Foreign Affairs (MOFA) (P 126) (the former Royal Palace). In this section, the longest periods are located near the Tri-Chandra Campus (TC). Profile S5-N5 (Figure 4.19) passes through the Patan Industrial Area (PIA) (P 21), Singha Darbar (P 97) (the central administration complex of the Nepal Government), Dillibazar (P 116) (commercial as well as old residential area), and the Prime Minister's Quarter (PMQ) (P 138). In this section, the longest dominant periods are found at Dillibazar. Similarly, profile W5-E5 (Figure 4.20) passes through Kalimati (P 94) (commercial area), Singha Darbar (P 97), and Sarswotikhel (P 105) (Bhaktapur) in an east-west direction. A greater variation in the dominant periods is found along this section but longer periods belong to the central part. Similarly, Figure 4.21 shows the profile W6-E6, passes through Ashon (P 114) and Kathmandu's Darbar Square area, a commercial area and the old core Kathmandu city (a UNESCO cultural heritage site), the Tri-Chandra Campus (TC) (P 115), Dillibazar (P 116), and the Pasupati Nath Temple area (P 118) (UNESCO cultural heritage site). This figure also demonstrates that the dominant period increases towards the center of the city and reaches a maximum in the Dillibazar area.

In comparison with other parts of the valley, the central part was analyzed minutely as it consists of very important places, as highlighted in Figure 4.14 and mentioned elsewhere in this paper. The old residential area in the central part has been changing into the urban core, which mostly consists of historical cultural heritage sites and commercial areas with narrow streets and few open spaces. The dominant period of buildings here may be increased during earthquake shaking due to poor structural conditions and soil-structure interaction phenomenon (Clinton et al., 2006; Pirner et al., 2009). Hence, the chances of occurrence of resonance between ground and buildings are very high as ground dominant period is longer in this area.

As a cross check to verify the reliability of the microtremor data, the dominant period ground profiles in Figure 4.18 to Figure 4.21, were compared with the borehole exploration-based approximate ground profile prepared by Sakai et al. (2002) (Chapter 2, Figure 2.25). The two methods yield similar results: longer periods are found towards the center of the valley where sediment thicknesses are known to be deeper.

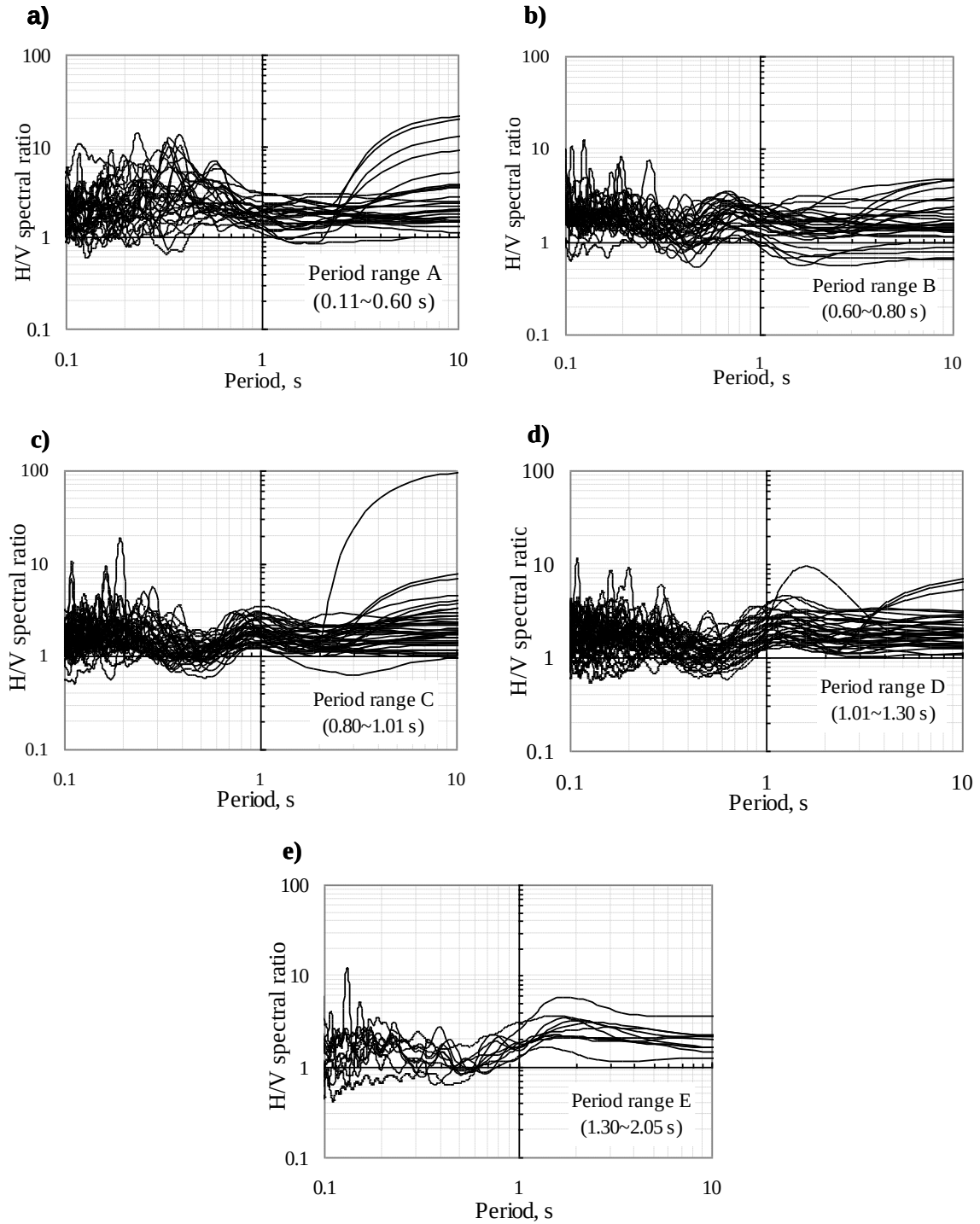


Figure 4.13: Horizontal-to-vertical spectral ratio of microtremor for different period ranges in the study area; a) Period range A: dominant periods from 0.11 s to 0.60 s; b) Period range B: dominant periods from 0.60 s to 0.80 s; (c) Period range C: dominant periods from 0.80 s to 1.01 s; (d) Period range D: dominant periods from 1.01 s to 1.30 s; (e) Period range E: dominant periods from 1.30 s to 2.05 s.

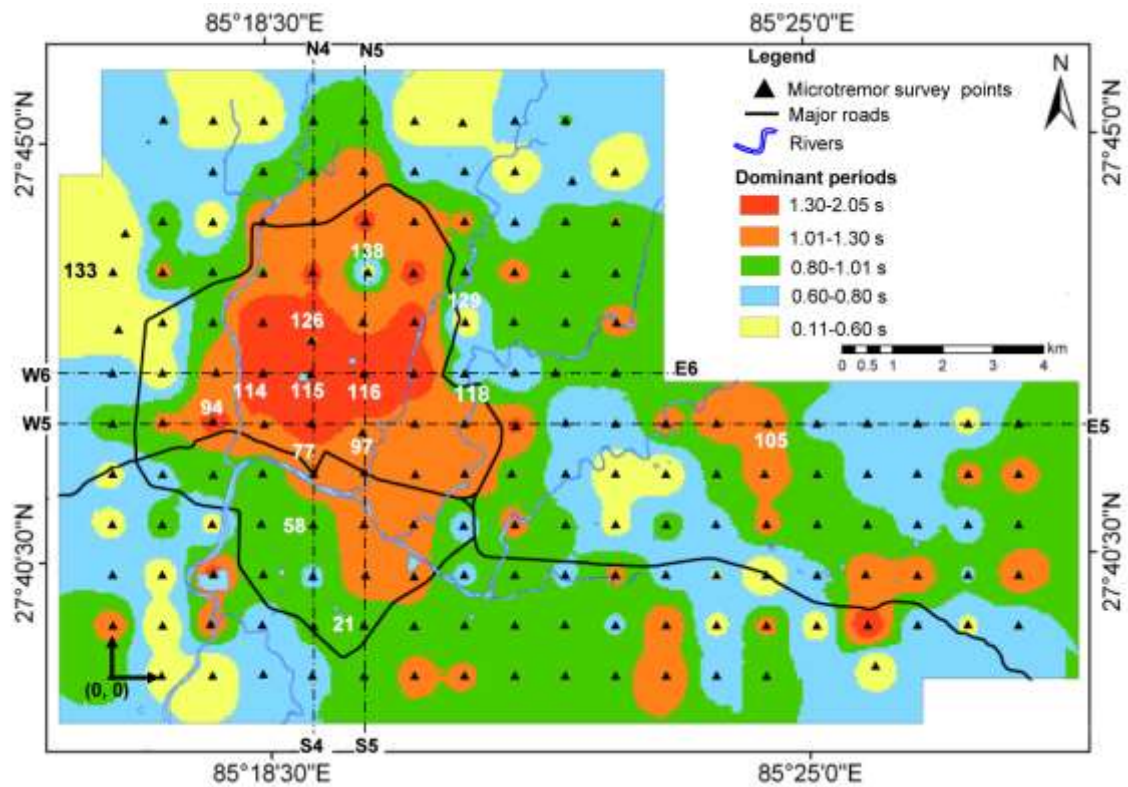


Figure 4.14: Estimated dominant period of shaking map (or Approximate Seismic Microzonation map) of the Kathmandu valley based on the variation of dominant period of ground. Lines S4-N4, S5-N5, W5-E5, and W6-E6 represent locations of typical cross-sections through the valley core (Figure 4.18-Figure 4.21). Various important places in the Kathmandu valley are also indicated by white color in the map.

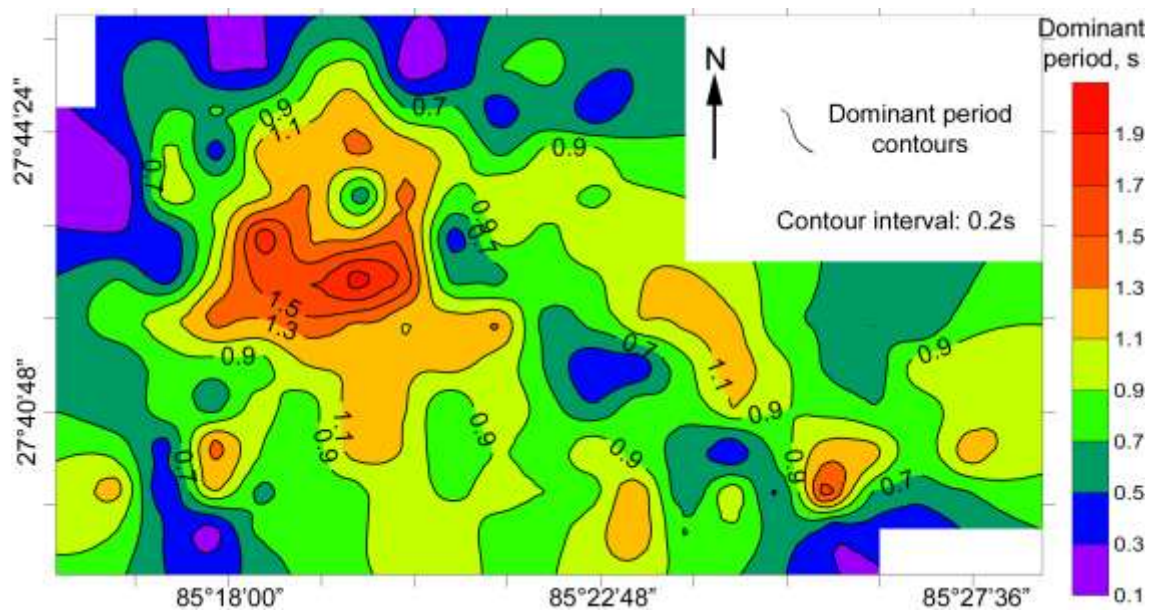


Figure 4.15: Dominant period contours for the Kathmandu valley.

Table 4.3: Classification of study area based on variation of dominant periods of ground.

Dominant period range	Description of zone
A	Period ranging from 0.11 s to 0.60 s
B	Period ranging from 0.60 s to 0.80 s
C	Period ranging from 0.80 s to 1.01 s
D	Period ranging from 1.01 s to 1.30 s
E	Period ranging from 1.30 s to 2.05 s

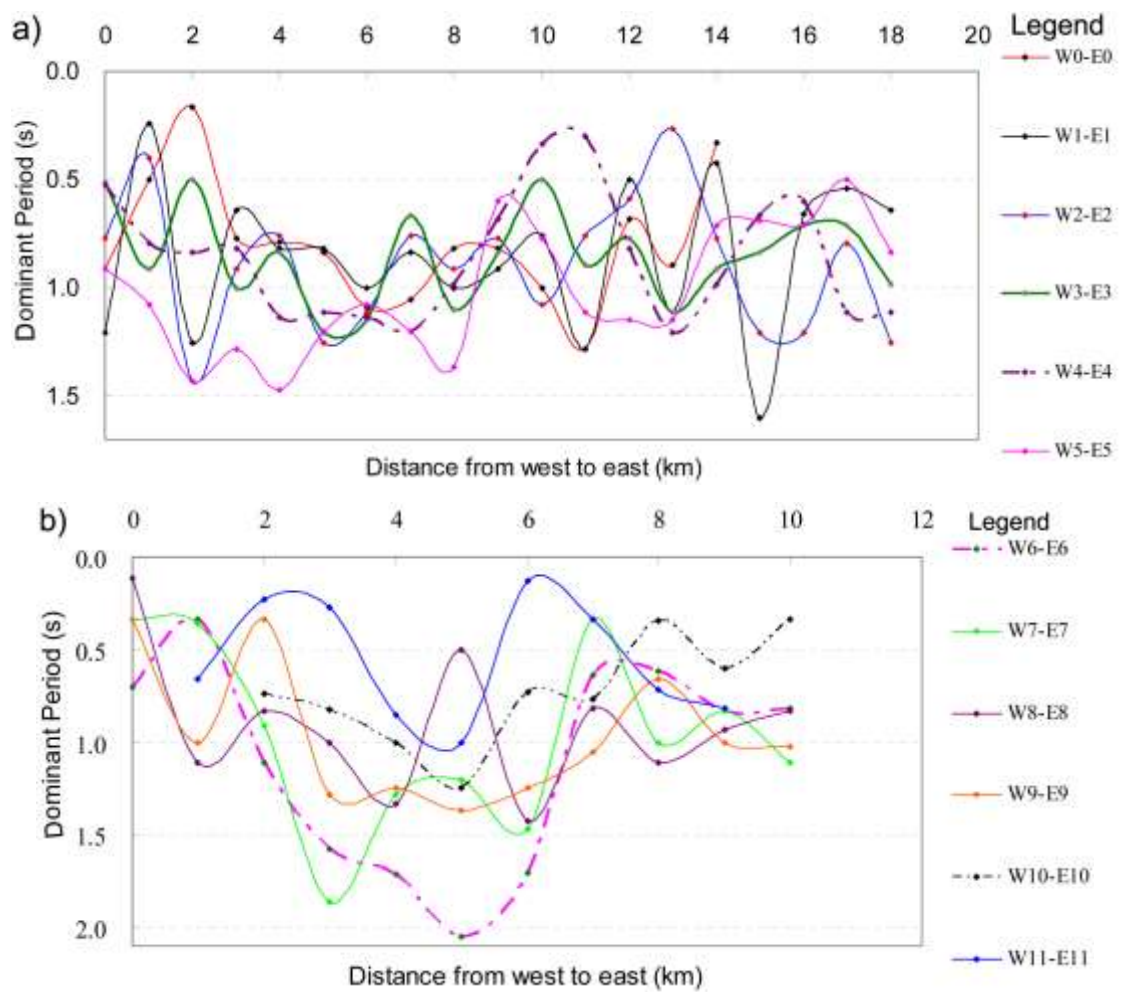


Figure 4.16: West to east profiles based on the dominant period of ground, a) Profiles W0-E0, W1-E1, W2-E2, W3-E3, W4-E4 and W5-E5, b) Profiles W6-E6, W7-E7, W8-E8, W9-E9, W10-E10 and W11-E11 (refer Figure 4.14).

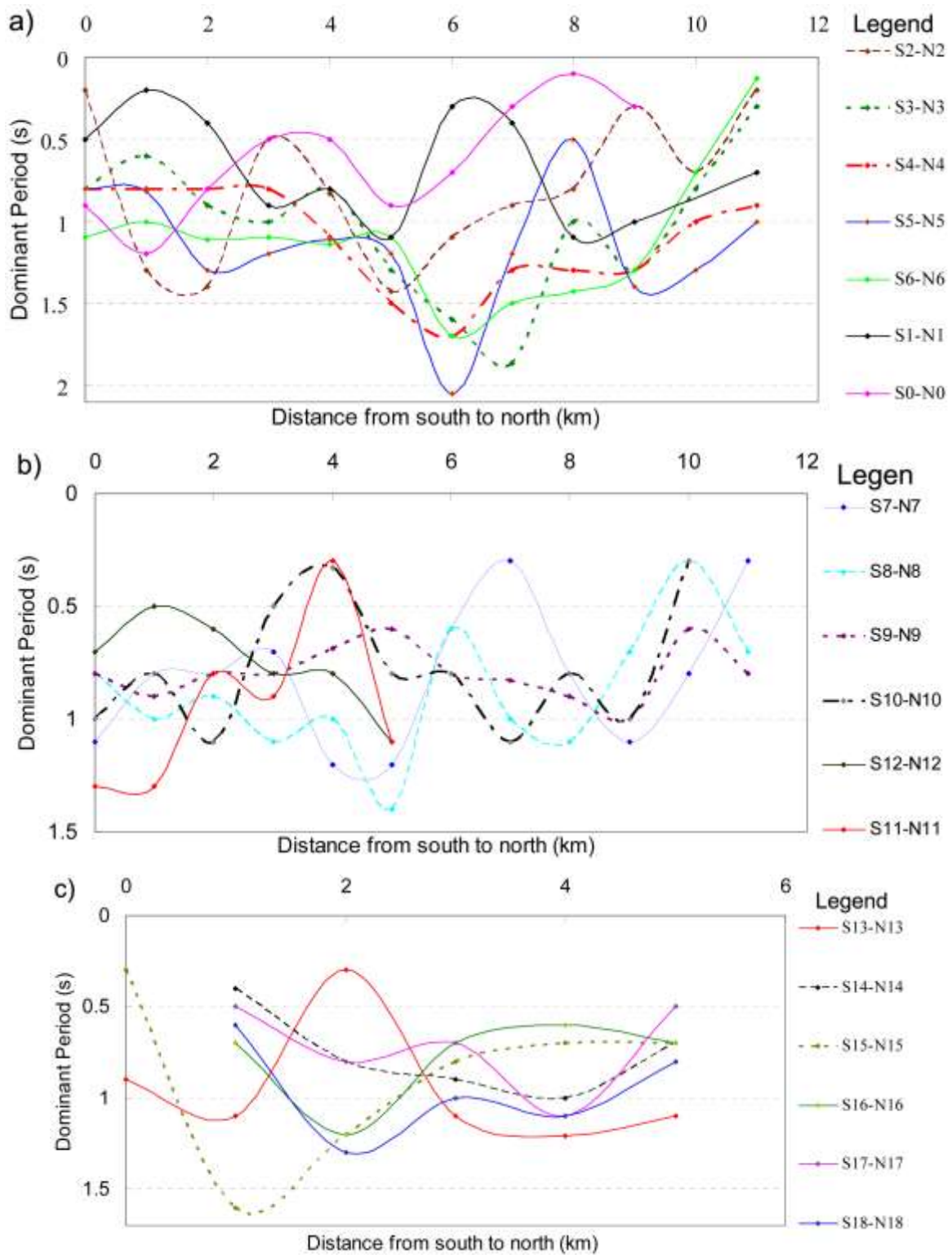


Figure 4.17: South to north profiles based on the dominant period of ground, a) Profiles S0-N0, S1-N1, S2-N2, S3-N3, S4-N4, S5-N5 and S6-N6, b) Profiles S7-N7, S8-N8, S9-N9, S10-N10, S11-N11 and S12-N12, c) Profiles S13-N13, S14-N14, S15-N15, S16-N16, S17-N17 and S18-N18 (refer Figure 4.14).

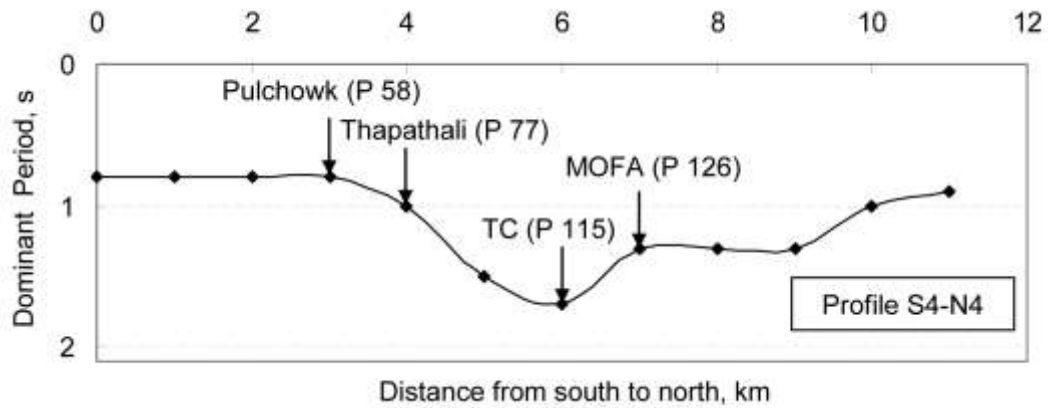


Figure 4.18: Variation of dominant periods along line S4-N4 (refer Figure 4.14). The numbered points indicate the microtremor measurement points on the line.

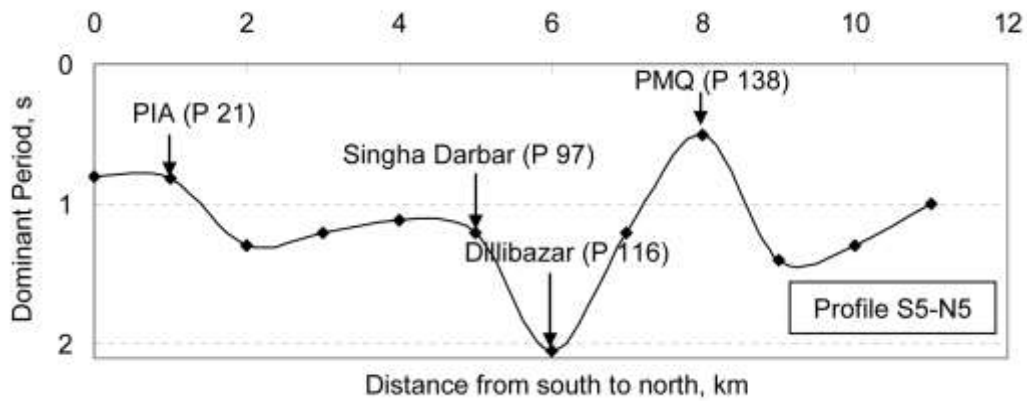


Figure 4.19: Variation of dominant periods along line S5-N5 (refer Figure 4.14). The numbered points indicate the microtremor measurement points on the line.

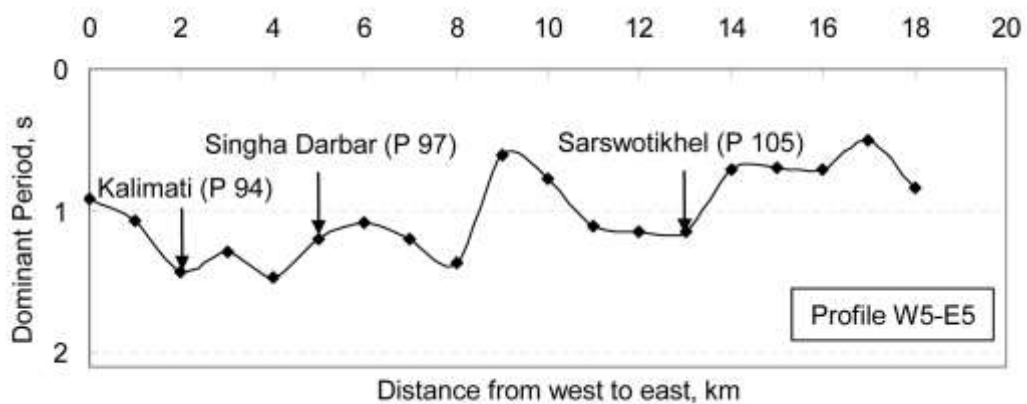


Figure 4.20: Variation of dominant periods along line W5-E5 (refer Figure 4.14). The numbered points indicate the microtremor measurement points on the line.

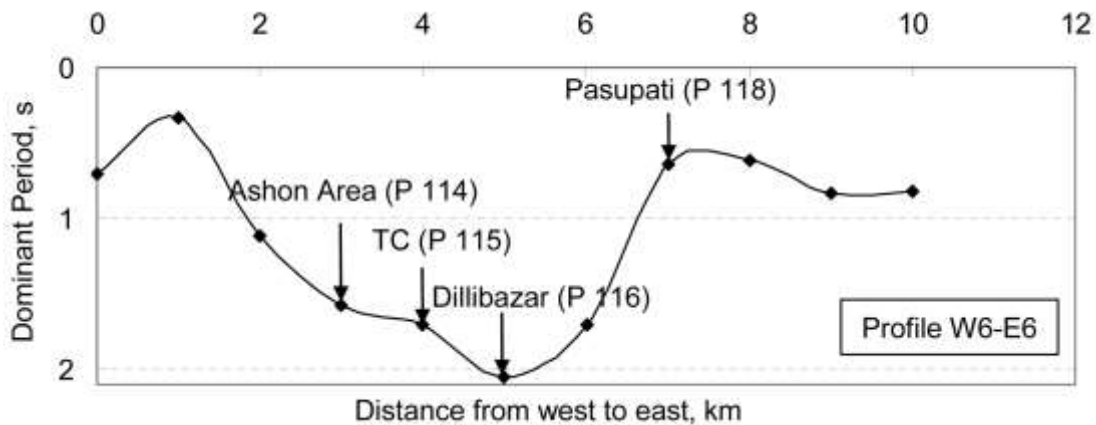


Figure 4.21: Variation of dominant periods along line W6-E6 (refer Figure 4.14). The numbered points indicate the microtremor measurement points on the line.

To sum up, based on the approximate microzonation map and the dominant period variation profiles in the Kathmandu valley, it is understood that the central area of the valley, the densely populated core city area, is characterized by dominant periods of up to 2 s while the outer less populated suburban area of the valley, lies in medium to short period range. Building heights in the city vary from a few stories to more than 20 stories. In general, the fundamental period of the N -story building can be approximately as $0.1 N$ s (Kramer, 1996). Based on this approximate relation, the dominant period of the Kathmandu Valley buildings can be estimated to vary from 0.1 s to more than 2 s. If the dominant period of a ground is equal or close to the period of a structure over the ground, resonance will take place leading to an enhanced vibration of structure. Therefore, it can be interpreted from the results that the central area of the valley (the core city area) is highly susceptible to seismic damage of long period structures, and the outer peripheral parts are susceptible to the damage of short- to medium-period structures.

4.3.3 Application of seismic microzonation map

Seismic microzonation map has a wider application in the field of engineering and urban planning. Some of the important applications of seismic microzonation map are as follows.

- ◆ Seismic microzonation map provides the information about geology, seismological and geotechnical aspects of the particular area. It gives the general geological information about the Kathmandu Valley.

- ◆ It provides information on the relative damage potential of the existing structures in a Valley. The microzonation map could be useful in establishing criteria for land-use planning and a strategy for the formulation of a systematic and informed decision-making process for the sitting and development of new communities in areas that are made hazardous by nature.
- ◆ Microzonation map provides information about requirements of further rigorous analysis for the ground response analysis in the particular area or not.

4.3.4 Limitation of seismic microzonation map

- ◆ Seismic microzonation map is an efficient tool to mitigate earthquake disaster risk by hazard-related land use management. However it does not replace the existing building and construction codes.
- ◆ It does not provide detailed hazard parameters at the level of the specific building site, but it does provide guidance on required site specific investigations.

4.4 CHAPTER SUMMARY

This chapter has addressed the issue of earthquake hazard mitigation in the Kathmandu valley of Nepal through interpretation of microtremor observation. The study area of the Kathmandu valley is built over soft soil layers of lacustrine origin, and has a repeated history of frequent earthquake damage. Despite a rising awareness level of earthquake disaster risk in the Kathmandu valley and a high probability of a major earthquake in the near future, much of the risk reduction efforts have focused only on management and legal issues leaving the scientific aspects of potential damage in the shadow. Particularly, the dynamic behavior of the sediment deposit must be considered while assessing the earthquake disaster risk in the Kathmandu valley. In this chapter, an approximate seismic microzonation map for the Kathmandu valley has been prepared out of dominant period of ground shaking obtained from microtremor observations at a total of 172 points for the very first time. The investigation results particularly reveal that the dominant period in the urban cores and peripheral settlements of the Kathmandu valley varies from 0.11 s to 2.05 s, and that the period decreases from a higher value in the central part of the valley gradually reaching a low in the outskirts. This trend is found to follow the distribution of sediment depth in the valley.

The central region of long period shaking corresponds to the concentration of 5- to 7-story old masonry and reinforced concrete-framed structures with fundamental resonance

periods close to the observed periods of sediment shaking determined in the present study. Additionally, because of increasing population and development into a greater commercial hub, the central part has seen a sharp rise in the number of mid-height to tall buildings, which have been constructed under inadequate geotechnical investigations. So, when an earthquake hits the valley, the concentration of structural damage and human casualties will be more in the valley core than any other part. In order to reduce the disaster risk, therefore, special considerations must be made in the seismic design of tall or long structures in the valley center. At the same time, it may also be necessary to consider enhanced seismic design criteria for medium- and short-period structures in the remaining part of the valley.

Finally, in absence of adequate subsurface data, an estimated dominant period shaking map, also referred to as an approximate seismic microzonation map in this chapter, for the Kathmandu valley has been prepared as the final output of this study. This map may serve as a first simple approximation that can and should be confirmed through further investigation in future. It may provide a certain basis for analyzing the damage risk of structures during an earthquake, and may also help planners and disaster managers adopt safer land-use planning, strengthen existing structures, and implement a building code in the design and construction of structures against earthquakes. Moreover, the map is a reference of dominant period-based ground characteristics, which does not provide specific hazard parameters at the building site level, but it does give guidance for the requirements of site specific investigation.

CHAPTER 5

5 ASSESSMENT OF SURFACE LAYER EFFECT ON GROUND MOTION AMPLIFICATION

5.1 INTRODUCTION

Kathmandu Basin sits on an ancient lake bed zone, with lacustrine sediment (Sakai 2001) in varying thickness from place to place. The detailed description about the geology and basin formation is given in Chapter 2. According to Sakai (2001), and Yoshida and Igarash (1984), these formation were formed in different geological periods, and types of the sediments in each formation are different from each other. Sakai (2001) has mentioned that the lower member of each formation mainly composed of boulder and cobble bed and minor amount of lenticular sand beds, middle member composed of lacustrine sediments (soft clay), and upper member composed of sandy and gravelly clay in thin layers. He has divided the sediments in the valley into three groups: 1) marginal fluvio-deltaic facies (i.e., typical riverbed layered deposits of clays, silts, sand and gravels) in the northern part, 2) open lacustrine facies (i.e., a thick organic mud, the black clay) in the central part, and 3) alluvial fan facies (i.e., lake deposits of clays and silts, usually sandwiched between thin layers of coarse sediments) in the southern part, as shown in Figure 5.1. From a review of the available literatures, it is understood that the uneven distribution of sediment deposits and complexity in deposition pattern are characteristic to the Kathmandu Basin.

As mentioned in Chapter 2, the cities of the Kathmandu Valley were repeatedly damaged by several earthquakes in the past, such as in 1255, 1408, 1681, 1803, 1810, 1833, 1866 1934, 1988, 2011 (Bilham et al., 1995; Chitrakar and Pandey, 1986; Pandey et al., 1995; CUEE, 2011). Observations of past earthquakes damage show wide variation of damage levels even in a relatively small area of the valley (Pandey and Molnar, 1988; Pandey et al. 1999; Rana, 1935). Moreover, Pandey and Molnar (1988) has mentioned that the damage distribution in the southern and eastern part is quite different than the central and northern part during 1934 Bihar-Nepal earthquake. Bhaktapur and neighboring villages in the eastern part of the valley were heavily damaged than other part of the valley. Mugnier et al. (2011) have recently conducted a study on the seismic response of the

Kathmandu Basin, and they have mentioned that the soft sediment deformation of the basin is mainly controlled by the fluidization of the silty layers during earthquake shaking. Moreover, Dixit et al. (1998), Hough and Bilham (2008), and Pandey and Molnar (1988) have pointed out that damage variation during the 1934 earthquake, in particular, was due to the presence of fluvio-lacustrine sediments in the Kathmandu Basin which play an important role in intensifying the ground motion.

Several studies reaffirm the fact that comparatively heavy earthquake damage in the Kathmandu Valley is associated with the valley ground structure (Hough and Bilham, 2008; Pandey and Molnar 1988; Mugnier et al., 2011). This wide variation of damage in the valley could be resulted from irregularities of radiation patterns due to source mechanism, as well as from local site effects associated with the near-surface geology. As summarized by Aki (1993), it has been widely observed that each soil type responds differently, when it is subjected to ground motion from earthquakes. One of the most studied cases of site amplification is represented by soft sediments, such as alluvial deposits, overlying a hard-rock basement; the impedance contrast causes magnification of both amplitude and duration of seismic motion at the surface, when seismic waves are traveling from underneath (Singh and Ordz, 1993).

As discussed in the above paragraphs, one of the main characteristics of the Kathmandu Valley is the complex geological structure, characterized by wide variability in the sediment thickness and material properties, which may cause trapping and focusing of seismic waves during an earthquake leading to an evident change in resonant frequency in over the short distances (Iannaccone et al., 1995). The resonant frequency of a site is particularly important because it indicates the frequency of the spectrum under which the near-surface soft sediment amplifies the earthquake ground motion. This phenomenon is also known as the site effect. The most important parameters obtained from site effect studies are the resonance frequency and the amplification factors of the ground motion.

According to Latch et al. (1996), and SESAME (2004), the resonant frequency of a ground is obtained by taking the value of the frequency at the first peak in the horizontal-to-vertical spectral (H/V) ratio of ground motion observed at a particular site, which is also the lowest frequency in the H/V spectrum. This is also referred to as the fundamental frequency of a particular location. Most of the past studies have showed that the damage distribution due to earthquakes is correlated with the fundamental frequency of the location (Gosar, 2007; Gosar and Martinec, 2009; Mucciarelli et al., 2004; Nguyen et al., 2004; Teves-Costa et al., 2007).

Gosar and Martinec (2009) have conducted the microtremors measurement both inside the buildings and in the free-field in Ilirska Bistrica Town area (S. Solvenia). They found that in northern part of the city, the fundamental frequency of the buildings is very close or similar with the fundamental frequency of the ground. From this observation they concluded that the damage caused by the 1995 earthquake ($M_L = 4.7$, IMAX= VI, EMS-98) is due to the resonance effect between ground and the structures. Similarly, Teves-Costa et al. (2007) have measured the microtremors in buildings and nearby ground in Angra do Heroismo-Azores. Their results are in good agreement with the surface geology and they show a good correlation with the observed damage during the $M_W = 6.9$, January 1, 1980 earthquake in the Azores with the fundamental frequency of the ground and the buildings. However, some of the studies, such as Fäh et al. (1994), Toshinawa et al. (1997), Gueguen et al. (1998, 2000) indicate that depending upon the soil conditions of underlain strata, a second amplified frequency is locally revealed which can play an important role to create a resonance with the structures built over the ground during earthquake. Furthermore, Guéguen et al. (1998) have linked the building damage distribution in Pujili (Ecuador), following an $M_{5.7}$ earthquake, with the second amplified frequency peak obtained using the horizontal-to-vertical spectral (H/V) ratio method proposed by Nakamura (1989), and have mentioned that the second peak represents the response of a soft surface layer of ground.

As mentioned in Chapter 2 and also in the above paragraphs, the geological structure and sediment depositional environment in the Kathmandu Basin consists of many strata of sand, silt and clay sediments, which bring forward a possibility that two or more amplified frequencies may occur during an earthquake. As the valley accommodates a number of low-rise to medium-rise buildings, historically important places and monuments, there are possibilities during an earthquake that the multiple amplified frequency causes a resonance with the structures in a broad frequency range, leading to an enhanced vibration of structures and possible collapse.

