Earthquake is one of the major hazards for our country. As per the existing seismic zoning map of the country, more than 50 percent area falls under the active seismic zone. The three Indian metro cities, Mumbai, Kolkata, and New Delhi, amongst the 20 largest urban agglomerations of the world, fall in Seismic Zones III and IV. Hence they belong to places in moderate damage risk (MSK VII) in Zone III to high damage risk (MSK VIII) in Zone IV.

As per recent estimate, about 30 percent of population lives in cities. The increasing population has led to occupation of hazardous regions. For the safety and sustainability of these urban areas, it is necessary to implement long-range urban planning and risk assessment tools that rely on an accurate and multidisciplinary urban modeling. This necessitates development of tools for hazard assessment especially for earthquake and subsequently mapping the parameters necessary for long-range planning of the cities. The difficulty of this challenge is manifested in the spatially irregular patterns of damage that are typically observed after major earthquakes. The challenge of urban hazard mapping is to predict the ground motion effects related to various source, path and site characteristics, not just at a single site but at all the places with an acceptable level of reliability.

Development of seismic microzonation of major urban centers has, therefore, been recognized as a priority area of seismic mitigation programme in India. Considering this, Ministry of Earth Sciences (MoES), constituted a National Steering Committee in March 2008 to provide overall guidance to undertake seismic microzonation studies for identified cities and urban centers of the country. First level seismic microzonation has already been completed for Jabalpur, Sikkim, Guwahati, Delhi, Bangalore, Ahmedabad, Dehradun, Chennai and Chandigarh. As there are no detailed guidelines for adopting appropriate investigations, all microzonation endeavors are not homogeneous. To this effect, I hope that the present "Seismic Microzonation Handbook", an MoES initiative that encapsulates detailed underlying principles and case studies on various geological, geophysical and geotechnical studies will prove useful to the seismic and earthquake engineering community of the country.

Shailesh Nayak
PREFACE

Natural hazards turn into disasters when human societies & built environment are affected by them. The havoc caused by the recent devastating Bhuj earthquake of 26th January 2001; Muzzaffarabad earthquake of 8th October 2005 and the tsunami-genic Sumatra earthquake of 26th December 2004 in Indian Ocean is still fresh in our memory. Moreover, the urban seismic risk is rapidly increasing with population growth and the unplanned urban development into areas susceptible to earthquakes. Over the past 50 years there has been remarkable progress in earthquake engineering research in line with the seismological advancements in the know-how of the earthquake ground shaking and the earthquake induced vibration of structures. The building codes have also undergone tremendous changes over the years.

Most of the countries developed their respective macro-seismic zonation maps based on critical evaluations of historic evidences of earthquakes, published and unpublished literatures and systematic seismotectonic mapping. These small scale zonation maps provide a broad picture of the levels and distribution of earthquake hazard to which a particular country is subjected to. They also provide a scientific basis for earthquake resistant design codes. These small scale maps, however, are not much useful in the assessment of area-specific seismic risks, necessitating large scale seismic zoning for earthquake disaster mitigation and management. There are various ways to look at the stages of seismic microzoning. The broad approach involve three levels of Seismic Microzonation expressed as Grade l: General Zonation, Grade 2: Detailed Zonation and Grade 3: Rigorous Zonation as favored by the Technical Committee on Earthquake Geotechnical Engineering of the International Society of Soil Mechanics and Foundation Engineering. The recommendation essentially meant making a beginning with readily available relatively small scale maps and move on to higher levels of microzonation by obtaining added quality inputs to meet the demands of large scale mapping.

Over the years, most of the urban complexes in India have undergone a phenomenal growth for various socioeconomic reasons. Thus, the vulnerability of our cities being exposed to different hazards has also increased considerably, necessitating a proper hazard evaluation, particularly of the high-population-density urban centers lying in higher seismic zones. Consequently, a scientifically assessed hazard and risk scenario is an essence faced by the urban agglomerations, particularly those, where numerous elements are at risk. Seismic microzonation, thus, is a principal component of pre-disaster mitigation efforts.

In India, seismic hazard and risk assessment studies of urban complexes at micro level are now more than a decade old. In this period, different Institutions and individuals have accomplished some basic work, particularly with regards to the selection of appropriate scales of maps and
development of methodology suitable in the Indian context. The methodology, now broadly adopted, follows a multidisciplinary hierarchical approach, where the sequence of studies aims at generating parameters for source, travel path, ground characteristics and vulnerability, and draw inputs from the disciplines of geology, geophysics, seismology, geotechnical engineering, engineering seismology and anthropology. It is contemplated that seismic microzonation and risk assessment of identified urban centers, falling in Seismic Zones V, IV and III, would be completed in the near future.

Even though Seismic microzonation is the need of the hour in earthquake disaster mitigation and management, there are no detailed guidelines laid out for adopting appropriate testing and investigation programs. To this effect the Ministry of Earth Sciences initiative to present the “Seismic Microzonation Handbook” prepared by Prof. Sankar Kumar Nath embodying the detailed explanation of all the Geological, Geophysical and Geotechnical methodologies available for systematic seismic microzonation studies will go a long way in producing useful Seismic Hazard and Microzonation Maps that can be adopted in the existing building codes and construction bylaws. It is expected that the documents will guide successful seismic microzonation endeavors in the country.

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PREAMBLE

The earthquake, wherein a group of waves, radiate outwards from a suddenly disturbed zone in the Earth’s crust, or mantle, and cause the earth surface to shake and vibrate, continues to be the greatest catastrophe happening any time in the human history, in as much as the human race from time immemorial has suffered its tragic consequences and bravely recovered from its severest damages. During the last millennium, earthquakes have been responsible for the deaths of at least eight million people. The catastrophic earthquakes that need mention in the present context are the 1755 Portugal earthquake of magnitude M 9, 1812 New Madrid earthquake of magnitude M>7, 1906 San Francisco earthquake of magnitude M 7.9, 1923 Kanto earthquake of magnitude M 7.9, Santa Barbara 1925 and Large Beach earthquake of 1933 of magnitude M 6.3, 1960 Chili earthquake of M 9.5, 1964 Prince William Sound Alaska earthquake of M 8.3, 1964 Niigata earthquake of magnitude M 7.5, 1985 Mexico earthquake of magnitude M 7.9, 1988 Armenia earthquake of magnitude M 7, 1989 Loma Prieta earthquake of M 7.1, 1994 North-Ridge earthquake of M 6.7, 1999 Turkey earthquake of M 7.4, 1999 Taiwan earthquake of M 7.5, and many more around the globe, most recent being the magnitude 7.0 Haiti earthquake of January 12, 2010 with an estimated casualties of 230,000. Indian earthquake problem cannot be overemphasized. The Indian plate has diverse tectonic environments along the plate boundaries, comprising a continent – continent collision in the Himalaya-Tibetan plateau region, an oblique subduction in the Burmese arc and a nascent plate boundary in the northeastern Indian Ocean deformation zone. The Indian shield within the plate is a mosaic of cratonic blocks sutured by paleo-rift valley zones, and is known to be a Stable Continental Region (SCR). Bilham et al. (2001) divided the central Himalaya into 10 regions with lengths roughly corresponding to those of great Himalayan ruptures (~220 km). With a convergence rate of 20 mm/yr along the arc, six of these regions currently have a slip potential of a least 4 m – equivalent to the slip inferred for the 1934 Nepal-Bihar earthquake. This implies that each of these regions now stores the strain necessary for such an earthquake. Moreover, the historic record has no great earthquake throughout most of the Himalaya since 1700, suggesting that the slip potential may exceed 6 m in some places. A replication of the 1950 Assam earthquake, now along the more populous segment of the Himalaya would be a devastating event.

Unsuccessful earthquake prediction research had been going on for several decades. Exactly when and where earthquakes will occur, and how large they will grow after they are initiated, depends on a myriad of very delicate and immeasurable details of the physical state of the Earth over a large volume, not just in the immediate vicinity of the fault. Keeping in mind the disaster management protocol given in Figure I, even if the prediction of individual large earthquakes were a goal that could be realized, it would still be of questionable utility. One possible way to reduce the intensity of calamity,
if not to prevent it totally, is to classify terrenes in order of progressively changing intensity of seismicity highlighting the more vulnerable areas prone to damage by earthquakes. Most casualties during earthquakes are caused by the collapse of structures, both engineered and non-engineered. The last few decades have seen a phenomenal growth of infrastructure including high-rise buildings, water and sewage works and transportation networks, in urban areas. Recent developments in technologies including Remote Sensing, Geographic Information System (GIS), Global Positioning System (GPS) and other computational tools have provided motivation to focus on devising new methodologies for management of urban assets and infrastructure to mitigate damage during disasters, such as earthquakes. Structural mitigation measures are the key to making a significant impact towards earthquake safety in our country. People would be far better off living and working in buildings having proper lifeline facilities that were designed to withstand earthquakes when they did occur. A schematic flow of an overall picture of the disaster management processes forming the disaster management life cycle is shown in Figure I.

**Figure I:** A schematic flow diagram illustrating an overall picture of the disaster management processes forming the disaster management life cycle.

Natural disasters inflicted by earthquakes can neither be prevented nor is there any possibility in the near future for accurate and socially useful short-term prediction for an impending earthquake. Under these circumstances, it is pragmatic to formulate strategies to minimize the primary and secondary effects arising out of large earthquakes. The strategy can only be in place if the following scientific enquiries are properly addressed:

- On a macro scale, classification of different parts of the country into various seismic zones and its continuous updating based on already experienced and expected severity of earthquake
shaking depending on seismotectonic set up and history of seismicity (currently four zones are considered: II-V prepared under the aegis of Bureau of Indian Standards),

- Recognition and delineation of active faults in contemporary deformation zones for source characterization, and

- Zonation of major urban centers (where the seismic risk is high due to population density) into smaller microzones (depending on local geology/site response and other seismological and geotechnical implications).

The seismic zonation at regional level does not incorporate local and secondary effects induced by the earthquakes leading to its infeasibility in applications for landuse development and planning, hazard mitigation and management regulations, and structural engineering applications at the local/site specific level. It is necessary to overcome the limitations of regional zonation practices, especially in urban seismic provinces with burgeoning population and unplanned urbanization practices in vogue. Study of seismic hazard and preparation of microzonation maps will, therefore, provide an effective solution for urban planning. Seismic hazard and microzonation of urban centers enable characterizing potential seismic vulnerability and risk, the likely knowledge of which ensures safe designing of new structures or retrofitting the existing ones.

Microzonation is a process that involves incorporation of geologic, seismologic and geotechnical concerns into economically, sociologically and politically justifiable and defensible landuse planning for earthquake effects so that architects and engineers can site and design structures that will be less susceptible to damage during major earthquakes. Microzonation is the subdivision of a region into zones that have relatively similar exposure to various earthquake related effects. This exercise is similar to the macro level hazard evaluation but requires more rigorous inputs about the site specific geological conditions, ground responses to earthquake motions and their effect on the safety of the constructions taking into consideration the design aspects of the buildings, ground conditions which would enhance the earthquake effects like the liquefaction of soils, the ground water conditions and the static and dynamic characteristics of foundation or of stability of slopes in the hilly terrain. To be useful, microzonation should provide general guidelines for the types of new structures that are most suited to an area, and it should also provide information on the relative damage potential of the existing structures in a region. It follows, therefore, that if the principles of microzonation are appropriately and judiciously applied, they could be useful in establishing criteria for landuse planning and strategies for the formulation of a systematic and informed decision making process, for the siting and development of new communities in areas that are made hazardous by nature. Seismic microzonation must address the following aspects to obtain a reliable and meaningful Microzonation Hazard Map of any terrain/urban centre.

**GEOLOGICAL CONSIDERATION**

- Regional and local geological studies including the basement configuration.
- Seismotectonic Mapping inclusive of Active-, Neo- and Paleo-faults.
SEISMICITY ANALYSIS

- Homogenization of earthquake catalogue and magnitudes at the local and regional (300-350 km radius around the study area) level for seismic source characterization.

GEOTECHNICAL STUDIES

- Geotechnical site characterization based on Standard Penetration Test (SPT), Cone Penetration Test (CPT), Shear wave velocity using Multichannel Analysis of Surface Waves (MASW), Spectral Analysis of Surface Waves (SASW), and Cross-hole surveys.
- Strain dependent modulus and damping parameters characterizing the soil properties of any site under consideration.

SEISMIC HAZARD ANALYSIS

- Deterministic Hazard Analysis.
- Probabilistic Hazard Analysis.

Advocate both the procedures based on the availability of data in constrained conditions. Recurrence interval/Return period may be judiciously considered in the analysis.

SEISMOLOGICAL STUDIES

- Strong ground motion synthesis for Peak Ground Displacement / Peak Ground Velocity / Peak Ground Acceleration (PGD/PGV/PGA).
- Duration of Peak Ground Acceleration.
- Ground Motion Attenuation Relations both regional and site specific.

SITE EFFECTS FROM SEISMOLOGICAL DATA

- Predominant Frequencies from Ambiant Noise Survey (Nakamura Technique). Predominant frequencies from Nakamura technique may be used as a starting model wherever strong earthquake events are not available.

SITE EFFECTS FROM GEOTECHNICAL ANALYSIS

- Equivalent linear model using the soil column and input bed rock Ground Motion. Topography and basin effects may also be considered.
- Amplification factor deduced from geotechnical studies should be used judiciously in conjunction with earthquake data driven site amplification.

LIQUEFACTION STUDIES

- Liquefaction Susceptibility map using Seed and Idriss (1971) Technique may be used as an initial model.
• Liquefaction Factor of Safety mapping due to Maximum Credible Earthquakes.

OTHER PARAMETERS
• Landslide Hazard Zonation Map to be prepared according to the Bureau of Indian Standards’ Guidelines.
• In the regions of moderate to strong seismicity, seismic surveillance should be done through a strong motion network.

All the above aspects will be the vector layers on GIS Platform in 1:25,000 / 1:10,000 scale or larger and accordingly due weights and ranks should be assigned for each layer/parameter based on local conditions and finally integrated.

Eventhough Seismic Microzonation is a burning issue in earthquake disaster mitigation and management there are no detailed guidelines laid out for adopting appropriate testing and investigation programs leading to inaccurate seismic hazard estimation and faulty deliverables provided thereof. In order to work out the appropriate implementation strategies to be adopted for initiating microzonation studies for selected cities in the country during the 11th Plan period, the Ministry of Earth Sciences (MoES), Government of India, the nodal Agency and National Custodian of Seismological Studies in the country constituted a National Steering Committee (NSC) for Seismic Microzonation of selected cities in India vide Office Memorandum No. MoES/P.O.(Seismo)/2(04)/2007 dated 27th March, 2008 (Annexure – I) drawing Faculty, Scientists, Engineers from Indian Institute of Technology Kharagpur (IITKGP), Indian Institute of Technology Roorkee (IITR), Indian Institute of Science (IISc), Bangalore, India Meteorological Department (IMD), National Geophysical Research Institute (NGRI), Hyderabad, Geological Survey of India (GSI) and the Officials of the Ministry of Earth Sciences. Subsequently a meeting was convened on 12th January, 2010 at Mahasagar Bhawan, MoES Head Quarters in New Delhi to discuss the current methodologies and practices of microzonation. A Seismic Microzonation Framework was presented and a small working group was formed (Annexure – II) drawing Scientists and Engineers from IIT Kharagpur, IISc. Bangalore, NGRI, IMD and MoES to prepare a “Seismic Microzonation Handbook” and a “Seismic Microzonation Manual”. The present endeavor is a fall out of these initiations and a few brain-storming sessions on several issues in seismic hazard and microzonation practices.

The present “Seismic Microzonation Handbook” (the concise version separately presented as “Seismic Microzonation Manual”) provides detailed explanation of all the Geological, Geophysical and Geotechnical methodologies available for systematic seismic microzonation studies. Details of procedures alongwith underlying principles are enunciated in Chapters – 1 through to 8 with typical case studies given in the Appendices – I through to V in the Handbook and only the underlying principles and protocols and worksheet in the respective Manual intended to serve as references and guidelines. Chapter – 1 of the Handbook provides Introduction and Background Preparation Studies for the purpose of Seismic Microzonation with a few typical Seismicity Analyses Case Studies given in Appendix – I. A review-cum-preview on Seismic Zonation and Microzonation has been performed in Chapter – 2 of the Handbook with Typical Geotechnical and Geophysical Investigations
Prescribed for Seismic Microzonation of NCT, Delhi given in Appendix – II. Chapter – 3 addresses Site Effects and Site Response Analyses with typical case studies given in Appendix – III. Chapter – 4 provides detailed methodologies of Strong Ground Motion Synthesis & Seismic Hazard estimation with typical case studies presented in Appendix – IV. Chapter – 5 encapsulates all aspects of Geological, Geophysical and Geotechnical Site Characterization with a few case studies presented in Appendix – V. Chapter – 6 deals with Earthquake Induced Hazard - Soil Liquefaction while Chapter – 7 deals with Earthquake Induced Hazard – Landslides. Finally all the Hazard Maps are integrated for Seismic Microzonation on GIS Platform as explained with some typical Indian Case Studies in Chapter – 8. The Handbook concludes with some remarks made in Chapter – 9. It is expected that both these documents will have snowball effects on any seismic microzonation endeavor in the country.

The author is indebted to the Ministry of Earth Sciences, especially the Secretary, Dr. Shailesh Nayak and Advisor, Dr. B. K. Bansal for entrusting him with this mammoth job of preparing both the Handbook and the Manual for Seismic Microzonation. This formidable task would not have been accomplished without receiving critical suggestions from the members of both the National Steering Committee and the Working Group. The help and support received from Dr. Vandana Chaudhary, MoES and the Doctoral & Project Students Kiran Kumar Singh Thingbaijam, Biswaranjan Khatua, Soumya Kanti Maiti, Manik Dasadhikari and Sukdev Biswas are thankfully acknowledged.

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SEISMIC MICROZONATION HANDBOOK

Chapter

Chapter 1  Introduction & Background Studies
Chapter 2  Seismic Zonation & Microzonation
Chapter 3  Site Effects & Site Response Analysis
Chapter 4  Strong Ground Motion Synthesis and Seismic Hazard
Chapter 5  Site Characterization
Chapter 6  Earthquake Induced Hazard - Soil Liquefaction
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GEOSCIENCE DIVISION
MINISTRY OF EARTH SCIENCES
GOVERNMENT OF INDIA
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CHAPTER – 1

Introduction & Background Studies

1.1 INTRODUCTION

Earthquakes are major menace to the mankind killing thousands of people every year in different parts of the globe. An estimated average of 17,000 persons per year has been killed in the 20th century. Statistics taken from the period 1973-1997 (http://www.cred.be), organized by 5-year bins, exhibits that earthquakes are among the disasters with larger death impact (Figure 1.1), although occurrences of flood events are twice per year.

![Natural Disasters (Reported killed)](chart)

Figure 1.1: Comparison amongst different types of natural catastrophes (after Ansal, 2006).

The pie chart in Figure 1.2 shows the distribution of natural disasters faced in the 20th century in terms of casualties (leaving out the fatalities by drought and famine). All the earthquakes occurring globally have caused fatalities irrespective of whether they are in the Trans-Alpide belt or the Circum-Pacific Ring
of Fire. The 1995 Kobe earthquake with unprecedented losses of $100 billion may only be a harbinger of
even greater losses if an earthquake strikes Tokyo, Los Angeles, San Francisco or other large urban
centers in the world. The catastrophic earthquakes that need mention in the present context are the
1755 Portugal earthquake of magnitude M 9, 1812 New Madrid earthquake causing catastrophic
earthquake of magnitude M>7, 1906 San Francisco earthquake of magnitude M, 7.9, 1923 Kanto
earthquake of magnitude M 7.9, Santa Barbara 1925 and Large Beach earthquake of 1933 of magnitude
Niigata earthquake of magnitude M 7.5, 1985 Mexico earthquake of magnitude M 7.9, 1988 Armenia
earthquake of magnitude M 7, 1989 Loma Prieta earthquake of M, 7.1, 1994 North-Ridge earthquake of
M, 6.7, 1999 Turkey earthquake of M, 7.4, 1999 Taiwan earthquake of M, 7.5, 2001 Bhuj earthquake of
M, 7.7 and many more around the globe. More than 50% of the casualties are inflicted by earthquakes
and in that, the region of Asia and Pacific suffers the most and the deaths amount to more than 85% of
the total loss by earthquakes. Some of the recent earthquakes that caused large damages are the 2004
Sumatra earthquake, 2005 Kashmir earthquake and the most recent, 2008 Sichuan earthquake. The
damages and loss of life in the Sumatra earthquake were not directly related to ground motions;
nonetheless, it was still responsible for the tsunami that was unleashed. The 12th May, 2008 Sichuan
earthquake (M, 7.9) caused casualties due to the toppling or collapse of buildings. The recent 12th January,
2010 Haiti earthquake of M, 7.0 with a casualty of over 220,000 and the 27th February, 2010 Chilean
earthquake of M, 8.8 with a casualty of about 700 and other recently occurring earthquakes have shown
how vulnerable and unprepared we are in the event of a large earthquake. Even today after gaining
considerable knowledge on the phenomenon of earthquake, over the past few decades, man is still
struggling to contain the earthquake that continues to cause severe damage and loss of lives.

Figure 1.2: Pie chart showing the number of casualties during the 20th century from natural hazards (without
drought and famine) and the regions being affected (after Science Council of Japan, 1989).
Across the history, there are many instances where the damaging earthquakes left behind its not-so-pleasant signatures, sometimes wiping out township (Yungay village, Peru during the 1970 Peruvian earthquake; destruction of the city of Tangshan during the 1976 Tangshan earthquake) and change in landforms (sudden upheaval of the Shillong Plateau during the 1897 Shillong earthquake; 7 m offset of the San Andreas Fault in the 1906 San Francisco earthquake). The devastating 12th January 2010 strike slip earthquake along a 50 kilometer portion of the Enriquillo-Plantain Garden fault zone (EPGFZ) near Port-au-Prince, Haiti, is the most strongly resonating, figuratively and literally. The earthquake, of magnitude 7.0 or 7.1, killed more than 220,000 people. More than 50 aftershocks to date of magnitude 4.5 or greater are part of the process of readjustment of the Earth’s crust that could take 2-3 years to reach equilibrium (Calais, 2010). Though the large earthquakes cause immense destruction, it also provides a good opportunity for the seismologists to get more insight into the internal structure of the earth and have a better understanding on the mechanisms of the earthquakes.

The number of occurrences of large earthquakes has remained fairly constant but the loss of life and property during the recent earthquakes has increased manifolds because the population is constantly on the rise. In the developed countries, the new constructions have better earthquake resistance but, not so, for the other developing or underdeveloped countries. So, there is an increase in the casualties even for the same sized earthquakes depending upon the construction. The life and property of hundreds of millions of people are at risk from the devastating effects of an earthquake. The number of fatalities in an earthquake is associated with the vulnerability of local buildings, population density and the intensity of ground shaking. Hough and Bilham (2005) gave a simple relation discerning between the earthquake magnitude since 1900 and the number of deaths per earthquakes (gray zone) (Figure 1.3) but the consequences of large earthquakes depend on its proximity to urban areas, vulnerability of the dwelling inhabitants, time of the day and on the energy released.

The studies carried out after the strong earthquakes have provided basic knowledge and information of the earthquakes and also acted as catalyst for the assessment of seismic hazard and its mitigation. These post-earthquake surveys gave insights into the destructive pattern of the earthquakes caused due to three complex processes. The first process is the solid earth system that is made of (a) seismic source, (b) propagation of the seismic wave through a medium and (c) the local geology. The second process is the anthropogenic system that consists of the man-made structures like buildings, dams, bridges, tunnels etc. and the quality of the construction and the last factor is the socio-economic development of the settlement before it is struck by an earthquake (Panza et al., 2001). The amount of loss of life and damages to human property depends not only on the magnitude of the earthquake but also on the aforesaid three processes. Due to the heterogeneous nature of the earth’s crust, the seismic waves undergo multiple reflections, refractions and transformations along their path from the source to the site of observation. The changes are more prominent near the surface underlain by soil, where the geological and geotechnical properties of the soil layers play an important role in the amplification of the seismic energy.
Prediction of an earthquake has been a subject of controversy with divided opinions. Earthquake prediction gained momentum with the prediction of the Blue Mountain Lake (1971) earthquake and the success claimed at Haicheng (1975) but proved to be short lived. Sykes et al. (1999) give an account on the possibility and limitation of earthquake prediction. Workers like Geller (1997) and Main (1995) argue that short-term prediction with certainty is inherently difficult and that very high resolution is required for mitigation measures. Usually a time scale is involved that corresponds to long-term prediction considering an earthquake with a return period of 50, 100 or 500 years. Prediction of individual earthquake may not be possible but the long-term rates of earthquakes can be forecasted with considerable accuracy especially in the regions of high seismic activities, like the plate margins, such as Japan, Italy, Turkey, Mexico and California.

The vulnerability of modern society towards earthquake hazards is increasing with time. Although the occurrence of earthquakes is inevitable, the reduction of the social and economic setback during earthquakes can be achieved through a comprehensive assessment of seismic hazard and risk. This can be accomplished by creating public awareness and by upgrading or retrofitting of the existing buildings and engineering structures as well as to the upcoming structures.

1.2 EARTHQUAKES IN SOUTH ASIA

Earthquakes in South Asia account for nearly 28.6% of globally incurred earthquake related fatalities during the last 100 years. Going back in history to as far as the first century, numerous large earthquakes have been reported to have occurred in the region; largest being the recent 2004
Sumatra Earthquake of $M_w$ 9.1. To accomplish a foremost and indispensable groundwork for seismic hazard studies, it is essential to compile existing records of earthquake occurrences. This region can be broadly classified into eight subregions according to major tectonic settings, namely East Iran (EI), Northwest India Eurasia Convergence (NWIEC), Central Himalaya (CH), Tibetan Plateau (TP), Northeast India Eurasia Convergence (NEIEC), Andaman-Nicobar (AN), Mid-Plate (MP), and Northwest Carlsberg Ridge (NCR) as depicted in Figure 1.4. These delineations are anticipated to enable investigate spatial variability of empirical relations between different magnitude types as well as a first order appraisal of the compiled data at subregional levels. The entire region comprises of different tectonics; that of inter-plate collisions, plate divergences, and intra-plate (stable continental) settings. Information regarding earthquake related fatalities extracted from NGDC database covering a period of 1900-2008 is depicted in Figure 1.5. Total earthquake related-deaths reported in the study region during the period 1900-2008 estimates to around 55 thousand, notwithstanding that the last decade accounts for nearly 32 thousand deaths. The trend indicates rapidly increasing seismic vulnerability and risk, which is apparently connected to burgeoning population and expanding urbanization.

Figure 1.4: The subregions of South Asia on a seismicity map is depicted along with the seismicity, covering a period 1976-2008, derived from Global Centroid Moment Tensor (GCMT, http://www.globalcmt.org, last accessed 10th April, 2009) database. The topographic information is adapted from global digital elevation model: GTOPO30 of US geological survey. The depicted beach balls are obtained from the GCMT database by selecting largest event within a sliding spatial window of $2^\circ \times 2^\circ$ (after Nath et al., 2011c).
In order to understand the seismicity of South Asia data sources considered are the instrumental data (1900 onwards) primarily from International Seismological Center (ISC), Global Centroid Moment Tensor (GCMT) database, and several other publications. The uniform magnitude scaling in generic $M_w$ scale is achieved through connecting relationships between the different magnitude types (after Nath et al., 2010a).

### 1.3 EARTHQUAKES IN INDIA

The Indian subcontinent has suffered much due to earthquakes being one of the most earthquake prone regions of the world. The Indian landmass, covering an area of about 3.2 million sq. km, has three broad morphotectonic provinces, namely,

i) Himalaya and Tertiary mobile belt

ii) Indo-Gangetic foredeep

iii) Peninsular shield.

All of these areas are characterized by distinctive stratigraphic, tectonic and deep crustal features (Pande, 2005). The Himalaya marks the largest active continent collision zone that has witnessed four great earthquakes in a short time span of 53 years between 1897 and 1950. The Peninsular India is a mosaic of Archaean nucleus with peripheral Proterozoic mobile belts, Cretaceous volcanism and rift-drift (Pande, 2005) and Mesozoic passive coastal basins. This complex landmass with varied rheology

![Earthquake related fatalities in South Asia based on global significant earthquake database of National Geophysical Data Center](image1.png)
behaves differently in response of far-field stress to generate intraplate earthquakes (Dasgupta et al., 2000). Nearly 56% of the subcontinent is prone to different levels of seismic hazard. This is amply demonstrated by the fact that more than 650 earthquakes in excess of M 5 have been recorded in India in the last one century. The major seismic zones that have resulted as a consequence of the collision between the Indian and Eurasian plates are the Kirthar Sulaiman on the western part, the Himalayan on the northern part and the Arakan-Yoma mountain ranges on the eastern part of India.

The ongoing collision between the two plates has resulted in some of the great earthquakes with magnitude 8.0 and above along the margins of the plates. The collision of the Indian and the Eurasian plate is at a rate of 5.5 cm/year (Khattri and Wyss, 1978). DeMets et al. (1994) predicts a relative motion of India with respect to Eurasia being about 52 mm/year, through the Global plate motion model Nuvel-1A. Michel et al. (2001) gave the relative expected motion between India and Sundaland block on the Myanmar boundary to be about 4.5 cm/year. With the help of Global Positioning System (GPS) measurements, Bilham et al. (1997) showed the convergence of Indian Plate with the southern Tibet to be at a rate of 20 ± 3 mm/year. Several devastating earthquakes have occurred during the last two centuries that imposed heavy casualties and economic setback to the country and its surrounding areas.

The important historic earthquakes of India are 1737 Calcutta, 1862 Eastern Bengal and the Arakan coast, 1819 Kutch, 1885 Kashmir, 1885 Benfal, 1897 Shillong and the recent ones are 1905 Kangra, 1934 (North) Bihar-Nepal, 1935 Baluchistan, 1945 Makran, 1950 Northeast Assam earthquakes and the 1960 Delhi earthquakes. The 1991 October 20 Uttarkashi earthquake of magnitude 6.5 claimed more than 1500 human lives. The significant earthquakes that adversely affected India are listed in Table 1.1. The earthquake casualties in India have been very high compared to other parts of the world. The death toll inflicted by major earthquakes in India goes beyond 10,000 and can be witnessed by the earthquakes of the 1905 Kangra (~25,000 deaths), 1934 Bihar-Nepal (~15,700 deaths), 2001 Bhuj (~20,000 death) and 2005 Kashmir (~80,000 deaths). Most of the deaths and casualties in India can be credited, to a major extent, to poor housing construction, in terms of design as well as the quality of materials, and improper planning. The Vulnerability Atlas of India prepared by the Building Materials and Technology Promotion Council (BMTPC) in 1997, Ministry of Urban Development, denotes that about 59% of the land area of India is vulnerable to seismic damage. The atlas further estimates that 10.9% of the land is likely to be affected by the earthquakes with an intensity of Medvedev-Spoonheuer-Karnik (MSK) IX or more, 17.3% of the land to MSK VIII (similar to Latur) and 30.4% to MSK VII (similar to Jabalpur) (BMTPC, 1997). There are about 11 million houses vulnerable in seismic zone V; while for seismic zone IV it is alarmingly 50 million.

Nearly 80 million building units are in the risk of being damaged in the event of an earthquake. The task is not only to restore the vulnerable houses in order to minimize the loss of human life and property but also come up with a method of estimating quantitatively the seismic vulnerability of existing built-up environment.
<table>
<thead>
<tr>
<th>Date</th>
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<tbody>
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<td>Kumaon</td>
<td>$M_w$: 8.0</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td>1819, June 16</td>
<td>Kutch, Gujarat</td>
<td>$M_s$: 8.3</td>
<td>USGS</td>
</tr>
<tr>
<td>1833, August 26</td>
<td>Kathmandu</td>
<td>$M_s$: 7.7</td>
<td>Bilham (1995)</td>
</tr>
<tr>
<td>1869, January 10</td>
<td>Near Cachar, Assam</td>
<td>$M_w$: 7.4</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td>1885, May 30</td>
<td>Sopor, Jammu and Kashmir</td>
<td>$M_s$: 7.0</td>
<td>USGS</td>
</tr>
<tr>
<td>1897, June 12</td>
<td>Shillong Plateau, Meghalaya</td>
<td>$M_s$: 8.7</td>
<td>USGS</td>
</tr>
<tr>
<td>1905, April 04</td>
<td>Kangra, Himachal Pradesh</td>
<td>$M_s$: 8.0</td>
<td>ISC</td>
</tr>
<tr>
<td>1906, August 31</td>
<td>Sibsagar, Assam</td>
<td>$M_s$: 7.0</td>
<td>ISC</td>
</tr>
<tr>
<td>1918, July 08</td>
<td>Srimangal, Assam</td>
<td>$M_s$: 7.6</td>
<td>ISC</td>
</tr>
<tr>
<td>1930, July 02</td>
<td>Dhubri, Assam</td>
<td>$M_s$: 7.1</td>
<td>ISC</td>
</tr>
<tr>
<td>1931, January 27</td>
<td>Assam</td>
<td>$M_s$: 7.6</td>
<td>ISC</td>
</tr>
<tr>
<td>1934, January 15</td>
<td>Bihar–Nepal Border</td>
<td>$M_s$: 8.3</td>
<td>ISC</td>
</tr>
<tr>
<td>1941, June 26</td>
<td>Andaman Island</td>
<td>$M_s$: 8.1</td>
<td>ISC</td>
</tr>
<tr>
<td>1943, October 23</td>
<td>Assam</td>
<td>$M_s$: 7.2</td>
<td>ISC</td>
</tr>
<tr>
<td>1947, July 29</td>
<td>Upper Assam</td>
<td>$M_s$: 7.7</td>
<td>USGS</td>
</tr>
<tr>
<td>1950, August 15</td>
<td>Arunachal Pradesh</td>
<td>$M_s$: 8.6</td>
<td>ISC</td>
</tr>
<tr>
<td>1967, December 10</td>
<td>Konya, Maharashtra</td>
<td>$M_s$: 6.5</td>
<td>ISC</td>
</tr>
<tr>
<td>1975, January 19</td>
<td>Kinnaur, Himachal Pradesh</td>
<td>$M_s$: 6.8</td>
<td>USGS</td>
</tr>
<tr>
<td>1988, August 06</td>
<td>Manipur–Myanmar Border</td>
<td>$M_s$: 7.3</td>
<td>USGS</td>
</tr>
<tr>
<td>1988, August 21</td>
<td>Bihar–Nepal Border</td>
<td>$M_s$: 6.8</td>
<td>USGS</td>
</tr>
<tr>
<td>1991, October 19</td>
<td>Uttarkashi, Uttar Pradesh</td>
<td>$M_w$: 6.8</td>
<td>HCMT</td>
</tr>
<tr>
<td>1993, September 29</td>
<td>Latur–Osmanabad, Maharashtra</td>
<td>mb: 6.3</td>
<td>USGS</td>
</tr>
<tr>
<td>1997, May 21</td>
<td>Jabalpur, Madhya Pradesh</td>
<td>mb: 6.0</td>
<td>USGS</td>
</tr>
<tr>
<td>1999, March 28</td>
<td>Chamoli, Uttar Pradesh</td>
<td>$M_s$: 6.5</td>
<td>ISC</td>
</tr>
<tr>
<td>2001, January 26</td>
<td>Kachchh, Gujarat</td>
<td>$M_s$: 8.0</td>
<td>Harvard Catalog</td>
</tr>
<tr>
<td>2004, December 26</td>
<td>Sumatra</td>
<td>$M_w$: 9.1</td>
<td>USGS</td>
</tr>
<tr>
<td>2005, October 08</td>
<td>Kashmir</td>
<td>$M_w$: 7.6</td>
<td>USGS</td>
</tr>
</tbody>
</table>

[Source: United States Geological Survey (USGS), International Seismological Centre (ISC), Harvard CMT catalog (HCMT)]
1.4 SEISMIC GAP HYPOTHESIS

The sections of the plate boundary that have not ruptured in the past 100 years are indicated as Seismic gaps and are identified as locales which may have high potential for future great earthquakes. Khattri and Wyss (1978) identified three seismic gaps as given in Figure 1.6, viz.,

i) Kashmir gap – Section west of the 1905 Kangra Earthquake,

ii) Central gap – Section between the 1905 Kangra and the 1934 Bihar Earthquakes, and

iii) Assam gap – Section between the 1897 and 1950 Assam Earthquakes.

The Seismic gap hypothesis (Khattri, 1987) states that major earthquakes are likely to occur along the sections of the plate boundaries, which have ruptured in the past but have not experienced great earthquakes at least in the past thirty years. The potential of experiencing a great earthquake in such sections rises proportionally with the time elapsed since the last great earthquake as a consequence of slow plate motions due to which the strains develop over decades or centuries.

Bilham et al. (2001) divided the central Himalaya into 10 regions; with lengths roughly corresponding to those of great Himalayan ruptures (~220 km). With a convergence rate of 20 mm/yr along the arc, six of these regions currently have a slip potential of at least 4 m – equivalent to the slip inferred for the 1934 earthquake. This implies that each of these regions now stores the strain necessary for such an earthquake. Moreover, the historic record has no great earthquakes throughout most of the Himalaya.
since 1700, suggesting that the slip potential may exceed 6m in some places. A replication of the 1950 Assam earthquake, now along the more populous segment of the Himalaya would be a devastating event (Figure 1.7, Bilham et al., 2001).

**Figure 1.7:** This view of the Indo-Asian collision zone shows the estimated slip potential along the Himalaya and urban populations south of the Himalaya (after Bilham et al., 2001). The bars are not intended to indicate the locus of specific future great earthquakes but are simply spaced at equal 220 km intervals, the approximate rupture length of the 1934 and 1950 earthquakes. (Inset) This simplified cross section through the Himalaya indicates the transition between the locked, shallow portions of the fault that rupture in great earthquakes and the deeper zone where India slides beneath southern Tibet without earthquakes.

The source regions along the plate margins mainly include the Himalaya-Tibetan plateau and the Pamir-Hindukush regions in the north, the Burma-Andaman arc region, including the northeastern India, and the northeastern Indian Ocean region further south, comprising the nascent India-Australia plate boundary zone. The regions within the SCR of the Indian plate mainly include the Narmada-Son paleo-rift valley zone in central India, the Godavari graben, Koyna - the Reservoir Induced Seismic (RIS) zone, and Latur – the site of the most devastating SCR earthquake.
1.5 SEISMOTECTONICS OF THE INDIAN SUBCONTINENT

The main seismogenic zones in the Indian subcontinent are caused by the collision of the Indian plate with the Eurasian plate. The Indian subcontinent can be divided into three main subregions based on the geologic and tectonic regime. The first subregion is formed by the Sulaiman and Kirthar mountain ranges in the west, the great Himalayan arc extending from north-west to the Arakan-Yoma mountain ranges covering a distance of 2500 km. The second subregion is formed by the vast alluvial plains in the north along the basin of river Ganges and Sindhu (Indus). The hills, mountains, and costal plains form the third subregion of the territory. The seismotectonic map of India and its adjoining regions is given in Figure 1.8.

Figure 1.8: Seismotectonic map of the Indian Subcontinent (modified from GSI, 2000; Nath et al., 2011c).
1.5.1 Great Himalayan Arc

The first subregion of the Himalaya is considered to be the product of ongoing collision of the Indian and Eurasian plates. The Himalaya and its vicinity have been the sites of great earthquakes in the past and one of the most seismically active regions in the world. The Himalaya from the south to the north is divided into well known litho tectonic units (Sharma, 1998) namely, (1) the sub or outer Himalaya forms the low altitude hills (average elevation 350 m to 1050 m) limited between Main Frontal Thrust (MFT) in the South and Main Boundary Thrust (MBT) in the north, (2) the lesser Himalaya limited between MBT in the south and Main Central Thrust (MCT) in the north, (3) the great or higher Himalaya limited between MCT in the south and Tethyan detachment fault (TDF) in the north, (4) the Tethys Himalaya confined between TDF in the south and Indus-Tsangpo (ITS) in the north, and (5) the Trans Himalaya or Indus-Tsangpo suture zone consisting the obducted slices of the oceanic crust of the Neo–Tethys. In the Northwest the Sulaiman Kirthar ranges consist of numerous Mesozoic and Tertiary arcuate faults and imbricated structures (Krishnan, 1968). The major faults’ system in the area is characterized by Chaman fault active along its entire length in NE-SW directions (Verma, 1991). The 1935 Quetta earthquake of $M_w$ 7.6 is the major event that nucleated from the Chaman fault systems indicating a strike-slip focal mechanism (Gupta and Singh, 1980).

The spatial distribution and focal-mechanism study of mantle earthquakes in the Hindukush Pamir region carried out by Billington et al. (1977) suggested a model which invoked subduction in two opposite directions and the Indian plate subducted under the Eurasian plate to the west and the Eurasian plate subducted under the Indian plate in the eastward direction. In the western Himalayan foothills most of the seismic activity related to MBT is having regional names such as Punjab thrust in the Kashmir Himalaya, the Jawalamukhi thrust, the Nahan and Karol thrust in Himanchal Pradesh. There were some evidences of earthquake trends’ in the Himalayan range. In the Garhwal Himalaya telesismically determined epicenters and locally determined earthquakes by Khattri et al. (1984) suggests that most of the seismicity is located to the North of the MBT. The cluster of the earthquakes located by microseismic network of $31^\circ$N and $78^\circ$E is mostly confined to a depth of 6-8 km and with strike-slip focal mechanism in the EW direction. Deeper earthquakes have hypocenters in the range of 15-20 km and located between MBT and MCT have thrust mechanism on the northward dipping planes. The Northeast Indian region is characterized by high seismic activity. The seismotectonics of Northeast Indian region is considered to be more complicated (Nandy, 2001). This region presents a juxtaposition of two mobile belts, namely the E–W trending Himalaya due to collision between the Indian and the Eurasian plates, and the N–S trending Arakan Yoma belt due to the underthrusting of the Indian plate below the Myanmar plate (Dasgupta et al., 2003). The Mishmi region is traversed by Mishmi thrust, Lohit thrust, Po Chu, Tutin and Barne faults, Tsangpo and Tidding sutures. Because of the influence of seismic forces associated with both the eastern Himalayas and Indo Myanmar arc, the Mishmi region is considered as a special zone of seismic activity with block tectonics (Gansser, 1966 and Dutta, 1964). The Shillong plateau is characterized by Dauki, Dhansiri, Dhubri, Sylhet, Dudhnai and Kulsi faults, the N-E trending Barapani Shear Zone and the Mikir hills to the north. The Shillong plateau is considered as a part of the peninsular India, which moved east along the Dauki fault. The Brahmaputra River separates the plateau from the Assam valley. The Brahmaputra basin lies on the northern territory edged by the NE-SW trending Naga thrust on the south-eastern flank. The Kopili fault is etched on the middle of the
Brahmaputra basin followed by Bomdila lineament on the northwest. Disang thrust is seen southeast of Kopili fault and adjacent to Naga thrust. The 1100 km long Burmese arc is evolved due to the eastward subduction of the Indian lithosphere at the continental margin of the Asian plate. The Indo-Myanmar ranges, which are convex westward, act like a translational link between the Himalayan ranges and the Sunda arc to the south (Mukhopadhyay, 1984). The Indo Myanmar arc, sidelined by Patkoi–Naga–Manipur–Chin hills, has been associated with oblique subduction seismicity with fault plane solutions of deep focus earthquakes.

1.5.2 Indo-Gangetic Plains

The Indo-Gangetic plains constitute the vast alluvium plains of the Ganges (Ganga), the Indus (Sindhu), and the Brahmaputra and their tributaries, and separate the great Himalayan arc from the peninsular India. The Indo-Gangetic alluvium plain is the E-W trending tectonic basin located along the southern margin of the Himalayan fold belt. The central part being the Gangetic plain separated from the Indus plain by Delhi-Aravali ridge in the west, and in the east from the Brahmaputra plains by Rajmahal hills. The other major subsurface ridges along the Indo-Gangetic plains are the Faizabad ridge, the Munger-Sharsa ridge and the Golpara ridge (Kayal, 2008). The structural patterns and gravity observations suggest that these ridges are bounded by subsurface faults (Kayal, 2008). Major faults identified in the Indo-Gangetic basin are the Moradabad, Lucknow, west Patna, east Patna, Munger-Sharsa fault, Malda-Kishanganj fault etc. It is believed that most of the faults extend northward transversely to the Himalayan belt (Valdiya, 1976). Recent 1833, 1906, 1934, 1987 earthquakes suggest that the Gangetic plains are neotectonically active and so provides a possibility of potential earthquakes in the near future.

1.5.3 Peninsular India

The peninsular shield of India is considered as one of the largest Precambrian shield areas of the world, separated by the extensive Indo-Gangetic plains from the great Himalayan arc. Indian shield was described as the stable land mass associated with slight seismicity. Number of earthquakes, to name few of them, 2001 $M_w 7.7$ Bhuj, 1993 $M_w 6.3$ Killari, 1997 $M_w 5.7$ Jabalpur, and $M_w 6.5$ Koyna earthquakes have visited the region. The events in the region are called interplate earthquakes. The tectonic feature of the Indian Peninsular shield is considered to be made up of three major cratonic regimes namely, the Aravali, the Dharwar, and the Singhbhum Protocontinents; these are separated by rifts (Burke et al., 1978). The major prominent rifts are the Narmada Son Lineament and the Tapti Lineament together called SONATA (Son-Narmada-Tapti Lineament) zone separating the northern and the southern blocks of the shield. Kutch rift in the western margin of the Indian shield up to the commencement of the last two decades was described as the stable landmass associated with slight seismicity. Northwest striking faults under the Deccan traps are believed to exist in this region (Chandra, 1977).

1.5.4 Earthquakes in different Tectonofrabics

There has been a consistent sequence of earthquakes in the Indian subcontinent since ancient times and it is one of the most earthquake prone regions of the world and susceptible to seismic vulnerability.
because of its high population density and urbanization. Most of the seismicity is concentrated along the 2500 km long Himalayan arc from Sulaiman-Kirthar zone to Arakan-Yoma subduction zones marking as the plate boundary of the Indian and Eurasian plate. The ongoing collision of these two plates has generated some of the devastating earthquakes of the history above M 7.0 along the plate margin. In recent times the occurrences of earthquakes in stable intra-plate region of the Indian peninsula have caused much concern over the years. The earthquakes in the peninsular India are having relatively lower magnitudes but equally devastating to that of the Himalayan earthquakes. Based on different tectonic regimes the earthquakes have been broadly categorized in two classes: (1) the Himalayan earthquakes, and (2) the earthquakes in the Indian peninsular shield.

### 1.5.4.1 Himalayan Earthquakes

The northward movement of the Indian plate towards the Eurasian plate closed the Tethys Ocean and caused extensive deformation along the plate boundary giving rise to the spectacular mountain chain of the Himalayas. This ongoing convergence of the Indian plate with the Eurasian plate is considered responsible for the generation of some of the devastating earthquakes causing wide spread damage to the populated regions in the foothills of the Himalayas. The northwestern zone of the Himalaya has been rocked by major earthquakes namely 1934 Nepal Bihar earthquake of $M_w$ 8.4, 1905 Kangra Earthquake of $M_w$ 7.8, 2005 Kashmir earthquake of $M_w$ 7.6, 1991 Uttarkashi earthquake of $M_w$ 6.7, 1999 Chamoli earthquake of $M_w$ 6.5 and 1988 Bihar Nepal earthquake of $M_w$ 6.6. Northeast India is one of the most earthquake prone regions in the subcontinent along the Himalayan arc. It has been delineated to be in zone V of the seismic zonation map of India (BIS, 2002). The Global Seismic Hazard Assessment Programme also classifies the terrain in the zone of high seismic risk; moreover rapid urbanization during the last two decades in the region has increased the vulnerability towards potential seismic threats. The significant devastating earthquakes nucleated along the Himalayan arc are given in Table 1.2.

### Table 1.2: List of significant earthquakes in last two centuries along the Himalayan Arc

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Mag ($M_w$)</th>
<th>Lat (ºN)</th>
<th>Lon (ºE)</th>
<th>Fatalities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kumaun</td>
<td>September, 1803</td>
<td>7.7</td>
<td>30</td>
<td>78.0</td>
<td></td>
</tr>
<tr>
<td>Kashmir</td>
<td>May 30, 1885</td>
<td>7.5</td>
<td>34.1</td>
<td>74.6</td>
<td></td>
</tr>
<tr>
<td>Cachar</td>
<td>January 10, 1869</td>
<td>7.4</td>
<td>25.0</td>
<td>93.0</td>
<td></td>
</tr>
<tr>
<td>Shillong</td>
<td>June 12, 1897</td>
<td>8.1</td>
<td>26.0</td>
<td>91.0</td>
<td></td>
</tr>
<tr>
<td>Kangra</td>
<td>April 4, 1905</td>
<td>7.8</td>
<td>32.3</td>
<td>76.3</td>
<td>1900</td>
</tr>
<tr>
<td>Dharchula</td>
<td>August 28, 1916</td>
<td>7.1</td>
<td>30.0</td>
<td>81.0</td>
<td>1500</td>
</tr>
<tr>
<td>Srimangal</td>
<td>July 8, 1918</td>
<td>7.6</td>
<td>24.5</td>
<td>91.0</td>
<td></td>
</tr>
<tr>
<td>Assam</td>
<td>January 27, 1931</td>
<td>7.6</td>
<td>25.6</td>
<td>96.8</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Date</td>
<td>Mag ($M_w$)</td>
<td>Lat (°N)</td>
<td>Lon (°E)</td>
<td>Fatalities</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------------</td>
<td>-------------</td>
<td>----------</td>
<td>----------</td>
<td>------------</td>
</tr>
<tr>
<td>Bihar Nepal</td>
<td>January 15, 1934</td>
<td>8.1</td>
<td>26.5</td>
<td>86.5</td>
<td>10700</td>
</tr>
<tr>
<td>Assam</td>
<td>October 23, 1943</td>
<td>7.2</td>
<td>26.0</td>
<td>93.0</td>
<td></td>
</tr>
<tr>
<td>Assam</td>
<td>August 15, 1950</td>
<td>8.7</td>
<td>28.5</td>
<td>96.5</td>
<td></td>
</tr>
<tr>
<td>Kinnaur</td>
<td>January 19, 1975</td>
<td>6.1</td>
<td>32.38</td>
<td>78.49</td>
<td></td>
</tr>
<tr>
<td>Dharmsala</td>
<td>April 26, 1986</td>
<td>5.4</td>
<td>32.15</td>
<td>76.4</td>
<td></td>
</tr>
<tr>
<td>Manipur</td>
<td>August 6, 1988</td>
<td>7.2</td>
<td>25.13</td>
<td>95.14</td>
<td></td>
</tr>
<tr>
<td>Uttarkashi</td>
<td>October 19, 1991</td>
<td>6.8</td>
<td>30.77</td>
<td>78.79</td>
<td>2000</td>
</tr>
<tr>
<td>Chamoli</td>
<td>March 28, 1999</td>
<td>6.5</td>
<td>30.38</td>
<td>79.21</td>
<td></td>
</tr>
<tr>
<td>Kashmir</td>
<td>October 8, 2005</td>
<td>7.6</td>
<td>34.38</td>
<td>73.47</td>
<td></td>
</tr>
</tbody>
</table>

### 1.5.4.2 Earthquakes in the Peninsular India

Until lately the Indian peninsular shield was considered as the Stable Continental Regions (SCR). However in the last two decades the region has witnessed several moderate to large earthquakes causing widespread damage to human lives and life line facilities. The major earthquakes those visited the peninsular shield of India in the last two centuries are given in Table 1.3.

#### Table 1.3: List of significant earthquakes in past two centuries in Peninsular India

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Mag ($M_w$)</th>
<th>Lat (°N)</th>
<th>Lon (°E)</th>
<th>Fatalities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gujarat</td>
<td>June 18, 1819</td>
<td>7.8</td>
<td>23.6</td>
<td>68.6</td>
<td>2000</td>
</tr>
<tr>
<td>Coimbatore</td>
<td>February 7, 1990</td>
<td>5.7</td>
<td>10.8</td>
<td>76.8</td>
<td></td>
</tr>
<tr>
<td>Son Valley</td>
<td>June 2, 1927</td>
<td>6.4</td>
<td>23.5</td>
<td>90.0</td>
<td></td>
</tr>
<tr>
<td>Anjar</td>
<td>July 21, 1956</td>
<td>6.0</td>
<td>23.0</td>
<td>70.0</td>
<td></td>
</tr>
<tr>
<td>Ongole</td>
<td>March 27, 1967</td>
<td>5.1</td>
<td>15.6</td>
<td>80.1</td>
<td></td>
</tr>
<tr>
<td>Koyna</td>
<td>December 10, 1967</td>
<td>6.3</td>
<td>17.5</td>
<td>73.8</td>
<td>177</td>
</tr>
<tr>
<td>Bhadrachalam</td>
<td>April 13, 1969</td>
<td>5.7</td>
<td>17.8</td>
<td>80.6</td>
<td></td>
</tr>
<tr>
<td>Broach</td>
<td>March 23, 1970</td>
<td>5.4</td>
<td>21.6</td>
<td>72.9</td>
<td></td>
</tr>
<tr>
<td>Killari</td>
<td>September 29, 1993</td>
<td>6.2</td>
<td>18.1</td>
<td>76.5</td>
<td>9748</td>
</tr>
<tr>
<td>Jabalpur</td>
<td>May 21, 1997</td>
<td>5.8</td>
<td>23.0</td>
<td>80.2</td>
<td>38</td>
</tr>
<tr>
<td>Bhuj</td>
<td>January 26, 2001</td>
<td>7.7</td>
<td>23.6</td>
<td>70.2</td>
<td>20023</td>
</tr>
</tbody>
</table>
The criss-cross lineaments existing in the Peninsular India plays an important role as inherent markers of defect for the accumulation and transmission of stress. Several possible explanations have been given about the stress accumulation source in the zone (Rajendran et al., 1992), which includes: (i) stresses resulting from forces of collision between India and northern Asia, (ii) potential gradients caused by surface topography and erosion, (iii) loading of sediments deposited in the Arabian Sea and the Bay of Bengal, and (iv) the effect of flexural bulge in central India resulting from its collision with Tibet. Further, the investigations of Gupta (1992) and Rastogi (1992) raised the issue of overall increase in seismicity of peninsular India due to construction of large dams.

In addition there are several damaging earthquakes around the subcontinent which have wide spread damages, they are 1935 Quetta earthquake of M 7.6, 1945 Makran earthquake of $M_W$ 8.2, 1965 Hindukush (Afghanistan) earthquake M 7.8, 2004 Sumatra earthquake of $M_W$ 9.1, to name a few.

1.6 SEISMOLOGY AND SEISMIC HAZARDS: A REVIEW

Ever since the concept of modern day seismology evolved with the study of seismic source, the waves they produce and the properties of the media through which those travel, the seed for earthquake hazard assessment was sown. The most important change for seismology since 1960 was the change in the picture of the earth created by sea floor spreading and plate tectonics. The debate between the single couple and double couple mechanisms was settled in 1964 by the demonstration that only the later was compatible with a dislocation source and the WWSSN made possible much more reliable solutions for even smaller earthquakes with much less effort.

The earthquake engineering is a 20th century development, several workers in many countries being involved in its advancements. The earthquake hazard assessment coupled with the development in seismological instrumentations and methodologies paved way for a speedy development of earthquake engineering. Over the past 50 years there has been remarkable progress in earthquake engineering research in line with the seismological advancements in the know-how of the earthquake ground shaking and the earthquake induced vibration of structures. The building codes have also undergone tremendous changes over the years. The science of seismic hazard assessment also underwent major developments keeping pace with the instrumental advancements and the concept of seismic wave propagation, the scenario at the seismic bedrock and engineering bedrock level and the soil structure interaction.

The local geology plays an important role in changing the characteristics of seismic waves. In general, large amplitude and long durations of ground motion have been observed on thick water saturated unconsolidated materials. In the report of State Earthquake Investigation Commission on the California earthquake of April 18, 1906, Wood (1908) concluded that the amount of damage produced by the earthquake depended mainly upon the geological character of the ground. This paper had been the initial step towards quantitative estimates of the role played by the geology of the San Francisco Bay area in changing the characteristics of seismic waves.

Earthquake ground motion hazard estimation necessitates the models of seismic sources, earthquake recurrence frequency or prediction of maximum credible earthquake/scenario earthquake,
ground motion attenuations and ground motion occurrence probability at a site and strong motion seismometry in a region. The seismic sources are defined based on the interpretations of available geological, geophysical and seismological data with respect to earthquake mechanisms and source structures that are likely to be common within specific geographic regions. Seismic source delineation is generally premised on geosciences knowledge that relates to geological structures. However, when causative earthquake faults and structures are not known with certainty, seismic source interpretations are not unique (Thenhaus, 1983). Frankel (1995) and Frankel et al. (1996 and 2000) proposed methods of seismicity smoothing to avoid arbitrary discussions regarding the placement of area-source boundaries and Woo (1996) presented the fractal geometry of distributed seismicity as a self-organized critical-state process. Area seismic sources define regions of the Earth’s crust that are assumed to have uniform seismicity characteristics distinct from neighboring zones, and are exclusive of active faults that are individually defined. The central and eastern United States regions are often cited as leading examples of regions where seismic hazard is defined through the use of area seismic source zones (e.g., Thenhaus, 1983; Reiter, 1990; Coppersmith, 1991; Coppersmith et al., 1993). Analogous regions of the world that are areas located large distances away from the active plate boundary zones are referred to as stable continental interior region (Johnston et al., 1994).

There are two fundamental approaches to assess earthquake recurrence frequency of the defined seismic sources in the Probabilistic Seismic Hazard Analysis (PSHA). These are historical and geological frequency assessments. Historical frequency assessments are based on statistical analysis of the historical catalog of earthquakes that have occurred within a region. Geological frequency assessments are generally based either on a prehistoric record of earthquake occurrence on faults termed paleoseismicity, which is compiled through detailed field geologic investigations or on physical estimates of seismic moment either on individual faults or distributed throughout broad regions. The diligent efforts of Lee and Wang (1988), Downes (1995), Stucchi (1993), Usami (1981), Ambraseys and Finkel (1995), and Ambraseys and Melville (1982) to research historic earthquakes are significant contributions to PSHA.

Earthquake prediction can be categorized on the basis of terms scale involved in the prediction process like long-term, short-term and so on. Generally a time scale involved corresponds to long-term prediction referring to earthquakes with a return period of 500 years. The stresses are reduced below self-organized critical-states (SOC) at the time of a large earthquake in the area surrounding the rupture zones. The SOC processes have been explained by Bak et al. (1988) using the analogy of grains of sand being added slowly to a sand pile. After the great earthquake of San Francisco 1906, a broad neighboring area was quiet for greater magnitude events for about 70 years (Sykes and Jaume’, 1990). Shocks of that size were more numerous before the earthquake from 1883 to 1906. The Gumbel’s methods (Gumbel, 1958) and b-value (Gutenberg and Richter, 1954) are the two techniques generally used for the prediction of maximum credible earthquake. Yegulalp and Kuo (1974) made an attempt to improve their early results for statistical prediction of the occurrence of maximum magnitude earthquakes by including all of the seismicity data covering the period of 1904-1965. The statistical search for non-random statistical features of the seismicity of strong earthquakes was investigated by Kagan and Knopoff (1976) by working out the space-time-magnitude relationship among world-wide earthquakes of magnitude greater than 7. Their
statistical procedure involved calculation of the second-order moment of sequences of aftershocks and foreshocks. Vorobieva and Panza (1993) investigated that the strong earthquake is followed by related strong earthquakes, which occur near the epicenter of the strong earthquake with an origin time rather close to the origin time of the strong earthquake. They developed an algorithm for the prediction of occurrence of a related strong earthquake to the Italian territory. Makropoulos and Burton (1985a) applied two different methods to the earthquake catalogue for Greece to evaluate Greek seismic hazard in terms of magnitude, earthquake strain energy release and Gumbel’s third asymptotic distribution of extreme values. In the same year Makropoulos and Burton (1985b) extended their seismic hazard methods beyond the magnitude to the estimation of expectations of levels of peak ground acceleration exceedance relevant to design and planning criteria. Reasenberg and Jones (1989) presented a stochastic parametric model that allows the determination of probabilities for aftershocks and larger mainshocks during intervals following the mainshock. Tsapanos and Burton (1991) estimated seismic hazard for fifty of the most seismically active countries of the world using the technique of Gumbel’s third asymptotic distribution of extreme values. Ambraseys (1995) presented relations for peak horizontal and vertical acceleration generated by earthquakes in the European area. Atkinson and Boore (1995) predicted relations for ground motions for the eastern North American earthquakes of magnitude between 4 and 7.25 at distances from 10-500 km. Talebian (1996) proposed a Poisson-type model to calculate ground motion for seven major cities in Japan. Lapajne et al. (1996) prepared the preliminary seismic hazard map of the territory of Slovenia based on an expansion of the basic approach laid out by Cornell in 1968. Musson and Winter (1996) prepared the seismic hazard maps of the United Kingdom with 90% probability of non-exceedance in fifty years. Sousa and Oliveira (1996) performed seismic hazard assessment based on macroseismic data considering the influence of geologic conditions. Tumarkin and Archuleta (1997) presented an overview of their recent results on utilizing small earthquakes in the earthquake engineering practice. Site-specific ground motion time-histories of large earthquakes could be successfully simulated by them using recordings of small earthquakes, which are often referred to as ‘empirical Green functions’ in seismology. Kebede and Van Eck (1997) performed PSHA for the Horn of Africa based on seismotectonic regionalization. The effects of local site condition on the characteristics of ground shaking thereby influencing the building codes in NEHRP recommended provisions. The use of $b$-values derived from the Gutenberg-Richter relationship as a phenomenological base for developing probabilistic seismic hazard analysis has been questioned for years. Speidel and Mattson (1997) used cumulative distribution probability and showed that seismic magnitude-frequency data can be well described as one or more populations, each of which is normally distributed with respect to magnitude.

in the Gulf of Izmit and its relation to the earthquake of August 1999. Kagan and Jackson (2000) presented long-term and short-term forecast for magnitude 5.8 and large earthquakes. Their forecasts were expressed as rate density anywhere in the Earth. Molchan (1997) presented earthquake prediction as a decision-making problem. Vorobieva (1999) presented an algorithm for prediction of a subsequent earthquake based on the analysis of the aftershock sequence following a first shock and of local seismic activity preceding it. The control on an earthquake size examined in a heterogeneous cellular automation that includes stress concentrations was examined by Steacy and McCloskey (1998).

There are large numbers of attenuation relations that have been and can be used to develop engineering estimates of strong ground motion throughout the world. It is impossible to list all of them. Several attenuation relationships have been published since 1990. These attenuation relations were chosen to represent a selection of those commonly used to estimate acceleration response spectra for engineering evaluation and design. For practical purposes and for emphasis on engineering, the discussion is restricted to attenuation relations and other related engineering models used to incorporate hanging wall, foot wall, and source directivity effects that provide estimates of Peak Ground Acceleration (PGA) and Pseudo-Acceleration (PSA). This excludes models that predict PGA only or some measure of seismic intensity, such as Modified Mercalli Intensity (MMI), Japan Meteorological Agency (JMA) intensity, or Medvedev-Sponheuer Karnik (MSK) intensity.

All of the ground-motion and seismological parameters used in the attenuation relations are presented in terms of a consistent set of symbols and functional forms that may not conform to those used in the original reference. This is intended to aid in the presentation and use of the models. The constants in the relations were adjusted to give a consistent set of units for the predicted strong-motion parameters, in this case function of \( g \) for acceleration and cm/sec for velocity. Pseudo-velocity was converted to pseudo-acceleration if the original relation was given in terms of PSV. Logarithms are consistently defined in terms of the natural logarithm in the attenuation relations. All other equations were left in terms of the common logarithm if that is how they were defined originally.

Some of the model’s functional forms were modified in order to simplify or generalize them so that a single table of regression coefficients could be used. Although this means that the attenuation relations depart from their original mathematical forms, the modifications provide a consistent framework for understanding and applying these relations in an unambiguous manner. The strong motion parameters are defined as either the largest horizontal component or the geometric mean of the two horizontal components, with no attempt to convert between the two. If such a conversion is necessary, the largest horizontal component can be estimated approximately from the average horizontal component by multiplying by 1.15 (Campbell, 1981; Ansari and Razdan, 1995).

All the engineering models have limitations that result from the availability of recordings, the criteria used to select the recordings, the theoretical assumptions used to develop the methods, and the seismological parameters used to define the source, path and site effects. It is dangerous to assume that an engineering model can be extrapolated beyond the data, the theoretical assumptions, or the geographic region of applicability, and still provide a reliable estimate of ground motion. In fact, some of these models
come with specific caveats regarding their use in engineering. Two specific parameters, site class and faulting mechanism, generically designated S and F, pose a particular challenge, because each model defines these parameters somewhat differently.

A developing country like India, with a variety of building practices and social and economic structure needs to evolve its own strategies for seismic hazard evaluation. Occurrence of few damaging earthquakes during the last decade has pointed to our shortcoming in risk reduction programmes. A meaningful programme must incorporate appropriate building codes and also create public awareness. Several initiatives are now being taken at research and management levels. An update of these initiatives and steps to strengthen disaster mitigation programmes are discussed here. The ten-year period of the International Decade for Natural Disaster Reduction (IDNDR), came as a good opportunity for the country to look back at what had been done in the past, new initiatives taken during the decade, and plan ahead for reducing the impact of natural hazards on its people, settlements and economic development.

Among the studies that characterize various regions of the territory of India, and in particular the Himalayan region, one should first mention the research on the attenuation of macroseismic intensity, based on isoseismal maps and available macroseismic data (e.g., Kaila and Sarkar, 1977; Chandra, 1980; Gupta and Trifunac, 1988). Instrumental data for strong motion in the Himalayan region were not available before 1986, and this impeded the determination of attenuation relationships of peak ground motion based on local data. This is the reason why the attenuation relationships of other regions (e.g., eastern United States) were adopted for seismic hazard studies, in order to estimate the expected strong ground motion amplitudes (Khattri et al., 1984). In choosing a particular ‘analogue region’, it is required that both the regions have similar attenuation with distance of the Modified Mercalli Intensity (MMI). Gupta and Trifunac (1988) used the probabilistic relations for the attenuation of MMI with distance in India and applied scaling equations for strong-motion parameters in terms of site intensity for some other regions, which have a similar definition of intensity.

In the recent years seismic hazard map of the territory of India and adjacent areas has been prepared by Parvez et al. (2003) using a deterministic approach based on the computation of synthetic seismograms complete with all main phases. They used structural models, seismogenic zones, focal mechanisms and earthquake catalogue for the purpose. There are few probabilistic hazard maps available for the Indian subcontinent, however, the study of Parvez et al. (2003) aimed at producing a deterministic seismic hazard map for the Indian region using realistic strong ground motion modeling with the knowledge of the physical process of earthquake generation, the level of seismicity and wave propagation in anelastic media.

The technique for seismic zoning, developed by Costa et al. (1993) is applied by Parvez et al. (2003) to prepare a first-order deterministic seismic hazard map for India and the neighboring areas. This technique has already been used to produce deterministic seismic hazard maps for many areas of the world (e.g., Orozova et al., 1996; Panza et al., 1996 and 1999; Alvarez et al., 1999; Aoudia et al., 2000; Bus et al., 2000; Markusic et al., 2000; Radulian et al., 2000; Zivcic’ et al., 2000; El-Sayed et al., 2001), proving its predicting capabilities wherever new strong earthquakes have occurred, such as, for instance, in Italy for the Umbria-Marche sequence which started in 1997.
1.7 SEISMIC HAZARD TERMINOLOGY AND TYPES

1.7.1 Seismicity Analysis

Typical Seismicity analyses comprise of the followings:

- Development of homogenous and consistent earthquake catalogue
- Spatial Seismicity patterns for geodynamic characterization
- Maximum Earthquake Prognosis
- Temporal Seismicity Pattern Analysis for Earthquake Precursors.

The existing and accessible earthquake catalogues generally suffer from temporal data completeness, and different magnitude types used. The latter is solved by homogenization to moment magnitude ($M_w$) scale using conversion equations that connect different magnitude types. The analyses are carried out using different sub-catalogues for each period of data completeness. The seismicity patterns allow identification of stress regimes, and earthquake precursors in the geodynamic framework.

A quantitative approach to seismicity analysis can be carried out with the assessment of seismicity parameters: a- and b-value, and correlation fractal dimension, $D_c$. The first two parameters are obtained from FMD given by Gutenberg-Richter (GR) relation. The relation as given by Gutenberg and Richter (1944) is,

$$
\log_{10} N = a - bm,
$$

(1.1)

Where $N$ is the cumulative frequency of occurrence of magnitude $m$ in a given earthquake database. The intercept and slope, a- and b-value respectively signify the background seismicity level, and the magnitude size distribution.

The maximum earthquake, $m_{\text{max}}$, is defined as the one that is assessed as physically capable of occurring within a defined seismic regime in an underlying tectonic setup (Thenhaus and Campbell, 2003). The implementation of seismic hazard assessment towards hazard mitigation and disaster prevention, in general, involves expensive infrastructure creation with planned growth. Therefore, a realistic and pragmatic approach is crucial. The estimation of $m_{\text{max}}$ can be either probabilistic or deterministic. The deterministic approach involves employing empirical relationships between $m_{\text{max}}$ and various tectonic and fault parameters or between $m_{\text{max}}$ and the strain rate or alternatively the rate of seismic-moment release (Wells and Coppersmith, 1994; Anderson et al., 1996; Anderson and Luco, 1983; Papastamatiou, 1980). However, in most cases deterministic approach associates large uncertainties with the estimated $m_{\text{max}}$. In the probabilistic approach, $m_{\text{max}}$ is estimated from the seismic event catalogues and appropriate statistical estimation procedures. Some probabilistic approaches are linear extrapolation of GR relation (Gutenberg and Richter, 1944), application of extreme value theory (Nath et al., 2005), application of gamma distribution for the FMD (Main, 1996) and seismic moment distribution (Kagan, 2002). The extreme value approach has strong drawbacks as most of the historical data are not used and large uncertainties
are involved in the estimation (Knopoff and Kagan, 1977). Furthermore, incorporation of seismogenic perspectives of the earthquakes as criticality phenomena has been a recent development (Main, 1995).

1.7.1.1 The b-value and Correlation Fractal Dimension

In several studies, b-value has been found to vary both spatially and temporally, and has often been employed as one parameter approach for seismicity analysis. A low b-value implies that majority of earthquakes are of higher magnitude, and a high b-value implies that the majority of earthquakes are of lower magnitude. The variation of b-value has been seen to be inversely related to stress distribution (Mogi, 1962; Wesnousky et al., 1983; Schorlemmer et al., 2005). Furthermore, large material heterogeneities have been reported with higher b-values (Scholz, 1968). Aftershocks have been reported to have high b-values while foreshocks have been associated with low b-values (Suyehiro et al., 1964; Nuannin et al., 2005). In spatial analysis, low b-value can be inferred as growing stress regime unleashing larger magnitude earthquakes while high b-value indicates low stress buildup with continued stress release through numerous smaller magnitude earthquakes.

The estimation of b-value is generally performed by maximum-likelihood method (Aki, 1965; Bender, 1983; Utsu, 1999) given as,

\[
b = \frac{\log_{10}(e)}{m_{\text{mean}} - \left( m_t - \frac{\Delta m}{2} \right)} \tag{1.2}
\]

Where \( m_{\text{mean}} \) is the average magnitude and \( \Delta m \) is the magnitude bin size. Schorlemmer et al. (2003) suggests a bootstrap method (Chernick, 1999) to estimate the associated standard deviation, \( \delta b \), of b-value. The approach involves computation of b-value repeatedly for a number of times, each time employing different replacement events drawn from the associated catalogue wherein any event can be selected more than once. The error is, then, estimated as the standard deviation associated with the computed values.

Another commonly used seismicity parameter is the fractal correlation dimension of earthquakes, \( D_c \). The parameter is implied as a power law exponent relating distance and the number of pair of points of either epicenters or focal depths within the distance (Kagan, 2007). The temporal variations in the fractal dimension can enable assessment of seismic processes (Kagan and Knopoff, 1980). A lower \( D_c \) value indicates higher clustering while a higher \( D_c \) value implies uniform or random spatial distribution. \( D_c \) has been implicated as a quantifier of crustal deformation in time and space (Turcotte, 1986). However, the variation of fractal dimension in different seismic zones has also been attributed to the Geological diversity (Aviles et al., 1987). An estimation of \( D_c \) can be achieved through correlation integral method (Grassberger and Procaccia, 1983). The fractal correlation dimension is given as,

\[
D_c = \lim_{r \to 0} -\log_{10} \frac{C(N, r)}{\log_{10} (r)} \tag{1.3}
\]
where

\[ C(N, r) = \frac{2}{N(N-1)} \sum_{i=1}^{N} \sum_{j=i+1}^{N} H(r - |y_i - y_j|) \]  \hspace{1cm} (1.4)

\( N \) is the total number of points in the query. The coordinates of the location of points are given by \( y_i \) and \( y_j \). Essentially, \( C(N, r) \) accounts for the number of points that are at a distance \( \leq r \) with respect to the total number of pairs of all the points.

A plot of \( \log_{10}(C(N, r)) \) against \( \log_{10}(r) \) is used to estimate \( D_C \) as the slope of the curve in its linear bound for a specific range of \( r \). In case of an infinite two-dimensional distribution of epicenters, the plot is a straight line. But practically for larger values of \( r \) a state of saturation is attained thereby decreasing the gradient. For smaller values of \( r \), an increase in the gradient is induced indicating a state of depopulation (Narenberg and Essex, 1990). The specific range of \( r \) is, therefore, projected from the bounds of saturation and depopulation limits.

The two parameters, \( b \)-value and \( D_C \) have been employed together in several studies. The results of Hirata (1989), Henderson et al. (1992) and Barton et al. (1999) indicated that the two parameters tend to correlate negatively. Oncel and Wilson (2002) also reported the correlations to be generally negative, though both positive and negative correlations could be transiently observed. The correlations are seen to be dependent on the modes of failure within the active fault complex. Wyss et al. (2004) purposed that heterogeneity of seismogenic volumes leads to variations in \( D_C \) and \( b \)-values, and indicated a positive correlation between the two parameters dominant in regions of creeping and locked patches as well. Nonetheless, conclusively \( D_C \) and \( b \)-value together form a composite indicator of the underlying dynamics of the region. The spatial correlations can be interpreted from individual inferences of the variations of \( D_C \) and \( b \)-value. Low values indicate clustering of mainly large earthquakes identifying regions of high stress concentrations. High values suggest random occurrence of mainly smaller earthquakes indicating low stress buildup. Low \( D_C \) and high \( b \)-value indicates clustering of mainly smaller earthquakes that may be implicated with creeping parts of the fault zones. High \( D_C \) and low \( b \)-value indicates random occurrence of mostly larger earthquakes suggesting formation of asperities across the underlying faults (Oncel and Wyss, 2000). The spatial variations of both parameters in conjunction can be related to the tectonic dynamics across a region.

A Homogeneous Earthquake Catalogue of South Asia in M_w scale together with the Seismogenic Source Framework for the Indian Subcontinent is given in APPENDIX – I.

1.7.2 Seismic Bedrock and Engineering Bedrock

The seismic waves are generated due to the shear dislocation of the seismic source fault, suffer the influences of the materials through which they propagate and arrive at the ground surface (Figure 1.9). In general, the earthquakes of the larger magnitude and of the shorter distance from the seismic source to observation site induce the stronger ground motion. Besides the magnitude and the distance, the ground
characteristics must be taken into account, because they have a firm relation with earthquake disaster due to the amplification of seismic waves in the soft deposit.

In order to bring the quantitative ground motion estimation into practical stage, it is convenient to divide this complex phenomenon into the following two parts. Suppose that there is a buried interface in that we can approximate the up-going waves vertical and laterally homogeneous in practically useful extent. Then, one of the divided two parts is the source effect with the influence of propagation path that gives the input wave to the interface. Another is the influence, i.e. amplification in the shallower sedimentary layers. The above mentioned interface is called “Seismic Bedrock” in general. The following two conditions are required for this interface,

i) the interface has a practically useful lateral extent and the physical properties of the underlying stratum are not varied along this interface, and

ii) the strata deeper than this interface is much more homogeneous in comparison with the layers shallower than the interface.

As the shear wave velocity of the upper earth crust is as homogeneous as from 3000 to 3500 m/sec, the top interface of the upper earth crust having 3000 m/sec of the shear wave velocity is called “Seismic Bedrock”.

In the viewpoint of Earthquake Engineering, it has been proposed, based on the followings, to use the shallower interface of which underlying stratum has from 300 to 760 m/sec of the shear wave velocity. This interface is called “Engineering Bedrock”.

i) The most important problem for earthquake engineering is not whether the long period components are dominant in the ground or the short period components, but whether the pre-dominant period of the ground is close to the proper period of the structures constructed or planned on the mentioned ground. Therefore, “bedrock” cannot be absolutely defined, but have to be defined in different ways even in the same site depending on the structure.

ii) “Seismic Bedrock” of the cities in Japan is buried at the depth where very few strong motion records have been obtained. In contrast, there is much more strong motion records obtained at “Engineering Bedrock”. Therefore, it is the practically useful way to consider “bedrock” at this depth.

iii) Except for the plains around the Tokyo metropolitan area, Osaka City and Nagoya City, there is not any information about such deep structure as “Seismic Bedrock”. Therefore, it is quite difficult to estimate amplification characteristics of the ground shallower than “Seismic Bedrock”.

Both the definitions of “bedrock” are widely used in the field of quantitative strong motion estimation. In the textbook of Earthquake Engineering, “Bedrock” and sometimes “Seismic Bedrock” is used in the context where “Engineering Bedrock” should be used. In contrast, the textbook of Seismology uses
“Bedrock” where it should be called “Seismic Bedrock”. It is important to distinguish the definition of the terms where they are used, by the context or the glossary.

1.7.3 Seismic Zonation

Seismic zonation discussed in details in Chapter - 2 is the process by which areas are subdivided into seismic zones based on historical and predicted intensity of ground motion, which is expressed in terms of the peak horizontal ground acceleration or velocity. Seismic design requirements for structures are generally constant within a seismic zone. Seismic zonation of ground motion in Canada and the United States for building codes is done using probabilistic models consisting of seismogenic zones, and recurrence relations for earthquakes within those zones, based primarily on historical records and limited geologic evidence of seismicity. Ground motion at any location is estimated using basic probabilistic procedures developed by Cornell (1968) based on the seismogenic zone activities and local attenuation functions. Shaking intensity is usually characterized as peak horizontal ground acceleration or velocity for rock or firm ground at a given probability of exceedance or return period.

1.7.4 Seismic Hazard

Seismic hazards are defined as those earthquake-related geologic processes that have the potential to “produce adverse effects on human activities” whether the threat is to life, constructed works, or real estate. For example, seismically induced liquefaction is considered to be a seismic hazard, as are the associated ground displacements. However, fire caused by a gas main ruptured by liquefaction-induced ground displacement is not, since it is not strictly a geologic process. Ground motion, the definitive characteristic of earthquakes, is a seismic hazard that causes damage to structures directly, by vibration, or indirectly, by inducing other seismic hazards such as liquefaction and landsliding.

1.7.5 Liquefaction

Liquefaction refers to the loss of shearing resistance or the development of excessive strains resulting from transient or repeated disturbances of saturated cohesionless soils. Liquefaction-induced horizontal ground movements can range from minor oscillations during ground shaking with no permanent displacement, to small permanent displacements, to lateral spreading and flow slides. Flow slides and submarine slope failures, which are presumed to be caused by liquefaction, are included in this category for convenience of analysis and mapping. Liquefaction can also induce vertical ground movements (settlement) by rearrangement of loose soils into a denser configuration.

1.7.6 Landslide

This hazard includes all types of seismically induced landslides (e.g., soil slumps, rock falls, debris flows, rock avalanches), except for those occurring directly as a result of ground liquefaction.
1.7.7 Amplification of Ground Motion

The localized amplification of ground motion due to subsurface and/or topographic conditions at a site is considered to be a seismic hazard over and above the firm ground seismic motions of the area. Amplification of ground motion often occurs at sites overlain by thick, soft soil deposits, especially when the predominant period of the earthquake motions matches the predominant period of the ground.

The fundamental phenomenon responsible for the amplification of motion over soft sediments is the trapping of seismic waves due to the impedance contrast between sediments and the underlying bedrock. 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. While these resonance patterns are very simple in the case of 1D media (vertical resonance of body waves), they become more complex in the case of 2D and a fortiori 3D structures.

1.7.8 Tsunamis and Seiches

This hazard includes waves in oceans, lakes, rivers or other bodies of water that are generated by tectonic subsidence or uplift, seismically-induced landslides or other seismic hazards.

1.7.9 Tectonic Subsidence or Uplift

Tectonic subsidence or uplift is the sudden relative elevation change of a large area of the earth’s surface due to an earthquake. Historically, the impact of subsidence has been more severe than uplift, especially where accompanied by flooding.

1.7.10 Ground Rupture

This category includes rupturing of the ground surface and/or the near-surface relative ground displacements that can occur during a seismic event.

The site-specific seismic hazard assessment necessitates estimation of ground motion, liquefaction potential and assessment of geotechnical hazard for probable integration to generate local specific microzonation of earthquake hazard.

1.8 ASSESSMENT OF GROUND MOTION

Assessment of ground motion is the first step of earthquake damage assessment, and generally consists of two processes as shown in Figure 1.9. The first process is the assessment of ground motion on engineering bedrock and the second process is the assessment of site effects. When both the processes are calculated, ground motion at the surface can be evaluated. The first process is further divided into two processes in several assessments (e.g., Midorikawa and Kobayashi, 1979).
1.8.1 Ground Motion on Bedrock

On the first process as shown in Figure 1.9 ground motion on engineering bedrock is calculated for a hypothetical source. The method of this process is classified into two groups:

(a) The method that considers rupture pattern of fault (e.g., Midorikawa and Kobayashi, 1979).

(b) The attenuation formula for ground motion (e.g., Fukushima and Tanaka, 1990 and 1991).

In the first group, the hypothetical seismogenic fault is divided into subfaults, and the rupture pattern is considered. Therefore, the method of this group explains the phenomenon that the observed waveforms at the same distance from the source are different from each other according to rupture pattern. However, there is a problem that it has uncertainty on assumption of rupture direction for hypothetical earthquakes that have never occurred.

In the second group, the empirical relationship is derived from regression analysis for the observed data. This is an easy method to calculate ground motion and there are many attenuation formulas for each ground motion index such as peak acceleration, peak velocity, response spectrum and so on (e.g., Fukushima and Tanaka, 1990 and 1991). Recently the observed data near source is used for regression analysis and the attenuation formula is revised to be adapted to calculation of ground motion near the source region. There are some attenuation formulae that can consider rupture direction also. However
the attenuation formula cannot fully explain the distribution of strong ground motion because the actual earthquake has complex rupture process.

1.9 SEISMIC MICROZONATION & RISK

Seismic microzonation involves the division of a region into subregions in which different safeguards must be utilized to reduce, and / or prevent damage, loss of life and societal disruptions during future earthquakes. Seismologists can help to mitigate the effects of an earthquake by quickly determining source parameters and acquiring information about local geology & soil profile, topography, depth of water table, characteristics of strong ground motions and their interaction with man-made structures. The success of microzonation for large urban centers in California is mainly due to the efforts directed towards gathering adequate geological and seismic information, which assisted in developing products having higher confidence level. Additional efforts were directed to establish a regulatory system empowered to guide landuse planning decisions and to influence design and construction practices. Moreover, the public awareness of earthquake hazards from time to time played an important role in influencing governmental decision on seismic hazards.

Seismic microzonation and hazard mitigation programs necessitate focused strategic research leading to preparation of user-friendly maps describing the current state-of-the-art knowledge about site specific ground shaking with their duration, frequency content, peak ground velocity and acceleration, as well as energy attenuation as a function of earthquake magnitude, epicentral distance, and faulting mechanism.

Decisions to mitigate seismic risk require a logical and consistent approach to evaluating the effects of future earthquakes on people and structures. To achieve this logic and consistency, it helps to view the methodology as consisting of four steps, as shown in Figure 1.10. First is the PSHA, which gives a probabilistic description of earthquake characteristics such as ground motion amplitudes and fault displacement. Second is the estimation of earthquake damage to artificial and perhaps natural structures. Third is the translation of the seismic hazards into seismic risks by using the selected damage or loss functions. Fourth is the formal or informal analysis of earthquake mitigation decisions, wherein the options, uncertainties, costs, decision criteria, and risk aversion of the decision maker are incorporated into the decision logic area. The ultimate goal of both the seismic hazard and seismic risk analysis is to develop the elements that can be used to make rational decisions on seismic safety. The decision process should incorporate uncertainties in the earthquake process and ground-motion characteristics, uncertainties in the effects of earthquakes on people and structures, costs of seismic safety and potential losses, and aversion to risk. Thus study of seismic hazard and preparation of microzonation maps will provide an effective solution for urban planning. Seismic hazard and microzonation of cities enable to characterize potential seismic vulnerability/risk that needs to be taken into account while designing new structures or retrofitting existing ones.
Figure 1.10: Steps in the mitigation of earthquake risk (after McGuire, 2004).
CHAPTER 2

Seismic Zonation & Microzonation

2.1 INTRODUCTION

The Seismic Zonation map of India presents a large scale view. The site behavior during an earthquake due to local variations in soil type and geology cannot be interpreted from it. Earthquake disasters are inevitable but it is possible to minimize the aftermath of an earthquake if the zones that are more susceptible to undergo maximum ground motion are identified. Seismic microzonation seems to be an answer to the need for mitigation against the seismic hazards as it gives a realistic answer in terms of ground motion at a higher resolution.

The developments in Geographic Information System (GIS) and its versatility saw GIS playing an important role in the natural disaster programs. The ability of GIS to handle large volume of data, its flexibility, accuracy and its capability to upgrade the database and integrate large amount of data with less time has proved to be indispensable in the field of seismic microzonation.

Rapid urbanization is a factor that calls for construction of mega-structures, and the main reason for human loss and property damage is when due importance is not given for adequate preparation for possible hazard. Seismic microzonation is needed in urban areas or the upcoming urban areas falling under the high hazard zone. Some of the seismic microzonation works carried out in important Indian cities are Delhi (Parvez et al., 2002; Mohanty et al., 2007), Sikkim (Nath, 2005; Pal et al., 2008) and Jabalpur (Mishra, 2004), while the seismic ground motion in large urban areas in other parts of the world are carried in Bursa (Topal et al., 2003), Bucharest (Moldoveanu et al., 2004), Algiers (Harbi et al., 2004), Alexandria (El-Sayed et al., 2004), Beijing (Ding et al., 2004), Napoli (Nunziata, 2004), Santiago de Cuba (Alvarez et al., 2004), Sofia (Slavov et al., 2004) and Zagreb (Herak et al., 2004; Panza et al., 2004).

The microzonation maps can serve many purposes for the Urban Development Authorities. It can offer valuable information to the engineers for the seismic design of buildings and structures, assessment of seismic risk to the existing structures and constructions, management of landuse and also for the future construction of defense installation, heavy industry, and important structures like dams, nuclear power stations and other public utility services.
2.2 SEISMIC ZONATION STUDIES IN INDIA

Earthquakes are not evenly distributed in India. In places like the southern India, the occurrence of earthquake is irregular whereas the northeastern, the northern and the northwestern part of India are subjected to regular earthquakes as they mark the boundary of the Eurasian and the Indian Plates. The frequency of earthquake occurrence is relatively high in these regions. The epicentral plot (Figure I.11) shows a dense clustering of earthquakes of varying magnitudes for the northeastern part of India and it continues following the junction of the Eurasian and the Indian Plates to the northwestern India. The earthquake occurrence is sparse for the central and southern India, though some few devastating earthquakes have occurred in that region. As there is a wide variation in the intensity of ground motion and also in the frequency of occurrence of earthquakes, there was a need to divide India into broad zones in terms of expected ground motion to represent seismic hazards. The following provides an account of the initial zoning process in India and how it was reviewed, revised and modified periodically with the availability of more data and with a better understanding of the dynamics of the earthquake.

Tandon (1956) first came up with the seismic zoning map of India. It consists of three zones based on the broad concept of space–time earthquake statistics and the prevailing understanding of geotectonics. Geological Survey of India (GSI) first came up with the national seismic hazard map of India in 1935 (Krishna, 1992). Then in 1962, the Bureau of Indian Standards (BIS) (earlier, Indian Standards Institution) published the seismic zonation map of India (IS: 1893-1962) based on earthquake epicenters and the isoseismal map published by GSI in 1935, where India was divided into seven zones ranging from 0 (no damage) to VI (extensive damage) as shown in Figure 2.1a. The Deccan Plateau was considered more or less a safe zone and the hazard level was assigned '0' while a larger part of the northeastern India was assigned VI. The zoning was reviewed in 1966 (IS: 18931966) and additional information like geology and tectonic features were taken into account for modifying the zones (Figure 2.1b).

The zonation map underwent major revision in 1970 after the 1967 Koyna earthquake. The magnitude of the earthquake was 6.5 and occurred at the Deccan Plateau, which was previously assigned the zone '0' in the earlier maps. There arose the need to utilize both geological and geophysical data to review zoning. The major change was the removal of the zone '0' as it was not appropriate scientifically to consider a region with zero possibility of earthquake shaking. Another addition was the merging of the zones V and VI (IS: 1893-1970). The zoning was, therefore, reduced to five zones (Figure 2.1c) as compared to the earlier classification of seven zones. The upgraded map placed Koyna in around zone IV.

In 1984, the zonation map was further modified (IS: 1893-1984) where the regions of different seismogenic potential were identified on the basis of past earthquakes and the regional tectonic features. However, the map does not show seismic hazard at different locations and failed to assess the return periods of the required design seismic coefficients for the source zones. The 1993 Latur earthquake of magnitude 6.3 caused intensity IX damages, but prior to the earthquake, Latur was placed in seismic zone I, where no such magnitude of earthquake was expected. The Latur earthquake further led to the revision of the seismic zonation map of India. The map was revised again in 2002 with only four zones: II, III, IV and V (IS: 1893 (Part 1): 2002) (Figure 2.2).
Figure 2.1: Seismic zonation map of India prepared in (a) 1962 (IS: 1893-1962), (b) 1966 (IS: 1893-1966), and (c) 1970.

Figure 2.2: Seismic Zonation Map of India (IS 1893-2002).
Zones I and II were combined and the Peninsular India was modified too. The new zone placed the 1993 Latur earthquake in zone III. The areas falling under zone V is most vulnerable to earthquakes. Some of the country’s most devastating earthquakes occurred in zone V. The areas under this zone are the Andaman and Nicobar Islands, entire northeastern part of India, parts of northwestern Bihar, the Kangra Valley in Himachal Pradesh, the eastern part of Uttarakhand, the Rann of Kutch in Gujarat and the Srinagar area in Jammu and Kashmir. Two major metropolitan cities, with a high population density, i.e. Delhi, lie in zone IV, and Kolkata, at the boundary of zone III and IV of the zonation map. The four seismic zones of India are assigned PGA values ranging from 0.1 \( g \) to 0.4\( g \) (Tables 2.1 & 2.2).

**Table 2.1:** The seismic zones and the expected PGA (IS : 1893 (Part 1) : 2002)

<table>
<thead>
<tr>
<th>Zone</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>III</td>
<td>0.20</td>
</tr>
<tr>
<td>IV</td>
<td>0.25</td>
</tr>
<tr>
<td>V</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Besides the ground motion, the expected maximum intensity of shaking in each zone was also estimated (Table 2.2) based on the Comprehensive Intensity Scale (CIS-64) (IS: 1893 (Part 1): 2002).

**Table 2.2:** The seismic zones and the expected Intensity in Comprehensive Intensity Scale (CIS–64) (IS: 1893 (Part 1): 2002)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>II</td>
<td>VI and below</td>
</tr>
<tr>
<td>III</td>
<td>VII</td>
</tr>
<tr>
<td>IV</td>
<td>VII</td>
</tr>
<tr>
<td>V</td>
<td>IX and above</td>
</tr>
</tbody>
</table>

Besides the zoning map of India by the BIS, other non-official seismic hazard maps have been available in literatures reported by various workers (Auden, 1959; Mithal and Srivastava, 1959; Guha, 1962; Gaur and Chouhan, 1968; Kaila and Rao, 1979; Khattri et al., 1984; Parvez and Ram, 1997 and 1999; Bhatia et al., 1999) based on the statistical or probabilistic models. The changes in the zonation map of India with the occurrence of significant earthquakes are an indication that the zoning at a national level does not provide the solution of tackling seismic hazards.

### 2.3 GLOBAL SEISMIC HAZARD ASSESSMENT PROGRAM (GSHAP)

An International program under the banner of Global Seismic Hazard Assessment Program (GSHAP) was launched during 1992–1998. The program was set up by the International Lithosphere Program (ILP) with the support from the International Council of Scientific Unions (ICSU) and endorsed as a demonstration
program in the framework of the United Nations International Decade for Natural Disaster Reduction (UN/IDNDR). The objective of the program was to improve the global standards in seismic hazard assessment. Almost all the countries participated in the program (Giardini and Basham, 1993). The GSHAP was coordinated at the global level and employed at regional/local scale through various centers of each region. The regions covered by GSHAP are Central-North America, South America, Central-Northern Europe, Middle East (Iran), Northern Eurasia, Eastern Asia and South-West Pacific. The GSHAP test areas are at the Northern Andes, Caucasus, Adriatic Sea, Ibero-Maghreb, East African Rift and India-China-Tibet.

The GSHAP compiled maps for the hazard estimation by bringing together the regional map produced for different GSHAP regions and test areas. The map showed the hazard level in terms of PGA. The expected PGA is shown with a 10% probability of exceedance in 50 years, corresponding to a return period of 475 years. The GSHAP map shows approximately 70% of the earth’s continental land mass to have low hazard, 22% have moderate hazard and 6% have high hazard, while the remaining 2% have highest PGA with an average return period of 475 years.

A probabilistic seismic hazard map of India and the bordering region was prepared by Bhatia et al. (1999). They identified eighty six potential seismic source zones (Figure 2.3) on the basis of the major

![SEISMIC SOURCE ZONE MAP OF INDIAN SUBCONTINENT](image)

**Figure 2.3:** The eighty six potential seismic source zones considered for the probabilistic seismic hazard map of India (after Bhatia et al., 1999).
tectonic features and seismicity trends. The hazard level was given in terms of PGA, which was derived from the Joyner and Boore (1981) attenuation relation. The PGA was computed for a probability of exceedence of 10% in 50 years. The PGA varies from 0.05g to 0.5g. The hazard level is higher along the plate margins as seen from Figure 2.4. The expected PGA for the Arakan-Yoma ranges is the highest with the value ranging from 0.35g to 0.4g. The PGA for the Peninsular India is of the order of 0.05g to 0.1g. The hazard map was broadly classified into four zones with the hazard parameter as 0.1g, 0.2g, 0.3g and 0.4g. Table 2.3 shows PGA distribution for the four broadly classified zones.

![Seismic hazard map of India and adjoining regions for 10% probability of exceedance in 50 years (after Bhatia et al., 1999).](image)

**Figure 2.4:** Seismic hazard map of India and adjoining regions for 10% probability of exceedance in 50 years (after Bhatia et al., 1999).

**Table 2.3:** The PGA predicted by GSHAP Model (Bhatia et al., 1999)

<table>
<thead>
<tr>
<th>Zone</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>up to 0.1</td>
</tr>
<tr>
<td>II</td>
<td>0.10 – 0.20</td>
</tr>
<tr>
<td>III</td>
<td>0.20 – 0.30</td>
</tr>
<tr>
<td>IV</td>
<td>0.30 – 0.40</td>
</tr>
</tbody>
</table>
The PGA of some past earthquakes has been compared with the expected PGA by GSHAP (Giardini, 1999) and it has been found that the observed PGA is much higher than that estimated by the GSHAP. Table 2.4 shows the comparison between the expected GSHAP and the observed PGA of the four major earthquakes (Panza, 2007). It can be seen that the ground motion given at a global or regional scale is insufficient for the hazard estimation and the hazard assessment has to be addressed at a local scale.

Table 2.4: Comparison of the PGA estimated by GSHAP with the observed PGA (Panza, 2007)

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Expected PGA (g)</th>
<th>Observed PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1995, January 17</td>
<td>Kobe, Japan</td>
<td>0.40 – 0.48</td>
<td>0.70 – 0.80</td>
</tr>
<tr>
<td>2001, January 26</td>
<td>Gujurat, India</td>
<td>0.16 – 0.24</td>
<td>0.50 – 0.60</td>
</tr>
<tr>
<td>2003, May 21</td>
<td>Boumerdes, Algeria</td>
<td>0.08 – 0.16</td>
<td>0.30 – 0.40</td>
</tr>
<tr>
<td>2003, December 26</td>
<td>Bam, Iran</td>
<td>0.16 – 0.24</td>
<td>0.70 – 0.80</td>
</tr>
</tbody>
</table>

2.4 SEISMIC MICROZONATION : PRINCIPLES AND PROTOCOL

Socio-economic and environmental impacts are implicit in the scientific and technological aspects towards the mitigation and management of seismic hazards outlining well-defined objectives: (i) evaluation of earthquake and related hazards, (ii) standardization of a global implementation scheme to facilitate uniform action plans towards adapting urbanization regulations and codes for design and construction practices, and (iii) seismic vulnerability assessment, and risk prognosis to enable preventive measures against the hazard. The seismic hazard defines the potentially damaging ground shaking in terms of Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and/or Peak Ground Displacement (PGD). The quantitative assessment can be achieved by both the deterministic and probabilistic approaches; the former delivers absolute values, while the latter estimates the same in terms of probability of non-exceedance corresponding to a certain determined level at a site of interest. A quasi-deterministic or quasi-probabilistic approach employs a hybrid seismological, geological, geomorphological, and geotechnically guided framework wherein all the potential hazard attributing features are considered with relative rankings in a logic tree, fuzzy set, or hierarchical concept (Nath, 2004 and 2005).

Regional hazard zonations do not incorporate local and secondary effects induced by the earthquakes leading to its infeasibility in landuse development and planning, hazard mitigation and management, and structural engineering applications at site-specific terms. It is necessary to overcome these limitations, especially in the highly populated urban centers with unplanned urbanization practices in vogue. Seismic microzonation is, therefore, envisaged to subdivide a region into subregions in which different safeguards must be applied to reduce, and/or prevent damages, loss of life and societal disruptions; in case a large devastating earthquake strikes the region.

Typically a Seismic microzonation project consists of two distinct parts:

- A technical report, which can and must be flexible according to the risk
A part concerning regulations, which makes the technical report applicable to third parties.

In order to succeed, both the parts need a framework and specific criteria and tools. Professionals are responsible for the first part and the public authorities for the second one.

### 2.4.1 Global Trends

In the recent times, several seismic microzonation projects across the globe have been reported. These mainly include assessment of predominant frequency, effective shear wave velocity \( V_s^{30} \) and deterministic seismic scenarios while site specific probabilistic hazard analysis is slowly emerging for engineering decisions. These aspects can summarily be reviewed as,

1. Single attribute characterization based on ambient noise derived predominant frequency
2. Geotechnical modeling of response spectra and site response based on borehole data
3. Shear wave velocity measurement to achieve \( V_s^{30} \) zonation
4. Estimation of Site response and predominant frequency from earthquake recordings
5. Strong ground motion simulations for seismic scenario and deterministic hazard assessments, and
6. Probabilistic hazard analysis based on local specific relationships.

Fundamentally, an approach depends on data availability, the study region/terrain under consideration, and the choice of techniques as have been practiced around the globe:

#### Global Scenario Approaches

<table>
<thead>
<tr>
<th>Approach</th>
<th>Example/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimation of predominant frequency from ambient noise survey</td>
<td>Bam City (Motamed et al., 2007)</td>
</tr>
<tr>
<td></td>
<td>Napoli (Nunziata, 2004)</td>
</tr>
<tr>
<td></td>
<td>Greater Bangkok (Tuladhar et al., 2004)</td>
</tr>
<tr>
<td>Site response estimation from earthquake data</td>
<td>Alaska (Nath et al., 2002a)</td>
</tr>
<tr>
<td>1D geotechnical modeling of predominant frequencies, and peak amplifications</td>
<td>Bucharest city (Ehret et al., 2004), Yenisehir-Bursa (Topal et al., 2003)</td>
</tr>
<tr>
<td>Probabilistic seismic microzonation maps for various return periods</td>
<td>Tashkent City (Sokolov and Chernov, 2001)</td>
</tr>
<tr>
<td>Predominant frequency and site amplifications from earthquake records &amp; ambient noise survey</td>
<td>City of Thessaloniki (Lachet et al., 1996)</td>
</tr>
<tr>
<td>Site response &amp; response spectra from synthetic seismograms</td>
<td>Santiago de Cuba (Alvarez et al., 2004), Beijing (Ding et al., 2004), Tehran (Hamzehloo et al., 2007)</td>
</tr>
<tr>
<td>( V_s^{30} ) based site classifications by means of shear wave velocity measurements</td>
<td>Las Vegas Valley (Scott et al., 2006), Linares (Montalvo-Arrieta et al., 2005)</td>
</tr>
</tbody>
</table>


### 2.4.2 Microzonation works carried out in India

Seismic microzonation for important Indian cities that is vulnerable to the earthquake hazards have been carried out by various workers. The microzonation works carried out at important cities of India by different workers are Delhi (Parvez *et al.*, 2002; Singh *et al.*, 2002; Mukhopadhyay and Bormann, 2004; Mohanty *et al.*, 2007), Sikkim (Nath, 2005; Pal *et al.*, 2008), Jablapur (Mishra, 2004), Guwahati (Nath *et al.*, 2008a), Bangalore (Sitharam *et al.*, 2006; Sitharam and Anbazhagan, 2007), Dehradun (Mahajan *et al.*, 2007) and Mumbai (Raghukanth and Iyengar, 2006).

### Indian Scenarios

<table>
<thead>
<tr>
<th>Study region</th>
<th>Approaches</th>
</tr>
</thead>
</table>
| **Guwahati** | Estimation of site response and predominant frequency from strong motion data and geotechnical borehole modeling (Nath *et al.*, 2008a).  
Seismic scenario for deterministic hazard (Nath *et al.*, 2008b).  
Landslide hazard and soil liquefaction zonation, site classification, and Thematic integration on GIS (Nath *et al.*, 2008c). |
| **Sikkim** | Estimation of site response and predominant frequency from strong motion array (Nath *et al.*, 2005).  
Deterministic seismic scenario and thematic integration based on analytical hierarchical process on GIS (Nath, 2004; Pal *et al.*, 2008). |
| **Delhi** | Geotechnical analysis, stochastic simulation of strong ground motion and shear wave velocity measurements with MASW (Rao and Satyam, 2005).  
Synthetic seismogram (Parvez *et al.*, 2004).  
Predominant frequency estimation from ambient noise survey (Mukhopadhyay *et al.*, 2002).  
Site response estimation from earthquake recordings (Nath *et al.*, 2003).  
Probabilistic and deterministic assessment from geotechnical borehole analysis (Iyengar and Ghosh, 2004). |
| **Bangalore** | Geotechnical data analysis & deterministic seismic scenario (Sitharam and Anbazhagan, 2008).  
Soil liquefaction potential mapping (Anbazhagan and Sitharam, 2006).  
GIS integration based on AHP (Anbazhagan *et al.*, 2010; Nath and Thingbaijam, 2009). |
| **Dehradun** | Seismic response from geotechnical modeling on the shear wave velocity derived from MASW and geotechnical data (Mahajan *et al.*, 2007). |


<table>
<thead>
<tr>
<th>Study region</th>
<th>Approaches</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chennai</td>
<td>Site response estimation from geotechnical modeling of the MASW derived shear wave velocity data and borehole information (Boominathan et al., 2008).</td>
</tr>
<tr>
<td>Chandigarh</td>
<td>$V_s^{30}$ mapping and soil liquefaction assessment (Kandpal et al., 2009).</td>
</tr>
<tr>
<td>Jabalpur</td>
<td>Deterministic seismic hazard analysis based on scenario earthquake; site response studies and evaluation of liquefaction potential of the area. Liquefaction susceptibility and frequency dependent amplification of the strata, Site classification based on $V_s^{30}$ data (Mishra, 2004; Menon et al., 2008).</td>
</tr>
</tbody>
</table>

### 2.4.3 Definition of a Microzonation Problem

Seismic microzonation can be defined as the subdivision of a region that has relatively similar exposure to various earthquake related activities or the identification of individual areas having different potential for earthquake effects. The important places of concern for which seismic microzonation needs to be carried out is the urban or upcoming urban area that falls under the high seismic hazard zone. The damage pattern due to an earthquake depends largely on the local site condition and the social infrastructures of the region with the most important condition being the intensity of ground shaking at the time of the earthquakes. Contrasting seismic response is observed even within a short distance over small changes in the geology of the site. Moreover, designing and constructing all structures everywhere to withstand conceivable future earthquake is economically not viable.

Development of seismic microzonation of major urban centers has been recognized as a priority area of seismic mitigation programme in India. Twenty seven cities in India have a population of one million or more. These 27 cities contain 25.6% of the total urban population of the country. This is compounded by the fact that while geographically, 57.1% of the country area is under seismic zones III, IV and V of BIS Seismic Zonation Map of India (BIS, 2002), 66% of the population and 63% of the housing are located within these zones. The three Indian metro cities Mumbai, Kolkata, and New Delhi are listed amongst the 20 largest urban agglomerations of the world. Their existence in Seismic Zones III and IV places them in moderate damage risk (MSK VII) in zone III to high damage risk (MSK VIII) in zone IV.

