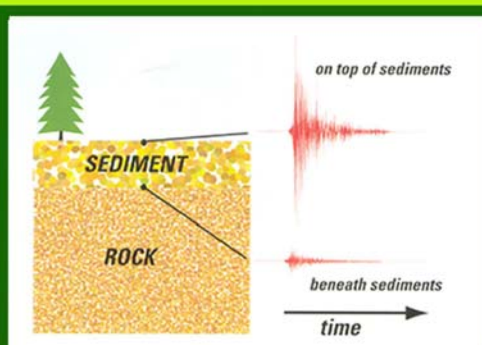
Geotechnical / Geophysical Investigations for Seismic Microzonation Studies of Urban Centres in India

TECHNICAL REPORT



August 2011

NATIONAL DISASTER MANAGEMENT AUTHORITY
GOVERNMENT OF INDIA

Geotechnical / Geophysical Investigations for Seismic Microzonation Studies of Urban Centres in India

Technical Report

National Disaster Management Authority
Government of India

Vision

Zero Tolerance
to
avoidable deaths due to earthquakes

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Member
National Disaster Management Authority
Government of India

PREFACE

While damaging effects of earthquakes have been known for long time, the contribution of soils to the magnitude and pattern of earthquake damage was not widely appreciated until recently. The local geological and soil conditions can profoundly influence all of the characteristics – amplitude, frequency content, and duration – of strong ground motion. Significant earthquake damage and loss of life has been directly related to the effect of local site conditions and liquefaction in several recent earthquakes (eg., 1985 Mexico earthquake, 1989 Loma Prieta Earthquake, 1994 Northridge, 1995 Kobe Earthquake, and Bhuj earthquake 2001). The local site effects play an important role in earthquake resistant design. Both the local site conditions and liquefaction related damage are very essential component of a comprehensive assessment of seismic hazard and are attributed to geotechnical / geophysical characteristics of soil overburden closure to the ground. A complete site characterization is essential for the seismic site classification and site response studies, which are carried out by detailed geotechnical/geophysical investigations. There is a clear role for geological and geotechnical data in evaluating the ground motions and in turn in the study of seismic hazard and preparation of geotechnical data driven microzonation maps to provide an effective solution for city planning.

During the national workshop organized by NDMA on 16th July 2008 clearly debated the need of preparation of technical document on geotechnical / geophysical investigations for seismic microzonation studies of urban centers in India. Even the present adopted practice of geotechnical investigations for seismic microzonation were discussed and felt the need for detailed guidelines as no such guidelines are available in the country. During the national workshop on seismic microzonation of Indian landmass held on 16th July 2008 at NDMA, at that time the existing working group for geotechnical investigations formed under the WCE-microzonation (vide No NDMA/BB/S&T-1/2007 dated 21st Sept 2007) was expanded by inducting additional members with Prof. T.G. Sitharam, Professor, Department of Civil Engineering, Indian Institute of Science (IISc), Bangalore as convener of the group and IISc as the nodal institution to prepare the technical document on geotechnical investigation for seismic microzonation studies in India. Further, the same was presented to the meeting of steering committee (geophysical hazard) which was held on 5th Nov 2008 and the TOR was approved by NDMA. NDMA has signed an MOU on 7th July 2009 with IISc to prepare the national document with the visit of officials from NDMA (Mr. Ajay Sule, Director- Mitigation) to IISc. The project was sanctioned to IISc as it had successfully carried out several mission mode projects of national importance. IISc has taken up the project in a mission mode from September 2009. Members of the new subgroup on geotechnical investigations met several times at NDMA, New Delhi and at Department of Civil Engineering, Indian Institute of Science, Bangalore under the chairmanship of Honorable member of NDMA Dr. Bhattacharjee and Prof. T.G.Sitharam of IISc as convener and has prepared this technical report.

This technical document is intended to serve as a reference document for the geotechnical aspects of seismic microzonation work in the country. Geotechnical and geophysical investigations required along with necessary number of tests, details of tests including methodology and precautions to be taken while carrying out these tests is elaborated in the various chapters of the technical document. The guidelines for carrying appropriate geotechnical and geophysical investigations for seismic microzonation studies are laid out in the report. It is noted that the procedure laid out in the guidelines document are well accepted internationally and has been used all over the world. The document addresses all the aspects of geotechnical and geophysical testing for characterizing the ground and methods of analyses available for estimating local site effects. The members of WCE-GT deliberated the contents of the report and the suggestions of the members were duly considered in the presentation of final report.

A core committee met at NDMA, New Delhi on 5th August 2011 and has decided that the final report is acceptable and can be presented in a National Workshop which has been planned for 19th August 2011, at NDMA Bhawan, New Delhi. The national workshop has been planned to disseminate and discuss the guideline document with the experts invited from all over the country. The present document is the Final Report incorporating the suggestions made by the members of WCE-GT during many deliberations since October 2009. I am glad to present the final report of this project. This type of effort is first of its kind in India and it is going to pave the way for taking up seismic microzonation studies on a scientific and rational basis.

The report along with recommended detailed investigations for selected two urban areas, which needs to be carried out in phase II of the project (as per the guidelines presented in this report) would be useful in establishing procedures for carrying out microzonation of the urban centres in the entire country and for rational design of structures, leading to building of safe and economical habitat. These results would also help in incorporation of seismic risk reduction strategies in selection of appropriate land use planning.

I would like to express my gratitude to General N.C. Vij, PVSM, UYSM, AVSM (Retd.), past Vice-Chairman of NDMA, for his valuable guidance at various stages of this project. Further, I also would like to thank Shri. M. Shashidhara Reddy the present Vice Chairman of NDMA for his valuable guidance at various stages of this project. I also place on record my sincere thanks to the distinguished members of the NDMA for their valuable insights and suggestions from time to time.

I would also like to express my sincere thanks to the Chairman and the members of WCE-GT, who have helped in achieving the objectives of the project. I thank Prof. P. Balaram, Director of Indian Institute of Science, Bangalore for agreeing to take up this project of national importance as a grant-in-aid project and in bringing out this report.

B. Bhattacharjee

Date: 19-08-2011
Place: New Delhi

National Disaster Management Authority

Executive Summary

Need for NDMA to prepare this National Document

Significant earthquake damage and loss of life has been directly related to the effect of local site conditions and liquefaction in several recent earthquakes (eg., 1985 Mexico earthquake, 1989 Loma Prieta Earthquake, 1994 Northridge, 1995 Kobe Earthquake, and Bhuj earthquake 2001). Even though there are potentially many factors contributing to earthquake damage, the site amplification of ground motion attributed to local site conditions and liquefaction are very important in terms of increasing seismic hazard at ground level and consequently the damage. Both of the local site conditions and liquefaction related damage are very essential component of a comprehensive assessment of seismic hazard and are attributed to geotechnical / geophysical characteristics of soil overburden closure to the ground. The importance of geotechnical / shallow geophysical characteristics of soil overburden in characterizing seismic ground motions has long been recognized. A major thrust of many studies was the classification of strong motion sites according to surface geology, average shear wave velocity in the upper 30 m (Vs-30, calculated as the ratio of 30 m to travel time for shear waves to travel from 30 m depth to the ground surface), and a geotechnical classification scheme. Further, seismic site response is also reported to be a function of soil depth. Thus ignoring soil depth may introduce a desirable level of uncertainty in ground motion prediction. Factors such as geologic age and cementation may also affect the nonlinear response of soil and rock, and the effect of soil nonlinearity is largely a function of soil type. Site effects have also been introduced into most of the current ground motion attenuation relationships and most of these relationships account site effects only through a broad site classification system that divides the sites into either rock or soil or with the additional soft soil (BIS 1893:2002). Potentially liquefiable sand deposits (Site class F) and soft soils (Site Class E) were excluded from the ground motion attenuation relationships, because response at these sites is mainly a function of whether or not liquefaction is triggered or partially triggered. Data from recent earthquakes clearly highlights that a detailed studies are required and also suggest that a refinement in the soil classification system is warranted to achieve the improved prediction of ground motions. For problems like zonation of a very large area like zonation of country or a continent, macro level is adopted and macrozonation is carried out considering seismicity, geology and larger scales without considering geotechnical aspects. But microzonation is generally done in smaller scale by considering regional seismicity, geology and local site conditions based on site characterization. It is also necessary to define the scale, which is acceptable for microzonation. Scale of geotechnical studies is very much dependent on heterogeneity and variation of site conditions and type of soils. A complete site characterization is essential for the seismic site classification and site response studies, which are carried out by detailed geotechnical/geophysical investigations. There is a clear role for geological and geotechnical data in evaluating the ground motions and in turn in the study of seismic hazard and preparation of geotechnical data driven microzonation maps to provide an effective solution for city planning.

Ground in particular subsurface overburden weathered soil is highly heterogeneous and also very complex. There are very large number of geotechnical and geophysical tests available for 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 the many factors starting from the purpose and the scope of the study, availability of resources (equipment and expertise personal), type of analysis to be carried, type of soil, etc. Geotechnical and geophysical investigations require for seismic microzonation are different from the conventional site investigations. The extent of area to be investigated for microzonation generally spans over several 10's of 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.

During the national workshop organized by NDMA on 16th July 2008 clearly debated the need of preparation of technical document on geotechnical / geophysical investigations for seismic microzonation studies of urban centers in India. Even the present adopted practice of geotechnical investigations for seismic microzonation were discussed and felt the need for detailed guidelines as no such guidelines are available in the country. Many institutions/agencies/individuals have reported based on one type of test or one type of results and called it as "seismic microzonation". Most of the geotechnical and geophysical studies carried earlier for seismic microzonation studies in India 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 adoption of proper interpretation techniques required for seismic microzonation. During the national workshop on seismic microzonation of Indian landmass held on 16th July 20008 at NDMA, at that time the existing working group for geotechnical investigations formed under the WCE-microzonation (vide No NDMA/BB/S&T-1/2007 dated 21st Sept 2007) was expanded by inducting additional members with Prof. Sitharam as convener of the group and IISc as the nodal institution to prepare the technical document on geotechnical investigation for seismic microzonation studies in India. Members of the new subgroup on geotechnical investigations met several times at NDMA, New Delhi under the chairmanship of Honorable member of NDMA Dr. Bhattacharjee and discussed the terms of reference of the committee and finalized the following terms of reference for preparation of geotechnical guidelines.

Terms of reference

Preparation of step by step guide lines for the three stages of seismic microzonation by highlighting input/data standards, analysis and output results and validations. Deliverables will be the detailed guidelines and recommendations for seismic microzonation experiments.

Guidelines will also address the resources required and quality assurance plans along with work breakdown structure and schedule to be followed. It will also highlight disadvantages and advantages of the different methods.

Experiment the developed procedures for one or two cities in India and confirm / validate the established procedures and also highlight all the problems and issues during field trials. Further, recommend the same guidelines to all the state government disaster teams for their future experiments.

The guidelines will be immediately useful for the detailed microzonation exercise of two selected cities in India. The guidelines so developed are also helpful to the owner/developer seeking approval of specific development projects within zones of required investigation and to the engineering geologist and/or civil engineer who must investigate the site and recommend mitigation of identified hazards. The guidelines so developed will also be helpful to engineers and other officials involved in the planning and development of approval process of major projects. Effective evaluation and mitigation ultimately depends on the combined professional judgment and expertise of the evaluating and reviewing professionals.

Further, the same was presented to the meeting of steering committee (geophysical hazard) which was held on 5th Nov 2008 and the TOR was approved by NDMA. NDMA has signed an MOU on 7th July 2009 with IISc to prepare the national document with the visit of officials from NDMA (Mr. Ajay Sule, Director- Mitigation) to IISc.

The WGE-GT Technical Document has provided the details of the methods to be adopted at the field and/or laboratory investigations along with the details of the corresponding equipment/instruments to be used and monitoring mechanism to be kept in place for ensuring the "quality" of the generated data. The methodology for interpretation of the data has also been spelt out for the various tests/investigations recommended. Tech Document has also clearly spelt out the detailed "Site –Specific Investigations" required to be undertaken for any important projects. This national document on geotechnical guidelines has highlighted the gaps in "Geotechnical Investigation" for seismic microzonation. It has also brought out the details of the items which need to be considered while carrying out the seismic microzonation and evaluate the seismic hazard at the ground level considering the local site effects.

Scope of this Document

The scope of the project includes preparation of a technical document giving all the relevant details about Geotechnical / Geophysical Investigations for seismic microzonation. The memorandum of understanding between IISc and NDMA has been enclosed in Annexure III. MOM clearly highlights that the Technical Document shall address geotechnical inputs needed by structural engineers for design, retrofitting and construction work at a given site that may have liquefaction potential. In addition, to the geotechnical work needed for Disaster Management work at NDMA/town planners, the Technical Document will, therefore, include, inter alia, the following (so that the Technical Document serves the purpose of an excellent Reference document for any detailed geotechnical investigation to be undertaken in the country). Details of the project objectives are listed in Annexure III.

This technical document is intended to serve as a reference document for the geotechnical aspects of seismic microzonation work in the country. Geotechnical and geophysical investigations required along with necessary number of tests, details of tests including methodology and precautions to be taken while carrying out these tests is elaborated in the various chapters of the technical document. Finally, guidelines for carrying appropriate geotechnical and geophysical investigations for seismic microzonation studies are laid out in the concluding chapter.

Overview of the Contents

Planning of site investigations (geophysical and geotechnical investigations) depends upon the scope of the seismic microzonation (eg. its scale, procedures adopted, etc) to be carried. Hence, proper planning of site investigation often requires to understand the underlying principles of seismic microzonation and its methodology. A General overview of the methodology for seismic microzonation is presented in the Chapter 1. This chapter also offers brief description of various components of the microzonation. It outlines the required geotechnical and geophysical investigations and their methodologies. It also lists the major cities which have high seismic risk (seismic zone III and above) with population over half a million.

Chapter 2 presents general procedure for carrying out site investigations for the purpose of seismic microzonation. It also provides guidelines for the proper planning of subsurface explorations. It discusses importance of the geotechnical and geophysical methods for microzonation and their suitability. Recommendations are given for selecting appropriate scales for microzonation studies based on vulnerability and population density of a region, in this chapter. Suitable scales for data collection using geotechnical and geophysical tests are also suggested here, based on the vulnerability of a region and heterogeneity in the soil subsurface.

Chapter 3 presents different in situ tests available for site characterization. The details of different low strain and high strain tests are also listed in this chapter. It also provides procedures for the evaluation of dynamic properties from the laboratory as well as in situ tests.

Local site conditions play significant role on the amplification of seismic waves and the resulted earthquake disasters. In depth discussions are made in Chapter 4, on the various local site conditions which influence the ground shaking. Different available methods for the assessing the local site conditions are presented in the Chapter 4. Various codal provisions for site classifications are also discussed here.

Various parameters which influence the liquefaction susceptibility are presented in the Chapter 5. It provides detailed procedures for evaluation of liquefaction potential based on field and laboratory tests. This chapter also explains various liquefaction phenomenon associated with earthquakes and the methods to quantify it.

The details of different geotechnical and geophysical studies carried for seismic microzonation in the India are presented in Annexure II. Problems associated with the current practices are highlighted here. The main issues in microzonation works and the codal provisions related to geotechnical and geophysical testing are also explained in this.

List of Abbreviations

The following list gives some of the important abbreviations used in this report. The symbols which are not given here are defined as and when they appear in text.

ASHA	Average Horizontal Spectral Amplification
CPT	Cone Penetration Test
CRR	Cyclic Resistance Ratio
CSR	Cyclic Shear Resistance
GIS	Geographical Information System
GSHAP	Global Seismic Hazard Analysis Program
HVSR	Horizontal to Vertical Spectral Ratio
JMA	Japan Meteorological Agency
LHZ	Landslide Hazard Zonation
LSA	Landslide Susceptibility Analysis
MASW	Multichannel Analysis of Surface Waves
MMI	Modified Mercalli Intensity
MSF	Magnitude Scaling Factor
MSK	Medevedev-Sponhever-Karnik
PGA	Peak Ground Acceleration
PHA	Peak Horizontal Acceleration
PVA	Peak Vertical Acceleration
RF	Rossi-Ferral scale
SASW	Spectral Analysis of Surface Wave
SEM	Spectral Element Mesh
SPT	Standard Penetration Test

Terminology and Definitions

Accelerogram

It is the record from an accelerograph showing acceleration as a function of time.

Accelerograph

It is an instrument recording ground or base acceleration.

Aftershock

One of a series of smaller quakes following the main shock of the earthquake.

Amplification

It is the increase in amplitude of vibration at surface level when compared to the rock level motion.

Amplitude

Maximum deviation from mean of centerline of a wave.

Anisotropy

It is the property of being directionally dependent, as opposed to isotropy, which implies identical properties in all directions.

Attenuation

Reduction of amplitude or change in wave due to energy dissipation over distance within time.

Base Isolation

A method using flexible bearings, whereby a building superstructure is detached from its foundation in order to reduce earthquake forces.

Base Shear or Equivalent Lateral Force (ELF)

Total shear force acting at the base of a structure.

Bed Rock

It is the solid rock that underlies loose material, such as soil, sand, clay, or gravel.

Brittle Failure

Failure in material due to limited plastic range, material is subjected to sudden failure without warning signs.

Capillary Action or Capillarity

A phenomenon where liquid spontaneously rises in a narrow space such as a thin tube, or in porous materials. This effect can cause liquids to flow against the force of gravity.

Compaction

Compaction is the process by which soil grains are brought to a closer packing (i.e. increasing density) by means of various types of loading.

Consolidation

It indicates the time dependent compressibility behavior of saturated fine grained soils.

Core

The central part of the earth below a depth of 2,900 kilometers. It is thought to be composed of iron and nickel and to be molten on the outside with a central solid inner core.

Creep (along a fault)

It is a very slow periodic or episodic movement along a fault trace without earthquakes.

Crust

The lithosphere, the outer 80 kilometers of the earth's surface made up of crustal rocks, sediment and basalt. The general composition is silicon-aluminum-iron.

Damping

The rate at which natural vibration decays as a result of the absorption of energy.

Critical Damping

The minimum damping that will allow a displaced system to return to its initial position without oscillation.

Deflection

The horizontal or vertical displacement of a member due to the application of external force.

Deformation

It is the permanent distortion due to seismic forces.

Depth of Focus

The depth of the focus or hypocenter beneath the earth's surface commonly classes Earthquakes: Shallow (0- 70 kilometers), intermediate (70-300 km), and deep (300-700 km).

Design Earthquake

It is generally defined as 2/3 of the maximum considered earthquake.

Diaphragm

It is generally a horizontal member, such as a floor or roof slab, which distributes lateral forces to vertical resisting elements.

Displacement

It is the lateral movement of the structure caused by lateral force.

Drift

It is the horizontal displacement of basic building elements due to lateral earthquake forces.

Ductility

Ability to withstand inelastic strain without fracturing. Ductility is a material property to fail only after considerable inelastic (permanent) deformation which process dissipates the energy from the earthquake by design.

Duration

The period of time for which ground acceleration (ground shaking) lasts.

Earthquake

A shaking of the earth's crust caused by breaking or shifting of rock beneath the ground surface.

Earthquake Hazard

Any physical phenomenon, such as ground shaking or ground failure, that is associated with an earthquake and produce adverse effects on human activities.

Earthquake Preparedness

Safety measures taken before an earthquake to reduce the damage.

Earthquake Risk

The expectation of loss of human life and properties during an earthquake. . Earthquake risk can be expressed as below:

Earthquake Risk = Earthquake Hazard*Vulnerability*Value at Risk

Earthquake Zoning

It is the identification of zones of similar levels of earthquake hazard. If the earthquake zoning is done on a national scale it is called macrozonation.

Effective Acceleration

The earthquake acceleration which is closely related to structural response and to damage potential of an earthquake. Effective acceleration is a function of size of loaded area, frequency content, weight, embedment, damping characteristics of the structure and its foundation.

Effective Stress

The net stress take by the soil skeleton is termed as the effective stress. For 100% saturated soil mass, the effective stress is equal to total stress minus pore water pressure. Effective stress, σ', at a point in a soil mass is equal to the total stress, σ , at that point minus the pore water pressure, μ , at that location, i.e.

$$\sigma' = \sigma - u$$

Energy Dissipation

Reduction in intensity of earthquake shock waves with time and distance, or by transmission through discontinuous materials with different absorption capabilities.

Engineering Bed Rock

It is defined as the layer of sedimentary rock of Pliocene or earlier age with shear wave velocity more than 700 m/s.

Epicenter

The point on the earth's surface directly above the focus or hypocenter of an earthquake.

Equivalent Lateral Force (ELF)

The representation of earthquake forces on a building by a single static force applied at the base of a building; also referred as Base Shear.

Failure Mode

The manner in which a structure fails (column buckling, overturning of structure, etc).

Fault

A fracture plane in the earth's crust across which relative displacement has occurred.

- ***Normal Fault*** – A fault under tension where the overlying block moves down the dip or slope of the fault plane.
- ***Strike-Slip Fault*** (or lateral slip) – A fault whose relative displacement is purely horizontal.
- ***Thrust (Reverse) Fault*** – A fault under compression where the overlying block moves up the dip or slope of the fault plane.
- ***Oblique-Slip Fault*** – A combination of normal and slip or thrust and slip faults whose movement is diagonal along the dip of the fault plane.

Faulting

The movement which produces relative displacement of adjacent rock masses along a fracture.

Fault Zones

The zone surrounding a major fault, consisting of numerous interlacing small faults.

Fine Grained soil

Soil in which the smaller grain sizes are predominate, such as silt, and clay.

Flexible System

A structural system that will sustain relatively large displacements without failure.

Focus / Hypocenter

The point at which the rupture occurs and marks the origin of the kinetic waves of an earthquake.

Frequency

The number of wave peaks or cycles per second.

Fundamental or Natural frequency

It is the frequency at which the response of structure is maximum. If the frequency of exciting force is equal to the natural frequency of the structure, that structure will sustain heavy damage.

Natural frequency $f_n = (1/2\pi)\sqrt{(K/M)}$ K – is the stiffness and M is the mass

Natural Period

is the inverse of fundamental frequency

$T_n = 1/f_n$

Graben (rift valley)

Long, narrow trough bounded by one or more parallel normal faults. These down-dropped fault blocks are caused by tensional crustal forces.

Ground Acceleration

Acceleration of the ground due to earthquake forces.

Ground Displacement

The distance that ground moves from its original position during an earthquake.

Ground Failure

A situation in which the ground does not hold together such as land sliding, mud flows and liquefaction.

Ground Movement

A general term which includes all aspects of ground motion: acceleration, particle velocity and displacement.

Ground Velocity

Velocity of the ground during an earthquake.

Ground Water Table

It indicates the level below which the ground is completely saturated with water.

Input Motion

A term representing seismic forces applied to a structure.

Inelastic

Behavior of an element beyond its elastic limit, having permanent deformation.

Inertial Forces

Earthquake generated vibration of the building's mass causing internally generated inertial forces and building damage. Inertial forces are the product of mass times acceleration (F = m a).

Intensity

It is the quantity defining the severity of ground shaking on the basis of observed effects in a limited area The principal scale used today is the Modified Mercalli, scale which is based on observation of the effects of the earthquake MM-I to MM-XII (MM-I = not felt, MM-XII = damage nearly total).

Isoseismals

Contours drawn to define limits of estimated intensity of shaking for a given earthquake.

Landslide

Earthquake triggering land disturbance on a hillside where one land mass slides over the other.

Lateral Force Coefficients

Factors applied to the weight of a structure or its parts to determine lateral force for seismic structural design.

Liquefaction

Transformation of a granular material (soil) from a solid state into a liquid state as a consequence of increased pore water pressure induced by sudden loading especially during earthquake.

Magnitude

A measure of earthquake size which describes the amount of energy released in the form of seismic waves.

Mantle

The main bulk of the earth between the crust and the core.

MCE- Maximum Credible Earthquake, the largest earthquakes that can reasonably be expected.

Microzonation

Microzonation is the process of subdividing a seismic zone into smaller zones according to certain criterions.

Mode

The shape of the vibration curve.

Moment Magnitude

It is the measure of total energy released by an earthquake and is based on the area of the fault that ruptured in the quake. It is calculated in part by multiplying the area of the fault's rupture surface by the distance the earth moves along the fault.

Mud Flow

It is a rapid movement of a large mass of mud formed from loose soil and water.

Nonstructural Components

Those building components, which are not intended primarily for the structural support and bracing of the building.

Period

The elapsed time in seconds of a single cycle of oscillation, inverse of frequency.

Performance Based Design

Performance based design tries to ensure that the entire system will perform in some predictable way in terms of safety and functionality. The seismic aspects of this performance based design are termed as the performance based earthquake engineering (PBEE).

Piezometric Profile

It is an imaginary surface that represents the static head of groundwater and is defined by the level to which water will rise. It is also known as isopotential level; piezometric surface; pressure surface.

Prevention

It includes all measures taken before an earthquake event in order to reduce the earthquake risk.

Resonance

When the vibrating frequency of the structure is very close to the natural frequency of the structure and the value of damping is very small, the amplitude of motion will be very high and this state is known as resonance.

Response Spectrum

The response spectrum is defined as the plot showing the peak response (in terms of displacement, velocity or acceleration) of a series of oscillators, which are having different natural frequencies and are put into motion by the same base vibration.

Return Period of Earthquakes

The time period (years) between large earthquakes at a particular site.

Richter Magnitude Scale

is defined as the logarithm (base 10) of the maximum trace amplitude (in micrometer) recorded on a Wood-Anderson seismometer located 100 Km from the epicenter of earthquake.

Rift

A fault trough formed in a divergence zone or in other areas in tension.

Seiche

A standing wave on the surface of water in an enclosed or semi-enclosed basin (lake, bay or harbor).

Seismic Zone

Areas defined on a map within which seismic design requirements are constant.

Seismicity

Seismicity refers to the geographic and historical distribution of earthquakes at a particular location.

Seismograph

A device which is used to detect and record ground motion.

Shear Strength

It is the magnitude of the shear stress that a soil can sustain without any shear failure.

Soil Compressibility

It indicates the overall load deformation behavior of soils resulted from elastic compression, compression of air voids and water (consolidation).

Soil Structure Interaction

The process in which the response of the soil influences the motion of the structure and vice-versa is termed as soil-structure interaction.

Spectra

A plot showing the amplitude as a function of period or frequency of a structure.

Stiffness

Rigidity or resistance to deflection or drift. It is the force required for unit deflection.

Subduction

The sinking of an oceanic plate under an overriding continental plate in a plate convergent boundary.

Time Dependent Response Analysis

Study of the behavior of a structure with respect to time as it responds to a specific ground motion.

Trench

A long and narrow deep trough in the sea floor; interpreted as marking the line along which a plate bends down into a subduction zone

Tsunami

A sea wave produced by large displacements of the seafloor as a result of earthquakes, volcanic activity or submarine slide.

Undisturbed Sampling

Undisturbed soil samples are those that are cut, removed, and packed with the least possible disturbance. They are samples in which the natural structures, void ratio, and moisture content are preserved as carefully as possible.

Vulnerability

It is the susceptibility to damage during an earthquake. The vulnerability of a structure depends mainly on size, mass, structure layout, irregularities, material types, construction details.

Wave Length

It is the distance between successive similar points on two wave cycles.

1 Introduction

1.1 Preamble

Natural calamities like earthquakes can neither be predicted nor be prevented. However, the severity of the damages can be minimized by proper infrastructure planning based on microzonation studies and by following appropriate construction procedures according to the earthquake resistant designs. Seismic microzonation studies provides the expected level of shaking in a region and associated seismic risks such as liquefaction, lateral spreading, landslides, tsunamis, etc.

India has experienced most disastrous earthquakes i.e. Assam 1897 (M=8.7), Kangra 1905 (M=8.6), Bihar-Nepal 1934 (M=8.4), Assam-Tibet 1950 (M=8.7), Uttarkashi 1991 (M=6.5), Latur 1993 (M=6.4), Jabalpur 1997 (M=6.0), Chamoli 1999 (M = 6.8), Bhuj 2001 (M= 7.6) and Kashmir 2005 (M=7.4) in the recent past. According to the zonation map of India (BIS-1893, 2002), entire country is divided into four seismic zones of various expected probable intensities of earthquake occurrence based on effective PGA values as well as comprehensive intensity scale (CIS-64) (BIS-1893, 2002).The Global Seismic Hazard Assessment Program (GSHAP) was launched in 1992 with a primary goal to create a global seismic hazard map in a harmonized and regionally coordinated fashion. The GSHAP strategy was to establish regional centers, which were responsible for the coordination and realization of the four basic elements of the seismic hazard analysis. The GSHAP has classified the Indian peninsula into several seismogenic zones based on the seismicity and tectonic setup. Bhatia et al. (1999) under GSHAP program estimated the seismic hazard for the Indian region using probabilistic approach.

After the devastating earthquake in Gujarat in 2001, the Government of India has paid serious attention to seismic microzonation and it has accepted its importance as a guiding tool in land use planning and safe construction practices to avoid the loss from the future earthquakes. As a result of this, seismic microzonation of various urban areas, like Delhi, Jabalpur, Chennai, Bangalore, Lucknow Ahmedabad etc. are being carried out by different researchers in the country. However, as there are no standard procedures and/or guidelines available for seismic microzonation, these studies are considerably differ from each other, particularly in site characterization studies. As there are number of geotechnical and geophysical studies available for dynamic site characterization, it is often confusing to select a particular test or methodology.

The purpose of this current document is to provide detailed information on the existing methodologies and procedures on geotechnical and geophysical methods, and then provide guidelines to carry out proper tests and methodologies to standardize the procedure for carrying out seismic microzonation. NDMA, as nodal agency, aimed to provide strategies to minimize the seismic hazards. As a part of the exercise, NDMA is developing the guidelines for carrying out seismic microzonation. Different committees formed by NDMA are looking after formation of guidelines for different phases of Seismic microzonation. The current document is focused at the geotechnical and geophysical aspects of seismic microzonation. Further the utility of this current document is elaborated in Sections 1.4 and 1.4.1.

Before initiating process of site investigations for seismic microzonation, it is required to sketch out the planning of geophysical and geotechnical investigations to be carried. Planning of site investigations is dependent upon the scope of the seismic microzonation (eg. its scale, procedures adopted, etc) to be carried. Proper planning of site investigation is often required to understand the underlying principles of seismic microzonation and its methodology. Hence, an overview of seismic microzonation and its procedure is provided here in this chapter with an emphasis on geotechnical/geophysical investigations.

1.2 Seismic Microzonation

The large and rapidly growing urban seismic risk, particularly in developing countries like India is a problem that needs to be solved quickly. This can be done through a comprehensive seismic microzonation of the area. Microzonation is a process of dividing a seismically active region into sub regions such that any characteristic of interest may be considered to be reasonably same over these micro zones. When such factors or characteristic related to seismic activity is mapped into micro zones and the process is called Seismic Microzonation.

The first attempt of seismic microzonation of any urban area i.e. an industrial as well as population center was carried out in city of Yokohama, Japan in 1954 considering various zones, corresponding soil conditions and design seismic coefficients for different types of structures located in that different zones. Subsequently, in view of the immense usefulness of microzonation studies were conducted in few earthquake prone areas of the World (Marcellini et al., 1998; Chavez-Garcia and Cuenca, 1998; Lungu et al., 2000; Faccioli and Pessina, 2001; Fäh et al., 2001; Alfaro et al., 2001; Ansal et al., 2004 etc).

Slob et al. (2002) presented a technique for microzonation for the city of Armenia in Colombia. In this study they used a 3D layer model in GIS, combined with a 1D calculation of seismic response using SHAKE to get the spatial variation in seismic response which was checked with the damage assessment of Armenia. They concluded that the amplification was high in the range of frequency of 5 Hz for houses with 2 stories, it become true after the earthquake, in which low rise building experienced more damage than high rise building. Topal et al. (2003) considered various parameters for microzonation such as geological, geotechnical, seismotectonic and hydrogeological conditions and on the basis of these, four different zones was proposed for the Yeneshir an urban area in Turkey. Ansal et al., (2004) adopted a probabilistic approach in a microzonation study for the city of Siliviri, Turkey and estimated the local geological and geotechnical on the basis of the available borehole data and laboratory test result. For site characterization the average shear wave velocity was used, that was determined from seismic refraction tests.

Need for Seismic Microzonation

Seismic microzonation is the first step in earthquake risk mitigation study and requires multidisciplinary approach with major contributions from the fields of geology, seismology, geophysics, geotechnical and structural engineering. This is very important to identify the tectonic and geological formations in the study area which is essential for determining the seismic sources and also for establishing a realistic earthquake hazard models for the investigation.

The very important issue affecting the applicability and the feasibility of any microzonation study is the reliability of the parameters for doing it. It is very clear based on the earthquake damage and strong motion records that there are numerous source and site factors i.e., near field effects, directivity, duration, focusing, topographical and basin effects etc which are important in assessing ground motion characteristics. The national seismic zoning maps which are prepared in small scales like 1:1,000,000 or less neglecting all the above factors and they does not consider the geological and geotechnical site conditions.

Seismic microzonation involves a very detailed field investigation to evaluate the hazard. It is very effective in delineating the spatial variations in the seismic hazard. They are also useful to evaluate the risk scenarios in the study area. This has been the most widely used method to map earthquake hazard at local scales which may incorporate a wide variety of information, including seismic response of different surface geological formations, liquefaction potential, topographic amplification of seismic waves, landslides, tsunamis etc. Seismic microzonation maps are very useful in urban planning because they help to predict the impact of future earthquakes and can also be used to locate key facilities like hospitals, fire stations, emergency operation centers etc. Microzonation studies are also very useful to save the heritage and important structures from future major earthquakes.

The Turkish codes insist that the seismic microzonation has to be done in vulnerable cities / regions with a population of 30,000 or more. However, since the population in India is very high, the microzonation need to be carried out for the cities based on population and macro hazard maps available in the country. For example microzonation can be taken up for the urban centres with a population of 500,000 to 1,000,000 in an area and which falls in seismic zone five. While selecting the regions for microzonation, the population of the region, population density, its socio-economic characteristics, gross domestic product (GDP) of the region and the strategic importance etc. also need also to be considered. Seismic microzonation of urban centers requires map of city with a scale of 1:5,000 to 1:25,000.

1.3 General Procedure for Seismic Microzonation

The scale of zonation depends mainly on the available database and on the quality of the zonation map required. Several inputs regarding seismicity, geology, geomorphology and geotechnical characteristics are needed for doing seismic microzonation of an area.

In essence, it is necessary to estimate response of soil layers under earthquake excitations and thus the variation of earthquake ground motion characteristics on the ground surface. A general methodology in doing the seismic microzonation of a region can be divided into the following four major heads.

- Estimation of the ground motion parameters using the historical seismicity and recorded earthquake motion data which includes the location of potential sources, magnitude, mechanism, epicentral distances.
- Site characterization using geological, geomorphological, geophysical and geotechnical data.
- Assessment of the local site effects which includes site amplification, predominant frequency, liquefaction hazard, landslides, tsunami etc.
- Preparation of the seismic microzonation maps

The methodology that can be adopted in carrying out the detailed microzonation studies in an area is shown in Figure1.1. Seismic microzonation, which is subdivision of an area into smaller zones depending upon site-specific seismic response, is an effective mitigation method. The detailed geology, seismo-tectonics and geotechnical characteristics of the area are the basic inputs to be considered in the microzonation process. The geotechnical soil properties and the depth of the bedrock are very important in the site response studies. Also, it influences the amplification and attenuation of strong motion amplitudes. So, detailed geological, geophysical and geotechnical data is required for seismic site characterization of the region. The site characterization and assessment of site response during earthquakes is one of the crucial phases of seismic microzonation**.** There are many factors that influence the way a site will respond to earthquake ground motion. A qualitative and quantitative estimation of local site effects is often expressed by the amplification factor and fundamental frequency, which can be estimated effectively through microtremor studies. Seismic hazard assessment with respect to ground shaking, liquefaction, site amplification, landslides, tsunamis etc are very crucial in microzonation studies. The current document mainly deals with site characterization, assessment of site amplifications through site response analysis, and estimation of liquefaction hazards. Seismic hazards associated with landslides and tsunamis are dealt with other research groups of NDMA.