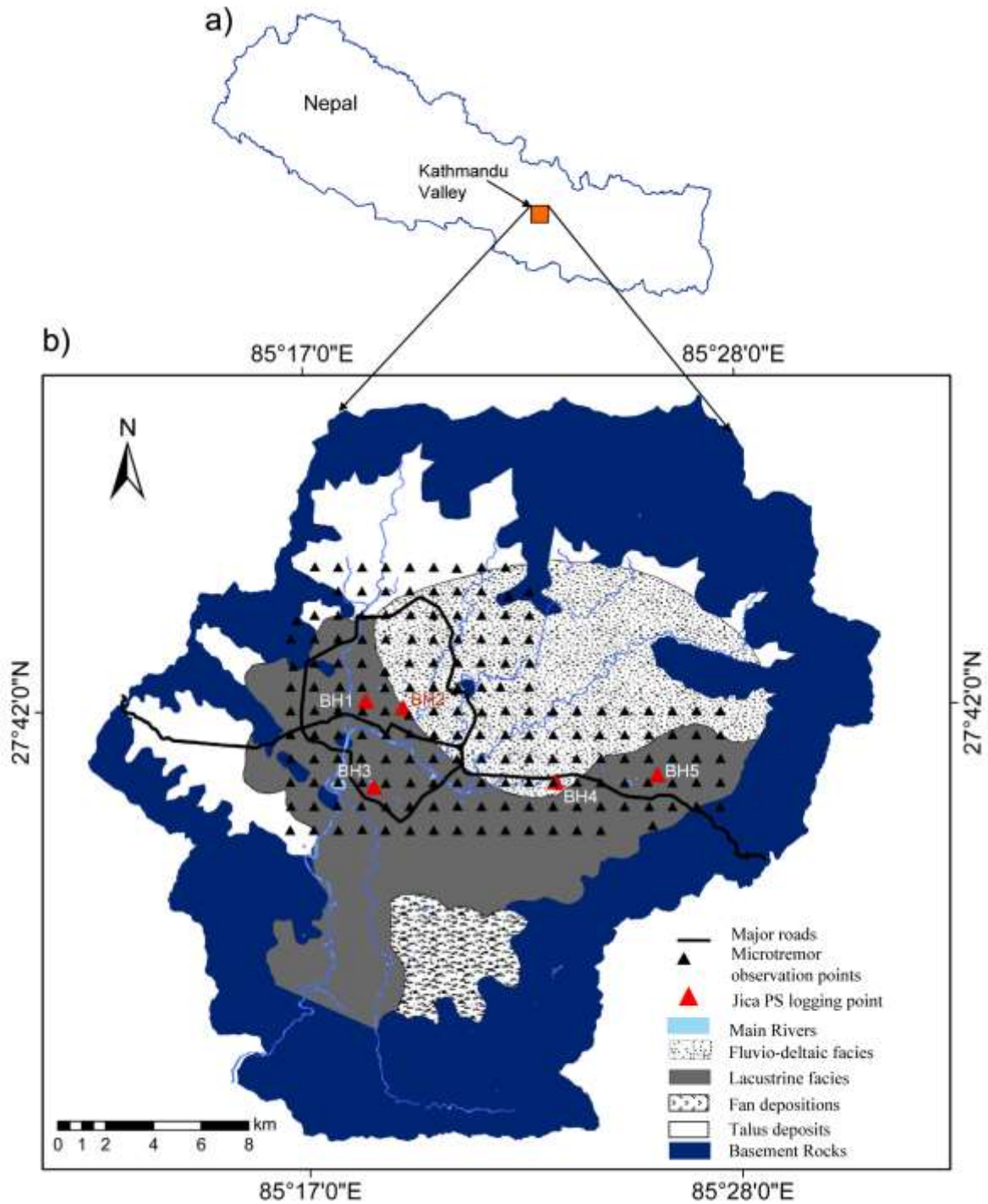


Figure 5.1: Study area, showing microtremor observation points and JICA PS logging point (BH1, BH2, BH3, BH4, and BH5).

5.2 CAUSES OF MULTIPLE RESONANCE IN A SEDIMENTARY BASIN

If soft sediments overlies resistant bedrock, an impedance contrast develops at the boundary which causes incoming seismic energy to be absorbed at a rate faster than it can

be transferred through the upper layers, causing significant amplification of ground motions. Large impedance contrast between soft soil overlying bedrock causes strong reflections as a result the seismic waves are trapped within the soil layers overlying the bedrock. These trapped waves start interfering (when waves meet while they try to travel through same medium) with each other, which may result in resonance.

In earth, a change in impedance occurs primarily in the vertical direction. Seismic impedance is a product of density of the layer(s) (ρ) and shear wave velocity (V_s), and it measures the stiffness of the soil layer. If the stiffness of the soil layer is high, there is strong contrast, and hence strong reflection of the seismic wave occurs. Therefore, difference in impedance contrast in the soil layer is very important in order to understand the conservation of seismic waves. If the impedance becomes higher, then resistance to seismic motion increases and vice versa.

It has been known that site amplification is a function of the impedance ratio between the valley fill and the underlying basement rock. According to Mresca et al. (2003), if the sedimentary layer of site has vertical heterogeneity that can generate high frequency trapped waves in the very shallow sediments giving rise to observed amplifications in the spectra. Guéguen et al. (2006) have mentioned that the highest frequency is more unstable because of the extremely high variability in the short distances of the upper sedimentary layers. These instabilities are the consequence of the fluvial infill sequences. Zaslavsky et al. (2006) and Hofstetter et al. (2008) have done a detailed investigation about the local site effect using ambient vibrations as well as analytical method. According to them, a simple two-layer model, where a soft layer overlay hard rock yield an H/V ratio which is characterized by a single spectral peak as shown in Figure 5.2. It is understood that the fundamental frequency is proportional to the S-wave velocity in the soft layer and inversely proportional to its thickness. They also illustrate that the maximum peak on the H/V spectral ratio depends on the seismic impedance between the soil and the rock. Similarly, they describe other type of H/V spectra which is characterized by having two inseparable spectral peaks. Such two peaks spectral ratio can be obtained at some sites located on exposure of fluvial lacustrine sediments (river bed materials). This type of H/V spectral ratio corresponds to a three-layer model, when soft and intermediate hard layer overlay hard rock (Figure 5.3).

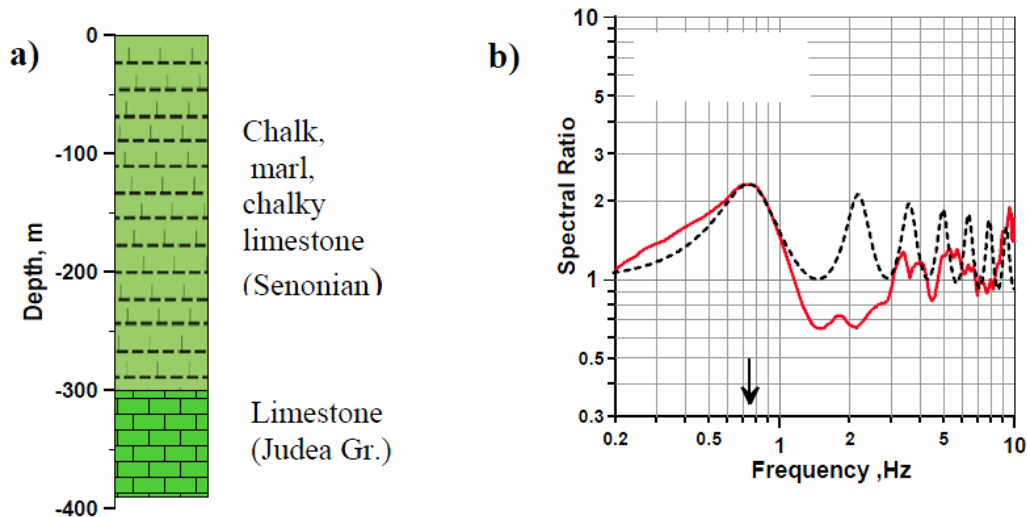


Figure 5.2: a) Lithostratigraphic section of the well; b) comparison between H/V spectrum using microtremor analysis (red line) and the analytical transfer function computed using well data (black line) (after Zaslavsky et al., 2006).

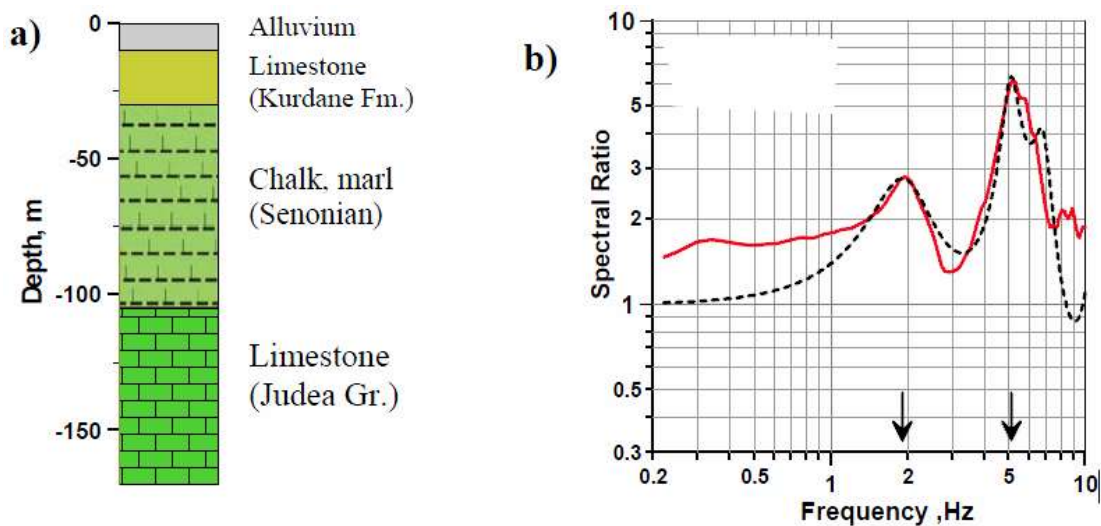


Figure 5.3: a) Lithostratigraphic section of the well; b) comparison between H/V spectrum using microtremor analysis (red line) and the analytical transfer function computed using well data (black line) (after Zaslavsky et al., 2006).

The first resonance frequency is related to the deeper bed rock while the second peak is related to the intermediate hard layer. The second peak depends mainly on thickness of the intermediate layer. The amplitudes of these peaks depend mainly on the S-wave velocity in the soft soil. Moreover, the multiple peaks H/V spectral ratio is generally due to the higher shear wave velocity of the intermediate layer which result the higher impedance contrast in the boundary. Zaslavsky et al. (2006) have also mentioned that these two peaks

in the H/V spectral ratios are associated with two reflectors. The deep reflector, which is associated with deep basement rock, is responsible for the first peak and the shallow reflector, the gravely sandy layer, is related to the second peak.

Many theoretical and numerical investigations, such as Zaslavsky et al. (2006) have shown that multiple peaks resonance frequency occurs for two large impedance contrasts (say, around 4 minimum for each), at two different scales: one for a thick structure, and the other one for a shallow structure. The two frequencies, f_0 and f_1 (with $f_0 < f_1$), may then be interpreted as their characteristics at each scale, f_0 being the fundamental frequency. According to SESAME (2004), when there exists two clear frequencies, it is likely that a) the surface velocity is low; b) the deep bedrock is very hard, and c) there exists two large impedance contrasts (at least, approx. 4) at two different scales, so that the amplification should be significant over a broad frequency range starting at f_0 and extending beyond f_1 . If the total thickness is known, the average S-wave velocity of the surface layers may be estimated with the formula $V_{s,av} \simeq f_0 4h$; if a reliable estimate of the S-wave velocity $V_{s,surf}$ is available close to the surface, then a lower bound estimate of the thickness of the topmost layer may be obtained with the formula $h_{1,min} \simeq V_{s,surf}/4f_1$ (SESAME, 2004).

5.3 PURPOSE OF THIS STUDY

Based on the above observation, it is understood that there is a possibility of occurrence of multiple resonance phenomena in the Kathmandu Basin during earthquakes. Therefore, in this chapter, an attempt is made to investigate the response of the surface and the layer underneath it during earthquake using horizontal-to-vertical spectral (H/V) ratio method. It is very important to know the areas which have single resonance frequency and multiple resonance frequency, because the multiple resonance frequency areas can amplify the ground motion over a broad-range of frequencies and hence their effect can not be limited for a particular building type. Therefore, the main objective of this work is to delineate the area of single and multiple resonance frequencies in the Kathmandu Valley. In Chapter 4, we have analyzed the microtremor observation data of the whole study area and obtained the predominant frequency of each site. Therefore, in this chapter, we particularly focus on the ground response in the areas which show multiple amplified frequencies. There is a wider application of multiple resonance frequencies, particularly in the seismic hazard assessment of the particular area. In this chapter, we also try to calculate the thickness of the responsible surface soil layer for the amplification of seismic waves in the Kathmandu Valley.

5.4 MATERIALS AND METHOD

5.4.1 Study area

The study area is shown in Figure 5.1 in which the location of the microtremor observation points and the location of the JICA borehole sites are also shown which are presented in detail in the section 5.4.2. The detail description of the location map is similar as mentioned in Chapter 2, Section 2.4.

5.4.2 Shear wave velocity in the Kathmandu Basin.

Although there are many boreholes which have drilled for different purposes, mostly in water supply from different agencies as mentioned in Chapter 2, the shear wave velocity up to basement rock has not been known till data in the Kathmandu Basin. As per Sakai (2001), the boreholes which were drilled before 2001 might not give accurate information about the subsoil condition (i.e., lithology and stratigraphy), because these boreholes were not fully recovered during drilling. The soil type and other parameters, which are need for the geotechnical study, are almost missing. Therefore, the detail analysis of the geotechnical as well as geophysical parameters using borehole data, and also calculation of the shear-wave velocity in the required site may not be possible. However, there are five locations, as denoted by BH1, BH2, BH3, BH4 and BH5 in Figure 5.1, in the basin, where shear-wave velocities were measured to a depth of 30 m using P-S Logging by the Japan International Cooperation Agency in 2002 during the study of Earthquake Risk Disaster Mitigation in the Kathmandu Valley (JICA, 2002). The soil profile and shear-wave velocity profile of sites (refer Figure 5.1) are shown in Figure 5.4 and also the average shear-wave velocity of the sites are also shown in Table 5.1.

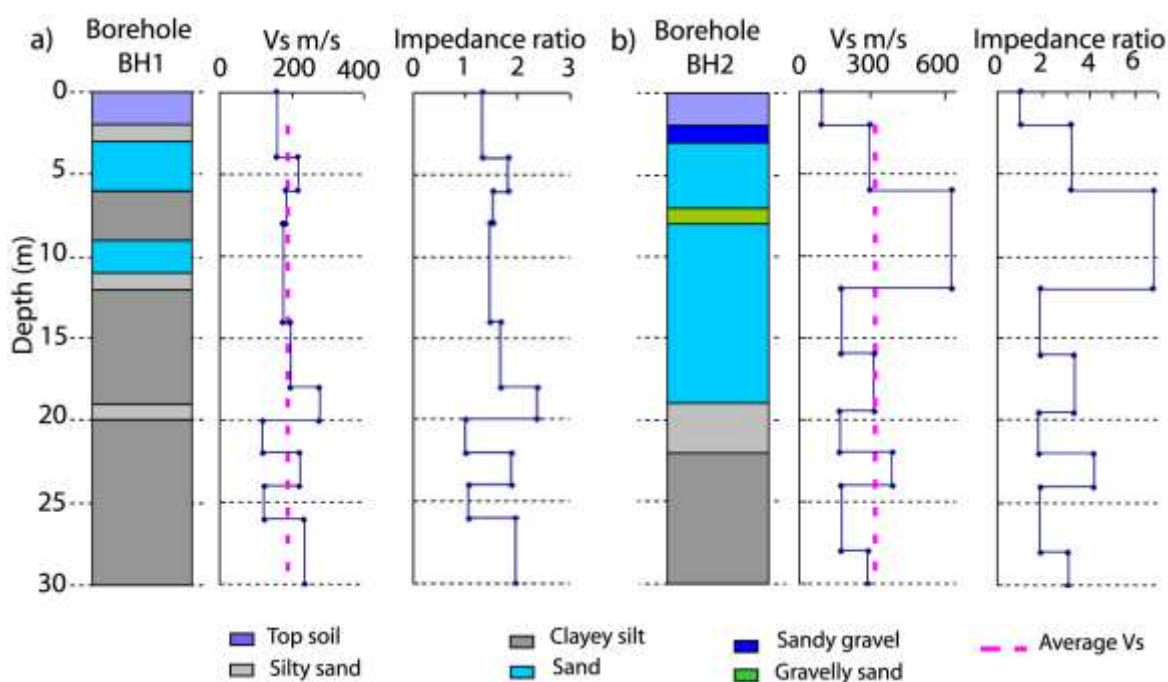
After detailed study of the JICA P-S Logging sites, it is understood that the shear-wave velocity varies from area to area. The average shear-wave velocity (i.e., 188.87 m/s) is found minimum in Borehole first (BH1) at Kathmandu Durbar Square site (Figure 5.4a and Table 5.1), which lies at the center part of the valley and dominated by soft clay soil. It also lies at the center of the commercial hub of the Kathmandu Valley and accommodates many historical buildings and monuments which is also one of the UNESCO Cultural Heritage sites of Nepal. As shown in Table 5.1 and Figure 5.4b the maximum average shear-wave velocity (i.e., 310.85 m/s), in comparison to other drilled sites in the valley, is found at second borehole site (BH2) which also lies at the center part of the valley, but it comparatively dominated by the sandy soil than first borehole. Third Borehole (Figure

5.4c) lies at Jawolakhel in the Lalitpur city. The borehole fourth and fifth lie at the Thimi and Bhaktapur durbar square in Bhaktapur district, in which the borehole fifth is laid at the center of the Bhaktapur city, which is dominated by historical palaces and monuments and is also a another UNESCO Culture Heritage site in the Kathmandu Valley.

Similarly, the borehole data shows that the area in borehole fourth and fifth is mostly dominated by clay soils. Figure 5.4a-e also show the variation of impedance ratio in different levels along the depth of each boreholes which is calculated from the available density and shear wave velocity variation along the depth of the boreholes (JICA, 2002).

Table 5.1: Shear wave velocity in the Kathmandu Basin.

SN	Borehole Location	Borehole ID (as in Figure 5.1)	Average shear wave velocity (m/sec)
1	New Road	BH1	188.87
2	Singha Durbar	BH2	310.85
3	Jawolakhel	BH3	247.33
4	Thimi	BH4	234.40
5	Durbar Square, Bhaktapur	BH5	252.93



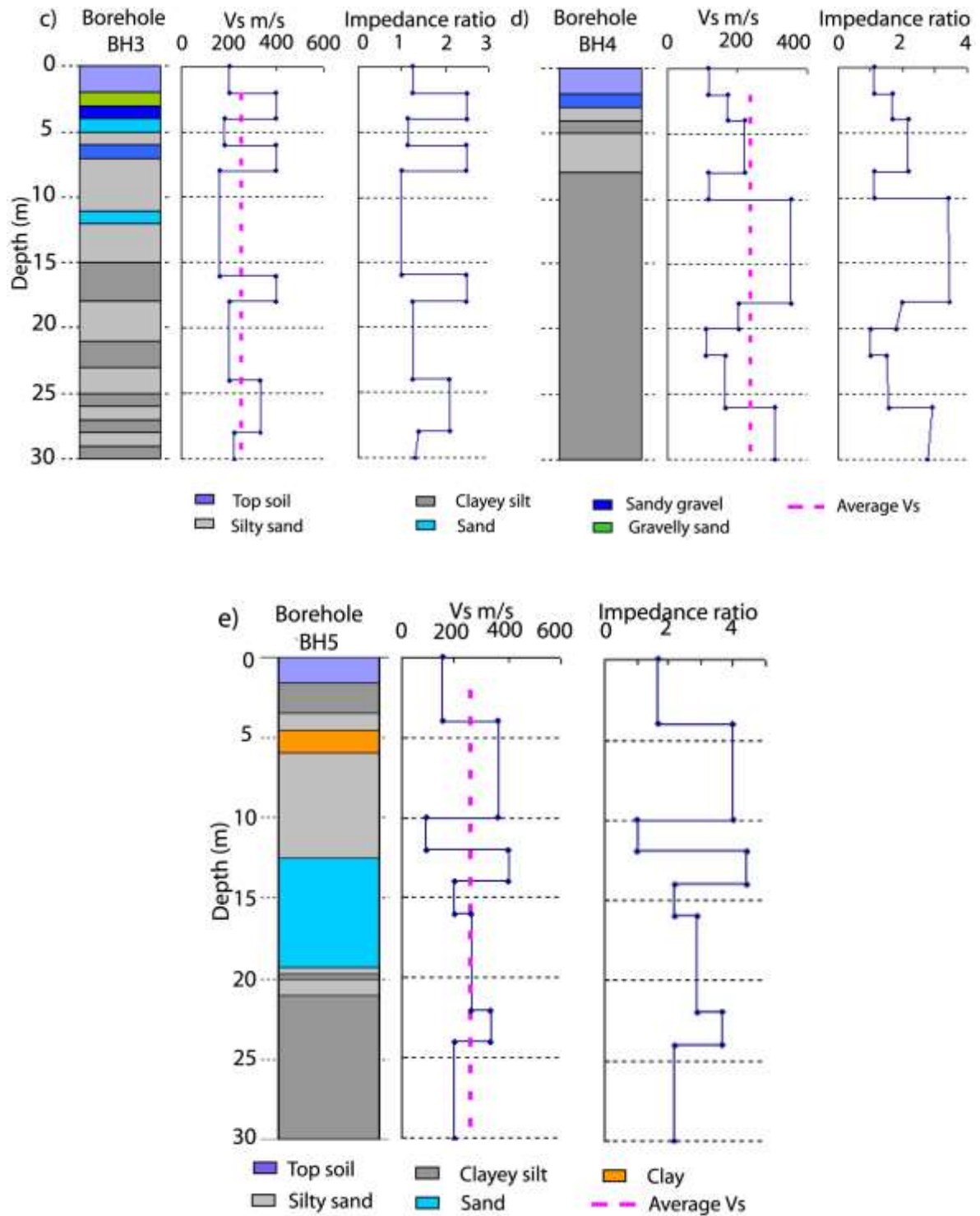


Figure 5.4: Borehole profile and shear wave velocity in the Kathmandu Valley (after JICA, 2002).

5.4.3 Methodology adopted

The overall methodology adopted in this study consists of the analysis of microtremor data of all sites in the study area and minutely analyzed the multiple resonance frequencies from the curves of the horizontal-to-vertical spectral ratio of 172 observation points. And

finally we have used these frequencies for the calculation of the thickness of the top soil layer in the study area. The study area and location of data collection (refer Figure 5.1) and their acquisition is similar as mentioned in Chapter 4, Section 4.1.2.3.

It has been known that the resonance frequency obtained based on microtremor data analysis using H/V technique has a wider application in the field of geotechnical earthquake engineering, such as in microzonation, sediment depth calculation, and shear wave velocity and so on. Ibs-Von Seht and Wohlenberg (1999) have demonstrated that there is a strong correlation between shear wave velocity, resonance frequency and thickness of the sediment, and they have proposed a very useful relationship between these parameters for western Lower Rhin Embayment (Germany). Many other studies, such as D'Amcio et al. (2004), Delgado et al. (2000), Dinesh et al. (2010), Gosar and Lenart (2010), Özalayboy et al. (2011), Parolai et al. (2002), Sukumaran et al. (2011) have developed an empirical relationship for determining the depth of a sediment layer over bedrock using the H/V method for different geographical and geological regions of the world.

In order to calculate the thickness of a sediment layer over bedrock or the thickness of the topsoil layer, Kramer (1996) derived an equation for one dimensional (1D) ground response analysis. According to his derivation, the 1D-resonance frequency, denoted by f_0 , is usually defined as the function of S-wave velocity (V_s) and the thickness H of the soil deposit, as in Equation (5.1).

$$f_0 = \frac{V_s}{4H} \dots\dots\dots (5.1)$$

In the case of a multi-stratified half-space (n layers), resonance frequency f_0 can be easily expressed by considering a single equivalent layer as follows.

$$f_0 = \frac{\sum_{i=1}^n V_{s_i} H_i}{4(\sum H_i)^2} \dots\dots\dots (5.2)$$

Therefore, by using the equations (5.1) and (5.2), in this research, the thickness of the uppermost soil layer is obtained for the area of known shear wave velocity and resonance frequency.

5.5 RESULTS AND DISCUSSION

5.5.1 Frequency distribution in the Kathmandu Valley

The results of the microtremor survey are expressed in terms of resonance frequency of the ground at each survey point, as typically shown in Figure 5.5a and b. As seen in the figures, microtremor observation point P91 has a single amplified frequency while microtremor observation point P82 has two amplified frequency (refer Figure 5.8).

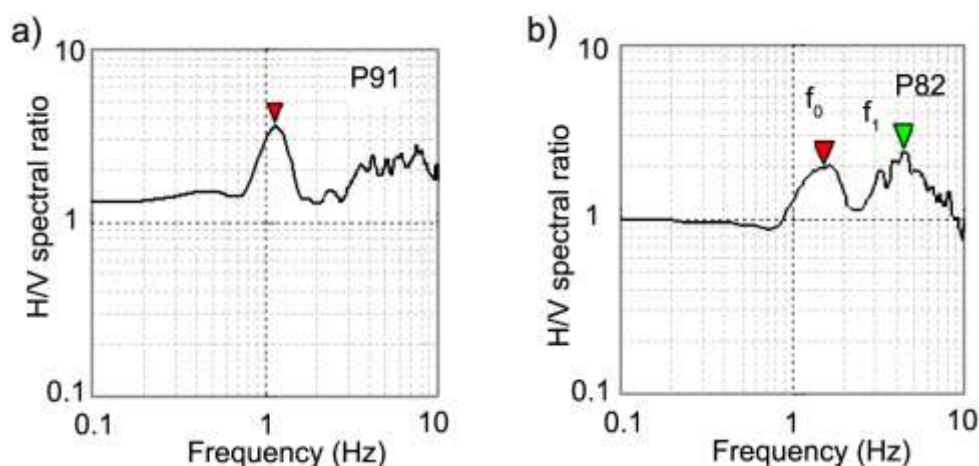


Figure 5.5: A typical a) single peak H/V spectral ratio, b) multiple peaks H/V spectral ratio in the study area.

Likewise, Figure 5.6 shows the H/V spectral ratio versus frequency graphs and the multiple amplified frequencies for some of the microtremor observation points in the Kathmandu Valley. Variations in the shapes of H/V curves can be easily noticed in this figure. The first resonant frequency can be easily identified because the first peak is smoother than the second peak, but it may be difficult to exactly identify the second resonant frequency in the multiple-peak H/V spectral-ratio curves. In order to accurately identify the second resonant frequency at a particular location, a midpoint in the H/V curve with maximum amplitude is taken, but in some cases, the midpoint of the curve may not lie in the position of maximum amplitude. Both frequencies, i.e. first resonant frequency or predominant frequency and second resonant frequency as indicated by f_0 and f_1 , as shown in Figure 5.6, are the representative to a particular site condition. The occurrence of the multiple peaks may be due to the presence of vertical heterogeneity in the soil column at that location, and they describe the overall seismic site response at two scales, including the deep and the upper soil profile response.

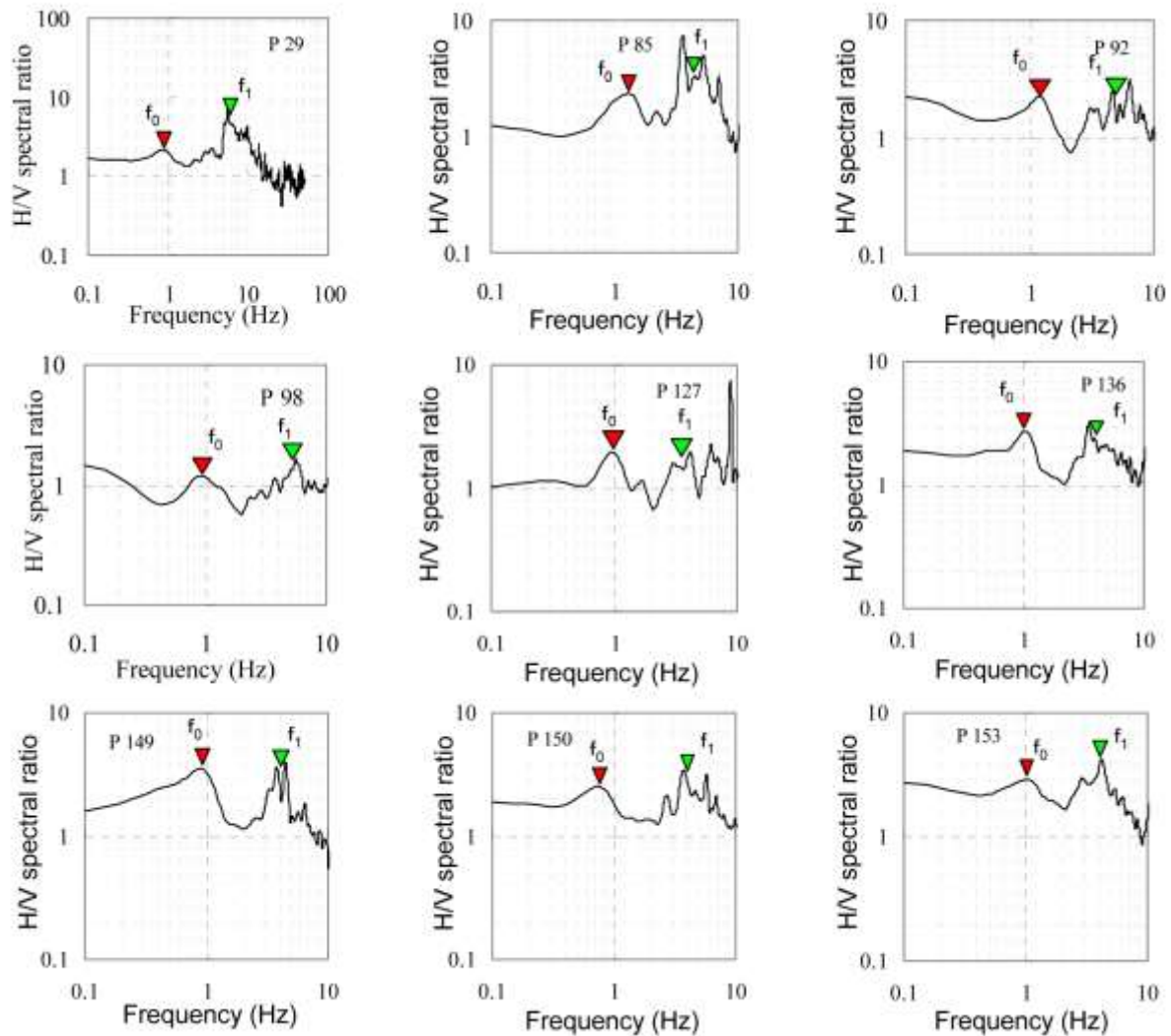


Figure 5.6: Example of H/V spectral ratios with multiple amplified frequencies, red and green triangles indicate the first and second resonant frequencies respectively.

The microtremor analysis results show that about 80% measurement points in the study area exhibit single amplified frequencies that vary from 0.58 Hz to 8.9 Hz, whereas the remaining sites exhibit two amplified frequencies, in which the first amplified frequencies vary from 0.48 Hz to 1.52 Hz, and the second amplified frequencies vary from 3.1 Hz to 7.5 Hz. Most of the H/V spectral-ratios with multiple amplified frequencies lie in and around the central and the northern part of the basin, which is dominated by the marginal fluvio-deltaic facies (river bed materials) (Sakai, 2001), and a few of them are found near the bank of the river and also in the areas consisting of organic clay with a sandy soil layer. A spatial distribution of single and multiple amplified frequencies in the study area is shown in Figure 5.7. Only two multiple amplified frequency points (5 and 18)

lie in Lalitpur City and 6 points (29, 43, 50, 53, 62 and 85) lie in the Bhaktapur City area. The remaining points lie in the Kathmandu City area where the population density is comparatively high (about 19,000 per km²).

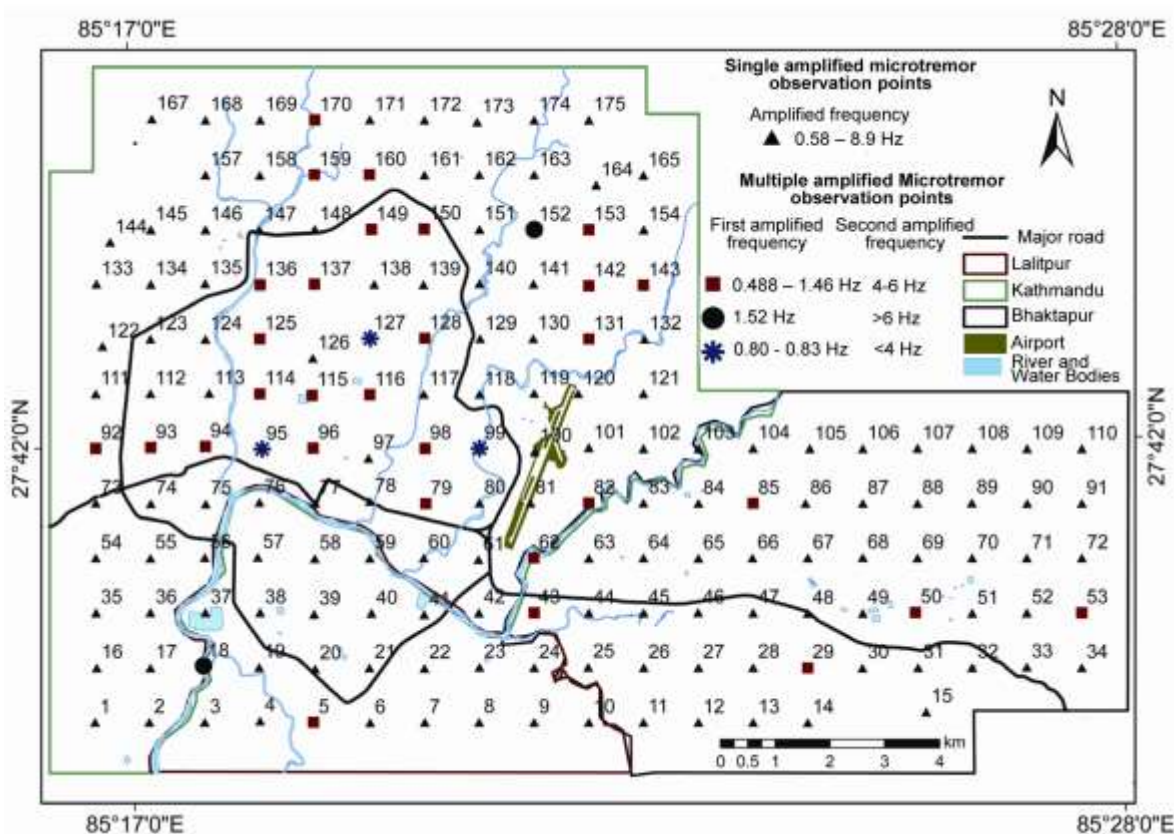


Figure 5.7: Distribution of single and multiple resonance frequencies in the study area.

This indicates that the Kathmandu City area has a wide range of resonance frequencies (i.e. 0.48 Hz to 8.9 Hz) than other areas of the valley. The Kathmandu Durbar Square area (the UNESCO cultural heritage site) is situated close to the points 95 and 114. Similarly, the Bhaktapur Durbar Square area (another UNESCO cultural heritage site) is situated very close to the point 50. Both areas encompass temples of historical and cultural importance, archeologically important palace buildings, and the UNESCO-recognized world cultural heritage sites. Moreover, points 50, 79, 93, 94, 95, 96, 98, 114, 115, 116, 125, 127, 128, 136, 137, 149 and 150 (refer Figure 5.7) lie in highly commercial areas, and consist of very important structures such as multistory apartments, departmental stores, public places, government offices and also old residential areas of the Kathmandu Valley. Similarly, points 131, 142, 143, 152, 153, 159, 160 and 170 (refer Figure 5.7) lie in the recently-urbanized area of the valley.

The distribution of the second peak resonant frequencies is mostly observed in the frequency range of 4 Hz to 6 Hz, as shown in Figure 5.7 and Figure 5.8, in the central and northern part of the basin, and also near the bank of the river. The ratio of second resonant frequencies to the first resonant frequencies is found to be approximately 5. The amplitude of the second peak frequencies vary from place to place but are found higher than the amplitude of the first peak frequencies in some of the locations (Figure 5.9). These frequencies are linked with the presence of the surface layer. This indicates that the bottom strata of the surface layer should have a sufficient impedance contrast compared to the layer underneath. Most of the measurements in those locations were carried out in the night and morning times when the vehicle movement was very low, in order to minimize the effects of nearby noise sources. The distribution pattern of the H/V spectral-ratios in the above areas convinces us that these second peak frequencies were not simply due to the vehicle movement near the microtremor observation points. It consolidates the possibility that the second frequency peak could be due to the thin surface layer corresponding to the old alluvial deposits, which may behave as an independent structure in the soil column. This could have resulted due to the presence of a stiff sediment layer, mainly composed of gravel and sand and presenting spatial and vertical heterogeneities. All this result shows two scales of amplification effects: 1) at low frequencies due to the global effect of the deep sedimentary basin and 2) at high frequencies due to the uppermost surface layers (i.e., the consequences of the lacustrine and fluvial deposit processes).

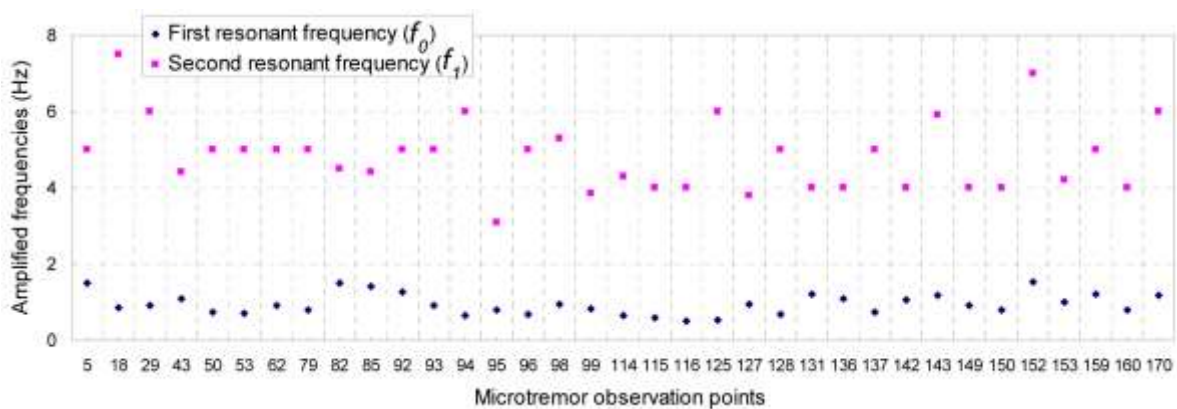


Figure 5.8: Distribution of multiple amplified frequencies in the study area.