Cities in India, which falls in seismic zone III, IV and V and having a population exceeding half a million, are recommended to have Level C microzonation maps. The list of these cities is given in Table 2.5 below:

**Table 2.5**

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>State</th>
<th>Name of the City</th>
<th>District</th>
<th>Seismic Zone</th>
<th>Zone Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Uttaranchal</td>
<td>Dehradun</td>
<td>Dehradun</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>2</td>
<td>Delhi</td>
<td>Delhi</td>
<td>New Delhi</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Sl. No.</td>
<td>State</td>
<td>Name of the City</td>
<td>District</td>
<td>Seismic Zone</td>
<td>Zone Factor</td>
</tr>
<tr>
<td>--------</td>
<td>---------------</td>
<td>------------------</td>
<td>----------</td>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>3</td>
<td>Gujarat</td>
<td>Jamnagar</td>
<td>Jamnagar</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>4</td>
<td>Gujarat</td>
<td>Rajkot</td>
<td>Rajkot</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>5</td>
<td>Gujarat</td>
<td>Bhavanagar</td>
<td>Bhavnagar</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>6</td>
<td>Gujarat</td>
<td>Surat</td>
<td>Surat</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>7</td>
<td>Maharashtra</td>
<td>Greater Mumbai</td>
<td>Mumbai</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>8</td>
<td>Maharashtra</td>
<td>Bhiwandi</td>
<td>Thane</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>9</td>
<td>Maharashtra</td>
<td>Nashik</td>
<td>Nashik</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>10</td>
<td>Maharashtra</td>
<td>Pune</td>
<td>Pune</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>11</td>
<td>Orissa</td>
<td>Bhubaneshwar</td>
<td>Khurda</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>12</td>
<td>Orissa</td>
<td>Cuttack</td>
<td>Cuttack</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>13</td>
<td>Tamil Nadu</td>
<td>Chennai</td>
<td>Chennai</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>14</td>
<td>Bihar</td>
<td>Patna</td>
<td>Patna</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>15</td>
<td>West Bengal</td>
<td>Asansol</td>
<td>Bardhaman</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>16</td>
<td>Assam</td>
<td>Guwahati</td>
<td>Kamrup</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>17</td>
<td>Gujarat</td>
<td>Vadodara</td>
<td>Vadodara</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>18</td>
<td>Gujarat</td>
<td>Ahmedabad</td>
<td>Ahmedabad</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>19</td>
<td>Tamil Nadu</td>
<td>Coimbatore</td>
<td>Coimbatore</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>20</td>
<td>Uttar Pradesh</td>
<td>Agra</td>
<td>Agra</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>21</td>
<td>Uttar Pradesh</td>
<td>Varanasi</td>
<td>Varanasi</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>22</td>
<td>Uttar Pradesh</td>
<td>Bareilly</td>
<td>Bareilly</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>23</td>
<td>Uttar Pradesh</td>
<td>Meerut</td>
<td>Meerut</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>24</td>
<td>Uttar Pradesh</td>
<td>Lucknow</td>
<td>Lucknow</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>25</td>
<td>Uttar Pradesh</td>
<td>Kanpur</td>
<td>Kanpur nagar</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>26</td>
<td>West Bengal</td>
<td>Kolkata</td>
<td>Kolkata</td>
<td>III</td>
<td>0.16</td>
</tr>
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### 2.4.4 Objectives of a typical Seismic Microzonation Project

A typical microzonation research aims at achieving and assimilating several of the following aspects in the evaluation of seismic hazard and building up levels of seismic microzonation of an urban center.

i) Establish the geological and geomorphologic units of the region and its surrounding areas, including the lithological characteristics and Seismic & Engineering Bedrock configurations at different locations.

ii) Identify and characterize major/minor faults, lineaments, and seismotectonic units that are seismically active.

iii) Characterize the seismic activities based on historical seismicity and recorded ground motion data. The attributes pertinent include location of potential sources, magnitude, intensity, focal mechanism and epicentral distances, etc. Selection of suitable attenuation laws.

iv) Evaluate spatial variation of shear wave velocity (i.e., average $V_s$ profiles) through geophysical survey, geotechnical borehole logging, and HVSR modeling to develop a database of shallow subsurface stratigraphy information.

v) Evaluate the spatial distribution of predominant frequency of the soil through Nakamura’s technique.

vi) Establish seismic (ground) response through theoretical and numerical modeling of wave propagation to identify amplification effects associated with near surface ground motions from alluvium deposits during an earthquake, during which critical facilities and infrastructure are expected to remain operational.

vii) Determine the ground motion parameters such as peak ground acceleration/velocity distribution at different locations. Determine the ground response spectrum, duration and the time history of earthquake inputs.

viii) Assess seismic stability and estimation of permanent ground deformation within typical structures.

ix) Site classification on the basis of shear wave velocity model and geotechnical assessments.

x) Assess the liquefaction potential from the detailed borehole geotechnical data and shear wave velocity profiles using suitable model studies. Identification of the threshold value, maximum epicentral distance and depth based on field and laboratory studies.
xi) Local specific deterministic and probabilistic seismic hazard analyses.

xii) Generate thematic hazard maps in terms of site classification, site response, predominant frequency, spectral accelerations, and peak ground accelerations on GIS platform.

xiii) Prepare inventory for the demographic, building typology, and landuse patterns.

xiv) Identify building hazards from resonance phenomenon on the basis of predominant frequency of the basin to demarcate zones of varying risk.

xv) Risk assessment of the urban center through scenario generations to identify damage levels and achieve a probabilistic account of the same.

The deliverables envisaged from a typical seismic microzonation project are:

- Regional seismotectonism
- Geological & Geomorphological mapping
- Seismic & Engineering Bedrock Configurations
- Predominant frequency distribution map
- Site classification map
- Site Amplifications map
- Site specific ground motion prediction equations
- Deterministic and Probabilistic hazard maps
- Socio-economic impact evaluations: Vulnerability and Risk assessment
- Provisions and regulations for landuse planning, structural developments, and recommendations for hazard mitigation actions.

The output maps of microzonation will include:

- Site characterization/ classification maps
- Spatial variation of PHA and Pseudo Spectral Acceleration (PSA) values at the bed rock level
- Spatial variation of PGA and PSA values at the surface level
- Liquefaction hazard maps
- Landslide and tsunami maps, and
- Comprehensive seismic hazard map prepared by combining all the above given maps (after giving proper weightage for each factors).

2.4.5 How to Implement the Parts

A seismic microzonation project extends from elementary to exhaustive data analyses involving innumerous technical aspects underlying the knowledge base with methodological diversity, but culminating ultimately into recommendations defining constraints on the national/global regulations with local ones. A framework that addresses the pertinent technical issues highlighted through the state-of-the-art practices and methodologies are:
• **Source Characterization**

Seismic sources are generally characterized by well defined physical parameters like corner frequency \( f_c \), seismic moment \( M_0 \), and stress drop \( \Delta \sigma \) derived directly from waveform data (Nath *et al.*, 2008b). Fault rupture attributes also include rupture dimensions, slip distributions, maximum slip, strike and dip. The source accounts for the shape of the amplitude spectra as well as the duration and directivity of ground motions.

The regional level study would involve compilation of fault based seismic sources within the purview of the city (both near- and far-sources) with associated geometry and, if possible, slip rates. Areal seismic sources can also be defined incase of the associated unknown complexity or distributed fault system. Development of homogenous moment magnitude based earthquake catalogues spanning over several hundred years (historical and instrumental periods inclusive) is an essential work component to quantify seismicity distribution, the magnitude frequency relation and the maximum magnitude prognosis.

• **Characterization of Wave Path Model**

The attenuation of seismic waves along the propagation path connecting the earthquake source and the recording site (observatory) is attributed to the degradation in the elastic properties such as shear and compressional moduli, and the scattering of seismic waves caused by heterogeneities in the earth’s interior. The strong motion attenuation feature as defined by Wu and Aki (1985) is an exponential function of the type \( Q_s = Q_o f^n \). Shear wave Quality factor \( Q_s \) is related to frequency dependent attenuation of spectral amplitudes, and scattering within the crustal structure. The geometric spreading function accounts for the variations in the seismic radiations, which evidently is inversely related to a factor of the distance from the source in the direction of wave propagation. The evaluation of path attributes is envisaged from the available waveform data and crustal models.

• **Site Characterization**

Site response estimation from geotechnical investigations involves combination of wave propagation theory with the material properties and the expected ground motion computed at the site of interest. Several algorithms are available for seismic response analysis for horizontally layered soil deposits in which recurrent and circular soil behavior can be simulated using linear equivalent model of a nonlinear phenomenon (Kramer, 1996). Techniques used widely to quantify site response from waveform data include the standard spectral ratio, generalized and horizontal-to-vertical spectral ratio (Field and Jacob, 1993).

The ground characterization of a terrain will include the following:

1. **Geo-hazard attributes**
   - Geomorphological (including relief) features
   - Landuse patterns
• Near surface lithological structure
• Tectonic and structural organization.

2. Geotechnical parameters

• Geo-hydrological properties
• Standard Penetration Test (SPT) N-values
• Factor of safety against soil liquefaction
• Density
• Shear Wave Velocity.

\[ V_s^{10}, V_s^{30}, V_s^{50} \text{ and } V_s^{\text{Avg}} \] distributions corresponding to average shear wave velocity from the ground surface to the depths of 10 m, 30 m, 50 m and the basement.

3. Site Response

• Response Curves or transfer functions
• Response spectra
• Interpreted thematic distribution of:
  - Predominant Frequency \((F_0)\), and
  - Peak Amplification \((A_0)\) at the predominant frequency.

At present there is some fundamental disagreement amongst researchers on the details of a procedure for assessing earthquake site response of several thousand meter deep alluvium deposits (Pyke, 1979).

• Ground motion simulation and hazard prediction

The quantitative assessment of seismic hazard necessitates measurement of peak ground motion parameter, such as peak ground acceleration from earthquake records. Paucity of strong ground motion data records under conditions similar to design earthquakes in terms of tectonic regime, earthquake size, local geology, and near fault conditions necessitates analytical or numerical approach for a realistic prognosis of the possible seismic effects. The strong ground motion modeling must accommodate: (i) the seismic wave radiation from a fault rupture, (ii) propagation through the crust, and (iii) modifications by the site conditions. Several techniques are available that differ in the theoretical considerations, data and computational requirements. Realistic results vis-à-vis complexity and computational as well as data requirement are the deciding factors for the applicability of a particular technique. Some of the existing techniques include: (i) stochastic approach (Boore and Atkinson, 1987; Beresnev and Atkinson, 1997; Motazedian and Atkinson, 2005), (ii) Green functions method (Bouchon and Aki, 1977), (iii) Empirical Green functions method (Hartzell, 1978; Irikura 1983), (iv) Finite Difference Method (Panza, 1985; Oprsal and Zahradnik, 2002), (v) Finite Element Method (Frankel, 1989), and (vi) Spectral Finite Element Method (Komatitsch and Tromp, 1999). The strong ground motion study would involve development of a database comprising of realistic synthetic strong ground motion data, site specific ground motion prediction equations, and the interpreted thematic distributions of:
- Peak Ground Acceleration (PGA), and
- Period-specific pseudo spectral acceleration (PSA) according to the targeted building typology.

2.5 MICROZONATION FRAMEWORK

As discussed till so far a seismic microzonation process is initiated with rudimentary assessments based on existing regional level hazard estimation, seismotectonic and macro-seismic studies. Several local specific hazard factors are, thereafter, evaluated and mapped on a Geographical Information System (GIS) platform with a uniform and consistent georeferencing scheme. From the structural viewpoint, GIS is very similar to conventional Data Base Management System (DBMS), except for the fact that the database of GIS is more sophisticated and has the capability to associate and manipulate enormous volume of spatially referenced interrelated data (Star et al., 1997; Foresman, 1997; Longley and Batty, 1997; Hanna and Culpepper, 1998). The components of a GIS are pictorially represented in Figure 2.5.

GIS stores spatial and aspatial data in two different databases. The geocoded spatial data defines an object that has an orientation and relationship with other objects in two (2D) or three-dimensional (3D) space. It is also known as topological data and stored in topological database. The data that describe the objects are known as attribute data stored in a relational database. GIS links the two databases by maintaining one-to-one relationship between records of object location in the topological database and records of the object attribute in the relational database by using end-user defined common identification index or code (Marble and Pequet, 1983; Korte, 1997; Hohl, 1998).

GIS uses three types of data to represent a map or any geo-referenced data, namely, point type, line type, and area or polygon type. It can work with both the vector and the raster geographic models. The vector model is generally used for describing the discrete features, while the raster model does it for the continuous features (Burrough, 1986; Davis, 1996; Burrough and McDonnell, 1998).

A GIS approach comprises of three distinct phases (Sander, 1998): (1) data acquisition, (2) data processing, and (3) data analysis. There are several ways of digitizing the map data for its incorporation in a GIS. The data can be directly digitized from the map using a digitizing table or the outline of the required classes may be traced on a transparent overlay by image processing software.

The GIS can be used for hazard management at different levels of development planning. At the national level, it can provide a general familiarity with the study area, giving planners a reference to the overall hazard situation. At the regional level, it can be used in hazard assessments for resource analysis and project identification. And at the local level, it can be used to formulate investment projects and
specific mitigation strategies. It is hard to conceive a micro-seismic programme without its very intimate coupling with GIS. It is the most efficient way to integrate a set of factor or state of nature maps into hazard maps, and thereafter, marry the hazard maps with the respective infrastructure map, to deliver risk assessments. It is proposed to be developed as a powerful decision support system for planning of human settlement in earthquake prone areas, as well as for earthquake disaster management.

A holistic Seismic Microzonation Framework is depicted in Figure 2.6 along with the seismological and geological attributions depicted in Figures 2.7 and 2.8 respectively. The scheme outlines compilation of information related to seismicity, identification of potential seismic source zones, development of seismicity models, and maximum earthquake prognosis in the regional level supported by earthquake catalogues and other relevant data such as fault database. The local level assessments involve mapping of surficial geological and geomorphological features supported by 2D/3D subsurface models, development of geotechnical database, and evaluation of different surficial soil attributes (e.g., density, rigidity, compressibility, damping, water content etc.), and the basement topography. The prevalent seismic characteristics, in terms of predominant frequency, site response, path and source attributes, are generally established through analytical and numerical treatment of the waveform, micro-tremor and geotechnical data, and thereupon, deterministic assessment is performed by means of strong ground motion simulations. Additional evaluations include that of relevant earthquake induced effects such as soil liquefaction and landslides. Eventually, a composite assessment is taken up of the geological, geotechnical, and seismological attributes to deliver seismic microzonation map in terms of a hazard index map.

Figure 2.6: Overall perspective of a seismic microzonation project: (a) data considerations and flow, and (b) framework outlining regional to local hazard assessment (modified after Nath and Thingbaijam, 2009).
Figure 2.7: Seismological aspects in the seismic microzonation model (after Nath and Thingbaijam, 2009).

Figure 2.8: Geological aspects in the seismic microzonation model (after Nath and Thingbaijam, 2009).
2.5.1 Microzonation Levels with Scale

A microzonation study is designed to map the different components of local seismic hazard at the scale of a study area, generally between 1:5,000 and 1:25,000 (Bard et al., 1995). These include:

- Active tectonic structures
- Modification of the seismic signal due to local geomorphological conditions
- Induced phenomena such as liquefaction, settlements, landslides etc.

The methods used to achieve these vary in complexity according to the degree of precision sought. Throughout the document, three levels of precision are adopted. These are A, B & C, and the range from the most rudimentary to the most refined level of precision.

**Level A**

The most rudimentary level of study is generally based on a compilation and interpretation of available data. It is the least expensive study and the scale of microzonation ranges from 1:100,000 to 1:50,000.

**Level B**

This intermediate level of study provides much more reliable results than those for Level A. Specific surveys are generally carried out during this study level, including drilling, trenching, geological sampling etc. The cost of this study remains reasonable, and the scale of microzonation generally ranges from 1:25,000 to 1:10,000.

**Level C**

This study is only carried out in areas where a very detailed level of mapping is required. Specific surveys and detailed calculations are involved. The cost of this study is high, but may be necessary in areas of high earthquake hazard risk. This scale of microzonation ranges from 1:10,000 to 1:5,000.

2.6 REGIONAL ASSESSMENTS

The regional level analysis encompasses the seismicity, seismic sources, and earthquake potential based on available historical and instrumental data covering hundreds of years, micro- and macro-seismicity, regional tectonics and neo-tectonics (faults/lineaments network), seismotectonics, geology, geo-hydrology, crustal structure, landslide incidents, observed soil liquefactions etc. (Nath et al., 2008b). Long-term earthquake catalogues are associated with two important issues namely data completeness and magnitude scale inhomogeneity, the former necessitating a temporal segregation of the data according to its completeness (Kijko, 2004) while in the latter, empirical relations connecting the different magnitude scales are used to homogenize the magnitude scale to moment magnitude, $M_w$, owing to its applicability to all magnitude ranges, faulting types, and hypocentral depths of the earthquakes. A large scale seismicity analysis is envisaged to examine spatial patterns (represented by b-value, and fractal correlation dimension of the epicenters or hypocenters), which along with the tectonic background delivers a broad seismic source zonation (e.g., Thingbaijam et al., 2008). Another alternative is seismicity smoothening of Woo (1996) that caters to spatial distribution of event activity rates.
The maximum, characteristics, or maximum credible earthquakes in a region can be estimated from maximum fault-rupture projected on faults/lineaments known or otherwise established from paleoseismic investigations (Wells and Coppersmith 1994; Hanks and Bakun 2002; Rajendran et al., 2004). Likewise, the estimation may also be derived from the slip deficiency based on the historical events and geodetic studies (Anderson et al., 1996). However, association of unknown fault complexities presents limitations in such deterministic assessments. A general technique employs homogenous earthquake catalogue to derive appropriate seismicity models (Kijko, 2004) to establish the annual recurrences. The faults likely to generate major earthquakes can be inferred from observed geological deformation episodes. Large scale micro-seismicity recordings can enable detecting active faults on the geological and geomorphologic signatures.

2.6.1 Summarily Regional Studies: Precursor to local ones

- Faults/lineaments database (historical and instrumental records)
- Geological features and crustal structure
- Earthquake Cataloging and seismicity analysis (e.g., Thingbaijam et al., 2008; Thingbaijam and Nath, 2008; Thingbaijam et al., 2009)
  - Data completeness
  - Scale homogeneity
- Development of up-to-date seismotectonic map
- Seismic source zonation
  - Based on the tectonic background and distribution of seismicity parameters (e.g., Thingbaijam et al., 2008, Nath and Thingbaijam, 2011b)
  - Seismicity smoothening (e.g., Woo, 1996)
  - The faults likely to generate major earthquakes can be inferred from observed geological deformation episodes. Large scale micro-seismicity recordings can enable detecting active faults on the geological and geomorphologic signatures.
- Earthquake potential: Assessment of maximum earthquakes
  - Maximum fault-rupture projected on fault/lineament known or otherwise established from paleoseismic investigations (Wells and Coppersmith, 1994; Hanks and Bakun, 2002; Rajendran et al., 2004)
  - Slip deficiency based on the historical events and geodetic studies (Anderson et al., 1996).
  - Appropriate seismicity models to establish the annual recurrences, and establish maximum earthquakes (Kijko, 2004).
- Earthquake recurrence study and prediction of seismicity parameters (a-value, b-value).
2.7 LOCAL SPECIFIC ASSESSMENTS

2.7.1 Geology and Geomorphology

The geology and geomorphology serves as a significant attribute towards seismic ground motion depiction at a site of interest (Aki, 1988; Panizza, 1991; Hartzell, 1992; Nath et al., 2002a; Nath et al., 2008b). In the geological and geomorphological studies, the near-surface signatures pertaining to the recent sedimentary deposits - alluvium, flood plains, cliffs, slope aspects etc. can be complemented by borehole litholog, exploratory drill holes, surface elevation model, land-cover, and basement topography derived from vertical electrical resistivity soundings and other geophysical investigations.

- The geology and geomorphology serves as a significant attribute towards seismic ground motion depiction at a site of interest (Aki, 1988; Panizza, 1991; Hartzell, 1992; Nath et al., 2002a; Nath et al., 2008b).
- The assessments include:
  - The near-surface signatures pertaining to the recent sedimentary deposits - alluvium, flood plains, cliffs, slope aspects etc.
  - Complementary studies using borehole litholog, exploratory drill holes, surface elevation model, and land-cover data
  - Development of basement topography from vertical electrical resistivity soundings and other geophysical investigations
  - Higher resolution mapping from analysis of satellite imageries.

2.7.2 Shear Wave Velocity

The shear wave velocity profile of soil column is used for site response modeling as well as site classification adhering to National Earthquake Hazard Reduction Program (NEHRP, Building Seismic Safety Council 2001) and Uniform Building Code (UBC) (ICBO, 1994) terminology. Several techniques are available to obtain subsurface shear wave velocity profiles that include (i) using empirical equations between SPT-N values obtained from geotechnical borelog, and the average shear wave velocity (e.g., Fumal and Tinsley, 1985; Imai and Tonouchi, 1982), (ii) Multi-channel Analysis of Surface Waves (MASW) (Park et al., 1999), (iii) Spectral Analysis of Surface Waves (SASW) (Stokoe et al., 1994), and (iv) Cone Penetration Test (CPT). These techniques are often employed in combination to authenticate and maintain consistency within specified uncertainty in the interpretations of different observations. The spatial mapping is generally done either with the average values for the sediment depth, or 30 m from the surface of the soil column. The latter is widely used for site classification.

2.7.3 Seismicity of the Study Area and Estimation of its Ground Motion Parameters

The first step in seismic microzonation is to assess the characteristics of ground motion at the bedrock level considering the seismic sources and the path effects. Generally, the seismic study area extending up to 300 km from the boundary of the study area should be identified. However, if there are any seismic sources which can create a very large earthquake (mega earthquake), this distance should be increased.
The details of seismic events and seismic sources need to be identified from the region of interest. The initial estimates of ground motion parameters (PGA, PGV or PGD) and/or bed rock motions should be obtained at the bed rock level using deterministic and probabilistic methods.

2.7.4 Site Response

The site amplification of ground motion is primarily attributed to either the geomorphological features that produce scattering, focusing, or defocusing of incident energy (topographic effect) or thick alluvium-filled terrain that causes reverberations due to trapped energy (basin effect). Techniques used widely to quantify site response in terms of site amplification factors include (i) Horizontal-to-Vertical Spectral Ratio (HVSR), (ii) Generalized Inversion approach, (iii) Standard Spectral Ratio method with reference to a rock site, (iv) coda wave technique, and (v) geotechnical evaluation of soil transfer function using available software viz. SHAKE, SHAKE 2000, WESHAKE, ShakeEdit etc. (Nath et al., 2002b; Kramer 1996; Kato et al., 1995; Lermo and Chavez-Garcia, 1993; Hartzell, 1992). The site amplification factor estimated using the flow chart shown in Figure 2.9 from geotechnical data analysis can complement the ones assessed from strong ground motion waveform. An example of calibration of HVSR site response with those estimated through geotechnical analysis is depicted in Figure 2.10. Application of multiple techniques allows resolving ambiguity associated with the estimation. Validations can be achieved through macro-seismic intensity distribution for previous/historical earthquakes (e.g., Hough and Bilham, 2008).

![Figure 2.9: Site Response Study Framework.](image-url)
2.7.5 Predominant Frequency

The predominant frequency corresponds to the maximum amplitudes of the ground motion in frequency domain. The proximity of predominant frequency of soil layers and natural frequency of the buildings indicate higher vulnerability of the built-environment owing to resonance effects (Navarro and Oliveira, 2006).

The assessment of predominant frequency distribution in a particular terrain can be performed through waveform data analysis that may be strong ground motion (acceleration), broadband (velocity) or ambient noise (microtremors). The Horizontal-to-Vertical Spectral Ratio (HVSR) analysis on ambient noise measurements proposed by Nakamura (1989) offers advantage of easy data acquisition, besides being inexpensive and reliable so far as the basin response effect is considered. Local specific relations between average shear wave velocity and predominant frequency with basement depth can be established to enable durability in the spatial extrapolation/interpolation of these parameters (e.g., Ibs-Von Seht and Wohlenberg, 1999; Parolai et al., 2002). Figure 2.11 depicts such relations observed in the Guwahati city.

Liquefaction, landslides and subsidence are other topics connected to soil performance with great importance essentially for stabilization of foundations and performance of lifelines. The level of water content in the soil stratum is of major importance and, consequently, the epoch of the year when the event takes place does have a great influence on the potential for liquefaction, landslide etc.
Figure 2.11: Local specific relations between averages shear wave velocity, and predominant frequency with the basement depth (after Nath and Thingbaijam, 2010).

2.7.6 Site Characterization

Site class specifications are employed to characterize generic subsurface conditions towards seismic response of the soil. A site characterization framework is shown in Figure 2.12. The average shear wave velocity of the upper soil column is widely used for the purpose; the ranges of values for each site class correspond to a specific class of soil. NEHRP (Building Seismic Safety Council, 2001 discussed in details in Chapter–5) specifies site class specifications (A-F) on the shear wave velocity averaged for the upper 30 metres of the soil column $V_s^{30}$ with the exceptions of site class E and F. The former is identified with the velocity less than 180 m/s or with more than 3 m of soft clay and Plastic index greater than 20, water content more than 40%, and corresponding average undrained shear strength less than 25 kPa. Site class F requires site-specific evaluations to identify any of four categories: (1) soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils, (2) peats and/or highly organic clays (soil thickness greater than 3 m) of peat and/or highly organic clay, (3) very high plasticity clays (soil thickness greater than 8 m with plasticity index greater than 75), and (4) very thick soft/medium stiff clays (soil thickness greater than 36 m). Universal Building Code (UBC) provisions (discussed in details in Chapter–5) also employ average shear wave velocities to describe soil coefficients (ICBO, 1994). Geological attributes are often connected to shear wave velocity in view of limited number of observation sites (e.g., Wills and Silva, 1998; Wills and Clahan, 2006). On the basis of the overlapping ranges of $V_s^{30}$ accorded to different geological units, Wills et al., 2000 introduced intermediate classes namely BC, CD, and DE corresponding to an average $V_s^{30}$ of 760 m/s, 360 m/s, and 180 m/s, respectively.
2.8 INDUCED HAZARD ASSESSMENTS

The secondary phenomena associated with ground shaking include ground spreading, slumping, soil liquefaction, landslide, rockfalls etc. that contributes to overall seismic risk.

2.8.1 Liquefaction Susceptibility Mapping

Soil liquefaction is triggered when loose or soft saturated unconsolidated soil transforms from a solid state to a viscous state due to increase in pore water pressure and consequent decrease of effective stress. It tends to reoccur at the same sites during successive earthquakes where geological and hydro-geological conditions remain fairly stable. Such sites need to be identified and mapped as a part of rudimentary study supported by a geological map providing detailed Quaternary (recent) deposits at high
Seismic Zonation & Microzonation

Recent sediments especially fluvial and Aeolian deposits, water table information along with physical soil characteristics such as type of soil, degree of water saturation, grain size, and plasticity are key inputs for liquefaction hazard assessment.

The fine grain criterion for sands (e.g., Wang, 1979; Seed and Idriss, 1982; Bray and Sancio, 2006) allows quick assessment while the standard geotechnical evaluation prospects into mechanical properties of the soil. A widely used technique is the simplified procedure of Seed and Idriss (1971) and its upgraded versions (Seed and Idriss, 1982). The factor of safety against liquefaction (FSL) is evaluated as ‘Cyclic Stress Ratio’/’Cyclic Resistance Ratio’ i.e., the earthquake induced loading divided by the liquefaction resistance of the soil. Other techniques using CPT or MASW also exist (e.g., Shibata and Teparaska, 1988; Lin et al., 2004). The probabilistic and deterministic assessment of the hazard can be found in Cetin et al. (2004) and Moss et al. (2006). The detailed zonation generally places four classes of hazard namely ‘no liquefaction’ (FSL>=2.0), ‘moderate’ (1.5<=FSL<2), ‘high’ (1<=FSL<1.5), and ‘very high’ (FSL<1.0).
Earthquakes can activate slope failures in the undulating terrains leading to landslides with catastrophic effects. These depend on several factors inherent to soil conditions such as geology, hydro-geology, topography, and slope stability. The groundwork consists of appraising existing information from newspapers, local reporting, and concerned organizations followed by association of the historical landslides into classes of failures and movement. The zonation of landslide hazard defining four degrees of hazards: ‘nil or low’, ‘moderate’, ‘high’, and ‘very high’ can be achieved through several ways – from simplistic analysis based on the preparatory factors i.e. soil and slope conditions, seismicity, water content, rainfall, etc to pseudo-static analysis and finite-element methods for non-linear behavior of the soil response. The deterministic landslide susceptible zones can be developed through preparatory factors without considering the triggering factors (Saha et al., 2005). The probabilistic evaluation of seismic landslide hazard dealing with the occurrence of an event with specific intensity at a site during a time interval has been considered by Fell (1994), Hungr (1997) and Perkins (1997). The advanced techniques recently proposed by several researchers like Del Gaudio et al. (2003) and Jibson et al. (2000) are inherently rigorous with extensive data inputs comprising of triggered landslides inventory, strong-motion records, geological maps alongwith engineering properties of different units, and digital elevation models of the topography, and employs dynamic model based on Newmark’s permanent-deformation (sliding-block) analysis.

2.9 COMPOSITE HAZARD EVALUATION

The evaluation of damages of selected elements in risk, for each scenario, is obtained using a simulation model that integrates all the above mentioned aspects, by summing up all possible contributions. The damages of each element in risk are classified in several different limit states, usually defined at five levels: no damage, slight damage, moderate damage, heavy damage, and collapse. Depending on the element under study, this damage classification may be lumped into coarser categories or adapted to operational/non-operational terms, as in the case of sections of lifelines etc.

Scenario studies are viewed in different perspectives according to the objectives to be targeted. These define the scale of intervention which is very critical in the way to obtain the elements necessary for the analysis.

A holistic multiple hazard considerations to deliver a decision support tool for landuse planning and developmental projects such as,

- The composite hazard assessment incorporates multiple attributes through multi-criteria evaluation technique for the spatial delineation.
- Fuzzy sets enabled scheme for the representation and manipulation of uncertainty related to the classification of individual locations according to their attribute values can be aided by the Analytic Hierarchy Process (AHP) - a mathematical method introduced by Saaty (1980) to determine priority of criteria in the decision making process (Nath, 2005).
- AHP uses hierarchical structures to represent a problem and then develop priorities for the alternatives based on the judgment of the experts. Pair-wise comparisons are employed to
form judgments between two particular elements rather than attempting to prioritize an entire list of elements.

- The process of allocating weights is a subjective one and can be done in the participatory mode in which a group of decision makers may be encouraged to reach a consensus of opinions about the relative importance of factors.
- The values within each thematic map/layer varying significantly are classified into various ranges or types, which are referred to as the features of a layer. These features are then assigned ranks or scores within each layer, normalized to ensure that no layer exerts an influence beyond its determined weight.

### 2.10 SEISMIC VULNERABILITY AND RISK ASSESSMENT

The exposures of the vulnerability components such as human population, buildings, etc. to the seismic hazard characterize seismic risk of a region. The seismic hazard is generally assumed to be stable over a long geological time while the typical vulnerability (and therefore, the risk) to hazard changes (McGuire, 2004). At a simple formulation, the risk is assessed as a convolution function of the hazard and the vulnerability:

\[
\text{Risk} = \text{Hazard} \times \text{Vulnerability}
\]

The risk appraisals, aimed at promoting reasonable hazard mitigation regulations, are generally based on vulnerability aspects such as landuse, demographic distributions, building typology etc. The computation of risk is fundamentally influenced by that of the hazard. Likewise, seismic risk assessment could be deterministic or probabilistic.

The former involves direct assessment of possible losses based on the results of deterministic hazard analysis with no involvement of reference time period but yielding to the current status. The assessment could, otherwise, follow either mean values or take into account the uncertainties related to frequency of event occurrences (hazard) and damage levels (vulnerability) yielding to a probabilistic account of the expected losses (Giacomo et al., 2005). These approaches allow estimation of risk on a reference period of time. Another approach is to generate the probable damage scenario by random simulations based on post earthquake damage studies (Barbat et al., 1996).

Typical geotechnical and geophysical investigations being carried out for seismic microzonation of Delhi as documented by Earthquake Risk Evaluation Center (EREC), India Metallurgical Department (IMD), New Delhi through an expert committee formed by the Ministry of Earth Sciences (MoES), New Delhi is given in Appendix – II as a guideline for conducting seismic microzonation investigations in any terrain.
3.1 INTRODUCTION

It has long been known that each soil type responds differently when subjected to ground motion from earthquakes. Usually the younger softer soils amplify ground motion relative to older more competent soils or bedrock. The potentially severe consequences of this phenomenon were demonstrated in the damage patterns of the 1985 Michoacan, Mexico earthquake (Singh et al., 1988), the 1988 Armenian earthquake (Borcherdt et al., 1989), the 1989 Loma Prieta earthquake (Hough et al., 1990; Borcherdt and Glassmoyer, 1992), and the Northridge earthquake in Los Angeles, California. Numerous other studies have also demonstrated the ability of surface geologic conditions to alter seismic motions (Borcherdt, 1970; King and Tucker, 1984; Aki, 1988; Field et al., 1992). There are many factors that influence the way a site will respond to earthquake ground motion (Aki, 1988; Aki and Irikura, 1991; Bard, 1995). These include: (i) the source location, (ii) the prevalence of energy focused or scattered from lateral heterogeneity, and (iii) the degree to which sediments behave nonlinearly, which causes the response to depend on the level of input motion. However, the site effects in the assessment of seismic hazard follow a simple approach wherein, for the potential sources of earthquake ground motion in a region the unique behavior of one site in relation to others is calculated.

Site effects play a very important role in characterizing seismic ground motions because they may strongly amplify (or de-amplify) seismic motions at the last moment just before reaching the surface of the ground or the basement of man-made structures. The greatest challenge in estimating site response from earthquake data is removing the source and path effects. Borcherdt (1970) introduced a simple procedure to divide the spectrum observed at the site in question by the same observed at a nearby reference site, preferably on competent bedrock. The resulting spectral ratio constitutes an estimate of site response if the reference site has a negligible site response. Andrews (1986) introduced a generalized inverse technique to compute site response by solving data of a number of recorded events for all source/path effects and site effects simultaneously. These techniques for computing site response depend on the availability of an adequate reference site (on competent bedrock) with negligible site response. Since such a site may not always be available, it is desirable to develop alternative methods that do not depend on a reference site. Boatwright et al. (1991) suggested a generalized inversion scheme where shear-wave spectra are represented with a parameterized source- and path-effect model and a frequency-dependent site response term for each station.
Another non-reference-site-dependent technique involves dividing the horizontal-component shear-wave spectra at each site by the vertical-component spectrum observed at that site (Lermo and Chávez-García, 1993). This method is analogous to receiver function technique (Langston, 1979) to study the upper mantle and crust from teleseismic records, assuming that the local site conditions are relatively transparent to the motion that appears on the vertical component. Nakamura (1989) introduced another technique for analyzing ambient seismic noise. He hypothesized that site response could be estimated by dividing horizontal-component noise spectra by vertical-component noise spectra. Several studies have since shown that Nakamura’s procedure can be successful in identifying fundamental resonant frequency of sedimentary deposits (Omachi et al., 1991; Lermo and Chávez-García, 1992; Field and Jacob, 1993; Field et al., 1995a).

The receiver function analysis exploits the fact that teleseismic P-waves that are incident upon the crustal section below a station produce P to S conversions at crustal boundaries as well as multiple reverberations in the shallow layers. By deconvolving the vertical-component signal from the horizontal-components, the obscuring effects of source function and instrument response can be removed, leaving a signal composed of primarily S-wave conversions below the station. The deconvolved horizontal component called receiver function trace is a best representative of the site response as the local site conditions are relatively transparent to the motion that appears on the vertical component.

Surface geology and geotechnical characteristics of soil deposits have a paramount importance on seismic ground shaking. The variation of ground shaking in space, amplitude, frequency content and duration are called “site effects”. Site effects include primarily effects of impedance contrast of surface soil deposits to the underlined bedrock, or firm soil considered as rock, which is rather well modeled using 1D ground models (i.e., linear elastic, equivalent linear or non-linear). They also include deep basin effects, and basin edge effects, produced from strong lateral geological discontinuities (i.e., geological anomalies, faults etc.). These effects which are dominated by the presence of surface waves additionally to body waves can only be studied using 2D and 3D models. Finally, site effects are also dealing with spatial variation of ground shaking characteristics due to surface topography.

The physics and the importance of site effects are well understood and quantified with the increasing number of strong motion measurements in the dense accelerometric arrays all over the world. Advanced numerical models using powerful computer facilities have also contributed significantly to the progress during the last two decades. Mexico City (1985) and Loma Prieta (1989) earthquakes, recorded in many stations located in different and well constrained ground conditions, relieved for the first time in a very precise experimentally documented way, the importance of the impedance contrast. Additional evidence of the significance of more complex site effects on seismic ground motions has been brought from the recent destructive earthquakes (Armenia 1988, Philippines 1990, Northridge 1994, Kobe 1995, Kozani 1995, Aegion 1995, Kocaeli and Duzce, Turkey 1999, Athens 1999, Ji-Ji Taiwan 1999 etc.).

3.2 BASIC PHYSICAL CONCEPTS AND DEFINITIONS

Earthquake recordings at soil surface include “information” that is related to three stage of the earthquake phenomenon evolution: a) the source activation (fault rupture), b) the propagation path of seismic energy,
and c) the effect of local geology on the wave-field at the recording site. The physical amplitude \( r(t) \),
potentially representing acceleration, velocity or displacement, which is recorded at a site, can be written in
the time domain in the form of the convolution of three factors:

\[ r(t) = e(t) * p(t) * s(t) \] (3.1)

Where \( e(t) \) is the source signal, \( p(t) \) is the function that characterizes the propagation from the source
to the site and \( s(t) \) expresses the effect of local soil conditions on ground motion (which from now on
will be denoted as site effects). In the frequency domain, Equation (3.1) is written in the form of a product

\[ R(f) = E(f) . P(f) . S(f) \] (3.2)

Where \( R(f) \), \( E(f) \), \( P(f) \), and \( S(f) \) are the Fourier transform of the time depended functions \( r(t) \), \( p(t) \), and \( s(t) \)
respectively. All of the above mentioned factors contribute to overall site response, either independently or
in combination with the others.

The term “site effects” introduce the effect of local geology in the modulation of seismic wave-field at
a recording site; where local geology consists of surface sedimentary sites and surface topography. The
main parameters that characterize a site are the geometry of the soil stratigraphy (thickness and lateral
discontinuities), the shape of the topographic relief and the dynamic, physical and mechanical properties
of soil and rock materials.

Surface soil formations are the product of the long-lasting process of erosion, weathering and
deposition: they are responsible for significant amplification and spatial variation of surface ground motion.
Surface topography in its simplest form consists of convex (ridges, mountains, hill etc.) or concave
surfaces (valley, basins, canyons etc.) with different behaviors during an earthquake. In case of convex
topographies, significant amplification is observed at the crest compared to that at the foot, while in the
concave ones, the amplification varies at the lateral than at the base.

The effect of local geology on ground motion also depends on other parameters such as the intensity,
the frequency and the incidence angle of the incoming wave-field (for strong or weak earthquakes) which
might introduce non-linear phenomena. Generally, it could be stated that there is a large variety of
parameters according to which, someone could categorize site effects, a fact that confirm the complexity
and the need to understand the background physics of this phenomenon.

3.2.1 Site effects due to low stiffness surface soil layers

It has long been recognized that the amplitude of earthquake ground motion is affected by both the
properties and configuration of the near surface material through which seismic waves propagate. These
properties are impedance – resistance to particle motion – (Aki and Richards, 1980) and damping
(attenuation).
Influence of Impedance and Damping in frequency and time domain

For horizontally polarized shear wave (SH) impedance can be defined (Equation 3.3) as the product of the density ($\rho$), the shear wave velocity ($V_s$) and the cosine of the angle of incidence (Figure 3.1b)

$$I = \rho V_s \cos \theta, \quad \cos \theta \approx 1 \quad \text{thus} \quad I = \rho V_s$$

Incidence angle, $\theta$, is usually small near the surface of the earth and its cosine can be assumed to be equal to unity. As a seismic wave passes through a region of decreasing impedance, the resistance to motion decreases and, to preserve energy, the amplitude of seismic wave increases. When there are sharp changes (decrease) in impedance below the earth’s surface (such as sediments/rock interfaces), some of the seismic waves transmitted into the upper layer get trapped in this layer and begin to reverberate.
Damping or inelastic attenuation is substantially greater in soft soils than in hard rocks. The fundamental phenomenon responsible for the amplification of motion in soil sediments is the trapping of seismic waves due to the impedance contrast between sediments and the underlying bedrock. For the simplest case of a soil layer with density $\rho_1$ and shear wave velocity $V_{s1}$ overlying a stiffer layer with density $\rho_2$ and shear wave velocity $V_{s2}$ (Figure 3.1a), the impedance contrast is expressed by the formula:

$$C = \frac{\rho_2 V_{s2}}{\rho_1 V_{s1}}$$  \hspace{1cm} (3.4)

To understand the basic concept of site effects, the simplification of the physical complex phenomena is instructive. Thus, when the structure is horizontally layered (1-dimensional structures), this trapping affects only body waves travelling up and down in the surface layers (Figure 3.1). When the sediments form a 2- or 3-dimensional structure due to soil thickness variations, this trapping also affects surface waves which develop on the sediment bedrock interfaces and thus reverberate back and forth. In all cases, this effect is the maximum when the reverberating waves are in phase with each other. The interference between these trapped waves leads to resonance which is a frequency-dependent phenomenon related to the geometrical and mechanical (density, P-wave and S-wave velocities, damping) characteristics of the soil structure. While these resonance patterns are very simple in the case of a 1D structure (vertical resonance of body waves), they become more complex in 2D and 3D structures. The fundamental resonant frequency may vary between 0.2 Hz (for very thick deposits or extremely soft materials) and 10 Hz or more for very thin layers of deposits or weathered rocks.

The amplitude of fundamental resonant peaks is mainly related to the impedance contrast between surface soil layers and underlying bedrock, to material damping of sediments and to a lesser degree with the characteristics of incident wave-field (type of waves, incidence angle, near or far field,...) for the simplest case, the amplification at the fundamental resonant frequency is given by the formula:

$$A_0 = \frac{2}{\frac{1}{C} + 0.5 \pi \zeta_1}$$  \hspace{1cm} (3.5)

Where C is the impedance contrast and $\zeta_1$ the material damping of the sediments. For the case of very small damping ($\zeta_1=0$), the maximum amplification is simply double the impedance contrast. Another interesting observation is that when the wavelength, $\lambda$, given in Equation (3.6), is much longer than the thickness of the layer (meaning that $\omega h / v_{s1} \approx 0$), the amplitude of surface displacements is doubled. This is called the free surface effect and is caused by upcoming seismic waves being reflected off the free surface of the earth. At the surface, both the upcoming and downgoing reflected waves are exactly in phase and the resultant amplitude at that location doubled.

$$\lambda = V_{s1} T$$  \hspace{1cm} (3.6)
Figure 3.1 provides an illustration of the effects of resonance in the frequency domain, particularly a low resistance sedimentary layer overlying hard rock (impedance contrast c=5). Without taking into account the free surface effect (where the amplification would be doubled as mentioned previously), a 100 m thick layer produces peaks of amplification at about 0.5, 1.5, 2.5 Hz and higher. On the other hand, a 50 m thick layer produces peaks at 1.0, 3.0 Hz and higher. It can be stated, therefore, that, the amplification of higher peaks decreases with increasing frequency, due to the consideration of inelastic attenuation or damping, which in this specific case takes a relatively large value. It has been shown, both experimentally and theoretically that this amplitude very often reaches values between 6 and 10, while in the extreme cases, exceeds 20 (high impedance contrast and small damping).

In case of 2D and 3D structures, fundamental frequency depends also on the geometry of the soil structures. The lateral geometry of these structures is affecting the amplification level at resonant frequencies especially when the material damping is small. Complex effects that are introduced due to the consideration of finite lateral extent are due to the locally generated ones at the discontinuities (edges, faults etc.) and laterally propagated surface waves. The effect of these surface waves is manifested in two ways:

Figure 3.2: Spectral responses computed at the basin center for 1D, 2D and 3D models of semi-shaped basin (after Riepl, 1997; Ansal, 2006).
When the semi-length of the soil structure is much larger than its maximum thickness (shallow basins), the waves have the same frequency characteristics as 1D resonance, thus increasing the 1D amplification level. When the semi-length of the soil structure is comparable to its thickness (deep basins), and the reverberating back and forth surface waves are in phase, the waves interfere with each other leading to 2D resonance patterns. The same resonance effects are involved in the seismic wave modulation due to 3D soil structure. The consideration of the second and third lateral dimension in the wave propagation phenomena, in case of 2D and 3D resonance, leads to an increase in ground motion amplification and a shift towards higher values of peak frequencies. An interesting comparison between 1D, 2D and 3D resonance, spectral peaks of amplification is presented in Figure 3.2. The difference between 1D and 2D resonance are much more pronounced than that between 2D and 3D cases. This means that the consideration of the third dimension in the simulation of ground motion leads to quantitative differences relative to 2D analysis (much larger amplification and a small shift in resonant frequencies).

In the domain, these resonance patterns affect the peak amplitude: of ground motion (mainly Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV)), the waveforms and the motion duration, especially in 2D soil structures. Experimental evidence (records) from recent earthquakes (Mexico, Loma Prieta, Northridge etc.) showed that PGA were up to 4 times larger at soil than at rock sites. Statistical analyses of records have shown that PGA is most likely to be amplified when the fundamental resonant frequency of a site exceeds 2-3 Hz. On the other hand, it was also observed that liquefied sandy deposits induce important reduction of peak acceleration (i.e., Kobe case). Therefore, PGA values on sediments cannot be predicted straightforwardly from PGA values on rock and these issues are strongly related to non-linear phenomena in soil behavior. A general trend however does exist, for moderate acceleration levels (<0.2-0.3g), in the sense that amplification of PGA is expected at soil sites compared to rock sites.

This behavior of PGA amplification may be attributed to (a) the fact that in soils with low S wave velocity, the accumulated energy results in amplification and therefore, as the ground becomes “softer” amplification becomes larger (elastic range) and (b) the fact that under strong dynamic loading the ground becomes “softer”, (strength decreases) and hence, the peak acceleration becomes smaller and the predominant period of soil profiles is shifted to higher values (non-linear behavior of soil materials). Consequently, amplification occurs under small ground shaking with decreasing absolute value as the ground shaking level is increased.

### 3.3 METHODS TO ESTIMATE SITE EFFECTS

There are various methods that may be used for site effect evaluation. The choice of the method is usually related to the significance of the engineering project for which it is applied. Generally, the methods are classified into five main categories.

- **Experimental empirical techniques** that utilize recordings of ground motion or ambient noise to estimate the basic characteristics of the expected ground motion usually in the frequency domain.
- **Empirical methods** that evaluate parameters of earthquake motions such as acceleration, velocity and response spectra based on site classification, average S-wave velocity, topography,
earthquake magnitude and existing amplification relationships; usually these methods are incorporated in seismic code provisions.

- Semi-empirical methods that compute time histories of earthquake motion by combining recorded earthquake motion of smaller earthquakes as element motion (i.e., Green’s Functions); these methods may account for detailed fault rupture process and the effects of asperities.

- Theoretical methods where site effects are computed through an analytical and more often numerical 1D, 2D or 3D wave propagation model; different wave types with different incident angles may be used; the main advantage of these methods is the possibility to use complex constitutive relationships for describing soil behavior under dynamic loading condition and the ability to model accurately site stratigraphy inclusive of basin topography.

- Hybrid methods that compute time histories of earthquake motions by coupling a longer period component determined by a theoretical seismic fault model with a computational seismic wave propagation model having a shorter period component is determined by a semi-empirical method.

The use of each method depends on many parameters and, in any case, requires an increased level of expertise. In the following paragraphs some aspects of the first four methods will be briefly discussed.

### 3.3.1 Theoretical Considerations and Site Response Estimation

The seismograms of the selected events are first corrected for the system response. Next the S-wave packets recorded by the seismographs are windowed with a window width containing the maximum amplitude. The window length is selected following the results of Seekins et al. (1996). A Hanning taper is applied to the time windowed data and then butterworth bandpass filtered before the amplitude spectra are computed.

Let the S-wave spectral amplitude and that of the background noise be $O(r_{ij}, f_k)$ and $B(r_{ij}, f_k)$ respectively at the hypocentral distance $r_{ij}$. Then the signal amplitude spectrum at the frequency $f_k$ can be expressed as,

$$A(r_{ij}, f_k) = O(r_{ij}, f_k) - B(r_{ij}, f_k)$$  \hspace{1cm} (3.7)

The corrected spectra are smoothed in order to reduce the data variance using a five-point smoothing window and a spline interpolator at 0.1 Hz interval.

Suppose a network has recorded $I$ events by $J$ stations. The amplitude spectrum of the $i^{th}$ event recorded at the $j^{th}$ station for the $k^{th}$ frequency, $A(r_{ij}, f_k)$ can be written in the frequency domain as a product of a source term $S_0(f_k)$, a propagation path term $P(r_{ij}, f_k)$, and a site effect term $S_I(f_k)$, (Lermo and Chávez-García, 1993; Nath et al., 2002a and 2002b):

$$A(r_{ij}, f_k) = S_I(f_k) \cdot P(r_{ij}, f_k) \cdot S_0(f_k)$$  \hspace{1cm} (3.8)
3.3.2 Standard Spectral Ratio (SSR) Technique

The most popular and widely used technique to characterize site amplification has been the Standard Spectral Ratio (Borcherdt, 1970), which is defined as the ratio of the Fourier amplitude spectra of a soil-site record to that of a nearby rock-site record from the same earthquake and component of motion (Figure 3.3). Suppose, we have a reference site \( j=R \) that is assumed to have a negligible site response \( \ln S_R = 0 \) and if the interstation spacing is too small compared to the epicentral distances, so that \( P(r_j, f_k) \approx P(r_R, f_k) \), then the site response at each seismological station can be estimated from,

\[
\ln S^R_{SR}(r_j, f_k) = \frac{1}{l} \sum_{i=1}^{l} \ln \left( \frac{A(r_j, f_k)}{A(r_R, f_k)} \right) = \frac{1}{l} \sum_{i=1}^{l} (\ln A(r_j, f_k) - \ln A(r_R, f_k))
\]

Equation (3.9) constitutes the geometric average spectral ratio. If the reference site has a non-negligible site response, then the spectral ratios become relative site-response estimates or site amplification factor. In the classical spectral ratio method, the source and path effects are considered identical for every site and the reference one only when the epicentral distance is appreciably larger compared to the interstation spacing. As a result the normalized SSR method is expected to perform better for the regional events.

The usual option for the selection of the reference station is a site of outcropping rock, while less frequently, a bedrock site having a downhole accelerometer installed in a borehole is used for this purpose.

**Figure 3.3:** General description of the Standard Spectral Ratio (SSR) Technique (after Ansal, 2006).
The basic conditions for the application of this particular technique are: a) the existence of simultaneous recordings at a soil site and at the reference site, b) the reference site has to be free of any kind of site effects (sediments and topography), and c) the distance between the soil site and the reference one ought to be small (i.e., smaller than the epicentral distance), in order to consider that the effect of the propagating path of the seismic energy is the same for the two sites.

However, the condition that an outcrop rock reference site should be free of any kind of site effects often is not valid. For this reason, a careful examination of the reference site is obligatory in order to correctly estimate amplification in sedimentary sites (Stiedl et al., 1996).

### 3.3.3 Generalized Inversion Scheme

Andrews (1986) having in mind that the Standard Spectra Ratio technique is reliably applicable only to data from dense, local arrays, proposed a generalized technique to look for all source, path and site effects in large data sets recorded in local or regional networks, by applying the solution of a large inverse problem.

In Equation (3.8) the propagation path term can be expressed as,

\[
P(r_i, f_k) = G(r_i) e^{Q_s G(r_i) f_k f_k}
\] (3.10)

Where \( G(r_i) \) accounts for geometrical spreading, \( Q_s(f_k) \) and \( \beta \) are S-wave frequency-dependent quality factor and velocity, respectively. Following Ordaz and Singh (1992), Castro et al. (1996) and others, we considered,

\[
G(r_i) = \frac{1}{r_i} \quad \text{for } r < 100 \text{ km}
\] (3.11)

Or,

\[
G(r_i) = (r_i * 100)^{-0.5} \quad \text{for } r > 100 \text{ km}
\] (3.12)

It is to take into account possible arrival of surface waves in the windowed data. The Equation (3.8) can further be written as,

\[
SO_i(f_k) SI_j(f_k) = \frac{A(r_i, f_k)}{P(r_i, f_k)} = A'(r_i, f_k)
\] (3.13)

Where \( A'(r_i, f_k) \) is the spreading and attenuation corrected amplitude spectrum. Taking the natural logarithm on both the sides of Equation (3.13) and multiplying by \( \sigma_{ijk} \) (Hartzell, 1992), we can write,

\[
\sigma_{ijk} \ln SO_i(f_k) + \sigma_{ijk} \ln SI_j(f_k) = \sigma_{ijk} \ln A'(r_i, f_k)
\] (3.14)

Or,

\[
S_i(f_k) + R_j(f_k) = D_{ij}(f_k)
\] (3.15)

Where, for the \( k \)th frequency \( f_k \), \( S_i = \sigma_{ijk} \ln SO_i(f_k), R = \sigma_{ijk} \ln SI_j(f_k) \) and \( D_{ij} = \sigma_{ijk} \ln A'(r_i, f_k) \). \( \sigma \) is a weighted factor and an estimate of the standard deviation of the data given by the ratio of the signal spectrum to a noise sample spectrum, \( N_{ij}(f_k) \). The algorithm for estimating \( \sigma \) tried in this study is,
\[ \sigma_{ij} = \max \left( \min \left( A(r_{ij}, f_k) / N(r_{ij}, f_k), 5.0 \right), 1.0 \right) / 5.0 \]  

(3.16)

Where \( \sigma \) is normalized and limited to the range of 1.0 to 0.2. This imposes a balance in the quality of the data with the need to include as many observations as possible to average out radiation-pattern and source-directivity effects. \( N_i(f_k) \) is obtained from noise sample immediately preceding the shear-wave arrival. Equation (3.15) can be written in the matrix form following the notation of Menke (1989) and solved by generalized inversion scheme using singular value decomposition (Nath et al., 2002b). The method requires a reference site with known response value in the frequency range of interest to minimize the trade-off between the source and site parameter values as observed by Andrews (1986).

\[
\begin{bmatrix}
G \\
S_i \\
R_j
\end{bmatrix}
= 
\begin{bmatrix}
(f_k) \\
(f_k)
\end{bmatrix}
= \begin{bmatrix} D_y (f_k) \end{bmatrix}
\]

(3.17)

Where \( G \), the data kernel matrix, is a sparse matrix, having two nonzero elements per row or column for each frequency. Equation (3.17) is solved using singular value decomposition technique and the source and site terms are computed.

\[ \ln SO_j (f_k) + \ln SI_j (f_k) = \ln A(r_{ij}, f_k) \]

(3.18)

The main advantage of this technique is the reliable estimation of the source and site effect terms from the whole data set especially in cases where there are no simultaneous recording of all earthquakes at all sites of the network (Field and Jacob, 1995).

### 3.3.4 Horizontal to Vertical Spectral Ratio (HVSR) Technique

All techniques mentioned above are using a reference site but in practice, appropriate reference sites are not always available. For this reason different methods that are not depending on reference sites have been developed. One of them is the Horizontal to Vertical Spectral Ratio. This extremely simple technique consists of using the spectral ratio of the horizontal to the vertical component of ground motion and estimates the Fourier amplitudes in different frequencies accordingly. The receiver function HVSR \( r_{ij}(f_k) \) can be computed at each \( j \) site for the \( i \)th event at the central frequency \( f_k \) from the root mean square average of the amplitude spectra as,

\[
HVSR_{ij}(f_k) = \frac{1}{2} \sqrt{\frac{\text{abs}H_{ij}(f_k)_{\text{NS}}^2 + \text{abs}H_{ij}(f_k)_{\text{EW}}^2}{\text{abs}V_{ij}(f_k)}}
\]

(3.19a)

Where,

\[ H_{ij}(f_k)_{\text{NS}} \] - Fourier spectra of the NS component,
\[ H_{ij}(f_k)_{\text{EW}} \] - Fourier spectra of the EW component, and
\[ V_{ij}(f_k) \] - Fourier spectra of the vertical component.

Finally the event average receiver function \( HVSR_{ij,\text{ave}}(f_k) \) (Field and Jacob, 1995) is computed at each \( j \) site for the \( k \)th frequency to consider the contribution of all the seismic events recorded at that
station. The basic assumption of the method is that the vertical component of the ground motion in case
where the soil stratigraphy is flat and horizontal is supposed to be free of any kind of influence related to
the soil conditions at the recording site. Figure 3.5 shows the general layout of the method which was first
applied to the S wave portion of the earthquake recordings obtained at three sites in Mexico City by Lermo
and Chavez-Garcia (1993). Generally, the Fourier spectra ratio exhibit similarities between SSR and
HVSR technique, with a better fit in frequencies rather than amplitudes of the resonant peaks.

**Figure 3.4:** Description of the Horizontal to Vertical Spectral Ratio (HVSR) Technique (after Ansal, 2006).

Detailed comparisons between SSR technique and other reference station techniques (Field *et al.*, 1992; Stiedl, 1993; Field and Jacob, 1995) led to a few basic qualitative conclusions such as: a) the
estimation of site effects with the use of SSR technique is relatively stable even if records are quite noisy,
b) the process should be based on a significant number of earthquake recordings (the use of a limited
number of records should be avoided), and c) the amplification level determined with SSR technique is
quite similar with that determined from other techniques and especially with the GIS.

Other comparison between the results of SSR and HVSR techniques led to controversial conclusions.
As it is already stated above, Lermo and Chavez-Garcia (1993) applied HVSR to the S wave portion of the
earthquake recordings and found similarities between standard spectral ration and HVSR with a good fit
in both frequencies and amplitude of the resonant peaks. Some other researchers used HVSR technique
on data sets from weak and strong motion records and concluded that the shape of the spectral ratio
presents very good statistical stability with minor dependency on source and path effects and that it is
quite well correlated with surface geology, while their amplification level seems to depend on the type of
incident wave, a fact that does not affect the fundamental resonant frequency. Field and Jacob (1995)
after systematic comparisons with other techniques concluded that the shape of the transfer function is satisfactorily reproduced by HVSR technique, although there is an underestimation of the amplification factor compared to SSR. On the same issue Raptakis et al. (1998 and 2000), using a large and high quality data set from EUROSEISTEST experimental site, proved that the significant differences between SSR and HVSR amplitudes at the fundamental frequency are attributed to the considerable amplification of the vertical component due to diffracted Rayleigh waves at the lateral discontinuities of the basin.

In conclusion, both SSR and HVSR techniques are reliable in estimating the fundamental frequency of the soil profile. However the amplification amplitude is comparable only when the soil layering is horizontal and there aren’t lateral geometrical variations. In those cases, due to the presence of in-ward propagating surface waves, it is expected that part of them will affect the vertical component and hence the amplitude of the HVSR method.

### 3.3.5 Horizontal to Vertical Response Spectral Ratio (HVRSR) Technique

The Horizontal to Vertical Response Spectral Ratio (HVRSR) is obtained using geometric mean of the response spectra of the horizontal components and the response spectra of the vertical component as follows,

\[
HVRSR_j(f_k) = \left( \frac{A_t(j, f_k) | T A_t(j, f_k) | R}{A_v(j, f_k) | V} \right)^{0.5}
\]

Where \( A \) stands for spectral acceleration at each \( j \) site for the \( i \)th event at the central frequency \( f_k \), \( T \), \( R \) and \( V \) representing the transverse, radial and vertical components respectively.

### 3.3.6 Empirical Methods

Empirical methods are practically used either for preliminary analyses or in the frame of seismic code prescriptions with well specified amplification factors defined according to the soil classification and the earthquake intensity. Simple relationships giving the amplification factor for the peak acceleration or/and velocity with the average shear wave velocity of the soil profile are proposed in the literature (Joyner and Fumal, 1984; Midorikawa, 1987; Borcherdt et al., 1991). All these relationships should be used only for preliminary studies and with extreme caution.

### 3.3.7 Semi-Empirical Methods

The semi-empirical methods compute time histories of earthquake motions caused by large scenario earthquake by combining recorded earthquake motions by smaller seismic event. The Green’s function technique is based on the idea that the total motion at a particular site is equal to the sum of the motions produced by a series of independent ruptures of many small parts on a causative fault. The method requires an approximate definition or estimation of certain parameters such as the geometry of the source, the slip functions describing the slip displacement vector with time for each elementary source, the velocity structure of the crustal materials between the source and the site and the Green’s Functions that describe the motion at the site due to an instantaneous unit slip at each elementary source. Normally the Green’s function at each site, account implicitly the particular site specific ground behavior in the linear elastic range.
The Empirical Green’s Function (EGF) technique (Hartzell, 1978) bypasses these complicated computations by using the weak motions of small earthquakes as empirical Green’s Functions to simulate strong motion. The method is essentially a deterministic one as it computes time histories for a defined earthquake scenario and other parameters. However it is possible to use statistical Green’s Functions which are computed as the statistical average of the recorded earthquake motions for different small seismic events.

3.3.8 Theoretical (Numerical and Analytical Methods)

When the geological structure of an area and the geotechnical characteristics of the site are known, site effects can be estimated through theoretical analysis. The prerequisite of sufficient geotechnical knowledge of the soil structure including surface and deep topography is, therefore, obvious. Such an approach requires an in-depth understanding of the constitutive models describing the soil behavior under dynamic solicitations and the methods used to solve the wave propagation problem in 1, 2 or 3 dimensions.

3.4 SITE EFFECTS AND SEISMIC CODES

Seismic ground response characteristics, defined generally as “site effects”, are inevitably reflected in seismic code provisions. The selection of appropriate elastic response spectra according to soil categories and seismic intensity is the simplest way to account for site effects both for engineering projects and for a general purpose microzonation study.

Modern seismic codes (IBC2000, UBC97, NEHRP, EC8) all introduced in the last few years, after the recent strong earthquakes in America, Europe, Japan and worldwide, which produced numerous valuable data, have incorporated the most important experimental and theoretical results with the adjustments and simplifications for purely practical reasons.

The main improvement is that amplification factors of spectral values are varying with seismic intensity; lower shaking intensity earthquakes introduce higher amplification factors due to more linear elastic soil behaviors, contrary to higher intensities where soils are exhibiting non-linear behavior resulting in a decrease of peak spectral values. Additionally, a more accurate soil categorization is introduced based on a better description of soil profiles using standard geotechnical parameters (i.e., plasticity index PI, undrained shear strength Su) and average $V_s$ values. In IBC2000 and other codes of the same family, a special attention is given to near-field conditions introducing higher amplification factors for the same earthquake magnitude. Also for soil layers of small thickness presenting high impedance contrast, the new version of codes attribute higher amplification factors which is compatible with observations and theory.

In general the parameters describing site effects in seismic codes are expressed through (a) soil categorization, and (b) spectral amplification factors and shapes. 1D site effect computations using the equivalent linear model is the main and almost universal tool for all improvement and modifications introduced so far (i.e., Dickenson and Seed, 1996; Pitilakis, 2003). The increasing number of records during the last two decades gave the minimum necessary validation background to these theoretical efforts and simulations. Full non-liner and elasto-plastic models are not really used mainly because of the difficulty in defining soil parameters for all soil categories, while it is known that probably for strong
earthquakes, the use of elasto-plastic models shall lead to an even more important decrease of ground amplification, especially in low resistance soil layers (Pitilakis et al., 1999).

Modern codes are certainly the serious steps forward for a better evaluation of design input motions at least for ordinary buildings. Future improvements should be addressed to the following issues: Azimuthal effects (i.e., different spectral values for the two horizontal components), basin edge and deep basin effects, evaluation of ground motion for large and very large shear strains, vertical component, topographic effects, velocity and displacement spectra, spatial variability of ground motion etc.

Seismic codes should always reflect the basic knowledge and technology of the present time, keeping in mind that they must be simple and realistic, having an acceptable level of accuracy, compatible among others, with the tools used for the seismic design of the structures.

3.5 1D GROUND RESPONSE ANALYSIS USING EQUIVALENT LINEAR APPROACH

Geotechnical site response is computed by considering (a) a geological unit which is defined by shear-wave velocity, damping, total unit weight, and thickness and (b) an input motion at rock level which has to be computed synthetically or the observed earthquake. Kramer (1996) has given the general theory for the computation of geotechnical site response.

Any input signal can be treated as the sum of sine waves of different amplitude, frequency and phase. A simpler solution of the site response at each frequency is used to compute the response of soil at each of the input sine waves. In the single layer case consider a uniform soil layer lying on an elastic rock surface of infinite depth. The horizontal displacement of harmonic S-waves in each material can be written as,

\[ u_s(z_r, t) = A_s e^{i(\omega t + k^*_s z_s)} + B_s e^{i(\omega t - k^*_s z_s)} \]  
\[ u_r(z_r, t) = A_r e^{i(\omega t + k^*_r z_r)} + B_r e^{i(\omega t - k^*_r z_r)} \]  

Where subscripts s and r refer to soil and rock respectively. \( \omega \) is the circular frequency of the harmonic wave and \( k^*_s \) is the complex wave number. Applying the boundary conditions on the above equations, transfer function can be written as,

\[ F(\omega) = \frac{2}{(1 + \alpha_s^*) e^{i k^*_s H} + (1 - \alpha_s^*) e^{-i k^*_s H}} \]  

Where \( \alpha_s^* \) is the complex impedance ratio. Since it is a complex function, it can be rewritten using Eular’s law as,

\[ F(\omega) = \frac{1}{\cos k^*_s H + i \alpha_s^* \sin k^*_s H} \]  

Site amplification of multilayer can be computed using the above formulations so that the transfer function for this type of soil deposit must account for the transmission and reflection of waves at boundaries between the adjacent layers. Due to constant change in shear modulus of the soil; the nonlinear behavior can’t be ignored in the analysis. The effect of nonlinear behavior in seismically induced dynamic loading
has been a debatable issue amongst seismologists. The prime assumption in the geotechnical analysis is that site response is depended on strain amplitude (Finn, 1991), however seismologists have avoided the nonlinearity due to lack of direct evidence from strong motion data (Aki and Richards, 1980).

As proposed by Kramer, an equivalent linear model of non-linear behavior of soil can be taken into consideration. It performs the iterative adjustment on the soil properties which is to be consistent with effective level of shear strain in the soil. The relationship between shear modulus and shear strain can be characterized by a modulus reduction curve. The first iteration in the equivalent linear analysis is performed using shear modulus and damping ratio. The effective shear strain can be defined as,

\[ \gamma_{\text{eff}} = R_\gamma \gamma_{\text{max}} \]  

(3.24)

Where \( R_\gamma \) is a stress reduction factor often taken as,

\[ R_\gamma = \frac{M - 1}{10}, \text{ } M \text{ is the magnitude of earthquake.} \]  

(3.25)

The process is repeated until there is not much variation from one iteration to the next and this is called convergence of the solution. It must be noted that although it is a linear equivalent of nonlinear phenomenon, it is still a linear analysis. However, this approach has been shown to provide a reasonable estimate of soil response. Site response using above algorithm has been coded in many software (SHAKE 2000, SHAKE91, PROSHAKE, DEEPSOIL etc.).

![Figure 3.5: Equivalent Linear Approach.](image)
In the equivalent linear approach as defined in Figure 3.5 the non-linearity of shear modulus and damping is accounted for the use of equivalent linear soil properties using an iterative procedure to obtain values for modulus and damping compatible with the effective strains in each layer. In this approach, first, a known time history of bedrock motion is represented as a Fourier series, usually using the Fast Fourier Transform (FFT). Second, the Transfer Functions for the different layers are determined using the current properties of the soil profile. The transfer functions give the amplification factor in terms of frequency for a given profile. In the third step, the Fourier spectrum is multiplied by the soil profile transfer function to obtain an amplification spectrum transferred to the specified layer. Then, the acceleration time history is determined for that layer by the Inverse Fourier Transformation in step four. With the peak acceleration from the acceleration time history obtained and with the properties of the soil layer, the shear stress and strain time histories are determined in step five. In step six, new values of soil damping and shear modulus are obtained from the damping ratio and shear modulus degradation curves corresponding to the effective strain from the strain time history. With these new soil properties, new

![Figure 3.6: Shear modulus reduction and damping ratio curves considered for sand (after Seed and Idriss, 1970).](image)

![Figure 3.7: Shear modulus reduction and damping ratio curves considered for rock (after Schnabel, 1973).](image)
transfer functions are obtained and the process is repeated until the difference between the old and new properties fit in a specified range. The basic approach of one dimensional site response study is the vertical propagation of shear waves through soil layers lying on an elastic layer of the rock which extends to infinite depth.

Soil behavior under irregular cyclic loading is modeled using modulus reduction \(\frac{G}{G_{\text{max}}}\) and damping ratio \((\beta)\) vs. strain curves. The non-linearity of shear modulus and damping is accounted for by the use of equivalent linear soil properties using an iterative procedure to obtain values for modulus and damping compatible with the effective strains in each layer as discussed above. The degradation curves for sand and rock generally follow spectral response analysis used for those proposed by Seed and Idriss (1970) and Schnabel (1973) respectively. These curves are shown in Figures 3.6 and 3.7 respectively. These are included in the SHAKE database and can be selected as input using appropriate option command.

A few typical Case Studies on site amplification / response assessment are given in APPENDIX – III.
CHAPTER – 4

Strong Ground Motion Synthesis and Seismic Hazard

4.1 INTRODUCTION

Recordings of high-frequency ground motion provide the information about seismic sources. At the same time geotechnical engineers use strong motion data for structural design, urban planning and management. The paucity of strong ground motion recordings at all magnitude ranges necessitates the generation of synthetic data. The shape, scaling of source spectra and seismic moment form the basis for simulating ground motions. Several techniques are available that differ in theoretical considerations, data and computational requirements. Realistic results vis-à-vis complexity and computational as well as data requirements are the deciding factors for the applicability of a particular technique. Some of the techniques widely used include: (i) Stochastic approach (Boore and Atkinson, 1987; Beresnev and Atkinson, 1997; Motazedian and Atkinson, 2005), (ii) Green Functions method (Bouchon and Aki, 1977), (iii) Empirical Green Functions method (Hartzell, 1978; Irikura, 1983), (iv) Finite Difference method (Panza, 1985; Oprsal and Zahradnik, 2002), (v) Finite Element method (Frankel, 1989), and (vi) Spectral Element method (Komatitsch and Tromp, 1999). The Frequency-Wave number (F-K) integration method has proved to be very useful in understanding wave progression in layered media as well as in the complex geological domain. Most often, the F-K integration approach accurately reflects the wave propagation phenomena and is useful for site-specific simulations. In the empirical Green’s Function method, a moderate event is simulated taking into account some a priori information about source and path using small recorded events.

The quantitative assessment of seismic hazard necessitates measurement of peak ground motion parameter (e.g., PGA) from earthquake records. Paucity of strong ground motion data records under conditions similar to design earthquakes in terms of tectonic regime, earthquake size, local geology, and near fault conditions necessitates analytical or numerical approach for a realistic prognosis of the possible seismic effects. The strong ground motion modeling must accommodate: (i) the seismic wave radiation from a fault rupture, (ii) propagation through the crust, and (iii) modifications by site conditions as depicted in Figure 4.1.
4.2 SIMULATION OF STRONG GROUND MOTION

Two types of modeling exist in seismology: forward modeling and inverse modeling. Forward modeling deals with the estimation of ground motion at the ground surfaced by modeling the earthquake faulting process, the medium of propagation between the earthquake source and the station, and local site effects near the station, such as modeling of topography, basin structure, and soft soil conditions. Inverse modeling, on the other hand, uses recorded ground motion data and tries to model earthquake source processes. For engineering purposes, estimation of ground motion at a location for a future large earthquake is important. Therefore, forward modeling is used on many occasions for strong ground motion estimation, using the results of inverse modeling as input, if available. In the following paragraphs we will deal with forward modeling only.

4.2.1 Stochastic Simulation

The high frequency accelerations are highly incoherent, and are the result of irregularities of the rupture front, heterogeneity of co-seismic slip distribution, fault geometry irregularity, propagation transfer, rupture bifurcation, and sudden changes in rupture velocity and slip amplitude. These details are unknown and unpredictable and probably impossible to model in a deterministic manner. Therefore, in strong ground motion simulations the incoherence of ground accelerations is accounted for using stochastic methods. Stochastic approaches of simulation of strong ground motion filter and window the white-noise time series according to seismologically determined average spectra and duration (Boore, 1983). This is a widely used and easily applicable method for the simulation of higher-frequency motions; however, it lacks low frequency information. In a recent study by Tumarkin and Archuleta (1997), there have been efforts towards incorporating rupture directivity effect into stochastic method by adjusting corner frequency, and towards increased low frequency content by the use of a new filter.

The stochastic approach is a powerful simulator of high frequency ground motion (f>0.1 Hz). It has been widely used to predict ground motion around the globe where earthquake recordings are meager. This method has been used to match observed waveform even up to very high magnitude or large seismic moment in different tectonic environments. One of the essential characteristics of the method is that it uses the simple functional forms of various factors affecting the ground motions (source, path, and site).
It uses the convolution model to model spectral acceleration. Time domain simulation is done as in the following steps:

- White noise (Gaussian or uniform) is generated for a given duration of strong motion
- It is windowed to give a shape like accelerograms by some shaping window. Boxcar function or window given by Saragoni and Hart (1974) can be used for the purpose
- The windowed noise is then transformed into frequency domain and it is normalized
- The convolution model is then multiplied with this normalized spectrum, and
- Inverse Fourier transform is taken to obtain the signal in time domain.

The above process has been depicted in Figure 4.2. In the entire process (Safak and Boore, 1988), it has been shown that the order of windowing and filtering is important; if the white noise is first filtered and then windowed the long-period level of the motion is distorted.

Figure 4.2: Time domain simulation steps in stochastic modeling (after Boore, 2003).
4.2.1.1 **Stochastic Simulations for Point Source Earthquakes**

This is essentially an engineering approach to the simulation of strong ground motion. Ground acceleration is modeled as a filtered Gaussian white noise modulated by a deterministic envelope function (Safak, 1988). The filter parameters are determined by either matching the empirical properties of the spectrum of the strong ground motion (e.g., Trifunac et al., 1988), theoretical spectral shapes (i.e., the Kanai-Tajimi spectrum; Housner and Jennings, 1964), or are determined on the basis of reliable physical characteristics of earthquake source and propagation media (e.g., Hanks and McGuire, 1981; Boore, 1983). The model used in the latter stochastic simulations is essentially an S-wave ground motion spectrum based on the far-field model of Brune (1970). It has been found to satisfy the main parameters of high frequency ground motion for earthquakes within a wide magnitude range (McGuire and Hanks, 1980). The theoretical considerations are given below.

The amplitude spectrum $A(\omega)$ can be written, in the frequency domain, as the product of source function $SO(\omega, \omega_c)$, a propagation path term $P(\omega)$, and a site function $SI(\omega)$ (Nath et al., 2005; Lermo and Chavez-Garcia, 1993; Boore, 1983) as,

$$ A(\omega) = SO(\omega, \omega_c) \cdot SI(\omega) \cdot P(\omega) $$  \hspace{1cm} (4.1)

Acceleration spectra often show a sharp decrease with increasing frequency so a high cut filter $F(\omega, \omega_m)$ is incorporated in Equation (4.1) such that,

$$ A(\omega) = SO(\omega, \omega_c) \cdot SI(\omega) \cdot P(\omega) $$  \hspace{1cm} (4.2)

Papageorgiou and Aki (1983) related $\omega_m$ to source processes while Hanks (1982) associated $\omega_m$ to attenuation near the recording site. However, the high-cut filter $F(\dot{u}, \dot{u}_m)$ has been taken to be,

$$ F(\omega, \omega_m) = \left[1 + (\omega/\omega_m)^2s\right]^{-1/2} $$  \hspace{1cm} (4.3)

Where ‘s’ controls the decay rate at higher frequencies. After comparing with several observed spectra, it has been assigned a value of ‘4’ (Boore, 1983).

Sometimes the high-cut filter $F(\omega)$ given by Anderson and Hough (1984) has also been used

$$ F(\omega) = e^{-k\omega^2/2} $$  \hspace{1cm} (4.4)

Where $k$ is a spectral decay parameter and controls the decay rate at higher frequencies.

Brune’s $\omega^2$-circular crack source model (Hwang and Huo, 1997) defines earthquake source to be a circular fault for which the ground acceleration spectrum decays with $\omega^2$ for frequencies higher than source corner frequencies. The point-source spectrum of this model is given as,

$$ \overline{SO}(\omega, \omega_c) = \left[\frac{R_{wp}FS.\omega^2}{\sqrt{2}(4\pi p\beta^2)}\right]_{\ast} \frac{M_0}{1+(\omega/\omega_c)^2} $$  \hspace{1cm} (4.5)

where $R_{wp}$ (=0.63) is the radiation pattern averaged over an appropriate range of azimuths and take-off angles, $FS$ (=2.0) accounts for the amplification of the seismic wave at the free surface, $\rho$ is the crustal...
density of the continental crust at the focal depth and $\beta$ is the shear-wave velocity at the source region. $M_0$ and $\omega_c$ are the scalar moment and corner frequency respectively. The corner frequency ($f_c = \omega_c / 2\pi$) given in terms of moment is,

$$f_c = 4.9 \times 10^8 \beta \left( \frac{\Delta\sigma}{M_0} \right)^{1/3}$$  \hspace{1cm} (4.6)

Where $\Delta\sigma$ is the stress drop in bars, $f_c$ is in Hertz, $\beta$ in kilometers/sec and $M_0$ in dyne-cm (Brune, 1970).

The propagation path term can be expressed as,

$$P(\omega) = G(R) e^{-\omega R}$$  \hspace{1cm} (4.7)

Where $G(R)$ accounts for geometrical spreading, and $Q(\omega)$ is a shear wave frequency-dependent quality factor of the medium. Following Ordaz and Singh (1992) and Castro et al. (1996) the following has been considered to take into account the possible arrival of surface waves in the windowed data,

$$G(R) = \begin{cases} 
1/R & \text{for } R<100 \text{ km} \\
(R*100)^{-0.5} & \text{for } R>100 \text{ km}.
\end{cases} \hspace{1cm} (4.8)$$

### 4.2.1.2 Random Vibration Theory

Peak ground acceleration can be predicted using random vibration theory (Lai, 1982) without calculating time series. Although we have generated time series but this theory is preferred, since we can avoid reliance on the additional comparison that may be required for calculating the time series. Here $E(a_{rms})$ is determined by $a_{rms}$ using the following relations (Boore, 1983). The $k$th moments $m_k$ of energy density spectrum of acceleration are defined as,

$$m_k = \frac{1}{\pi} \int_0^{\infty} \omega^k |A(\omega)|^2 d\omega$$  \hspace{1cm} (4.9)

The rms acceleration is given by,

$$a_{rms} = (m_0 / T)^{1/2}$$  \hspace{1cm} (4.10)

Where $T$ is the duration of accelerograms. For accelerograms in which number of extremes $N$ is less than 2, $N$ is arbitrarily set equal to 2. If the value of $N$ exceeds 20, an asymptotic approximation given below should be used for $E(a_{max})$ (Cartwright and Longuet-Higgins, 1956).

$$\frac{E(a_{max})}{a_{rms}} = 2\ln(N)^{1/2} + \gamma/[2\ln(N)]^{1/2}$$  \hspace{1cm} (4.11)

where $N$ is determined by

$$N = 2 \tilde{f} T$$  \hspace{1cm} (4.12)

And $\tilde{f}$ is predominant frequency and is calculated by
The accelerograms have been computed by stochastic simulation both in time as well as frequency domains taking into consideration the source, site and path effect to verify the derived parameters from strong motion data.

**4.2.1.3 Stochastic Simulations for Finite Fault Rupture Earthquakes**

Beresnev and Atkinson (1997) have developed a procedure (FINSIM) for the stochastic simulation of strong ground motion from finite fault ruptures. The fault rupture plane is modeled with an array of subfaults. The radiation from each subfault is modeled as a point source with a $\omega^2$-spectrum, similar to Boore (1983). The fault rupture initiates at the hypocenter and spreads uniformly along the fault plane with a constant rupture velocity triggering radiation from subfaults in succession. Simulations from each subfault, properly lagged and summed at the observation point, provide the simulation of ground motion from the modeled finite fault rupture. The size of the subfaults controls the overall spectral shape at medium frequencies. Optimal subfault size ($\Delta l$) is given as (Beresnev and Atkinson, 1999):

$$\log (\Delta l) = -2.0 + 0.4 M_w$$  \hspace{1cm} (4.14)

The total number of subfaults is controlled by the constraint that the total seismic moment of the subfaults must be equal to the target seismic moment. The quantitative assessment of seismic hazard necessitates measurement of peak ground motion parameter (e.g., PGA) from earthquake records. Paucity of strong ground motion data records under conditions similar to design earthquakes in terms of tectonic regime, earthquake size, local geology, and near fault conditions necessitates analytical or numerical approach for a realistic prognosis of the possible seismic effects.

The current trend in ground-motion prediction is shifting towards an extended, or finite-fault, source representation. The point source approximation is clearly unable to characterize key features of ground motions from large earthquakes, such as their long duration and the dependence of amplitudes and duration on the azimuth to the observation point (source directivity). Finite-fault effects contribute not only to the duration and directivity of ground motions; they also affect the shape of the spectra of seismic waves. In this model, the finite-fault plane is subdivided into elements (subfaults), and the radiation from a large earthquake is obtained as the sum of contributions from all elements, each of which acts as an independent (sub) source. In the typical implementation, the rupture starts at a hypocentral point on the fault and propagates radially from it, triggering the subfaults as it passes them. The fields from all the subevents are geometrically delayed and added together at the observation point.

The stochastic method for strong ground motion simulation of Beresnev and Atkinson (1997) employs a finite fault. The dimension of the sub-fault can be calculated using a relation given by Beresnev and Atkinson (1997). However, a large uncertainty is associated with this equation due to paucity of large earthquake magnitude data. To overcome this limitation to a large extent, the algorithm is further modified
by Motazedian and Atkinson (2005) by introducing dynamic corner frequencies. The enhanced approach conserves the radiated energy at high frequencies at any sampling of sub-fault size and controls the relative amplitude of higher versus lower frequencies.

Accordingly the Fourier amplitude spectrum due to the $n^{th}$ sub-fault is given as,

$$A_n(f) = CM_n H_n \left[ \frac{(2\pi f)^2}{1 + \left( \frac{f}{f_{on}(t)} \right)^2} \right] e^{\frac{-\pi f f_{on}(t)}{\beta Q}} G \quad (4.15)$$

Where $C$ is a scaling factor, $M_n$ is the $n^{th}$ sub-fault moment, $H_n$ is the spreading factor responsible for conserving the energy at high frequency spectral level of the sub-fault, $f_{on}(t)$ and is the dynamic corner frequency. The scaling factor is given as,

$$C = \frac{R_{\text{eq}} \sqrt{2}}{4\pi \rho eta^3} \quad (4.16)$$

The moment $M_n$ of the $n^{th}$ sub-fault is calculated using the slip distribution as follows,

$$M_n = \frac{M_o D_n}{\bar{d}(D_n)} \quad (4.17)$$

Where $D_n$ is the corresponding average slip. The dynamic corner frequency is expressed as follows,

$$f_{on}(t) = 4.9 \times 10^6 (N_0(t))^{1/3} N^{1/3} (\frac{\Lambda \sigma}{M_o})^{1/3} \quad (4.18)$$

Where $N_0(t)$ is the number of rupture sub-faults at a time, $t$. The spreading factor in Equation (4.15) is computed as,

$$H_n = \left( \sum_i \frac{f^2}{1 + (f/f_{on}(t))^2} \right)^2 \left( \sum_i \frac{f^2}{1 + (f/f_{on}(t))^2} \right) \quad (4.19)$$

Where $f_0$ is the corner frequency at the end of the rupture.

The strong ground motion modeling must accommodate:

- Seismic wave radiation from a fault rupture,
- propagation through the crust, and
- Modifications by the site conditions.
4.2.1.4 Dynamic Corner Frequency Concept

In finite-fault modeling, ruptured area is time dependent. It is initially zero and is finally considered equal to the entire fault area. If the rupture stops at the end of the first subfault, the corner frequency is inversely proportional to the area of the first subfault. If the rupture stops at the end of the ninth subfault, the corner frequency is inversely proportional to the ruptured area, which is the area of N subfaults. Eventually, when the rupture stops at the end of the Nth subfault, the corner frequency is inversely proportional to the entire ruptured area. Thus, if corner frequency is inversely proportional to ruptured area, it follows that the corner frequency in finite-fault modeling can be considered as a function of time. Similarly, it also follows that corner frequency should be decreasing as the signal duration builds. Thus, at each moment of time the corner frequency depends on the cumulative ruptured area. The rupture begins with high corner frequencies and progresses to lower corner frequencies.

4.2.1.5 Pulsating Area

In an actual earthquake rupture, the slip may only be occurring on part of the fault at any time. “Self-healing” model proposed by Heaton (1990) considers that duration of slip at any location on the fault is short. In stochastic modeling, this is possible only when some part of the fault is actively pulsing at any time. In such a model, the areas that are actively pulsating contribute to the ground-motion radiation, but the other areas on the fault are passive. The passive cells will have no effect on the dynamic corner frequency. The active area will move along the fault as rupture progresses. By decreasing the pulsating area, the amplitudes at low frequencies decrease; thus, a narrow pulsating area on the fault results in lower amplitudes at longer periods and lower energy radiation. Variation of this parameter can be used to adjust relative amplitudes of low-frequency motion in finite-fault modeling. Note that the pulsating length cannot be less than the length of one subfault (Motazedian and Atkinson, 2005).

4.3 WAVEFORM SYNTHESIS USING F-K INTEGRATION

For the computation of synthetic accelerogram, impulsive source has been used as a first approximation for near-field effect. The Wavenumber integration method of Herrmann and Mandal (1986) is then followed.

The generation of synthetic seismograms for point sources in simply layered structures has made rapid advances in the past decade. Two approaches involving Laplace transform and Fourier transform are actively being pursued. The Laplace transform or Cagniard-de Hoop technique, usually referred to as the generalized ray method (Helmberger, 1968), constructs the solution by tracking individual seismic arrivals ray by ray from the source to receivers. This method is valid at higher frequencies and works well at predicting particular phases, but is poorly suited to models with many layers and larger distances when a complete seismogram is desired. The other approach involves expressing the solutions in terms of a double integral transformation over wavenumber and frequency (Hudson, 1969). The complete solution rather than individual rays is considered in such a full wave theory approach. This method can handle a larger number of plane layers, but requires considerable computational efforts especially at higher frequencies.
Suppose that an earthquake can be represented by a double-couple without moment source model with the symbols ‘n’ for the vector normal to the fault and ‘f’ for the direction of force as used by Haskell (1964). The Fourier transform of vertical, radial and tangential components of the displacement can be given as,

\[
\begin{align*}
    \mathbf{u}_z(r, \phi, o, \omega) &= ZSS[(f_1 n_1 - f_2 n_2) \cos 2\phi + (f_1 n_1 + f_2 n_1) \sin 2\phi] \\
    &+ ZDS[(f_1 n_3 + f_3 n_1) \cos \phi + (f_2 n_1 + f_3 n_2) \sin \phi] \\
    &+ ZDD[f_3 n_3]
\end{align*}
\]

\[(4.20)\]

\[
\begin{align*}
    \mathbf{u}_r(r, \phi, o, \omega) &= RSS[(f_1 n_1 - f_2 n_2) \cos 2\phi + (f_1 n_1 + f_2 n_1) \sin 2\phi] \\
    &+ RDS[(f_1 n_3 + f_3 n_1) \cos \phi + (f_2 n_1 + f_3 n_2) \sin \phi] \\
    &+ RDD[f_3 n_3]
\end{align*}
\]

\[(4.21)\]

\[
\begin{align*}
    \mathbf{u}_\theta(r, \phi, o, \omega) &= TSS[(f_2 n_1 + f_3 n_1) \cos 2\phi - (f_1 n_1 - f_2 n_2) \sin 2\phi] \\
    &+ TDS[(f_2 n_1 + f_3 n_1) \cos \phi - (f_1 n_1 + f_3 n_2) \sin \phi]
\end{align*}
\]

\[(4.22)\]

Where ZDD, ZDS, ZSS, RSS, RDS, RDD, TSS and TDS are referred as Green’s Functions. RSS and RDS in Equation (4.21) also include near-field terms. These terms decrease faster than the others, and therefore, are important only at short distances.

The inverse Fourier transform (4.20), (4.21) and (4.22) on multiplication of \(-\omega^2\), needs a convolution of the source spectra for the generation of acceleration time history of the vertical, radial and tangential components of ground motion as given below,

\[
\begin{align*}
    \mathbf{u}_{r, z, \phi}(r, \phi, 0, t) &= \int_{-\infty}^{\infty} S(\omega) u_{r, z, \phi}(r, \phi, 0, \omega) \exp(i\omega t) d\omega / 2\pi
\end{align*}
\]

\[(4.23)\]

Where \(S(\omega)\) is the Fourier spectra of the impulse source function as described by Herrmann (1979).

### 4.4 SPECTRAL FINITE ELEMENT METHOD

#### 4.4.1 Equation of Motion

Let us consider an earth model with volume \(\Omega\) and outer free surface \(\partial \Omega\). The displacement wave field introduced by an earthquake is \(\mathbf{u}(x, t)\), where \(x\) denotes the material points in the earth model and \(t\) time, is determined by the seismic wave equation

\[
\rho \frac{\partial^2 \mathbf{u}(x, t)}{\partial t^2} = \nabla \cdot \mathbf{T} + \mathbf{f}
\]

\[(4.24)\]

Where \(\rho\) denotes the distribution of mass density. The stress tensor \(\mathbf{T}\) is determined in terms of strain as \(\mathbf{T}_{ij} = C_{ijkl} \mathbf{e}_{kl}\), where \(C_{ijkl}\) being the fourth order elastic tensor and the strain tensor \(\mathbf{e}\) is associated with the displacement vector \(\mathbf{u}\) by the relation,

\[
\mathbf{e} = \frac{1}{2} \left[ \nabla \mathbf{u} + (\nabla \mathbf{u})^T \right]
\]

\[(4.25)\]
On the earth’s free surface $\partial \Omega$ the traction should vanish, i.e. on $\partial \Omega$
\[ \hat{n} \cdot T = 0 \]  \hspace{1cm} (4.26)

Where $\hat{n}$ denotes the unit outward normal on the surface. On solid-solid boundary both the traction and the displacement need to be continuous. In addition to the above boundary condition, the seismic wave equation must be solved subject to the following initial conditions,
\[ u(x,0) = 0, \quad \frac{\partial^2 u(x,t)}{\partial t^2} = 0 \]  \hspace{1cm} (4.27)

The force $f$ in Equation (4.24) represents the earthquake. In the case of a simple point source it may be written in terms of moment tensor $M$ as,
\[ f = -M \cdot \nabla \delta(x - x_s)S(t) \]  \hspace{1cm} (4.28)

Where the location of the point source is denoted by $x_s$, $\delta(x - x_s)$ denoting Dirac delta distribution located at $x_s$ and $S(t)$ is the source time function.

### 4.4.2 Weak Formulation

For computational purpose, in spectral finite element method we deal with an integral or weak formulation of the strong form of equation of motion. It is obtained by taking dot product of the momentum Equation (4.24) with an arbitrary test vector $w$, and integrating by parts over the volume $\Omega$ of the earth we get,
\[ \int_{\Omega} \rho \left( \frac{\partial^2 u}{\partial t^2} \right) d^3x = \int_{\Omega} w \cdot \nabla T d^3x + \int_{\Gamma} w \cdot f d^3x \]  \hspace{1cm} (4.29)

Equation (4.29) is equivalent to the strong formulation (4.24) because it holds for any test vector $w$. Integrating the first term of the right hand side and denoting boundary of $\Omega$ by $\Gamma$, we obtain:
\[ \int_{\Omega} \rho w \cdot \ddot{u} d^3x = \int_{\Gamma} w \nabla \cdot T d^3x - \int_{\Omega} \nabla w \cdot T d^3x + \int_{\Gamma} w \cdot f d^3x \]  \hspace{1cm} (4.30)

On the free surface the stress term becomes zero (i.e., the free surface boundary condition or Neumann condition). Thus the integral over $\Gamma$ vanishes. Thus we get the final form of the weak formulation as,
\[ \int_{\Omega} \rho w \left( \frac{\partial^2 u}{\partial t^2} \right) d^3x = -\int_{\Omega} \nabla w \cdot T d^3x + M \cdot \nabla w (x_s)S(t) \]  \hspace{1cm} (4.31)

The term on the left side gives rise to the mass matrix and the first term on the right side is related to the stiffness matrix. The 2nd term on the right side is related to the source term.

### 4.4.3 Formulation of Spectral Element Mesh and Mapping Function

We subdivide the model domain $\Omega$ into a number of non-overlapping elements $\Omega_e$, $e=1,2,\ldots,n_e$, such that $\Omega = \bigcup_{e}^{n_e} \Omega_e$. In SEM 2D the elements are generally quadrilateral ones whereas in SEM 3D the elements are restricted to the hexahedral shapes (deformed cube).
Figure 4.3: Two dimensional mesh of curved structures using size doubling in the middle layer (Bernhard Schuber, 2003).

Figure 4.4: A “cubed sphere” mesh of the globe.

Figure 4.5: Illustration of a three dimensional (Komatitsch and Tromp, 2002) cube (Muller-Hannemann, 2000)

Each element domain $\Omega_e$ in 3D is mapped onto a reference cube $\Lambda \otimes \Lambda \otimes \Lambda$, where as in 2D it is mapped onto a reference square $\Lambda \otimes \Lambda$ and in 1D onto a standard interval $\Lambda$, where $\Lambda$ is an one dimensional standard interval $[-1,1]$.

$$x(\xi) = \sum_{a=1}^{M} x_a N_a(\xi)$$  \hspace{1cm} (4.32)

Points $x = (x, y, z)$ within each hexahedral element $\Omega_e$ may be uniquely related to points $\xi = (\xi, \eta, \zeta)$, $-1 \leq \xi, \eta, \zeta \leq 1$, in a reference cube based upon the invertible mapping. $a=1,2,\ldots,M$ anchors $x(\xi_a, \eta_a, \zeta_a)$ and shape functions $N_a(\xi)$ define the geometry of an element $\Omega_e$. In general, the shape functions are typically $N_a(\xi)$ products of Lagrange polynomials of degree either $N=1$ (for straight edge elements) or
N=2 (for curved elements) which are defined on the anchor nodes of the elements, \( n_d \) being the number of dimensions of the considered problem. The \( n+1 \) Lagrange polynomials of degree \( n \) are defined in terms of \( n+1 \) control points \(-1 \leq \xi_{\alpha} \leq 1, \alpha=0,1,\ldots,n \) by

\[
\ell_{\alpha}^{n}(\xi) = \frac{(\xi - \xi_{\alpha-1}) \cdots (\xi - \xi_{0})(\xi - \xi_{1}) \cdots (\xi - \xi_{n})}{(\xi_{\alpha} - \xi_{\alpha-1}) \cdots (\xi_{\alpha} - \xi_{0})(\xi_{\alpha} - \xi_{1}) \cdots (\xi_{\alpha} - \xi_{n})}
\]  

(4.33)

The two Lagrange polynomials of degree 1 with two control points \( \xi = -1 \) and \( \xi = 1 \) are,

\[
\ell_{0}^{1}(\xi) = \frac{1}{2}(1 - \xi), \quad \ell_{1}^{1}(\xi) = \frac{1}{2}(1 + \xi)
\]

(4.34)

And the three Lagrange polynomials of degree 2 with three control points \( \xi = -1, \xi = 0, \xi = 1 \) are,

\[
\ell_{0}^{2}(\xi) = \frac{1}{2} \xi (\xi - 1), \quad \ell_{1}^{2}(\xi) = 1 - \xi^{2}, \quad \ell_{2}^{2}(\xi) = \frac{1}{2} \xi (\xi + 1)
\]

(4.35)

Thus the shape functions \( N_{\alpha}(\xi) \) are defined on the anchor nodes \( x_{\alpha} \) inside the element. For one dimensional element two anchor nodes per element are sufficient to define a linear structure, one node at each point. Using two points per dimension two dimensional elements would result in \( 2^{2} = 4 \) anchor nodes whereas the three dimensional elements would result in \( 2^{3} = 8 \) anchor nodes. In general, the geometry of hexahedral elements may be controlled by \( M = 8, 20, \) or \( 27 \) anchor nodes. The shape functions of three dimensional elements are triple products of the corresponding Lagrange polynomials. The shape functions for 2-D four node element based on Lagrange polynomials of degree \( N = 1 \) is as follows:

\[
N_{1}(\xi, \eta) = \ell_{0}^{1}(\xi) \ell_{0}^{1}(\eta) \quad N_{2}(\xi, \eta) = \ell_{1}^{1}(\xi) \ell_{0}^{1}(\eta) \quad N_{3}(\xi, \eta) = \ell_{0}^{1}(\xi) \ell_{1}^{1}(\eta) \quad N_{4}(\xi, \eta) = \ell_{1}^{1}(\xi) \ell_{1}^{1}(\eta)
\]

(4.36)

Figure 4.6: Mapping of 2D elements on the reference square \( \Omega_{2} = [-1,1] \otimes [-1,1] \). Left: straight element with four anchor nodes. Right: curved element with nine anchor nodes.
The weak form (4.31) involves volume integrals over elements $\Omega_e$. Using the mapping (4.32), an element of volume $d^3x = dx dy dz$ within a given element $\Omega_e$ is related to an element of volume $d^3\xi = d\xi d\eta d\zeta$ in the reference cube by,

$$d^3x = dx dy dz = \mathcal{J} d^3\xi$$  \hspace{0.5cm} (4.37)

Where the Jacobian of the mapping (4.32) is given by,

$$\mathcal{J} = \frac{\partial (x, y, z)}{\partial (\xi, \eta, \zeta)}$$  \hspace{0.5cm} (4.38)

### 4.4.4 Interpolation functions on the elements

In a traditional FEM, low-degree polynomials are also used as basis functions for the representation of functions on an element. In a SEM, a higher-degree Lagrange interpolant is used to express functions on the elements, and the control points $\xi_\alpha, \alpha = 0, 1, \ldots, n$, needed in the definition of the Lagrange polynomials of degree $n$ (4.33) are chosen to be $n+1$ Gauss-Lobatto-Legendre (GLL) points, which are the roots of the equation,

$$\left(1 - \xi^2\right)P_n'(\xi) = 0$$  \hspace{0.5cm} (4.39a)

Where $P_n'$ denotes the derivative of the Legendre polynomial of degree $n$. Equation (4.39) implies that +1 and -1 are always GLL points, irregardless of the degree $n$. Therefore, some GLL points always lie exactly on the boundaries of the elements. An SEM typically uses Lagrange polynomials (4.33) of degree 4-10 for the interpolation of functions. As an example, 5 Lagrange polynomials of degree 4 as had shown in Figures 4.6, 4.7a, 4.7b illustrate the distribution of the associated GLL points on the face of a hexahedral element.
Figure 4.7b: Left: The five Lagrange interpolants of degree $N=4$ on the reference segment $[-1,1]$. The $N+1=5$ Gauss-Lobatto-Legendre (GLL) points, can be distinguished along the horizontal axes. All Lagrange polynomials are, by definition, equal to 1 or 0 at each of the GLL points. When Lagrange polynomials of degree $n$ are used to discretize functions on an element, each 3D spectral element contains a grid of $(n+1)^3$ GLL points, and each 2D face of an element contains a grid of $(n+1)^2$ GLL points, as illustrated here for the degree 4 polynomials shown on the left.

In the weak form of the wave Equation (4.31), we expand functions $f$, e.g., a component of the displacement field $s$ or the test vector $w$, in terms of degree-$n$ Lagrange polynomials (4.33) with GLL control points determined by (4.39a):

$$f(x(\xi, \eta, \zeta)) = \sum_{\alpha=0}^{n} \sum_{\beta=0}^{n} \sum_{\gamma=0}^{n} f^{\alpha\beta\gamma} L^\alpha_\alpha(\eta) L^\beta_\beta(\zeta) L^\gamma_\gamma(\zeta)$$

(4.39b)

Where $f^{\alpha\beta\gamma} = f(x(x(\xi, \eta, \zeta))$ denotes the value of function $f$ at the GLL points $x(\xi, \eta, \zeta)$. In a SEM, polynomials of degree 4 or 5 provide the best tradeoff between accuracy and time-integration stability.

The weak form (4.31) involves gradients of the displacement field $u$ and the test vector $w$. Using the polynomial representation (4.39b), such gradients may be expressed as

$$\nabla f(x(\xi, \eta, \zeta)) = \sum_{i=1}^{n} \sum_{\alpha=0}^{n} \sum_{\beta=0}^{n} \sum_{\gamma=0}^{n} \nabla \tilde{u}^\alpha_\alpha(\eta) \nabla \tilde{u}^\beta_\beta(\zeta) \nabla \tilde{u}^\gamma_\gamma(\zeta) +$$

$$+ i_{\alpha}(\xi) j_{\beta}(\eta) k_{\gamma}(\zeta)$$

(4.39c)

Where a prime denotes the derivative of a Lagrange polynomial, and where we have used the index notation $\delta^i_i = \delta$, $i=1,2,3$, and $x_1 = x, y_1 = y, z_1 = z$.

4.4.5 Integration over elements

The next step is to evaluate the integrals in the weak form (4.31) at the elemental level. In a SEM, a Gauss-Lobatto-Legendre integration rule is used for this purpose, because this leads to a diagonal mass
matrix when used in conjunction with GLL interpolation points. Based on this approach, integrations over elements $\Omega_s$ may be approximated as

$$
\int_{\Omega_s} \rho \mathbf{w} \cdot \frac{\partial}{\partial t} \mathbf{x} \mathbf{d}^2 = \int_{\xi}^{\xi_f} \int_{\zeta}^{\zeta_f} \rho(x(\xi),w(\xi)) \frac{\partial}{\partial t} s(x(\xi),\mathbf{w}(\xi)) \mathbf{3}(\xi) d\xi d\zeta
$$

$$
= \sum_{i=0}^{n} \sum_{j=0}^{m} \sum_{k=0}^{m} \omega_i^{a} \omega_j^{b} \omega_k^{c} \mathbf{3}^{a b c} \rho^{a b c} \sum_{i=1}^{3} \sum_{j=0}^{m} \sum_{k=0}^{m} \omega_i^{a} \omega_j^{b} \omega_k^{c}
$$

(4.39d)

Where a dot denotes differentiation with respect to time.

After determining the mass matrix and stiffness matrix the governing ordinary differential equation with time dependent can be written a

$$
M \ddot{\mathbf{U}} = -K \mathbf{U} + \mathbf{F}
$$

(4.39e)

Where $M$ denotes the global mass matrix, $K$ the global stiffness matrix, $\mathbf{U}$ the displacement vector, and $\mathbf{F}$ the global source vector.

Taking the full advantage of the fact that the global mass matrix is diagonal, time discritization of the second order ordinary differential equation (4.39e) can be done by a classical explicit second-order finite difference method scheme, which is a particular case of the more general Newmark scheme. The scheme is conditionally stable is given by the predictor-corrector phase (predictor phase at the beginning of each time step and corrector phase at the end of each time step) as:

**Predictor:**

$$
\dot{a}^{n+1} = \dot{a}^n + \ddot{a}^n \Delta t + \frac{1}{2} \dddot{a}^n (\Delta t)^2
$$

$$
\dot{v}^{n+1} = \dot{v}^n + \ddot{v}^n \Delta t
$$

$$
\dot{a}^{n+1} = 0
$$

**Corrector:**

$$
\dot{a}^{n+1} = (M)^{-1}(B+T+F^{n+1})
$$

$$
\dot{v}^{n+1} = \dot{v}^{n+1} + \ddot{v}^{n+1} \Delta t
$$

$$
\dot{a}^{n+1} = \dot{a}^{n+1}
$$

(4.39f)

4.4.6 SEM simulation for Earthquakes in the Sikkim Himalaya

The Sikkim Himalaya is situated in the eastern edge of the rupture zone of the 1934 Great Nepal-Bihar earthquake of $M_w$ 8.1. The earthquake is attributed to a strike-slip fault striking at N285°E and dipping at 6°S at the focal depth of 20 km (Hough and Bilham, 2008) located 150 km away from Gangtok. The estimated maximum earthquake in the Sikkim region is of magnitude $M_w$ 8.3 (Bilham et al., 2001;
Thingbaijam and Nath, 2008). We simulate both the 1934 Nepal-Bihar Earthquake and the MCE of $M_w 8.3$ in the 3D integrated Sikkim Himalaya model. For constructing the source parameters the Havard CMT solution is used as follows:

\[
M_{rr} = -M_0 \left( \sin \delta \cos \lambda \sin 2\phi + \sin 2\delta \sin \lambda \sin^2 \phi \right) 
\]

\[
M_{rp} = M_0 \left( \sin \delta \cos \lambda \sin 2\phi - \sin 2\delta \sin \lambda \cos^2 \phi \right) 
\]

\[
M_{rr} = -M_{rr} + M_{rp} 
\]

\[
M_{rp} = -M_0 \left( \sin \delta \cos \lambda \cos 2\phi - 0.5 \sin 2\delta \sin \lambda \sin 2\phi \right) 
\]

\[
M_{rt} = M_0 \left( \cos \delta \cos \lambda \cos \phi - 0.5 \cos 2\delta \sin \lambda \sin \phi \right) 
\]

\[
M_{rp} = M_0 \left( \cos \delta \cos \lambda \sin \phi - \cos 2\delta \sin \lambda \cos \phi \right) 
\]

Where $\phi$ is the Strike, $\delta$ is the Dip and $\lambda$ is Rake of the earthquake source.

The simulation parameters are enlisted in Table 4.1.