Role of Geotechnical and Geophysical Investigations in Seismic Microzonation

In seismic microzonation of urban centres, in spite of the inherent uncertainty and risk, geotechnical /geophysical investigations form an important step. Geotechnical information becomes invaluable in the evaluation of hazards at ground surface, local site effects and liquefaction More accurate the exploration, the greater freedom the planner will have as this will provide an more accurate estimate of hazards. One of the most difficult and controversial aspects of any geotechnical/geophysical investigation is deciding how much exploration to do. This is mostly an art or at best an inexact science. An adverse geology might require more investigations than the average. Since there are no standards and no "handbook solutions" to the amount of investigation that should be done, in this document an attempt is made to recommend the course of action to determine all the needs of geotechnical/geophysical exploration. Investigation spans over several square kilometers to sever hundreds of square kilometers. Hence, careful planning of testing program is very essential, to obtain representative site characterization with the minimum number of tests. These data will be of very high importance in evaluating the induced earthquake effects such as site amplification, local site effects, liquefaction susceptibility and slope stability. The smaller the grid size the accuracy will be more in evaluating hazard in an area. But this will increase the cost and the man power requirement for field testing, data collection and the analysis.

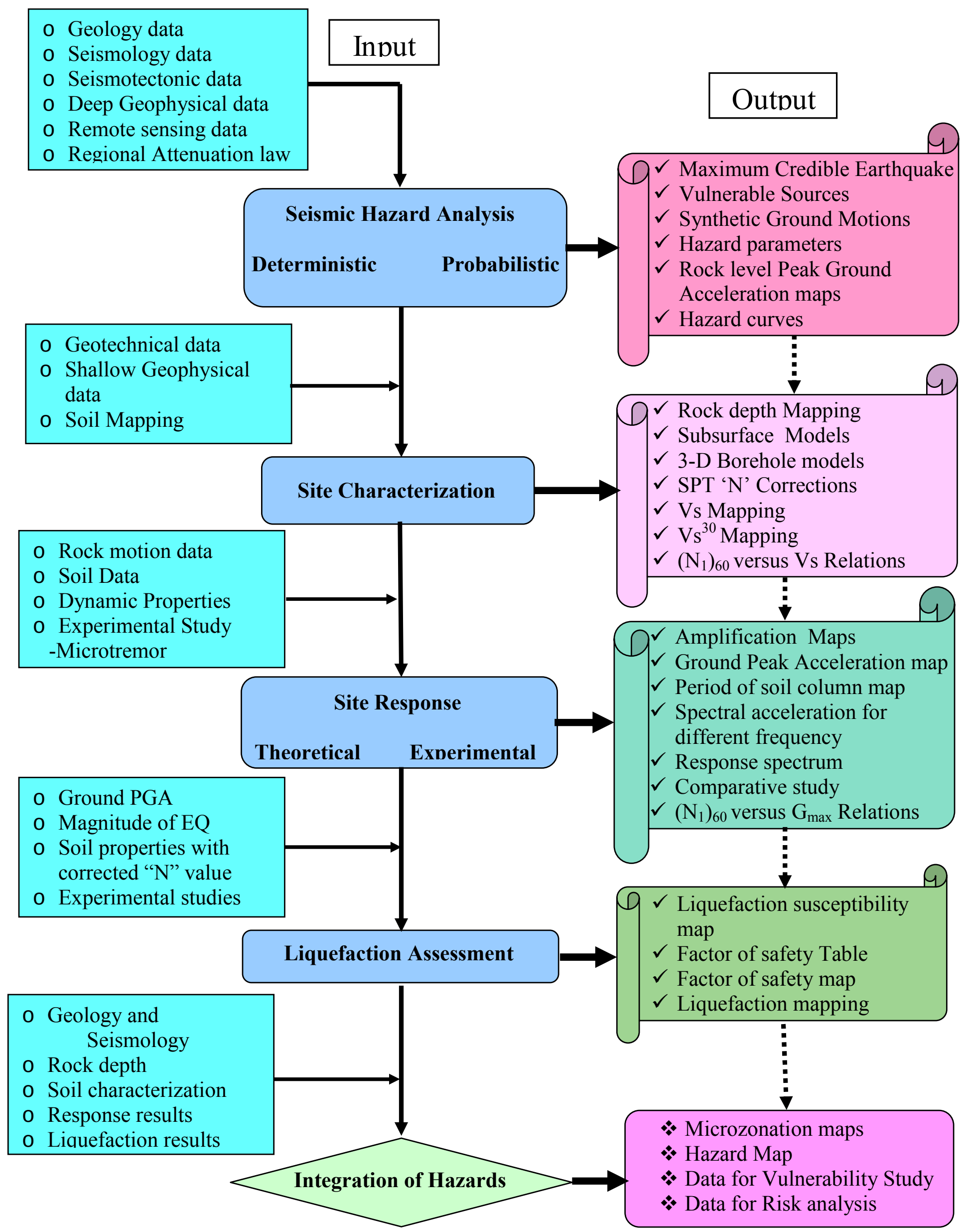

Figure 1.1: General Methodology for Seismic Microzonation studies

Different Grades/Levels of Seismic Microzonation

The local site effects during earthquakes are related to geotechnical characteristics such as amplification, liquefaction, landslide and mudflow. To assess these geotechnical characteristics three grades of approaches to zonation was suggested by ISSMGE Technical Committee for Geotechnical Earthquake Engineering along with the accepted scales of mapping (Table 1.1). The level of zonation depends mainly on the available database and on the quality of the zonation map required. Several inputs regarding seismicity, geology, and geotechnical characteristics are needed for doing seismic microzonation of an area.

ISSMGE Technical Committee for Geotechnical Earthquake Engineering classified the seismic microzonation studies into three grades/levels. Details of these three classes of seismic microzonation studies are given in Table 1.1. The appropriate selection of grade of seismic microzonation is very important starting point of the seismic microzonation study. Selection of the grade of seismic microzontion is mainly depends upon the seismicity of the region, extent of coverage area, and population density/importance of the study. The following paragraphs briefly explains each of these three levels of zonation and also provides the guidelines on selection of the appropriate level of zonation for the purpose of seismic microzonation of urban centers in India.

Level – I/Grade - I

The procedures to carry out seismic microzonation at this Grade-1, is similar to macrozonation studies carried for National seismic zoning. The main inputs required are available seismological data, regional attenuation relation (if available else attenuation relation of similar regions can be used), to obtain bedrock/input motions. The site effects(surface ground motions) are estimated usually from the available geological and geomorphological maps. Rarely, actual geophysical and geotechnical field investigations are carried for accounting actual site conditions.

Table 1.1: Three Levels of Zonation (Modified from TC4-ISSMGE4, 1999)

Geotechnical Phenomenon	Grade-1	Grade-2	Grade-3**
Ground Motions	▪ Historical Earthquakes and Existing Information ▪ Geological Maps ▪ Interviews with Local Residents	▪ Microtremor ▪ Simplified Geotechnical Study	▪ Geotechnical Investigation ▪ Ground Response Analysis
Slope Stability	▪ Historical Earthquakes and Existing Information ▪ Geological and Geomorphological Maps	▪ Air Photos and Remote Sensing ▪ Field Studies ▪ Vegetation and Precipitation Data	▪ Geotechnical Investigation ▪ Analysis
Liquefaction	▪ Historical Earthquakes and Existing Information ▪ Geological and Geomorphological Maps	▪ Air Photos and Remote Sensing ▪ Field Studies Interviews with Local Residents	▪ Geotechnical Investigation ▪ Analysis
Accepted Scale of Mapping	1:10,00,000 to 1:50,000	1:1,00,000 to 1:10,000	1:25,000 to 1:10,000
Grid size for testing geophysical survey and bore hole	▪ Homogeneous sub-surface – 2 km x 2km to 5 km x 5km ▪ Heterogeneous Sub-surface – 0.5 km x 0.5 km to 2 km x 2km (selectively)	▪ Homogeneous sub-surface – 1 km x 1km to 3 km x 3km ▪ Heterogeneous Sub-surface – 0.5 km x 0.5 km to 1 km x 1km	▪ Homogeneous sub-surface – 0.5 km x 0.5km to 2 km x 2km ▪ Heterogeneous Sub-surface – 0.1 km x 0.1 km to 0.5 km x 0.5k

** For seismic microzonation of urban centers with high seismicity (in seismic zones III, IV and V), Grade – 3 level of zonation is to be adopted.

This level of seismic microzonation studies are not recommended for urban centers. This Level-I microzonation may only be adopted only for cities with low seismic hazard (Zone-II and Zone-III) and less population. Scale of study and mapping varies from 1:10,00,000 to 1:50,000 under this level of microzonation.

Level – II/Grade – II

The inputs required for this level of seismic microzonation is more elaborate than the previous scale. The required data will include.

- seismological data and models
- Specific attenuation relation and recurrence relation
- Geotechnical data and the reports of geotechnical studies
- Validation of collected and available data
- Laboratory studies
- Field and experimental studies

This level of microzonation is to be adopted for regions in higher seismic hazard (zones IV and V) with less population. For the regions with moderate seismic hazard (zone III), but with population exceeding 100,000 this scale can be adopted. Scale of study and mapping is larger than the previous level of microzonation. ISSMGE recommended scale of study and mapping is 1:1,00,000 to 1:10,000.

Level III

This level of seismic microzonation is to be adopted for highly vulnerable areas with high population density. This level of microzonation requires rigorous and thorough inputs from geotechnical and geophysical investigations. The PHA values need to be evaluated using probabilistic methods and site response analysis using equivalent linear and non linear analysis. A detailed liquefaction hazard analysis based on deterministic and probabilistic methods need to be carried. Both the site response analysis and liquefaction hazard analysis needs the inputs from both field and laboratory tests. The tsunami and landslide analysis has to be done in those regions which are prone to these hazards.

The output maps of microzonation in this scale will include

- Site classification maps
- Spatial variation of PHA and Sa values at bed rock level
- Spatial variation of PHA and Sa values at surface level
- Liquefaction hazard maps
- Landslide and tsunami maps
- Comprehensive seismic hazard map prepared by combining all the above given maps (after giving proper weightage for each factors)

For seismic microzonation of urban centers with high seismicity (in seismic zones III, IV and V), Grade – 3 level of seismic microzonation is to be adopted. 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 III microzonation maps. The list of these cities is given in the Table 1.2. The recommended scale of study and mapping is 1:25,000 to 1:10,000.

The national seismic zoning maps (macro zonation studies) are mostly at small scales such as 1:1,000,000 or less and are mostly based on seismic source zones defined at similar scales. Procedures that are usually adopted for such studies are similar to grade-I seismic microzonation studies. However, seismic microzonation for a city / town requires 1:10,000 to 1:25,000 scale studies and needs to be based on seismic hazard studies at similar scales (eg. grade-III, microzonation studies). The country wide macrozonation maps which are produced by national experts will go through a careful review process. However it may not be possible to adopt such a review system for the large number of seismic microzonation studies in the country. Geological Survey of India maps are available at 1:50,000 scale with regard to base map, contours, geomorphology, geology etc. It is advisable to explore these maps for microzonation studies immediately. For some important mega cities this exercise can be repeated at large scales like 1:10000. This is presently being done for National Capital Region Delhi. One possible solution for this scale incompatibility is to increase the scales of seismic macrozonation maps steadily with the accumulation of geological and seismological data as implemented in USA and Japan. Geological formations, local site classification, equivalent shear wave velocity, spectral acceleration, spectral amplification and their variation are some of the parameters studied during a seismic microzonation studies. A consistent approach has to be implemented to assess each parameter with respect to all other parameters. The objective of seismic zonation is to establish a seismic hazard map at a scale of 1:10000 taking into account earthquake source and local site conditions. Thus estimation of the earthquake induced forces and their variation in the investigated area is the main target in seismic microzonation.

Table 1.2: List of Cities with Over Half a Million Population in Seismic Zone III IV and V (BMPTC, Govt. of India, UNDP and MHA)

Sl. No.	State	Name of the City	District	Seismic Zone	Zone Factor
1	Uttaranchal	Dehradun	Dehradun	IV	0.24
2	Delhi	Delhi	New Delhi	IV	0.24
3	Gujarat	Jamnagar	Jamnagar	IV	0.24
4	Gujarat	Rajkot	Rajkot	III	0.16
5	Gujarat	Bhavanagar	Bhavnagar	III	0.16
6	Gujarat	Surat	Surat	III	0.16
7	Maharashtra	Greater Mumbai	Mumbai	III	0.16
8	Maharashtra	Bhiwandi	Thane	III	0.16
9	Maharashtra	Nashik	Nashik	III	0.16
10	Maharashtra	Pune	Pune	III	0.16
11	Orissa	Bhubaneshwar	Khurda	III	0.16
12	Orissa	Cuttack	Cuttack	III	0.16
13	Tamil Nadu	Chennai	Chennai	III	0.16
14	Bihar	Patna	Patna	IV	0.24
15	West Bengal	Asansol	Bardhaman	III	0.16
16	Assam	Guwahati	Kamrup	V	0.36
17	Gujarat	Vadodara	Vadodara	III	0.16
18	Gujarat	Ahmedabad	Ahmedabad	III	0.16
19	Tamil Nadu	Coimbatore	Coimbatore	III	0.16

Sl. No.	State	Name of the City	District	Seismic Zone	Zone Factor
20	Uttar Pradesh	Agra	Agra	III	0.16
21	Uttar Pradesh	Varanasi	Varanasi	III	0.16
22	Uttar Pradesh	Bareilly	Bareilly	III	0.16
23	Uttar Pradesh	Meerut	Meerut	IV	0.24
24	Uttar Pradesh	Lucknow	Lucknow	III	0.16
25	Uttar Pradesh	Kanpur	Kanpur nagar	III	0.16
26	West Bengal	Kolkata	Kolkata	III	0.16
27	Jammu & Kashmir	Srinagar	Srinagar	V	0.36
28	Jammu & Kashmir	Jammu	Jammu	IV	0.24
29	Madhya Pradesh	Indore	Indore	III	0.16
30	Madhya Pradesh	Jabalpur	Jabalpur	III	0.16
31	Punjab	Amritsar	Amritsar	IV	0.24
32	Punjab	Jalandhar	Jalandhar	IV	0.24
33	Andhra Pradesh	Vijayawada	Krishna	III	0.16
34	Jharkhand	Dhanbad	Dhanbad	III	0.16
35	Karnataka	Mangalore	South Canara	III	0.16
36	Kerala	Kochi	Ernakulam	III	0.16
37	Kerala	Kozhikode	Kozhikode	III	0.16
38	Kerala	Trivandrum	Trivandrum	III	0.16

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

As discussed earlier, first step in the 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 the seismic events and the seismic sources need to be identified from this region. The initial estimates of ground motion parameters (PHA, PHV or PHD) and/or bed rock motions should be obtained at the bed rock level using deterministic and probabilistic methods. The required information will be furnished by the other group, PSHA.

It is now the responsibility of the geotechnical engineer to estimate the influence of local soil conditions and topography on the earthquake motions and obtain the surface ground motions.

Site Characterization

Local geological conditions are having considerable effect on ground motion at a given site. When subjected to the earthquake ground motions the response of different soil types differ. Usually the younger softer soil amplifies ground motion relative to older, more compact soils or bedrock. Local amplification of the ground is often controlled by the soft surface layer, which leads to the trapping of the seismic energy, due to the impedance contrast between the soft surface soils and the underlying bedrock. Natural frequency of each soil layer depends on the physical properties of soil and the depth to bedrock. The main aim of the site response study is to evaluate the amplification of ground motion and determination of natural resonance frequency of the soil. The site characterization need to be done based on geotechnical, geophysical and seismological inputs.

Geotechnical studies

- Geotechnical site characterization should be done based on SPT, CPT and shear wave velocity measurements from MASW or SASW tests. However the selection of site classes need to be done based on the average shear wave velocity obtained from insitu measurements. If the data is obtained by converting SPT or CPT data, the error involved in these relations will deteriorate the quality of the average shear wave velocity values and this in turn will affect the accuracy of the results.
- Determination of strain dependent modulus and damping parameters characterizing the soil properties of any site under consideration may also be carried out.
- Equivalent linear model using the soil column and input bed rock motion should be used for site effect estimation. The input bed rock motion should be provided by PSHA group. Topography and basin effects also may be considered.
- Amplification factor deduced from geotechnical studies should be used judiciously in conjunction with earthquake data driven site amplification.
  - One of the important parameter to be considered in geotechnical studies is the scale of geotechnical data collection. It varies from 0.5 km x 0.5km to a maximum 5 km x 5km. Details of selection of appropriate scales for geotechnical studies is discussed at length in Chapter 2.

Local Topographical/Site Effects

- Surface level ground motion parameters, preferably peak ground acceleration (PGA) values should be evaluated considering local site effects.
- For regions where the terrain is uneven, the topographical effects need to be considered.
- The response spectra and the smoothed design response spectra need to be developed based on the site class.
- The development of response spectra may be done based on codal provisions.

Liquefaction Studies

- The factor of safety against liquefaction can be evaluated based on the geotechnical data obtained from the field.
- The liquefaction potential evaluation can be done based on the deterministic or probabilistic methods.
- Liquefaction potential can be reported in terms of SPT and CPT values required to prevent liquefaction can also be developed (if required).
- Liquefaction factor of safety evaluated based on the available geotechnical data and using probabilistic techniques can then be used for the liquefaction hazard assessment.

Other Parameters

- In hilly terrain, Landslide Hazard Zonation Map to be prepared according to the Bureau of Indian Standards guidelines
- In coastal areas, possibility of a Tsunami needs to be considered. The effects of Tsunami would be inundation of large area, making the ground completely saturated and in turn susceptible to liquefaction and lateral sliding.
- 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 scale or larger and accordingly due weights and ranks should be assigned for each layer/ parameter based on local conditions.

1.4 Utility of the Current Technical Document

The final objective of the seismic microzonation is to assess the surface level ground motions and resulted seismic hazards such as liquefaction, landslides. The input motions (ie. bed rock motions) at the bed rock level, can be obtained by carrying probabilistic seismic hazard analysis. The required bed rock motions or the procedures required to generate bed rock motions, will be supplied by the other group, PSHA. Then, the next step would be to obtain surface ground motions after accounting local site effects. The current technical document provides the detailed information on how to account these local site effects on the input or bed rock motions to obtain surface ground motions. Guidelines are laid in this document on how to estimate local site effects through various geotechnical and geophysical tests. Detailed descriptions and procedures are also given for assessing liquefaction potential of a given site.

As the scale of study for the purpose of seismic microzonation is very large compared to conventional geotechnical investigations, we need very careful and judicious planning of geotechnical and geophysical tests. Detailed information is given in Chapter 2, on how to plan the required geotechnical and geophysical investigations. Further, the nature of investigations and testing required are different from investigations required for static testing. This document also provides the guidelines over the selection of required methods. Some guidelines are also included on the required number of boreholes and other tests necessary for seismic microzonation studies, to minimize the ambiguity over the quantum of required investigations. Finally, it provides the details of various ground response analysis and other methods to quantify local site effects related site amplification and liquefaction.

The current document provides the planning of geotechnical and geophysical tests that are required for seismic microzonation purpose. The results from such studies are very useful in the urban infrastructure planning. The information is basically useful in the initial planning for identifying suitable locations for various important structures such as hospitals, fire stations, schools, colleges, community centers, cinema theaters and etc. These studies also help to adopt appropriate construction practices to avoid seismic hazards. However, based on the importance of the structure and seismicity of the region, site specific studies are required to be carried out. For such studies, the information from the microzonation studies becomes part of the preliminary investigations. However, detailed geotechnical and geophysical site investigations are required to be carried out. The information from the geotechnical and geophysical explorations from the seismic microzonation studies is not adequate for such projects. As the soil is very erratic and complex in nature, every site is unique in its composition of geology of materials, geometry and properties of subsurface layers. Considering these facts, site specific studies are required to be carried for important structures based upon the significance of the structure to be built.

The information contained in this document is also useful for carrying out geotechnical/geophysical tests required for site specific studies. Information such as selection of geotechnical and geophysical methods and their procedures to obtain the required properties of subsurface layers given in this document is useful. The main difference between seismic microzonation and site specific studies is the scale of the study. Area under site specific studies usually spans over several hundred square meters unlike several square kilometers in seismic microzonation studies. Site specific studies require very detailed site investigations to completely characterize the site from earthquake loading point of view. Hence, the information given in this document over the selection of appropriate tests and procedures would be useful for site specific studies. However, the required number of tests and other details over the planning of location of bore holes etc. would be different. These details would not be available in this document.

This report also helps to identify the seismic risks associated with the existing structures in the urban centers apart from providing planning of new structures. Based on the seismic microzonation study, it is possible to find out the vulnerability of the existing structures. If the existing important structures such hospitals, fire stations and other public utility buildings are found at high risk based on seismic microzonation studies, it may be further required to carry out the site specific studies as the actual local site conditions may be different from the seismic microzonation studies. These studies help to establish the actual involved seismic risk and its magnitude at these sites. Based on these site specific studies, the actual need of remedial measures can be assessed. Based on the results (actual expected level of shaking at the site), retrofitting of the existing structures or modification of the structure and/or its utility is taken up to minimize the seismic risks.

Utility for the Nodal Agencies

This document provides very important information that is required for the nodal agencies to decide upon the necessity and importance of seismic microzonation for the given region. It guides the level/grade of the seismic microzonation required for the chosen region. It provides the preliminary information over the extent of geotechnical and geophysical investigations required. It provides the detailed guidelines on the selection of appropriate geotechnical/geophysical tests and the methodology to be followed. It provides the various procedures to estimate local site effects to finally provide surface ground motions and liquefaction potential of the region. The final outcome of seismic microzonation studies usually provided in the form of hazard maps, useful for the urban planning of various important structures. It is also useful for the estimating seismic risks involved with the existing structures. Based on the need, site specific studies may be further required to be carried out to estimate required extent of strengthening for the existing structures and/or new structures.

1.5 Summary

The general methodology for seismic microzonation has been discussed in detail here along with the different aspects of seismic microzonation. Different levels of the seismic microzonation and their required inputs have been discussed. Guidelines have been provided for selection appropriate grade of seismic microzonation. Emphasis has been given for specifying the different scales required for geotechnical studies and microzonation. The list of Indian cities (with more than half a million population), which fall in seismic zone III and above are also listed. Utility of the current document and its scope of applicability is discussed in detail.

2 General Procedures for Geotechnical/Geophysical Investigations for Seismic Microzonation

2.1 Introduction

Earthquake is one of the most deadly natural disasters because of uncertainties involved in predicting it. Seismic microzonation plays an important role in effective evaluation of the hazard and it is an excellent seismic hazard mitigation tool. Seismic microzonation, which is subdivision of an area into smaller zones depending upon site-specific seismic response. When the division is done considering any criteria related to earthquakes, like seismic intensity, damages of building, liquefaction potential, it is called seismic microzonation. Seismic microzonation requires multi-disciplinary expertise to evaluate the hazard at the ground surface. It also requires a comprehensive understanding of the effects of earthquake ground motions on man-made structures, slopes and soil sites. It can be considered as the process for estimating the response of soil layers under earthquake excitations and thus the variation of earthquake ground motion characteristics on the ground surface.

Site characterization is very important aspect of the seismic microzonation. The accuracy of expected levels of surface shaking and associated earthquake hazards depends upon the accuracy of the determination of site characteristics. There are very large number of site investigations methods are available for site characterization. Each of the method has advantages and limitations over other methods. Hence, selection of a method requires understanding of the each of the method useful for seismic microzonation. Details of each of the test along with its procedure and limitations are provided in Chapter 3. Apart from the selection, it is also important to decide upon the number of tests required. In seismic microzonation, the extent of area under investigation is very vast. It spans over several square kilometers to sever hundreds of square kilometers. Hence, careful planning of testing program is very essential, to obtain representative site characterization with the minimum number of tests. This chapter discusses about these issues, which are required for proper planning of site investigations.

2.2 Seismic zonation

Zonation is a process of dividing a seismically active region into regions or zones according to a certain criterion to evaluate the hazard. The purpose of this division is to get an area for which various characteristics may be considered fairly uniform. Seismic microzonation may be defined as the assessment of variations in earthquake hazards resulting from differing foundation conditions within a larger area that is prone to seismic activity. Earthquake hazards refer to those effects which have the potential to cause harm to people, property or the environment. Seismic microzonation procedure may provide the general guidelines to be followed in the region and may form the basis for regulations for most of the structures. Seismic microzonation is the evaluation and assessment of different inputs from different fields of earthquake geotechnical engineering. In most general terms, seismic microzonation is the process of estimating the response of soil layers under earthquake excitations and thus the variation of earthquake characteristics on the ground surface. However, it is also very important to select appropriate ground motion parameters for microzonation that correlate with the observed structural damage (Finn, 1991).

Generally, the response of the structures during an earthquake is not only related to structural features but also controlled by two main factors i.e., earthquake source characteristics and local site conditions. One way of resolving this problem is to consider the interaction among all three components, the structures, site conditions and incoming seismic waves. The characteristics of incoming earthquake waves may be changed during their propagation through soil layers. In addition, soil characteristics may also change due to the induced cyclic excitations. It requires multi-disciplinary contributions as well as comprehensive understanding of the effects of earthquake generated ground motions on manmade structures (Ansal and Slejko, 2001). The main objective in microzonation studies is to estimate more accurately the possible damage during an earthquake in the future taking into account all the main controlling factors.

As discussed earlier, it is required to choose appropriate scale of the study, before seismic microzonation is carried. As the selection of this scale influences the scale of geotechnical/geophysical investigations, the details of various scales are discussed in the next section.

2.3 Field Data Collection Scales

One of the areas in microzonation studies which have not received its due attention is the scale with which the geophysical and geotechnical data need to be collected. These data will be of very high importance in evaluating the induced earthquake effects such as site amplification, local site effects, liquefaction susceptibility and slope stability. With the advancement of computational facilities it is easy to evaluate the peak horizontal acceleration (PHA) values at bed rock level for any scale (grid size). But to get an accurate subsurface profile, the scale at which the field investigation and data collection has to be done is a matter of concern. The smaller the grid size the accuracy will be more in evaluating hazard in an area. But this will increase the cost and the man power requirement for field testing, data collection and the analysis.

It is advisable to analyze the PHA value to get an initial assessment of seismic hazard at a location. In addition to this the existing data available with different agencies or organizations can be analyzed to check the spatial variability of the soil profile. If the spatial variability of soil profile is relatively less with low hazard levels, then a coarser grid can be selected for data collection else a finer grid has to be selected. For those areas which are vulnerable against liquefaction or slope failure, a more detailed geotechnical and geophysical data collection can be done.

The amount of geotechnical testing need to be done for any microzonation work is usually determined by experience and budgetary concerns. As of now there are no standards and handbook solutions available to determine the scale of geotechnical investigation that should be done. Since major or complex microzonation projects demand a greater level of geotechnical effort, the first step is to determine level of microzonation. This will determine whether the project will likely require a high level or a relatively lower level of geotechnical effort. Naturally, some aspects of geotechnical investigations are similar for all the microzonation works. Some of the general guidelines which can be followed are

- Determine geotechnical parameters needed for the present study and prioritize them

- Use the available geologic and geotechnical data to the maximum extent possible
- Conduct exploration in at least two phases
  - In phase one the stress need to be given to development of regional geology and initial site investigation.
  - The initial exploration phase should be well-funded so that sufficient geotechnical data is obtained and this should further guide the future explorations.

Use the geophysical testing also when ever applicable. This will reduce the cost and a large area can be covered in a shorter time.

First step before initiating of the seismic microzonation is to choose appropriate grade/level of seismic microzontion to be carried out. Details and guidelines for selection of appropriate level of seismic microzonation is already discussed in the Section 1.3.2. As discussed, seismic microzonation for urban centers require to carry out level-III seismic microzonation studies. This requires a map of a city with a scale of 1:5,000 to 1:25,000. Other levels of seismic microzonation studies are recommended based on the extent of the study area, seismicity and population density. The required field data collection scales for these three levels of microzonation are discussed in the following paragraphs.

Level – I

This level of seismic micro zonation is suitable for regions with low hazard (regions in zone – II) with low population. The inputs required are available seismological data, regional attenuation relation (if available else attenuation relation of similar regions can be used) and the available geological, geotechnical, geomorphological and microtremor data. If no such geotechnical and geophysical data is available, the following proposed grid size may be chosen for geotechnical data collection:

- Homogeneous sub-surface – 2 km x 2km to 5 km x 5km
- Heterogeneous Sub-surface – 0.5 km x 0.5 km to 2 km x 2km

Level - II

This level of seismic microzonation is to be adopted for regions in higher seismic hazards. For the regions with moderate seismic hazard, but with population exceeding 100,000 this scale can be adopted. This level of microzonation requires some inputs from actual geotechnical and geophysical site investigation data. The field data is collected according to the following grid sizes.

- Homogeneous sub-surface – 1 km x 1km to 3 km x 3km
- Heterogeneous Sub-surface – 0.5 km x 0.5 km to 1 km x 1km

Level III

This level of seismic microzonation is to be adopted for highly vulnerable areas with high population density as discussed in the Section 1.3.2. This grade of study is recommended for the seismic microzonation of urban centers with high seismicity (in seismic zones III, IV and V) with the population exceeding half a million. This level of studies require detailed site characterization using rigours and thorough geotechnical and geophysical investigations using finer data collection grids. The proposed scale for geotechnical data collection for the Level - III microzonation is as follows:

- Homogeneous sub-surface – 0.5 km x 0.5km to 2 km x 2km

- Heterogeneous Sub-surface – 0.1 km x 0.1 km to 0.5 km x 0.5km

Site response analysis using equivalent linear or non linear analysis is to be carried using inputs obtained from geotechnical and geophysical investigations. A detailed liquefaction hazard analysis based on deterministic and probabilistic methods need to be carried. This level of seismic microzonation provides detailed maps of site classification, spatial variation of bedrock as well as surface amplifications, liquefaction, landslide and tsunami, as discussed in the Section 1.3.2.

Note: Level – III seismic microzonation and field data collection is recommended for the seismic microzonation of urban centers with high seismicity (in seismic zones III, IV and V) with the population exceeding half a million.

2.4 Site Characterization

As discussed, the site characteristics and its geomorphology (eg. valley, basin, ridge effects, etc), play an important role on the observed response of surface ground motions. Site specific ground response analysis account for these effects and estimate the surface ground motions resulted from the soil response to bed rock motions. Selection of ground response analysis (eg. 1D, 2D or 3D ground response analysis), depends upon mainly topography (geomorphology) of the site. The required soil properties for site characterization are obtained from either geotechnical tests and/or geophysical tests. Broadly, site investigation required for seismic microzonation consists of following two components:

- Arriving at the surface mapping (local topography) of the site/region
- Planning and execution of subsurface Investigations

Various Phases in the Site Investigation:

Scope of the any investigation always depends upon the purpose of the study. Site investigations for the purpose of seismic microzonation are similar in many respects to routine geotechnical site investigations. The main difference is in the scale of the investigations. General geotechnical investigations are of smaller size covering few meters to hundreds of acres depending upon the size of the building or any other proposed construction. Where as in case of seismic microzonation, the extent of the investigations ranges from few square kilometers to hundreds of square kilometers. The other difference is that site investigations for seismic microzonation require mainly dynamic site characterization from the perspective of earthquake loading.

Similar to the regular site investigations, one has to first carry out geologic reconnaissance study for proper planning of the investigations. It can be done through collecting available data, terrain analysis through topographic maps. Then Site reconnaissance to confirm the data. Finally, preparation of subsurface exploration program for site investigations is done based on the reconnaissance. These four phases of site investigation are discussed below.

(i)Collection of DATA:

One should gather as much material as possible, before initiating investigations. Often, large amount of information is available in the literature for a given location. One should look for the following information from the literature.

- Bedrock geology, including major structural features such as faults

- Surficial geology in terms of soil types on a regional or if possible, local basis
- Climate conditions, which influence soil development, ground water fluctuations, erosion, flooding, slope failures, etc.
- Associated geological hazards such as ground subsidence and collapse, slope failures

(ii) Terrain Analysis:

It is most important part of the site investigation. Landforms and other surface characteristics are strong indices of geologic conditions and help to choose appropriate ground response model. Characteristic terrain features reveal several useful information such as rock type, structural forms where rock is shallow and weathering conditions, erosion, or represent typical soil formations in terms of their origin, mode of deposition, thickness of the deposits. Engineering geology maps can be prepared from the terrain analysis that provides information about the geologic conditions over an entire study area.

(iii) Site Reconnaissance:

One should have fair idea about the site conditions, before planning the investigations. If necessary, one should carry reconnaissance survey of the region under the interest for seismic microzonation. From the survey, one should gather information regarding geology, terrain and accessibility to exploration equipment, existing structures and their condition, existing utilities and potentially hazard conditions. This will help for the proper planning of scale of investigation, selection of a method and number of tests to be carried out.

While carrying the local site exploration one should carry reconnaissance on the following issues.

- Examine exposures of soils and rocks in cuts for possible visual classification of site
- Contact local contractors, engineers and local well drillers for the information on the foundation and local soil conditions
- Gather information from the well and bore wells regarding the depth of ground water table

(iv) Subsurface explorations:

There are different methods available for subsurface explorations require for seismic microzonation. Some of them are indirect methods like geophysical methods, and some of them are direct methods like geotechnical exploration methods, which allow the examination of materials, recovery of samples and their subsequent testing.

Subsurface exploration methods are required to obtain and/or to confirm subsurface profile from other preliminary studies. They also provide information on the ground water conditions. Various subsurface methods can be grouped into indirect (geophysical) and direct (geotechnical) methods. Brief explanation of these techniques is provided in the following sections.

Geotechnical Explorations

Scope of the investigation is always depends upon the purpose of the study. Investigations for the seismic microzonation are similar in many respects to regular investigations. The main difference is the scale of the study. In general geotechnical investigations are of smaller size covering few meters to hundreds of acres depending upon the size of the building or any other proposed construction. In case of seismic microzonation, the extent of the investigations to be covered is varying form few square kilometers to hundreds of square kilometers.

Geotechnical engineering analysis and evaluation is valid only if the measured values are representative of in situ conditions. Properties of some materials are best measured in the laboratory, while others in field tests. The general objective of the geotechnical/geophysical investigation for the microzonation is to account all the significant factors that influence on the seismic hazards. This objective is achieved only through the proper planning of in situ and laboratory testing.

Geotechnical investigation involves the following tasks for the purpose of microzonation.

- Need to define/identify the bedrock depth, which is very important as purpose of the geotechnical investigations is to assess the influence of local site conditions on the bedrock/earthquake motions.
- Obtain surface mapping to account influence of topography features and geomorphology conditions on the expected levels of earthquake shaking. Arrive at the topography and identify geological hazards if any, such as unstable slopes, faults, floodplains
- Define groundwater table conditions considering seasonal variations
- Perform in situ testing and procure samples for laboratory testing

Various geotechnical tests including both in situ and laboratory, available for seismic microzonation are discussed at length in the Chapter 3. The recommended methods and their procedures for the purpose of seismic microzonation are explained in the Chapter 6. Various boring and sampling techniques are explained in detail in Chapter 3. However, it is important to mention that such geotechnical investigations are suitable for depths up to 50 to 60 m. Beyond these depths, undisturbed sampling becomes difficult.

Geophysical Explorations

General geotechnical investigations involve the drilling of holes in the ground, sampling at discrete points, and in situ or laboratory testing. This methodology suits for exploration of smaller volume of soil and rock. However, for seismic microzonation, one needs to carry explorations on larger volumes. Geophysical methods overcome this drawback and some of the other problems inherent in conventional geotechnical investigation techniques.

There are many geophysical methods available today. Most of the methods can provide the profiles of continuous sections. Some of the techniques can also provide stiffness properties of the ground, which are useful for seismic microzonation. Geophysical techniques also help in locating cavities, backfilled mine shafts and subsurface geological features such as faults and discontinuities.

In seismic microzonation, it is required to obtain detailed subsurface profile over the region of interest. It is difficult to carry conventional geotechnical site explorations over such a large region. In addition, carrying geotechnical site explorations over a large area is very expensive. Geophysical methods are only alternative to avoid these difficulties. These methods provide lateral variability of the near-surface materials beneath a site.