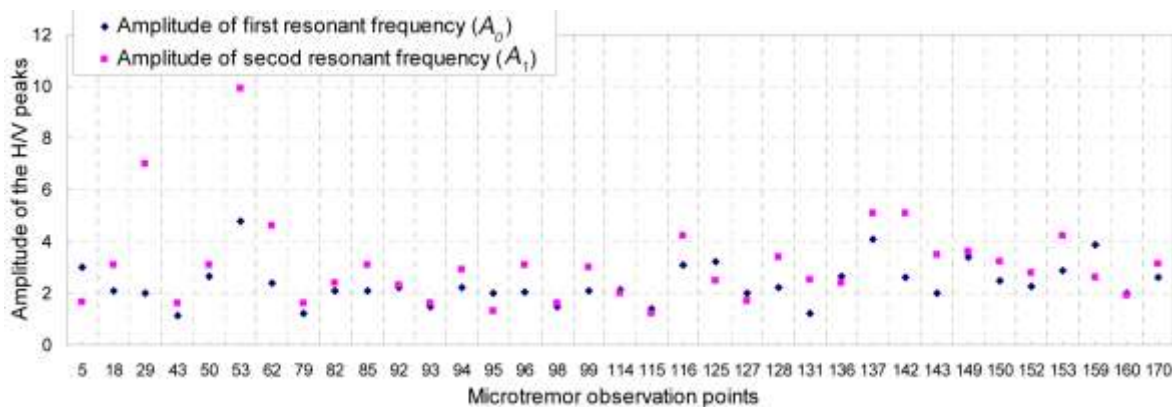


Figure 5.9: Distribution of amplitude of multiple amplified frequencies in the study area.

5.5.2 Estimation of soil layer thickness

Except for a countable number of boreholes drilled for the purpose of ground water exploration and investigation of soil parameters during building design, detailed geotechnical information of the deep structure of the Kathmandu Valley is not available. The shear wave velocity profile up to the basement rock is also not available. As a result, the correlation of fundamental frequency with sediment depth is not possible. However, there are five boreholes, as denoted by BH1, BH2, BH3, BH4 and BH5 in Figure 5.1 and Figure 5.10, in the valley where shear wave velocities were measured up to a depth of 30 m using P-S Logging by the Japan International Cooperation Agency in 2002 (JICA, 2002) during the study of Earthquake Risk Disaster Mitigation in the Kathmandu Valley.

In order to understand the level of impedance contrast and to determine the depth of the uppermost layer, which is responsible for the amplification of soil, a specific analysis was carried out of the soil profile and shear wave velocity of the boreholes, which are close to the two-peak H/V spectral ratio sites. As shown in the Figure 5.11a and Figure 5.11b, two double-peak H/V spectral ratio sites (i.e., point 50 and point 95) were found close to the JICA PS logging sites (i.e., BH5 and BH1). The H/V spectrum of Point 50, as shown in Figure 5.11a, has two peaks, which represent the multiple amplified frequencies at this point. Figure 5.12a shows a soil profile, shear wave velocity profile and impedance ratio of the borehole BH5, which is close to the microtremor observation point 50 (refer Figure 5.10). Using 252.93 m/s as an average shear wave velocity in borehole BH5 (refer Figure 5.12a; Table 5.1; JICA, 2002) and the frequency of the second peak at microtremor observation point 50 ($f_1=5.0$ Hz), Equation (5.1) yields a thickness $H_1 = 12.64$ m, which is equal to the thickness of the uppermost layer in that area. The change in impedance contrast at various levels along the soil profile is clearly seen in Figure 5.12a, in which the

maximum change in impedance ratio is seen at a level of about 12.6 m from the surface and hence the strong impedance contrast (greater than 4) can be expected at that level. The calculated depth of the uppermost soil layer from the microtremor data analysis is found nearly equal to the depth of impedance contrast (refer Figure 5.12a), which shows good agreement between the calculated depth and the observed depth. Similarly, Figure 5.11b shows the H/V spectral ratio of Point 95, in which two peaks are seen, which represent the multiple amplified frequencies in that area. Using 188.86 m/s as an average shear wave velocity from the shear wave velocity profile for BH1 (Figure 5.12b); Table 5.1; JICA, 2002) and the frequency ($f_1 = 3.1$ Hz) (refer Figure 5.11b)) of the second peak observed near the borehole profile, Equation (5.1) yields a thickness $H_1 = 15.23$ m, which is equal to the thickness of the uppermost layer in that location. Figure 5.12b shows the level of impedance contrast in which the change in impedance ratio is seen at about 17 m from the surface which is nearly equal to the above calculated depth of the uppermost layer. The microtremor observation point 95 is about 300 m south of the borehole (i.e., BH1) location, and that may be the cause for the difference in calculated depth based on the microtremor data analysis with the depth of impedance contrast in the borehole profile.

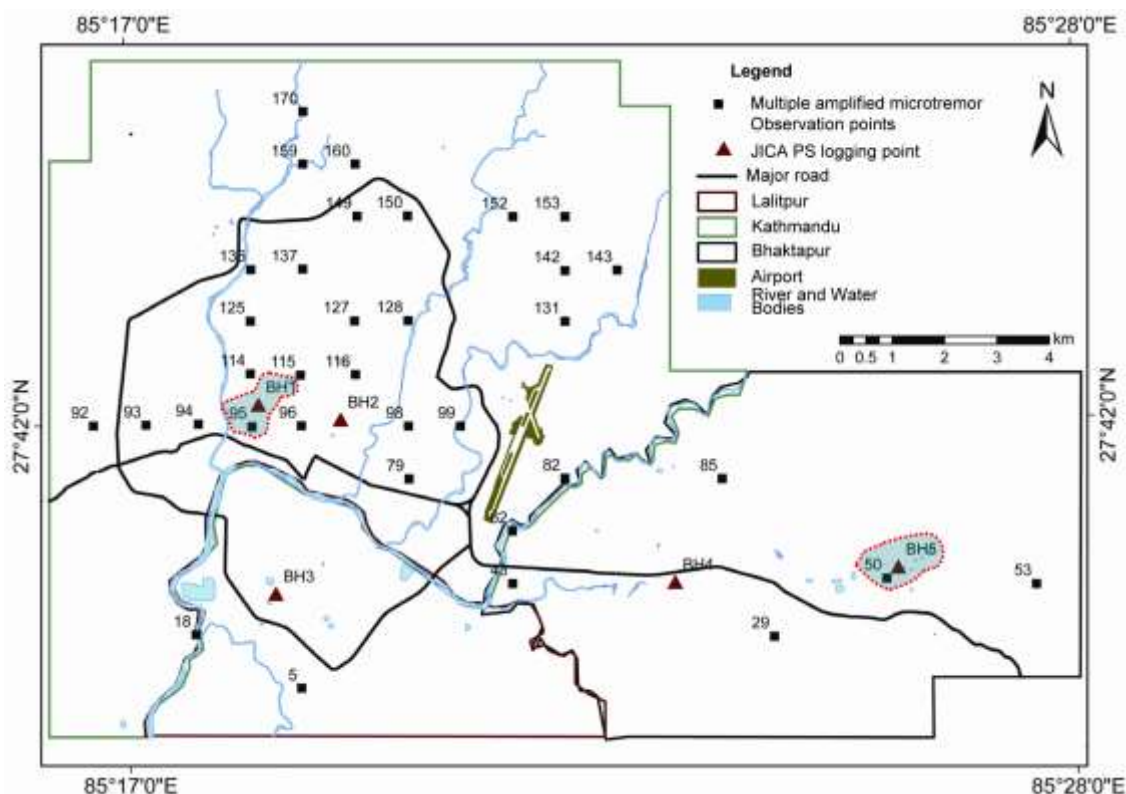


Figure 5.10: Distribution of multiple amplified frequencies and location of JICA PS logging sites in study area, red dash boundary indicates the borehole and microtremor point taken for the analysis of depth of impedance contrast.

Additionally, based on the shear wave velocity profile in all five sites (refer Figure 5.4, Table 5.1), an average shear wave velocity up to a depth of 30m for the Kathmandu Valley soil was obtained to be 246.87 m/sec (JICA, 2002), while the second resonance frequency of the valley soil was mostly found to vary from 4 Hz to 6 Hz. Using this average shear wave velocity and the frequency of the second peak, the Equation (5.1) yields a thickness of 10 m to 16 m, which is equal to the thickness of the uppermost layer of the Kathmandu Valley.

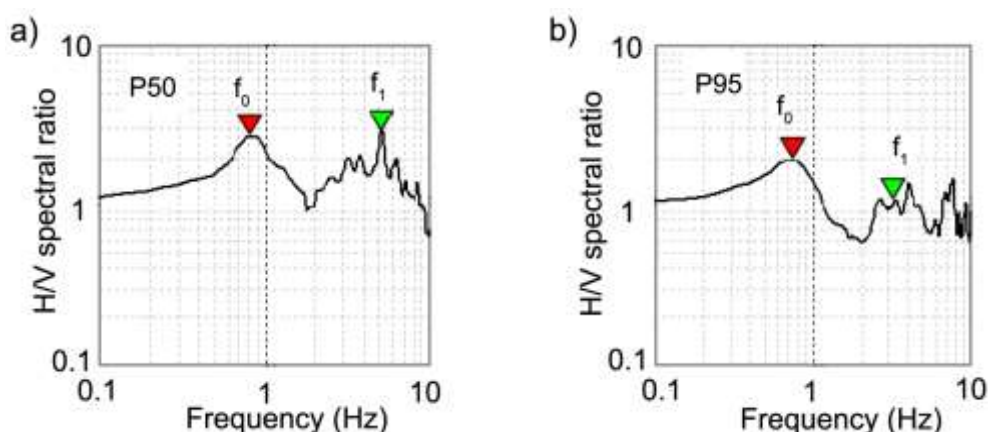


Figure 5.11: Horizontal-to-vertical spectral ratio, a) in microtremor observation point 50, b) in microtremor observation point 95.

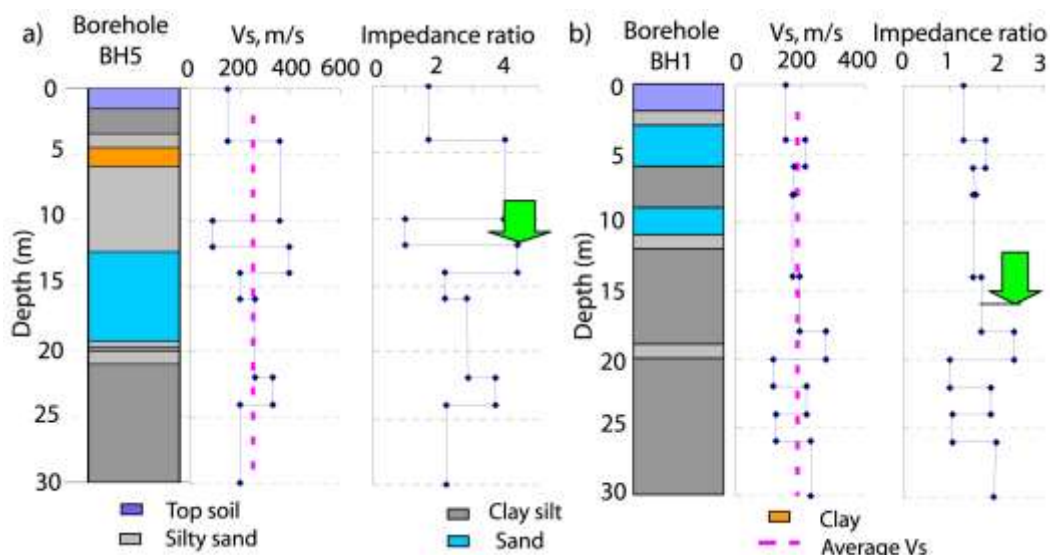


Figure 5.12: Borehole profile, velocity profile and impedance ratio in a) site BH5 and b) site BH1 (refer Figure 5.10). Green downward arrows indicate the calculated depth of the top soil layer based on the microtremor observation point at Point 50 and 95 respectively.

From the above results, it can be said that the basin areas, especially in the central and northern part, have a wider range of frequency than the other part of the valley. Based on the theoretical and numerical investigations, SESAME (2004), Toshinawa et al. (1997), and Gueguen et al. (1998) have mentioned that two-peak H/V spectral ratio occurs due to two impedance contrasts, at two different scales: one for a thick structure, and the other for a shallow structure. Moreover, Lebrun et al. (2001) have shown in Grenoble (France) that the seismic ground motion is amplified in a wide frequency range due to the 2D or 3D effects and/or to the presence of an uppermost sediment layer. They have also mentioned that the wide frequency range is due to the fluvial deposits in the area and that the lateral variability can be strong from one point to another. In the Kathmandu Valley, the northern part generally consists of riverbed materials (Sakai, 2001). Due to the impedance contrast at a shallow depth, the uppermost layer behaves independently from the rest of the sediment-filled column. This assumption was also confirmed by simulation (2D-model) and experimental data (using Standard Spectral Ratio method) performed by Fäh et al. (1994) in the case of the Mexico basin, where the second peak was attributed to the presence of a very soft uppermost clay layer resting on semi-infinite media.

The Kathmandu Valley accommodates a variety of buildings, especially in terms of building materials used and height (i.e., number of stories) of the structures. Most of the places consist of low-rise (1-5 stories) to medium-rise (6-10 stories) buildings. In general, Kramer (1996) has proposed that the fundamental frequency of an N-story building is approximately $10/N$ Hz, which results in an estimated predominant frequency of the Kathmandu Valley buildings to be around 1 Hz to 10 Hz. Moreover, the urban area mostly consists of 3 to 7 storey buildings made of adobe, brick masonry and reinforced cement concrete. When the frequency of a building is equal or close to the frequency of the ground, resonance will take place, leading to an enhanced vibration of the structure and higher possibilities of collapse.

Finally, it can be said that the sediments in the Kathmandu Valley may cause two kinds of problems for the valley buildings during earthquakes, especially in the central and northern part of the basin. One may be from the surface layer and other from the layer underneath. As higher resonant frequencies are characteristic to the surface sediment layer, this may mostly affect the low-rise to medium-rise buildings while the lower frequencies are characteristic to the layer underneath, which may affect the medium-rise to high-rise buildings during an earthquake event.

5.6 CHAPTER SUMMARY

This chapter also has tried to address the issue of earthquake hazard mitigation in the Kathmandu Valley of Nepal using microtremor observations. Being a lacustrine basin with complexity in depositional environment, the Kathmandu Valley has a wide range of sediment layers. As a result, there is inhomogeneity in the sediments, and their response to the seismic waves is different. In this study, we have investigated the response of the surface and the underneath layer during an earthquake using horizontal-to-vertical spectral (H/V) ratio method of microtremor analysis. Single and multiple amplified frequencies were delineated based on the results of the microtremor observations at a total of 172 points covering about 210 km². The investigation results have shown that two amplified frequencies appear on about 20% of the sites, which are mainly distributed in the central and the northern part of the basin. The predominant frequencies vary from 0.5 Hz to 8.9 Hz, whereas the second resonant frequencies vary mostly from 4 Hz to 6 Hz. These two resonance frequencies are characteristic to particular site condition, and they describe the overall seismic site response in two scales, including the deep and surface soil layer. Depending upon the area, especially in the central and northern part, the top 10-20 m of the sediment layer plays an important role in making the second resonant effect in the basin.

It can be said that the sediment in the Kathmandu Valley may create two kinds of problems for valley buildings, one may be from the topmost layer and other may be from the layer underneath. The Urban area of the Kathmandu Valley is mainly composed of low-rise to medium-rise buildings and/or improperly designed houses with high resonance frequencies as well as tall buildings with low resonance frequencies. The multiple amplified frequencies in a particular area can make a resonance effect both for low-rise as well as tall buildings. Therefore, the behavior of the surface layer as well as the layer underneath should be taken into consideration during seismic risk studies in the valley.

CHAPTER 6

6 ESTIMATION OF BASEMENT TOPOGRAPHY OF THE KATHMANDU BASIN

6.1 INTRODUCTION

The intensity of ground motion is a function of earthquake magnitude and distance from the seismic source, as well as local geological condition and topography of the area (Kramer, 1996). The subsurface geology, sediment depth, and material properties in deep sedimentary basins have considerable influence on the ground motion induced by an earthquake, causing significant amplification (i.e., Su et al., 1992; Bonilla et al., 1997; Nath and Thingbaijam, 2011). Seismic waves generated at a source (i.e. in the fault plane) are modulated, when they are propagated through the medium. Although, geological structure of the area is an important factor, local site condition is known to have a great influence on the potential damage resulting from earthquakes (Seed and Idriss, 1969). According to Delgado et al. (2000), soft sediments are the root of numerous geotechnical problems due to their low density, high compressibility and low strength. The characteristics of soft sediments make them potentially collapsible, causing huge settlements of foundations, even under low magnitude of seismic loads.

Besides the geotechnical properties of the local soil site, the behavior of the soil during earthquake depends upon the depth of the sedimentary column and its shear wave velocity. Geometry, shear-wave velocity in the soft sediments and depth of the bedrock are the key parameters, which control the site amplification (Akamatsu et al., 2000; Seed and Alonso, 1974). Seed et al. (1976) have computed response spectra from ground motions recorded at sites underlain by four categories of site conditions: rock sites, stiff sites (less than 61 m deep), deep cohesionless soil sites (greater than 76 m deep), and sites underlain by soft to medium-stiff clay deposits. Figure 6.1 shows the normalized response spectra (by dividing spectral accelerations by the peak ground acceleration) which clearly illustrate the effects of local soil condition on the shapes of the spectra. Figure 6.1 also illustrates that deep and soft soil deposits produce greater proportions of long-period (low-frequency) motion. This effect can be very significant, particularly when long-period structures, such

as tall buildings and long span bridges, are founded on such deposits.

Evaluation of site incorporating the total thickness of sediments, geotechnical properties of the sediment layers as well as other seismic effects, such as edge effects, topographic effects is technically complex and required huge data, computational cost, and time. For instant, generation of three-dimensional real earth model is complex and required much computational time and budget, and also difficult to apply for the wider geological area. Therefore, a simple one-dimensional (1D) model is generally considered by assuming the medium to be horizontally homogeneous and isotropic, but vertically inhomogeneous as accounted for by the impedance contrasts due to younger sediments overlying the older ones (Kramer, 1996).

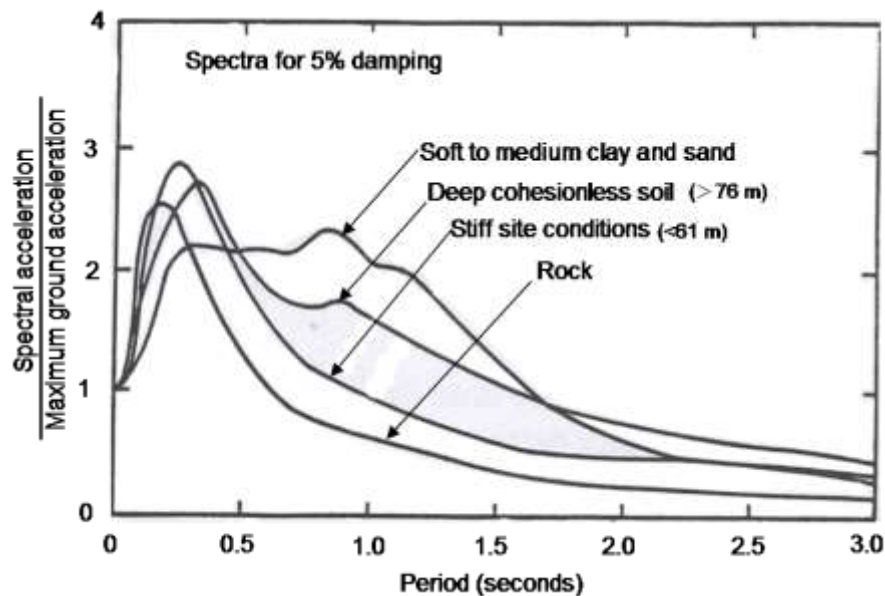


Figure 6.1: Average normalized response spectra for 5% damping for different local site conditions (after Seed et al., 1976).

From an engineering perspective, it is preferred to predict the ground motions at “engineering bedrock” on which the foundations of built structures are laid. From geotechnical point of view, the shear wave velocity in the top 30 m of the soil column is responsible for the unusual amplification of the ground motion (Finn, 1991). According to Satoh et al. (1995), the engineering bedrock is the sedimentary rock of Pliocene or earlier age with the S-wave velocity of 500 m/sec or larger, and a standard penetration test (SPT) blow count of 50 or more. Conventionally, the engineering bedrock has been assumed to be at the uppermost layer, having an S-wave velocity greater than or equal to 760 m/sec of the soil column, in accordance with the National Earthquake Hazard Reduction Program

(NEHRD) provisions (e.g., ICBO, 1997). However, an average S-wave velocity in the uppermost 30 m only gives amplification information relating to 1D site effects and is insensitive to 2D and 3D site effects.

Realistic strong motion estimation at sites located in thick soil deposits depends on a well-known subsurface structure modeling. The thick sediment deposited basin is potentially subject to enhanced seismic hazard by means of trapping and focusing of seismic waves resulting in higher level and longer duration strong ground motions. Moreover, the fundamental phenomenon responsible for the amplification of seismic waves is due to the impedance contrast between sedimentary deposits and the underlying hard-strata or bedrock. Field et al. (2000) have mentioned that the thickness and velocity of soft sedimentary layers overlying the bedrock are considered among the most critical parameters that influence strong ground motions during large earthquakes. Therefore, estimation of sub-surface thickness is very important, particularly in regions where sub-surface geology is poorly known and which is under earthquake threat.

6.1.1 Effect of sediment depth on seismic damages

There are many example of effect of thick sediment deposit for the amplification of ground motion and hence damage of infrastructure during past earthquakes. Some of them are illustrated as follows.

1) 1934 Bihar-Nepal Earthquake

The 1934 Bihar-Nepal earthquake ($M_w=8.1$) (Ambraseys and Douglas, 2004; Hough and Bilham, 2008) which has epicenter about 300 km east from Kathmandu Valley induced heavy damage with a maximum intensity of X in Modified Mercalli Intensity Scale in the Kathmandu Valley (Dunn et al., 1939; Pandey and Molnar, 1988). This earthquake reportedly destroyed about 19% and damaged about 38% of the valley's buildings (Pandey and Molnar, 1988; Rana, 1935). Hough and Bilham (2008) and Pandey and Molnar (1988) have pointed out that the amplification of ground motion and hence heavy damage of infrastructure in the valley is due to the presence of thick fluivo-lacustrine sediments in the Kathmandu Basin.

2) 1967 Caracas Earthquake

The Caracas earthquake ($M_w = 6.6$) of 1967 (Suárez and Nábělek, 1990) caused heavy damage to multi-story buildings and showed that variations of damage to structures can be

directly related to the interacting effects of the structure and the underlying subsurface conditions. According to Seed and Alonso (1974), the damage intensity was highest for 5 to 9 story buildings in which the depth of foundation soil was less than 100 m but for soil depths over 150 m, the structural damage intensity was about eight times higher for buildings with over 14 stories than for structures in the 5 to 9 story range. Figure 6.2 illustrates the strong influence of soil depth on structural performance during an earthquake. Based on the field data and analytical studies, Seed and Alonso (1974) have concluded that soil-structure interaction effects had a large influence on the intensity of structural damage in Caracas in the 1967 earthquake.

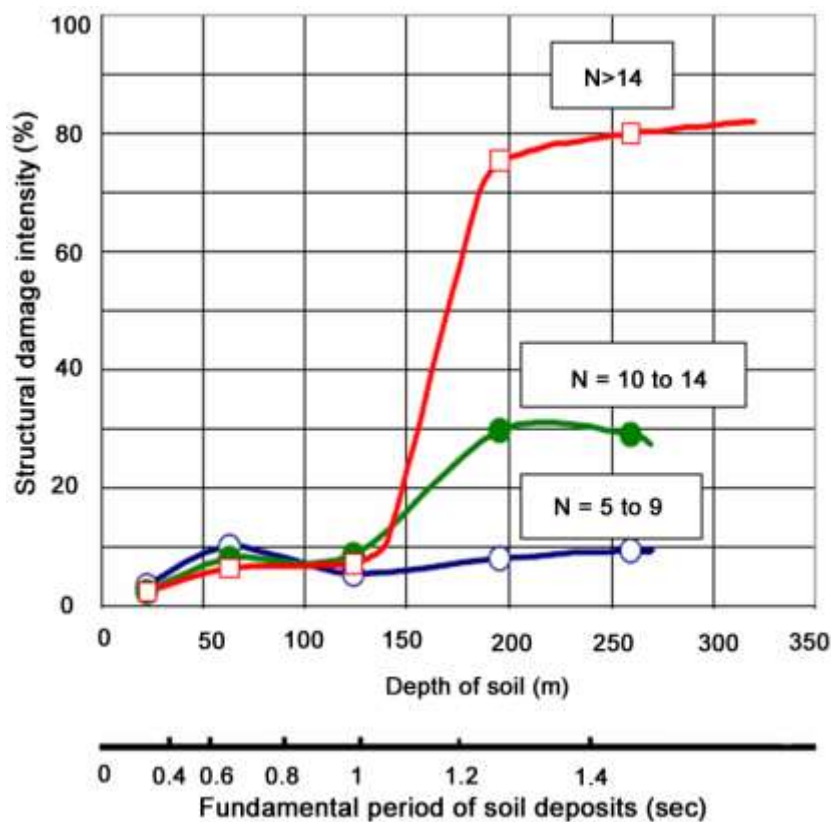


Figure 6.2: Relationship between structural damage intensity and soil depth in the 1967 Caracas earthquake (after Seed and Alonso, 1974).

3) 1985 Michoacan Earthquake

The 1985 Michoacan ($M_s=8.1$) earthquake caused extensive damage some 350 km away of its epicenter in Mexico City but caused only moderate or minor damage in the vicinity in Acapulco City (near the Pacifica cost of Mexico) (Kramer, 1996). From geotechnical as well as other geophysical study, it has been understood that the destruction

in Mexico City was mainly due to the presence of thick pile of loose lake sediments (Kramer, 1996) in the ancient Texcoco Lake. The damage distribution showed that the greatest damage occurred in those portions of the lake zone underlain by 38 to 50 m of soft soil (Stone et al., 1987), where the characteristic site periods were estimated at 1.9 to 2.8 sec but within this area, damage to buildings of less than five stories and modern buildings greater than 30 stories was slight. In such area, most buildings in the range of five-to 20 stories were found either severely damaged or collapsed.

4) 1989 Lima Prieta Earthquake

The 1989, a $M_s=7.1$ earthquake occurred near Mt. Lima Prieta located about 100 km south of San Francisco and Oakland, California. The Loma Prieta earthquake produced MMI VIII shaking in the epicentral region, but intensities were higher, MMI IX, in portions of San Francisco and Oakland which caused extensive damage in certain areas in San Francisco (Kramer, 1996). Rainera et al. (1990) have pointed out that the higher in intensity and extensive damage of infrastructures in San Francisco was due to the presences of largely filled alluvial deposits of clay and silty to sandy clays with some layers of sandy and gravelly soils.

5) 1995 Hyogoken-nambu Earthquake

The Hyogoken-nambu (Kobe) earthquake of January 17, 1995 ($M_w=6.9$) (Kitagawa and Hiraishi, 2004) brought destructive damage to Kobe and Hanshin area in the northern margin of the Osaka Basin. The earthquake damage was also distributed in the eastern part of the Osaka Basin located more than 30 km from source region. The damage of buildings and other infrastructure suggest that the bedrock configuration may affect the damage distribution significantly. Akamatsu et al. (1996) have pointed out that the step-like configuration of bedrock may cause focusing of seismic waves as well as interference of secondary-generated surface waves and incident S-waves which brings large amplitude of seismic waves responsible for the distribution of earthquake damage.

From above observation, it can be said that sediment thickness and bedrock topography play an important role for the amplification of ground motion and hence damage and destruction of infrastructure during strong earthquakes. Akamatsu et al. (2000) have concluded that it is very important to clarify the detailed structure of the bedrock and its effects on ground motion characteristics during strong ground motion in a sedimentary basin. Therefore, knowledge of the thickness of soft soil layers as well as their spatial

variation within a region is of great interest for land-use planning and for geotechnical earthquake engineering.

6.1.2 Bedrock topography of the Kathmandu Basin

The Kathmandu Complex is composed of the Shivapuri Gneiss and Marbles of the Bhimphedi group (Stöckline and Bhattarai, 1981). As mentioned in Figure 2.19 of Chapter 2, the northern slope of the Kathmandu Basin is mainly composed of gneiss, schist and granite, but the other slopes and the central part of the valley consists of weakly metamorphic rock. According to Moribayashi and Maruo (1980), the density of rocks in the Kathmandu Basin varies from 2.61 to 2.75 g/cm³, which also represent the density of rocks such as Paleozoic Sandstone, Phyllite, Limestone and Precambrian gneiss.

Morbayashi and Maruo (1980) were estimated the basement topography of the central portion of the Kathmandu Basin using the Gravitational method. They carried out the gravity measurements in November 1976 at 112 gravity stations along the road side with in the valley and prepared a gravity contour map as shown in Figure 6.3. This map showed prominent gravity lows around the central part of the study area, and gravity values increase towards the peripheries of the valley. They have proposed the basement topography contour map of the center portion of the basin as shown in Figure 6.4 by assuming the density contrast of 0.8 g/cm³: 2.67 g/cm³ for bedrock and 1.87 g/cm³ for lacustrine sediments. They assumed that the average density of the Quaternary sediments might be greater than 1.5 g/cm³. According to their study, the maximum thickness of the sediment at the central part of the basin reaches upto 650 m.

6.1.3 Comments in basement topography proposed by Moribashi and Maruo (1980)

- ◆ According to JICA (2002) report, the average density of lacustrine sediments upto 30 m depth is found 1.55 g/cm³ which is less than the assumed value (i.e., 1.87 g/cm³) by Moribayashi and Maruo (1980). Therefore, the basement contour value estimated based on the assumed value may not represent the actual depth of the basement in the Kathmandu Basin.
- ◆ The density contrast between basement rock and the sediment may not be constant everywhere in the basin, since thickness and properties of the sediments are varied within the short distances.
- ◆ As mentioned by Stöckline and Bhattarai (1981) and Moryabashi and Maruo (1980), the types of the basement rocks are found different from area to area in

the Kathmandu Basin, and hence their density will be different according to the rock types. Therefore, it is very difficult to expect constant density contrast everywhere in the basin.

- ◆ The gravity survey points were not uniformly distributed in their study area. It seemed that data were taken sparsely along the roadside of the basin which may not give in-depth information of the area.
- ◆ Study area covered only the central part of the Kathmandu City and some part of the Lalitpur City. The eastern part of the Kathmandu Basin (i.e. Bhaktapur City) was not covered.

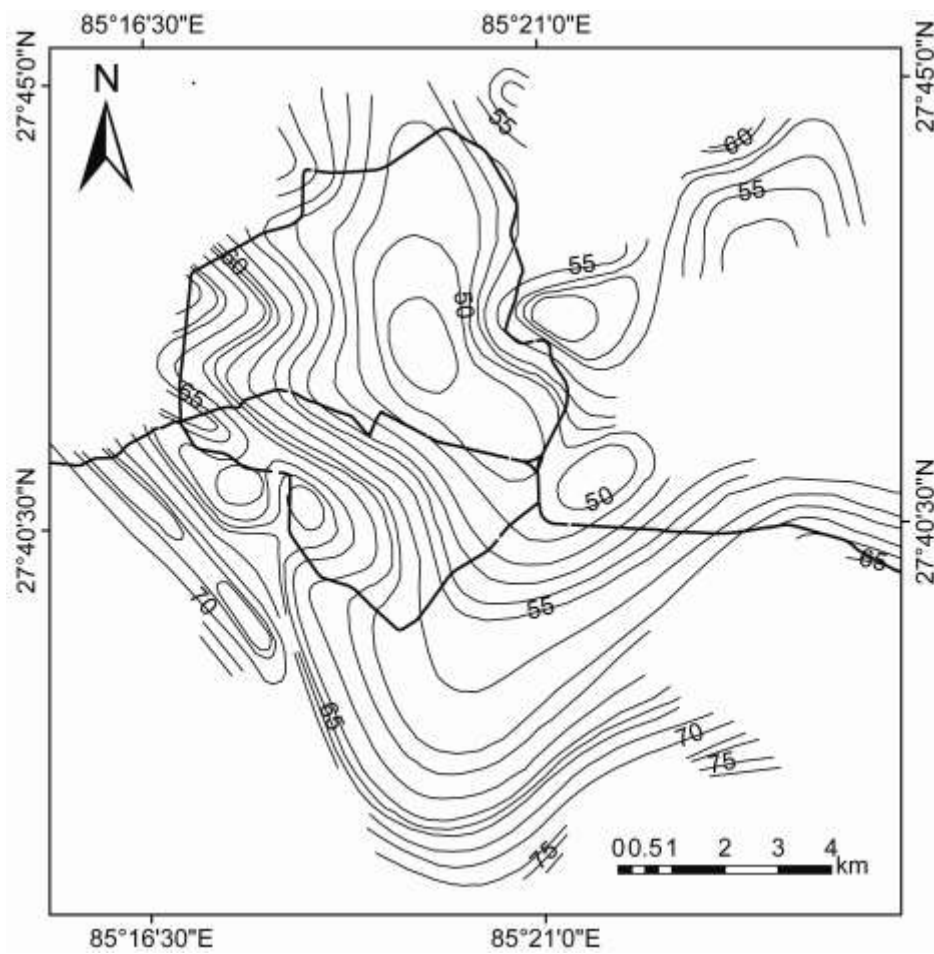


Figure 6.3: Gravity contour map in the Kathmandu Valley (redrawn after Moribayashi and Maruo, 1980).

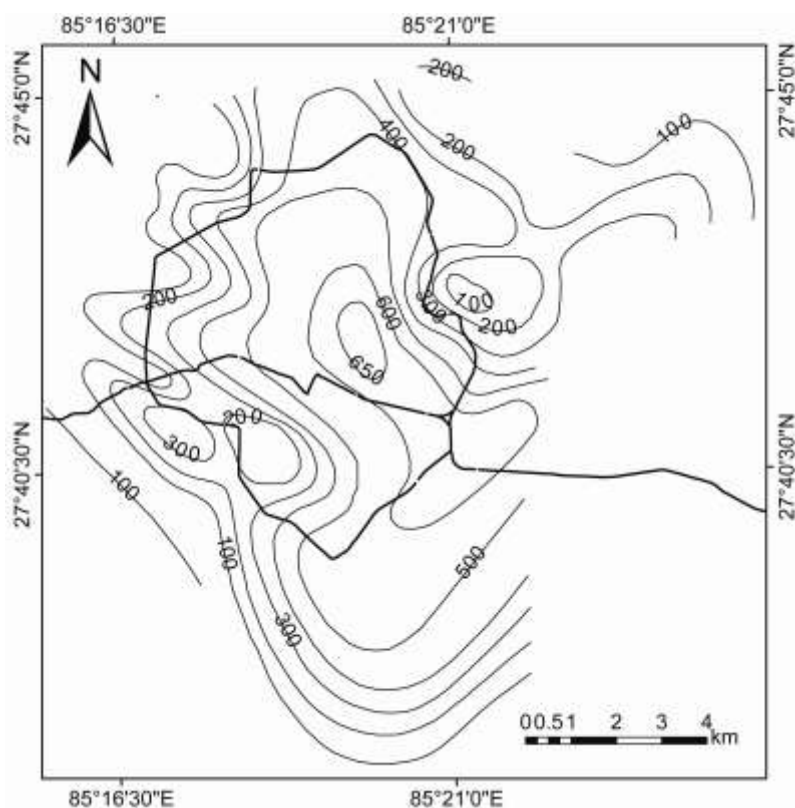


Figure 6.4: Basement contour map in the Kathmandu Valley estimated from density difference of 0.8 g/cm³ (redrawn after Moribayashi and Maruo, 1980).

6.2 PURPOSE OF THIS STUDY

In spite of the known fact about the high seismic hazard in the Kathmandu Valley, there are very few scientific research-activities towards the mitigation of earthquake hazard risk in the valley. As stated elsewhere in this chapter, the sediment thickness and its properties play a major role in intensifying the ground motion. However, the Kathmandu Basin still lacks adequate information on spatial variation of the lacustrine sediments. The basement topography proposed by Moribayashi and Maruo (1980) has not yet reconfirmed and is still debatable. The present study is the first attempt to explore the seismic hazard risk in different location of the valley by estimating the depth of lacustrine sediment which plays an important role for amplification of ground motion during earthquakes. The main objectives of this study are.

- ◆ To evaluate the amplification effect of the thick sediment and its role for the damage of buildings.
- ◆ To study of the use of horizontal-to-vertical spectral ratio method for the evaluation of sediment depth in different geological and geographical region of the world.

- ◆ To derive an empirical relationship between the resonance frequencies obtained from the H/V technique and the thickness of the lacustrine sediment for the Kathmandu Basin.
- ◆ To generate an approximate basement topography of the Kathmandu Basin.
- ◆ To generate the cross profiles along north-south and west-east direction to see the variation of soft sediments in the valley.

6.3 MATERIALS AND METHOD

6.3.1 Area covered

The detailed description of the study area is mentioned in Chapter, 2, Section 2.4. Figure 6.5 shows the microtremor observation points, sediment distributions, and the location of the borehole that is taken for the verification of the calculated depth in this study.