**Table 4.1:** Source parameters for 1934 Bihar-Nepal earthquake and MCE of $M_w 8.3$

<table>
<thead>
<tr>
<th>Parameters</th>
<th>1934 Bihar-Nepal Earthquake</th>
<th>MCE of $M_w 8.3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latitude</td>
<td>25.55°N</td>
<td>27.22°N</td>
</tr>
<tr>
<td>Longitude</td>
<td>87.09°E</td>
<td>88.64°E</td>
</tr>
<tr>
<td>Moment magnitude</td>
<td>8.1</td>
<td>8.3</td>
</tr>
<tr>
<td>Strike</td>
<td>291°</td>
<td>287°</td>
</tr>
<tr>
<td>Dip</td>
<td>34°</td>
<td>27°</td>
</tr>
<tr>
<td>Rake</td>
<td>64°</td>
<td>126°</td>
</tr>
<tr>
<td>Depth</td>
<td>20 km</td>
<td>19 km</td>
</tr>
<tr>
<td>$M_{rr}$ component of moment tensor</td>
<td>$1.48*10^{28}$ dyne-cm</td>
<td>$2.322*10^{28}$ dyne-cm</td>
</tr>
<tr>
<td>$M_{rt}$ component of moment tensor</td>
<td>-$0.9997*10^{28}$ dyne-cm</td>
<td>-$2.653*10^{28}$ dyne-cm</td>
</tr>
<tr>
<td>$M_{pp}$ component of moment tensor</td>
<td>-$0.4819*10^{28}$ dyne-cm</td>
<td>$0.331*10^{28}$ dyne-cm</td>
</tr>
<tr>
<td>$M_{rp}$ component of moment tensor</td>
<td>$0.041017*10^{28}$ dyne-cm</td>
<td>-$0.628968*10^{28}$ dyne-cm</td>
</tr>
<tr>
<td>$M_{rp}$ component of moment tensor</td>
<td>-0.817787*10^{28} dyne-cm</td>
<td>$1.283686*10^{28}$ dyne-cm</td>
</tr>
<tr>
<td>$M_{lp}$ component of moment tensor</td>
<td>$0.819621*10^{28}$ dyne-cm</td>
<td>-$0.13564*10^{28}$ dyne-cm</td>
</tr>
</tbody>
</table>
The SPECFEM3D version 2.0.0 software package available from Computation Infrastructure for Geodynamics is used. In each spectral element we use a poly-nomial degree of N = 4, thus each element contains \((N + 1)^3 = 125\) Gauss–Lobatto–Legendre integration points. The simulations are carried out on both the SUN SOLARIS and LINUX platforms. The simulation is performed at 13 stations of the Sikkim region. The name of the stations and the average distance from the source of MCE \(M_w 8.3\) are given in Table 4.2. Snapshots of the simulated velocity wave field are displayed in Figures 4.8 and 4.9 (Bihar-Nepal earthquake at various time steps are shown in the Figure 4.8 and MCE of \(M_w 8.3\) at various time steps are shown in the Figure 4.9). The synthetic seismograms are computed at the surface level at various stations in Sikkim for MCE of \(M_w 8.3\) as shown in Figure 4.10. The effect of topography on wave propagation in the Sikkim Himalaya is depicted in Figure 4.11.

**Table 4.2:** Average distances (km) of Sikkim stations from the source of MCE \(M_w 8.3\)

<table>
<thead>
<tr>
<th>Name of the stations</th>
<th>Average distance (km) from the source of MCE (M_w 8.3) nucleating from the source of 14th February, 2006 Sikkim earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gangtok</td>
<td>11.470</td>
</tr>
<tr>
<td>Chungthang</td>
<td>42.103</td>
</tr>
<tr>
<td>Lachen</td>
<td>55.367</td>
</tr>
<tr>
<td>Lingza</td>
<td>43.782</td>
</tr>
<tr>
<td>Mangan</td>
<td>31.832</td>
</tr>
<tr>
<td>Gezing</td>
<td>39.619</td>
</tr>
<tr>
<td>Singtam</td>
<td>15.021</td>
</tr>
<tr>
<td>Padamchen</td>
<td>14.583</td>
</tr>
<tr>
<td>Aritar</td>
<td>4.854</td>
</tr>
<tr>
<td>Melli</td>
<td>23.050</td>
</tr>
<tr>
<td>Jorthang</td>
<td>34.568</td>
</tr>
<tr>
<td>Uttare</td>
<td>45.571</td>
</tr>
<tr>
<td>Namchi</td>
<td>27.485</td>
</tr>
</tbody>
</table>
Figure 4.8: Wave propagation snap shots for Bihar-Nepal Earthquake of $M_w 8.1$ at different elapse time.
Figure 4.9: Wave propagation for MCE $M_w$ 8.3 Sikkim Earthquake of February 14, 2006 triggered at 27.22° N, 88.64° E at different elapse time step.
Figure 4.10: Seismogram at various stations for MCE $M_w$ 8.3 Sikkim earthquake (a) Aritar, (b) Chungthang, (c) Gangtok, (d) Gezing, (e) Jorthang, (f) Lingza, (g) Mangnan, and (h) Sinhtam.
4.5 FACTORS AFFECTING EARTHQUAKE STRONG GROUND MOTION

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

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

Recorded strong ground motion in the near-field incorporates all the heterogeneities, complex arrivals and the high frequency content of the source process. Patches on the fault plane with higher slips are called asperities. Asperities are highly stressed regions surrounded by weak or zero stress zones. The fault plane is then composed of patches of high stress and low or zero stress, which leads no a non-uniform stress drop during an earthquake event. Barriers are identified as those portions of the fault plane that do not rupture. Simple source models assume that main fault rupture parameters (rupture velocity, rise time and stress parameters) are homogenous and coherent over the plane of dislocation.

Seismic Moment, Stress Drop, Effective Stress and Corner Frequency are the main parameters of earthquake source that influences strong ground motion characteristics. A short review of these parameters is provided below.

Seismic Moment, $M_0$, is the most recognized measure of the earthquake size given by the multiplication of the shear modulus (Lame’s constant) of the medium, the average total dislocation (i.e., mean fault offset or slip) and the area of the dislocation surface (i.e., fault rupture surface). The seismic moment, $M_0$, is generally regarded as the best available single number to describe the size of an earthquake and can be estimated from the low frequency asymptote of the Fourier transform of the displacement seismogram. The moment magnitude ($M_w$) is derived from seismic moment on the basis of the following equation (Kanamori, 1977)

$$M_w = (2/3) \log M_0 - 10.73$$  \hspace{1cm} (4.46)

Effective Stress is the difference between the initial static stress and frictional stress in existence during the rupture process.

Corner Frequency is the frequency where the high and low frequency trends of the Fourier Amplitude Spectrum are related to the inverse of the rise time (rate of growth in dislocation or roughly, time duration of rupture). By measuring the corner frequency from the Fourier Amplitude Spectrum the apparent duration of faulting at the source and hence the fault dimension can be estimated. A large magnitude earthquake will generally have a large fault dimension and hence a small corner frequency, implying a longer rupture and also strong motion duration.

4.5.2 Subduction Zone and Shallow Crustal Earthquakes

The collision of tectonic plates in subduction zones causes large and deep earthquakes. Ground motion data from subduction zone earthquakes are associated with slower rate of attenuation compared to those from shallow crustal earthquakes. Analysis of ground motion data also indicates that the response spectral shapes obtained from subduction zone earthquakes have smaller amplitudes in the log-period range than the response spectral shapes from shallow crustal earthquakes.

4.5.3 Effects of Distance

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

$$\exp\left[-\left(\pi f / Q_v\right)X\right]$$

4.6 GROUND MOTION PREDICTION EQUATIONS

An evaluation of seismic hazard, whether deterministic (scenario based) or probabilistic, requires an estimate of the expected ground motion at the site of interest. The most common means of estimating this ground motion in engineering practices, including probabilistic seismic hazard analysis, is the use of attenuation relationship. An attenuation relation is a mathematical based expression that relates a specific strong motion parameter of ground shaking to one or more seismological parameters of an earthquake. These seismological parameters quantitatively characterize earthquake source, wave propagation path between the source and the site, and the soil and geological profile beneath the site. Attenuation relationships have been established for many ground motion parameters including –

- Peak Ground Acceleration (PGA)
- Peak Ground Velocity (PGV)
- Peak Ground Displacement (PGD)
- Spectral Quantities.

Attenuation relationships are developed, by statistical evaluation of a large set of ground motion data, for different regions and fault types (e.g., interplate versus intraplate, strike-slip versus subduction). Various investigators have proposed attenuation relationships for PGA. All the commonly used attenuation relationships (Figure 4.12) assume that the ground motion values are log-normally distributed (i.e., the logarithm of the value is normally distributed).

![Figure 4.12: Example of ground motion prediction equations (after Nath et al., 2005 and 2009; Pal et al., 2008).](image)
4.7 GROUND MOTION PREDICTION EQUATIONS (GMPE) – A REVIEW

Typically, the ground motion prediction equation are formulated as follows,

\[ \ln \hat{Y} = f(M_w, R, \sum [Source], \sum [Site]) \]  

(4.47)

Where \( \ln \hat{Y} \) is the natural logarithm of the ground motion parameter of interest, \( M_w \) is the magnitude of the earthquake defined in terms of moment magnitude, \( R \) is a measure of distance representing the path of seismic energy from the earthquake source to the site of interest, \( \sum [Source] \) forms the set of the parameters relating to the earthquake source such as type of faulting, rupture width and depth, and fault dip, and \( \sum [Site] \) comprises the parameters relating to the site of interest such as average shear wave velocity, geologic characteristics, or depth to bedrock (Kramer, 1996; Abrahamson and Shedlock, 1997; Douglas, 2003; Abrahamson et al., 2008).

The standard response variables in ground motion prediction relations are Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and 5% damped elastic pseudo-response spectral acceleration (Sa). PGA is the absolute maximum value of ground acceleration and PGV is similarly defined for velocity. PGA represents a short-period parameter related to horizontal force from earthquakes on one-to-two story buildings. It is typically expressed in \( g \) i.e., acceleration due to gravity (1 g = 981 cm/s²). On the other hand, the maximum response in terms of Sa comprises the response spectra for single degree of freedom structure to a particular input motion as a function of natural period and damping ratio generally assumed 5% for structural material (Kramer, 1996).

Source parameters capture the effects of earthquake size, defined by magnitude \( M_w \) and the characteristics of the fault rupture. Moment magnitude \( (M_w) \) is a preferred measure since it is based on seismic moment, which connects to the characteristics of the fault rupture (Hanks and Kanamori, 1979). The parameters defining finite source models include fault-rupture width, fault-rupture length, depth-to-top of the fault rupture \( (Z_{TOR}) \), seismogenic depth \( (H_{SEIS}) \), and average dip of the fault-rupture plane \( (\delta) \) (Abrahamson and Shedlock, 1997). Most of the GMPEs developed for tectonically active regions have faulting types incorporated either via flag variables or in terms of rake angle of the fault movement \( (\lambda) \). Three broad categories of faulting types are usually defined – normal, strike-slip and reverse. These are respectively associated with lower, intermediate and highest ground motions for a specific source to a site (Bommer et al., 2003; Ambraseys et al., 2005).

Figure 4.13: Vertical cross-section through a fault rupture plane illustrating the different distance metrics described in the text (after Abrahamson and Shedlock, 1997; Scasserra et al., 2008).
The path attributes, defining the propagation of seismic waveform for source-to-site, are incorporated by distance metrics (Abrahamson et al., 2008; Boore and Atkinson, 2008). Older GMPEs and many of those developed for subduction zones and stable tectonic regions usually employ epicentral distance (R_{EPIC}) or hypocentral distance (R_{HYPO}). Recent ones have different distance measures in order to accommodate the finite source model considerations as depicted in Figure 4.13. These include the distance of site to the closest point on the rupture plane is referred to as rupture distance (R_{RUP}) and the horizontal distance to the surface projection of the rupture referred to as Joyner-Boore distance, (R_{JB}). R_{RUP} approximately equals to hypocentral distance while R_{JB} equals to epicentral distance for smaller events with magnitude M_W<6.5. Campbell (1997) used seismogenic distance (R_{SEIS}) defined as the distance from the site to the closest point on the rupture plane at or below the seismogenic depth in order to segregate non-seismogenic fault rupture. Meghawati and Pan (2010) employed distance from the site to the centre of the fault rupture plane (R_{CF}). Abrahamson and Silva (2008) and Chiou and Youngs (2008) introduced site-coordinate R_x defined by the horizontal projection to the top edge of the rupture perpendicular to the strike of the rupture plane and is negative when station is located opposite to the direction of the fault dip. The distance metric is used to explicitly incorporate the hanging wall effect, which entails higher ground motion levels at the sites located on the hanging wall compared to the footwall of the fault. In case of subduction zones, two types of earthquakes are generally identified – interface and intraslab (Youngs et al., 1997). The former ascribes to shallow angled events occurring at the interface between subduction and overriding plates. On the other hand, the latter is described as typically high-angled earthquakes occurring within the subducting plate.

Site attributes strongly influence ground motions; soft soil sites are more likely to suffer from higher ground motion amplifications compared to the rock sites. In the simplest form, GMPEs use flag variables to indicate soil or rock sites (e.g., Abrahamson and Silva, 1997; Campbell, 1997; Sharma et al., 2009). In the recent developments, the incorporation has been mostly based on V_{S30}, averaged shear wave velocity of 30 m soil column (Boore et al., 1997; Powers et al., 2008). However, due to general difficulty in evaluating depth to seismic bedrock at site, auxiliary measurements defining the depths at which shear wave velocity is approximately 1.0 km/s or 2.5 km/s are used to define the change from rock to hard rock (Abrahamson and Silva, 2008; Chiou and Youngs, 2008; Campbell and Bozorgnia, 2008). The site-response phenomenon is also known to be non-linear in effect to its relations with the level of ground motions at the bedrock level and cyclic loads on the soil (Kramer, 1996; Abrahamson and Silva, 1997). In the Next Generation Attenuation (NGA) models, the nonlinear part of site amplification has been constrained by analytical models, use of amplification factors, or by the employed data (Abrahamson et al., 2008). Some Typical Global Attenuation Relationships are given in Table 4.3.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Data Source</th>
<th>Relationship</th>
<th>Reference</th>
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<tbody>
<tr>
<td>1.</td>
<td>San Fernando earthquake</td>
<td>log PGA = 190 / R^{1.63} Where R is the distance to the source in km.</td>
<td>Donovan (1973)</td>
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<tr>
<td></td>
<td>(February 9, 1971)</td>
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<td>2.</td>
<td>California earthquake</td>
<td>PGA = y_o/(1+ (R' / h)^2) Where log y_o = – (b+3) + 0.81M – 0.027M^2 and b</td>
<td>Blume (1965)</td>
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<tr>
<td></td>
<td></td>
<td>is a site factor,</td>
<td></td>
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<td>Data Source</td>
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<td>3.</td>
<td>California and Japanese earthquakes</td>
<td>( PGA = \frac{0.0051}{\sqrt{T_G}} \times 10^{(0.61M – p\log R + 0.167 – 1.83 / R)} ) Where ( M = ) Earthquake Magnitude, ( R = ) Epicentral Distance, ( T_G = ) Time period.</td>
<td>Kanai (1966)</td>
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<td>4.</td>
<td>Cloud (1963)</td>
<td>( PGA = 0.0069e^{1.64M} / (1.1e^{1.1M} + R^2) ) Where ( M = ) Earthquake magnitude, ( R = ) the distance to the source in km.</td>
<td>Milne and Davenport (1969)</td>
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<td>5.</td>
<td>Cloud (1963)</td>
<td>( PGA = 1.254e^{0.8M} / (R + 25)^2 ) Where ( M = ) Earthquake magnitude, ( R = ) the distance to the source in km.</td>
<td>Esteva (1970)</td>
</tr>
<tr>
<td>6.</td>
<td>U.S.C. and G.S.</td>
<td>( \log PGA = (6.5 – 2 \log(R' + 80)) / 981 ) Where ( R' = ) Epicentral Distance in km.</td>
<td>Cloud and Perez (1971)</td>
</tr>
<tr>
<td>7.</td>
<td>303 Instrumental Values</td>
<td>( PGA = 1.325e^{0.67M} / (R + 25)^{1.6} ) Where ( M = ) Earthquake magnitude, ( R = ) the distance to the source in km.</td>
<td>Donovan (1973)</td>
</tr>
<tr>
<td>8.</td>
<td>Western U.S. records</td>
<td>( PGA = 0.0193e^{0.8M} / (R^2 + 400) ) Where ( M = ) Earthquake magnitude, ( R = ) the distance to the source in km.</td>
<td>Donovan (1973)</td>
</tr>
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<td>9.</td>
<td>U.S., Japan</td>
<td>( PGA = 1.35e^{0.58M} / (R + 25)^{1.52} ) Where ( M = ) Earthquake magnitude, ( R = ) the distance to the source in km.</td>
<td>Donovan (1973)</td>
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<tr>
<td>10.</td>
<td>Western U.S. records, USSR and Iran</td>
<td>( \text{In } PGA = – 3.99 + 1.28M – 1.75 \ln [R + 0.147e^{0.732M}] ) Where ( M = ) the surface wave magnitude for ( M ) greater than or equal to 6, or it is the local magnitude for ( M ) less than 6, ( R = ) fault distance.</td>
<td>Campbell (1981)</td>
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<td>Sl. No.</td>
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<td>11.</td>
<td>Western U.S. records and worldwide</td>
<td>$\log PGA = -1.02 + 0.249M - \log \sqrt{R^2 + 7.3^2} - 0.00255\sqrt{R^2 + 7.3^2}$&lt;br&gt;Where PGA is peak horizontal acceleration in g, M is moment magnitude, R is the closest distance to the surface projection of the fault rupture in km.</td>
<td>Joyner and Boore (1981)</td>
</tr>
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<td>12.</td>
<td>Western U.S. records and worldwide</td>
<td>$\log PGA = 0.49 + 0.23(M - 6) - \log \sqrt{R^2 + 8^2} - 0.0027\sqrt{R^2 + 8^2}$&lt;br&gt;Where, M is moment magnitude, R is the closest distance to the surface projection of the fault rupture in km.</td>
<td>Joyner and Boore (1982)</td>
</tr>
<tr>
<td>13.</td>
<td>Western U.S. records</td>
<td>$\ln PGA = \ln \alpha(M) - \beta(M) \ln(R+20)$&lt;br&gt;Where M is the surface wave magnitude for M greater than or equal to 6, or it is the local magnitude for smaller M, R is the closest distance to source for M greater than 6 and hypocentral distance for M smaller than 6, $\alpha(M)$ and $\beta(M)$ are magnitude-dependent coefficients.</td>
<td>Idriss (1985)</td>
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<td>14.</td>
<td>Italian records</td>
<td>$\ln PGA = -1.562 + 0.306M - \log \sqrt{R^2 + 5.8^2} + 0.169S$&lt;br&gt;Where M is Earthquake Magnitude, $R$ is the closest distance to the surface projection of the fault rupture in kilometers, S is 1.0 for soft sites or 0.0 for rock.</td>
<td>Sabetta and Pugliese (1987)</td>
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<td>15.</td>
<td>Western U. S. and worldwide (soil sites)</td>
<td>For M less than 6.5,&lt;br&gt;$\ln PGA = -2.611 + 1.1M - 1.75 \ln[R + 0.822e^{0.418M}]$&lt;br&gt;For M greater than or equal to 6.5,&lt;br&gt;$\ln PGA = -2.611 + 1.1M - 1.75 \ln[R + 0.316e^{0.629M}]$&lt;br&gt;Where M is Earthquake Magnitude, $R$ is the closest distance to the surface projection of the fault rupture in kilometers.</td>
<td>Sadigh et al. (1986)</td>
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<tr>
<td>Sl. No.</td>
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| 16.    | Western U. S. and worldwide (rock sites) | For $M$ less than 6.5,  
\[\ln \text{PGA} = -1.406 + 1.1M - 2.05 \ln (R + 1.353e^{0.406M})\]  
For $M$ greater than or equal to 6.5,  
\[\ln \text{PGA} = -1.406 + 1.1M - 2.05 \ln (R + 0.579e^{0.537M})\]  
Where, $M$ is Earthquake Magnitude,  
$R$ is the closest distance to the surface projection of the fault rupture in kilometers. | Sadigh et al. (1986) |
| 17.    | Worldwide earthquakes | $\ln \text{PGA} = -3.512 + 0.904M - 1.328$  
$\ln \sqrt{R^2 + [0.149e^{0.647M}]^2} + [1.125 - 0.112 \ln R - 0.0957M] F + [0.440 - 0.171 \ln R]S_{sr} + [0.405 - 0.222 \ln R]S_{hr}$  
Where $F = 0$ for strike-slip and normal fault earthquakes and 1 for reverse, reverse-oblique, and thrust fault earthquakes,  
$S_{sr} = 1$ for soft rock and 0 for hard rock and alluvium,  
$S_{hr} = 1$ for hard rock and 0 for soft rock and alluvium,  
$R$ = fault distance,  
$M$ = Earthquake Magnitude. | Campbell and Bozorgnia (1994) |
| 18.    | Western North American | $\ln \text{PGA} = b + 0.527(M - 6.0) - 0.778$  
$\ln \sqrt{R^2 + (5.570)^2} - 0.371 \ln \frac{V_s}{1396}$  
Where $b = 0.313$ for strike-slip earthquakes,  
$-0.117$ for reverse-slip earthquakes, and $-0.242$ if mechanism is not specified,  
$V_s$ is the average shear wave velocity of the soil in (m/sec) over the upper 30 meters,  
The equation can be used for magnitudes ($M$) of 5.5 to 7.5 and for distances not greater than 80 km,  
$R$ is fault distance. | Boore et al. (1997) |
4.7.1 Next Generation Attenuation Models

The NGA equations were developed for shallow crustal earthquakes in active tectonic region of Western United States. However, these new equations have been purported to be valid in other regions of similar tectonics. The models aimed at the prediction of different ground motion parameters namely, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), Peak Ground Displacement (PGD), and 5% damped elastic pseudo-absolute response spectral acceleration (PSA) for 0-10 sec. Abrahamson et al. (2008) compared five NGA equations developed by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008) on the datasets used, adopted model parameters and constraints, and the predictions of the ground motions. The difference was pointed out in the use/discard of aftershocks data, incorporation of rupture characteristics (i.e., buried or not), modeling of soil effects, treatment of the magnitude dependence and nonlinear site response. Owing to the data employed, NGA models has been normally considered valid for shallow continental earthquakes in shallow active tectonic regimes with magnitude $M_w > 4.0$, $M_w \leq 8.5$ for strike-slip faulting, $M_w \leq 8.0$ for reverse faulting, and $M_w \leq 7.5-8.0$ for normal faulting. Applicability of NGA has been tested and found reasonably suitable in several regions across the globe, e.g., Shoja-Taheri et al. (2010) in the Iranian Plateau, Stafford et al. (2008) in the Euro-Mediterranean region, and Campbell and Bozorgnia (2006) in Europe. Scasserra et al. (2008) reported the necessity of minor adjustments of the models for Italy. Akkar and Bommer (2007) also observed that the systematic differences between the NGA models and those from Europe to be insignificant. Conformity of ground motion characteristics across tectonically active regions of shallow crustal seismicity has been suggested.

4.7.2 Developments in India

Non-availability of data has been a major reason for abated developments of GMPEs in India. Nonetheless, several models have been developed for earthquakes occurring in the Himalayas, viz. Singh et al. (1996), Sharma (1998), Jain et al. (2000), Nath et al. (2005), Sharma and Bungum (2006) and Sharma et al. (2009). The model developed by Sharma et al. (2009) addresses horizontal response spectra for 5% damping for two different site categories: rock and soil, and two faulting types: strike-slip and reverse. They used combined data for shallow earthquakes in the Himalayas and Zagros regions to achieve wider magnitude range coverage as well as to overcome lack of near-field data in the Himalayas on the premises that seismotectonics of two regions have considerable similarity. However, the Himalayas have low angled thrust regions unlike Zagros. Moreover, the data from the Zagros region is more than that from the Himalayas making the model less oriented to the Himalayas. Nonetheless, the authors asserted that similarity in general faulting types has been considered and the predictions of new equation do not differ from the previous ones while introducing lower standard deviations. Nath et al. (2005) provided a region specific ground model prediction equation for the Sikkim Himalayas. They employed recordings of small to moderate earthquake of magnitude $3.0 \leq M_w \leq 5.6$ and simulated recordings for earthquakes with magnitude $6.0 \leq M_w \leq 8.3$. In the intraplate regions, GMPE has been mostly developed using synthetic database owing to lack of sufficient strong ground motion recordings. Iyengar and Raghukanth (2004) developed equations for the peninsular India using a synthetic database developed by point source
stochastic modeling, and observed that the predictions of their model is similar to those of available models for other intraplate regions. Raghukanth and Iyengar (2007) extended the previous model to include predictions of spectral accelerations. Mandal et al. (2009) employed 239 strong-motion records of 32 significant aftershocks of $3.1 \leq M_w \leq 5.6$ at epicentral distances of $1 \leq km \leq 288$ to develop a GMPE in Kutch region, western India. They noted higher uncertainty in the predictions for near-source distance range ($< 25$ km) and far source distance range ($> 100$ km), respectively. In the northeastern Indian plate margin, Nath et al. (2009) employed strong motion records derived from finite source stochastic simulations to develop a model for the region. They observed that the PGA-distance trend exhibited by the attenuation model is close to the one developed for Eastern North America for distances greater than 30 km. Baruah et al. (2009) provided an empirical attenuation relation for the small to moderate earthquakes ($2.5 \leq M_S \leq 5.0$) in the Shillong region, northeast India. Gupta (2010) used limited strong motion data recorded in northeast India for earthquakes in Indo-Myanmar subduction zone to derive region specific adjustments to the global model given by Atkinson and Boore (2003). The author observed that the earthquakes in the zone have larger ground motion amplitudes compared to the other subduction zones around the world owing to differences in source attributes, propagation path and site geology.

Probabilistic Seismic Hazard Analysis (PSHA) for the entire as well as in parts of the country and the adjoining regions has been reported by several workers. Bhatia et al. (1999) employed the equation given by Joyner and Boore (1981) for the entire country; the reason cited being no reliable estimates of attenuation values available for the region during that time. Das et al. (2006) use a pseudo-spectral velocity scaling equation, which they developed for northeast India. Sharma and Malik (2006) employed the equations developed by Sharma and Bungum (2006) and Youngs et al. (1997). The selection criteria being cited as the former being pertinent to the Himalayas and the latter to the subduction zones to account for shallow active tectonics and Indo-Myanmar subduction regime, respectively. Anbazhagan et al. (2009a) employed the equation given by Iyengar and Raghukanth (2004) in the Bangalore city. For the probabilistic seismic hazard analysis in the northwest Himalayas, Mahajan et al. (2010) considered the models given by Abrahamson and Litcheister (1989), Hasegawa et al. (1981) and Peng et al. (1985) on the basis of their conformity to the recordings of historical earthquakes in the specific source regions of the study region. However, unaccounted is the compatibility adjustment between the models; e.g., vertical accelerations employed by the first one and horizontal accelerations employed by the last one. MonaLisa et al. (2007) used two equations given by Ambraseys et al. (1996) and Boore et al. (1997), respectively but preferred the second one for seismic hazard assessment of the northwest Himalayan Belt in Pakistan. Apparently the aforementioned probabilistic hazard analyses do not employ multiple GMPEs usually undertaken to address epistemic uncertainty associated with ground motion estimation.

Jaiswal and Sinha (2007) employed three GMPEs given by Iyengar and Raghukanth (2004), Atkinson and Boore (1995) and Toro et al. (1997) respectively for PSHA in the peninsular India. Intraplate seismic environment has been considered as the criteria for the selection. A subjective weightage based on tectonic affinity to the study region has been considered by the authors. Menon et al. (2010) delivered PSHA of Tamil Nadu, which lies in the intraplate region of the Peninsular India. They considered three equations given by Abrahamson and Silva (1997), Campbell and Bozorgnia (2008) and Raghukanth and
lyengar (2007) respectively. Notwithstanding the comparison of prediction with the limited available strong-motion data from two events undertaken by the authors – one being an aftershock event, the GMPEs were developed for active tectonic regions except for the last one. The differences in the tectonic environments between the developed and target regions have been overlooked. Similarly, Kanagarathinam et al. (2008) considered four GMPEs for hazard analysis in the eastern coast of India. The equations were given by Sabetta and Pugliese (1996), Boore et al. (1993), Sharma (1998) and lyengar and Raghukanth (2004) respectively. The same case is also observed in the employment of equations pertaining to the tectonically active regions. In Gujarat, Petersen et al. (2004) examined two equations given by Frankel et al. (1996) and Toro et al. (1997) respectively for hazard analysis. Both the equations were developed for intraplate environment. They observed that the model given by Frankel et al. (1996) yields ground motions similar to those recorded from 2001 Gujarat earthquake. Further, the authors noted discrepancies in ground motion estimates in the updated hazard analysis in Gujarat compared to those computed by Bhatia et al. (1999) and suggested that the differences are due to the differently employed GPMEs. Intraplate ground motions are likely to be higher than interplate ground motions owing to the associated higher stress drops and lower attenuation properties. In Afghanistan, Boyd et al. (2007) employed five relations for shallow earthquakes given by Ambraseys et al. (1996), Abrahamson and Silva (1997), Boore et al. (1997), Sadigh et al. (1997) and Campbell and Bozorgnia (2003) respectively according to applicable faulting types in a logic tree framework. The authors use different sets comprising of the GMPEs given by Atkinson and Boore (2003) and Youngs et al. (1997) for subduction zone earthquakes. In southeast Asia, Petersen et al. (2004) considered the models given by Sadigh et al. (1997) and Youngs et al. (1997) with adjustments for rupture distance >200 km for shallow interplate and subduction zone earthquakes, respectively for seismogenic sources along Sumatra belt. A summary of globally available GMPEs for different seismogenic regimes can be found in Allen and Wald (2009). Typical Ground motion prediction equations used for seismic hazard estimation in Indian conditions developed for various tectonic provinces are given in Table 4.4.

Table 4.4: Typical Ground Motion Prediction Equations used for Seismic Hazard Estimation in Indian conditions developed for various tectonic provinces

<table>
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<th>Sl. No.</th>
<th>Region</th>
<th>Relationship</th>
<th>Reference</th>
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</table>
| 1.      | Sikkim Himalaya, Shallow active tectonic crust | First order attenuation relation:  
\[ \ln(Y) = -3.6 + 0.72M - 1.08 \ln r + 0.007r \]  
 where ‘Y’ is the strong motion parameter (PGA),  
 ‘M’ is earthquake magnitude,  
 ‘r’ is a measure of source to site distance.  
Second order attenuation relation:  
\[ \ln(PGA) = \ln(SA) - a_1 - (a_2 - a_4M) + a_5h - a_6 \ln(SR) \]  
 where ‘h’ is the site elevation,  
 ‘SR’ the site response,  
 ‘SA’ spectral acceleration at respective frequencies. | Nath et al. (2005) |
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<th>Sl. No.</th>
<th>Region</th>
<th>Relationship</th>
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<tr>
<td>2.</td>
<td>Peninsular India, SCR</td>
<td>( \ln(\gamma_{br}) = C_1 + C_2 (M - 6) + C_3 (M - 6)^2 - \ln(r) - C_4 + \ln(\epsilon_{br}) ) Where ( C_1, C_2, \ldots, C_4 ) are regression coefficients, ( \gamma_{br} = (Sa/g) ) stands for the ratio of spectral acceleration at bedrock level to acceleration due to gravity, ( 'M' ) and ( 'r' ) refers to moment magnitude and hypocentral distance respectively, ( \ln(\epsilon_{br}) ) is the error term.</td>
<td>Raghukanth and Iyengar (2007)</td>
</tr>
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<td>3.</td>
<td>Himalayas and Zagros region, Shallow active tectonic crust</td>
<td>( \log A = b_1 + b_2 M_w - b_3 \log \sqrt{R_{jb}^2 + b_4^2} + b_5 S + b_6 H ) Where ( b_1, b_2, \ldots, b_6 ) are regression coefficients, ( 'A' ) is the spectral acceleration in terms of ( m/sec^2 ), ( 'S' ) is 1 for rock site and 0 otherwise, ( 'H' ) is 1 for strike slip mechanism and 0 for reverse mechanism, ( 'M_w' ) is the magnitude and ( 'R_{jb}' ) represents the distance to the surface projection of the rupture.</td>
<td>Sharma et al. (2009)</td>
</tr>
<tr>
<td>4.</td>
<td>Gujarat, SCR</td>
<td>( \ln(Y) = -7.9527 + 1.4043 M_w - \ln(r_{jb}) + 19.822 ) For ( 3.1 &lt; M_w \leq 7.7 ) std.dev. (( \sigma )): ± 0.8243 Where ( 'Y' ) is peak horizontal acceleration in ( 'g' ), ( 'M_w' ) is moment magnitude, ( 'r_{jb}' ) is the closest distance to the surface projection of the fault rupture in kilometers, ( 'S' ) is a variable taking the values of 0 and 1 according to the local site geology, ( 'S' ) is 0 for a rock site, and, ( 'S' ) is 1 for a soil site.</td>
<td>Mandal et al. (2009)</td>
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<td>5.</td>
<td>Northeast India, Active intraplate deformation</td>
<td>Spectral attenuation relation: ( \ln(PGA) = C_1 + C_2 M + C_3 (10 - M)^3 ) + ( C_4 \ln(r_{rup}) + C_5 \exp(C_6 M) ) + ( C_7 S_v + C_8 \ln(SR) + C_9 \ln(SA) ) Where ( PGA ) is in ( 'g' ), ( 'M' ) is the earthquake moment magnitude, ( 'r_{rup}' ) is the rupture distance (km), ( 'S_v' ) represents the effective shear wave velocity averaged over the top 30 meters overburden, ( 'SR' ) is the site response, and ( 'SA' ) is the spectral acceleration.</td>
<td>Nath et al. (2009)</td>
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<td>6.</td>
<td>Northeast India, Shallow crust &amp; Subduction zone</td>
<td>( \log[PSV(T)] = C_1(T) + C_2(T)M + C_3(T)h + C_4(T) \log \left( \sqrt{R^2 + h^2} \right) + C_5(T)v )</td>
<td>Das et al. (2006)</td>
</tr>
<tr>
<td></td>
<td>Where ‘( M )’ is the earthquake magnitude, ‘( R )’ is the epicentral distance, ‘( h )’ is the focal depth, ‘( v )’ is an index variable taken as zero for horizontal motion and 1 for vertical motion. The spectral amplitude PSV(( T )) at natural period ‘( T )’, of a single-degree-of freedom oscillator is defined in the units of m/s. Further, ( c_1(T) ), ( c_2(T) ), ( c_3(T) ), ( c_4(T) ), and ( c_5(T) ) are the coefficients to be evaluated by the regression analysis on the available PSV data.</td>
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<td>7.</td>
<td>Shillong region, Active Intraplate deformation</td>
<td>Equation for peak ground acceleration ( a_{PG} ) estimated for Shillong plateau is ( \lg a_{PG} = 0.086M_s - 0.547\lg R + 0.185 \pm 0.18 ) Where ‘( a_{PG} )’ indicates peak ground acceleration, ( M_s ) earthquake magnitude, ‘( R )’ epicentral distance. Predominant period for the Shillong plateau which is expressed by: ( \lg t = 0.04M_s + 0.01\lg R - 1.06 \pm 0.09 )</td>
<td>Baruah et al. (2009)</td>
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<td>8.</td>
<td>Indo-Myanmar Arc, Subduction zone</td>
<td>( \log Y = C_1 + C_2M + C_3h + C_4R - g \log R + C_5sIc + C_6sIsD + C_7sIsE ) Where ‘( Y )’ represents the random horizontal component of the peak ground acceleration or 5% damped pseudo-acceleration in cm/sec², ‘( M )’ is the moment magnitude (limited to 8.5 for interface and to 8.0 for in-slab events with larger magnitude), ‘( h )’ the focal depth in km (limited to 100 km for deeper events), ‘( R )’ a distance metric with near source saturation effects taken into account, ‘( g )’ the geometric attenuation factor, The last three terms in equation accounts for the</td>
<td>Gupta (2010)</td>
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<td>effect of local soil condition defined by the NEHRP site classes based on the average shear wave velocity, $V_{S30}$, in the top 30 m of ground. All these terms are taken as zero for the generic rock type of site (NEHRP class ‘B’ defined by $V_{S30} &gt; 760$ m/s), where as only one of $S_C$, $S_D$, and $S_E$ is taken equal to 1 for soil classes ‘C’, ‘D’, and ‘E’, respectively. Further, the scaling factor $s_l$ in these terms accounts for the nonlinear soil behavior, which is defined as a function of peak ground acceleration.</td>
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<td>9.</td>
<td>Entire India</td>
<td>$\log A = -1.02 + 0.249M - \log r - 0.00255r + 0.26P$</td>
<td>Bhatia et al. (1999)</td>
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<td></td>
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<td>$r = (d^2 + 7.3^2)^{1/2}$</td>
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<td></td>
<td></td>
<td>$5.0 \leq M \leq 7.7$</td>
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<td>$\log V = -0.67 + 0.489M - \log r - 0.00256r + 0.17S + 0.22P$</td>
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<td></td>
<td></td>
<td>$r = (d^2 + 4.0^2)^{1/2}$</td>
<td>(by Joyner and Boore, 1981)</td>
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<td></td>
<td></td>
<td>$5.3 \leq M \leq 7.4$</td>
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<td>Where ‘A’ is peak horizontal acceleration in ‘g’, ‘V’ is peak horizontal velocity in cm/sec, ‘M’ is moment magnitude, ‘d’ is the closest distance to the surface projection of the fault rupture in km, ‘S’ takes on the value of zero at rock sites and one at soil sites, and ‘P’ is zero for 50 percentile values and one for 84 percentile values.</td>
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<td>10.</td>
<td>Gujarat</td>
<td>$\ln Y = C_1 + C_2 (M - 6) + C_3 (M - 6)^2$</td>
<td>Petersen et al. (2004)</td>
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<td>$- C_4 \ln R_M - (C_5 - C_4) \max \left[ \ln \left( \frac{R_M}{100} \right), 0 \right] \varepsilon - C_6 R_M$</td>
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<td></td>
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<td>$+ \varepsilon_e + \varepsilon_a$</td>
<td>(After Toro et al. 1997)</td>
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<td>$R_M = \sqrt{R_3 + C_7}$</td>
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<td>In the above equation, ‘M’ is moment magnitude, ‘R’ is horizontal distance, $\varepsilon_e$ is epistemic uncertainty, and $\varepsilon_a$ is aleatory uncertainty.</td>
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<td>11.</td>
<td>Northeast India</td>
<td>$\ln (A) = C_2 M - b \ln (X + \exp (C_3 M)) + C_4 S_{SR}$</td>
<td>Sharma and Malik (2006)</td>
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<td></td>
<td></td>
<td>(After Sharma and Bungum, 2006)</td>
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<td></td>
<td></td>
<td>Where ‘M’ is the magnitude, ‘X’ is the distance parameter; b is fixed to be 1.21, and $S_{SR}$ is the site condition.</td>
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<td>( \ln(PGA)<em>{ij} = C</em>{1}^{<em>} + C_{2}M_{i} + C_{3}^{</em>} \ln \left( r_{rup} \right)<em>{ij} + \epsilon + C</em>{4} \frac{C_{2}^{2}}{C_{3}^{2}} M_{i} ) + ( C_{5}Z_{ss} + C_{6}Z_{t} + C_{7}H_{i} + \eta_{i} + \epsilon_{ij} ) )</td>
<td>After Youngs et al., 1997</td>
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<td></td>
<td></td>
<td>( C_{1}^{<em>} = C_{1} + C_{3}C_{4} - C_{3}^{</em>} C_{4}^{*} )</td>
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<td></td>
<td></td>
<td>( C_{3}^{*} = C_{3} + C_{6}Z_{s} )</td>
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<td></td>
<td></td>
<td>( C_{4}^{*} = C_{4} + C_{7}Z_{s} )</td>
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<td></td>
<td></td>
<td>Where 'i' is the earthquake index,</td>
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<td>'J' is the recording station index for the ( i^{th} ) event,</td>
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<td>'PGA' (in units of g) is the geometrical mean of the two horizontal components of peak ground acceleration,</td>
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<td></td>
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<td>'M' is the moment magnitude,</td>
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<td></td>
<td></td>
<td>'rup' is source to site distance (in km),</td>
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<td></td>
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<td>'H' is the focal depth (in km). The terms 'H' and Zss are addition to the Youngs et al. (1988) model.</td>
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<td>'( \eta_{i} )' Representing earthquake to earthquake variability of ground motions, and an intra event component '( \epsilon_{ij} )' representing within earthquake variability of ground motions.</td>
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<td>( \ln(PSA) = C_{1} + C_{2} (M - 6) + C_{3} (M - 6)^{2} - \ln(R) - C_{4}R + \ln \epsilon )</td>
<td>Jaiswal and Sinha (2007)</td>
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<td>( \ln(Y) = C_{1} + C_{2} (M - 6) + C_{3} (M - 6)^{2} - C_{4} \ln R_{M} - (C_{5} - C_{4}) \max \left[ \ln \left( \frac{R_{M}}{100} \right), 0 \right] \epsilon + C_{6} \epsilon_{a} )</td>
<td>Toro et al., 1997</td>
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<td>12.</td>
<td>Peninsular India</td>
<td></td>
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</table>
| 13     | Northwest Himalays, Pakistan  | \[ R_{\text{M}} = \sqrt{R_y^2 + C_7^2} \]  
In the above equation, 'M' is moment magnitude, 'R' is horizontal distance, '\( \varepsilon_e \)' is epistemic uncertainty, and '\( \varepsilon_a \)' is aleatory uncertainty. | MonaLisa et al. (2007)     |
|        |                               | \[ \log(y) = C_1 + C_2 M + C_3 r + C_4 \log(r) + \sigma P \]  
(After Ambraseys et al., 1996)  
Where 'y' is the parameter being predicted, in this case peak horizontal ground acceleration (PGA) in 'g', 'Ms' is the surface wave magnitude, and  
\[ r = \sqrt{d^2 + h_0^2} \]  
Where ‘d’ is the shortest distance from the station to the surface projection of the fault rupture, in km, and ‘h0’ is a constant to be determined with C1, C2, C3 and C4. The standard deviation of log(y) is 0, and the constant ‘P’ takes a value of 0 for mean values and 1 for 84-percentile values of log(y). |                           |
| 14     | Bangalore                     | \[ \ln(y) = C_1 + C_2 (M - 6) + C_3 (M - 6)^2 - \ln(R) - C_4 R + \ln \varepsilon \]  
(After Raghukanth, 2005)  
Where 'y', 'M', 'R' and '\( \varepsilon \)' refer to PGA/spectral acceleration (g), moment magnitude, hypocentral distance, and error associated with the regression, respectively. | Anbazhagan et al. (2009a) |
| 15     | Eastern coast of India        | \[ \log_{10}(Y) = a + bM + c \log_{10}(r^2 + h^2)^{1/2} + e_1 S_1 + e_2 S_2 + \sigma \]  
(After Sabetta and Pugliese, 1996)  
Where 'M' is magnitude,  
'R' is distance (fault or epicentral) in kilometers,  
'a' is the standard deviation of the logarithm of 'Y',  
The dummy variables 'S1' and 'S2' refer to the site classification and take the value of 1 for shallow and deep alluvium sites, respectively, and zero otherwise,  
The parameter h is a fictitious depth determined by the regression and incorporates all of the factors that | Kanagarathinam et al. (2008) |
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<td>tend to limit the motion near the source, a property</td>
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<td>normally referred to as “saturation with distance”.</td>
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<td></td>
<td></td>
<td>( \log Y = b_1 + b_2 (M - 6) + b_3 (M - 6)^2 + b_4 r + b_5 \log r + b_6 G_B + b_7 G_C + \varepsilon_r + \varepsilon_e, )</td>
<td>(After Boore et al., 1993)</td>
</tr>
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<td></td>
<td>Afghanistan</td>
<td>( \log(A) = -1.072 + 0.3903M - 1.21\log(X + e^{0.5873M}) )</td>
<td>Sharma, 1998</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \ln y = C_1 + C_2 (M - 6) + C_3 (M - 6)^2 - \ln(R) - C_4 R + \ln \varepsilon )</td>
<td>(After Iyengar and Raghukanth, 2004)</td>
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<td></td>
<td>( \log(y) = C_1 + C_2 M + C_3 r + C_4 \log(r) + \sigma P )</td>
<td>Boyd et al. (2007)</td>
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<td></td>
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<td>( r = \sqrt{d^2 + h_0^2} )</td>
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<td></td>
<td>( d ) is the shortest distance from the station to the surface projection of the fault rupture, in km, and</td>
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'|ho'| is a constant to be determined with $C_1$, $C_2$, $C_3$ and $C_4$. The standard deviation of $\log(y)$ is 0, and the constant 'P' takes a value of '0' for mean values and '1' for 84-percentile values of $\log(y)$.

\[
\ln \text{Sa}(g) = f_1(M, r_{rup}) + Ff_3(M) + HWf_4(M, r_{rup}) + Sf_5(P\text{ga}_{\text{rock}})
\]

(After Abrahamson and Silva 1997)

Where ‘Sa(g)’ is the spectral acceleration in ‘g’, ‘M’ is moment magnitude, ‘rrup’ is the closest distance to the rupture plane in km, ‘F’ is the fault type, ‘HW’ is the dummy variable for hanging wall sites, and ‘S’ is the dummy variable for site class.

\[
\log Y = C_1 + f_1(M_w) + C_4 \ln \left( f_2(M_w, r_{seis}, S) + f_3(F) + f_4(S) + f_5(HW, F, M_w, r_{seis}) + \varepsilon \right)
\]

(After Cambell and Bozorgnia, 2003)

Where ‘Y’ is either the vertical component, ‘Y_v’, or the average horizontal component, ‘Y_h’, of PGA or 5% damped PSA in g ($g = 981 \text{ cm/sec}^2$),

‘$M_w$’ is moment magnitude,

‘r_{seis}’ is the closest distance to seismogenic rupture in kilometers,

‘r_{fb}’ is the closest distance to the surface projection of fault rupture in kilometers,

$\varepsilon$ is a random error term with zero mean and standard deviation equal to $\sigma_{\ln Y}$.

\[
\log Y = fn(M) + c_3h + c_4R - 0.1\log R + c_5sI_c + c_6sI_d + c_7sI_E
\]

(After Atkinson and Boore, 2003)

Where $Y =$ peak ground acceleration or 5% damped pseudoacceleration (PSA) in cm/sec random horizontal component,

M = moment magnitude use M 8.5 for interface events of M >8.5, M 8.0 for in-slab events of M ≥8);

$fn(M) = c1 + c2M$,

$h =$ focal depth in kilometers.

\[
R = \sqrt{D_{fault}^2 + \Delta^2}
\]

with $D_{fault}$ being closest distance.
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| 118    |        | to fault surface, in kilometers and \( \Delta a \) near source saturation term, \( S_c = 1(\text{NEHRP C}) \), \( S_d = 1(\text{NEHRP D}) \), \( S_e = 1(\text{NEHRP E}) \).  
\[
\ln(\text{PGA}) = C^*_1 + C^*_2 M + C^*_3 \ln \left( \frac{r_{rup}}{a} \right) + C^*_4 \frac{\Delta a}{a} + C^*_5 Z_{ss} + C^*_6 Z_s + C^*_7 H + \eta_i + \epsilon_{ij} ;
\]
\[
C^*_1 = C_1 + C_3 C_4 = C^*_3 C^*_4 \\
C^*_3 = C_3 + C_6 Z_s \\
C^*_4 = C_4 + C_7 Z_s
\]
(After Youngs et al., 1997)

Where \( 'i' \) is the earthquake index, \( 'j' \) is the recording station index for the \( i^{th} \) event, PGA (in units of g) is the geometrical mean of the two horizontal components of peak ground acceleration, \( 'M' \) is the moment magnitude, \( r_{rup} \) is source to site distance (in km), \( 'H' \) is the focal depth (in km). The terms \( 'H' \) and \( 'Zss' \) are addition to the Youngs et al. (1988) model, \( '\eta_i' \) representing earthquake to earthquake variability of ground motions, and an intra event component \( '\epsilon_{ij}' \) representing within earthquake variability of ground motions.

| 17. | Northwest Himalaya, India | \[
\log_{10} a_y(g) = -1.15 + 0.245 M - 1.096 \log_{10} (r + (r_0^{0.256M}) + 0.096 F - 0.0011 E r
\]
(After Abrahamson and Litechiser, 1989)  
Where \( 'M' \) is magnitude, \( 'r' \) is the distance in kilometers to the closest approach of the zone of energy release, \( 'F' \) is a dummy variable that is 1 for reverse or reverse oblique events and 0 otherwise, and \( 'E' \) is a dummy variable that is 1 for interpolate events and 0 for intraplate events. The standard error of \( \log_{10} a_y \) is 0.296. | Mahajan et al. (2010) |
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<tr>
<td>18.</td>
<td>Tamil Nadu</td>
<td>( \ln \text{Sa}(g) = f_1(M, r_{rup}) + F_f(M) + HW f_4(M, r_{rup}) + S_f(\text{pga}_{rock}) ) (After Abrahamson and Silva, 1997)</td>
<td>Menon et al. (2010)</td>
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<td>Where ‘\text{Sa}(g)’ is the spectral acceleration in ‘g’, ‘M’ is moment magnitude, ‘r_{rup}’ is the closest distance to the rupture plane in km, ‘F’ is the fault type, ‘HW’ is the dummy variable for hanging wall sites, and ‘S’ is the dummy variable for site class.</td>
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<td>( \ln(y_{br}) = C_1 + C_2 (M \text{–} 6) + C_3 (m \text{–} 6)^2 - \ln(r) - C_4 r + \ln(e_{br}) ) (After Raghukanth and Iyengar, 2007)</td>
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<td>Where ( y_{br} = (\text{Sa}/g) ) stands for the ratio of spectral acceleration at bedrock level to acceleration due to gravity, ‘M’ and ‘r’ refers to moment magnitude and hypocentral distance respectively, ( \ln(e_{br}) ) is the error term.</td>
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<td>19.</td>
<td>West Bengal</td>
<td>( \log Y = C_1 + f_1(M_w) + C_4 \ln \sqrt{f_2(M_w, r_{seis}, S)} + f_3(F) + f_4(S) ) (Modified Cambell and Bozorgnia, 2003)</td>
<td>Nath et al. (2011d)</td>
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<td>Where ‘Y’ is either the vertical component, ‘\text{Y}_V’ , or the average horizontal component, ‘\text{Y}_H’ , of PGA or 5% damped PSA in g (g 981 cm/sec²), ‘\text{M}<em>W’ is moment magnitude, ‘r</em>{seis}’ is the closest distance to seismogenic rupture in kilometers.</td>
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<td>( \log \text{PSA} = C_1 + C_2 M + C_3 M^2 + (C_4 + C_5 M) f_1 + (C_6 + C_7 M) f_2 + (C_8 + C_9 M) f_0 + C_{10} R_{cd} )</td>
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<td>Where ( f_0 = \max(\log(R_0 / R_{cd}), 0) ); ( f_1 = \min(\log R_{cd}, R_1) ); ( f_2 = \max(\log(R_{cd} / R_2), 0) ); ( R_0 = 10, R_1 = 70, R_2 = 140 ). (After Atkinson and Boore, 2006)</td>
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<td>‘M’ is the Earthquake magnitude, ‘\text{R}_{cd}’ is the closed distance to the faults.</td>
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4.7.3 Ranking of Prediction Equations

Methodology

Quantitative suitability assessment of a particular GPME to a particular region is decisive in providing a ranking order for a suite of GMPEs towards proper selection as well as assignment of appropriate weights in the logic tree implementation. The assessments are generally based on the efficacy of the GPME to the observed earthquakes in the region. In the present study, we employ the Information-Theoretic approach proposed by Scherbaum et al. (2009). The method supersedes an earlier approach given by Scherbaum et al. (2004a) that uses exceedance probabilities to quantify the appropriateness of candidate models with respect to the reference data. It has been observed that the previous scheme suffers from subjective decisions such as definition of thresholds for acceptability as well as dependency on sample size. The information theoretic approach overcomes these limitations. Furthermore, the approach has been tested successfully in California by Delavaud et al. (2009). Detail discussions on the technique can be found in the respective papers. The fundamental concept is the usage of relative Kullback-Leibler (KL) divergence as a measure of efficacy between the two models. The divergence between the two models $f$ and $g$ is given as follows,

$$\Delta_{KL}(f, g) = E_f[\log_2(f)] - E_g[\log_2(g)]$$  \hspace{1cm} (4.48)

Where $E_f$ is the statistical expectation with respect to $f$. The information unit, namely bit, is considered with base 2 logarithm. Relative KL divergence between the two models with respect to the true model $i.e.$ nature, which is ultimately unknown, results in canceling out the unknown model. Therefore, the second expectation indicates the level of efficacy between different models given a set of observations $x = \{x_i\}$, $i = 1, ..., N$ and can be estimated by the average sample log-likelihood as,

$$LLH = -\frac{1}{N} \sum_{i=1}^{N} \log_2 \left( g(x_i) \right)$$  \hspace{1cm} (4.49)

This estimator is used as ranking criterion with the lower value indicating greater efficacy. The efficacy tests are preferably performed using a set of data different from the one used for the construction of the model to avoid tendency to over-fit and underestimate the information loss in mapping the true model. Otherwise, corrective measures can be taken as suggested by Scherbaum et al. (2009).

4.7.4 Macroseismic Intensity Dataset

The earthquake recordings available in the country do not comprise a volume significant enough to support effective appraisal of ground motions. We, therefore, consider the available macroseismic data for the purpose. As such, globally macroseismic intensity data are gaining acceptance in the seismic hazard studies (e.g., Albarello and Mucciarelli, 2002; Mucciarelli et al., 2000 and 2008; D’Amico and Albarello, 2008). They are also often used in constraining ground motion predictions and has been suggested applicable for ground motion prediction model appraisal (e.g., Nath et al., 2009; Scherbaum
et al., 2009). Martin and Szeliga (2010) recently compiled a catalogue of macroseismic intensities in India and adjoining regions totaling 8331 observations from 570 earthquakes. The attributes have been given in European Macroseismic Scale (EMS) (Grünthal, 1998), intensities with the geographic coordinates of the observation sites.

Macroseismic intensity provides an indirect measure of observed structural damages but unlike PGA, it lacks physical parameter descriptive of the performance of the building in terms of force or displacement. Nevertheless, several relations between PGA and intensity are available; only a few connect PGA and EMS intensity scale (Margottini et al. 1992; Wald et al., 1999; Atkinson and Sonley, 2000; Souriau, 2006; Atkinson and Kaka, 2007; Sorensen et al., 2008). Seismic intensity values are often connected one-to-one with PGA since PGA is also a measure of intensity. However, Souriau (2006) employed hypocentral distance as an additional predictor parameter. The dependence of intensity on distance has been attributed to affect frequency on the variation of the intensity i.e., lower frequency being suggestive of larger distances while higher frequency entailed to shorter distances. Further, the amount of PGA to generate a given intensity would also differ according to the source-to-site distance.

4.8 SEISMIC HAZARD ANALYSIS

The seismic hazard analysis is concerned with getting an estimate of the strong-motion parameters at a site for the purpose of earthquake resistant design or seismic safety assessment. For generalized applications, seismic hazard analyses can also be used to prepare macro or micro-zoning maps of an area by estimating the strong-motion parameters for a closely spaced grid of sites. Two basic methodologies used for the purpose are the "deterministic" and the "probabilistic" seismic hazard analysis approaches. In the Deterministic Seismic Hazard Analysis (DSHA), the strong-motion parameters are estimated for the maximum potential earthquake, assumed to occur at the closest possible distance from the site of interest, without considering the likelihood of its occurrence during a specified exposure period. On the other hand, the Probabilistic Seismic Hazard Analysis (PSHA) integrates the effects of all the earthquakes expected to occur at different locations during a specified life period, with the associated uncertainties and randomness taken into account.

4.8.1 Conventional Deterministic Seismic Hazard Analysis (DSHA)

In the early years of geotechnical earthquake engineering, the use of Deterministic Seismic Hazard Analysis (DSHA) was prevalent. A DSHA involves the development of a particular seismic scenario upon which a ground motion hazard evaluation is based. The scenario consists of the postulated occurrence of an earthquake of a specified size occurring at a specified location. A typical DSHA can be described as a four-step process consisting of:

1. Identification and characterization of all earthquake courses capable of producing significant ground motion at the site. Source characterization includes definition of earth source’s geometry (the source zone) and earthquake potential.
2. Selection of a source-to-site distance parameter for each source zone. In most DSHAs, the shortest distance between the source zone and the site of interest is selected. The distance may be expressed as an epicentral distance or hypocentral distance, depending on the measure of distance of the predictive relationship(s) used in the following step.

3. Selection of the controlling earthquake (i.e., the earthquake that is expected to produce the strongest level of shaking), generally expressed in terms of some ground motion parameter, at the site. The selection is made by comparing the levels of shaking produced by earthquakes (identified in step 1) assumed to occur at the distances identified in step 2. The controlling earthquake is described in terms of its size (usually expressed as magnitude) and distance from the site.

4. The hazard at the site is formally defined, usually in terms of the ground motions produced at the site by the controlling earthquake. Its characteristics are usually described by one or more ground motion parameters obtained from predictive relationships of the types presented in the previous sections. Peak acceleration, peak velocity, and response spectrum ordinates are commonly used to characterize the seismic hazard.

**Figure 4.14:** Four steps of a deterministic seismic hazard analysis (after Kramer, 1996).

The DSHA procedure is shown schematically in Figure 4.14. Expressed in these four compact steps, DSHA appears to be a very simple procedure, and in many respects it is. When applied to structures for which failure could have catastrophic consequences, such as nuclear power plants and...
large dams, DSHA provides a straightforward framework for the evaluation of worst-case ground motions. However, it provides no information on the likelihood of occurrence of the controlling earthquake, the likelihood of it occurring where it is assumed to occur, the level of shaking that might be expected during a finite period of time (such as the useful lifetime of a particular structure or facility), or the effects of uncertainties in the various steps required to compute the resulting ground motion characteristics.

4.8.2 Probabilistic Seismic Hazard Analysis (PSHA)

PSHA rectifies a number of the problems inherent in DSHA by quantifying uncertainty and the probability of earthquake occurrence. As noted by Kramer (1996), PSHA follows similar steps to DSHA but uncertainty is quantified by a probability distribution at every step in the process. Probability distributions are determined for the magnitude of each earthquake on each source, the location of the earthquake in or along each source, and the prediction of the response parameter of interest. Kramer (1996) describes PSHA as a four-step process as enumerated and depicted in Figure 4.6.

- **Identify and characterize** (geometry and potential) all earthquake sources capable of generating significant shaking at the site. Three sources surrounding the site are shown in Figure 4.15. For each source, develop the probability distribution of rupture locations within the source. (A uniform probability distribution is generally chosen, which means that earthquakes are equally likely of occurring at any point along or in the source). Combine these distributions with the source geometry to obtain the probability distribution of source-to-site distance. (Contrast this with DSHA that assumes that the probability of occurrence is 1 at the points in each source zone closest to the site and 0 elsewhere).

- Develop seismicity or temporal distribution of earthquake occurrence. A recurrence relationship, which specifies the average rate at which an earthquake of some size will be exceeded, is used to characterize the seismicity of each source zone. (The recurrence relationship may accommodate the maximum earthquake but is not limited to that earthquake, as DSHA often does).

- The ground motion produced at the site by earthquakes of any possible size (magnitude) occurring at any possible point in each source zone must be determined with the use of predictive (attenuation) relationships. (The uncertainty inherent in the attenuation relationship is also considered explicitly in PSHA unlike DSHA).

- The uncertainties in earthquake location, size, and ground motion prediction are combined to obtain the probability that the ground motion parameter (e.g., PHA, spectral acceleration) will be exceeded in a particular time period (say 10% in 50 years).
4.8.3 Establishment of Seismic Occurrence Scenarios

The establishment of feasible scenarios requires the study of past seismic occurrences, the definition of different seismogenic zones affecting the area under study, and the characterization of their most important parameters (maximum expected magnitude and frequency or probability). It may also involve the knowledge of attenuation functions, if scenarios have to do anything with ground motion parameters, or the minimization of any other objective function, as total losses inflicted within a given time period.

In many applications not a single criterion prevails. Hypotheses to be analyzed are; the consideration of a largest historical event, as a measure of an extreme type event; the 50, 100 or 1000 year mean return period event, requiring an hazard and deaggregation analysis; the most probable measure of impact over the entire stock in the study area; or the eventual rupture of a possible fault structure.

All the seismological and geological themes have been generated using the Geographical Information System (GIS), a tool for computer based data and storage and manipulation that can link geological, seismological and geotechnical data with information on urban development for studies of the impact of hypothetical/historical/maximum credible/scenario earthquakes on human activities. The GIS facilitates numerous repetitive calculations required to produce individual seismic hazard map that the overall relative hazard map is based on, as well as linking seismic hazard data with information on buildings and infrastructure of damage and loss assessment.

A few typical CASE STUDIES are given in APPENDIX – IV.
5.1 INTRODUCTION

Building code based on seismic design forces on various seismic hazard parameters that describe the intensity of ground shaking during an earthquake. The design parameter is typically described by the acceleration, velocity or spectral acceleration with a specified probability of exceedance. These parameters are mapped on a national scale for a standard ground condition, usually rock or stiff soil.

Damage patterns in past earthquakes show that soil conditions at a site may have major effects on the level of ground shaking. Mapping of seismic hazard at local scales incorporating the effects of local soil conditions is imperative to calculating the probability of exceeding different levels of the mapped ground motion parameter in seismic microzonation.

5.2 EFFECTS OF LOCAL SOIL CONDITIONS

Site conditions play a major role in establishing the damage potential of incoming seismic waves from major earthquakes. Damage patterns in Mexico City after the 1985 Michoacan earthquake demonstrated conclusively the significant effects of local site conditions on seismic response of the ground. Peak accelerations of incoming motions at the rock level were generally less than 0.04 g and had predominant periods of around 2 s. Many clay sites in dried lakebed on which the original city was founded had site periods also of around 2 s and were excited into resonant response by the incoming motions. As a result the bedrock outcrop motions were amplified about 5 times. The amplified motions had devastating effects on structures with periods close to the site periods.

The nonlinear behavior of soils causes amplification factors to be dependent on the intensity of shaking. This was demonstrated very clearly by Jarpe et al. (1989) by comparing the amplification factors for site on Treasure Island in San Francisco Bay relative to the rock motions at adjacent Yerba Buena Island, using data from the main shock of the 1989 Loma Prieta earthquake and 7 aftershocks. The amplification factors for surface motions recorded at the Treasure Island site during 1989 Loma Prieta earthquake are shown in Figure 5.1. The solid line shows the variation in NS spectral ratio for the first 5 seconds of the shear wave in the main shock before any liquefaction took place at the site.
The shaded area in Figure 5.1 shows the 95% confidence region for the spectral ratios of 7 aftershocks. The amplification factors are drastically reduced in the strong motion phase, although still 2 or greater over a wide frequency band of engineering interest. The reduction in amplification with increased intensity of shaking is due to the nonlinear stress-strain response of the soil, resulting from the reduced effective shear moduli and increased damping. The peak acceleration at the surface is only 0.16 g, as shown in Figure 5.2 so the amplification factors are associated with fairly low levels of earthquake shaking.

5.3 IN-SITU FIELD TESTING FOR SITE CHARACTERIZATION

Various in-situ field tests available for site characterization are discussed in the subsequent sub-sections. The most common field tests conducted for site characterization are Standard Penetration Test (SPT), Cone Penetration Test (CPT), Spectral Analysis of Surface Wave (SASW), and Multichannel Analysis of Surface Wave (MASW).

5.3.1 Standard Penetration Test (SPT)

The standard penetration test is done using a split-spoon sampler in a borehole/auger hole. This sampler consists of a driving shoe, a split-barrel of circular cross-section (longitudinally split into two parts) and a coupling. The standard procedure for carrying out the standard penetration test is given below (as prescribed by BIS:2131, 1981) (Figure 5.3).
A borehole is made to the required depth and the bottom of the hole is cleaned.

The split-spoon sampler, attached to the drill-rods of required length is lowered into the borehole and is relaxed at the bottom.

The sampler is then driven to a distance of 450 mm in three intervals of 150 mm each. This is done by dropping a hammer of 63.5 kg from a height of 762 mm (BIS: 2131, 1981). The number of blows required to penetrate the soil is noted down for the last 300 mm, and this is recorded as the N value. The number of blows required to penetrate the sampler through the first 150 mm is called the seating drive and is disregarded. This is because the soil for the first 150 mm is disturbed and is ineffective for the SPT-N value.

The sampler is then pulled out and is detached from the drill rods. The soil sample, within the split barrel, is collected taking all precautions so as not to disturb the moisture content and is then transported to the laboratory, for tests. Sometimes, a thin liner is placed inside the split barrel. This makes it feasible for collecting the soil sample, within the liner, by sealing off both the ends of the liner with molten wax and then taking it away for laboratory test of the contained soil.

The standard penetration test is performed at every 0.75 m intervals in a borehole. If the depth of the borehole is large the interval can be made 1.50 m. In case, the soil under consideration consists of rocks or boulders, the SPT-N value can be recorded for the first 300 mm. The test is stopped if:

- 50 blows are required for any 150 mm penetration
- 100 blows are required for any 300 mm penetration
- 10 consecutive blows produce no advance.

However, it should be noted that SPT-N value obtained from the above set of procedures has to be corrected before it can be used for any of the empirical relations. These corrections and their values for certain conditions have been discussed in details in the next section.

**Necessary Corrections to be applied for the SPT Values**

The SPT-N value that is collected from the field is without applying any corrections. The N values that are obtained in the field are corrected for various errors, such as: overburden pressure, hammer energy, borehole diameter, presence of line, rod length and fines content.
The SPT-N value is corrected as follows:

\[(N_{1/60}) = (N) \cdot C_N \cdot C_R \cdot C_S \cdot C_B \cdot C_E\] (5.1)

Where \(C_R\) - correction for rod length; \(C_S\) - correction for sampler configuration; \(C_B\) - correction for borehole diameter; \(C_E\) - correction for hammer energy efficiency; It should also be noted that in the current studies, the corrected SPT-N value \((N_{1/60})\) is further corrected for its fine contents as follows.

\[(N_{1/60,CS}) = (N_{1/60}) \cdot C_{FINES}\] (5.2)

The fines correction is 1 for fine contents of \(FC < 5\%) and reaches a maximum value of \(FC > 35\%\) (where \(FC\) is the percentage fine content). The regressed relation for \(C_{FINES}\) is however given by the regression relationship as follows:

\[C_{FINES} = (1 + 0.004 FC) + 0.05 \left( \frac{FC}{N_{1/60}} \right)\] (5.3)

Where \(FC\), the percentage fines content is expressed as an integer (e.g., 10% fines is expressed as 10); \(N_{1/60}\) is in units of blows/ft. The correction factors to be used are given in Tables 5.1 – 5.4.

**Table 5.1: Hammer Correction factor (CE)**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Donut Hammer</td>
<td>0.5-1.0</td>
</tr>
<tr>
<td>Safety Hammer</td>
<td>0.7-1.2</td>
</tr>
<tr>
<td>Automatic-trip Donut Hammers</td>
<td>0.8-1.3</td>
</tr>
</tbody>
</table>

**Table 5.2: Correction for B.H. Diameter (CB)**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Size</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole Diameter</td>
<td>65-115 mm</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>150 mm</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>200 mm</td>
<td>1.15</td>
</tr>
</tbody>
</table>

**Table 5.3: Correction for Rod Length (CR)**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Size</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;3 m</td>
<td>0.75</td>
</tr>
<tr>
<td>Rod Length</td>
<td>3-4 m</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>4-6 m</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>6-10 m</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>10-30 m</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Table 5.4: Correction for Sampler based on method ($C_s$)

<table>
<thead>
<tr>
<th>Factor</th>
<th>Size</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard samplers</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Sampling method</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sampler without liners</td>
<td>1.1-1.3</td>
<td></td>
</tr>
</tbody>
</table>

The formula used to find the correction for energy ratio is:

$$C_E = \frac{ER}{60\%}$$  \hfill (5.4)

Where ER (efficiency ratio) is the fraction or percentage of the theoretical SPT impact energy that is actually transferred to the sampler.

The sources of some of the common errors while carrying out SPT tests are listed in Table 5.5 (Kulhawy and Mayne, 1990).

Table 5.5: Source of errors in SPT test

<table>
<thead>
<tr>
<th>Cause</th>
<th>Effects</th>
<th>Influence on SPT–N value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inadequate cleaning of the hole</td>
<td>SPT is not made in original in-situ soil. Therefore, spoils may become trapped in sampler and be compressed as sampler is driven, reducing recovery</td>
<td>Increases</td>
</tr>
<tr>
<td>Failure to maintain adequate head of water in borehole</td>
<td>Bottom of borehole may become quick and soil may rinse into the hole</td>
<td>Decreases</td>
</tr>
<tr>
<td>Careless measure of hammer drop</td>
<td>Hammer energy varies</td>
<td>Increases</td>
</tr>
<tr>
<td>Hammer weight in accurate</td>
<td>Hammer energy varies</td>
<td>Increases or Decreases</td>
</tr>
<tr>
<td>Hammer strikes drill rod collar eccentrically</td>
<td>Hammer energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Lack of hammer free fall because of ungreased sheaves, new stiff rope on weight, more than two turns on cat head, incomplete release of rope each drop</td>
<td>Hammer energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Cause</td>
<td>Effects</td>
<td>Influence on SPT–N value</td>
</tr>
<tr>
<td>--------------------------------------------</td>
<td>-----------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>Sampler driven above the bottom of casing</td>
<td>Sampler driven in disturbed, artificially densified soil</td>
<td>Increases greatly</td>
</tr>
<tr>
<td>Careless blow count</td>
<td>Inaccurate results</td>
<td>Increases or Decreases</td>
</tr>
<tr>
<td>Use of non-standard sampler</td>
<td>Corrections with standard sampler not valid</td>
<td>Increases or Decreases</td>
</tr>
<tr>
<td>Coarse gravel or cobbles in soil</td>
<td>Sampler becomes clogged or impeded</td>
<td>Increases</td>
</tr>
<tr>
<td>Use of bent drill rods</td>
<td>Inhibited transfer of energy of sampler</td>
<td>Increases</td>
</tr>
</tbody>
</table>

5.3.2 Cone Penetration Test (CPT)

Cone Penetration Test (CPT) is an in-situ test done to determine the soil properties and to get the soil stratigraphy. This test was initially developed by the Dutch Laboratory for Soil Mechanics (in 1955) and hence it is sometimes known as the Dutch cone test. On a broad scale the CPT test can be divided into two – Static Cone Penetration Test (BIS:4968, Part - 3, 1976) and Dynamic Cone Penetration Test.

(i) Static Cone Penetration Test

The cone with an apex angle of 60° and an end area of 10 cm² will be pushed through the ground at a controlled rate (2 cm/sec) (Figure 5.4). In the static test the cone is pushed into the ground and not driven. During the penetration of cone penetrometer through the ground surface, the forces on the cone tip (q<sub>c</sub>) and sleeve friction (f<sub>s</sub>) are measured. The measurements are carried out using electronic transfer and data logging, with a measurement frequency that can secure the detailed data about soil contents and its characteristics. The Friction Ratio (FR = f<sub>s</sub>/q<sub>c</sub>), varies with the soil types and is an important parameter.

![Figure 5.4](http://geosystems.ce.gatech.edu/Faculty/Mayne/Research/devices/cpt.htm)
(ii) **Dynamic cone Penetration Test**

Dynamic test is conducted by driving the cone by hammer blows. The dynamic cone resistance is estimated by measuring the number of blows required for driving the cone through a specified distance. Usually this test will be performed with a 50 mm cone without bentonite slurry or using a 65 mm cone with bentonite slurry. The hammer weighs 65 kg and the height of fall is 75 cm. The test is done in a cased borehole to eliminate the skin friction.

There are many correlations available to evaluate soil properties based on the CPT value (either static or dynamic).

### 5.3.3 Seismic Cone Penetration Test (SCPT)

The seismic cone penetration test uses a standard cone penetrometer with two geophones. One set of geophones is located behind the friction jacket and the other set is located one meter above the first set (Figure 5.5).

![Figure 5.5: Seismic Cone Penetration test (Courtesy of Fugro Company).](image)

The test method consists of measuring the travel time of seismic waves propagating between a wave source and ground surface. These waves will comprise of shear waves and compressional or primary waves. The velocity of seismic waves in the ground will give shear modulus and Poisson’s ratio and the soil profile.
5.3.4 Field Vane Shear Test

Vane shear test is done on fully saturated clays for the evaluation of undrained shear strength. This is suitable for soft clays whose shear strength (less than 100 kPa) is changed considerably by sampling. The equipment consists of stainless steel vane (with four blades) connected to the end of a high tensile rod. The rod is enclosed by a sleeve packed by grease. The typical dimension of the vanes is usually 50 mm by 100 mm or 75 mm by 150 mm and the diameter of the rod should not exceed 12.5 mm (BIS:4434, 1978). After pushing the vane and the rod into the clay, torque is gradually applied to the top end of the rod till the clay fails in shear. The shear strength can be calculated based on the torque applied and the test is repeated at desired depths.

5.3.5 Dilatometer Test

Dilatometer consists of a stainless steel blade with a thin flat circular expandable steel membrane on one side. Dilatometer is advanced into a borehole from the ground surface and tests are conducted at an interval of 10 to 20 cm. At each interval, the dilatometer is stopped and the membrane is inflated under gas pressure. The readings of inflation of the membrane and the corresponding gas pressure are recorded. There are correlations available to relate the test results with low-strain soil stiffness and liquefaction resistance of the soil.

5.3.6 Pressuremeter Test

Pressuremeter test is conducted using a pressuremeter. It is a cylindrical device that uses flexible membrane for the application of uniform pressure to the walls of a borehole. It measures the stress-deformation behavior of the soil by measuring volume of fluid injected into the flexible membrane and the corresponding pressure applied. This is the only in-situ test that can measure stress-strain as well as strength behavior of in situ soil.

5.3.7 Suspension Logging Test

This test is most commonly used in petroleum explorations. It principally consists of a probe of length 5 to 6 m long. It is lowered into an uncased borehole filled with water and drilling fluid. A reversible-polarity solenoid located at the end of the probe produces impulsive pressure wave in the drilling fluid, which travels to the bottom of the borehole and produces P and S waves in the surrounding soil. Thus produced P and S waves transmit through the soil and reach the geophones attached to the probe 1m away from the solenoid. This technique allows measurements of P- and S-waves in a single, uncased borehole, but at very high frequencies (1000 to 3000 Hz for P-waves and 500 to 2000 Hz for S-waves), which are much higher than the frequency range of interest of geotechnical earthquake engineering.

5.3.8 Spectral Analysis of Surface Wave (SASW)

This method is used to measure the shear wave profiles of soil. This method depends on the dispersive characteristics of Rayleigh waves traveling through layered soil medium. A dynamic source is used to create surface waves of different frequencies and these are monitored by two receivers at known distances. Using the SASW methods large area can be covered and the soil profiles can be obtained. However, more advanced MASW method is preferred to SASW as it is very tedious to conduct SASW tests which depend upon only two geophones for exploration.
5.3.9 Multi-channel Analysis of Surface Wave (MASW)

Shear wave velocity ($V_s$) is an essential parameter for evaluating the dynamic properties of soil in the shallow subsurface. A number of geophysical methods have been proposed for near-surface characterization and the measurement of shear wave velocity by using a great variety of testing configurations, processing techniques, and inversion algorithms. The most widely-used technique is Multichannel Analysis of Surface Waves (MASW). The MASW method was first introduced by Park et al. (1999). In this survey an active seismic source (e.g., a sledge hammer) and a linear receiver array are used to collect data in a roll-along mode. The MASW has been found to be an efficient method for unraveling the shallow subsurface properties. In particular, MASW is used in geotechnical engineering for the measurement of shear wave velocity and dynamic properties, identification of subsurface material boundaries and spatial variations of shear wave velocity. It is a seismic method that can be used for geotechnical characterization of near-surface materials.

MASW system consists of a number of geophones (usually more than twelve) and usually they are arranged at equal placing. The seismic waves are created by an impulsive source (sledge hammer). These waves are captured by the geophones/receivers. The captured Rayleigh wave is further analyzed using suitable software to generate $V_s$ data. This is being done in three steps i) preparation of a Multi-channel record (some times called a shot gather or a field file), ii) dispersion curve analysis, and iii) inversion. The term “Multi-channel record” indicates a seismic data set acquired by using a recording instrument with more than one channel. MASW has been effectively used with highest signal-to-noise ratio (S/N) of surface waves.

![Figure 5.6: Dispersion Analysis-Multichannel Approach (2D Wave-field Transformation) (after Sitharam and Anbazhagan, 2007; Anbazhagan et al., 2009b).](image)

The generation of a dispersion curve is a critical step in the MASW method. A dispersion curve (Figure 5.6) is generally displayed as a function of phase velocity versus frequency. Phase velocity can be calculated from the linear slope of each component on the swept-frequency record. The lowest analyzable frequency in this dispersion curve is around 4 Hz and the highest frequency of 75 Hz has been considered.
Empirical relations have been proposed to correlate the penetration test results between CPT and SPT (Robertson et al., 1983) as well as with the shear-wave velocities (Ohta and Goto, 1978; Mayne and Rix, 1995; İyisan, 1996). A list of some of the relationships proposed to calculate shear wave velocity in terms of SPT-N value is given in Table 5.6.

The laboratory tests conducted on soil and rock samples retrieved from boring operations could also be considered in two groups. The first group of tests (i.e., grain size distribution, water content, consistency limits) is needed to determine the soil classification, grain size characteristics and index properties of the soil and rock layers encountered in the soil profile. These tests would allow the classification of soil layers to determine site classification according to different site classes proposed in different earthquake codes. The second group of tests is conducted to obtain shear strength characteristics of soil specimens under cyclic excitations (Kokusho, 1980; Ishihara, 1993). The three basic types of tests are resonant column, impulse wave velocity measurements, and low frequency cyclic loading tests (cyclic triaxial, cyclic simple shear, cyclic torsional triaxial). It would be preferable to determine dynamic shear modulus curves based on these laboratory tests.

**Table 5.6:** Proposed Relationships to Estimate Shear Wave Velocity from SPT-N Values

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( V_s ) (m/sec)</th>
<th>Author</th>
<th>Region</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>( V_s = 5.3 N + 134 )</td>
<td>Fumal and Tinsley (1985)</td>
<td>USA</td>
</tr>
<tr>
<td></td>
<td>( V_s = 114.4 N^{0.31} )</td>
<td>Lee (1990)</td>
<td>USA</td>
</tr>
<tr>
<td></td>
<td>( V_s = 165.7 N^{0.19} )</td>
<td>Pitilakis et al. (1992)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>( V_s = 105.7 N^{0.33} )</td>
<td>Raptakis et al. (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>( V_s = 184.2 N^{0.17} )</td>
<td>Raptakis et al. (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>( V_s = 27.0 N^{0.73} )</td>
<td>Jafari et al. (2002)</td>
<td>South of Tehran</td>
</tr>
<tr>
<td></td>
<td>( V_s = 80.2 N^{0.292} )</td>
<td>Imai (1977)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>( V_s = 97.9 N^{0.269} )</td>
<td>Hasancebi and Ulusay (2007)</td>
<td>Turkey</td>
</tr>
<tr>
<td></td>
<td>( V_s = 132(N60)^{0.178} )</td>
<td>Pitilakis et al. (1999)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>( V_s = 80(N)^{0.33} )</td>
<td>Anbazhagan and Sitharam (2010)</td>
<td>Bangalore, India</td>
</tr>
<tr>
<td></td>
<td>( V_s = 89.31 N^{0.358} )</td>
<td>Uma Maheswari et al. (2010)</td>
<td>Chennai, India</td>
</tr>
<tr>
<td></td>
<td>( V_s = 44 N^{0.48} )</td>
<td>Dikmen (2009)</td>
<td>Western Taiwan</td>
</tr>
<tr>
<td></td>
<td>( V_s = 100.0 N^{0.33} )</td>
<td>Jafari et al. (2002)</td>
<td>Japan</td>
</tr>
<tr>
<td>Sand</td>
<td>( V_s = 57.4 N^{0.49} )</td>
<td>Lee (1990)</td>
<td>USA</td>
</tr>
<tr>
<td></td>
<td>( V_s = 56.4 N^{0.50} )</td>
<td>Seed et al. (1983)</td>
<td>USA</td>
</tr>
<tr>
<td>Soil type</td>
<td>$V_s$ (m/sec)</td>
<td>Author</td>
<td>Region</td>
</tr>
<tr>
<td>-----------</td>
<td>--------------</td>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>Sand</td>
<td>$V_s = 162.0 \text{ N}^{0.17}$</td>
<td>Pitilakis et al. (1992)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>$V_s = 100.0 \text{ N}^{0.24}$</td>
<td>Raptakis et al. (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>$V_s = 123.4 \text{ N}^{0.29}$</td>
<td>Raptakis et al. (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>$V_s = 80.6 \text{ N}^{0.331}$</td>
<td>Imai (1977)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 90.8 \text{ N}^{0.319}$</td>
<td>Hasancebi and Ulusay (2007)</td>
<td>Turkey</td>
</tr>
<tr>
<td></td>
<td>$V_s = 22.0 \text{ N}^{0.76}$</td>
<td>Chein et al. (2000)</td>
<td>Western Taiwan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 79.0 \text{ N}^{0.434}$</td>
<td>Hanumantharao and Ramana (2008)</td>
<td>Delhi</td>
</tr>
<tr>
<td></td>
<td>$V_s = 32 \text{ N}^{0.5}$</td>
<td>Shibata (1970)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 87 \text{ N}^{0.36}$</td>
<td>Ohta et al. (1972)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 145 \text{ N}^{0.178}$</td>
<td>Pitilakis et al. (1999)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>$V_s = 57 \text{ N}^{0.434}$</td>
<td>Anbazhagan and Sitharam (2010)</td>
<td>Bangalore, India</td>
</tr>
<tr>
<td></td>
<td>$V_s = 100.53 \text{ N}^{0.265}$</td>
<td>Uma Maheswari et al. (2010)</td>
<td>Chennai, India</td>
</tr>
<tr>
<td></td>
<td>$V_s = 5.1 \text{ N+152}$</td>
<td>Fumal and Tinsley (1985)</td>
<td>USA</td>
</tr>
<tr>
<td></td>
<td>$V_s = 88.0 \text{ N}^{0.34}$</td>
<td>Ohta and Goto (1978)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 73 \text{ N}^{0.33}$</td>
<td>Dikmen (2009)</td>
<td>Western Taiwan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 125 \text{ N}^{0.3}$</td>
<td>Okamota et al. (1989)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s = 80.0 \text{ N}^{0.33}$</td>
<td>Jafari et al. (2002)</td>
<td>Japan</td>
</tr>
<tr>
<td>All</td>
<td>$V_s = 91.0 \text{ N}^{0.34}$</td>
<td>Imai (1977)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 85.35 \text{ N}^{0.348}$</td>
<td>Ohta and Goto (1978)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 121.0 \text{ N}^{0.27}$</td>
<td>Jafari et al. (2002)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s = 61.0 \text{ N}^{0.50}$</td>
<td>Seed and Idriss (1981)</td>
<td>USA</td>
</tr>
<tr>
<td></td>
<td>$V_s = 107.6 \text{ N}^{0.36}$</td>
<td>Athanasopoulos (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td></td>
<td>$V_s = 22.0 \text{ N}^{0.85}$</td>
<td>Jafari et al. (1997)</td>
<td>Iran</td>
</tr>
<tr>
<td></td>
<td>$V_s = 116.1 \text{ (N + 0.3185)}^{0.202}$</td>
<td>Jinan (1987)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s = 51.5 \text{ N}^{0.516}$</td>
<td>Iyisan (1996)</td>
<td>Turkey</td>
</tr>
<tr>
<td>Soil type</td>
<td>$V_s$ (m/sec)</td>
<td>Author</td>
<td>Region</td>
</tr>
<tr>
<td>----------------------------</td>
<td>---------------</td>
<td>---------------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>All</td>
<td>$V_s=97.0\ N^{0.314}$</td>
<td>Imai and Tonouchi (1982)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s=82.0\ N^{0.39}$</td>
<td>Ohsaki and Iwasaki (1973)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s=92.1\ N^{0.337}$</td>
<td>Fujiwara (1972)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s=90.0\ N^{0.309}$</td>
<td>Hasancebi and Ulusay (2007)</td>
<td>Turkey</td>
</tr>
<tr>
<td></td>
<td>$V_s=82.6\ N^{0.43}$</td>
<td>Hanumantharao and Ramana (2008)</td>
<td>Delhi</td>
</tr>
<tr>
<td></td>
<td>$V_s=76\ N^{0.33}$</td>
<td>Imai and Yoahimura (1970)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s=19\ N^{0.6}$</td>
<td>Kanai et al. (1966)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s=92\ N^{0.329}$</td>
<td>Imai and Yoahimura (1975)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s=90\ N^{0.341}$</td>
<td>Imai et al. (1975)</td>
<td>Japan</td>
</tr>
<tr>
<td></td>
<td>$V_s=243.8\ \sigma e^{0.4}$</td>
<td>LIQUFAC</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s=152.4\ \sigma e^{0.3}$</td>
<td>LIQUFAC</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_s=95.64\ N^{0.301}$</td>
<td>Uma Maheswari et al. (2010)</td>
<td>Chennai, India</td>
</tr>
<tr>
<td></td>
<td>$V_s=32.8\ N^{0.51}$</td>
<td>Sisman (1995)</td>
<td>Turkey</td>
</tr>
<tr>
<td></td>
<td>$V_s=68.3\ N^{0.292}$</td>
<td>Kiku et al. (2001)</td>
<td>Turkey</td>
</tr>
<tr>
<td></td>
<td>$V_s=58\ N^{0.39}$</td>
<td>Dikmen (2009)</td>
<td>Western Taiwan</td>
</tr>
<tr>
<td></td>
<td>$V_s=76.2\ N^{0.24}$</td>
<td>Kalteziotis et al. (1992)</td>
<td>Greece</td>
</tr>
<tr>
<td>Silt loam/sandy clay</td>
<td>$V_s=4.3\ N+218$</td>
<td>Fumal and Tinsley (1985)</td>
<td>USA</td>
</tr>
<tr>
<td>Silty Sand/Sandy slit</td>
<td>$V_s=86.0\ N^{0.42}$</td>
<td>Hanumantharao and Ramana (2008)</td>
<td>Delhi</td>
</tr>
<tr>
<td>Silts</td>
<td>$V_s=106\ N^{0.32}$</td>
<td>Lee (1990)</td>
<td>USA</td>
</tr>
<tr>
<td>Silts</td>
<td>$V_s=22.0\ N^{0.77}$</td>
<td>Jafari et al. (2002)</td>
<td>South of Tehran</td>
</tr>
<tr>
<td>Alluvial Soil</td>
<td>$V_s=84\ N^{0.31}$</td>
<td>Ohba and Toriumi (1970)</td>
<td>Japan</td>
</tr>
<tr>
<td>Fine grained Soil</td>
<td>$V_s=19.0\ N^{0.85}$</td>
<td>Jafari et al. (2002)</td>
<td>South of Tehran</td>
</tr>
<tr>
<td>Cohesionless Soils</td>
<td>$V_s=59\ N^{0.47}$</td>
<td>Ohsaki and Iwasaki (1973)</td>
<td>Japan</td>
</tr>
<tr>
<td>Granular Soils</td>
<td>$V_s=100.5\ N^{0.29}$</td>
<td>Sykora and Stokoe (1983)</td>
<td>USA</td>
</tr>
</tbody>
</table>
### Soil Characterization

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$V_s$ (m/sec)</th>
<th>Author</th>
<th>Region</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sands and gravelly</td>
<td>$V_s=152+5.1N^{0.27}$</td>
<td>Fumal and Tinsley (1985)</td>
<td>USA</td>
</tr>
<tr>
<td>sand soils</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granular Soils</td>
<td>$V_s=175+3.75N$</td>
<td>Kayabali (1996)</td>
<td>Turkey</td>
</tr>
<tr>
<td>Gravel</td>
<td>$V_s=94.0N^{0.34}$</td>
<td>Ohta and Goto (1978)</td>
<td>Japan</td>
</tr>
<tr>
<td>Gravel</td>
<td>$V_s=192.4N^{0.13}$</td>
<td>Raptakis <em>et al.</em> (1995)</td>
<td>Greece</td>
</tr>
<tr>
<td>Holocene gravels</td>
<td>$V_s=63.0N^{0.43}$</td>
<td>Sykora and Koester (1988)</td>
<td>USA</td>
</tr>
<tr>
<td>Pleistocene gravels</td>
<td>$V_s=132N^{0.32}$</td>
<td>Sykora and Koester (1988)</td>
<td>USA</td>
</tr>
<tr>
<td>Holocene gravels</td>
<td>$V_s=63.0N_{60}^{0.43}$</td>
<td>Rollons <em>et al.</em> (1998)</td>
<td>USA</td>
</tr>
<tr>
<td>Pleistocene gravels</td>
<td>$V_s=132.0N_{60}^{0.32}$</td>
<td>Rollons <em>et al.</em> (1998)</td>
<td>USA</td>
</tr>
<tr>
<td>Cohesive Soils</td>
<td>$V_s=76.6N^{0.45}$</td>
<td>Kalteziotis <em>et al.</em> (1992)</td>
<td>Greek</td>
</tr>
<tr>
<td>Cohesionless Soils</td>
<td>$V_s=49.1N^{0.50}$</td>
<td>Kalteziotis <em>et al.</em> (1992)</td>
<td>Greek</td>
</tr>
</tbody>
</table>

Where $N$ - SPT value; $\sigma_e$ - effective vertical stress of the soils; $V_s$ - shear wave velocity; P- Pleistocene.

### 5.3.10 Other Low Strain Field Tests

**Low Strain Tests:** The strain levels in these types of tests will be around 0.0001% some of the important low strain tests are discussed below.

**Seismic Reflection Test:** This test is used to evaluate the wave propagation velocity and the thickness of soil layers. The test setup will consist of a source producing a seismic impulse and a receiver to identify the arrival of seismic waves and the travel time from source to receiver is measured. Based on these measurements, the thickness of soil layer can be evaluated.

**Seismic Refraction Test:** This test will use the arrival time of the first seismic wave at the receiver. Using the results obtained from this test the delineation of major stratigraphic units is possible.

**Steady state vibration test:** In this test the wave propagation velocities are measured from steady state vibration characteristics. However these tests can be useful for determining the near-surface shear wave velocity and they fail to provide the details of highly variable soil profiles.

**Seismic cross-hole test:** In seismic cross-hole test the wave velocities are measured using more than one borehole (Figure 5.7). In the simplest case two boreholes are used – one with an impulse source and the other with a receiver and both are kept at the same depth. The test is repeated at various depths to get the soil profile.
Seismic Down-hole (up-hole) test: This test is used to measure the travel time of seismic waves from source to receiver. It is performed using a single borehole. In seismic down-hole test the receiver is kept at the ground surface and the impulse source is kept at different depths. The up-hole test is done with receiver at the ground surface and the impulse source in the borehole. This test is not effective for depths greater than 30 to 60 m.

Resonant column test: This is the most common low strain test which is being used to evaluate dynamic properties of soil. In this test the soil sample, either solid or hollow cylindrical samples are subjected to torsional or axial loading using an electro magnetic loading system. The fundamental frequency of the sample can be evaluated and this in turn will give the value of shear modulus of the soil specimen. Even though the resonant column test is very good in evaluating the damping and strain dependent properties of soils, the response will depend on the response of the apparatus also.

Bender Element Test: in this test the shear wave velocity of laboratory specimen can be measured using a pezoelectric bender element. A transmitter and receiver elements (piezoelectric) are placed at each end of the sample. There will be changed in dimension of these piezoelectric elements when subjected to change in voltage. An electric pulse applied to the transmitter causes it to deform rapidly and produce a stress wave that will travel through the specimen toward the receiver. When the stress
wave reaches the receiver, it generates a voltage pulse and this is measured. The wave speed is calculated from the arrival time and the known distance between the transmitter and the receiver. Since the soil specimens are not disturbed during the tests using the piezoelectric elements, these are incorporated in various soil testing devices, such as conventional triaxial devices, oedometers and direct or simple shear devices.

**Cyclic Triaxial Tests:** The test device consists of the standard triaxial testing equipment with a cyclic axial loading unit. In some cases, the cell pressure is also applied cyclically and it is possible to simulate isotropic or anisotropic initial stress conditions. The values of the shear modulus and damping ratio can be obtained from the stress strain response of the samples. Average slope of hysteresis loop gives shear modulus and the area under the loop provides the damping ratio. Typically, five to ten specimens are tested under different levels of cyclic shear strain amplitudes chosen in the range of $10^{-4}$ to $10^{-1}$ for the determination of dynamic properties. Dynamic deformation characteristics are influenced by effective confining pressure during the test. When an undisturbed sample of normally consolidated soil is obtained, the effective vertical pressure at the depth of sampling is isotropically applied by cell pressure to avoid the influence of over consolidation. In order to obtain in-situ shear modulus of the soil from the laboratory test results, correction of these results is necessary so that a shear modulus corresponding to the average effective principal stress at the sample depth is obtained. Cyclic triaxial test is very useful in determining the liquefaction potential of the soil. In cyclic triaxial test, the principal stress axes remain either vertical or horizontal. Hence do not represent the true seismic loading where principal stress axes rotate continuously.

**Cyclic Direct Simple Shear Test:** The cyclic direct simple shear test can simulate the earthquake loading more precisely than the cyclic triaxial test and hence this is one of the tests which is commonly used for liquefaction testing. When a cyclic shear stress is applied to the top or bottom of the specimen, the deformation is similar to that of a soil element in which there is a vertical propagation of S wave.

**5.3.11 Model Tests**

**Centrifuge Modeling:** In usual lab tests it is extremely difficult to simulate very high body forces. This deficiency can be overcome with the use of centrifuge techniques, in which models are subjected to predetermined, high acceleration levels to simulate the field conditions satisfactorily. Some of the important problems in which the centrifuge modeling can be applied are earth dams, tunnels, offshore foundations, geo-environmental problems, problems of nuclear waste disposal, seismic studies of earth structures and foundations etc.

**Shake Table:** Characteristics of earthquake ground motions and the behavior of structures during earthquakes can be simulated more precisely by constructing scaled models on a shake table and subjecting them to real time ground motions. The results obtained can be used to improve seismic design of various structures. Shake table models are usually built in laminar boxes, to reduce frictional effects and to facilitate smooth relative displacement between the adjacent layers. Single to six degrees of freedom shake tables in various sizes are available.
5.4 ROUTINE GEOTECHNICAL LABORATORY TESTS

Routine geotechnical laboratory tests (following relevant IS codes wherever applicable) for soils and rock samples are as follows:

5.4.1 Index properties of Soil and Rock Samples

For Soil samples: Grain Size Analysis of representative samples can be obtained from Sieve and Hydrometer analysis, (BIS:2720 Part 4-1985) or by deploying laser analyzer (BIS:2720 Part 4-1985). This is to evaluate the soil particle sizes and gradation. Coarser particles are separated in the sieve analysis portion, and the finer particles are analyzed with a hydrometer (75 μm size is chosen to make a distinction between the coarse and fine particles). The sieve analysis is done using an automatic sieve shaker wherein the sample passes progressively through to smaller mesh sizes to assess its gradation. The hydrometer analysis uses the rate of sedimentation to determine particle gradation.

The Atterberg limits are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in any of the four states: solid, semi-solid, plastic and liquid. In each state the consistency and behavior of a soil is different and so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil’s behavior and they are represented by Atterbergs Limits (Liquid Limit - LL; Plastic Limit - PL; Shrinkage Limit - SL). These Atterberg limits can be determined in the laboratory following BIS:2720(Part 5)-1985. The difference between the liquid limit and plastic limit is called the plasticity index (I_p). The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction.

Natural water content (w%) can be calculated as per BIS:2720 Part 2-1973. Specific Gravity, In-situ Density and Moisture Content can be obtained as per BIS:2720 Part 3-1980. Relative Density of cohesionless soils can be evaluated as described in BIS:2720 Part 14-1983. Free swell index of soil as per BIS:2720 (Part XL)–1977 also termed as free swell or differential free swell is the increase in the volume of soil without any external constraint when subjected to submergence in water. Bulk density (γ) is defined as the mass of soil particles of the material divided by the total volume they occupy.

Permeability characteristics of the soils can be determined using falling head or fixed head permeameter as per BIS:2720(Part 17)-1986. Compressibility characteristics can be obtained from odometer tests as per Bureau of Indian Standards (BIS:2720 (Part 15)-1986). Strength characteristics can also be obtained using triaxial, direct shear and vane shear tests.

For Rock Samples: Following tests alongwith BIS codes are used for rock samples:

- Unconfined Compressive Strength of rock samples [BIS:9143-1979]
- Dynamic Modulus of rock core specimen [BIS:10782-1983]
- Modulus of Elasticity, Poisson’s Ratio in uniaxial compression [BIS:9221-1979]
- Point Load Strength Index [BIS:8764-1998].
5.4.2 Tests for shear strength parameters and consolidation characteristics

Tests for shear and consolidation shall be preferably performed on undisturbed samples and in some cases on remolded samples. The direct shear test (Direct shear Test: BIS:2720 Part 13-1986) determines the consolidated drained strength properties of a sample. Test is performed with different normal loads to evaluate the shear strength parameters (c and \( \phi \)). The methods of testing for soils for the determination of Shear Strength parameters of soil from consolidated undrained triaxial compression test with or without pore water measurement are provided in BIS:2720 (Part XII)–1981. Triaxial Shear tests comprise UU, CU (Consolidated Undrained test with and without Pore Water Pressure Measurement) or CD (consolidated drained) tests.

5.5 GROUND RESPONSE ANALYSIS USING ‘SHAKE’ / ‘DEEPSOIL’

SHAKE/ DEEPSOIL are the most commonly used computer programs for carrying out one dimensional ground response analysis of horizontal or gently sloping grounds. SHAKE was initially developed by Schnabel et al. (1972) which was later modified by Idriss and Sun (1992). SHAKE implements equivalent linear approach to model nonlinear behavior of soils. The details of the method have been discussed in Chapter – 3 in Section 3.5.

5.6 EUROCODE-8 AND NEHRP

A site classification scheme based on \( V_{s}^{30} \) values was proposed by Borcherdt (1994) and a similar scheme was adopted by the National Earthquake Hazard Reduction Program (NEHRP) also. The NEHRP (BSSC, 2003) site classification scheme is given in Table 5.7. Eurocode-8 (2003) has also classified the site based on \( V_{s}^{30} \) standard penetration test (SPT) and cone penetration test (CPT) values. The classification given by Eurocode-8 is given in Table 5.8. Eventhough both the schemes use similar methods to identify site classes, the range of \( V_{s}^{30} \) values specified for each site class is different in both the methods.

Table 5.7: NEHRP site classification scheme (BSSC, 2001); \( V_{s}^{30} \) denotes average shear-wave velocity for 30 m upper soil column

<table>
<thead>
<tr>
<th>Site class</th>
<th>( V_{s}^{30} )</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>&gt;1500 m/s</td>
<td>Hard rock</td>
</tr>
<tr>
<td>B</td>
<td>760-1500 m/s</td>
<td>Rock site</td>
</tr>
<tr>
<td>C</td>
<td>360-760 m/s</td>
<td>Soft rock, hard or very stiff soils or gravels</td>
</tr>
<tr>
<td>D</td>
<td>180-360 m/s</td>
<td>Stiff soils</td>
</tr>
<tr>
<td>E</td>
<td>&lt;180</td>
<td>More than 3 m of soft clay defined as soil with plasticity index (PI) &gt; 20, moisture content (w) ( \geq ) 40 percent, and average undrained shear strength (Su) &lt; 25 kPa</td>
</tr>
</tbody>
</table>
Either of the following our categories of soil are considered:

(i) soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils,

(ii) peats and/or highly organic clays (soil thickness > 3 m) of peat and/or highly organic clay,

(iii) very high plasticity clays (soil thickness > 8 m with PI > 75, and

(iv) very thick soft/medium stiff clays (soil thickness > 36 m).

<table>
<thead>
<tr>
<th>Site class</th>
<th>$V_s^{30}$</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>Site-specific evaluation of the soil</td>
<td>Either of the following our categories of soil are considered: (i) soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils, (ii) peats and/or highly organic clays (soil thickness &gt; 3 m) of peat and/or highly organic clay, (iii) very high plasticity clays (soil thickness &gt; 8 m with PI &gt; 75, and (iv) very thick soft/medium stiff clays (soil thickness &gt; 36 m).</td>
</tr>
</tbody>
</table>

Table 5.8: Site classification adopted by Eurocode – 8 (2003)

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Description of Stratigraphic Profile</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$V_s^{30}$ (m/s)</td>
</tr>
<tr>
<td>A</td>
<td>Rock or other rock-like geological formation, including utmost 5 m of weaker material at the surface.</td>
<td>&gt;800</td>
</tr>
<tr>
<td>B</td>
<td>Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.</td>
<td>360 – 800</td>
</tr>
<tr>
<td>C</td>
<td>Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.</td>
<td>180 – 360</td>
</tr>
<tr>
<td>D</td>
<td>Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.</td>
<td>&lt;180</td>
</tr>
<tr>
<td>E</td>
<td>A soil profile consisting of a surface alluvium layer with $V_s^{30}$ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s^{30}$&gt;800 m/s.</td>
<td></td>
</tr>
</tbody>
</table>
### Description of Stratigraphic Profile

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Description of Stratigraphic Profile</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Deposits consisting, or containing a layer at least 10m thick of soft clays/silts with a high plasticity index (PI &gt; 40) and high water content</td>
<td>$V_s^{30}$ (m/s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt;100 (indicative)</td>
</tr>
<tr>
<td>S2</td>
<td>Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1</td>
<td></td>
</tr>
</tbody>
</table>

In many locations the rock depth will be shallow (less than 30 m) and hence the evaluation of $V_s^{30}$ value will not be possible. In those cases, extrapolation of available $V_s$ values has to be done to evaluate the $V_s^{30}$ values. The method proposed by Boore (2004) can be used for this purpose. He suggested different models to extrapolate the shear wave velocities for depths less than 30 m to get the $V_s^{30}$ value. The first method is extrapolation based on constant velocity. In this model it is assumed that the shear wave velocity remains constant from the deepest velocity measurement to the 30 m level as

$$V_s^{30} = \frac{30}{tt(d) + (30 - d)/V_{eff}}$$

(5.5)

Where $tt(d)$ is the travel time to depth $d$ and $V_{eff} = V_s(d)$, $V_s(d)$ is the timed average velocity to a depth of $d$.

Eventhough this method is simple, it is found to underestimate the $V_s^{30}$ values, since in most of the soils, the shear wave velocity is found to increase with depth. Another relation proposed by Boore (2004) was based on a power law relation as:

$$\log V_s^{30} = a + b \log \bar{V_s}(d)$$

(5.6)

Where $\bar{V_s}(d)$ is the velocity at a depth of $d$ m $(10 < d < 30)$. The values of the regression coefficients $a$ and $b$ can be obtained from Boore (2004). The extrapolation of $V_s$ values can also be done based on the velocity statistics (Boore, 2004).

$$P(\xi > V_{eff} / V_s(d)) = a(V_{eff} / V_s(d))^b$$

(5.7)

Where $P(\xi > V_{eff} / V_s(d))$ is the probability of exceedance of $V_{eff} / V_s(d)$. More details of this analysis is reported in Boore (2004).

A modified site classification system based on geotechnical data was proposed by Rodriguez-Marek et al. (2001). In this system the stiffness of soil was also taken into account for the site classification. This system is presented in Table 5.9. The main advantage of this system is that it correlates the $V_s^{30}$ values with the geotechnical and surface geological features.
### Table 5.9: Classification based on geotechnical features (after Rodriguez-Marek et al., 2001)

<table>
<thead>
<tr>
<th>Site</th>
<th>Description</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock</td>
<td>Crystalline bedrock; $V_s^{30} \geq 1500$ m/s</td>
</tr>
<tr>
<td>B</td>
<td>Competent bed rock</td>
<td>$V_s^{30} &gt; 600$ m/s or $&lt; 6$ m of soil. Most unweathered California rock cases</td>
</tr>
<tr>
<td>C1</td>
<td>Weathered rock</td>
<td>$V_s^{30} \sim 300$ m/s increasing to $&gt; 600$ m/s, weathering zone $&gt; 6$ m and $&lt; 30$ m</td>
</tr>
<tr>
<td>C2</td>
<td>Shallow stiff soil</td>
<td>Soil depth $&gt; 6$ m and $&lt; 30$ m</td>
</tr>
<tr>
<td>C3</td>
<td>Intermediate depth stiff soil</td>
<td>Soil depth $&gt; 30$ m and $&lt; 60$ m</td>
</tr>
<tr>
<td>D1</td>
<td>Deep stiff Holocene soil</td>
<td>Soil depth $&gt; 60$ m and $&lt; 200$ m</td>
</tr>
<tr>
<td>D2</td>
<td>Deep stiff Pleistocene soil</td>
<td>Soil depth $&gt; 60$ m and $&lt; 200$ m</td>
</tr>
<tr>
<td>D3</td>
<td>Very deep stiff soil</td>
<td>Soil depth $&gt; 200$ m</td>
</tr>
<tr>
<td>E1</td>
<td>Medium thickness soft clay</td>
<td>Thickness of soft clay layer $3 – 12$ m</td>
</tr>
<tr>
<td>E2</td>
<td>Deep soft clay</td>
<td>Thickness of soft clay layer $&gt; 12$ m</td>
</tr>
<tr>
<td>F</td>
<td>Potentially liquefiable sand</td>
<td>Holocene loose sand with high water table, $Z_w \leq 6$ m</td>
</tr>
</tbody>
</table>

### 5.7 ASSESSMENT OF DYNAMIC PROPERTIES BASED ON OTHER SOIL PROPERTIES

There may be some cases when the evaluation of the shear modulus ($G_{\text{max}}$) is not possible based on any of the above mentioned methods. In those cases the evaluation can be done based on other soil properties which are available. One of the initial relations to find the value of $G_{\text{max}}$ based on void ratio was proposed by Richart et al. (1970).

\[
G_{\text{max}} = 700 \left( \frac{2.17 - e^2}{1 + e} \right) (P^n)^{0.5} \text{ (kgf / cm}^2\text{)} \quad (\text{for rounds sand})
\]

\[
G_{\text{max}} = 330 \left( \frac{2.97 - e^2}{1 + e} \right) (P^n)^{0.5} \text{ (Kgf / cm}^2\text{)} \quad (\text{for angular sand})
\]

Where $e$ is the void ratio and $P^n$ is the effective mean principle stress.

The shear modulus varies with effective stress alone. The relation showing the variation of $G_{\text{max}}$ was proposed by Chung et al. (1984). The effect of soil types on damping ratio was studied by Kokusho (1987).
5.8 SPECTRAL SHAPES

Within the scope of Eurocode 8, the earthquake motions are represented by an elastic ground acceleration response spectrum, dependent on sub-soil class defined and on the magnitude value.

For the application purposes only, two different response spectra, Type 1 and Type 2, have been introduced, Figures 5.8 and 5.10, to be adopted respectively in high (\(M_w > 5.5\)) and low seismicity regions (\(M_w < 5.5\)). The epicentral distance is not considered because it controls the amplitude of the spectra but not their shape, and, therefore, has no effect on normalized spectra. For other applications and tectonic environments, it may be recommended the adoption of more spectral shapes, especially for the very large magnitude values (\(M > 7.5\)), for which \(T_c\) and \(T_d\) may be higher.

![Spectral Shapes](image)

**Figure 5.8:** Type 1 elastic response spectra for the 5 subsoil classes and corresponding soil parameter (S) and control periods (Tb, Tc, Td) (after Ansal, 2006).