The general objective of the geophysical/geotechnical investigation for the microzonation is to account all the significant factors that influence on the seismic hazards. This objective is achieved only through the proper planning of in situ and laboratory testing. Geotechnical engineering analysis and evaluation is valid only when they are based on truly representative values of natural materials. It is very difficult to obtain undisturbed samples particularly in case of sandy soils. These problems are eliminated in the geophysical methods. These methods are generally, carried on the ground at in situ conditions. Geophysical methods carried for the purpose of seismic microzonation, should be aimed at the following information

- ❖ Depth of the bedrock
- ❖ Very small strain stiffness of the ground
- ❖ To study variability of soils

The methods useful for seismic microzonation are explained in the Chapter 3. These methods can be used in the subsurface explorations even up to the depths of 100 to 150 m below ground level. Beyond these depths especially in alluvial belts, there are no techniques for evaluating the subsurface. In such cases numerical techniques with different case scenarios will help in understanding deep basin effects. Guidelines for selecting appropriate geophysical method for seismic microzonation are provided in the concluding Chapter 6.

It is desirable to create a 3-D ground profile based on the geotechnical or geophysical exploration. The 3-D profile can be generated based on GIS platform. A representation of boreholes in 3-D is shown in Figure 2.1. It is represented as several donut elements of thickness 0.5m placed one over the other. The entire geotechnical data for the boreholes can be linked to this and this will be extremely useful in retrieving the data.

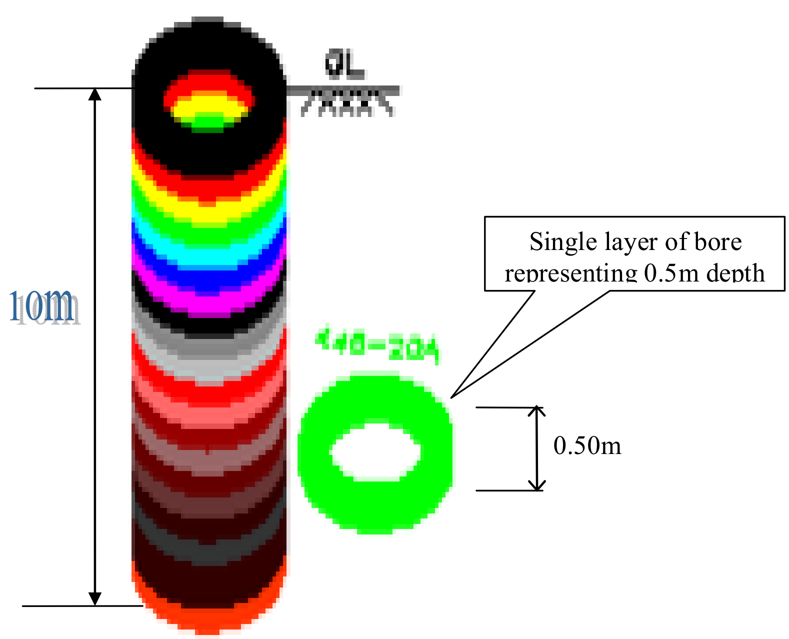

Figure 2.1: A typical 3-D borelog

2.5 Required field and laboratory data for seismic microzonation

Dynamic properties of soils such as shear modulus and damping are important in the design of geotechnical engineering problems involving dynamic loading. These dynamic properties of soils are strongly affected by the magnitude of shear strain amplitude induced in the soil deposits during strong earthquakes motions. Therefore it would be necessary to evaluate the dynamic properties of soil deposits for a wide range of shear strains. For the measurements of strain dependent dynamic properties, several laboratory and field techniques are being used. The recent developments in the numerical analysis for non-linear dynamic responses of grounds due to strong earthquake motions have increased the demand for the dynamic soil properties corresponding not only at small strain levels but also at large strain levels. The shear modulus and the damping of soils estimated at large strains would serve as a key factor for the assessment of ground response and geotechnical engineering related structures due to strong earthquake motions.

In addition to the evaluation of dynamic properties of the soils (shear modulus and damping), it is essential to evaluate liquefaction potential of the in-situ soil deposits. Which require estimation of liquefaction resistance of the in-situ soils as well as estimation of the expected levels of shaking due to earthquake excitation. There are number of methods available for estimation of liquefaction resistance of the soils using various geotechnical and geophysical tests.

Apart from the dynamic properties and liquefaction characteristics of the in-situ soils, it is essential to obtain the information about geometry of the subsurface soil layers, ground water table conditions, geology and depth of the bedrock.

2.6 General Guidelines for the Planning of Geotechnical and Geophysical Tests (e.g. Number and Locations of Tests)

The main purpose of geotechnical and geophysical investigations is to obtain the bed rock depth, geometry and properties of subsurface layers. The number of tests should be sufficient enough to establish bed rock and subsurface profile of the region under the investigation.

Number requirement of tests depends upon the nature of heterogeneity of subsurface layers. Hence, planning of location and number of tests require some preliminary understanding over the site conditions. For this purpose, one is required to obtain information from the available literature on the region, previous geotechnical/geophysical investigations carried in the region for various other purposes, and available geology and geomorphology maps.

First step before planning of tests, would be select the grid size. As discussed in the Section 1.3.1, selection of grid size depends upon the grade of seismic microzonation to be carried out, and the nature of heterogeneity of the subsurface layers. The following criteria may be adopted for planning of number of boreholes and other tests:

1. Choose the maximum possible coarse grid size suggested for the grade of seismic microzonation to be carried out (See Section 2.3).

2. Carryout out minimum number of geophysical and geotechnical tests at the selective grids to understand the heterogeneity of the subsurface conditions.

3. If the subsurface conditions are observed to be fairly homogeneous, then proceed with the same grid size otherwise revise the selected of grid size to finer grid.

4. Plan geophysical tests (eg. MASW, SASW, seismic reflection, seismic refraction. Suggestion of a specific method is given in the final chapter six) one at the center of the each of the grid, until unless site conditions prevents.

5. Drill boreholes and carry out geotechnical in-situ (SPT, CPT) & geophysical direct tests (such as cross hole or seismic up, down or SCPT tests which provides direct measurement of seismic wave velocity. Further details on recommendation of a specific test are provided in the concluding chapter). Plan at least one borehole for a set of nine grids(set of 3x3 grids).

6. Depth of drilling ideally should go up to the bedrock level. However, the availability of funds and resources often prohibits drilling of the bore hole to great depths. Hence, the minimum recommended depth of borehole for the purpose of seismic microzonation should be between 30 – 40 m until unless bedrock is met before.

7. At the same bore hole location try to obtain shear wave velocity profile from the indirect tests such as MASW, SASW, seismic refraction, reflection etc. (specific recommendation over these methods is made in the concluding chapter), so that the test results from the direct and indirect geophysical tests can be compared at the selective grid points.

8. It is also required to carry cross checking of the results between geotechnical in-situ and geophysical tests.

Here, the general broad selection of geotechnical and geophysical methods are only discussed. Exact selection or recommendation of specific tests is discussed later in the corresponding sections and in the concluding chapter. Annexure I gives the field tests for site characterization including BIS code numbers.

2.7 Summary

Various field data collection grid sizes required for the seismic microzonation are presented.. The grid sizes will vary depending on the degree of heterogeneity in subsurface conditions, extent of zonation and the purpose. Guidelines for the selection of the grid size based on the heterogeneity and the grade of the seismic microzonation are provided. General procedures for carrying out site investigations for the purpose of seismic microzonation are discussed. Guidelines for proper planning of subsurface explorations are also included. Importance and suitability of various geotechnical and geophysical methods for seismic microzonation are presented.

3 Site Characterization for Seismic microzonation

3.1 Introduction

Evidences from past earthquakes clearly shows that the damages due to an earthquake and its severity are controlled mainly by three important factors i.e., earthquake source and path characteristics, local geological and geotechnical characteristics, structural design and construction features of structures. Seismic ground response at a site is strongly influenced by local geological and soil conditions. The exact details of the geological, geomorphological and geotechnical data along with seismotectonic background and seismicity are needed to evaluate the ground response and site effects. The damage pattern during an earthquake depends on the soil characteristics at a site and it may have a major effect on the level of ground shaking. This point highlights the importance of site characterization in microzonation studies. The regional tectonic maps as well as surface geology maps and vertical geological profiles would be the essential ingredients for the seismic microzonation study. The characteristics and thickness of site's soil conditions are to be identified based on borings, in-situ geophysical and geotechnical tests. The geological, geomorphological and geotechnical databases are needed for assessing the local site effects for site amplification as well as for liquefaction and landslide susceptibility.

The geometry of the subsoil structure, the soil types and the variation of their properties with depth, the lateral discontinuities and the surface topography influence the amplification of ground motion and hence intensity of damage during destructive earthquakes. For this the accurate knowledge of the geology, geomorphology, geophysical data and geotechnical details are the key parameters controlling the damage severity during an earthquake.

3.2 Site Investigation

Site investigation provides samples for visual description, index testing. This helps to identify the strata at the site. Subsequent laboratory tests help to obtain index properties as well as engineering properties of the soils.

While the general ground exploration process will have different objectives, the following are the main objectives of site investigation carried for the purpose of seismic microzonation.

- To determine ground water conditions
- To carry index tests for classifying soils
- To obtain engineering properties of the soils
- To confirm lateral variability of soils

Site exploration consists of the following steps:

Borings reveals direct geologic evidence of the subsurface site conditions. Borings are necessary to obtain samples for laboratory tests, in addition to exploring geologic stratigraphy and structures. Borings also provide information on ground-water data. They are required to perform in-situ tests, install instruments, and explore the condition of existing structures. Boring is carried out in the relatively soft and uncemented ground (soils), close to ground surface. There are large numbers of methods available for making boreholes in soils strata. The most common boring methods are auger drilling, power augering, wash boring and light percussion drilling. This latter technique is well adapted to stony soils.

Boring is suitable for soils only, where as rotary drilling is used for both rocks and firm soils. In rotary drilling, rotary action along with downward force is used to grind the material using core cutter fitted with diamond drill bit. Even though it can be used for both rock and soil, it is easier to use in strong intact rock rather than in a weak weathered rocks and soils. It requires number of elements such as drilling machine or rotary rig, a flush pump, string of hollow drill rods and a drilling tool such as core cutter fitted with diamond drill bit.

Drilling has traditionally been used in the more competent and cemented, deeper deposits (rocks). It is now also widely used to obtain high-quality samples of heavily overconsolidated clays, for specialist laboratory testing. This method can produce holes to great depths, which can be used for in situ tests as well as for sampling, and can allow the installation of instrumentation.

Using trial pits, very detailed information on the stratification, strength and discontinuities in the soil can be obtained. It is the best method to obtain very high quality block samples, which can be used for laboratory testing. These pits can be excavated using either hand digging tools or machinery. Shallow trial pits provide insitu examination of near surface deposits at lower costs. However, cost increases significantly with the depth as deeper cuts needs supporting sides.

3.3 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), Seismic Cone Penetration Test (SCPT), Spectral Analysis of Surface Wave (SASW), and Multichannel Analysis of Surface Wave (MASW). The widely accepted method for site characterization is based on V_S^{30} values. In the absence of direct shear wave velocity measurement results, these values can be reduced from SPT and CPT values and can be used for site characterization.

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 procedure for carrying out the standard penetration test is discussed as follows (as given by BIS: 2131, 1981) (Figure 3.1):

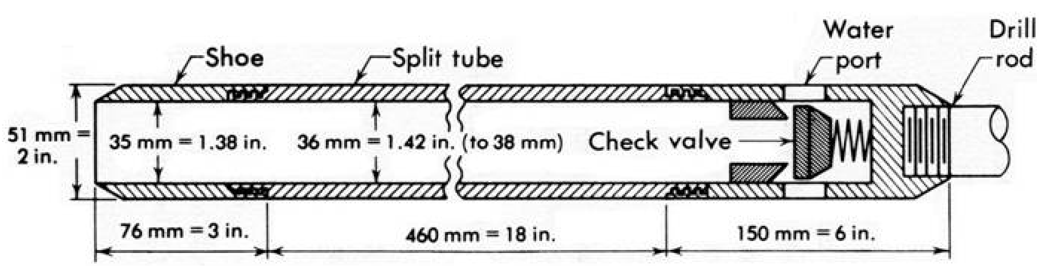

Figure 3.1: A Standard Penetrometer

- 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 to not 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, however, 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 the 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 detail 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 corrections, such as: overburden pressure, hammer energy, borehole diameter, presence of liners, rod length and fines content.

The SPT-N value is corrected as follows:

$$(N_1)_{60} = (N).C_N.\,C_R.\,C_S.\,C_B.\,C_E \quad (3.1)$$

Where, C_R - correction for rod length; C_S - correction for sampler configuration; C_B - correction for borehole diameter; C_F - correction for hammer energy efficiency;

The correction factors to be applied for the used are given in tables 3.1 – 3.4.

Table 3.1: Hammer Correction factor (C_E)

Factor	Correction
Donut Hammer	0.5-1.0
Safety Hammer	0.7-1.2

Automatic-trip	0.8-1.3

Table 3.2: Correction for B.H. Diameter (C_B)

Factor	Size	Correction
Borehole Diameter	65-115 mm	1.00
	150 mm	1.05
	200 mm	1.15

Table 3.3: Correction for Rod Length (C_R)

Factor	Size	Correction
Rod Length	<3 m	0.75
	3-4 m	0.80
	4 -6 m	0.85
	6-10 m	0.95
	10-30 m	1.00

Table 3.4: Correction for Sampler based on method (C_S)

Factor	Size	Correction
Sampling method	Standard samplers	1.00
	Sampler without liners	1.1-1.3

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

$$C_E = \frac{ER}{60\%} \quad (3.2)$$

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

It should also be noted that in the current studies, the corrected SPT- N value ($N_{1,60}$) is further corrected for its fines contents as follows.

$(N_1)_{60,CS} = \alpha + \beta(N_1)_{60}$ (3.3)

Where α and β are coefficients, which can be determined from the following relations.

$\alpha = 0$ for $FC \leq 5\%$ (3.4a)

$\alpha = \exp[1.76 - (190/FC^2)]$ for $5\% < FC < 35\%$ (3.4b)

$\alpha = 5.0$ for $FC \geq 35\%$ (3.4c)

$\beta = [0.99 + (FC^{1.5}/1000)]$ for $5\% < FC < 35\%$ (3.4d)

$\beta = 1.2$ for $FC \geq 35\ \%$ (3.4e)

Where, FC is the fines content in percentage and $N_{1,60}$ is in units of blows/ft.

The sources of some of the common errors while carrying out SPT tests are listed in Table 3.5 (Kulhawy and Mayne, 1990).

Table 3.5: Source of errors in SPT test

Cause	Effects	Influence on SPT-N value
Inadequate cleaning of hole	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	Increases
Failure to maintain adequate head of water in borehole	Bottom of borehole may become quick and soil may rinse into the hole	Decreases
Careless measure of hammer drop	Hammer energy varies	Increases
Hammer weight inaccurate	Hammer energy varies	Increases or Decreases
Hammer strikes drill rod collar eccentrically	Hammer energy reduced	Increases
Lack of hammer free fall because of increased sheaves, new stiff rope on weight, more than two turns on cathead, incomplete release of rope each drop	Hammer energy reduced	Increases
Sampler driven above bottom of casing	Sampler driven in disturbed, artificially densified soil	Increases greatly
Careless blow count	Inaccurate results	Increases or Decreases
Use of non-standard sampler	Corrections with standard sampler not valid	Increases or Decreases
Coarse gravel or cobbles in soil	Sampler becomes clogged or impeded	Increases
Use of bent drill rods	Inhibited transfer of energy of sampler	Increases

Advantages and disadvantages of SPT

The main advantage of SPT test is that it is the most widely used insitu test and lots of correlations are available to evaluate various soil properties. A disturbed sample is obtained from the test and this will help in identifying the soil type and percentage of fines. Since this is a large strain test, lots of correlations are available to evaluate the liquefaction potential. The SPT test is not possible in dense sand with boulders. There are lots of variability in the results and the results will depend on the type of execution also. More over the SPT is performed only at discrete increments and does not provide continuous soil profile. In addition to this the large number of corrections, which are to be applied to the SPT values, adds lots of uncertainty to the results.

Use of SPT values in evaluating the site effects

The major use of SPT data is in evaluating various soil parameters, soil profiling and liquefaction potential evaluation. There are lots of correlations available for obtaining other soil properties from SPT values. These include correlations to evaluate the relative density, angle of internal friction, specific weight, unconfined compressive strength etc. Readers can refer to Bowles (1997) for more details. In addition to this the consistency of soil can also be obtained from the SPT values. The methods for estimating shear wave velocity values from the SPT values are discussed in next chapter. In the absence of shear wave velocity data, these values can be obtained by converting the shear wave velocity values, which in turn can be used for the site classification purposes. A detailed description of various methods available to evaluate the liquefaction susceptibility based on SPT values is discussed in detail in chapter 5.

Cone Penetration Test (CPT)

Cone Penetration Test (CPT) is an insitu 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 cm^2 will be pushed through the ground at a controlled rate (2 cm/sec) (Figure 3.2). In 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_c) and sleeve friction (f_S) 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 ($FR = f_s/q_c$), will vary with soil type and it is also an important parameter. Even though CPT test doesn't provide any soil samples for direct soil classification, it is possible to obtain an approximate soil classification using the correlation developed by Robertson and Campanella (1983). Soil classification based on cone penetration resistance is shown in the Figure 3.3.

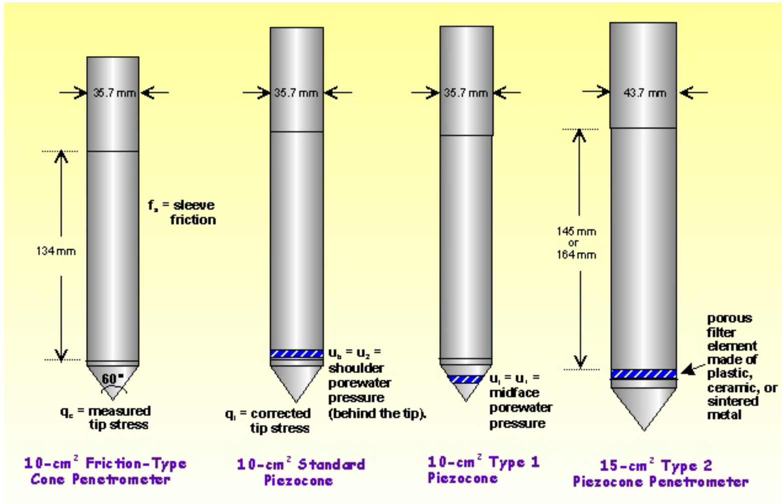

Figure 3.2: Different types of Cones used in CPT test
(http://geosystems.ce.gatech.edu/Faculty/Mayne/Research/devices/cpt.htm)

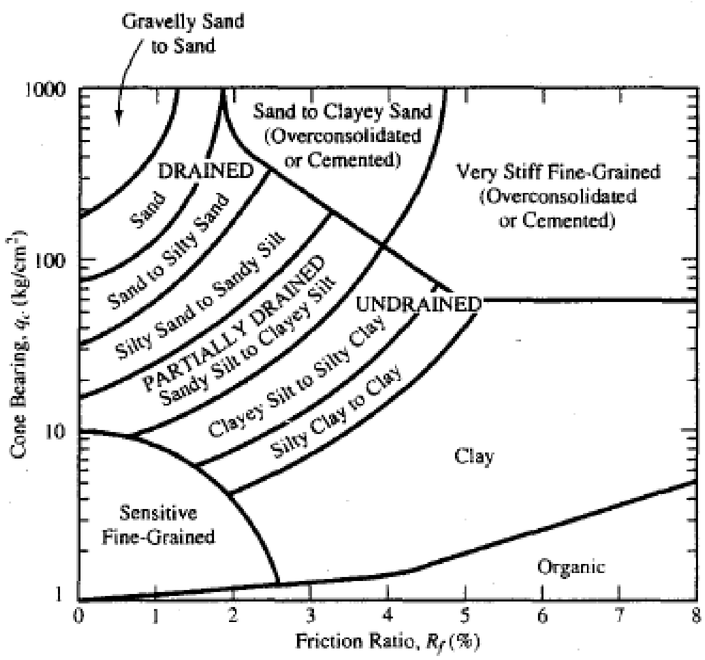

Figure 3.3: Soil classification based on penetration resistance (Adapted from Robertson and Campanella, 1983)

Correction of CPT values

Overburden Pressure: the measured cone resistance needs to be corrected and normalized for overburden pressure.

$$q_{c1} = \left(q_c / P_{a2}\right) C_Q \qquad (3.5)$$

Where q_c is the measured cone tip resistance, $C_Q = (P_a/\sigma_{v0})^n$, correction factor for overburden stress. Due to the low overburden pressure the value of C_Q will become higher and values higher than 1.7 should not be applied (Youd et al., 2001). P_a is the atmospheric pressure (in the same unit as σ_{v0} and its value is 100 kPa if σ_{v0} is also in kPa) and the value of *n* varies with soil type (0.5 for sand to 1.0 for clay, Olsen, 1997).

P_{a2} is also atmospheric pressure and its unit should be same as q_c (0.1 MPa, if q_c is in MPa).

The q_{c1} values corrected for percentage of fines is given as (Andrus et al., 2004)

$$(q_{c1})_{cs} = K_c \, q_{c1} \tag{3.6}$$

The value of K_c is calculated using the following relationships

$$K_c = 1.0 \text{ for } I_c \leq 1.64 \tag{3.7}$$

$$K_c = -0.403 I_c^4 + 5.581 I_c^2 - 21.63 I_c^2 + 33.75 I_c - 17.88 \quad I_c \geq 1.64 \tag{3.8}$$

Where I_c is the soil behaviour type index (Andrus et al., 2004).

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F)^2 \right]^{0.5} \tag{3.9}$$

Where $Q = \left[(q_c - \sigma_{v0}) / P_a \right] \left[(P_a / \sigma_{v0}')^n \right]$ (3.10)

$$F = \left[f_s / (q_c - \sigma_{v0}) \right] \times 100\% \tag{3.11}$$

Where f_s is the measured cone sleeve resistance

Using the CPT results, the soil type can be identified from the soil behaviour type index (I_c). The classification of soil based on soil behaviour type index is given in Table 3.6.

Table 3.6: Soil Behaviour type Index

Soil behaviour type index, I_C	Soil behaviour type
Ic < 1.31	Gravelly sand to dense sand
1.31 < Ic < 2.05	Sands: clean sand to silty sand
2.05 < Ic < 2.60	Sand mixtures: silty sand to sandy silt
2.60 < Ic < 2.95	Silt mixtures: clayey silt to silty clay
2.95 < Ic < 3.60	Clays: silty clay to clay
Ic > 3.60	Organic soils: peats

(ii) Dynamic cone Penetration Test

Dynamic test will be conducted by driving the cone by hammer blows. The dynamic cone resistance will be 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 will be done in a cased borehole to eliminate the skin friction.

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

Correlations between soil properties and CPT measurements

There are different correlations available in the literature relating CPT values with soil properties. Robertson and Campanella (1983) correlated CPT results and shear strength of the soils. The correlation chart is shown Figure 3.4. Similarly there are also correlations available between SPT and CPT tests as shown in the Figure 3.5.

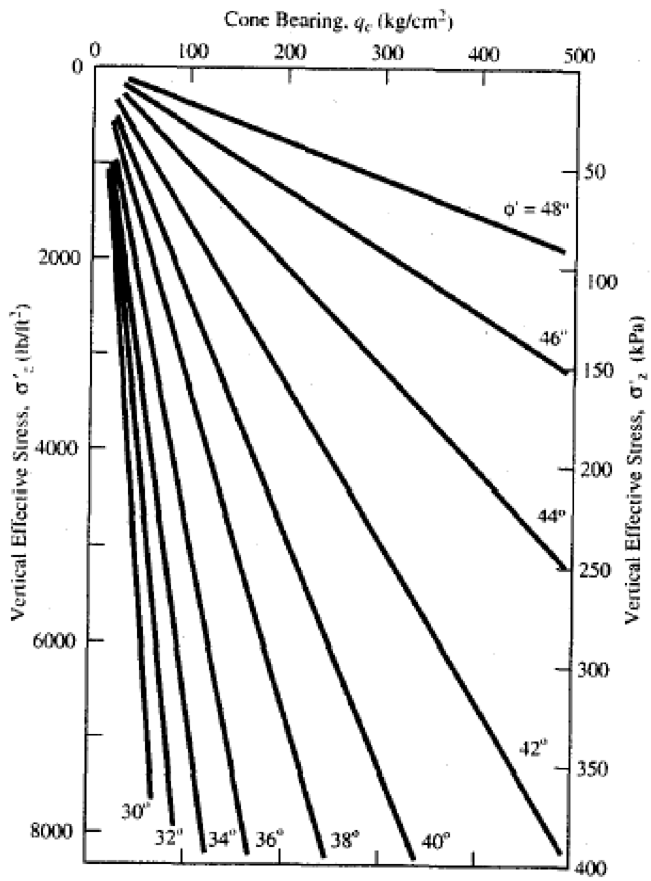

Figure 3.4: Relationship between friction angle and CPT values for uncemented, normally consolidated sands (Adapted from Robertson and Campanella, 1983)

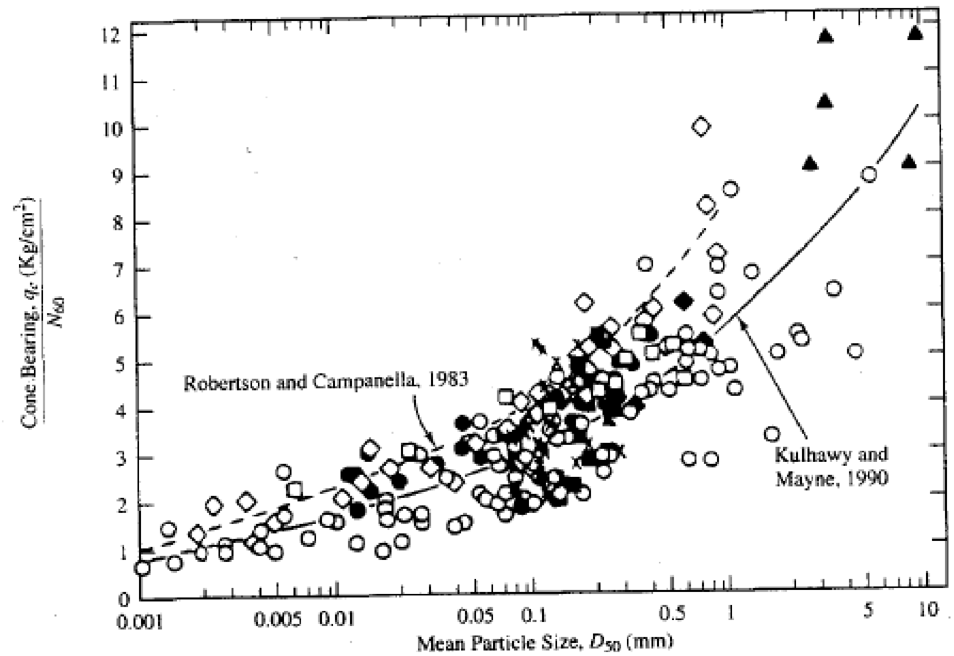

Figure 3.5: Correlation between mean grain size of the soil and q_c / N_{60} (Adapted from Kulhawy and Mayne, 1990)

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 sleeve and the other set is located one meter above the first set (Figure 3.6). It can also be fitted with a geophone or an accelerometer fitted above the friction sleeve.

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 (S waves) and compressional or primary waves (P-waves). The velocity of seismic waves in ground will give the properties like shear modulus and poisson's ratio and soil profile.

CPT is very fast and various correlations are available for obtaining different soil properties (Bowles, 1997). However CPT can be used only in soft to medium stiff ground without boulders.

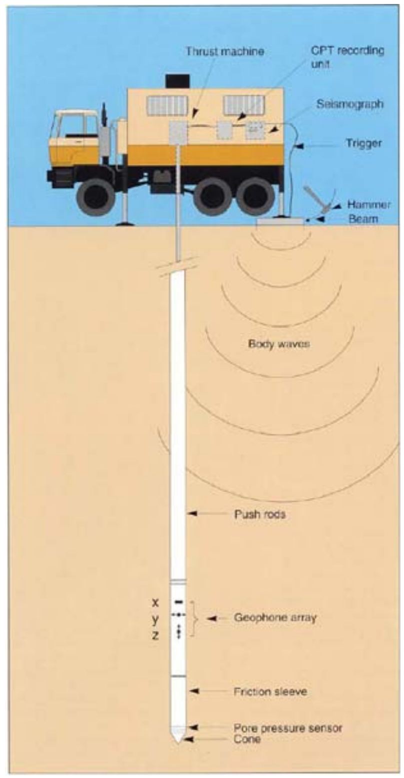

Figure 3.6: Seismic Cone Penetration test (Courtesy of Fugro Company)

Advantages and Disadvantages of CPT

This test can be performed very fast and it is relatively inexpensive. More over CPT provides a continuous soil profile and this will detect very thin layers of soil (which may go undetected in SPT). With the help of additional transducers, the measurement of pore water pressure during penetration and/or after attainment of equilibrium at depths of interest are commonly obtained. This test is particularly suited for soft soils. The main disadvantage of CPT is that it is difficult to use in very dense or stiff soil. In these types of soils, larger pushing force will be required and sometimes cones with diameter 50 mm is being used.

Use of CPT Data

The data obtained from CPT test is mainly used for assessing the soil properties, soil profiling and liquefaction potential evaluation. The cone resistance will be high in sands and low in clays, and the friction ratio will be low in sands and high in clays. The CPT values may not provide accurate predictions of soil type based on physical characteristics, such as, grain size distribution but will act as a pointer for obtaining the mechanical characteristics (strength and stiffness) of the soil.

Geophysical Tests

General geotechnical investigations involve the drilling of holes in the ground, sampling at discrete points, and in situ or laboratory testing. This methodology suits for exploration of smaller volume of soil and rock. However, for seismic microzonation, one needs to carry explorations on larger volumes. Geophysical methods overcome this drawback and some of the other problems inherent in conventional geotechnical investigation techniques. There are many geophysical methods available today. Most of the methods can provide the profiles of continuous sections. Some of the techniques can also provide stiffness properties of the ground, which are useful for seismic microzonation. Geophysical techniques also help in locating cavities, backfilled mine shafts and subsurface geological features such as faults and discontinuities.

In seismic microzonation, it is required to obtain detailed subsurface profile over the region of interest. It is difficult to carry conventional geotechnical site explorations over such a large region. In addition, carrying geotechnical site explorations over a large area is very expensive. Geophysical methods are only alternative to avoid these difficulties. These methods provide lateral variability of the near-surface materials beneath a site.

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 distance. Using the SASW methods large area can be covered and the soil profile can be obtained.

Spectral Analysis of Surface Wave (SASW) method (Stokoe et al., 1989; Foti, 2000) and Multichannel Analysis of Surface Wave (MASW) method (Park et al., 1999; Xia et al., 1999) are developed to estimate shear wave velocity profile from surface wave energy. Surface wave methods is non-invasive field tests that are executed from the ground surface without drilling any boreholes. Surface wave methods are increasingly used in civil engineering applications to evaluate soil shear modulus with depth (O'Neill et al., 2003). This approach has advantages over invasive subsurface measurements (Xia et al., 2002), because it can be easily implemented along linear sections to obtain a two dimensional shear-wave velocity profile of shallow layers (Lin et al., 2004; Hayashi and Suzuki, 2004). It can be used as a tool for imaging subsurface heterogeneity (Shtivelman, 2002). The number of surface-wave profiling applications is growing, but there are questions about experimental and theoretical limitations (O'Neill, 2004). They include seismic refraction, seismic reflection and surface wave methods. Seismic refraction and reflection methods are based on the analysis of body wave propagation and by considering either P or S waves. P wave refraction is used to locate underlying bedrock formations and S wave refraction can be used to obtain the small strain stiffness profiles. Shear wave velocity structure is an important parameter in geotechnical earthquake engineering and its use is becoming very common in recent years (Foti and Butcher, 2004; Socco and Strobbia, 2004; Xia et al., 2004). Even though shear wave velocity profile can be obtained from both SASW and MASW, MASW is preferred over SASW as it is very tedious to conduct SASW tests which depends upon only two geophones for explorations.

Multichannel Analysis of Surface Wave (MASW)

Shear wave velocity (Vs) 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 measurement of shear wave velocity by using a great variety of testing configurations, processing techniques, and inversion algorithms. The most widely-used technique is MASW (Multichannel Analysis of Surface Waves). The MASW method was first introduced in Geophysics by Park et al. (1999). It is a very conventional mode of survey using an active seismic source (e.g., a sledge hammer) and a linear receiver array, collecting data in a roll-along mode. The MASW has been found to be an efficient method for unraveling the shallow subsurface properties. In particular, the 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 is a geophysical method, which generates a shear-wave velocity (*Vs*) profile (i.e., *Vs* versus depth) by analyzing Raleigh surface waves on a multichannel record. MASW identifies each type of seismic waves on a multichannel record based on the normal pattern recognition technique that has been used in oil exploration for several decades. The identification leads to an optimum field configuration that assures the highest signal-to-noise ratio (S/N). Effectiveness in signal analysis is then further enhanced by in the data processing step (Ivanov et. al, 2005). MASW is also used to generate a 2-D shear wave velocity profile.

MASW system consists of number of geophones (usually more than twelve) and usually they are arranged with 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 *Vs* data. This is being done in three steps i) preparation of a Multichannel record (some times called a shot gather or a field file), ii) dispersion-curve analysis, and iii) inversion. The term "Multichannel record" indicates a seismic data set acquired by using a recording instrument with more than one channel using geode seismograph. MASW has been effectively used with highest signal-to-noise ratio (S/N) of surface waves.

The generation of a dispersion curve is a critical step in MASW method. A dispersion curve (Figure 3.7) 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 4Hz and highest frequency of 75Hz has been considered. Each dispersion curve obtained for corresponding locations has a very signal to noise ratio of 80 and above.

Dispersion properties of all types of waves (both body and surface waves) are imaged through a wave field-transformation method that directly converts the multichannel record into an image where a dispersion pattern is recognized in the transformed energy distribution, as illustrated at left. Then, the necessary dispersion property (like that of the fundamental mode) is extracted from a specific pattern. All other reflected/scattered waves and ambient noise are automatically removed during the transformation.

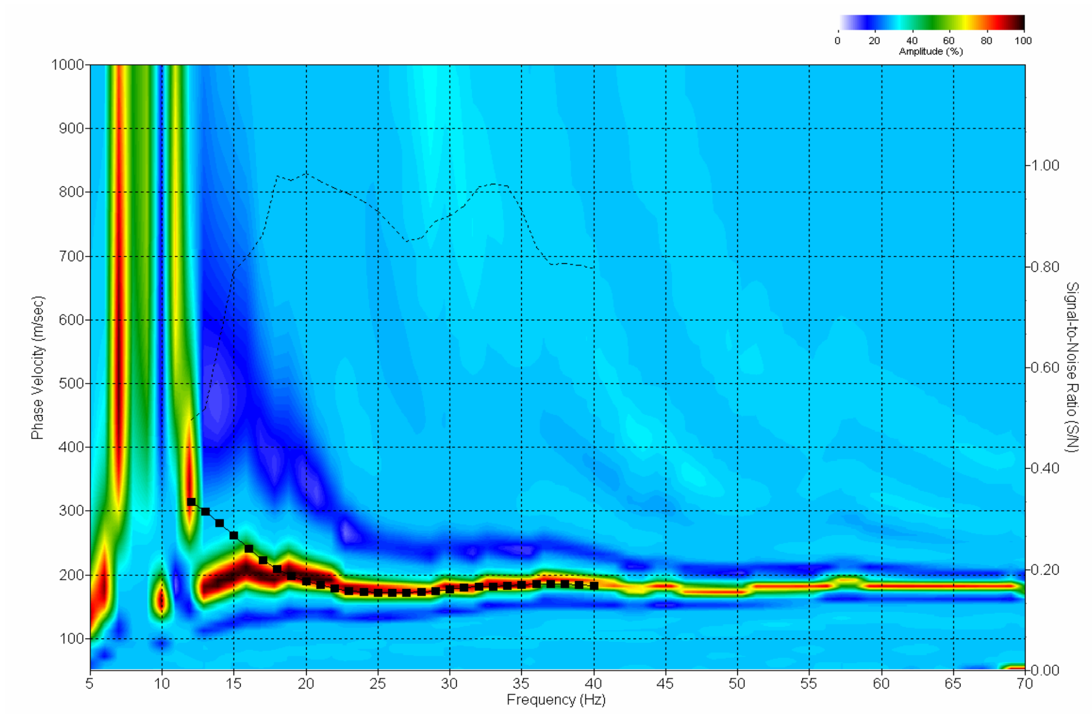

Figure 3.7: Dispersion Analysis--Multichannel Approach (2-D Wave field Transformation)

MASW tests are very fast and economical. In the SASW method, the dispersion curve is obtained by using a two-receiver test configuration. But in MASW multi station recording permits a single survey of a broad depth range and high levels of redundancy with a single field configuration. MASW method overcomes the drawbacks associated with SASW method. Wills (1998) and Boore and Brown (1998) have given the reliability of shear wave velocities measured using different techniques. Hunter et al. (2002) compared the shear wave velocity measured through borehole method and MASW technique for several sites in the Fraser river delta and found that the difference between the results was only 9 %. Under normal conditions, MASW can provide the profile upto 30 to 80 m. However, with special equipments it can go up to 250 – 300 m, as depth of profile depth mainly depends upon the source energy and geophone spacing. These tests are very useful when the rock depth is more than 30 m. However the interpretation of results require experience and special software. More over the soil profile obtained need to be cross checked with the profile obtained from borehole data at selected points.