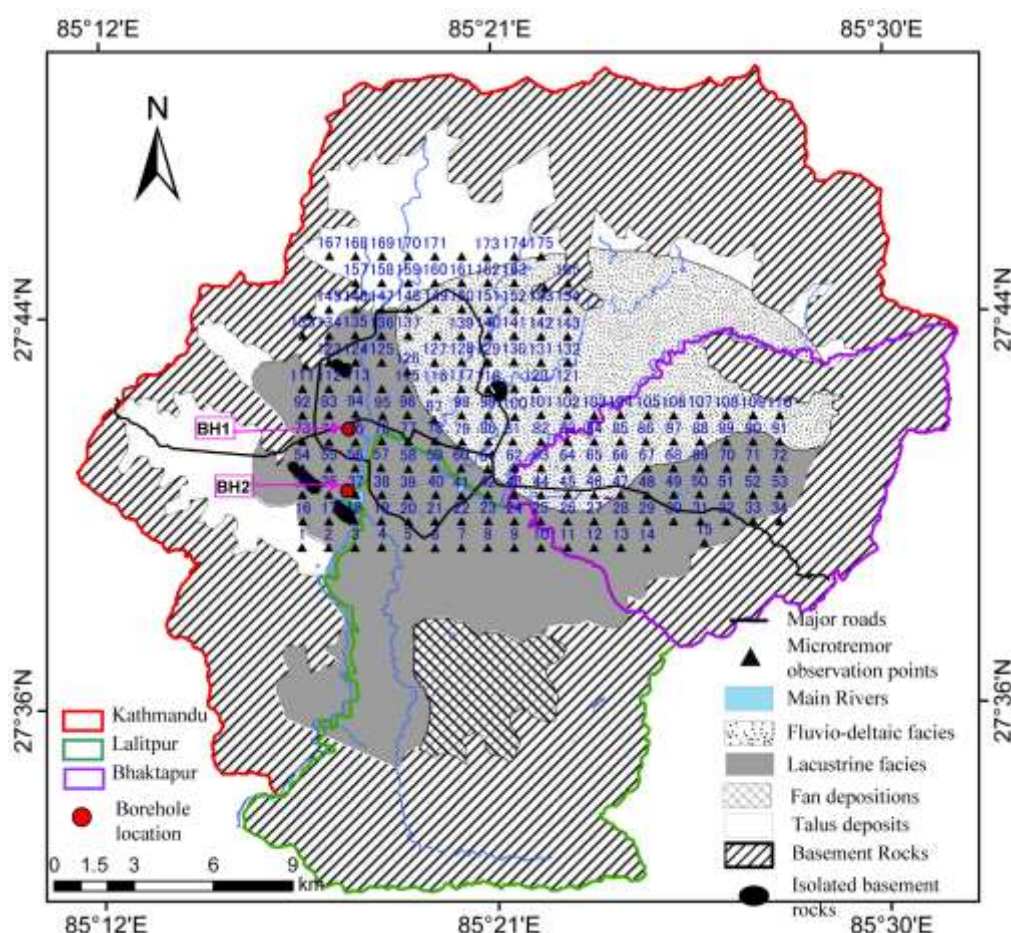


Figure 6.5: Location map of the Kathmandu Valley (study area). A sediment distribution map of the Kathmandu Valley, microtremor measurement points and borehole location (BH1 and BH2) in the study area are shown (modified after Fujii and Sakai, 2002).

6.3.2 Methodology adopted

The methodology adopted in this study mainly consists of collection of microtremor data and analyze them using horizontal-to-vertical (H/V) spectral ratio technique to obtain the resonance frequency of each predetermined location of the Kathmandu Basin. As mentioned in Chapter 3, Section 3.4.2.4, the H/V method is a low-cost and effective tool to estimate the fundamental resonant frequency of sediments by measuring the microtremors in a single-station. It has widely applied in the last two decade for different purposes, such as site effects evaluation, wave amplification estimation, liquefaction vulnerability assessment, sediment depth estimation and microzonation studies in different geographical and geological regions of the world (Bour et al., 1998; Delgado et al., 2000; Field and Jacob, 1993; Hardesty et al., 2010; Hasancebi and Ulnsay 2006; Ibs-von Seht and Wohlenberg, 1999; Langston et al. 2009; Mucciarelli, 2011; Tuladhar et al., 2004). Several studies, such as Bour et al. (1998), Cadet et al. (2011), Chatelain et al. (2008), Fäh et al., (2001), Field and Jacob (1993), Gosar (2007), Gueguen et al. (2007), Herak (2011), Hasancebi and Ulnsay (2006), Huang (2002), Huang et al. (2002), Lunedei and Albarello (2009, 2010), Mucciarelli et al. (2003), Sánchez-Sesma et al. (2011), Theodulidis et al. (1996), Theodulidis et al. (2005), Theves-Costa et al. (1996), Tuan et al. (2011), Walling et al. (2009), and Zandieh and Pezeshk (2011) have concluded that the ambient noise H/V spectral ratios provide reliable estimates of the fundamental frequencies of soil deposits and also demonstrated that the frequency of the resonating waves in poorly consolidated sediments is related to the shear wave velocity as well as the thickness of the sediment. Moreover, Birgöen et al. (2009), D'Amico et al. (2004), Delgado et al. (2000), Dinesh et al. (2010), Garcia-Jerez et al. (2006), Gosar and Lenart (2010), Hinzen et al. (2004), Ibs-von Seht and Wohlenberg (1999), Özalayboy et al. (2011), Parolai et al. (2002), and Sukumaran et al. (2011) have applied microtremor H/V spectral ratio technique for site investigation and measuring the thickness of the soil cover over the hard stratum or bedrock.

In this study, the fundamental resonant frequency of the soil layer is used to map the thickness of the soft sediments in the Kathmandu Basin. The detail analysis method to obtain the resonance frequency is presented in Chapter 4, Section 4.2.3.3. Typical results of microtremor data analysis and calculation of predominant frequency of the sites in some of the location of Kathmandu Basin is shown in Figure 6.6.

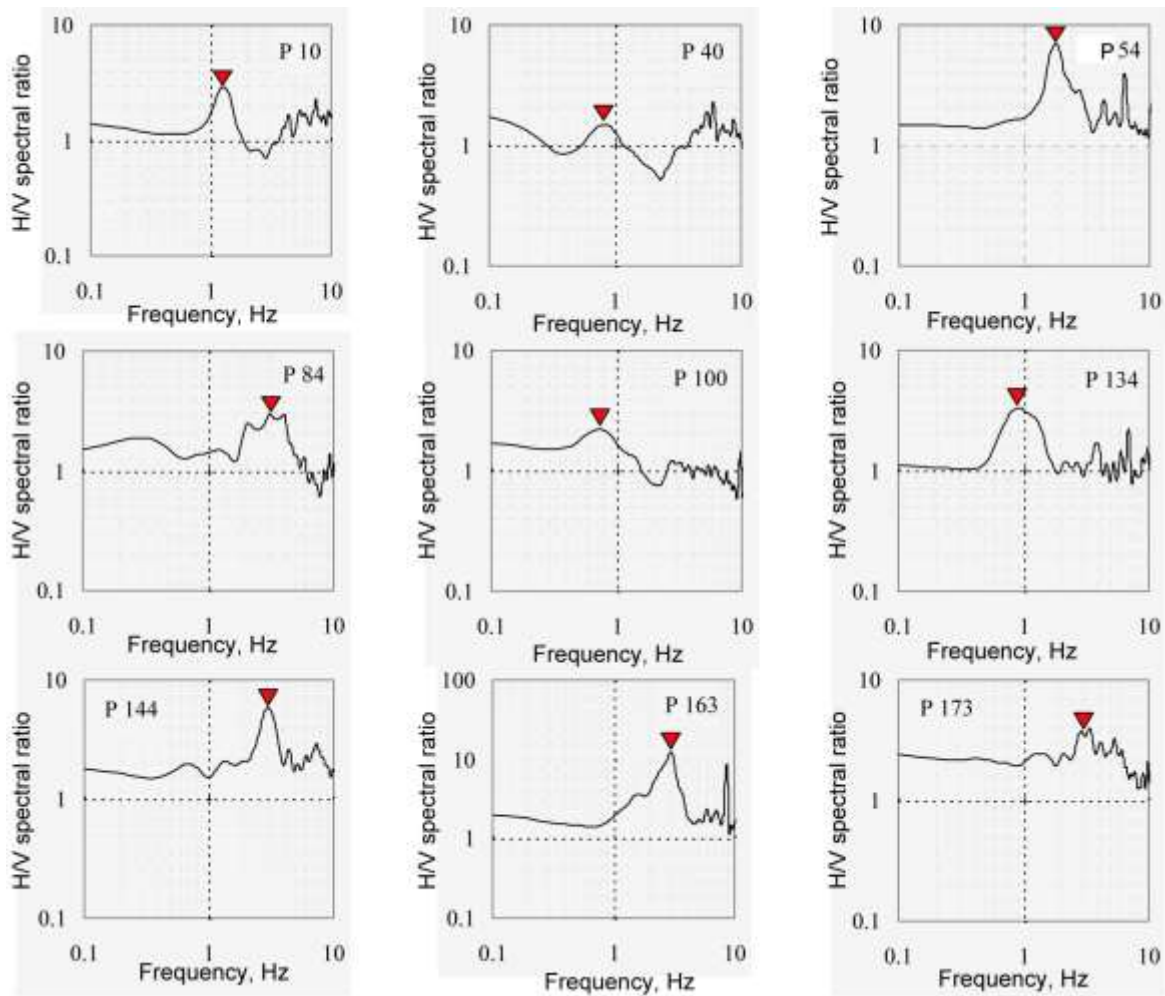


Figure 6.6: Typical H/V spectral ratio of some microtremor measurement points in the study area. Red pointed triangle represents the predominant frequency taken from H/V spectral ratio.

6.3.3 Limitation and assumptions

In this study, it is assumed that the H/V spectral ratio depends primarily on the source/site characteristics rather than geographical location. According to Bonnefoy-Claudet et al. (2006a), microtremors are small scale vibrations of ground caused by multiple random sources such as water waves, turbulent wind, effects of wind on trees or buildings, industrial machinery, and car and heavy vehicles or human activities. Bonnefoy-Claudet et al. (2006a), and Gosar and Martinez (2009) have mentioned that the characteristics of microtremor are different from site to site, and types of waves in microtremor are still debatable. In the context of Kathmandu Basin, it is assumed that the main source of microtremors can be vehicles which are moving 24-hours in the cities of Kathmandu, Lalitpur and Bhaktapur, and the industries in and around the Kathmandu

Valley. Heavy vehicles such as buses, heavy lorry, heavy trucks, and cars cross the basin edge, which is very near and on the bed rock of the Kathmandu Basin, and are continuously running in all direction of the valley. The source may be the water movement in rivers in the valley and also the waver movement along the rivers which pass along the bedrock of the basin. Other possible sources of microtremor may also be due to the human activities (i.e., construction activities etc.) and the effects of winds on trees and buildings inside the valley and around the basin. It is assumed that the recorded microtremors are not only from the nearby sources but also they have traveled some distance and recorded in the instrument after traveling up to the basement depth of the basin.

6.4 RESULTS AND DISCUSSION

6.4.1 Sediment depth and predominant frequency relationship for the Kathmandu Basin

Ibs-von Seht and Wohlenberg (1999) have showed that the fundamental resonant frequency of the soil layer is closely related to thickness of the soil layer as given in Equation (6.1).

$$h = af_r^b \dots\dots\dots (6.1)$$

Where ‘a’ and ‘b’ are the standard errors of the correlation coefficients.

Ibs-von Seht and Wohlenberg (1999) have studied both parameters (i.e, h and f_r) and demonstrated that it is possible to establish a direct functional relationship between them without knowing shear wave velocity (V_s). They have estimated the value of ‘a’ and ‘b’ and proposed an empirical relationship (Equation 6.2) relating the fundamental resonant frequency (f_r) to the thickness of soft sediment cover (h) (Quaternary sediments), based on 34 boreholes ranging in depth from 15 m to 1,257 m and data from 102 seismic station in the western Lower Rhine Embayment in Germany which is covered with Tertiary and Quaternary sediments overlaying Palaeozoic bedrock. Similarly, Parolai et al. (2002) developed an empirical relationship (Equation 6.3) between thickness of sediment with resonant frequency for the Cologne area in Germany based on 32 boreholes with a depth range from 20 m to 402 m and 337 data from seismic stations. Recently, Birgöen et al. (2009) have derived an empirical relationship (Equation 6.4) between the thickness of Tertiary-Quaternary sediments overlying Palaeozoic bedrock and their resonance frequencies for the Istanbul region based on the H/V ratio from 15 measurements at the borehole locations and velocity profile of 2 microtremor array measurements sites. The

results obtained by Birgöen et al. (2009) showed very strong relationship (R^2 value: 0.995) between resonant frequency and thickness of the sediment which varies from 20 m to 449 m. Similarly, Özalaybey et al. (2011) have derived an equation (Equation 6.5) for sediment cover in Izmit Basin in Turkey which has the sedimentary cover thickness about 1200 m at the deepest part.

$$h = 96 f_r^{-1.388} \dots\dots\dots (6.2)$$

$$h = 108 f_r^{-1.551} \dots\dots\dots (6.3)$$

$$h = 150.99 f_r^{-1.1531} \dots\dots\dots (6.4)$$

$$h = 141 f_r^{-1.27} \dots\dots\dots (6.5)$$

To map the soft sediment thickness in the Kathmandu Basin, we adopt terrain specific equations given by above researchers. A theoretical thickness for the sediments of the Kathmandu Basin (refer Table 6.1) is calculated using above equations, based on the fundamental frequency (f_r) obtained for each stations using microtremor observations.

Table 6.1: Calculated thickness of sediments over hard stratum or bedrock using Ibs-von Seht and Wohlengerg (1999), Parolai et al. (2002), Birgöen et al. (2009), and Özalaybey et al. (2011) equations for 172 microtremor measurement points in the Kathmandu Basin.

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
1	1.10	84	93	135	125
2	2.00	37	37	68	58
3	5.00	10	9	24	18
4	1.30	67	72	112	101
5	1.40	60	64	102	92
6	1.20	75	81	122	112
7	0.92	108	123	166	157
8	0.95	103	117	160	150

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
9	1.22	73	79	120	110
10	1.22	73	79	120	110
11	1.00	96	108	151	141
12	0.78	136	159	201	193
13	1.46	57	60	98	87
14	1.12	82	91	132	122
15	3.05	20	19	42	34
16	0.83	124	144	187	179
17	3.60	16	15	34	28
18	0.85	120	139	182	173
19	1.80	42	43	77	67
20	1.22	73	79	120	110
21	1.22	73	79	120	110
22	1.10	84	93	135	125
23	1.20	75	81	122	112
24	1.00	96	108	151	141
25	1.10	84	93	135	125
26	1.46	57	60	98	87
27	0.78	136	159	201	193
28	2.00	37	37	68	58
29	0.90	111	127	170	161
30	2.35	29	29	56	48
31	0.60	195	239	272	270
32	1.52	54	56	93	83
33	1.85	41	42	74	65
34	1.56	52	54	90	80
35	1.30	67	72	112	101

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
36	2.33	30	29	57	48
37	0.68	164	196	236	230
38	1.10	84	93	135	125
39	1.32	65	70	110	99
40	0.80	131	153	195	187
41	0.90	111	127	170	161
42	1.32	65	70	110	99
43	1.10	84	93	135	125
44	1.30	67	72	112	101
45	0.93	106	121	164	155
46	1.32	65	70	110	99
47	1.70	46	47	82	72
48	4.00	14	13	31	24
49	1.30	67	72	112	101
50	0.79	133	155	198	190
51	0.83	124	144	187	179
52	1.26	70	75	116	105
53	0.70	157	188	228	222
54	1.95	38	38	70	60
55	1.05	90	100	143	133
56	1.97	38	38	69	60
57	1.00	96	108	151	141
58	1.40	60	64	102	92
59	0.80	131	153	195	187
60	0.87	116	134	177	168
61	1.50	55	58	95	84
62	0.83	124	144	187	178

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
63	1.20	75	81	122	112
64	2.00	37	37	68	58
65	1.12	82	91	132	122
66	1.30	67	72	112	101
67	1.00	96	108	151	141
68	1.10	84	93	135	125
69	1.20	75	81	122	112
70	1.41	60	63	102	91
71	1.41	60	63	102	91
72	1.02	93	105	148	137
73	1.70	46	47	82	73
74	0.98	99	111	155	145
75	1.05	90	100	143	133
76	1.22	73	79	120	110
77	0.88	115	132	175	166
78	0.75	143	169	210	203
79	0.80	131	153	195	187
80	0.83	124	144	187	179
81	1.02	93	105	148	137
82	1.46	57	60	98	87
83	3.00	21	20	43	35
84	3.32	18	17	38	31
85	1.22	73	79	120	110
86	0.73	149	176	217	210
87	1.02	93	105	148	137
88	1.60	50	52	88	78
89	1.80	42	43	77	67

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
90	0.90	111	127	170	161
91	1.20	75	81	122	112
92	1.26	70	75	116	105
93	0.88	114	131	174	165
94	0.62	186	226	262	258
95	0.73	149	176	217	210
96	0.68	164	196	236	230
97	0.83	124	144	187	179
98	0.93	107	121	165	155
99	0.83	124	144	187	178
100	0.73	148	175	216	210
101	1.66	48	49	84	74
102	1.30	67	72	112	101
103	0.90	111	127	170	161
104	0.87	116	134	177	168
105	0.87	116	134	177	168
106	1.40	60	64	102	92
107	1.45	57	61	98	88
108	1.42	59	63	101	90
109	2.00	37	37	68	58
110	1.40	60	64	102	92
111	1.66	48	49	84	74
112	3.00	21	20	43	35
113	0.90	111	127	170	161
114	0.63	180	219	255	251
115	0.59	202	248	280	279
116	0.49	260	328	345	350

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
117	0.59	202	247	280	278
118	1.56	52	54	90	80
119	1.69	46	48	82	72
120	1.32	65	70	110	99
121	1.17	77	85	126	116
122	3.00	21	20	43	35
123	2.80	23	22	46	38
124	1.42	59	63	101	91
125	0.54	228	283	309	311
126	0.78	136	159	201	193
127	0.90	111	127	170	161
128	0.68	164	196	236	230
129	3.66	16	14	34	27
130	1.00	96	108	151	141
131	1.20	75	81	122	112
132	0.90	111	127	170	161
133	8.90	5	4	12	9
134	0.90	111	127	170	161
135	1.20	75	81	122	112
136	1.10	84	93	135	125
137	0.75	143	169	210	203
138	2.50	27	26	52	44
139	0.63	180	219	255	251
140	1.22	73	79	120	110
141	0.90	111	127	170	161
142	1.07	87	97	140	129
143	1.17	77	84	126	115

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
144	3.00	21	20	43	35
145	1.00	96	108	151	141
146	3.00	21	20	43	35
147	0.80	131	153	195	187
148	0.80	131	153	195	187
149	0.90	111	127	170	161
150	0.80	131	153	195	187
151	0.95	103	117	160	150
152	1.52	54	56	93	83
153	1.00	96	108	151	141
154	1.12	82	90	132	122
157	1.36	63	67	106	95
158	1.21	74	80	121	111
159	1.10	84	93	135	125
160	0.80	131	153	195	187
161	1.37	62	67	105	95
162	1.61	50	52	87	77
163	3.00	21	20	43	35
164	1.81	42	43	76	67
165	3.00	21	20	43	35
167	1.51	54	57	94	84
168	4.40	12	11	27	21
169	3.81	15	14	32	26
170	1.17	77	85	126	116
171	1.12	82	90	132	122
172	8.00	5	4	14	10
173	3.00	21	20	43	35

Microtremor observation point	Frequency (f_r) (Hz)	Thickness (m)			
		(Ibs-von Seht and Wohlenberg, 1999) $H = 96f_r^{-1.388}$	(Parolai et al., 2002) $h = 108f_r^{-1.551}$	(Birgöen et al, 2009) $H = 150.99f_r^{-1.1531}$	(Özalaybey et al., 2011) $h = 141f_r^{-1.27}$
174	1.40	60	64	102	92
175	1.20	75	81	122	112

From Table 6.1, it has been observed that the variations of estimated depth, based on above four equations, was not much comparable with each other and the analysis clearly showed an average standard deviation of 41.88 m in thickness as shown in Figure 6.7.

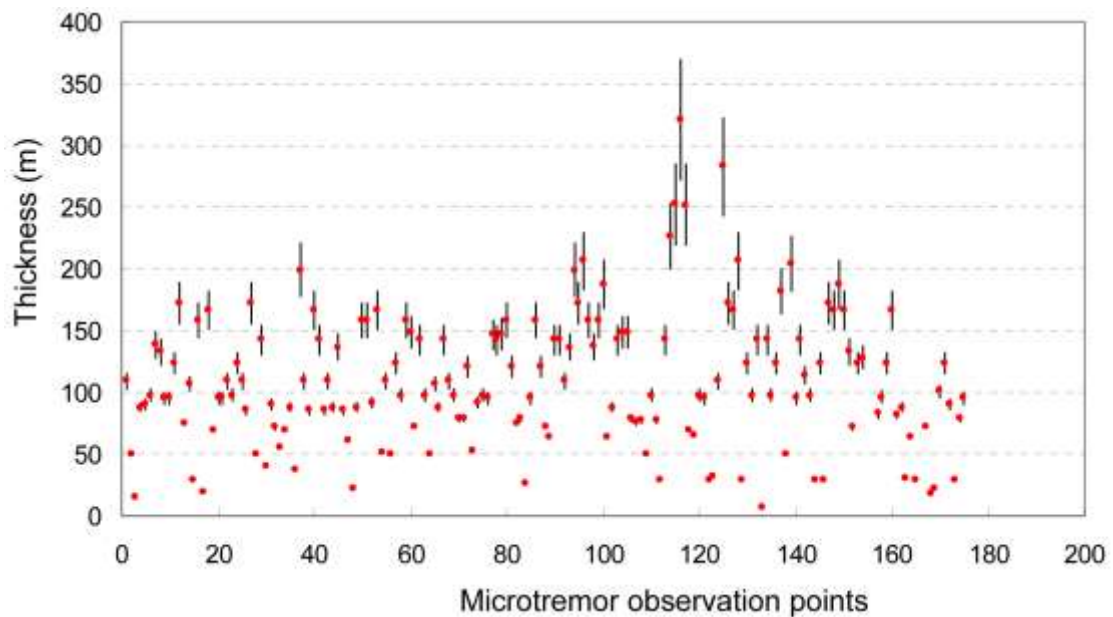


Figure 6.7: Comparison between depths calculated using Ibs-von Seht and Wohlenberg (1999), Parolai et al. (2002), Birgöen et al. (2009) and Özalaybey et al. (2011) relationships (Equations (6.2), (6.3), (6.4), and (6.5)). The circle indicates the average value whereas the length of the line suggests deviation from the average.

In order to minimize the value of standard deviation and thereby to obtain the reliable results, the depth estimated based on the non-linear regression equations proposed by the

four researchers (refer Table 6.1) are divided into two groups emphasizing the less variations of the estimated depth in each group. In this context, in first group, the depth of the Kathmandu Basin sediments is estimated using Ibs-von Seht and Wohlenberg (1999) and Parolai et al. (2002) equations, since these equations showed more comparable results with each other (refer Table 6.1). Similarly, in second group, the depth is estimated using Birgöen et al. (2009), and Özalaybey et al. (2011) equations. The standard deviation of each group is obtained as shown in Figure 6.8 and Figure 6.9. These Figures show that the standard deviation (i.e., 48.55 m in thickness) of group first (i.e. Ibs-von Seht and Wohlenberg, 1999; Parolai et al., 2002) (Figure 6.8) is higher than (i.e., 7.44 m in thickness) the group second (Birgöen et al., 2009; Özalaybey et al., 2011) (Figure 6.9). The results also show that the values of thickness obtained from Birgöen et al. (2009) and Özalaybey et al. (2011) are more compatible with each other. In other words, the depths calculated using Birgöen et al. (2009) and Özalaybey et al. (2011) show significantly smaller variations in the thickness due to the comparable geotechnical characteristics of the geological formation.

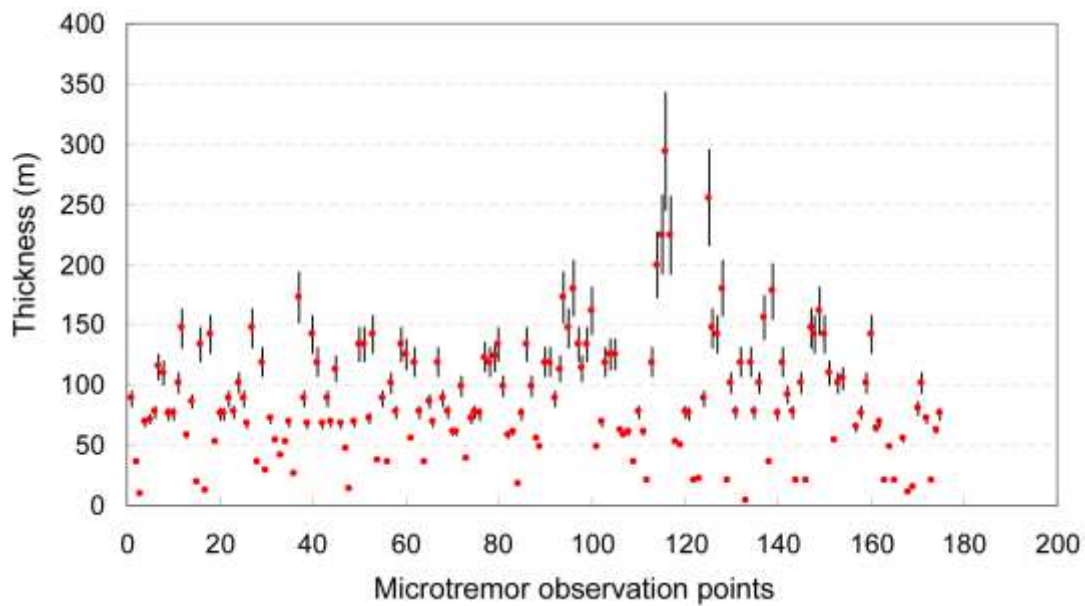


Figure 6.8: Comparison between depths calculated using Ibs-von Seht and Wohlenberg (1999), and Parolai et al. (2002) relationships (Equations (6.2) and (6.3)). The circle indicates the average value whereas the length of the line suggests deviation from the average.

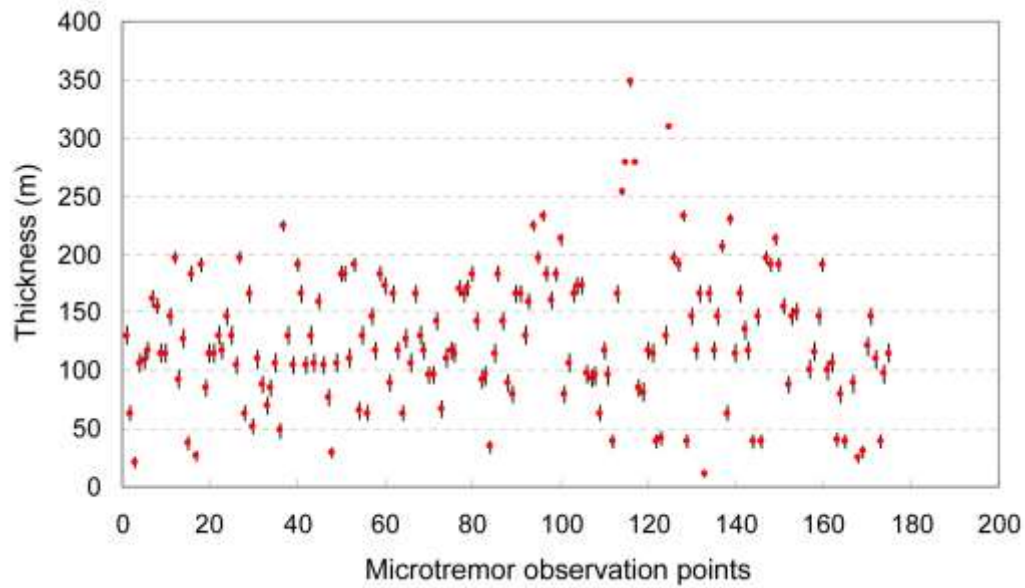


Figure 6.9: Comparison between depths calculated using Birgöen et al. (2009) and Özalaybey et al. (2011) relationships (Equations (6.4) and (6.5)). The circle indicates the average value whereas the length of the line suggests deviation from the average.

We further averaged the values estimated using Equation 6.4 and Equation 6.5 to obtain the best fit equation which we purposed for the Kathmandu Basin as follows.

$$h = 146.01 f_r^{-1.2079} \dots\dots\dots (6.6)$$

The obtained equation (i.e. Equation 6.6) is further used for obtaining primary information on the relative depth variation (refer Table 6.2) of the interface between the two physically contrasting layers of lacustrine sediment and the underlain hard strata (or bedrock) in the Kathmandu Basin.

Table 6.2: Relative variation of the thickness of sediment in the Kathmandu Basin.

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
1	85.276	27.655	1331	1.10	130
2	85.286	27.655	1280	2.00	63
3	85.296	27.655	1359	5.00	21

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Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
4	85.307	27.655	1318	1.30	106
5	85.317	27.654	1331	1.40	97
6	85.327	27.654	1325	1.20	117
7	85.337	27.654	1290	0.92	161
8	85.347	27.654	1307	0.95	155
9	85.357	27.654	1301	1.22	115
10	85.367	27.654	1306	1.22	115
11	85.378	27.654	1315	1.00	146
12	85.388	27.654	1312	0.78	197
13	85.398	27.654	1327	1.46	92
14	85.408	27.654	1375	1.12	127
15	85.430	27.655	1350	3.05	38
16	85.277	27.664	1338	0.83	183
17	85.286	27.664	1301	3.60	31
18	85.296	27.664	1274	0.85	178
19	85.307	27.664	1280	1.80	72
20	85.317	27.663	1321	1.22	115
21	85.327	27.663	1346	1.22	115
22	85.337	27.663	1273	1.10	130
23	85.347	27.663	1284	1.20	117
24	85.357	27.663	1285	1.00	146
25	85.367	27.663	1299	1.10	130
26	85.378	27.663	1303	1.46	92
27	85.388	27.663	1309	0.78	197
28	85.398	27.663	1322	2.00	63
29	85.408	27.663	1334	0.90	166
30	85.418	27.662	1348	2.35	52

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
31	85.429	27.662	1326	0.60	271
32	85.439	27.662	1313	1.52	88
33	85.449	27.662	1302	1.85	69
34	85.459	27.662	1322	1.56	85
35	85.276	27.673	1351	1.30	106
36	85.287	27.673	1282	2.33	53
37	85.296	27.673	1272	0.68	233
38	85.307	27.673	1301	1.10	130
39	85.317	27.672	1347	1.32	104
40	85.327	27.672	1367	0.80	191
41	85.337	27.672	1273	0.90	166
42	85.347	27.672	1269	1.32	104
43	85.358	27.672	1303	1.10	130
44	85.368	27.672	1285	1.30	106
45	85.378	27.672	1291	0.93	159
46	85.388	27.672	1289	1.32	104
47	85.398	27.672	1292	1.70	77
48	85.408	27.672	1310	4.00	27
49	85.418	27.671	1334	1.30	106
50	85.428	27.671	1348	0.79	194
51	85.439	27.671	1310	0.83	183
52	85.449	27.671	1337	1.26	110
53	85.459	27.671	1328	0.70	225
54	85.277	27.682	1325	1.95	65
55	85.287	27.682	1316	1.05	138
56	85.297	27.682	1271	1.97	65
57	85.307	27.682	1305	1.00	146

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Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
58	85.317	27.681	1290	1.40	97
59	85.327	27.681	1278	0.80	191
60	85.337	27.681	1275	0.87	173
61	85.347	27.681	1244	1.50	89
62	85.358	27.681	1285	0.83	183
63	85.368	27.681	1319	1.20	117
64	85.378	27.681	1311	2.00	63
65	85.388	27.681	1328	1.12	127
66	85.398	27.681	1302	1.30	106
67	85.408	27.681	1303	1.00	146
68	85.419	27.681	1305	1.10	130
69	85.429	27.680	1307	1.20	117
70	85.439	27.680	1315	1.41	96
71	85.449	27.680	1315	1.41	96
72	85.459	27.680	1317	1.02	143
73	85.277	27.691	1287	1.70	77
74	85.287	27.691	1290	1.10	130
75	85.297	27.691	1300	1.05	138
76	85.307	27.691	1280	1.22	115
77	85.317	27.691	1265	0.88	170
78	85.327	27.691	1270	0.75	207
79	85.338	27.690	1291	0.80	191
80	85.348	27.690	1290	0.83	183
81	85.357	27.690	1311	1.02	143
82	85.368	27.690	1293	1.46	92
83	85.378	27.690	1296	3.00	39
84	85.388	27.690	1322	3.32	34

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
85	85.398	27.690	1324	1.22	115
86	85.408	27.690	1326	0.73	214
87	85.419	27.690	1326	1.02	143
88	85.429	27.689	1350	1.60	83
89	85.439	27.689	1308	1.80	72
90	85.449	27.689	1333	0.90	166
91	85.459	27.689	1329	1.20	117
92	85.277	27.700	1322	1.26	110
93	85.287	27.700	1282	0.88	170
94	85.297	27.700	1298	0.62	260
95	85.307	27.700	1304	0.73	214
96	85.317	27.700	1283	0.68	233
97	85.327	27.698	1289	0.83	183
98	85.338	27.699	1318	0.93	160
99	85.348	27.699	1298	0.83	182
100	85.358	27.699	1327	0.73	213
101	85.368	27.699	1325	1.66	79
102	85.378	27.699	1306	1.30	106
103	85.388	27.699	1293	0.90	166
104	85.398	27.699	1309	0.87	173
105	85.409	27.699	1331	0.87	173
106	85.419	27.699	1366	1.40	97
107	85.429	27.699	1332	1.45	93
108	85.439	27.698	1323	1.42	96
109	85.449	27.698	1336	2.00	63
110	85.459	27.698	1339	1.40	97
111	85.277	27.709	1318	1.66	79

Chapter 6: Estimation of basement topography of the Kathmandu Basin

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
112	85.287	27.709	1315	3.00	39
113	85.298	27.709	1296	0.90	166
114	85.307	27.709	1314	0.63	253
115	85.317	27.708	1297	0.59	279
116	85.328	27.708	1344	0.49	347
117	85.338	27.708	1300	0.59	278
118	85.348	27.708	1295	1.56	85
119	85.358	27.708	1304	1.69	77
120	85.366	27.708	1330	1.32	105
121	85.378	27.708	1321	1.17	121
122	85.278	27.717	1453	3.00	39
123	85.287	27.718	1321	2.80	42
124	85.297	27.718	1284	1.42	96
125	85.307	27.718	1299	0.54	309
126	85.317	27.714	1298	0.78	197
127	85.328	27.718	1221	0.9	166
128	85.338	27.718	1318	0.68	233
129	85.348	27.717	1348	3.66	30
130	85.358	27.717	1305	1.00	146
131	85.368	27.717	1308	1.20	117
132	85.379	27.717	1301	0.90	166
133	85.277	27.727	1354	8.90	10
134	85.287	27.727	1292	0.90	166
135	85.297	27.727	1327	1.20	117
136	85.308	27.727	1279	1.10	130
137	85.318	27.727	1285	0.75	207
138	85.329	27.727	1308	2.50	48

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
139	85.338	27.726	1327	0.63	253
140	85.348	27.726	1281	1.22	115
141	85.358	27.726	1320	0.90	166
142	85.368	27.726	1336	1.07	135
143	85.378	27.726	1327	1.17	120
144	85.280	27.734	1346	3.00	39
145	85.287	27.736	1321	1.00	146
146	85.297	27.736	1337	3.00	39
147	85.308	27.736	1295	0.80	191
148	85.318	27.736	1300	0.80	191
149	85.328	27.736	1337	0.90	166
150	85.338	27.736	1330	0.80	191
151	85.348	27.735	1328	0.95	155
152	85.358	27.735	1308	1.52	88
153	85.368	27.735	1345	1.00	146
154	85.379	27.735	1368	1.12	127
157	85.298	27.745	1320	1.36	101
158	85.308	27.745	1304	1.21	116
159	85.318	27.745	1302	1.10	130
160	85.328	27.745	1340	0.80	191
161	85.338	27.745	1348	1.37	100
162	85.348	27.744	1328	1.61	82
163	85.358	27.744	1352	3.00	39
164	85.370	27.743	1404	1.81	71
165	85.379	27.744	1432	3.00	39
167	85.288	27.754	1323	1.51	89
168	85.298	27.754	1303	4.40	24

Microtremor observation point	Latitude (in degree)	Longitude (in degree)	Elevation (m)	Predominant Frequency (f_r) (Hz)	Relative variation of thickness of soft sediment in the Kathmandu Basin (this study) based on the relationship $h=146.01 f_r^{-1.2079}$ (m)
169	85.308	27.754	1334	3.81	29
170	85.318	27.754	1293	1.17	121
171	85.328	27.754	1295	1.12	127
172	85.338	27.754	1322	8.00	12
173	85.348	27.753	1353	3.00	39
174	85.359	27.753	1329	1.40	97
175	85.369	27.753	1358	1.20	117

This observation is validated by comparing the results of the gravity contour map proposed by Moribayashi and Mauro (1980) and also with the depth of the bedrock based on the borehole drilled for academic purposes (Sakai et al., 2001) in the Kathmandu Basin.

6.4.2 Digital Elevation model and cross-sections

The results of this study are expressed in terms of thickness of the lacustrine sediment and its variation in different areas of the Kathmandu Basin. The sediment depth in various locations in the basin is calculated using Equation 6.6. The contour map of the estimated soft sediment thickness and a digital elevation model (DEM) for the study area (Kathmandu Basin) are shown in Figure 6.10, Figure 6.11 and Figure 6.12. The calculated values give a deep interface of soft sediment (unconsolidated) and basement layer in the center of the Kathmandu Basin and shallow in and around the outskirts of the valley. The sudden abrupt change in the sediment thickness is found at points A (i.e. thickness of sediment about 48 m) and B (i.e. thickness of sediment about 30 m) (Figure 6.10) which are about 2 km and 3 km along north and east from the central part of the Kathmandu respectively, which indicates the presence of basement rock in the shallow depth in those areas.

The calculated depth of the interface between two layers is used to plot the cross-profiles and digital elevation model (DEM) for the Kathmandu Basin. Figure 6.12a, and Figure 6.12b show the profiles along west to east and south to north direction respectively.

The west to east profiles (Figure 6.12a) along P167-P175, P157-P165, P144-P154, P35-P53, P16-P34, and P1-P15 and show gentle slope of basement topography, whereas the profiles along P133-P143, P122-P132, P111-P121, P92-P110, P73-P91, and P54-P72 show steeper slope and increase in the soft sediment thickness mainly towards the center location. The variation of the soft sediment thickness in the valley can also be described using Figure 6.12b, the profiles along P3-P168, P4-P169, P5-P170, P6-P171, P7-P172, and P8-P113 give information about the distribution of sediment thickness along south to north direction and clearly indicates the steeper slope of basement floor and increase of sediment thickness towards the center. The DEM of the hard stratum further reveal that the thickness of sediment in depression (I) (refer Figure 6.5, Figure 6.11, and Table 6.2) in points P94, P95, P96, P114, P115, P116, P117, P125, P126, P127, P128 shows relative depth variations of 214 m, 246 m, 233 m, 253 m, 279 m, 347 m, 278 m, 309 m, 197 m, 166 m, 233 m and depression (II) in points P50, P86, P103, P104, P105, P132 show the depth variation of 194 m, 214 m, 166 m, 173 m, 173 m, 166 m respectively.