It is important to point out that the value of S is large for the Type 2 spectrum than for the Type 1, for all classes other than A. This reflects the non-linear response of soil layers and the fact that weak motion is amplified more than strong motion (Rey et al., 2002).

5.9 CONSTRUCTION OF DESIGN RESPONSE SPECTRA

The design response spectra is defined as a smoothened plot of maximum acceleration as a function of frequency or time-period of vibration for specific damping ratio for earthquake excitations at the base of a single degree of freedom system. They are useful in analyzing the performance of structures under earthquake loading. The procedure for the development of design response spectra as given by Bureau
of Indian Standards (BIS, 2002) anchors the spectral shape to the peak ground acceleration (i.e., zero period spectral acceleration or effective peak ground acceleration). Three spectral shapes are specified for rocky or hard soil sites, medium soil site, and soft soil sites, respectively as depicted in Figure 5.9.

![Figure 5.9: Design response spectra for 5% damping at different soil types; the spectral acceleration coefficient \((Sa/g)\) represents the spectral acceleration scaled by the effective peak ground acceleration defined as the zone factor by BIS (2002).](image)

On the other hand, the scheme given by IBC (2006 and 2009) scales the design spectrum by two spectral ordinates at 0.2 sec and 1.0 sec corresponding to short and long periods, respectively. The amplification factor of acceleration response spectrum for different site class defined by National Earthquake Hazard Reduction Program (NEHRP) are defined for two different spectral periods as listed in Tables 5.10 and 5.11, respectively. The description of NEHRP site classes and comparisons to soil types defined by BIS (2002) is provided in Table 5.7.

**Table 5.10:** Amplification factor for acceleration response spectra at 0.2 sec (after BSSC, 2001). \(S_s\) denote spectral acceleration at 0.2 sec period

<table>
<thead>
<tr>
<th>Site class</th>
<th>(S_s \leq 0.25)</th>
<th>(S_s = 0.50)</th>
<th>(S_s = 0.75)</th>
<th>(S_s = 1.00)</th>
<th>(S_s \geq 1.25)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
<td>1.4</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
<td>1.7</td>
<td>1.2</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>F</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Site specific analysis</td>
</tr>
</tbody>
</table>
Table 5.11: Amplification factor for acceleration response spectra at 1 sec (after BSSC, 2001). $S_i$ denotes spectral acceleration at 1 sec period

<table>
<thead>
<tr>
<th>Site class</th>
<th>Mapped spectral acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_i \leq 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>Site specific analysis</td>
</tr>
</tbody>
</table>

The procedure given by IBC (2006 and 2009) is as follows:

1. Compute the maximum considered earthquake spectral response acceleration at 0.2 sec and 1 sec periods, respectively.

$$S_{MS} = F_a S_s$$

$$S_{ML} = F_v S_i$$

$F_a$ and $F_v$ correspond to amplification factor for acceleration response spectra at 0.2 sec and 1 sec periods, respectively as listed in Tables 5.10 and 5.11. $S_s$ and $S_i$ denote the spectral accelerations at the respective periods.

2. Compute the design basis earthquake spectral response acceleration at 0.2 sec and 1 sec periods respectively.

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{DL} = \frac{2}{3} S_{ML}$$

3. Determine the characteristic time-periods.

$$T_o = 0.2 \frac{S_{DL}}{S_{DS}}$$

$$T_S = \frac{S_{DL}}{S_{DS}}$$

4. Construct the design response spectra as follows.

$$S_a = \begin{cases} 
0.6 \left( \frac{S_{DS}}{T_o} \right) T + 0.4 S_{DS} & T \leq T_o \\
S_{DS} & T \geq T_o \text{ and } T \leq T_S \\
S_{DL}/T & T \geq T_S 
\end{cases}$$
where $S_a$ is the design spectral response acceleration and $T$ is the fundamental time-period of the structure.

![Figure 5.10: Type 2 elastic response spectra for the 5 subsoil classes and corresponding soil parameter (S) and control periods (Tb, Tc, Td) (after Ansal, 2006).](image)

<table>
<thead>
<tr>
<th>ECS-00 TYPE 2</th>
<th>S</th>
<th>Tb</th>
<th>Tc</th>
<th>Td</th>
</tr>
</thead>
<tbody>
<tr>
<td>soil A $V_s &gt; 800$ m/s</td>
<td>1,0</td>
<td>0,05</td>
<td>0,25</td>
<td>1,2</td>
</tr>
<tr>
<td>soil B $360 &lt; V_s &lt; 800$ m/s</td>
<td>1,2</td>
<td>0,05</td>
<td>0,25</td>
<td>1,2</td>
</tr>
<tr>
<td>soil C $180 &lt; V_s &lt; 360$ m/s</td>
<td>1,5</td>
<td>0,10</td>
<td>0,25</td>
<td>1,2</td>
</tr>
<tr>
<td>soil D $V_s &lt; 180$ m/s</td>
<td>1,8</td>
<td>0,10</td>
<td>0,30</td>
<td>1,2</td>
</tr>
<tr>
<td>soil E (h &lt; 20 m)</td>
<td>1,6</td>
<td>0,05</td>
<td>0,25</td>
<td>1,2</td>
</tr>
</tbody>
</table>

In situations where the 30 m depth are not enough to reach a bed-rock type soil formation, as in deep sedimentary basins it might be necessary to introduce more classes, which are essentially the same as above but with specific deep ground geological constitution.

In conclusion, one can say that there are several possibilities to deal with the soil problem in impact studies, depending on the working scale, the knowledge of the geotechnical situation and the software availability. But it also depends on the detail of knowledge of the other components of the entire process for estimating the seismic impact.

**Some typical case studies related to Site Characterization are given in Appendix – V.**
6.1 INTRODUCTION

The secondary phenomena associated with ground shaking during a large earthquake include ground spreading, slumping, soil liquefaction, landslide, rockfalls etc. that contributes to overall seismic risk.

Soil liquefaction is normally associated with large earthquakes. In common usage, liquefaction refers to the loss of strength in saturated, Cohesionless soils due to increasing pore water pressures during dynamic loading. A more precise definition of soil liquefaction is given by Sladen et al. (1985):

“Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shock loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance as shown in the cartoon of Figure 6.1.” In a more general manner, soil liquefaction has been defined as the transformation of saturated Cohesionless soil from a solid state to a liquefied state as a result of increased pore pressure and reduced effective stress.

6.2 LIQUEFACTION HAZARD ASSESSMENT

Liquefaction is one of the most important, interesting, complex, and controversial topics in geotechnical earthquake engineering. Its devastating effects sprang to the attention of geotechnical engineers in a
three-month period in 1964 when the Good Friday earthquake \((M_w = 9.2)\) in Alaska was followed by the Niigata earthquake \((M_s = 7.5)\) in Japan. Both earthquakes produced spectacular examples of liquefaction-induced damage, including slope failures, bridge and building foundation failures, and flotation of buried structures. In the 30 years since these earthquakes, liquefaction has been studied extensively by hundreds of researchers around the world. Different terminologies, procedures, and methods of analysis have been proposed, and a prevailing approach has been slow to emerge.

The term *liquefaction* has been used to describe a number of different, though related phenomena. Rather than try to trace the convoluted development of the current state of knowledge regarding liquefaction, this chapter will present a basic framework for the conceptual understanding of liquefaction-related soil behavior and use it to describe the various methods by which liquefaction hazards can be evaluated.

### 6.2.1 Liquefaction Related Phenomena

The term *liquefaction*, originally coined by Mogami and Kubo (1953) has historically been used in conjunction with a variety of phenomena that involved soil deformations caused by monotonic, transient, or repeated disturbance of saturated cohesionless soils under undrained conditions. The generation of excess pore pressure under undrained loading conditions is a hallmark of all liquefaction phenomena. The tendency for dry cohesionless soils to densify under both static and cyclic loading is well known. When cohesionless soils are saturated rapid loading occurs under undrained conditions, so the tendency for densification causes excess pore pressures to increase and effective stresses to decrease. Liquefaction phenomena that result from this process can be divided into two main groups: *flow liquefaction* and *cyclic mobility*.

Both flow liquefaction and cyclic mobility are very important, and any evaluation of liquefaction hazards should carefully consider both. In the field, flow liquefaction occurs much less frequently than cyclic mobility but its effects are usually far more severe. Cyclic mobility, on the other hand, can occur under a much broader range of soil and site conditions than flow liquefaction; its effects can range from insignificant to highly damaging.

### 6.2.2 Flow Liquefaction

Flow liquefaction produces the most dramatic effects of all the liquefaction-related phenomena – tremendous instabilities known as *flow failures*. Flow liquefaction can occur when the shear stress required for static equilibrium of a soil mass (the *static shear stress*) is greater than the shear strength of the soil in its liquefied state. Once triggered the large deformations produced by flow liquefaction are actually driven by static shear stresses. The cyclic stresses may simply bring the soil to an unstable state at which its strength drops sufficiently to allow the static stresses to produce the flow failure. Flow liquefaction failures are characterized by sudden nature of their origin, the speed with which they develop, and the large distance over which the liquefied materials often move.

### 6.2.3 Cyclic Mobility

*Cyclic mobility* is another phenomenon that can also produce unacceptably large permanent deformations during earthquake shaking. In contrast to flow liquefaction, cyclic mobility occurs when the static shear
stress is less than the shear strength of the liquefied soil. The deformations produced by cyclic mobility failures develop incrementally during earthquake shaking. In contrast to flow liquefaction, the deformations produced by cyclic mobility are driven by both cyclic and static shear stresses. These deformations, termed \textit{lateral spreading}, can occur on very gently sloping ground or on virtually the flat ground adjacent to bodies of water. When structures are present, lateral spreading can cause significant damage.

A special case of cyclic mobility is \textit{level-ground liquefaction}. Because static horizontal shear stresses that could drive lateral deformations do not exist, level-ground liquefaction can produce large, chaotic movement known as \textit{ground oscillation} during earthquake shaking, but produces little permanent lateral soil movement. Level-ground liquefaction failures are caused by the upward flow of water that occurs when seismically induced excess pore pressures dissipate. Depending on the length of time required to reach hydraulic equilibrium, level-ground liquefaction failure may occur well after ground shaking has ceased. Excessive vertical settlement and consequent flooding of low-lying land and the development of sand boils are characteristic of level-ground liquefaction failure.

\section*{6.3 EVALUATION OF LIQUEFACTION HAZARDS}

Both flow liquefaction and cyclic mobility can produce damage at a particular site, and a complete evaluation of liquefaction hazard requires that the potential for each be addressed. When faced with such a problem, the geotechnical earthquake engineer can systematically evaluate potential liquefaction hazards by addressing the following questions:

1. Is the soil susceptible to liquefaction?
2. If the soil is susceptible, will liquefaction be triggered?
3. If liquefaction is triggered, will damage occur?

If the answer to the first question is no, liquefaction hazard evaluation can be terminated with the conclusion that liquefaction hazards do not exist. If the answer is yes, the next question must be addressed. In some cases it may be more efficient to reverse the order of the second and third questions, particularly when damage appears unlikely. If the answers to all three are yes, a problem exists; if the anticipated level of damage is unacceptable, the site must be abandoned or improved or on-site structures strengthened. These questions pertain to the three most critical aspects of liquefaction hazard evaluation: \textit{susceptibility, initiation, and effects}. All three must be considered in a comprehensive evaluation of liquefaction hazards.

\section*{6.4 LIQUEFACTION SUSCEPTIBILITY}

Not all soils are susceptible to liquefaction; consequently, the first step in a liquefaction hazard evaluation is usually the evaluation of liquefaction susceptibility. If the soil at a particular site is not susceptible, liquefaction hazards do not exist and the liquefaction hazard evaluation can be terminated. If the soil is susceptible, however, the matters of liquefaction initiation and the effects must be addressed. There are
several criteria by which liquefaction susceptibility can be judged, and some are different for flow liquefaction and cyclic mobility. These include historical, geologic, compositional, and state criteria.

6.4.1 Historical Criteria

A great deal of information on liquefaction behavior has come from post earthquake field investigations, which have shown that liquefaction often recurs at the same location when soil and groundwater conditions have remained unchanged (Youd, 1984). Thus liquefaction case histories can be used to identify specific sites, or more general site conditions, that may be susceptible to liquefaction in future earthquakes. Youd (1991) described a number of instances where historical evidence of liquefaction has been used to map liquefaction susceptibility.

Postearthquake field investigations have also shown that liquefaction effects have historically been confined to a zone within a particular distance of the seismic source. Ambraseys (1988) compiled worldwide data from shallow earthquakes to estimate a limiting epicentral distance beyond which liquefaction has not been observed in earthquakes of different magnitudes (Figure 6.2). The distance to which liquefaction can be expected increases dramatically with increasing magnitude. While relationships of the types shown in Figure 6.2 offer no guarantee that liquefaction cannot occur at greater distances, they are helpful for the estimation of regional liquefaction hazard scenarios.

![Figure 6.2: Relationship between limiting epicentral distance of sites at which liquefaction has been observed and moment magnitude for shallow earthquakes. Deep earthquakes (focal depths > 50 km) have produced liquefaction at greater distances (after Ambraseys, 1988).](image)

6.4.2 Geological Criteria

Soil deposits that are susceptible to liquefaction are formed within a relatively narrow range of geological environments (Youd, 1991). The depositional environment, hydrological environment, and age of a soil deposit all contribute to its liquefaction susceptibility (Youd and Hoose, 1977).
Geologic processes that sort soils into uniform grain size distributions and deposit them in loose states produce soil deposits with high liquefaction susceptibility. Consequently, fluvial deposits, and colluvial and aeolian deposits when saturated, are likely to be susceptible to liquefaction. Liquefaction has also been observed in alluvial-fan, alluvial-plain, beach, terrace, playa, and estuarine deposits, but not as consistently as in those listed previously. The susceptibility of older soil deposits to liquefaction is generally lower than that of newer deposits. Soils of Holocene age are more susceptible than soils of Pleistocene age, although susceptibility decreases with age within the Holocene. Liquefaction of pre-Pleistocene deposits is rare.

Liquefaction occurs only in saturated soils, so the depth to groundwater (either free or perched) influences liquefaction susceptibility. Liquefaction susceptibility decreases with increasing groundwater depth; the effects of liquefaction are most commonly observed at sites where groundwater is within a few meters of the ground surface. At sites where groundwater levels fluctuate significantly, liquefaction hazards may also fluctuate.

Human-made soil deposits also deserve attention. Loose fills, such as those placed without compaction, are very likely to be susceptible to liquefaction. The stability of hydraulic fill dams and mine tailings piles, in which soil particles are loosely deposited by setting through water remains an important contemporary seismic hazard.

6.4.3 Compositional Criteria

Since liquefaction requires the development of excess pore pressure, liquefaction susceptibility is influenced by the compositional characteristics that influence volume change behavior. Compositional characteristics associated with high volume change potential are associated with high liquefaction susceptibility. These characteristics include particle size, shape, and gradation.

For many years, liquefaction-related phenomena were thought to be limited to sands. Finer-grained soils were considered incapable of generating high pore pressures commonly associated with liquefaction, and coarser-grained soils were considered too permeable to sustain any generated pore pressure long enough for liquefaction to develop. More recently, the bounds on gradation criteria for liquefaction susceptibility have broadened.

Liquefaction of nonplastic silts has been observed (Ishihara, 1984 and 1985) in the laboratory and the field, indicating that plasticity characteristics rather than grain size alone influence the liquefaction susceptibility of fine-grained soils. Coarse silts with bulky particle shape, which is nonplastic and cohesionless, are fully susceptible to liquefaction (Ishihara, 1993); finer silts with flaky or platelike particles generally exhibit sufficient cohesion to inhibit liquefaction. Clays remain nonsusceptible to liquefaction, although sensitive clays can exhibit strain-softening behavior similar to that of liquefied soil. Fine-grained soils that satisfy each of the following four Chinese criteria (Wang, 1979) may be considered susceptible to significant strength loss:

- Fraction finer than 0.005 mm \( \leq 15\% \)
- Liquid limit, LL \( \leq 35\% \)
- Natural water content \( \geq 0.9 \) LL
- Liquidity index \( \leq 0.75 \).
To account for the differences in Chinese and U.S. practice, the U.S. Army Corps of Engineers modified the measured index properties (by decreasing the fines content by 5%, increasing the liquid limit by 1%, and increasing the natural water content by 2%) before applying the Chinese criteria to a clayey silt in the foundation of Sardis Dam (Finn et al., 1994).

On the other end of the grain size spectrum, liquefaction of gravels has been observed in the field (Coulter and Migliaccio, 1966; Wong, 1984; Youd et al., 1985; Yegian et al., 1994) and in the laboratory (Wong et al., 1974; Evans and Seed, 1987). The effects of membrane penetration are now thought to be responsible for high liquefaction resistance observed in early laboratory investigations of gravelly soils.

Liquefaction susceptibility is influenced by gradation. Well-graded soils are generally less susceptible to liquefaction than poorly graded soils; the filling of voids between larger particles by smaller particles in a well-graded soil results in lower volume change potential under drained conditions, and consequently lower excess pore pressures under undrained conditions.

Particle shape can also influence liquefaction susceptibility. Soils with rounded particle shapes are known to densify more easily than soils with angular grains. Consequently, they are usually more susceptible to liquefaction than angular-grained soils. Particle founding frequently occurs in the fluvial and alluvial environments where loosely deposited saturated soils are frequently found, and liquefaction susceptibility is often high in those areas.

6.4.4 State Criteria

Even if a soil meets all of the preceding criteria for liquefaction susceptibility, it still may or may not be susceptible to liquefaction. Liquefaction susceptibility also depends on the initial state of the soil (i.e., its stress and density characteristics at the time of the earthquake). Since the tendency to generate excess pore pressure of a particular soil is strongly influenced by both density and initial stress conditions, liquefaction susceptibility depends strongly on the initial state of the soil. These liquefaction susceptibility criteria are different for flow liquefaction and cyclic mobility.

6.4.4.1 Critical Void Ratio

![Figure 6.3: (a) Stress-strain and (b) stress-void ratio curves for loose and dense sands at the same effective confining pressure. Loose sand exhibits contractive behavior (decreasing void ratio) and dense sand exhibits dilative behavior (increasing void ratio) during shearing. By the time large strains have developed, both specimens have reached the critical void ratio and mobilize the same large-strain shearing resistance (after Kramer, 1996).](image)
At large strains, all specimens approach the same density and continue to shear with constant shearing resistance. Figure 6.3 depicts the stress-strain and stress-void ratio curves for loose and dense sands. The void ratio corresponding to this constant density was termed the critical void ratio, $e_c$. By performing tests at different effective confining pressures, it has been found that the critical void ratio is uniquely related to the effective confining pressure, and called the locus the Critical Void Ratio (CVR) line (Figure 6.4). By defining the state of the soil in terms of void ratio and effective confining pressure, the CVR line could be used to mark the boundary between loose (contractive) and dense (dilative) states.

**Figure 6.4:** Use of the CVR line as a boundary between loose contractive states and dense dilative states (after Kramer, 1996).

Since the CVR line marked the boundary between contractive and dilative behavior, it is considered to mark the boundary between the states in which a particular soil is or is not susceptible to flow liquefaction (Figure 6.5). Saturated soils with initial void ratios high enough to plot above the CVR line are considered susceptible to flow liquefaction, and soils with initial states plotting below the CVR line are considered nonsusceptible.

**Figure 6.5:** Use of CVR line as a boundary between initial states that are and are not susceptible to flow liquefaction (after Kramer, 1996).

### 6.4.4.2 Steady State of Deformation

Performing (Castro, 1969) static and cyclic triaxial tests on isotropically consolidated specimens and several static tests on anisotropically consolidated specimens, three different types of stress – strain behavior, illustrated for anisotropically consolidated specimens in Figure 6.6 are observed. Very loose specimens (such as specimen A in Figure 6.6) exhibited a peak undrained strength at a small shear strain and then “collapses” to flow rapidly to large strains at low effective confining pressure and low large-strain strength. Dense specimens (specimen B) initially contracted but then dilated until a relatively high constant effective confining pressure and large-strain strength was reached. At intermediate densities (specimen C) the exceedance of peak strength at low strain was followed by a limited period of strain-softening behavior, which ended with the onset of dilation at
intermediate strains. Further loading produced continued dilation to higher effective confining pressures and, consequently, higher large-strain strengths. This type of behavior is termed *limited liquefaction.*

![Figure 6.6](image)

**Figure 6.6:** Liquefaction, limited liquefaction, and dilation in monotonic loading tests (after Kramer, 1996).

![Figure 6.7](image)

**Figure 6.7:** Three-dimensional steady-state line showing projections on \( e - \tau \) plane, \( e - \sigma' \) plane, and \( \tau - \sigma' \) plane. A similar plot can be developed using the stress path parameters \( q \) and \( p' \) instead of \( \tau \) and \( \sigma' \) (after Kramer, 1996).

The locus of the points describing the relationship between void ratio and effective confining pressure in the steady state of deformation is called the *Steady-State Line* (SSL). In its most general form, the SSL can view as a three-dimensional curve in \( e - \sigma' - \tau \) (Figure 6.7) or \( e - p' - q \) space. The SSL can also be projected onto planes of constant effective confining pressure (\( \sigma' \) constant) and density (\( e \) constant). The SSL can also be expressed in terms of the steady-state strength, \( S_{su} \); since the shearing resistance of the soil in the steady state of deformation is proportional to the effective confining pressure, the strength-based SSL is parallel to the effective confining pressure-based SSL when both are plotted on logarithmic scales (Figure 6.8).
Proportionality of $S_{su}$ to $\sigma'_3$ produces strength-based and effective confining pressure-based steady-state lines with identical slopes (after Kramer, 1996).

State criteria for flow liquefaction susceptibility. Soils with combinations of initial density and stress conditions that plot above the SSL are susceptible to flow liquefaction when the static shear strength is greater than the steady-state strength. Initial conditions that plot below the SSL are not susceptible to flow liquefaction (after Kramer, 1996).

The SSL is useful for identifying the conditions under which a particular soil may or may not be susceptible to flow liquefaction (Figure 6.9). Soils whose condition plots below the SSL are not susceptible to flow liquefaction. A soil whose state lies above the SSL will be susceptible to flow liquefaction only if the static shear exceeds its steady state (or residual) strength. Since the SSL can be used to evaluate the shearing resistance of liquefied soils, it is also useful for evaluating the potential effects of liquefaction. Although determination of the position of the SSL can be difficult in practice, the SSL is very useful for understanding the basic concepts of liquefaction.

Cyclic mobility, on the other hand, can occur in soils whose state plot above or below the SSL. In other words, cyclic mobility can occur in both loose and dense soils.

The location of the SSL is sensitive to the compositional characteristic of the soil – its vertical position is strongly influenced by gradation and its slope by particle angularity. Soils with rounded particles usually have flat SSLs – a characteristic that often leads to difficulty in the estimation of in-situ steady-state strength.

6.4.4.3 State Parameter

The nature of the steady-state line illustrates the limited applicability of absolute measures of density, such as void ratio and relative density, for characterization of a potentially liquefiable soil. As illustrated in Figure 6.9, an element of soil at a particular void ratio (hence a particular density and relative density) can be susceptible to flow liquefaction under a high effective confining pressure but nonsusceptible at a low effective confining pressure.
Using concepts of critical-state soil mechanics, the behavior of a cohesionless soil should be more closely related to the proximity of its initial state to the steady-state line than to absolute measures of density (Roscoe and Poorooshab, 1963). In other words, soils in states located at the same distance from the steady-state line should exhibit similar behavior. Using the logic, a state parameter (Been and Jeffries, 1985) can be defined as

$$\psi = e - e_{ss}$$  \hspace{1cm} (6.1)

Where $e_{ss}$ is the void ratio of the steady-state line at the effective confining pressure of interest (Figure 6.10). When the state parameter is positive, the soil exhibits contractive behavior and may be susceptible to flow liquefaction. When it is negative, dilative behavior will occur and the soil is not susceptible to flow liquefaction. The state parameter has been related to friction angle, dilation angle, CPT resistance (Been et al., 1986 and 1987; Sladen, 1985), PMT results (Yu, 1994), and DMT results (Konrad, 1988). Ishihara (1993) showed that the ability of the state parameter to characterize soil behavior of very loose sands under low effective confining pressures may be limited and proposed an analogous parameter (the state index) based on the relative distance between the initial state and the quasi-steady-state line (a line analogous to and located slightly below the SSL which corresponds to the stress and density conditions at the phase transformation points observed in cases of limited liquefaction).

The concept of the state parameter is very useful and the possibility of determining its value from in-situ tests is appealing. The accuracy with which the state parameter can be determined, however, is influenced by the accuracy with which the position of the SSL can be determined.

### 6.5 IDENTIFICATION OF LIQUEFIABLE SOIL AND LIQUEFACTION STUDIES

The following soils must be considered as potential subjects of liquefaction:

- Muddy sand and silt with the following characteristics –
  - degree of water saturation around 100%
  - Diameter at 50%, $D_{50}$, between 0.05 mm and 1.5 mm.
b) Clayey soil with the following characteristics –
  - diameter at 15%, $D_{15}$, greater than 0.005 mm
  - liquid limit, $W_L$, less than 35%;
  - water content greater than 0.9 $W_L$;
  - Representative point on the plasticity diagram located above the straight line A (Figure 6.6).

In contrast, the following soil types may be considered as exempt from risk:

a) Soil with a grain-size diameter at 10% $D_{10}$, greater than 2mm.

b) Soils which simultaneously have:
  - $D_{70} < 74 \mu m$;
  - $I_p > 10\%$ (Plasticity index).

Where the indications above reveal a possibility of liquefaction, there are good grounds for quantifying the liquefaction risk by the methods described below.

### 6.5.1 Level A studies

An initial qualitative study of the sensitivity to liquefaction is carried out using geological, hydrogeological and geomorphological data. The following geological and piezometric criteria are added to the above, in order to judge the susceptibility to liquefaction:

### SUSCEPTIBILITY OF SEDIMENTARY DEPOSITS TO LIQUEFACTION AS A FUNCTION OF THE TYPE AND AGE OF THE DEPOSIT

(after Youd and Perkins, 1978)

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>Possibility of liquefaction occurrence in saturated non-cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;500 ys</td>
</tr>
<tr>
<td>CONTINENTAL DEPOSITS</td>
<td></td>
</tr>
<tr>
<td>Fluvial</td>
<td>very high</td>
</tr>
<tr>
<td>Alluvial plain</td>
<td>high</td>
</tr>
<tr>
<td>Aeolian</td>
<td>moderate</td>
</tr>
<tr>
<td>Marine</td>
<td>-</td>
</tr>
<tr>
<td>terraces</td>
<td>high</td>
</tr>
<tr>
<td>Delta</td>
<td>high</td>
</tr>
<tr>
<td>Lacustrine</td>
<td>high</td>
</tr>
<tr>
<td>Colluvium</td>
<td>high</td>
</tr>
</tbody>
</table>
### CONTINENTAL DEPOSITS

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>Possibility of liquefaction occurrence in saturated non-cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;500ys</td>
</tr>
<tr>
<td></td>
<td>&lt;10,000ys</td>
</tr>
<tr>
<td>Dunes</td>
<td>high</td>
</tr>
<tr>
<td>Loess</td>
<td>low</td>
</tr>
<tr>
<td>Glacial till</td>
<td>high</td>
</tr>
</tbody>
</table>

### COASTAL AREAS

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>Possibility of liquefaction occurrence in saturated non-cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Recent</td>
</tr>
<tr>
<td>Delta</td>
<td>very high</td>
</tr>
<tr>
<td>Estuary</td>
<td>high</td>
</tr>
<tr>
<td>Beach</td>
<td>moderate/high</td>
</tr>
<tr>
<td>Lagoon</td>
<td>high</td>
</tr>
</tbody>
</table>

### ARTIFICIAL BACKFILL

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>Possibility of liquefaction occurrence in saturated non-cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>uncompacted</td>
<td>very high</td>
</tr>
<tr>
<td>compacted</td>
<td>low</td>
</tr>
</tbody>
</table>

### SUSCEPTIBILITY OF SEDIMENTARY DEPOSITS TO LIQUEFACTION AS A FUNCTION OF THEIR AGE AND THE POSITION OF THE WATER TABLE

(after Youd and Perkins, 1978)

<table>
<thead>
<tr>
<th>Age of deposit</th>
<th>Water Table depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 - 3</td>
</tr>
<tr>
<td>Recent</td>
<td>high/very high</td>
</tr>
<tr>
<td>Holocene</td>
<td>high</td>
</tr>
<tr>
<td>Holocene</td>
<td>high</td>
</tr>
<tr>
<td>Recent</td>
<td>low</td>
</tr>
</tbody>
</table>

The zonation at Level A will result in a map of susceptibility to liquefaction. It will comprise of three categories:

- zones of very low or non-existent susceptibility
- zones of low to moderate susceptibility
- zones of high to very high susceptibility.
6.5.2 Level B studies

Level B studies, and, therefore, those at Level C, use the mechanical characteristics of the soil to create a “liquefaction” hazard map.

In order to accomplish this, a simplified method is used, originally developed by Seed and based on the use of Standard Penetration Tests (SPT). A certain number of boreholes, and SPT and/or Cone Penetration Tests (CPT) are required.

The cyclic shear stress $\tau_e$ generated by an earthquake at a given depth where the total vertical stress is $\sigma_v$ is

$$\tau_e = 0.65 \frac{a_{\text{max}}}{g} \sigma_v r_d$$  \hspace{1cm} (6.2)

Where $a_{\text{max}}$ is the maximum acceleration at the surface, derived from the seismic-hazard study, possibly modified to take into consideration the site effects, and $r_d$ is a coefficient of stress attenuation as a function of soil rigidity. We can assume that:

$$r_d = 1 - 7.5 \times 10^{-4} z^2 \quad 0 \leq z \leq 20 \text{ m}$$  \hspace{1cm} (6.3)

The soil cyclic shear strength is calculated from its resistance to standard penetration. If $N$ is the number of SPT blow, we can define:

$$N_i = \left( \frac{100}{\sigma'_{v}} \right)^{1/2} \cdot N$$  \hspace{1cm} (6.4)

Where $\sigma'_{v}$, the effective vertical stress is expressed in kN/m$^2$.

Cyclic shear strength is given by the equation:

$$\tau_e = A(N_i)^{0.755} \sigma'_{v}^{0.06N_i}$$  \hspace{1cm} (6.5)

Where the coefficient $A$ is dependent on the fines content of the soil, and on the zone of seismicity. The following values adapted from Ambraseys (1988) are appropriate:

<table>
<thead>
<tr>
<th>Fines content of soil</th>
<th>$\approx 5%$</th>
<th>$&gt; 10%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismicity zone</td>
<td>$\text{I}$</td>
<td>$\text{II}$</td>
</tr>
<tr>
<td>A</td>
<td>$1.7 \times 10^{-2}$</td>
<td>$1 \times 10^{-2}$</td>
</tr>
</tbody>
</table>

N.B.: The values in zone III for material with more than 10% fines are slightly conservative due to the lack of experimental data.
The safety coefficient is given by:

\[ F_S = \frac{\tau_r}{\tau_c} \]  

(6.6)

The result of the analysis is generally presented in the map format, with four zones distinguished according to the value of the safety coefficient:

- no liquefaction: non-liqueifiable zones or \( F_S \geq 2.0 \)
- liquefaction unlikely: \( 1.5 \leq F_S < 2.0 \)
- liquefaction probable: \( 1.0 \leq F_S < 1.5 \)
- liquefaction almost certain: \( F_S < 1.0 \)

The maximum investigation depth should be 25 m.

The geotechnical investigations required for the evaluation of liquefaction hazard are not generally sufficiently numerous to allow the tracing of precise zone boundaries, particularly where the study area is very large. Geological criteria are, therefore, used making sure that quantitative mechanical information according to the type of sedimentary deposit is included.

### 6.5.3 Level C studies

This level of study is distinguished from Level B by the volume of investigations involved. It will commonly be desirable to carry out complementary drilling with SPT and CPT. Undisturbed samples should be taken in suspect formations.

The cyclic shear stress produced by an earthquake will be calculated on some characteristic stratigraphic profiles by dynamic computation of wave propagation. These calculations will enable further refinement of the variation of the coefficient \( r_d \), particularly where profiles are very different.

Undrained cyclic shear strength will be further controlled through cyclic triaxial laboratory tests, for areas where Level B studies revealed a safety coefficient between 1 and 1.5 (liquefaction probable).

For shallow foundations of small buildings, the impact of depth and thickness of the liquefiable layer will be evaluated. The coefficient of safety, \( F_S \), being calculated as indicated in the following integral can be evaluated:

\[ I_c = \int_0^{20} (10 - 0.5z) F_L \, dz \]  

(6.7)

Where,

\[ F_L = 1 - F_S \quad \text{When } F_S \leq 1 \]

\[ F_L = 0 \quad \text{When } F_S \geq 1 \]
The value \( I_L \) varies between 0 for a non-liquefiable zone and 100 for an area where liquefaction is highly likely. The result of the analysis will be presented in map format, with four zones distinguished according to their \( I_L \) value:

- no liquefaction non-liquefiable zone or \( I_L = 0 \)
- liquefaction unlikely \( 0 < I_L \leq 5 \)
- liquefaction probable \( 5 < I_L \leq 15 \)
- liquefaction almost certain \( I_L > 15 \).

### 6.6 LIQUEFACTION SUSCEPTIBILITY MAP

Liquefaction ‘susceptibility’ is a measure of a soil’s inherent resistance to liquefaction, and can range from not susceptible, regardless of seismic loading, to highly susceptible, which means that very little seismic energy is required to induce liquefaction. Susceptibility has been evolved by comparing the properties of top soil deposits of the region under study to the other soil deposits where liquefaction has been observed in the past (based on Seed et al., 1985). Liquefaction susceptibility is evaluated based on the primary relevant soil properties such as grain size, fine content, and density, degree of saturation, SPT-N values and age of the soil deposit in each of the borelogs. These susceptible areas are identified by considering the approach of Pearce and Baldwin (2005). Soil is susceptible for liquefaction if (1) there are presence of sand layers at depths less than 20 m, (2) it encounters water table depth less than 10 m, and (3) SPT value ‘N’ blow counts less than 20. By interpolation, susceptibility of map can be prepared.

### 6.7 FACTOR OF SAFETY AGAINST LIQUEFACTION ASSESSMENT

Factor of Safety against liquefaction of soil layer are evaluated based on the simplified procedure (Seed and Idriss, 1971) and subsequent revisions of the simplified procedures (Seed et al., 1983 and 1985; Youd et al., 2001; Cetin et al., 2004). In this study, the earthquake induced loading is expressed in terms of cyclic shear stress and this is compared with the liquefaction resistance of the soil. Liquefaction estimation requires two variables for evaluation of liquefaction resistance of soils. Two variables are defined based on cyclic stress approaches which are as follows:

1. The seismic demand of a soil layer is represented by a Cyclic Stress Ratio (CSR).
2. The capacity of soil to resist liquefaction represented by Cyclic Resistance Ratio (CRR).

Here liquefaction resistance is estimated using an in-situ test based on corrected SPT-N values. Steps involved in the calculation of liquefaction hazard are shown in Figure 6.11 as a flow chart.
6.7.1 Factor of Safety against Liquefaction

If the cyclic stress ratio caused by the earthquake is greater than the cyclic resistance ratio of in-situ soil, then liquefaction could occur during an earthquake. The factor of safety against liquefaction is defined as follows

\[
FS = \left( \frac{CRR_{7.5}}{CSR} \right)^{MSF}
\]  

(6.8)

Here subscript 7.5 for CRR denotes that CRR values calculated for the earthquake moment magnitude of \( M_w 7.5 \). MSF is the magnitude scaling factor. The higher factor of safety means that soil is having more resistance to liquefaction.

6.7.2 Peak Ground Acceleration

Estimation of factor of safety against liquefaction of soil layer requires the ground level peak acceleration due to an earthquake. Ground level peak horizontal accelerations have been estimated by the site
response studies using equivalent linear response analysis software (using SHAKE 2000 program). This peak ground acceleration is further used to estimate the cyclic stress ratio (CSR).

**6.7.3 Cyclic Stress Ratio (CSR)**

The excess pore pressure generation to initiate liquefaction depends on the amplitude and the duration of the earthquake induced cyclic loading. In the cyclic stress approach the pore pressure generation is related to the cyclic shear stresses, hence the earthquake loading is represented in terms of cyclic shear stresses. The earthquake loading can be evaluated by using Seed and Idriss (1971) simplified approach. The earthquake loading is evaluated in terms of uniform cyclic shear stress amplitude and it is as given below:

\[
Cyclic \ Stress \ Ratio \ (CSR) = 0.65 \left( \frac{a_{\text{max}}}{g} \right) \left( \frac{\sigma_{\text{vo}}}{\sigma_{\text{vo}'}} \right) r_d
\]  

(6.9)

In this equation \(0.65 \frac{a_{\text{max}}}{g}\) represents 65% of the peak cyclic shear stress, \(a_{\text{max}}\) is the peak ground surface acceleration, \(g\) is the acceleration due gravity, \(\sigma_{\text{vo}}\) and \(\sigma_{\text{vo}'}\) are the total and effective vertical stresses and \(r_d\) = stress reduction coefficient. For the calculation of stress reduction coefficient many correlations are available which are discussed in detail in a 1996 NCEER workshop report (Youd et al., 2001). Youd et al. (2001) recommends that for routine practice and non-critical projects, the equations given by Liao and Whitman (1986) may be used to estimate average values of given below:

\[
r_d = 1.0 - 0.0765z \quad \text{for } z \leq 9.15 \text{ m} \quad (6.10)
\]

\[
r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \quad (6.11)
\]

**6.7.4 Cyclic Resistance Ratio (CRR)**

Liquefaction resistance of soil depends on how close the initial state of soil is to the state corresponding to “failure”. The liquefaction resistance can be calculated based on laboratory tests and in-situ tests. Here, liquefaction resistance using in-situ test is based on SPT ‘N’ values usually attempted. Cyclic Resistance Ratio (CRR) is arrived at based on the corrected ‘N’ value as per Seed et al. (1985), Youd et al. (2001) and Cetin et al. (2004). Seed et al. (1985) presents a plot of CRR versus corrected ‘N’ value from a large volume of laboratory and field data. However, the corrected ‘N’ values are used to calculate CRR for the magnitude of \(M_w 7.5\) earthquakes using the equation proposed by Idriss and Boulanger (2005) as given below:

\[
CRR = \exp \left[ \left( \frac{(N_1)_{60\text{cm}}}{14.1} \right) + \left( \frac{(N_1)_{60\text{cm}}}{126} \right) - \left( \frac{(N_1)_{60\text{cm}}}{23.6} \right) + \left( \frac{(N_1)_{60\text{cm}}}{25.4} \right)^4 - 2.8 \right]
\]  

(6.12)

**6.7.5 Magnitude Scaling Factor (MSF)**

The CRR curves are generated either using the SPT-N values or CPT \(q_c\) values or shear wave velocity \(V_s\) corresponding to an earthquake of magnitude \(M_w 7.5\). Seed and Idriss (1982) suggested the use of
Magnitude Scaling Factors (MSF) for earthquakes of magnitude other than $M_w$ 7.5. The available MSF are Seed and Idriss (1982) scaling factors; Revised Idriss (1985) scaling factors; Ambraseys (1988) scaling factors; Arango (1996) scaling factors; Andrus and Stokoe (1997) scaling factors and Youd and Noble (1997) scaling factors. Detailed discussion and comparison of these scaling factors are available in Youd et al. (2001) and Bhandari et al. (2003). An NCEER - 1996 and 1998 NCEER/NSF workshop (Youd et al., 2001) recommends the revised Idriss (1985) scaling factors and it was used by Yilmaz and Bagci (2006) for soil liquefaction susceptibility and hazard mapping in the residential area of Kutahya (Turkey). The magnitude-scaling factor used in the present study is the revised Idriss scaling factor for the magnitude less than $M_w$ 7.5 and it is given as

$$MSF = \left[ \frac{10^{2.24}}{M_w^{2.50}} \right]$$ (6.13)

### 6.7.6 Factor of Safety Calculation

Using the available geotechnical borelog data base, after applying necessary corrections to SPT-N values, corrected ‘N’ $(N_{100})$ values are obtained. A simple excel spread sheet is developed to automate these calculations for all the borelogs with depth. The factor of safety for each layer of soil is arrived at considering corresponding '(N<sub>100</sub>)' values. Typical liquefaction analysis is shown in (Table 6.1). It is to be noted here that, apart from Seed et al. (1983) recommendation, the fines content in the soil are considered using representative parameters such as Liquid Limit (LL) and Plasticity Index (PI). The soil having the liquid limit of more than 32 is recommended (Boulanger and Idriss, 2004) for the detail study (DS), which can account for the strength loss in plastic silts or clays during cyclic or seismic loading.

**Table 6.1: Typical liquefaction analysis for a borehole**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Corrected N value $(N_{100})$</th>
<th>$\sigma_{vo}$ kN/m$^2$</th>
<th>$\sigma_{vo}'$ kN/m$^2$</th>
<th>$r_d$</th>
<th>CSR</th>
<th>FC %</th>
<th>Liquid Limit</th>
<th>CRR</th>
<th>MSF</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.50</td>
<td>4</td>
<td>30.00</td>
<td>30.00</td>
<td>0.99</td>
<td>0.22</td>
<td>46.2</td>
<td>0</td>
<td>0.08</td>
<td>2.68</td>
<td>0.94</td>
</tr>
<tr>
<td>3.20</td>
<td>3</td>
<td>64.00</td>
<td>47.32</td>
<td>0.98</td>
<td>0.30</td>
<td>40.9</td>
<td>0</td>
<td>0.08</td>
<td>2.68</td>
<td>0.69</td>
</tr>
<tr>
<td>4.20</td>
<td>21</td>
<td>84.00</td>
<td>74.19</td>
<td>0.97</td>
<td>0.25</td>
<td>53.3</td>
<td>26</td>
<td>0.22</td>
<td>2.68</td>
<td>2.36</td>
</tr>
<tr>
<td>5.20</td>
<td>20</td>
<td>104.00</td>
<td>94.19</td>
<td>0.96</td>
<td>0.24</td>
<td>53.1</td>
<td>31</td>
<td>0.21</td>
<td>2.68</td>
<td>2.35</td>
</tr>
<tr>
<td>7.00</td>
<td>44</td>
<td>140.00</td>
<td>122.34</td>
<td>0.95</td>
<td>0.25</td>
<td>57.1</td>
<td>25</td>
<td>19.54</td>
<td>2.68</td>
<td>NL</td>
</tr>
<tr>
<td>8.50</td>
<td>102</td>
<td>170.00</td>
<td>155.29</td>
<td>0.93</td>
<td>0.23</td>
<td>59.2</td>
<td>27</td>
<td>NL</td>
<td>2.68</td>
<td>NL</td>
</tr>
</tbody>
</table>
6.7.7 Liquefaction Hazard Map

Liquefaction hazard mapping has been done by many researchers using SPT data (Palmer et al., 2003; Brankman et al., 2004; Pearce and Baldwin, 2005; Yilmaz and Bagci, 2006). The liquefaction hazard map is prepared for the desired maximum moment magnitude. The minimum factor of safety from each borelog is considered to represent the factor of safety against liquefaction at that location, which are used for mapping. These factors of safety against liquefaction are grouped into 4 as shown in Table 6.2.

<table>
<thead>
<tr>
<th>Group</th>
<th>Factor of safety range</th>
<th>Severity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;1</td>
<td>Very Critical</td>
</tr>
<tr>
<td>2</td>
<td>1 to 2</td>
<td>Critical</td>
</tr>
<tr>
<td>3</td>
<td>2 to 3</td>
<td>Low critical</td>
</tr>
<tr>
<td>4</td>
<td>&gt;3</td>
<td>Non liquefiable</td>
</tr>
</tbody>
</table>

6.8 SAMPLE COMPUTATION OF LIQUEFACTION HAZARD IN TERMS OF FACTOR OF SAFETY

The simplified procedure for liquefaction analysis of Seed and Idriss (1971) is widely used globally. A soil column is considered to be a rigid body, which gets excited at the base with the propagation of shear wave to the ground surface. The shear stress is generated in the soil column and can be calculated through the following,

\[
(\tau_{\text{max}})_{r} = \sigma_{0} \times \frac{a_{\text{max}}}{g}
\]

(6.14)

Where \( (\tau_{\text{max}})_{r} \) denotes maximum shear stress for rigid body, \( \sigma_{0} \) is the total overburden pressure, \( a_{\text{max}} \) is the peak horizontal acceleration on the ground surface, and \( g \) represents acceleration due to gravity.

In reality, soil behaves as a deformable body instead of as a rigid body. Hence, the rigid body shear stress should be reduced with a correction factor to give the deformable body maximum shear stress \( (\tau_{\text{max}})_{d} \). This correction factor is called the stress reduction coefficient \( r_{d} \) and can be computed as follows,

\[
(\tau_{\text{max}})_{d} = r_{d} (\tau_{\text{max}})_{r}
\]

(6.15)

The value of the stress reduction coefficient decreases with depth to a value of unity on the ground surface as shown in Figure 6.12.
The following equations have been derived for average value of $r_d$ (Cetin et al., 2004),

$$
\begin{align*}
\frac{1}{r_d(d, M_w, a_{max})} &= \left[1 + \frac{-9.147 - 4.173 \cdot a_{max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.069(-d^{0.328} - 7.760\ln(a_{max} + 78.576))}}\right] ^{-1}, & d < 20 \text{ m} \\
\frac{1}{r_d(d, M_w, a_{max})} &= \left[1 + \frac{-9.147 - 4.173 \cdot a_{max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.069(-7.760\ln(a_{max} + 78.576))}}\right] ^{-0.0014(d^{3.28} - 65)}, & d \leq 20 \text{ m}
\end{align*}
$$

(6.16) (6.17)

The above relations do not incorporate shear wave velocity. Different relations are available in case shear wave velocity is to be employed (Cetin et al., 2004).

On substituting, Equation (6.14) in Equation (6.15), the maximum shear stress for the deformable body $(\tau_{max})_d$ can be calculated as,

$$
(\tau_{max})_d = r_d \cdot \sigma_0 \cdot \frac{a_{max}}{g}
$$

(6.18)

Seed and Idriss (1971) suggested that a value of 65% of the maximum shear stress $(\tau_{max})$ is reasonably accurate. They based their prediction by appropriate weighting of laboratory test data. Therefore, Equation (6.18) can be written in terms of equivalent average of shear stress $(\tau_{ave})$. 

**Figure 6.12:** The variation of stress reduction factor $(r_d)$ with depth (after Cetin et al., 2004).
Earthquake Induced Hazard - Soil Liquefaction

If the equivalent average of shear stress ($\tau_{ave}$) is normalized with the initial effective overburden pressure $\sigma_0'$, the term is called the seismic demand of a soil layer or CSR (Cyclic Stress Ratio).

$$CSR = \frac{\tau_{ave}}{\sigma_0'} = 0.65 * \frac{\sigma_0' * a_{max} * r_d}{g}$$  \hspace{1cm} (6.20)

Another parameter that is employed is the Cyclic Resistance Ratio (CRR). It represents the capacity of the soil to resist liquefaction. The SPT-N value data has been computed at the depth spacing of 1.5m at each of the boreholes in the study region, say Guwahati region in the present case (Figure 6.13). The effective overburden pressure is given as below,

$$\sigma_{io} = d_j \left( \sum d_i * \gamma_i \right) - \left[ (d_j - w.l.) * 9.81 \right]$$  \hspace{1cm} (6.21)

Where $d_j$ is the depth interval of each N-value, $d_i$ is the depth at which the soil lithology changes and $\gamma_i$ is the average unit weight of the material.

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<td>0.45</td>
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</table>

Figure 6.13: Represent a computation table at a borehole site (after Sahai, 2009).

According to the classifications given by Youd et al. (2001) (Figure 6.14), three relations can be formulated for the increasing level of silt content or fine content (FC) in the soil. These are given as follows,
y = 0.00000031x^5 - 0.000021x^4 + 0.00053x^3 - 0.0054x^2 + 0.03x + 0.011, FC = 5\% \quad (6.22)

y = 0.0000012x^5 - 0.000087x^4 + 0.0024x^3 - 0.03x^2 + 0.18x - 0.32, FC = 15\% \quad (6.23)

y = 0.00000127x^5 - 0.00001x^4 + 0.00018x^3 - 0.0016x^2 + 0.02x + 0.04, FC = 35\% \quad (6.24)

where y = CRR and X = (N_{1})_{60}.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{figure6_14.png}
\caption{Correlations between CRR and Corrected blow count (N_{1})_{60} (after Youd et al., 2001).}
\end{figure}

Thereafter, the Factor of Safety (FOS) against the liquefaction hazard is computed as,

\[ \text{FOS} = \frac{\text{CRR}}{\text{CSR}} \quad (6.25) \]

The factor of safety represents a criteria which is given as:

(i) FOS < 1 for liquefiable soil i.e., failed state.
(ii) FOS = 1 at the point of failure indicating vulnerability, and
(iii) FOS > 1 for non liquefiable soil i.e., stable state.

6.9 LIQUEFACTION SUSCEPTIBILITY OF SOILS WITH PLASTIC FINES

Since liquefaction requires the development of excess pore pressure, liquefaction susceptibility is influenced by the compositional characteristics that influence volume change behavior composition characteristics.
associated with high volume change potential tending to be associated with high susceptibility. These characteristics include particle size, shape and gradation.

According to the observations after earthquakes in China, Wang (1979) established that any clayey soil containing less than 15–20% particles by weight smaller than 0.005 mm and having water content (wc) to liquid limit (LL) ratio greater than 0.9 is susceptible to liquefaction. Based on these data, Seed and Idriss (1982) stated that clayey soils could be susceptible to liquefaction only if all three of the following conditions are met: (1) percent of particles less than 0.005 mm is 15%, (2) LL<35, and (3) wc/LL>0.9. Due to its origin, this standard is referred to as ‘Chinese Criterion’. It has been modified by different workers since then. Based on the results of the cyclic testing performed in the study (Bray and Sancio, 2006), a soil may be susceptible to liquefaction if the ratio of the water content to liquid limit is greater than 0.85 (wc/LL>0.85) and the soil plasticity index is less than 12 (12<PI). Soils that do not meet these conditions but have plasticity index less than 18 (PI<18) and water content to liquid limit ratio greater than 0.8 (wc/LL>0.8) may be moderately susceptible to liquefaction. These soils, especially those satisfying the first set of requirements, should be tested in the laboratory to assess their liquefaction susceptibility and strain potential under the loading conditions existing in the field. Soils with PI>18 did not liquefy at low effective stresses. However, structures founded on these soils, and for that matter, any soil, may undergo significant deformations if the cyclic loads approach or exceed the dynamic strength of the soil. The plasticity index versus wc/LL plot depicting clustering of points in different liquefaction potential zones are shown in Figure 6.15 and the chart to determine volume strain as a function of safety is given in Figure 6.16.

The physical and shear parameters of sediment as obtained from geotechnical borehole data are used to evaluate these criteria.

Figure 6.15: Plasticity Index versus wc/LL exhibiting boreholes associated with different liquefaction potentials – non-susceptible, moderate susceptible and susceptible (after Sahai, 2009).
Finally, all the results from the application of simplified procedure as well as the fine sand criteria are merged to prepare a FOS zonation map of Guwahati region as depicted in Figure 6.17 on Geographical Information System (GIS) to create buffer area of 500 m around observation site and thereafter modified according to surface geology of the site.

### 6.10 FACTOR OF SAFETY BEFORE EARTHQUAKE

Factor of safety has been calculated using volumetric strain. Volumetric strain is calculated using (Ishihara and Yoshimine, 1992).

\[
\text{Specific density } Dr(\%) = 25 \left( \frac{N_1}{\sigma_v} \right)^{0.46} \left( \frac{e_0}{\sigma_v} \right)^{0.12} \tag{6.26}
\]

\[
e_0 = e_f + \Delta e
\]

where \( e_f = 1 - \left( \frac{Dr}{2} \right) \), \( \Delta e = e_{vol}(1 + e_f) \)

and \( Dr(\%) \) Before Earthquake = \( (1 - e_0)^2 \)

So from \( Dr(\%) \) before Earthquake, we can calculate the \( N_{1/60} \) before earthquake. Where \( e_{vol} \) is volumetric strain.

After getting the factor of safety before and after earthquake, we can see the effect of the earthquake and interpret its impact on the settlement in terms of liquefaction potential.
Figure 6.17: A Factor of Safety (FOS) against soil liquefaction hazard assessed for a deterministic seismic hazard scenario of Mw 8.7 nucleating from the Shillong plateau (after Nath, 2007a).
CHAPTER – 7

Earthquake Induced Hazards - Landslides

7.1 LANDSLIDES AND OTHER MASS MOVEMENTS

The effect of earthquakes on slope stability depends on numerous factors inherent to site environment, including geology, hydrogeology, topography, mechanical characteristics of the material making up the slope, and pre-existing stability conditions. In addition to these factors, stability is also affected by the characteristics of seismic motion: maximum acceleration, frequency, and duration.

The methods available for the evaluation of slope stability during an earthquake range from simple pseudo-static analysis using a conventional stability method to finite-element studies involving non-linear behavior of the material.

The displacements caused by slope instability can either be very rapid and catastrophic, or very slow. Once movement is started, it may continue over long distances, particularly in the case where the moving mass is subjected to pore pressure leading to liquefaction of the underlying ground.

7.1.1 Landslides of unconsolidated material

7.1.1.1 Level A studies

The first stage in hazard evaluation is to establish a map of the classic types of hazard, i.e., a map without consideration of seismic aspects. In order to do this, a bibliographical study should be carried out concerning the different landslides of the region, classifying them by type of movement. For each landslide, the determining factor of the origin of movement should, if possible, be identified, such as hydrogeology, topography, or type of material, based on criteria that do not require special methods (in-situ or laboratory tests). The information gathered must allow generalization of particular cases registered over the whole area by comparison with other events using pattern-recognition methods. In this way, zones will be identified where the “probability” that a given type of landslide will occur is identical. Mapping of these data is to be carried out at the appropriate scale for the study area.

The zonation will identify the different types of movement and assign four degrees of hazard:

- Level 1 : Nil or Low
- Level 2 : Moderate
- Level 3: High
- Level 4: Very High.

In addition, a bibliographical study of the main earthquakes of the region will include an evaluation of movements produced, wherever possible.

The hazard map incorporating seismic risk will include the same four hazard levels as the map excluding seismic influence. It will be established by integrating the latter map with the data of earthquake movements. This may result in:

- Extension of the zones affected by landslides
- Addition of specific phenomena such as sub-horizontal slides along stream banks, as well as liquefaction slides
- Extension of the zones affected by rock falls.

### 7.1.1.2 Level B studies

For Level B studies, it is assumed that the information necessary for Level A studies is already available; this will be complemented by the application of a simplified pseudo-static method that determines an increase in the hazard. This method does not apply to areas where a notable increase in pore pressure could occur.

Zonation excluding seismic events represents the attribution of an approximate “static safety coefficient” $FS$. It is suggested that this coefficient should be related to the hazard level as follows:

<table>
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<tr>
<th>Level</th>
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<tr>
<td>Level 1</td>
<td>$FS &gt; 2.0$</td>
</tr>
<tr>
<td>Level 2</td>
<td>$1.5 &lt; FS &lt; 2.0$</td>
</tr>
<tr>
<td>Level 3</td>
<td>$1.25 &lt; FS &lt; 1.5$</td>
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<tr>
<td>Level 4</td>
<td>$FS &lt; 1.25$</td>
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</tbody>
</table>

This coefficient is a function of only the mechanical ground characteristics, slope geometry, and hydrogeology.

Seismic action is incorporated into a pseudo-static calculation by the introduction of a horizontal seismic coefficient $k$, which varies with the seismicity of the site. The following coefficients are adopted.

<table>
<thead>
<tr>
<th>Level</th>
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<tr>
<td>Level I</td>
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</tr>
<tr>
<td>Level II</td>
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</tr>
<tr>
<td>Level III</td>
<td>0.20</td>
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</table>
The pseudo-static safety coefficient $FD$ of the slope is a function of the mechanical characteristics of the deposit, geometry of the slope, hydrogeology and the seismic coefficient:

$$FD = A \phi (k, \alpha )$$  \hspace{1cm} (7.1)

Where $\alpha$ is the mean gradient of the slope.

This is also the case for:

$$FS = A \phi (0, \alpha )$$

This leads to:

$$FD = FS \frac{\phi (k, 0)}{\phi (k, 0)}$$  \hspace{1cm} (7.2)

Where slope geometry ($\alpha$), regional seismic hazard ($k$), and the hazard level of the slope excluding earthquakes ($FS$), are known, $FD$ may be calculated. Regarding the notion of the four hazard levels in Table 7.1, the function $\phi (k, \alpha)$ is determined under schematic conditions for the following cases:

- Cohesive soil with sliding parallel to the slope gradient (Figure 7.1a)
- Cohesive soil with flat sliding that goes beyond the foot of the slope (Figure 7.1b)
- Non-cohesive soil with sliding parallel to the slope gradient (Figure 7.1c).

**Figure 7.1a - SLOPE STABILITY**

Cohesive soil * plane sliding

![Graph](image-url)
Figure 7.1b - SLOPE STABILITY
Cohesive soil: slides beyond slope toe

Figure 7.1c - SLOPE STABILITY
Non-cohesive soil

Figure 7.1: Slope Stability (after Keefer, 1984; Keefer and Wilson, 1989).
7.1.1.3 Level C studies

These studies are similar to those carried out for Level B, but the $\phi (k, \alpha)$ functions will be determined for certain typical cross-sections. In addition, the static-safety coefficient $FS$ will be evaluated from hydrological data and the soil’s mechanical characteristics, to support the choice of zonation criteria defined in (Table 7.1). More detailed analytical methods may be applied wherever thoroughly justified and calibrated.

7.1.1.4 Rock and Boulder Falls

In the case of a large-scale rock face, seismicity causes an increase in hazard by release of material during an earthquake. Each individual case will help in appreciating whether this increase can be ignored, or if it must be shown on the map.

The most commonly observed characteristics of slope instability during an earthquake, have been summarized by Keefer (1984), and Keefer and Wilson (1989).

- **Boulder falls, rock falls and collapse** correspond to extremely rapid, surficial movements. The transported mass is broken into a certain number of blocks, ranging from small to very large.

- **Rock slides along discontinuities** correspond to extremely rapid to very rapid movements; they may be either surficial or deep-seated. The transported mass is very fragmented.

- **Slope stripping and surficial landslides** correspond to moderate to rapid movements; they affect both the dry and moist soil. The transported mass is strongly fragmented.

- **Rotational soil slides** are slow to rapid movements, generally deep-seated, which affect both dry and moist soil. The transported mass is generally monolithic.

- **Sub-horizontal slides** occur either by movement along a layer with very weak mechanical characteristics or where the layer undergoes liquefaction; the associated movements have a moderate to high velocity and occur at various depths. The transported mass is generally not very fragmented.

- **Collapse or subsidence** in Karstic zones.

The mechanical analysis of slope stability under seismic load remains largely unsolved. The reasons for this lie in the three-dimensional aspect, the imprecise knowledge of the mechanical, hydraulic, and seismic limits, and the initial state of geometry and mechanical stress. Numerical methods for the evaluation of displacement, possibly taking into account the excess pore pressure, are laborious and difficult to confirm, and suffer from the limitations described above. Mapping using geological or morphological criteria is not yet very well-developed for earthquake engineering, as not much experience has been accumulated to date.

Under these conditions, the hazard zonation for landslides, made worse where seismicity is involved, remains relatively crude.

Finally, the observation of slope instability during major earthquakes reveals that certain phenomena are specific to seismic load, as they occur under morpho-geological conditions that would represent little or no
threat without the earthquake. At present, no satisfactory, proven diagnostic method exists that fully accounts for such movements.

7.2 EARTHQUAKE INDUCED LANDSLIDES

Landslides occur on a regular basis throughout the world as part of the ongoing evolution of landscapes. Many landslides occur in natural slopes, but slides also occur in man-made slopes from time to time. At any point in time, then, slopes exist in states ranging from very stable to marginally stable. When an earthquake occurs, the effects of earthquake-induced ground shaking is often sufficient to cause failure of slopes that were marginally to moderately stable before the earthquake.

Earthquake induced landslides, which have been documented from as early as 1789 B.C. has caused tremendous amounts of damage throughout history. In many earthquakes, landslides have been responsible for as much or more damage then all other seismic hazards combined. In the 1964 Alaska earthquake, for example, an estimated 56% of the total cost of damage was caused by earthquake-induced landslides (Youd and Perkins, 1978; Wilson and Keefer, 1985). Kobayashi (1981) found that more than half of all deaths in large (M>6.9) earthquake in Japan between 1964 and 1980 were caused by landslides. The 1920 Haiyuan earthquake (M=8.5) in the Ningxia Province of China produced hundreds of large landslides that caused more than 100,000 deaths.

7.2.1 Types of Earthquake Induced Landslides

Many factors, including geologic and hydrologic conditions, topography, climate, weathering, and landuse influence the stability of slopes and the characteristics of landslides. A number of procedures for the classification of landslides have been proposed; that of Varnes (1978) is perhaps most widely used in the United States. Similar principles and terminology can be used to classify earthquake-induced landslides (Table 7.3) on the basis of material type (soil or rock), character of movement (disrupted or coherent), and other attributes, such as velocity, depth and water content. Earthquake induced landslides can be divided into three main categories: disrupted slides and falls, coherent slides, and lateral spreads and flows.

Table 7.3: Types and Characteristics of Earthquake induced Landslides (after Keefer, 1984)

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of Movement</th>
<th>Internal Disruption^a</th>
<th>D</th>
<th>U</th>
<th>PS</th>
<th>S</th>
<th>velocity^c</th>
<th>Depth^d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disrupted Slides and Falls</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock falls</td>
<td>Bounding, rolling, free fall</td>
<td>High or very High</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Extremely rapid</td>
<td>Shallow</td>
</tr>
<tr>
<td>Rock slides</td>
<td>Translational sliding on basal shear surface</td>
<td>High</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Rapid to extremely rapid</td>
<td>Shallow</td>
</tr>
<tr>
<td>Name</td>
<td>Type of Movement</td>
<td>Internal Disruption(^a)</td>
<td>D</td>
<td>U</td>
<td>PS</td>
<td>S</td>
<td>velocity(^c)</td>
<td>Depth(^d)</td>
</tr>
<tr>
<td>-------------------------</td>
<td>----------------------------------------------------------------------------------</td>
<td>----------------------------</td>
<td>---</td>
<td>---</td>
<td>----</td>
<td>-----</td>
<td>------------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Rock avalanches</td>
<td>Complex, involving sliding and/or flow, as stream of rock fragments</td>
<td>Very high</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Extremely rapid</td>
<td>Deep</td>
</tr>
<tr>
<td>Soil falls</td>
<td>Bounding, rolling, free fall</td>
<td>High or very high</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Extremely rapid</td>
<td>Shallow</td>
</tr>
<tr>
<td>Disrupted soil</td>
<td>Translational sliding on basal shear surface or zone of weakened, sensitive clay</td>
<td>High</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Moderate to rapid</td>
<td>Shallow</td>
</tr>
<tr>
<td>Soil avalanches</td>
<td>Translational sliding with subsidiary flow</td>
<td>Very high</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Very rapid to extremely rapid</td>
<td>Shallow</td>
</tr>
</tbody>
</table>

**Coherent Slides**

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of Movement</th>
<th>Internal Disruption(^a)</th>
<th>D</th>
<th>U</th>
<th>PS</th>
<th>S</th>
<th>velocity(^c)</th>
<th>Depth(^d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock slumps</td>
<td>Sliding on basal shear surface with component of headword rotation</td>
<td>Slight of moderate</td>
<td>?</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Slow to rapid</td>
<td>Deep</td>
</tr>
<tr>
<td>Rock block slides</td>
<td>Translational sliding on basal shear surface</td>
<td>Slight or moderate</td>
<td>?</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Slow to rapid</td>
<td>Deep</td>
</tr>
<tr>
<td>Soil slumps</td>
<td>Sliding on basal shear surface with component of headword rotation</td>
<td>Slight or moderate</td>
<td>?</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>Slow to rapid</td>
<td>Deep</td>
</tr>
<tr>
<td>Soil block slides</td>
<td>Translational sliding on basal shear surface</td>
<td>Slight or moderate</td>
<td>?</td>
<td>?</td>
<td>×</td>
<td>×</td>
<td>Slow to rapid</td>
<td>Deep</td>
</tr>
<tr>
<td>Slow earth flows</td>
<td>Translational sliding on basal shear surface with minor internal flow</td>
<td>Slight</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td></td>
<td>Very slow to moderate with very surges</td>
<td>Generally shallow occasionally deep</td>
</tr>
</tbody>
</table>
## Water content

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of Movement</th>
<th>Internal Disruptiona</th>
<th>D</th>
<th>U</th>
<th>PS</th>
<th>S</th>
<th>velocityc</th>
<th>Depthd</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Spreads and Flows</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil lateral</td>
<td>Translation on basal zone of liquefied sand, or silt or weakened, sensitive clay</td>
<td>Generally moderated, occasionally slight, occasionally high</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid soil flows</td>
<td>Flow</td>
<td>Very high</td>
<td>?</td>
<td>?</td>
<td>?</td>
<td>x</td>
<td>Very rapid</td>
<td>shallow</td>
</tr>
<tr>
<td>Subaqueous landslides</td>
<td>Complex, generally involving lateral spreading, and/or flow; occasionally involving slumping and/or block sliding</td>
<td>Generally high or very high, occasionally moderate or slight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Generally rapid to extremely rapid</td>
<td>Generally rapid to extremely rapid, occasionally show to moderate</td>
</tr>
</tbody>
</table>

Disrupted slides and falls include rock falls, rock slides, rock avalanches, soil falls, disrupted soil slides, and soil avalanches. The earth materials involved in such failures are sheared, broken, and disturbed into a nearly random order. These types of failures, usually found in steep terrain, can produce extremely rapid movements and devastating damage; rock avalanches and rock falls have historically been among the leading causes of death from earthquake induced landslides.

Coherent slides, such as rock and soil slumps, rock and soil black slides, and slow earth flows, generally consist of a few coherent blocks that translate or rotate on somewhat deeper failure surfaces in moderate to steeply sloping terrain. Most coherent slides occur at lower velocities than disrupted slides and falls. Lateral spreads and flows generally involve liquefiable soils, although sensitive clays can produce landslides with very similar characteristics. Due to the low residual strength of these materials, sliding can occur on remarkably flat slopes and produce very high velocities.

The different types of earthquake induced landslides occur with different frequencies. Rock falls, disrupted soil slides, and rock slides appear to be the most common types of landslides observed in historical earthquakes (Table 7.4). Subaqueous landslides, slow earth flows, rock block slides, and rock avalanches are least common, although the difficulty of observing subaqueous slides may contribute to their apparent rarity.
Table 7.4: Relative Abundance of Earthquake-induced Landslides from Study of 40 Historical Earthquakes Ranging from $M_s = 5.2$ to $M_w = 9.5$ (after Keefer, 1984)

<table>
<thead>
<tr>
<th>Abundance</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very abundant (&gt;100,000 in the 40 earthquakes)</td>
<td>Rock falls, disrupted soil slides, rock slides</td>
</tr>
<tr>
<td>Abundant (10,000 to 100,000 in the 40 earthquakes)</td>
<td>Soil lateral spreads, soil slumps, soil block slides, soil avalanches</td>
</tr>
<tr>
<td>Moderately common (1000 to 10,000 in the 40 earthquakes)</td>
<td>Soil falls, rapid soil flows, rock slumps</td>
</tr>
<tr>
<td>Uncommon</td>
<td>Subaqueous landslides, slow earth flows, rock block slides, rock avalanches</td>
</tr>
</tbody>
</table>

For preliminary stability evaluations, knowledge of the conditions under which earthquake induced landslides have occurred in the past earthquakes is useful. It is logical to expect that the extent of earthquake-induced landslide activity should increase with increasing earthquake magnitude and that there could be a minimum magnitude below which earthquake-induced landsliding would rarely occur. It is equally logical to expect that the extent of earthquake-induced landslide activity should decrease with increasing source to site distance and that there could be a distance beyond which landslides would not be expected in earthquakes of a given size.

A study of 300 U.S. earthquakes between 1958 and 1977 showed that the smallest earthquakes noted to have produced landslides had magnitudes of about 4.0 (Keefer, 1984). Minimum magnitudes for different types of landslides were estimated as shown in Table 7.5. Where magnitudes were not available, minimum Modified Mercalli Intensity (MMI) values of IV and V have been observed for disrupted slides or falls and other types of slides, respectively. Although these empirically based limits are useful, their approximate nature must be recognized; failure of slopes that are near the brink of failure under static conditions could be produced by quite weak earthquake shaking.

Table 7.5: Estimates of the Smallest Earthquakes Likely to Cause Landslides (after Keefer, 1984)

<table>
<thead>
<tr>
<th>ML</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>Rock falls, rock slides, soil falls, disrupted soil slides</td>
</tr>
<tr>
<td>4.5</td>
<td>Soil slumps, soil block slides</td>
</tr>
<tr>
<td>5.0</td>
<td>Rock slumps, rock block slides, show earth flows, soil lateral spreads, rapid soil flows, and subaqueous landslides</td>
</tr>
<tr>
<td>6.0</td>
<td>Rock avalanches</td>
</tr>
<tr>
<td>6.5</td>
<td>Soil avalanches</td>
</tr>
</tbody>
</table>
The maximum source-to-site distance at which landslides have been produced in historical earthquakes are different for different types of landslides. Disrupted slides or falls, for example, have rarely been found beyond epicentral distances of about 15 km for M=5 events but have been observed as far as about 200 km in M=7 earthquakes. Note that the curve for lateral spreads and flows correlates reasonably well with the magnitude-distance curve for liquefaction. Similarly, the area over with earthquake-induced landsliding can be expected also increases with increasing earthquake magnitude. Regional differences in attenuation behavior have little apparent influence on the area of earthquake-induced landsliding.

7.3 EVALUATION OF SLOPE STABILITY

The stability of slopes is influenced by many factors, and a complete slope stability evaluation must consider the effects of each. Geological, hydrological, topographical, geometrical, and material characteristics all influence the stability of particular slope. Information on these characteristics is needed to reliably perform and interpret the results of both static and seismic slope stability analyses. Review of available documents, field reconnaissance, field monitoring, subsurface investigation, and material testing can all be used to obtain this information.

For many sites, considerable useful information can be obtained from previously published documents such as geologic maps, soil survey and/or agricultural maps, topographic maps, natural hazard maps, and geologic and geotechnical engineering reports. Additional information may be obtained from aerial photographs (particularly stereo-paired aerial photographs) and from other remote sensing.

Field reconnaissance involves careful observation and detailed mapping of a variety of site characteristics associated with the existing or potential slope instability. Features such as scarps; tension cracks; walls, or pavements; displaced ditches, channels, and fences; cracked foundations, walls, or pavements; and leaning trees or poles can be identified and mapped as the evidence of instability. The locations of streams, springs, seeps, ponds, and moist areas, and differences in vegetative cover, can provide evidence of altered or disrupted water flow caused by slope instability.

If time permits, slope movement can be monitored. Surface monument can be installed at points on and near the slope and surveyed periodically to identify the magnitude and direction of surface movement. Photogrammetric methods can be used to determine relative movements from sets of stereo-paired aerial photographs taken at different times. Inclinometers are very useful for monitoring lateral deformation patterns below the ground surface. In many cases, crack gauges, tilt meters, and extensometers can also be used to observe the effects of slope movement. When as is commonly the case, pore pressures are important, piezometers and/or observation wells can provide important information on pore pressures and their variation with time.

Subsurface investigation includes excavation and mapping of test pits and trenches, boring and sampling, in-situ testing, and geophysical testing. Such investigations can reveal the depth, thickness, density, strength, and deformation characteristics of subsurface units, and the depth and variation of the groundwater table. In-situ and geophysical tests are particularly useful for determining the location of an existing failure surface.
7.4 LANDSLIDE HAZARD AND RISK ANALYSIS IN INDIA AT A REGIONAL SCALE

7.4.1 Materials and Method

Many methodologies have been proposed for the spatial landslide hazard zonation. Two approaches are the most promising ones, namely, the methods based on the statistical analysis of geo-environmental factors related to the occurrence of landslides (Information Value Based Method), and the deterministic modeling based on simple mechanical laws that control slope instability (Guzzetti et al., 1999). The Information Value Based Method (Yin and Yan, 1988; Wu et al., 2000) is the one used for the present investigation (Dasadhikari et al., 2011). This statistical approach is based on the observed relationship between each factor and the past and present landslide distributions (Carrara et al., 1991). The input thematic datasets are converted into a grid format (i.e., rasterization) and various factor layers that control the landsliding are prepared in ArcGIS platform. The Landslide Hazard Zonation and Risk Assessment have been accomplished in six stages as shown in a Logic Tree Framework in Figure 7.2 (Dasadhikari et al., 2011).

Figure 7.2: Methodology for landslide hazard zonation and risk analysis. 1, 2, …, 5, 6 are represent six stage are respectively; 1. Thematic Layer, 2. Data Quantification, 3. Calculation of Score Factor, 4. Integration, 5. Model Testing Zonation & Classification, and 6. Risk analysis (modified after Landslide Hazard Zonation Atlas of India, 2003; Dasadhikari et al., 2011).
Thematic Data Layer (Factor Map)

Selection of factors and preparation of corresponding thematic data layers are the crucial components of any model for Landslide Hazard Zonation mapping. The factors governing instability in a terrain are primarily engineering geology, slope, geomorphology, geotectonic, landuse, and climatic condition. The selection of these factors and their classes are primarily based on the existing landslides and their associated terrain factors. Based on the information collected from available maps, field investigations, and published & unpublished reports the thematic data layers are generated. A landslide distribution map is also prepared. The mapping is done at a grid size of 1 km x 1 km.

Engineering Geology

Engineering Geological Maps cover various types of rocks in terms of geological eras. The main engineering geological attributes are lithology and structures. The lithological attributes are very important in determining the shear strength, permeability, susceptibility to chemical and physical weathering which in turn affects the stability. Structures include the features of inhomogeneity and discontinuity in rocks or soils. On the engineering geological viewpoint Flysch, the weathered mantle of the metamorphic rocks, Quaternary, and Neogene deposits affect landslides. Indian mountain regions are mainly covered by basement crystallines, volcanic rock, central crystallines, metamorphic and igneous rock as depicted in Figure 7.3. Landslide frequency distributions as observed in this region are very high about 96.03% of the total landslide occurrences in the country.

Figure 7.3: Engineering Geology map of India (after Dasadhikari et al., 2011).
Geomorphology

Two geomorphological features covering the Indian mountain regions are basically the alluvial plain and the foredeep zone of structural ridge and valleys. This geomorphology is more prone to landslides to the tune of about 68.02% since these regions are situated under steep to very steep slopes. Indian plateau and some plain land are low to moderately prone to the tune of about 26.40% to landslides since these regions are geologically covered by the hard rock and moderately steeping to gentle slopes as shown in Figure 7.4. The deltaic plains and desert plains or dune regions are favorable to low landslide/stable region as these areas are situated at low altitude as well as at a flatly dipping slopes. Deltaic/coastal sedimentary regions are considered unlikely to be landslide prone since these regions are not subjected to weathering processes.

Slope

Slope angle is one of the key factors in inducing slope instability. The interrelation between slope gradient and stability is complex and nonlinear. Theoretically speaking, steeper the slope higher will be its probability of sliding. Correlation of the landslide frequency with the slope angle exhibited that the landslide frequency increases with increasing slope angle attaining to about to 74.88% of the total landslide occurrences in the country. So the slope maps are classified into thirteen categories depending on sliding and the degree of slope. In this region steep to very steep slopes mainly cover the hilly areas which affect the
sliding process. Rest of the area is covered by flat to moderately steeping slopes prohibiting significant sliding as shown in Figure 7.5.

Rainfall

India consists of extraordinary variety of climatic regions. Broadly it can be divided into four climatic zones namely Alpine, Subtropical, Tropical and Arid. Types and severity of slope failures differ very significantly from region to region depending on the climatic patterns, particularly the temperature and the precipitation. In the temperate regions, having moderate but prolonged rainfall, seasonal fluctuations in groundwater flow and pressure may lead to periodic activation of deeper slides. In subtropical and tropical regions in which deluges due to monsoon or cyclonic storms occurs damaging debris, slides may occur over considerable areas in a short time as in the case of Peninsular India. On the other hand, water is commonly the primary factor triggering a landslide. Slides often occur following intense rainfall, when storm water runoff saturates soils on steep slopes or when infiltration causes a rapid rise in the groundwater levels. Groundwater may rise as a result of heavy rains or a prolonged wet spell. As water tables rise, some slopes become unstable. These hydrological aspects are generalized for the present...
study by considering the annual rainfall (<100 mm to >10000 mm) distribution as depicted in Figure 7.6. It varies in the East to West direction. Mainly Eastern, North-Eastern, South-Western, Southern, and Northern India are affected due to heavy rainfall (>1200 mm) enhancing landslide frequency distribution to a very high percentage to the tune of about 83.25%.

Figure 7.6: Rainfall Map of India (after Dasadhikari et al., 2011).

Geotectonic

tectonic features, such as thrusts and faults, are usually associated with extensive fractured zones and steep relief anomalies. Therefore, major structural discontinuities produced by faults and fractures are included as important parameters in this study. To account for the influence of fault zones a distance function is applied to create buffer zones at a distance of 1 km around the fault/fracture. Two broad
classes: ‘active regions’ containing about 86.18% landslide frequency or ‘unlikely regions’ containing about 15.82% landslide frequency is shown in Figure 7.7.

Landuse / Landcover

Landuse change has been recognized throughout the world as one of the most important factors influencing the occurrence of rainfall-triggered landslides. The effect of landuse mainly vegetation on slope stability appears to be complex. Generally speaking a vegetative cover promotes stability, whereas a barren slope is more prone to landslide due to excessive erosion and weathering. USGS published a Global
Landuse/Landcover (1 km * 1 km) digital remote sensing data (http://glcfapp.glcf.umd.edu) which classified 45 landuse / landcover (LULC) categories, Figure 7.8 depicting the Indian condition. In this LULC data, Indian mountain region covers mainly different type of vegetation (tropical semi-evergreen, temperate conifer, tropical evergreen, subtropical conifer, degraded forest) and snow-field which play causative and protective factor for the landslide by the seasonal variation. On the other hand agricultural land (slope and irrigated intensive agriculture) and settlement are too favorable for landsliding due to the regional variation.

Figure 7.8: Landuse/Landcover of India (after Dasadhikari et al., 2011).

7.4.2 Actual Landslides observed or reported

It is most important that such observations are considered in the mapping of all landslides, old or new, active or dormant. The evidence of past instability is frequently the best guide to consider the future behavior in the locality. It is more important for the estimation of accuracy assessment in between landslide hazard zonation map and actual landslide observed in-situ. Landslide Hazard Zonation Atlas of India provides major landslide inventory data as shown in Figure 7.9 that is used in the present study.
7.4.3 Histogram Analysis

The histogram analysis is performed to generate the statistical relationship between the observed landslide and each thematic layer attributes. For calculating landslide frequency distribution, we identified total number of landslide occurrences under a particular thematic layer’s attributes and calculate total number of landslides under each thematic layer. Each feature in a theme containing different landslide frequencies is shown in Figure 7.10 and detailed in Table 7.6. It is important for data quantification and assigning appropriate score factors.

Figure 7.9: Landslide Inventory Map of India (after Dasadhikari et al., 2011).
Figure 7.10: Histogram analysis of different factors contributing to the landslide frequency distribution (after Dasadhikari et al., 2011).
### Table 7.6: Thematic weighted calculation and ranking for LHI (after Dasadhikari et al., 2011)

<table>
<thead>
<tr>
<th>FACTORS</th>
<th>TEMATIC WEIGHTED*</th>
<th>CATEGORIES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>EG*</td>
</tr>
<tr>
<td>Alluvium</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Deltaic/ Coastal Sediment</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>Aeolian Sand</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Residual Soil</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Laterite and Lateritic Soil</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Piedmont deposit</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Sub Himalayan / Naga Lushai Belt of soft to moderately hard sedimentaries</td>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Engineering Geology 0.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metamorphic and igneous rocks of lesser Himalaya / Arakan Yoma Axial Zone</td>
<td></td>
<td>8</td>
</tr>
<tr>
<td>Greater / Trans / lohit Himalayan mountains of Central Crystallines, Tethyan sediments</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td>Younger sediments</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Older sediments</td>
<td></td>
<td>11</td>
</tr>
<tr>
<td>(Gondwana) sediments</td>
<td></td>
<td>12</td>
</tr>
<tr>
<td>Volcanic rock</td>
<td></td>
<td>13</td>
</tr>
<tr>
<td>Deformed metasedimentary, metavolcanics with minor granitoid and other intrusives</td>
<td></td>
<td>14</td>
</tr>
<tr>
<td>Basement crystallines</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>Geom*</td>
<td>Landslide Frequency (%)</td>
<td></td>
</tr>
<tr>
<td>Alluvial plain</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Deltaic plain</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>Desert plain / dune</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Thick regolith cover</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Geomorphology 0.13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateritic cover</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Himalayan and Naga Lushai Mountain Belts / foredeep zone of structural ridges and valleys</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Structural hills and flats on unmetamorphosed to feebly metamorphosed sediments with minor intrusive and volcanic</td>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Flat to gently sloping undeformed sediments and faults basin</td>
<td></td>
<td>8</td>
</tr>
<tr>
<td>Plateau on basalt with thin sedimentary places</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td>Peneplain, plateau and residual hills with structural hill ranges</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Slope*</td>
<td>Landslide Frequency (%)</td>
<td></td>
</tr>
<tr>
<td>VERY STEEP SLOPE: more than 600 metres per kilometer</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>STEEP SLOPE: 300-600 metres per kilometer</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>STEEP SLOPE: 150-300 metres per kilometer</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>MODERATELY STEEP SLOPE: 80-150 metres per kilometer</td>
<td></td>
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<tr>
<td>MODERATELY STEEP SLOPE: 20-80 metres per kilometer</td>
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<tr>
<td>MODERATELY STEEP SLOPE: 20-80 metres per kilometer at elevation 100-500 metres above msl</td>
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<td>6</td>
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<tr>
<td>Slop 0.3</td>
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<td>MODERATELY STEEP SLOPE: 20-80 metres per kilometer at elevation 100 metres above msl</td>
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<td>7</td>
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<tr>
<td>GENTLE SLOPE: 10-20 metres per kilometer at elevation over 500 metres above msl</td>
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<td>GENTLE SLOPE: 10-20 metres per kilometer at elevation 100-500 metres above msl</td>
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<td>9</td>
</tr>
<tr>
<td>GENTLE SLOPE: 10-20 metres per kilometer at elevation upto 100 metres above msl</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>FLAT: less than 10 metres per kilometer at elevation over 500 metres above msl</td>
<td></td>
<td>11</td>
</tr>
<tr>
<td>FLAT: less than 10 metres per kilometer at elevation 100-500 metres above msl</td>
<td></td>
<td>12</td>
</tr>
<tr>
<td>FLAT: less than 10 metres per kilometer at elevation upto 100 metres above msl</td>
<td></td>
<td>13</td>
</tr>
</tbody>
</table>
**Earthquake Induced Hazards - Landslides**

* EG, Geom, Slope, Rainfall, Fdist, and LULC are represent thematic attribute of Engineering Geology, Geomorphology, Slope, Rainfall, Fault Distance, and landuse/landcove thematic layer respectively.*

<table>
<thead>
<tr>
<th>FACTORS</th>
<th>TEMATIC WEIGHTED</th>
<th>CATEGORIES</th>
<th>Rainfall*</th>
<th>Landslide Frequency (%)</th>
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<tr>
<td></td>
<td></td>
<td>0-400</td>
<td>1</td>
<td>0.00</td>
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<td></td>
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<td>Rainfall (in mm) 0.15</td>
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<td>1.48</td>
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<td></td>
<td></td>
<td>1000-4000</td>
<td>4</td>
<td>82.23</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4000-&gt;10000</td>
<td>5</td>
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<td>Fdist*</td>
<td>Landslide Frequency (%)</td>
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</tr>
<tr>
<td></td>
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<td>0-1 km</td>
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<td></td>
<td></td>
<td>&gt;1 km</td>
<td>2</td>
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<td>LULC*</td>
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<td>3.39</td>
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<tr>
<td>Subtropical Evergreen</td>
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<td>3</td>
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<tr>
<td>Tropical Moist Deciduous</td>
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<td>Landuse</td>
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<td>Thorn Forest/Scrub (Southern)</td>
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<td>2.87</td>
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<td>3</td>
<td></td>
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<tr>
<td>Abandoned Jhum</td>
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<td>1.83</td>
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<td></td>
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<td>Sparse woods</td>
<td>19</td>
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<td>Bush</td>
<td>20</td>
<td>1.83</td>
<td>2</td>
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<tr>
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<td>Savannah</td>
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<td>4</td>
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<td></td>
</tr>
<tr>
<td>Alpine Grasslands</td>
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<td>2.09</td>
<td>3</td>
<td></td>
</tr>
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<td>Sparse vegetation (cold)</td>
<td>28</td>
<td>1.31</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Thorn Forest/Scrub (Southern)</td>
<td>29</td>
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<td>3</td>
<td></td>
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<tr>
<td>Shrub</td>
<td>30</td>
<td>2.61</td>
<td>3</td>
<td></td>
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<tr>
<td>Sparse woods</td>
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<td>1.04</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Bush</td>
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<td>1.31</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Coastal vegetation</td>
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<td>3.02</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Savannah</td>
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<td>2.09</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Plain Grasslands</td>
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<td>6.27</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Slope Grasslands</td>
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<td>0.52</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Desert (cold)</td>
<td>37</td>
<td>1.31</td>
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<td></td>
</tr>
<tr>
<td>Thorn Scrub / Desert (hot)</td>
<td>38</td>
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<td>Irrigated Intensive Agriculture</td>
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<td>Coral reef</td>
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<td>Water Bodies</td>
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<tr>
<td>Snow</td>
<td>47</td>
<td>6.53</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Barren</td>
<td>48</td>
<td>0.52</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Bare Rock</td>
<td>49</td>
<td>0.26</td>
<td>1</td>
<td></td>
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<tr>
<td>Salt Pan</td>
<td>50</td>
<td>0.52</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Mud Flats</td>
<td>51</td>
<td>0.52</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Settlement</td>
<td>52</td>
<td>5.74</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>
7.4.4 Data Quantification

By using remote sensing and GIS, all thematic maps are quantified and rasterized to specific pixel size (1 km x 1 km). Entire qualitative thematic information (vector base) has been converted to quantitative (raster base) data sets. A binary method is used for cross matching of each parameter with respect to the landslide occurrence map.

7.4.5 Calculation of Score Factor

The score factor is calculated by determining susceptible factor of each variable for landslide failure. Landslide hazard analysis involves overlaying of various themes of continuous and discrete datasets using the principle of convergence of evidences. These analyses are mostly grid based requiring the input datasets to be quantified based on their individual contributions. For instance, a particular type of landuse or lithology would indicate its contribution towards causing slope instability in the region only qualitatively. Similarly between the slope and the rainfall theme, the slope would contribute more significantly towards slope instability than the rainfall one. For example a high slope angle contributes more towards landslide susceptibility than low slope angle, thus maximum percentage of landslide frequency distribution associated with high slope angle will have maximum ranking amongst the thematic attributes compared to the minimum percentage of landslide frequency distribution as depicted in Table 7.6. More closely, associated contributing units are taken together to reduce the number of units to be ranked. A suitability scale ranging from 1 to 5 values is devised. The value of ‘1’ is assigned to the lowest contributing unit in a particular theme and the highest contributing unit is assigned a value of ‘5’. The thematic weight values in a ten point scale and rank values from 1 to 5 helps to bring out the differences in contributions between the themes and between units within a theme during the GIS analysis. The thematic weighted \( X_j \) is calculated by the formula:

\[
X_j = \frac{R_j - R_{\text{min}}}{R_{\text{max}} - R_{\text{min}}}
\]  

Where \( R_j \) is the row score, \( R_{\text{max}} \) and \( R_{\text{min}} \) are the minimum and maximum scores of a particular layer.

7.4.6 Weighted Thematic Data Layers

Based on the score factors a numerical ranking is implemented and all the thematic layers are reclassified using spatial analysis tools on ARCGIS platform. The categories which depict dominant landslides are assigned highest number in the numerical scale as 5,4,3,2 and 1 shown in Table 7.6. The numerical scale is assigned in the weighted thematic data layers as severe to very high, high, moderate, low and unlikely as shown in Figure 7.11.
Figure 7.11: Weighted Thematic Data Layers (after Dasadhikari et al., 2011).
7.4.7 Integration

All the influencing factors of different variables are overlaid and added spatially together to find out the probable area of landslide occurrences. The maps from NATMO and GSI are used as the source for creating various thematic maps. Since the source maps are of different scales, the foremost task is to achieve one single boundary for all the themes. Various slope/landuse/rock types were assigned different unit values in the respective thematic maps viz., slope, rainfall, Landuse, engineering geology, geomorphology and geotectonics. The vector datasets were stored as Arcinfo coverages. The coverages are cleaned and built for generating correct polygons (units) and ensuring data integrity. The landslide zonation analysis involves grid data sets (the discrete data sets from different themes are converted to grids) to compute the final landslide susceptibility grid through weighted and integration analysis of the source themes. The classes of different data layers are assigned the corresponding rating value as attribute information in the GIS and an ‘attribute map’ is generated for each data layer. These attribute maps are then multiplied by the corresponding thematic weights ($X_j$) and then added up to yield the landslide hazard index (LHI) for each cell. The methodology for this operation is systematically shown in Figure 7.2. The Landslide Hazard Index is calculated by the formula:

$$LHI = \sum \sum \sum \sum \sum \text{Weighting} \times \text{data layer (attribute)}$$  \hspace{1cm} (7.4)

7.4.8 Landslide Hazard Zonation & Classification

The LHI frequency diagram shown in Figure 7.12 is subsequently used for landslide hazard zonation. The LHI threshold boundaries used are: 2, 3, 4 and >4. Using the ‘slicing’ operation, a Landslide Hazard Zonation (LHZ) map is prepared. Thereafter, a $3 \times 3$ ‘majority filter’ has been applied to the map as a post-classification filter to reduce the high frequency variation. The LHZ map is prepared for the entire country using four point hazard scales, which are ‘Severe to Very High’, ‘High’, ‘Moderate’ and ‘Unlikely’ zones as shown in Figure 7.13.

**Figure 7.12:** Reclassification of the landslide hazard index into the 4 (threshold 2, 3, 4 and > 4) classes (after Dasadhikari et al., 2011).
7.5 COMPARISON OF LANDSLIDE HAZARD ZONATION MAP WITH FIELD DATA/ACCURACY ASSESSMENT

Accuracy assessment is a general term for comparing the classification of geographical data that are assumed to be true, in order to determine the accuracy of the classification process. Usually, the assumed-true data are derived from ground truth data. Therefore, a set of reference pixels is usually used for accuracy assessment. Reference pixels are the points on the classified image for which actual data are (or will be) known. The reference pixels are randomly selected (Congalton, 1991). The distribution of existing (active) landslides (Reference pixels or inventory data) is used to evaluate the validity of landslide hazard zonation map. For accuracy assessment Erdas Imagine 8.5 is used. In this study overall classification accuracy is found to be 75.25% and overall Kappa Statistics is 0.6180% as shown in Table 7.7.
### Table 7.7: Calculation for accuracy assessment (after Dasadhikari et al., 2011)

<table>
<thead>
<tr>
<th>Class Name</th>
<th>Reference Totals</th>
<th>Classified Totals</th>
<th>Number Correct</th>
<th>Producers Accuracy</th>
<th>Users Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unlikely</td>
<td>27</td>
<td>104</td>
<td>27</td>
<td>100.00%</td>
<td>25.96%</td>
</tr>
<tr>
<td>Moderate to High</td>
<td>13</td>
<td>23</td>
<td>13</td>
<td>100.00%</td>
<td>56.52%</td>
</tr>
<tr>
<td>High</td>
<td>266</td>
<td>171</td>
<td>170</td>
<td>63.91%</td>
<td>99.42%</td>
</tr>
<tr>
<td>Severe to Very High</td>
<td>89</td>
<td>86</td>
<td>86</td>
<td>96.63%</td>
<td>100.00%</td>
</tr>
</tbody>
</table>

Overall Classification Accuracy = 75.25%

Overall Kappa Statistics = 0.6180

### 7.6 LANDSLIDE RISK ANALYSIS

In the study area, landslides mainly affect the population, buildings, transportation networks, and agricultural activities. Landslide risk can be estimated based on the following equation:

\[
\text{Risk} = \text{hazard} \times \text{exposure}
\]  

The exposure factor is estimated by the combination of population density, landuse/landcover, and road network data layers. The above mentioned data layers are classified and ranked as shown in Table 7.8. Generally, larger values are assigned to population areas, built-up areas and road networks to reflect their higher significance in determining the impact of landslides.

### Table 7.8: Thematic Weighted Calculation for Risk Analysis (after Dasadhikari et al., 2011)

<table>
<thead>
<tr>
<th></th>
<th>Population Density</th>
<th>Landuse/ Landcover</th>
<th>Road Networks</th>
<th>Thematic Weighted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Population Density</td>
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<td></td>
<td></td>
<td>0.4</td>
</tr>
<tr>
<td>Landuse/ Landcover</td>
<td>1.11</td>
<td>1</td>
<td></td>
<td>0.3</td>
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<tr>
<td>Road Networks</td>
<td>1.43</td>
<td>0.77</td>
<td>1</td>
<td>0.3</td>
</tr>
</tbody>
</table>

### 7.7 INFERENCES

Based on the above studies, a landslide hazard zonation and risk map has been prepared here using integrated Remote Sensing and GIS techniques. The entire India has been divided into four landslide susceptibility zone (very low/unlikely, moderate, high and severe to very high) as shown in Figure 13. Based on this susceptibility zone, we calculate vulnerability index using risk elements (population density,
landuse/landcover, and road network) and divided into three risk zones (very low, moderate, high) as shown in Figure 7.14. The hazard and risks are correlated with each other. Using these techniques the areas which are vulnerable to landslides are predicted and suitable mitigation measures are adopted. The risk maps produced by that procedure wisely point the land occupied by the villages and by the arable-cultivated areas surrounding them as high to very high risk areas. Also the areas in the proximity of the road network are characterized as moderate to high risk areas. All these areas are characterized by continuous human presence and they are related to economic activities.

Figure 7.14: Landslide Risk Map of India. The risk map is produced by using the simple LHZ map. The risk threshold boundaries used are: 10, 12.5, and >12.5. Using the ‘slicing’ operation, a landslide risk map is prepared showing the three point risks scale, (1) High, (2) Moderate, and (3) Very low zones (after Dasadhikari et al., 2011).
CHAPTER 8

Integration of Hazard Maps on GIS Platform for Seismic Microzonation

8.1 INTRODUCTION

A very preliminary process of reducing the effects of earthquake is by assessing the hazard itself. Seismic microzonation is the first and foremost step to minimize seismic related damages and loss of lives. Seismic microzonation is the generic term for subdividing a region into smaller areas having different potential for hazardous earthquake effects, defining their specific seismic behavior for engineering design and landuse planning. Usually seismic microzonation includes approaches for assessing local ground response, slope instability and liquefaction. Microzonation studies involve experimental techniques together with theoretical approaches involving ground motion modeling.

As part of the national level microzonation programme, Department of Science and Technology, Govt. of India has initiated microzonation of 63 cities in India. As an initial experiment, seismic hazard analysis and microzonation was taken up for Jabalpur city in Madhya Pradesh. Further, for many other regions such as Sikkim, Mumbai, Delhi, Northeast India, Guwahati, Ahmedabad, Bhuj, Dehradun, Haldia and Chennai, an attempt has been made to carryout microzonation considering geomorphological features and detailed geotechnical studies. Among the above Jabalpur, Sikkim, Guwahati and Bangalore microzonation works have been completed. Nath (2004 and 2005) used GIS as integration tool to map seismic ground motion hazard for the Sikkim Himalaya in India. In this study, similar approach of Nath (2004 and 2005) is used to develop a hazard index map wherein the seismic hazard parameters are integrated and coupled with ground information. The hazard index maps are prepared using both the deterministic and probabilistic approaches. There are very large number of geotechnical and geophysical tests available for the characterization of subsurface soils. Each of the method has its own advantages and also limitations over other methods. There is always an ambiguity over the selection of a particular method for site characterization. Selection of a particular method depends upon many factors starting from the purpose and the scope of the study, availability of resources (equipment and expert personnel), type of analysis to be carried out etc. The extent of the area to be investigated for microzonation generally spans over several kilometers unlike routine geotechnical site investigations. Characterizing such a huge area using conventional site investigations is not possible. Moreover, microzonation studies require dynamic site characterization from the perspective, of earthquake loading. So, it is very important to plan site investigations properly using appropriate geotechnical/geophysical methods. Most of the geotechnical and geophysical studies carried out earlier for seismic microzonation lacked either in the selection of appropriate geotechnical/geophysical tests or planning of sufficient number of tests, or following appropriate measures/
methodologies for testing, or in the adoption of proper interpretation techniques required for seismic microzonation. Analyses based on such studies with either improper data or insufficient data or lack of proper methodology, can often lead to improper estimate of seismic hazards. In this chapter, case studies of geotechnical and geophysical investigations carried out for microzonation of a few cities in India are discussed. An integration of all the developed maps is attempted based on weights and ranks. A final hazard index map is developed using Analytic Hierarchy Process (AHP) on GIS (Geographical Information System) platform. Application of GIS for microzonation mapping is amply demonstrated by many researchers all over the world.

8.2 GEOGRAPHICAL INFORMATION SYSTEM (GIS)

Geographical Information System (GIS) platform was adopted as a primary and other working tool in preparing the seismic hazard microzonation map for the Guwahati regions. Multitasking functionality of GIS makes it ideally suited to seismic microzonation as it enables automation of data manipulation and information of maps. The complex spatial analyses associated with seismic microzonation necessitate GIS technology for data dissemination and its management.

![Figure 8.1: A schematic process flow of GIS based Thematic Mapping.](image)

The GIS framework allows accounting for added levels of details and complexity. It is very important that the relevant data layers are consistent in their level of details, in order to successfully combine them and cross analyze them in the pair-wise comparison process. A schematic process flow is shown in Figure 8.1. The methodologies used to produce the integrated seismic hazard maps are already discussed in Chapter
8.3 EARTHQUAKE HAZARD PARAMETERS

Seismic microzonation is subdividing a region into smaller areas having different potential for hazardous earthquake effects. The earthquake effects depend on ground geomorphological attributes consisting of geological, geomorphology and geotechnical information. The parameters of geology and geomorphology, soil coverage/thickness, and rock outcrop/depth are some of the important geomorphological attributes. Other attributes are the earthquake parameters, which are estimated by hazard analysis and effects of local soil for a hazard (local site response for an earthquake). The Peak Ground Acceleration (PGA) [from deterministic or probabilistic approach], amplification/site response, predominant frequency, liquefaction and landslide due to earthquakes are some of the important seismological attributes. Weight of the attributes depends on the region and decision maker, for example flat terrain has weight of “0” value for landslide and deep soil terrain has highest weight for site response or liquefaction.

Followings are the Geomorphological, Geotechnical and Seismological themes considered for the microzonation of Greater Guwahati City (Nath, 2007a; Nath et al., 2008b; Nath and Thingbaijam, 2009):

i) Geology and Geomorphology (Base map)
ii) Effective Shear-wave Velocity (calculated from SPT data)
iii) Liquefaction Potential
iv) Landuse Map
v) Basement configuration and Thickness of Valley Fill
vi) Landslide Hazard Zonation
vii) Site Response (considered at Predominant Frequency)
viii) Predominant Frequency
xi) Peak Ground Acceleration (PGA calculated using F-K integration).

8.3.1 Landuse

For the preparation of Landuse map of Guwahati City, the 1990 landuse map prepared by the Assam State Remote Sensing Application Center, Guwahati was used as primary guide to carry out the baseline classification and ground checking to arrive at an appropriate landuse. Satellite image was used for detailed classification of different landuse classes. A detailed road to road GPS based survey was conducted to arrive at the cultural parameters such as residential, commercial and institutional areas in order to further sub-classify the broad habitation areas shown in greenish hue on the image. Ground checking was also conducted to collect data on other landuse classes such as water bodies, swamps, and agricultural field. A survey of the industrial areas was also conducted and shown in Figure 8.2.
The followings are the landuse classes derived – Residential Areas, Commercial Areas, Industrial Areas, Public Utility Areas, Educational Areas, Residential Areas in hill, Army/Police reserve, Airport, Field/Open areas, Agricultural land, Hills with Dense Forest, Hills with Light Forest, River, Sandbars, River Island, Waterbody/Bell and Swampy areas.

Figure 8.2: Landuse map of Guwahati region (after Nath, 2007a and 2007b).

8.3.2 Geology and Geomorphology (BASE MAP)

In order to facilitate compilation and collation of data and maps from varied sources on a single platform, the first task was to bring out a Base map of the area surrounding Guwahati where microzonation was contemplated. For this purpose, Department of Science & Technology, Govt. of India (DST) took up the matter with the Survey of India who provided 1:25,000 scale topographic sheets. The required information, coverage wise, such as roads, contour, railways, streams and certain administrative boundaries were traced on a transparency and digitized using onscreen digitization techniques. Further, IRS PAN & LISS III satellite images were acquired in digital format for the Greater Guwahati region. The images were
corrected and georeferenced with respect to the topographic map points and GPS readings taken of the
ground control points. Thereafter, the LISS and the PAN scenes were merged to produce a sharpened
high resolution FCC image having a ground resolution of 5.8 m. The stream network, road network, river
and water bodies were, thereafter updated using the satellite image. Further, an extensive GPS based
point survey was done in the greater Guwahati to capture all possible road networks, important places,
and other requisite features on the ground. Based on the above exercise, a comprehensive map of
Guwahati was prepared. It is this map that has been used all throughout the microzonation exercise. All
other maps, data and GPS point surveys were collated and corrected with respect to this base map,
which made it possible to weed out errors in various data sets for Guwahati Seismic Microzonation.

Broadly the Guwahati region consists of two main geological formations, viz. –

a) Precambrian granitic rocks forming the hill tracts and isolated hillocks, and
b) Quaternary alluvium occupying the valleys, deposited over the uneven eroded and faulted
   basement of granitic rocks.

The granite and granite gneissic rocks are well foliated and jointed, allowing deep weathering along
the joint and fault planes, and covered in most places by 1 to 3 m thick ferruginous soil capping.

The Quaternary alluvium, perhaps, form the flood plain deposit of the Brahmaputra River. This deposit
could be classified into 5 aggradational units based on lithological characters, state of weathering, order
of superposition and unconformity between them. They are, in order of increasing antiquity.

1. Active flood plain and levee deposit (AFP)
2. Digaru Surface (T1)
3. Bordang Surface (T2)
4. Sonapur Surface (T3), and
5. Pediment Surface (PD).

The rocks of active flood plains consist of alternate layers of silt/fine sand and clay/silty clay. The
rocks of Digaru Surface are represented mostly by silt and fine sand and are underlain by the Bordang
Surface below an unconformity. The Bordang Surface consists of white silty clay at the top and medium
to coarse grained sand at the bottom. The Sonapur Surface constitute the oldest unit of all the fluvial
deposits of Brahmaputra valley, mostly exposed near the foot hills showing contact with the pediment
forming colluviums or with the granite rocks. It comprises of bedded sand, silt and clay in varying proportion
with maximum amount of clay. The Pediment Surface is formed of weathered and eroded alluvial materials
deposited mostly along the foothills.

It is difficult to distinguish these litho units at depth from the study of sludge collected during digging
of boreholes up to 120 m depths for ground water exploration. However, it has been seen that sand,
silt, clay and gravel alternate in irregular proportion with extensive lateral variations. Mention may be made here that seismic resistivity sounding surveys by GSI could identify three layers of rocks at depth with resistivity characteristics of 200, 100 and 25 Ohm meter in the central and western part of the valley.

The important geological structures relevant to seismic microzonation are faults inferred to have cut across the valley fill of Quaternary Alluvium. Important faults are (1) NE-SW trending fault cutting across the Dipar Beel, which runs for about 15 km along Chotanagar – Maligaon area. It demarcates the boundary between the Nilachal and Fatasil hills; (2) a N10°E-S10°W tending fault running for about 10 km between the Kalapahar hill and the Fatasil hill; (3) a N40°E-S40°W trending fault passing along the Tepar Beel, running for about 20 km from the southern foot hill to the river Brahmaputra and (4) another important fault runs almost E-W from near Khanapara westward to the Dipar Beel in between the southern hills and the isolated hills of Kalapahar and Fatasil etc.

8.3.3 Geomorphology / Terrain Configuration

The overall topography of the area is rugged with high relief due to presence of steep sided hillocks carved out of the Meghalaya Plateau that occupies the southern and the eastern fringe of the area under consideration. The denuded and continuous hill tracts from Rani to Khanapara RF in the south and the Amchang Hills RF in the east rise in altitude from 200 m to 400 m above MSL. Isolated hillocks within the valley occasionally rise up to 300 m above MSL. The highest hillock is located in the north-western part of the area (Silapahar-381 m) and the lowest elevated hillock, Odalbakra – 145 m, lies in the center of the area. The relative relief is high varying between 80 m and 300 m. The general elevation of the valley area varies from 25 m to 50 m above MSL.

The major landform units can be mapped as

a) Denuded hills
b) Valley filled alluvium with almost flat surface, and
c) Swampy landmass/bels/water bodies.