Overburden Pressure Correction for Vs

The V_S value corrected for overburden pressure, V_{s1}, is calculated as follows:

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{v0}} \right)^{0.25} \quad (3.12)$$

Where, P_a - atmospheric pressure (approximated as 100 kPa), σ'_{v0} - initial effective overburden stress in kPa.

Advantages and Disadvantages of V_S Measurements

The V_S measurements are possible in very stiff and gravelly soils. The V_S values can be obtained in sites where borings are not permitted. The shear wave velocity is directly related to the mechanical property of soil and it is related to small strain shear modulus G_{max}. No soil sample is obtained in the test and hence visual inspection of soils may not be possible. Very thin layer of loose soil strata may go unnoticed in the test. Various models are assumed for interpretation of results and the results can be influenced by ground water conditions, presence of clay. The Vs measurement is a low strain test and the pore pressure buildup and liquefaction initiation are high strain behaviors. Hence the correlations available between Vs values and liquefaction susceptibility are relatively less.

The advantages and disadvantages of various insitu tests explained above are listed in Table 3.7 (Youd et al., 2001).

Table 3.7: Comparison of major insitu tests

Features	SPT	CPT	*Vs*
Past measurements at liquefaction sites	Abundant	Abundant	Limited
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain
Quality control and repeatability	Poor to good	Very good	Good
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All
Soil sample retrieved	Yes	No	No
Test measures index or engineering property	Index	Index	Engineering

3.4 Selection of Appropriate Test

The selection of appropriate insitu test method for site characterization will depend of geological and economical conditions. The relative merits, demits and suitability of each of these techniques has been discussed in detail in the previous sections. Based on this the appropriate method can be selected for site characterization. The preference of methods for liquefaction potential evaluation is based on SPT, CPT or shear wave velocity (in this order). The applicability of various insitu tests for different site conditions are shown in Figure 3.8. It is always advisable to cross validate the results obtained from geophysical test with the geotechnical test data for selected points.

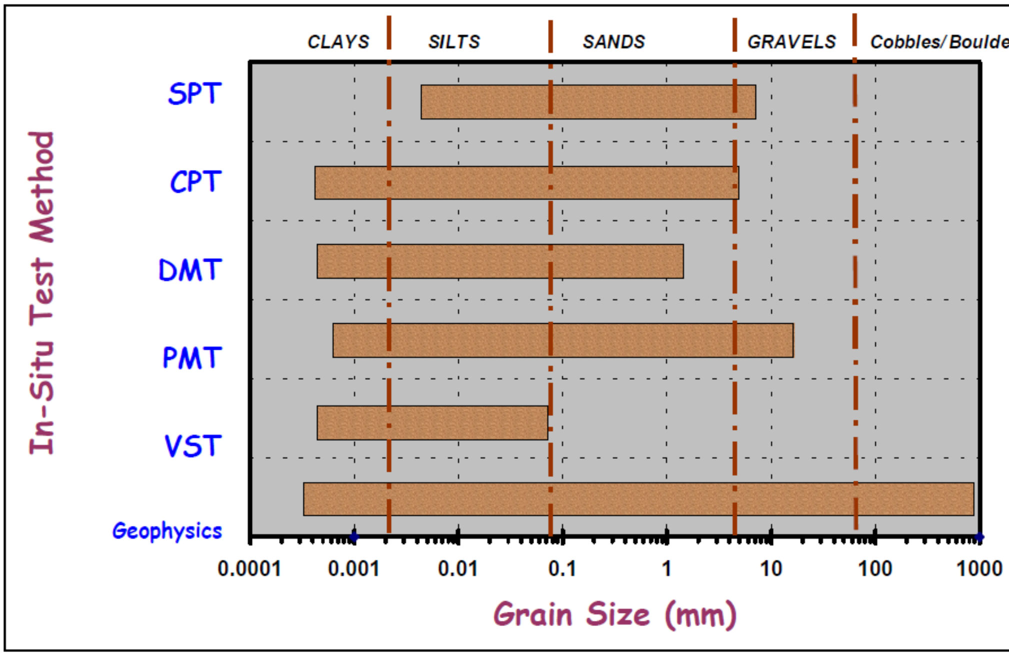

Figure 3.8: Relevance of insitu soil test to different types of soils

3.5 Routine Geotechnical Laboratory Tests

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

Index properties of Soil and Rock Samples

For Soil samples: Grain Size Analysis of the representative samples can be obtained from Sieve and Hydrometer analysis, (BIS: 2720 Part 4-1985) or 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 coarse and fine particles). The sieve analysis is done using an automatic sieve shaker where in the sample passes through progressively 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 thus 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 Atterberg 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 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 (Section 1 and 2). 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 40),

(1977) also termed as free swell or differential free swell is the increase in 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 permeability test as per BIS:2720 (Part17),1986. Compressibility characteristics can be obtained from oedometer tests as per Bureau of Indian Standards (BIS:2720 (Part15),1986). Strength characteristics can also be obtained using triaxial, direct shear and vane shear tests.

For Rock Samples: Following tests along with 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]

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 remoulded 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 ϕ). Methods of test for soils for 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 12), 1981. Triaxial Shear tests comprise UU, CU (Consolidated Undrained test with and without Pore Water Pressure Measurement) or CD (consolidated drained) tests.

3.6 Evaluation of Dynamic Properties

The response of the soil to cyclic loading strongly influences the earthquake damage. Hence the evaluation of the dynamic properties of soils is of utmost importance in geotechnical earthquake engineering. The dynamic properties can be evaluated based on field and lab tests and these tests can be broadly divided into two – High strain and Low strain tests. The selection of the testing technique should depend on the soil property which we intend to measure. Since soil properties are highly non linear, the properties which influence wave propagation, stiffness, damping, poisson's ratio etc. need to be evaluated at low strain. The high strain tests are most commonly used to measure the soil strength, which need to be evaluated at higher strain levels.

Field Tests

The field tests or the in situ tests measure the dynamic soil properties without altering the chemical, thermal or structural condition of the soil. The field test can be broadly divided into two – low strain and large strain tests.

(i) 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. Even though it is more commonly used than seismic reflection test, its major application is for delineation of major stratigraphic units.

Seismic Refraction Test: The successful application of seismic refraction and reflection for profiling depends upon how well we modeled the wave propagation in the surface layers. The propagation of seismic waves through near surface deposits is very complex. The particulate, layered and fractured nature of the ground means that waves undergo not only reflection and refraction but also diffraction, making modeling of seismic energy transmission impractical. 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.

Suspension Logging Test: This test is most commonly used in the 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 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 transmits through the soil and reach the geophones attached to the probe at 1m away from the solenoid. This techniques 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 2,000 hz for S-waves), which are much higher than the frequency range of interest of geotechnical earthquake engineering. This test is very effective at higher depths (up to 2 km)

Seismic Cross-hole Test: Crosshole testing provides useful information on dynamic soil properties required for site-specific ground response analyses, liquefaction potential studies and dynamic machine foundation design. Perhaps it is best insitu method used for obtaining the variation of low strain shear wave velocity with depth. Unlike MASW/SASW testing, it does not rely on any indirect methods to determine the wave velocities. Very few justifiable assumptions like presence of horizontal layers and applicability of Snell's laws of refraction are only inherent in the crosshole testing method. Other parameters, such as moduli and Poisson's ratios, can be easily determined from the measured shear and compressional wave velocities. Only disadvantage with this method is that it requires drilling of bore holes for its testing. And hence, it is expensive, time consuming and also very tedious compared to other geophysical seismic methods like MASW/SASW.

Cross-hole testing is typically carried out using three parallel in-line boreholes with plastic lining. Borehole should not be more than 7m, as increased spacing enhances errors in travel times due influence of refracted waves. Typically 3 to 6 m of spacing is adopted. Energy source is lowered in one of the more hole at a depth of 1m into the stratum being investigated. Receivers (three component geophones) are lowered in the other boreholes to the same depth and clamped. Seismic waves are generated at the source and their arrival times at the receivers are noted. Both the body waves P and S waves are used in the test. Based on the distance between the boreholes and the travel times of different phases of seismic waves, their velocity is calculated.

Down-hole and Up-hole Tests: Procedure for testing of down-hole seismic method is similar to the seismic cone test discussed in the previous section. The borehole drilled for cross-hole testing can be used for this test. Three orthogonally oriented geophones are lowered in to the borehole to different depths. At each depth, geophones are clammed to the bore hole and are used in recording arrival times of seismic waves. Source is placed at the ground surface. Data interpretation is similar to seismic cone test. To increase the speed of investigation, an array of geophones separated by a distance of 1 m, are used to cover measurements corresponding to different depths in one short. Test set up for uphole is just opposite to downhole. In uphole test, source is lowered and receiver is placed at the surface. Rest of the procedure is similar.

It is easy to generate the S-waves in a down hole test than in an uphole test and hence it is more preferred. Since the waves have to travel through all the layers to reach the receive, it can detect soil layers, which may go undetected in other tests and hence the results obtained from these tests are more reliable. There are difficulties in conducting the test and the interpretation of the results due to the casing and borehole fluid effects.

There are other field insitu tests that use principle of wave reflection and refraction. Seismic reflection test uses an impulse (usually rich in p-waves) at the source and measure the arrival time at receiver. It provides the wave propagation velocity and thickness of surficial layers. Even though it can provide deeper layer information, it application is limited due to difficulty of determining arrival time of the reflected waves. This test is not effective for depths greater than 30 to 60 m.

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.

MASW/SASW Tests: Even though both MASW and SASW tests provides in situ shear wave velocity, utilizing SASW to obtain detailed velocity structure of a region is very tedious as it uses only two geophones for explorations. The measured shear wave velocity from these tests can directly be used to obtain the in situ shear modulus at low strains (maximum shear modulus). Details of these tests are already discussed in the previous sections.

High Strain Tests

The most commonly used in situ soil tests are high strain test. There are correlations available between the results of high strain tests and low strain tests as described in the previous sections. Based on these correlations, one can easily calculate in-situ shear modulus (Eq. 4.6). Some of the commonly used high strain tests are given below.

i. Standard penetration test (SPT)

ii. Cone penetration test (CPT)

iii. Dilatometer test

iv. Pressuremeter test

v. Field vane shear test

Out of these tests, SPT and CPT are discussed in detail in the previous sections. A brief description of other tests are given below.

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 bore hole from the ground surface and tests are conducted at an interval of 10 to 20 cm. At each interval, the dilatometer is stopped and membrane is inflated under gas pressure. The readings of inflation of the membrane and corresponding gas pressure are recorded. There are correlations available to relate the test results with low-strain soil stiffness and liquefaction resistance of soil.

Pressuremeter Test : Pressuremeter test is conducted using a pressuremeter. It is a cylindrical device that uses flexible membrane for 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 insitu test that can measure stress-strain as well as strength behavior of insitu soil.

Field Vane Shear Test: Vane shear test is done on fully saturated clays for evaluation of undrained shear strength. This is very suitable for soft clays whose shear strength (less than 100 kPa) will be 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 the desired depths.

Laboratory Tests

They are usually performed on very small representative samples. The success of the laboratory testing depends heavily on simulating the actual field conditions (initial state of the sample and the loading conditions). Since the dynamic soil properties depend on lots of factors, the preparation of soil sample for testing need to be done with utmost care. Some of the laboratory tests used to evaluate the dynamic properties of soil are given below.

Resonant Column Test (*low strain test*): This is most common low strain test which is being used to evaluate the 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 electromagnetic loading system. The fundamental frequency of the sample can be evaluated and this in turn will give the value of the 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 (*low strain test*): In this test the shear wave velocity of laboratory specimen can be measured using a piezoelectric bender element. A transmitter and receiver elements (piezoelectric) are placed at each end of the sample. There will be a change 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 transmitter and 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 (*high strain test*)**:** 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 area under the loop provides the damping ratio. Typically, five to ten specimens are tested under to different levels of cyclic shear strain amplitudes chosen in the range of 10^{-4} to 10^{-1} for determination of dynamic properties. Dynamic deformation characteristics are influenced by the 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 (details are given later in Section 5.3.1). 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 (*high strain 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.

3.7 Assessment of Dynamic Properties Based on Other Soil Properties

There may be some cases when the evaluation of the shear modulus (G_{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_{max} based on void ratio was proposed by Richart et al. (1970).

$$G_{\max} = 700\frac{(2.17-e)^2}{1+e}(P')^{0.5}\ (kgf/cm^2)\ (for\ round\ sand) \tag{3.13}$$

$$G_{\max} = 330\frac{(2.97-e)^2}{1+e}(P')^{0.5}\ (kgf/cm^2)\ (for\ angular\ sand) \tag{3.14}$$

Where e – void ratio and P' – effective mean principle stress

The shear modulus varies with effective stress also. The relation showing the variation of G_{max} with effective confining stress was proposed by Chung et al. (1984) (Figure 3.9). The effect of soil types on damping ratio was studied by Kokusho (1987). The curves developed for different types of soils are presented in Figure 3.10.

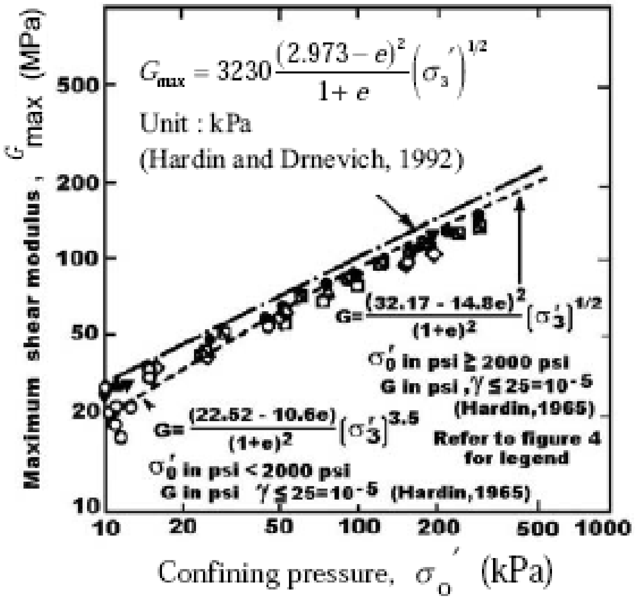

Figure 3.9: Variation of G_{max} with confining pressure

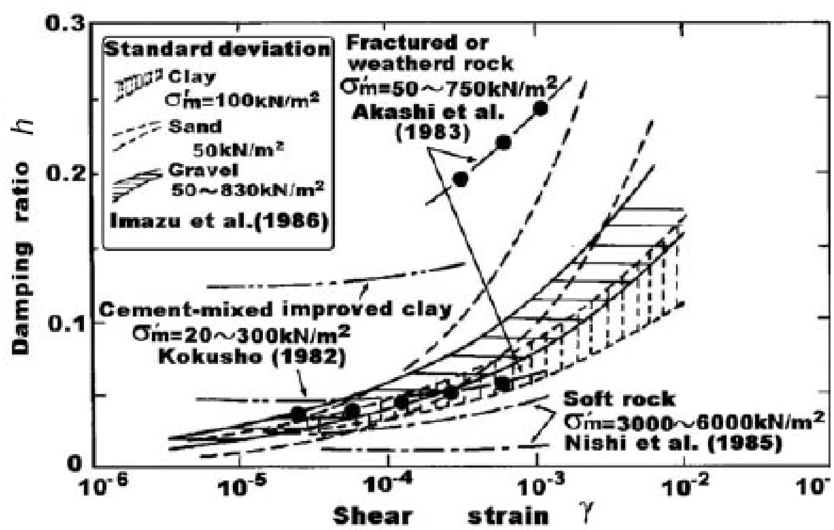

Figure 3.10: Damping ratio of different soils (Kokusho, 1987).

3.8 Site Characterization Methods

One of the first and most important steps in earthquake hazard evaluation is site characterization. This involves acquisition, synthesis and interpretation of qualitative and quantitative information about the site of interest. Seismic site characterization can be carried out based on following methods.

Based on Geology

The first step in a seismic microzonation is the geological investigation of the region to determine the geological settings. Geological maps at different scales are available and can provide important information on regional and local geology. Aerial photographs from governmental agencies can reveal important aspects of site geomorphology. Field reconnaissance performed by geologists provides topographic features and past occurrences of ground failures. The purpose is to identify the tectonic formations that may produce future earthquakes (Barka and Kadinsky-Cade, 1988; Barka, 1992; Yeats et al., 1997; Jackson, 2001). One of the reasons for the excessive damage during some of the major earthquakes (Spitak 1988; Kobe 1995) was because of the improper estimation of the earthquake characteristics of the region. In evaluating the seismicity of an area two factors need to be considered. The first one is the tectonic and geologic formations that can produce earthquakes in the region and the second factor is the seismic history. To understand the possible mechanisms that can generate earthquakes, detailed geological and seismological studies are necessary (Bolt, 1999). Surface mapping of faults, outcrops, joints, bedding planes, slides and other features are essential during a geological site characterization.

Based on Geotechnical Data

Site response and ground failure are strongly influenced by the properties of soil. Site exploration usually begins with a thorough review of the available information about the site and its surroundings. Geotechnical reports for the sites may be available from various governmental or non governmental agencies. Geotechnical site characterization requires a full 3D representation of stratigraphy, estimates of geotechnical parameters and hydrogeological conditions and properties. The traditional methods like drilling and undisturbed sampling can provide adequate stratigraphic details and estimates of geotechnical parameters. But they cannot provide useful estimates of hydrogeological conditions which are very important in estimating the seismic hazards like liquefaction, landslides etc.

Site response is primarily influenced by the properties that influence wave propagation, particularly stiffness and damping. Ground failure is influenced by the shear strength of the soil. Soils are highly nonlinear even at very low strains. This nonlinearity causes soil stiffness to decrease and damping to increase with increasing strain amplitude. Both site response and ground failure are parts of the same continuous spectrum of nonlinear soil behavior.

Many researchers have proved that geotechnical site conditions play an important role in damage distribution as well as in the recorded strong motion records (Ishihara, 1997; Aki, 1998; Tertulliani, 2000; Hartzell et al., 2001, Ozel et al., 2002). The determination of geotechnical site conditions requires identification of the soil stratification and properties of soil layers based on various in-situ tests and laboratory tests on soil and rock samples (Kokusho, 1987; Atkinson and Sallfors, 1991; Pitilakis et al., 1992).

In evaluating the behavior of soils under cyclic loads, it is preferable to consider stress-strain and shear strength properties separately. Dynamic shear modulus, damping ratio and their variation with shear strain may be regarded as the dynamic stress-strain properties of soils and used for site response analyses (Ishihara, 1982; Dobry and Vucetic, 1987). Cyclic stress amplitudes and number of cycles leading to failure or excessive deformations may be defined as dynamic shear strength properties and may be used to determine areas that would experience such shear strength degradation during earthquakes.

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

The behavior of soils subjected to cyclic loading has been studied by various researchers (Seed and Idriss, 1970; Castro and Christian, 1976; Idriss et al., 1978; Sangrey and France, 1980; Yasuhara et al., 1982; Ishihara et al., 1983; Fujiwara et al., 1985; Hatanaka et al., 1988; Ansal and Erken, 1989; Vucetic and Dobry, 1991; Matasovic and Vucetic, 1992; Talesnick and Frydman, 1992; Viggiani and Atkinson, 1995; Lanzo et al., 1997; Vucetic et al., 1998; Vrettos and Savidis, 1999; Lanzo and Vucetic, 1999; Pecker, 2007). Numerous studies were carried out to evaluate stress-strain behavior of soils utilizing both laboratory and in-situ tests.

The stress strain parameters of soil can be obtained using formulas, which are experimentally determined, or based on design curves (Vucetic and Dobry, 1991; Kagawa, 1992; Ishibashi and Zhang, 1993; Kallioglou et al., 1999). For evaluating the maximum shear modulus, Gmax, several expressions has been proposed and one of them proposed by Hardin and Drnevich (1972) is given below.

$$\left(\frac{G_{\max}}{p_a}\right) = 3.21\frac{(2.97-e)^2}{1+e}(OCR)^n\left(\frac{\sigma'_c}{p_a}\right)^n \qquad (3.15)$$

Where, e is the void ratio (< 2.0), n is the exponent ranging between 0 and 0.5 depending on the PI; n is equal to 0.5 for most sandy and clayey soils, σ_c' is the mean effective stress and P_a is the reference atmospheric pressure.

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

The in-situ tests generally conducted to identify the soil stratification and engineering properties of the soil layers are penetration tests. Two methods that have been widely used are the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). SPT is generally used to investigate cohesionless or relatively stiff soil deposits, whereas CPT is used to identify soil properties in soft soil deposits (Lunne et al., 1997). The variability of the Standard Penetration Test equipment and procedures used has significant effects on the obtained blow counts (Seed et al., 1985; Skempton, 1986). The energy delivered to the split-spoon sampler, is strongly influenced by many factors such as hammer type, borehole diameter, rod length, rod diameter, tightness of the rod joints, verticality of the rod string, and type of sampler etc. Therefore it is very important to have sufficient information to estimate the energy ratio correction for SPT blow counts before using these results for assessing the properties of soil layers.

Empirical relations have been proposed to correlate the penetration test results between CPT and SPT (Robertson et al., 1983) as well as with the shear-wave velocities (Ohta 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 3.8.

Table 3.8: Different Relationships to Estimate Shear Wave Velocity from SPT Value

Author	Data	Soil type	V_S (m/sec)
Kanai et al. (1966)	Not known	All soils	$V_S = 19\ N^{0.6}$
Shibata (1970)	Not known	Sandy soils	$V_S = 32\ N^{0.5}$
Imai and Yoshimura (1970)	Not known	All soils	$V_S = 76\ N^{0.33}$
Ohba and Toriumi(1970)	Not known	Alluvial soils	$V_S = 84\ N^{0.31}$
Ohta et al. (1972)	Not known	Sandy soils	$V_S = 87\ N^{0.36}$
Ohsaki and Iwasaki (1973)	Not known	All soils Cohesionless soils	$V_S = 82\ N^{0.39}$ $V_S = 59\ N^{0.47}$
Imai and Yoshimura (1975)	Not known	All soils	$V_S = 92\ N^{0.329}$
Imai et al. (1975)	Not known	All soils	$V_S = 90\ N^{0.341}$
Imai (1977)	Not known	All soils	$V_S = a\ N^b$ a = 102 b = 0.29(H.clay) a = 81 b = 0.33(H.sand) a = 114 b = 0.29(P.clay) a = 97 b = 0.32(P.sand)
Ohta and Goto (1978)	Not known	All soils	$V_S = 69\ N^{0.17}\ D^{0.2}\ E\ F$ E = 1 (H) ; = 1.3(P) F = 1 (clay); 1.09(f.sand); 1.07(m.sand); 1.14(c.sand); 1.15(g.sand); 1.45(gravel)
Seed and Idriss (1981)	Not known	All soils	$V_S = 61\ N^{0.5}$
Imai and Tonouchi (1982)	1654 sets of data (Japan)	All soils	$V_S = 96.9\ N^{0.314}$
Seed et al. (1983)	Unknown	Sands	$V_S = 56.4\ N^{0.5}$
Sykora and Stokoe (1983)	229 sets of crosshole data	Granular soils	$V_S = 106.7\ N^{0.27}$
Fumal and Tinsley (1985)	Not known	Sands and gravelly sand Soils	$V_S = 152 + 5.1\ N^{0.27}$
Sykora and Koester (1988)	186 points (7m, 7 sites) 186 points (7m, 7 sites)	Holocene gravels Pleistocene gravels	$V_S = 63\ N^{0.43}$ $V_S = 132\ N^{0.32}$
Okamoto et al. (1989)	Not known	All	$V_S = 125\ N^{0.3}$ (P.sand)
Lee (1990)	Not known	a) Clays b) Sands c) Silts	$V_S = 114\ N^{0.31}$ $V_S = 57\ N^{0.49}$ $V_S = 106\ N^{0.32}$
Imai and Yoshimura (1990)	Not known	All soils	$V_S = 76\ N^{0.33}$
Yokota et al. (1991)	Not known	All soils	$V_S = 121\ N^{0.27}$
Jafari et al. (1997)	Not known	All soils	$V_S = 22\ N^{0.85}$
LIQUFAC	Unknown	All	$V_S = 243.8\ \sigma_e^{\ 0.4}$
LIQUFAC	Unknown	All	$V_S = 152.4\sigma_e^{\ 0.3}$

Author	Data	Soil type	V_S (m/sec)
Pitilakis et al. (1999)		a) Sand b) Clay	$Vs = 145 (N_{60})^{0.178}$ $Vs = 132 (N_{60})^{0.178}$
Hanumantha Rao (2006)	SPT and MASW	a) Sand b) Silty sand / sandy silt c) All Soils	$Vs = 79 (N)^{0.434}$ $Vs = 86 (N)^{0.42}$ $Vs = 82.6 (N)^{0.43}$
Anbazhagan and Sitharam (2010)	SPT and MASW data	a) Sands b) Clays	$Vs = 57 ((N_1)_{60,cs})^{0.44}$ $Vs = 80 (N)^{0.33}$
Uma Maheswari (2010)	SPT and MASW data	a) all Soils b) Sands c) Clays	$Vs = 95.64 N^{0.301}$ $Vs = 100.53 N^{0.265}$ $Vs = 89.31 N^{0.358}$

Where, N - SPT value; σ_e - effective vertical stress of the soils; VS -shear wave velocity; D - depth (m); H- Holocene; P- Pleistocene; f - fine; m - medium; c- coarse; g – gravel.

Depending upon the type of soil the respective relation can be adopted. Preference can be given to the relations developed for a similar region. If the shear wave velocity data is available for some points of the study area, then it is advisable to cross check the values obtained from the selected relations with the actual values.

In the absence of shear wave velocity values, the same can be obtained from CPT data also. The correlation between CPT values and shear wave velocity values for uncemented soils is shown in Figure 3.11 (Robertson and Cabal, 2010).

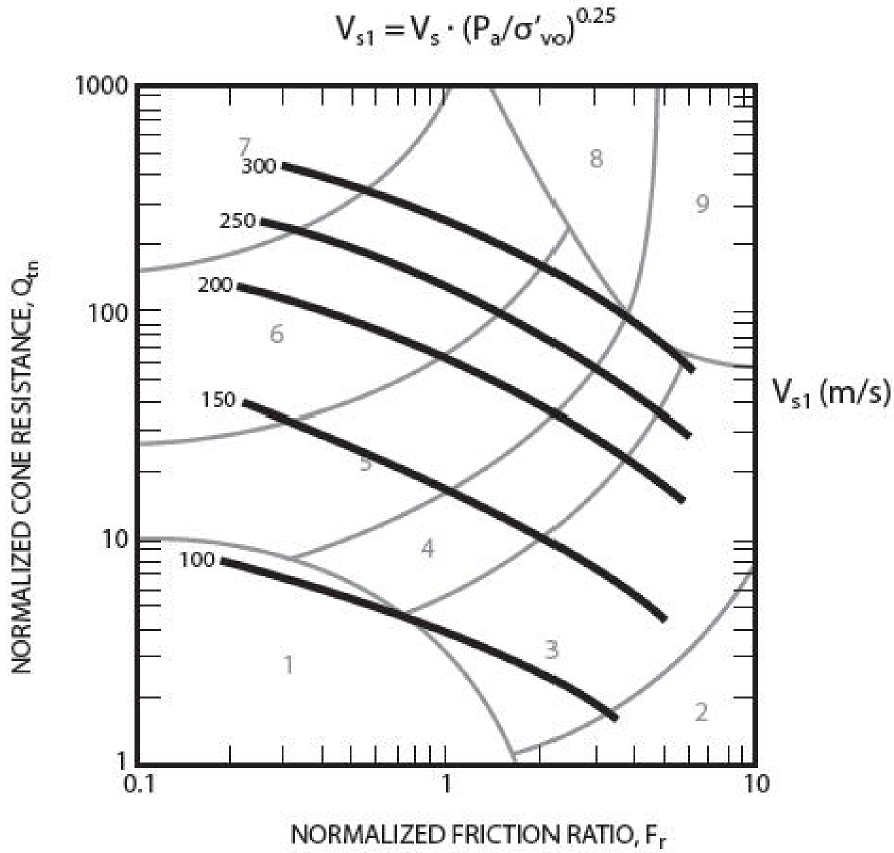

Figure 3.11: Correlation between CPT values and Vs

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

Based on Geophysical Investigations

Seismic tests are classified into borehole (invasive) and surface (non invasive) methods. They are based on the propagation of body waves and surface waves, which are associated to very small strain (< 0.001%). The shear wave velocity (Vs) is a soil property used to determine the shear modulus (G) of the soil as below:

$$G=\rho V_s^2 \qquad (3.16)$$

Seismic tests are also used to determine the material damping ratio by measuring the spatial attenuation of body or surface waves.

$Do = \alpha V / 2\pi f$ (Do < 10%) (3.17)

Where, Do - small strain damping ratio, α - attenuation coefficient; V - velocity respectively of P,S or R waves; f - frequency.

The most widely used borehole methods are P-S logging, down-hole logging and cross-hole logging (Mancuso, 1994; Raptakis et al., 1994). In down-hole logging, the travel time used by vertically propagating shear-waves from a source on the surface to a subsurface receiver along a borehole is measured. Cross-hole logging is based on subsurface measurements in which the travel time is measured by horizontally propagating shear-waves from inside a borehole to neighboring boreholes (Hall and Bodare, 2000). Cross-hole logging also has the advantage of identifying the properties of the soil deposit between the boreholes.

Recently most of codes like Eurocode-8 (2003), NEHRP (BSSC, 2003), International Building Code (IBC, 2009) etc. specify the site classification based on the average shear wave velocity values in the top 30 m (V_s^{30}). The amplification of shear waves mainly depends on the density and the shear wave velocity of the overlying soil layer. Since the variation in density of soil is comparatively less, the amplification depends heavily on the shear wave velocity near the earth surface. There are two methods to denote the near surface shear wave velocity (V_s) – depth corresponding to one quarter wave length of the period of interest and the average shear wave velocity in the top 30 m. The main disadvantage with quarter wavelength V_s is that the depths associated with this will be very deep. Hence the classification based on V_s^{30} is being used more commonly now a days. It is calculated using the equation

$$V_s^{30} = \frac{30}{\sum_{i=1}^{N}\left(\frac{d_i}{v_i}\right)} \quad (3.18)$$

Where d_i - thickness of the i^{th} soil layer in metres; v_i - shear wave velocity for the i^{th} layer in m/s and N – no. of layers in the top 30 m soil strata which will be considered in evaluating V_s^{30} values.

A sample table showing the evaluation of V_S^{30} value is shown in Table 3.9

Table 3.9: Average shear wave velocity calculation (From the data obtained from Bangalore, Anbazhagan, 2007)

Depth (m)	***Vs* (m/s)**	**Soil thickness [d_i] (m)**	**Average *Vs* Soil-7.2m**	**Average *Vs*-5m**	**Average *Vs*-10m**	**Average *Vs*-15m**	**Average *Vs*-20m**	**Average *Vs*-25m**	**Average *Vs*-30m**
-1.22	316	1.2	259	265	286	310	338	362	306
-2.74	250	1.5							
-4.64	255	1.9							
-7.02	241	2.4							
-10.00	388	3.0							
-13.71	355	3.7							
-18.36	435	4.6							

Depth (m)	*Vs* (m/s)	Soil thickness [d_i] (m)	Average *Vs* Soil-7.2m	Average *Vs*-5m	Average *Vs*-10m	Average *Vs*-15m	Average *Vs*-20m	Average *Vs*-25m	Average *Vs*-30m
-24.17	527	5.8							
-31.43	424	7.3							
-39.29	687	7.9							

Eurocode-8 and NEHRP

A site classification scheme based on V_s^{30} values was proposed by Burckhardt (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 3.10. 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 3.11. Even though both the schemes use similar methods to identify the site classes, the range of V_s^{30} values specified for each site class is different in both the methods.

Table 3.10: Site classification as per NEHRP scheme. (BSSC, 2003)

NEHRP Site Class	Description	V_s^{30}
A	Hard rock	> 1500 m/s
B	Firm and hard rock	760 – 1500 m/s
C	Dense soil, soft rock	360 – 760 m/s
D	Stiff soil	180 – 360 m/s
E	Soft clays	< 180 m/s
F	Special sandy soils, eg. liquefiable soils, sensitive clays, organic soils, soft clays > 36 m thick	

Table 3.11 : Site classification adopted by Eurocode – 8 (2003)

Ground Type	Description of stratigraphic profile	Parameters		
		V_s^{30} (m/s)	SPT	C_U (KPa)
A	Rock or other rock-like geological formation, including utmost 5 m of weaker material at the surface.	> 800		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive	< 180	< 15	< 70

Ground Type	Description of stratigraphic profile	Parameters		
		V_s^{30} (m/s)	SPT	C_U (KPa)
	layers), or of predominantly soft-to-firm cohesive soil.			
E	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} > 800 m/s.			
S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)		10 - 20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

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 3.12. The main advantage of this system is that it correlates the V_s^{30} values with the geotechnical and surface geological features.

Table 3.12: Classification based on geotechnical features (Rodriguez-Marek et al., 2001)

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

Extrapolation Techniques to Obtain V_s^{30}

In many locations the rock depth will be shallow (less than 30 m) and hence the evaluation of Vs30 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 has 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.

$$V_S^{30} = \frac{30}{tt(d) + (30 - d)/V_{eff}} \tag{3.19}$$

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.

Even though this method is simple, it is found to under estimate 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, the V_s^{30} value can be estimated as:

$$\log \mathrm{V_s^{30}} = a + b \log \overline{V_S}(d) \tag{3.20}$$

Where $\overline{V_S}(d)$ 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)

$$P\left(\xi > V_{eff} / V_s(d)\right) = a\left(V_{eff} / V_s(d)\right)^b \tag{3.21}$$

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

Note: 1. Even though the extrapolation techniques by Boore (2004) provides means to obtain V_s^{30} for those sites where soil layer thickness is less than 30m, use of these techniques require engineering judgment. Like for example, if the soil stratum thickness is available only up to 27m, extrapolation using Boore techniques provides conservative values of V_s^{30} rather than considering bedrock velocities. However, use of these techniques for very shallow soil stratums is questionable. For example, if soil stratum thickness is only 5m (i.e. bedrock depth is shallow at 5m), applying this technique would provide over conservative site classification, which is unacceptable.

2. Use of V_s^{30} for classification of sites where soil stratum thickness is substantially more than 30 m is also questionable, particularly with presence of soft soils below 30m. More over, recent studies by Krishna Kumar and Boominathan (2010) brought of the importance of these deep soil effects even in case of stiff soils. These deep soil effects are further discussed in detail in Section 4.5.

3.9 Conclusions

This chapter discusses in details about different tests required for site characterization. The site characterization methodologies based on Geology, geotechnical and geophysical methods are discussed in detail. Even though the average shear wave velocity values are not available, a good estimate of these values can be obtained from the local geology and methods for obtaining these are discussed in this chapter. The main recommendations of the chapter are given below.