The sediment depth elevation model suggests that the sediment distribution in the basin is far from uniform and have an undulating topography with steep relief in many locations in the basin. Hagen (1969) mentioned that during the pre-lake formation in the Kathmandu Basin, drainage systems originating in the northern slope of the Shivapuri hill and termed as “Proto Bagmati River” were very active. These river systems were responsible for the deposition of coarse-grained sediments (gravels, and coarse sand) below the lake deposits in the entire valley. The influence of Proto Bagmati river systems appear more in the northern and central part than in the other part of the valley. According to Yoshida and Igarashi (1984), this deposition took place some 2.5 Ma ago (i.e., during middle to late Pliocene period).

After analyzing the digital elevation model of the basement topography in the Kathmandu Basin, there arise mainly two possible explanations. Firstly, the calculated depth of the sediment represents the total depth of lake deposit which is overlain by basement rock in the Kathmandu Basin. The calculated depth may not necessarily indicate the presence of hard rock, rather it is a representation of the presence of basement layer at that depth, beyond which the sediment do/may not contribute the amplification of the ground motion. From geotechnical point of view, this contrast corresponds to the bedrock.

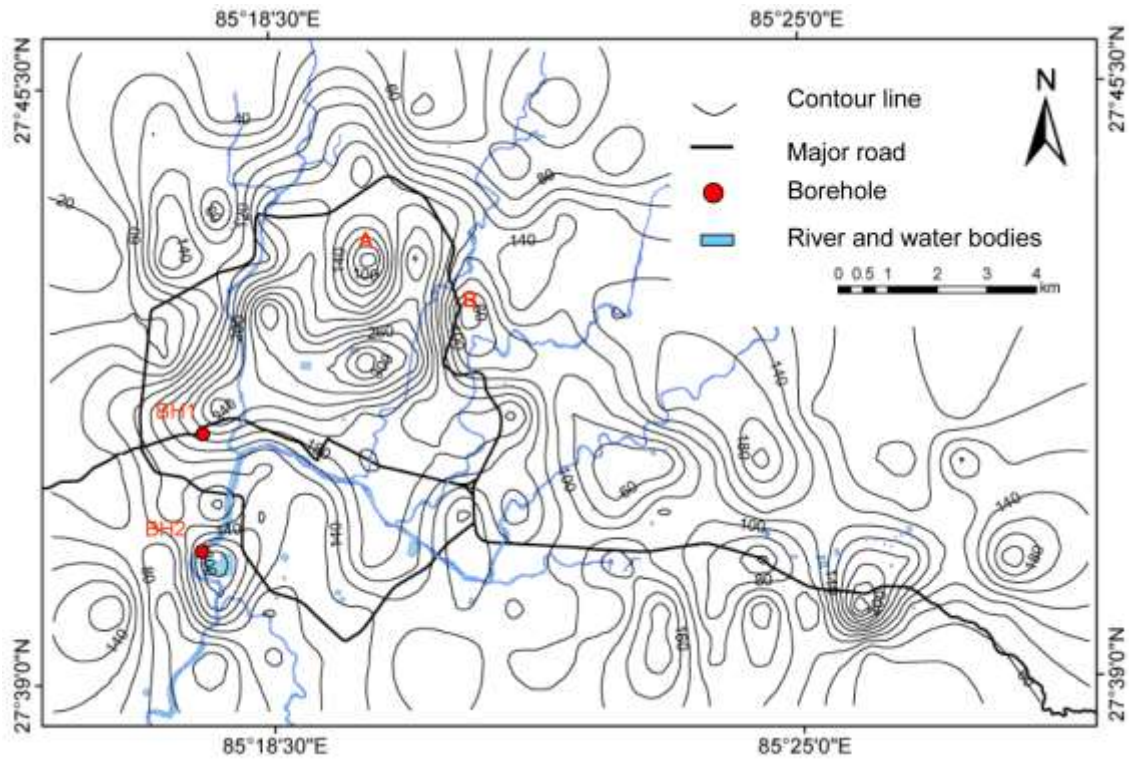


Figure 6.10: Contour map of the basement topography of the Kathmandu Basin.

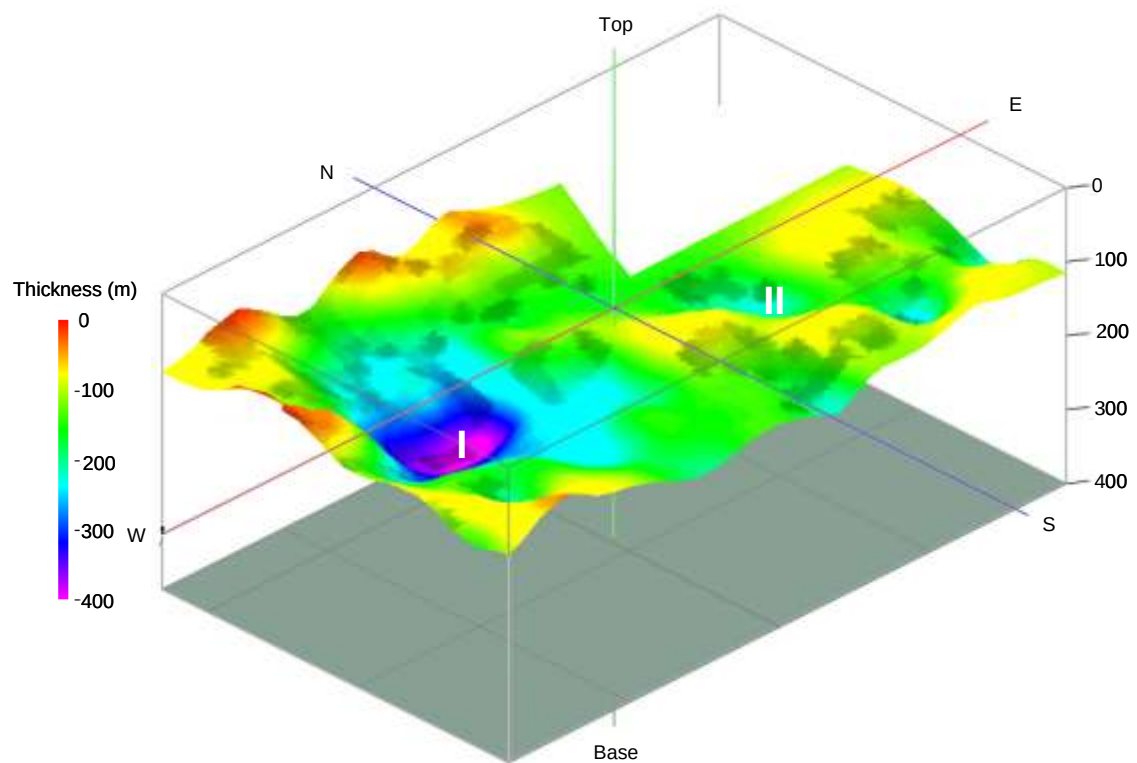


Figure 6.11: Bedrock-soft sediment palaeo-topography of the Kathmandu Valley Basin (the vertical scale is 15 times exaggerated). I and II are the depressions carved over Bedrock-soft sediment surface forming the sites of thickest deposit in the study area.

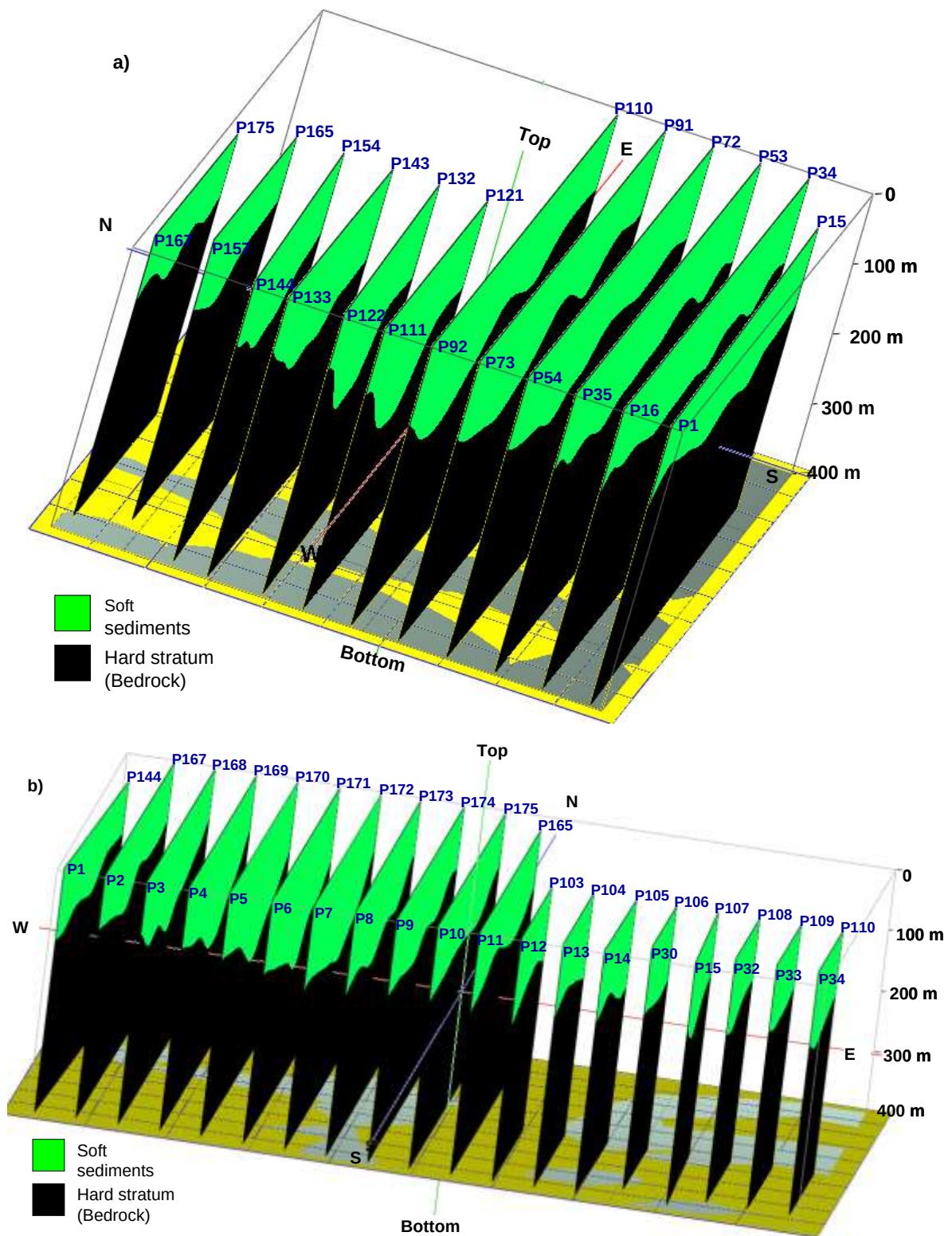


Figure 6.12: Cross profiles showing the contact of soft sediment and bedrock, and variations of soft sediment (the vertical scale is 15 times exaggerated); (a) W–E profiles, and (b) S–N profiles.

Secondly, although the Figure 6.10 and Figure 6.11 show a number of small depressions in the whole study area, two large depressions are found. First depression at the central part of the Kathmandu City denoted by I in Figure 6.11 which is wider and deeper and represents the main ancient lake of the Kathmandu Basin, while other is along the eastern part denoted by II which is relatively shallow and its catchments area elongated from northwest to southeast. Similarly, there are number of buried ridge which separated/connected the depressions with each others. The longest buried ridge which separated/connected the central large and deep depression with the eastern shallow depression is extended from northwest to southeast part of the valley (Figure 6.11).

The distribution of sediment indicates that the deepest part of the lake existed mainly in the center part of Kathmandu where, at present, the main core city exists, which is one of the oldest residential areas in the Kathmandu Valley. It accommodates number of departmental stores, Government Offices, historical monuments including UNESCO cultural world heritage sites. Moreover, due to increasing population and development into a greater commercial hub, the center part has seen a sharp rise in the number of mid-height to tall buildings without adequate geotechnical investigation. Depending upon the type and stories of buildings, the predominant frequencies are different. The thickness distribution map shows that the main lake part (i.e. central part) has a considerable thickness of the soft sediments and hence prone to higher amplification of seismic wave at the corresponding predominant frequencies.

6.4.3 Validation of the results

6.4.3.1 Comparison of results with gravity measurement data

We compared the basement topography estimated in this study with the basement topography proposed by Moribayashi and Maruo (1980) using the Gravitational method. Figure 6.3 shows a gravity contour map prepared by Moribayashi and Maruo (1980) for the center area of the Kathmandu Basin. The basement topography obtained based on the microtremor observation in this study (Figure 6.10) is found quite similar with the results of the gravity survey by Moriyabashi and Mauro (1980) because the low gravity is obtained in the center part of the valley where the thickness of the lacustrine sediments is high and gradually increases towards the marginal area where the sediment thickness is low. Moreover, the first depression along north to south through center part of the valley and second depression along northwest to southeast proposed in this study matched quite

well with the gravity data as mentioned by Moribayashi and Maruo (1980).

Moribayashi and Maruo (1980) were proposed the basement contour map (Figure 6.4) of the central area of the Kathmandu Basin based on the assumption that density-contrast was 0.8 g/cm^3 : 2.67 g/cm^3 for bedrock and 1.87 for lacustrine sediments. They have estimated the maximum thickness of sediment about 650 m at the central part of the basin. In reality, however, the density of lacustrine sediments is lower (JICA, 2002) than this assumed value by Moribayashi and Maruo (1980) and also the actual density distribution of bedrock and the overlain sediment will be different according to the sediment types, depth of the sediment, bedrock type as well as amount of consolidation of sediment in the particular area. According to JICA (2002), the average density of the Kathmandu Valley sediment layer up to 30 m is about 1.52 g/cm^3 , and it varies from 1.43 g/cm^3 to 1.68 g/cm^3 according to the sediment types and depth of the sediment layer, which is quite lower than the assumed value by Moribayashi and Maruo (1980). This shows that the basement contour map estimated from the assumed density difference of 0.8 g/cm^3 for whole area seems to be relatively higher, and also may not represents the actual depth of the basement by assuming constant density-contrast for the whole area of the Kathmandu Basin.

6.4.3.2 Comparison of results with borehole exploration based ground profile

Moreover, in order to verify the estimated sediment thickness distribution map of the Kathmandu Basin, the thickness variation profiles along south-north direction (Figure 6.12b, profiles P3-P168, P4-P169, P5-P170, P6-P171, P7-P172, P8-P173) through center of the Kathmandu Basin was compared with the schematic geological cross section of sediment distribution along N-S direction based on borehole exploration (Figure 2.25, Chapter 2) proposed by Sakai et al. (2002). These sections correlated well and showed that the distribution of the sediment thickness based on this study is in good agreement with the distribution of the soft sediment proposed by Sakai et al. (2002).

As mentioned in the Chapter 2, there have been various attempts of borehole exploration in the Kathmandu Valley for different purposes, however due to lack of enough information, particularly in relation to the coordinates, ground strata, and geotechnical data, only 340 boreholes data were employed in analyzing the sediment distribution in the valley (refer to Figure 6.13). As shown in the Figure 6.13, most of the boreholes were concentrated in the center part of the valley and only very few boreholes were reached up to bedrock, however there is still debate regarding the depth of bedrock

(Sakai et al. 2001). These sparsely distributed boreholes in the Kathmandu Valley make it largely difficult to accurately predict the ground profiles of the whole valley, mainly because almost all boreholes were not recovered during drilling (Sakai et al. 2001), which have led to dissimilar evaluation of the ground stratification which ultimately make very difficult to compare with the results of such study. However, precise evaluation of such boreholes may give some pattern of soft sediment distribution in the valley. For preliminary verification purposes, the soft sediment depth estimated in this study (i.e. based on microtremor observations) was compared with the ground profiles that were drawn based on the borehole exploration. Figure 6.13 shows the location of the microtremor observation points, location of borehole exploration, and cross section taken for comparison of the results. In this figure, the microtremor observation points were represented by black triangles where as borehole exploration points were represented by red circle, in which some of the boreholes that were taken for profiles generation were shown by red color number. In drawing borehole exploration based ground profiles, it was assumed that the boreholes which were located nearby these sections would represent the ground profiles of that area. Figure 6.14, Figure 6.15, Figure 6.16, and Figure 6.17 show typical profiles of the ground through lines S₄-N₄ and S₅-N₅ in south-north direction, and lines W₆-E₆ and W₇-E₇ in west-east direction (refer to Figure 6.13) respectively.

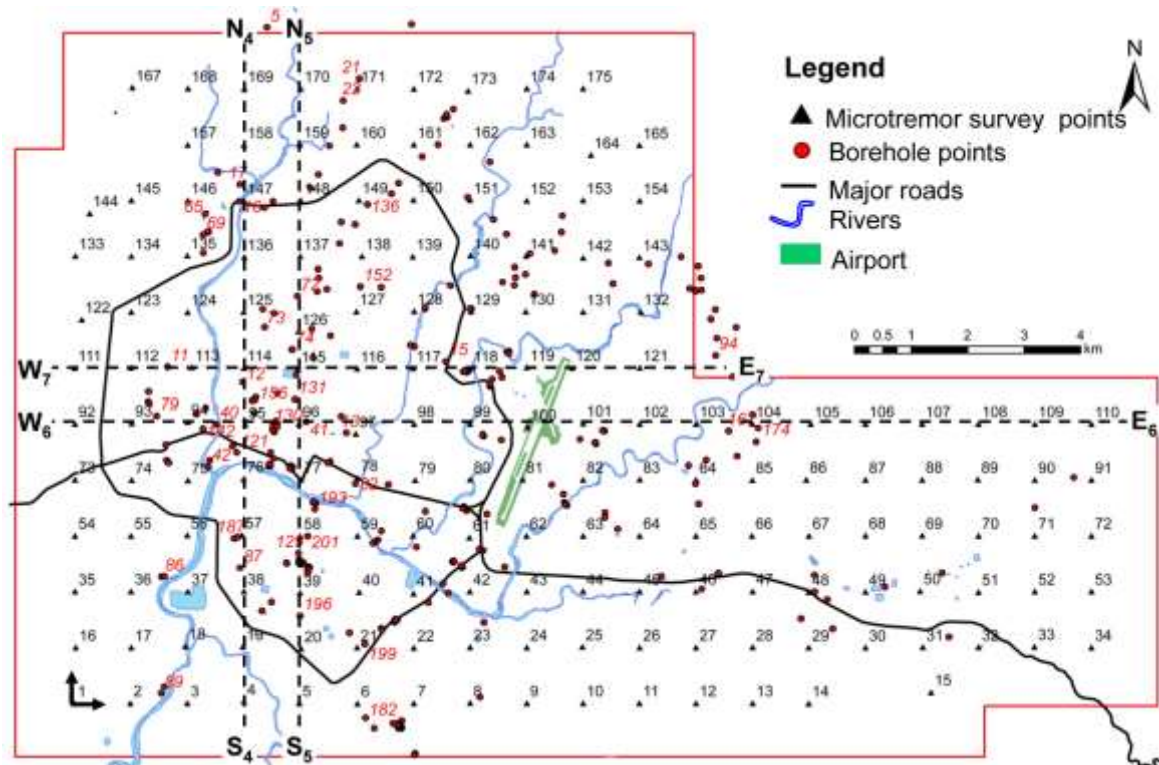


Figure 6.13: Distribution of microtremor observation points and borehole in the study area.

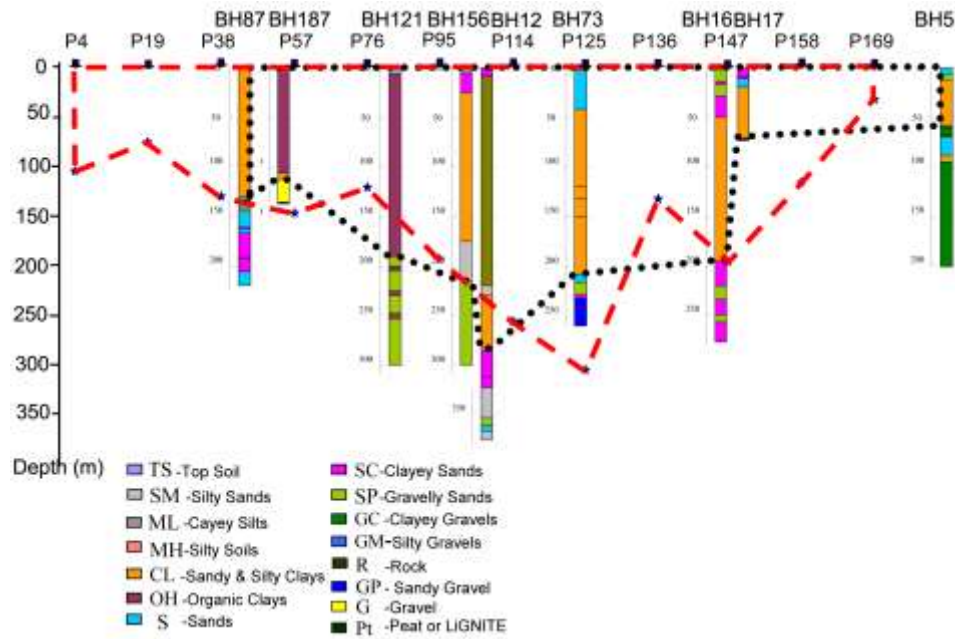


Figure 6.14: Section S₄-N₄ (Comparison of soft sediment depth based on geo-information data base system and microtremor survey (refer to Figure 6.13). Black dotted boundary and red dashed boundary represent the depth of soft sediment based on the geo-information data base system and microtremor survey respectively. BH and P represent the borehole and microtremor observation points respectively).

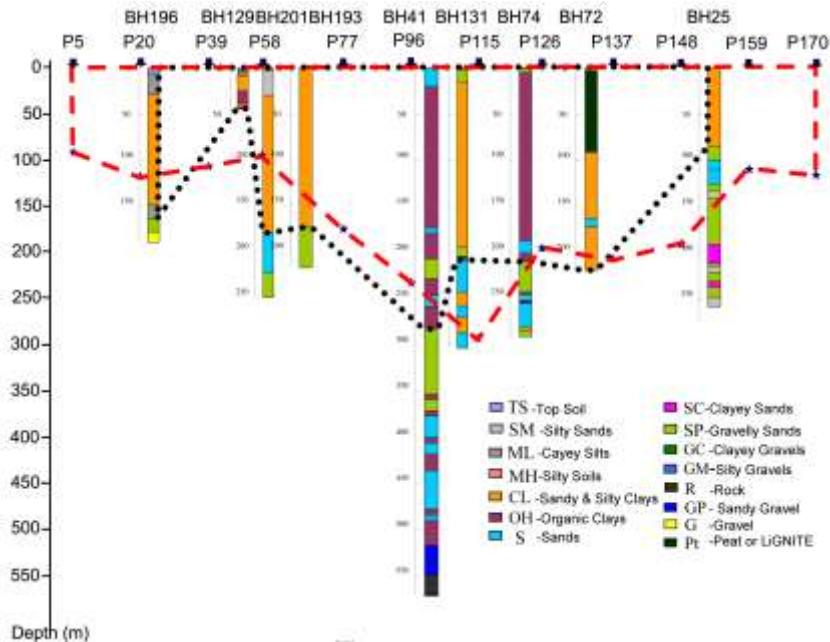


Figure 6.15: Section S₅-N₅ (Comparison of soft sediment depth based on geo-information data base system and microtremor survey (refer to Figure 6.13). Black dotted boundary and red dashed boundary represent the depth of soft sediment based on the geo-information data base system and microtremor survey respectively. BH and P represent the borehole and microtremor observation points respectively).

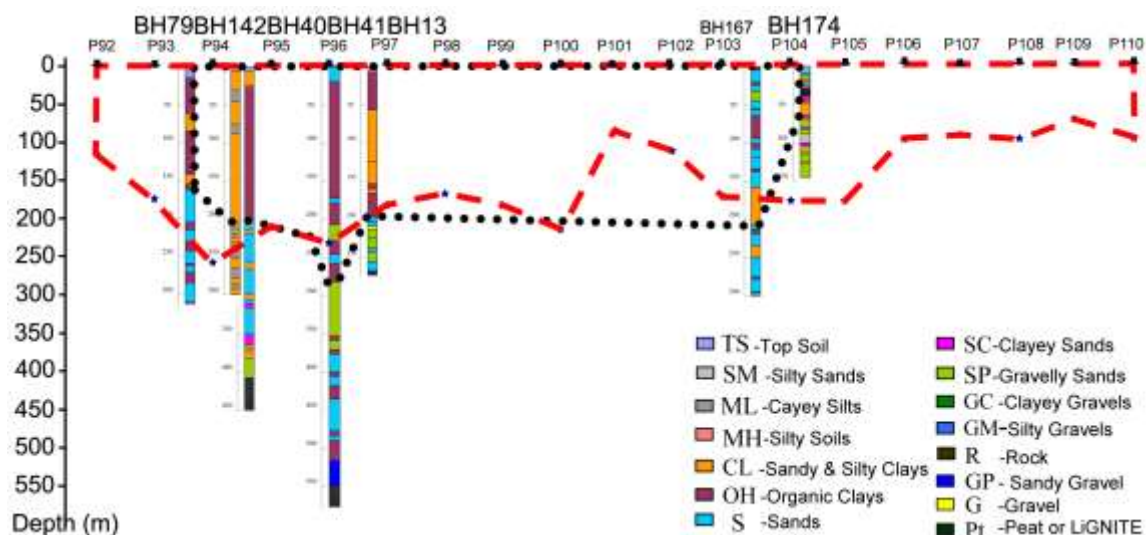


Figure 6.16: Section W₆-E₆ (Comparison of soft sediment depth based on geo-information data base system and microtremor survey (refer to Figure 6.13). Black dotted boundary and red dashed boundary represent the depth of soft sediment based on the geo-information data base system and microtremor survey. BH and P represent the borehole and microtremor observation points respectively).

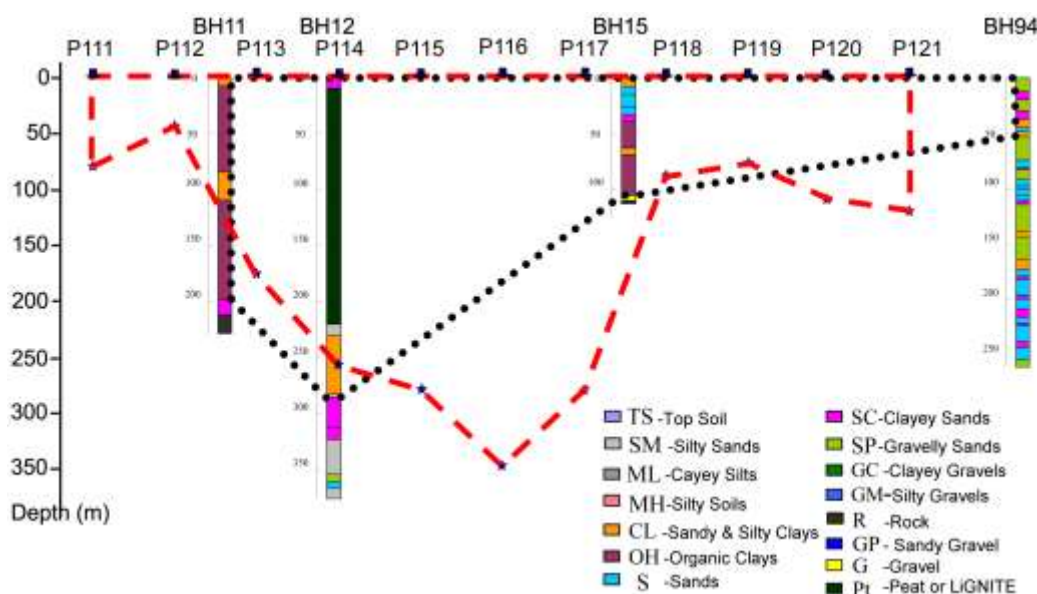


Figure 6.17: Section W₇-E₇ (Comparison of soft sediment depth based on geo-information data base system and microtremor survey (refer to Figure 6.13). Black dotted boundary and red dashed boundary represent the depth of soft sediment based on the geo-information data base system and microtremor survey respectively. BH and P represent the borehole and microtremor observation points respectively).

Although, not very perfect, the comparison of these ground profiles based on the microtremor observations indicate that there is some degree of reliability. These results comparison showed that the ground profile based on microtremor observation correlated well with the distribution of soft sediment in the valley.

According to Stöcklin and Bhattari (1981) and Sakai et al. (2001), the basement of the Kathmandu Basin consists of weakly metamorphosed Phulchauki Group (Chapter 2, Figure 2.19), which consists of Paleozoic sandstone, phyllite and weathered rocks. Moreover, Sakai et al. (2001) have mentioned that almost all sand and gravels at the lower part of the core are composed of detritus to weakly metamorphosed sedimentary rocks derives from the underlying Kathmandu Complex.

There is always confusion about the type of the lithological layer below the clay (unconsolidated) layer and hence it is always difficult to differentiate the sediment of bedrock with the basal conglomerate or gravelly soil in the Kathmandu Basin from the borehole data (Sakai et al., 2001). Figure 6.18a and Figure 6.18b show the lithostratigraphy in two boreholes (BH1 and BH2) that were used by Sakai (2001) and Sakai et al. (2001) for academic purposes in which the depth of the basement rock is shown at about 252 m in borehole BH1 (Figure 6.18a) and thick layer of the sand is shown below 232 m in borehole BH2 (Figure 6.18b). These Figures also showed that the depth of the basement rock (or hard sediment) estimated from the result of microtremor observation are at 196 m below the surface at borehole BH1 and 188 m at borehole BH2. The difference in depth of the bedrock with the estimated value may be due to the change in basement topography abruptly in nearby areas of these boreholes. Due to marginal area of the bowl shaped Kathmandu Basin, the basement contour values also showed varied abruptly with in the short distance in and around these borehole locations. The borehole BH2 lies at the edge of the Kathmandu Basin and steep slope of basement topography is observed in west, north and south direction (Figure 6.10). Moreover, the calculated depth of the basement rock in microtremor observation point near the borehole BH1 is found 260 m and similarly 233 m near the borehole BH2 (refer Figure 6.5, Figure 6.10, and Table 6.2). This indicates that the depth estimated in this study provides comparatively accurate result for the basement topography of the Kathmandu Basin and confirms the conclusions of previous studies (e.g., Birgöen et al. (2009), D'Amcio et al. (2008), Delgado et al. (2000), Hinzen et al. (2004), Ibs-von Seht and Wohlenberg (1999), Özalayboy et al. (2011), Parolai et al. (2002), and Sukumaran et al. (2011)) in encouraging the use of microtremor observations for approximate estimation of sediment depth over wide basin areas.

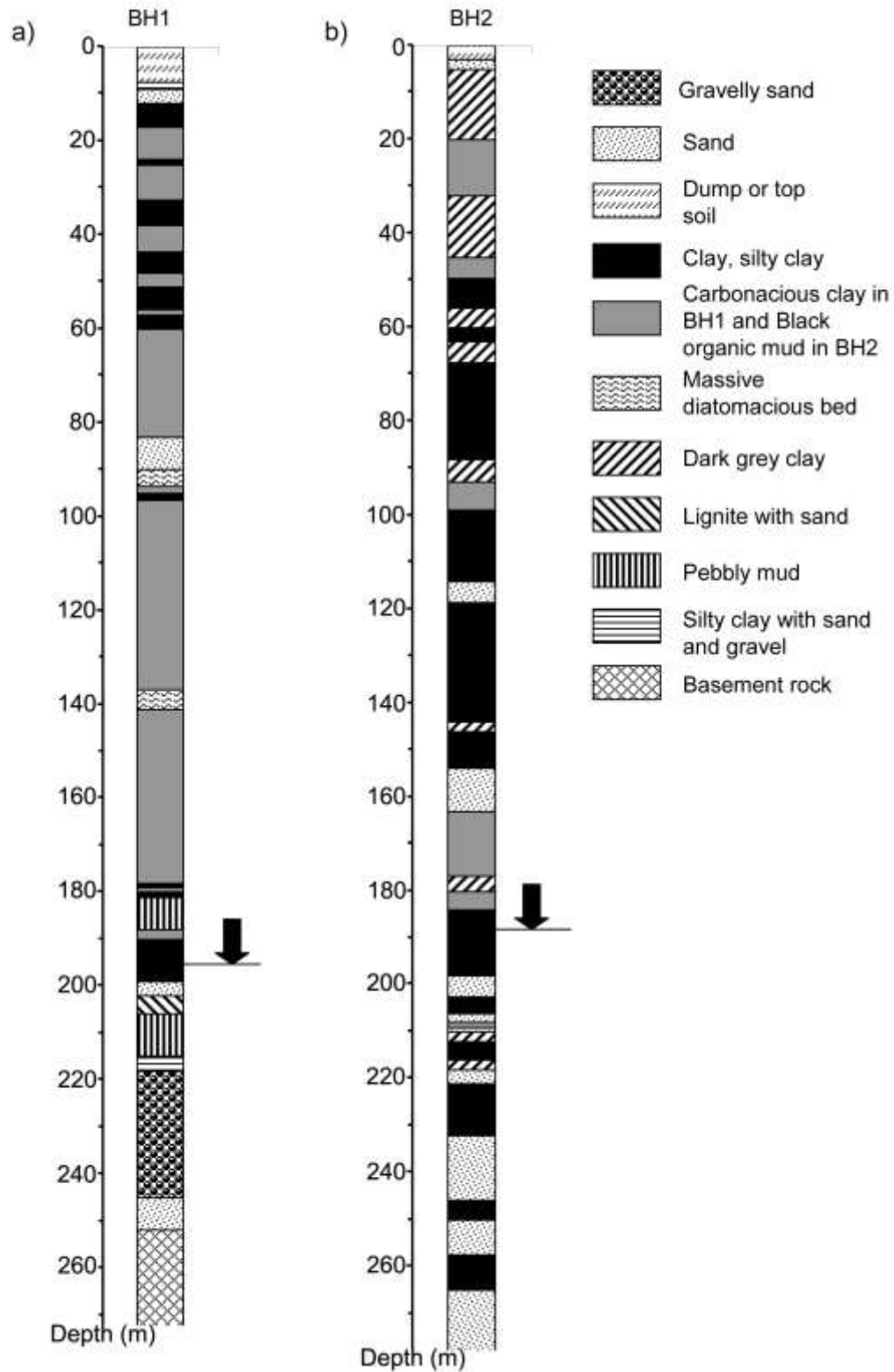


Figure 6.18: a) Litho-stratigraphic section in central part of the Kathmandu Basin, on the basis of drill-cores (BH1) (refer Figure 6.5, and Figure 6.10) collected by the Paleo-Kathmandu Lake Project in 2000; b) Litho-stratigraphic section of borehole BH2 (refer Figure 6.5, and Figure 6.10). Downward black arrow showed the depth of the basement as per present study.

6.5 CHAPTER SUMMARY

In order to explore the hazard level as well as to estimate the risk due to future expected earthquake disaster in the Kathmandu Valley, the study on the floor variation of the lacustrine sediments in the Kathmandu Basin was done. Due to lack of adequate and precise scientific studies on the floor variation of sediments in the Kathmandu Basin, it makes always difficulty to ascertain their characteristics during earthquakes, which ultimately makes to use erroneous and assumed data for the ground modeling, and analysis and design of the infrastructures. This study tries to solve this problem by proposing approximate basement topography of the Kathmandu Basin using microtremor observation in 172 sites. This study enables to estimate the soft sediment variation in the Kathmandu Basin using a non-linear regression equation ($h=146.01 f_r^{-1.2079}$), and provides the hidden basement topography of the Kathmandu Basin. The sediment/rock below this basement topography may not take part for the amplification of the ground motion during earthquake in the Kathmandu Valley.

The map thus depicted shall not be meant as a detailed and highly constrained representation of the valley bedrock; however, it represents the first reliable reconstruction of the subsurface morphology of the Kathmandu Basin, which shows a good consistency with available geological/log data. This study represents a useful starting point for future activities such as future more detailed surveys, numerical modeling, and seismic hazard or microzonation studies.

CHAPTER 7

7 SEISMIC VULNERABILITY ASSESSMENT OF BUILDINGS

7.1 INTRODUCTION

The main source of earthquakes in Nepal and the Himalayan Region is the subduction of the Indian plate underneath the Eurasian plate at rate of 16-18 mm/year (e.g., Banerjee and Burgmann, 2002; Bettinelli et al., 2006; Bilham and Szeliga, 2008; Jouanne et al. 2004). The detail about damage of buildings during past earthquakes in Nepal and Kathmandu Valley has been presented in Chapter 2. Pandey and Molnar (1988), and Bilham et al. (1995) have mentioned that the uneven distribution of damage in the Kathmandu Valley during past earthquakes, especially during 1934 Bihar-Nepal earthquake is due to the local geological as well as buildings condition in the valley.

Kathmandu Valley is considered one of the most risky cities in terms of earthquake lethality potential (GeoHazards and UNCRD, 2001). According to the report of Geo-Hazard International (2001), a person living in Kathmandu is about nine times more likely to be killed by an earthquake than a person living in Islamabad, Pakistan and about 60 times more likely than a person living in Tokyo, Japan. Rapid population growth, poor land use planning, precarious settlement pattern, and inadequate enforcement of building code make Kathmandu Valley vulnerable against earthquakes. Most of the structures are constructed by traditional construction materials with traditional technology (indigenous technology, which applies least of the engineering techniques). Traditional earthquake resistant measures have lost their relevance as more people use modern construction techniques and building materials. In general, most of the structures, especially traditional buildings are vulnerable to earthquake damage compared to the modern buildings. Despite the knowledge of historical seismicity, and continued geological research in the Nepal Himalayas, public awareness of earthquake hazard and risk was minimal until a few years ago and implementation of earthquake hazard risk management efforts are still almost nonexistent.