It is important in the present context that many of the hillocks have been cut into small terraces and are occupied for habitation, and the swampy lands/water bodies have been filled up in many places for construction of houses due to tremendous pressure of population. A few of the hillocks have scarp or steeply inclined face. The isolated hillocks have NE-SW trending dendritic pattern of drainage of moderate density. Due to deep weathering along joints and faults the hill slopes have become unstable making them vulnerable to rock and debris slides, especially during heavy rains accompanied by earthquake shaking. Figure 8.3 depicts the Geology and Geomorphology map of Guwahati.
8.3.4 Basement Configuration and thickness of Valley Fill

The basement zonation map shown in Figure 8.4 forms N10°W-S10°E trending steep ‘V’ shaped valley with maximum depth of –150 m in between the Fatasil and Kalapahar hillocks that is traversed by a fault. Another NE-SW trending valley occurs in between the engineering College hill and the Fatasil hill that extend up to the junction of Neelachal hill and the Fatasil hill with maximum depth of –250 m. This valley is also underlain by a fault. A NE-SW trending valley passing along the Tepar beel by the western side of the Japorong hills reaches the depth of –100 m and it is relatively wide. An E-W deep valley passes from Panjabri to Kahabari across the Deepar bell. Toward east the basins has highs and lows but to the west the basin gradually get deeper and attains the depth of – 300 m. A fault also runs along this valley. Thus it is seen the valley area has variable depth of basement having steep gradient along some zones like the western, eastern and the southern margins of the Fatasil hill; western margin of the Kalapahar hill and the northern periphery of the Rani-Khanapara hill tract. These zones will have pronounced basin edge effect during earthquake shaking.
The city of Guwahati is located in an area surrounded on all sides by highly active tectonic blocks. Generally speaking the area is buttressed in between the Himalayan collision zone to the north and the north-east, Indo-Myanmar subduction interface of Indian plate to the east and the Meghalaya Plateau – Mikir hills tectonic block to its south. Strictly speaking the Guwahati area falls in the domain of Meghalaya Plateau and Mikir hill block. Juxtaposition of ongoing collision-subduction tectonic processes has made the area one of the most intense seismic zones of the world.

Analysis of contemporary tectonics in the region reveals that active Himalayan frontal thrusts and cross faults cutting across these thrusts have been generating many relatively shallow, small and moderate earthquakes. Strike-slip movements along a NW-SE trending fault (Po Chu) in the Mishmi block had produced the 1950 Great Assam earthquake ($M_w 8.6$) inflicting catastrophic damage in the Upper Assam area. All the other thrusts/faults in this block viz. Mishmi thrust, Lohit thrust and Tidding suture may be classified as capable faults.

The active subduction process along the Indo-Myanmar mobile belt and the conjugate faults lying across this belt has been producing many large and major earthquakes that shake the Guwahati area.
The tectonic block of Maghalaya Plateau-Mikir hill represents the north-eastern most exposed element of Indian shield, occupying a crucial position in between the northern collision and the eastern subduction zones of the Indian plate this block, is under tremendous stress and is seismically very active. The Guwahati area being located in the northern margin of this block is vulnerable to severe earthquake damage. The N-S faults cutting across the Plateau such as the Jamuna or Dhubri fault, Dhudnoi/Chedrang fault, and Kulsi fault in the Meghalaya Plateau are very active; NW-SE Kopili fault passing in between the Plateau and the Mikir hill and the NW-SE Bomdila fault passing along the northern margin of Mikir hill, both traversing across the Himalayan thrust and fold belt as well as the Indo-Myanmar mobile belt are very important.

The Jamuna or Dhubri fault has been the source for 1931 (M_w 7.1) Dhubri earthquake. Movement along the Dhudnoi or Chedrang fault generated the 1897 great Assam earthquake. The Kopili fault has the record of producing the 1869 Cachar earthquake (M_w >7) and 1943 earthquake (M_w >7). Moreover, recent recording of earthquake events clearly demonstrate that Kopili fault is highly active at present. Another important active fault is the NE-Sylhet and its associated faults falling in the tectonic domain of Bengal basin. This fault had generated the 1918 Srimangal earthquake (M_w 7.6). Similar earthquake may affect the Guwahati city. The E-W Dauki fault system, a regional structure of great importance, though seems to be dormant at present may produce large earthquakes that may affect the Guwahati city. Focal mechanism solutions of the past earthquakes reveal that most of the events were due to strike slip motion in these terrains (Nandy and Dasgupta, 1991; Nandy, 2001).

Thus it is seen that the Guwahati city area is vulnerable to catastrophic near source great and large earthquakes. It may be mentioned here that recent release of stress along a 1200 km long subduction interface in the southern part of the Indo-Maynmar-Andaman-Sunda subduction zone by the 26th December, 2004, 9.1 magnitude earthquake has made it highly probable that next rupture may take place along the northern sector of the subduction zone in Indo-Myanmar region as shown in Figure 8.5.

8.3.6 Landslide Hazard Zonation

Detailed work by Keffer, 1984 on 40 historical earthquakes and numerous landslides induced by them revealed that most of the slides get triggered at intensity VI and above (M.M. Scale) with very few slides occurring at lower intensity zones. The thickness of weathered zone/soil is fairly high in the granite hillocks in and around the Guwahati city. Many of the landslide incidences (baring a few in the Kalapahar area) are due to anthropogenic activities. A total of 6 landslides of different categories have been recorded. Most of them are slump type followed by debris slide and rock fall.

Twenty two landslides are concentrated in and around the Guwahati city, especially in its central part. Kalapahar area experienced 10 slides followed by Dhirenpara having 4 slides. Area along the G-S road has the record of 9 landslides. Based on geo-environmental parameters like slope angle, lithology, structure,
Figure 8.5: Seismotectonic map of the Guwahati region (after Nath, 2007a).
relative relief, landuse, landcover, hydrological correlation, seismicity, rainfall and landslide incidences were considered for preparation of landslide hazard zonation map. Eight thematic maps were first prepared viz., facet map, slope morphometry map, relative relief map, lithological map, structural map, landuse, landcover map, drainage map, landslide incidence map, on 1:50,000 scales and then enlarged to 1:25,000 scales. The landslide hazard zonation map has been prepared according to total estimated hazard (TEH) of each face by superimposing the slope facet map successively one by one over all the thematic maps. The TEH of the facet is calculated after adding the values of landslide hazard evaluation factor (LHEF) of all 9 geo-environmental parameters encompassing the particular facet. The derived landslide hazard zonation map thus prepared showed only three categories of hazard zone such as Low Category of Hazard Zone (LHZ), Moderate Hazard Zone (MHZ) and High Hazard Zone (HHZ) as shown in Figure 8.6.

Figure 8.6: Landslide Hazard Zonation map of Guwahati region (after Nath, 2007a).

8.3.7 Shear Wave Velocity ($V_s^{30}$) Map

Shear Wave Velocity distribution map of Guwahati region is shown in Figure 8.7.
Figure 8.7: Shear Wave Velocity distribution map of Guwahati region (after Nath, 2007a and 2007b).

Figure 8.8: Predominant Frequency distribution map of Guwahati region (after Nath, 2007a).
8.3.8 Predominant Frequency Map

Predominant Frequency Distribution map of Guwahati region is given in Figure 8.8.

8.3.9 Site Response Map

The Site Response is another seismological theme. Site Response Distribution map is presented in Figure 8.9.

![Site Response Distribution Map of Guwahati Region](image)

**Figure 8.9:** Site Response distribution map of Guwahati region (after Nath, 2007a).

8.3.10 Factor of Safety Map

A component subjected to a solitary load will be considered in the first instance. This load is interpreted in the context of the component’s nature and duty - thus load usually implies a transverse force in the case of a beam component, or a longitudinal compressive force in a column, or a torque in the case of a shaft, or a pressure in a fluid containment vessel, and so on.

There are two completely different manifestations of the load, which have important consequences for the component:
• The extrinsic actual load is the load exerted on the component by its surrounds,
• The intrinsic maximum load is the largest load that the component can withstand without failure; the maximum load is a property of the component, a function of its dimensions and material.

Clearly a component is safe only if the actual load applied to the component does not exceed the component’s inherent maximum sustainable load. The degree of safety is usually expressed by the safety factor ‘n’.

\[ n = \frac{\text{maximum load}}{\text{actual load}} = \frac{F_{\text{max}}}{F} \]

and it follows that:

if \( n = 1 \) then the component is on the point of failure
if \( n < 1 \) then the component is in a failed state
if \( n > 1 \) then the component is safe.

The intrinsic details of liquefaction hazard assessment have been discussed in Chapter – 6 in Section 6.3.

Accordingly Guwahati is classified in two zones namely safe and unsafe. Factor of safety distribution map of Guwahati region is shown in Figure 8.10.

Figure 8.10: Factor of Safety zonation map of Guwahati region (after Nath, 2007a).
8.3.11 Site Classification Map

Detailed site classification of Guwahati City is given in Appendix – V and shown in Figure 8.11.

![Site Classification Map of Guwahati Region](image)

Figure 8.11: Site Classification map of Guwahati region (after Nath, 2007a).

8.3.12 Peak Ground Acceleration (PGA) Map

PGA is estimated in the Guwahati region by two methods, but for microzonation purposes PGA estimated by F-K integration as shown in Figure 8.12 is used.

8.4 GIS INTEGRATION LOGIC

The representation and interpretation of uncertainty related to the classification of individual locations provided by the fuzzy logic based on location attribute values. Fuzzy logic implements classes or groupings of data with boundaries. The central idea of fuzzy sets is aided by the Analytic Hierarchy Process (AHP). AHP is a multi-criteria decision method that uses hierarchical structures to represent a problem and then develop priorities for the alternatives based on the judgment of the user (Saaty, 1980). The idea of
multi-criteria decision-making was based on the concept of McHarg (1968). McHarg (1968) introduced a systematic land use planning by using the concept of compatibility of multiple land uses. He mentioned that the factors affecting land and its relative values are different and, therefore, it is difficult to think of optimizing them for a single use. It can be optimized for multiple compatible uses. He introduced simple matrix system for determining the degree of compatibility.

Saaty (1968) has shown that weighting activities in multi-criteria decision-making can be effectively dealt with hierarchical structuring and pair-wise comparisons. Pair-wise comparisons are based on forming judgments between two particular elements rather than attempting to prioritize an entire list of elements (Saaty, 1980). For multi-criteria evaluation, Saaty’s Analytical Hierarchy Process (AHP) is used to determine the weights of each individual criterion (Saaty, 1990). AHP is a mathematical method to determine priority of criteria in the decision making process. It is a popular tool used by decision makers in the multi-attribute decisions.

Saaty’s Analytical Hierarchy process constructs a matrix of pair-wise comparisons (ratios) between the factors of Earthquake Hazard Parameters (EHP). The constructed matrix shows the relative importance of the EHP based on their weights. If 9 earthquake hazard parameters are scaled as 1 to 9, 1 meaning that the
two factors are equally important, and 9 indicating that one factor is more important than the other, the reciprocals of 1 to 9 (i.e., 1/1 to 1/9) show that one is less important than the others. The allocation of weights for the identical EHP depends on the relative importance of factors and participatory group of decision makers. Then the individual normalized weights of each EHP are derived from the matrix developed by pair-wise comparisons between the factors of EHP. This operation is performed by calculating the principal Eigen vector of the matrix. The results are in the range of 0 to 1 and their sum adds up to ‘1’ in each column. The weights for each attribute can be calculated by averaging the values in each row of the matrix. These weights will also sum to ‘1’ and can be used in deriving the weighted sums of rating or scores for each region of cells or polygon of the mapped layers (Jones, 1997).

Since EHP vary significantly and depends on several factors, they need to be classified into various ranges or types, which are known as the features of a layer. Hence each EHP features are rated or scored within EHP and then this rate is normalized to ensure that no layer exerts an influence beyond its determined weight. Therefore, a raw rating for each feature of EHP is allocated initially on a standard scale such as 1 to 10 and then normalized using the relation,

\[ X_i = \frac{R_i - R_{\text{min}}}{R_{\text{max}} - R_{\text{min}}} \]  (8.1)

Where \( R_i \) is the rating assigned for features with single EHP, \( R_{\text{min}} \) and \( R_{\text{max}} \) is minimum and maximum rate of particular EHP.

For seismic microzonation and hazard delineation of the above themes of Greater Guwahati region, both geomorphological and seismological are reclassified into a 1st phase: geohazard map and 2nd phase: seismic microzonation map with PGA distribution for a SEM of \( M_w 8.7 \). A typical two phase microzonation procedure is followed from hazard zonation to regional hazard zonation mapping on GIS platform and finally to seismic microzonation as shown in the detailed roadmap given in Figures 2.6, 2.7 and 2.8.

The geological vector layer that have been used for microzonation are Geology and Geomorphology (GG), Basement (BS), Landslide hazard zones (LS) and Landuse (LU), where as the seismological themes are Shear Wave Velocity (\( V_{s30} \)), Site Response (SR), Peak Ground Acceleration(PGA), Predominant frequency (PF) and Factor of Safety (FS).

The geological and seismological themes are in weight scale of 9:1 depending on their contribution to seismic hazard, the highest being attached to shear Geological & Geomorphological layer with a normalized weight of 0.2000. Basement has got the next weightage with a normalized value of 0.1778; Landslide hazard and Landuse have got the next weightage with normalized value of 0.1556 and 0.1333 where as the Shear Wave Velocity (\( V_{s30} \)), Peak Ground Acceleration (PGA), Site Response, Predominant Frequency and Factor of Safety are assigned the values 0.1111, 0.0889, 0.0667, 0.0444 & 0.0222 respectively.

In this method, a matrix of pair-wise comparisons (ratio) between the factors is built, which is used to derive the individual normalized weights of each factor. The pair-wise comparison is performed by calculating the principal eigen vector of the matrix and the elements of the matrix are in the range of 0 to 1 summing to ‘1’
in each column. The weights for each theme can be calculated by averaging the values in each row of the matrix. These weights will also sum to ‘1’ and can be used in deriving the weighted sum of rating or scores of each region of cells or polygon of the mapped layers. Since the values within each thematic map/layer vary significantly, those are classified into various ranges or types known as the features of a layer. These features are then assigned ratings \( r \) or scores within each layer, normalized to 0-1 as shown in Table 8.1 below.

<table>
<thead>
<tr>
<th>Theme</th>
<th>Weight</th>
<th>Feature</th>
<th>Rating</th>
<th>Normalized Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geology (GG)</td>
<td>0.2000</td>
<td>River, Water Bodies &amp; Swampy area</td>
<td>8</td>
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<td></td>
<td></td>
<td>Active Flood Plain</td>
<td>7</td>
<td>0.8571</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Natural Levee</td>
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<td>0.7143</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pediment</td>
<td>5</td>
<td>0.5714</td>
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<tr>
<td></td>
<td></td>
<td>Sonapur Surface</td>
<td>4</td>
<td>0.4286</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Digaru Surface</td>
<td>3</td>
<td>0.2857</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bordang Surface</td>
<td>2</td>
<td>0.1429</td>
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<tr>
<td></td>
<td></td>
<td>Denuded Hills</td>
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<td>0.0000</td>
</tr>
<tr>
<td>Basement (BS)</td>
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<td>600</td>
<td>7</td>
<td>1.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>500 – 600</td>
<td>6</td>
<td>0.8333</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400 – 500</td>
<td>5</td>
<td>0.6667</td>
</tr>
<tr>
<td></td>
<td></td>
<td>300 – 400</td>
<td>4</td>
<td>0.5000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 – 300</td>
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<td>0.3333</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100 – 200</td>
<td>2</td>
<td>0.1667</td>
</tr>
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<td></td>
<td></td>
<td>&lt;50 –100</td>
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<td>0.0000</td>
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<tr>
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<td></td>
<td>Medium Hazard Zone</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Low Hazard Zone</td>
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<td>0.3333</td>
</tr>
<tr>
<td></td>
<td></td>
<td>River, Sand Bar &amp; Hazard Free Zone</td>
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<td>0.0000</td>
</tr>
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<tr>
<td></td>
<td></td>
<td>Educational, Army/ Police Reserve, Commercial area</td>
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<td>0.8333</td>
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<td></td>
<td>Sandbars, River Island &amp; Swampy area</td>
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<td></td>
<td>Field/ open space &amp; Agricultural Area</td>
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<td>Residential Areas in Hill</td>
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<td>0.3333</td>
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<td></td>
<td></td>
<td>Hill with dense &amp; light forest</td>
<td>2</td>
<td>0.1667</td>
</tr>
<tr>
<td></td>
<td></td>
<td>River, water bodies/ Beel</td>
<td>1</td>
<td>0.0000</td>
</tr>
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<td>Theme</td>
<td>Weight</td>
<td>Feature</td>
<td>Rating</td>
<td>Normalized Rating</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>--------</td>
<td>---------------</td>
<td>--------</td>
<td>-------------------</td>
</tr>
<tr>
<td>Shear Wave Velocity (V_s^{30})</td>
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<td>200 – 240</td>
<td>4</td>
<td>1.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>240 – 280</td>
<td>3</td>
<td>0.6667</td>
</tr>
<tr>
<td></td>
<td></td>
<td>280 – 320</td>
<td>2</td>
<td>0.3333</td>
</tr>
<tr>
<td></td>
<td></td>
<td>320 – 360</td>
<td>1</td>
<td>0.0000</td>
</tr>
<tr>
<td>Peak Ground Acceleration (PGA)</td>
<td>0.0889</td>
<td>&gt;= 0.75</td>
<td>6</td>
<td>1.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.60 – 0.75</td>
<td>5</td>
<td>0.8000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45 – 0.60</td>
<td>4</td>
<td>0.6000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.30 – 0.45</td>
<td>3</td>
<td>0.4000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.15 – 0.30</td>
<td>2</td>
<td>0.2000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt; 0.15</td>
<td>1</td>
<td>0.0000</td>
</tr>
<tr>
<td>Site Response (SR)</td>
<td>0.0667</td>
<td>&gt;= 5.5</td>
<td>5</td>
<td>1.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5 – 5.5</td>
<td>4</td>
<td>0.7500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0 – 4.5</td>
<td>3</td>
<td>0.5000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.5 – 3.0</td>
<td>2</td>
<td>0.2500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt;1.5</td>
<td>1</td>
<td>0.0000</td>
</tr>
<tr>
<td>Predominant Frequency (PF)</td>
<td>0.0444</td>
<td>&lt;0.5</td>
<td>8</td>
<td>1.0000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5 – 1.0</td>
<td>7</td>
<td>0.8571</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0 – 2.0</td>
<td>6</td>
<td>0.7143</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.0 – 3.0</td>
<td>5</td>
<td>0.5714</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0 – 4.0</td>
<td>4</td>
<td>0.4286</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.0 – 5.0</td>
<td>3</td>
<td>0.2857</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.0 – 7.0</td>
<td>2</td>
<td>0.1429</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 7.0</td>
<td>1</td>
<td>0.0000</td>
</tr>
<tr>
<td>Factor of Safety(FS)</td>
<td>0.0222</td>
<td>≤1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;1</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The probabilistic seismic hazard zonation map is obtained through the integration of all the above themes using the following relation.

\[
PSSH = GG_wGG_r + B_wB_r + LU_wLU_r + V_s^{30}wV_s^{30}r + PGA_wPGA_r + SR_wSR_r + PF_wPF_r + FS_wFS_r/\Sigma_w \tag{8.2} \]

The notations have their usual meanings. The integration scheme is shown in Figure 8.13. Four zones are mapped as shown in Figure 8.14 where average PSHI index are >0.55, 0.44, 0.35, 0.26 and <0.16. We termed these zones as very high, high, moderate, low and very low hazard regions. Figure 8.14 shows microzonation map of Guwahati region overlaid by PGA computed by Green’s Function approximation.
Figure 8.13: The seismic microzonation scheme for Guwahati city with the weights assigned to each theme labeled accordingly leading to the final hazard map. A representative accelerogram simulated at a borehole location for the projected maximum earthquake of $M_w 8.3$ nucleating from the Shillong seismic zone is also depicted (after Nath and Thingbaijam, 2009).

Figure 8.14: Seismic Microzonation Map of Guwahati regions (using PGA computed by F-K integration) (after Nath, 2007a and 2007b; Nath and Thingbaijam, 2009).
8.5 EARTHQUAKE HAZARD PARAMETERS FOR BANGALORE

Different attributes considered for Bangalore microzonation (Anbazhagan et al., 2010) are presented below:

8.5.1 Geomorphological Attributes

The geomorphological attributes considered in this study are the geology and geomorphology (GG), rock depth/soil thickness (RD/ST), soil type and strength (represented in terms of average shear wave velocity) (SS), drainage pattern (DP) and elevation level (EL).

8.5.1.1 Geology and Geomorphology (GG)

As the study area is densely covered by buildings, it is very difficult to obtain the detailed geological/geomorphological maps. A simple geological and geomorphology map of Bangalore is prepared and presented in Figure 8.15 for the study area.

Figure 8.15: Geology and Geomorphology of the study area of Bangalore City (after Anbazhagan et al., 2009b and 2010).
Bangalore lies on top of the south Karnataka Plateau (Mysore Plateau) and its topology is almost flat with the highest point being at Doddabettahalli (954 m above Mean Sea Level) in the direction of a NNE-SSW trending ridge lies east of the Vrishabhavathi river. The study area falls under the expanse of the Peninsular Gneissic Complex. The main rock types in the regions are Gneissic country rock and as well as intrusions of Granites and Magmatites. Bangalore city lies over a hard and moderately dense Gneissic basement dated back to the Archean era (2500-3500 mya). A large granitic intrusion in the south-central part of the city extends from the Golf Course in the north central to Vasantpur V V Nagar in the south of the city (almost 13 km in length) and on an average 4 km from east to west along the way. A magmatite intrusion formed within the granitic one extends for approximately 7.3 km running parallel with Krishna Rajendra Road/ Kanakpura Road from Puttanna Chetty Road in Chamrajpet till Bikaspura Road in the south. A 2.25 km Quatrzite formation is found in Jalahalli East (Figure 8.15).

Dike swarms are seen around the western outskirts of the city (west of the Outer Ring Road) majority striking approximately oriented on N15°E. However random east west trending ones are also seen. They appear to strike parallel to the strike of the vertical foliation of the country rock. These basic intrusions dated back to close of the Archean era (Lower Proterozoic; 1600-2500 mya) mainly constitute of hard massive rocks such as Gabbro, Dolerite, Norite and Pyroxenite.

Bangalore city is subjected to a moderate annual soil erosion rate of 10 Mg/ha. The basic geomorphology of the city comprises of a central Denudational Plateau and Pediment (towards the west) with flat valleys that are formed by the present drainage patterns. The central Denudational Plateau is almost void of any topology and the erosion and transportation of sediments carried out by the drainage network gives rise to the lateritic clayey alluvium seen throughout the central area of the city. The pediment/pediplain is a low relief area that abruptly joins the plateau.

### 8.5.1.2 Rock Depth/ Soil Thickness (RD/ST)

Another important theme is the overburden thickness of soil; it can be represented as rock depth or soil thickness. The overburden thickness of Bangalore is estimated using drilled borehole information at selected 170 locations from 850 borehole data. The overburden thickness varies from 1 m to 33 m in the study area and it is shown in Figure 8.16. The south central part and north eastern part has largest overburden thickness compared to other areas. On average, Bangalore has the overburden thickness of less than 5 m on the western side and about 15 m in the rest of the places. These overburden thickness obtained from boreholes does not represent thickness of soil from the true engineering rock (shear wave velocity is 700 m/s) level. They correspond to thickness of overburden above the weathered rock. Hence the overburden thickness of Bangalore is represented in the form of engineering rock level using MASW results. Figure 8.17 shows the overburden thickness/engineering rock depth using MASW. Engineering rock depth varies from 5 m to 55 m. An average engineering rock depth is found to be about 15 m from the original ground level.
Figure 8.16: Soil thickness using borehole data (after Anbazhagan et al., 2009b and 2010).

Figure 8.17: Engineering rock depth using MASW (after Anbazhagan et al., 2009b and 2010).
8.5.1.3 Soil Type and Strength (SS)

Seismic response and liquefaction analyses require the mechanical and geometrical parameters of the overburden soil above the engineering rock depth. Mechanical and geometrical parameters represent the properties of overburden soil in terms of geology, average shear wave velocity of the medium and standard penetration test ‘N’ values. The site/area can be classified/characterized for site response or liquefaction behavior based on the above three parameters. During site characterization, it is necessary to determine the variations in soil stratification and engineering properties of soil and rock layers encountered at the site preferably based on in-situ tests and laboratory tests conducted on the samples obtained during soil exploration.

Initially site classification is achieved based on the characteristics of geologic units taking into consideration the possible variations in each unit.

Wills and Silva (1998) suggested that average shear wave velocity in the upper 30 m can be used as one parameter to characterise the geological units. Equivalent (average) shear wave velocity became popular criterion for site characterization. Equivalent (average) shear wave velocity is defined as the weighted average of shear wave velocities of soil and rock layers in the top 30 meters. Equivalent shear

![30 m Average Shear Wave Velocity](image)

**Figure 8.18:** 30 m Average Shear Wave Velocity (after Anbazhagan et al., 2009b and 2010).
wave velocities are being used in earthquake codes for the purpose of evaluating the design earthquake characteristics on the ground surface (Borcherdt, 1994). In addition, it is also possible to use empirical relationships to estimate spectral amplifications based on equivalent shear wave velocity. Equivalent shear wave velocity can be calculated by conducting in-situ seismic wave velocity measurements or by using correlations developed in terms of SPT-standard penetration or CPT-cone penetration tests. Equivalent (average) shear wave velocity of the study area is calculated based on in-situ measured shear wave velocity using MASW. Figure 8.18 shows the 30 m average shear wave velocity of Bangalore.

It is observed that the average of 30 m velocity pertains to the major part of the BMP area that can be classified as “site class D”, and “site class C” and a smaller part in and around Lalbagh Park is classified as “site class B”. The average shear wave velocity for soil overburden in the study area is shown in Figure 8.19 that depicts the whole study area has a medium to dense soil with an average velocity range of 180 m/s to 360 m/s falling into “site class D”. These two average shear wave velocities maps are considered as separate themes for GIS integration.

Figure 8.19: Soil Overburden average Shear Wave Velocity (after Anbazhagan et al., 2009b and 2010).
8.5.1.4 Drainage Pattern (DP) and Elevation Level (EL)

The geotechnical attributes presented above are based on a large number of geotechnical data and experiments. But the geological and geomorphologic information is presented as one map based on available information. This map does not have sufficient information and account for other factors such as impedance contrast, 3-dimensional basin and topographical effects. Hence to account for the above parameters, the other important parameters of drainage pattern (DP) and elevation level (EL) are considered as separate themes based on the recent available information. Figure 8.20 shows the drainage pattern of the study area with water bodies and Figure 8.21 shows the elevation levels in the form of contours at 10 m intervals.

Figure 8.20: Tanks and drainage features in the study area (after Anbazhagan et al., 2009b and 2010).
8.5.2 Seismological Attributes

The seismological thematic maps have been generated based on detailed studies of seismic hazard analysis, site response studies and liquefaction analysis. From these studies different earthquake hazard parameters are mapped. But for final Index map preparation and GIS integration only selected maps are considered as the following themes:

- Peak Ground Acceleration (PGA) at rock level based on synthetic ground motions from MCE based on DSHA.
- PGA at rock level at 10% probability of exceedance in 50 years based on PSHA.
- Amplification factor based on ground response analysis using SHAKE 2000.
- Predominant frequency based on site response and experimental studies.
- Factor of safety against Liquefaction potential.

Figure 8.21: Terrain slope based on the elevation contour (after Anbazhagan et al., 2009b and 2010).
8.5.2.1 PGA Map for MCE from DSHA

From detailed deterministic seismic hazard analysis maximum credible earthquake for Bangalore is $M_w$ of 5.1. A synthetic ground motion model has been generated for MCE considering the Mandya-Channapatna-Bangalore lineament (L15) as the source. The synthetic ground motions are generated at rock level at 653 borehole locations using rock depth information obtained from geotechnical data. The peak ground acceleration at each borehole locations is obtained from synthetic ground motions. PGA at rock level from ground motions using DSHA is as shown in Figure 8.22.

![Peak Ground Acceleration using DSHA](after Anbazhagan et al., 2009b and 2010).

Figure 8.22: Peak Ground Acceleration using DSHA (after Anbazhagan et al., 2009b and 2010).

8.5.2.2 PGA Map from PSHA

A detailed probabilistic seismic hazard analysis has been performed using six seismogenic sources identified in DSHA. The hazard curves and UHRS at 10% probability of exceedance in 50 years are calculated for
about 1400 grid points in the study area having the size of 0.5 km × 0.5 km. Further to define seismic hazard at rock level for the study area, PGA at each grid point has been estimated. These values are used to prepare PGA maps for 10% probability of exceedance in 50 years, which corresponds to a return period of 475 years. Rock level PGA map for Bangalore is shown in Figure 8.23 depicting that the PGA values vary from 0.17g to 0.25g, which is considered as a theme in GIS for probabilistic hazard index mapping.

8.5.2.3 Amplification Factor

The basic intention of site response analysis is to estimate the effect of local site conditions in assessing the site amplification with respect to ground shaking. Amplification factor is used as a theme to represent the ground behavior during the earthquake. In this study the term “Amplification Factor” is referred to as
the ratio of the peak horizontal acceleration at the ground surface to the peak horizontal acceleration at
the bedrock. This factor is evaluated for all the boreholes using PGA at bedrock obtained from synthetic
acceleration time history at each borehole location and the peak ground surface acceleration obtained
as a result of ground response analysis using SHAKE 2000. Figure 8.24 shows the amplification factor
theme for GIS integration.

![Amplification Factor Map](image)

**Figure 8.24:** Amplification Factor Map (after Anbazhagan *et al.*, 2009b and 2010).

### 8.5.2.4 Predominant Frequency

Eventhough amplification is a major concern in ground response analysis, building failure depends on
the period of ground motion at which the resonance can occur (i.e., predominant period/frequency of soil
column). Hence it is necessary to add the predominant period/frequency of soil column as a theme to
represent index of the study area. The study area has predominant frequency ranges from 3 Hz to 12 Hz.
Figure 8.25 shows the predominant frequency theme for the study area.
8.5.2.5 Factor of Safety against Liquefaction

Factor of Safety against liquefaction (FS) is added as a theme to represent the liquefaction behavior of soil in the study area. At about 620 borehole locations, FS has been calculated using amplified PGA values at the ground level considering simplified procedure of Seed and Idriss (1971) with subsequent revisions after Seed et al. (1983 and 1985), Youd et al. (2001) and Cetin et al. (2004). Figure 8.26 shows the factor of safety against liquefaction theme on GIS.
8.5.3 Integration of different Layers (Themes)

For seismic microzonation and hazard delineation all the themes presented above are integrated to generate seismic microzonation maps. The final microzonation maps can be represented in three forms: (1) hazard map, (2) vulnerability map, and (3) risk map; because earthquake loss not only depends on the hazard caused by earthquakes, but also on exposure (social wealth) and its vulnerability. Usually hazard map gives the hazard index (HI) based on hazard calculation and site conditions. Vulnerability map presents the expected degree of losses within a defined area resulting from the occurrence of earthquakes and is often expressed on a scale from 0 (no damage) to 1 (full damage). Vulnerability study includes all the exposure such as man-made facilities that may be impacted in an earthquake. It includes all residential, commercial, and industrial buildings, schools, hospitals, roads and railroads, bridges, pipelines, power plants, communication systems, and so on. Risk map is the combination of hazard classes and vulnerability classes to yield risk classes.
Hazard index is the integrated factor depending on the weights and ranks of the seismological and geomorphological themes. Theme weights can be assigned based on their contribution to the seismic hazard. Rank can be assigned within each theme based on their values closer to hazards. Usually higher rank will be assigned to values, which is more hazardous in nature, for example larger PGA will have the higher rank. The contributing themes and their weights are listed in Table 8.2.

**Table 8.2: Themes and its weights for GIS integration**

<table>
<thead>
<tr>
<th>Index</th>
<th>Themes</th>
<th>Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>Rock level PGA using DSHA-DPGA</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Rock level PGA using PSHA-PPGA</td>
<td>9</td>
</tr>
<tr>
<td>AF</td>
<td>Amplification factor</td>
<td>8</td>
</tr>
<tr>
<td>ST</td>
<td>Soil Thickness using MASW</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Soil Thickness using borehole</td>
<td>7</td>
</tr>
<tr>
<td>SS</td>
<td>Equivalent Shear Wave Velocity for Soil</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Equivalent Shear Wave Velocity for 30 depth</td>
<td>6</td>
</tr>
<tr>
<td>FS</td>
<td>Factor of Safety against liquefaction</td>
<td>5</td>
</tr>
<tr>
<td>PF</td>
<td>Predominant period / frequency</td>
<td>4</td>
</tr>
<tr>
<td>EL</td>
<td>Elevation levels</td>
<td>3</td>
</tr>
<tr>
<td>DR</td>
<td>Drainage pattern</td>
<td>2</td>
</tr>
<tr>
<td>GG</td>
<td>Geology and Geomorphology</td>
<td>1</td>
</tr>
</tbody>
</table>

Once the identical weights are assigned then normalized weights can be calculated based on the pair-wise comparison matrix. Some of the attributes (like PGA and $V_s$) have two values for the same theme; hence both are given same weights with different percentages. The normalized weights are calculated using Saaty's Analytical Hierarchy Process (Nath, 2004). In this method, a matrix of pair-wise comparisons (ratio) between the factors is built, which is used to derive the individual normalized weights of each factor. The pair-wise comparison is performed by calculating the principal Eigen vector of the matrix and the elements of the matrix are in the range of 0 to 1 summing to ‘1’ in each column. The weights for each theme can be calculated by averaging the values in each row of the matrix. These weights will also sum to ‘1’ and can be used in deriving the weighted sum of rating or scores of each region of cells or polygon of the mapped layers. Since the values within each thematic map/layer vary significantly, those are classified into various ranges or types known as the features of a layer. Table 8.3 shows the pair-wise comparison matrix of the themes and the calculated of normalized weights.
### Table 8.3: Pair-wise comparison matrix of Themes and their normalized weights

<table>
<thead>
<tr>
<th>Theme</th>
<th>PGA</th>
<th>AF</th>
<th>ST</th>
<th>Vs</th>
<th>FS</th>
<th>PF</th>
<th>EL</th>
<th>DR</th>
<th>GG</th>
<th>Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>1</td>
<td>9/8</td>
<td>9/7</td>
<td>9/6</td>
<td>9/5</td>
<td>9/4</td>
<td>9/3</td>
<td>9/2</td>
<td>9/1</td>
<td>0.200</td>
</tr>
<tr>
<td>AF</td>
<td>8/9</td>
<td>1</td>
<td>8/7</td>
<td>8/6</td>
<td>8/5</td>
<td>8/4</td>
<td>8/3</td>
<td>8/2</td>
<td>8/1</td>
<td>0.178</td>
</tr>
<tr>
<td>ST</td>
<td>7/9</td>
<td>7/8</td>
<td>1</td>
<td>7/6</td>
<td>7/5</td>
<td>7/4</td>
<td>7/3</td>
<td>7/2</td>
<td>7/1</td>
<td>0.156</td>
</tr>
<tr>
<td>Vs</td>
<td>6/9</td>
<td>6/8</td>
<td>6/7</td>
<td>1</td>
<td>6/5</td>
<td>6/4</td>
<td>6/3</td>
<td>6/2</td>
<td>6/1</td>
<td>0.133</td>
</tr>
<tr>
<td>Fs</td>
<td>5/9</td>
<td>5/8</td>
<td>5/7</td>
<td>5/6</td>
<td>1</td>
<td>5/4</td>
<td>5/3</td>
<td>5/2</td>
<td>5/1</td>
<td>0.111</td>
</tr>
<tr>
<td>PF</td>
<td>4/9</td>
<td>4/8</td>
<td>4/7</td>
<td>4/6</td>
<td>4/5</td>
<td>1</td>
<td>4/3</td>
<td>4/2</td>
<td>4/1</td>
<td>0.089</td>
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<td>EL</td>
<td>3/9</td>
<td>3/8</td>
<td>3/7</td>
<td>3/6</td>
<td>3/5</td>
<td>3/4</td>
<td>1</td>
<td>3/2</td>
<td>3/1</td>
<td>0.067</td>
</tr>
<tr>
<td>DR</td>
<td>2/9</td>
<td>2/8</td>
<td>2/7</td>
<td>2/6</td>
<td>2/5</td>
<td>2/4</td>
<td>2/3</td>
<td>1</td>
<td>2/1</td>
<td>0.044</td>
</tr>
<tr>
<td>GG</td>
<td>1/9</td>
<td>1/8</td>
<td>1/7</td>
<td>1/6</td>
<td>1/5</td>
<td>1/4</td>
<td>1/3</td>
<td>1/2</td>
<td>1</td>
<td>0.022</td>
</tr>
</tbody>
</table>

Within individual theme a grouping has been made according to their values. Then rank is assigned based on the values. Usually these ranks vary from 1 to 10, highest rank is assigned for values more hazardous in nature. These ranks are normalized to 0-1 using the Equation (8.1). The assigned ranks with normalized values are given in Table 8.4.

Based on above attributes, two types of hazard index maps are generated. One is Deterministic Seismic Microzonation map (DSM), which is basically a deterministic hazard index map using PGA from deterministic approach and other themes. Another map is the Probabilistic Seismic Microzonation map (PSM). Probabilistic hazard index are calculated similar to DSM but PGA is obtained from probabilistic seismic hazard analysis.

### Table 8.4: Normalized ranks of the themes for Bangalore Microzonation (after Anbazhagan et al., 2010)

<table>
<thead>
<tr>
<th>Themes</th>
<th>Values</th>
<th>Weight</th>
<th>Ranks</th>
<th>Normalized Ranks</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA (g)</td>
<td></td>
<td>0.200</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt;0.12</td>
<td></td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.12 to 0.13</td>
<td></td>
<td>2</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>0.13 to 0.14</td>
<td></td>
<td>3</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>0.14 to 0.15</td>
<td></td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>AF</td>
<td>1-2</td>
<td>0.178</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2-3</td>
<td></td>
<td>2</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>3-4</td>
<td></td>
<td>3</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>&gt;4</td>
<td></td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>ST (m)</td>
<td>1-5</td>
<td>0.156</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td></td>
<td>2</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>10-15</td>
<td></td>
<td>3</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Integration of Hazard Maps on GIS Platform for Seismic Microzonation

<table>
<thead>
<tr>
<th>Themes</th>
<th>Values</th>
<th>Weight</th>
<th>Ranks</th>
<th>Normalized Ranks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15-20</td>
<td>4</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20-25</td>
<td>5</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>$V_s$ (m/s)</td>
<td>$&lt;100$</td>
<td>0.1333</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>100-200</td>
<td>2</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>200-300</td>
<td>3</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td></td>
<td>300-400</td>
<td>4</td>
<td>1</td>
<td></td>
</tr>
<tr>
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<td>$&lt;1$</td>
<td>0.111</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1-2</td>
<td>2</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$&gt;2$</td>
<td>1</td>
<td>0</td>
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<td>0</td>
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<td></td>
<td>3.5-5</td>
<td>2</td>
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<tr>
<td></td>
<td>5-7.5</td>
<td>3</td>
<td>0.5</td>
<td></td>
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<tr>
<td></td>
<td>7.5-9</td>
<td>4</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9-11</td>
<td>5</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>EL</td>
<td>Steep slope</td>
<td>0.067</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Moderate slope</td>
<td>3</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Low slope</td>
<td>2</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flat ground</td>
<td>1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>DR</td>
<td>Lake area</td>
<td>0.0444</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Drainage area</td>
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<td>0.5</td>
<td></td>
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<tr>
<td></td>
<td>Flat area</td>
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<td>0</td>
<td></td>
</tr>
<tr>
<td>GG</td>
<td>Granitic intrusion</td>
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<td>4</td>
<td>1</td>
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<tr>
<td></td>
<td>Magnetic instruction</td>
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<td>0.66</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Swarms</td>
<td>2</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Granitic rock</td>
<td>1</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

### 8.5.4 Deterministic Seismic Microzonation Map

Deterministic seismic microzonation map is the hazard index map for the worst scenario earthquake. Important factor of PGA (weight is 9) is estimated from synthetic ground motions, which are generated based on MCE of 5.1 in moment magnitude for the closest vulnerable source of Mandya-Channapatna-Bangalore lineament. Hazard index values are estimated based on normalized weights and ranks through the integration of all themes using the following equation:

$$DSM = \left( \frac{DPGA_w + DPGA_r + AF_w AF_r + ST_w ST_r + SS_w SS_r + FS_w FS_r + PF_w PF_r + EL_w EL_r + DR_w DR_r + GG_w GG_r}{\sum w} \right)$$  \hspace{1cm} (8.3)

Using estimated values deterministic seismic microzonation map has been generated using the
scheme as shown in Figure 8.27. Figure 8.28 shows the deterministic seismic microzonation map for Bangalore. Integrated GIS map shows that hazard index values vary from 0.10 to 0.66. These values are grouped into four categories <0.1, 0.10-0.15, 0.15-0.30, 0.3-0.45, 0.45-0.6 and 0.6 to 0.66. The maximum hazard is attached to the seismic hazard index greater than 0.6 at the western part of Bangalore. Eastern part of the city attached to a minimum hazard compared to other areas. Western and southern parts have mixed hazard and northern part has moderate hazard.

8.5.5 Probabilistic Seismic Microzonation Map

Similar to DSM hazard index calculation, probabilistic hazard index has been estimated, but PGA values are taken from the probabilistic seismic hazard analysis. PGA at 10% probability of exceedance in 50 years has been estimated considering six seismogenic sources and regional recurrence relations. Based on probabilistic hazard index values Probabilistic Seismic Microzonation map (PSM) has been generated. Probabilistic hazard index values are estimated based on normalized weights and ranks through the integration of all themes using the following equation:
Figure 8.28: Deterministic seismic microzonation map of Bangalore (after Sitharam and Anbazhagan, 2008; Anbazhagan et al., 2010; Nath and Thingbaijam, 2009).

Figure 8.29 shows the probabilistic seismic microzonation map based on the calculated hazard index. Probabilistic hazard index values vary from 0.10 to 0.66 and has been grouped to four categories such as <0.1, 0.10-0.15, 0.15-0.30, 0.3-0.45, 0.45-0.6 and 0.6-0.66. These values are lesser than the deterministic hazard index. The maximum hazard is attached to the seismic hazard index greater than 0.6 at the south-western part of Bangalore. Lower part (south) of Bangalore is identified as moderate to maximum hazard when compared to the northern part.
The microzonation study in the region has been formulated into two different aspects geomorphological and seismological. The former is derived from thematic layers comprising of surface geology, soil cover, slope, rock outcrop, and landslide hazard, which are integrated to achieve geological hazard distribution. The corresponding thematic layers have been developed by Nath (2004), Nath (2005) and Pal et al. (2008). The major datasets include IRS–1C LISS III digital data, toposheets from Survey of India, surface geological maps, soil taxonomy map based on composition, grain size and lithology from the National Bureau of Soil Survey and seismic refraction profiles. The percent slope mapping has been done with Triangulated Irregular Network (TIN) on GIS. Rock outcrop and landslide scarp region had been identified and vectorized into two separate polygon coverage. The latter highlights the relevant hazard conduced from seismic activities instead of geotechnical landslide hazard zonation. The seismological themes, namely surface consistent peak

Figure 8.29: Probabilistic seismic microzonation map of Bangalore (after Sitharam and Anbazhagan, 2008; Anbazhagan et al., 2010; Nath and Thingbaijam, 2009).

8.6 CASE STUDY FROM THE SIKKIM HIMALAYA

The microzonation study in the region has been formulated into two different aspects geomorphological and seismological. The former is derived from thematic layers comprising of surface geology, soil cover, slope, rock outcrop, and landslide hazard, which are integrated to achieve geological hazard distribution. The corresponding thematic layers have been developed by Nath (2004), Nath (2005) and Pal et al. (2008). The major datasets include IRS–1C LISS III digital data, toposheets from Survey of India, surface geological maps, soil taxonomy map based on composition, grain size and lithology from the National Bureau of Soil Survey and seismic refraction profiles. The percent slope mapping has been done with Triangulated Irregular Network (TIN) on GIS. Rock outcrop and landslide scarp region had been identified and vectorized into two separate polygon coverage. The latter highlights the relevant hazard conduced from seismic activities instead of geotechnical landslide hazard zonation. The seismological themes, namely surface consistent peak
ground acceleration and predominant frequency were, thereafter, integrated with the geological hazard distribution to obtain the seismic hazard microzonation map of the Sikkim Himalaya. Site response in the region is attributed mainly to different source radiation patterns, scattering, diffraction and undulating topographic effects. The study in the region by Nath et al. (2005) comprises of HVSR and GINV techniques based on 80 local earthquakes (3_ML_5.6) during 1998–2003 recorded by nine stations of IIT Kharagpur Sikkim Strong Motion Array (SSMA). The finite fault stochastic simulation has been used to deliver the surface consistent peak ground acceleration due to the maximum earthquake of Mw 8.3 projected to be nucleating from Main Boundary Thrust (MBT) trending NW-SE in the southern part of the region at a depth of 26.3 km with a fault plane solution providing 310° strike and 35°NNE dip (Nath et al., 2005; Pal et al., 2008).

The adopted simulation parameters are as: the designated epicenter at 27.25°N and 88.46°E, rupture dimensions of 250 km length and 80 km width, crustal shear wave velocity of 4.0 km/s, crustal density of 2.7 g/cm³, stress drop of 65 bars, Quality factor $Q = 167 f^{0.47}$ where $f$ is the frequency in Hz, and geometrical spreading factor is given by $1/R$ for $R<100$ km and $1/R^{0.5}$ for $R \geq 100$ km where $R$ is the hypocentral distance in km. The final integrated hazard map, as presented in Figure 8.30, exhibits five broad qualitative hazard classifications: “Low”, “Moderate”, “High”, “Moderately High” and “Very High”.

Figure 8.30: The seismic microzonation scheme for Sikkim Himalaya with the weights assigned to each theme labeled accordingly leading to the final hazard map. A representative accelerogram simulated at the Gangtok Strong Motion Station for the projected maximum earthquake of Mw 8.3 nucleating from a depth of 26.3 km on MBT is also depicted (after Nath, 2006; Pal et al., 2008; Nath and Thingbaijam, 2009).
8.7 MICROZONATION OF DELHI

The seismic microzonation maps were prepared for Delhi on 1:50,000 scales and this includes details regarding geology, seismotectonic details, ground water, bedrock depth, site response, liquefaction susceptibility, shear wave velocity and peak ground acceleration. The area was grouped into three hazard zones i.e., low, moderate and high. Apart from this there were lots of other works that were done for quantifying the seismic hazard of Delhi region. Iyengar and Ghosh (2004) carried out seismic hazard analysis of Delhi region based on deterministic and probabilistic methods and evaluated the peak horizontal acceleration values at rock level. More over they have evaluated site amplification and local site effects using the geotechnical borehole data and SHAKE91. In another study the bed rock level PGA maps for Delhi were developed by Rao and Satyam (2005) by considering five seismic sources in Delhi region. A Geotechnical site characterization was carried out based on borehole, geophysical data and $V_s^{30}$. Estimation of soil amplification factors were carried out using DEGTRA software and microzonation map for amplification factors was generated. The seismic response of the soil was estimated using the microtremor measurements at different locations in Delhi. Based on the shape of the resonance spectra and H/V amplitude, the predominant frequency and fundamental frequency map of the Delhi was prepared. Based on the SPT values obtained from the borehole data, the liquefaction potential of Delhi was also evaluated (Rao and Satyam, 2007).

8.8 SEISMIC MICROZONATION OF JABALPUR

The first microzonation work done in India was for the Jabalpur urban area. This work was carried out by the national agencies like Geological Survey of India (GSI), Central Region Nagpur, India Meteorology Department (IMD), New Delhi, National Geophysical Research Institute (NGRI), Hyderabad, Central Building Research Institute (CBRI), Roorkee and Government Engineering College, Jabalpur. Seismic

![Synthesized Hazard Map for Jabalpur](after Mishra, 2004).
hazard analysis was carried out based on DSHA and the peak ground acceleration maps were developed based on the attenuation relation developed by Joyner and Boore (1981). Using the information obtained from the geological and geotechnical studies, the first level microzonation map was prepared. The liquefaction hazard assessment was carried out using the geotechnical data based on the simplified approach proposed by Seed and Idriss (1971). The site classification of the study area was done based on average shear wave velocity at 30m depth ($V_{s30}$). The predominant frequency and peak amplification maps were also prepared for Jabalpur. The final hazard map prepared for Jabalpur is shown in Figure 8.31.

8.9 SEISMIC VULNERABILITY OF BUILDINGS

The primary emphasis in disaster management has been placed on the performance of structures, the failure of which would pose direct risk to life and property. Survey and assessment of existing building stocks for earthquake vulnerability risk is necessary to formulate the seismic hazard map of an urban center.

The seismic vulnerability study comprises mainly of a review of the existing buildings in the light of guidelines for earthquake resistant construction in India, construction practices being adopted in urban areas, building typologies, designing of questionnaire for detail survey of buildings of the region, selection of representative building samples for detailed analysis and NDT, and the creation of database. Subsequently, seismic vulnerability of existing building stock are estimated quantitatively and qualitatively. The quantitative approach covers demand-capacity computation, while qualitative procedure estimates structural scores using Rapid Screening Procedures (RSP). The results are mapped using ArcInfo and GIS, which are later synergized with seismic hazard microzonation to deliver seismic risk.

8.10 SEISMIC EVALUATION METHODOLOGY

Indian buildings built over past two decades are seismically deficient because of lack of awareness regarding seismic behavior of structure, constant upgradation of knowledge as regards to earthquake resistant design & construction. Also seismic design is not practiced in most of the buildings being built. It calls for seismic evaluation of existing building stocks in an area.

Evaluation is a complex process, which has to consider not only the design of building but also the deterioration of the material and damage caused to the building, if any. The difficulties faced in the seismic evaluation of a building are manifold. There is no reliable information/database available for existing building stock, construction practices, in-situ strength of material and components of the building. The seismic evaluation mainly relies on a set of general evaluation statements. The unavailability of a reliable estimate of earthquake parameters, to which the building is expected to be subjected during, its residual life poses another challenge. Probabilistic approach to evolve needful parameters, would call for elaborate studies. Hence, for preliminary appraisal, the ground motion parameters available in the present code (IS: 1893-2002) have been estimated at the macro level. As regards the effect of local soil conditions, which are known to greatly modify the earthquake ground motion, experiences of ground accentuation and data generated through collateral studies on site response have been considered. Also, in view of
above constraints, the present study is limited to seismic evaluation of representative buildings of different
typology viz. Type-A (Mud/RR Masonry, Adobe), Type-B (Brick Masonry Buildings), and Type-C (RCC
Buildings), and projects a generalized pattern of building response to future seismic ground motion in
different wards/zones of the urban area.

The approach for the assessment of seismic vulnerability of buildings involves estimation of seismic
vulnerability of existing building stock quantitatively and qualitatively. The quantitative approach covers
demand-capacity computation (ATC-40, 1996), while qualitative procedure estimates structural scores
based on national & international state-of-the-art procedures viz. Rapid Screening Procedure (ATC-21,
1988; ATC-21-1, 1988). The seismic evaluation leading to seismic vulnerability of existing building stock
at the study region is estimated quantitatively and qualitatively. The general procedures for seismic
evaluation of existing buildings adopted in the present study are: site visit & data collection; selection &
review of evaluation statements; follow-up fieldwork; and analysis of buildings by quantitative and qualitative
approach.

8.11 RISK ASSESSMENT

The exposures of the vulnerability components such as human population, buildings, etc to the seismic
hazard characterize seismic risk of a region. The seismic hazard is generally assumed to be stable over
a long geological time while the typical vulnerability (and, therefore, the risk) to the hazard changes
(McGuire, 2004). The risk is assessed as a convolution function of the hazard and the vulnerability i.e.,
Risk = Hazard* Vulnerability. The risk appraisals, aimed at promoting reasonable hazard mitigation
regulations, are generally based on vulnerability aspects such as landuse, demographic distributions,
building typology etc.

The computation of risk is fundamentally influenced by that of the hazard. Likewise, seismic risk
assessment could be deterministic or probabilistic. The former involves direct assessment of possible
losses based on the results of deterministic hazard analysis with no involvement of reference time
period but yielding to the current status. The assessment could, otherwise, follow either mean values or
take into account the uncertainties related to frequency of event occurrences (hazard) and damage
levels (vulnerability) yielding to a probabilistic account of the expected losses (Giacomo et al., 2005).
These approaches allow estimation of risk on a reference period of time. Another approach is to generate
the probable damage scenario by random simulations based on post earthquake damage studies (Barbat
et al., 1996). An example of deterministic assessment leading to first cut examination of the current risk
distribution for the Guwahati city based on landuse patterns using multi-criteria evaluation technique is
depicted in Figure 8.32.
Figure 8.32: A preliminary current risk map drawn from the thematic integration of seismic microzonation (hazard) and Landuse (vulnerability) maps (after Nath, 2007a).
Concluding Remarks

The seismic microzonation has emerged as an important issue in high risk urban centers across the globe. The compilation of data pertaining to geological, geophysical, geotechnical and seismological aspects comprises a major part of the venture, which necessitates a consortium of several public and private organizations engaged in diversified but related domains. The effort towards enhancing our understanding of seismic hazard and related effects is an on-going process, and therefore, the framework and tools for seismic microzonation studies presented in this Handbook needs to be continuously updated in the light of ongoing advancements as well as experiences gained during earthquakes. It is expected that seismic microzonation will enable updating building codes as well as formulate actions for hazard mitigation at subregional and local levels. Active programs related to infrastructural improvements and response planning can lead to the reduction of seismic risk.

Structural mitigation measures are the key to making a significant impact towards earthquake safety in our country. For successful earthquake mitigation, it has to be ensured that all new constructions in the seismic zones are complaint with the BIS Codes and for this purpose a techno legal regime has to be put in place. Though Bureau of Indian Standards (BIS) has laid down the national standards for construction in seismically vulnerable areas, these are not mandatory in nature. In many States, building bylaws are non-existent, and even in states where there are bylaws, which have considerations for seismic safety, the enforcement mechanisms leave a lot to be desired.
Appendix

Appendix I
Seismicity Analysis

Appendix II
Typical Geotechnical and Geophysical Investigations Prescribed for Seismic Microzonation Investigations for National Capital Territory (NCT), Delhi

Appendix III
A few Typical Case Studies on Site Effects

Appendix IV
A few Typical Case Studies on Ground Motion Synthesis and Seismic Hazard Scenario

Appendix V
Typical Case Studies in Site Characterization
I.1 A Unified Earthquake Catalogue for South Asia Covering the period 1900 - 2008
(Nath et al., 2011c)

Records of earthquake occurrences, in the form of a catalog, constitute an important database for seismic hazard studies. Events occurring prior to 1900 have been mostly reported on the basis of macroseismic intensity. These are generally categorized as ‘historical’ events. The ‘instrumental’ era started from 1900 onwards with gradual deployment of seismological observatories across the globe and since the advent of the World-Wide Standard Seismograph Network (WWSSN) during 1963-1964, data collection have surged ahead (Basham and Giardini, 1993; Johnston and Halchuk, 1993). This was accompanied by progresses in data-processing as well as in the theoretical understanding. However, most of the available global and local databases suffer from magnitude type inhomogeneity (i.e., usage of different magnitude types), and temporal heterogeneity in the data completeness.

Earliest works of earthquake cataloging in India and adjoining regions include that of Oldham (1883), Tandon and Srivastava (1974), Chandra (1977), Bapat et al. (1983), Rao and Rao (1984), and Srivastava and Ramachandran (1985). Quittmeyer and Jacob (1979) reported historical and instrumental earthquakes in Pakistan, Afghanistan, Northwest India, and Southeast Iran. Several workers, Lee et al. (1976), Gu (1983) amongst others, have cataloged earthquakes occurring in China. Recently, there has been several efforts to produce homogenous earthquake catalogs in India and adjoining regions: Ambraseys and Bilham (2003a) for Afghanistan, Jaiswal and Sinha (2004) for peninsular India, Thingbaijam et al. (2008) and Yadav et al. (2009) separately for northeast India, and Thingbaijam et al. (2009) for northwest frontier province of Indian-Eurasian plate convergence. In such studies, four major considerations are significant – (1) data sources, (2) data uncertainty, (3) uniform magnitude scaling by means of a single magnitude type, and (4) data completeness. Accordingly, rigorous examinations of these aspects are undertaken in the present study to compile a homogenous earthquake catalog for India and the adjoining regions. In case of instrumental records, entire south Asia is considered to impose a broader spatial scope in the analysis. The possible variations in the data attributes such as relations between different magnitude types, and temporal variation of data completeness across sub-regional tectonic provinces, are also investigated. Additionally, the records of historical events in the study region are independently compiled.

APPENDIX – I

Seismicity Analysis

I.1 A Unified Earthquake Catalogue for South Asia Covering the period 1900 - 2008 (Nath et al., 2011c)

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Regional context and data sources

Three major data sources, namely International Seismological Centre (ISC, http://www.isc.ac.uk, last accessed August 2009), US Geological Survey/National Earthquake Information Center (USGS/NEIC, http://neic.usgs.gov.us, last accessed April 2009), and Global Centroid Moment Tensor (GCMT, http://www.globalcmt.org, last accessed April 2009) are considered. Predominantly employed magnitude types for earthquake reporting are listed in Table I.1. As depicted in Figure I.1(a), the catalog from ISC is seen having higher data volume with longer temporal coverage corroborating the similar observation by Willemann and Storchak (2001). The total number of events in the region reported by ISC is ~7.4% of the total global events (Figure I.1b). The catalog from ISC is employed as primary data source owing to its higher data volume and other advantages such as magnitude error provided for the number of entries, multiple entries in different magnitude types, and multiple estimates for the same magnitude type. India Meteorological Department (IMD) reports regional and local earthquakes at the website: http://www.imd.ernet.in, since 2006. Jaiswal and Sinha (2004) prepared an earthquake catalog for south India based on Rao and Rao (1984), Seeber et al. (1999), and USGS/NEIC (Jaiswal and Sinha, 2007).

Table I.1: Different magnitude types that are predominantly employed for earthquake reportings (after McGuire, 2004). Approximate magnitude at which the magnitude type saturates is also listed.

<table>
<thead>
<tr>
<th>Magnitude type</th>
<th>Saturation</th>
<th>Reference/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local/Richter, $M_L$</td>
<td>6.8</td>
<td>Richter (1935)</td>
</tr>
<tr>
<td>Short period body-wave, $m_b$</td>
<td>7.0</td>
<td>Kanamori (1983)</td>
</tr>
<tr>
<td>Surface-wave, $M_S$</td>
<td>8.0</td>
<td>Gutenberg (1945a)</td>
</tr>
<tr>
<td>Duration, $M_D$#</td>
<td>-</td>
<td>Real and Teng (1973)</td>
</tr>
<tr>
<td>Vertical p-wave, $m_{pv}$</td>
<td>7.0$^\ddagger$</td>
<td>Hori (1969), Bune et al. (1973)</td>
</tr>
<tr>
<td>Moment, $M_w$</td>
<td>None</td>
<td>Hanks and Kanamori (1979)</td>
</tr>
<tr>
<td>Long-period body-wave, $m_B$</td>
<td>8.0</td>
<td>Gutenberg and Richter (1956)</td>
</tr>
<tr>
<td>$m_B$ based moment, $M_{W(mB)}$</td>
<td>8.3</td>
<td>Bormann and Saul (2008)</td>
</tr>
<tr>
<td>Lg wave, $M_n$ or $M_{bg}$</td>
<td>7.0$^\ddagger$</td>
<td>Nuttli (1973)</td>
</tr>
<tr>
<td>Vertical surface-wave, $M_{LV}$</td>
<td>8.0$^\ddagger$</td>
<td>Hori (1969)</td>
</tr>
</tbody>
</table>

#Duration magnitudes are used for small earthquakes; $^\ddagger$Based on characteristics of short period P-waves; $^\ddagger$comparable to $M_L$; $^\ddagger$vis-à-vis characteristics of surface waves.

Figure I.2 depicts spatial coverage of the datasets for the different magnitude types. The dataset for $M_w$ is derived from the GCMT database, and those for other magnitude types, except $M_{L,IMD}$ and $M_{W,JS}$, are
Figure I.1: (a) Annual reportings of earthquakes with magnitude (all types) $\geq 3.0$ in South Asia from three major global agencies: ISC, USGS/NEIC, and GCMT database. The inset depicts the temporal coverages. (b) The reported events by ISC is $\sim 7.4\%$ of the reported global events for magnitudes (all types) $\geq 3.0$ during 1999-2008.

Figure I.2: Spatial coverage of the derived datasets for the magnitude type indicated on each map.
derived from the ISC catalog. $M_W$, $m_b$, $M_S$, and $M_L$ are widespread, although $M_W$ and $M_S$ are scanty in the mid-plate regions and $M_L$ is negligible in the Northwest Carlsberg Ridge province. The $m_{pv}$ specified events are confined to northern parts of the study region in the Hindukush-Pamir province. Events reported in $M_D$ are seen across the northwest and central Himalayas, Andaman-Nicobar, and peninsular India. It is noted that the $M_L$ magnitude type in ISC are standard (i.e., scaled to Richter’s definition for California) within the organizational framework. The data from IMD are also considered with the magnitude type designated as $M_{L,IMD}$ to differentiate it from the local magnitudes given by ISC. The records from Jaiswal and Sinha (2004) with the magnitude type designated as $M_{W,JS}$ are seen clustered within south India.

**Figure I.3:** Comparison plots between the entries of the same magnitude types based on datasets derived for South Asia from the reportings of ISC, USGS and GCMT using a bisector (1:1), and a linear fit with intercept=0.

**Magnitude Errors**

The three data sources, namely ISC, USGS and GCMT, are compared to each other in context of present study region as depicted in Figure I.3. The catalogs from GCMT and USGS exhibit compatibility for all the three magnitude types (i.e., $M_S$, $M_W$, and $m_b$). On the other hand, USGS and ISC exhibit higher affinity in case of $M_S$ as compared to those for $m_b$ and $M_W$. Scordilis (2006) found the catalogs from USGS and ISC to be almost
equivalent 'in case of $M_b$ entries. The present observations indicate that this holds true for all the global catalogs. Magnitude (2009) reported that $M_b$ magnitudes are systematically lesser in ISC catalog as compared to those in USGS catalog. This has been attributed to minor differences between the magnitude-calibration terms used by the two agencies. In the present study, the average discrepancy between $M_b$ entries in the two catalogs is observed to be about 0.10 units. This is higher than that estimated on a global scale by Utsu (2002), who found the discrepancy to be about 0.05 units. On the other hand, the observed concurrence between $M_b$ reported by USGS and those by GCMT suggests higher uncertainty in the magnitude type reported by ISC.

![Figure I.4](image-url)

**Figure I.4:** (a-e) Top panel: Histograms of the errors (i.e., standard deviation, $\sigma$) found in ISC catalog associated with magnitude types. In case of $M_b$, histograms for shallow (focal depth, $H<70$ km) and deeper ($H \geq 70$ km) earthquakes are given in the inset. Bottom panel: Bootstrap dataset for the corresponding magnitude type.

The magnitude estimation improves with the number of recording stations, which is likely to be more for higher magnitude earthquakes. Kagan (2003) pointed out other factors such as focal depth, focal mechanism, and catalog period. However, the available data do not support development of error functions for magnitudes. Nonetheless, not only several error entries for $M_b$, $M_s$, $M_L$, and $M_d$ estimates are available in the ISC catalog, but also multiple magnitude type entries as well as multiple entries for same magnitude type. Figure I.4 depicts histograms of standard deviations. The average error for a particular magnitude type, as listed in Table I.2, is approximated as a maximum likelihood estimate from 1000 datasets generated by bootstrapping technique (Chernick, 1999). To estimate the error associated with magnitude types having no error entry, generalized orthogonal regression is used assuming a linear model between the variables to estimate ratio of error variance, $\eta (=\sigma_y^2/\sigma_x^2)$. The unknown error value is searched over a range of likely values such that slope has minimum standard deviation (e.g., Thingbaijam et al., 2009). The errors estimated with $M_s$ for shallower and deeper earthquakes are comparable. On the other hand, shallower earthquakes associate higher error as compared to deeper ones in case of $M_{pv}$. Kagan (2003) suggested that $M_b$ and $M_s$ in the global catalogs have errors of 0.20-0.25 units, while $M_{w}$ in the GCMT database has an average error of ~0.09 units. The present study reveals marginally higher uncertainties.
Table I.2: The magnitude type and the estimated average error; the data source is indicated by means of subscript notation

<table>
<thead>
<tr>
<th>Magnitude type</th>
<th>Average σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>mb,ISC</td>
<td>0.303</td>
</tr>
<tr>
<td>mb,ISC (for H &lt; 70 km)</td>
<td>0.304</td>
</tr>
<tr>
<td>mb,ISC (for H ≥ 70 km)</td>
<td>0.300</td>
</tr>
<tr>
<td>Ms</td>
<td>0.200</td>
</tr>
<tr>
<td>Ml</td>
<td>0.299</td>
</tr>
<tr>
<td>Md</td>
<td>0.380</td>
</tr>
<tr>
<td>MW,ISC</td>
<td>0.100</td>
</tr>
<tr>
<td>MW,GCMT</td>
<td>0.095&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>mb,GCMT</td>
<td>0.150&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>mpv (for H&lt;70 km)</td>
<td>0.400&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>mpv (for H&gt;70 km)</td>
<td>0.300&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>c</sup>Based on MW,ISC; <sup>a</sup>based on mb,ISC

Regression Analyses

The usage of a magnitude type in reporting is decided by several factors such as the data recorded, earthquake-source distance from the station deeming to be local, near-field or far-field etc. While MW, MS and mb are generally used for teleseismic events, ML and MD are used for local ones. Uniform magnitude scaling by means of a single magnitude type is necessary to establish data homogeneity for meaningful statistical analysis. MW is preferred owing to its applicability for all ranges of earthquakes; large or small, far or near, shallow or deep focused. The magnitude type does not suffer from saturation effect unlike other types. Furthermore, MW has been widely used in the recently developed ground motion prediction equations (e.g., Power et al., 2008).

Appropriate correlations between different magnitude types through regression analysis are envisaged to enable magnitude type conversions (e.g., Castellaro and Bormann, 2007; Bormann et al., 2007; Thingbaijam et al., 2008, 2009; Yadav et al., 2009). Castellaro et al. (2006) demonstrated that Generalized Orthogonal Regression (GOR) technique is more appropriate for earthquake magnitudes compared to the standard linear least square technique. However, in situations necessitating treatment of difference in magnitude errors, more complicated approach such as chi-square regression of Stromeyer et al. (2004) may be necessary. Nonetheless, GOR is employed in the present analysis owing to the use of instrumental dataset and lack of comprehensive error estimates. Moreover, the technique has been successfully employed in the earlier studies. The regression is based on error variance ratio (η = σ<sup>y</sup>²/σ<sup>x</sup>²) between the error variances of the variables on vertical and horizontal axis, respectively. The main objective is the functional interchangeability of the variables i.e., M<sub>y</sub> = M<sub>x</sub> + α and M<sub>x</sub> = M<sub>y</sub> - α. The analyses make use of the average errors computed from sample error distributions available on the applicable data subset or the estimated average value in case of non-availability of associated...
errors. Linear models of the type: \[ M_y = \beta M_x + \alpha \] are adopted. In case of the expected linear compatibility between the connecting magnitudes, linear fits with slope = 1 are also examined.

**Correlation between \( M_S \) and \( M_W \)**

Owing to its consistency for all the global catalogs, the data for \( M_S \) magnitude type from ISC and GCMT catalogs are combined in a single dataset for the regression analysis. The correlations between \( M_S \) and \( M_W \) in each sub-region, as presented in Figure I.5(a), do not differ statistically entailing an average relation to be coherent. Figure I.5(b) depicts the correlation,

\[
M_W = 0.700(\pm 0.018) M_S + 1.865(\pm 0.090), \quad n = 1824, \quad \eta = 0.226 \quad (I.1)
\]

On the global scale, Scordilis (2006) observed that the correlation between \( M_W \) and \( M_S \) magnitudes have a bi-linear trend differentiating lower and higher magnitudes such that both magnitude types are almost equivalent for \( M_W \geq 6.2 \). In the present study, while \( M_S \) corresponds to \( M_W \) at around \( M_W \) 6.2, the scanty data do not rule out systematic deviations for \( M_S > 7.5 \) and hence, \( M_W \) conversion on the basis of one-to-one connection is more practical for this magnitude range (e.g., Thingbaijam et al., 2008).

**Figure I.5:** (a) The results of the regression between \( M_W \) and \( M_S \) for each sub-region, (b) Correlation for the entire region, (c) Comparison between GOR (indicated by the square boxes) and regressions with slope = 1 between \( m_b \) and \( M_W \) for each of the sub-region, (d) \( M_W \) is seen with an average deviation of \( 0.159\pm0.249 \) units from \( m_b \) as depicted by the normal bold line.
Correlation between $m_b$ and $M_w$

High uncertainties associated with GOR fits, as depicted in Figure I.5(c), pose non-linearity behavior between $m_b$ and $M_w$ (Castellaro et al., 2006; Thingbaijam et al., 2008); however, the associated large uncertainties do not contradict an average deviation model. The two models are compatible for sub-regions - TP, MP and NCR, albeit very high uncertainties are associated with the GOR fits. Figure I.5(d) indicates that the average difference between $M_w$ and $m_b$ to be about 0.159 units as given below.

\[
M_w = 1.000(\pm0.049) \, m_b + 0.159(\pm0.249), \, m_b < 6.3, \, n = 1002 \tag{1.2}
\]

Correlation between $M_L$ and $M_w$

The compatibility of the two magnitude types is observed by Cassidy et al. (2005) for the continental regions. Heaton et al. (1986) also suggested $M_L$ and $M_w$ to be roughly equivalent. On the other hand, Ristau et al. (2005) showed that $M_w$ is higher than $M_L$ on an average of 0.6 units for offshore events owing to different crustal properties. A linear average deviation model has also been employed by Atkinson and McCartney (2005). In the present study, the available data pairs are not sufficient for correlation analysis. Nonetheless, several entries are found to have $M_L$ and $M_S$ as well as $M_L$ and $m_b$. In case of $M_L$-$M_S$, large scattering in the available data-pairs, as depicted in Figure I.6(a), indicates linear correlation to be infeasible.

Figure I.6: (a) $M_L$-$M_S$ data pairs exhibit high scattering, (b) Correlation between $M_L$ and $M_D$, (c) Correlations between $M_L$ and $M_D$, (d) Both GOR and linear fit with slope = 1 indicates $m_b$ to be higher than $M_D$, (e) Correlations between $m_b$ and $m_{pv}$ for focal depth, $H<70$ km, and (f) Correlations between $m_b$ and $m_{pv}$ for $H \geq 70$ km.
Correlation between $m_b$ and $M_L$

The correlations between $m_b$ and $M_L$ are examined with linear fits with slope $= 1$ for each sub-region with the data restricted between $M_L$ 2.0 and $M_L$ 6.5 considering scale saturation and high scattering outside the range. The results listed in Table I.3 indicate compatibility of the magnitude types while the associated uncertainties do not support statistically significant correlations for the sub-regions. The error entries in the dataset yields average uncertainty of 0.15 units and 0.29 units for $m_b$ and $M_L$, respectively. The obtained linear relations, as depicted in Figure I.6(b), are given as follows,

$$ m_b = 1.00(\pm 0.129) M_L + 0.007(\pm 0.446), \quad n = 3130 \tag{I.3} $$
$$ m_b = 1.069(\pm 0.034) M_L - 0.250(\pm 0.127), \quad n = 3130, \eta = 0.27 \tag{I.4} $$

Table I.3: The correlation of $m_b$ and $M_L$ such that $m_b = M_L + \alpha$, where $n$ is the number of data-pairs used for the regression

<table>
<thead>
<tr>
<th>Sub-region (Figure 1.4)</th>
<th>$\alpha$</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EI</td>
<td>0.020±0.411</td>
<td>20</td>
</tr>
<tr>
<td>NWIEC</td>
<td>0.223±0.431</td>
<td>404</td>
</tr>
<tr>
<td>CH</td>
<td>0.139±0.375</td>
<td>184</td>
</tr>
<tr>
<td>TP</td>
<td>0.144±0.399</td>
<td>479</td>
</tr>
<tr>
<td>NEIEC</td>
<td>0.036±0.430</td>
<td>266</td>
</tr>
<tr>
<td>AN</td>
<td>-0.106±0.426</td>
<td>1756</td>
</tr>
<tr>
<td>MP</td>
<td>0.633±0.896</td>
<td>18</td>
</tr>
<tr>
<td>NCR</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Combined</td>
<td>0.007±0.446</td>
<td>3130</td>
</tr>
</tbody>
</table>

Table I.4: The correlation between $M_L$ and $M_D$ such that $M_L = M_D + \alpha$, where $n$ denotes the number of data-pairs used for the regression

<table>
<thead>
<tr>
<th>Sub-region</th>
<th>$\alpha$</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EI</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>NWIEC</td>
<td>-0.053±0.367</td>
<td>88</td>
</tr>
<tr>
<td>CH</td>
<td>0.001±0.279</td>
<td>156</td>
</tr>
<tr>
<td>TP</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>NEIEC</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>AN</td>
<td>0.191±0.555</td>
<td>58</td>
</tr>
<tr>
<td>MP</td>
<td>-0.043±0.314</td>
<td>436</td>
</tr>
<tr>
<td>NCR</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Combined</td>
<td>-0.017±0.346</td>
<td>74</td>
</tr>
</tbody>
</table>

Correlation between $M_L$ and $M_D$

Table I.4 lists the results of the linear fits with slope $= 1$. Negligible intercepts are observed in all the sub-regions, except for the Andaman-Nicobar one. The associated standard deviations do not indicate any
statistically significant variations in the estimated values. The linear fits on the entire dataset, as depicted in Figure I.6(c), yields highly comparable models as given below,

\[
M_L = 1.000(\pm0.140)M_D - 0.017(\pm0.346), \quad n = 741 \quad (I.5)
\]

\[
M_L = 0.991(\pm0.011)M_D - 0.009(\pm0.031), \quad n = 741, \eta = 0.62 \quad (I.6)
\]

**Correlation between \(m_B\) and \(M_D\)**

\(M_D\) could not be correlated with \(M_W\) directly owing to insufficient data. However, \(m_B\) could be connected to \(M_D\). The plots given in Figure I.6(d) exhibits close-match between the average deviation model fit and that of GOR. The worked out relations are as follows,

\[
m_B = 1.00(\pm0.148)M_D + 0.189(\pm0.498), \quad n = 46 \quad (I.7)
\]

\[
m_B = 1.031(\pm0.070)M_D + 0.063(\pm0.294), \quad n = 46, \eta = 0.62 \quad (I.8)
\]

**Correlation between \(m_B\) and \(m_{pv}\)**

There are no events reported to have both \(m_{pv}\) and \(M_W\) magnitude entries. This necessitated indirect connectivity through \(m_B\). The regressions performed between \(m_B\) and \(m_{pv}\), depicted in Figures I.6(e-f), yield two distinct relationships for the shallow and deep-rooted earthquakes, respectively as follows.

\[
m_B = \begin{cases} 
0.782(\pm0.003)m_{vp} + 0.992(\pm0.011), & H < 70 \text{ km}, \ n = 948 \\
1.287(\pm0.019)m_{pv} - 1.890(\pm0.068), & H \geq 70 \text{ km}, \ n = 233
\end{cases} \quad (I.9)
\]

**Figure I.7:** (a) The \(M_L\) magnitudes from ISC and IMD are seen to be comparable, (b) The correlation between \(M_w(m_B)\) and \(m_B\) indicates that the two magnitude corresponds to each other, (c) The correlation between \(M_w(m_B)\) and \(m_B\).

**Other Magnitude types**

The magnitudes reported by IMD do not have error estimates; a conjectural average error of 0.3 units is assigned owing to the compatibility with those reported by ISC (Figure I.7a). Three magnitude types - intermediate-period/broadband body-wave magnitude \(m_B\) (Gutenberg and Richter, 1956), its offshoot moment magnitude \(M_w(m_B)\), vertical surface-wave magnitude \(M_{Lv}\), and 1-sec period \(L_g\) wave magnitude \(M_n\) (Nuttli, 1973), are found with some reported events with single magnitude type entries making it necessary to connect them to the
moment magnitude type. The sample errors found in the ISC catalog yields average error for $M_{W(mb)}$, $M_{LV}$ and $m_b$ to the tune of 0.38, 0.45 and 0.40 units, respectively. Although based on scanty data, $M_{W(mb)}$ seems to be comparable to $m_b$ with a minor deviation as depicted in Figure I.7(b). The relation is as given below,

$$m_b = 1.00 \pm 0.031 M_{W(mb)} - 0.022\pm 0.192, m_b \leq 6.2, n = 9 \ (I.10)$$

The consistency is also corroborated by GOR fit, which gives

$$m_b = 0.964 \pm 0.154 M_{W(mb)} + 0.21\pm 0.797, \eta = 0.623, n = 9 \ (I.11)$$

The correlation between $m_b$ and $M_{W(mb)}$ as depicted in Figure I.7(c) yields,

$$M_{W(mb)} = 1.113\pm 0.045 m_b - 0.898\pm 0.267, \eta = 0.902, n = 26 \ (I.12)$$

Equation I.12 agrees reasonably well with that of Bormann and Saul (2008) in the higher magnitude ranges ($M_W > 6.0$) while it deviates at lower magnitudes. The data-pairs associated with $M_{LV}$ are seen to be highly scattered, and hence, are not feasible for any correlation. Hori (1969) related $M_{LV}$ and $m_{pv}$ as follows,

$$M_{LV} = 1.13\pm 0.13 m_{pv} - 0.18\pm 0.06 \ (I.13)$$

In case of $M_n$, no correlating pairs are found with the available 106 entries. Patton (2001) suggested that $M_n$ and $M_L$ have one-to-one correspondences based on a dataset prepared from events associated with diverse sources. $M_n$ and $M_L$ are, therefore, considered to be practically equivalent. This also agrees with the $M_n - M_w$ relation (within the uncertainty bounds) given by Atkinson and Sonley (2005) for southeast Canada.

Data Compilation

To implement uniform magnitude scaling for the instrumental catalog, $M_w$ entries found in GCMT are retained. Magnitude entry from ISC catalog is selected maintaining a preference order of: $M_w$, $M_p$, $m_b$, $M_L$, $m_{pv}$, $M_{W(mb)}$, $M_D$, $m_b$, and $M_{LV}$. Equation I.1 is used to convert $M_S$ into $M_w$. Equation I.2 is used to convert $m_b$ to $M_w$. $M_L$ is converted to $M_W$ via Equation I.4. Similarly, $M_D$ is converted to $M_w$ using Equation I.8, and $m_{pv}$ converted to $M_w$ using Equation I.9. The entries given in $M_{W(mb)}$ are converted to $M_w$ using Equation I.10. In case of $m_b$, Equation I.12 is used to derive $M_{W(mb)}$ and thereafter, employed to obtain $M_w$ estimate. $M_{LV}$ is converted to $m_{pv}$ values, which is then connected to $M_w$. Following Thingbaijam et al. (2009), events having magnitude <4.0 with focal depth ~0.0 are excluded in order to minimize arbitrary content due to likely blasting activities. Furthermore, likely duplicate records are eliminated by searching the events occurring on the same date, hour and minute within spatial bound of 90 km, and retaining the one with largest magnitude.

In order to assimilate the data from IMD into the present compilation, each record is manually correlated according to the date, time and reported epicenter. Only 45 events do not have a clear match with the entries of ISC. The catalog prepared by Jaiswal and Sinha (2004) exhibits different magnitude coverages during the overlapped period with respect to the present compilation (Figure I.8a). A comparison with the present compilation depicted in Figure I.8(b) shows that $M_w$ in the present study is, on an average, 0.095 units smaller implicating an uncertainty of 0.46 units with $M_{W,JS}$. Accordingly, the correction factor of -0.095 and uncertainty of 0.46 units are adopted for the data source. Additionally, several reportings
are consulted to improve the data content. These include Pacheco and Sykes (1992), Chung and Gao (1995), Singh and Gupta (1980), Johnston (1993), Ambraseys (2000), Ambraseys and Bilham (2003a), Ambraseys and Douglas (2004), Mandal et al. (2004), Bilham et al. (2005), Wallace et al. (2005), Ulomov et al. (2006), Thingbaijam et al. (2008), and Amateur Seismic Centre (2009).

Accordingly, the following criteria are implemented for selecting the entries into the compilation:

1. The records derived using the dataset from ISC are employed as primary data.
2. Any entry found to match in the dataset from GCMT, the entry is replaced with the one obtained from it.
3. Entries in the dataset from GCMT not found in the compilation (after step 2) are directly adopted in the compilation.
4. If an entry in the catalog given by Jaiswal and Sinha (2004) does not have a match in the compilation (after step 3), it is accepted in the compilation.
5. The events reported by IMD without any clear match with entries of the compilation (after step 4) are entered in the compilation.
6. Entries in the present compilation are updated with respect to the published reporting of magnitude in $M_{W,JS}$ if available and are found having one-to-one correspondence event wise.
7. In case the reported event is not found in the compilation (after step 6), it is inserted into the compilation with appropriate magnitude scaling.

The present compilation is, thus, achieved with higher data volume compared to the original sources. Figure I.9 depicts the spatial distributions of the events.

**Seismicity Declustering**

The statistical modeling of main-shock earthquake occurrences is generally based on the independency of the events since the space-time correlation of seismicity is exhibited by foreshock and aftershock
clusters. Main-shock catalogs are, therefore, derived by eliminating the clusters. Windowing algorithms are generally used for the purpose. These are based on the space-time-magnitude considerations on each event and mostly differ in the selection of spatiotemporal window parameters (e.g., Gardner and Knopoff, 1974; Knopoff et al., 1982; Reasenberg, 1985; Uhrhammer, 1986). However, deciding optimal parameters is difficult in the light of diverse seismotectonic conditions (Baiesi and Paczuski, 2005; Gomberg et al., 2003).

Figure I.9: A seismicity map prepared using the compiled catalog.

Knopoff (2000) presented a straightforward approach - windowing technique for smaller earthquakes ($M_w < 6.4$) while for the larger earthquakes, aftershocks are identified by searching till the end of the catalog using Omori's decay rate law after demarcating the aftershock zone based on the aftermath seismicity. The law is given as below,

$$\lambda_a = k (t + c)^{-p}$$

(1.14)

Where $\lambda_a$ is the average rate of earthquakes for time $t$ following the main shock event, $k$ and $c$ are model coefficients (Utsu et al., 1995; Omori, 1894). In the present study, a similar technique is adopted since (1) there is higher likelihood of aftershocks of larger main shock event being recorded in the catalog compared to those for the smaller ones, and (2) the spatial spans of aftershocks, especially for those associated with larger earthquakes, are dynamic depending not only on the magnitude of the event but also on the geological background.
Table I.5: Windowing parameters employed for seismicity declustering (modified after Knopoff et al., 1982; Knopoff, 2000; Jaiswal and Sinha, 2007)

<table>
<thead>
<tr>
<th>Magnitude ($M_w$)</th>
<th>Distance bin (km)</th>
<th>Time bin (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.5 - 6.5</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>4.5 - 5.5</td>
<td>34</td>
<td>90</td>
</tr>
<tr>
<td>3.5 - 4.5</td>
<td>12</td>
<td>35</td>
</tr>
<tr>
<td>&lt; 3.5</td>
<td>4</td>
<td>12</td>
</tr>
</tbody>
</table>

The parameters listed in Table I.5 are adopted in the declustering for main-shock events with magnitudes $M_w < 6.5$. For events with higher magnitude ($M_w > 6.4$), the aftershock zone is identified by inspecting continuous spatial windows of $0.25^\circ \times 0.25^\circ$ for the presence of at least one event within 12 days of main-shock occurrence. Once the zones are demarcated, the events found within the zone from the advent till the end of the catalog are examined with cumulative number of events against time. Nyffenegger and Frohlich (2000) observed that the aftershock sequences for intermediate as well as deep earthquakes do not behave differently from those of the shallower ones. The algorithm, therefore, remains the same for deeper (hypocentral depth $\geq 70$ km) earthquakes, except for the maximum decay time that is restricted to one year. In case of small number of events ($<10$), the events are directly marked as aftershocks if found within 300 days.

The p-value is estimated by nonlinear fit of the integral form of Equation I.14 with $c = 0$. The time series is employed with a cumulative number of events rather than discrete distribution of aftershock counts per unit time in accordance with Ogata (1983). Instead of posterior assignment of p-value=1, the fit is evaluated by restricting the associated standard deviations within 10% of the estimated coefficient in order to accommodate observations of different p-values in the region (e.g., Mandal et al., 2006; Rao et al., 2006; Mishra et al., 2007). The termination of the aftershock sequences are decided accordingly. The analysis has uncertainties due to errors associated with epicentral locations, time and magnitudes. In the processing, the epicenters are grouped within a distance bound and consequently the errors associated are significantly reduced and so is with the case of time bins while the magnitude-wise correlation between the events is done with the assigned magnitudes. The present analysis is restricted to the identification of the most likely aftershocks, and henceforth errors in the magnitudes are not given additional treatment. The same approach, but on the other way round with the examination of events preceding the main shock, is used for the detection of likely foreshocks. The time window is bounded to one year for events with magnitudes: $M_w 6.5 – 7.4$ as well as for the deeper earthquakes (focal depth $\geq 70$ km), while the window is restricted to a maximum of three years for events with magnitudes $M_w > 7.4$. The former time window criterion concurs to that of Papadopoulos et al. (2000).

The plots of aftershock sequences for a few major earthquakes are depicted in Figure I.10. The p-values estimated for the aftershock sequences vary from 0.50 to 1.02, and are not significantly different from that given by previous studies (Mandal et al., 2006; Rao et al., 2006; Mishra et al., 2007; Ramana et al., 2009). The aftershocks of 2004 $M_w 9.1$ Sumatra and 2005 $M_w 7.6$ Kashmir earthquakes continue to the end of the catalog. Overall, the declustering algorithm identified 39.05% of the total events as likely aftershock or foreshocks in the original catalog.
Hypocentral Depth Corrections

The hypocentral depth entries in the compiled catalog (only main-shock events considered) have either been fixed (i.e., uncertainty not provided) or assigned standard deviation values. In order to reduce the associated uncertainties, Engdahl-van der Hilst-Buland (EHB) catalog from International Seismological Centre (2009) is consulted. The EHB catalog was prepared using data from International Seismological Summary, ISC and Preliminary Determination of Epicenters of USGS/NEIC; hypocentral depth recomputed using algorithm given by Engdahl et al. (1998). The EHB catalog spans 1960 – 2006 and do not recalculate the magnitudes. The epicentral location and the depth entries along with the uncertainty (standard deviation) in the present compilation are updated on the basis of event-to-event comparison with the entries in the EHB catalog. Additionally, records given by Stork et al. (2008) are selectively used to update entries not covered by the EHB catalog.

The spatial distribution of earthquakes at different hypocentral depth ranges is depicted in Figure I.11. Figure I.12(a) depicts the distributions of the hypocentral depth uncertainty (δ depth) for main-shock events with $M_w \geq 4.0$. Lowered average standard deviation suggests significant improvement. A histogram of hypocentral depth for the events is depicted in Figure I.12(b). Data are observed to be clustered at four different depth-ranges (in km): 0 – 25, 25 – 70, 70 –180 and 180 – 300. The data confidence levels are evaluated as percentage of data within the depth-range as given by the ratio between the total entries found to be within the specified range considering the uncertainty and that without considering the uncertainty. Mean absolute deviation for Gaussian distribution is considered, which is equal to about 0.8 times the standard deviation. The inset in Figure I.12(b) depicts the histogram for the depth ranges. The confidence levels are found to be 87, 85, 86 and 89 percent for the depth bins, respectively. These define the limitations of the data employed in the present analysis, and are significantly close to the acceptable 90% confidence bounds; as such the degree of uncertainty associated with the earthquake data is generally high (e.g., Wiemer and Wyss, 2000; Thingbaijam et al., 2008).
Figure I.11: A seismicity map of India and the adjoining regions with the main-shock events with magnitude $M_W \geq 4.0$ during the period 1900 – 2008 categorized according to the hypocentral depths.

Figure I.12: (a) Distribution of hypocentral depth uncertainty ($\sigma$ depth) for the main-shock events with $M_W 4.0$. (b) Hypocentral depth distribution for the main-shock events binned at 5 km spacing. The inset depicts histograms for the different depth ranges. Inscribed are % entries found within the range taking into account the associated data uncertainty.
Data Completeness

A seismicity map prepared using the derived main-shock catalog is depicted in Figure I.13. Earthquake catalogs are generally characterized by spatiotemporal heterogeneity in the data completeness mostly due to non-availability of relevant accounts or records as well as irregularities in the spatial and temporal coverage of monitoring networks. Quantitative assessment of data completeness is essential in order to facilitate unbiased assessment of background seismicity rates with the data segregated based on its completeness (e.g., Thingbaijam and Nath, 2008). A commonly applied statistical method considers time-independent seismogenic process to be stable (e.g., Stepp, 1972; Mulargia and Tinti, 1985). This postulates stability in the reportings to be a factor for data completeness, notwithstanding that possibility exists that the entire data-set may not be complete at all (Albarello et al., 2001). The fundamental hypothesis is that the main shock events follow Poissonian behavior, and consequently, the data completeness test is essentially applicable for the declustered catalogs.

Figure I.13: Declustered seismicity of South Asia covering a period: 1900 – 2008 comprising of 30364 events.

The present analysis is preformed for broad sub-regional level assessment of the temporal variation of data completeness on the sub-catalogs using the method employed by Mulargia and Tinti (1985). Minimum magnitudes at which stable seismicity is observed (referred to here as ‘threshold magnitude’) are inferred from the plot of the cumulative number of events against time, which runs to the end of the catalog. Non-cumulative plots are also used for corroborative evaluations. Figure I.14 depicts the data completeness plots for each of the sub-regions. The period 1900–2008 in the individual sub-regions exhibit threshold magnitudes in the range of $M_W$ 5.5–6.4 while the entire period shows threshold magnitudes between $M_W$ 4.5 and $M_W$ 5.3. The data completeness is observed to have improved in all the cases during the last two decades.
Records for historical events, i.e. those occurring prior to 1900, in the study region are rather scanty. The National Geophysical Data Center database (NGDC, http://www.ngdc.noaa.gov) lists historical earthquakes but the database suffers from data inconsistencies (Dunbar et al., 1992). Martin and Szeliga (2010) delivered a catalog of macroseismic intensity data for events occurring as early as 1636 in India and adjoining regions. Szeliga et al. (2010) found the data to be complete only for those with magnitude >8.0 since 1800. They also noted that higher uncertainty is associated with magnitude and location of the earthquakes estimated using macroseismic intensity owing to sensitivity of methodology and involvement of subjective interpretations. Furthermore, historical reportings are dependent on several factors such as documentation procedure, population and built-up environment prevailing during the period. Nonetheless, they comprise of useful data for the long-term seismicity modeling (e.g., Kijko and Sellevoll, 1989; Stucchi et al., 2004). A compilation of reported historical events is, hence, performed by consulting

**Figure I.15:** The historical earthquakes are depicted with approximate location and magnitude ($M_w \geq 5.0$), respectively; dashed circle indicate unknown attributes but otherwise reported event.

Several historical earthquakes do not have sufficient information (e.g., Bapat et al. 1983; Szeliga et al. 2010). In the northeast India, several historical earthquakes have been reported although their estimate of magnitudes and location are unknown (Rajendran et al., 2004). Likewise, Heidarzadeh et al. (2008) reported historical earthquakes in the Makran province with unknown magnitude during: 326 BC, 1008, 1483, 1668, 1765, and 1851. In case of NGDC, reported event is adopted only if information about the associated damage/deaths is also listed along with the magnitude. As far as date of publication is concerned, Lee et al. (1976) is rather old but the data-source has been referred by Bapat et al. (1983) and Pun and Ambraseys (2007) citing the magnitudes to be $M_s$ equivalent. Ambraseys and Bilham (2003a) estimated $M_s$ values for historical earthquakes in Afghanistan based on perceived intensity of damage and thereafter, employed an empirical bilinear relation to compute seismic moment and consequently $M_w$ estimates. Figure I.15 depicts the approximate location and magnitude ($M_w \geq 5.0$) of known historical
earthquakes in the study region; oldest events that are significant and well-established in the Indian context date back to roughly 500 – 600 years only.

Discussion

Relations between Magnitude types

Source spectral properties across different tectonic provinces entail different waveform properties that are source dependent. This aspect has been accommodated in the relations between different magnitude types by the uncertainties defined in the model parameters. The correlation between $M_w$ and $M_s$ is seen to be consistent with the previous studies. That between $M_L$ and $M_D$ by Thingbaijam et al. (2009) was based on the constraint of minimum error with the linear slope and is affected by sparse data on the higher magnitude range $M_D>4.0$. The relation has been, consequently, revised in the present study. Magnitude types: $m_b$, $M_D$, and $M_L$ are seen to have linear correspondence to each other. This agrees to some extent with the observations of Sitaram and Borah (2007) in northeast India. Considering the linear compatibility between $M_w$ and $m_b$ with a difference of 0.159 units, $M_L$ is seen to be deviating about 0.6 – 0.8 units from $M_w$. The latter observation reasonably agrees with that of Atkinson and McCartney (2005). On the other hand, higher difference (of about 0.46 units) has been observed between $M_w$ and $m_b$ by Braunmiller and Nábelek (2002) for the explorer region.

Magnitude uncertainties and Data completeness

The magnitude error distribution for the main shock events compiled in the present study indicates uncertainty to the order of 0.4 units to be predominant, owing to the bulk of $m_b$ entries. The mean error is estimated to be about 0.38 units implying that general practice of using 0.30 units for instrumental events to be inadequate. On the other hand, the data completeness is seen considerably improved since the last two decades conforming to the ongoing advancements in the earthquake data collection, processing and reportings.

Historical Earthquakes

Major shortcomings with the historical earthquake reporting include: (1) possible inhomogeneity in magnitude scaling owing to lack of calibrations amongst different studies (data sources), and (2) high data incompleteness and uncertainties warranting appropriate treatment in the applications, thereof.

Note: The Homogenous Earthquake Catalogue of South Asia in $M_w$ scale can be accessed at [http://earthqaz.net/sacat](http://earthqaz.net/sacat/) for the downloadable files for the catalogue as: Early instrumental and instrumental period (1900-2008) [txt format, 3.6 MB], Pre-instrumental period (Prior to 1900) [txt format, 18.0 KB] & readme file [pdf format, 20.0 KB].

I.2  A Seismogenic Source Framework for the Indian Subcontinent

(Thingbaijam and Nath, 2011)

Seismogenic source modeling in the Indian subcontinent has come a long way from seismic intensity based analysis to deterministic source zonations and seismicity parameterizations. As late as 1950s,
Tandon (1956) employed concept of spatiotemporal earthquake occurrences linked with tectonics for seismic hazard zonation. Similarly, Guha (1962) considered seismic intensity in liaison with geology and tectonics for the same purpose. This was further explored by Gubin (1968) to define 16 seismogenic zones considering the historical earthquakes as well. Kaila et al. (1972) performed seismicity analysis accounting data for the period of 1954-1967 to prepare maps of seismicity parameters namely, \( a\)-value, \( b\)-value and return periods for earthquakes with magnitude \( \geq 6.0 \). One of the earliest probabilistic seismic hazard analysis efforts of the country is that of Khattri et al. (1984). They employed 24 broad seismogenic source zones identified on the basis of historical seismicity, seismotectonics and geology wherein the seismicity parameters were estimated. However, statistical approaches have been somewhat limited in view of scanty data available during the period. Bhatia et al. (1999), under the GSHAP framework, delivered considerable improvement on the seismogenic source zonation. They demarcated a total of 86 seismogenic source zones in the broad region on the basis of major tectonic features and seismicity trends. The employed earthquake data were derived from National Oceanic and Atmospheric Administration database and local catalogs after subjecting to processing tasks such as removing duplicates, aftershocks and earthquakes without any magnitude. They used maximum likelihood method to estimate \( a\)-value and \( b\)-value while the maximum earthquake for each zone was construed from the past seismicity. They observed that the sparse seismicity data, especially in the Indian shield region, necessitated merging some zones as a single unit for the assessment of \( b\)-value. On the other hand, Parvez et al. (2003) defined 40 seismogenic zones for the Indian subcontinent on the basis of seismicity, tectonics and geodynamics. At the same time, they pointed out that the seismogenic zones considered by Khattri et al. (1984) suffer from ambiguity due to likely existence of inhomogeneity within each zone owing to rather broad spatial bounds. Furthermore, they found that the seismic source zones demarcated by Bhatia et al. (1999) lack parity with the observed events in few cases. Nonetheless, the previous studies including Parvez et al. (2003) have a common shortcoming - lack of comprehensive seismotectonic detailing. Comparatively more detailed assessment is performed by Gupta (2006). The author defined a total of 81 seismogenic source zones according to correlation of the seismicity with tectonic features. More recently, Jaiswal and Sinha (2007) applied zoneless approach, i.e. seismicity smoothening, to compute spatial distribution of seismic activity rates in the peninsular India. They also employed broad zones defined on the basis of large-scale geological features proposed by Seeber et al. (1999) to establish maximum possible earthquake. Incidentally, seismicity analysis at subregional levels across the study region can be found in several recent publications; viz. northeast India (Thingbaijam et al., 2008; Thingbaijam and Nath, 2008), northwest frontier province (Thingbaijam et al., 2009), and the Himalayas (Shanker and Sharma, 1998), among others. As the data quality improves with the deployment of seismological observatories, the impetus is drawn to the data quality decided by data homogeneity and completeness for meaningful statistical analysis; the homogeneity being entailed by the usage of single magnitude type, preferably moment magnitude \( M_w \). Likewise, most of the previous studies employ data from multiple sources and thus posed heterogeneous compilations. This aspect calls for the consideration of data uncertainty \textit{vis-à-vis} the different data sources (Nath et al., 2011c). Furthermore, the previous studies constitute substantial tectonical simplifications overlooking likely disparity of tectonics at different seismogenic depths.
Seismological Data

Three kinds of data are considered for the present study, namely earthquake catalog, fault database, and geodetic data in terms of fault-slip rates. The earthquake chronology in the form of a catalog is indispensable for seismicity modeling. At the same time, the seismicity of a region is mostly rendered by the underlying active fault network while information related to fault-slip rates allows predictions of earthquake recurrences and magnitudes.

Earthquake Catalog

Nath et al. (2011c) compiled an earthquake catalog of South Asia for 1900-2008 period in a homogenous $M_w$ magnitude framework. The considered data sources include International Seismological Center, Global Centroid Moment Tensor database, and several other publications. The uniform magnitude scaling in $M_w$ is achieved through connecting relationships between the different magnitude types. These relationships are derived by regression analysis on the available data-pairs. It is seen that on average, $m_b$ is lower than $M_w$ by 0.16 units while $M_L$ is lower than $M_w$ by 0.6-0.8 units depending on the magnitude range. The compiled instrumental catalogue is declustered to remove foreshocks and aftershocks to derive a mainshock catalogue. A windowing algorithm is employed for the lower magnitude range ($M_w<6.5$). For higher magnitudes, declustering is done on the zone of the aftermath seismicity and subsequent assessment of the temporal coverage of events occurring within the zone based on the Omori's decay rate law. Aftershock sequences of major earthquakes prior to 2008 are observed to have terminated, except those for 2004 Sumatra $M_w$ 9.1, and 2005 Kashmir $M_w$ 7.6 earthquakes. The temporal variations of the data completeness for the time-period 1900-2008 evaluated on the basis of stable number of events for the magnitude range indicate three verges: first one spanning the entire period, the second one from 1964 onwards and the third one with considerable improvement in the reportings during the last two decades.

Fault Database, Focal Mechanisms, and Slip Rates

The fault database is compiled on Geographical Information System (ArcGIS 9.1) platform. The sources include seismotectonic map of India published by Geological Survey of India (Dasgupta et al., 2000), seismotectonics map of Afghanistan and adjacent areas published by U.S. Geological Survey (Wheeler and Rukstales, 2007) and that given by Wellman (1966), the tectonic framework of Andaman-Sumatra belt given by Curray (1991), and the fault map of Tibet compiled by He and Tsukuda (2003).

The focal mechanism data employed in the present study are primarily derived from GCMT Global Centroid Moment Tensor (GCMT, www.globalcmt.org, last access April 2009), which covers a period from 1976 to 2008. Those published by several researchers are also consulted, which include Ben-Menahem et al. (1974), Singh et al. (1975a), Banghar (1976), Chen et al. (1976), Chandra (1977), Singh and Gupta (1980), Byrne et al. (1992), Rastogi (1992), Chung (1993), Chung and Gao (1995), Chen and Molnar (1990), Ramesh and Estabrook (1998), Dasgupta et al. (2000), and Bilham and England (2001).

A seismotectonic map of the study region is depicted in Figure I.16. The crustal deformation rates across the study region are discussed in Annexure B.
Figure I.16: A seismotectonic map of India and the adjoining regions prepared with fault patterns adapted from Wellman (1966), Curay (1991), Dasgupta et al. (2000), and He and Tsukuda (2003). The depicted earthquake rupture areas were adapted from Bilham (pers. comm.), Bendick et al. (2007), Ortiz and Bilham (2003), Byrne et al. (1992), and Cummins (2004) except those enclosed with dashed lines, which are conjectured. Those in lighter shades have higher uncertainty. Epicenters of events occurring post 1900 are depicted with stars. The beach balls are adapted from different reporting (cited in the text) (after Thingbaijam and Nath, 2011).

Seismotectonic Context

The seismotectonics regimes across the study region are significantly diverse and have been discussed in details by Kayal (2008), Balasubrahmanyan (2006), Gupta (2006), Dasgupta et al. (2000), Chandra (1978) and amongst others. The Indian plate boundary encompasses transverse fault system on the northwest, Himalayan arc on the north, and complex underthrusting and subduction along Indo-Myanmar arc, Hindukush-Pamir zone, and Andaman-Nicobar-Sumatra tracts. The Himalayan tracts run E-W and comprise of several north dipping thrust zones with interspersed topographic discontinuities. The major
features include Main Frontal Thrust (MFT), Main Boundary Thrust (MBT), Main Central Thrust (MCT), and Indus Tsangpo Suture Zone (ITSZ) located south to north. Most seismicity is concentrated on the shallow north dipping planes that settle on a gently dipping decollement surface belong to the underthrusting Indian plate (Seeber et al., 1981; Pandey et al., 1995; Bilham et al., 1998; Lave and Avouac, 2000). The shear zone of decollement surface runs at shallower depths under the lesser and higher Himalayas and at higher depths below the Tethys Himalayas.

Nevertheless, earthquakes also occur in the Indian plate within the lower crust zones exhibiting compression and strike-slip (Bilham, 2004). These seismogenic tracts have ruptured several times causing large and damaging earthquakes (Dasgupta et al., 2000; Singh and Gupta, 1980; Chen and Molnar, 1977). The major events include 1505 Lo Mustang Mw 8.6-8.8, 1720 Uttar Pradesh Mw ~7.5 (± 0.4), 1803 Kumaon Mw 7.5 (± 0.2), 1833 Kathmandu Mw 7.6 (±0.2), 1905 Kangra Mw 7.8 (±0.2), 1916 Dharchula Mw 7.1 (0.3), 1934 Bihar-Nepal Mw 8.1 (±0.2), 1945 Chamba Mw 6.3 (±0.3), 1975 Kinnaur Mw 6.8 (±0.2), 1986 Dharamsala Mw 5.5 (±0.1), 1988 Udayapur Mw 6.8 (± 0.1), 1991 Uttarkashi Mw 6.8 (± 0.1), and 1999 Chamoli Mw 6.6 (± 0.1) (Singh et al., 1975a and 1975b; Singh and Gupta, 1980; Pacheco and Sykes, 1992; Bilham, 1995; Wallace et al., 2005; Ambraseys and Douglas, 2004; Bilham and Ambroseys, 2005; Ghimire and Kasahara, 2007).

The underlying seismicity in Tibetan plateau is attributed to the active collision of the India-Eurasia tectonic plates. Episodes of large earthquakes with magnitude >7.0 have been reported as early as 0180 AD in the terrain (Lee et al., 1976). Schulte-Pelkum et al. (2005) observed that strong seismic anisotropy exists above the decollement surface while suggesting that the deep seismicity in Tibet can be attributed to Eclogitization and loss of the denser material into the mantle. In the northeast India, the seismic activities are accredited to the juxtaposition of E–W trending Himalaya and N–S trending Arakan Yoma belts. The former is an extension of the entire span of the collision between the India and the Eurasia plates while the latter is ascribed by the underthrusting of the Indian plate below the Myanmar plate. The region is associated with several major events such as 1869 Cachar Mw 7.4 (±0.2), 1897 Shillong Mw 8.1(±0.1), 1906 north Myanmar Mw 6.8 (±0.2), 1908 north Myanmar Mw 7.4 (±0.2), 1918 Srimangal Mw 7.4 (±0.3), 1930 Dhubri 7.1 (±0.4), 1931 north Myanmar Mw 7.6(±0.4), 1943 Assam Mw 7.2(±0.2), 1947 Subansiri Mw 7.7(±0.4), 1950 Assam Mw 8.6(±0.2), and 1988 Manipur Mw 7.2(±0.1) (Bilham and England, 2001; Rajendran et al., 2004; Ambraseys and Douglas, 2004; Thingbaijam et al., 2008; Thingbaijam and Nath, 2008; Nath et al., 2011c; Molnar and Deng, 1984). The tectonics of Shillong plateau is characterized by a protruding mechanism associated with the 1897 Great Shillong earthquake (Chandra, 1978; Bilham and England, 2001). The convergence of the Himalaya and Arakan Yoma belts is the Assam syntaxis/Mishmi zone, where the seismicity has been attributed to inter-plate wedge motions (Ben-Menahem et al., 1974; Nandy, 2001). The Indo-Myanmar arc on the eastern plate convergence is believed to be associated with an oblique subduction. A major earthquake reportedly occurred during 1762 rupturing ~700 km along the Arakan coast (Cummins, 2004). The tectonic grains along the arc include Indo-Myanmar ranges to the north, and Andaman-Nicobar ridges to the south. These features are delineated by Shan-Sagaing fault on the east, Western Andaman fault, and Sumatran fault system on the southeast (Dasgupta et al., 2000 and 2003). The Shan-Sagaing fault has been associated with several major
earthquakes - 1908 $M_w$ 7.3 ($\pm$0.2), 1929 $M_w$ 7.0 ($\pm$0.3), 1930 $M_w$ 7.2 ($\pm$0.2), 1931 $M_w$ 7.6 ($\pm$0.4), and 1945 $M_w$ 7.4 ($\pm$0.3) (Molnar and Deng, 1984).