- The site exploration by borings give the best details of soil layering but the main disadvantage is that it is more time consuming and expensive.
- Standard Penetration Test, even though it is having some disadvantages it is the most widely used technique.
- Cone Penetration Test is ideal for soft soils and it is less expensive. The piezo cone penetration test with pore pressure measurements will be helpful in identifying undrained and drained layers.
- The geophysical techniques are faster and convenient in getting the soil profile. They will be extremely useful to get the soil profile in the case of deep soil deposits, where depth of bed rock is more than 30 m.
- The results obtained from the cross-hole test are more accurate than MASW but the cost is high. Hence it is recommended in critical areas. The up-hole or down hole test are less expensive, when compared to cross hole test, but the results are less accurate.
- It is recommended to crosscheck the soil profile obtained from the geophysical methods with the actual borehole data at some of the locations. The accuracy of the geophysical methods depends on the experience and expertise of the person carrying out the survey and interprets the results.
- In the absence of shear wave velocity values, these values can be obtained using the correlations between SPT and CPT. However while using these correlations, it is advisable to use the relation developed for similar regions.
- The liquefaction susceptibility need to be assessed based on the insitu tests like SPT and CPT.

4 Local Site Effects and Ground Response Analysis

4.1 Introduction

It has been evident from the past earthquake events all over the world that the amplification of ground motion is highly dependent on the local geological, topography and geotechnical conditions. Many researchers have worked on the estimation of local site effects (Phillips and Aki, 1986; Wills and Siliva, 1998; Semblat et al., 2000; Slob et al., 2002; Stewart et al., 2003; Topal et al., 2003; Pitilakis, 2004). It is observed that large concentration of damage in specific areas during an earthquake is due to site dependent factors related to surface geological conditions and local soil altering seismic motion. After the 1923 Kanto earthquake, it is very clear that major seismic damage was controlled by local geology. It is well known that each soil type responds differently, when subjected to the ground motions, imposed due to earthquake loading.

Generally, the soil layers over the firm bedrock may attenuate or amplify the bed rock earthquake motion depending upon geotechnical characteristics, their depth and arrangement of soil layers. Usually the younger softer soils amplify ground motion relative to older, more competent soils or bedrock. Local amplification of the ground is often controlled by the soft surface layer, which leads to the trapping of the seismic energy, due to the impedance contrast between the soft surface soils and the underlying bedrock. Various studies have demonstrated the ability of geological and geotechnical conditions in changing the seismic motion. It has been reported that the damaging effects associated with such soft deposits, may lead to local intensity increments as large as 2 to 3 degrees in MM scale (Aki and Irikura, 1991; Finn, 1991). Local soil conditions has significant role on amplification of seismic waves, and being experienced in the past earthquakes (Street et al., 2001; Slob et al., 2002; and Ansal et al., 2004). The nature and depth of the soil layers have a great influence on the intensity at ground level.

4.2 Physics of Site Amplification

In most of the cases, the density of the soil will be less near the surface of the earth and it will increase with the depth. In the case of an earthquake, the seismic waves will be generated at a very large depth and when they travel towards the ground surface, the waves will be moving towards soils with lower density. The velocity of wave (V_s) in each soil layer will depend on the shear modulus and the density of the soil. Since the stiffness of soils decreases towards the ground surface, the wave propagation velocity will also decrease. When the seismic waves moves from one soil layer to the other, with different stiffness, its energy has to be conserved. The wave energy per unit wave length can be obtained from the following equation (Towhata, 2008)

Energy per wavelength = $\omega\pi\rho V_s E^2$

Where E is the amplitude of motion, ω is the frequency of the wave and ρ is the density of soil. When the seismic waves move from one soil layer to the other, having different V_s values, the value of E will change to conserve the energy. As the waves approach the earth surface, the value of V_s will decrease and there will be a corresponding increase in the value of E, the amplitude of motion. This will cause in increasing the amplitude of vibration at the ground surface and in most of the case this will be higher for soft soils. This phenomenon is similar to the amplification of wave height of tsunami waves when it approaches the shore. In the case of tsunami waves approaching the shore, the wave velocity will decrease and the amplitude of wave, wave height, will increase to conserve the energy. For a more detailed description of this phenomenon readers can refer to Towhata (2008).

4.3 Effect of Local Site Conditions on Ground Motion

Many historical evidences of the influence of local geological characteristics on the intensity of ground shaking and damage caused due to an earthquake are found. Data dating back to about 200 years supports this claim. However, it was not until the early 1970s that the local site conditions were included in building codes and provisions. Evidence for the existence of local site effects is quite overwhelming. In addition to theoretical evidence, amplification functions developed from measurements of surface and bedrock motions at the same location, and comparisons of surface motion characteristics from nearby sites with different subsurface conditions, all confirm the effects of local site conditions of earthquake ground motions.

The effects of site amplification was evident at some locations during the Mexico (1985) and Loma Prieta (1989) Earthquakes. The time history of acceleration and response spectra obtained from strong motion records (UNAM – rock site, SCT – soft soil site) of Mexico earthquakes are shown in Figure 4.1. This clearly shows the amplification due to soft overlying soil. In the case of Loma Prieta also, the site amplification effects were significantly high in the central San Francisco Bay area (Figure 4.2). At this area, the sites are underlain by thick deposits of soft clayey soils.

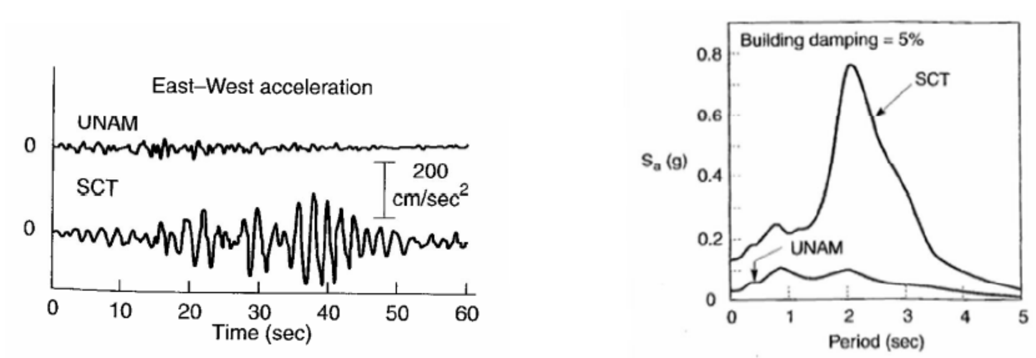

Figure 4.1: Site amplification during Mexico earthquake (SCT – soft soil site and UNAM – rock site) (Stone et al., 1987)

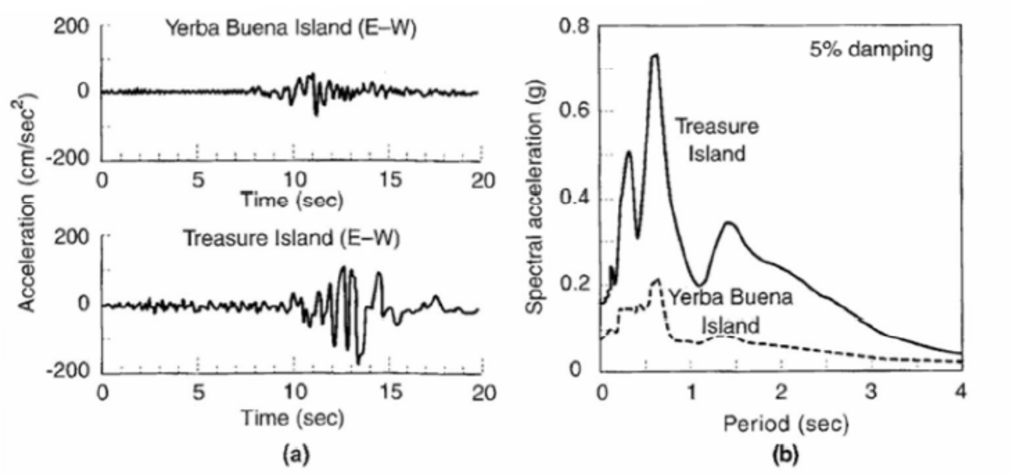

Figure 4.2: Site amplification during Loma Prieta earthquake (Seed et al., 1990)

Both the above examples clearly stress the need for proper assessment of site amplification values. For some frequency, the ground motion is amplified by four times, when compared to the rock level. The local site conditions can influence the amplitude, frequency content and the duration of motion. The major factors which will influence the site effects are discussed in the following sections.

Effect of Topography

Local site effects produce significant amplifications of the ground motion during an earthquake. One of the important local site effects is due to the effect of topography. Irregular topography can substantially affect the amplitude and frequency characteristics of seismic motion. Macro seismic observations of destructive earthquakes often show higher damage intensity at the tops of hills, ridges and canyons than at lower elevations and on flat areas.

Two general types of topography have to be distinguished

- Surface topography, mainly characterized by mountainous features, such as the presence of rock ridges or steep soil slopes,
- Subsurface (or subsoil) topography, either caused by lateral heterogeneities of the subsoil layers or by sharp basin geometry (Fah et al., 1997; Mayer-Rosa & Jimenez, 1999).

The curvature of a sediment-filled basin structure in particular can capture body waves and cause some incident body waves to propagate through the alluvium as surface waves resulting in stronger shaking effects and longer duration of strong ground motion (Kramer, 1996).The effect of topography on the amplification has been studied by field experiments. Trifunac and Hudson (1971), Davis and West (1973) recorded significant amplifications. But the amplifications recorded were much higher than those predicted by the theoretical models. The main problem involved was development of adequate soil models that can sufficiently represent the uncertainties involved in the geology and geotechnical properties of the area (Bard and Tucker, 1985, Geli et al., 1988). Aki (1988) pointed out that significant correlations exist between the amplification, geological and geotechnical features. Aki also proposed that these complex relationships can be represented by the seismic microzonation maps. Pedersen et al. (1994) found that the diffracted waves found at the reference station might explain the amplification of spectral ratios. The quantitative value of the ground motion at any particular site depends upon the source of the seismic waves.

The main effects of topography as pointed out by Brune (1984) and Sanchez-Sesma (1983, 1985) are:

- Amplification of S_H waves near the crests of canyons.
- The presence of a shear fundamental resonance in a 3-5 Hz frequency band.
- Dependence on the radiation wave field, angle of incidence and canyon dimensions.

In contrast to subsoil topography, it is the surface topography that causes more serious effects. According to numerous reconnaissance studies after strong earthquakes, an increase of damage to buildings can be observed on steep slope situations which extend towards the plateau. Damaging events like Bingol earthquake of May 1, 2003 in Turkey and Northern Algerian earthquake of May 21, 2003 displayed heavy damage concentrations along the top of steep slopes. According to different scientific groups (Aki, 1988; Bard, 1995, 1997; Geli et al., 1988) that deal with instrumental and theoretical investigations of surface topography on ground motion characteristics, the following are stated:

- Mountain tops or ridge crests, and more generally, convex topographies (such as cliff borders), lead to an amplification of seismic ground motion, while valleys or foothills (concave topographies) tend to de-amplify the seismic signals.
- The effects of surface topography are larger on horizontal components than on vertical ones, thus indicating that S motion is more affected by surface topography than P motion.
- The influence of surface topography on ground motion is directly related to the sharpness of topography. According to this theoretical model, amplification of incoming seismic waves increases as the wedge angle becomes sharper.
- Amplification and de-amplification of seismic ground motion on topographic features are both frequency-dependent and band-limited: maximum effects can be observed for wavelengths that approximately agree with the horizontal dimensions of the topographic shape.

The approach adopted by AFPS (1995) was to introduce an additional empirical parameter as the topographical amplification factor in the definition of the design spectrum to account for the topographical effects:

$$\tau = 1 + 0.8(I - i - 0.4) \quad \text{where} \quad 1.0 \leq \tau \leq 1.4 \ (1) \qquad (4.1)$$

Where, I and i are the gradients of the lower and upper slopes respectively. During the Athens earthquake that occurred in 1999, amplification studies on sites were carried out to find the effect of topography. It was concluded that the average topographic amplification is about 50% for time periods of about 1second at a site (Bouckovalas and Kouretzis, 2001). The effect of slope topography on the seismic ground motion is studied by George and Papadimitriou (2004). The following important observations are to be noted:

1. Even a purely horizontal excitation, as a vertically propagating SV wave, results in considerable vertical motion at the ground surface near the slope. This

component of ground motion is independent of any vertical excitation induced to the base of the slope by the earthquake itself and, consequently, it has to be superimposed to it. The results of the parametric analyses show that the vertical component of seismic motion may become comparable to the horizontal free-field motion.

2. The topography aggravation of the horizontal ground motion, expressed through the peak acceleration ratio $A_h = Z_{ah} / a_{h,ff}$, fluctuates intensely with distance away from the crest of the slope, alternating between amplification (A_h>1.0) and de-amplification (A<1.0) within very short horizontal lengths.

3. The horizontal ground motion is de-amplified at the toe of the slope and amplified near the crest. As a result, topography aggravation may be seriously overestimated, when calculated as the peak seismic ground motion at the crest over that at the toe of the slope.

The above two points can be directly associated to reflection of incoming S_V waves on the inclined free surface of the slope which leads to reflected P and S_V waves impinging obliquely at the free ground surface behind the crest, as well as Rayleigh waves. Topography effects become important for normalized height ratios $H/\lambda > 0.16$ and slope inclinations $i > 17$. If these conditions are met, the peak values of topography aggravation factors for the horizontal and the vertical ground acceleration behind the crest usually vary between $A_{h,max}$ = 1.20–1.50 and $A_{v,max}$ = 0.10–1.10, while free field conditions behind the crest are usually met at a distance D_{ff} = (2–8)H.

Based on the records obtained from the dense strong motion array in Taipei basin for two earthquakes(M_L=6.5 and 6.57) Chin-Hsiung et al., (1998) reported that there were significant differences among the peak ground accelerations, durations, and spectral accelerations in different parts of the basin as well as among the records for two earthquakes. The authors have concluded that for estimating ground motion variations for microzonation of the basin one must be very careful to draw conclusions by using only few seismic events and that it is necessary to collect more data from the array to perform a more detailed microzonation study. The last two decades has witnessed significant progress in estimation of local site effects by different researchers all over the world (Campillo et.al, 1988, Sanchez-Sesma et al. 1988, Borcherdt and Glassmoyer 1994, Trifunac and Todorvska 2001).

The effect of Central Mountain Range in Taipei basin was evaluated by Shiann-Jong Lee et al (2008). The Spectral Element Mesh method was adopted to simulate the seismic wave propagation. Realistic topography and complex subsurface topography can be effectively incorporated using SEM. It was concluded that in the event of a shallow earthquake in the area the central mountain range will scatter the surface waves there by reducing the magnitude of strong ground motion. The occurrence of a deep earthquake, topography scatters the body waves which propagate as body waves resulting in increase of PGA values by over 50%.

Shuo Ma et al. (2008) studied the effect of San Gabriel mountains which are bounded by the Mojave segment of the San Andreas fault on the north and by the Los Angeles Basin on the south. It was observed that effect of mountain topography reduces the value of ground motion for some areas in the basin. The topography of the mountains scatters the surface waves generated by the rupture on the San Andreas fault, leading to less-efficient excitation of basin-edge generated waves and natural resonances within the Los Angeles Basin.

When the top width of the geological formation is shorter than the base width (Figure 4.3), the amplification produced will be higher than one (Towhata, 2008). The amplification factor can be calculated by taking the ratio of amplitude at the hill top and the surface (base). It is clear from the above figure that a triangular hill with zero top width achieves the greatest amplification and the top width equal to B is of amplification equal to one at all frequencies.

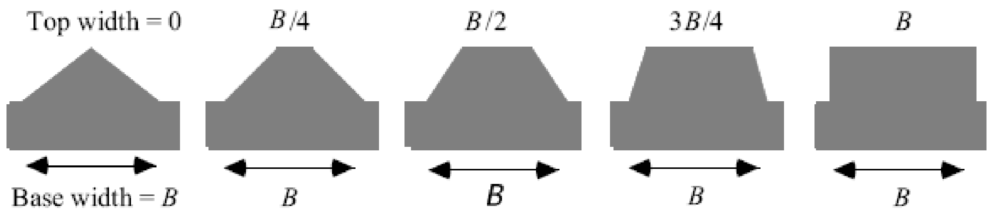

Figure 4.3: Hypothetical shapes of hills

It is very important to understand the local geotechnical and geological conditions when performing seismic microzonation studies. Site classification can be done based on geological units (Nath et., al 1997). But in microzonation even further detailed study of the variations in the geological units is necessary. Wills & Silva (1998) suggested and utilized the average shear wave velocity in the upper 30 m as one parameter to characterize the geologic units admitting the importance of other factors such as impedance contrast, 3-dimensional basin and topographical effects, and source effects such as rupture directivity on ground motion characteristics. Thus the influence of local soil stratification on earthquake characteristics is one of the major factors in evaluating the earthquake forces and thus the structural response. Local soil conditions may amplify or deamplify the earthquake forces in different regions. Sometimes an appropriate simplification may be needed to reflect the complex stratification characteristics in describing the soil amplification phenomena.

Basin and Valley Effects

Many of the modern civilization and cities were developed in river basins. In most of these locations, the depth of bed rock is very high and the soil overburden consists of loose soil. These facts points towards the importance of evaluation of site amplification due to basin effects. The presence of softer alluvial soils and the curvature of basin will amplify the ground motion and it will also increase the duration of motion also. The study by King and Tucker (1984) has found that the one dimensional ground response analysis can predict the ground response only at the centre of the basin and not at the edges. This variation will have significant effect on the design of long span structures like bridges, pipe lines etc. which are crossing the valley.

Depending on the curvature of the terrain, focusing and defocusing of the seismic waves will occur (Figure 4.4). Waves are scattered or trapped depending on whether the topography is concave or convex (Lay and Wallace, 1995). The slope angle is one of the important parameters affecting the site amplification. Because this angle determines the angle of reflection and refraction of seismic waves. Based on the study on the effects of slope angle on site response, Ashford et al. (1997) has reported that the slope angle of 15% - 25% will create the maximum amplification (Figure 4.5). It has been found that the ridges at the top of the hills will amplify the seismic waves and the valleys will attenuate the seismic waves.

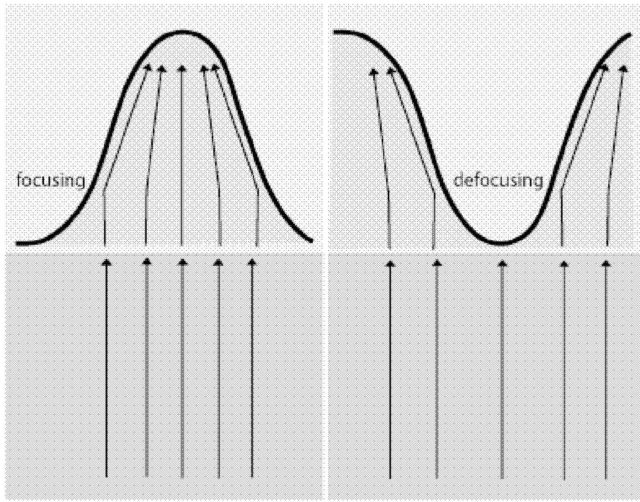

Figure 4.4: Focusing and defocusing of seismic waves

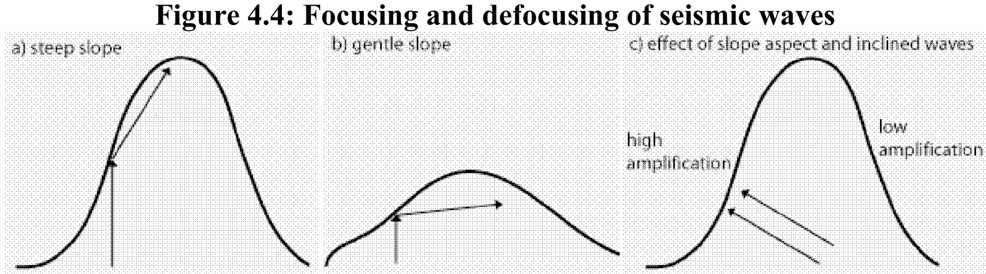

Figure 4.5: Effect of slope on site response

Due to the importance of the surface topography in the site amplification, there have been attempts to evaluate the site amplification at regional scale using remote sensing images (Lee et al., 2009; Anggraeni, 2010).

Effect of Slope Angle

The effect of the slope angle and the depth has been studied by various researchers (Nguyen and Gatmiri, 2007; Gatmiri et al., 2008; Fiore, 2010 etc.). In a more recent work Fiore (2010) has evaluated the site amplification values for different slope angles. This study has found that the amplification due to topography will be more at the upper break f the slope that at the bottom of the slope or on the slope itself. When the slope angle is between 28° - 41° the amplification will be maximum for the frequency range of 4 – 16 Hz. For slope angles below 24° the amplification does not show much variation with frequency. Fiore (2010) has also reported that the amplification is a linear function of slope and can be evaluated using the following equation

$$Amplification_d = 0.02 * d + 1.10 \tag{4.2}$$

Where d – slope angle in degree (between 10° - 41°)

For a very wide and shallow basin, one dimensional analysis can be done at the centre while a two dimensional analysis will be required at the edges. However, in the case of a deep narrow basin, one dimensional analysis will not give accurate results. The effects of topographic and subsurface irregularities are given in table 4.1

Table 4.1: Effects of Topographic and Subsurface irregularities (Modified from Silva et al., 1988)

Structure	Effect on seismic waves	Effect
Sediment filled valleys	Local changes in shallow sediment thickness	Increased duration of Significant motion
	Generation of long period surface waves from body waves at shallow incident angle	Increased amplification and duration due to trapped surface waves.
Shallow and wide (depth/width < 0.25) sediment filled valleys	More pronounced effects near the edge. Largely vertically propogating shear waves away from edge	Amplification near edges due to the generation of surface waves. One dimensional analysis may under predict the amplification values.
Shallow and wide (depth/width > 0.25) sediment filled valleys	Effects throughout the valley width	Amplification throughout the valley. One dimensional model may under predict the values

Effect of Ground Water

The relation between groundwater and earthquakes is not new and is well documented. The Chinese noticed centuries ago that water levels in wells can vary in association with earthquake activity, and used this behavior with some success to anticipate earthquakes. More recent research in the United States has attempted to use monitoring of groundwater levels in wells to help predict earthquake activity (Moyle, 1980). The 1983 Borah Peak earthquake in southeastern Idaho caused groundwater located near its epicenter to erupt as much as 25 feet into the air (CGER, 1992). However, within the past 50 year researchers have noted that groundwater can play a direct role in earthquake occurrence and earthquake related damage. In addition, it is known that earthquakes such as the 2004 Indonesian quake can cause measurable changes on groundwater levels in places thousands of miles from the epicenter (USGS, 2005). Research on earthquake mechanisms indicates that groundwater likely plays a significant role in many large earthquakes. Furthermore, groundwater can magnify the damaging effects of ground surface.

Groundwater can also play a significant role in how earthquakes affect the ground surface when an earthquake occurs. The most well known effect is liquefaction. During the shaking cause by an earthquake, certain types of fluid saturated sediments can lose their structure and become liquefied. These phenomena led to increased structural damage. The geologic and hydrologic factors that affect liquefaction susceptibility are the age and the type of sedimentary deposits, the looseness of cohesions less sediments, and the depth to the ground water table. The liquefaction is mostly limited to water-saturated, cohesions less, granular sediments at depths less than 15m. Noack and Fah (2001) gave weight according to the depth of water table (Table 4.2, more weight more damage, less weight less damage).

Table 4.2: Weightage factor for assessing liquefaction susceptibility

Depth of water table	Weightage
10 m – 20 m	2
3 m -10 m	3
1 m -3 m	4

During an earthquake, base rock movements generate shear waves that propagate through overlying soils. Liquefaction results when these shear waves, passing though saturated sand layers, distort the granular structure and cause loosely packed grains to collapse. This densification causes an increase in pore pressure if drainage cannot occur. If these pore pressures exceed roughly sixty percent (60%) of the soil's effective stress, large settlements and translational deformations can occur.

Groundwater level influences the ground response significantly and cannot be neglected for site effect analyses. In the presence of confined aquifers liquefaction can take place in the subsurface. This result in attenuation of the propagation of shear waves and can reduce the ground shaking. In the presence of Ground water the shear wave cannot reach the surface as they can't pass through the water, because water offers no shear resistance.

Effect of Bedrock

In many earthquakes, the local geology and soil condition have had profound influence on site response. The term local is somewhat vague one, generally meaning local when compared to the total terrain transverse between the earthquake source and the site. This based on the assumption that the gross bedrock vibration will be similar at two adjacent sites. Local differences in geology and soil properties will cause different surface ground motions at the two sites. Factors influencing the local site effects are the topography, nature of bedrock and the nature and geometry of the depositional soils. Thus, the term local may involve a depth of a kilometer or more, and an area within a horizontal distance of several kilometers from the site. Soil conditions and local geological features affecting site response are numerous, and are now discussed as below:

- The greater the horizontal extent of the softer soils, the less the boundary effects of the bedrock on the site response.
- The depth of soil overlaying bedrock affects the dynamic response, the natural period of vibration of the ground increasing depth.

This helps to determine the frequency of the wave amplified or filtered out by the soils and is also related to the amount of soil structure interaction that will occur in an earthquake. The Mexico earthquake of 1957 and 1985 witnessed extensive damage to long-period structures in the former lake bed area of Mexico City where the flexible latchstring deposits caused greater amplification of long period waves.

- The slope of the bedding plates of the soils overlying bedrock obviously effects the dynamic response, but it is less easy to deal rigorously with non-horizontal strata.
- Changes of soil type horizontal across a site affect the response locally within that site, and may profoundly affect the safety of a structure straddling the two

soil types.

- The topography of both the bedrock and the deposited soils has various effects on the incoming seismic waves, such as reflection, refraction, focusing, and scattering.

The mapping of bed rock depth can be done from the geotechnical or geophysical data collected. A sample figure showing the rock depth and overburden thickness for Bangalore is shown in Figure 4.6.

4.4 Methods of Estimating Local Site Effects

There are different techniques available for the estimation of site response. Broadly they can be categorized as empirical, experimental and numerical methods. The researchers, engineers and seismologists aim to find convenient and low cost methods for the estimation of the effects of local geology.

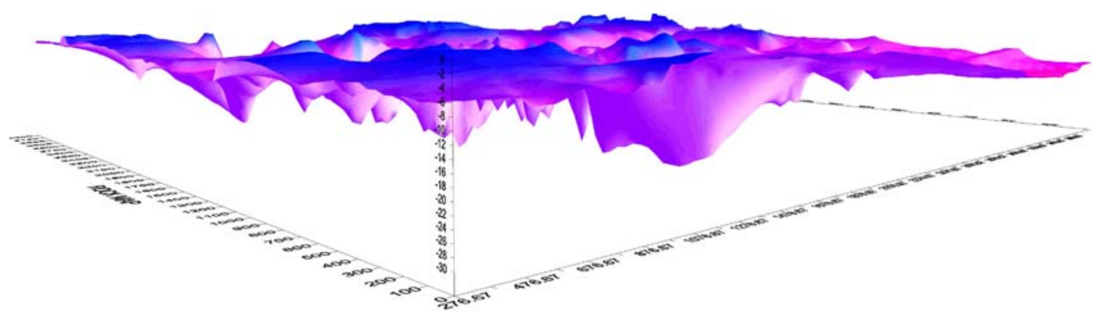

Figure 4.6: Rock depth and overburden thickness for Bangalore

Empirical Methods

Several researchers have developed empirical relations between surface geology and various ground motion parameters. These relationships are developed from one particular set of data where both earthquake observations and information on surface geology are available, which can be applied at other sites where only geological information is known. The below mentioned methods will give only an approximate value of site amplification and related effects. The site amplification will depend on various other factors also and hence these methods should be used when there is not enough geotechnical data to evaluate the site amplification.

(i) Earthquake Intensity Increment Based on Geology

The relationship between surface geology and seismic intensity increments has been developed by various researchers. The correlations given by Medvedev (1962), Evernden and Thomson (1985), Kagami et al. (1988) and Astroza and Monge (1991) are based on the seismic observations in Asia, California, Japan and Chile respectively. Medvedev's (1962) empirical relation (Table 4.3) has been extensively used in eastern European countries for microzonation studies while the one developed by Evernden and Thomson (1985) is used in the Western Europe (Table 4.4). Owing to the wide availability of the geological maps, zonation can be done using these correlations.

Table 4.3: Intensity Increment for each geological unit (Medvedev's, 1962)

Lithology	MSK Scale
Granites	0.0
Lime stone Sand Stone Shale	0.2 – 1.3
Gypsum, Marl	0.6 – 1.4
Coarse material ground	1.0 – 1.6
Sandy Ground	1.2 – 1.8
Clayey Ground	1.2 – 2.1
Fill	2.3 – 3.0
Moist Ground (gravel, sand, Clay	1.7 – 2.8
Moist fill and Soil ground	3.3 – 3.9

Table 4.4: Intensity Increment for each geological unit (Everden and Thompson, 1985)

Lithology	MM Scale
Granitic and metamorphic rock	0.0
Paleozoic rocks	0.4
Early Mesozoic rocks	0.8
Cretaceous to Eocene rocks	1.2
Undivided tertiary rocks	1.3
Oligocene to middle Pliocene rocks	1.5
Pliocene – Pleistocene rocks	2.0
Tertiary Volcanic Rocks	0.3
Quaternary volcanic rocks	0.3
Alluvium (water table (< 30 ft)	3.0
30 ft < water table < 100 ft	2.0
100 ft < water table	1.5

(ii) Spectral Amplification Based on Geology

Borcherdt and Gibbs (1976) proposed the concept of relative amplification which can be used to evaluate the effect of local geology quantitatively. Relative amplification is defined as the amplification with respect to reference site. They measured ground motions generated by nuclear explosions at sites with various geological conditions to obtain the spectral amplifications of the motions with respect to granitic rock sites. Based on this they developed an equation between intensity increment and the average horizontal spectral amplification (AHSA), which is the average of the spectral amplification in the frequency range of 0.5 to 2.5 Hz, as shown below.

$$II = 0.27 + 2.7 \log (AHSA) \qquad (4.3)$$

Where, II is Intensity increment and AHSA is the average horizontal spectral amplification.

Shima (1978) has given the relative amplification factors for different soil types based on analytical computations of the seismic ground response. He defined the relative amplification as the ratio of the maximum value of ground response in the frequency range of 0.1 – 10 Hz with respect to that on Loam. Midorikawa (1987) used similar procedure and proposed the values of the relative amplification factor for various geological types. The amplification factor here is defined as the mean of ground amplification in the frequency range of 0.4 to 5 Hz. Table 4.5 gives the summary of the relative amplification factors with geology.

Table 4.5: Correlations between Surface Geology and Relative Amplification (TC4-ISSMGE, 1999)

Geological Unit	Relative Amplification
Borcherdt and Gibbs (1976)	
Bay mud	11.2
Alluvium	3.9
Santa clara formation	2.7
Great valley sequence	2.3
Franciscam formation	1.6
Granite	1.0
Shima (1978)	
Peat	1.6
Humus soil	1.4
Clay	1.3
Loam	1.0
Sand	0.9
Midorikawa (1987)	
Holocene	3.0
Pleistocene	2.1
Quaternary volcanic rocks	1.6
Miocene	1.5
Pre- Tertiary	1.0

Tryantafilidis et al. (1999) proposed correlations between intensity increment (II) and the average horizontal spectral amplification (AHSA) for different frequency ranges using the instrumental data and microseismic observations in the Thessaloniki city.

$$II = 0.83 + 2.35 \log (AHSA_{0.25\text{-}3.0}) \quad (4.4)$$

$$II = 0.21 + 3.40 \log (AHSA_{1.0\text{-}10.0}) \quad (4.5)$$

$$II = 0.13 + 3.12 \log (AHSA_{3.0\text{-}6.0}) \quad (4.6)$$

These correlations indicate that the locations with average spectral amplifications of around 10 have experienced local intensity increments as large as 2 to 3 times where as for average amplifications of 4 it reduce to 1.5 to 2 times. Teramo et al., (2005) has given an empirical equation for estimating site amplification factor using shear wave velocities, slope of the ground surface and water table depth.

(iii) Relative Amplification Based on Geotechnical Parameters

The most important geotechnical parameters, which can be used to estimate the amplification factors are average shear wave velocity and SPT 'N' value. In the early days measurement of shear wave velocity using cross hole tests were expensive where as the SPT 'N' values were available abundantly. Shima (1978) found that the analytically calculated amplification factor is linearly related with the ratio of shear wave velocity of the surface layer to that of bedrock. When the bedrock shear wave velocity is found to be relatively constant over a wide area, the relative amplification in each locality can be obtained from the shear wave velocity of the surface layer. Various researchers (Joyner and Fumal (1984), Midorikawa (1987) and Borcherdt et al. (1991)) proposed relations between the average shear wave velocity of surficial layers and the relative amplification as shown in Table 4.6.

Table 4.6: Correlations between Average Shear Wave Velocity and Relative Amplification (TC4- ISSMGE, 1999)

Author	Proposed equation
Joyner and Fumal (1984)	$RA = 23\ V_{S'}^{-0.45}$
Midorikawa (1987)	$RA = 68\ V_{S30}^{-0.6}$ (V_{S30} < 1100 m/s)
	$= 1.0$ (V_{S30} > 1100 m/s)
Borcherdt et al. (1991)	$AHSA = 700/\ V_{S30}$ (for weak motion)
	$= 600/\ V_{S30}$ (for strong motion)

Where, RA is the relative amplification, AHSA is the average horizontal spectral amplification in the frequency range of 0.25 to 5 Hz, V_{S30} is the average shear wave velocity over a depth of 30 m (m/s) and $V_{S'}$ is the average shear wave velocity over a depth of one quarter wave length for a 1 sec period wave (m/s).

An approximate relation to evaluate the surface level acceleration values for soft soils based on rock level acceleration values is shown in Figure 4.7.

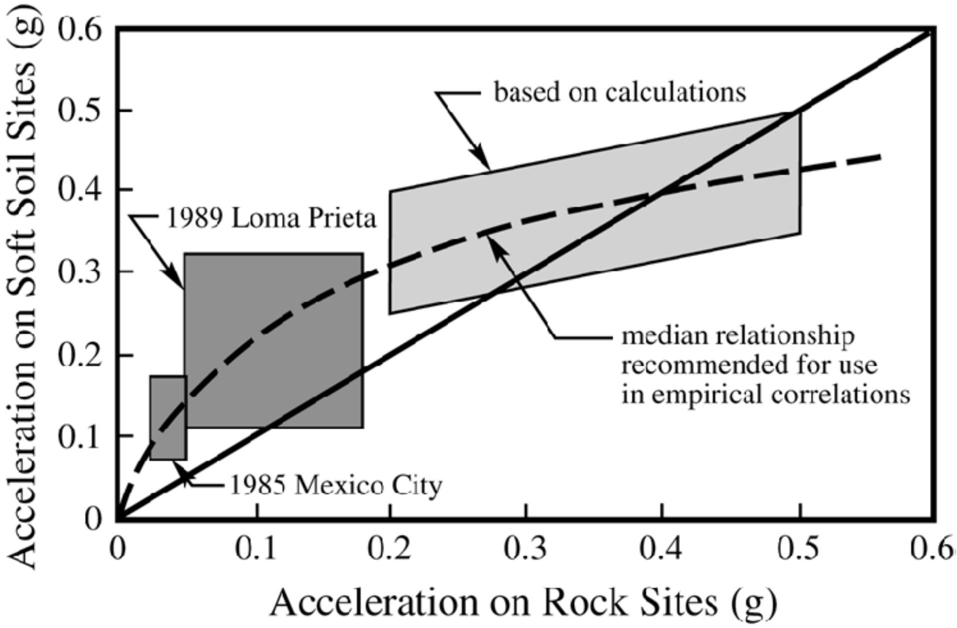

Figure 4.7: Relation between rock level and surface level acceleration values (Idriss, 1990)

The period of vibration corresponding to the fundamental frequency is called the characteristic site period. Approximate value of the site period can be evaluated using the average shear wave velocity using the following equation,

$$T_s = \frac{2\pi}{\omega_0} = \frac{4H}{v_s} \quad (4.7)$$

The haracteristic period calculated using above expression is very useful in the site amplification studies. This value provides an indication of the period of vibration at which the most significant amplification would occur.