Because of haphazard urbanization and poorly engineered buildings, if an earthquake of the 1934 class recurs, the damage scenario in the Kathmandu Valley will be

unimaginable and the fatalities will be unexpectedly high. For such an earthquake scenario, the Japan International Cooperation Agency (JICA, 2002) has estimated some 59,000 houses to be destroyed, 20,000 deaths, and 59,000 serious injuries. An another study carried out in the framework of the Kathmandu Valley Earthquake Risk Management Project (Dixit et al., 2000) estimates a total of 40, 000 deaths, 95,000 injuries, and more than 600,000 homeless for the same scenario of earthquake in the valley.

The main objective of earthquake mitigation program is to minimize the structural damage and reduce the casualties when a large earthquake hits the area. For this purpose, first of all, assessment of vulnerability of the existing structures and services should be performed. This will help in predicting possible scenario of earthquake disaster. Moreover, this will help in proper planning, pre-disaster measures in the form of retrofitting, preparedness and other preventive steps. Planning well before onset of disaster is the best step in mitigating the earthquake disaster. In this study, an attempt is made to assess the preliminary vulnerability of buildings from expected earthquake disaster in the Kathmandu Valley.

7.2 PURPOSE OF THE STUDY

The main purpose of this study is to evaluate the building condition in the Kathmandu Valley. Detail evaluation is time consuming and required detail drawings and huge budget. In this study, the vulnerability condition of buildings is done by visual inspection using standard tool. The followings are the objectives of the present study.

- ◆ To review the building types and condition of existing buildings in the Kathmandu Valley.
- ◆ To analyze the vulnerability condition of existing buildings in the Kathmandu Valley
- ◆ To analyze the correlation between building damage and local geological condition during 1934 Bihar-Nepal earthquake in the Kathmandu Valley.

7.3 BUILDING TYPES IN THE KATHMANDU VALLEY

The Kathmandu Valley has a wide variety of buildings located in both urban and rural area. Most of the buildings in the valley are non-engineered and totally dependant on masons for the construction. From the analysis of past earthquakes damage and also from the structural analysis of buildings, it is found that the lateral loading system which is governed by construction methods and the type of materials used in the construction are

the main reason to damage the buildings (UNDP, 1994). A building inventory is important for building damage estimation due to earthquake, and so for earthquake disaster risk management. In Kathmandu Valley the following building types have been found which are also classified by Japan International Cooperation Agency in 2002 (JICA, 2002) during the study of earthquake disaster risk mitigation in the Kathmandu Valley.

7.3.1 Mud/stone masonry buildings (AD & ST)

Mud buildings are made of sun dried (unfired) brick (earthen) with mud mortar joints which are generally used for traditional houses such as residential buildings and temples. Generally mud buildings are constructed in rural area, however, such buildings are also seen urban areas as well. The typical mud building is shown in Figure 7.1.



Figure 7.1: An example of Adobe building.

Mud buildings consist of load bearing walls with mud mortar joints and wooden joists floor systems and roof of tiles or corrugated galvanized iron (CGI) Sheet. The wall thickness is generally one and half or two bricks. They generally do not have damp proofing course. Because of lack of integrity in horizontal components, they have very low resistance to lateral forces and hence they are high fragile during an earthquake.

Stone masonry buildings are not in practice in urban area, but in the periphery they are seen. The stone masonry walls consist of various sizes of stone and mud joint. During earthquake, the behavior of stone masonry buildings is similar to mud buildings as mentioned above.

7.3.1.1 Vulnerability of mud/stone masonry buildings

The vulnerability of these buildings is mainly due to the weak construction materials and the technology used in the construction of buildings. The wall construction starts right from the ground level with small foundation or without foundation and goes generally up to two stories and more (Gupta, 1988). Adobe buildings may get cracked at earthquake Intensity VI. Wide cracks and even partial collapse may occur at VII, and collapses are widespread under an intensity of VIII (IAEE, 2004).

The weak seismic performance of these buildings is due to the following reasons (UNDP, 1994; IAEE, 2004).

- ◆ Weak materials used in construction (i.e., low tensile, compressive and shearing strength)
- ◆ Non-uniformity/irregularity in plan (e.g. L, E,T shaped) as well as structural system leading it to torsion effects
- ◆ Large and unsymmetrical openings
- ◆ Lack of maintenance
- ◆ Use of irregular shape of walling units, blocks (especially in stone masonry)
- ◆ Lack of diaphragm
- ◆ Lack of integrity between load-bearing elements and lack of bands
- ◆ Long, unstable and heavy walls used
- ◆ De-lamination of walls
- ◆ Mixed construction (some part in fired bricks and some part in mud/stone)

7.3.1.2 Typical damage pattern in earthen building

The earthen buildings are categorized as a very weak building type with respect to their resistance of later load during strong ground shaking. The typical damage patterns in the earthen buildings are shown in Figure 7.2.

- ◆ Large cracks on corner of the walls in earthen buildings may occur due to the diagonal compression and tension in the joints.
- ◆ Lack of integrity between gable and main wall cause the collapse of gable in the buildings.
- ◆ Unsymmetrical (T, L, U shape) earthen buildings may collapse due to torsion effect in the buildings.
- ◆ Longer cantilever may cause cracks in the junction between wall and cantilever member during strong shaking.

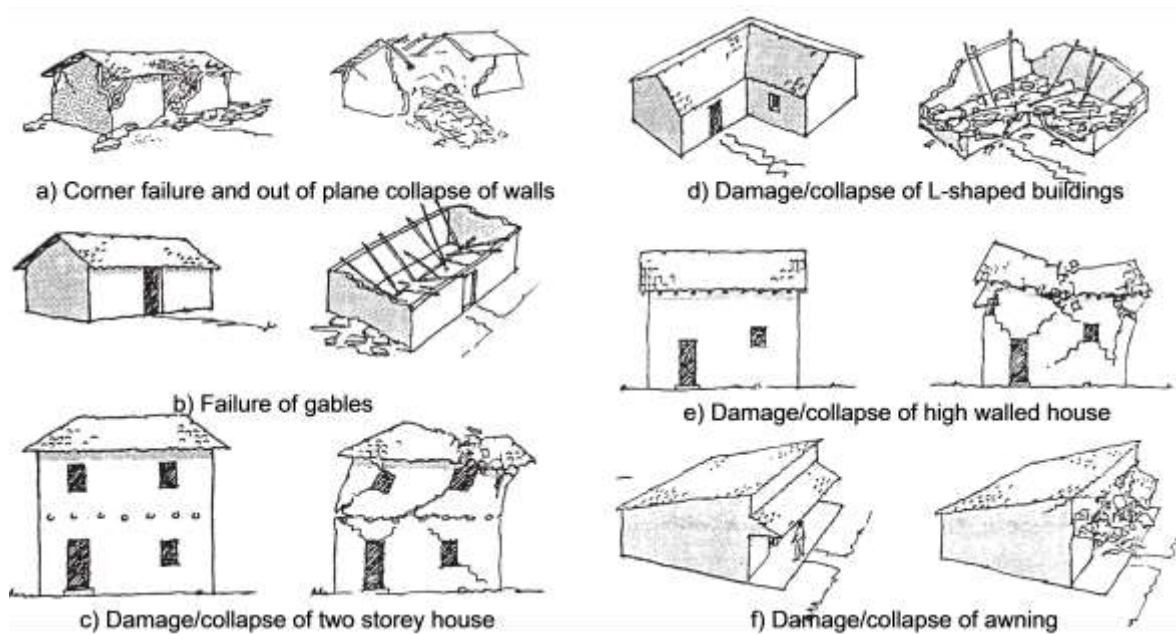


Figure 7.2: Typical damages and collapse of mud buildings.

7.3.2 Brick masonry buildings

Brick has been used for constructing various kinds of buildings in the Kathmandu Valley, such as residential buildings and schools since a long time ago. These are basically two types, namely- brick masonry with mud mortar (BM) and Brick masonry with cement sand mortar

7.3.2.1 Brick masonry with mud mortar (BM)

These are brick masonry buildings with fired bricks in mud mortar. The floor is made of wood or wood and mud. The roof of these buildings is made of tiles or corrugated galvanized iron (CGI) sheet. Large numbers of such type of buildings are still remaining in urban and rural areas of the Kathmandu Valley (JICA, 2002). The number of stories of these buildings varies from 2 to 4 stories. Before cement had come in use, BM buildings were a common type of construction in the Kathmandu Valley. A typical example of brick masonry with mud mortar buildings is shown in Figure 7.3.

These buildings have very poor horizontal rigidity because of the low bond strength, high water absorption at the mud joints and the wooden floors and roof. And hence they are very weak for lateral forces resistance capacity during earthquake. Brick Masonry buildings of less than three stories seem fragile and BM buildings more than three stories

or higher seem very fragile (JICA 2002) during earthquake. According to their seismic vulnerability, JICA (2002) classified BM buildings as brick masonry building with mud mortar and brick masonry building with mud mortar well built. Some buildings in Kathmandu Valley have been constructed using good wooden joists and nails for floor and roof. Wooden bands are also used in plinths and in lintels. If we study the past earthquake, these buildings (BM with well built) showed better performance than simple BM buildings.



Figure 7.3: An example of BM building.

7.3.2.2 Brick masonry buildings with cement mortar joints (BC)

Brick masonry building with cement mortar joint (BC) is located in most of the area of Kathmandu Valley. Most of the people are still constructing this type of building in urban as well as in rural areas. According to JICA (2002), BC type buildings comprise more or less half of the buildings in the Kathmandu Valley. These are load bearing masonry buildings with fired bricks in cement or lime mortar for the wall construction. This type of building is generally with rigid reinforced concrete slabs floor and roof (Figure 7.4). They usually have strip footing of stone masonry with mud mortar or brick masonry in cement sand mortar (UNDP, 1994). BC buildings in Kathmandu Valley have been constructed for the last 30 years (JICA, 2002). They have improved structural system than brick masonry buildings with mud joints. Due to lack of structural integrity and poor workmanship, they are weak in tension (not ductile), they are still seemed vulnerable.



Figure 7.4: An example of BC building.

7.3.2.3 Vulnerability of brick masonry buildings

The vulnerability of these types of buildings is mainly due to low ductility, weak bond and lack of integrity between two orthogonal walls. The following are the main weakness in the materials and unreinforced masonry construction, and other reason for the extensive damage of such buildings (UNDP, 1994; IAEE, 2004).

- ◆ Heavy weight and very stiff buildings, attracting large seismic forces.
- ◆ Non-uniformity/irregularity in plan and elevation as well as structural system leading to torsion effects.
- ◆ Very low tensile and low shear strength, particularly with poor mortars.
- ◆ Brittle behavior in tension as well as compression.
- ◆ Undefined load paths: shifting of upper storey walls.
- ◆ Re-entrant corners.
- ◆ Large and unsymmetrical openings.
- ◆ Mix construction (some part in mud mortar and other in cement mortar).
- ◆ Weak connection between wall and wall.
- ◆ Weak connection between wall and roof.
- ◆ Lack of integrity between the load-bearing elements.
- ◆ Lack of diaphragm action.

7.3.2.4 Typical damage and failure of brick masonry buildings

The creation of tensile and shearing stresses in walls of masonry buildings is the primary cause of different types of damage suffered by such buildings. The typical damage and failure modes of masonry buildings are as follows

◆ Damage and failure of bearing walls

Diagonal cracks on masonry wall may occur due to diagonal compression or diagonal tension in the walls. Such failure may be either through the pattern of joints or diagonally through masonry units. These cracks usually initiate at the corner of openings. This kind of failure can cause partial or complete collapse of the structure. Figure 7.5 illustrates such kind of failure pattern.

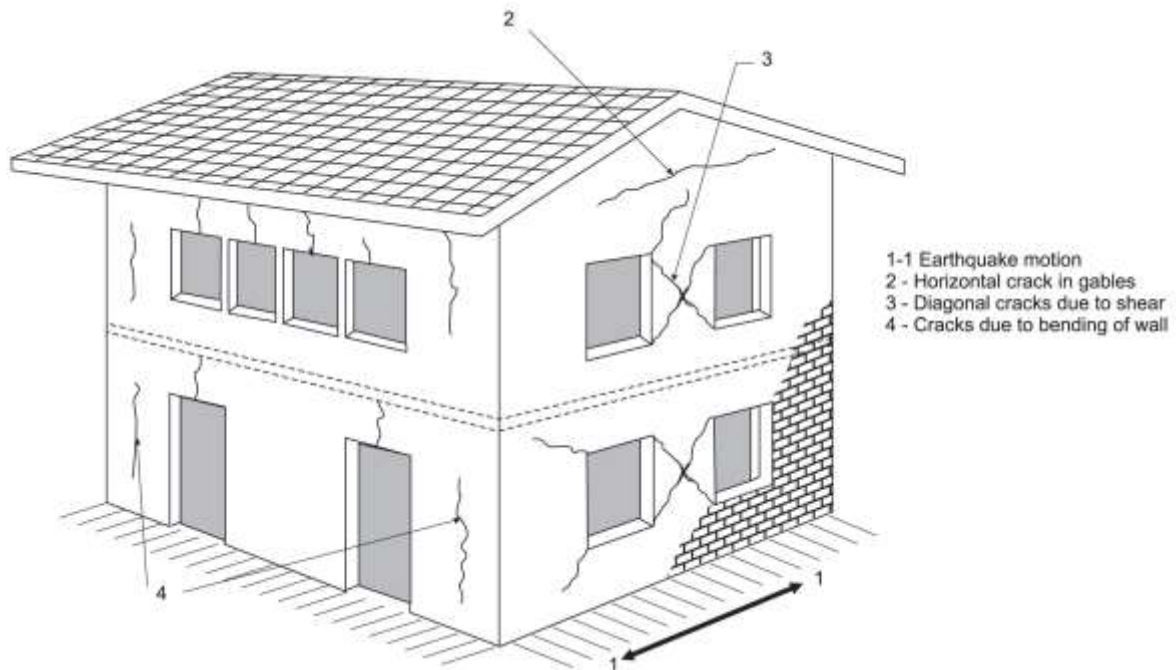


Figure 7.5: Typical failure pattern in brick masonry building.

- ◆ A wall can fail as a bending member loaded by seismic inertia forces on the mass of the wall itself in a direction transverse to the plane of the wall.
- ◆ Tension cracks occur vertically at the centre, ends or corners of the walls. Longer the wall longer the opening, and more prominent is the damage. Failures in the piers occur due to combined action of failure and shear during earthquake loading.
- ◆ Horizontal bending tension cracks are caused in the gables and finally collapse of the gable wall may occur

- ◆ Walls can be damaged due to the seismic force of the roof, which can cause the formation of tension cracks and separation of supporting walls.
- ◆ The unsymmetrical buildings may damage due to torsion are warping in an earthquake. This mode of failure causes extensive cracking due to shear in all walls. Large cracking can be seen near the corner of the building.

7.3.3 Reinforced cement concrete (RCC) buildings

Reinforced cement concrete (RCC) buildings are most common types of building in the Kathmandu Valley which have been constructing for last 30 years in urban areas. Nowadays, people in urban as well as rural areas have been using reinforcement and cement as major building materials. Most of RCC buildings, even in urban areas, are non-engineered. During construction, people are totally dependent on masons and most of the buildings are designed without a structural engineer and constructed by unskilled masons who do not have sufficient knowledge of RCC structural works. RCC buildings in the Kathmandu Valley can be divided into two types, namely- Reinforced concrete buildings with/without masonry wall more than three stories (RC_5), and Reinforced concrete buildings with/without masonry wall less than or equal to three stories (RC_3).

7.3.3.1 Reinforced concrete buildings less than or equal to three stories (RC_3).

These are the buildings that we find mostly in urban and rural area of the Kathmandu Valley. In the RCC building, concrete columns, beam, slabs are the main load carrying elements (Figure 7.6). If the building is designed and constructed as per National Building Code of Nepal, it would resist a moderate earthquake, and sustain major earthquake without life threatening damage (NBC, 105). Since, most of the buildings in the Kathmandu Valley are designed without consulting with structural engineer and are constructed by unskilled masons. Moreover, the building materials used for construction are not as per specification, and hence there is concern about their stability during great earthquake.



Figure 7.6: An example of RC₃ building.

7.3.3.2 Reinforced concrete buildings more than three stories (RC₅)

Because of rapid increase of urban population, people have started to construct building more than three stories (Figure 7.7) without consulting with qualified engineer. The floors of the second and upper stories on the roadside extend beyond the floors of the lower stories. The walls of cantilever slab are supported on cantilever beams and in some areas column also started from the slab. Therefore, load path may not regular which makes the building more vulnerable during earthquake.

7.3.3.3 Vulnerability of reinforced cement concrete (RCC) buildings

The main defect in RCC buildings in Kathmandu Valley is the construction with a small size of column and lack of ductile detailing of reinforcement and tendency to provide large cantilever (JICA, 2002). Construction of RCC buildings has developed a myth in Nepal that these are infinitely strong and can go up to unlimited heights. Such wrong notion has led to severe deficiency in strength of such buildings.

The weak seismic performance of these buildings is due to the following reasons (UNDP, 1994; IAEE, 2004).

- ◆ Low -quality of materials used in construction: low quality of sand, aggregate, cement and steel as well as workmanship.
- ◆ Non-uniformity/irregularity in plan and elevation as well as structural system leading to torsion effects.
- ◆ Strong column-weak beam system not maintained.

- ◆ Undefined load path.
- ◆ Short column effect.
- ◆ Soft-storey effect.
- ◆ Lack of ductile detailing (lack of confining/shear bars, anchorage problem, splicing of bars etc).



Figure 7.7: An example of RC₅ building.

7.3.3.4 Typical damage pattern of reinforced cement concrete (RCC) buildings

The following types of damage pattern are quite common in reinforced concrete buildings

- ◆ Falling of infill walls- the infill wall in the RCC buildings may fail due to out of plane failure. It will overturn outside the framework if not held tight or connected with the frames
- ◆ Crushing of column ends and virtual hinging - during strong shaking, the column ends are subjected to heavy eccentric compressive stresses under which the concrete gets crushed and spalled off from the outer surface. In repeated cycles of loading, thus effective section gets much reduced, and then the column ends virtually start behaving as pins and whole framework collapse like a mechanism.
- ◆ Short column effect – when infill masonry walls with wide openings are attached to the columns, the portion of the column that will deform under lateral seismic

loads, become very short as compared to their normal height, such column attract much larger shear forces as they become stiffer than other column. During strong shaking due to earthquake, they get sever diagonal tension cracks which may lead to failure of the column (Figure 7.8).

- ◆ Diagonal cracking of column beam joint - due to improper connection or due to improper placement of reinforcement in column beam connection, diagonal cracking occurs through the junction of the column with the beam which seriously impairs the strength of the frame.
- ◆ Pulling out of reinforcing bars – when spacing of the stirrups is not sufficient or not as per design code, and also where the anchor length of column bars or overlaps between the longitudinal bars are not adequate for developing full tensile strength of the bar, they often pulled out due to tension caused in the column under sever reversible of stress.
- ◆ Foundation sinking and tilting - when foundation of building is laid on soft soils, the sinking or tilting of foundation of columns may occur during strong shaking and lead to severe cracking of the super structure and even collapse of the building.

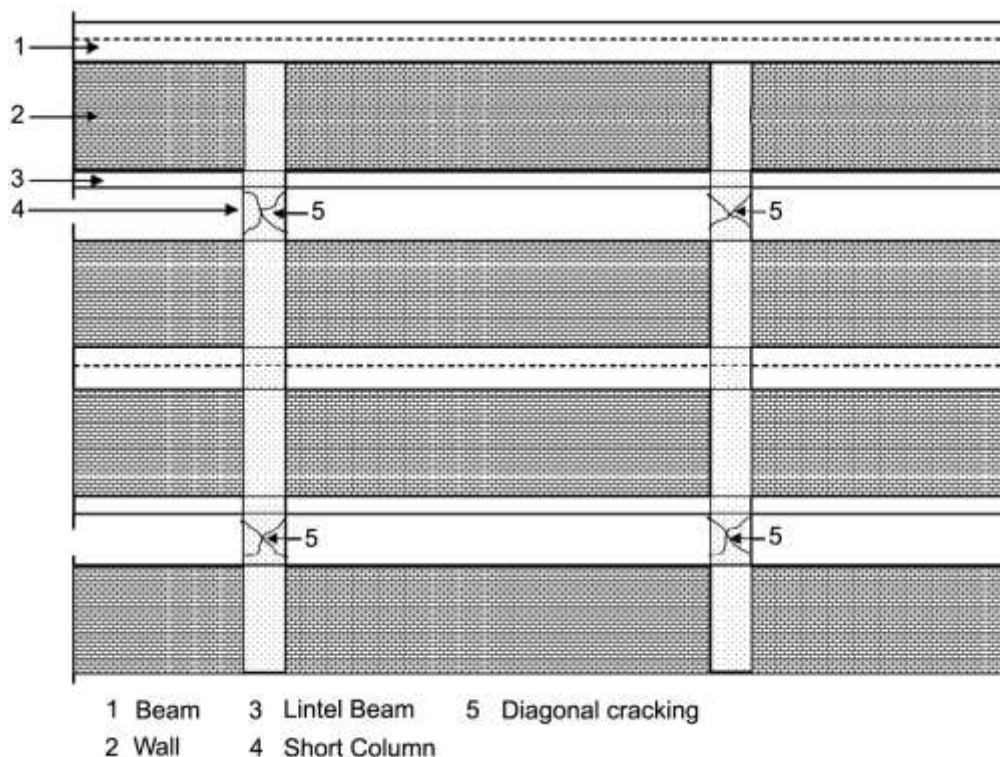


Figure 7.8: Shear failure in short column.

7.4 GENERAL METHODS OF BUILDING VULNERABILITY ASSESSMENT AND THEIR APPLICABILITY IN NEPAL

There exist many approaches for vulnerability assessment of buildings. Some methods are simple to understand and easy to use but some are complicated and required advance technology and manpower. Most of the methods are developed according to the requirement of the organization and experts to fulfill the local need. Some of the widely used methods for the vulnerability assessment of buildings are described below:

7.4.1 Analytical method

Finite Element Method (FEM) is an analytical method to assess the vulnerability of buildings. This method uses the complete load deflection characteristics of the buildings. This method works very precisely for the engineered buildings and it is very difficult to model the non-engineered buildings. In this method, individual elements of a structure are analyzed for exceedance in stress due to specified demand. This method is accurate and useful if we do the detail analysis of individual building and may not be possible to carry out for large numbers of buildings. Furthermore, this method requires very fine details of buildings and such details are not easily available in the case of the buildings of the Kathmandu Valley.

7.4.2 Federation Emergency Management Agency (FEMA) method

Simple method that has been developed by Federation Emergency Management Agency (FEMA) is a very useful tool to find the vulnerability of individual building. This tool can be used rapidly and easily to identify those buildings that might pose a risk of life or injury. In this method, a structural score “S” which is related to the probability of the building sustaining major life threatening damage is generated based on the experts opinion ATC-13(1982). Vulnerability of an individual building is analyzed base on the structural score. The method is fast and relatively inexpensive. The users can make decision based on the building score. The inspection, data collection and decision making process will occur at the building site. It is a single page tool specially covering building type and their vulnerability score. This tool is developed and applied by Applied Technical Council (ATC) especially for the vulnerability assessment of American building topology so it is suitable for building types of American society ATC-13 (1982). If we use FEMA Rapid Visual Screening procedure for vulnerability assessment in Nepali building topology, we are unable to cover major Nepalese building types and outcome will be far form reality.

7.4.3 HAZARD US (HAZUS) method

HAZUS is a software program that uses mathematical formulas and information about building stock, local geology and the location and size of potential earthquakes, economic data and other information about a community. This is a versatile method developed in the United States by the Federal Emergency Management Agency (FEMA) under a cooperative agreement with the National Institute of Building Science (HAZUS-MH MR1, 2003). Ground shaking, liquefaction and landslides are the earthquake hazards models are used in this method. Ground motions in terms of spectral acceleration (SA) and peak horizontal ground acceleration (PGA) are estimated based on the location, size and type of earthquake, and the local geology. HAZUS is most recent and comparatively more accurate method for earthquake loss estimation and is used for earthquake-hazard mitigation, emergency preparedness, response and recovery planning, and disaster response operations. To replicate this model in other cities outside USA, similar type of detail and precise hazard and inventory data are required. This tool can also be used for damage estimation after an earthquake. The building types given in HAZUS are not common building type found in developing countries like Nepal. To adopt this method in a new area having other than the 36 building types as mentioned in HAZUS, new fragility curve has to be derived from building capacity curve and demand spectrum analysis, which require more research in this field. Although this method is versatile and can be used in more accurate manner, it is expensive; the procedures are of a highly technical nature and required high level of skilled manpower.

7.4.4 Risk Assessment Tools for Diagnosis of Urban Areas against Seismic Disasters (RADIUS) method

Risk Assessment Tools for Diagnosis of Urban Areas against Seismic Disasters (RADIUS) project has been launched in 1996, with the financial and technical assistance from the Government of Japan. The aim of this project is to promote worldwide activities for reduction of urban risk which is growing particularly in the developing countries. The main purposes of the RADIUS project were to raise awareness and provide practical tools for earthquake risk reduction.

Scenario earthquakes, ground conditions, demographic data and vulnerability functions are critical input data for the earthquake damage estimation. The damage levels considered in this method are collapse and heavy damage. The ground is classified into four types of soil: Hard Rock, Soft Rock, Medium Soil, and Soft Soil. The study area is

divided into zones of 0.5 to 2 km (recommended) using software based on Excel and for each of the meshes the amount of houses, lifelines and probable population are mapped. The input and output data in the grid cells can be visualized spatially as a simple raster maps. In this program, buildings are classified into 10 different classes according to material type, construction type, applied code, uses and number of stories. This classification is based on the common building types in Latin American cities. The number of each type of building in each mesh is estimated by density of buildings with a weight called “Mesh Weight”. This method is simple, easy to understand and easy to implement. The building types and fragility curve described in RADIUS methodology are for common buildings found in most developing countries. RADIUS method seems more appropriate for the study of the preliminary earthquake hazard and vulnerability assessment (RADIUS, 1999). This method is more appropriate to prepare fast earthquake scenarios, which better fits the needs of earthquake-threatened cities in developing countries. This method is not suitable for individual buildings. This method required generating the mesh. In one mesh there should be only one building types but in reality there may be more than one type of buildings. This method is developed based on the common building types in Latin American cities. Since the building types are different in our case, it will not give accurate result.

From the review of above methods it can be understood that the existing methods do not cover the major building topology of the Kathmandu Valley. They were developed as per the local need and some of them are not suitable for individual building and some of them required skilled manpower. Hence it is necessary to develop a method that can be used for specific condition of Kathmandu Valley of Nepal.

7.5 RAPID VISUAL SEISMIC VULNERABILITY ASSESSMENT

As mentioned in Section 7.3, Kathmandu Valley consists of varieties of buildings in terms of building materials and methodology adopted for the construction of buildings. Past studies showed poor performance of buildings during earthquakes. In 2002, Japan International Cooperation Agency (JICA) has done the study for expected intensities in the valley for different probable earthquakes scenarios. The damage by an earthquake will be different depending on the type and location of the earthquake. Based on the historical earthquake catalogue, recent seismicity, and tectonic aspects in and around Nepal and the Kathmandu Valley, four “Scenario Earthquakes” were chosen and expected earthquake intensity maps in Modified Mercalli Intensity have been prepared for the whole

Kathmandu Valley (JICA 2002). In these scenarios, the Kathmandu Valley might experience minimum MMI VI to maximum MMI IX. In this context, it is necessary to identify the buildings which can sustain the damage and destruction from the expected maximum scenario of earthquake disaster in the valley.

Rapid visual vulnerability assessment of buildings can help to delineate the vulnerable and non-vulnerable buildings for a particular input ground motion, from which we can identify the buildings which need detail vulnerability assessment and also the buildings which can sustain the scenario earthquake without life threatening damage.

7.5.1 Area covered

Kathmandu Valley is one of the oldest cities in Nepal and famous for its historical and cultural richness. It consists of many historic temples and monuments. For preliminary vulnerability assessment of buildings in the valley, the following 4 places: 1) Shanti Nagar Tole in the central part of the valley (i.e., eastern part of the Kathmandu City) which is a newly developed urban area of the valley, 2) Mangal Bazar in the center of the Lalitpur City and 3) Bhaktapur Durbar Square in Bhaktapur City which are the oldest residential area in the valley, and 4) Duwakot which represents a typical village area in the Kathmandu Valley, are selected as shown in the Figure 7.9. The generation of the building database in this study is based on the interpretation of Google Earth Image together with field survey. A total of 413 buildings (i.e. Shantinagar – 112, Mangal Bazar, Lalitpur – 99, Durbar Square, Bhaktapur – 98 and Duwakot, Bhaktapur – 104) are surveyed and analysis of the data is done to find their vulnerability level.

7.5.2 Methodology adopted

For preliminary vulnerability assessment of buildings in the Kathmandu Valley, a Rapid Visual Seismic Vulnerability Assessment Tool as shown in Table 7.1 developed by Paudyal (2008) has been used. This tool has been prepared based on the analysis of fragility curve for the Kathmandu Valley (Figure 7.10) (JICA, 2002), and reviewing FEMA 154 and FEMA 155 as well as other different building vulnerability assessment tool proposed by experts for the different geographical region of world. As illustrated in Table 7.1, the tool mainly consists of two kinds of score, namely the basic structural score and modifiers score. The final score of a building is calculated by adding or subtracting the modification score from basic score depending upon the types of modifiers. According to Paudyal (2008), and Paudyal et al. (2010), the basic scores represent comparative

vulnerability of buildings without consideration of different parameters (elements) of buildings such as plan irregularity or vertical irregularity. It has been known that the performance of buildings during earthquakes depend upon various factors. The most common for the buildings of the Kathmandu Valley of which are the code or guidelines followed, plan irregularities, vertical irregularities, number of stories, column size in frame buildings, opening size in masonry buildings and condition (maintenance) of building which are incorporated as a performance modification factors. The score of these performance modification factors can be positive or negative depending upon their influence in building vulnerability.

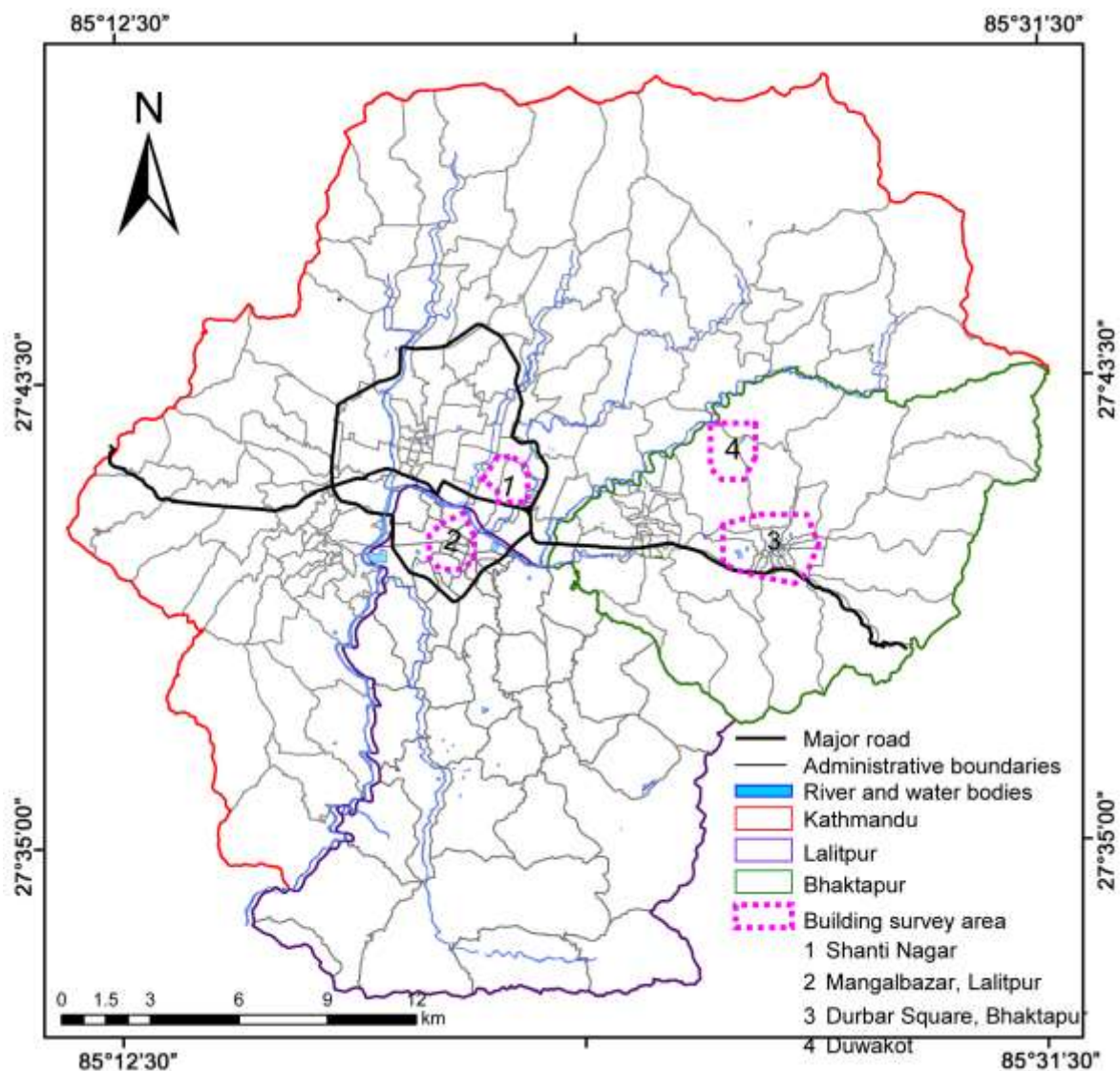


Figure 7.9: Study area for the vulnerability survey of buildings in the Kathmandu Valley.

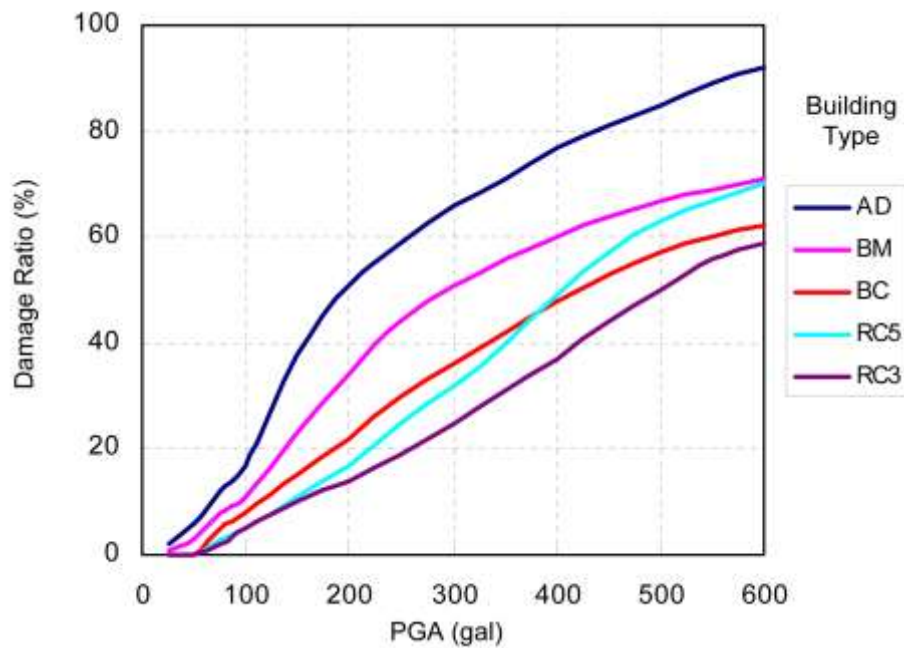


Figure 7.10: Fragility curve for Kathmandu Valley (redrawn after JICA, 2002).

Vulnerability of a building is evaluated based on the total score of the building. The structural score 2 is taken as a cutoff score for the evaluation of building vulnerability (Paudyal, 2008). The cutoff score of building is equivalent to the score of the brick masonry with cement sand mortar building as recommended by Nepal National Building Code (i.e. NBC, 2002) up to 2 stories with symmetrical plan and elevation (i.e. no vertical and horizontal irregularity) and which can resist the probable scenario earthquake (i.e. about 300 gal) (JICA, 2002) for the Kathmandu Valley. Therefore, total score of building less than 2 indicates vulnerable buildings requiring detailed evaluation and score equal or greater than 2 indicates non-vulnerable buildings and hence safe and can sustain life threatening damage. The probable damage of building can be estimated based on European Micro-Seismic Scale, 1998 (EMS-98) on the score of developed tool as follows:

- ◆ Score ≥ 2 – High probability of 2 or less Grade damage (slight or moderate damage but not structural damage or collapse of building).
- ◆ Score < 2 – High probability of 3 or more Grade damage (heavy damage, structural damage or may collapse of building).

Table 7.1: Rapid Visual Seismic Vulnerability Assessment Tool for Kathmandu Valley (Paudyal, 2008).