The plate convergent belt along the Himalayas extends southwards through Myanmar to the Andaman and Nicobar Islands and the Sumatra, where the Indian plate is believed to be underthrusting along the Sumatra fault system (Fitch, 1972; Curray et al., 1979; McCaffrey, 2009). The seismicity in the terrain has been observed to follow down-dip extension of subduction interface, characterized by extensive faulting, and numerous large shallow and intermediate earthquakes of different mechanisms (Kumar et al., 1996; Ortiz and Bilham, 2003; Dasgupta et al., 2003; Kayal et al., 2004). This region in the southeastern boundary of Indian plate has been visited by destructive earthquakes in the past; 1847 Nicobar $M_w$ 7.5-7.9, 1881 Car Nicobar $M_w$ 7.9 ($\pm$0.1), 1941 Andaman $M_w$ 7.7 ($\pm$0.1), 2002 Diglipur $M_w$ 6.5 ($\pm$0.1), 2004 Sumatra $M_w$ 9.1 ($\pm$0.1), and 2005 Sumatra-Andaman $M_w$ 8.6 (Rajendran et al., 2004; Kayal et al., 2004; Bilham et al., 2005). The stress orientation along Andaman-Sumatra trench is decidedly of active subduction dominated by shallow thrust events attributed to regional compressional stress.

The northwest frontier province of the India-Eurasia plate convergence comprises of the Himalayan tectonic features, namely the MBT and MCT and is bordered by the Hazara Syntaxis and the Hindukush-Pamir region to the northwest. The Hindukush-Pamir region experiences high seismicity constituting shallow to intermediate-depth earthquakes. The western part of the Himalayan arc, consisting of the Kashmir ranges, turns to the south near Nanga Parbat at the Hazara syntaxis (Meltzer et al., 2001). This region is highly seismogenic with occurrences of numerous major earthquakes, to name a few, 1555 Srinagar $M_w \geq$7.6, 1878 Abbottabad $M_w$ 6.8 ($\pm$0.3), 1885 Kashmir $M_w$ 6.3 ($\pm$0.3), 1974 Pattan $M_w$ 6.2 ($\pm$0.2), and 2005 Kashmir $M_w$ 7.6 ($\pm$0.1) (Ambraseys and Douglas, 2004; Bendick et al., 2007). The prominent tectonic features along the plate boundary region include the Kirthar-Sulaiman ranges, and the transverse fault systems of, the Chaman and Ornach Nal faults. The Chaman fault zone has been associated with two major earthquakes; namely 1931 Mach $M_w$ 7.3 ($\pm$0.3), and 1935 Quetta $M_w$ 7.7 (0.3) (Ambraseys and Bilham, 2003a; Ambraseys and Douglas, 2004; Szeliga et al., 2009). The Arabian plate to the south of Pakistan apparently is subducted northwards beneath the Eurasian plate forming the E-W trending Makran subduction zone (Byrne et al., 1992). The recent episodes of major to great earthquakes in the eastern parts of the zone include 1945 $M_w$ 8.1 ($\pm$0.2), 1947 $M_w$ 7.1 ($\pm$0.2), and 1983 $M_w$ 6.7 ($\pm$0.1).

Most parts of the peninsular India, including the Indian Shield region, are characterized by diffused seismicity. However, localized seismicity associated with several rift zones and shear/thrust zones can be observed in the region. There have been episodes of several major earthquakes attributed to reactivation of pre-existing fault and fault-failure due to the compression building in the regime. These include 1819 Kutch $M_w$ 7.7 ($\pm$0.2), 1900 Coimbatore $M_w$ 5.7 ($\pm$0.3), 1927 Son Valley $M_w$ 6.4 ($\pm$0.2), 1938 Satpura $M_w$ 6.2($0.2), 1956 Anjar $M_w$ 6.0 ($\pm$0.1), 1967 Ongole $M_w$ 5.1 ($\pm$0.1), 1969 Bhadrachalam $M_w$ 5.7 ($\pm$0.1), 1970 Broach $M_w$ 5.4 ($\pm$0.1), 1984 Bangalore $M_w$ 4.5 ($\pm$0.3), 1988 Idukki $M_w$ $\sim$4.5 ($0.1), 1993 Latur $M_w$ 6.2 ($\pm$0.1), 1997 Jabalpur $M_w$ 5.8 ($\pm$0.1), and 2001 Bhuj $M_w$ 7.7 ($\pm$0.1) (Chandra, 1977; Singh et al., 1975a; Rastogi, 1992; Rajendran et al., 1996; Chung, 1993; Chung and Gao, 1995; Bilham, 1999; Bendick et al., 2001). The Narmada-Son lineament is a prominent tectonic grain of the Indian shield trending.
ENE-WSW, and has been associated with major earthquakes in the past, viz. 1997 Jabalpur and 1927 Son Valley. The Eastern Ghat region shows diffused seismicity with shallow focus earthquakes. Earthquakes have also nucleated off the coast in the Bay of Bengal with focal plane solutions indicating left-lateral strike-slip motions. On the other hand, the western Ghat region shows a prominent seismicity cluster in the Koyna-Warna region, which has been associated with reservoir triggered seismicity (Gupta et al., 1997). However, existence of rift valley and vertical crust block movements in the region has also been suggested (Brahmam and Negi, 1973; Kailasam et al., 1976). At the same time, high seismicity in the region has been also attributed to the geometry of the fault zones and their interactions through stress transfer (Gahalaut et al., 2004). The region has been visited by moderate-to-strong earthquakes, viz. 1967 Koyna $M_w$ 6.3 ($\pm$0.2), 1993 Koyna $M_w$ 5.2 ($\pm$0.2), and 2000 Koyna $M_w$ 5.0 ($\pm$0.1). The western end of the Narmada-Son lineament also exhibits considerable seismicity. The Latur region in central India experienced an earthquake of $M_w$ 6.2 ($\pm$0.1) in 1993. Thrust type faulting has been indicated with this event. The Godavari Graben region experienced 1969 Bhadrachalam $M_w$ 5.7 earthquake; otherwise, has relatively lower level of seismicity. The southern most parts of India shows shallow strike-slip earthquakes predominantly. The earthquakes in Bengal Basin exhibit strike-slip faulting types indicating NNE-SSW compressive regime (Kayal, 2008). The 1964 Sagar Island $M_w$ 5.4 earthquake is associated with Eocene Hinge, Bengal Basin (Nath et al., 2010). Major earthquakes have been noted in region during 1822, 1842, 1846, and 1885 (Szeliga et al., 2010). Except for 1885 event ($M_w$ ~5.6-6.2), the other events have magnitude derived from macroseismic intensity data in the range $M_w$ ~6.4-7.3. Most seismicity in Indo-Gangetic plains are observed in the zone of the Delhi-Aravalli belt. The western margin of the Indian plate exhibits a neo-tectonic compressive stress regime against the collision front to the north. This seismogenic province of Kutch/Gujarat has been visited by several moderate to large earthquakes namely 2001 Bhuj, 1956 Anjar and 1819 Kutch. The seismicity has been attributed to reactivation of rift related extensional structures under compressive stresses and exhibits reverse faulting predominantly. Overall, the seismic productivity observed in several parts of the peninsular India suggests intra-plate deformation to be significant.

**General Methodologies**

**Time independent Seismicity Model**

Earthquake occurrences across the globe is universally accepted to follow Gutenberg and Richter (GR, 1944) relationship,

$$\log_{10} \lambda (m) = a - bm$$

where $\lambda (m)$ is the cumulative number of earthquakes with magnitude $\geq m$. The parameters $a$- and $b$-value relates to the background seismicity level and the magnitude size distribution, respectively. The $b$-value is often employed as an indicator of stress disparity (Scholz, 1968; Schorlemmer et al., 2005) due to subtle tectonic reinforcement (Mogi, 1967), and hence can be utilized to define seismotectonic fabric of the seismogenic provinces (Thingbaijam et al., 2008; Gulia and Wiemer, 2010). Also, it has been observed to be relatively stable in comparison to the $a$-value (Tinti and Mulargia, 1985; Qin et al., 1999).
A maximum likelihood method for estimating $b$-value given by Aki (1965) and later modified by Bender (1983) is as follows,

$$
b = \frac{\log_{10} (e)}{m_{\text{mean}} - m_t - \frac{m}{2}}$$  \hspace{1cm} (I.16)

where $m_{\text{mean}}$ is the average magnitude, $m_t$ is the minimum magnitude of completeness, and $\Delta m$ is the magnitude bin size ($= 0.1$ in the present study). The standard deviation of $b$-value $\delta b$ can be computed by means of a bootstrapping approach suggested by Schorlemmer et al. (2003) which involves the repeated computation each time employing different replacement events drawn from the catalog. A minimum magnitude constraint is generally applied on the GR relation given by Equation (I.15) on the basis of the magnitude of completeness entailed by the linearity of the GR relation on the lower magnitude range. Apparently, data incompleteness is attributed to instrument insensitivity to lower magnitudes. At the same time, an upper magnitude has been also suggested in accordance with physical dissipation of energy and constraints due to the tectonic framework (e.g., Kagan, 2002; Kijko, 2004; Thingbaijam and Nath, 2008). This is achieved by establishing the maximum earthquake $m_{\text{max}}$, physically capable of occurring within a defined seismic regime in an underlying tectonic setup. The magnitude distribution is, therefore, truncated at $m_{\text{max}}$ such that $m_{\text{max}} \gg m_{\text{min}}$. A modified version of Equation (I.15) is a truncated exponential distribution incorporating an upper bound as follows,

$$
\lambda(m) = \lambda(m_{\text{min}}) \frac{10^{-b(m-m_{\text{min}})} - 10^{-b(m_{\text{max}}-m_{\text{min}})}}{1 - 10^{-b(m_{\text{max}}-m_{\text{min}})}}$$  \hspace{1cm} (I.17)

This equation was formulated by Cornell and Van Marke (1969) and can be found in Berril and Davis (1980), Coppersmith (1991), and Reiter (1990). Berril and Davis (1980) observed that the expression satisfies the maximum entropy considerations imposed by the $m_{\text{max}}$ constraint. Different truncation forms have also been proposed by several researchers (e.g., Main, 1996; Utsu, 1999; Kagan, 2002). Fault geodetic measurement often supports characteristic earthquakes near the maximum earthquake to have lower recurrence periods than the one predicted by GR relation. In such cases, a delta function or a Gaussian spike in the region of the $m_{\text{max}}$ is used to formulate characteristic recurrence models (e.g., Schwartz and Coppersmith, 1984; Youngs and Coppersmith, 1985).

Commonly employed approaches for the estimation of $m_{\text{max}}$ are either based on seismicity data, fault dimension, geodetic measures or a hybrid one. An earthquake catalog based approaches include the maximum likelihood method based on seismicity models, extreme value distribution, and linear extrapolation on the GR trend (e.g., Kijko 2004; Nath et al., 2005). Probabilistic extrapolation with the truncation of the frequency-magnitude curve at specific value of annual probability of exceedance is also used (Nuttli, 1981). Alternatively, fault based approach involves estimation of $m_{\text{max}}$ based on maximum fault-rupture dimensions using the relations between magnitude and fault rupture dimensions (e.g., Wells and Coppersmith, 1994; Hanks and Bakun, 2002). The procedure purposed by Ward (1997) considers the fault-rupture constrained by the strength and configuration of the faults. Fault slip rates are used
often to assess $m_{max}$ as well as constrain the recurrence periods (e.g., Youngs and Coppersmith, 1985; McGuire, 2004; Pace et al., 2006). In the present study, three approaches are considered according to the data feasibility: (1) published assessments in study region discussed in Annexure B, (2) catalog based maximum likelihood method, and (3) probabilistic extrapolation for annual probability of exceedance of 0.001.

**Seismogenic Source Zonation**

A popular approach in the seismogenic localization process is areal source zonation, wherein the objective is to capture uniform seismicity. However, given the temporal span of seismicity records and varying completeness thereof, the uniformity of the long-term seismicity within a classified zone cannot be fully confirmed or dismissed. A viewpoint given by Woo (1996) is that the earthquake magnitudes have spatial organization in accordance with the fractal law, which the magnitude-independent areal zonation fails to address. He explains that $b$-value for the demarcated zone should be same as those of sub-zones existing within the zone, and the source geometry should pertain to magnitude distribution. This does not necessarily conflict with the regional-tectonic specific $b$-value, which has been often employed in seismic hazard analysis, e.g. Frankel et al. (2002), Jaiswal and Sinha (2007), Petersen et al. (2008) among others. An implication is drawn from the fact that the $b$-value conforms to the prevalent fault kinematics within the demarcated zone and vice versa. Incidentally, the maximum earthquake $m_{max}$ has also been entailed as region specific constrained by the tectonic framework.

In the real situations, it is usually difficult to establish definite tectonic classification for a given seismogenic zone. In most of the cases, observed faulting mechanisms are non-uniform necessitating provisions to account for the epistemic (or aleatory?) uncertainties. While seismic source zonation becomes a case of tectonic dismantling, the standard zoning procedure is prone to possibly biased hazard estimates (Mucciarelli et al., 2008). The segregation of uniform tectonic regimes leading to reduced seismogenic zone dimensions with sparse earthquake occurrence would obscure the seismicity parameterization. In that respect, seismicity smoothening or zone-free approach has been viewed as a pragmatic one, which accounts for tectonic uncertainties while adhering to the observed spatial distribution of earthquake occurrences. The approach also complies with the fact that the locations of future large earthquakes tend to follow the spatial distribution of the past seismicity (e.g., Kafka, 2007; Parsons, 2008). The zone-free methodology has been in vogue since the works of Vere-Jones (1992), Kagan and Jackson (1994), and Frankel (1995). Several researchers have come up with different procedural treatments (e.g., Woo, 1996; Cao et al., 1996; Jackson and Kagan, 1999; Frankel et al., 2002; Lapajne et al., 2003). Mucciarelli et al. (2008) observed that the standard areal zonation approach provides hazard estimates relatively lower than zone-free approach. On the other hand, Beauval et al. (2006) asserts that in high seismic activity regions, the smoothing approach yields systematically lower estimates than the zoning method. In that respect, recent most studies have combined the techniques – say seismicity smoothing for small-moderate earthquakes and fault specific zonation for larger earthquakes (e.g., Petersen et al., 2008; Kalkan et al., 2009). Alternately, unified approach can be formulated that accounts for regional seismicity models based on area zonation associated with larger earthquakes for assessment
of $b$-value and $m_{\text{max}}$ while seismicity smoothing is used to establish activity rate in order to accommodate absence of fault associability. This delineates the grid cells according to regions of homogenous seismotectonic characteristics. Eventually, the methodology adopted in the present study can be outlined into three aspects: (1) delineation of areal source zones on the basis of seismicity distribution and fault patterns complemented by available focal mechanism data, (2) formulation of seismicity model and associated uncertainty values for each source zone, and (3) application of seismicity smoothening algorithm to obtain activity rates for specific threshold magnitude/s.

**Data Treatment for the Magnitude Uncertainties**

In the present study, the magnitude uncertainties are accounted for by mean of corrections suggested by Felzer (2008). These include magnitude rounding to a single decimal place and the magnitude error, which, if not accounted for, can lead to incorrect estimation of the seismicity rate (Tinti and Mulargia, 1985; Rhoades, 1996; Felzer, 2008). Most magnitudes in the instrumental records are generally rounded to the nearest 0.1, while considerable rounding of the decimal point as high as 0.5 can be found with the historical earthquakes prior to 1900. The technique for the corrections of magnitude rounding involves generation of new magnitudes corresponding to the catalog ones based on a randomly generated correcting factor based on the $b$-value. Computations are done with new sets of magnitudes generated 500 times and average of the results is considered. At the same time, incorrect assessment of the seismicity rate can also occur due to the magnitude errors owing to the symmetric nature of the Gaussian magnitude error being imposed on the asymmetric distribution of the magnitudes. A simple tool for the magnitude error suggested by Felzer (2008) is given as,

$$\Delta m = \frac{b^2 \sigma^2}{2 \log_{10}(e)} \quad (1.18)$$

where $\sigma$ is the standard deviation of the magnitude error, $b$ refers to $b$-value, and $\Delta m$ is a correcting factor required to be subtracted from each magnitude.

**Seismogenic Source Models**

*Layered Seismogenic Zones*

The seismicity patterns and seismic source dynamics has been observed to have significant variations with depth (e.g., Prozorov and Dziewonski, 1982; Christova 1992; Tsapanos, 2000; Allen et al., 2004) that assuming a single set of seismicity parameters over the entire depth range can produce incorrect hazard estimations. Accordingly, four hypocentral depth ranges are considered: upper crust (0-25 km), lower-crust (25-70 km), lower crust (70-180 km), and deep-seated (beyond 180 km). The source zonation at each layer is carried out by considering the seismicity patterns, fault networks and similarity in the style of focal mechanisms (e.g., Cáceres et al., 2005). The assessment yields a total of 172 seismic source zones as depicted in Figure I.17. The layered seismogenic source model in the present study is expected to facilitate resolving the source characteristics more precisely than single layer schemes that has been considered hitherto in the study region.
In order to establish the seismotectonic description at each layer, representative focal mechanism tensor (i.e., $F$) is considered by calculating the weighted average of the known moment tensors as follows,

$$ F_{ij} = \frac{\sum_{n=1}^{N} M^n_0 F^n_{ij}}{\sum_{n=1}^{N} M^n_0} \quad (1.19) $$

where $N$ is the total number of the focal mechanisms; $M^n_0$ is the scalar moment of the $n$th focal mechanism, and $F^n_{ij}$ is a function of the strike, dip, and rake of this focal mechanism (Aki and Richards, 1980). The average focal mechanism thus obtained for each zone is depicted in Figure I.17. These enable us to characterize each zone (wherever data is available) in terms of focal mechanism and their most likely rupture mechanism of the future earthquakes, especially large magnitude ones (e.g., Meletti et al., 2008).

Figure I.17: A layered seismogenic source model for India and adjoining regions considering hypocentral depth ranges: (a) 0-25 km, (b) 25-70 km, (c) 70-180 km, and (d) 180-300 km (after Thingbaijam and Nath, 2011).
**Seismicity Parameters**

Two aspects of the seismicity parameters are looked into: correlation between the parameters, and tectonic affinity of the $b$-value distribution. The $b$-value is estimated by means of Equation (I.16). The $a$-value and consequently the annual activity rate are based on Equations (I.15) and (I.19). In several cases, zones with similar tectonics are merged to overcome lack of sufficient number of events i.e., $\geq 50$ while achieving acceptable uncertainty with the estimated $b$-values eventually producing 103 zones out of the 172 zones. The assessment of $m_{\text{max}}$ across the zones and the results for the zone-wise analyses are given in Annexure B. Figure I.18 depicts a summary of the seismicity parameters by means of cross plots. It is seen that $a$-value and $b$-value have strong positive correlation while $b$-value have weak negative correlation with the maximum earthquake $m_{\text{max}}$ and the observed maximum earthquake $m_{\text{max,obs}}$. These observations, however do not imply casual relationships. On the other hand, the correlation between $a$-value and $m_{\text{max}}$ or $m_{\text{max,obs}}$ is insignificant. The annual rate activity for mean magnitude, considered $M_{W}$ 5.0 for the ranges of the magnitudes, exhibits no correlation with the $b$-value. Figure I.19 (a-e) depicts the mean $b$-value calculated from the estimated values for each zone grouped according to the underlying predominant fault type. The classification of fault type has been done according to the rake, as depicted in Figure I.19(f). In the zones with predominantly reverse faulting type, $b$-value ranges between 0.72-0.96 with lowest mean value of 0.82($\pm$0.07) while mean $b$-value of 1.05($\pm$0.11) is seen with the strike-slip regimes, where it ranges from 0.90 to 1.28. The regimes with predominantly normal faulting have $b$-values in the range 1.28-1.57 and associate the highest mean $b$-value of 1.37($\pm$0.10). The zones with assorted reverse and strike-slip faulting and those with mixed normal with strike-slip faulting types, respectively have fuzzy b-value ranges exhibiting affinity towards that of strike-slip faulting type. These observations suggest entailment of the $b$-value with the underlying tectonics while existence of assorted faulting types in a zone can generate fuzzy association between $b$-value vis-à-vis the seismotectonic classes.

![Figure I.18](image)

**Figure I.18:** The correlations between the zone-specific seismicity parameters indicated by Pearson’s linear correlation coefficient $r$, such that $r \rightarrow -1$ (perfect negative correlation) to $r \rightarrow +1$ (perfect positive correlation) and $r \rightarrow 0$ for no relationship (after Thingbaijam and Nath, 2011).
Construction of Seismicity Grids

The seismicity smoothening allows modeling of discrete earthquake distributions into spatially continuous probability distributions. The fundamental gridded seismicity smoothening technique of Frankel (1995) is adopted. The approach has been previously employed by several workers across the globe; to name a few, Stirling et al. (2002) in New Zealand, Frankel et al. (2002) for nation-wide seismic hazard mapping in United States, Akinci et al. (2004) to generate hazard maps for Northern and Central Italy, Pelaez Montilla et al. (2003) in Northern Algeria, Lapajne et al. (2003) in Slovenia, and Jaiswal and Sinha (2007) in Peninsular India.

In the present analysis, the study region is gridded at a regular interval of 0.1°; each grid point encompassing a cell of 0.1° x 0.1°. The seismic activity rate is computed as follows,

$$N(m_r) = \frac{\sum n_j(m_r) e^{-(\psi/c)^j}}{\sum e^{-(\psi/c)^j}}$$

Where $n_j(m_r)$ is the number of events with magnitude $\geq m_r$, $\Delta_j$ is the distance between $i^{th}$ and $j^{th}$ cells, and $c$ denotes the correlation distance. The annual activity rate $\lambda m_r$ is computed each time as $N(m_r)/T$ where $T$ is the (sub) catalog period. Different models are generally envisaged according to sub-catalogs pertaining to different data completeness to accommodate the pertinent model uncertainties. The threshold magnitudes of $M_w$ 4.0, $M_w$ 4.5, and $M_w$ 5.5 are employed. Description of the sub-catalogs pertaining to each seismogenic depth sections alongwith the time-frames for the threshold magnitudes is given in Table I.6. The temporal coverages conform to the earlier assessments of Nath et al. (2011c), Thingbaijam et al. (2008 and 2009), and Thingbaijam and Nath (2008). The correlation distances of 55 km, 65 km, and
85 km are decided for the respective cases after calibrating output of several runs of the smoothing algorithm with the observed seismicity. The corrections for the magnitude uncertainty are applied as well. Figure I.20 depicts the models obtained at each seismogenic layer. Significant disparity of seismic activity is observed across the study region suggesting spatial clustering to be evident with the seismicity. At same time, remarkable conformity to the underlying tectonics is observed. The activity rates are observed to be higher in the plate boundary regions of Hindukush-Pamir, Himalayas, Indo-Myanmar arc, and Andaman-Sumatra zone. In the intra-plate terrains, Kutch province and Koyna-Warna zone exhibits higher activity rates. While the activity rates at the intermediate hypocentral depth range are high along the Indo-Myanmar arc and Andaman-Sumatra tracts, the Hindukush-Pamir tracts shows exceptionally high activity rates. The seismicity at hypocentral depths >180 km are mostly confined along these three seismogenic provinces.

Table 1.6: The sub-catalogs for the three different threshold magnitudes considered for the construction of seismicity grids

<table>
<thead>
<tr>
<th>Depth-range (km)</th>
<th>Sub-catalog</th>
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<tr>
<td></td>
<td>M&lt;sub&gt;w&lt;/sub&gt; 4.0</td>
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Figure I.20: Smoothened seismicity models computed at the different hypocentral depth ranges in terms of activity rate for 100 years considering different sub-catalogs with threshold magnitudes of $M_w$ 4.0, $M_w$ 4.5, and $M_w$ 5.5 respectively (after Thingbaijam and Nath, 2011).

Discussion

The aspects of seismogenic source zones are looked into considering hypocentral depth distributions – a perspective not attempted before in the study region. Similar approach can be found in Stirling et al. (2002), and Suckale and Grünthal (2009). Previous subregional groundwork studies, e.g., Thingbaijam et al. (2008 and 2009), and Thingbaijam and Nath (2008) have been revisited here to deliver meliorated results.

Hypocentral Depth Classification

The earthquakes observed in the present study pertain to shallow (< 70 km) and intermediate (70-300 km) hypocentral depths as far as generally accepted classification is concerned (e.g., Thingbaijam et
However, followed nomenclature in the context of tectonic settings allocates the hypocentral depth distribution as pertinent to ‘upper crust’, ‘lower crust/interface’, ‘intraslab’, and ‘deep-seated zones’. The first three categories roughly agree with the empirically established depth sections: 0-25 km, 25-70 km, and 70-300 km, respectively in the present study; the intraslab depth-range further demarcated into two stratified regimes of different source potential: 70-180 km and 180-300 km, respectively. The conformity of these depth classifications to the underlying seismotectonic provinces across the study region is examined as follows.

Earthquakes in the upper crust at the depths up to 22-25 km suffer less attenuation in the seismic wave propagation and have larger spreading. In fact, owing to the higher significance from the seismic hazard point of view, the earthquakes for tectonically active upper crust have been addressed exclusively in the Next Generation Attenuation models (Power et al., 2008). Most earthquakes in the intraplate regions occur at focal depth range of 10–15 km; however, deeper (>25 km) have also been reported (Johnston, 1996; Klose and Seeber, 2007). The upper crust classification addresses the shallow seismogenic zones of higher significance to the seismic hazard in most of the cases. Maggi et al. (2000) interpreted the depth variation of seismic activity in terms of strength of continental lithosphere with the effective elastic thickness, which approximately corresponds to the seismogenic thickness. The crustal thickness across the peninsular India ranges 35-45 km whereas the thickness in the Himalayas are observed to be around 75-80 km (Mahadevan, 1994; Reddy et al., 1999). In the regions of lesser and higher Himalayas, the seismogenic upper crust is bounded by a decollement plane formed by the Indian plate that dips gently northwards at depths of around 20-25 km (Seeber et al., 1981). Across the Tibetan plateau, Karakoram, and Hindukush-Pamir regions, the crustal thicknesses are found to be in the range from 63-72 km (Holt and Wallace, 1990). In the intraplate region, earthquakes occurring in the lower crust (hypocentral depth > 25 km) are significantly rare compared to those occurring in the upper crust. Chen and Molnar (1983) suggested that the continental crust comprises seismically active upper layer and aseismic lower layer over a brittle uppermost mantle. On the other hand, earthquakes in the subduction zones are generally categorized as ‘interface’ or ‘intraslab’ (Youngs et al., 1997). The interface earthquakes are generally observed to occur within 50 km depth (Ticheler and Ruff, 1993) and this corresponds to the depth range 25–70 km in the present study. The planes beneath the Himalayas extend to Tibetan plateau where the lower crust is known to be seismically active with occurrences of interface earthquakes (e.g., Maggi et al., 2000; Bilham, 2004). In fact, the depth of interface earthquakes of moderate to large earthquakes in the subduction zones has been observed to be mostly in the range of 35-70 km (Pacheco et al., 1993). These observations support segregation of lower crust as different seismic source layer, which extend to a maximum depth close to 45 km and 70 km in the intraplate and interplate regions, respectively. Intraslab earthquakes occur at higher depths >70 km, and in the present study, they are concentrated in Andaman-Sumatra, Indo-Myanmar, and Hindukush-Pamir subduction zones. The seismotectonic patterns in these regions have been a subject of extensive research (e.g., Dasgupta et al., 2003; Satyabala, 2003; Stork et al., 2008; Pegler and Das, 1998; Liu, 1999). Present depth-wise delineations in the Indo-Myanmar arc and Andaman-Sumatra region reasonably agree with the observations of Stork et al. (2008), and Dasgupta et al. (2003). The subduction zone up to about 180 km depth has been noted to be more
discernible in the Indo-Myanmar arc (Dasgupta et al., 2003). Along the Sumatra-Myanmar arc, the upper crust exhibits lower seismic activity compared to the deeper subduction zones representing transitions of stratified regions of seismic potential (e.g., Byrne et al., 1988). Likewise, the classification of hypocentral depth distribution of Pegler and Das (1998), and Liu (1999) in the Hindukush-Pamir subduction zone is similar to the one in the present study. Eventually, the empirically established depth-ranges in the present study are seen to be reasonably consistent not only with the different seismotectonic regimes but also with previous region specific studies.

**Regional Seismotectonics and Seismicity Parameters**

**The Himalayas and Southern Tibetan Plateau:** Low angled thrusting events are predominant in the upper crust zones of the Himalayas, which is typical of the collision tectonics. Exceptional is the eastern Himalayas, where strike-slip motions are observed. The apparent transverse motions have been implicated to the deformations of Indian plate to the south (Drukpa et al., 2006). Across the Himalayan arc, lower b-values are observed ranging 0.73–0.94, which corresponds to higher seismic stress in the region. The northwestern regions extending from Kangra to the Hazara syntaxis exhibits high seismic activity and is associated with recurrence period of ~500 years for $M_w > 8.0$ according to the seismicity model. The recurrence period of ~555 years for major earthquakes given by Malik and Nakata (2003) concurs with great earthquakes, while projection by Malik et al. (2010) of ~1160 years recurrence period approximately agrees with magnitude $M_w \geq 8.2$. Bendick et al. (2007), and Kondo et al. (2008) placed the recurrence period for events with magnitudes similar to Kashmir earthquakes to 680(±150) years, and ~2000 years respectively. The seismicity model based on the areal seismogenic zone in the present study places the recurrence period of similar-sized event to be around 200(±60) years. Fault specific assessment is expected to predict higher recurrence period than areal source one if the associated seismic activity is high. The observed seismic activity rate in the upper crust zones of Kangra seismic province is high and the trend connects to the Hazara syntaxis along the MBT tracts. The zone of western Nepal (Garhwal Himalaya) is another region of high seismic activity along the Himalayan arc and is observed with a recurrence period of ~1000 years for $M_w \geq 8.0$ events, which corresponds to the paleoseismicity estimated by Malik and Nakata (2003). However, paleoseismic events (with $M_w > 7.0$) identified by Kumar et al. (2001) yield lower recurrence period. At the same time, higher uncertainty for the upper magnitude range has been implicated by Kumar et al. (2006). The slip rate of 17 mm/yr in the region yields recurrence rates closer to the GR model compared to the TGR model. The former model predicts a recurrence period of ~3600 years for the maximum earthquake. For the recurrence period, the latter model connects to magnitude of $M_w \sim 8.5$. Furthermore, mega earthquakes in the upper crust zone would be susceptible to surface rupturing; occurrences of such events in the terrain have been asserted by Wesnousky et al. (1999). In the east Nepal (central Himalayas), about 100 years recurrence time for $M_w 7.0$ events and ~700 years for the Bihar-Nepal earthquake conforming to uncertainty bounds for large/great earthquakes in the region based on the palaeoseismic events identified by Sukhija et al. (2002) is observed. The recurrence period of mega events $M_w > 8.5$ is seen in order of 2000 years higher than period of 1000-1500 years suggested by Lave et al. (2005). In the eastern Himalaya, b-value is found to be 0.94 (±0.11). The minimum recurrence period for great earthquakes ($M_w \geq 8.0$) in the terrain is observed to be in order
of ~2500 years. To the eastern end where feature of MBT and Mishmi thrust interacts, the significant seismic activity is seen in the upper crust zone and recurrence period of $M_w \geq 7.6$ events is observed to be in order of 500 years while great earthquakes (Rajendran et al., 2004) have minimum recurrence period of ~900 years. The events in the Kinnanur province exhibit strike slip and normal faulting type events; $b$-value ~1.0 is observed in the region. The recurrence period for the largest known event in the zone $M_w \sim 6.1$ Kinnaur earthquake is observed to be about 60-100 years.

The events occurring in the lower-crust zones along the central and eastern Himalayas exhibit strike-slip elements unlike those in the northwest Himalayas. Along the Himalayas, the lower crust zones exhibit lower $b$-values in the range of 0.78-0.96. In the east central Himalayas, the hypocenter of the 1505 Himalayan earthquake $M_w$ 8.6-8.8 is conjectured to be located at lower crust depth range i.e., >25 km owing to the associated rupture width and fault dip. The seismicity models give a recurrence period of ~2500–9000 years (TGR) for the prehistoric earthquake. Lower recurrence period of 80-100 years for major earthquakes $M_w \geq 7.0$ is observed in the central Himalaya while it is as low as ~550 years and as high as ~2000 years in the northwest Himalayas. The major earthquakes in Po-Chu fault zone is seen to have exceedingly high recurrence running into several millennia. The earthquakes occurring in the zones of depths >70 km in the terrain are scanty and insignificant from the hazard point of view. They are characterized by low magnitudes and high return periods. The sub-mantle events along the eastern Himalayas exhibit strike-slip mechanisms with normal faulting components and normal faulting. North-south compression and approximate east-west extension is indicated, which support a strong upper mantle and conforms to the observations of Chen and Molnar (1983), Zhu and Helmberger (1996), Priestley et al. (2008), and Monsalve et al. (2009). Higher stress prevailing in the upper crust zones compared to the lower crust and mantle zones is suggested by lower $b$-values.

In the crustal zones (depths <70 km), Tibetan Plateau is seen predominantly with normal faulting earthquakes suggesting east-west extension stress regimes. The seismic activity is seen more or less uniformly distributed except for a highly active pocket along the eastern flanks of Karakoram-Jiali fault. The estimated $b$-value and $m_{max}$ across the upper crust of region is in the range 1.05-1.37 and $M_w$ 7.2-8.0, respectively while the same for the lower crust is 1.14-1.28 and $M_w$ 6.5-7.8, respectively. The seismicity parameters indicate that the seismogenic stress prevailing in the region is higher at upper crust compared to the lower crust similar to the stress regimes in the Himalayan zones. Molnar and Chen (1983) observed that most seismicity in the region are clustered in the shallow upper crust (<15 km) and deeper crust (50-70 km) suggesting ductile middle crust. The depth classification in the present study broadly differentiates the two seismogenic zones and overlooks the aseismic middle crust zones. The recurrence period projected by the seismicity models for major earthquakes ($M_w > 7.0$) across the upper crust zones is found to be as low as ~100 years and as high as ~600 years. The same for lower crust zones ranges from one to several millenniums. The earthquakes with strike-slip faulting indicative of lateral shears can be seen on the western and eastern Himalayan flanks underlying Karakoram and the extent of the Po-Chu fault zones, respectively. The former zone is observed with high seismic activity rate. This agrees with clockwise large rotation in the deformation patterns on the east (e.g., Molnar and
Lyon-Caent, 1989). The Main Karakoram fault zone is seen with low $b$-value of $0.90(\pm0.11)$ and $m_{\text{max}}$ estimate of $M_w 7.6(\pm0.4)$. The fault zone is also observed to be active in both the span of upper and lower crust. The recurrence period of $M_w \geq 7.0$ events is observed to be around 230-280 years. At the depths of 70-90 km in the mantle portion of the lithosphere, the seismicity is sparse and has been attributed to low mantle heating alongwith high strain rates (Priestley et al., 2008). Normal faulting earthquakes have been reported by Molnar and Chen (1983) in the zone.

**Northwestern Frontier Province**: Along the northwestern frontier region of India-Eurasia plate convergence, the upper crust zones have $b$-values ranging between 0.77 and 0.98, except in central Pamir region where it is higher with estimated value of $1.01(\pm0.09)$ for the entire crustal volume. The entire terrain is observed to be seismically highly active. The Hindukush-Pamir seismogenic province exhibits intense deformations attributed to the continental collision. In the upper crust regimes, the central Pamir is seen with normal faulting with strike slip component indicating east-west extension and shearing motion on the north. To the west of Pamir, thrust faulting with strike-slip component is predominant along the Herat fault. On the northern and northeastern flanks, thrusting is also evident. Pamir mountain range has been associated with frontal thrusting and rear normal faulting similar to the settings in Himalayas (Brunel et al., 1994). In the Hindukush ranges, the upper crust seismogenic zone exhibits predominant thrust faulting events. The earthquakes in the zone follow active subduction tectonics. Shallow thrusting with small strike-slip component is visible along MBT in the region of Hazara syntaxis. The western margin of the syntaxis is associated with wedge decollement (Bendick et al., 2007). Low angled thrust faulting, similar to ones in shallow thrust planes of the Himalayas, is observed along the lobed Sulaiman-Kirthar ranges while strike-slip faulting is predominant in the Chaman and Ornach-Nal fault zones.

In the lower crust zones at the depth range of 25-70 km, strike-slip faulting is observed in all the major fault zones across the northwest frontier region of India-Eurasia plate convergence, except in central Pamir where normal component in the predominant faulting is considerable and in the western Kunlun Shan fault zone at the northern flanks where thrusting is observed. Higher $b$-values ranging 1.05-1.22 are observed in these zones, except in the western Kunlun Shan fault zone where it is 0.95 and in the Salt range thrust zone where it is 0.93. The observation is indicative of higher stress acting at the northern and southern flanks of Hindukush-Pamir in the region. The distribution of estimated $m_{\text{max}}$ varies from $M_w$ 7.3 to $M_w$ 8.3. At the higher depths (>70 km), the seismicity is clustered in Hindukush-Pamir zone. Strike slip motion is predominant in Pamir where the $b$-value are found to be $1.28(\pm0.09)$; otherwise the events are mostly reverse faulting with strike-slip components in the Hindukush and adjoining regions of MBT, where the $b$-value corresponds to $1.10(\pm0.03)$, and $1.07(\pm0.09)$ respectively. At the depths >180 km, seismicity is seen mostly in two zones: Hindukush and Pamir. In the former zone, the events show predominantly thrusting and the $b$-value is estimated to be $0.84(\pm0.04)$. Predominantly normal faulting with strike-slip component is observed in the latter zone which associates a higher $b$-value of $1.35(\pm0.1)$. A northward subduction of the Indian plate beneath the Eurasian plate for the Hindu Kush and a southward subduction of the Eurasian plate beneath the Pamirs has been postulated (Burtman and Molnar, 1993; Fan et al., 1994). However, Pegler and Das (1998) found that the intermediate depth seismicity in the
region can be explained by single contorted zone of seismicity with most activity concentrated at 100-300 km depth. Hindukush is associated with subduction while Pamir is characterized by the deformations due to flow in upper mantle. Higher stress regimes is inferred at Hindukush, especially in the upper crust and mantle fault zones compared to those at Pamir. Furthermore, the observed strike slip components in the lower crust and upper mantle in lower stress (i.e., higher *b*-value) condition are consistent with the contorted seismogenic zone model.

**Northeast India and Indo-Myanmar Arc:** The composite fault plane solutions derived microseismicity data in the Shillong Plateau exhibit predominant reverse faulting conforming to the underlying ‘popup’ tectonics (Kayal, 2001; Kayal *et al.*, 2006). The seismic activity in the terrain is significantly high in the lower crust compared to the upper crust. Likewise, the *b*-value is observed here is higher about 0.94(±0.11) in the upper crust compared to that to lower crust, which is around 0.91(±0.10), suggesting higher stress in the lower crust zone. The recurrence period of 500-600 years for major/large earthquakes in the region by Sukhija *et al.* (1999a) is seen conforming to the magnitude range $M_w$ 6.5-7.0 according to the seismicity models. In the lower crust zone, the long recurrence period of ~3000 years for the great earthquakes projected by Bilham and England (2001) agrees with the seismicity models. On the other hand, lower recurrence period of ~1200 years for the great earthquakes suggested by Rajendran *et al.* (2004) implicates a lower *b*-value around ~0.70. The local seismicity exhibits assorted focal mechanisms - reverse, strike-slip, and normal faulting correlated to number of faults, shear and lineaments traversing across the region (Nandy, 2001). Reverse and strike-slip faulting type events are mainly observed across the Tripura-Mizoram folds. The southern parts of Indo-Myanmar arc exhibits transcurrent motions while reverse faulting is predominant in northern parts. These correspond to NNE–SSW compressional stress in the region as observed by Chen and Molnar (1990). On the northern margin, the Indo-Myanmar features are buttressed by the Mishmi Himalayas. The NW-SE trending Po Chu fault in the terrain is believed to be associated with a dextral shear movement (Ben-Menaham *et al.*, 1974). The *b*-value observed here corresponds to mean values of 1.03 and 1.21 in the upper and lower crust, respectively. Unlike the Shillong zone, the upper crust zones in rest of the northeast India are associated with the *b*-values in the range of 0.86-0.88. At the same time, the lower crust zones have higher *b*-values ranging 0.96-1.13. On the northern margin, the Indo-Myanmar features are buttressed by the Mishmi Himalayas. These parts of Indo-Myanmar arc have a high *b*-value of 1.13(±0.11) in the lower crust zone suggesting likely underlying weak geology. Overall, the observations indicate that to the contrary in the Shillong plateau, the other parts of northeast India experience higher stress in the upper crust zones. Further, earthquake recurrence period of ~500 years in these parts corresponds to $M_w$ 7.0-7.5 events while in the lower crust zones, it connects to $M_w$ 6.5-7.2 events. Along the Shan Sagaing fault, strike slip faulting to the south and reverse faulting to the further north is observed in the upper crust zone. However, the similar *b*-values of ~0.9 are observed along the fault indicating comparable stress distribution. The recurrence period of 500 years corresponds to $M_w$ 7.7–8.0 events in the zone. At the same time, the entire traverse of the fault exhibit predominantly strike-slip faulting in the lower crust zones with recurrence period of ~500 years corresponding to $M_w$ 6.7-6.8 events.
At the higher depth section (70-180 km), the clustered seismicity along the arc has reverse faulting with strong strike slip components in the north and south. The $b$-value observed here falls in the range of 0.90-1.07. The recurrence period of $M_w \geq 8.0$ events in the central and north parts is observed to be ~1000-2500 years according to the seismicity models. Incidentally, the present observation concurs with Stork et al. (2008) that both reverse and strike-slip faulting occurs at different depth sections along the arc. The fault kinematics of the Shan Sagaing fault exhibits characteristics of a transform fault while underthrusting at the higher depths along the northern Indo-Myanmar arc is observed to have approximate N-S direction. The compression regime extending to the descending slab of the Indian plate beneath the arc conforms to the oblique subduction of the Indian plate (Satyabala, 2003; Dasgupta et al., 2003; Angelier and Baruah, 2009).

**Andaman-Nicobar and northern Sumatra:** Earthquakes occurring in the upper crust zones along the Andaman-Nicobar exhibit predominantly reverse faulting conforming to the active subduction (dip-slip) tectonics. The dip is seen in northeast direction. Similar observation has also been made by Kayal et al. (2004). The $b$-value is observed to be about 0.82($\pm$0.07) while the recurrence period of great earthquakes is seen to be about 300 years in the zones. On the other hand, the seismicity is sparse in the region of Arakan offshore (15°N-20°N), especially at the higher depths; this has been implicated to likely aseismic deformations caused by thick sediment of Bengal basin (Kayal, 2008). On the western flanks, the two strike-slip faults namely, Shan-Sagaing and western Andaman, respectively are connected by a complex back arc where the events exhibit normal faulting types – typical of spreading ridges; a complex tectonic setting amidst the active subduction (Curray et al., 1979). High $b$-values close to 1.35($\pm$0.1) are observed here while the western Andaman fault and northern Sumatra zone associates $b$-value equal to 0.95($\pm$0.06). The seismicity in the zones is observed to be dominated by lower magnitude events. In the lower crust zones, the observed distribution of the fault kinematics is similar to that in the upper crust. The pertinent mean $b$-values is seen ranging 0.84-1.10. The zone of 2004 Sumatra earthquake exhibits $b$-value equal to ~0.87 with the recurrence period for events $M_w \geq 8.5$ to be about 1000 years. These estimates are within the time-period of 900-1000 years estimated by Rajendran et al. (2007a). The stretch from the southern Indo-Myanmar arc to the Nicobar Islands has predominantly normal faulting type with small strike-slip component and $b$-value of 1.36($\pm$0.14). The normal faulting events are likely due to extensional tectonics underlying the back-arc convergences. East to the Andaman Islands, the normal faulting zone has high $b$-value of 1.57($\pm$0.13). At depths of 70-180 km, the seismicity is highly clustered with distinct segregation between Nicobar and Andaman-Sumatra seismogenic provinces. The $b$-value in both the zones is close to ~1. A variation in the orientation of the compressional stress is seen with N-S in the north and NW-SE in the south forming a gentle arc. Kumar et al. (1996) also reported differences in the tectonic patterns across the Andaman-Nicobar Islands. The seismicity at the depths >180 km occurs mostly in the Andaman-Sumatra tracts and predominantly have normal faulting with strike-slip component. The tectonic patterns conform to the down-dip tensional events of slab-pull extensional tectonics, which contribute to the subduction of the Indian plate (Dasgupta et al., 2003).

**Intraplate regions:** The intraplate deformations in the northeast India has been discussed earlier. In western India including the highly seismogenic zone of Kutch, the upper crust events exhibit predominant
reverse faulting typical of compressive stresses although some faults in the region have strike-slip faulting. The interconnected rift systems and reactivation of ancient rifts play an important role in the seismogenic process of the region. The $b$-value of $0.95(\pm 0.05)$ is associated with the zone. The seismicity models indicate the recurrence period of 2000-3000 years for M$_w$ 7.7 earthquake (magnitude similar to that of Bhuj earthquake). Rajendran and Rajendran (2001), on the other hand, suggested that the recurrence period to be much lower to the tune of 900-1000 years supporting characteristics earthquake model for the pertinent fault zones. The events in southern Gujarat exhibit predominantly transcurrent motions with $b$-value of 0.86(±0.07). Similar fault kinematics is observed along the western parts of south India. In the western Ghat, the events in highly seismogenic Koyna-Warna zone have predominant strike-slip faulting mechanism. However, normal faulting events are also observed in the region, especially in the vicinity of Warna reservoir (e.g., Bhattacharya, 2007; Gahalaut et al., 2004). High seismic activity rate is observed here with higher $b$-value of 1.09(±0.12) implicating predominance of lower magnitude earthquakes. The underlying tectonics has been attributed to fluid filled fractures responding to the reservoir loadings (Gupta et al., 1997; Talwani, 1997) combined with the interactions of the fault zones (Gahalaut et al., 2004). Reservoir triggering in the region is seen spatially confined and influenced by rate of loading implicating a stress relieving mechanism (Gupta, 2005). Most shallow earthquakes associated with discrete faults in South India exhibits strike-slip faulting mechanisms; however, the zone of upper Godavari fault in Latur region is implicated to reverse faulting (Ramesh and Estabrook, 1998). Leaving aside Koyna-Warna zone, the observed $b$-value in the region ranges from 0.78 to 1.19. Except for Kaddam fault zone with $b$-value of 1.19(±0.12), the estimates of $b$-value do not have statistically significant differences with estimated value 0.81(±0.05) for the combined zones. The estimate $b$-value in this case conforms to high stress regimes associating reverse faulting type. Jaiswal and Sinha (2007) suggested different $b$-values for rift and cratonic zones in south India corresponding to 0.85 and 1.0, respectively. They noted that the proportion of (relatively) larger earthquakes to the smaller ones is higher in the rift zones. However, that is not the case as the magnitude distribution of the main-shock events is observed to be more or less similar for the different zones in the upper crust barring Koyna-Warna, Kutch, and Kaddam fault zones. The observed values are higher than those compared to previous studies (e.g., Kaila et al., 1972). Higher $b$-value implies larger number of smaller earthquakes in the catalog, and the present cases may be attributed to the increased instrumentation and improved earthquake reportings (cf. Jaiswal and Sinha, 2007). On the other hand, it is noted that the regional stress under compression regime is pertinent to the seismogenic zones across the entire south India, even in the Koyna-Warna zone where the high seismic activity has been implicated to reservoir triggered seismicity. The intraplate regions adjacent to the plate boundary tracts encompassing the Indo-Gangetic plains and Bengal Basin as a whole yields a low $b$-value of 0.72(±0.1) conforming to prevailing high stress. Thrust components indicative of compressional stress oriented in N-S on the western parts and NNE-SSW on the eastern parts are evident. At the same time, the offshore earthquakes in Bay of Bengal exhibit predominantly strike slip faulting and have $b$-value estimate of 0.87(±0.12).

Seismic activities in the lower crust zones (depth ~25-40 km) across the intraplate region are sparse. The $b$-value for the entire south India is found to be 0.89(±0.13) while the parts adjoining the active
regions to the north exhibits b-value of 0.93((±0.10). The seismogenic lower crust in the Narmada-Son lineament has been implicated to possible presence of high pore pressure, fractured rocks at depth, high strain rate, and influenced by the N-S compression due to motion of Indian plate (Gahalaut et al., 2004). The predominant faulting type has been construed to be reverse faulting in the zone. Similar faulting type is also observed at the off coast zones in the Bay of Bengal but with strike-slip component. The compressional stress is indicated in the NNE-SSW direction. On other hand, strike slip faulting similar to pattern observed along the western Indian plate margins is observed at Kutch seismogenic province. The northwestern India-Eurasia plate convergence is suggested to be undergoing transcurrent motions. The Indian intraplate seismicity has been explained in terms of stress within Indian plate driven by the plate collision in the north causing intermittent faulting in the weak zones. The stress concentrations believed to be localized in those zones, especially pertaining to ancient rifts. Reservoir triggered phenomena has been also observed. Bilham et al. (2003) suggest that Indian plate is under a flexural force, which is indicated by approximately EW trending features: depression at Himalayan foothills, bulge at central India, and another depression across south India. They asserted that the flexures are responsible for the prevailing stress regime within the Indian plate.

**Issues for the Hazard Studies**

The regional seismotectonics is evaluated with weighted average moment tensors for the delineated seismic zones. Extensive analysis of the stress fields (e.g., Angelier and Baruah, 2009) is beyond the scope of the present study. On the other hand, the premise of long-term b-value derived from data of comparatively shorter time period forms the fundamental time-independent seismicity model derived using the earthquake catalog. The damaging (presumably) large earthquakes occupy the tail of the frequency magnitude distribution. In the geological based analysis, the estimation of $m_{max}$ has significant effect on the associated slip-rate as well as b-value. However, the present study considers only areal zones while assuming that b-values for the main-shock events in a specific seismotectonic province and (truncated or) linear GR relation are reasonably consistent over the long time-period. The zone-specific seismicity models have been examined with GR and Truncated GR relations. The available average fault slip rates for particular zone are employed to observe viability of the characteristic recurrence models. Although the recurrence models can be influenced by the choice of seismic source zones, these perspectives concur with the concept of Main (1996) based on the self organized criticality property of the earthquakes wherein ‘critical’ is when a linear fit of GR relation is seen appropriate, ‘sub-critical’ is when the largest events occur lesser than that entailed by the linear fit, and ‘super-critical’ is when the largest earthquakes are seen more than those indicated by the linear fit. The regression performed on the aggregated focal mechanisms indicates tectonic implications of the b-value. The overall patterns of b-value vis-à-vis the predominant faulting types in the present study are comparable with those of Schorlemmer et al. (2005), Thingbaijam et al. (2008), and Gulia and Wiemer (2010). Spatial mapping of the b-value at a higher resolution based on the concept that the parameter inversely correlates with the stress have led to the development of Asperity Likelihood Models (Wiener and Schorlemmer, 2007). In the overall assessment, poor data quantity in several zones have attributed to the b-values exhibiting
higher standard deviations in the order of 0.2. Such uncertainties are usually handled by either logic tree approach or Monte-Carlo technique (e.g., Cramer et al., 1996 and 2002; McGuire, 2004; Suckale and Grünthal, 2009).

The minimum magnitudes across the seismic zones recorded in the earthquake catalog often reflect data incompleteness and deviates from linear trend of the generally accepted GR relation. In the seismicity parameters assessment a threshold (minimum) magnitude of completeness is searched for which the frequency magnitude trend is linear conforming to the GR relation (e.g., Wiemer and Wyss, 2000; Thingbaijam et al., 2008 and 2009). In the seismic hazard computation, the contributing earthquakes are considered above a minimum magnitude below which the earthquake hazard on engineering structures is insignificant. The seismic intensity attenuation models provided by Szeliga et al. (2010) based on observed macroseismic intensity data in India indicate that magnitude $M_w \approx 4.5$ for shallow earthquakes yields seismic intensity of hazard interest, i.e. in the vicinity of European Macro Seismic (EMS) VII (damaging) and VI (slightly damaging), at the hypocentral distance of 10-20 km. This observation implies use of lower magnitude bound $M_{\text{Lower}}$ equal to $M_w \approx 4.5$ considering the shallow earthquakes for the seismic hazard analysis in India to be pragmatic. In case of deeper earthquakes, lower magnitude bound for damaging earthquakes would be around $M_w 5$ or higher.

In order to construct spatial feature of the earthquake occurrences, seismicity smoothening has been applied to predict the activity rates for the different seismogenic depth considerations. Although the activity rates have been defined at grid points, finite fault rupturing can be applied with the rupture parameters formulated for magnitude and the underlying tectonics for the hazard assessment. The considered elementary concepts of areal seismic source have the mean rates of seismic activity varying spatially within the source zone while other parameters, namely $m_{\text{max}}$ and $b$-value, remain fixed. This assumes $b$-value and the activity rate for the mean magnitude to be uncorrelated and non-uniform distribution of earthquake probability within a zone. To sum up, the present study is a step towards development of database of seismogenic sources within the purview of India and its adjoining regions. Further work envisaged is development of fault specific sources based on active tectonics to provide comprehensive assortment of the seismogenic sources controlling the seismic hazard in the territory.

**Inferences**

The present study aimed at establishing seismicity-derived hazard components using updated earthquake catalog supplemented by fault database, centroid moment tensor database, fault slip rate information, and results of palaeoseismic investigations. Better methodological approach have been adopted with segregation of seismogenic zones pertaining to the four hypocentral depth sections presenting a first order approximation towards volumetric seismogenic sources superseding the conventional 2D zoning. The estimated seismicity parameters across the seismogenic zones have been appraised in the light of the underlying tectonics and observed to be consistent in terms of different stress regimes. The present results constitute fundamental inputs for the subsequent analyses towards the seismic hazard analysis of the region.
Annexure – A

Historical and Pre-historical Earthquakes in India and adjoining regions

**Historical Earthquakes**

**Table I.A1:** Historical earthquakes occurring prior to 1900 with magnitudes >6.5 in India and the adjoining regions; approximate epicentral locations are given. For the magnitudes with unspecified uncertainty, 0.6 magnitude units are assumed.

<table>
<thead>
<tr>
<th>Date</th>
<th>Lat (°N)</th>
<th>Lon (°E)</th>
<th>M_w</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sept., 0180</td>
<td>39.40</td>
<td>99.50</td>
<td>7.4</td>
</tr>
<tr>
<td>June, 0819</td>
<td>36.40</td>
<td>65.40</td>
<td>7.3</td>
</tr>
<tr>
<td>June 06, 1505</td>
<td>29.50</td>
<td>83.00</td>
<td>8.6-8.8</td>
</tr>
<tr>
<td>July 06, 1505</td>
<td>34.50</td>
<td>69.10</td>
<td>7.2</td>
</tr>
<tr>
<td>June 17, 1515</td>
<td>26.60</td>
<td>100.80</td>
<td>7.4</td>
</tr>
<tr>
<td>Sept., 1555</td>
<td>33.50</td>
<td>75.50</td>
<td>≥7.6</td>
</tr>
<tr>
<td>July 11, 1609</td>
<td>39.20</td>
<td>99.00</td>
<td>6.6</td>
</tr>
<tr>
<td>May 26, 1618</td>
<td>18.90</td>
<td>72.90</td>
<td>6.9</td>
</tr>
<tr>
<td>July 13, 1652</td>
<td>25.40</td>
<td>100.50</td>
<td>6.6</td>
</tr>
<tr>
<td>July 15, 1720</td>
<td>30.00</td>
<td>80.00</td>
<td>7.5(±0.4)</td>
</tr>
<tr>
<td>1751</td>
<td>31.30</td>
<td>80.00</td>
<td>7.0</td>
</tr>
<tr>
<td>Sept. 01, 1803</td>
<td>30.00</td>
<td>78.00</td>
<td>7.5(±0.2)</td>
</tr>
<tr>
<td>June 16, 1819</td>
<td>23.60</td>
<td>68.60</td>
<td>7.7(±0.2)</td>
</tr>
<tr>
<td>Jan. 22, 1832</td>
<td>36.50</td>
<td>71.00</td>
<td>7.3</td>
</tr>
<tr>
<td>Aug. 26, 1833</td>
<td>28.00</td>
<td>85.70</td>
<td>7.6(±0.2)</td>
</tr>
<tr>
<td>Feb. 19, 1842</td>
<td>35.00</td>
<td>71.00</td>
<td>7.4</td>
</tr>
<tr>
<td>Oct. 31, 1847</td>
<td>~7.00</td>
<td>?</td>
<td>7.5-7.9</td>
</tr>
<tr>
<td>Jan. 24, 1852</td>
<td>29.30</td>
<td>68.60</td>
<td>6.6</td>
</tr>
<tr>
<td>May 23, 1866</td>
<td>27.70</td>
<td>85.30</td>
<td>6.0-7.0 (?)</td>
</tr>
<tr>
<td>Jan. 10, 1869</td>
<td>25.50</td>
<td>93.00</td>
<td>7.4(±0.2)</td>
</tr>
<tr>
<td>April 11, 1870</td>
<td>30.00</td>
<td>99.00</td>
<td>7.3</td>
</tr>
<tr>
<td>Oct. 18, 1874</td>
<td>35.10</td>
<td>69.20</td>
<td>6.9</td>
</tr>
<tr>
<td>Mar. 02, 1878</td>
<td>34.00</td>
<td>73.20</td>
<td>6.7(±0.3)</td>
</tr>
</tbody>
</table>
The reportings of earthquakes prior to 1900 in the region are rather scanty. Most events have inadequate information related to epicentral location as well as magnitude (e.g., Bapat et al., 1983). Recently, Martin and Szeliga (2010) compiled a catalog of macroseismic intensity data for the earthquakes occurring as early as 1636 in India and adjoining regions. Szeliga et al. (2010) computed magnitude and epicentral location of 53 historical earthquakes occurring during 1762-1898 listed in the catalog. Reportedly, the catalog has been observed to be complete for only the events with magnitude > 8.0 since 1800. At the same time, the magnitude and location of the earthquakes estimated using the macroseismic intensity data have high uncertainty owing to their sensitivity to the applied methodologies and involvement of subjective interpretations. A compilation of the events with $M_W > 6.5$ is given in Table I.A1; a more comprehensive list is accessible at http://earthqHz.net/sacat. The magnitude uncertainty for the historical earthquake wherever cannot be established is assumed to be around 0.6 magnitude units. The $M_W$ values are evaluated for the events available in $M_s$ scale using the $M_W-M_s$ relationship given by Nath et al. (2011c). The National Geophysical Data Center database (NGDC, http://www.ngdc.noaa.gov) provides a global database but suffers from data inconsistencies (Dunbar et al., 1992). The earliest event listed occurred in China with the magnitude of $M_s \approx 7.5$ (Lee et al., 1976; Bapat et al., 1983; Pun and Ambraseys, 2007). Other events include $1515 \ M_s \approx 7.5$, $1609 \ M_s \approx 6.7$, and $1652 \ M_s \approx 6.7$. Further, two more events are reported by NGDC, namely $1870 \ M_s \approx 7.3$, and $1896 \ M_s \approx 6.6$. Ambraseys and Bilham (2003a) estimated $M_s$ values for historical earthquakes in Afghanistan based on perceived intensity of damage and thereafter, employed an empirical bilinear relation to compute seismic moment and consequently the equivalent $M_W$ estimates. Amongst all the events, $1505 \ M_w \approx 7.2$, $1832 \ M_w \approx 7.3$, $1842 \ M_w \approx 7.4$, $1852 \ M_w \approx 6.6$, and $1874 \ M_w \approx 6.9$ are found to be pertinent. An event of $M_s \approx 6.8$ reportedly occurred during 1892 in West Pakistan (Ambraseys, 2000). Ambraseys and Douglas (2004) calibrated the magnitude of several earthquakes occurring in the northern India – $1505 \ M_w \approx 8.6-8.7$, $1555 \ M_w \geq 7.6$, $1720 \ M_w \approx 7.5(\pm 0.4)$, $1803 \ M_w \approx 7.7(\pm 0.2)$, and $1833 \ M_w \approx 7.6(\pm 0.2)$. Apparently, the 1505 earthquake was assigned $M_w = 8.2$ according to the macroseismic intensity data (Ambraseys and Jackson, 2003). However, Bilham and Ambraseys (2005) proposed that magnitude $8.6 < M_w < 8.8$ would be more appropriate in accordance to the estimated rupture length of 600 km, rupture width of 70–90 km and slip of 7–15 m. An event of magnitude $M_s \approx 7.0$ during 1751 has also been observed (Ambraseys and Jackson, 2003). The records from Geological Survey of India mention an event during 1885 in the Kashmir area of $M_s \approx 7.0$. A strong earthquake has been noted
in Nepal during 1866 associated with an intensity of VIII in Modified Mercalli Scale (Chandra, 1977; Chitrakar and Pandey, 1986). An event of $M_S \approx 6.7$ occurred in the northwest frontier province during 1878, and an event of $M_S \approx 6.8$ occurred in Bangladesh during 1885 (Amateur Seismic Centre, 2009). Bilham (1999) recount 1819 Allah-bund earthquake of $M_W 7.7(\pm 0.2)$, and 1869 Cachar earthquake of $M_W 7.4(\pm 0.2)$. Rao (2000) reported an event of magnitude of $M_S \approx 6.9$ in south India that occurred during 1618. The 1897 Shillong earthquake is designated with $M_W 8.1(\pm 0.1)$ by Bilham and England (2001). Ambrasey and Jackson (2003) reported two larges earthquakes during 1751 ($M_S \geq 7$) and 1806 ($M_S \approx 7.7$) in the upper northeast India. A major earthquake occurred along the Arakan Coast during 1762 but the exact epicentral location is unknown while magnitude could be anywhere near $M_W \approx 8.8$ (Cummins, 2004; Okal and Synolakis, 2008). Along the Andaman tracts, Bilham et al. (2005) reported the occurrence of a major earthquake during 1881 having magnitude $M_W \approx 7.9$. In Makran seismogenic province, Heidarzadeh et al. (2008) list several historical earthquakes with unknown magnitude during the years: 326 BC, 1008, 1483, 1668, 1765, and 1851 in the region.

The compilation is likely to suffer from inhomogeneity in the magnitude estimation procedure since calibration across different assessments is not possible due to lack of common reportings. Apart from seismic intensities, the magnitudes has been estimated on the basis of slip parameters inferred from the associated deformation, viz. 1819 Kachchh earthquake (Bilham, 1999) and 1897 Shillong earthquake (Bilham and England, 2001). The reportings of historical earthquakes in a region are dependent on several factors such as availability of the written reports, and the distribution of the population and built-up environment during the period that can cause ambiguous reportings in space and time.

**Pre-historical Earthquakes**

Paleoseismic data are crucial in defining earthquake recurrences and realistic projection of maximum earthquakes, especially in the context of limited temporal span covered by reportings of the historical earthquakes. Several palaeoseismic investigations have been reported across India, especially in the Himalayas. The Main Frontal Thrust (MFT) fault is recognized to be active in the central Himalayas (Wesnousky et al., 1999; Lave and Avouac, 2000). Although the known historical earthquakes have not associated surface ruptures, Wesnousky et al. (1999) suggested that there has been prehistoric earthquakes sufficiently large to produce surface ruptures. Lave and Avouac (2000) inferred from microseismicity and geodetic monitoring of over a decade that interseismic strain accumulates in the higher Himalayas locking MHT over a ductile decollement beneath southern Tibet and is eventually released by great ($M_w > 8.0$) earthquakes. Lave et al. (2005) presented evidence of an earthquake occurring during 1100 AD in the Frontal thrust fault. The rupture extent of the earthquake is inferred to be exceeding 240 km and magnitude close to $M_w 8.8$. The authors asserted that $M_w < 8.2$ events in the frontal Himalayas folding would not associate surface rupture. On the other hand, Wobus et al. (2005) suggested surface faults on the MFT as alternate to the single fault model of MFT consistent with available geodetic, geomorphic and microseismicity data.
In northwestern Himalaya, Kumar et al. (2001) identified sites of three large surface rupture earthquakes 260 AD, 1294 AD and 1423 AD along Black Mango fault that connects two segments of the active MFT. The last two events have estimated displacement of 4.6 m and 2.4-4.0 m, respectively while larger displacement has been implicated to the 260 AD event. These events correspond to magnitudes $M_w \geq 7.0$ according to displacement-slip relationship of Wells and Coppersmith (1994). Kumar et al. (2006) found paleoseismic evidences of surface rupture along MFT taking place during 1200-1700 AD associating single event displacements of $\sim 11-38$ m. They estimated recurrence period of at least 1330-3250 years for the events with similar sized displacements in the terrain. A recent study in Kangra meizoseismal zone by Malik et al. (2010) also supported that HFT to be more active than any other fault in the region. They inferred that two major events occurred in the zone; recent one during 1500-1600 AD. The events associate net displacements of $\sim 9.0$ m and magnitudes $M_w > 7.0$ with recurrence period of 1160±250 years. Malik et al. (2003) reported that preliminary trench investigation at the base of the Chandigarh Fault Scarp revealed total displacement of 3.5 m possibly resulting from one large magnitude ($M_w \sim 7.0$) prehistoric earthquake. They observed that active faults branching out of major fault systems i.e., MBT and MFT exhibit fault scarps of 12–50 m suggesting continued tectonic movement. Bendick et al. (2007) studied the 2005 Kashmir earthquake based on surface displacements and locations of aftershocks to infer association of multiple fault planes and suggested recurrence interval of 680 ± 150 years for similar sized events. On the other hand, Kondo et al. (2008) performed paleoseismological trench excavations across the middle section of the 2005 surface rupture and inferred ~2000 years interval for similar sized events. They observed that the event occurred in sub-Himalayas while stressing that accumulated elastic strain around the complex northwestern margin of the Indo-Asian collision zone has not been significantly released by the 2005 earthquake. In the Dehra Dun Valley, Malik and Nakata (2003) found at least two events to be evident from palaeoseismic investigations at Sirmurital fault demonstrating a recurrence period of 938±59 years. Similarly, Malik and Mathew (2005) observed that the low angle thrust fault Pinjore Garden Fault, which is an offshoot of the Main Boundary Thrust (MBT) system, associates at least two prehistoric earthquakes. Sukhija et al. (2002) found evidence for two prehistoric seismic events in the meizoseismic zone of 1934 earthquake. They suggested that the region has been continuously seismically active.

In the northeast India, several historical earthquakes have been reported; although their estimate of their magnitudes and location are not precisely known (Rajendran et al., 2004; Ambraseys and Jackson, 2003; Iyengar et al., 1999; Bapat et al., 1983). A very large earthquake (magnitude $\geq 8.0$) during 825-835 AD located near Guahati-southern Brahmaputra valley; moderate earthquakes during 1548 AD and 1596 AD; minor tremors during 1642 AD, 1649 AD and 1663 AD; very large earthquake (magnitude $\geq 8.0$) in the vicinity of source location of 1950 earthquake during 1697 AD; large earthquakes located in the higher Himalayas during 1713 AD and 1806 AD; series of tremors moderate earthquakes during 1870-1880 AD. Rajendran et al. (2004) suggested that the sand vents in the Lower Brahmaputra Valley were formed by the 1697 earthquake. Steckler et al. (2008) pointed out that the possible rupture of 1548 earthquake might be in Bangladesh. Reddy et al. (2009) performed field investigations in the meizoseismal area of 1950
Assam earthquake and identified more than a dozen prominent liquefaction features as evidences of large to great earthquakes. The data have been observed to corroborate the 1548 AD event. Bilham and England (2001) postulated a south dipping hidden fault ‘Oldham fault’ to be the source location of the 1897 Shillong earthquake. Rajendran et al. (2004) concurred to the south dipping structure with a minor deviation on its location. However, different recurrence period of earthquakes in the zone has been postulated; major/large earthquakes (perhaps M_w >6.5) in 500-600 years (Sukhija et al., 1999b), great earthquakes (M_w >8.0) in 3000-8000 years (Bilham and England, 2001), and great earthquakes (M_w >8.0) in about 1200 years (Rajendran et al., 2004). This aspect warrants further exposition on the pertinent earthquake occurrence model according to statistical treatment of the recent seismicity in the region.

In the northwestern part of the Rann of Kutch, western India, Bilham (1999) observed that the 1819 earthquake M_w 7.7(±0.2) with an apparent fault-displacement greater than 11 m creating a visible fault scarp. Based on age data of liquefaction features, Rajendran and Rajendran (2001) observed that a predecessor of the 1819 earthquake occurred 800-1000 years ago. They listed several historical earthquakes in the region; moderate event 1668, 1821 M_w ~5, 1845 M_w >6.0, 1856 M_w ~5, and 1864 M_w ~6. Rajendran et al. (2008a) combined the palaeo-liquefaction features with archeological data to infer occurrence of two previous earthquakes in source region of 2001 Bhuj earthquake about 4000 and 9000 years ago. Additionally, parts of Kutch and Cambay basins are also observed to be potential seismic source locations. In the meizoseismal area of the 1993 Latur earthquake that occurred in cratonic regions of south India, Sukhija et al. (2006) found geological evidences of a paleoseismic event of similar size that took place during 190 BC-410 AD placing an interval of roughly 2300 years for similar sized events in the zone.

In Andaman-Sumatra tracts, Bilham et al. (2005) suggest that large thrust earthquakes in 1847 (M_w 7.5-7.9), 1881 (M_w 7.9±0.1), 1881 (M_w 7.0±0.1), and 1941 (M_w 7.7±0.1) to have occurred on intermediate regions of the down-dip boundary zones located within the 2004 Sumatra earthquake rupture. Two great earthquakes also occurred in the Sumatra region which has been inferred by Natawidjaja et al., 2006 as corresponding to magnitudes M_w 8.5-8.7. Rajendran et al. (2007b) on basis of paleoseismological/tsunami evidence suggested that at least one predecessor of the 2004 earthquake occur 900-1000 years ago. They also observed that such events are infrequent and follow variable intervals. Rajendran et al. (2008b) analyses morphological features along the coast of the Andaman and Nicobar Islands and observed ~600 years and ~1000 years old coastal uplifts. The authors attributed the level changes to two major previous subduction earthquakes in the region.

Most of the paleoseismicity investigations are indicated with higher uncertainty in the magnitude, location and recurrence period of the earthquakes. At the same time, such studies are generally fault specific and have higher significance in fault specific seismicity modeling. Nonetheless, the reportings provide supplementary information towards the earthquake recurrence and constraining the maximum earthquakes in areal source zone based time-independent seismicity modeling.
Annexure – B

Seismicity Models for Source Zones in India and the adjoining regions

Source Zones

The demarcated seismogenic sources zones for the four different seismogenic layers in India and adjoining regions, as depicted in Figure I.17, encompass areas with implicit fault multiplicity wherein the frequency magnitude distribution can be expected to be uniform.

Time Independent Recurrence Model

In the Global Seismic Hazard Assessment Programme exercise for the India and adjoining regions, Bhatia et al. (1999) employed conjectural $m_{\text{max}}$ based on the past seismicity but did not elaborate on the applied estimation technique. More recently, there have been several investigations on the distribution of $m_{\text{max}}$ in the study region. Thingbaijam and Nath (2008) carried out assessment for broad seismogenic zones in the northeast India and observed that seismicity models conform to $m_{\text{max}}$ of $M_W$ 8.2-8.7 in the region. Jaiswal and Sinha (2008) defined the distribution of $m_{\text{max}}$ estimated from seismicity data according to the geologic zones of craton and rift in the peninsular India with the mean values ranging from $M_W$ 5.6 to $M_W$ 8.3. Bilham et al. (2001) observed that the slip potential across the Himalayan tracts could yield to several great earthquakes of $M_W$ ~8.6. In the central Himalayan terrains, Feldl and Bilham (2006) observed that recurrence of the historical 1505 Nepal Earthquake would cause an earthquake of magnitude in order of $M_W$ ~8.6. Thingbaijam et al. (2009) observed that the broad source zones in the northwest frontier province of the India-Eurasia plate convergence zone covering the fault tracts of Sulaiman-Kirthar and Hindukush-Pamir have $m_{\text{max}}$ estimates in the range of $M_W$ 8.05-8.31. These regional assessments have been taken into account in the present study. Apart from this, the catalog based maximum likelihood method (Kijko, 2004; Thingbaijam and Nath, 2008) and the probabilistic extrapolation for annual probability of exceedance of 0.001 is employed in according to the data feasibility. In case of the zone associated with the 2004 Sumatra earthquake of $M_W$ 9.2($\pm$0.1), the catalog based maximum likelihood method yields a maximum earthquake of $M_W$ 9.4($\pm$0.2).

Table I.B1: A comparison of the return period for the maximum earthquake, $m_{\text{max}}$, in selected zones estimated from fault slip rate $S_t$ through slip ratio technique, and model fit of the linear GR and the tapering GR relations. These are denoted as $T_{SR}$, $T_{GR}$ and $T_{TGR}$, respectively. The value of $S_t$ for each zone is decided from the estimated values as given in the cited reference/s

<table>
<thead>
<tr>
<th>Zone</th>
<th>$m_{\text{max}}$ ($M_W$)</th>
<th>$S_t$ (mm/yr)</th>
<th>$T_{SR}$</th>
<th>$T_{GR}$</th>
<th>$T_{TGR}$</th>
<th>Reference#</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.0</td>
<td>16.0</td>
<td>4.97e02</td>
<td>1.56e03</td>
<td>1.56e04</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>8.0</td>
<td>16.0</td>
<td>4.97e02</td>
<td>6.28e02</td>
<td>6.27e03</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>8.0</td>
<td>16.0</td>
<td>4.97e02</td>
<td>2.72e02</td>
<td>2.72e03</td>
<td>1</td>
</tr>
<tr>
<td>Zone</td>
<td>$m_{\text{max}}$ ($M_W$)</td>
<td>$S_r$ (mm/yr)</td>
<td>$T_{SR}$</td>
<td>$T_{GR}$</td>
<td>$T_{TGR}$</td>
<td>Reference*</td>
</tr>
<tr>
<td>------</td>
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<tr>
<td>4</td>
<td>7.4</td>
<td>12.0</td>
<td>2.19e02</td>
<td>7.63e02</td>
<td>7.63e03</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>7.8</td>
<td>7.0</td>
<td>7.85e02</td>
<td>7.16e02</td>
<td>7.15e03</td>
<td>3</td>
</tr>
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<td>6</td>
<td>8.2</td>
<td>10.0</td>
<td>1.17e03</td>
<td>2.61e04</td>
<td>2.67e03</td>
<td>3, 4</td>
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<tr>
<td>7</td>
<td>7.6</td>
<td>6.0</td>
<td>6.3e02</td>
<td>2.0e03</td>
<td>2.00e04</td>
<td>5</td>
</tr>
<tr>
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<td>8.0</td>
<td>1.73e03</td>
<td>1.37e03</td>
<td>1.37e04</td>
<td>4*</td>
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<tr>
<td>9</td>
<td>8.0</td>
<td>14.0</td>
<td>6.12e02</td>
<td>9.80e02</td>
<td>9.00e03</td>
<td>4</td>
</tr>
<tr>
<td>10+34</td>
<td>7.1</td>
<td>8.0</td>
<td>6.12e02</td>
<td>9.80e02</td>
<td>9.05e03</td>
<td>6</td>
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<tr>
<td>11+33</td>
<td>7.8</td>
<td>6.0</td>
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<td>3.57e02</td>
<td>3.56e03</td>
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<td>12</td>
<td>8.6</td>
<td>14.0</td>
<td>1.60e03</td>
<td>5.34e02</td>
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<td>1.45e04</td>
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<td>2.51e02</td>
<td>3.1e02</td>
<td>3.005e03</td>
<td>11</td>
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<td>21</td>
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<td>6.3</td>
<td>3.81e03</td>
<td>2.87e03</td>
<td>2.87e04</td>
<td>9</td>
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<tr>
<td>22</td>
<td>8.8</td>
<td>12</td>
<td>2.89e03</td>
<td>1.17e03</td>
<td>1.17e03</td>
<td>12</td>
</tr>
<tr>
<td>23+27+29</td>
<td>7.7</td>
<td>3.0</td>
<td>1.44e03</td>
<td>7.10e02</td>
<td>6.0e03</td>
<td>13</td>
</tr>
<tr>
<td>24+25+26</td>
<td>8.5</td>
<td>10</td>
<td>2.0e03</td>
<td>3.67e03</td>
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<td>28+30+31</td>
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<td>6.5</td>
<td>6.65e02</td>
<td>2.31e02</td>
<td>1.99e03</td>
<td>14</td>
</tr>
<tr>
<td>41+42</td>
<td>7.60</td>
<td>1.0</td>
<td>3.80e03</td>
<td>7.73e03</td>
<td>7.73e04</td>
<td>15</td>
</tr>
<tr>
<td>47+48+53 +54+55</td>
<td>8.3</td>
<td>14.0</td>
<td>9.89e02</td>
<td>1.40e03</td>
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<td>63</td>
<td>8.2</td>
<td>6.0</td>
<td>1.15e03</td>
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<td>7.65e03</td>
<td>12, 13</td>
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<td>119</td>
<td>8.3</td>
<td>0.7</td>
<td>1.97e04</td>
<td>7.20e03</td>
<td>7.20e04</td>
<td>15</td>
</tr>
</tbody>
</table>


* Average of Chaman-Gardiz-Konar system, Darvaz-Karakul fault zone, and Herat and Talas-Ferghana faults.
Figure I.B1 depicts the frequency magnitude distribution plots for main-shock events in each of the seismogenic source zones. The $b$-value and $a$-value are estimated by applying the maximum likelihood method (Aki, 1965; Bender, 1983) on the instrumental catalog. The incomplete data (including the historical data) are rendered return periods according to the models, namely Gutenberg-Richter (GR) and Truncated Gutenberg-Richter (TGR) models. The return periods for the earthquakes (near $m_{\text{max}}$) have also been evaluated using average lower estimate of the fault slip rates in the zone. While the linear GR relation can statistically accommodate large events if seismic source zone is of appropriate size and the temporal coverage of the catalog is also long enough (e.g., Molchan et al., 1997), TGR model is reckoned to be more appropriate considering the energy dissipations at the larger magnitudes. Possible cases for characteristic models are also noted. However, the overall average slip rates in the source zones could be still lesser than those employed here owing to the consideration of the large spatial bounds as well as likely presence of creeping blocks. As such Characteristic recurrence models entail fault-specific analyses, e.g., Petersen et al. (2004) in the Gujarat seismic province.
Figure I.B1: Frequency magnitude distribution plots for the seismogenic source zones (after Thingbaijam and Nath, 2011).
APPENDIX – II

Typical Geotechnical and Geophysical Investigations
Prescribed for Seismic Microzonation Investigations
for National Capital Territory (NCT), Delhi

Section – I: Work Components & Guidelines

Section – II: Technical Specifications

SECTION – I

Work Components & Guidelines

II.1 WORK COMPONENTS

EREC, IMD, New Delhi has taken up a programme of rigorous exercise for Geotechnical Characterization of NCT Delhi conducting High strain in-situ field tests including drilling and sampling, low strain field tests shear wave velocity measurement & GPR profiling geotechnical examination of disturbed (DS) and undisturbed (UDS) samples, Index and Dynamic Properties of Soil.

In reference to above, considering the requirements of precision level fixed for final map generation (1:10,000 Scale) and the problems and issues to be addressed, exploratory studies shall have to be conducted following state-of-the-art technology. EREC plans to conduct geotechnical and geophysical studies exercising field-testing at 500 locations in different domains of NCT, Delhi. According to the nature of work, the work components have been divided in four categories (I, II, III, IV) as given below and detailed in Tables II.1 – II.4.

Geotechnical investigation (Category I): boreholes up to 30 m depth, conducting SPT (with or without energy measuring device), collecting disturbed and undisturbed sampling, Spread over NCT and Routine laboratory tests for Index Properties and DCPT.

Geotechnical investigation (Category II): Deep boreholes depth varying up to 120 meters to bed rock depth (>300 m), conducting SPT with or without Energy measurement, collecting disturbed (DS from SPT) and undisturbed sampling (UDS), Routine laboratory tests for Index Properties and DCPT. Locations of boreholes however, spread in whole NCT, but mostly in North and Central Delhi area including Lutyens Delhi falling within Delhi triangle between western limit of ridge and Yamuna River, Chatterpur area and along domain of Delhi Ridge fill.
**Geotechnical investigations (Category III)** (OPTIONAL): Special laboratory test for dynamic soil property (OPTIONAL): Resonant Column test, Cyclic Triaxial tests on soil samples collected at selected locations of category A and B. These tests may be undertaken by EREC in collaboration with Government/autonomous institutions and if it is materialized, Samples are to be collected by bidder/bidders and delivered to designated agency. Rates for handling charges and testing are therefore quoted separately.

**Geophysical investigations (Category IV)** (OPTIONAL): Shear wave velocity using Designated Multichannel Analysis of Surface Waves method (MASW), Cross-hole Shear wave velocity test (CHT), Down-hole shear wave velocity (DHT), up to minimum 30m depth. Ground penetration Radar profiling

1.1 However, quotes are invited for all the above work components, but for some of the work component marked as OPTIONAL, EREC/IMD may opt to undertake in collaboration with government and autonomous institutions. In this case Special laboratory test (Category III) requires collection of suitable samples and arranges to deliver at specified laboratory; therefore, separate rates are to be quoted for handling and testing charges.

1.2 To accomplish the task of geotechnical characterization in stipulated time period, it is proposed to conduct studies simultaneously at least three sites/domains. Details of task to be carried out and **tentative quantum of work are** indicated Tables II.1 – II.4.

## II.2 DETAIL TASK AND TENTATIVE QUANTUM OF WORK

**Table II.1 (Category I):** Details of the tests to be conducted in NCT, Delhi and rates to be quoted by each bidder

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Description of the items</th>
<th>Quantum of Work</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Field test – drilling, sampling</td>
<td>Per meter depth considering varied site conditions likely to be encountered in Delhi (approx. 400 x 30 = 12000 Running meter - RM) (400 locations to be specified by EREC officials)</td>
<td>Water table is to be located if found within 30 m. If rock is encountered within 30 m then rock drilling has to continue at least 3 m within rock</td>
</tr>
<tr>
<td>1a</td>
<td>Mobilization of machinery and equipment, and drilling of boreholes - up to 30 m [IS:5313-1980] depth or up to 3 m in rock strata if encountered within 30 m depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1b</td>
<td>Conducting Standard Penetration Test [SPT] (as per IS:2131-1981) and collecting undisturbed samples (UDS) alternately at every 1.5 m interval or change of strata thereby having an SPT or a UDS at every</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------</td>
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<td>---------</td>
</tr>
<tr>
<td>1c</td>
<td>Same as <strong>Item 1a &amp; 1b</strong> along with SPT energy measurements at specified location</td>
<td>(Nos.) 40 locations</td>
<td>Among the 400 boreholes, 40 will be selected for energy measuring device</td>
</tr>
<tr>
<td>1d</td>
<td>Drilling and rock core sampling, Rock classification in terms of RQD [IS:11315 pt. 11-1985] up to 3 m of rock strata if encountered within 30 m depth</td>
<td>(150 RM)</td>
<td>Subject to the field conditions wherever Rock is encountered within 30 m depth</td>
</tr>
<tr>
<td>2</td>
<td>Disturbed sampling [DS] at 3.0 m interval from SPT sampler</td>
<td>For each borehole up to 30 m depth with DS @ 3.0 m intervals =&gt; 10 x 400 = 4000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Undisturbed sampling [UDS] (IS:2132-1986, IS:763-1978, IS:9640-1980, IS:10108-1982) at 3 m interval or at the changes of soil strata (Using appropriate soil samplers)</td>
<td>For each borehole up to 30 m depth with UDS @ 3 m intervals =&gt; 10 x 400 = 4000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>DCPT at or near to selected borehole locations up to depth of 30 m or refusal as per IS:4968 part-II (Dynamic Method using cone and Bentonite slurry)</td>
<td>400 locations</td>
<td>DCPT will be performed at or near to selected borehole locations</td>
</tr>
<tr>
<td>5</td>
<td>Excavation of Open test pit of 1.5 m x 1.5 m x 1.5 m depth or 3.0 m x 3.0 m x 3.0 m depth and thin walled sampling [IS:2132-1986] at the pit bottom followed by auguring up to 6 m / 4.5 m depth respectively along with disturbed sampling at 1.5 m interval</td>
<td>About 4 open pits per main BH location x 400 = 1600 (Nos.) Total length of auguring @ 6 m / 4.5 m per pit x 1600 = 9600/7200 rm DS @ 1.5 m interval =&gt; 4/3 per pit x 1600 = 6400/4800 (Nos.) Rate for each open pit should be quoted separately. The size of pit shall be decided by EREC Official</td>
<td>At least 4 open pit within 100 m distances from the master borehole. Exact no. of open pit and borehole drilling upto 6 m are subject to the</td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
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</tr>
<tr>
<td>6</td>
<td>Checking and recording water table if found within 30 m depth (IS:6935-1973) and within open pit up to (1.5 + 6 = 7.5 m depth)</td>
<td>400 + 4 x 400 = 2000 (Nos.)</td>
<td>availability of permissible open space surrounding each borehole location</td>
</tr>
<tr>
<td>B</td>
<td>Routine Laboratory tests on sample collected in 400 boreholes</td>
<td></td>
<td>Routine laboratory tests on soil samples</td>
</tr>
<tr>
<td>1a</td>
<td>Complete Grain size analysis up to clay size by Sieve and Hydrometer/ Pipette analysis (IS:2720 Part 4-1985)</td>
<td>DS (4000) + DS open pit (6400) + UDS (4000) = 14400 (Nos.)</td>
<td>Special laboratory tests such as Cyclic triaxial and Resonant column tests will be undertaken by EREC in collaborative mode through Government / Autonomous institutions at selected locations. Required samples are to be collected and delivered to designated laboratory / laboratories</td>
</tr>
<tr>
<td>1b</td>
<td>Grain size by laser analyzer (IS:2720 Part 4-1985)</td>
<td>About 100 representative samples as decided by EREC officials</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Atterberg limits (LL, PL, SL) (IS:2720 Part 5 &amp; 6 -1985)</td>
<td>DS (4000) + DS open pit (6400) + UDS (4000) = 14400 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Specific gravity, G (IS:2720 Part 3-1980)</td>
<td>2000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Natural water content, w% (IS:2720 Part 2-1973)</td>
<td>UDS – 4000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Bulk density, γ (UDS)</td>
<td>UDS – 4000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Relative density for Cohesionless Soil (IS:2720 Part 14-1983)</td>
<td>1000 (Nos.) Subject to the availability of sandy soils (only on samples from pits)</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Coefficient of consolidation, Cc (UDS)– for cohesive soils (IS:2720 Part XV-1965)</td>
<td>About 500 Nos. as decided by EREC official</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Direct Shear [DS/remolded] (IS:2720 Part 13-1986) on Sandy type soils</td>
<td>Min. 3 specimen per sample to be tested. Approximate no. of samples – 400. Representative samples will be scrutinized by EREC official, which are to be obtained from 400 borehole and 1600 open pit</td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------</td>
<td>----------------</td>
<td>---------</td>
</tr>
<tr>
<td>9a</td>
<td>Tri-axial shear (UDS) [Unconsolidated Undrained] / UCC (For cohesive material or where UDS is possible)</td>
<td>A min. of 3 specimens per sample shall be tested. Total number of Tri-axial tests shall be about 1500. The type of Tri-axial test (UU, CU or CD) to be conducted on each sample shall be decided by EREC Official based on type of soil encountered. Bidder may quote rates UCC and for each category of Tri-axial test</td>
<td></td>
</tr>
<tr>
<td>9b</td>
<td>(UDS) [Consolidated Undrained] with pore water pressure measurement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9c</td>
<td>(UDS) [Consolidated drained] (for sandy soil and some cohesive soil samples if considered necessary)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9d</td>
<td>Tri-axial shear (DS) [Consolidated Undrained]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9e</td>
<td>(DS) [Consolidated drained]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Tests on Rock samples from rock core at &lt;30 m depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Unconfined Compressive strength of Rock sample [IS:9143-1979]</td>
<td>50 (Nos.)</td>
<td>Subject to the site conditions found in 400 borehole locations</td>
</tr>
<tr>
<td>2</td>
<td>Laboratory determination of $V_p$ (Primary wave velocity), $V_s$ (Shear wave velocity) &amp; dynamic modulus of Rock core specimen [IS:10782-1983]</td>
<td>50 (Nos.)</td>
<td>——do——</td>
</tr>
<tr>
<td>3</td>
<td>Modulus of elasticity and Poisson’s ratio in uni-axial compression [IS:9221-1979]</td>
<td>50 (Nos.)</td>
<td>——do——</td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------</td>
<td>-----------------</td>
<td>---------</td>
</tr>
<tr>
<td>4</td>
<td>Point load strength index [IS:8764-1978]</td>
<td>50 (Nos.)</td>
<td>——do——</td>
</tr>
<tr>
<td>D</td>
<td>Reporting: Site details with Lat-Long /photograph/digital map, Physical Borelog (1:10), borelog chart containing equipment used, starting and completion date, N values, GL, GWT; laboratory and field test data, laboratory test results/plots/data interpretations, such as Plotting of grain size distribution, Dynamic Cone Penetration results, Preparation of Borelogs, Section/Fence Diagram along Boreholes, corrected SPT table alongwith the computation of average ‘N’ value, Analysis and Interpretation of test results of the sample tested</td>
<td>Borelog (1:10) at 20 specific site as per EREC directives. Requisite format for data presentation will be supplied by EREC official. Report is to be presented in both soft and hard copy</td>
<td></td>
</tr>
</tbody>
</table>

**Table II.2 (Category II):** Details of the tests to be conducted in Specific Areas of NCT, Delhi and rates to be quoted by each bidder

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Description of the items</th>
<th>Quantum of Work</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Field test – Deep drilling, sampling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1a</td>
<td>Mobilization of machinery, equipments and Deep drilling (i) boreholes up to 120 m (Approx. 50 Nos.), (ii) Drilling boreholes up to bed rock level (Approx. 50 Nos.) from existing ground level, for collecting disturbed (DS) and undisturbed sampling (UDS) as per IS:8763 IS-1978, IS:9640-1980, IS:10108-1982</td>
<td>Per run meter 100 boreholes at specific locations with average of 165 m (100 x 165) = 16500 m (Approx.) running meter (RM) drill length, locations to be specified by EREC officials</td>
<td>At some places bed rock may be &gt;300 m. At least 3 m has to be drilled within bed rock. Water table level is to be recorded if found</td>
</tr>
<tr>
<td>1b</td>
<td>Conducting Standard Penetration Test [SPT] (as per IS:2131-1981) and collecting undisturbed samples (UDS) alternately at every 1.5 m interval thereby having an SPT or</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Item No.</strong></td>
<td><strong>Description of the items</strong></td>
<td><strong>Quantum of Work</strong></td>
<td><strong>Remarks</strong></td>
</tr>
<tr>
<td>-------------</td>
<td>---------------------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>1a</td>
<td>a UDS at every 3.0 m intervals. Samples obtained from SPT shall be treated as Disturbed Samples (DS)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1c</td>
<td>Same as Item 1a &amp; 1b along with SPT energy measurements at specified locations and depths</td>
<td>Extra cost for energy measurements 10 location</td>
<td>Among the above 100 boreholes. Selected test locations will be specified by EREC officials</td>
</tr>
<tr>
<td>1d</td>
<td>Drilling and rock core sampling, Rock classification in terms of RQD, [IS:11315 pt. 11-1985] up to 3 m of rock strata if encountered within 30 m depth</td>
<td>(150 RM)</td>
<td>Subject to the field conditions wherever Rock will be encountered</td>
</tr>
<tr>
<td>2</td>
<td>Disturbed sampling [DS] at 3.0 m interval from SPT samples till drill depth.</td>
<td>For each borehole up to drill depth with DS @ 3.0 m intervals =&gt; 55 x 100 = 5500 Nos. (approx.)</td>
<td>(Average 165 m has been considered due to variable bedrock depth (at some location bedrock depth may be &gt;300 m)</td>
</tr>
<tr>
<td>3</td>
<td>Undisturbed sampling [UDS] (IS:2132-1986, IS:8763-1978, IS:9640-1980, IS:10108-1982) at 3 m interval or at the changes of soil strata till 30 m (using appropriate soil sampler)</td>
<td>UDS at every 3 m interval up to 30 m depth; 10 x 100 = 1000 Nos. (Approx.)</td>
<td>——do—-</td>
</tr>
<tr>
<td>4</td>
<td>Excavation of Open test pit of 1.5 m x 1.5 m x 1.5 m depth or 3.0 m x 3.0 m x 3.0 m depth and thin walled sampling [IS:2132-1986] at the pit bottom followed by auguring up to 6 m / 4.5 m depth respectively along with disturbed sampling at 1.5 m interval</td>
<td>About 4 open pits per main BH location x 100 = 400 (Nos.) Total length of auguring @ 6 m / 4.5 m per pit x 400 = 2400 rm / 1800 rm DS @ 1.5 m interval =&gt; 4/3 per pit x 400 = 1600/1200 (Nos.)</td>
<td>At least 4 open pit within 100 m distances from the master borehole. Exact no. of open pit and borehole drilling upto 6 m</td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------</td>
<td>----------------</td>
<td>---------</td>
</tr>
<tr>
<td>5</td>
<td>Checking and recording water table if found within 30 m depth (IS:6935-1973) and within open pit up to (1.5 + 6 = 7.5 m depth).</td>
<td>100 + 4 x 100 = 500 (Nos.)</td>
<td>are subject to the availability of permissible open space surrounding each borehole location</td>
</tr>
<tr>
<td>B</td>
<td>Routine Laboratory tests on samples collected in 100 boreholes</td>
<td>Routine Laboratory tests on soil samples</td>
<td></td>
</tr>
<tr>
<td>1a</td>
<td>Complete Grain size analysis by Sieve and Hydrometer/Pipette analysis (IS:2720 Part 4-1985)</td>
<td>DS (5500) +DS open pit (1600) + UDS (1000) = 8100 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>1b</td>
<td>Grain size by laser analyzer (IS:2720 Part 4-1985)</td>
<td>About 25 representative samples as decided by EREC officials</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Atterberg limits (LL, PL,SL)</td>
<td>DS (5500) + DS open pit (1600) + UDS (1000) = 8100 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Specific gravity, G (IS:2720 Part 3-1980)</td>
<td>500 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Natural water content, w% (IS:2720 Part 2-1973)</td>
<td>UDS – 1000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Bulk density, $\gamma$ (UDS)</td>
<td>UDS – 1000 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Relative density (UDS-Cohesion-less Soil) (IS:2720 Part 14-1983)</td>
<td>250 (Nos.) Subject to the availability of sandy soils</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Coefficient of consolidation, Cc (UDS)</td>
<td>About 100 Nos. as decided by EREC official</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Direct Shear [DS/remolded] (IS:2720 Part 13-1986) on Sandy type soils</td>
<td>Min. 3 specimen per sample to be tested. Approximate no. of samples – 100. Representative samples will be scrutinized by EREC official, which are to be obtained from 100 borehole and 400 open pit locations. Exact No. of samples are subject to the availability of permissible open space surrounding each borehole location</td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>to the quality field sampling and their successful operation with the sophisticated laboratory equipment</td>
<td></td>
</tr>
<tr>
<td>9a</td>
<td>Tri-axial shear (UDS) [Unconsolidated Undrained] / UCC</td>
<td>A min. of 3 specimens per sample shall be tested. Total number of Tri-axial tests shall be about 400. The type of Tri-axial test (UU, CU or CD) to be conducted on each sample shall be decided by EREC Official based on type of soil encountered. Bidder may quote rates UCC and for each category of Tri-axial test</td>
<td></td>
</tr>
<tr>
<td>9b</td>
<td>Tri-axial shear (UDS) [Consolidated Undrained with pore water pressure measurements]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9c</td>
<td>Tri-axial shear (UDS) [Consolidated drained]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9d</td>
<td>Tri-axial shear (DS) [Consolidated Undrained with pore water pressure measurements]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9e</td>
<td>Tri-axial shear (DS) [Consolidated Drained]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Tests on Rock samples from rock core at &lt;30 m depth</td>
<td>Routine Laboratory tests on rocks</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Unconfined Compressive strength of Rock sample [IS:9143-1979]</td>
<td>50 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Laboratory determination of dynamic modulus of Rock core specimen [IS:10782-1983]</td>
<td>50 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Modulus of elasticity and Poisson’s ratio in uni-axial compression [IS:9221-1979]</td>
<td>50 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Point load strength index [IS:8764-1978]</td>
<td>50 (Nos.)</td>
<td></td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
<td>------------------------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>D</td>
<td>Reporting: Site details with Lat-Long/photograph/digital map, Physical Borelog (1:10), borelog chart containing equipment used, starting and completion date, N values, GL, GWT; laboratory and field test data, laboratory test results/plots/data interpretations, such as Plotting of grain size distribution, Dynamic Cone Penetration results, Preparation of Borelogs, Section/ Fence Diagram along Boreholes, corrected SPT table alongwith the computation of average ‘N’ value, Analysis and Interpretation of test results of the sample tested.</td>
<td></td>
<td>Borelog (1:10) at 50 specific site as per EREC directives. Requisite format for data presentation will be supplied by EREC official. Report is to be presented in both soft and hard copy.</td>
</tr>
</tbody>
</table>

Table II.3 (Category III): Special laboratory tests on representative samples (OPTIONAL)

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Description of the items</th>
<th>Quantum of Work</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Cyclic Tri-axial test [UDS/DS] (standard sinusoidal pulse/ signature)</td>
<td>Min. 3 specimens per sample and approx. 50 samples (Handling and testing charges should be quoted separately)</td>
<td>This test is proposed to be EREC in undertaken by collaboration with Government/ Autonomous agencies. Field samples are to be collected by bidder/ bidders and delivered to designated agencies.</td>
</tr>
<tr>
<td>1b</td>
<td>Cyclic Tri-axial test [UDS/DS] (using Bhuj (2001)/ Chamoli (1999) Earthquake signatures)</td>
<td>Same as above (Nos.)</td>
<td>Same as above</td>
</tr>
<tr>
<td>Item No.</td>
<td>Description of the items</td>
<td>Quantum of Work</td>
<td>Remarks</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------</td>
<td>----------------</td>
<td>---------</td>
</tr>
<tr>
<td>2</td>
<td>Resonant column test - RCT</td>
<td>Same as above (Nos.)</td>
<td>Same as above</td>
</tr>
</tbody>
</table>

**Table II.4 (Category D): Geophysical tests at specific sites (OPTIONAL)**

<table>
<thead>
<tr>
<th>Description of work</th>
<th>Quantum of work (Tentative)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Penetrating Radar - GPR</td>
<td>50 points (length of profile lines ~100 m)</td>
<td>Exact no., location, type and nature of test will be decided by EREC officials</td>
</tr>
<tr>
<td>Cross-hole shear wave velocity test – CHT with min 3 holes to a depth of 30 m. $V_s$ readings required @ 1.5 m interval or change of strata</td>
<td>25 points</td>
<td></td>
</tr>
<tr>
<td>Down-hole Shear wave velocity test – DHT up to 30 m @ 1.5 m or change of strata</td>
<td>25 points</td>
<td></td>
</tr>
<tr>
<td>Multiple Spectral Analysis of Surface Waves - MASW</td>
<td>2000 RM</td>
<td></td>
</tr>
</tbody>
</table>

### II.3 GUIDELINES TO BE FOLLOWED DURING FIELD OPERATIONS

Borehole drilling at 500 locations in the entire NCT area of Delhi (excepting rock outcrop) down to 30 m depth from existing ground level and conducting Standard Penetration Test (SPT) at 1.5 m intervals or at the changes of strata (IS:2131-1981). The standard penetration test (SPT) is performed during the advancement of a soil boring to obtain an approximate measure of the dynamic soil resistance, as well as a disturbed drive sample (split barrel type). DCPT will also be performed at or near to all borehole locations. The details of the field tests are given in Tables II.1 to II.4. The following conditions are to be noted while carrying out field investigations:

i. Mobilization of drilling equipment, machineries etc. suitting site conditions will have to be done alongwith drilling as running meter.

ii. In the same borehole DS, UDS and SPT (with/without energy measuring devices) will have be conducted as per IS code. The vendor will have to prepare borelogs for at least 50 selected boreholes at 1:10 scale. Data sheet for preparing borelogs for each of 500 borehole locations are given in IS:2131-1985. EREC official will make necessary changes in the borelog format suitting the same for incorporation in GIS format.

iii. SPT equipment need to be calibrated prior to under taking work from the agency specified by EREC/IMD.
iv. Wherever rocky strata is encountered within 30 m depth, boring will have to be made at least 3 m within the rock strata along with rock core sampling followed by laboratory tests for Designated Scientific classification (IS:11315 pt. 11-1985).

v. Collecting disturbed (DS) and undisturbed (UDS) samples in all 500 boreholes as per IS:2132-1972, which after relevant laboratory tests and visual examinations, will be used for the Designated Scientific classification of the soil.

vi. In some of the selected sites the vendor(s) has/have to demonstrate the SPT energy transfer efficiency (%) using suitable devices to measure hammer energy, which is being transferred through the SPT assembly. To validate the energy transfer efficiency (%) data, it will be required to perform such tests at 3 m intervals along the entire 30 m depth of a particular borehole and also in several boreholes at different depths. Tests results may vary at locations having varied soil strata and also it may vary due to the type of equipment used. These tests are required to confirm the reliability of vendor’s SPT equipment.

vii. There may be some unavoidable or unforeseen circumstances in field works wherein operations such as SPT, DS, UDS, rock boring etc. might go beyond control. In some cases extraction of the samples from samples might go wrong in the laboratory. It may also happen that results may not be acceptable due to poor quality samples/samplers. Under such situations (if any) the total no. of field and laboratory tests in each or all the boreholes are subject to the site conditions encountered during actual operations. However, any vital operation/information if not provided or obtained by the vendor during field operation, it will be considered as a lapse on the part of vendor. The vendor shall be required to repeat some of the vital field/laboratory tests wherever found necessary by EREC at Vendor’s own cost.

viii. Due to various unforeseen situations met during field test it may not be possible to obtain undisturbed samples at all specified locations as per IS code. It may also happen that at some particular site only clayey (where sieve analysis is not required) or sandy soils (where hydrometer analysis is not required) are sampled. In such cases, no. of tests actually performed will be counted for payment as per unit rate. Failure to provide any reliable sample or tests data either in the field or laboratory will be considered as lapse on the vendor’s part. In vital cases, if relevant samples or information or data are missing or failed, then the vendor will be held responsible. Repeating of some of the tests (field or laboratory) by vendor will be at the discretion of EREC.

ix. At each borehole location, 4-6 open pits around the selected borehole shall have to be made. Exact no. of open pits will be decided after inspection of the site. Open drive sampler in 4 thin walled tubes (IS:9680-1980) and disturbed samples from the pit bottom (1.5 m depth) have to be collected for laboratory test (such as grain size, Atterberg limit, bulk density, moisture content, triaxial shear tests etc.).
x. In-situ density at each pit using Sand-replacement and mini-penetrometer devices. Disturbed samples to be collected at 1m intervals up to a depth of 5 m from the open pit bottom using hand or motorized auger.

II.4 SUBMISSION OF THE REPORT

Upon completion of the field investigation and laboratory-testing program, the vendor, in consultation with EREC official will compile, evaluate, and interpret the data and represent all data in GIS platform. The specifications for the GIS applications will be made available by EREC officials in due course of time. Additionally, the vendor will be responsible for producing a report that presents the subsurface information obtained from the site investigations and provides specific technical recommendations. The need for multiple types of reports on a single project depends on the project size, phasing and complexity.

Report shall include:

(a) Borelogs, site plan alongwith Latitude, Longitude, lithological section of borehole, local site geology, field and lab test data in a prescribed tabular format alongwith all graphical interpretations, supporting calculations, figures, formula, practical and theoretical considerations/documents for the interpretation of tests results.

(b) All laboratory test results in a suitable software format for checking and reproduction in other desired interpretation by the EREC.

(c) On completion of all the field and laboratory work, the contractor shall submit a draft report containing all field and laboratory data and their useful interpretation, summarized test data, graphs, chart, conclusion and recommendations.

(d) Contractor’s qualified Geotechnical Designated Scientist/specialist shall visit EREC office and vice-versa for detailed discussion on the draft report.
SECTION – II

TECHNICAL SPECIFICATIONS

BROAD OUTLINE OF GROUND INVESTIGATIONS IN NCT, DELHI

ERECE has evolved a comprehensive ground investigation program in the NCT area (about 1485 sq km) of Delhi for collection of seminal data set on:

I. **Geotechnical characterization** of sub-soil/rock up to 30 m depth with an objective of identifying Liquefaction Potential/susceptibility, earthquake induced ground settlement and recommending mitigation measures for safe built environment and devising retrofitting measures for the important structures by resorting to:

1. **Field tests (SPT - Standard Penetration Test, DCPT - Dynamic Cone Penetration Test):**
   
   to know the penetration resistance of the granular and silty soil depending on its consistency, density and cementing bond due to ageing effect, overburden pressure and depth of water table. These data will be used for obtaining liquefaction potential of sub-soil of NCT, Delhi, for which following parameters are also required:

   i. Earthquake magnitude, source distance, MSF (Magnitude Scaling Factor), PGA (Peak Ground Acceleration) at base rock and Spectral acceleration amplification factor in case bed rock is underlain by soft sediment cover. This data will be obtained separately.

   ii. Designated Scientific soil classification type.

   iii. Borelog data up to 30 m depth and identification of liquefying layer, if any.

   iv. In-situ Density and initial overburden pressure. These are either obtained from correlations with SPT-N or directly from the undisturbed samples collected at specific depth within the boreholes.

   v. Water table depth. The same will be checked with CGWB data wherever feasible (IS:6935-1973).

   vi. Grain size distribution and amount of fines. These are obtained from routine laboratory tests on disturbed samples, which are usually collected at 1.5 m intervals or at the changes of strata.

   vii. SPT (IS:2131-1985)/ DCPT (IS:4968-1976 Pt. I & II) count number at different depth. Any one of SPT/DCPT data is enough for liquefaction calculation. However, both these test will be conducted to avoid ambiguity in results and also to workout empirical relation, so that any one test can be used in future to cover more no. of locations for better representation of sub-soil profile.
viii. Dynamic Cone Penetration Test (DCPT – IS:4968-1976 Pt. I & II) is quick test and it gives a continuous record of the penetration resistance of the soil with depth. This test uses same monkey weight and height of fall as used in SPT and test results help to understand the uniformity or variability in the subsoil profile which very useful in the preliminary exploration for extensive sites. The blow count for 75 mm penetration is recorded.

Note: Based on the 2450 borehole data collected from different agencies and 1st level microzonation study made by EREC and subsequently on the basis of reconnaissance survey in the NCT area of Delhi, it is decided to carry out soil investigation at 500 locations. In all these locations, SPT/DCPT and proper sampling will have to be made as per relevant IS codes (IS:8763-1978, IS:9640-1980, IS:10108-1982). At locations where rock strata will be encountered within 30 m depths, rock coring shall be continued at least up to two runs or 3 m within the rock. Brief outline of major in-situ tests are given as under.