(iv) Amplification Based on Surface Topography

Topographical effects can play important role on ground motion characteristics and this has been highlighted by many researchers (Geli et al., 1988; Faccioli, 1991; Chávez-García et al., 1996, Reinoso et al., 1997; Athanasopoulus et al., 1999). Theoretical and experimental studies on topographical effects on ground motion have been made by Aki (1988) who showed the effects of topography using a simple structure of a triangle edge model. Facioli (1991) used Aki's model as ridge-valley topography and addressed the relative amplification at the crest of the ridge compared to the base and also the deamplification in the valley. Amplification of the motion of the crest of ridge relative to the base is pointed out by Brambati et al. (1980) dealing with damage patterns during the 1980 Friuli earthquake in Italy and in the Chilean earthquake of 1985. It has been predicted numerically that in the valley, there is deamplification of the amplitude due to the defocusing effect. The intensity in a valley may be 1-2 times lesser as compared with the surrounding, if it is free from the soil deposits. The approach adopted by AFPS (1995) was to introduce an additional empirical parameter as the topographical amplification factor (A_T) in the definition of the design spectrum to account for the slope variation effect on topographical effects;

$$A_T = 1+0.8(I-i-0.4) \qquad 1.0 \le A_T \le 1.4 \quad (4.8)$$

where, I and i are the gradients of the lower and upper slopes respectively. The effects of basins and sediment filled valleys on earthquake ground motion were investigated by various researchers. (Bard and Bouchon, 1985; Wen et al., 1995; Rassem et al., 1997, Gao et al., 1996; Su et al., 1998; Wald and Graves, 1998; Kawase, 1998; Amirbekian and Bolt, 1998; Paolucci et al., 2000; Sokolov et al., 2000; Chávez- García and Faccioli, 2000).

Experimental Methods

These methods are based on different kinds of data based on microtremor measurements, weak seismicity survey and strong motion data. Detailed description for estimating site effects using the above methods is given below.

(i) Microtremor Data

The site effects are often expressed by the amplification factor and resonance/fundamental frequency. Usually there are various vibrations in the ground which are caused by natural or ambient noise like wind, sea waves, traffic, industrial machinery etc. The range of vibration frequencies of ambient noises is from 0.1 Hz to 10 Hz (i.e. 10 sec to 0.1 sec period). The vibrations that have comparatively small periods of less than 1 second are called microtremors and those that have a larger period range is called microseisms. Due to close relation between spectral features of microtremors and site's geological conditions, these small vibrations are very useful in earthquake geotechnical engineering. Kanai and Tanaka (1961) have explained a theoretical interpretation and practical engineering application of microtremors as a convenient tool for evaluating frequency properties of surface ground. The use of microtremor method for site response evaluations has been restricted to Japan with many controversies in USA and Europe. Applications of this method include site response analysis, natural frequency of structures etc. However, the real generation and nature of microtremors have not yet been established.

Kanai et al. (1954) proposed a method to classify the ground into four categories, which is used by the Japan Building Code. This classification is based on the detailed comparison of microtremor results and ground conditions. A general wave pattern and period-frequency (pf) diagrams for four types are shown in Figure 4.8. It is clear that a stiff ground/ rock is inclined to have low predominant period and soft sediments have high predominant period. Kanai and Tanaka (1961) proposed two methods based on microtremor records. One is based on the largest period and the mean period, the other based on the largest amplitude in microns and the predominant period. This can be used to identify site categories associated with various levels of seismic damage due to strong shaking. Table 4.7 gives the detailed description of soil type in each category.

Omote and Nakajima (1966) classified pf diagrams from more than 500 sites into three categories. Also, the distribution of SPT 'N' value with depth for three categories has different patterns.

Type A: $T = 0.1 - 0.25$ sec (no peak over 0.3 sec)

Type B: $T = 0.25 - 0.4$ sec (a few peaks between 0.5 and 0.6 sec)

Type C: $T \geq 0.4$ sec (flat as a whole)

Table 4.7: Microzones for Japan Building Code (Kanai and Tanaka 1961)

Zones	Soil Description
I	Ground consisting of rock, hard sandy soils or gravely deposits
II	Ground consisting of sandy gravel, hard sandy clay, loam or alluvial gravel with thickness of 5m or more
III	Standard ground other than Zone I, Zone II or Zone IV
IV	Ground consisting of soft alluvial delta deposits, top soils or mud thickness of 3m or more where less than 30 years has elapsed since the time of reclamation.

Udwadia and Trifunac (1978) investigated the use of microtremor method to determine site amplification characteristics for earthquake shaking by comparing data from strong motion earthquake records and microtremor measurements in California. They found little correlation between the ground motions due to earthquake shaking and microtremor excitation. Nakamura (1989) carried out extensive microtremor study in Kanonomiya and Tabata, Japan and proposed a method, which is widely used in response studies. According to this the H/V ratio is a reliable estimation of not only resonant frequencies but also about the corresponding amplification. On soft soil sites they exhibit a clear peak that is well correlated with the fundamental resonant frequency which is supported by several researchers (Field and Jacob, 1993; Lachet and Bard, 1994; Lermo and Chavez-Garcia, 1994). Ohmachi et al. (1991) used Nakamura's technique to analyse microtremor measurements conducted at sites of heavy damage in San Francisco after the Loma Prieta earthquake in 1989. The estimated frequencies and amplification factors for the Marina district are correlated with three degrees of damage.

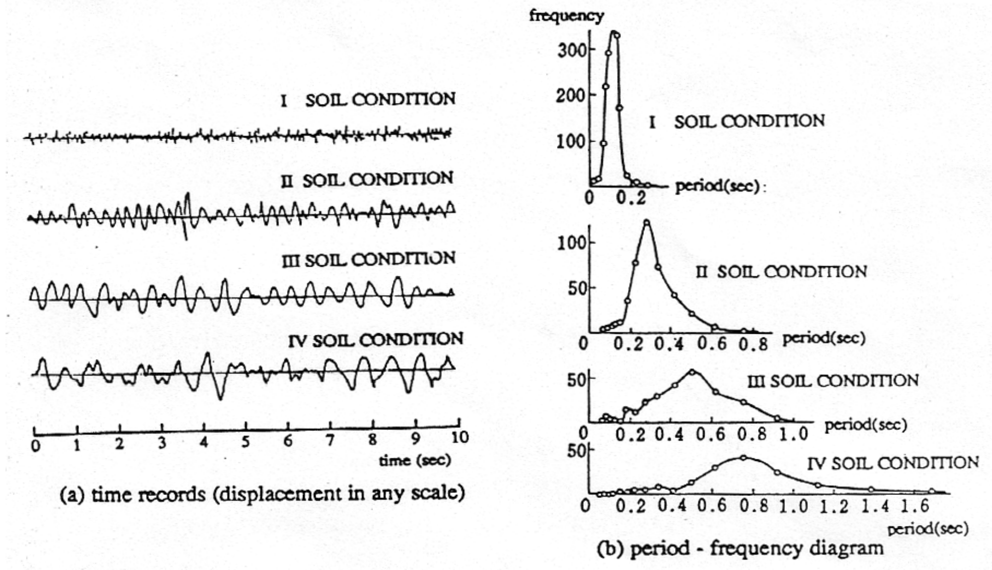

Figure 4.8: Classification of Microtremor Types (Kanai et al., 1966)

These studies conclude that the peak H/V amplitude is not well correlated with the S wave amplification at the site's resonant frequency. It is highly sensitive to some parameters like Poisson's ratio near the surface. A one to one average correlation is developed by Konno and Ohmachi (1998) on the basis of a comparison between observed H/V peaks and numerical estimates of 1D transfer functions. Also, an empirical relationship between H/V peak amplitude and local intensity increment (MM scale) was developed by Toshinawa et al. (1997) based on his experimental studies. Bard et al. (1997) compared the amplifications derived from earthquake records and H/V peak amplitudes for more than 30 sites and demonstrated that the later is always smaller than the former. This Nakamura method provides a lower bound estimate to the actual amplification which is proved by large experimental data. Thus it can be concluded that microtremor method provide reliable estimate of both site periods and amplifications in the long period range only that is when source and path effects are the same for all the sites considered.

There exists one more indirect use of microtremor recordings. Aki (1957) showed that the noise recordings on small aperture arrays could be used to measure phase velocities of surface waves by analyzing the spatial correlation of the microtremors. Then, the P and S wave velocity profiles can be derived by inversion which makes it possible to estimate the site response by 1D modeling. Several studies in Italy (Malagnini et al., 1993; Chouet et al., 1998), Japan (Miyakoshi and Okada, 1996) and Israel (Gitterman et al., 1996) illustrate the practical use of this technique with other methods used in geotechnical engineering in estimating velocity profiles. In particular, its coupling with the Nakamura technique (array measurements at few sites and H/V ratios at many sites) may lead to a reliable 2D and 3D mapping of the subsurface conditions (Gitterman et al., 1996; Tokimatsu et al., 1994).

Mukhopadhyay et al. (2002) conducted microtremor studies in Delhi and compared the resonance frequency obtained by microtremor with that estimated from strong motion records. The strong motion records are from Singh et al. (2003) for the same locations (14 sites). Tuladhar et al. (2004) did seismic microzonation of greater Bangkok area using microtremor observations.

The microtremor survey using a single point micro tremor can be used for preliminary site classification, estimation of bed rock depth, validation of geological profile etc. even though the single point micro tremor survey is widely being used, there are limitations in interpretation of results. The results may be misleading in the case of complex soil profiles.

(ii) Weak Motion Data

Weak motion data are the records from small to moderate, natural or artificial seismic events (small magnitude earthquakes, aftershocks of big events, mine or quarry blasts, nuclear tests). Such data can be recorded by digital, high sensitive instruments identical to those used by seismologists for microseismicity and seismotectonic studies. Field and Jacob (1993) quoted that the greatest challenge in the estimation of site response from such instrumental recordings is removing the source and path effects. Two techniques are developed depending on whether or not they need a reference site with respect to which the particular effects at other sites are estimated.

(a) Reference Site Technique

The three important factors which will affect the ground motion are the source, path and the site characteristics. The identification and removal of these effects is the greatest challenge in evaluating the site response. The simplest method to evaluate the site response is to divide the response spectrum obtained at the site with that of the bed rock (reference site). If the recording in the rock is at a close distance to the soil site, then the three governing factors, which will affect the ground motion, will be the same for both the soil site and the rock. However when the reference site and the site under consideration (soil site) are not nearby, then the influence of source, path and the site characteristics will be different for these two sites. More over the geometric spreading of the seismic waves will also need to be accounted (Borcherdt & Glassmoyer, 1994; Borcherdt, 1996; Hartzell et al. 2000; Borcherdt 2002). To evaluate the source and site terms simultaneously, a generalized inversion scheme developed by Andrews (1986) can be applied. In the generalized inversion technique a relatively large dataset can be used (Stewart et al., 2003) but the nonlinear response of sedimentary deposits cannot be predicted accurately when the weak motion data are dominating the input data.

(b) Non Reference Site Technique

In practice, adequate reference sites are not always available. For this reason, different methods without reference sites have been developed. It consists of taking the spectral ratio between the horizontal and the vertical components of the shear wave part. This technique is a combination of Langston's (1979) receiver function method for determining the velocity structure of the crust from the horizontal to vertical spectral ratio (HVSR) of teleseismic P waves, and the Nakamura's (1989) method. It was first applied to the S wave portion of the earthquake recordings obtained at three different sites in Mexico City by Lermo and Chavez- Garcia (1993). The same technique has been applied on various sets of weak and strong motion data (Lachet et al., 1996; Theodulidis et al., 1996; Bonilla et al., 1997; Chavez- Garcia et al., 1996; Riepl et al., 1998; Zare et al., 1999) from which it is concluded that The HVSR shape exhibits a very good experimental stability, it is well correlated with surface geology and less sensitive and comparisons with theoretical 1D computations, the absolute level of HVSR depends on the type of incident waves. Also, the determination of the absolute level of amplification from only HVSR is not straight forward. Field and Jacob (1993) applied this technique and found that the method reproduces well the shape of the site response, but underestimates the amplification level. They also found very different results when applying this technique to the P wave part of the recordings.

(iii) Strong Motion Data

The development of strong motion arrays makes it possible to evaluate site effects in mega cities like Los Angeles, Tokyo, Taipei and Mexico city using the strong motion data. While using this method, even the non-linear site effects are included in the recordings. Recent studies show that there is a fairly good agreement between the old and new techniques. With the increase in strong motion data, it is possible to determine soil amplification and prepare site response maps based on ground motion records (Boore et al., 1993; Beresnev et al., 1995; Theodulidis & Bard, 1995; Hartzell et al., 1997; Atkinson & Cassidy, 2000). Khoubbi and Adams (2004) estimated soil amplification in Ottawa, Canada using strong motion records.

Numerical Methods

The most commonly used methods for one two and three dimensional analysis of site response are discussed below.

(i) One dimensional site response analysis

In most of the soils the wave propagation velocities increase with depth. Hence when a seismic wave strikes the softer overlaying material, it will be reflected towards the vertical direction. When the seismic wave reached the ground it might have under gone so many refractions and as a result its direction will be nearly vertical. The basic assumption of one dimensional analysis is that all the boundaries are horizontal and the response is caused by the wave propagating in a vertical direction. The one dimensional site response can be evaluated using linear or nonlinear approaches (Kramer, 1996).

The linear approach is the simplest approach to evaluate the ground response. It relies on the principle of superposition. This approach is only suitable for analysis of linear systems. However, the nonlinear behavior of the soil can be approximated by iterative procedure with equivalent linear soil properties. This linear approach has been implemented in the following procedures, which are commonly used for ground response analysis. (Kramer, 1996).

- Transfer functions
- Equivalent linear approximation of non linear response
- Deconvolution

Transfer Functions:

Calculation of transfer functions is the key to any linear analysis. It requires solving the wave propagation equation for the case of one dimensional soil model. Basically, it provides the amplification or deamplification factor to the input bedrock motion, to arrive the surface ground motions. Typically, the procedure to arrive the surface motions from the input bedrock motion consists of the following steps.

The time history of bedrock motion is converted to frequency domain using Fast Fourier Transforms (FFT). Transfer function is then applied to the each of frequency (Fourier Series), to obtain Fourier series of surface/ground motion. Then, inverse FFT is applied to obtain represent surface motions in the time domain.

Equivalent linear approximation of non linear response:

As the soil is a nonlinear material, the linear approximation is not appropriate with problems involving soils such as ground response analysis. Hence, a modified approach called equivalent linear analysis is most commonly adapted to approximate non linear response of the soils. The shear modulus and damping are depended upon the level of strain to which soil is subjected. The evaluation of these equivalent shear modulus and damping properties of soils are already discussed in the Section 3.6.

In equivalent linear analysis, it is required to first estimate the level of shaking in each layer assuming some constant initial values of shear modulus and damping. Later in successive iterations, the shear modulus and damping of the soils that corresponds to the estimated levels of shaking (shear strains) are used in the analysis. Thus, the method considers nonlinear stress strain response of the soils indirectly. It is worth remember that even though it consider equivalent shear modulus and damping, these values are constant though out the shaking (analysis) in any iteration. A widely used computer program, SHAKE (Schnabel et al., 1972; Idriss and Sun, 1992) and DYNEQ (Yoshida and Suetomi, 1996) implemented this equivalent linear approach.

Deconvolution:

It is often required to obtain actual earthquake motions at the bedrock from the measured/recorded surface ground motions. Deconvolution is the technique to obtain the bedrock motions from the surface ground motions. It uses same transfer functions discussed earlier to back calculate bed rock motions. This technique can include strain dependent modulus and damping properties similar to equivalent linear approach (Roesset, 1977).

Nonlinear Approach:

The linear approach is very simple and it is easy to compute, but it cannot evaluate the non linear response of the soil precisely. This issue can be overcome by using the nonlinear response of soil using direct numerical integration (in small time intervals) in time domain. The integration of motion in small time intervals will permit the use of any linear or non linear stress-strain models.

The data from borings or measurements of shear-wave velocity are used to construct the soil model. When such data are not available, generic ground conditions can be used (Shima and Imai, 1982). Since all soils have highly nonlinear properties, non-linearity in site characterization and analysis has to be taken under serious consideration. Moreover nonlinear behavior can also be observed in the earthquake ground motion records (Tokimatsu and Midorikawa, 1982; Chang et al., 1991). There are many softwares, which can incorporate the nonlinear response of soils such as PLAXIS, SASSI2000, FLAC, QUAKE/W and etc.

(ii) Two dimensional site response analysis

The one dimensional site response analysis will be useful for level or gently slopping ground with parallel soil layers. Since these conditions are not so common, the on dimensional analysis may not give very accurate results in most of the cases. In the case of sites where embedded structures like pipe lines or tunnels are there, one dimensional analysis will not yield the desired results. The two dimensional analysis can be done either based on frequency domain or time domain methods. This analysis can be done using dynamic finite element methods adopting either equivalent linear approach or nonlinear approach (Kramer, 1996). Numerical modeling software like PLAXIS, FLAC, QUAKE/W, etc can be used for modeling two dimensional cases.

Due to the high computational cost involved in the dynamic finite nonlinear element methods, various researchers proposed number of alternatives to this approach such as shear beam approach and layered inelastic shear beam approach.

Shear Beam Approach:

This approach is being widely used for the analysis of earth dam. Comprehensive literature review is provided my Gazetas (1987) on this method. Basic assumption in this method is that a dam deforms in a simple shear and thereby produces only horizontal displacements.

(iii) Three dimensional site response approach

There may be cases in which there is variation in soil profile in three dimensions and the two dimensional approach may not be adequate. This is ideal for studying the response of three dimensional structures. The method and the approaches adopted is similar to the two dimensional approach. The important approaches adopted are equivalent linear finite element approach, nonlinear finite element approach, and etc. Softwares like ABAQUS, PLAXIS, FLAC, SASSI2000, etc can used for the modeling purpose.

4.5 Deep Soil Effects

The analysis of strong motion records have found out that the difference of stiffness between the overlying soil and the underlying bedrock will affect the amplitude, frequency and duration of seismic waves. These effects were observed during the Mexico (1985) and the Loma Prieta (1989) earthquakes. The city of Mexico was underlain by a thick deposit of soft clay which extends up to more than 50m. The spectral acceleration history of Mexico earthquake at two site, deep soil and rock sites (Figure 4.9) clearly illustrates the deep soil effect on the seismic waves (Chen and Scawthorn, 2003). Figure 4.9 clearly illustrates the deep soil effect on amplitude, frequency content and duration of seismic waves. During Loma Prieta (1989) earthquake also, severe damage was observed at those locations which were underlain by thick layers of alluvium and bay mud. Based on the records of Mexico and Loma Prieta earthquakes, Idriss (1990) developed a method to predict the approximate value of surface level acceleration of deep soil sites based on the rock level peak horizontal acceleration values.

The evaluation of surface level acceleration values is (amplification factor) is a complicated issue. This is due to the variation of soil properties over short distance and the soil may not behave in elastic manner. The shear modulus of soil will also vary with the strain levels. Hence for very accurate evaluation of surface level acceleration values, site specific analysis is required. These should be done by considering the non-linear site response and using suitable numerical models.

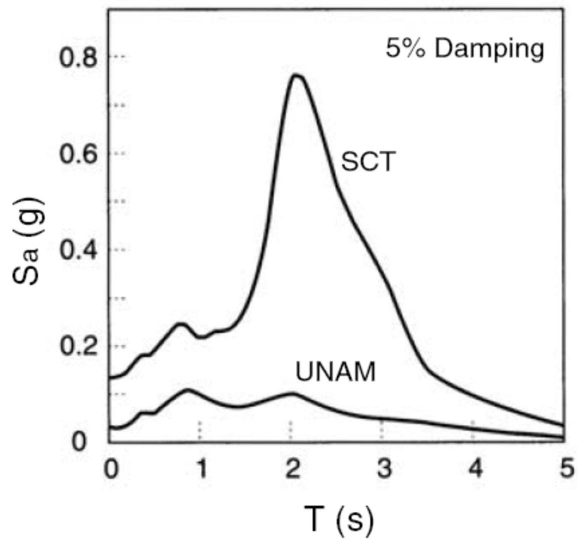

Figure 4.9: Spectral accelerations at SCT site (soft soil) vs. UNAM site (rock) (Chen and Scawthorn, 2003)

One of the most widely used computer programs for evaluating the one-dimensional seismic response of horizontally layered soil deposits, SHAKE 2000 can be used to evaluate deep soil effects. This software uses the equivalent-linear model for evaluation of non linearity of shear modulus and damping. There is another similar one dimensional ground response analysis software, DEEPSOIL (Hashash et al. 2009) which can also be used for obtaining deep soil effects. DEEPSOIL can perform both 1D nonlinear and equivalent linear analysis. There are also other software packages for evaluating the site response using finite element method (PLAXIS, SASSI2000, QUAKE/W etc.).

4.6 Ground Response Analysis

The site amplification factors at any location can be determined using two approaches – based on the observed intensity measures (*IM*) and theoretical approach. The first method compares the *IM* for various site conditions with that of a reference site condition for determining the amplification factors. The theoretical analysis methods will be useful in extending the amplification factor models to the site conditions which are poorly represented in empirical data sets (Annie and Stewart, 2006). The frequency of ground motion, which will be influenced by the site condition, depends on the thickness of the over lying soil deposit also. If the thickness of the soil deposit is small then the amplification will occur for the waves with higher frequency and vice versa. The amplification factors can be evaluated using the observational methods and this can be divided into two types – using a reference site and without using a reference site.

The three important factors which will affect the ground motion are the source, path and the site characteristics. The identification and removal of these effects is the greatest challenge in evaluating the site response. The simplest method to evaluate the site response is to divide the response spectrum obtained at the site with that of the bed rock (reference site). If the recording in the rock is at a close distance to the soil site, then the three governing factors, which will affect the ground motion, will be the same for both the soil site and the rock. However when the reference site and the site under consideration (soil site) are not nearby, then the influence of source, path and the site characteristics will be different for these two sites. More over the geometric spreading of the seismic waves will also need to be accounted (Borcherdt & Glassmoyer, 1994;

Borcherdt, 1996; Hartzell et al. 2000; Borcherdt 2002). To evaluate the source and site terms simultaneously, a generalized inversion scheme developed by Andrews (1986) can be applied. In the generalized inversion technique a relatively large dataset can be used (Stewart et al., 2003) but the nonlinear response of sedimentary deposits cannot be predicted accurately when the weak motion data are dominating the input data.

Another method of site response, which does not depend on the reference site, is based on horizontal to vertical spectral ratio (HVSR). In this method the horizontal component of the response spectra is normalized using the vertical component of the spectra for the site under consideration. This method can be applied for both the noise recordings (Nakamura, 1989; Field and Jacob, 1993) and the earthquake recordings. The attenuation relationships used to predict the ground acceleration levels will give the acceleration values at the bed rock level. These values may change considerably when the surface level peak ground acceleration (PGA) values are evaluated. It has been found that the basins and sediment filled valleys are also having significant effect on earthquake ground motion. Researchers like Field (2000), Lee and Anderson (2000) and Steidl (2000) proposed methods for modifying the attenuation relations for predicting the peak ground acceleration (PGA) and spectral acceleration values at the ground surface level. Stewart et al. (2003) has developed empirical amplification factors for active tectonic regions. They have developed separate sets of amplification factors for different site classes, which were identified using different methods. Even though these amplification factors were developed for active tectonic regions, the same can be applied to other tectonic regions after further studies (Stewart et al., 2003).

Atkinson and Boore (2006) had proposed a method to evaluate the amplification factors based on empirical studies of ground motion data for east of North America (ENA). Similarly, the amplification factor equations for Peninsular India were developed by Raghu Kanth and Iyengar (2007).

Numerical modeling techniques provide the most accurate ground response analyses, considering the actual measured local soil properties. These techniques are most commonly adopted for estimation of local soil effects. However, these methods require recorded or estimated bedrock motions. These motions will be provided by the PSHA group. Details of the various numerical techniques and their procedures are discussed in detail earlier in the Section 4.4.3. Here, the most commonly used computer program SHAKE is explained in the following subsection.

Ground Response Analysis using SHAKE

SHAKE is most commonly used computer program for carrying one dimensional ground response analysis of level 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. Details of the equivalent linear approach are discussed in detail in the Section 4.4.3.

The SHAKE assumes the soil layers are subjected to transient, vertically traveling shear waves. This assumption is a justifiable, as the waves propagate from the bed rock to the surface, they encounter softer materials and thereby they directed to vertical direction due to multiple reflections and refractions. To model any site, the information regarding subsurface layering and their characterization is required. Particularly, the shear wave velocity profile of the local soil is required for estimation of in situ shear modulus. Further to model nonlinear behavior of the soil, program requires the strain dependent shear behavior of soils and their damping ratio expressed in terms of modulus reduction and damping ratio curves respectively. Details for the determination of these properties are later discussed in the Section 3.6. SHAKE program includes the standard curves for the different types of soils based on their grain size distribution and plasticity. Some of these curves are shown in the Figures. 4.10 and 4.11. These curves can also be used in case no detailed experimental study is carried.

The program calculates the transfer functions based on the subsurface soil profile and their properties. The program/method also requires the input bedrock motion as discussed earlier (Details regarding appropriate ground motion generation will be furnished by the other group, PSHA). The input bedrock motion will be converted to Fourier series using Fast Fourier Transforms (FFT). Surface Fourier series is calculated by multiplying transfer function with bedrock Fourier series. Later, inverse FFT is applied to the calculated Fourier series of surface motion to obtain time history of the surface ground motion.

There has not been much work done in India with reference to the local site effects. For the seismic hazard assessment of Chennai, site characterization was done using the SPT data and MASW techniques by Boominathan et al. (2008). The ground motion parameters were evaluated based on the one-dimensional ground response analysis, carried out using the data collected from 38 representative sites by the equivalent linear method using the SHAKE91 program. In addition to this the characteristic site periods for Chennai were also evaluated by Boominathan et al. (2008).

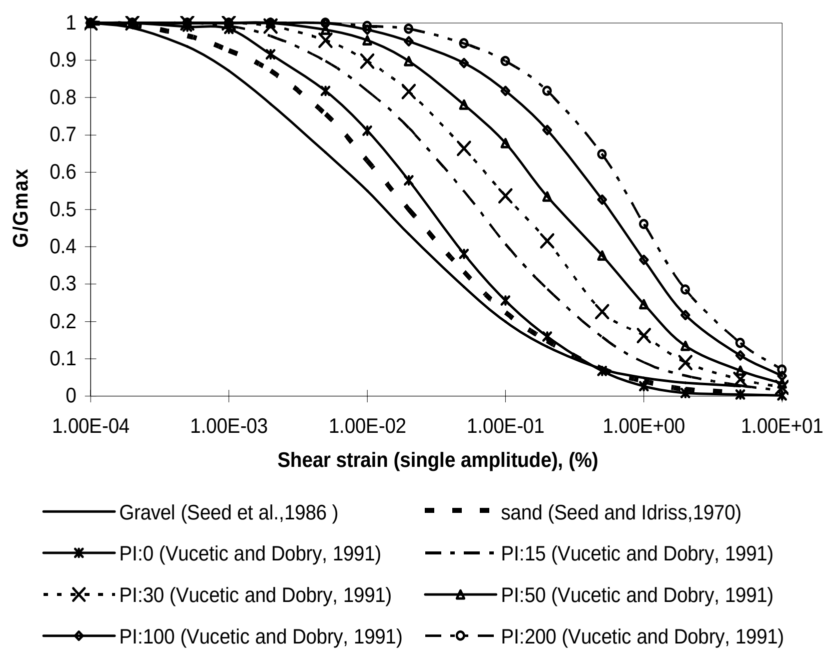

Figure 4.10: Modulus reduction curves available in the literature for soils of different plasticities.

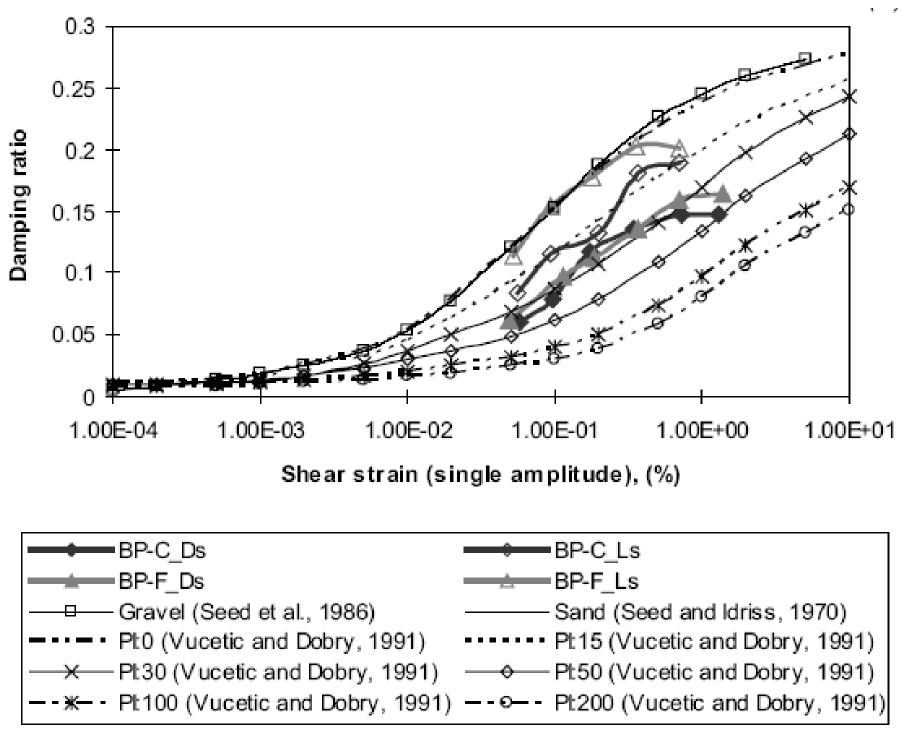

Figure 4.11: Damping ratio curves available in the literature for soils of different plasticities.

The site characterization for Bangalore was done based on the SPT and MASW test results. Local site effects were assessed by conducting one dimensional ground response analysis using SHAKE 2000. For this the SPT data as well as the MASW results were used. Based on the average shear wave velocity in the top 30 metre (V_s^{30}) values obtained from the MASW survey, the site class for Bangalore was classified as site class C and D (Anbazhagan et al., 2009) based on NEHRP site classification scheme.

4.7 Approach Adopted for the Evaluation of Ground Response Spectra and PGA Values for Different Site Classes

When the study area is very vast, it is extremely difficult to classify the region into different site classes based on the geotechnical or geophysical data. Hence in those cases the PGA values can be evaluated for different NEHRP or Eurocode site classes. Based on site investigation we can identify the site class at any location can be determined and for the appropriate site class the PGA or Sa values can be obtained from the respective maps.

There are attenuation relations which will give the PGA values for different site classes based on the rock level PHA values. One such relation is available for South India, proposed by Raghu Kanth and Iyengar (2007). The amplification factors can be evaluated based on the following equation.

$$\ln F_s = a_1 y_{br} + a_2 + \ln \delta_s \qquad (4.9)$$

Where a_1 and a_2 are regression coefficients, y_{br} is the spectral acceleration at rock level and δ_s is the error term. The values of the regression coefficients a_1 and a_2 will vary for different site classes and for different time periods. These values were derived based on the statistical simulation of ground motions. Four NEHRP site classes A to D, ten random samples of soil profiles were considered in evaluating the amplification factors. The values of a_1, a_2 and δ_s for different site classes for evaluating the PGA values are given in Table 4.8. The amplification factor for soft and medium dense soil will vary with the rock level PHA values. The value of the damping ratio, ξ, and the modulus reduction, G_{red}, will vary with shear strain (Idriss, 1990). The variation of G_{red} and the damping ratio with shear strain is shown in Figure 4.12. The method adopted for evaluation of amplification factor (F_s) values considers this effect and the value of F_s varies with the rock level PHA values. The variation of F_s values with site class and rock level PHA values are given in Figure 4.13. The value of spectral acceleration for different site classes can be obtained from:

$$y_s = y_{br} F_s \tag{4.10}$$

Where F_s is the amplification factor y_s is the spectral acceleration at the ground surface for a given site class.

Table 4.8: Amplification factors used in evaluating PGA values for different site classes in Peninsular India (Raghu Kanth and Iyengar, 2007)

Site Class	a_1	a_2	$\ln(\delta_s)$
A	0.00	0.36	0.03
B	0.00	0.49	0.08
C	-0.89	0.66	0.23
D	-2.61	0.80	0.36

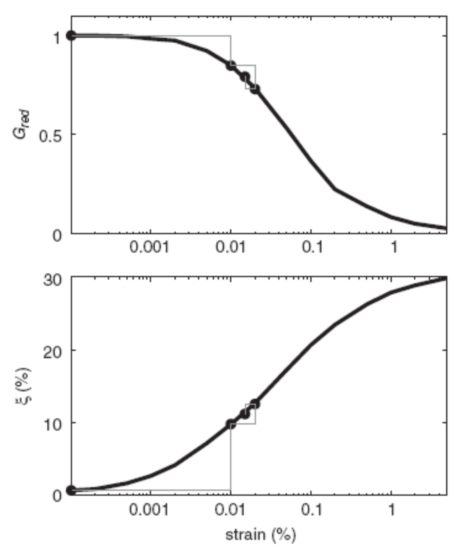

Figure 4.12: Variation of modulus reduction (G_{red}) and damping ratio (ξ) with shear strain (Robinson et al., 2006)

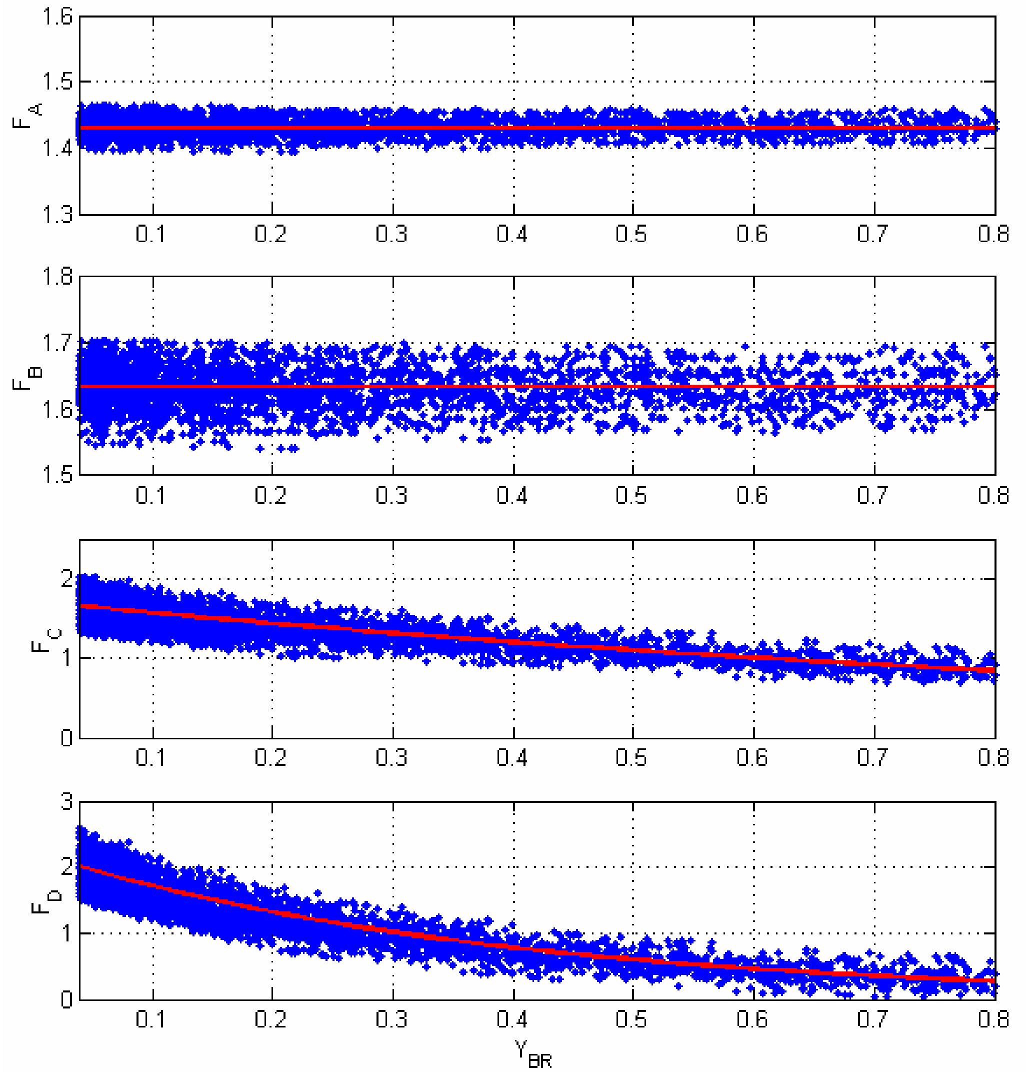

Figure 4.13: Variation of site amplification factors for different site classes with rock level PHA values (Raghu Kanth and Iyengar, 2007)

4.8 Conclusions

This chapter discusses the effects of local soil profile on site effects. This brings out the major factors which need to be checked for evaluation of site effects. Various codal provisions for site classifications are also discussed here. The major recommendations of this chapter are listed below:

- Soil topography need to be ascertained and the local site effects need to be calculated by considering the topographical effects.
- The local site effects can be estimated based on the available geotechnical data. If the data available is less, then the assessment can be done using empirical relations.
- The site amplification due to slope is a linear function of the slope angle.