Building Sketch		Surveyor: _____				
		Building Use _____		Date of Survey _____		
		Address: _____				
		House No: _____		Year built: _____		
		No of Stories: _____		Photograph No. _____		
		Occupancy				
		Commercial ☐	Historic ☐	Office ☐		
		Residential ☐	School ☐	Others ☐		
		Max. Number of Persons		Falling Hazards		
		0-10 ☐	11-20 ☐	21-50 ☐	Large Cantilever Slab ☐	
		51-100 ☐	101-1000 ☐		Chimneys ☐ Gamala ☐	
		>1000 ☐			Parapets ☐ Others ☐	
Basic Score, Modifiers and Final Score						
Building Type	ST & AD	BM	BC	RC3	RC5	
Basic Score	1	1.5	2	3	2	
Code or Guidelines Followed	+1 Yes	+1 Yes	+1 Yes	+1.5 Yes	+2 Yes	
	0 No	0 No	0 No	0 No	0 No	
Building Stories (BS)	-0.5 BS>2	-0.5 BS>2	-0.5 BS>2	NA	NA	
	0 BS≤2	0 BS≤2	0 BS≤2			
Column Size (CS)	NA	NA	NA	-1 CS≤9"	-1 CS≤9"	
				0 CS>9"	0 CS>9"	
Verticularity Irregularity	-1 Yes	-1 Yes	-1 Yes	-1 Yes	-1 Yes	
	0 No	0 No	0 No	0 No	0 No	
Plan Irregularity	-0.5 Yes	-0.5 Yes	-0.5 Yes	-0.5 Yes	-0.5 Yes	
	0 No	0 No	0 No	0 No	0 No	
Poor Condition	-0.5 Yes	-0.5 Yes	-0.5 Yes	-0.5 Yes	-0.5 Yes	
	0 No	0 No	0 No	0 No	0 No	
Opening > 40% for BC & >30% for Others	-0.5 Yes	-0.5 Yes	-0.5 Yes	NA	NA	
	0 No	0 No	0 No			
Final Score						
Comments:				Is Building Vulnerable		
				Yes ☐ No ☐		

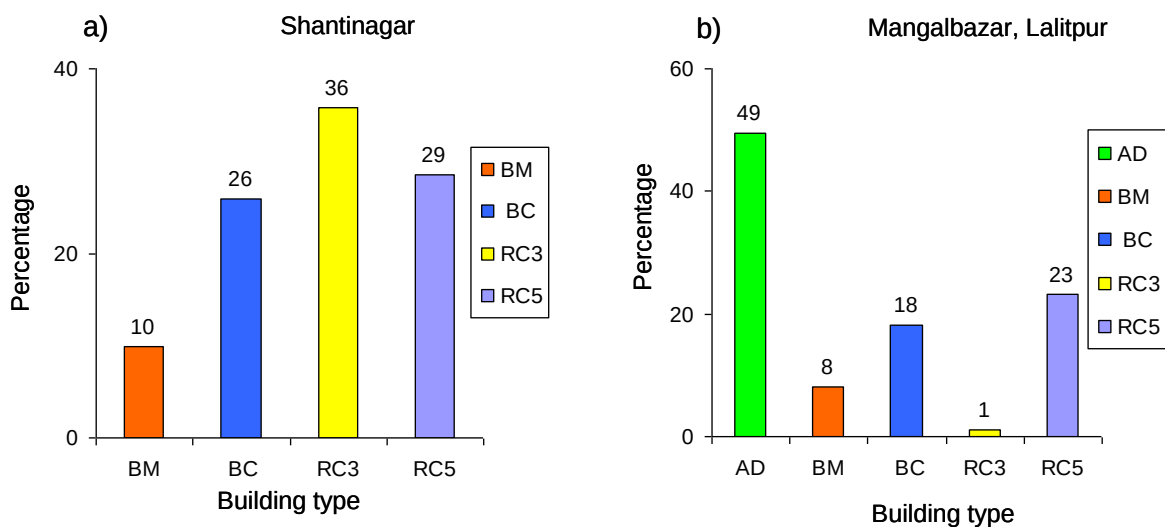
Note: If buildings have more than 5 stories and roof having Tile, RBC and Slate please notify in the comments

- ST & AD – Stone Masonry & Adobe Building
- BM – Brick Masonry with Mud Mortar
- BC – Brick Masonry with Cement or Lime Mortar
- RC3 – Reinforced Concrete Frame with Masonry (RC) of 3 stories or less
- RC5 – Reinforced Concrete Frame with Masonry (RC) of 4 stories or more

7.5.3 Results and Discussion

7.5.3.1 Types of buildings

Figure 7.11 shows the building types in each survey area and average building types in the Kathmandu Valley. Figure 7.11a shows building types in the Shantinagar area, in which the most dominated building types are reinforced concrete buildings (i.e. 65%), mud type buildings may not be seen in that area. Shantinagar area is a newly developed urban center which is connected with the main urban center of the Kathmandu Valley. Most of the buildings are recently constructed. However buildings in Mangalbazar in Lalitpur, Durbar Square in Bhaktapur, and Duwakot areas are still dominated by mud and brick masonry buildings. In Mangal Bazar area about 49% buildings are adobe (Figure 7.11b), and similarly, in Bhaktapur Durbar Square area about 63% percent are adobe buildings (Figure 7.11c). Recently these old residential areas are in the process of replacement by concrete buildings. Duwakot area is also dominated by brick masonry and adobe buildings (Figure 7.11d). After analyzing building types in four survey areas, it can be said that the Kathmandu Valley is still dominated by mud and brick masonry buildings (Figure 7.11e) which are highly vulnerable during strong ground shaking as they are very weak for the resistance of lateral force.



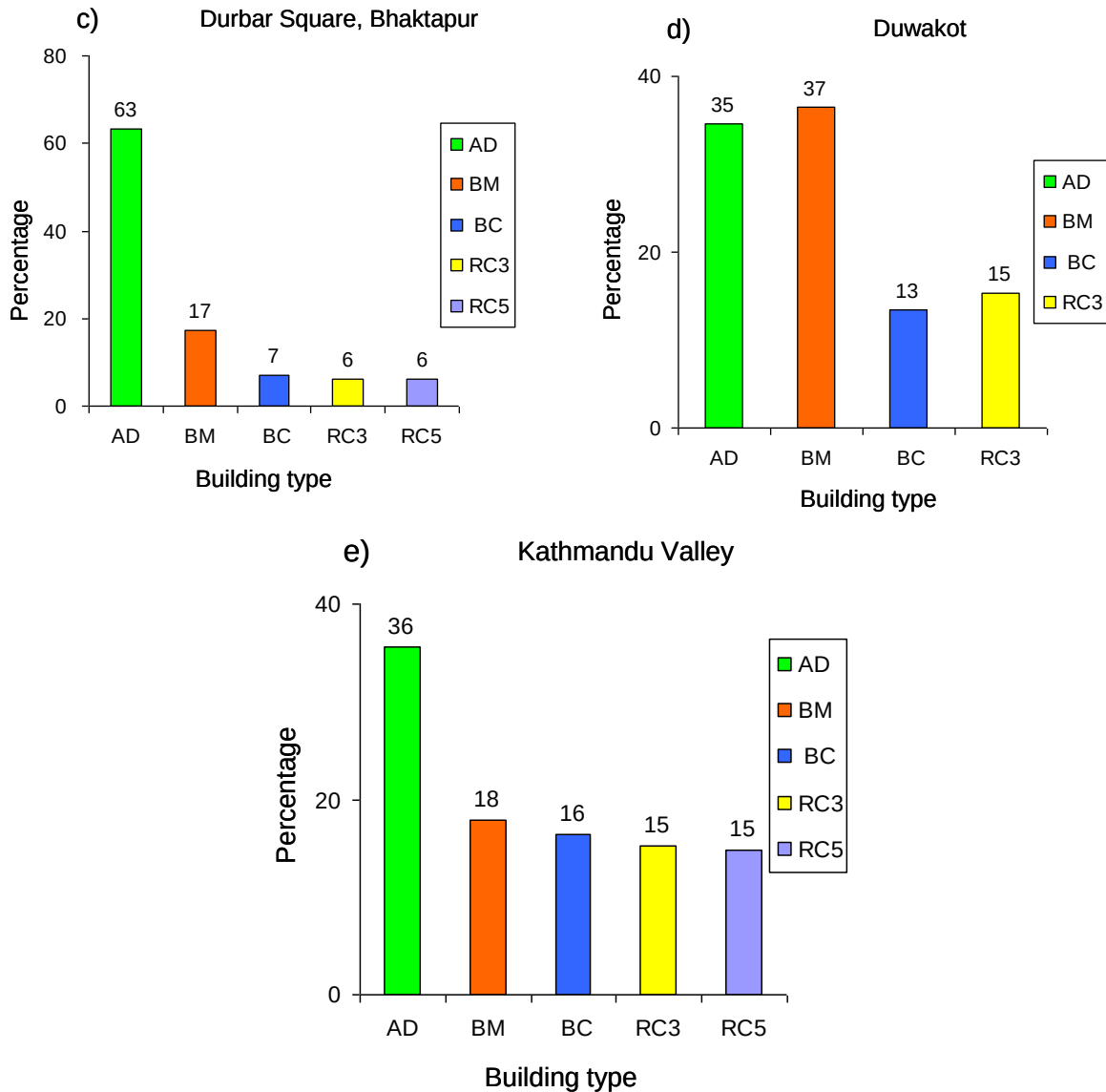
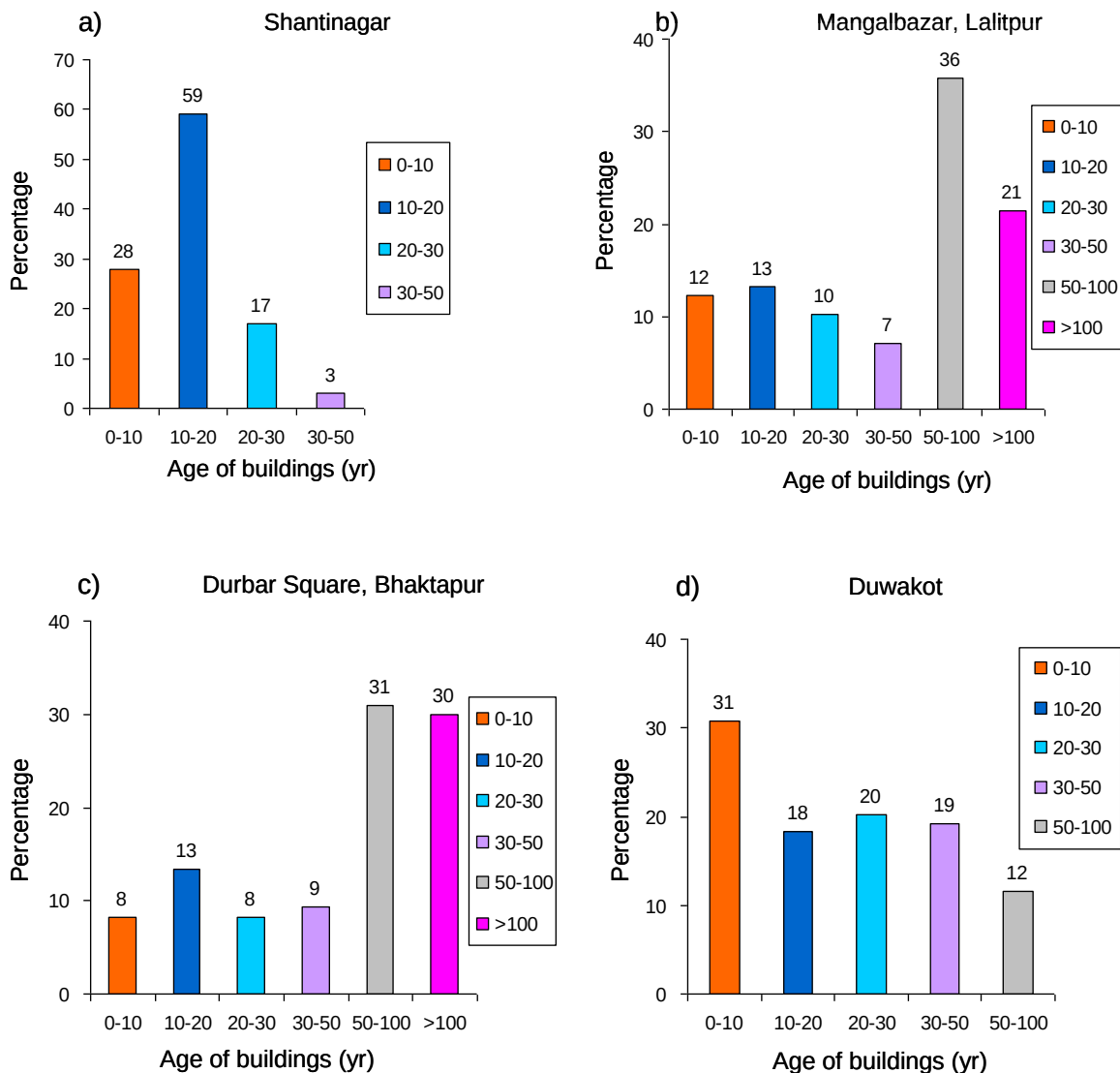


Figure 7.11: Building type in the study area.

7.5.3.2 Age of buildings

Figure 7.12 shows the age of buildings in the study area in the Kathmandu Valley, in which most recently constructed buildings to the very old (i.e. >100 years) buildings can be seen. The age of the buildings are categorized into 6 groups as shown in the Figure 7.12 (i.e., age from 0-10, 10-20, 20-30, 30-50, 50-100, and >100 years). From Figure 7.12a, it is seen that Shantinagar area is mostly dominated by recently constructed buildings, in which most of the buildings (i.e. about 87%) age are less than 20 years. Similarly, Duwakot area is also mostly dominated by recently constructed buildings in which the buildings age are seen within 30 years (Figure 7.12d). As mentioned above, the Shantinagar area is recently developed urban area while the Duwakot area is a typical village which is in the process of

becoming an emerging urban area in the valley in which most of the buildings (i.e. about 49%) are recently constructed (i.e., less than 20 years). Although Mangal Bazar in Lalitpur city and Bhaktapur Durbar Square in Bhaktapur city are highly populated urban center, these areas are dominated by very old buildings, in which most of them are more than 50 years. In Mangal Bazar area about 57% buildings (Figure 7.12b) are more than 50 years, while about 61% (Figure 7.12c) buildings are more than 50 years in Bhaktapur Durbar Square area. As mentioned in the above section, these areas are dominated by mud and brick masonry buildings. Therefore, these areas can be a very dangerous area for the future expected earthquake disaster in the valley. If we observe total buildings in the valley, based on the survey results from the four study area, about 45% buildings are seen less than 20 years in age which is in increasing trend, as the urban area of the Kathmandu Valley is expanding rapidly.



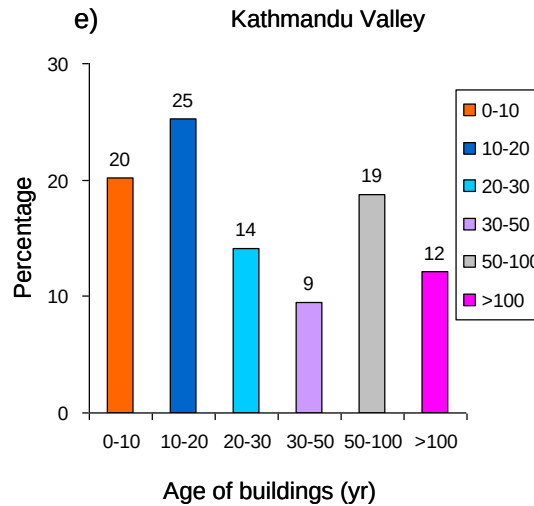
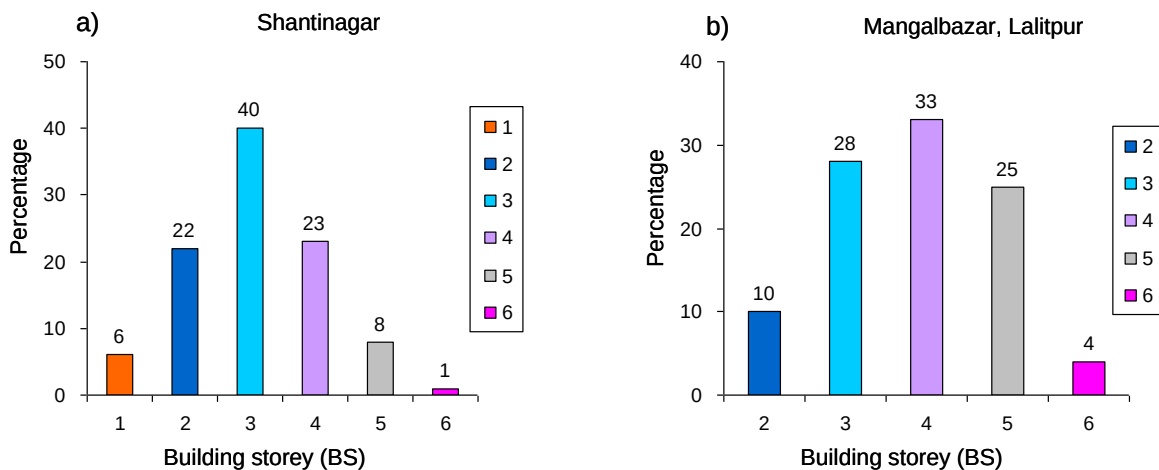


Figure 7.12: Age of the buildings in the study area.

7.5.3.3 Stories of buildings

Kathmandu Valley consists of varieties of buildings in terms of number of stories. Figure 7.13 shows the distribution of number of stories of buildings in the survey areas. The Shantinagar area is mostly dominated by 3 stories buildings (about 40%) (Figure 7.13a), while the Mangal Bazar area in Lalitpur city and Durbar Square area in Bhaktapur city are dominated by 3 to 5 stories buildings (Figure 7.13b, and Figure 7.13c), and Duwakot area mainly consists of 2 and 3 stories buildings (Figure 7.13d). From the Figure 7.13e, it is seen that the Kathmandu Valley is dominated by 3 to 5 stories buildings. From the results of the survey and field study, it is observed that most of the 4 to 6 stories buildings in urban center (i.e. Mangalbazar in Lalitpur and Durbar Square in Bhaktapur) are still dominated by old masonry buildings.



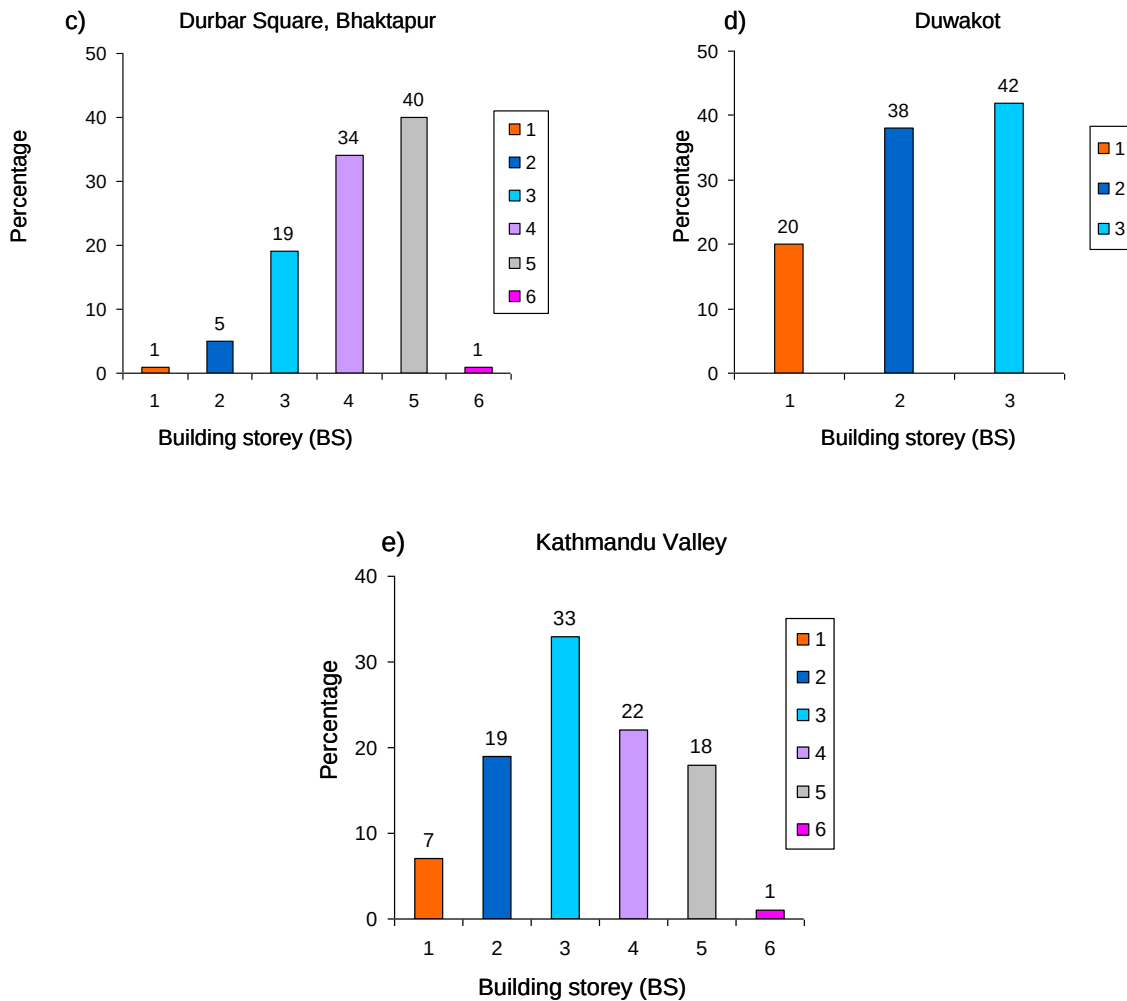


Figure 7.13: Building story in the study area.

7.5.3.4 Vulnerability of buildings

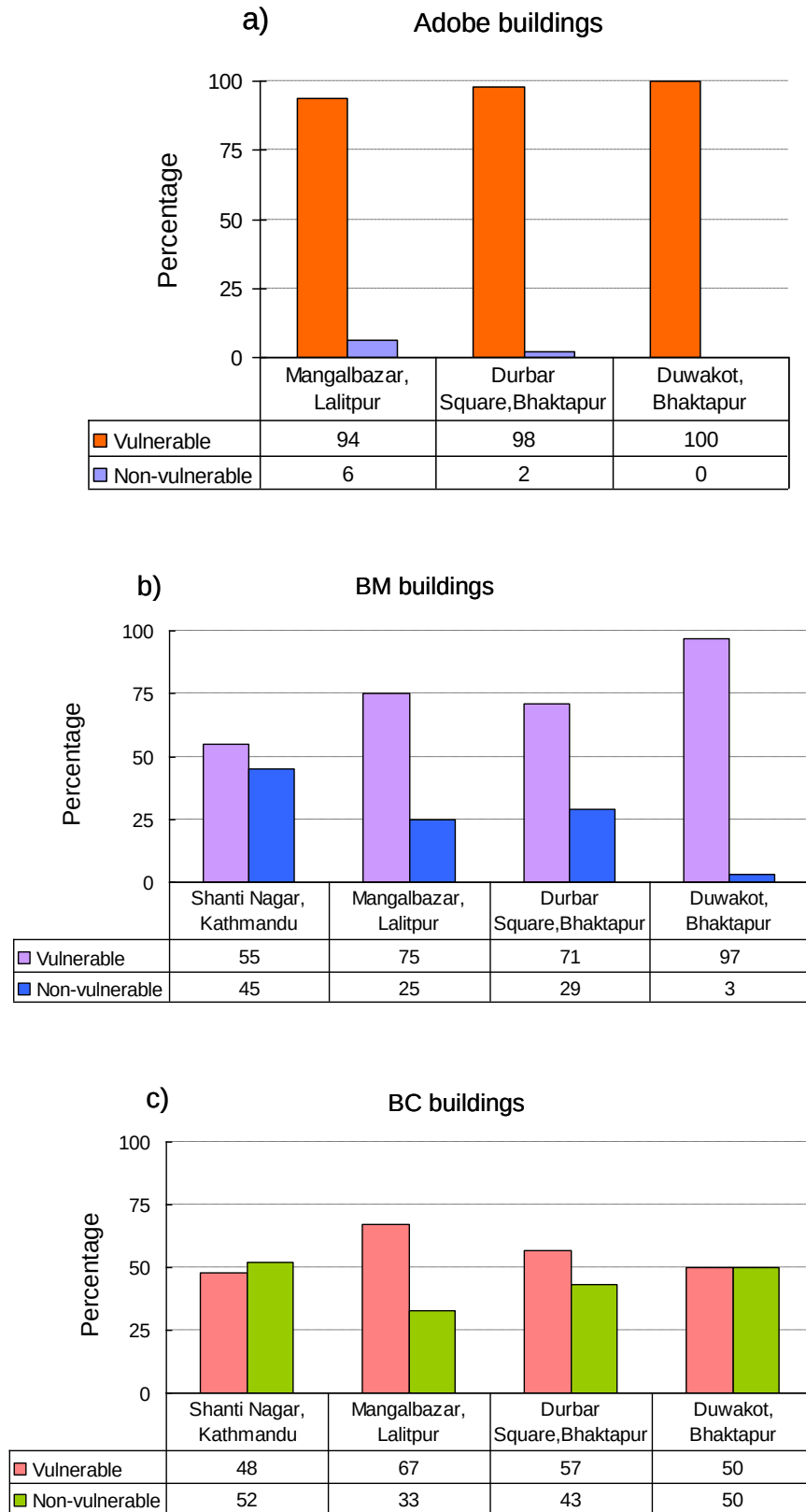
Figure 7.14 shows the results of vulnerability assessment of the buildings in the survey area in the three cities of the Kathmandu Valley. Vulnerability of buildings in each survey areas and condition of each building types are shown in Figure 7.14 a, b, c, d, e, f, and g. Mud buildings are found highly vulnerable in comparison to other building types as shown in Figure 7.14a. In Duwakot area, almost all mud buildings are found vulnerable. Brick Masonry buildings in mud mortar are another type of buildings which are existed mostly in urban and sub-urban area of the valley. They also found vulnerable because of the old age and non-engineered construction practices. About 97% of these buildings in Duwakot area, about 75% in Mangal Bazar area, and about 71% in Bhaktapur Durbar Square area are found vulnerable (Figure 7.14b). This may be due to the presence of unsymmetrical as well as hybrid construction practices of buildings in those areas. Figure 7.14c shows the vulnerability condition of brick masonry in cement sand mortar (BC)

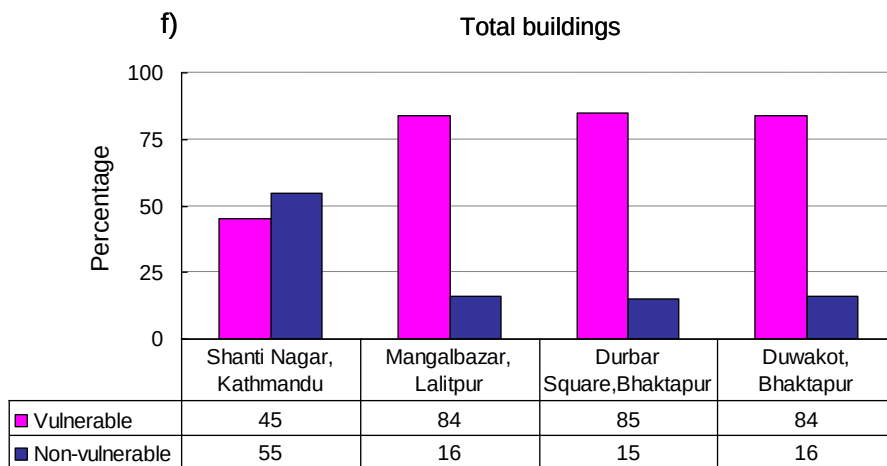
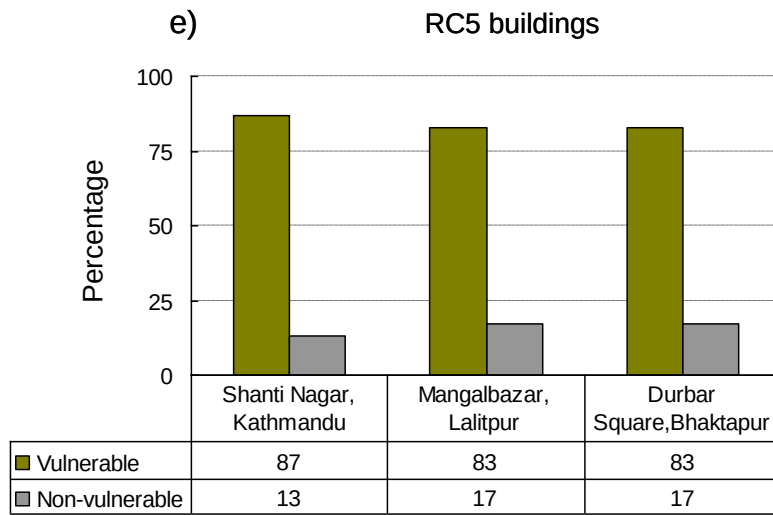
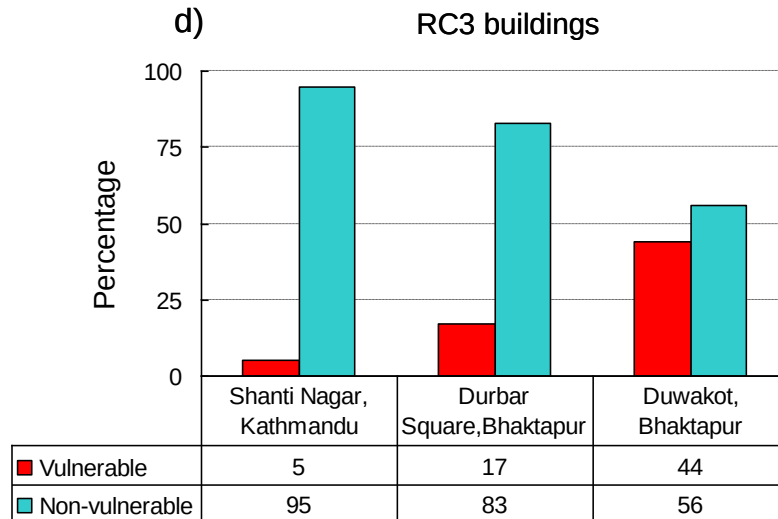
buildings in the study areas. Although they are found stronger than brick masonry buildings with mud mortar, they are made vulnerable by addition of long cantilevers and unsymmetrical floor applying hybrid construction practices. The hybrid construction, such as ground floor wall with mud mortar and upper floor wall with cement sand mortar, or ground floor with cement sand mortar and upper floor with mud mortar wall, practices in the valley made building vulnerable to earthquake. About 52% BC buildings are found non-vulnerable in Shantinagar area; where as only 33% of such buildings are non-vulnerable in Mangal Bazar area (Figure 7.14c).

In comparison to other types of buildings, reinforced concrete buildings upto 3 stories (RC3) are found in good condition. For instant, in Shantinagar area about 95% of such buildings (Figure 7.14d) are found non-vulnerable to earthquake. Similarly in Bhaktapur Durbar Square area and Duwakot area, about 83% and 56% of RC3 type buildings are found non-vulnerable respectively (Figure 7.14d). Similar to mud and brick masonry with mud mortar buildings, reinforced concrete buildings more than 3 stories (RC5) are found most vulnerable in the study areas. As shown in Figure 7.14e, about 87%, and 83% of such buildings are found vulnerable in Shantinagar area, Bhaktapur Durbar Square area and Mangal Bazar area respectively. This is mainly because of the insufficient load bearing systems in the RC5 type buildings. Most of such buildings consist of 9" column with inadequate steel reinforcement. In the Kathmandu Valley, most of such buildings are also found with unsymmetrical in plan and elevation. In comparison to other areas, buildings in Shantinagar area are found good and they can sustain life threatening damage during earthquakes. About 55% buildings in Shantinagar area, about 16% in Mangal Bazar area, about 15% in Bhaktapur Durbar Square area, and about 16% in Duwakot area are found non-vulnerable (Figure 7.14f) to maximum expected earthquake scenario in Kathmandu Valley. As illustrate in Figure 7.14f in comparison to Shantinagar area, buildings in other areas are found highly vulnerable to earthquake.

To sum up, reinforced concrete buildings up to 3 stories and brick masonry buildings with cement sand mortar are found good condition than other types of buildings in the Kathmandu Valley which can be seen from the Figure 7.14g. About 78% of reinforced concrete buildings up to 3 stories are found non-vulnerable in whole study areas (Figure 7.14g). The mud buildings (93%) and reinforced concrete buildings more than 3 stories (84%) are found highly vulnerable buildings in the valley as illustrated in the Figure 7.14g. The result of the survey in whole study area shows that about 73% of buildings are vulnerable and about 27% of buildings are non-vulnerable in the valley (refer Figure

7.14h). Vulnerability of such buildings in the Kathmandu Valley is mainly due to the lack of integrity in the whole structural system and use of poor construction materials.





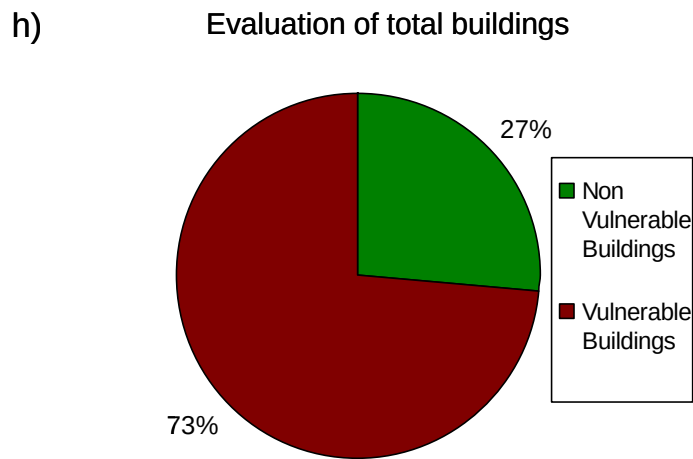
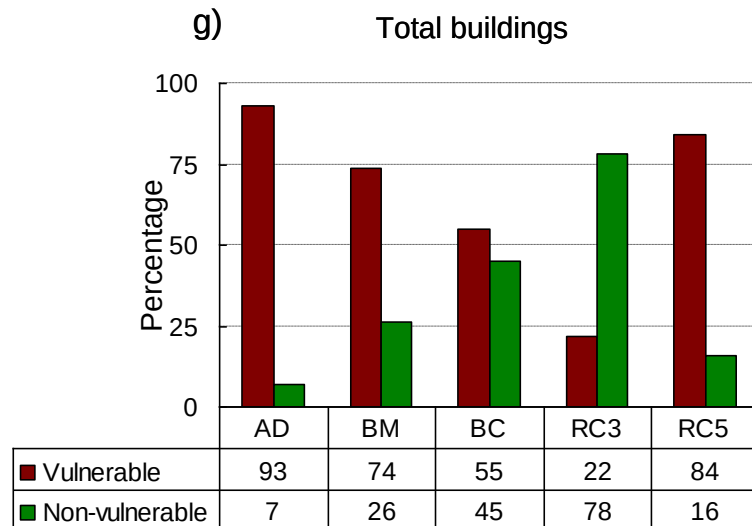


Figure 7.14: Results of the rapid visual vulnerability assessment of buildings in the Kathmandu Valley.

7.6 ANALYSIS OF BUILDING DAMAGE DISTRIBUTION DURING 1934 EARTHQUAKE WITH GEOLOGICAL CONDITION OF THE KATHMANDU BASIN

As mentioned in Chapter 2, Section 2.4.4.2, Kathmandu Valley is an intermontane basin and consists of thick lacustrine sediments. Sakai et al. (2001) have mentioned that the geological condition of valley is similar with the geology of Mexico City which was severely damaged during 1985 Michoacan earthquake ($M_s=7.1$).

In 1934, an earthquake ($M_w=8.1$) was occurred at about 300 km east of the Kathmandu

Valley. This earthquake has caused heavy damage and destruction in the valley than other parts which are near the epicenter area. Pandey and Molnar (1988), and Bilham et al. (1995) have mentioned that this is mainly due to the effect of thick soft lake deposits in the valley which play an important role for the amplification of ground motion. The damage pattern of buildings in the valley was varied from area to area. Most of the buildings were damaged heavily while some are slightly fractured. In whole Nepal, about 207, 000 buildings were affected in which more than 80,000 buildings were completely destroyed. In the case of Kathmandu Valley, about 56,000 buildings were affected which were about 27% of the total affected buildings in whole Nepal. Damage pattern of buildings in the valley were not uniform. In some part, completely destroyed buildings and somewhere slightly fractured buildings were found with in the short distance in the valley (Pandey and Molnar, 1988; Rana, 1935).

Figure 7.15 shows damaged map of the Kathmandu Valley by 1934 Bihar-Nepal Earthquake. Data shows that about 2,359 buildings were completely destroyed in the Bhaktapur City while about 1000, and 725 were completely destroyed in Lalitpur and Kathmandu City. The destruction was very heavy at Bhaktapur and neighboring village in the eastern part of the valley. The ratio of completely destroyed buildings to the much fractured buildings were found more than one in Bhaktapur City, while it was much lower than one in other two cities. This indicated that much of the buildings were severely damaged in Bhaktapur City than other part of the valley. Although Bhaktapur City was the smallest city with less population in comparison to Kathmandu and Lalitpur, total 1,172 people were killed in Bhaktapur City while only 479 and 547 people were killed in Kathmandu and Lalitpur City respectively.

The above discussion shows that the uneven distribution of damage and casualties in the valley may be due to the resonance effect between the ground and the structures built over it. The following section describes in detail about the resonance effect in buildings during earthquakes.