2. Exploration open pit: At some borehole locations, subject to availability of space and depending the variation in the subsoil profile encountered up to 30 m depth 4 to 6 open pits around the borehole shall be made. Exact no. of open pits will be decided after inspection of the site. Open drive sampler in 4 thin walled tubes (IS:9680-1980) and disturbed samples from the pit bottom (1.5 m depth) have to be collected for laboratory test (such as grain size, Atterberg limit, bulk density, moisture content, tri-axial shear tests etc.). In-situ density at each pit has to be obtained by conducting Sand-replacement, core cutter and mini-penetrometer devices.

Note: The above exercise will help mapping of sub-soil profile within 100 m range of each of 500 borehole location. Sub-soil profile obtained in above manner will provide complete information, which is otherwise not possible with 500 boreholes in the entire NCT area of Delhi.

3. Laboratory test on disturbed (DS), undisturbed (UDS) and remolded (RM) samples: Grain size, plasticity, coefficient of consolidation, strength test such as Direct shear, Tri-axial shear test – Unconsolidated Undrained (UU), Consolidated Undrained (CU), Consolidated Drained (CD).

i) Dynamic test: Resonant column - To determine the shear modulus \((G_{max} \text{ or } G_0)\) and damping \((D)\) characteristics of soils at small strains for cases where dynamic forces are involved, particularly seismic ground amplification and machinery foundations. Recent research has shown the results are also applicable to static loading at very small strains \(<10^{-6} \text{ percent}\).

ii) Cyclic tri-axial test - To check shear strength of the soil under cyclic loading and also to check no. of cycles required to cause liquefaction under recreated field condition, Cyclic tri-axial tests are used for projects with repeated and/or cyclic loading, resilient modulus determinations, and/or liquefaction analysis of soils. In each of these tests, the specimen is initially consolidated to the effective vertical overburden stress \((\sigma_{vo}')\) prior to shear.
Note: With limitations in the sampling system and boring techniques used in our country it might not be possible to extract undisturbed sample at 3 m intervals from all of 500 locations as per IS code. Not all undisturbed samples are really undisturbed. There is every possibility that proper extraction of the sample for cyclic tri-axial test may fail. Under so many uncertainties, only few samples may be possible to be tested in cyclic tri-axial system. Therefore, exact no. of laboratory test will be decided after eliminating all errors encountered in the process.

4. Physical model test to experience Liquefaction phenomena in the laboratory: Shock test on recreated Liquefying soil model tank; Shaking table test on model building/walls subjected to synthetic/recorded earthquake signature.

5. Numerical modeling using FLAC-3D/ANSYS CIVIL 3D/PLAXIS to check occurrence of liquefaction, post-liquefaction settlement ground settlement due to strong motion, lateral spreading. Analyzing building performance resting on known sub-soil strata by taking into consideration of dynamic soil-structure interaction and devising methodology for retrofitting measures of important buildings/monuments of Delhi.

Note: Item 4 & 5 as above: these tests do not come under the purview of soil investigation. They are mentioned here for the sake of explaining the goal of seismic hazard investigations. As a matter of fact, relevant ground data will be fitted into the software for the calculation of earthquake-induced ground settlement.

II. Generate geophysical parameters by conducting GPR – Ground Penetrating Radar, ER – Electrical resistivity, CHT – Cross-hole Test, DHT – Down-hole Test, SASW – Spectral Acceleration of Surface Wave, MASW – Multi-channel Analysis of Surface Waves (accelerated hammer type) to obtain base rock/soil profile and Shear wave velocity of subsoil. Salient features of some of the important geophysical tests are given below:

i. The GPR surveys provide a quick imaging of the subsurface conditions, leaving everything virtually unchanged and undisturbed. This can be a valuable tool used to define subsoil strata, underground tanks, buried pipes, cables, as well as to characterize archaeological sites before soil borings, probes, or excavation operations. It can also be utilized to map reinforcing steel in concrete decks, floors, and walls. The GPR surveys are particularly successful in deposits of dry sands with depths of penetration up to 20 m or more, whereas in wet saturated clays, GPR is limited to shallow depths of only 3 to 6 meters.

ii. The cross-hole testing (CHT) involves the use of a down-hole hammer and one or more down-hole vertical geophones in an horizontal array of two or three boreholes spaced about 3 to 6 meters apart to determine the travel times of waves in different strata. The boreholes are most often cased with plastic pipe and grouted in place. After setup and curing of the grout, the borehole verticality must be checked with an inclinometer to determine changes in horizontal distances with depth, particularly if the investigations extend to depths exceeding 15 m. Special
care must be exercised during testing to ensure good coupling of the geophone receivers with the surrounding soil medium. Usually, inflatable packers or spring-loaded clamps are employed to couple the geophone to the sides of the plastic casing. A special down-hole hammer is preferably used to generate a vertically-polarized horizontally-propagating shear wave. An up strike generates a wave that is a mirror image of a down strike wave. The test is advantageous in that it may be conducted to great depths of up to 300 meters or more.

iii. Down-hole surveys (DHT) can be performed using only one cased borehole. Here, S-waves are propagated down to the geophone from a stationary surface point. No inclinometer survey is needed as the vertical path distance (R) is calculated strongly on depth. In the DHT, a horizontal plank at the surface is statically loaded by a vehicle wheel (to increase normal stress) and struck lengthwise to provide an excellent shear wave source. The orientation of the axis of the down-hole geophone must be parallel with the horizontal plank (because shear waves are polarized and directional). The results are paired for successive events (generally at 1 m depth intervals) and the corresponding shear wave at mid-interval is calculated as \( V_s = \frac{DR}{Dt} \), where \( DR \) = the hypotenuse distance from plank to geophone and \( Dt \) = arrival time of the shear wave. A recent version of the down-hole method is the Seismic Cone Penetration Test (SCPT) with an accelerometer located within the cone of penetrometer. In this manner, no borehole is needed beforehand.

iv. The Multi-channel Analysis of Surface Waves (MASW) method is a nondestructive seismic method to evaluate linear elastic modulus of underground materials. It analyzes dispersion properties of certain types of seismic surface waves (fundamental-mode Rayleigh waves) propagating horizontally along the surface of measurement directly from impact point to receivers. It gives this shear-wave velocity \( (V_s) \) (or stiffness) information in either 1D (depth) or 2D (depth and surface location) format in a cost-effective and time-efficient manner. The main advantage with the MASW method is to take a full account of the complicated nature of seismic waves that always contain harmful noise waves such as higher modes of surface waves, body waves, scattered waves, traffic waves etc. These noise waves may result in a significant portion of the recorded data being dubious if not properly accounted for. The fundamental framework of the MASW method is based on the multi-channel recording and analysis approach long used in seismic exploration surveys. These techniques can discriminate useful signal against all other types of noise by utilizing pattern-recognition techniques. Due to multi-channel recording and processing schemes employed, results \( (V_s) \) information of the survey are highly reliable even under the presence of higher modes of surface waves and various types of cultural noise. For the same reason, the processing steps can be fully automated. Therefore, the method is extremely easy and fast to implement.

Note: Not all tests will be performed at same location. However, No. & type of tests and their strategic locations will be decided after or during field investigations at 500 prime borehole locations. Some of these tests would be required to check SPT-N data and some will be performed at
locations other than those of SPT boreholes. The reason for conducting several tests at specific locations; is to obtain complete mapping of sub-soil profile of Delhi. There are several correlations in literatures, which help obtaining one parameter from different tests and vice-versa.

III. Profiling of Geometry of the **Chattarpur basin** area and Delhi ridge boundary by exploration up to bed rock depth numerical modeling of response pattern on earthquake shaking. Exploration depth may vary between 5 m to >300 m. About fifty borehole locations will be selected in the Chattarpur basin area with provision for undisturbed sampling up to top 30 m depth and disturbed sampling at 1.5 m c/c till bed rock depth. Details of the tests are given in (Table II.2, Category II). Laboratory testing of samples collected from above explorations for different parameters viz. Grain size analysis; Atterberg Limit tests, Specific gravity, natural moisture content, permeability, Consolidation test Direct shear and Triaxial shear test, resonant column test with simulated dynamic loading, for ascertaining frequency dependent damping ratio and other dynamic soil properties. About fifty boreholes (Table II.2, Category II) will also be conducted up to the depth of 120 meters in central Delhi.
A few Typical Case Studies on Site Effects

III.1 Estimation of S-Wave Site Response in and around Delhi Region from Weak Motion Data (Nath et al., 2003)

The Delhi ridge, which is the northernmost extension of the Aravalli Mountains, consists of Alwar quartzite rocks belonging to Pre-Cambrian age and extending from southern parts of the territory to western bank of Yamuna for about 35 km. The alluvial formations overlying the quartzitic bedrock have different nature on either side of the ridge. The Yamuna flood plains contain a distinct river deposit. The nearly closed Chattarpur alluvial basin covering an area of about 48 sq km is occupied by alluvium derived from the adjacent quartzite ridge.

The exploratory drilling undertaken by various agencies has brought out the subsurface configuration of rock formation and depth to bedrock in different parts of Delhi region. The nature of bedrock topography is rendered uneven due to the existence of subsurface ridges. The thickness of alluvium overlying the quartzites increases away from the outcrops.

Data Processing and Site Response Study in Delhi Region

India Meteorological Department (IMD) is operating a network of 13 stations of VSAT based digital telemetry systems around Delhi as shown on the base map in Figure III.1. Ridge observatory has a 3-component, broadband (CMG-40T) system with a 24-bit digitizer at 20 samples per second. The other telemetry systems have 3-component sensors (S-13, short period) with 24-bit digitizer at 50 samples per second. Nine local ($M_L \geq 2.3$) and nine regional ($M_L \geq 3.9$) events enlisted in (Table III.1) are used for the proposed site amplification study in and around Delhi. Figure III.1 also depicts the epicentral location of all the local events.

Table III.1: Local and Regional Events used in the Site Amplification Factor Estimation

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Date</th>
<th>Origin Time</th>
<th>Epicenter</th>
<th>Magnitude $M_L$</th>
<th>Region</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>05.11.2000</td>
<td>04:05:01.9</td>
<td>28.72ºN/76.73ºE</td>
<td>2.3</td>
<td>Haryana</td>
</tr>
</tbody>
</table>
### Regional Events

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Date</th>
<th>Origin Time</th>
<th>Epicenter</th>
<th>Magnitude $M_L$</th>
<th>Region</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>26.01.2001</td>
<td>03:16:42.9</td>
<td>23.40N/70.28E</td>
<td>$M_L=6.9$</td>
<td>Rann of Kuchch, Gujrat</td>
</tr>
<tr>
<td>2.</td>
<td>26.01.2001</td>
<td>03:33:35.3</td>
<td>23.40N/70.18E</td>
<td>$M_L=5.5$</td>
<td>Rann of Kutch, Gujrat</td>
</tr>
<tr>
<td>3.</td>
<td>25.02.2001</td>
<td>02:21:37.0</td>
<td>36.50N/70.60E</td>
<td>$M_b=6.7$</td>
<td>Afghanistan-Tazikistan Border</td>
</tr>
<tr>
<td>4.</td>
<td>15:04:2001</td>
<td>08:36:38.0</td>
<td>29.00N/81.40E</td>
<td>$M_L=3.9$</td>
<td>Nepal</td>
</tr>
<tr>
<td>5.</td>
<td>28.04.2001</td>
<td>10:37:58.4</td>
<td>28.40N/87.10E</td>
<td>$M_b=5.2$</td>
<td>Nepal-Tibet Border</td>
</tr>
<tr>
<td>6.</td>
<td>16.05.2001</td>
<td>11:00:08.0</td>
<td>30.70N/68.90E</td>
<td>$M_b=5.0$</td>
<td>Pakistan Near Quetta</td>
</tr>
<tr>
<td>7.</td>
<td>30.06.2001</td>
<td>11:48:37.0</td>
<td>30.97N/69.95E</td>
<td>$M_b=5.0$</td>
<td>Pakistan</td>
</tr>
<tr>
<td>8.</td>
<td>16.07.2001</td>
<td>16:07:18.0</td>
<td>32.80N/72.90E</td>
<td>$M_L=5.0$</td>
<td>Pakistan</td>
</tr>
<tr>
<td>9.</td>
<td>16.07.2001</td>
<td>16:12:48.2</td>
<td>28.13N/84.72E</td>
<td>$M_L=5.4$</td>
<td>Nepal</td>
</tr>
</tbody>
</table>

In this study, the results obtained on Site Amplification Factor (SAF) in and around Delhi are estimated by classical standard spectral ratio for nine regional events (Ridge Delhi [NDI] Observatory being the reference station), normalized standard spectral ratio for 3 local events, receiver function and generalized inversion for nine local and nine regional events in the frequency range of 0.5 to 7.5 Hz. In the classical spectral ratio method, the source and path effects are considered identical for every site and the reference one only when the epicentral distance is appreciably larger compared to the inter-station spacing. As a result the normalized SSR method is expected to perform better for the regional events. In the present context the standard spectral ratio technique is applied for SAF estimation using 9 regional events only. For the local events, however, a normalized SSR technique is evolved for the earthquakes with the network azimuthal coverage of $270^\circ$ or more. For each central event, the station spectral amplitudes are normalized to a 50 km epicentral distance. This gives weighted path effect calibrated spectral amplitude.
at every station to a 50 km epicentral distance. Referring to Figure III.1, the events 2, 6 and 9 are selected for the normalized SSR computation in the present study. Considering the local sources to be random, the multiple event normalized amplitude spectra at each station are stacked for the elimination of source effect. The spectral ratio is then determined from the stacked amplitude with respect to the reference site (NDI here). We considered the shear-wave velocity $\beta = 4.06$ km/sec as an average one on the velocity models for the Garhwal Himalaya assuming $\frac{\alpha}{\beta} = 1.73$ (IMD, 2000), where $\alpha$ represents the P-wave velocity.

![Figure III.1](image_url)

**Figure III.1**: Telemetry network and local events on the base map of greater Delhi.

The filtered seismograms at the stations Sohna (SONA), Ridge New Delhi (NDI) and Kurukshetra (KKR) are presented in Figures III.2(a), (c) and (e) for the local event 05112000 ($M_L = 2.3$) together with the Fourier spectra of the respective S-envelope in Figures III.2(b), (d) and (f) in log-log scale.
Figure III.2: The local event 05112000 ($M_L = 2.3$) as recorded by the sites SONA (EW component), NDI (EW component) and KKR (Vertical component) and the corresponding Fourier spectra of the S-wave envelopes.
We have treated $Q_s$ as predetermined (Nath et al., 2003) in the inversion procedure. The relation used is,

$$Q_s(f) = 58.79f \pm 6.72$$  \hspace{1cm} \text{(III.1)}

As displayed in Figure III.3 the Equation (III.1) has been obtained by the regression analysis on the S-wave envelope of the weak motion data of the local and regional events used in this study.

![Regression analysis for the Q-factor estimation from the direct S-wave envelope.](image)

**Figure III.3:** Regression analysis for the Q-factor estimation from the direct S-wave envelope.

All the curves in Figures III.4(a) and (b) for the horizontal and vertical components exhibit a minimum of 0.32 at 0.5 Hz and 1.5 at 7.5 Hz, a mild departure from the mean value 1.0, the expected site amplification for a competent bedrock site. As a result NDI can be used as the reference site for the SAF determination at all the stations of the network.

The representative site response curves at KALG, SONA and AGRA are presented in Figures III.5 and III.6(a) and (b) respectively. A detailed analysis for KALG is presented in Figure III.5, wherein the SAF curves by GINV and SSR for the horizontal and vertical component seismograms of regional events, HVSR for local and regional earthquakes are displayed together with the data scatter.

It is evident from the diagrams that GINV estimated horizontal component SAF from the regional events follow those using the local events (Figures III.5(a) and (b)).
Figure III.4: (a) Site amplification factor at New Delhi (NDI) estimated by generalized inversion (GINV) and receiver function (HVSIR) from the horizontal amplitude spectra of the seismograms recorded for the regional and local earthquakes. The solid straight line parallel to frequency axis at SAF 1.0 represents spectral amplification for a reference site, (b) Vertical spectral amplification by GINV for regional and local events at New Delhi (NDI).
Figure III.5: Site amplification factor at Kalagarh (KALG) by (a) Generalized inversion (GINV) of the horizontal and vertical amplitude spectra of the seismograms for the local and regional events, (b) SAF value scatter with frequency for the horizontal spectra by GINV for local and regional events. The shaded areas represent ± one standard deviation (SD) from the mean, (c) Standard spectral ratio estimate (SSR) for the regional events and the normalized spectral ratio estimate for local events using horizontal and vertical amplitude spectra, (d) SAF value scatter with frequency for the horizontal spectra by SSR for local and regional events. The shaded areas represent ± one standard deviation (SD) from the mean, (e) The receiver function estimate (HVSR) of the site amplification factor for the local and regional events, (f) The scatter of site amplification factor by HVSR with frequency for the local and regional events. The vertical components also show similar trend for both the local and regional events.
The vertical components also show similar trend for both the local and regional events (Figure III.5(a)), with reduced amplification compared to the horizontal ones. The GINV estimated SAF for the local and regional earthquakes exhibit mild scatter as depicted in Figure III.5(b), the shades indicating ± one standard deviation from the mean. The SSR estimated site response values at KALG from the horizontal component seismograms of the regional events presented in Figure III.5(c) closely follow those by the normalized SSR using horizontal component seismograms of the local events 2, 6 and 9 (Table III.1). The vertical spectral amplification by SSR method from the local and regional events show a trend following the horizontal ones but with defused amplification. The site response values from regional events by standard spectral ratio and local events by normalized standard spectral ratio also exhibit a mild scatter as depicted in Figure III.5(d) with ± one standard deviation from the mean. The HVSR computed SAF from the regional and local events together with the shaded scatter for ± one standard deviation displayed in Figures III.5(e) and (f) follow the trend of the curves by GINV and SSR. The predominant frequency at Kalagarh is 5 Hz. Similar analysis is performed at all the sites before calculating the average SAF values. Incidentally SONA and AGRA recorded only local events. Their site amplification by generalized inversion and normalized standard spectral ratio from the horizontal and vertical component amplitude spectra and receiver function are presented in Figures III.6(a) and (b) respectively. SONA has a predominant frequency of 3.5 Hz while at AGRA it is 1 Hz. The GINV and SSR curves exhibit a good similarity being in the close proximity of each other. The vertical spectral amplification by GINV and SSR is appreciably less compared to the horizontal ones. HVSR generated SAF curves at both SONA and AGRA seem to be following the GINV estimated horizontal amplification curves as well.

![Figure III.6: SSR, GINV, HVSR estimates of the site response for the local events at (a) SONA and (b) AGRA.](image-url)

The GINV computed site amplification factors for the horizontal component spectra at the central frequencies 1.5, 4 and 6.5 Hz at all the 13 stations are enlisted in (Table III.2). Unless otherwise mentioned, the site amplification SAF will mean those computed using SSR (standard or normalized) and GINV from horizontal component amplitude spectra only.
Table III.2: Comparison of SAF values at 13 sites in and around Delhi at the central frequencies 1.5, 4 and 6.5 Hz as estimated by GINV

<table>
<thead>
<tr>
<th>Stations</th>
<th>1.5 Hz (0.5 -2.5 Hz)</th>
<th>4 Hz (3-5 Hz)</th>
<th>6.5 Hz (5.5-7.5 Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SONA</td>
<td>2.28</td>
<td>3.18</td>
<td>2.48</td>
</tr>
<tr>
<td>KALG</td>
<td>4.65</td>
<td>4.68</td>
<td>2.81</td>
</tr>
<tr>
<td>AGRA</td>
<td>5.17</td>
<td>4.78</td>
<td>3.21</td>
</tr>
<tr>
<td>AYAN</td>
<td>1.88</td>
<td>1.68</td>
<td>2.56</td>
</tr>
<tr>
<td>KKR</td>
<td>2.93</td>
<td>4.26</td>
<td>6.44</td>
</tr>
<tr>
<td>ASR</td>
<td>2.31</td>
<td>3.44</td>
<td>6.51</td>
</tr>
<tr>
<td>RTK</td>
<td>5.65</td>
<td>6.09</td>
<td>1.67</td>
</tr>
<tr>
<td>BIS</td>
<td>3.22</td>
<td>2.84</td>
<td>4.52</td>
</tr>
<tr>
<td>BHGR</td>
<td>1.96</td>
<td>2.86</td>
<td>5.52</td>
</tr>
<tr>
<td>UGON</td>
<td>0.53</td>
<td>0.58</td>
<td>1.51</td>
</tr>
<tr>
<td>RTUL</td>
<td>1.89</td>
<td>2.21</td>
<td>3.31</td>
</tr>
<tr>
<td>KUDL</td>
<td>3.76</td>
<td>4.51</td>
<td>2.62</td>
</tr>
<tr>
<td>NDI</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Results and Discussions

In the greater Delhi and its surroundings, all the stations (except UGON) exhibit site amplification except NDI at all frequencies. As discussed in the preceding section the site amplification values estimated by standard spectral ratio for the regional events, normalized standard spectral ratio for the local events, generalized inversion and receiver function are highly comparable. The horizontal component spectral amplification is more pronounced than the vertical ones as evident in Figures III.5 and III.6 for the sites KALG, SONA and AGRA. This is true for all the sites. The variation of the vertical site amplification factor with frequency indicates that all the components are influenced by the near-surface inhomogeneity in soil conditions. The site amplification at KALG at the resonance frequency (5 Hz) is 6.45, while at SONA it is 5.75 at 3.5 Hz. At AGRA the SAF value at the resonance frequency 1 Hz is 6.47. The GINV estimated vertical spectral amplification (Figure III.6(b)) show a peak at 1.5 Hz while the resonance frequency of the basin is 1 Hz (SSR, HVSR, GINV_H all peaking at 1 Hz). This may be attributed to a local optimisation
problem associated with the generalized inversion scheme and hence a numerical anomaly. The scatter plot between the site amplification factors computed by SSR and GINV as depicted in Figure III.7(a) shows a good 1:1 correspondence. Since the vertical components are not free from the near-surface influences, the HVSR estimated site amplification factors show a large scatter with respect to the GINV estimated site response as shown in Figure III.7(b).

![Graph showing comparison of site amplification factors](image)

**Figure III.7:** The comparison of site amplification factor calculated by GINV versus those obtained from (a) the SSR method, and (b) the HVSR technique. The diagonal line indicates 1:1 correspondence.

Because detailed geological, geotechnical, and geophysical information relevant to the soil column underlying the considered stations are not available, we did not attempt to correlate soil characteristics with calculated site response, but make a subjective interpretation based on the values yielded by generalised inversion technique. As listed in (Table III.2), at 1.5 Hz an appreciable site response ($\approx 5.17$) is computed at the station AGRA, while KALG exhibited a value of 4.65. AGRA again shows higher response ($\approx 4.78$) at 4 Hz, while RTK exhibited the highest response of 6.09. At 6.5 Hz a high response ($\approx 6.51$) is estimated at ASR and KKR ($\approx 6.44$). At BHGR the site response is 5.52, while at BIS it is 4.52. Using the estimated site response at the central frequencies 1.5, 4 and 6.5 Hz we prepared the contour maps as shown in Figures III.8(a), (b) and (c) in order to have a broader perspective of the regional site response variation. At 1.5, 4 and 6.5 Hz we observed two low site amplification zones, one at New Delhi and the other at Oonchagaon. At all the central frequencies, the site amplification is higher in the greater Delhi region as shown in the zoomed site contour maps of greater Delhi in Figures III.9(a), (b) and (c). However, the site response gradient is steeper towards east Yamuna sector on the fluvial deposits at 1.5 Hz. At higher frequencies, the SAF gradient is mild. This subjective interpretation, eventhough is very preliminary, definitely yields an amplification trend and calls for detailed investigation using dense network in and around Delhi using both weak and strong motion stations.
Figure III.8: (a) Site response contour maps in the greater Delhi region at the central frequency 1.5 Hz, (b) Site response contour maps in the greater Delhi region at the central frequency 4 Hz, and (c) Site response contour maps in the greater Delhi region at the central frequency 6.5 Hz.
Figure III.9: Site response contour maps in the greater Delhi region at (a) 1.5 Hz, (b) 4 Hz, and (c) 6.5 Hz.
III.2 Site Response Study in the Sikkim Himalaya using Strong Motion Data
(Nath et al., 2005)

Geological Considerations of the Sikkim Region

The Sikkim region is located in the earthquake-prone part of the eastern Himalayas along Darjeeling-Sikkim tract, where fast and unplanned urbanization is still active with the incidence of a good number of large earthquakes in this terrain. Most workers have divided the Himalayas into a series of longitudinal tectono-stratigraphic domains, such as, (1) Sub Himalayas, (2) Lesser Himalayas, (3) Higher Himalayas, and (4) Tethys Himalayas (Figure III.10(a)), which are separated by major dislocation zones. The detailed petrographic provinces along with the geo-morphotectonic features are given in Figure III.10(b) (Narula et al., 2000).

Figure III.10(a): Generalized Geological map of the Himalayas, showing the different geotectonic domains and lithological units. Inset shows the location of the Sikkim Himalaya. MBT, Main Boundary Thrust; NP, Nanga Parbat; ND, Nanda Devi (after Gansser, 1964)
Figure III.10(b): Schematic Geological map of the Sikkim Himalaya displaying detail petrographic provinces, morphotectonic features and drainage patterns. The locations of strong motion stations are shown as solid triangles (after Nath, 2005 and 2006; Pal et al., 2008).
Data Source for Site Response Study in Sikkim

A semi-permanent nine-station strong motion array in Sikkim established by Indian Institute of Technology (IIT), Kharagpur, India has been operative since 1998. One Kinemetrics Altus K2 and eight Kinemetrics Altus ETNA high dynamic range strong motion accelerographs have been installed to continuously monitor the signals that satisfy event detection criteria. A trigger level of 0.02% of the full-scale (2g) and a sampling of 200 samples per second is set for the data recording. The dynamic range of the systems is 108 dB at 200 samples/sec with 18-bit resolution. The data for 80 local earthquakes ($3 \leq M_L \leq 5.6$) have been recorded during 1998-2003 shown in Figure III.11 on IRS LISS III image of Sikkim. The analysis is based on 80 earthquakes, which are recorded with good signal-to-noise ratio (signal-to-background noise ratio $\geq 3$). The data recording history for 80 events is given in (Table III.3) with epicentral distance ranging from 10-100 km.

Figure III.11: Sikkim Strong Motion Array and Epicentral locations of 80 earthquakes on IRS LISS III image of Sikkim (after Nath, 2005 and 2006; Pal et al., 2008).
Table III.3: Sikkim Strong Motion Array recording history for 80 earthquakes with signal-to-background noise ratio $\geq 3$  

<table>
<thead>
<tr>
<th>Sl. No.</th>
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**Strong Motion Data Processing**

The uncorrected acceleration time series, x(n), recorded by a given station were corrected for the instrument response and baseline following the standard algorithm as outlined in the software package of Kinemetrics Inc. The onset of S-wave arrival time (t_o) was estimated in x(n). Then, x(n) was bandpass filtered between 0.1 and 30.0 Hz. From the filtered dataset b(n), a time window of 5.0 sec duration, starting from t_o and containing the maximum of S-wave arrival was selected for all the analysis undertaken in this study.

**Analysis of Strong Motion Data**

*Path effect*

![Figure III.12](image)

**Figure III.12:** The plots of Q_s versus frequency. The dashed lines represent the zone of scatter of the data points with respect to the power law relation between Q_s and frequency with ± one standard deviation in the constant and the exponent (after Nath, 2005 and 2006; Nath et al., 2005).
Considering all the magnitude earthquakes between 3 to 5.6 involved in the present analysis we have determined $Q_s$ and its dependence on frequency in the form of a power law through regression analysis of the direct S-wave data recorded by SSMA as given in Figure III.12. The equation thus established with very small uncertainty in the constant and the exponential terms is as follows:

$$Q_s = (167.69 \pm 28.25)f^{(0.47 \pm 0.06)}$$

(III.2)

$Q_s$ obtained in the present study represents the overall attenuation of the seismic wave energy, which includes the direct S-wave, early coda, and possibly $L_g$ phase of the recorded data from events with focal depths more than 10 km, except for a few shallow foci earthquakes. Since in our case there is no deeper event with a focal depth beyond 35 km the observations are restricted to only one seismic wave energy attenuation relation with a trade off between the source and the attenuation factors built in the convolution model.

**Site Response Analysis**

Station site amplification has been computed from 80 strong motion events with signal-to-background noise ratio greater than 3. The site response has been calculated by using both the HVSR & GINV techniques for all the events at each site for different source azimuths, the results for Singtam and Gezing being presented here. Figure III.13 exhibits site response spectra for both N-S and E-W components computed by HVSR for event no. 3 (Table III.3) recorded at a source azimuth of 36ºN. The results obtained by GINV for the same are given in Figure III.13. It is observed that site response spectra of both the N-S and E-W components computed by either HVSR (Figure III.13(a)) or GINV (Figure III.13) have similar trend with minor or no variation at all in the amplitude of the respective site response curves and hence an RMS of N-S and E-W component site amplifications can be considered as the representative of site response spectra at the station under consideration. We can also draw a comparison between the estimates by both GINV and HVSR as shown in Figure III.13, wherein the RMS site spectral function obtained by GINV mimics the trend of the RMS site amplification spectra obtained by HVSR, the former having comparatively mild spectral fluctuations. The smooth nature of GINV as compared to HVSR is attributed to the fact that the site spectral function of a reference station has not been deconvolved from that of the recording site. Similar exercise is carried out for event no. 5 (Table III.3) recorded at Singtam at a source azimuth of 270ºN. The NS and EW components of response spectra estimated by HVSR are given in Figure III.13(d) and that by GINV in Figure III.13(e). The RMS site effect components obtained by both the techniques presented in Figure III.13(f) show a similar trend already reported for the source azimuth 36ºN in Figure III.13(g). In order to study the dependence of site response on source azimuth due to non-uniform radiation pattern of the source, difference in scattering and diffraction patterns at different azimuths and anisotropy of the medium through which the wave propagates we examined a composite variation of the RMS site amplification by GINV in Figure III.13(g) and HVSR in Figure III.13(h) for the events with the source azimuths 36ºN and 270ºN respectively. A pronounced variation of the site response curves by both the methods at strikingly different angles has been observed in these diagrams.
Figure III.13: Comparison of N-S and E-W component of site response for source azimuth 36°N by (a) HVSR, (b) GINV, (c) RMS site response by HVSR & GINV. Comparison of N-S and E-W component site response for source azimuth 270°N by (d) HVSR, (e) GINV, (f) RMS site response by HVSR & GINV. RMS site response for both the source azimuths 36°N and 270°N by (g) GINV, and (h) HVSR (after Nath et al., 2005).
The scatter plot between the GINV & HVSR estimates as shown in Figure III.14 exhibit good cluster around 45° degree line with ± one standard deviation signifying 1:1 correspondence. From this point onwards we consider HVSR estimates to be the representative of the non-reference site spectral amplification.

Figure III.14: Scatter plot between HVSR and GINV depicting data clustering around the 45° 1:1 correspondence line (after Nath et al., 2005; Nath, 2006).
Figure III.15: Variation of Site amplification with different source azimuthal coverage at Gezing by receiver function technique. Comparison between N-S & E-W component site response for the event recorded at (a) 104°N, (b) 122°N, (c) 290°N, (d) 330°N. Comparative plot between the RMS site response for the events at (e) 104°N and 122°N, (f) 290°N and 330°N. (g) The average RMS site response for the events at 115°N and 310°N azimuths (after Nath et al., 2005).

In order to further analyze the azimuthal dependency of site amplification at different stations we simulated experiments to study the effect on both the horizontal components of each event and dependency of the RMS site amplification on minor and large variation in source azimuth using HVSR technique only. The results at Gezing are presented here. In Figures III.15(a) and (b) we have presented the HVSR plots at two source azimuths 104°N and 122°N respectively recorded at Gezing station for the NS and EW components. The similar experiment is performed for two more earthquakes coming from azimuths 290°N and 330°N. The NS and EW site spectral amplification for these events at Gezing are presented in Figures III.15(c) and (d) respectively. Here also the NS and EW components follow each other closely thereby ruling out the dependency of site amplification on station azimuth i.e., 90° difference between two components. The RMS site response components for the events coming from 104°N and 122°N azimuths are compared in Figure III.15(e), those coming from 290°N and 330°N in Figure III.15(f) and finally those coming from strikingly different average azimuthal directions of 115° (between 104° and 122°N) and 310° (between 290° and 320°N) in Figure III.15(g).
It is evident from Figures III.15(e) and (f) that for the events coming from the azimuthal direction with minor deviation of 18°-40°, site amplifications do not reflect these azimuthal variations. It is not true when the events are coming from strikingly different azimuthal directions as exhibited in Figure III.15(g) for a deviation of 165° in source azimuth. The site response curves in this figure do not replicate each other. This significant difference in the spectral amplification may be attributed to different source radiation patterns, scattering, diffraction and topographic effects that influence the site effects in a hilly terrain.

In order to illustrate spatial distribution of site response in the Sikkim Himalaya, we subdivided HVSR values in three frequency bands, namely, low frequency band, LFB (<5 Hz) with a geometrically central frequency 2.5 Hz, moderate frequency band, MFB (5-10 Hz) with a geometrically central frequency 7.5 Hz and high frequency band, HFB (10-20 Hz) with a geometrically central frequency 15 Hz. The averaged site response values at these frequencies are plotted, contoured and presented in Figures III.16(a), (b) and (c) respectively. It is observed that high site response contours at LFB are at the foothills of the Himalayas in the Siwalik soft rock terrain. The site amplification at Singtam, which is at an elevation of 500 m above mean sea level, is to the tune of 3.4. It is further evident from Figure III.16(a) that as the altitude increases towards the northern districts of Sikkim, namely, Mangan, Chunthang and Lachen with the elevations of 1000, 2200 and 3800 m respectively, the site response diminishes at 2.5 Hz. For the MFB with the geometrical central frequency of 7.5 Hz we observe in Figure III.16(b) a different scenario of spatial variation of site amplification in contrast to that at low frequency band. Here, the higher HVSR values peak near Mangan at an elevation of 1000 m. In this region the hill slope is much steeper, the maximum site amplification observed in this case is 4.6.
In the HFB with geometrically central frequency 15 Hz the spatial variation pattern of site amplification shifts further northeast towards Chungthang with an altitude of 2200 m, site response peaking to a value of 5.8 on the high hill scarp between Mangan and Chungthang. It is noted that the Gangtok lineament is striking very close to Chungthang and is seismogenic in the area. This side of the hill is also very steep thereby contributing to the diffraction wave trains in the coda part of the S-wave. This variation in the SR contour pattern at three distinct frequency bands seem to follow the topography for the moderate to high frequency regime and is in consistence with the findings of Bouchon and Barker (1996), when they simulated an experiment to observe ground motion amplification at or near top of the hill depending on the slope and the resonance frequency of the respective morphometric signature of the terrain.

Even the experiment simulated by Pedersen et al. (1994) show that the level of amplification can be significantly higher exceeding a factor of 10 at the slopes and base of the mountain as a function of frequency. In our case we see the amplification variation from low to high frequency bands at different topographic platforms with low to higher resonance frequency in line with the works of Bouchon and Barker (1996), and Pedersen et al. (1994).
III.3 Site-Specific Modeling of SH and P-SV Waves for Microzonation Study of Kolkata (Vaccari et al., 2010)

![Figure III.17: Tectonic scheme of the Bengal Basin and its surroundings. Malda-Kishanganj Fault (MKF), Dhubri Fault (DbF), Jangipur Gaibandha Fault (JGF), Rajmahal Fault (RF), Sainthia–Bahmani Fault (SBF), Garhmayna-Khandagosh Fault (GKF), Debagram Bogra Fault (DBF), Pingla Fault (PF), Eocene Hinge Zone (EHZ), Main Central Thrust (MCT), Main Boundary Thrust (MBT), MainFrontal Thrust (MFT), Po Chu Fault (PCF), Naga Thrust (NT); Disang Thrust (DT), Dauki Fault (DF), Kulsi Fault (KF), Dudhnoi Fault (DhF), Sylhet Fault (SF), Lohit Thrust (LT), Dhansiri Kopili Fault (DKF), Mishmi Thrust (MT), Kathihar-Nailphamari Fault (KNF), Tista Fault (TF), Mat Fault (MaT), Shan-Shagaing Fault (SSF), Eastern Boundary Thrust Zone (EBTZ), Munger-Saharsha Ridge Marginal Fault (MRMF) (modified after GSI, 2000). The square box (inset) shows the location of the Bengal Basin and its surrounding region in the Indian subcontinent context (after Vaccari et al., 2010).]

Regional Geology and Tectonic Settings

The tectonic feature of the Bengal Basin is a curvilinear Eocene Hinge Zone (EHZ), also known as the Calcutta–Mymensing hinge zone. The EHZ has a NE–SW trend, a width of 25 km and an extension of 550 km. It terminates at the E–W striking Dauki Fault (DF) at the southern boundary of Shillong Plateau (Figure III.17). The other major fault systems of the basin are the Garhmayna–Khandagosh Fault (GKF), Jangipur–Gaibandha Fault (JGF), Pingla Fault (PF), Sainthia–Bahmani Fault (SBF), Malda–Kishanganj Fault (MKF), Rajmahal Fault (RF), and Debagram–Bogra Fault (DBF) (Figure III.17). The EHZ is a regional feature that demarcates the continent-ocean transition beneath the Bengal Fan and also divides, tectonically, the Bengal Basin into two major units: the shelf and the geosynclinal area. The EHZ demarcates a zone of differential thickening and a subsidence rate of the overlying Oligocene and Miocene section (Salt et al., 1986). In West Bengal, the hinge is cut across by numerous en-echelon faults and by moderate flexures. From the seismic pros-pecting records, across the EHZ, there is a sharp change in facies and pressure regimes in the Upper Paleogene and Neogene sections (Ganguly, 1997).
In the Bengal Basin, three structural domains are recognized: the western scarp zone, the middle shelf zone and the eastern deeper basin. The western scarp zone is defined by a series of N–S trending subsurface faults originally identified through deep drilling and gravity modeling, and recently imaged by deep seismic profiling (GSI, 2000). A series of buried basement ridges marks the western margin of the Bengal Basin. To the east of these ridges, there are rows of basin margin en-echelon faults and scarps. The western part of the Bengal Basin constitutes a broad shelf zone bounded by the Basin margin fault zone to the west and northwest and by the Eocene Hinge Zone to the east and southeast.

The total sedimentary thickness below Kolkata, above the crystalline basement, is around 7 km (Murty et al., 2008); of this, the top 0.35–0.45 km is quaternary, which overlies, from top to bottom, 4.5–5.5 km of tertiary sediments, 0.5–0.7 km of Cretaceous Trap and 0.6–0.8 km of Permo-carboniferous Gondwana rocks. Mitra et al. (2008) show at very shallow depth, the very sharp increase in S-wave velocity that is visible in our section models.

Seismic Activity of Kolkata

Kolkata city lies at the boundary of Indian seismic zones III and IV, which comes under high seismic risk (IS:1893–2002 Part 1). The Global Seismic Hazard Assessment Program (GSHAP, 1999) estimates a PGA of 0.08g for firm soil site in greater Kolkata.
The area has been affected by distant as well as nearby earthquakes. Near sources include the earthquakes of 29th September, 1906 and 15th April, 1964 Calcutta earthquake (located south of Kolkata over the EHZ), which were strongly felt in and around Kolkata and caused considerable damage. A maximum intensity of VII, in the Rossi–Forel scale, was felt for the September 1906 earthquake.

Figure III.19: The cross section AA’ with its different geological units. The colors represent different layers as shown in the legend. The numbers at the left top and bottom are the length and depth of the AA’ profile, respectively (after Vaccari et al., 2010).

Figure III.20: Cross sections BB’, CC’ and DD’ with their different geological units. The colors represent different layers as shown in the legend. The numbers at the left top and bottom are the length and depth of the respective profiles (after Vaccari et al., 2010).
(Middlemiss, 1908) and an intensity of VII, in the Mercalli scale (Jhingran et al., 1969) and VII in the Modified Mercalli (MM) scale for the April 1964 earthquake (GSI, 2000). In other words, the maximum observed intensity is VII in the Mercalli-Cancani-Sieberg (MCS) or VI in the Medvedev–Sponheuer–Karnik (MSK) scales, which roughly corresponds to the acceleration range 0.025–0.05g (Panza et al., 1997). Though both earthquakes caused severe cracks in buildings, the April 1964 earthquake was the most damaging event and was felt over an area of 67,000 km².

Distant earthquakes that shook Kolkata include those of 1st September 1803, 26th August 1833 and 31st December 1881. The best documented event was the great 1897 Shillong earthquake where a maximum intensity of VIII (MM scale) was felt over the Bengal Basin (Seeber and Armbruster, 1981). The Shillong earthquake was located at an epicentral distance of 470 km from Kolkata but caused considerable damage, to the extent of partial collapse of buildings. Oldham (1899) reported an intensity of isoseist 3 on the Oldham scale for the Shillong earthquake, which is equivalent to an intensity of VII in the MSK scale or to VIII on the MCS scale (e.g., Decanini et al., 1995). The 8th July 1918 Srimangal earthquake, located about 350 km from Kolkata, also caused cracks in many old and new buildings. Similar damage was observed in Kolkata during the Dubhri earthquake of 3rd July 1930. The Bihar–Nepal earthquake of 15th January 1934, which was about 480 km distant, also caused substantial damage to buildings. An intensity of VI (MM) was assigned to Kolkata since the damage pattern was similar to that observed after the nearby-source Kolkata earthquake of 1964 (Dunn et al., 1939).

Methodology

The hybrid method used in this study (Fäh, 1992; Fäh et al., 1993; Fäh and Panza, 1994; Fäh et al., 1994) based on the Modal Summation (MS) (Panza, 1985; Florsch et al., 1991; Panza et al., 2001), and Finite Difference (FD) methods (Virieux, 1984 and 1986; Levander, 1988). The methodology belongs to the Neo-Deterministic Seismic Hazard Assessment (NDSHA) procedure, developed, and so far, applied to the realistic modeling of the seismic input in 13 mega-cities and large urban areas in Europe, Central America, Africa and Asia. The procedure and some results are discussed in detail by Panza et al. (2001 and 2002), and a recent example of its application is given by Zuccolo et al. (2008).

The input parameters are the earthquake source and the geological structures through which seismic waves propagate from the source to the site of interest. The structural model used in the computations consists of two parts: simple 1D ‘bedrock’ or ‘regional’ structure and 2D lateral heterogeneous structure. The source is located in the bedrock structure and the calculations are performed in two stages. The seismic wave-field is propagated from the earthquake source to the boundaries of the laterally heterogeneous area applying the MS method. The resulting time series are used to excite the wave propagation in the laterally heterogeneous medium where the seismic wave-field is propagated with the Finite Difference (FD) technique. The hybrid approach thus allows the calculation of the local seismic wave-field for short (few kilometers) as well as for long (several hundred kilometers) epicentral distances.

The bedrock model is defined as a stack of horizontal layers, each characterized by its thickness, P- and S-wave velocities, density and Q factor, controlling the anelastic attenuation.
To optimize the setting of relevant parameters in the FD model (optimal model depth, grid step, proper removal of reflections at boundaries), the results obtained from the MS computation are compared with those given by FD for the case of the 1D regional model. When the differences between these two results do not exceed 5%, then a 2D simulation is performed with the obtained optimal values of parameters.

**Table III.4:** Mechanical properties of the various soil layers in profile AA’

<table>
<thead>
<tr>
<th>Formation</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$V_p$ (m/s)</th>
<th>$V_s$ (m/s)</th>
<th>$Q_p$</th>
<th>$Q_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top soil</td>
<td>1.5</td>
<td>400</td>
<td>230</td>
<td>40</td>
<td>19</td>
</tr>
<tr>
<td>Sand fine</td>
<td>1.6</td>
<td>500</td>
<td>290</td>
<td>44</td>
<td>20</td>
</tr>
<tr>
<td>Clay</td>
<td>1.8</td>
<td>550</td>
<td>320</td>
<td>60</td>
<td>25</td>
</tr>
<tr>
<td>Silt</td>
<td>1.8</td>
<td>550</td>
<td>320</td>
<td>60</td>
<td>25</td>
</tr>
<tr>
<td>Gravel</td>
<td>1.82</td>
<td>600</td>
<td>345</td>
<td>62</td>
<td>26</td>
</tr>
<tr>
<td>Sand medium</td>
<td>1.82</td>
<td>700</td>
<td>405</td>
<td>64</td>
<td>28</td>
</tr>
<tr>
<td>Sand coarse</td>
<td>1.83</td>
<td>800</td>
<td>460</td>
<td>64</td>
<td>28</td>
</tr>
<tr>
<td>Sand fine to medium</td>
<td>1.83</td>
<td>900</td>
<td>520</td>
<td>66</td>
<td>29</td>
</tr>
<tr>
<td>Sand fine to coarse</td>
<td>1.83</td>
<td>1000</td>
<td>580</td>
<td>66</td>
<td>29</td>
</tr>
<tr>
<td>Sand coarse with gravel</td>
<td>1.85</td>
<td>1100</td>
<td>635</td>
<td>70</td>
<td>30</td>
</tr>
</tbody>
</table>

**Table III.5:** Mechanical properties of the various soil layers in profiles BB’, CC’ and DD’

<table>
<thead>
<tr>
<th>Formation</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$V_p$ (m/s)</th>
<th>$V_p$ (m/s)</th>
<th>$Q_p$</th>
<th>$Q_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top soil</td>
<td>1.5</td>
<td>240</td>
<td>140</td>
<td>40</td>
<td>18</td>
</tr>
<tr>
<td>Dark grey silty clay</td>
<td>1.8</td>
<td>260</td>
<td>150</td>
<td>45</td>
<td>20</td>
</tr>
<tr>
<td>River Channel deposit</td>
<td>1.9</td>
<td>460</td>
<td>265</td>
<td>50</td>
<td>23</td>
</tr>
<tr>
<td>Bluish grey silty clay with kankar</td>
<td>1.85</td>
<td>325</td>
<td>185</td>
<td>60</td>
<td>27</td>
</tr>
<tr>
<td>Yellowish grey silt with clay binders</td>
<td>1.9</td>
<td>415</td>
<td>240</td>
<td>60</td>
<td>27</td>
</tr>
<tr>
<td>Mottled brown/grey silty clay</td>
<td>1.9</td>
<td>460</td>
<td>265</td>
<td>60</td>
<td>27</td>
</tr>
<tr>
<td>Light grey clay</td>
<td>1.9</td>
<td>485</td>
<td>280</td>
<td>60</td>
<td>27</td>
</tr>
<tr>
<td>Dense greyish brown silty sand</td>
<td>1.9</td>
<td>615</td>
<td>355</td>
<td>64</td>
<td>29</td>
</tr>
</tbody>
</table>

**Input Data**

The input data considered for the earthquake ground motion simulations using the hybrid approach are (a) the earthquake source model, (b) the regional (bedrock) model parameter describing the average properties of the various subsurface lithologies, and (c) the local model parameters.
The 15th April 1964 Calcutta earthquake, along the EHZ, was taken as the source for the computation. The source is at an epicentral distance of 96.5 km from the nearest profile (i.e., BB’). For modeling ground motion, the earthquake hypocenter is placed at 36 km of depth with the focal mechanism parameters dip = 32°, strike = 232° and rake = 56°, as reported for the 15th April 1964 Earthquake (GSI, 2000; Chandra, 1977), but the magnitude is taken as \( M_w = 6.5 \), that is, the maximum expected magnitude in this region, as suggested by Mohanty and Walling (2008b).

The regional bedrock structure is taken from the comprehensive work of Parvez et al. (2003) compiled from the available geological and geophysical information. The Indian subcontinent is divided into 15 regional polygons based on their structural model and Q-structure; structural polygon 7 is considered for the present study.

For the 2D local structural model, four N–S trending profiles are considered: AA’, BB’, CC’ and DD’ (Figure III.18). Profile AA’ is obtained from GSI (Chatterjee et al., 1964) (Figure III.19) and BB’, CC’ and DD’ are compiled from different sources (Ghosh and Gupta, 1972; Som, 1999; C. E. Testing Company Pvt. Ltd., 2002; Sengupta, 2000; Pal, 2006) (Figure III.20). The AA’ profile runs from Daulat, Triveni to Children Park, and Bhatpara with a length of about 10.5 km and a depth of 120m. The cross-sectional profile was prepared using data from 6 boreholes. The BB’ (4 km), CC’ (4.5 km) and DD’ (4.5 km) profiles run through the length of the metro track with a total length of 13 km from the Tollygunj to Shyam Bazar station and reach a depth of 60m. The mechanical properties of S-wave velocity \( (V_s) \), density \( (q) \) and the quality factor \( (Q_p \text{ and } Q_s) \) for the local structural model of various soil layers along each profile are shown in Tables III.4 and III.5. The 2D structural model used for computation is a realistic model which reflects the sharp jump in the S-wave velocity \( (V_s) \) followed by a continuous 1D structure model (Mitra et al., 2008). In the computation for BB’, CC’ and DD’, merged the very thin topmost layer is merged with the layer below, to avoid digitization problems. However, this does not influence the results, given the wavelengths involved. The relation between \( V_p \) and \( V_s \) is considered to be \( V_p = 1.73 \ V_s \), approximately.

**Results**

For the four profiles AA’, BB’, CC’ and DD’, synthetic seismograms were generated with a cutoff frequency of 6 Hz, using the scaled point-source approximation by Gusev (1983) as reported in Aki (1987). The hazard factor was computed in terms of acceleration and Response Spectra Ratio (RSR). The RSR was used as an estimate of the amplification at each site and is expressed as:

\[
\text{RSR} = \frac{\text{RS}(2D)}{\text{RS}(1D)} \tag{III.3}
\]

\( \text{RS}(2D) \) is the response spectrum (at 5% damping) of the signals in the laterally varying local structure and \( \text{RS}(1D) \) is the response spectrum calculated for the bedrock regional reference model. The site amplification is estimated in terms of RSR as a function of frequency and epicentral distance.
The maximum ground acceleration $A_{\text{MAX}}$ along the AA’ profile, located at a distance of about 132 km from the source, is observed to be 0.05g for the radial component (Figure III.21), 0.02g for the transverse component and 0.025g for the vertical component. A maximum amplification of 7 is observed for the radial component at 0.7 Hz, while the vertical and transverse components show similar amplifications (around 6) at 1.4 and 0.7 Hz, respectively (Figure III.22).
Figure III.22: Response Spectra Ratio (RSR) versus frequency and epicentral distance along the AA’ profile. A 5% damping is assumed (after Vaccari et al., 2010).

The maximum $A_{\text{MAX}}$ for the BB’ profile is observed in the radial component equal to 0.17g at an epicentral distance of about 97 km that corresponds to the river channel (Figure III.23). The $A_{\text{MAX}}$ for the
vertical and transverse components are 0.10 g and 0.03 g, respectively. Figure III.24 depicts the RSR with the maximum amplification observed for the radial component with RSR of 8 at 1.0 Hz followed by a vertical component with RSR equal to 6 at 1.7 Hz and a transverse component with RSR of 5 at 1.0 Hz. The maximum amplification is consistently within the frequency range of 1.0–2.0 Hz (Figure III.24).

**Figure III.23:** Accelerograms along the BB’ profile for the three components of ground motion. The maximum amplitude $A_{\text{MAX}}$ is given in cm/s$^2$ (after Vaccari et al., 2010).
Figure III.24: Response Spectra Ratio (RSR) versus frequency and epicentral distance along the BB’ profile. A 5% damping is assumed (after Vaccari et al., 2010).

The maximum $A_{\text{max}}$ for the CC’ profile is 0.15 g, observed for the radial component, followed by 0.12 g for the vertical component and 0.05 g for the transverse component (Figure III.25). The RSR distribution for the CC’ profile shows amplifications as large as 10 for the radial component at 1.0 Hz. For the vertical and transverse components, the amplification is 8 at 1.7 Hz and 6 at 1.0 Hz, respectively (Figure III.26).
Figure III.25: Accelerograms along CC’ profile for the three components of ground motion. The maximum amplitude $A_{\text{MAX}}$ is given in cm/s$^2$ (after Vaccari et al., 2010).
Figure III.26: Response Spectra Ratio (RSR) versus frequency and epicentral distance along the CC’ profile. A 5% damping is assumed (after Vaccari et al., 2010).
For the DD’ profile, the maximum $A_{\text{MAX}}$ for the vertical component is about 0.1 g at an epicentral distance of around 105 km from the source. The $A_{\text{MAX}}$ for radial and transverse components are 0.10 and 0.05 g, respectively (Figure III.27). The RSR distribution versus frequency and epicentral distance shows clear amplification for the transverse and vertical components at specific frequencies. For the transverse component, the maximum computed RSR is 7 at 1.2 Hz while for the vertical component; the computed RSR is 8 at the slightly higher frequency of 1.7 Hz. The RSR at the radial component shows distinct amplifications of 10 at 1.0 Hz, and 7 at 2.5 Hz (Figure III.28).

Figure III.27: Accelerograms along DD’ profile for the three components of ground motion. The maximum amplitude $A_{\text{MAX}}$ is given in cm/s$^2$ (after Vaccari et al., 2010).
Figure III.28: Response Spectra Ratio (RSR) versus frequency and epicentral distance along the DD' profile. A 5% damping is assumed (after Vaccari et al., 2010).
Inferences

Site specific synthetic seismograms are generated along four profiles oriented in a N–S direction across the northern and southern part of Kolkata. Profile AA’ samples the northern part while BB’, CC’ and DD’ sample the southern part of the megacity. Looking at the nearby seismicity, the source used to generate the synthetic seismogram along the four profiles has been placed in the epicentral area of the 1964 Calcutta earthquake ($M_b$: 5.2). The source is located to the south of the profiles at a distance of 96.5 km. The hazard is estimated in terms of $A_{\text{MAX}}$ and RSR. The computation of the $A_{\text{MAX}}$ of the three components for the four profiles shows that the peak acceleration varies in the range from 0.05 to 0.17 g. A comparative study of various seismic intensity scales with respect to acceleration shows that the expected intensity range for Kolkata is from VII to X (Lliboutry, 2000; Murphy and O’Brien, 1977; Medvedev, 1977; Richter, 1958; Panza et al., 1997).

III.4 Macroseismic-Driven Site Effects in the Southern Territory of West Bengal, India (Nath et al., 2010)

Historical macroseismic data are valuable especially in the regions where earthquakes are experienced rather at low frequency. In the case of West Bengal (Figure III.29), India considering the low seismicity in the region and non-availability of strong motion data for site characterization.

Figure III.29: A geological map covering the southern territory of West Bengal (after Geological Survey of India, 2000) while the inset depicts the three earthquakes considered in the present study (after Nath et al., 2010).

Following Hough and Bilham (2008), the site amplification distribution is anticipated from macroseismic field in terms of residual intensity, i.e. the difference between the observed intensity (at the surface) and the estimated ones at the bedrock. Three major earthquakes namely, 12th June 1897 Great Shillong of $M_W$ 8.1, 15th January 1934 Great Nepal-Bihar of $M_W$ 8.1, and 15th April 1964 Sagar
Island of $M_w$ 5.4 have been considered. The simulations are performed considering the source and the path attributes for each of the historical earthquakes. The large epicentral distances (>200 km) associated with these earthquakes accounts for long period components of the simulated seismogram that are generally associated with basin effects. The modeling is considered by comparing two popular techniques - stochastic simulations vis-à-vis frequency wave-number (F-K) integration or Green’s function approach; the former being generally used in the simulations of high frequency ground motion attributes (Motazedian and Atkinson, 2005; Herrmann, 2002; Boore, 1983).

**Historical Earthquakes**

The source attributes of the historical earthquakes as listed in Table III.6 have been studied by several researchers. A reverse faulting through a protrusion mechanism bounded by at least two major faults has been established as the tectonic framework that caused the 1897 Shillong earthquake (Bilham and England, 2001). The rupture geometry for this earthquake is constrained by geodetic and geological data that indicates 16±5 m of reverse slip on 110±10 km ESE fault; implicating a high stress drop equal to 159 bars (Nath et al., 2009). The rupture is assumed to have propagated up-dip from about 35 km to 9 km depth, extending through the crust accompanied by a 10m slip of normal faulting on the Chedrang fault to the southern extension (Oldham, 1899; Hough and Bilham, 2008). The second event is modeled to incorporate the effect of seismic slip on the fault with rupture parameters as postulated in an earlier study by Hough and Bilham (2008). A stress drop of 86 bars has been assigned to the event as estimated from the slip and rupture length. The intensity residual map is prepared by selecting higher predicted values at each point. The source geometry of the 1934 Nepal-Bihar earthquake ascribes to a 150 km fault-rupture length oriented in the 285°N with a shallow dip of 6° and the frontal thrusts at shallow depth alongwith a stress drop of 275 bars (Nath et al., 2009). The 1964 Sagar Island earthquake is believed to have originated due to the reactivation of NNE trending fault located over the Eocene Hinge zone (Chandra, 1977). The earthquake mechanism suggests thrusting with minor strike-slip component. The associated rupture dimensions are estimated through magnitude-source scaling relation of Hanks and Bakun (2002).

Table III.6: The source parameters of the three earthquakes used in the present study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1897 Shillong$^\diamond$</th>
<th>1934 Nepal-Bihar$^\Phi$</th>
<th>1964 Sagar Island</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epicentral location</td>
<td>26.0°N 91.0°E</td>
<td>25.9°N 91.7°E</td>
<td>21.6°N 91.0°E</td>
</tr>
<tr>
<td>Magnitude ($M_w$)</td>
<td>8.1</td>
<td>7.1$^#$</td>
<td>8.1</td>
</tr>
<tr>
<td>Strike</td>
<td>112°</td>
<td>292°</td>
<td>285°</td>
</tr>
<tr>
<td>Dip</td>
<td>50°</td>
<td>50°</td>
<td>6°</td>
</tr>
<tr>
<td>Focal depth (km)</td>
<td>35.0</td>
<td>0.0 (?)</td>
<td>20.0</td>
</tr>
<tr>
<td>Rupture length (km)</td>
<td>110.0</td>
<td>35.0</td>
<td>150.0</td>
</tr>
<tr>
<td>Rupture width (km)</td>
<td>35.0</td>
<td>9.0</td>
<td>80.0</td>
</tr>
<tr>
<td>Fault depth (km)</td>
<td>9.0</td>
<td>9.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

$^\#$triggered by the $M_w$ 8.1 earthquake; $^\diamond$after Hough and Bilham (2008), $^\Phi$after Nath et al. (2009).
Strong Ground Motion Simulations

The earthquakes occurring at large epicentral distances from the site can cause significant devastations owing to basin effects, which are primarily associated with long period components of the seismogram. The waveforms are also generally affected by the source directivity and path characteristics. However, the former influence is considerably less at large epicentral distances. The optimal assessment is considered from two modelling techniques, namely finite-fault stochastic and frequency wave-number integration. The algorithm given by Motazedian and Atkinson (2005) has been employed.

In F-K integration method, the synthetic seismogram for an impulse source in a layered medium is generated by double integral transforms over wave number and frequency (Hudson, 1969).

In this study, the crustal velocity, density, and attenuation profiles, as depicted in Figure III.30, taken from Ghose and Bhattacharya (2004), Prasad et al. (2005), and Parvez et al. (2003), respectively are used for the simulation purpose. Both the velocity and acceleration time histories are synthesized.

![Crustal profiles depicting (a) crustal velocity, (b) crustal density, (c) attenuation coefficients](image)

**Figure III.30:** Crustal profiles depicting (a) crustal velocity, (b) crustal density, (c) attenuation coefficients (after Ghose and Bhattacharya, 2004; Prasad et al., 2005; Parvez et al., 2003).

The S-wave parts of the radial and transverse components of the synthetic seismograms are used to compute the peak ground motion using random vibration theory. The theory enables prediction of peak ground motion (acceleration or velocity) without using time series which eliminates additional comparisons required to generate time series (Vanmarcke and Lai, 1980). The peak ground parameters are correlated to the root mean square (rms) parameters, which can be calculated using Parseval’s theorem by relating the total energy in frequency domain to the total energy in time domain.

Figure III.31 depicts the amplitude spectra derived from the accelerograms for $M_w$ 5.4 Sagar Island Earthquake simulated at the epicentral distances of 50 km, 100 km, 150 km and 200 km. At the epicentral distance of 50 km, the amplitude spectra obtained from stochastic simulation are observed to be overall on the higher side compared to those derived from FK integration method. The two methodologies exhibit reasonable conformity in the lower frequency band at the epicentral distances of 100 km and 150 km.
However, at 200 km epicentral distance, the stochastic simulation yields lower amplitude spectra. Another comparison between the two simulation techniques is depicted in Figure III.32. The acceleration spectra computed for the three earthquakes, namely 1964 Sagar Island, 1934 Nepal-Bihar and 1897 Shillong earthquakes at Kolkata (22.61°N, 88.24°E) corresponding to the epicentral distances of 305 km, 558 km and 467 km indicate that F-K integration technique yields higher acceleration spectra in all the cases.

**Figure III.31:** Comparison between simulated acceleration spectra derived from synthetic accelerograms at different epicentral distances using stochastic (bold curves) and frequency wave-number integration (light curves) algorithms for $M_w$ 5.4 1964, Sagar Island earthquake (after Nath et al., 2010).
The Green’s Function approach is, therefore, deemed more appropriate in synthesizing long period components of the seismograms as compared to the stochastic method for far-source/teleseismic events. Subsequently, the technique is employed for the estimation of peak ground motions for the historical earthquakes, which are located at the epicentral distances >200 km.

**Residual Intensity Estimation**

The relationships given by Wald *et al.* (1999) have been used to estimate seismic intensity levels from Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV). The relations are preferred owing to their extensive usage in the generation of shake maps across the globe. In the present study, the correlation

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**Figure III.32:** The acceleration spectra derived from synthetic accelerograms by both the algorithms at Kolkata for 1964 Sagar Island, 1934 Nepal-Bihar, and 1897 Shillong Earthquakes (after Nath *et al.*, 2010).
characteristics for both the peak motion parameters are examined; as such Wald et al. (1999) pointed out that PGV is a better predictor of intensity than PGA at higher intensity levels. The three earthquakes namely $M_w$ 8.1, 1987 Shillong, $M_w$ 8.1, 1934 Nepal-Bihar and $M_w$ 5.4, 1964 Sagar Island are simulated using F-K integration technique across the study region at bed rock level as shown in Figure III.33, and the peak parameters are estimated with RVT. Contour maps of the observed macroseismic field are prepared to designate the corresponding seismic intensity levels. The available data are either in MMI and MSK intensity scales. However, one-to-one correspondence exists between MMI and MSK scales (Scawthorn, 2003). The two scales can be considered equivalent without any significant aberrations. Eventually, the residual intensity is obtained by subtracting the intensity at the bedrock level from the intensity at the surface level.

$$\text{MMI}_{\text{residual}} = \text{MMI}_{\text{surface}} - \text{MMI}_{\text{bedrock}} \quad (\text{III.4})$$
MMI\textsubscript{residual} represents the residual intensity while MMI\textsubscript{surface} refers to the seismic intensity at the surface obtained from the observed isoseismals. The present study considers the macroseismic distribution given by Ambraseys and Bilham (2003b) for the Shillong earthquake. For the 1934 Nepal-Bihar and 1954 Sagar Island earthquakes, we consult the compilations of Geological Survey of India given by Dasgupta \textit{et al.} (2000). Following Wald \textit{et al.} (1999), the seismic intensity at bedrock are computed as below,

\begin{align}
\text{MMI}_{\text{bedrock}} &= 3.66 \log (\text{PGA}) - 1.66 \quad \text{(III.5)} \\
\text{MMI}_{\text{bedrock}} &= 3.47 \log (\text{PGV}) + 2.35 \quad \text{(III.6)}
\end{align}

PGA and PGV correspond to the peak ground acceleration and velocity, respectively at the bedrock level.

\textbf{Figure III.34}: The plot between residual intensities calculated from Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV) in MMI scale exhibits a reasonable 1:1 correlation between the two (after Nath \textit{et al.}, 2010).
A plot between residual intensities calculated from PGA and PGV exhibits a reasonable one-to-one correspondence trend as depicted in Figure III.34. A mean deviation of 0.084 (±0.2) is observed indicating an overall uncertainty to an order of 0.2 in the estimations and, otherwise, statistically insignificant difference between the two predictions. Therefore, residual intensities calculated from PGA are employed to assess the site amplification factor distribution in the study region considering that PGA is an attribute of strong ground motion and more compliant to the near-surface seismic modulations.

Figure III.35(a-c) depicts the estimated residual intensity distributions for the three earthquakes. The final residual intensity distribution map as depicted in Figure III.35(d) is generated by overlaying all the three maps and selecting the maximum at each grid point.

**Inferences**

It is seen that residual intensity distributions for the 1897 Shillong and 1934 Nepal-Bihar earthquakes intensify with higher values to the northwest of the study region, and that of 1964 Sagar Island earthquake exhibiting higher values to the southeast, on the contrary. In the former cases, it is seen that the source directivity and geological setting controls the regional damage pattern to a large extent. The basement exposures along the Rajmahal trap and likewise, the extent of Shillong Plateau to the northeast can be thought of as seismic delimiters when it arrives to the zone of thicker sediments to the southeastern extent of the territory. Nevertheless, high residual intensity values in the Eocene Hinge zone are exhibited by the Sagar Island earthquake indicating rather high site amplifications, typical of basins and deltas possessing very thick Holocene alluvium deposits. The site amplifications, in terms of twice the residual...
intensity (Hough and Bilham, 2008), could be as high as 4 to 6 times in the Eocene hinge zone and to the southeast of the territory. Lower site amplifications in the range of 2 to 3 times have been found to be significantly manifested in SW-NE extent of the basin. These observations ascribe to potential high seismic threats in the West Bengal basin. The present characterization of site amplification effects is anticipated to deliver first-hand assessment of potential damage patterns from future earthquakes. Macroseismic based analysis is found to be a simple and rapid approach in groundwork assessment of site effects at a regional level.
APPENDIX – IV

A few Typical Case Studies on Ground Motion Synthesis and Seismic Hazard Scenario

IV.1 Ground Motion Synthesis and Seismic Scenario in Guwahati City – A Stochastic Approach (Nath et al., 2009)

The Guwahati City is fast emerging as a multiethnic cosmopolitan with burgeoning population and rapid unplanned urbanization. It is a commercial hub, catering to eight states in the northeast Indian region, located in lower Brahmaputra valley underlain by thick alluvial deposits ranging from 25 to 600 m with granitic exposure at places. The regional seismotectonism of the northeast Indian province exhibits rather high magnitude earthquake potential as reported by Thingbaijam et al. (2008). As such large earthquakes in the past have caused widespread damage to life and properties in the region. Presently, the urban agglomeration in the terrain stands quite vulnerable to possible future great earthquakes. In the perspective of the devastations caused by recently occurring Sichuan earthquake of May 12, 2008 with $M_w$ 7.9 that has caused death toll of nearly 70,000 fatalities and destructions worth billion of dollars, it is imperative that realistic hazard scenario is envisaged in this extremely vulnerable seismic province of the world.

Seismic hazard quantification at a site is essentially based on understanding of the site response and propagation path of seismic waves as well as the source characterization of a damaging earthquake. A typical deterministic hazard model encompasses these physical attributes towards estimating of Peak Ground Acceleration (PGA). The source kinematics and wave dynamics implicates the scientific emphatic towards holistic understanding of the hazard level. This, therefore, is the definitive case of a finite fault stochastic modeling wherein strong ground motion synthesis has been carried out not only for predicted maximum earthquake in the terrain as reported by Thingbaijam and Nath (2008), but also workout regional as well as site specific attenuation relations for the Guwahati city through successive modeling for strong ($M_w$ >5) to a great earthquake (e.g., predicted maximum earthquake $M_w$ >8) at the hypocenters of the historical earthquakes.
Regional Seismotectonics

Figure IV.1: A seismotectonic map of northeast India adapted from Thingbaijam et al. (2008) wherein the fault rupture zones of four scenario earthquakes have been depicted with two dimensional boxes (after Nath et al., 2009).

The major tectonic background in northeast Indian region comprises the eastern Himalayan structures, the Mishmi massif (Nandy, 2001), the Indo Myanmar arc, the Brahmaputra valley, and the Shillong plateau. The Shillong plateau has been the main contributor to the seismicity of the Guwahati region. Because of its proximity to the great 1897 earthquake of $M_w$ 8.1 as shown in the Figure IV.1, the seismicity of the Guwahati city is expected to be dominated by the prevailing seismotectonism of the plateau. Based on the seismicity analysis in the northeastern India, Thingbaijam et al. (2008) classified four seismic source zones namely Eastern Himalayan Zone (EHZ), Mishmi Block Zone (MBZ), Eastern boundary Zone (EBZ) and Shillong Zone (SHZ) with a history of the occurrence of a great earthquake of $M_w$ 8.1 (Bilham and England, 2001). Subsequently, Thingbaijam and Nath (2008) estimated the corresponding maximum earthquake in the zones to be $M_w$ 8.35 ($\pm$0.59), $M_w$ 8.79 ($\pm$0.31), $M_w$ 8.20 ($\pm$0.50), and $M_w$ 8.73 ($\pm$0.70) respectively. The 1934 Nepal Bihar earthquake of $M_w$ 8.4 shares the similar tectonics as EHZ. The earthquake was felt over an area of 4,920,000 sq.km in India, Nepal, and Tibet. In the meizoseimal region, the earthquake caused major destruction, created numerous fractures, landslides and slump belt (Singh and Gupta, 1980). In the MBZ, the great Assam earthquake of 1950 $M_w$ 8.6 is believed to be originated due to right shear lateral movement in Po Chu Fault (Ben-Menaham et al., 1974) while 1988 Manipur earthquake of $M_w$ 7.2 is associated with Indo Myanmar arc in the EBZ.

Data Source

The data for the present analysis includes strong motion waveform recorded by the Indian Institute of Technology Guwahati (IITG) strong motion network, and geotechnical data acquired by Assam Engineering College at 200 borehole locations. The waveform data has been used for source, path and site characterization in the Guwahati city by Nath et al. (2008a) while geo-technically derived site amplification calibrated by that estimated from strong event recorded by IITG network has been utilized to deduce the site amplification distribution. A map of
study region depicting road networks, strong motion stations, and borehole sites is presented in Figure IV.2. The geotechnical analysis involves a combination of wave propagation theory with the material properties and the expected ground motion computed at the site of interest. The horizontally layered soil deposits in which recurrent and circular soil behavior can be simulated using a linear equivalent model of a nonlinear phenomenon through an iterative process to compute shear modulus and damping compatible with the equivalent uniform strain induced in each sub-layer accounting for the nonlinear behavior of soil (Kramer, 1996). Each geological unit, e.g. soil profile, is defined by its shear wave velocity, damping, total unit weight, and thickness. The initial estimate of damping has been taken to be 5% for soil while SHAKE 2000 used for geotechnical analysis. Two representative plots for site application against frequency has been given in Figure IV.3(a) while the histogram of the site response obtained at all the observation points 200 boreholes has been depicted in Figure IV.3(b).

Figure IV.2: The geological and geomorphological map of Guwahati depicting road networks, strong motion stations, and borehole sites (after Nath, 2007a; Nath et al., 2009).

Figure IV.3: (a) Two representative site amplification plot observed in boreholes located in high amplification zone, (b) The site amplification distribution exhibits dominance of site amplification factor in the range of 3.5-5.0 times (after Nath et al., 2009).
Methodology

The prediction of strong ground motion in engineering design is based both on the empirical and theoretical approaches. As understanding of the seismic wave-fields that shake man-made structures progresses, the latter methods gained momentum during the last two decades. The stochastic approach (Housner and Jennings, 1964; Hanks and McGuire, 1981; Boore, 1983; Boore and Atkinson, 1987; McGuire et al., 1984) is one of the expedient methods of synthesizing strong ground motion and is modeled with Gaussian noise with a spectrum that is either empirical or based on physical model of the earthquake source (Halldorsson et al., 2002). This method has been used to match the observed waveform even up to very high magnitude or seismic moment in different tectonic environment. Another approach, the frequency-wavenumber (F-K) integration method has been proved to be a very powerful simulator of wave progression in layered media as well as in complex geological domain (Bouchon and Aki, 1977). This method employs elastodynamic representation theorem to compute the motion (e.g., Aki and Richards, 1980) wherein the rupture process is modeled by postulating a slip function on a fault plane. Most often, the F-K integration approach more accurately reflects the wave propagation phenomena. However, due to simplistic consideration and formulation of strong motion affecting parameters, the stochastic algorithm has been found to be effective in the generation of high frequency ground motion (f > 0.1 Hz) and has been widely used to predict the ground motion around the globe where earthquake recordings are scanty.

Stochastic algorithm uses the standard convolution theorem to model spectral acceleration. The amplitude spectrum $A(\omega)$ can be written, in the frequency domain, as the product of source function $SO(\omega, \omega_c)$, a propagation path term $P(\omega)$, and a site function $SI(\omega)$ (Boore, 1983; Nath et al., 2005) as given below,

$$A(\omega) = SO(\omega, \omega_c). SI(\omega). P(\omega) \quad \text{(IV.1)}$$

Where $\omega_c (\omega = 2\pi f_c)$ refers to corner frequency. Acceleration spectra often shows a sharp decrease with increasing frequency so a high cut filter $F(\omega, \omega_m)$ is incorporated in the above equation such that,

$$A(\omega) = SO(\omega, \omega_c). SI(\omega). P(\omega). F(\omega, \omega_m) \quad \text{(IV.2)}$$

Papageorgiou and Aki (1983) related $\omega_m$ to source processes while Hanks (1982) associated $\omega_m$ to attenuation near the recording site. However, generally the high-cut filter $F(\omega, \omega_m)$ has been taken to be,

$$F(\omega, \omega_m) = \left[1 + \left(\frac{\omega}{\omega_m}\right)^2\right]^{-1/2} \quad \text{(IV.3)}$$

Where ‘s’ controls the decay rate at higher frequencies. The high-cut filter $F(\omega)$ given by Anderson and Hough and Anderson (1988) is presented as,

$$F(\omega) = e^{-ks\omega^2} \quad \text{(IV.4)}$$

where $k$ is a spectral decay parameter and controls the decay rate at higher frequencies.

The conventional point source approximation is unable to characterize key features of ground motions from large earthquakes, such as their long duration and the dependence of amplitudes and duration on the azimuth to the observation point (source directivity). Finite source model is, thus, used to simulate the ground motion that contributes not only to the duration and directivity of ground motions; they also affect the
shape of the spectra of seismic waves. In this model, the finite-fault plane is subdivided into elements (subfaults), and the radiation from a large earthquake is obtained as the sum of contributions from all elements, each of which acts as an independent subsource. The stochastic method for strong ground motion simulation of Beresnev and Atkinson (1997) employs a finite fault in which sub-source moment depends on the sub-fault dimension. The dimension of the sub-fault used in this technique can be calculated using a relation given by Beresnev and Atkinson (2001). However, a large uncertainty is associated with the suggested relation due to paucity of large earthquake magnitude data. To overcome this limitation to a large extent, the algorithm is further modified by Motazedian and Atkinson (2005) by introducing dynamic corner frequencies. The enhanced approach conserves the radiated energy at high frequency at any sampling of the sub-fault size and thus controls the relative amplitude of higher versus lower frequencies.

Accordingly, the Fourier amplitude spectrum due to the \( n \text{th} \) sub-fault is given as,

\[
A_n(f) = CM_nH_n \left[ \frac{(2\pi)^2}{1+\left(\frac{f}{f_{on}(t)}\right)^2} \right] e^{\frac{nR_g}{QG}}
\]

(IV.5)

Where \( C \) is a scaling factor, \( M_n \) is the \( n \text{th} \) sub-fault moment, \( H_n \) is the spreading factor responsible for conserving the energy at high frequency spectral level of the sub-fault, and \( f_{on}(t) \) is the dynamic corner frequency. \( G, R_n, \) and \( Q \) refer to Geometric spreading factor, hypocentral distances corresponding to \( n \text{th} \) subfault, and Quality factor respectively. The scaling factor is given as,

\[
C = \frac{R_g \sqrt{2}}{4\pi \rho \beta^3}
\]

(IV.6)

Where \( R_g, \rho, \) and \( \beta \) refer to hypocentral distance, average crustal density and shear wave velocity respectively. The moment \( M_n \) of the \( n \text{th} \) sub-fault is calculated using the slip distribution as follows,

\[
M_n = \frac{M_0 D_n}{a(D_n)}
\]

(IV.7)

Where \( D_n \) is the corresponding average slip and \( a(D_n) \) represents the slip of the subfault. The dynamic corner frequency is expressed as follows,

\[
f_{on}(t) = 4.9 \times 10^6 (N_R(t))^{3/4} N^{1/2} \beta \left( \frac{\Delta \sigma}{M_0} \right)^{1/3}
\]

(IV.8)

Where \( N_R(t) \) is the number of rupture sub-faults at a time, \( t \). The total number of subfaults is given by \( N \) while \( \Delta \sigma \) is the stress drop The spreading factor in Equation (IV.5) is computed as,

\[
H_n = \left( \sum \frac{f^2}{1+(f/f_{on})^2} \right)^2 \left( \frac{1}{N} \sum \frac{f^2}{1+(f/f_{on}(t))^2} \right)
\]

(IV.9)
Where \( f_0 \) is the corner frequency of the end of the rupture.

The seismic scenario in the Guwahati region has been generated by characterizing the maximum earthquake, source parameters, path attenuation and site effects.

**Source, Path and Site Characterization**

Seismic sources characterized by physical parameters like corner frequency \( f_c \), seismic moment \( M_o \), and stress drop \( \Delta\sigma \) have been estimated from the waveform data using Brune’s source approximation. The detailed study of source parameters have been made by Nath *et al.* (2008a) that shows a stress drop range of 49.76 to 442.24 bars in the Guwahati region. A very high stress drop observed in the region implicates large strong motion amplitude during an earthquake. The actual stress drop during any scenario event cannot be predicted from the available strong motion data but an average stress drop can be estimated that can be used in the scenario earthquake simulation as has been done by Nath *et al.* (2008b). The derived stress drop is especially useful where there is not any strong earthquake, however, in case of simulation of historical earthquake, observed parameters are preferable. In the present investigation, stress drop for scenario earthquake are either computed by observed slip value or taken from the published reports.

On the basis of the recorded strong motion data in the region, the shear wave quality factor \( Q_s \) shows a wide range of value varying from \( 180 f^{0.86} \) to \( 733 f^{0.35} \) (Nath *et al.*, 2008a). Accordingly, a power law frequency dependent relationship has been estimated for the region as given by

\[
Q_s = 342 f^{0.726}
\]  

(IV.10)

The \( Q_s \) obtained in the present study comprises of overall attenuation of the seismic wave energy which includes the direct S-wave, early coda, and possibly \( L_g \) phase of the recorded data.

On the basis of geotechnical investigation in the region, amplification of ground motion varies from 1 to 15 times (Nath *et al.*, 2008a). The site amplification computed through linear equivalent of non-linear system (Kramer, 1996) has been calibrated with the Horizontal to Vertical Spectral Ratio (HVSR) site amplification and found to exhibit a reasonable agreement in the required frequency range. However, site amplifications derived from the geotechnical data have been taken to account for site specific seismic hazard owing to its dense availability in the region.

**Seismic Scenario**

Finite source approximation based on dynamic corner frequency (Motazedian and Atkinson, 2005) has been used for the generation of seismic hazard scenario in the greater Guwahati region. As already discussed, the algorithm considers not only finite rupture but also provides increased resolution for lower frequency band. The robustness of the algorithm has been proved by simulating time history and spectral acceleration using observed source, path and site parameters. A representative simulated output for magnitude \( M_w \) 4.84 at AMTRON and AEC strong motion stations has been depicted in Figure IV.4. The simulated earthquake occurred on February 02, 2006 with the epicenter located at 27.20ºN 92.00ºE and magnitude of \( M_w \) 4.84 at a focal depth of 33 km. The source, path, and site parameters for the
simulations have been taken from Nath et al. (2008a). In the present study, the simulation has been performed using the finite fault simulator code EXSIM of Motazedian and Atkinson (2005). Seismic scenario in the territory has been generated based on the stochastic simulation of strong ground motion for the predicted maximum earthquake assumed to be nucleating from the hypocenter of the historic great or the largest reported earthquakes namely, the 1897 Shillong earthquake, 1934 Nepal Bihar earthquake, 1950 Assam, and 1988 Manipur earthquakes.

**Figure IV.4:** Accelerograms in time and frequency domain simulated through finite fault modeling using derived source parameters for Mw 4.84 at (a) AMTRON, and (b) AEC (after Nath et al., 2009).

### 1934 Nepal Bihar Earthquake

It is caused by strike slip faulting that was striking 100°N, dipping 30°S and focal depth of 20 km with stress drop of 275 bars as suggested by Singh and Gupta (1980). The other simulation parameters like shear wave velocity, density and quality factor has been taken from Nath et al. (2008a). The Kappa has been assumed to be 0.05 for the northeastern Indian region while geo-technically derived site response as the representative site amplification with 5% damping has been considered for the simulation. Table IV.1 enlists the detail simulating parameters used for the present modeling. The estimated PGA distribution for this scenario earthquake of Mw 8.4 has been presented in Figure IV.5(a) in grey scale with maximum being 0.24g in the Guwahati region as estimated from the scenario.
Table IV.1: Parameters used for strong ground motion simulation in Guwahati

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>1897 Shillong</th>
<th>1950 Assam</th>
<th>1934 Nepal</th>
<th>1988 Manipur</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strike</td>
<td>112°N</td>
<td>333.5°N</td>
<td>100°N</td>
<td>284°N</td>
</tr>
<tr>
<td>Dip</td>
<td>40°ESE</td>
<td>57.5°SW</td>
<td>30°S</td>
<td>45°E</td>
</tr>
<tr>
<td>Focal depth (km)</td>
<td>35</td>
<td>35</td>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>Source (Lat, Long)</td>
<td>26.00°N, 91.00°E</td>
<td>28.38°N, 96.68°E</td>
<td>26.60°N, 86.80°E</td>
<td>26.19°N, 94.89°E</td>
</tr>
<tr>
<td>Scenario Earthquake Magnitude (M_w)</td>
<td>8.7</td>
<td>8.8</td>
<td>8.4</td>
<td>8.2</td>
</tr>
<tr>
<td>Fault length (km)</td>
<td>330</td>
<td>600</td>
<td>312</td>
<td>200</td>
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<tr>
<td>Fault width (km)</td>
<td>150</td>
<td>100</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>No. of subfaults along Strike</td>
<td>11</td>
<td>18</td>
<td>14</td>
<td>10</td>
</tr>
<tr>
<td>No. of subfaults along Dip</td>
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<td>3</td>
<td>4</td>
<td>4</td>
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<td>Stress (bar)</td>
<td>159</td>
<td>66</td>
<td>275</td>
<td>83</td>
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<td>Shear Wave Velocity (km/s)</td>
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<td></td>
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<td>Crustal density (g/cm³)</td>
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<tr>
<td>Pulsating area</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>0.05</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Q_s</td>
<td>342f^{0.72}</td>
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<td></td>
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<tr>
<td>Geometrical spreading</td>
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<tr>
<td>Windowing function</td>
<td>Saragoni and Hart</td>
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<tr>
<td>Site Amplification</td>
<td>Geotechnical</td>
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<td></td>
</tr>
<tr>
<td>Damping</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1950 Assam Earthquake

It is the largest earthquake, known to have ever occurred, in the region. Richter (1958) assigned instrumentally determined magnitude of 8.7 to the earthquake. Three million square kilometer area in India, Myanmar, Bangladesh, Tibet, and China were affected accompanied by approximately 1500 deaths. The spread of aftershock activities extended from 91° to 97°E and 24° to 33°N. It is believed that the earthquake was caused by a motion of the Asian plate relative to the eastern flank of the Indian plate where the Northeast Assam block is imparted a tendency of rotation with fracture lines being developed along its periphery. According to Ben-Menahem *et al.* (1974), the earthquake had a strike-slip rupture with the velocity of 3 km/sec striking of 330-337° East of North and dipping at 55-60°ENE. The stress drop corresponding to this event has been estimated to be 66 bars. A seismic scenario has been generated in
the Guwahati city for the scenario magnitude of $M_w$ 8.8 assumed to be nucleating from source region of this event. The simulation parameters have been listed in the Table IV.1. The spatial PGA distribution map, as presented in Figure IV.5(b), is accordingly generated. It is seen that spatial pattern of PGA is quite similar to the previous one but the PGA value to be ranging between 0.008g to 0.038g unlike the one simulated for the 1934 Nepal Bihar earthquake.

Figure IV.5: The spatial distribution of Peak ground acceleration obtained through extended finite source stochastic simulation across the study region for scenario source region of (a) 1934 Nepal Bihar, (b) 1950 Assam, and (c) 1988 Manipur Earthquakes respectively (after Nath et al., 2009).

1988 Manipur Earthquake

The Manipur-Burma border earthquake of $M_w$ 7.2 rocked the entire northeastern region of the country early in the morning of August 6, 1988. The tremor was felt throughout northeast India, Bangladesh and parts of Burma. It lasted for approximately two minutes, and caused a loss of four human lives, considerable
damage to buildings, railway tracks, roads etc. Field surveys showed that the maximum intensity reached VIII on the Modified Mercalli Intensity Scale near the epicentral region (Kayal and De, 1991). For the scenario earthquake simulation of $M_w$ 8.3 in this source region, a rupture dimension has been considered to be equal to 200 x 80 km with 100 km focal depth since most of the earthquakes in this regime have focal depth of intermediate focus ranging from 90 km to 110 km. The stress drop for this event has been determined empirically to be equal to 83 bars while other parameters are same as depicted in Table IV.1. The simulated PGA distribution as depicted in Figure IV.5(c) shows a variation from 0.015g to 0.105g.

1897 Shillong Earthquake

This largest intra-plate earthquake in the Indian subcontinent raised the northern edge of the Shillong Plateau by more than 10 m, resulting in the destruction of structures over much of the plateau and the surrounding areas. The earthquake also caused widespread liquefaction and flooding in the Brahmaputra and Sylhet Floodplains. Source parameters for this great earthquake has been analyzed by many researchers (Ambraseys and Bilham, 2003b; Hough et al., 2005; Bilham and England, 2001). The strike and dip of the fault have been taken as 292°N and 40°ESE respectively as reported by Bilham and England (2001) while the fault dimension of 330 x 150 km has been modeled for the simulation considering the extent of the fault. The earthquake has been implicated to have originated due to pop-up mechanism bounded by at least two major faults Oldham and Dauki faults. The scenario earthquake of $M_w$ 8.7 has been projected in the rupture plane of Oldham fault. The 1897 Earthquake produced a gigantic slip of 16 m (maximum up to 21 m), which implicated a stress drop equal to 159 bars computed through the relation,

$$\Delta \sigma = \mu \frac{u(\infty)}{L} \quad \text{(IV.11)}$$

Where $\mu$ is the shear modulus of the crust, $u(\infty)$ is the final slip and $L$ is the sub-fault length. Other simulation parameters have been listed in Table IV.1, which have been applicable to this region (Nath et al., 2008b). The final peak ground acceleration maps thus produced at rock level and at the surface have been presented in Figures IV.6(a) and (b) respectively. It is apparent from Figure IV.6(b) that the PGA value at surface ranging from 0.22g to 1.27g has been enhanced due to site response while a maximum PGA of 0.4g has been estimated at rock level. The time and spectral domain presentation of a representative accelerogram have been presented for this earthquake in Figure IV.7. Also, the effect of site amplification on the simulated acceleration has been depicted in the diagram. Time history of the acceleration computed at bedrock level at the IITG station (Figure IV.2) has been depicted in Figure IV.7(a) while Figures IV.7(b) shows the effect of geotechnical amplification. Figure IV.7(c) depicts the site specific spectral acceleration simulated using geotechnical as well as HVSR site response. It can be observed that the geotechnical amplifications, which has been calibrated with those of strong motion data, exhibit to be an averaging input, so far as the spectral amplitudes are concerned (Nath et al., 2008a). Apparently, the Shillong region represents major contributor to the seismic hazard in the Guwahati city compared to other three seismic sources.
Figure IV.6: The spatial distribution of Peak ground acceleration produced for the scenario earthquake of $M_w$ 8.7 at (a) rock level, and (b) at surface (after Nath et al., 2009).

Figure IV.7: Accelerogram generated through stochastic finite fault modeling (a) at bedrock level, (b) at surface using site amplification derived through geotechnical data, (c) Spectral acceleration computed at surface using site amplification derived from geotechnical analysis (bold line) with the acceleration spectra simulated using HVSR site amplification (lighter shade) (after Nath et al., 2009).
Regional and Site Specific Attenuation Relations

The most common means of estimating ground motion at the site of interest is the use of an attenuation relationship which relates a specific strong motion parameter of ground shaking to one or more seismological parameters of an earthquake like the source, the wave propagation path between the source and site, the soil and geologic profile beneath the site.

In the present analysis, first order attenuation relationship has been derived by regression analysis considering the similar relation given by Campbell and Bozorgnia (2003).

\[
\ln(\text{PGA}) = C_1 + C_2 M + C_3 (10-M)^3 + C_4 \ln(\text{rup} + C_5 \exp(C_6 M))
\]  
(IV.12)

Where \(C_i\) (i=1 to 6) are the regression coefficients, The PGA is in g, \(M\) is the earthquake moment magnitude and \(\text{rup}\) is the rupture distance (km). However, for estimating the site specific attenuation relationship, local site parameters need to be incorporated in the fundamental equation. Therefore, site response, shear wave velocity and spectral response have been incorporated as follows,

\[
\ln(\text{PGA}) = C_1 + C_2 M + C_3 (10-M)^3 + C_4 \ln(\text{rup} + C_5 \exp(C_6 M)) + C_7 S_v + C_8 \ln(\text{SR}) + C_9 \ln(\text{SA})
\]  
(IV.13)

Where \(S_v\) represents effective shear wave velocity averaged over the top 30 meters, \(\text{SR}\) is the site response and \(\text{SA}\) is the spectral acceleration. The rupture distance, \(\text{rup}\), refers to that closest distance to the fault rupture from the observation point.

For the computation of regional attenuation relationships, PGA has been estimated by simulating all the observed earthquakes in the region from magnitude \(M_w\) 4.8 to \(M_w\) 8.1. A grid of 0.2\(^o\) x 0.2\(^o\) for near-field effects have been considered for the purpose. Thereafter, multiple regression analysis has been performed to estimate the model. A semi-empirical approach has been used to minimize the difference between the observed and the predicted values of ground motion using the least square error energy minimization. The estimated regional attenuation relationship, thus, derived as follows,

\[
\ln(\text{PGA}) = 9.143 + 0.247 M - 0.014 (10-M)^3 - 2.697 \ln(\text{rup} + 32.9458 \exp(0.0663 M))
\]  
(IV.14)

The PGA attenuation with rupture distance using this equation for magnitude \(M_w\) 7 have been shown in Figure IV.8(a) along with some already available attenuation relationships around the globe. The relation for Guwahati is quite comparable to those with Singh et al. (1996) for the Himalaya and of Nath et al. (2005) for the Sikkim region. However, relation given by Atkinson and Boore (1995) of Eastern North America matches with our relation for hypocentral distance greater than 30 km. Other relations estimated by Parvez et al. (2002), and Chandrasekran (1994) for different tectonic environments depict higher values than the present estimate while attenuation relationship of Joyner and Boore (1981), Ambrasey (1995), and Sharma (1998) underestimate. The log residuals computed with respect to rupture distance and magnitude has been depicted in the Figures IV.8(b) and (c) respectively. The log residual in the present estimate varies between ±0.6 that reflects the uncertainty bound with respect to the rupture distance as well as magnitude. Furthermore, site specific attenuation coefficients, that consider the local site attributes, have been derived at various frequencies as listed in the Table IV.2. The stochastic
ground motion simulation has been employed in the present study generally underestimates the near-
source peak amplitudes as compared to deterministic/analytical assessments. However, the diversion
as observed between Atkinson and Boore (1995) could be due to lack of near source data required for
calibration of the technique. However, further investigations through deterministic/analytical approaches
are envisaged and will be reported in the near future.

Figure IV.8: (a) Comparison of the present attenuation relation (bold line) with other relations estimated around the
globe. Log residual of PGA with respect to (b) rupture distance and (c) moment magnitude (Mw) (after
Nath et al., 2005 and 2009).

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>C₁</th>
<th>C₂</th>
<th>C₃</th>
<th>C₄</th>
<th>C₅</th>
<th>C₆</th>
<th>C₇</th>
<th>C₈</th>
<th>C₉</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>13.043</td>
<td>-0.107</td>
<td>-0.027</td>
<td>-2.713</td>
<td>10.722</td>
<td>0.198</td>
<td>-1.468</td>
<td>-0.621</td>
<td>0.047</td>
</tr>
<tr>
<td>0.5</td>
<td>10.606</td>
<td>-0.067</td>
<td>-0.030</td>
<td>-2.297</td>
<td>4.343</td>
<td>0.281</td>
<td>-1.273</td>
<td>-1.581</td>
<td>0.017</td>
</tr>
<tr>
<td>1</td>
<td>5.373</td>
<td>0.159</td>
<td>-0.017</td>
<td>-1.366</td>
<td>0.079</td>
<td>0.685</td>
<td>-3.345</td>
<td>-2.795</td>
<td>0.052</td>
</tr>
<tr>
<td>2</td>
<td>5.240</td>
<td>0.456</td>
<td>-0.011</td>
<td>-2.115</td>
<td>0.720</td>
<td>0.493</td>
<td>-0.175</td>
<td>-0.214</td>
<td>0.099</td>
</tr>
<tr>
<td>4</td>
<td>11.421</td>
<td>-0.161</td>
<td>-0.031</td>
<td>-2.338</td>
<td>7.388</td>
<td>0.216</td>
<td>-1.994</td>
<td>0.249</td>
<td>0.016</td>
</tr>
<tr>
<td>5</td>
<td>8.076</td>
<td>0.104</td>
<td>-0.023</td>
<td>-2.087</td>
<td>1.709</td>
<td>0.378</td>
<td>-2.720</td>
<td>0.445</td>
<td>0.043</td>
</tr>
</tbody>
</table>

Table IV.2: Regression Coefficient of Spectral Attenuation in the Guwahati City
### Frequency C 1 C 2 C 3 C 4 C 5 C 6 C 7 C 8 C 9 (Hz)

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>C₁</th>
<th>C₂</th>
<th>C₃</th>
<th>C₄</th>
<th>C₅</th>
<th>C₆</th>
<th>C₇</th>
<th>C₈</th>
<th>C₉</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>6.339</td>
<td>0.285</td>
<td>-0.018</td>
<td>-2.181</td>
<td>1.613</td>
<td>0.389</td>
<td>-0.655</td>
<td>0.363</td>
<td>-0.020</td>
</tr>
<tr>
<td>8</td>
<td>5.278</td>
<td>0.191</td>
<td>-0.018</td>
<td>-1.623</td>
<td>0.441</td>
<td>0.505</td>
<td>-2.639</td>
<td>0.718</td>
<td>-0.018</td>
</tr>
<tr>
<td>10</td>
<td>7.229</td>
<td>-0.049</td>
<td>-0.022</td>
<td>-1.609</td>
<td>1.093</td>
<td>0.380</td>
<td>-2.298</td>
<td>0.064</td>
<td>0.074</td>
</tr>
<tr>
<td>12</td>
<td>8.815</td>
<td>0.122</td>
<td>-0.021</td>
<td>-2.277</td>
<td>3.634</td>
<td>0.296</td>
<td>-1.134</td>
<td>0.311</td>
<td>0.024</td>
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<tr>
<td>18</td>
<td>2.932</td>
<td>0.690</td>
<td>-0.006</td>
<td>-2.043</td>
<td>0.170</td>
<td>0.652</td>
<td>-1.847</td>
<td>0.269</td>
<td>0.001</td>
</tr>
<tr>
<td>20</td>
<td>3.888</td>
<td>0.566</td>
<td>-0.006</td>
<td>-1.887</td>
<td>0.264</td>
<td>0.594</td>
<td>-2.953</td>
<td>0.412</td>
<td>0.070</td>
</tr>
<tr>
<td>Composite</td>
<td>9.131</td>
<td>0.040</td>
<td>-0.024</td>
<td>-2.231</td>
<td>3.621</td>
<td>0.295</td>
<td>-1.269</td>
<td>0.078</td>
<td>0.031</td>
</tr>
</tbody>
</table>

### Inferences

Peak ground acceleration in the Greater Guwahati region has been estimated through the simulation of four scenario earthquakes. The maximum earthquake predicted for the region has been considered for deterministic seismic hazard estimation that has been assumed to be nucleating from the hypocenter of historical earthquakes but with projected maximum magnitude. It is seen that the areas of high acceleration correspond to those of high site amplification that suggest that the site amplification is the most predominant factor. Furthermore, directivity effects could not conceived in all the cases due to overwhelming control by the site amplification over the estimated peak ground acceleration. Estimated site specific spatial PGA distributions depict a maximum of 1.27g for the predicted Mw 8.7 at the 1897 Shillong earthquake source region. The intensity for this scenario earthquake following the relation of Murphy and O’Brien (1977) predicts a maximum of MMI XI in the Guwahati region, which is significantly higher than the observed intensity value of MMI VIII for the earthquake of Mw 8.1 (Ambraseys and Bilham, 2003b). Furthermore, areas of high PGA (>1g) corresponds to Bordang surface geologically that corresponds to lower predominant frequency (>1.0 Hz) and lower shear wave velocity for 30 m depth profiles (V₃₀) in the range of 240-280 m/s. However, PGA distribution due to the Nepal Bihar earthquake source of magnitude Mw 8.4 is no less vulnerable in seismic terms with maximum PGA of 0.25g and MMI VIII in the region. The high intensity estimated for these earthquakes may be attributed to the consideration of scenario magnitude as well as local site effects. The higher PGA distribution has been observed along the Brahmaputra River flood plain. Finally, site specific attenuation relationship has been worked out that can be utilized for the prediction of PGA in the region at a denser mesh. The results achieved in this study can be considered a benchmark for similar investigation elsewhere towards updating universal building codal provisions that is to be adopted for rapid but planned urban growth with sustainable developments.

### IV.2 Probabilistic Seismic Hazard Assessment of India

Seismic hazard analysis studies are facilitated by appraisal of underlying seismotectonic regimes. The usually employed data comprises of earthquake catalog (instrumental and historical), fault map, focal mechanism data, palaeoseismic information, and fault-slip rates.
Seismogenic Sources and Seismicity Models

The Indian subcontinent encompasses different seismotectonic regimes. Figure IV.9 depicts a seismicity map of the region. The tectonically active interplate regions include the Himalayas, southern Tibetan Plateau, northwest frontier province, Indo-Myanmar arc, and Andaman-Sumatra region. Distinct from the tectonic plate boundaries is the intraplate margin of northeast India that encompasses the Shillong plateau. On the other hand, Peninsular India is delineated as Stable Continental Region (SCR). The subduction zones include that of Hindukush-Pamir in the northwest frontier province, Indo-Myanmar arc, and Andaman-Sumatra seismic belt. Subduction interface earthquakes are also observed across the Himalayas and the northwestern flanks of the Indian plate boundary.

Thingbaijam and Nath (2011) carried out an extensive study to demarcate and parameterize the underlying seismogenic source zones in the Indian subcontinent. The authors employed the earthquake catalog (Nath et al., 2011c), supplemented by records of historical earthquakes (with reported significant events occurring as late as 0819 AD), focal mechanism data, fault-slip rates and palaeoseismicity findings. They formulated a layered seismogenic source zonation corresponding to four hypocentral

![Figure IV.9: A seismicity map of India and adjoining regions depicting epicentral locations of the main-shock events covering the period 0819-2008; broadly classified tectonic provinces excluding active shallow crustal and interface regions are delineated with different shades (after Thingbaijam and Nath, 2011b).]
depth ranges (in km): 0-25, 25-70, 70-180 and 180-300. In total, 172 areal source zones were delineated on the basis of seismicity, fault patterns, and similarity in fault plane solutions. Owing to the data inadequacy, several zones were merged to facilitate computation of the seismicity parameters (Figure I.17). The Gutenberg and Richter (GR) parameters, i.e. $a$-value and $b$-value, for the zones were estimated by means of maximum likelihood method while the maximum earthquakes $m_{\text{max}}$ decided according to results of earlier investigations or the seismicity models. The earthquake occurrences are considered to be random. Smoothened seismicity models represented by activity rates at regular interval of 0.2° were also constructed for threshold magnitudes of $M_w$ 4.0, $M_w$ 4.5 and $M_w$ 5.5, respectively.

**Table IV.3: Selected Ground Motion Prediction Equations**

<table>
<thead>
<tr>
<th>Tectonic province</th>
<th>Reference and code in brackets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tectonically active shallow crust</td>
<td>Akkar and Bommer, 2010 (AKBO10); Boore and Atkinson, 2008 (BOAT08); Campbell and Bozorgnia, 2008 (CABO08); Sharma et al., 2009 (SHAR09)</td>
</tr>
<tr>
<td>Active shallow crust/ Subduction</td>
<td>Kanno et al., 2006 (KAN06); Zhao et al., 2006 (ZHAO06)</td>
</tr>
<tr>
<td>Subduction</td>
<td>Atkinson and Boore, 2003 (ATBO03); Atkinson and Macias, 2009 (ATMA09); Gupta, 2010 (GUPT10); Lin and Lee, 2008 (LILE08); Youngs et al., 1997 (YOU97)</td>
</tr>
<tr>
<td>Stable continental region</td>
<td>Toro, 2002 (TOR02); Campbell, 2003 (CAM03); Atkinson and Boore, 2006 (ATBO06); Raghukanth and Iyengar, 2007 (RAIY07)</td>
</tr>
<tr>
<td>Intraplate margin</td>
<td>Toro, 2002 (TOR02); Atkinson and Boore, 2006 (ATBO06); Sharma et al., 2009 (SHAR09); Nath et al., 2011a (NATH11)</td>
</tr>
</tbody>
</table>

**Ground Motion Prediction Equations**

16 Ground Motion Prediction Equations (GMPEs) as listed in Table IV.3 are adopted in the present analysis for hazard computations. The GMPEs are selected according to the assessment carried out by Nath and Thingbaijam (2011a). Number of candidate GMPEs were selected for the different seismotectonic provinces across India based on the criteria given by Bommer et al. (2010), and then subjected to suitability test proposed by Scherbaum et al. (2009). Nath and Thingbaijam (2011a) reported that there is general conformity amongst the GMPEs developed for tectonically active shallow crust while those developed for intraplate regions catering to higher ground motions have lower ranks (or suitability), and the subduction zones have significant regional implications. In the present selection, included lower-ranked equations are included such that they are either equally matched or outnumbered by those of higher ranks. Figure IV.10 depicts the logic tree constructed for the GMPEs in the present analysis. The ranking analyses were carried out using macroseismic intensity data due to which the ranking parameter (i.e., log-likelihood) does not have large variations in several cases (cf., Delavaud et al. 2009). Nevertheless, the ranking
analysis indicated important considerations to be taken while adopting the relevant GMPEs such as regional corrections suggested by the developers, and elimination due to rather poor conformity. The decision to assign equal weights is taken in order to avoid clear-cut preference. Higher influence by the GMPEs with higher rank collectively, nonetheless, is achieved owing to the selection.

![Diagram](image)

**Figure IV.10:** The logic tree framework for the ground motion prediction equations; the assigned weights are given inside the square brackets and the references for the codes used for the equations are listed in Table IV.3 (after Nath and Thingbaijam 2011b).

The equations selected for tectonically active shallow crust regions include that of Akkar and Bommer (2010), Boore and Atkinson (2008), Campbell and Boore (2008), and Sharma et al. (2009). The equations developed by Kanno et al. (2006) and Zhao et al. (2006) addresses tectonically active shallow crust and
subduction zones. For the SCR, the adopted equations include that of Atkinson and Boore (2006), Toro (2002), Raghukanth and Iyengar (2007) and Campbell (2003). In case of intraplate margin region of northeast India, the equation developed by Nath and Thingbaijam (2011a) for the region is included. In order to appropriately associate active deformations, the equation given by Sharma et al. (2009) is also included.

For the subduction zones, several considerations are imposed according to the observations of Nath and Thingbaijam (2011a). The equation developed by Atkinson and Boore (2003) is incorporated with correction for Japan in case of the Himalayas and northwest India-Eurasia convergence, and with correction for Cascadia in case of Indo-Myanmar and Andaman-Sumatra subduction zones, respectively. The equation developed for Atkinson and Macias (2009) is restricted to interface earthquakes with magnitude $M_W \geq 7.5$. Likewise, the equations developed by Gupta (2010) and Lin and Lee (2008) are restricted to intraslab regions of Indo-Myanmar, and Himalayas/Hindukush-Pamir respectively. The equation developed by Youngs et al. (1997) based on the world-wide data is also limited to the intraslab-subduction zones.

The adjustments for compatibility between the GMPEs are applied in the same manner as carried out by Nath and Thingbaijam (2011a). The mean peak horizontal component of the ground motion is homogenized in terms of new geometric mean definition, namely GMRotl50, as given by Boore et al. (2006). The conversion factors given by Beyer and Bommer (2006), and Campbell and Bozorgnia (2008) are used accordingly. The different source-to-site distance measures namely $R_{JB}$ (Joyner-Boore distance), $R_{EPIC}$ (epicentral distance), $R_{RUP}$ (rupture distance), $R_{HYPO}$ (hypocentral distance), and $R_{CF}$ (distance to the site from center of the fault rupture) for larger earthquakes ($M_W > 6.4$) are calculated by constructing finite-fault models. The predominant focal mechanisms as depicted in Figure I.17 are used for the purpose. This approach has been adopted instead of using the relations developed by Scherbaum et al. (2004) for tectonically active shallow crustal regions owing to different seismotectonic regimes in the present case. In case of smaller magnitude earthquakes ($M_W < 6.5$), coincidence is assumed between $R_{JB}$ and $R_{EPIC}$, $R_{RUP}$ and $R_{HYPO}$, and $R_{CF}$ and $R_{HYPO}$ respectively. To estimate the rupture dimensions (i.e., length and width), we use the relations given by Wells and Coppersmith (1994) for crustal events and those given by Strasser et al. (2010) for the subduction earthquakes. For the large intraplate earthquakes with reverse faulting, the fault-rupture area estimated from the magnitude is constrained by a factor of 2 (Nath and Thingbaijam, 2011b). Following the observation of Mai et al. (2005) that the hypocenters in strike-slip and crustal dip-slip events mostly occur in deeper sections of the fault plane, the location of hypocenter is placed on the plane decided by 0.5 and 0.8 (reverse faulting), 0.5 and 0.4 (strike-slip faulting), and 0.5 and 0.2 (normal faulting) of the rupture length and width, respectively from the fault location. The fault location is the top corner of the fault plane such that the dip is on the right-hand side. Depth to shear-wave velocity $V_S = 1.0 \text{ km/s} (Z_{1.0})$ is estimated using the relation between $Z_{1.0}$ and $V_S^{30}$ given by Chiou and Youngs (2008) while depth to $V_S = 2.5 \text{ km} (Z_{2.5})$ is assigned 2 km following Boore and Atkinson (2008). The computed scenarios consider varying hypocentral depths with homogenous distribution of source- to-site distance not less than 15 km for the shallow crustal zones. This is to avoid estimating ground motions at very near-source locations. Resolving the uncertainty associated with near-source ground
Seismic Site Conditions

Site conditions considered in previous PSHA studies in India discussed earlier include hard-rock, rock, and stiff soil conditions. Nath and Thingbaijam (2010) observed that the engineering bedrock in Guwahati city, north east India conforms to average shear-wave velocity for upper 30 m soil-column $V_s^{30}$ values ranging from 760-1500 m/s. In Garhwal Himalaya, Mahajan and Rai (2011) observed that sandstone and conglomerate bedrock have $V_s$ in the range of 750 - 800 m/s and 950 - 1000 m/s. In Bangalore, south India, Anbazhagan and Sitharam (2009) observed that the bedrock profile for weathered (soft) rock with $V_s \sim 330 \pm 30$ m/s extends from 1m to ~21 m while that of engineering bedrock with $V_s \sim 760 \pm 60$ m/s extends from 1m to 50 m. Maheswari et al. (2010) reported site class D, C and B in Chennai, south India; shallow bedrock areas conforming to site class B.