5 Liquefaction

5.1 Introduction

During an earthquake, soil can fail due to liquefaction with devastating effect such as land sliding, lateral spreading, or large ground settlement. The phenomenon of seismic soil liquefaction had been observed for many years, but was brought to the attention of researchers after Niigata (1964) and Alaska earthquakes (1964). Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or by other sudden dynamic loading. This is due to the reduction in effective stress of the soil due to the sudden earthquake loading. In the case of saturated soils, the sudden loading will cause an increase in porefluid pressure and this will reduce the effective stress. This sudden reduction in effective stress will reduce the shear strength drastically and the soil will behave like a fluid. A study by Poulos et al. (1985) finds that a phenomena similar to liquefaction can occur in dry sand or silt that are very loose. At the time of sudden loading, the escape of air from the voids are restricted and this can cause a running soil of running ground, which is similar to liquefaction. Liquefaction and related phenomena has caused tremendous amounts of damage in historical earthquakes around the world (Yanagisawa, 1983; Morales et al., 1995). During the Bhuj earthquake on 26th January 2001 (M_W - 7.7) lot of damages had been reported due to liquefaction and other ground failures (Rao and Mohanty, 2001).

The basic definition of liquefaction starts with a soil mass saturated with pore fluid, for typical geotechnical considerations with water as pore fluid, and the excessively large development of this pore water pressure inducing a critical hydraulic gradient to reduce the effective stress to zero in soils naturally or artificially deposited. But the term liquefaction always associates the presence of water in the soil mass as a primary requirement and one cannot usually think of liquefaction without the presence of water. In the initial stages of research on liquefaction of soils after the Niigata earthquake liquefaction was related with the development of excessive pore water pressure and in turn reducing the effective stress in soil to zero, but these studies were mainly focused on sandy soils, which naturally have larger permeability to successively develop the excess pore water pressure for liquefaction. As the research progressed in the liquefaction study, sandy soils with presence of non plastic fines (silt) posed a problem to the definition of liquefaction because of their higher deformation but failing to produce pore water pressure to make effective stress zero, therefore after this the definition of liquefaction is defined considering the either of two main requirements, 1) the development of excessive pore water pressure to reduce effective stress to zero or 2) to undergo a limited but extensive deformation.

In the study of liquefaction of soils effective stress (σ') is most basic parameter for definition or analysis of the liquefaction, along with strain. This concept of effective stress is defined as $\sigma' = (\sigma - u)$, where σ'- effective stress (kPa), σ - total stress (kPa), u - pore pressure (kPa).Very interesting question about effective stress concept is, what are the situations under which one can use this effective stress concept. As an answer to this question, the term u (pore pressure parameter) can be interpreted for other possibilities i.e pore fluid associated with u is not only water but any fluid filling the pore space can also be considered, it may be water, air or any other fluid. However in this document the phenomenon of liquefaction in saturated soil is emphasized.

A large number of investigations have been carried out for understanding the phenomenon of soil liquefaction in the last four decades. From these investigations it was observed that a vast majority of liquefaction occurrences were associated with sandy soils and silty sands of low plasticity.

Mechanism of Soil Liquefaction

It is necessary to understand the mechanism of soil liquefaction, where it occurs and why it occurs so often during earthquakes. Figure 5.1 clearly depicts the mechanism of soil liquefaction. Liquefaction of soil is a process by which sediments below water table temporarily loose shear strength and behaves more like a viscous liquid than as a solid. The water in the soil voids exerts pressure upon the soil particles. If the pressure is low enough, the soil stays stable. However, once the water pressure increases, it decreases the effective stress and this will reduce the shear strength of the soil. During an earthquake, the sudden load is taken by the water in the form of excess pore water pressure. If increased pore water pressure cannot be released, it will continue to build up and it will lead to reduction of effective stress to zero. In this state the soil layer losses its shear strength and it behaves like a viscous liquid and it cannot take the load coming from overlying soil or structures. Thus the upper layers of soils move down and this will lead to the collapse of structures on the ground.

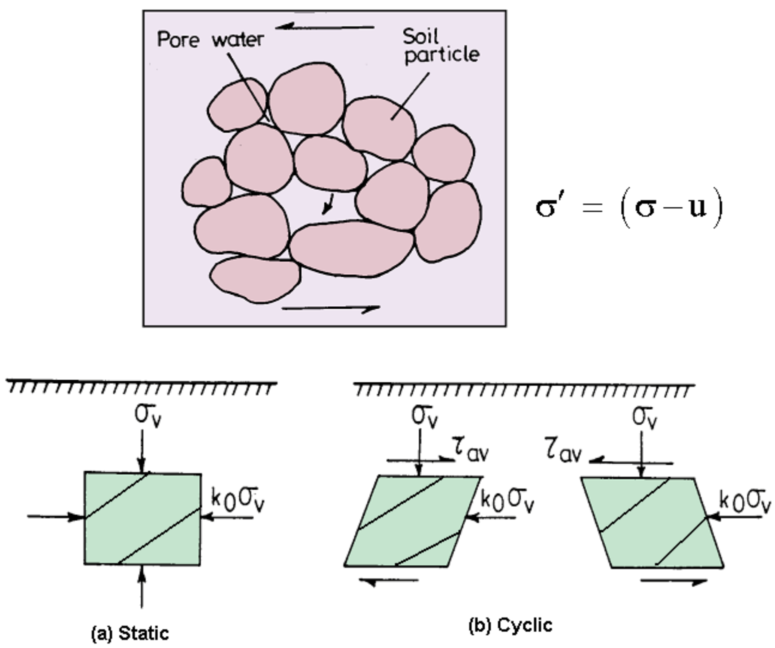

Figure 5.1: Mechanism of Soil Liquefaction

(i) Liquefaction Caused by Seepage Pressure Only: Sand Boils

If the pore water pressure in a saturated sand deposit reaches and exceeds the overburden pressure, the sand deposits will float or boil and lose its bearing capacity. This process is nothing to do with the density and volumetric contraction of sand. Therefore, it has been usually considered as a phenomenon of seepage instability. However, according to the mechanism behavior of the material, it also belongs to the category of soil liquefaction.

(ii) Liquefaction Caused by Monotonous Loading or Shearing: (Flow Slide)

The concept of critical void ratio has been suggested by Casagrande. The skeleton of loose saturated sand exhibits irreversible contraction in bulk volume under the action of monotonous loading or shearing, which will cause increase of pore water pressure and decrease of effective stress and finally brings about an unlimited flow deformation.

(iii) Liquefaction Caused by Cyclic Loading or Shearing: Cyclic Mobility

With various experimental techniques and testing apparatus it has been found out that cohesionless soil always show volumetric contraction at low shear strain level, but may dilate at higher shear strain level depending upon the relative density of soil. Therefore, under the action of cyclic shearing a saturated cohesionless soil could show liquefaction at time intervals when shear strain is low, but may regain shear resistance in time intervals when the shear strain level is higher. A sequence of such sort of intermittent liquefaction would bring about the phenomenon of cyclic mobility with limited flow deformation. If the saturated cohesionless soil was loose enough to keep contraction at high shear strain level, then it also could come out to be an unlimited flow deformation.

5.2 Evaluation of Liquefaction Susceptibility

The liquefaction susceptibility is evaluated by considering the soil properties alone, without considering the earthquake loading. If a soil at a particular site is susceptible to liquefaction then only is it prone to the liquefaction hazards. The important factors that will decide the susceptibility of soil liquefaction at the site are discussed in the following sections.

Type of Soil (Index Properties of Soil)

The phenomenon of liquefaction is most commonly observed in loose, cohesionless soils. Earlier it was believed that liquefaction was limited to sands only. Fine grained soils were considered incapable of generating the high pore pressure and coarse grained soils were considered too permeable to sustain any generated pore pressure long enough for liquefaction to occur. However, it has been found out that liquefaction takes place in gravelly soils as well (Yoshimi and Kuwabara, 1973).

Lab experiments have shown that majority of clays remain unsusceptible to liquefaction. Sensitive clays can exhibit strain softening behavior which makes it vulnerable to liquefaction. It has been deduced from some major earthquakes that for cohesive soils to liquefy they must fulfill the following conditions (Chinese criteria, Wang, 1979; Figure 5.2)

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

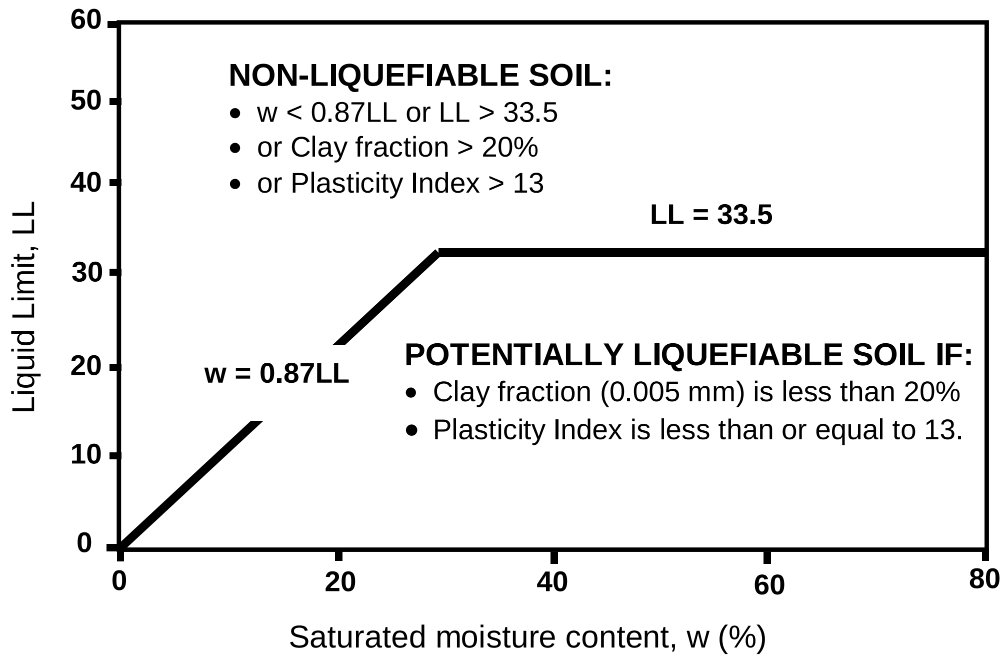

Figure 5.2: Chinese criteria for liquefaction of clayey soil

Although this is the criteria deal with the liquefaction of the clayey soil are based, Andrews and Martin (2000) re-conditioned the criteria based on the data received from the subsequent earthquakes. The tailored criteria are as follows:

Clay soils are susceptible to liquefaction if:

Fraction finer than 0.002 mm ≤ 10%

Liquid limit, LL ≤ 32%

Clay soils are unlikely to be susceptible to liquefaction if:

Fraction finer than 0.002 mm > 10%

Liquid limit, LL > 32%

For intermediate cases, i.e. for percentage of sample finer than 0.002 mm being more than 10% (or less than 10%) and LL being less than 32 (or more than 32) , the soil needs to be tested further for its liquefaction characteristics.

Shape of Soil Particles

Rounded particles are known to densify more easily than angular shaped particles. Hence, the ease of densification is directly proportional to the susceptibility of soil to liquefaction. This implies that rounded particles are more prone to liquefaction than angular shaped particles, during an earthquake.

Permeability of Soil

Permeability of soil mass leads to dissipation of pore water pressure very quickly. Hence, if this dissipation of pore water pressure occurs very quickly then the soil may not liquefy. Based on the observations, it can be concluded that presence of highly permeable soil layers can reduce liquefaction potential of adjacent soil layers. In other words, the liquefaction susceptibility of a loose sand layer will be reduced if it is surrounded by a highly permeable gravel layer.

Presence of Seismic Waves

An earthquake is characteristically a kind of vibration energy that includes impacts or shocks. The vibration it produces varies according to the characteristics of the ground through which the vibrations transmit. Earthquake on the surface of the ground is strongly dependant on the vibration characteristics of the layers underneath.

It is known that the most common cause for liquefaction is the presence of vibration/ seismic energy released during an earthquake. Based on observations, it has been concluded that the potential for liquefaction increases with the increase in seismic energy. The energy thus produced, is the cause for the cyclic shear stress that acts on the soil mass and this causes the acceleration of the soil mass in the horizontal direction. The acceleration is known as the peak acceleration exerted by earthquake at the ground surface. It has been stated that the shaking threshold required for liquefaction is a local shaking magnitude of about 5 and a peak acceleration of 0.1g.

Depth of Ground Water Table

Another criterion which makes soil susceptible to liquefaction is its degree of saturation. This clearly indicates the importance of depth of water table and it must be near the surface of the ground. Normally the soil located above the water table is unsaturated and hence the chances of liquefaction are less (Day, 2002). Day (2002) suggests that it is better to consider that the liquefaction will occur only in soil which is located under water. For those locations where the depth of water table is very deep, the liquefaction susceptibility will be low. More over if the level of the ground water table keeps on changing the liquefaction susceptibility of the soil will also fluctuate.

The details of ground water table at various locations in the country can be obtained from Central Grown Water Board (CGWB) or from the local authorities. The ground water level used for the liquefaction analysis should be the historically shallowest (neared to the ground surface) groundwater level. In the absence of any of these information the worst expected ground water level (preferably at surface level) need to be considered.

Historical Environment

Observations from earlier earthquakes in the region can offer a great amount of information regarding the liquefaction susceptibility of the soil mass at the site. There has been contradicting observations on this topic by various researchers. The soil deposit which has already subjected to seismic shaking will have a better resistance against liquefaction than a newly formed soil deposit with same density (Day, 2002). If the soil has already liquefied in the past due to an earthquake, then it can liquefy again in the future earthquakes (Youd, 1993). This observation is mainly based on the fact that at such locations the soil condition is susceptible to liquefaction and the there is possibility of occurrence of earthquakes which are strong enough to generate liquefaction. The information on the past liquefaction can be obtained in the form of maps, which provides information on the previous earthquakes (and the earthquakes expected to occur in the future at the site) which caused the liquefaction of soil.

It has also been observed that liquefaction effects are historically confined to a zone that lies within a particular distance from the tremor source. It is also obvious that the distance to which the effect of liquefaction can be expected is directly related to the magnitude of the earthquake. Ambraseys (1988) suggested a method to estimate the limiting epicentral distance for liquefaction and the magnitude of earthquake. Relationship between limiting epicentral distance of sites at which liquefaction has been observed and moment magnitude for shallow earthquakes is shown in the Figure 5.3. Please note that this figure is only helpful for estimation of regional liquefaction hazards. Deep earthquakes whose focal depths are more than 50 km have produced liquefaction at greater distances.

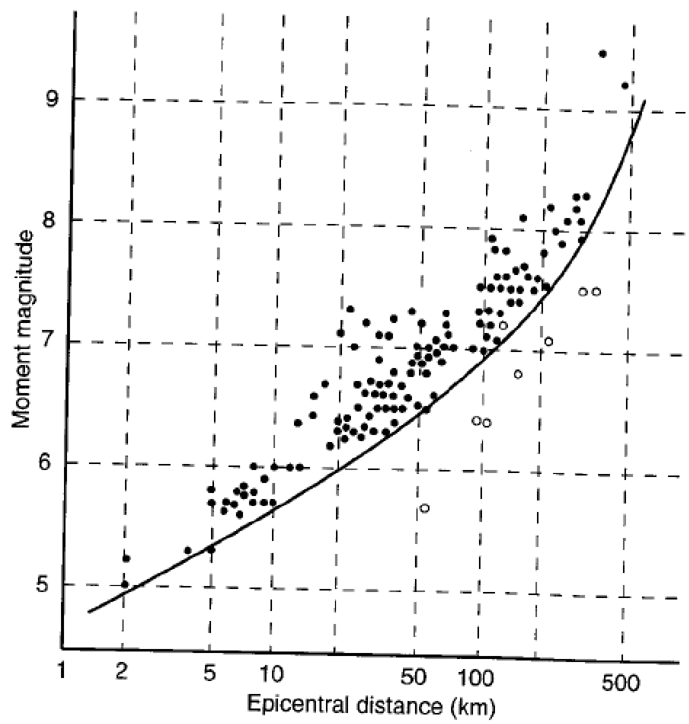

Figure 5.3: Relationship between limiting epicentral depths where liquefaction occurred, and the moment magnitudes of shallow earthquakes (After Ambraseys (1988)

Age of Soil

The age of the soil plays a pivotal role in determining the liquefaction susceptibility. The liquefaction resistance of a soil increases, with the age of the soil. New soils liquefy more easily than old soil deposits, because the old soil deposits have more stable particle arrangement due to the compression of the soil particles with time. More over the particle bonding in the case of old deposits will be more. Thus old soil deposits have a greater liquefaction resistance comparatively.

Confining Pressure

Soils at depths of more than 15 metre generally do not liquefy due to the high effective confining pressure on it. The higher the effective confining pressure, lesser is the probability of the soil to liquefy. As already mentioned, if the ground water table depth is more than 15 m from the ground surface, the soil lying below the water table generally does not liquefy. But this doesn't stop us from determining the liquefaction susceptibility of the soil layer that is at a depth of more than 15 m from the ground surface. In case of a sloping dam, the soil deposit has to be analyzed for its liquefaction susceptibility even if the depth of deposit is more than 15 m.

Relative Density of Soil

The soil layer which has a low relative density is more susceptible to liquefaction than dense soil. Loose soils densify easily during earthquake and this will cause an increase in pore water pressure which leads to the liquefaction of soil. Where as dense sands will dilate during earthquake and this will reduce the pore water pressure and hence the liquefaction susceptibility will be less..

Natural Soil Deposits in Water Bodies

Soils in lakes, rivers or oceans are highly prone to liquefaction due to their loose and segregated structure. Hence soils which are susceptible to liquefaction are formed in marine depositional environments. Tailing dams are also susceptible to liquefaction.

5.3 Evaluation of Liquefaction Potential

There are number of approaches available to evaluate the liquefaction potential of the soils such as cyclic strain approach, cyclic stress approach, energy dissipation approach, effective stress-based response analysis approach, probabilistic approach. Out of these several approaches, cyclic stress approach is most commonly used due to its simplicity and robustness to accurately model earthquake induced stresses within the ground. Because of these reasons, over the years many design charts and correlations were developed based on cyclic stress approach for the estimation of liquefaction resistance of soils through laboratory as well as in situ tests. Further discussions here in this document on the evaluation of liquefaction potential of the soils is limited to the cyclic stress approach only.

The evaluation of liquefaction potential involves two stages – (i) evaluation of earthquake loading and (ii) evaluation of soil strength against earthquake loading. The earthquake loading on soil is expressed using the term cyclic stress ratio (CSR) and the soil strength (capacity of soil) to resist liquefaction is expressed using cyclic resistance ratio (CRR). The capacity of the soil to resist liquefaction was denoted using different symbols. Seed and Harder (1990) used the symbol *CSRℓ*, Youd (1993) used the symbol *CSRL*, and Kramer (1996) used the symbol *CSRL* to denote this. But the NCEER workshop (Youd et al. 2001) recommends the use of CRR to denote the soil resistance against liquefaction.

One of the first methods to quantify the liquefaction potential of soils is the simplified procedure developed by Seed and Idriss (1971). The "simplified method" suggested by Seed and Idriss (1971) to evaluate the cyclic stress ratio (CSR) values is

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} \frac{r_d}{MSF} \tag{5.1}$$

Where a_{max} is the peak ground acceleration (at surface level), σ_{vo} and σ'_{vo} are the total and effective over burden pressure, r_d is the depth reduction factor and *MSF* is the magnitude scaling factor. The above relation was developed for an earthquake of magnitude M_w – 7.5 and if the magnitude of earthquake is different from this, it is being taken care by the *MSF*.

The term r_d , used in the evaluation of CSR, is to account for the flexibility of the soil. There are many methods available for evaluating the r_d value. The initial studies in this direction were done by Seed and Idriss (1971) and later on lots of relations were developed to predict the depth reduction factor. In a recent work, Cetin and Seed (2004) has developed a new relation to evaluate the depth reduction factor. In this relation, the value of r_d is taken as a function of depth, earthquake magnitude, ground acceleration and the soil stiffness.

Evaluation of the liquefaction resistance can be obtained from both laboratory and field tests. The former method is seldom used because of the cost and the difficulties involved in getting undisturbed samples. Laboratory testing becomes complicated due to difficulties associated with the disturbance of samples during sampling and it is almost impossible to obtain an undisturbed soil sample from the field. This problem can however be controlled, to some extent, by the use of suitable *frozen* sampling techniques, and consequent testing of these samples in a high quality cyclic simple shear or triaxial shear apparatus. The complexities of these techniques are beyond the budget and scope of most geotechnical engineering studies. More over the frozen sampling is not feasible in soils with noteworthy fines content, as the low permeability of these types of soils can lead to ice expansion causing disturbance to the soil structures instead of preventing the disturbance.

Liquefaction Resistance of Soils from Laboratory Tests

Initial understanding on liquefaction and factors affecting liquefaction has been thoroughly studied by various researchers through laboratory studies using cyclic triaxial tests (Vaid et al. , 1999; Baziar and Dobry, 1995; Lade and Yamamuro, 1997; Zlatovic and Ishihara, 1997; Thevanayagam, 1998; Yamamuro and Lade, 1998; Amini and Qi, 2000; Thevanayagam et al. , 2000; Polito and Martin, 2001; Xenaki and Athanasopoulos, 2003), cyclic simple shear tests (Vucetic et al., 1998; Sanin and Wijewickreme, 2006) and torsional shear tests (Yoshimi et al., 1977). Of late, shake table tests (Koga and Matsuo, 1990; Hesham and Ludwig, 2003; Yao et al., 2004), centrifuge model tests (Dobry et al., 1995; Zeghal et al., 1999; Famiyesin et al., 2001; Sharp et al., 2003; Pitikilas et al., 2004; Yang et al., 2004; Elgamal et al., 1996, 2005; Sharp and Adalier, 2006), cyclic multi directional simple shear tests (Wichtmann et al., 2007) and full scale blast induced liquefaction tests (Ashford et al., 2002, 2004; Rollins et al., 2005) are extensively being used to understand liquefaction behaviour of soils. Details of these different laboratory tests are presented in the following subsections.

(i) Cyclic Triaxial Test

Because of its more widespread availability and the greater simplicity in testing procedure, cyclic triaxial tests are often used to evaluate the liquefaction characteristics of saturated sands. Cyclic traixial tests are conventionally performed on isotropically consolidated specimens to simulate stress conditions on horizontal planes beneath level ground. In these tests liquefaction failure is usually defined as the point at which initial liquefaction (where pore water pressure becomes equal to cell pressure) is reached or at which some limiting cyclic strain amplitude (commonly 5%) is reached.

The cyclic triaxial test results are influenced by the non uniform stress conditions imposed by end restraints, formation of shear band and necking. Maintaining height to diameter ratio of the specimen between 2 to 2.5 and using of latex membrane reduces the effect of end restraints. Formation of necking has significant influence on the estimation of liquefaction resistance of dense specimens only, as liquefaction and development of large strains occur almost simultaneously in loose specimens. Hence, test data is discarded after formation of necking. The cyclic triaxial test results are influenced by the size of the specimen to a small extent. Wong et al. (1975) observed only 10 % variation in results between 300 *mm* and 70 *mm* diameter specimens. However, as per the recommendations of ASTM 5311, minimum diameter of the specimen is 50 mm.

Typically, five to ten specimens are tested under the different levels of cyclic axial stress amplitudes and corresponding liquefied cycle is recorded. Finally, cyclic strength curves in the form of cyclic stress ratio ($CSRL_{triaxial}$) vs the corresponding number of cycles causing liquefaction is prepared from the cyclic triaxial tests.

As the stress conditions in the field are different compared to cyclic triaxial tests, , corrections are required to the laboratory determined curves to arrive the in situ liquefaction resistance (*CRR*) of the soils. (Kramer, 1996).

$$(CRR)_{field} = 0.9\, c_r\, (CRR)_{triaxial} \tag{5.2}$$

$$\frac{\tau_{cyl}}{\sigma'_{vo}} = 0.9 c_r \frac{\sigma_d}{2\sigma_c} \tag{5.3}$$

where, *CRR* is the cyclic stress ratio. τ_{cyl} and σ'_{vo} are respectively cyclic shear stress and effective vertical stress in the field. σ_d and σ_c are respectively the applied cyclic deviator stress and effective confining pressure in the triaxial tests. c_r is the correction factor to account the differences in the application of cyclic shear stresses in the cyclic triaxial tests and cyclic simple shear tests. The factor 0.9 is to account the multidirectional shaking present in the field (Pyke et al., 1975; Seed et al., l975).

The value of c_r, depends upon the earth pressure coefficient (k_0 = 1 - sinϕ), according to the formulae suggested by Finn et al (1971) and Castro (1975).

$$c_r = \left(\frac{1+k_0}{2}\right) \quad \text{(Fin et al., 1971)} \tag{5.4}$$

$$c_r = \left(\frac{2(1+k_0)}{3\sqrt{3}}\right) \text{(Castro, 1975)} \tag{5.5}$$

(ii) Cyclic Direct Simple Shear Test

The cyclic direct simple shear test is capable of reproducing earthquake stress conditions much more accurately than the cyclic triaxial test. It is most commonly used for liquefaction testing. In this test, a short cylindrical specimen is restrained against lateral expansion by rigid boundary platens, a wire reinforced membrane or with a series of stacked rings. By applying cyclic horizontal shear stresses to the top or bottom of the specimen, the test sample is deformed in the same way as an element of soil subjected to vertically propagating S waves. In recent years, simple shear devices that allow independent control of vertical and horizontal stresses have been developed. To simulate the actual earthquake conditions, Pyke (1979) used a large-scale simple shear apparatus. It was found that cyclic strength is related to the relative density of the soil and cyclic stresses that cause liquefaction in simple shear were less than those causing liquefaction in triaxial shear.

(iii) Cyclic Torsional Shear Test

Many of the difficulties with cyclic triaxial and cyclic shear test can be overcome with cyclic torsional shear test. This is mostly used to determine stiffness and damping characteristics over a wide range of strain levels. It allow isotropic or an isotropic initial stress conditions and can impose cyclic shear stresses on horizontal planes with continuous rotation of principal axes. Dobry et al. (1995) used strain controlled cyclic torsional loading along with stress controlled axial loading of solid specimens and has proven effective for measurement of liquefaction behavior. Torsional testing of solid specimens, however produces shear stresses that range from zero along the axis of the specimen to a maximum value at the outer edge. To increase the radial uniformity of shear strains, a hollow cylindrical cyclic torsional shear apparatus were also developed. While hollow cylinder tests offer perhaps the best uniformity and control over stresses and drainage. Ishihara and Li (1972) developed a torsional triaxial shear test and conducted strain controlled tests on solid cylinders of saturated sands. These tests helped in establishing relationship between cyclic triaxial tests, cyclic simple shear tests and the torsional triaxial test.

Liquefaction Resistance of Soils from Field Tests

The difficulties and the high cost involved in the laboratory test makes the insitu-tests convenient method for evaluating liquefaction potential. The four major in-situ test methods which are considered for the liquefaction potential evaluation are given below.

1. The Standard Penetration Test (SPT)
2. The cone penetration test (CPT)
3. Measurement of in-situ shear wave velocity (Vs)
4. The Becker penetration test (BPT).

Of the above mentioned tests, the most commonly used tests are SPT and CPT.

(i) Based on Standard Penetration Test (SPT) Values

Standard penetration test is widely used as an economical, quick and convenient method for investigating the penetration resistance of non-cohesive soils. The use of SPT as a tool for evaluation of liquefaction potential began to evolve in the wake of a pair of devastating earthquakes that occurred in 1964; the 1964 Great Alaskan Earthquake (M=9.2) and 1964 Niigata Earthquake (M=7.5), both of which produced significant liquefaction related damage.

The SPT values obtained from the site investigation must be corrected to remove the errors and this has been discussed in detail in the previous chapter. The most widely used methods to evaluate the liquefaction potential based on SPT values were proposed by Seed and Idriss (1971), Seed and Peacock (1971), Seed et al. (1984 & 1985). This relationship is between corrected SPT- N values (corrected for overburden stress, instrument errors and other factors that affect the SPT testing) and intensity of cyclic loading which is expressed in the form of uniform cyclic stress ratio required to cause liquefaction (i.e. liquefaction resistance). Details of these methods are presented below.

Seed and Idriss (1971) Method

The initial approach for evaluating behavior of soils in the field during dynamic loading was developed by Seed and Idriss (1971). The procedure is referred to as the simplified procedure, and involves the comparison of the seismic stresses imparted onto a soil mass during an earthquake (Cyclic Stress Ratio, CSR) to the resistance of the soil to large magnitude strain and strength loss (Cyclic Resistance Ratio, CRR). The CSR estimation is based on the estimated ground accelerations generated by an earthquake, the stress conditions present in the soil, and correction factors accounting for the flexibility of the soil mass (Youd and Idriss 1997). Seed and Idriss developed this empirical method by combining the data on earthquake characteristics and in-situ properties of soil deposits, which is widely used all over the world for the assessment of liquefaction hazard. For earthquakes of other magnitudes, the appropriate cyclic strength is obtained by multiplying with a factor called magnitude scaling factor MSF. The factor of safety against liquefaction, F_L can then be estimated as the ratio of CSR and CRR..

Seed and Peacock (1971) Method

In the Seed and Peacock (1971) method, the average shear stress τ_{av} will be computed same as in Seed and Idriss method. Using corrected SPT 'N' value and the proposed chart by Seed and Peacock, τ_Z can be calculated at the desired depth of the soil strata. If $\tau_{av} > \tau_Z$ then soil will liquefy at that zone.

Iwasaki et al. (1982) Method

Iwasaki et al. (1982) proposed a simple geotechnical method as outlined in the Japanese Bridge Code (1991). In this method, soil liquefaction capacity factor R, is calculated along with a dynamic load L, induced in a soil element by the seismic motion. The ratio of both is defined as 'liquefaction resistance'. The soil liquefaction capacity is calculated by the three factors, which take into account the overburden pressure, grain size and fine content. In this method it is assumed that the severity of liquefaction should be proportional to the thickness of the liquefied layer, proximity of the liquefied layer to the surface, and the factor of safety of the liquefied layer.

The prediction by the liquefaction potential index is different than that made by the simplified procedure of Seed and Idriss (1971). According to Toprak and Holzer (2003), the simplified procedure predicts what will happen to a soil element whereas the index predicts the performance of the whole soil column and the consequences of liquefaction at the ground surface. Sonmez (2003) modified this method by accepting the threshold value of 1.2 of factor of safety as the limiting value between the categories of marginally liquefiable to non-liquefiable soil.

NCEER Workshops – 1996 & 1998 (Youd et al., 2001)

The NCEER workshops in 1996 and 1998 resulted in a number of suggested revisions to the SPT based procedure. Cetin et al. (2000) reexamined and expanded the SPT case history database. The data set by Seed et al. (1984) had 125 cases of liquefaction/ no liquefaction in 19 earthquakes, of which 65 cases pertain to sands with fines content ≤ 5%, 46 cases had fines content between 6 and 34% and 14 cases had ≥ 35%. Cetin et al. (2000) used their expanded data set and site response calculations for estimating CSR to develop revised relationships. Idriss and Boulanger (2004) presented a revised curve between CSR and modified SPT value based on the reexamination of the available field data.

The curves between CRR and the corrected SPT values suggested by Youd et al. (2001) are given in Figure 5.4. In this work it was noted that the CRR value will increase with fines content. A slightly modified curve between CSR value and the corrected SPT value ($N_{1,60}$) was proposed by Idriss and Boulanger (2006) (Figure 5.5). Cyclic resistance ratio (CRR) can be calculated based on the corrected "N" values using the equation proposed by Idriss and Boulanger (2006) as given below:

$$CRR = \exp\left\{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right\} \tag{5.6}$$

Probabilistic Methods

Although these methods are widely used, these doesn't accommodate conditions by earthquakes that produce very high peak ground shaking levels (CSR>2.5). The relationships do not have a formal probabilistic approach as well; i.e. the relationships do not provide the probability or uncertainty of occurrence of liquefaction. A number of researchers have developed correlations based on probabilistic methods to evaluate the liquefaction potential. One of the first to develop a relation was Liao et al. (1988) and more recently Youd and Noble (1997) (Figure 5.6), and Toprak et al. (1999) (Figure 5.7). The relationships provided by these researchers are in the form of probability contours (probability of triggering soil liquefaction). In the following figures these curves are superimposed over the Seed et. al. (1984) deterministic relationship for comparison.