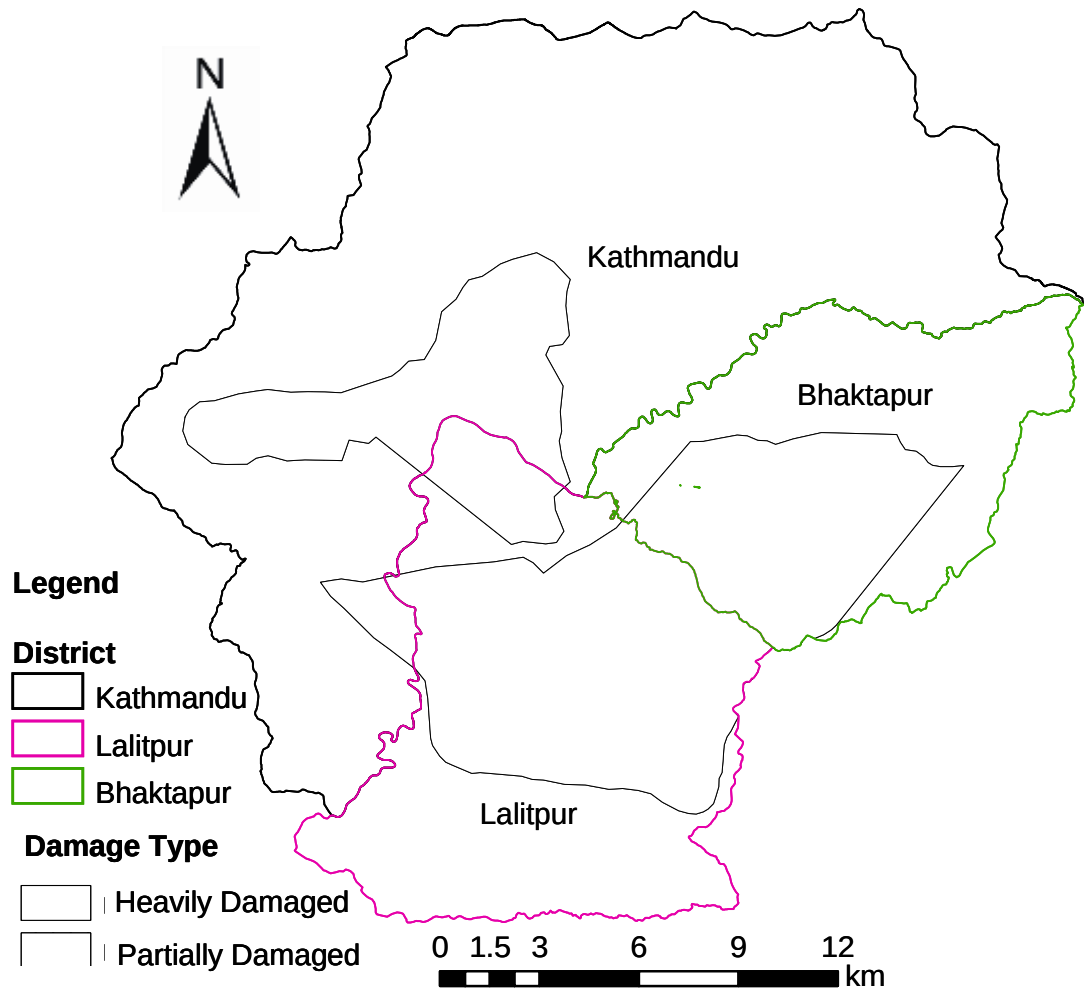


Figure 7.15: Damage map of the Kathmandu Valley by 1934 Bihar-Nepal Earthquake (redrawn after Rana, 1935).

7.6.1 Resonance frequency and drop of resonance frequency in buildings

The dynamic response of the building to earthquake ground motion is the most important cause of earthquake-induced damage to the buildings. Seismic waves generated at the time of earthquake (i.e., faults movement) are not all of a uniform character. As these waves pass through the earth on their way from the fault to the building site, they are modified by the soil and rock through which they pass. Once the seismic waves reach the building site they undergo further modifications that are different on characteristics of the ground and soil beneath the building. Therefore, amount of damage and response of the earthquake on the building can depend on source effects, path effects and local site effects. The characteristics of ground motions which have been the greatest importance for buildings are the duration, amplitude (of displacement, velocity and acceleration) and frequency of ground motion (Kramer, 1996).

Surface ground motion at the building site is actually a complex superposition of different vibration frequencies. Therefore, the response of building to ground motion is also a complex phenomenon which also depends upon the vibration characteristics of the buildings. However the building's vibrations tend to center around one particular frequency that is known as its natural or fundamental frequency. Generally, the shorter a building is the higher its natural frequency, and the taller the building is the lower its natural frequency.

When the frequency contents of the ground motion are centered around the buildings natural frequency, it can be said that the building and the ground motion are in resonance with one another. Resonance tends to increase or amplify the buildings response. Therefore, the buildings suffer the greatest damage from ground motion at a frequency close or equal to their own natural frequency. Damage of buildings during Mexico earthquake of 1985 is the one of the examples of resonance effect. A majority of the buildings which collapsed during this earthquake were around 20 stories tall. They had a natural period of around 2.0 seconds (ICBO, 1997). These 20 stories buildings were in resonance with the frequency contents of the earthquake ground motion in the particular area. However, other buildings which have different heights and with different vibration characteristics were often found undamaged, even though they were located right next to the damaged 20 story buildings. From these observations, it can be said that different buildings can respond in widely differing manners to the same earthquake ground motion. Chopra (1995) defined three cases for soil structure interactions based on the relationship between natural frequency of vibration of buildings (ω) and the forcing frequency of external forces (ω_0):

1. if $\omega_0/\omega \cong 0$, i.e., the forcing frequency is much smaller than the natural frequency, then dynamic effects are negligible and maximum responses depend on the structure stiffness.
2. if $\omega_0/\omega > 1$, i.e., the forcing frequency is much higher than the natural frequency, then response approach zero and depend mainly on the mass.
3. if $\omega_0/\omega = 1$, i.e., the forcing frequency is equal to the natural frequency of the buildings, then response is maximum and governed by the damping ratio.

Therefore, the damage distributions are often seen to be dependent on the predominant frequency of soil layers and natural frequency of the buildings (e.g., Navarro and Oliveiram, 2006). Evaluation of these two parameters is very important in computations for seismic planning of structures. Moreover, the natural frequency of a structure is directly proportional to the square root of stiffness and inversely proportional to the square root of

mass. It has been observed that the system mass has not change significantly during the life span but the system stiffness change significantly due to age (deterioration) and earthquakes effects (cracks in buildings). The decrease of natural frequency in structure may be due to the various causes. The reduction of natural frequency may be brought about by the reduction of stiffness, by the increase of weight (due to increase of moisture) or by the change of the structure (formation of yielding hinges, changes in subsoil). In the case of concrete structure the reduction in stiffness may be due to:

- ◆ the degradation of concrete
- ◆ the creep of concrete
- ◆ the cross-section weakening of cracks
- ◆ change in boundary condition etc.

The decrease in natural frequency in a structure is also depend on the level of ground shaking and the associated level of non-linearity in the soil-structure system. Many researchers had conducted experimental tests and also measured response of buildings during earthquakes to evaluate the natural frequency of structures. Some of the example of period elongation (or drop in frequency) in structure are discussed below:

Mucciareli et al. (2004) were measured the response of a four story building during the M5.3 earthquake on November 1 2002 in southern Italy. This building was already suffered substantial damage (European Macro Seismic Scale, EMS, damage 3) during the earthquake M5.4 just one day before i.e. on Oct 31 2002, and ambient vibration tests were being carried out to evaluate the frequency of the damaged building when the second earthquake took place (which increase the damage to level EMS 4). The strong ground motion measurement that was obtained was then processed using four different techniques: Short-Time Fourier transform (STFT), wavelet transform (WT), horizontal-to-vertical moving window ratio (HVMWR), and horizontal-to-vertical spectral ratio (HVSr). The frequency shift has then been calculated using Equation (7.1) considering the difference between the initial frequency (F_{in}) and the minimum frequency (F_{min}) in the record. The obtained results are shown in Table 7.2.

$$(\Delta F)_{\max} = (F_{in} - F_{\min}) / F_{in} \dots\dots\dots (7.1)$$

Table 7.2: Estimation of frequency shift using different techniques from the measured response of a damaged building after the 2002 Molise earthquake (adapted from Mucciarelli et al., 2004).

Method	North-South Frequency (Hz)				East-West Frequency (Hz)			
	F_{in}	F_{end}	F_{min}	$(\Delta F)_{max}$	F_{in}	F_{end}	F_{min}	$(\Delta F)_{max}$
STFT	2.5	1.9	1.3	-48%	3.2	2	1.8	-43%
WT	1.5	1.3	1	-33%	2	1.3	1	-50%
HVMWR	2.5	2.1	1.8	-28%	2.7	2	1.5	-44%
HVSR	2.45	1.9	1.5	-38%	2.8	1.9	1.8	-35%

As mentioned in the above table, the frequency shift varies between 28% and 48% in the N-S direction and between 35% and 50% in the E-W direction. Mucciarelli et al. (2004) have mentioned that such a large reduction in frequency can be attributed to the nonlinear behavior of the frame members and the phenomena such as the formation of plastic hinges and crack opening which causes a reduction of the frequency and also lead to an increase in structural damping and decelerate the oscillation of the building. According to them, this reduction in the seismic excitation causes a subsequent reduction in the nonlinear effects and thus the frequency is observed to increase again. Nevertheless, the final values of frequency are smaller than the initial values due to the accumulation of damage.

Naeim (1998) has applied moving window Fast Fourier Transform (FFT) on the response of 20 instrumented buildings subjected to the 1994 Northridge Earthquake and observed that in the buildings without structural damage the period of the building did not elongate significantly, or elongated and then came back to the vicinity of the initial value towards the end of the ground motion. In the buildings which suffered structural damage, however, the period elongated significantly, as shown in Figure 7.16 below. As illustrated in Figure 7.16a, there was a 50% increase in the period of vibration (from 1.4s to 2.1s) from the beginning to the end of the record, which means a 32% change in the frequency (0.71Hz to 0.48Hz). However, the period reaches approximately 2.3s during the response which can be interpreted as a 64% period elongation and a 40% frequency shift. Anderson and Filippou (1997) have recorded a similar period elongation in a 17 storey steel building as shown in Figure 7.16b (Naeim, 1998). The period of vibration of this building increases

by 53% in the north-south direction and 20% in the east-west direction. These observations illustrate that the decrease in the frequency and increase in the period in each direction is related to the orientation of the building and also the principle direction of the earthquake.

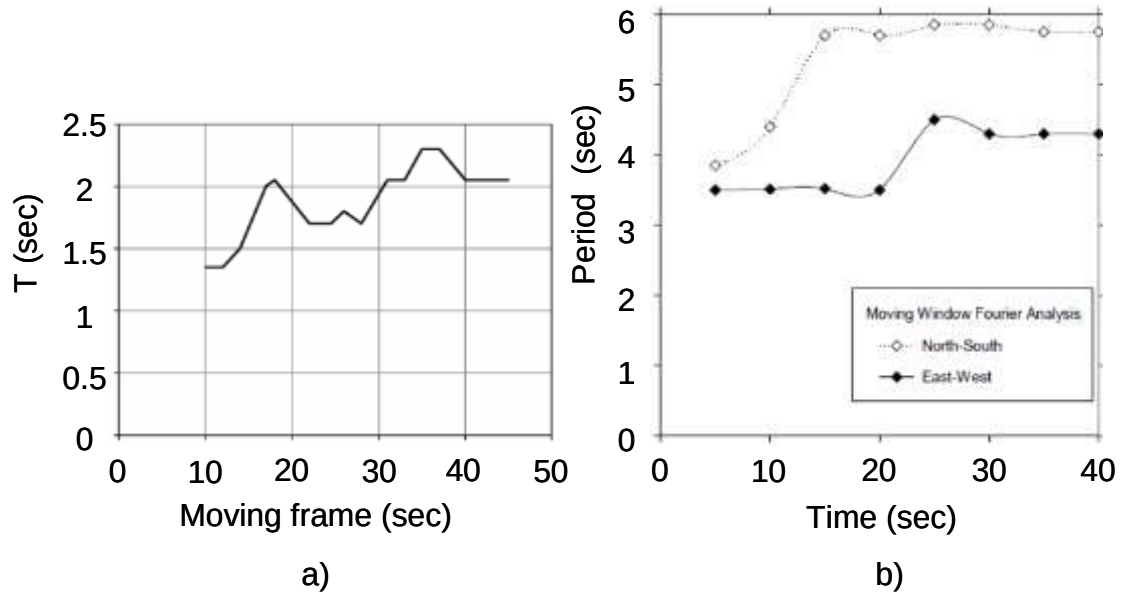


Figure 7.16: Period elongation (or frequency reduction) due to structural damage (a) Van Nuys 7-storey building (adapted from Naeim, 1998), and (b) 17-storey steel building (adapted from Anderson and Filippou, 1997).

Iwan and Cifuentes (1986) have made a model to study the dynamic behavior of degrading hysteretic structures subjected to damaging earthquake motions in which the variation of the effective stiffness of the structure is displayed as a function of the amplitude of the oscillation. The identify model has been applied to the measured records from two reinforced concrete buildings. Figure 7.17a and b show the effective stiffness diagrams for the Bank of California and the Imperial Country Services buildings, respectively. The initial value and the final value of the effective stiffness can be observed to be very different and shows that the structures did not recover their initial stiffness after the respective earthquakes to which they were subjected. In both structures, about 60% loss in stiffness was seen which relates to a 37% frequency shift and a 58% period elongation (Iwan and Cifuentes, 1986).

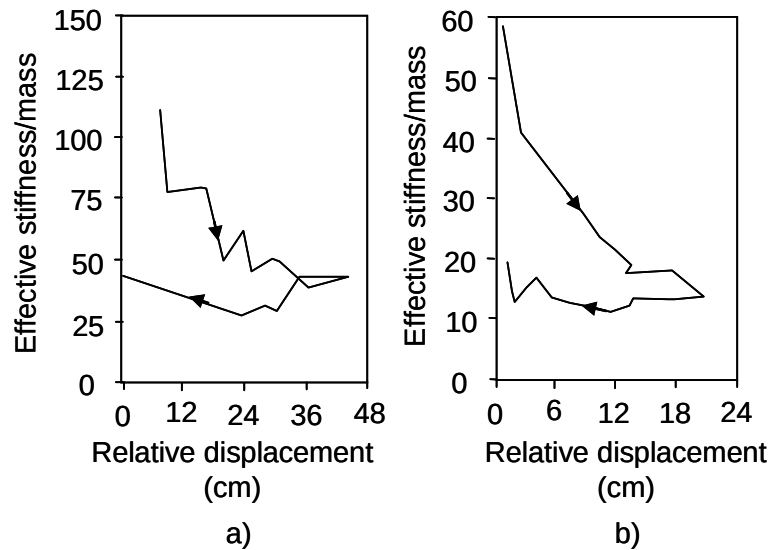


Figure 7.17: Reduction of frequency due to decrease in effective stiffness in the buildings.

Hashemi and Mosalam (2006) presented the shaking table test on a one-storey prototype in-filled RC frame which was designed as ACI 2002 NEHRP recommendations. Three different ground motions scaled to different levels of intensity were used as an input motion and the experiment was performed in three distinct stages of input ground motions. The detailed of each stage can be found in Hashemi and Mosalam (2006). The decrease in the natural frequency for each stage of the test is shown in Figure 7.18, and it can be observed that at the end of this sequence of testing (AWR-DUZ 9), the structure had experienced a frequency decrease from approximately 5.5Hz to 2Hz (a 64% frequency shift and a 175% period elongation). At this stage of the test, partial collapse of the infill panel had occurred and slight cracking was observed in the foundation, and beam-column joints. Due to the large shift in the natural frequency, in order to further increase the demand, a different ground motion with low-frequency spectral response was required and the Loma Prieta ground motion was thus chosen. Further damage at the bases of the columns and beam-column joints was observed and significant yielding of the rebar was recorded in the “LomaPr 7” test. The third and final stage of the test was performed to investigate the collapse sequence of the structure without the infill wall and a natural frequency of approximately 1.5Hz was reached. The overall frequency shift and period elongation were found to be 83% and 267%, respectively.

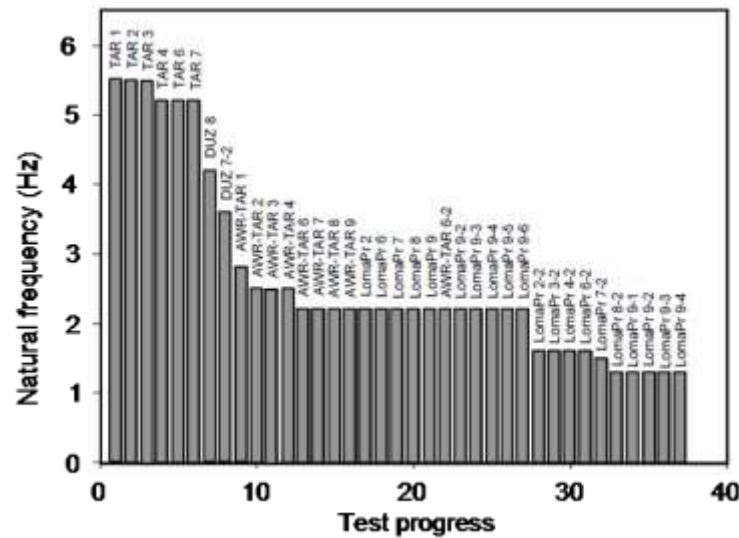


Figure 7.18: Decrease in natural frequency of a one storey frame for different stages and levels of the shake table test (adapted from Hashemi and Mosalam, 2006).

There are many examples about the reduction of frequency of structure due to age. Michel et al. (2011) have investigated the fundamental frequency drop for un-reinforced masonry buildings from dynamic tests. They mentioned that the structural frequency drop depends mainly on shaking amplitude, and also the soil-structure interaction. The opening of cracks in the construction materials produces a temporary or permanent decrease in stiffness and therefore a decrease in the fundamental frequency and possibly an increase in the damping ratio. The observed frequency in the buildings may also be affected by other phenomenon such as variation of the frequency content of the input motion (Michel and Gueguen, 2010), temperature (Clinton et al., 2006), or the soil-structure interaction (SS) effects (Todorovska, 2009).

7.6.2 Analysis of buildings damage distribution during 1934 earthquake

In Chapter 4, we have described the seismic microzonation of the Kathmandu Valley. Microzonation study describes the variation in seismic response of the subsurface and subsequently determines whether the soil is being amplified to a level that may damage existing buildings or other structures at that location.

Based on the microtremor observations in three cities of the Kathmandu Valley, it is observed that predominant period of ground varies from 0.1 s to 2 s. As illustrated in Figure 7.19, predominant time period varies from 1.0 s to 2.0 s around the center core area of the Kathmandu city. Similarly, the period for the Lalitpur city varies from 0.8 s to 1.3 s and for the Bhaktapur city varies from 0.5 s to 1.0 s. The characteristics site period

provides a very useful indication of the period of vibration at which the most significant amplification can be expected. The largest amplification of the soil will occur at the lowest natural frequency (i.e., at corresponding characteristic site period). The response of a building to shaking of ground depends ‘of course’ on the design quality of the construction. However, what is very important factor is the height of the buildings. It has been shown that the tall buildings react totally different than smaller buildings.

As mentioned elsewhere in this chapter (i.e., Section 7.6.1), the damage and destruction of the buildings during earthquakes mainly depend on the resonance effect, which only happen when the period of seismic ground motion is similar or close to the period of the buildings. The period of the buildings primarily depend upon the height (stories) and also the other factors which may increase the period of vibrations, such as age of buildings, structural condition of buildings, soil structure interaction during shaking etc.

The result of microtremor observation indicates that the possibility of devastation of higher period structure is high in the core city area of the Kathmandu, where as the short to medium period structure is high in the other two cities. According to the result of the survey of buildings in the Kathmandu Valley, it has been found that core city area encompasses buildings that are mainly 3 to 5 stories in height as illustrated in Figure 7.13. Most of them are old masonry buildings (refer Figure 7.11 and Figure 7.12) and some of them have already experienced earthquakes in the past. Using the crude rule of thumb, the fundamental period of an N-story building is approximately $N/10$ sec, which results in an estimated predominant period of the Kathmandu Valley buildings to be around 0.3 s to 0.5 s as shown in Table 7.3. However according to the different experimental and numerical studies from the different researches, as mentioned in previously (i.e. Section 7.6.1), have shown that the time period of such buildings can be increased up to 60 percent and/or more, depending upon the structural condition of buildings. As mentioned in the earlier paragraphs, the main reasons for increase of time period of buildings in its life span is due to the decrease of stiffness because of increase in weight (due to increase of moisture content) or cracking in building (due to poor maintenance or earthquakes). The elongation of period can also be observe during strong shaking due to non linearity effect in soil and structure itself.

In this study, it is assumed that the increase in period of buildings may be around 60 percent. Then, the new period of buildings is calculated around 0.48 s to 0.96 s as shown in Table 7.3. If we observe the predominant period variation of ground in three cities, it can be seen that the new time period of buildings are nearly equal or very close to the time

period in the eastern part of the valley especially around Bhaktapur City. This causes the resonance between ground and the buildings in that area and therefore buildings vibrate with higher amplitude and finally damage of buildings is occurred. This could be concluded that the heavy devastation of buildings in the Bhaktapur City during 1934 Bihar-Nepal Earthquake was definitely due to the resonance effect between ground and the buildings in that area.

Table 7.3: Time period of buildings of Kathmandu Valley based on crude rule of thumb.

SN	Building Type	Story	Period (T) s	New Period (T_n) due to stiffness degradation (s)
1	Masonry	3	0.3	0.48
2	Masonry	4	0.4	0.64
3	Masonry	5	0.5	0.80
4	Masonry	6	0.6	0.96

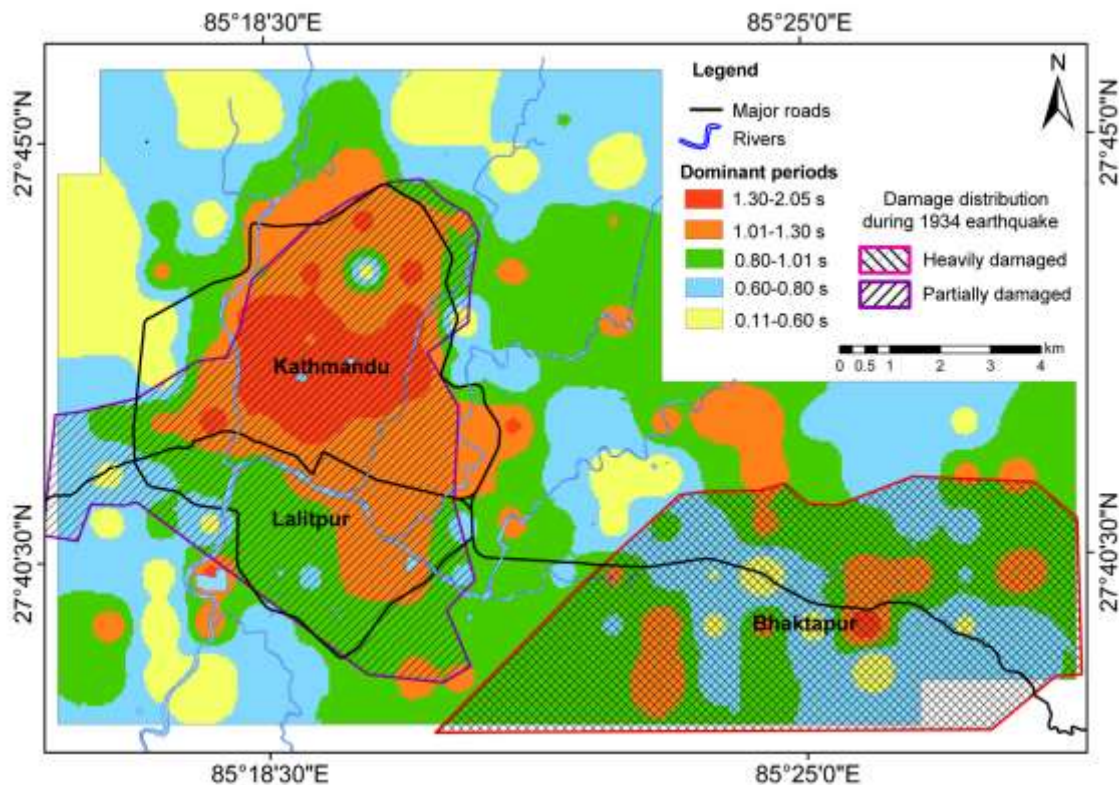


Figure 7.19: Dominant period variation map of the Kathmandu Valley. Single and double hatched area show the partially damaged and heavily damaged of buildings during 1934 Bihar-Nepal Earthquake.

7.7 CHAPTER SUMMARY

In order to explore the earthquake hazard damage risk in the Kathmandu Valley, the study about the earthquake vulnerability condition of the buildings was carried out in four typical locations of the Kathmandu Valley. As an oldest urban and commercial center as well as some emerging urban areas in an around the core urban center, the valley consists of various types of buildings in terms of building materials used in construction and number of stories. The analysis results show that the valley core center is still dominated by masonry buildings with 3 to 5 stories. Although the replacement of old buildings with new RCC structures is going on, most of the buildings are still very old (more than 50 years), and most of them are in poor condition (lack of maintenance, cracks in building etc.).

The vulnerability assessment results show that most of the buildings in the urban center of the Kathmandu Valley are vulnerable to earthquake. The results demonstrated that comparatively Shantinagar area is found less vulnerable to earthquake. This may be due to the presence of recently constructed buildings which are mostly about 3 stories and also absence of mud buildings. Bhaktapur Durbar Square area is found highly vulnerable to earthquakes which mostly consist of old masonry buildings. In total about 27% of buildings are found non-vulnerable where as about 73% buildings are found vulnerable in the Kathmandu Valley. This is because the most dominant types of buildings in that area are adobe (AD) (mud) and brick masonry in mud mortar (BM) and the current construction practices still use mud mortar in brick masonry buildings without paying due consideration to earthquake resistant features.

In this chapter, based on the result of microtremor observations, comparison between predominant period of ground and buildings in the Kathmandu Valley is also performed. The result shows that the predominant periods of ground in and around Bhaktapur City is found equal or very close to the predominant periods of the buildings. From this observation, it could be concluded that the higher damage of buildings in and around the Bhaktapur City during 1934 Bihar-Nepal Earthquake was due to the resonance effect between the ground and the structure during strong ground shaking.

CHAPTER 8

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

Kathmandu Valley with an estimated current population of 5 million has been shaken many times in the past by damaging earthquakes. Despite a rising awareness level of earthquake disaster risk in the valley, and a high probability of a major earthquake in the future, much of the risk reduction efforts have focused only on management and legal issues leaving the scientific aspects of potential damage in shadow. Particularly, the dynamic behavior of the sediment deposit must be considered while assessing the earthquake disaster risk in the Kathmandu Valley.

There are many problems regarding the earthquake hazard risk in the Kathmandu Valley. Rapid population growth, poor land use planning, precarious settlement patterns, and inadequate enforcement of building code(s) are important factors for making Kathmandu vulnerable to earthquakes. This information elsewhere mentioned in this study creates an urgent need to implement the earthquake disaster risk mitigation program in the valley. The main objectives of earthquake disaster mitigation program are to minimize the structural damage and reduce the casualties, when a great earthquake hits the area.

In the present study, the earthquake disaster risk evaluation of the Kathmandu Valley has been done for considering the three major objectives: 1) evaluation of geological condition of the Kathmandu Valley, based on the borehole data analysis and literature review, 2) evaluation of ground response characteristics of the Kathmandu Basin during an earthquake, based on microtremor data analysis, and 3) evaluation of building vulnerability, based on rapid visual seismic vulnerability assessment of buildings in the valley. The study is carried out step by step to draw some conclusion to meet these objectives. After processing and analyzing the data, the following important conclusions are drawn.

Kathmandu Valley sits on an ancient lake deposits as high as 500 m in the central part of the valley. The evaluation of geological condition of sediment deposit through borehole data analysis shows variation of sediment thickness and properties within the short distances of the valley. Due to lack of enough geotechnical information in the borehole

data, which were mainly explored during water supply purposes and cores were not fully recovered, much of the subsurface structure of the Kathmandu Valley is still unknown. However, the compilation and analysis of borehole data in this study shows that the bedrock level in the valley is ranging in depth from 48 m in Sanepa at southern part to 545 m in the central part of the valley. From this analysis, it is observed that the depositional environment of the sediments within the valley varies from place to place which is strongly controlled by the topographic situation of the valley. The sediment in the northern part of the basin are poorly sorted with highly micaceous coarse sand, and gravels, while in the central and southern half part mainly consist of a thick sequence of cohesive soil consisting of dark gray to black highly and medium plastic organic clay, and southern part mainly consists of alluvial fan facies. This indicates that the valley sediment thickness and properties vary from place to place, which may cause trapping and focusing of seismic waves during an earthquake and leading to an evident change of resonance frequency over the short distance.

There are two groups of methods for analyzing the variation of ground motion due to subsoil characteristics: 1) analytical techniques, and 2) instrumental techniques. The analytical techniques, such as borehole drilling are good and can give comparatively accurate information of the site response, however it is time consuming and required large number of input parameters which may not be possible while covering large area like Kathmandu Valley. Evaluation of ground characteristics through installation of seismic observation at required site is also not feasible in the densely urbanized area of the Kathmandu Valley. Therefore, the horizontal-to-vertical spectral ratio technique using microtremor observation at required site is only the possible option for site effect study for the valley; since microtremor can be available anywhere and any place and not depend on the seismic events. Moreover, the measurement of microtremor and data acquisition is easy and economical, and hence this method becomes one of the widely used techniques for ground response analysis for developing countries like Nepal.

Microtremor data analysis at a total of 172 measurement points within the area of 210 square kilometers in the densely populated urban area of the Kathmandu Valley is performed to address the issue of earthquake hazard risk mitigation in the Kathmandu Valley. Based on the obtained dominant period in each predetermined location (i.e., 1-km grid points), an estimated dominant period of shaking map of the Kathmandu Valley is prepared in which the study area is divided into five dominant period ranges (i.e. period range A: 0.11 s to 0.60 s, period range B: 0.60 s to 0.80 s, period range C: 0.80 s to 1.01 s,

period range D: 1.01 s to 1.30 s, and period range E: 1.30 s to 2.05 s). This result reveals that the central part of the valley that cover about 30 percent of the urban area of the valley falls under dominant period range from 1.0 s to 2.0 s (i.e., dominant period range D and E), while the outer parts fall under dominant period range from 0.1 s to 1.0 s (i.e. dominant period range A to C). The longer period range in the central part correlates with the extent of depth of Pliocene and Quaternary deposits, while the shorter dominant period in the outskirts corresponds to shallow stiffer near the rocks flanking the valley sides.

The evaluation of 172 horizontal-to-vertical spectral ratio curves reveal that the valley sediment is characterized by two amplified frequencies, which vary from 0.48 Hz to 8.89 Hz in first resonance and from 3.1 Hz to 7.5 Hz (mostly from 4 Hz to 6 Hz) in second resonance. These two amplified frequencies were detected at about 20% of the sites that are mostly distributed in the central and northern part of the valley. These two resonance frequencies are characteristic to particular site response in two scales, including the deep and surface soil layer. Depending upon the area, especially in the central and northern parts, it is found that the top 10-20 m of the sediment layer plays an important role in causing the second resonant effect in the valley.

The study of the correlation between the depth of sediment with the obtained predominant frequencies reveals that the soft-sediment depth of the Kathmandu Valley can be estimated using a non-linear regression equation $h=146.01f_r^{-1.2079}$ and provides the hidden basement topography of the Kathmandu Valley. The Digital Elevation Model (DEM) of the sediment distribution profiles along west-to-east and south-to-north direction are presented. The calculated depth of the sediment represents the total depth of lake deposit which is overlain by basement rock in the Kathmandu Valley which may not necessarily indicate the presence of hard rock, rather it is a representation of the presence of basement layer at the depth, beyond which the sediment may or may not contribute the amplification of seismic waves. The obtained map represents the first reliable reconstruction of the subsurface morphology of the Kathmandu Valley, which shows a good consistency with available geological log data.

The constructed maps, based on the derived sediment depth and frequency relationship for the Kathmandu Valley, show a number of smaller depressions in the study area; however two large depressions are found. The first depression at the central part of the Kathmandu City which is wider and deeper and represents the main ancient lake of the Kathmandu Valley, while the other is along the eastern part which is relatively shallow, and its catchments area elongated from northwest to southeast. Similarly there are number

of buried ridges which separated/connected the depressions with each other. The longest buried ridge which separated/connected the central large and deep depression with the eastern shallow depression is extended from northwest to southeast part of the valley.

From the results of ground response analysis in the Kathmandu Valley, it can be said that the uneven distribution of sediment in the Kathmandu Basin may create two kinds of problems for the valley buildings during strong ground shaking. One may be from the surface layer at the central as well as at the northern part of the valley and the shallow depth of sediment above bedrock in the outer area, and other may be from the thick sediment in and around the central part. As higher resonant frequencies (i.e., lower dominant periods) are characteristic to the thin sediment layer, this may mostly affect the low- to medium-rise buildings while the lower frequencies (i.e., longer periods) are characteristic to the thick sediment, which may affect medium- to high-rise buildings during an earthquake events. More specifically, if the predominant frequency (or period) of a ground is equal or close to predominate frequency (or period) of a structure over the ground, resonance will take place leading to an enhanced vibration of the structure and then it collapses. Additionally, because of increasing population and development into a greater commercial hub, the central part has seen a sharp rise in the number of mid- height to tall buildings, which have been constructed under inadequate geotechnical investigations. In order to reduce the disaster risk, therefore, the multilayer ground effect must be considered in seismic design of long period structures in the valley center. At the same time, it may also be necessary to consider enhanced seismic design criteria for medium- and short-period structures in the valley.

In order to explore the earthquake hazard damage risk of buildings in the Kathmandu Valley, a study about the rapid visual seismic vulnerability assessment of the buildings is also carried out in four typical locations of the three cities of the valley. The analysis results show that the valley consists of various types of buildings in terms of building materials used in construction and number of stories of buildings, however urban core center is still dominated by masonry buildings (i.e. about 70%) with 3 to 5 stories (i.e., about 73%). Although the replacement of old masonry buildings with new reinforced cement concrete buildings is taking place, most of the buildings are still very old (i.e., about 31% more than 50 years), and most of them are in poor condition (lack of maintenance, cracks in buildings etc.). The vulnerability assessment results show that most of the buildings in the urban center of the valley are vulnerable to earthquake. The results demonstrated that comparatively Shantinagar area, a recently developed urban center in the

valley, is found less vulnerable (i.e., about 45%) to earthquake, which is due to the newly constructed (i.e. about 87% less than 20 years) buildings with three or less stories (i.e. about 68%) and also absence of mud buildings. Bhaktapur Durbar Square (eastern part of the valley) area is found highly vulnerable (i.e., about 85%) to earthquake which mostly consists of old buildings (i.e., about 61% more than 50 years) and accommodates mostly masonry (i.e. about 87%) buildings. In total, about 27% of the buildings are found non-vulnerable where as about 73% buildings are found vulnerable to earthquake in the whole survey area. This large number of vulnerable buildings in the survey area may be due to two reasons. Firstly, the three core urban centers of the valley (i.e., Kathmandu, Lalitpur and Bhaktapur) still accommodates old mud (i.e. about 36%) and brick masonry in mud mortar (i.e. about 18%) buildings, and the current construction practices in the valley is still not follow the building code, and more specifically modern buildings type such as reinforced concrete buildings are constructed without paying due consideration to earthquake resistant features. Secondly, buildings vulnerability survey areas in this study are mostly located in the old urban center of the valley, and in some of the areas still use brick masonry with mud mortar for construction of buildings, which are very weak for lateral load resistant during earthquake shaking.

The obtained results, such as estimated dominant period of shaking map for the valley, multiple resonance frequency area in the valley, frequency-depth relationship for the Kathmandu Valley sediment, and vulnerability condition of Kathmandu's buildings are the major findings of this study. These provide a basis for analyzing the earthquake disaster risk to structures in the valley. The obtained maps and results act as a reference for evaluation of ground motion characteristics and building vulnerability assessment in the valley, but they do not provide specific hazard parameters for specific sites in the valley.

8.2 LIMITATIONS AND RECOMMENDATIONS

Kathmandu Valley has been facing lots of problems for the mitigation of the risk due to earthquake hazard. Due to lack of precise data/information in this field, most of the studies were based on the assumption and most of them used data from different sources, as a result the outcomes might not be reliable. The present study has tried to solve the earthquake hazard disaster risk mitigation in the valley by presenting some useful information, such as dominant period of shaking map, multiple resonance frequency area map, sediment distribution map etc.; however there are still many unsolved problems in this field and need to be explored. From this study, it is understood that Kathmandu Valley

is a natural laboratory in doing research in the field of earthquake disaster risk mitigation. Many research works such as, actual floor variation of the basement rock, response of the lacustrine sediment during actual ground shaking, basin edge effects, topographic effects, soil structure interaction effects are still not evaluated. Some of the studies/works which were not possible to pursue in this study are hence recommended as a future study.

The basement topography and sediment layer model of the lake deposit could be developed if a large number of boreholes with bedrock information were available. There were very few boreholes in the southern and eastern part of the valley. Hence more deep boreholes with adequate geotechnical/geophysical information are recommended to be constructed. There are no such responsible organizations to maintain borehole data base in valley, which makes collecting reliable data for research works in this field very difficult. It is recommended to establish an organization that has sufficient geotechnical expertise and help in providing reliable borehole database easily.

In this study, it is assumed that the main source of microtremors can be continuous vehicles movement in the cities of Kathmandu, Lalitpur and Bhaktapur, and the industries in and around the Kathmandu Valley. Heavy vehicles such as buses, heavy lorry, heavy trucks, and cars cross the basin edge, which is very near and on the bed rock of the Kathmandu Basin, and are continuously running in all direction of the valley. Another source may be the water movement in rivers in the valley and also the waver movement along the rivers which pass along the bedrock of the basin. Other possible sources of microtremor may also be due to the human activities (i.e., construction activities etc.) and the effects of winds on trees and buildings inside the valley and around the basin.

The predominant period based ground shaking map, multiple resonance frequency distribution map, sediment distribution map prepared in this study is a preliminary attempt for the evaluation of the earthquake hazard risk mitigation in the Kathmandu Valley. These maps have been prepared based on the microtremor observation in a grid of 1 km spacing. The results showed variation of ground response within the short distances of the valley. Hence, data measurement in a closer grid (i.e. 500 m interval) could give the better results and help to evaluate the characteristics of the basin precisely.

In order to evaluate the earthquake hazard risk in the valley, the response of the valley buildings during strong ground shaking is equally important. Due to time limitation, microtremor measurement inside the buildings is not possible in this study. The microtremor measurement in the buildings should be done so that appropriate relationship between story and period of the building can be developed. At the same time the

relationship between the ground period and the building period should be performed in order to study its possible damage during earthquakes.

Evaluation of earthquake hazard risk only by the interpretation of resonance frequency of ground may not be sufficient for the basin like Kathmandu Valley. Therefore, basin edge effects, topographic effects should be performed for precise evaluation.

Due to inadequate geotechnical/geophysical data, verification of the microtremor results with the numerical analysis was not possible. Therefore, the present results should be verified through numerical investigation too.

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