Nath et al. (2011b) attempted a first-order nation-wide assessment of site conditions in India. They observed that ~70% of the total landmass comes under site classes D and C. In view of the site characterization studies across the country, firm rock site condition (standard engineering bedrock) is considered to be more realistic for the regional hazard computations. The standard engineering bedrock conforms to $V_s^{30} \sim 760$ m/s (defined as boundary site-class BC). The GMPEs employed in the present analysis are accordingly adopted for the respective site condition (cf. Nath and Thingbaijam, 2011b). At the same time, a correction factor of 1 with negligible uncertainty is considered between site-class BC and site-class B (e.g., Boore and Atkinson, 2008).

General Methodology

In the probabilistic seismic hazard analysis, annual rate of ground motion exceeding a specific value is computed to account for different return periods of the hazard. Contributions from all the relevant sources and possible events are considered. The computational formulation as developed by Cornell (1968), Esteva (1970), and McGuire (1976) is given as follows,

$$\lambda(a > A) = \sum_i \nu_i \int_{m_{\min}}^{m_{\max}} \int_{r_{\min}}^{r_{\max}} P(a > A \mid m, r, \sigma) f_{m}(m) f_{r}(r) f_{\sigma}(\sigma) \, dm \, dr \, d\sigma$$  \hspace{1cm} (IV.15)

where $\lambda (a > A)$ is the annual frequency of exceedance of ground motion amplitude $A$, $\nu_i$ is the annual activity rate for $i^{th}$ seismogenic source for a threshold magnitude, function $P$ yields probability of the ground motion parameter $a$ exceeding $A$ given magnitude $m$ at source-to-site distance $r$. The standard deviation of the residuals (in log-normal distribution) associated with GMPE, denoted by $\sigma$ is also considered. The corresponding probability density functions are represented by $f_{m}(m)$, $f_{r}(r)$ and $f_{\sigma}(\sigma)$. The probability density function for the magnitudes is generally derived from the GR relation (Gutenberg and Richter, 1944). The present implementation makes use of the truncated exponential density function given by Cornell and Vanmarcke (1969),

$$f_{m}(m) = \frac{\beta \exp[-\beta (m - m_{\min})]}{1 - \exp[-\beta (m_{\max} - m_{\min})]} \hspace{1cm} (IV.16)$$
where $\beta = b \ln(10)$, $b$ refers to the $b$-value of GR relation. The distribution is bounded within minimum magnitude $m_{\text{min}}$ and maximum magnitude $m_{\text{max}}$. Instead of considering probability function for the source-to-distance measure explicitly, point source locations are adopted, wherein finite fault-ruptures are constructed based on the magnitude and underlying tectonic regime as discussed earlier. Two schemes adopted are smoothened gridded seismicity and uniform-seismicity areal zones (or uniformly smoothened), respectively. The smoothened seismicity is used to predict the activity rates representing spatial feature of earthquake occurrences. Each grid-point is associated with finite-fault rupturing formulated for the magnitude and the underlying focal mechanism. In this implementation, the annual activity rates vary spatially within the source zone but $b$-value and $m_{\text{max}}$ remain fixed. This assumes $b$-value and activity rate to be uncorrelated and a non-uniform distribution of earthquake probability within a zone. On the other hand, the uniform-seismicity consideration postulates each point within the areal zone to have equal probability for earthquake occurrences.

Ground motion variability constitute aleatory uncertainty intrinsic to the definition of GMPEs and consequently to that of PHSA. Computations based only on the median ground motions ignoring the associated variability are known to underestimate the hazards, especially at low annual frequencies of exceedance (Bender, 1984; Bommer and Abrahamson, 2006). Explicit treatment of uncertainty is generally achieved by integrating a number of times the standard deviation; the number of standard deviations is denoted by $\varepsilon$ and its maximum value by $\varepsilon_{\text{max}}$. The ground motions become unrealistically high with increasing $\varepsilon_{\text{max}}$ necessitating truncation at a specific value. Truncation at $\varepsilon_{\text{max}} < 3$ has been found to be inappropriate (e.g., Bommer and Abrahamson, 2006; Strasser et al., 2008). The upper bound of the ground motions (or constraining the physical limits on the ground motion values) is a topic of ongoing research (Bommer et al., 2004; Strasser and Bommer, 2009). The values of $\varepsilon_{\text{max}}$ ranging from 2 to 4 are usually employed (e.g., Bernreuter et al. 1989; Romeo and Prestininzi, 2000; Marin et al., 2004). These aspects allow considering $\varepsilon_{\text{max}} = 3$ to be pragmatic, and is accordingly adopted uniformly for all the GMPEs in the present study.

The hazard computation is performed on grid-points covering the entire study region at a spacing of 0.2°. Logic tree framework is employed in computation at each site to incorporate multiple models in source considerations, GMPEs and seismicity parameters. Figure IV.11 depicts a logic tree formulation at a site. In the present study, the seismogenic source framework represented by smoothed-gridded seismicity is collectively assigned weight equal to 0.6. The adopted two models corresponding to the threshold magnitude of $M_w$ 4.5 and $M_w$ 5.5 are further assigned weights equal to 0.45 and 0.55, respectively. The latter was derived using earthquake catalog having a longer period compared to the former, and therefore, entail higher weight. The seismicity model parameters are assigned weights of 0.36 while the respective $\pm 1$ standard deviation gets weight equal to 0.32. Similar weight allotment is assigned to $m_{\text{max}}$.

The computations are performed with the minimum magnitude equal to $M_w$ 4.5. This consideration is corroborated by seismic intensity attenuation models provided by Szeliga et al. (2010). The hazard distributions are computed for the source zones at each depth-section separately, and thereafter, integrated.
Figure IV.11: A logic tree formulation at a site for the source specified as that of tectonically active shallow crust region with predominant strike-slip faulting; the GPME framework for different tectonic regions is given in Figure IV.10 (after Nath and Thingbaijam 2011b).

Deliverables

Figure IV.12 depicts spatial distributions of PGA at 10% probability of exceedance in 50 years estimated for each hypocentral depth-section across active Himalayan tracts and northeast India. The smoothed-gridded seismicity source zonations have been exclusively considered in this case. The hazard contributions from the upper crust zones (0-25 km hypocentral depth range) cover the entire region. In case of lower crust zone, i.e. 25-70 km hypocentral depth range, higher hazards are concentrated in two regions namely west-central Himalayas, and northeast India. The intraslab earthquakes in Indo-Myanmar arc occurring in the depth range of 70-180 km constitute a major hazard contributor in northeast India. On the other hand, western parts of Kashmir are exposed to shallow as well as subduction earthquakes. Although not depicted in the figure, hazards from shallow as well as deep-seated earthquakes can also be noted for Andaman-Nicobar Islands. The results obtained for each depth range are integrated to establish the overall hazard distribution in the country. Figure IV.13 depicts hazard curves obtained at major cities in India.

The seismic hazard maps are presented in Figure IV.14. These correspond to spatial distribution of PGA, PSA at 0.2 sec and 1 sec computed for 10% and 2% probability of exceedance in 50 years, which correspond to return periods of 475 years and 2475 years, respectively. In the tectonically active region, higher hazard areas include extent of the Garhwal Himalayas, parts of western Kashmir, and northeast India. Western Gujarat and Koyna-Warna regions in the stable continental region exhibit higher hazard.
Furthermore, regions in and around Delhi, Jabalpur, Satpura, Latur, Bhadrachalam, Ongole, Bangalore, Chennai, Coimbatore, and Bengal basin, respectively have relatively higher hazards.

**Figure IV.12:** The spatial distribution of peak ground acceleration estimated for each hypocentral depth ranges. The computation corresponds to a return period of 475 years and is exclusively based on smoothed-gridded seismicity source zonations (after Nath and Thingbaijam 2011b).
Figure IV.13: The seismic hazard curves for selected cities (as indicated on each plot) computed for PGA, PSA at 0.2 sec and 1 sec, respectively for uniform firm rock site (after Nath and Thingbaijam 2011b).

Seismic Hazard Perspectives

Table IV.4 compares the computed PGA with those indicated by BIS (2002), GSHAP and earlier studies carried out at selected major cities in India. The computation carried out for 10% probability of exceedance in 50 years is considered for the purpose of comparisons. The present study yields comparatively higher hazard, which is pronounced in the high seismogenic regions. Jaiswal and Sinha (2007) estimated lower hazard in the highly seismogenic zones of Kutch, Gujarat, and Koyna-Warna regions. Same is the case with Menon et al. (2010) in Tamil Nadu. The present results are, however, similar to those of Anbazhagan et al. (2009) in Bangalore. In the northwestern Himalayas, Mahajan et al. (2010) estimated maximum PGA as high as 0.75 g. The present analysis associates the region with a maximum of about 0.60 g. In northeast India, the differences between results of Sharma and Malik (2006) and the present study are observed to be about 1.3 – 2.0 times less.
Figure IV.14: Seismic hazard distribution in India in terms of PGA, PSA at 0.2 sec. and 0.1 sec. for firm rock site conditions. Also included in the maps are the data for Nepal, Bhutan, Bangladesh and Srilanka (after Nath and Thingbaijam 2011b).
Table IV.4: Estimated peak ground accelerations with 10% probability of exceedance in 50 years at selected major cities across India by Bureau of Indian Standards (BIS, 2002), Global Seismic Hazard Assessment Program (GSHAP, Bhatia et al., 1999), present study, and other independent studies are listed. Except for BIS and the present study, the PGA estimate is the largest value obtained from the published map contours. The estimate is given for rock site condition or is otherwise indicated.

<table>
<thead>
<tr>
<th>City (Latitude, Longitude)</th>
<th>BIS (zone)(^1)</th>
<th>GSHAP</th>
<th>Present study</th>
<th>Additional notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ahmedabad (23.03°N, 72.61°E)</td>
<td>0.08 (III)</td>
<td>0.05</td>
<td>0.11</td>
<td>0.10 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
<tr>
<td>Bangalore (12.98°N, 77.58°E)</td>
<td>0.05 (II)</td>
<td>0.05</td>
<td>0.11</td>
<td>0.10 (Jaiswal and Sinha (2007)(^2), 0.15 (Anbazhagan et al., 2009)</td>
</tr>
<tr>
<td>Bhuj (23.25°N, 69.66°E)</td>
<td>0.18 (V)</td>
<td>0.20</td>
<td>0.42</td>
<td>0.25 (Jaiswal and Sinha, 2007)(^2), 0.20–0.70 (Petersen et al. 2004)(^3)</td>
</tr>
<tr>
<td>Chennai (13.00°N, 80.18°E)</td>
<td>0.08 (III)</td>
<td>0.05</td>
<td>0.12</td>
<td>0.089 (Menon et al., 2010), 0.10 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
<tr>
<td>Dehradun (30.33°N, 78.04°E)</td>
<td>0.12 (IV)</td>
<td>0.30</td>
<td>0.47</td>
<td>0.45 (Mahajan et al., 2010)</td>
</tr>
<tr>
<td>Guwahati (26.18°N, 91.73°E)</td>
<td>0.18 (V)</td>
<td>0.30</td>
<td>0.66</td>
<td>0.50 (Sharma and Malik 2006)</td>
</tr>
<tr>
<td>Hyderabad (17.45°N, 78.46°E)</td>
<td>0.05 (II)</td>
<td>0.05</td>
<td>0.09</td>
<td>0.08 (Jaiswal and Sinha 2007)(^2)</td>
</tr>
<tr>
<td>Imphal (24.78°N, 93.94°E)</td>
<td>0.18 (V)</td>
<td>0.45</td>
<td>0.68</td>
<td>0.50 (Sharma and Malik, 2006)</td>
</tr>
<tr>
<td>Jabalpur (23.20°N, 79.95°E)</td>
<td>0.08 (III)</td>
<td>0.10</td>
<td>0.19</td>
<td>0.15 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
<tr>
<td>Kolkata (22.65°N, 88.45°E)</td>
<td>0.08 (III)</td>
<td>0.10</td>
<td>0.15</td>
<td>0.10 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
<tr>
<td>Koyna (17.40°N, 73.75°E)</td>
<td>0.12 (IV)</td>
<td>0.25</td>
<td>0.47</td>
<td>0.25 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
<tr>
<td>Mumbai (19.11°N, 72.85°E)</td>
<td>0.08 (III)</td>
<td>0.10</td>
<td>0.16</td>
<td>0.15 (Jaiswal and Sinha, 2007)(^2)</td>
</tr>
</tbody>
</table>
In order to evaluate the updated seismic hazard analysis vis-à-vis the current provisions, eight cities are selected; two located in the specific seismic zone classified by BIS (2002). Figure IV.15 depicts plots of design response-spectra at 5% damping for firm rock site conditions at each city. The PSA at 0.2 sec and 1 sec, respectively for 2475 years return period are employed following the scheme outlined by IBC (2006 and 2009). It is observed that the provision given by BIS (2002) greatly underestimate the hazard distribution. The differences in the estimated hazard distribution compared to previous studies can be attributed to several factors:

1. In the present study, the GMPEs have been used as appropriate for different seismotectonic regimes. This aspect has been overlooked in most of the earlier studies; for instance Bhatia et al. (1999) employed a single equation for the entire country disregarding the different seismotectonic provinces, and Menon et al. (2010) inappropriately employed equations developed for tectonically active regions although their study region comes under stable continental region. More details have been given by Nath and Thingbaijam (2011b).

2. The layered seismogenic source framework based on hypocentral-depth distribution for the areal zonation, and smoothed-gridded seismicity models employed in the present study conforms to the variation of seismotectonic attributes with hypocentral depth, especially in subduction zones. This is a significant improvement over the previous studies where seismotectonic attribution has been oversimplified.

3. Multiple models for the seismogenic source, and seismicity parameters were not considered in most of the previous studies, except for Jaiswal and Sinha (2007) and Menon et al. (2010).
(4) The hazard computation in the present study incorporates the ground-motion variability, which incidentally could be a major reason for the estimation of comparatively higher hazard. Previous studies delivered the hazard estimates in terms of median (or mean) ground-motion values. As depicted in Figure IV.16, the observations at different cities indicate that the median ground-motion values are significantly lower, especially at lower annual exceedance rates.

Figure IV.15: Design response spectra (5% damped) for selected cities (after Nath and Thingbaijam 2011b).
Figure IV.16: Seismic hazard curves for PGA derived using present logic tree formulation with the ground motion truncated at different levels of standard deviation at major cities in India (after Nath and Thingbaijam 2011b).
APPENDIX — V

Typical Case Studies on Site Characterization

V.1 Assessment of Seismic Site Conditions: A Case Study from Guwahati City, Northeast India (Nath and Thingbaijam, 2010)

Guwahati City covers an area of about 600 km² bounded between the longitude 91°30′-91°50′E and latitude 26°05′-26°12′N. Owing to its strategic location, significant socioeconomic importance is attached to the City, which is the largest in the region and a home to a diversified and multi-ethnic population. Besides being a political state capital of Assam, it is also an active business hub and a host to several educational and economical institutions.

The City is located on the lower Brahmaputra basin between tectonic grains of the Himalayan mobile belt to the north and the Shillong plateau to the south. The geological and geomorphological background of the City is depicted in Figure V.1. The terrain is manifested mainly by interspersed Precambrian granitic rocks forming the hill tracts and isolated hillocks, and the valley filled with Quaternary alluvium deposited over the basement of hard granitic rocks. The steep sided hillocks in the southern and eastern fringes rise to an average height of 400 m above mean sea level (msl) while those within the valley elevate up to 300 m approximately above msl at several places. The hard granitic rock basement of the shallow basin exhibits the coincidence of the engineering and the seismic bedrock in most parts of the region. The alluvium deposits could be classified into five aggradational units based on lithological characters, state of weathering, order of superposition and unconformity. They, in order of increasing antiquity, those are (i) active flood plain and Levee deposit, (ii) Digaru surface, (iii) Bordang surface, (iv) Sonapur surface, and (v) Pediment surface. The sludge collected during drilling of boreholes up to 120 m depths for ground water exploration indicates that sand, silt, clay and gravel alternate in irregular proportion with extensive lateral variations. Significant disparities in the geological and geomorphological features implicate a likely variation in seismic response across the terrain.
Figure V.1: Geological and geomorphological map of Guwahati region depicting the locations of geotechnical boreholes, microtremor recording stations, seismic stations, and VES/lithological section profiles as shown by the lines AA', BB' and CC', respectively (after Nath et al., 2008b; Nath and Thingbaijam, 2010).

The damage patterns associated with the historical earthquakes are generally considered to be the indicator of seismic susceptibility of an urban establishment. Oldham (1899) reported widespread destructions in the City during the 1897 Great Shillong Earthquake (M_w 8.1) that occurred at a hypocentral distance of about 70 km away from the City; the seismic intensity observed being VII-VIII in Medvedev-Sponheuer-Karnik scale (Ambraseys and Bilham, 2003b; Bilham, 2008). Reportedly, the earthquake triggered liquefaction of considerable areas adjoining the Brahmaputra River inducing land-fissures, landslides, rockfalls, widespread slumping, and spreading of the ground between the hilly tracts on the southern bank of the river. In the recent times, population outburst in the City caused unplanned habitations and rampant construction of houses; encroachment of the hilly terrains, and reclamation of natural water bodies inadvertently increasing the seismic vulnerability of the terrain.

Data

With the aim of seismic hazard microzoning of the City, a consorted effort towards data compilation were undertaken involving deployment of geotechnical instruments, borehole drills, and extensive ground survey (DST, 2007). Geological Survey of India (GSI) conducted Vertical Electrical- Resistivity
Sounding (VES) employing the standard Schlumberger and Wenner techniques (e.g., Patra and Nath, 1999) at selected traverses in the City. Three layers of rocks have been identified at depths with resistivity values of 200, 100 and 25 Ohm-meter. The electrical resistivity observations and litho-stratigraphic data from selected 30 boreholes drilled to the basement for ground water exploration and those located closer to the hillocks have been used to derive the basement topography of the study region. The onset of the hard granitic layer overlying the alluvium overburden is considered as the basement. The basement, thus observed (exposed at several places), comprises of hard rock of granitic composition, wherein the engineering and seismic bedrock merges into one entity prohibiting identification of critical contrast between the two hazard related terminologies. Accordingly, the engineering bedrock considered in this study is defined by the rock condition having S-wave velocity between 760 m/s and 1500 m/s. SPT-N value data were collected from 200 boreholes drilled across the terrain, as depicted in Figure V.1, with the penetration up to 24-30 m in most of the cases. Typical subsurface conditions in the region are presented in Figure V.2, which depicts section profiles and basement zonation.

In another geophysical investigation, microtremor (ambient noise) survey was conducted jointly by India Meteorological Department (IMD) and GSI in the City. Several CMG-6TD (sensitive to 60-80 Hz) and SS-1 (sensitive to 1-50 Hz) seismometers with digital (Reftek and Kinematics) recorders were deployed at the locations as depicted in Figure V.1 with a total of 141 recording stations to account for approximately one station per square km. The exact locations of all the stations were determined through built-in GPS system with a precision of 0.0001 degree. The data acquisitions were conducted according to the recommendations of Bard (2000), and Mucciarelli (1998). Sufficient data covering 24 hours to several days were collected at each site addressing day, night and local variations to generate hourly data for spectrum analysis, each of which were sub-divided into 8 to 10 minutes providing 6 to 8 data files, and the pertinent uniform wave-train analyzed for any spurious inconsistency that may be induced by heavy traffic or spurious transients. New waveform data files were created by separating a portion of smooth common wave-train of about 120 seconds from each of the waveform data files. Appropriate data portion of each file were used for spectrum analysis according to the time window length selection criteria. A generally accepted rule of thumb is that the window length should have at least 10 cycles for the lowest frequency analyzed (e.g., Bard, 1998). In this study, 30 seconds time window length has been used.

Additionally, Kinematics Altus K2 strong-motion accelerographs were installed at twelve sites of significant landmarks/installations across the City by Indian Institute of Technology Guwahati (IITG), as a part of the seismic microzonation initiative. The dynamic range of the systems is 108 dB, 200 samples/sec and 18-bit resolution. Each of the systems has been set with a trigger level of 0.1% of the full-scale sensitivity (2g). The present analyses make use of five earthquakes that occurred during 2006 and were recorded with good signal-to-noise ratio (≥ 3) at several stations.
Figure V.2: (a) Typical subsurface conditions in the Guwahati City depicted by section profiles as indicated in Figure V.1: A-A’ (adapted from Nath, 2007a), B-B’, and C-C’ derived from VES and geotechnical borehole litholog data, (b) The basement zonation map of the region derived from VES and selected borehole data (adapted from Nath, 2007a).
Methodology and Results

The local soil is known to exhibit different predominant frequency depending upon its physical property and the sediment thickness, as well as variations due to focusing at the soil-bedrock interface and diversity in the layering and lateral continuity within various soil layers. The shear-wave velocity often increases with the confining pressure (e.g., Dobry and Vucetic, 1987), which in turn is proportional to the basement depth. As revealed by laboratory tests, increase in pressure leads to soil compaction and hence, an increase in the shear wave velocity. However, the variation of shear wave velocity within the stratum column in the sedimentary basins may not have monotonic trend owing to different material properties. The relation between shear wave velocity and the basement depth is, therefore, appropriate when average shear wave velocity is considered as a representative of the overall overburden. Similarly, predominant frequency \( f_r \) and the basement depth \( h \) can be connected by means of an average shear-velocity in a simplistic manner. For a single layer over a half space earth model, the average shear-wave velocity \( V_{S}^{av} \) can be written as,

\[
V_{S}^{av} = 4. h . f_r
\]

The predominant frequency is known to have inverse correlation with the basement depth. Ibs-Von Seht and Wohlenberg (1999) suggested the relation between the two parameters to follow a power law model,

\[
h = a \left( f_r \right)^b
\]

The average shear-wave velocity can also be related to the thickness in a similar fashion following Delgado et al. (2000),

\[
V_{S}^{av} = x . h^y
\]

The relations given by Equations (V.2) and (V.3) are reckoned to introduce durability in geo-spatial extrapolations/interpolations of the parameters. Further, they provide an expedient tool in the regions with similar geology for the first-order assessment of either parameter in case one parameter is known. Such relations act as thumb rule in geotechnical and geophysical investigations at regional and global perspectives, e.g. Cologne Area, Germany (Parolai et al., 2002), Bajo Segura Basin, SE Spain (Delgado et al., 2000), and Lower Rhine Embayment, Germany (Ibs-Von Seht and Wohlenberg, 1999), and in Southern Italy (D’amico et al., 2004).

Predominant Frequency

Horizontal-to-Vertical Spectral Ratio (HVSR) analysis on microtremor (ambient noise) measurements offers advantage of easy data acquisition, besides being inexpensive and reliable (e.g., Nakamura, 1989 and 2000; Luzón et al., 2001). The theoretical basis has been contentious and the approach mostly experimental. The HVSR curve has been implicated to body waves (e.g., Nakamura, 2000; Herak, 2008) as well as surface waves (e.g., Konno and Ohmachi, 1998; Arai and Tokimatsu, 2005). On the basis of theoretical modeling, Albarello and Lunedei (2009) observed surface-waves’ approximation to be consistent with the larger frequencies while the body waves’ interpretation to be concurring with the predominant frequency and its vicinity. At the same time, the dependence of HVSR on source distribution yields minor aberrations to the location of HVSR maximum. Several studies have demonstrated applicability of microtremor analysis, especially on soft soils where clear HVSR peaks can be ascribed to the contrast at the soil-bedrock interface (e.g., Field et al., 1995a; Lachet et al., 1996; Bard et al., 1997; Mucciarelli, 1998; Mucciarelli et al., 2003; SESAME, 2004; Gosar et al., 2009; Gallipoli et al., 2009a). The results from Site Effect Assessment using Ambient Excitations (SESAME, 2004) project under the European Commission delivered assessments on the HVSR technique, guidelines, and recommendations to probable applications.
In the present analysis, the SPEC program tagged with SEISAN (Havskov and Ottemoeller, 2000) has been used for the computation of Fourier spectra of individual component and relative spectra of horizontal versus vertical (H/V) components. The spectral amplitudes were accordingly smoothened with a one-third of octave band filter before computing HVSR, which is done till a distinct peak is obtained, and at times, going to 300 iterations (e.g., Al Yuncha et al., 2004). Figure V.3(a) depicts a few representative plots of the average HVSR against frequency (after Singh et al., 2008). The correlation between basement depth and predominant frequency is established as given by Equation (V.2). The basement depths obtained from VES at 34 points located in the proximity of the microtremor survey stations are used for the analysis. The correlation through orthogonal distanced regression assigning equal weightage to both the abscissa and ordinate, as depicted in Figure V.3(b), is deduced as,

\[ h = 125 (\pm 1.22) f_r^{-1.495 (\pm 0.407)} \]  

Equation (V.4) is similar to those given by Ibs-Von Seht and Wohlenberg (1999), and Parolai et al. (2002) observed respectively at Lower Rhine Embayment and Cologne Area, Germany. The estimated basement depths are consistent with the predominant frequency in the range of 0.5-3.5 Hz. It is to be noted that the exponent term amongst the three regional relations (Figure V.3(b)) does not vary significantly implying that in the generic model given by Equation (V.2), the coefficient 'a' is region specific (e.g., Delgado et al., 2000). The spatial distribution of predominant frequency in the study region is depicted in Figure V.4. Overall, different frequency responses are seen to conforming to the geomorphological signatures in the study region.
Figure V.3:  (a) Representative plots of HVSR against frequency in the Guwahati City - the mean values are given by continuous curves while the standard deviation of each is depicted by the grey shades, (b) The basement depth against predominant frequency plot wherein different correlating models have been depicted (after Nath and Thingbaijam, 2010).

Figure V.4: Predominant frequency distribution map of the Guwahati City (after Nath and Thingbaijam, 2010).
Shear-Wave Velocities

The shear-wave velocity is used in earthquake engineering for site response modeling (e.g., Schnabel et al., 1972). The parameter relates to physical properties of the surface deposit/weathered layer such as fabric, aging, compaction, structure, constituents, degree of weathering, and alterations of seismic motions. The fundamental model given by Equation (V.3) connects average shear-wave velocity to basement depth. We compute the average shear-wave velocity through Equation (V.1) using the basement depths and the observed predominant frequency at the selected microtremor observation sites. Non-linear regression analysis performed thereafter on the data-pairs of average shear-wave velocity and basement depth, as shown in Figure V.5, yields the following,

\[
V_s^{av} = 101 (\pm 14.9).h^{0.33 (\pm 0.052)} \tag{V.5}
\]

The relation shows higher uncertainties with the increasing basement depth. As depicted in the diagram, the function is close to the one given by Parolai et al. (2002). The average shear-wave velocity ascribes to the overburden thickness. However, site classification schemes are generally based on the shallower depth in view of the near-surface affinity to site effects besides the cost and conveniences for the engineering practices (e.g., Field et al., 2000; Street et al., 2001). In fact, Joyner et al. (1981) observed that the velocity to a depth equal to one-quarter wavelength of the period of the propagating wave train from resolution point of view represents site conditions appropriately. Several studies, to name a few, Borcherdt et al. (1991), Boore et al. (1997), Dobry et al. (2000), and Field et al. (2000) have formulated site amplifications as a function of average shear-wave velocity for the upper 30m of the subsurface, i.e. effective shear-wave velocity ($V_s^{30}$). The parameter is also used by both the NEHRP and the Uniform Building Code (UBC) to classify sites for earthquake engineering (BSSC, 2001). The parameter has also been incorporated as the site term in the ground motion prediction equations (e.g., Powers et al., 2008).
A straightforward and widely used technique for shear-wave velocity estimation is the conventional SPT data analysis. Several empirical relations between SPT N-values and lithological shear-wave velocity ($V_s$) are available as listed in Table V.1 that are often used to overcome the absence of in-situ measurements with explication of their applicability for a study region. The present study region with Archean hillocks exposed at places, and undulating basement topography bearing Precambrian gneissic complex falls in the category of thin alluvium characterized by smaller basins underlain by harder bedrock. Higher shear-wave velocity range is usually observed in such geological settings as compared to the deeper basins (e.g., Wills and Clahan, 2006).

**Table V.1:** Several relations between SPT N-value and shear wave velocity, $V_s^{30}$, available for different soil types

<table>
<thead>
<tr>
<th>Soil-Type</th>
<th>Relations</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>$V_s^L = 5.3 N + 134$</td>
<td>Fumal and Tinsley (1985)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 114.4 N^{0.31}$</td>
<td>Lee (1990)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 165.7 N^{0.19}$</td>
<td>Pitilakis et al. (1992)*</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 105.7 N^{0.33}$</td>
<td>Raptakis et al. (1995)*</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 184.2 N^{0.17}$</td>
<td>Raptakis et al. (1995)*</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 27.0 N^{0.73}$</td>
<td>Jafari et al. (2002)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 80.2 N^{0.292}$</td>
<td>Imai (1977)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 97.9 N^{0.269}$</td>
<td>Hasancebi and Ulusay (2007)</td>
</tr>
<tr>
<td>Sand</td>
<td>$V_s^L = 5.1 N + 152$</td>
<td>Fumal and Tinsley (1985)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 56.4 N^{0.50}$</td>
<td>Seed et al. (1983)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 57.4 N^{0.49}$</td>
<td>Lee (1990)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 162.0 N^{0.17}$</td>
<td>Pitilakis et al. (1992)*</td>
</tr>
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<td>$V_s^L = 100.0 N^{0.24}$</td>
<td>Raptakis et al. (1995)</td>
</tr>
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<td>$V_s^L = 123.4 N^{0.29}$</td>
<td>Raptakis et al. (1995)*</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 80.6 N^{0.331}$</td>
<td>Imai (1977)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 100.5 N^{0.29}$</td>
<td>Sykora and Stokoe (1983)</td>
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<td>$V_s^L = 90.8 N^{0.319}$</td>
<td>Hasancebi and Ulusay (2007)</td>
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<td></td>
<td>$V_s^L = 22.0 N^{0.76}$</td>
<td>Chien et al. (2000)*</td>
</tr>
<tr>
<td>Gravel</td>
<td>$V_s^L = 192.4 N^{0.13}$</td>
<td>Raptakis et al. (1995)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 63.0 N^{0.43}$</td>
<td>Sykora and Koester (1988)</td>
</tr>
<tr>
<td>Silt loam/ sandy clay</td>
<td>$V_s^L = 4.3N + 218$</td>
<td>Fumal and Tinsley (1985)</td>
</tr>
<tr>
<td>All</td>
<td>$V_s^L = 91.0 N^{0.34}$</td>
<td>Imai (1977)</td>
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<td></td>
<td>$V_s^L = 85.35 N^{0.348}$</td>
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<td>Yokota et al. (1991)*</td>
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<td>$V_s^L = 61.0 N^{0.50}$</td>
<td>Seed and Idriss (1981)</td>
</tr>
<tr>
<td>Soil-Type</td>
<td>Relations</td>
<td>Author(s)</td>
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<td>---------------------</td>
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<tr>
<td></td>
<td>$V_s^L = 107.6 N^{0.36}$</td>
<td>Athanasopoulos (1995)</td>
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<td></td>
<td>$V_s^L = 22.0 N^{0.85}$</td>
<td>Jafari et al. (1997)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 116.1 (N + 0.3185)^{0.202}$</td>
<td>Jinan (1987)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 51.5 N^{0.516}$</td>
<td>Iyisan (1996)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 97.0 N^{0.314}$</td>
<td>Imai and Tonouchi (1982)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 82.0 N^{0.39}$</td>
<td>Ohsaki and Iwasaki (1973)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 92.1 N^{0.337}$</td>
<td>Fujiwara (1972)</td>
</tr>
<tr>
<td></td>
<td>$V_s^L = 90.0 N^{0.309}$</td>
<td>Hasancebi and Ulusay (2007)</td>
</tr>
</tbody>
</table>

Φ: Soft clay, Θ: Stiff clay, ▽: Silty sand, Ψ: Loose sand, " Adapted from Jafari et al. (2002)

Figure V.6: Several relations between SPT N-value and shear wave velocity as depicted along with the median trend (bold line) and the dashed lines representing 95% prediction bounds. It is observed that: (a) in the case for clay and clayey soils, the median trend to be well aligned to that of Fumal and Tinsley (1985) while (b) in the case of sand and gravelly sand including gravel, the relation given by the same authors is reasonably consistent with the median at lower N-value however yields higher shear-wave velocity as the N-value increases, (c) Available relations between SPT N-value and the shear-wave velocity for all types of soil conditions plotted together with those of Fumal and Tinsley (1985) classified on the basis of soil textures.
It has been observed that $V_s$ values cluster around the mean values for different soil types (e.g., Ohsaki and Iwasaki, 1973; Tonouchi et al., 1983). Loose soil conditions such as that of sands are expected to exhibit lower $V_s^{\text{av}}$; however, for the same N-value, which is indicative of soil compactness, sand can exhibit higher velocity (e.g., Imai, 1977; Ohta and Goto, 1978; Fumal and Tinsley, 1985; Hasancebi and Ulusay, 2007). Fumal and Tinsley (1985) implicated ‘sandy clay’ and ‘loam soils’ to higher $V_s^{L}$ for similar N-values compared to other soil types. The control of N-value $V_s^{L}$ over, notwithstanding the influence of the soil type, yields average relations when soil types are not considered to account for a larger number of observations (e.g., Hasancebi and Ulusay, 2007). On the other hand, the soil classifications enable geologically constrained predictions. The equations of Fumal and Tinsley (1985) based on different soil textures namely ‘clay and silty clay’, ‘silt loam and sandy clay’, and ‘sand and gravelly sand’, are preferred in the present study due to their coherency with the observed soil lithology. The implications, thereupon, are examined through comparisons with that of several available correlations via assessment of a linear median trend through least squares fit that employs minimization of the least absolute residuals. The relation for ‘clay and silty clay’ is seen comparatively moderate as shown in Figure V.6(a). Although the relation for ‘sand, gravelly sands, and gravels’ yields higher $V_s$ values at larger N-values as depicted in Figure V.6(b), it is reasonably consistent with the median at the lower N-values. Figure V.6(c) presents an overall comparison between the three relations and those available for all soil types indicating that the relations for clayey and sandy soils are closer to the median trend while that for ‘silt loam and sandy clay’ yields higher $V_s$ estimates for the same N-values. However, the latter trend matches satisfactorily with that given by Athanasopoulos (1995). The estimation of shear-wave velocity is depicted with two representative borehole logs given in Figure V.7(a).

Eventually, $V_s^{30}$ is computed at each borehole site. However, in cases where the bedrock is found to be located within a depth of 30 m, the average shear-wave velocity of the overburden thickness, rather than $V_s^{30}$, is considered as the presence of rock would escalate $V_s^{30}$. The spatial distribution of $V_s^{30}$ is depicted in Figure V.7(b). Although the estimated velocities range from 180 to 360 m/s, the maximum number of observations is within 220-280 m/s.
Figure V.7: (a) The soil strata, SPT-N, and the corresponding shear-wave velocity profile depicted for two closely located boreholes, (b) The spatial distribution of $V_{S}^{30}$ across Guwahati City. The estimated $V_{S}^{30}$ at the borehole sites is seen to range between 180 and 360 m/s with the maximum number of observations exhibit a range of 220-280 m/s (after Nath and Thingbaijam, 2010).

Soil Liquefaction, Site Classification, and Site Response

NEHRP recommends provisions for seismic regulations according to site classes (A to E) defined for similar seismic responses. The site classes A and B are assigned to hard rock and rock site conditions with $V_{S}^{30} > 1500$ ms$^{-1}$, and within 760-1500 ms$^{-1}$, respectively while site class C is designated to soft rock, hard or very stiff soils or gravels exhibiting $V_{S}^{30}$ in the range of 360-760 ms$^{-1}$, and stiff soils with $V_{S}^{30}$ in the range of 180-360 ms$^{-1}$ designated to be in site class D. However, site class E is implicated to a soil profile with more than 3 m of soft clay defined as soil with plasticity index (PI) >20, moisture content (w) ≥40 percent, and average undrained shear strength (Su) <25 kPa while site-specific testing and evaluation is warranted for the identification of site class F. Either of the four categories of soil are considered to be indicative of site class F: (i) soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils, (ii) peats and/or highly organic clays (soil thickness >3 m) of peat and/or highly organic clay, (iii) very high plasticity clays (soil thickness >8 m with PI >75, and (iv) very thick soft/medium stiff clays (soil thickness >36 m). Geological attributes are often connected to shear-wave velocity in view of limited number of observation sites (e.g., Wills and Silva, 1998). On the basis of the overlapping ranges of $V_{S}^{30}$ accorded to different geological units, Wills et al. (2000) introduced intermediate classes namely BC, CD, and DE corresponding to an average $V_{S}^{30}$ of 760 ms$^{-1}$, 360 ms$^{-1}$ and 180 ms$^{-1}$, respectively.
Soil Liquefaction

In the present study, the alluvial deposits of mainly Pleistocene-Holocene sediments in the overall terrain predominantly come under site class D as inferred from $V_{30}^s$ distribution. However, soft soil sites and marshy lands (reclaimed at several places) over the active flood plain and levee deposits, and Digaru and Bordang surfaces are liable to come under site class E or F. While site class E is identified from geotechnical considerations, soil liquefaction susceptibility is evaluated in the study region to identify site class F. The resistance of the soil to earthquake-related liquefaction mainly depends on density, fabric, cementation and age of the soil, whereas the susceptibility increases with earthquake magnitude, duration, and the extent of shear-stress reversal. The present investigation is based on data acquired at each borehole at down-hole intervals of 1.5 m. The water table is seen mostly at 1.5 m but ranges from 0.0 to 6.6 m across the borehole dataset. The fine grain criteria based on two factors namely the ratio of water content to liquid limit ($w_c/LL$) and soil plasticity index (PI) as proposed by Bray and Sancio (2006) is used to evaluate the susceptibility of plastic soils. The susceptible zones are identified with $w_c/LL > 0.85$ and PI $< 12$ while moderately susceptible zones are identified with $w_c/LL > 0.8$ and PI $< 18$. The Factor of Safety (FOS) against soil liquefaction is designated to be less than 1 and equal to 1, respectively in the former and the latter cases. Thereafter, the methodology based on SPT data analysis given by Youd et al. (2001) is followed for non plastic soils. It involves estimation of Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) to arrive at the Factor of Safety (FOS) – ratio of the two factors whose value equals to one or less than one indicating soil liquefaction susceptibility. The seismic demand of a soil layer is represented by CSR while the capacity of soil to resist liquefaction is represented by CRR. The shear stress generated at the soil column excited by a propagation of shear wave velocity at the base can be computed as,

$$\left(\tau_{\text{max}}\right)_d = 0.65 r_d \sigma_0 \left(\frac{a_{\text{max}}}{g}\right)$$

(V.6)

Where $(\tau_{\text{max}})_d$ represents maximum shear stress, $\sigma_0$ is the total overburden pressure, $a_{\text{max}}$ is the peak horizontal acceleration, $r_d$ is a correction factor for deformable body, which is termed as stress reduction coefficient and $g$ represents acceleration due to gravity (Seed and Idriss, 1971). In the present study, the average values of $r_d$ are estimated using the relations given by Cetin et al. (2004). The CSR is computed as,

$$CSR = 0.65 \frac{\sigma'_0 a_{\text{max}}}{\sigma_0 g} r_d$$

(V.7)

Where $\sigma'_0$ is the initial effective overburden pressure, which is given as,

$$\sigma'_0 = d_j \left(\sum d_i \gamma_i \right) - 9.81 \left(d_j - w_l\right)$$

(V.8)

Where $d_j$ is the depth interval of each N-value, $d_i$ is the depth at which the soil lithology changes, $w_l$ is the depth of water table, and $\gamma_i$ is the average unit weight of the material. The CRR values are estimated from $N_{1(60)}$ values using the relationships provided by Youd et al. (2001). These relationship curves were developed for granular soils with the fine contents of 5% or less, 15%, and 35% for magnitude $M_w$ 7.5 earthquakes. The magnitude scaling, e.g. $M_w$ 8.7, is achieved by means of the magnitude scaling factor (MSF) such that,
Following Youd et al. (2001), the magnitude scaling factor (MSF) is computed as,

\[
MSF = \frac{CRR_{7.5}}{CSR}^{2.56} \tag{V.10}
\]

The scaling factor provides an approximate representation of the effects of shaking duration or equivalent number of stress cycles enabling the adjustment of the induced CSR during earthquake for a specific magnitude to an equivalent CSR for a magnitude of \( M_w 7.5 \).

For the assessment of peak horizontal accelerations, synthesized base motion for scenario earthquake magnitude of \( M_w 8.7 \) nucleating from the hypocenter of the 1897 Shillong earthquake simulated by Nath et al. (2009) is used. A representative synthetic accelerogram obtained at Cotton seismic station is depicted in Figure V.8.

**Figure V.8:** A representative synthetic accelerogram generated through stochastic finite fault modeling at the bedrock level for a reverse faulting earthquake with scenario magnitude of \( M_w 8.7 \) nucleating from Shillong plateau with a hypocentral distance of \(~70 \text{ km}\) (after Nath and Thingbaijam, 2010).

**Figure V.9:** Site classification map of Guwahati City prepared following an extended National Earthquake Hazard Reduction Program (NEHRP) nomenclature of Wills et al. (2000). The likely liquefaction sites during the 1897 Shillong earthquake are depicted with square boxes.
Soil liquefactions associated with 1897 Shillong earthquake in the Guwahati City was not fully reported probably due to the fact that the earthquake-related phenomenon was not well-known during that period of time. The 1897 Shillong earthquake induced wide-spread soil liquefaction and deformations in the Guwahati City can be inferred from the accounts of Oldham (1899), and Bilham (2008). The latter discussed several personal observations of La Touche, a geologist with GSI during the period. From these reports, we could gather several information such as (1) deformations were observed along the banks of the Brahmaputra River, some of which are likely to be associated with soil liquefaction, (2) sands and floods with fissures were reported with devastation of crops at unnamed locations, and (3) fissures, induced likely by soil liquefaction, developed across the road connecting Sukleswar Ghat and Bhorolumukh on the southern bank of the River. The likely liquefaction sites during the 1897 Shillong earthquake could be indicated with inscribing square boxes on the site classification map given in Figure V.9.

**Site Classification**

The site classification of the City is eventually achieved by demarcating the zones of site classes CD, D, E and F as presented in Figure V.9. For the liquefaction susceptibility zonation purpose, a buffer area of 500 m around each identified site has been created on Geographical Information System (GIS) platform and modified, thereafter, according to the geological conditions in the immediate vicinity. The zones identified as liquefiable soil (FOS < 1) and at the point of failure (FOS = 1) are, irrespectively, classified as site class F. On the other hand, the low lying Precambrian residual hills interspersed across the terrain are classified as rock sites. The narrow tracts underlying older (Pleistocene) alluvium, gravel and soft rocks are placed under the site class CD.

The HVSR can be inverted through Monte Carlo perturbation of the initial model parameters to generate best fit soil models (e.g., Herak, 2008). This analysis can provide a supplementary analysis to reinforce the present observations and provide yet another perspective to overcome geophysical ambiguity. In the present study, a representative geotechnical model is derived using the observed average HVSR spectra computed from the earthquake data. In case of ambient tremors, anthropogenic origin consisting mostly of horizontally traveling surface waves could violate the fundamental assumption of body-wave field and, therefore, needs critical examinations. A Matlab® code developed by Herak (2008) based on the excitation of the body-wave field has been used for the purpose. The algorithm for the inversion to determine the best fit soil model has been described by the author. The methodology is based on Monte Carlo search in the model space minimizing the misfit function,

\[
m = \sum_i \{ (HVSR_{obs} (f_i) - HVSR_{theo} (f_i)) W_i \}^2 \quad (V.11)
\]

\(HVSR_{obs}\) and \(HVSR_{theo}\) stand for the observed and the theoretical HVSR, respectively. \(W_i\) represents the weights assigned to each of the frequency with the highest being allocated to the largest HVSR value. The analysis at the COTTON station is illustrated as follows. The model space consists of six parameters for each layer (excluding the half-space) as listed in Table V.2. \(HVSR_{theo}\) is computed as the ratio of S-wave and P-wave amplifications, respectively considering that the horizontal and vertical motions are same on an average at the engineering bedrock level. The final model achieved is depicted in Figure
V.10. The results at the seismic stations indicate conformity of site classification with $V_s^{30} = 212$ m/s at IITG, $V_s^{30} = 238$ m/s at IRRIG, $V_s^{30} = 187$ m/s at COTTON, $V_s^{30} = 327$ m/s at AMTRON, $V_s^{30}$ and = 167 m/s at AEC. It is to be noted that inversion problems can be ill-posed i.e., different models could provide same solutions; in that respect, the present results corresponds to a solution constrained by the initial model parameters.

Table V.2: The initial model constructed using geotechnical data from the adjacent borehole site and the final models of the 1D soil profile at the COTTON station consisting of S-wave and P-wave velocity ($V_s$ and $V_p$), density $\rho$, and S-wave and P-wave attenuation coefficients ($Q_s$ and $Q_p$) respectively.

<table>
<thead>
<tr>
<th>$V_p$ (m/s)</th>
<th>$V_s$ (m/s)</th>
<th>$\rho$(gm/cc)</th>
<th>$h$ (m)</th>
<th>$Q_p$</th>
<th>$Q_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Initial model</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>596</td>
<td>266</td>
<td>1.78</td>
<td>6</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>466</td>
<td>208</td>
<td>1.78</td>
<td>6</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>671</td>
<td>300</td>
<td>2.2</td>
<td>6</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>648</td>
<td>290</td>
<td>2.23</td>
<td>6</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>614</td>
<td>274</td>
<td>2.25</td>
<td>6</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>1192</td>
<td>637</td>
<td>2.25</td>
<td>15</td>
<td>108</td>
<td>54</td>
</tr>
<tr>
<td>1192</td>
<td>637</td>
<td>2.25</td>
<td>15</td>
<td>108</td>
<td>54</td>
</tr>
<tr>
<td><strong>Final model</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>180 ($\pm$18)</td>
<td>50 ($\pm$05)</td>
<td>1.70 ($\pm$0.17)</td>
<td>6</td>
<td>29 ($\pm$3)</td>
<td>19 ($\pm$2)</td>
</tr>
<tr>
<td>435 ($\pm$44)</td>
<td>109 ($\pm$10)</td>
<td>1.74 ($\pm$0.17)</td>
<td>6</td>
<td>37 ($\pm$4)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>844 ($\pm$85)</td>
<td>383 ($\pm$38)</td>
<td>2.04 ($\pm$0.20)</td>
<td>6</td>
<td>37 ($\pm$4)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>577 ($\pm$58)</td>
<td>189 ($\pm$19)</td>
<td>2.3 ($\pm$0.23)</td>
<td>6</td>
<td>47 ($\pm$5)</td>
<td>16 ($\pm$2)</td>
</tr>
<tr>
<td>625 ($\pm$63)</td>
<td>206 ($\pm$21)</td>
<td>2.3 ($\pm$0.23)</td>
<td>6</td>
<td>47 ($\pm$5)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>1263 ($\pm$126)</td>
<td>316 ($\pm$32)</td>
<td>1.7 ($\pm$0.17)</td>
<td>15</td>
<td>47 ($\pm$5)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>1493 ($\pm$150)</td>
<td>540 ($\pm$54)</td>
<td>1.7 ($\pm$0.17)</td>
<td>15</td>
<td>60 ($\pm$6)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>1244 ($\pm$125)</td>
<td>719 ($\pm$72)</td>
<td>2.3 ($\pm$0.23)</td>
<td>15</td>
<td>60 ($\pm$6)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td>1765 ($\pm$177)</td>
<td>1020 ($\pm$102)</td>
<td>2.3 ($\pm$0.23)</td>
<td>15</td>
<td>37 ($\pm$4)</td>
<td>20 ($\pm$2)</td>
</tr>
<tr>
<td><strong>1762</strong>&lt;sup&gt;a&lt;/sup&gt;</td>
<td>788&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2.25&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$\infty$&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$\infty$&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$\infty$&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup>fixed values
Site Characterization Studies

**Site Response**

The HVSR, better known as Nakamura’s technique (Nakamura, 1989), is applicable only to ambient noise driven site effects in terms of predominant frequency. In case of the recorded earthquake waveform data, there are three widely used site response estimation techniques, namely classical standard spectral ratio (SSR), generalized inversion (GINV), and HVSR receiver function (RF) type. In case of SSR, there is requirement of reference site, whose site response is flat and modulated around the amplification factor of 1. At the same time, GINV necessitates constraining inversion scheme with a known site response at one or more sites. On the other hand, the RF type does not necessarily have the requirement of either a reference site or site/s with known site response. In the present study, we have a global problem in the application of the first two techniques due to the absence of a reference site. It has been observed globally by several workers that SSR and GINV are consistent in the assessment of site amplifications. However, there are several opinions in regards to site amplification estimate through RF methodology that has emerged globally through experimentations carried out in different terrains by several workers. Lermo and Chavez-Garcia (1993) proposed H/V ratio (RF type) using intense parts of S-wave envelope for site response assessment, and observed that the RF estimates mimic the site amplifications computed using the classical SSR on the basis of analyses carried out at three different cities in Mexico. Parolai et al. (2004), Nath et al. (2005, 2003 and 2002b), Bonilla et al. (1997), Chávez-García et al. (1997), Lachet et al. (1996), and Field and Jacob (1995) reported scattering in the estimated amplitudes computed using RF technique while compared to those estimated by SSR. Overall, the RF technique conforms to simple geological consideration where amplifications on the vertical components are negligible (Parolai et al., 2004; Nath et al., 2002b), especially in the lower frequencies range (<8 Hz), which does not have
severe contamination from surface waves (Lachet et al., 1996). Lachet et al. (1996) demonstrated that the consistency of RF technique is frequency dependent with the reliable range falling below 8Hz. The data from the site response studies in the Sikkim Himalayas, Delhi basin, and Garhwal Himalayas, respectively from Nath et al. (2005, 2003 and 2002b) as depicted in Figure V.11, depict reasonable compliance between the site amplifications derived using the RF type and GINV techniques. Castro et al. (2004) also observed that site functions computed by both the techniques are similar within the standard deviations of the estimates at several sites. Since the predominant frequency observed at the seismic stations comes within the pertinent lower frequency range in the present study at Guwahati City and seldom crossing the 8Hz limit as suggested by Lachet et al. (1996), the RF technique as suggested by Lermo and Chavez-Garcia (1993) can be expected to define at least the lower bound for the site amplification estimates with significant applicability in site characterization.

Figure V.11: A comparison between site amplification factor obtained using Generalized Inversion (GINV) technique and those obtain by means of Receiver function (RF) type method using the data from Nath et al. (2005, 2003 and 2002b).

The 1D site response distribution in the study region has been estimated by Nath et al. (2008a) through SHAKE 2000 (Ordonez, 2004) at the borehole sites. An equivalent linear model was used in the analysis, which accounts for recurring and systematic cyclic loading on horizontally layered soil deposits with expected elliptical hysteresis loops in the shear stress and strain behavior. The shear modulus and damping reduction curves were assigned on the basis of soil attributes from the geotechnical data according to inbuilt functions of the software. The curves used for clay and sand are those given by Sun et al. (1988), and Seed and Idriss (1970) respectively. The mean site amplification factor derived from Geotechnical modeling, as depicted in Figure V.12(a), is observed to be around 3 in the frequency range 2-4 Hz. The average site response curves from the earthquake data through frequency averaging at each seismic station is depicted in Figure V.12(b-f).
Figure V.12: (a) Mean site response amplification factor from the estimated amplification spectra by means of geotechnical modeling at 136 borehole sites in the site class D zones across the Guwahati City is depicted by bold curve while the normal curves indicate ±1 standard deviations. (b), (d), (c), (d), (e) and (f) Average site response curves through frequency averaging observed at the seismic stations (after Nath and Thingbaijam, 2010).
Discussion

A strong earthquake occurred on September 21, 2009 with a magnitude of $M_w$ 6.1 in the eastern Bhutan province at the epicenter 27.346°N, 91.412°E, which is around 133 km NNW of the Guwahati City. Several high rise buildings in the City developed cracks, and incidentally, a seven-storey building located at Bhangagarh area (under site class F) tilted due to the earthquake impact corroborating the present assessment of soil conditions.

Predominant Frequency

The structural damages inflicted by an earthquake are logically tied with appropriate seismic design code adaptations. Nevertheless, the damage distributions are often seen to be dependent on resonance effect due to proximity between the predominant frequency of soil layers and natural frequency of the buildings (e.g., Navarro and Oliveiram, 2006). In order to present a rudimentary assessment, a current building footprint map of the City as depicted in Figure V.13(a) is adapted from the building inventory compiled by Assam Engineering College. The building typology, namely Type A (mud, adobe, timber frame), Type B (brick masonry buildings), and Type C (reinforced concrete buildings), is followed according to BMTPC (1997). The categorization based on the number of stories of 1-3 is defined as ‘low rise’, 4-7 ‘moderately rise’, and ≥8 as ‘high rise’. The assortment of residential building distribution consists mostly of low rise buildings (Type A, B and C), while the commercial locations have predominantly moderate to high rise buildings (Type C).
Figure V.13: (a) The building footprints in the City adapted from the Inventory prepared by Assam Engineering College as a part of consorted effort towards seismic microzonation and risk assessment in the City, (b) The relationships between fundamental natural frequencies versus building heights (in terms of number of storeys and 1 storey ~3.0 m) are depicted. The grey shaded zone covers the predominant frequency range 0.5-3.5 Hz observed across the inhabited areas of Guwahati City, and dash lines denotes the average upper and lower values of the overall predominant frequency distribution. The categories of buildings are indicated as low, moderately and high rise, respectively according to BMPTC (1997).

Figure V.13(b) depicts different relations between height of the building and the fundamental natural period. Most of the inhabited parts of Guwahati City are interspersed between the hillocks. Low-rise to moderate-rise buildings are predominately observed there while the predominant frequency of the ground mostly lies in the range 0.5-3.5 Hz. The relationship given by Gallipoli et al. (2009b) puts the building distribution in seismically safe-zone while the other relationships, viz. those of Bureau of Indian Standards (BIS, 2002), indicate that moderate-rise to high-rise buildings are seismically vulnerable. The different predictions given by the relationships mentioned here can be attributed to the diversity of the construction practices across the globe, especially between India and Europe. All the same, the observations in the present study region suggest that moderate- to high-rise buildings in the City are seismically vulnerable. This warrants tackling of the implicated resonance effects with measures like structural retrofitting (e.g., stiffening, introducing damping in building components etc.) besides deciding building code provisions for future developments. In summary, the spatial distribution of predominant frequency constitutes a first cut hazard map not only useful in vulnerability analysis of built environment but also in the structural development and urban planning. The present appraisal is intended to be followed by building and ward specific analysis (e.g., Gosar et al., 2009; Gallipoli et al., 2009b).

Shear Wave Velocity, Site Classification, and Site Response

In the present study, region specific relations have been derived between shear-wave velocity, predominant frequency, and basement depth in line with those observed in other parts of the globe. The borehole logs of two sites, as depicted in Figure V.7(a) located adjacent to a microtremor survey location near Boragaon, exhibits the basement depths to around 27.5-29.0 m. On the other hand, the electrical resistivity sounding surveys places the basement depth at the site to be approximately 30 m. The corresponding microtremor
data analysis indicates a predominant frequency of 2.5 Hz that implicates a basement depth of
approximately 32 m through Equation (V.4). Consequently, average shear wave velocity in the range of
301-310 m/s could be estimated through Equation (V.5) with basement depths ranging between 27.5-
30.0 m. The estimated \( V_{S}^{30} \) at the boreholes ranges between 242 and 287 m/s. The relations apparently
yield reasonable estimates of the parameters suggesting consistency thereof.

In case of the applicability of relationships between shear-wave velocity and SPT N-value, a strategy
has been adopted to put forth in the knowledge of litho-stratigraphic setting in the study region through
several analyses involving comparisons of existing relations reported from across the globe by various
workers. In order to establish the practical viability of the strategy, the site responses are derived from 1D
géotechnical modeling and the earthquake recordings located in the close vicinity and calibrated. Figure
V.14(a) depicts a comparison between estimated site amplification factors from the 1D géotechnical
modelling using the borehole data, and that evaluated from the ground motion data. A frequency spacing
of 1 Hz has been used to compare the site response curves for a generalized assessment as explained
in Figure V.14(b). The simulations of accelerograms show that the géotechnically derived site response
yields average acceleration spectra compared to those simulated using the site response calculated
from the earthquake data (Figure V.14(b)), although lower compared to those estimated from the data in
the higher frequency band (>5 Hz). The difference might be attributed to the underestimation of high
frequency site amplifications from non-linear effects (e.g., Park and Hashash, 2004).

The present study adheres to the NEHRP site classification scheme. Nevertheless, we appraise
the observed average maximum site amplifications vis-à-vis predominant frequency and \( V_{S}^{30} \). Midorikawa
(1987) observed that the mean amplification factors for geological unit of Holocene origin to be around 3
in the frequency range of 0.4 to 5 Hz. The author correlated Site Amplification Factor (SAF) with \( V_{S}^{30} \) as,

\[
SAF = 68(V_{S}^{30})^{0.6}
\]  

(V.12)
The above relation implies amplification factor equal to 2.0-2.8 for site class D in the region, which is comparable to the mean site amplification factors obtained through 3-points frequency averaging of site response estimated through the geotechnical modeling pertinent to site class D. NEHRP site class F is qualified with the soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

Figure V.15(a) shows that the geotechnical modeling, earthquake ground motion and ambient noise data analyses provide efficacy to the 30 m depth soil column around the predominant frequency of 2 Hz. In case of IITG, the area underlies a shallow basement where the pertinent soil condition comes under site class F. As observed in Figure V.15(b), maximum site amplifications inversely correlate to predominant frequencies i.e., higher amplifications at the lower predominant frequencies and vice versa. A rather high site amplification factor of 7.5 on an average is seen at IRRIG (with predominant frequency of about 2.0 Hz at a shallow basement depth and relatively higher ground topography going up to 130 m) which could be indicative of basin edge effect. Although similar predominant frequency values are observed at COTTON (site class E) and AMTRON (site class D), the locations are far from edges of the basin. Overall, the study region consists of small valley interplays causing irregular micro-basin features in the parent Brahmaputra basin. In this respect, the surficial geometry along with varying predominant frequency may further subdivide the broad NEHRP site classification implementation. Indeed, a number of recent studies demonstrated that in case of diverse geological, and geotechnical contexts, the site effects may not always be accurately represented by a simple index such as $V_{s30}$ (e.g., Park and Hashash, 2004; Semblat et al., 2002; Bakir et al., 2002; Husker et al., 2006). The site effects may be further resolved in view of the bedrock topography and the basin complexities by probable 2-D/3-D basin modelling. The geometric parameters such as basement depth vis-à-vis predominant frequency and distance from the basin edge can be envisaged to enforce the basin effects in improving the present site classifications (e.g., Field, 2000; Raptakis et al., 2005).

Figure V.15: (a) The observed predominant frequency from the ground motion data at the monitoring station (square box) with respect to that derived from ambient noise survey (circle) and geotechnical modelling (asterisk) in the vicinity of the station. (b) The corresponding site amplification factor estimated from ground motion data at each seismic station with respect to that estimated at adjacent borehole from 1D geotechnical modeling (after Nath and Thingbaijam, 2010).
Inferences

The site conditions have significant influence on the overall seismic hazard potential. Even in the regions characterized by small basins, the disparities of seismic site conditions across the region could be significant. This entails the need to integrate various data for understanding the basin attributes such as basin geometry and depth. The present study appraised the seismic site conditions of the Guwahati City in terms of predominant frequency and shear wave velocity. The employed geophysical and geotechnical data include ambient noise measurements, earthquake recordings, and geotechnical borehole (SPT-N value) data. Alongwith the delineation of the basement topography based on vertical electrical-resistivity sounding and selected drilled boreholes, region specific relationships between predominant frequency, shear wave velocity, and basement depth could be formulated. The predominant frequency in most parts of the inhabited areas of the City are in the range of 0.5-3.5 Hz. The soil liquefaction potential assessment in the City shows widespread susceptibility. A site classification map of the City based on NEHRP provisions has been prepared which identifies the zones of site classes F, E, D, CD and generic rock in the region. The assessment of site response in the City has been accomplished by means of geotechnical modeling as well as single station standard spectral ratio analysis of earthquake recordings. Further resolution in the basin configuration can be ascertained by means of micro-gravity and other borehole geophysical tools. The distribution of seismic site conditions, and consequently, building vulnerability, is largely influenced by basin response defined by impedance contrast and the bedrock topography.

V.2 Topographic Gradient based Site Characterization in India complemented by Strong Ground Motion Spectral Attributes (Nath et al., 2011b)

Surficial geological conditions can induce significant amplification of the earthquake ground motions; this has been amply corroborated by macroseismic observations, instrumental studies, and theoretical modeling (e.g., Phillips and Aki, 1986; Aki, 1988; Su et al., 1992; Kato et al., 1995; Bonilla et al., 1997; Nath et al., 2010). Owing to significant contribution of site effects to earthquake ground motion, mapped zones of site amplification distribution constitute an important tool for landuse planning and hazard mitigational measures. Site classification to define similar seismic responses has been envisaged to enable generalization. Shear-wave velocity connects to strength and impedance contrasts between soil materials, and consequently can be related to expected amplification (Borcherdt, 1970). The average value for shallow profile of 30 m depth are considered mainly due to the near-surface affinity to site effects as well as lower cost and convenience for engineering practices (Borcherdt, 1994; Field et al., 2000; Street et al., 2001). In view of strong vertical and lateral heterogeneities in the surficial geology affecting the propagation of seismic waves, the site classifications based on $V_{30}$ has been found to be rather simplistic (e.g., Wald and Mori, 2000; Mucciarelli and Gallipoli 2006, Castellaro et al., 2008). Nonetheless, NEHRP site classifications have proven practical applications for seismic hazard studies. Several techniques are available for measuring shear-wave velocity at a site, which are often employed either individually or corroboratively for local specific studies; Geotechnical borelog, Multi-channel Analysis of Surface Waves (MASW), Cone Penetration Test (CPT), to name a few. However, these techniques are not conducive for wide regional coverage. Consequently, regional-level site characterization has been attempted by several
workers based on surrogate datasets (e.g., Wills and Clahan, 2006; Matsuoka et al., 2005; Yong et al., 2008; Wald and Allen, 2007). These include use of geological maps, geomorphological maps, topographic-gradient maps, and remotely-sensed image processing data. In case of identification of site classification pertinent to strong ground-motion stations, several methods in lieu of geological and geotechnical investigation/s have been proposed based on spectral analysis of the earthquake data. These include horizontal to vertical spectral ratio of S-wave, response spectra shape, and horizontal to vertical response spectral ratio (e.g., Lermo and Chávez-García, 1993; Seed et al., 1976; Yamazaki and Ansary, 1997).

A framework for site-conditions on a regional-level is expected to be consequential in the regional seismic hazard analysis. The regional-level perspective is, therefore, examined presently in the Indian context with appraisal for topographic gradient based approach formulated by Allen and Wald (2009). Available data from several cities across the country are employed for the purpose. Additionally, spectral analyses techniques for site characterization are experimented at strong ground motion stations located across the Himalayas and the northeastern region of the country.

Figure V.16: A map of India: the regions of stable tectonic (or relatively subdued topography) are indicated with the light grey shades (after Electrical Power Research Institute, 1994) while the locations of the cities are denoted by square boxes.
Nation-wide Site Conditions

Background information and available Data

Geological Survey of India (GSI) has been publishing hard copy geological maps of India at scales of 1:2,50,000, 1:10,00,000 and 1:50,00,000. Recently, as part of National Spatial Data Infrastructure programme\(^1\), GSI has taken the task delivering nationwide 1:50,000 scale geological maps for India. However, the higher resolution geological map for the entire country is not accessible currently. Topographical data, on the other hand, is available in higher resolution and therefore, preferred. The global 3 arcsec digital elevation data is given by Jarvis et al. (2008) and made available by the Consortium for Spatial Information of the Consultative Group for International Agricultural Research. The global data was derived by post-processing original digital elevation data from Shuttle Radar Topography Mission (SRTM, Farr and Kobrick, 2000) generated by National Aeronautics and Space Administration and National Imagery and Mapping Agency. The processing includes application of hole-filling algorithm (Reuter et al., 2007) to provide seamless and complete elevational surfaces for the continental regions while eliminating areas of no-data. The digital elevation model is exported from ArcGIS to text file and subsequently converted to grid file. The 3 arcsec grid files are, thereafter, down-sampled to a resolution of 9 arcsec and the slopes are estimated by ArcGIS spatial analysis tool. The data suffers from interpolations, especially in the mountainous regions of the Himalayas owing to data gaps leading to underestimation of true elevations. However, the underestimation of peaks in mountainous regions, where higher shear-wave velocity ranges is expected, is not going to have significant effect on the regional classification scheme. The entire territory of India can be divided into regions of active tectonic (or dynamic topography) and stable tectonic (or relatively subdued topography) as depicted in Figure V.16.

The site classifications based on $V_{S}^{30}$ are usually carried out at the local level in order to implement seismic microzonation studies (e.g., Nath et al., 2008a; Nath and Thingbaijam, 2010). Successfully executed seismic microzonation projects can be expected to yield useful information that could support regional-level assessments. A compilation of measured $V_{S}^{30}$ from different projects across the country is used in the present study. These include data from Bangalore, Chennai, Delhi, Dehradun, and Guwahati (located on the map given in Figure V.16). In the Bangalore city, Anbazhagan and Sitharam (2009a) generated $V_{S}^{30}$ distribution from MASW survey and subsequent data analysis. NEHRP site classes B, C and D are observed in the city with most sites falling under site class C (Anbazhagan et al., 2010). Sathyam and Rao (2008) carried out MASW survey to evaluate the shear-wave velocity distribution in the Delhi city. They observed that $V_{S}^{30}$ distribution ranges $<230$ m/s at the sites underlying loose sandy soil (alluvium) and $>350$ m/s at the sites of Quartzite lithology. Boominathan et al. (2008) carried out site characterization in the Chennai city and reported $V_{S}^{30}$ in the range of 230-320 m/s for sites with various depositional conditions of sand and clay. They employed corrected blow counts (standard penetration test) at boreholes verified with MASW measurements. Mahajan et al. (2007) conducted site characterization of Dehradun city. MASW surveys were conducted for near-surface shear-wave velocity measurements. Mahajan (2009) reported that shear-wave velocity distribution in the city pertaining to

\(^1\)http://nsdiindia.gov.in
clay and sand have $V_s^{30} < 180$ m/s, stiff soils have $V_s^{30}$ in the range of 180-270 m/s, conglomerate have $V_s^{30}$ in the range 270-360 m/s, and rock sites have $V_s^{30} > 360$ m/s. Nath and Thingbaijam (2010) reported site classification of Guwahati city. The authors observed that the blow count measurements from standard penetration tests at boreholes located across the city conform to $V_s^{30}$ ranging from 180 m/s to 280 m/s.

**Figure V.17**: The correlations between $V_s^{30}$ and topographic gradient for (a) active tectonic, and (b) stable continental regions; the polygons given in lighter shade are according to Allen and Wald (2009) while those given in thicker lines indicate the broader generalizations. Histogram depicting differences between measured $V_s^{30}$ values ($V_s^{30m}$) and those derived from topographic-gradient correlations ($V_s^{30T}$) for (c) active region, and (d) stable region. The $V_s^{30}$ derived from SPT-N values are excluded in the residual analysis (after Nath et al., 2011b).

**Appraisal of Topographic-gradient Approach**

Figure V.17 (a-b) depicts $V_s^{30}$ data from different cities vis-à-vis the correlations between $V_s^{30}$ and topographic gradients given by Allen and Wald (2009). The correlations are observed to improve substantially when wider range outlining the generic NEHRP site classes – B, C and D is considered limiting present study to NEHRP provision. The logarithm residuals, i.e. difference between the measured $V_s^{30}$ values and those derived from topographic-gradient correlations, is depicted in Figure V.17 (c-d). Another appraisal of the performance of the topographic-gradient based $V_s^{30}$ is performed using site classification maps
derived from geotechnical assessments for the cities. Figure V.18 depicts two maps based on geotechnical investigations and that derived from topographic-gradient based approach, respectively for each city.
Figure V.18: Site classification maps developed through geotechnical borehole analysis and/or MASW surveys for different cities: (a) Bangalore (after Anbazhagan and Sitaram, 2009a), (b) Chennai (after Maheswari et al., 2010), (c) Dehradun (after Mahajan, 2009), and (d) Guwahati (after Nath and Thingbaijam, 2010), respectively placed along the topographic gradient derived maps (after Nath et al., 2011b).

The comparisons between the two maps are carried out by means of statistical correlations between two map data, which is a widely used accuracy assessment technique in remotely sensed data analysis (Story and Congalton, 1986; Jensen, 1996; Patil and Taillille, 2003). The technique involves deriving an error matrix comprising of statistical indicators for each case. The indicators include ‘overall accuracy’ - percentage of matched data, ‘user’s accuracy’ - percentage of correctly matched data in the classified map, ‘producer’s accuracy’ - percentage of correctly matched data in the reference map, and kappa statistics representing a measure of the differences between the observed and the chance agreement between the two maps. The kappa statistics is computed as

\[
k = \frac{N \sum_{i=1}^{r} X_{ii} - \sum_{i=1}^{r} (X_{i+}X_{+i})}{N^2 - \sum_{i=1}^{r} (X_{i+}X_{+i})}
\]  

(V.13)

where \(N\) is the total number of sites in the matrix, \(r\) is the number of rows in the matrix, \(X_{ii}\) is the number in row \(i\) and column \(i\), \(X_{i+}\) is the total for row \(i\), and \(X_{+i}\) is the total for column \(i\) (Congalton and Mead, 1983; Jensen, 1996). The kappa statistics close to 1 suggests ‘perfect’ agreement and that close to 0 indicates ‘poor’ agreement. In the present study, the site classification maps based on geotechnical investigations and that
derived from topographic-gradient based approach are considered as reference and the classified data, respectively. Table V.3 presents the error matrix computed for the Guwahati city; the overall accuracy is observed to be 68.92% while the kappa statistics is found to be 0.44 suggesting that there is moderate chance agreement between the two maps. The estimated statistical indicators for the five cities are summarized in Table V.4.

**Table V.3:** Error matrix derived for the site classification maps of Guwahati city is listed; Map 1 and 2 refer to topographic-gradient and geotechnical based NEHRP site classifications maps, respectively. The notations used are UA: User’s accuracy, and PA: Producer’s Accuracy.

<table>
<thead>
<tr>
<th>Map/ NEHRP Site class</th>
<th>D</th>
<th>C</th>
<th>B</th>
<th>Total</th>
<th>UA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Map 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>162.00</td>
<td>74.00</td>
<td>12.00</td>
<td>248.00</td>
<td>65.32</td>
</tr>
<tr>
<td>C</td>
<td>3.00</td>
<td>34.00</td>
<td>0.00</td>
<td>37.00</td>
<td>91.89</td>
</tr>
<tr>
<td>B</td>
<td>2.00</td>
<td>10.00</td>
<td>28.00</td>
<td>40.00</td>
<td>70.00</td>
</tr>
<tr>
<td>Total</td>
<td>167.00</td>
<td>118.00</td>
<td>40.00</td>
<td>224.00</td>
<td></td>
</tr>
<tr>
<td>PA</td>
<td>97.01</td>
<td>28.81</td>
<td>70.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table V.4:** Indicators for the statistical correlations between the differently derived site classification maps.

<table>
<thead>
<tr>
<th>City</th>
<th>Producer’s Accuracy</th>
<th>NEHRP Site Class</th>
<th>Overall Accuracy</th>
<th>Kappa value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>C</td>
<td>B</td>
<td>D</td>
</tr>
<tr>
<td>Bangalore</td>
<td>90.91</td>
<td>42.06</td>
<td>05.00</td>
<td>12.35</td>
</tr>
<tr>
<td>Chennai</td>
<td>77.39</td>
<td>34.48</td>
<td>40.00</td>
<td>74.76</td>
</tr>
<tr>
<td>Dehradun</td>
<td>94.81</td>
<td>25.00</td>
<td></td>
<td>70.87</td>
</tr>
<tr>
<td>Guwahati</td>
<td>97.01</td>
<td>28.81</td>
<td>70.00</td>
<td>65.32</td>
</tr>
</tbody>
</table>

**Table V.5:** Types of foundation soil given by Bureau of Indian Standards (BIS, 2002), site classes specified by NEHRP and Japan Road Association (1980), respectively; N denotes for corrected standard penetration test N value (blow counts) at the foundation level while \( V_s^{30} \) represents the average shear-wave velocity in m/s for 30 m of the soil from the surface.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Site I (Rock/stiff soil)</td>
<td>Rock</td>
</tr>
<tr>
<td>Type I (N&gt;30)</td>
<td>A (( V_s^{30} &gt;1500 )) + B (760 ≤ ( V_s^{30} &lt;1500 ))</td>
<td>Site II (Hard soil)</td>
<td>Hard/very stiff soils/gravels</td>
</tr>
<tr>
<td></td>
<td>C (360 ≤ ( V_s^{30} &lt;760, N&lt;50 ))</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type II (10 ≤ N ≤ 30)</td>
<td>D (180 ≤ ( V_s^{30} &lt;360, 15 ≤ N ≤ 50 ))</td>
<td>Site III (Medium soil)</td>
<td>Stiff soils</td>
</tr>
<tr>
<td>Type III (N&lt;10)</td>
<td>E (( V_s^{30} &lt;180, N&lt;15 ))</td>
<td>Site IV (Soft soil)</td>
<td>Soft soils</td>
</tr>
</tbody>
</table>

* after Zhao et al. (2006b)
Overall, the topographic-gradient based approach performs fairly well except for the Bangalore city. For individual site classes, the correlations are seen to be poor for the rock sites in the stable regions. However, site classification maps of these regions are mostly based on SPT-N values with possible higher uncertainties. Visually, significant conformity can be observed between the two site-classification maps in the Chennai and Guwahati cities, respectively while it is moderate in Dehradun and poor in the Bangalore city. Apparently, overall consistency of the topographic-gradient based approach is about 60-70%. The level of accuracy required for effective local-specific assessment would be about 80-90%. Hence, the approach can be effective as a first-cut tool for regional level evaluation. In this implementation, site classifications pertaining to the rock sites ($V_{S30}^{30}$>760 m/s) can also include those of site class A, i.e. hard rock sites (i.e., $V_{S30}^{30}$ >1500 m/s). To indicate this consideration, $B^+$ notation is used hereinafter. Nonetheless, similar site classification scheme, where site classes A and B have been clubbed together, can also be found in the site class definition of Japan Road Association (1980) while that defined by Bureau of Indian Standards (BIS, 2002), approximately corresponds to either of the three site classes C, B and A of the NEHRP nomenclature. A summary of different site class definitions is presented in Table V.5. Figure V.19 depicts a preliminary nation-wide seismic site condition map derived on the basis of topographical gradients.

![Figure V.19: A preliminary NEHRP based site condition map of India (after Nath and Thingbaijam 2011b) derived from topographic-gradient based correlations of Allen and Wald (2009).](image-url)
Spectral Analyses

Existing Techniques

Spectral analyses of earthquake ground-motion recordings are often used for site classification at individual recording stations, especially in lieu of geotechnical data analysis, owing to the fact that different site conditions cause amplification in different period ranges (Mohraz, 1976). Three approaches are commonly used, namely Horizontal-to-Vertical Spectral Ratio (HVSR), Response Spectra Shape (RSS), and Horizontal-to-Vertical Response Spectral Ratio (HVRSR) (Lermo and Chávez-García, 1993; Borcherdt, 1970; Seed et al., 1976; Yamazaki and Ansary, 1997). HVSR, better known as Nakamura’s technique, is applicable to ambient noise data (Nakamura, 1989) as an expedient technique for site response approximation in terms of predominant frequency. Lermo and Chávez-García (1993) proposed that the HVSR, similar to receiver-function analysis (Langston, 1979), can be employed for site response assessment using S-wave envelope of the strong ground-motion data. Field and Jacob (1995) found that the HVSR could reveal overall frequency dependence of site response while Lachet et al. (1996) demonstrated that the consistency of the technique is frequency-dependent with the reliable range falling below 8 Hz. A reference rock site, where site response is modulated around amplification factor of 1, is required in case of standard spectral ratio technique. The rock site should be located in the close proximity for the technique to be feasible. In the absence of the reference site, the technique has been used for site response approximation (e.g., Nath et al., 2008a; Nath and Thingbaijam, 2010). The response spectrum, on the other hand, characterizes the ground motion indirectly in terms of maximum response (in terms of acceleration, velocity or displacement) of a single degree of freedom system as a function of natural frequency or period and damping ratio of the system. Seed et al. (1976) employed RSS computed from normalized 5%-damped acceleration response spectra for site classification. Recently, Lee et al. (2001) used an enhanced database to work out the mean response spectral shapes for NEHRP site classes. In the analysis, individual response spectral shapes are normalized using the observed peak ground acceleration. Yamazaki and Ansary (1997) observed that response spectra shapes significantly depends on earthquake magnitude, distance and focal depth, and suggested using ratio of 5% damped response spectra of the horizontal and vertical components respectively; a technique which is referred to as HVRSR. Zhao et al. (2006b) pointed out that the damping ratio of 5% provides smoothing effect on the spectral values, and thus, has a practical advantage over using undamped response spectrum. Fourier spectra are known to have sharp (not smooth) peaks while 5% damped response spectra provide smooth and more consistent shape with lesser effect of averaging over a large number of records. The method has been tested successfully and its usefulness in unambiguous in site classification corroborated by several workers across the globe (e.g., Zare et al., 1999; Fukushima et al., 2007; Lee et al., 2001; Zhao et al. 2006b; Ghasemi et al., 2009; Wen et al., 2010).

Strong Motion Stations and Data

The spatial distribution of the considered strong motion stations is depicted in Figure V.20(a). These comprise of Kangra array with 11 stations, Shillong array with 39 stations, Uttarakhand array in the northwest Himalayas with 19 stations, Sikkim array with 9 stations, and Guwahati array with 8 stations
Figure V.20: (a) The spatial distribution of the strong motion stations; the strong motion arrays namely Kangra and Uttarakhand, Sikkim, and Shillong and Guwahati, respectively are depicted, (b) The magnitude and hypocentral distance distribution of the earthquakes recorded by the arrays (after Nath et al., 2011b).

(Chandrasekaran and Das, 1992; Nath et al., 2005; Nath et al., 2008a). The Kangra, Uttarakhand, and Shillong arrays were operated by IIT Roorkee, and the strong ground motion data comprising recordings of eleven earthquakes with magnitude ranging $M_w$ 4.9-7.2 are accessible at the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) Virtual Data Center (http://db.cosmoseq.org, last accessed February, 2010). The recordings are available as three component acceleration time series, which have been band-pass filtered (0.17-0.20 Hz and 25.0-27.0 Hz). The Sikkim array operated by IIT Kharagpur in the Sikkim Himalaya comprises of one Kinematics Altus K2 and eight Kinematics Altus ETNA high dynamic range strong motion accelerographs. The instruments have been calibrated with trigger level of 0.02% of the full-scale (2g), dynamic range of 108 dB at 200 samples/sec and 18-bit resolution. The earthquake data include those of 20 events having magnitude in the range of $M_w$ 3.4-5.6. The Guwahati array is maintained by IIT Guwahati. The recordings of five small to moderate magnitude ($4.5 \leq M_w \leq 4.9$) by the array is available through Guwahati seismic microzonation consortium. The
configurations of the instruments in the array are similar to those of the Sikkim array. The entire data comprises of a total of 210 three-component acceleration time series, i.e. 111 from Shillong Array, 41 from Sikkim, 18 from Guwahati, 11 from Kangra, and 29 from Uttarakhand array, respectively. Figure V.20(b) summarized the recorded earthquakes. The site classes are identified for the stations. While for most of the stations, there is conformity to the known typical site geology i.e. either soil or rock, seven stations associated with soil geology are found to have been classified as site class B. These are Chamba, Dauki, Gunjung, Haflong, Harengajao, Laisong, and Sihunta, which are reclassified as pertaining to site class C.

**Horizontal-to-Vertical Spectral Ratio (HVSR)**

For all the filtered acceleration time series, the shear-wave phases are picked up using windows of varying length depending on the record with each window chosen to include the strongest shaking. The windowed shear-wave acceleration time series are transformed into frequency domain, and the Fourier amplitude spectra smoothed with the running weighted 9-point averaging algorithm. Thereafter, $HVSR_{ij}(f_k)$ is computed at each $j$ site for the $i^{th}$ event at the central frequency $f_k$ from the root mean square average of the amplitude spectra as where $H_{ij}(f_k)|_T$ is the Fourier spectra of the transverse component; $H_{ij}(f_k)|_R$ is the Fourier spectra of the radial component; and $V_{ij}(f_k)$ is the Fourier spectra of the vertical component (Nath et al., 2005). This can be written as,

$$HVSR_{ij}(f_k) = \left( \frac{\left( H_{ij}(f_k)|_T \right)^2 + \left( H_{ij}(f_k)|_R \right)^2}{2} \right)^{0.5} / V_{ij}(f_k) \quad (V.14)$$

In order to overcome the limitation of limited dataset, bootstrapping technique (Chernick, 1999) is adopted for computation of standard deviation. This conforms to the fact that a set of observations can be assumed to be from an independent and identically distributed population, and thus can be implemented by constructing samples of the observed dataset (and of equal size to the observed dataset), each of which is obtained by random sampling with the replacement from the original dataset. The computation is done by sampling 1000 times at all the $k$ frequencies.

The HVSR curves covering 0.1–20 Hz pertaining to the different terrains are depicted in Figure V.21. In cases of rock and very stiff soil sites (site classes B+ and C), there is perceptible conformity between HVSR trends and the curve given by Boore and Joyner (1997) for generic rock site ($V_S^{30} \sim 620$ m/s) for frequencies up to $\sim 4$ Hz in case of Sikkim and 7-8 Hz in case of other regions. In the Shillong array, which represents the northeast India (excluding the eastern Himalayan terrains), the mean curves for site classes B+ and C, respectively exhibits reasonable similarity with the predominant frequencies in the range of 3.0-5.0 Hz. The same can be said for site class C in Kangra region, notwithstanding the sparse data. However, the trend differs significantly in the Sikkim Himalayas with the curve exhibiting higher scattering, especially in the higher frequency band (>8 Hz), with the indication of two peaks: one at around 3 Hz and other at around 11-12 Hz. In the northwest Himalayas represented by combined Kangra and Uttarakhand array, the peak amplitudes of the mean curve fall within the frequencies 1.5–3.0 Hz indicating a shift to lower frequency band compared to the northeast region. The HVSR curve for site class D in the northeast India shows predominant frequency around 1 Hz while a subtle shift in the predominant frequencies to the range of 2.0–4.0 Hz is visible with the mean trend computed with the geotechnical based assessment by Nath and Thingbaijam (2010), which nonetheless comes within the uncertainty bounds.
Figure V.21: Horizontal-to-vertical spectral ratio plots from the strong motion arrays according to the different site classes. The curves for rock sites derived by Mandal et al. (2005), and Singh et al. (2007) as well as generic rock site response curve given by Boore and Joyner (1997) are considered for comparison purpose. In case of site class D, the mean site response model for the Guwahati city derived from geotechnical analysis by Nath and Thingbaijam (2010) is depicted.
Figure V.22: Average curves for the normalized spectral acceleration computed as geometric mean of those derived the two horizontal (longitudinal and transverse) components. For the respective soil class, the corresponding curves given by Seed et al. (1976), and Lee et al. (2001) are plotted for comparison purpose.
**Response Spectra Shape**

The computation of RSS is carried out following the formulation given by Nigam and Jennings (1969). Figure V.22 depicts the computed RSS for natural periods ranging 0.04 – 3.0 sec and 5% damping ratio. The mean curves for different site classes given by Seed et al. (1976), and Lee et al. (2001) are considered for comparative assessments. These curves are observed to be located within the uncertainty bounds with the RSS derived for all the cases. However, there is significant concurrence in case of site class B+ in the northwest Himalayas, represented by Kangra and Uttarakhand arrays. Similar observations can also be made for site class C in the Shillong array, albeit the curve given by Lee et al. (2001) exhibits higher peaks at the periods >0.3 sec. In case of site class B+ in the Shillong array, the trends match well with those of Seed et al. (1976), and Lee et al. (2001). However, the curve for site class C in Kangra region has higher peaks for the periods < 0.55 sec while it matches closely with the trend computed by Seed et al. (1976) for rock sites at longer periods. In the Sikkim Himalayas, two different clusters are observed with the peaks at ~0.085 and ~0.3 sec, respectively. The former shows no agreement with those given by Seed et al. (1976), and Lee et al. (2001), while the latter one has reasonable conformity at longer periods (>0.8 sec). On the other hand, the trend observed for site class D in Guwahati and Shillong arrays reveals a good match at lower periods (<0.5 sec) with that of Seed et al. (1976) for stiff soil sites while at the longer periods with that of Lee et al. (2001) for site class D.

**Horizontal-to-Vertical Response Spectral Ratio**

The Horizontal-to-Vertical Response Spectral Ratio (HVRSR) is obtained using geometric mean of response spectra of the horizontal components and the response spectra of the vertical component as follows,

\[
HVRSR_{ij}(f_{k}) = \left( \frac{A_{ij}(f_{k})}{A_{ij}(f_{k})_{T}} A_{ij}(f_{k})_{R} \right)^{0.5} / A_{ij}(f_{k})_{V}
\]

where \( A \) stands for spectral acceleration, and rest of the notations are as previously defined. The HVRSR curves obtained for different terrains pertaining to different site classes are depicted in Figure V.23. Also depicted are the HVRSR curves computed by Lee et al. (2001), and Zhao et al. (2006b) for different site classes.

The mean HVRSR curves for site class B+ pertaining to the Sikkim and Kangra-Uttarakhand arrays do not exhibit distinct peaks. In case of the Shillong array, there is marked differences from the trends computed by Lee et al. (2001), and Zhao et al. (2006b). Overall, the observed HVRSR curves for site class B+ at all the cases do not comply with the ones previously given by the authors. However, the HVSR curves exhibit similar traits i.e., peak values at the periods in the range of 0.2 sec to 0.3 sec with the ones given by the authors for site class C. The mean trend for site class C in the Shillong array concurs with the one given by Zhao et al. (2006b) while that observed for site class C in the Kangra array tallies with the one given by Lee et al. (2001). In case of site class D in the Guwahati and Shillong arrays, respectively the HVRSR curve reasonably agrees with the one given by Lee et al. (2001) for site class D.
Figure V.23: Average HVSR curves. For the respective soil class, the corresponding curves given by Lee et al. (2001), and Zhao et al. (2006b) are plotted as well for comparison purpose.
Statistical Analysis

The statistical significance between the spectral trends for different site classes obtained by the three techniques is examined by means of Welch’s t-test (Welch, 1947). The test is performed for a pair of site classes at different time periods. The t-statistic is given by the following formulae,

\[ t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \]  \hspace{1cm} (V.16)

where \( \bar{x}_1 \), \( s_1^2 \) and \( n_1 \) are the sample means, unbiased variances and sample sizes, respectively. The associated degrees of freedom is calculated as,

\[ DF = \frac{\left( \frac{s_1^2}{n_1} + \frac{s_2^2}{n_2} \right)^2}{\left( \frac{s_1^2}{n_1} \right)^2 / (n_1 - 1) + \left( \frac{s_2^2}{n_2} \right)^2 / (n_2 - 1)} \]  \hspace{1cm} (V.17)

Equations (V.16) and (V.17) are used to compute the probability of obtaining the t-statistic assuming a null hypothesis that the two means are equal. The null hypothesis can be rejected if the probability is found to be below a certain significance level (0.05 or 0.1). Figure V.24 depicts plots between the probability for the null hypothesis and time-period for each comparison. HVSR do not exhibit statistically significant discrimination between site classes B and C. On the other hand, HVSR and RSS are able to distinguish all the site classes. HVSR is able to distinguish between site amplifications of site classes B and C at the high frequencies. Additionally, the scatterings associated with the curves are expressed by means of standard deviations with respect to the observed means as depicted in Figure V.25. High scatterings are observed at short periods in case of HVSR while they exalt in the longer periods in case of RSS. The stable and lower oscillations can be seen in case of HVSR making it comparatively a better technique for regional analysis.

Discussion

The correlations between topographic gradients and \( V_s^{30} \) given by Allen and Wald (2009) using the data from several cities across India. While the available limited data volume restricts comprehensive region-specific regressions, the correlations exhibit reasonable agreement when considered for broad site class definitions of NEHRP provision overlooking the possible variation of \( V_s^{30} \) values. This allows development of a first generation site classification map at the regional level depicting NEHRP site classes. The classifications exhibit good match with the ones delivered by local specific studies for the regions of dynamic topography. For the regions of relatively subdued topography, the correlations yield lower \( V_s^{30} \) estimates, and hence, provide conservative implementations. The seismic wave velocities of Gondwana rocks published by Geological Survey of India (Sarkar et al., 2007) entail possible higher \( V_s^{30} \) values in the peninsular India. The reported velocity measures were determined in the laboratory using rock specimens from several boreholes across Gondwana formations of India–Damodar, Mahanadi and Gondwana valley rift zones, respectively. At several boreholes associated with the samples extracted at shallow depth (within 30 m from the surface), the estimated shear-wave velocity ranges \( \geq 1490 \) m/s with the rock types...
comprising of very fine grain to coarse sandstone. Future study could address site class A especially with the availability of higher resolution geological maps. The rock sites across in the country are not similar; e.g., rock sites in the Himalayas have younger geology and higher deformations compared to those located in the peninsular India. Overall, the preliminary site classification map exhibits site class D having 51%, site class C having 23%, and site class B⁺ having 25% coverage of the total area while that of site class E could be identified with a marginal 1% of the total area.

Figure V.24: Probability for the null hypothesis that the two curves are equal estimated for each time period; left, centre and right panels show comparisons between various sites obtained by HVSR, RSS and HVRSR techniques, respectively (after Nath et al., 2011b).
Three empirical techniques for site classification, namely HVSR, RSS and HVRSR, are experimented. The HVSR curves obtained for rock sites (including site class C) show similar traits in the amplification (or HVSR) values while the trends also agree with the theoretical one derived for generic rock at frequencies<8 Hz within the uncertainty bounds. Except that in case of Sikkim region, the aberrations are seen from 4 Hz onwards. The undulating topography across the Himalayan terrains is liable to have profound effect on the site response, especially at the higher frequencies (Nath et al., 2005). This is supported by significant oscillations at higher frequencies. While mean amplifications for rock sites are well represented by HVSR curves, the peak amplitudes and the associated frequency with the mean HVSR curve for site class D slightly differs from the one derived from geotechnical model. This difference may be due to the consideration of 30 m depth profiles for the geotechnical assessment. Nonetheless, the HVSR trends pertinent to the site class exhibits comparatively lower predominant frequency (~1 Hz) as compared to 3.0-5.0 Hz for the rock sites. Interestingly, the RSS shows remarkable consistency in the mean peak amplitudes across the rock sites, including site class C in all the regions, except Sikkim. In the Sikkim Himalayas, the ambiguity of the RSS with two different clusters having different trends could be due to the topographic effects. On the other hand, the mean RSS for site class D exhibits reasonable conformity. The results of HVRSR for the rock sites exhibits peaks of the mean curves at higher periods as compared to those computed for similar site conditions in other regions. At the same time, there is significant conformity as far as site classes C and D are concerned. The following observations are noted:

1. The correlations of \( V_{s}^{30} \) with topographic gradients provide estimation of broad NEHRP site classes with 60–70% of accuracy catering to preliminary regional-level site classifications.

2. RSS and HVRSR support different signatures for different site classes, and a pragmatic approach would be to employ both the techniques for corroborative interpretations.

**Figure V.25**: The distribution of dispersions associated with the three techniques in terms of ratio of standard deviation to the observed mean (after Nath et al., 2011b).
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Annexure – I

MINISTRY OF EARTH SCIENCES
MADHAGAR BHAVAN, BLOCK-12, C.G.O. COMPLEX, LODHI ROAD, NEW DELHI-110003

MoES/P.O. (Seismo)/2(04)/2007

Dated: 27th March 2008

OFFICE MEMORANDUM

Sub: Constitution of National Steering Committee (NSC) for seismic microzonation of selected cities, in India.

Ministry of Earth Sciences has constituted a National Steering Committee (NSC) to oversee the microzonation studies to be carried out for selected cities in the country during the 11th Plan period. Composition, terms of reference and tenure of NSC is given below:

A. COMPOSITION:

1. Prof. D.K. Paul,
Department of Earthquake Engineering,
Indian Institute of Technology, Roorkee- 247 667

2. Rep. of Director General
Geological Survey of India,
27-Jawahar Lal Nehru Marg,
Kolkata- 700 016

3. Dr. A.K. Bhatnagar,
ADG (Seismology),
India Meteorological Department
Mausam Bhawan, Lodhi Road, New Delhi- 110 003.

4. Dr. R.K. Chadha
Scientist-F
National Geophysical Research Institute
Uppal Road, Hyderabad-500 007

5. Prof. T. G. Setharam,
D/o Civil Engineering,
Indian Institute of Sciences, Bangalore- 560 012

6. Prof. S. K. Nath,
D/o Geology and Geophysics,
Indian Institute of Technology, Kharagpur- 721 302

Contd. P-2
7. Dr. B.K. Bansal, Programme Officer, MoES, New Delhi

Member-Secretary

B. TERMS OF REFERENCE:

- The committee will provide overall guidance to undertake the microzonation of different selected cities.

- The Committee will identify different working group to carry out the proposed studies for identified cities, which may be prioritized depending upon their locations and population density.

- Will help in evolving specific projects to facilitate carrying out the field as well as lab. oriented studies.

- The committee will screen in the individual project proposals and recommend for support.

- The committee may co-opt experts on specific theme/subject, required if any.

C. TENURE

The initial tenure of the committee will be three years from the date of its first sitting.

2. TA/ DA to Non-official members of the NSC will be paid by the MoES, as per the Govt. of India norms. A sitting fee of Rs.500/- per day of the meeting will be paid to all the Non-official members of NSC.

3. This issues with the approval of Secretary, MoES and with concurrence of IF Division, MoES.

Vandana Chaudhary
Scientist-C
Seismology Division
e-mail: v.chaudhary@nic.in

To,


2. Director General, Geological Survey of India, 27-Jawahar Lal Nehru Marg, Kolkata- 700 016, with a request to kindly nominate a representative of GSI in the committee.

Contd. P-3
3. Dr. A.K. Bhatnagar, ADG (Seismology), India Meteorological Department, Mausam Bhawan, Lodhi Road, New Delhi - 110 003.
4. Dr. R.K. Chadha, Scientist-F, National Geophysical Research Institute, Uppal Road, Hyderabad-500 007
5. Dr. T G Sitharam, D/o Civil Engineering, Indian Institute of Sciences, Bangalore- 560 012
6. Prof. S K Nath, D/o Geology and Geophysics, Indian Institute of Technology, Kharagpur- 721 302
7. Dr. B.K. Bansal, Programme Officer, MoES, New Delhi

Copy Forwarded for information to:

1) Pay and Accounts Officer, MoES, New Delhi
2) Sr. PPS to Secretary, MoES.
3) PS to JS (A), MoES.
4) PS to FA, MoES
5) Director (F) & DS(F), MoES/ Cash Section
6) Office Copy

Vandana Chaudhary  
Scientist-C  
Seismology Division  
e-mail: v.chaudhary@nic.in
Annexure – II

भारत सरकार
GOVERNMENT OF INDIA
पृथ्वी विज्ञान मंत्रालय
MINISTRY OF EARTH SCIENCES
‘महासागर भवन’ क्लाक-12, सी.जी.ओ. कॉम्पलेक्स, लोधी रोड
‘Mahasagar Bhavan’ Block-12, C.G.O. Complex, Lodhi Road

संख्या
No. ____________

MoES/P.O.(Seismo)/2(04)/2008

04.02.2010

Sub: Minutes of the meeting held on January 12, 2010 at Mahasagar Bhawan, to discuss the Methodologies and Practices of Microzonation.

Please find enclosed herewith the approved minutes of the meeting held on January 12, 2010 at Mahasagar Bhawan, to discuss the Methodologies and Practices of Microzonation for your kind perusal and necessary action, as appropriate.

(Vandan Chaudhary)
Scientist-C
Seismology Division
Tel: 011-24615528

To,

1. Dr. H. K. Gupta, Raja Ramanna Fellow, National Geophysical Research Institute, Uppal Road, Hyderabad-500007.
4. Dr. A. K. Bhatnagar, ADG, India Meteorological Department, Mausam Bhavan, Lodhi Road, New Delhi – 110 003
5. Prof. S.K. Nath, Head, Department of Geology & Geophysics, Indian Institute of Technology, Kharagpur- 721 302
6. Dr. T.G. Sitharam, Dept. of Civil Engineering, Indian Institute of Science, Bangalore 560 012.
7. Dr. R.K. Chadha, National Geophysical Research Institute, Uppal Road, Hyderabad- 500 007.
8. Dr. B.K. Bansal, Head, Geosciences/ Seismology Div., MoES, New Delhi
9. Dr. Ravi Kumar, National Geophysical Research Institute, Uppal Road, Hyderabad-500007.

Copy for information to:
- Sr. PPS to Secretary, MoES.
- Shri B. B. Bhattacharjee, Hon’ble Member, National Disaster Management Authority, New Delhi.
- Secretary, Ministry of Urban Development, New Delhi.
Minutes of the Meeting held on January 12, 2010 at Mahasagar Bhawan, to discuss the Methodologies and Practices of Microzonation.

A meeting was convened on 12\textsuperscript{th} January, 2010, at Mahasagar Bhawan, to discuss the current methodologies and practices of microzonation. Dr. Shailesh Nayak, Secretary, Ministry of Earth Sciences, chaired the deliberations. The meeting was attended by the representatives of IMD, IIT(Kh), IISc, NGRI, GSI, NDMA and MoUD.

Dr. Shailesh Nayak welcomed all the participants of the meeting and mentioned that all the major towns in the country required to be microzoned, which in turn, will help in the assessment of the earthquake hazard and preparing the necessary building codes. However, at present, the pace of work is very slow and there is no uniformity with regard to methodology as well as scale of maps. Also, the required base maps are not available and therefore, it is necessary to resolve these issues. He further mentioned that probably 1:25,000 scale may be ideal at this stage. As far as the base maps are concerned, Survey of India may be approached, and in case, they are not available, some alternate approach may be thought of. At this stage, we should not worry too much about the accuracy of the available maps and start working with the existing maps. The final maps may be updated later in future as and when information is available. He further mentioned that since the work is gigantic and is also not a mandate of academic institutions, it is necessary to involve the industry in this venture. However, prior to that, it is necessary that we are ready with the detailed document, especially, on methodologies including detailed steps to be followed in estimation of various parameters.

Dr. B. K. Bansal mentioned that as per the guidelines decided by MHA, sometime back, cities which fall in seismic zone IV and V, and have a population of half a million plus the capital cities of all the states, may be taken up for microzonation to start with. He also mentioned that a brief framework and methodology for microzonation studies was prepared initially by IIT (Kh) and the same was circulated amongst the members of the National Steering Committee (NSC).

Presentation by Prof. S. K. Nath:

Dr. Nath made a detailed presentation on the current practices of microzonation world wide. He mentioned that the world over, microzonation studies, in general, are being carried out at 3 different levels viz. Level –I on 1:1,000,000 to 1:50,000; Level-II on 1:100,000 – 1:10,000 and Level-III on 1:25,000 – 1:5,000 scale. However, there is hardly any uniformity in the approach. Also, no country has been following such a comprehensive approach, the way it is being done in India. In most of the cases, microzonation is based on 1 or 2 parameters only, whereas in India, most of the parameters are being used and also introspected to arrive at seismic hazard index maps. He briefly touched upon the methodology being used in India for estimating the different parameters, namely, site characterization, estimation of peak ground acceleration, determination of predominant frequency, computation of response spectra, etc. He also presented the detailed framework
of microzonation. Prof. Nath also highlighted the results of microzonation studies carried out systematically for Sikkim, Guwahati, Delhi, Bangalore, and Jabalpur, etc.

**Presentation by Dr. Sitharam:**

Prof. Sitharam also made a presentation on the methodology being adopted for geo-technical investigations. He detailed out the various approaches being followed worldwide, for geo-technical investigations including CPT, SPT, SASW and MASW etc. He also mentioned that while identifying the cities for microzonation, it is necessary to take into account the parameters like, hazard level, population, economics and importance. Prof. Sitharam also mentioned that a detailed document is being prepared by NDMA on geo-technical investigations where he has played a major role in preparation of the document.

Gen. Tyagi, from NDMA, mentioned that standard operating procedure for different level of maps needs to be prepared. First level maps, wherever available, also needs to be updated. He also impressed upon the need of preparing the building codes and their implementation in the country.

Shri Suresh, representative of Ministry of Urban Development (MoUD), mentioned that microzonation maps are very useful for MoUD, especially for land use planning. As the Ministry is in the process of preparing the Master Plan, it would be appropriate if the reports and the findings of the studies, which have already been carried out, may be passed on to the relevant State Government agencies.

Dr. Chadha from NGRI, mentioned that while carrying out these studies, especially, the simulation of peak ground acceleration, one must ensure that the findings are supported by instrumental data.

Dr. Ravi Kumar also mentioned that the estimated value needs to be calibrated with actual observations. He cited the example of Sikkim region, where the estimated value was much higher than the observed.

Dr. P. Pande mentioned that GSI had started the microzonation study sometime in 2001 and preliminary microzonation maps were prepared for Delhi as well. At present, GSI is engaged in microzonation studies of 13 cities in the country of which work on Dehradun, Chandigarh and Jammu has already been completed. Presently, the work is going on in Ahmedabad, Jamnagar, Bhavnagar, Bharuch, Siliguri, Jalandhar, Vishakapatnam, Chennai and Mumbai. The microzonation studies which have been carried out by GSI are mainly based on geology, soil characteristics, noise survey and resistivity survey etc. However, due to some constraints, viz. non-availability of large scale maps, difficulty in mapping in urbanized cities, lack of manpower etc., GSI could not standardize any methodology for microzonation. Dr. Pande also mentioned that NDMA, a few years back, had constituted a sub-committee for microzonation for bringing out the guidelines as well as the manuals for carrying out studies related to seismic hazard and microzonation etc. However, it is not very sure that how far the work has been completed so far.
He further informed that since GSI has prepared the geological map for the whole country on 1:50,000 scale, the same may be utilized as the base maps. Also, all these geological maps will be uploaded on the GSI portal by September 2010. He also mentioned that ground water maps are very essential for generating liquefaction maps and must be taken into consideration while preparing microzonation maps.

After a detailed discussion, the following points emerged:

1. Since the first level hazard maps (probabilistic) are being prepared by NDMA, MoES will not take up this work. However, once these maps are ready and require further refinement, if any, it may be taken up by the Ministry.

2. A list of cities which are required to be microzoned, may be prepared based on the hazard level, its importance and population density etc.

3. An inventory may be prepared on the work which has already been carried out by the different agencies related to microzonation.

4. MoES will concentrate on the microzonation of different cities at 1:25,000 scale and geological maps prepared by GSI may be used as base maps for this purpose. A confidence level must be attached to the all estimated parameters if certain inputs are not available at a particular scale.

5. The framework presented by Prof. Nath, was accepted. However, it was felt that a detailed document needs to be prepared on methodology for estimating the various parameters. A small group comprising Prof. S.K. Nath, Prof. T.G. Sitharam, Dr. A.K. Shukla, Dr. Ravi Kumar and Dr. B. K. Bansal, was identified to complete the task by 28th February, 2010.

6. For microzonation of Delhi on 1:10,000 scale, immediate concentration should be given to the areas where new development is going on presently. EREC may check with town planning department for getting the boundary maps.

7. Since MoUD prepares land use maps at 1:10,000 scale for urban areas, a meeting may be organized with the officials of MoUD by April, 2010 where the results from different areas in Delhi may be presented.

The meeting concluded with a thanks to the Chair.
List of participants

1. Dr. Shailesh Nayak, Secretary, MoES, New Delhi.
2. Dr. A. K. Bhatnagar, India Meteorological Department, Lodhi Road, New Delhi-110 003.
5. Dr. Prabhash Pandey, Dy. Director General, Op. Information & Technology, Geological Survey of India, 27, J. L. Nehru Road, Kolkata-16
6. Prof. T. G. Sitharam, D/o Civil Engineering, Indian Institute of Science, Bangalore 560 012.
7. Dr. B. K. Bansal, Head, Seismology Division, MoES, New Delhi.
9. Dr. R. K. Chadha, Scientist-F, National Geophysical Research Institute, Uppal Road, Hyderabad-500 007.
10. Dr. M. Ravi Kumar, Scientist-F, National Geophysical Research Institute, Uppal Road, Hyderabad-500 007.
11. Dr. A. K. Shukla, EREC, India Meteorological Department, Lodhi Road, New Delhi-110 003.
12. Dr. Vandana Chaudhary, Seismology Division, MoES, New Delhi.