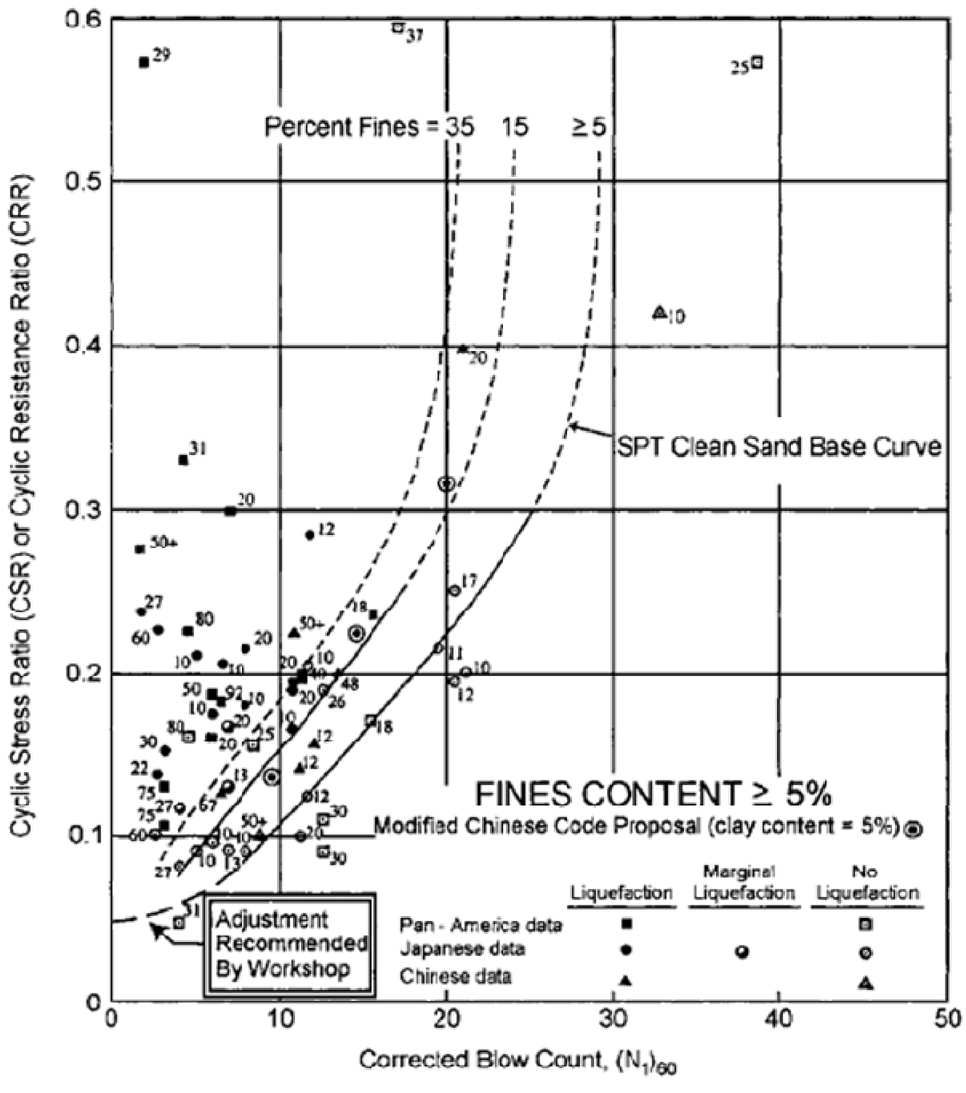

Figure 5.4: Deterministic cyclic resistance curves proposed by Youd et al. 2001

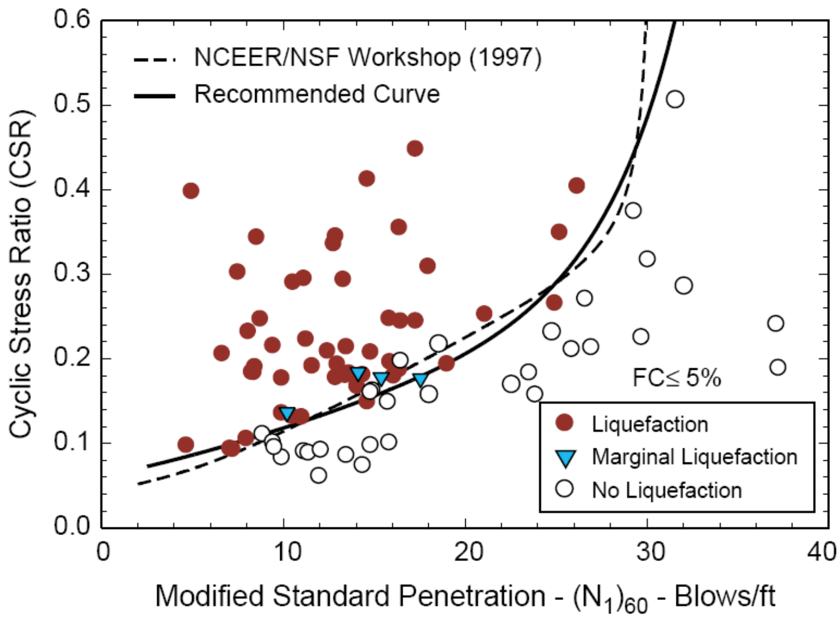

Figure 5.5: Deterministic cyclic resistance curves proposed by Idriss and Boulanger (2006)

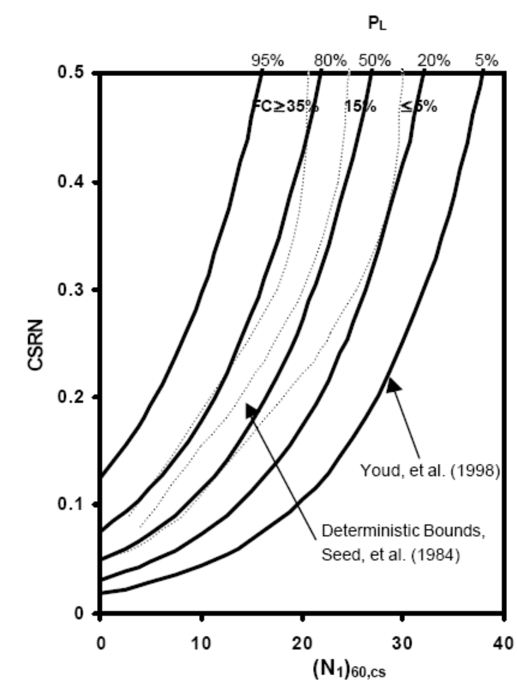

Figure 5.6: Correlation between Equivalent Uniform Cyclic Stress Ratio and SPT $N_{1,60}$ -Value (Youd and Noble, 1997)

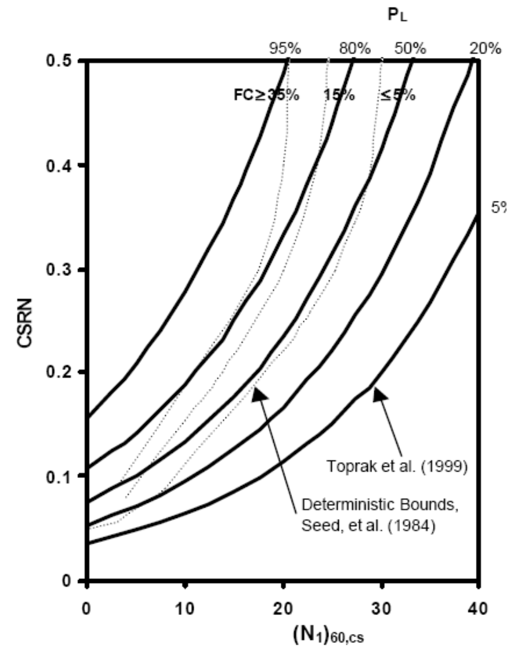

Figure 5.7: Correlation between Equivalent Uniform Cyclic Stress Ratio and SPT $N_{1,60}$ Value (Toprak et al, 1999)

The correlation by Liao et. al (1988) is based on the seismic data that were used by Seed et. al. (1984), which at that time, were of very low quality. This relationship was developed by using the binary regression of logistic models method, for probabilistic regression. The likelihood function prepared, overstated the overall variance or uncertainty of the proposed correlation. An additional inadequacy was that Liao et al. (1988) couldn't find, the effect of fines content in the soil on the regression relationship between SPT- N values and liquefaction resistance. So the relation developed was for sandy soils with less than 12 % fines.

The Youd and Noble (1997) correlation takes into account a number of field case history data points from the past earthquakes which have occurred since the earlier relationships have been developed. The correlation also excludes the limitations of the Liao et al. (1988) relationship as discussed above. This correlation is applicable to soils of varied fine contents and hence can be used for silty soils as well.

The correlation as proposed Toprak et al. (1999) also makes use of an extended field case history database, and excludes the most of the questionable data used by Liao et al. (1988). The regression method used here is also that of binary regression, and as a result the overall uncertainty is again very large. However in spite of all these shortcomings, all the above relationships discussed here are being widely used.

A recent and comprehensive work in the area of probabilistic liquefaction potential evaluation was done by Cetin et al. (2004). The probability of liquefaction can be evaluated using the procedure suggested by Cetin et al. (2004).

$$P_L = \Phi\left[-\frac{(N_1)_{60}(1+\theta_1 FC) - \theta_2 \ln CSR_{eq} - \theta_3 \ln Mw - \theta_4(\ln(\sigma'_{v0}/P_a) + \theta_5 FC + \theta_6}{\sigma_\varepsilon}\right] \quad (5.7)$$

Where P_L – Probability of liquefaction (as a fraction); Φ – standard normal cumulative distribution function; $(N_1)_{60}$ – corrected *N* value; *FC* – fineness content in percentage; CSR_{eq} – cyclic stress ratio without *MSF*; *Mw* – moment magnitude of earthquake; σ'_{v0} - effective vertical pressure at the given depth; *Pa* – atmospheric pressure (in the same unit as σ'_{v0}); $\theta_1 - \theta_6$ - regression coefficients and σ_ε - model uncertainty. The probabilistic curve suggested by Cetin et al. (2004) is shown in Figure 5.8.

The evaluation of earthquake loading in liquefaction potential evaluation requires the quantification of the uncertainties in earthquake loading. All the available methods, either probabilistic or deterministic, use a single ground acceleration and earthquake magnitude (a_{max} and M_W). The results obtained from the PSHA analysis show that several magnitudes contribute towards the ground acceleration and their percentage of contribution varies. This is clear from the seismic hazard curves given in Figure 5.9. From this figure it is clear that it will not be fair to conclude that a particular ground acceleration was produced by a certain magnitude, instead it is being contributed by different magnitudes. But the conventional liquefaction analysis methods fail to consider this aspect of earthquake loading. More over the annual frequency of occurrence of lower acceleration values will be more and that of higher acceleration values will be less. The conventional liquefaction analysis fails to account for such variations in frequency of occurrence of ground motions also.

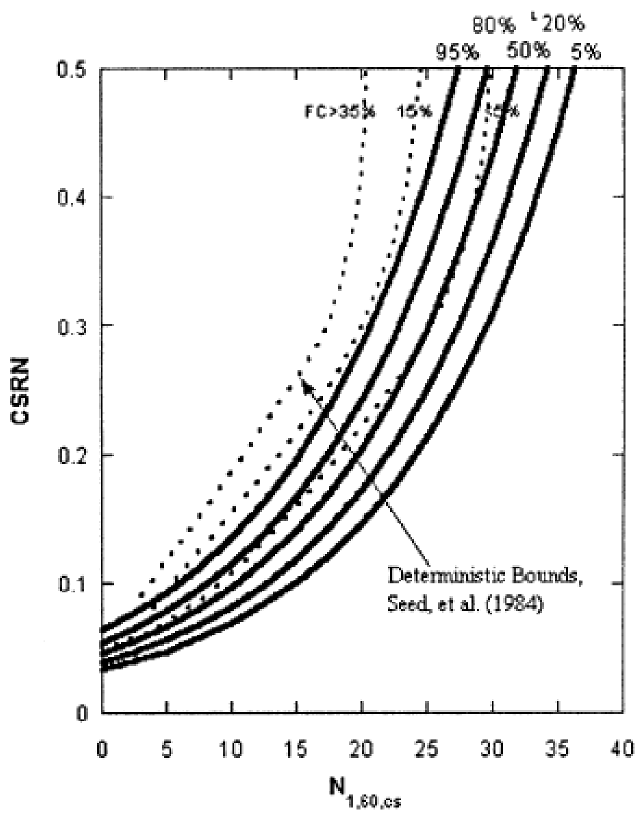

Figure 5.8: Correlation between CSR and SPT $N_{1,60,cs}$ Value (Cetin et al, 2004)

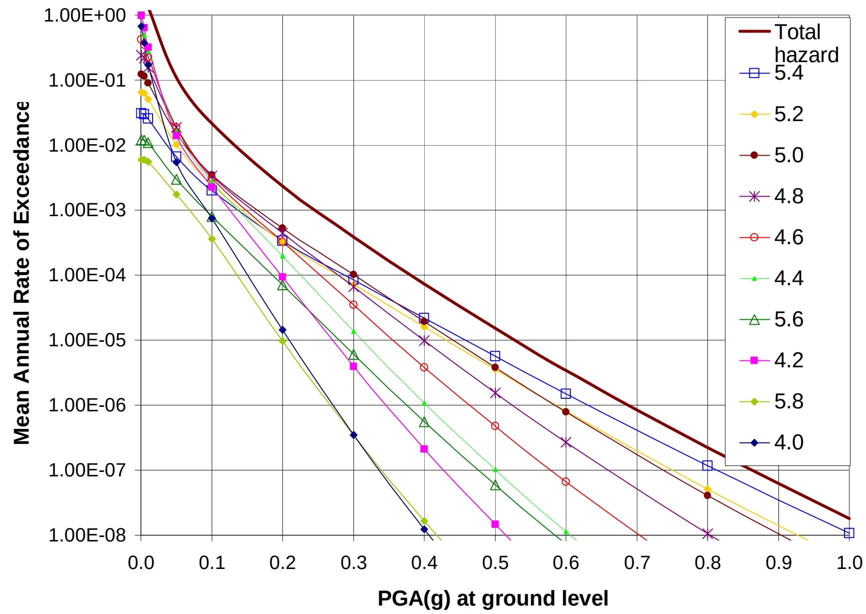

Figure 5.9: Deaggregated probabilistic seismic hazard curve with respect to magnitude (location at Bangalore)

Hence to account for these uncertainties in a better way, a probabilistic performance based approach was suggested by Kramer and Mayfield (2007). In this approach, the contributions from all magnitudes and all acceleration levels are considered. Thus, the uncertainty in the earthquake loading for the initiation of liquefaction is explicitly included in the analysis. This is achieved by discretizing the seismic hazard “space” into a large number of acceleration and magnitude bins (Figure 5.10). Thus instead of taking a single acceleration and earthquake magnitude, as in the conventional approach, it covers the entire acceleration and earthquake magnitude ranges. This method is formulated based on the probabilistic framework suggested by Kramer and Mayfield (2007).

$$\lambda_{EDP^*} = \sum_{i=1}^{N_{IM}} P\left[EDP > EDP^* \middle| \mathrm{IM} = \mathrm{im_i} \right] \Delta\lambda_{im_i} \qquad (5.8)$$

Where *EDP* – Engineering design parameter like factor of safety; EDP^* - a selected value of *EDP*; *IM* – intensity measure which is used to characterize the earthquake loading like peak ground acceleration; im_i – the discretized value of *IM*; λ_{EDP^*} - mean annual rate of exceedance of EDP^*; $\Delta\lambda_{im_i}$ - incremental mean annual rate of exceedance of the discretized value of the intensity measure *im*.

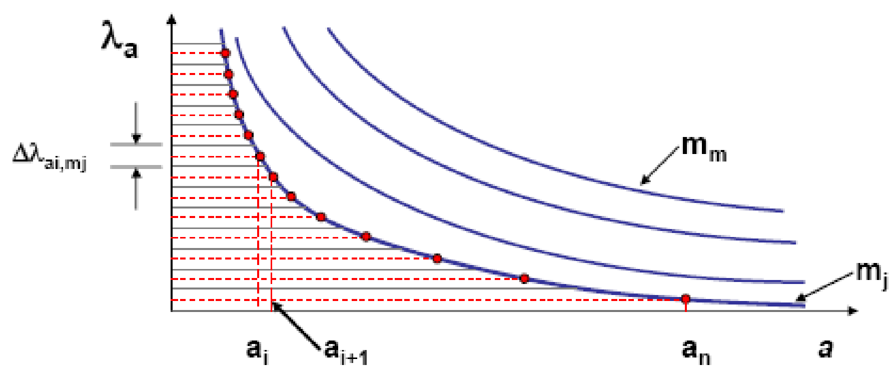

Figure 5.10: Deaggregation of seismic hazard with respect to magnitude and acceleration

The following equation can be obtained by considering the *EDP* as factor of safety and the intensity measure of ground motion as earthquake ground acceleration.

$$\Lambda_{FS_L^*} = \sum_{i=1}^{N_a} \sum_{j=1}^{N_M} P[\mathrm{FS_L} < \mathrm{FS_L^*} \mid \mathrm{a}_i, m_j] \Delta\lambda_{a_i}, m_j \tag{5.9}$$

Where $\Lambda_{FS_L^*}$ - annual rate at which factor of safety will be less than $\mathrm{FS_L^*}$; N_M - number of magnitude increments; N_a - number of peak acceleration increments; $\Delta\lambda_{a_i}, m_j$ - incremental annual frequency of exceedance for acceleration a_i and magnitude m_j (this value can be obtained from the deaggregated seismic hazard curve with respect to magnitude, similar to the one given in Figures. 5.9 & 5.10). The conditional probability in the previous equation can be written as (Kramer and Mayfield, 2007).

$$P[\mathrm{FS_L} < \mathrm{FS_L^*} \mid \mathrm{a}_i, m_j] = \Phi\left[-\frac{\begin{array}{c}(N_1)_{60}(1+\theta_1 FC) - \theta_2 \ln(CSR_{eq,i} FS_L^*) - \\ \theta_3 \ln(m_j) - \theta_4(\ln(\sigma_{v0}^{'} / P_a) + \theta_5 FC + \theta_6\end{array}}{\sigma_\varepsilon} \right] \tag{5.10}$$

Where the notations are same as that given in Eq. 5.7.

$$CSR_{eq,i} = 0.65 \frac{a_i}{g} \frac{\sigma_{vo}}{\sigma_{vo}^{'}} r_d \tag{5.11}$$

$CSR_{eq,i}$, the *CSR* value calculated without using the *MSF* for an acceleration a_i and this will be calculated for all the acceleration levels. The most widely used technique to calculate the stress reduction factor (r_d) was suggested by Seed and Idriss (1971). Further, Cetin and Seed (2004) evolved a method to evaluate the stress reduction factor as a function of depth, earthquake magnitude, ground acceleration and the average shear wave velocity of the top 12 m soil column. For a depth less than 20 m the value of r_d is given by:

$$r_d(d, Mw, a_{\max}, V_{s,12}^*) = \frac{\left[1 + \frac{-23.013 - 2.949 a_{\max} + 0.999 M_w + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341(-d + 0.0785 V_{s,12}^* + 7.586)}}\right]}{\left[1 + \frac{-23.013 - 2.949 a_{\max} + 0.999 M_w + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341(0.0785 V_{s,12}^* + 7.586)}}\right]} \pm \sigma_{\varepsilon_{r_d}} \quad (5.12)$$

Where a_{max} and M_w are the maximum acceleration (in g) and corresponding earthquake moment magnitude values; $V_{s,12}^*$ - average shear wave velocity in m/s for the top 12 m soil layer and $\sigma_{\varepsilon_{r_d}}$ is the standard deviation of model error.

The previous equation was developed for a single earthquake magnitude and acceleration. Since the discretized magnitude (m_j) and acceleration (a_i) values are considered for calculation, the above equation for calculating r_d has been modified in this study to account for all the acceleration and magnitude values. The modified equation is:

$$r_d(d, m_j, a_i, V_{s,12}^*) = \frac{\left[1 + \frac{-23.013 - 2.949 a_i + 0.999 m_j + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341(-d + 0.0785 V_{s,12}^* + 7.586)}}\right]}{\left[1 + \frac{-23.013 - 2.949 a_i + 0.999 m_j + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341(0.0785 V_{s,12}^* + 7.586)}}\right]} \quad (5.13)$$

Where a_i and m_j correspond to the discretized acceleration and magnitude values. Based on the shear wave velocity values available for the study area, the value of $V_{s,12}^*$ can be calculated using the following equation.

$$V_{s,12}^* = \frac{12}{\sum \frac{d_i}{V_{s_i}}} \quad (5.14)$$

Where V_{s_i} - Shear wave velocity at a depth d_i and $d_i \le 12$ m. In case if the $V_{s,12}^*$ values are not available the same can be assumed as 150 – 200 m/s (Cetin et al., 2004).

As an alternative to FS_L, liquefaction potential can be characterized by the SPT resistance required to prevent liquefaction, N_{req}, at a given location in the site and at a required depth and for a given return period. The probabilistic method can be applied to get the annual frequency of exceedance for $\mathrm{N}_{\mathrm{req}}^*$ (Kramer and Mayfield, 2007):

$$\lambda_{N_{req}^*} = \sum_{i=1}^{N_a} \sum_{j=1}^{N_M} P[\mathrm{N}_{\mathrm{req}} > \mathrm{N}_{\mathrm{req}}^* \mid \mathrm{a}_{\mathrm{i}}, m_j] \Delta\lambda_{a_i, m_j} \quad (5.15)$$

Where

$$\mathrm{P}[\mathrm{N}_{\mathrm{req}} > \mathrm{N}_{\mathrm{req}}^* \mid \mathrm{a}_{\mathrm{i}}, m_j] = \Phi\left[-\frac{\mathrm{N}_{\mathrm{req}}^* - \theta_2 \ln(CSR_{eq,i}) - \theta_3 \ln(m_j) - \theta_4(\ln(\sigma_{v0}' / P_a) + \theta_6}{\sigma_\varepsilon}\right] \quad (5.16)$$

The value of $\mathrm{N}_{\mathrm{req}}^*$ is the corrected N value (for energy, overburden pressure and percentage of fines) required to prevent liquefaction with an annual frequency of exceedance of $\lambda_{N_{req}^*}$. Cetin et al. (2004) has suggested two sets of values for the regression coefficients $\theta_1 - \theta_6$. Since the evaluation of SPT values will have some error, the coefficients with measurement/estimation errors are recommended for Indian conditions.

(ii) CPT Based Methods

In the previous section the discussion was centered around the correlations to evaluate the liquefaction potential based on SPT values. Due to the better accuracy and repeatability, several correlations are available for estimating the CRR values based on CPT values. The first to propose liquefaction triggering model based on CPT values was Robertson and Campanella (1985), Seed et al. (1985) and Seed and de Alba (1986). One of the most widely used correlations now a days is the one proposed by Robertson and Wride (1998) (Figure 5.11).

Robertson and Wride (1998) Method

Seed et al. (1985) developed initially a method to estimate the cyclic resistance ration (CRR) for clean sands and silty sands based on the CPT using normalized penetration resistance.

The cone penetration resistance q_c can be normalized as

$$q_{c1N} = C_Q (q_c / p_a) \quad (5.17)$$

$$C_Q = (P_a / \sigma_0')^n \quad (5.18)$$

where, C_Q is normalized factor for cone penetration resistance, P_a is the atmosphere of pressure in the same units as σ_0' and n ia an exponent that varies with soil type (= 0.5 for sands and 1 for clays) and q_c is the field cone penetration resistance at tip. The normalized penetration resistance (q_{c1N}) for silty sands is corrected to an equivalent clean sand value $(q_{c1N})_{CS}$ as

$$(q_{c1N})_{CS} = K_C\, q_{c1N} \quad (5.19)$$

where, K_C is the correction factor for grain characteristics and is defined as below by Robertson and Wride (1998).

$$K_C = 1.0 \qquad \text{for } I_C \leq 1.64 \quad (5.20)$$

$$K_C = -0.403\, I_C^4 + 5.581\, I_C^3 - 21.63\, I_C^2 + 33.75\, I_C - 17.88 \quad \text{for } I_C > 1.64 \quad (5.21)$$

If $I_C > 2.6$, the soil in this range are likely to clay rich or plastic to liquefy. I_C is the soil behavior type index and is calculated as

$$I_C = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \quad (5.22)$$

where Q is normalized penetration resistance

$$= [(q_c - \sigma_0)/P_a][P_a/\sigma_0']^n \quad (5.23)$$

$$F = [f_s / (q_c - \sigma_0)] * 100\% \quad (5.24)$$

where f_s being the sleeve friction stress

$$\mathrm{CRR}_{7.5} = 0.833\left[\frac{(q_{c1N})_{CS}}{1000}\right] + 0.05 \qquad \text{if } (q_{c1N})_{CS} < 50 \qquad (5.25)$$

$$\mathrm{CRR}_{7.5} = 93\left[\frac{(q_{c1N})_{CS}}{1000}\right]^3 + 0.08 \qquad \text{if } 50 \leq (q_{c1N})_{CS} < 160 \qquad (5.26)$$

where, $(q_{c1N})_{CS}$ is clean sand cone penetration resistance normalized to approximately 100 kPa (1atm). Then, using the equivalent clean sand normalized penetration resistance $(q_{c1N})_{CS}$, CRR can be estimated from the Figure 5.12.

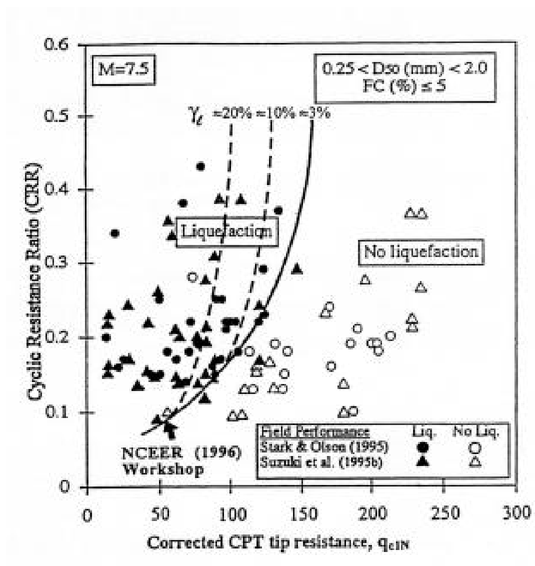

Figure 5.11: CPT- Based Liquefaction Triggering Correlation proposed by Robertson and Wride (1998) for clean, dry sands

The CPT based liquefcation correlation was reevaluated by Idriss and Boulanger (2004) using case history data compiled by Shibata and Teparaksa (1988), Kayen et al. (1992), Boulanger et al. (1997), Stark and Olson (1995), Suzuki et al. (1997) and Moss (2003).

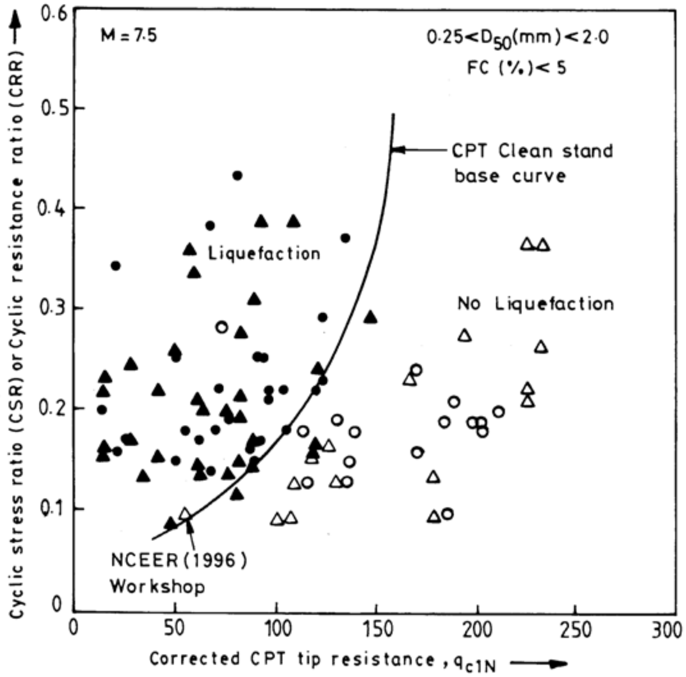

Figure 5.12: Calculation of CRR from CPT q_{c1N} (Youd et al., 2001)

Moss (2003) has provided a most comprehensive compilation of field data and associated interpretations. He used friction ratio R_f instead of the parameter I I_C, soil behavior type index and examined for the cohesion less soils with fines content greater than or equal to 35%.

Probability Method

A logistic regression approach was proposed for the liquefaction potential evaluation by Lai et al. (2006). The general form of the equation suggested by Lai et al. (2006) is:

$$P_L = \frac{1}{1 + \exp\left\{-\left[\beta_0 + \beta_1 \ln(CSR_{7.5}) + \beta_2 \sqrt{q_{C1N}}\right]\right\}} \qquad (5.27)$$

This work has proposed different values for the regression coefficients, based on the types of soils (sands, sand mixtures and silt mixtures), to evaluate the probability of liquefaction. In a more recent work, Moss et al. (2006) suggests a probabilistic approach to identify the liquefaction triggering based on CPT values. The probability of liquefaction can be obtained based on this method using the following relation.

$$\mathrm{PL} = \Phi\left[-\frac{q_{c1}^{1.045} + q_{c1}(0.110R_f) + 0.001R_f + c(1+0.850R_f) - 7.177\ln(CSR) - 0.848\ln Mw - 0.002(\ln(\sigma'_{v0}) - 20.923}{1.632}\right] \qquad (5.28)$$

Where *PL* - probability of liquefaction; q_{c1} is the normalized tip resistance; R_f - friction ratio (%); *c* - normalization exponent; *CSR* - cyclic stress ratio; *Mw* - moment magnitude of earthquake; σ'_{v0} - effective overburden pressure. In this method the earthquake loading is characterized by the ground acceleration and the earthquake magnitude; the soil resistance is characterized by normalized tip resistance, friction ratio, normalization exponent and effective overburden pressure. The probability contours for liquefaction triggering reported by Moss et al. (2006) is given in Figure 5.13.

A probabilistic performance based relation can be derived from the above equation for evaluating the factor of safety against liquefaction based on CPT values.

$$\Lambda_{FS_L^*} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P[\mathrm{FS_L} < \mathrm{FS_L^*} \mid \mathrm{a_i}, m_j] \Delta\lambda_{a_i}, m_j \tag{5.29}$$

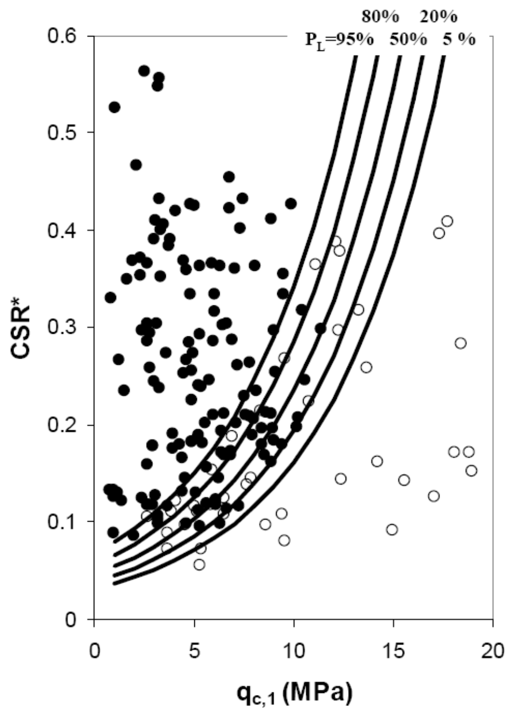

Figure 5.13: Probabilistic liquefaction triggering based on CPT values (Moss et al., 2006)

Where $\Lambda_{FS_L^*}$ is the annual rate at which factor of safety will be less than $\mathrm{FS_L^*}$; N_M is the number of magnitude increments; N_a is the number of peak acceleration increments; $\Delta\lambda_{a_i}, m_j$ is the incremental annual frequency of exceedance for acceleration a_i and magnitude m_j. The conditional probability in the above equation can be estimated using the following expression.

$$\mathrm{P}[\mathrm{FS_L} < \mathrm{FS_L^*} \mid \mathrm{a_i}, m_j] = \Phi\left[-\frac{q_{c1}^{1.045} + q_{c1}(0.110R_f) + 0.001R_f + c(1+0.850R_f) - 7.177\ln(CSR_{eqi}.FS_L^*) - 0.848\ln m_j - 0.002(\ln(\sigma'_{v0}) - 20.923}{1.632}\right] \tag{5.30}$$

In a similar way the CPT values required to prevent liquefaction for a given return period can be evaluated using the following equations.

$$\lambda_{q^*_{c1-req}} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P[\mathrm{q_{c1}} > q^*_{c1-req} \mid \mathrm{a_i}, m_j] \Delta\lambda_{a_i}, m_j \tag{5.31}$$

Where the conditional probability in the previous equation can be evaluated by:

$$P[q_{c1} > q^{*}_{c1-req} \mid a_{i}, m_{j}] = \Phi\left[-\frac{\left(q^{*}_{c1-req}\right)^{1.045} + q^{*}_{c1-req}(0.110R_{f}) + 0.001R_{f} + c(1+0.850R_{f}) - 7.177\ln(CSR_{eqi}) - 0.848\ln m_{j} - 0.002(\ln(\sigma'_{v0}) - 20.923}{1.632} \right] \quad (5.32)$$

(iii) Shear Wave Velocity (V_S) Based Methods

The use of shear wave velocity for site characterization studies has increased tremendously in the last two decades. The advantages of using Vs for evaluating liquefaction potential are (i) Vs measurements are possible in soils that are difficult to penetrate with CPT and SPT or to extract undisturbed samples, such as gravelly soils, and at sites where borings or soundings may not be permitted, (ii) Vs is a basic mechanical property of soil materials, directly related to small-strain shear modulus, and (iii) the small-strain shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil-structure interaction analyses.

There are correlations available to measure the liquefaction triggering potential based on the in-situ shear velocity (V_S) values. After considering the liquefaction case histories of 20 earthquakes and by analyzing about 193 liquefaction and non-liquefaction case histories, Andrews and Stokoe (1997) developed a relation to evaluate the cyclic resistance of soils based on V_S. This relation was modified by Andrews and Stokoe (2000) (Figure 5.14), for cemented and aged soils (> 10,000 years), by incorporating a correction factor and is given below.

$$CRR = \left\{ 0.022\left(\frac{V_{S1}}{100}\right)^{2} + 2.8\left(\frac{1}{V^{*}_{s1} - V_{S1}} - \frac{1}{V^{*}_{s1}}\right) \right\} MSF \quad (5.33)$$

Where V_{s1} – shear wave velocity corrected for overburden pressure, V^{*}_{s1} – limiting shear wave velocity value for liquefaction. The value of V^{*}_{s1} is assumed to vary linearly from 200 m/s for soils with percentage of fines 35% to 215 m/s for soils with percentage of fines less than 5 %. The model uncertainty in the above relation was evaluated using reliability methods by Juang et al. (2005). A probabilistic neural network (PNN) was adopted by Goh (2002) to evaluate the liquefaction susceptibility based on V_S.

Shear wave velocity (Vs), of late, is increasingly being used as a field index of liquefaction resistance as an alternative to SPT based approach (Andrus and Stokoe, 2000). However it should be noted that the procedure is not yet as standardized as that of SPT or CPT and the case histories in the database of validation against field studies are relatively less. In addition to this, it should be noted that there are different ways of finding Vs values (based on in situ tests) and the variation of values are up to 15% (Nazarian and Stokoe, 1984; Dennis et al., 1988; Stokoe et al., 1988; Stokoe et al., 1999; Brown et al., 2002). Three concerns arise when using Vs for liquefaction-resistance evaluations: (i) seismic wave velocity measurements are made at small strains, whereas pore-water pressure buildup and the onset of liquefaction are medium- to high-strain phenomena, (ii) seismic testing does not provide samples for classification of soils and identification of non-liquefiable soft clay-rich soils, and (iii) thin, low Vs strata may not be detected if the measurement interval is too large. Therefore the preferred practice is to drill sufficient boreholes and conduct in situ tests to detect and delineate thin liquefiable strata.

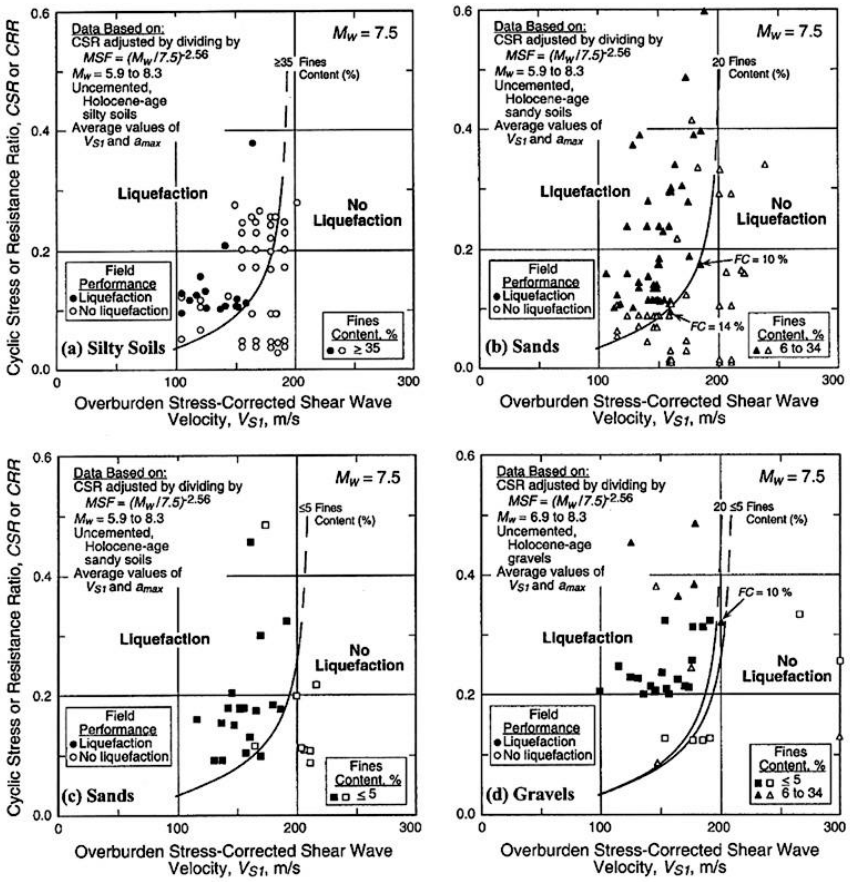

Figure 5.14: V_S based Liquefaction Triggering Correlation as proposed by Andrus and Stokoe (2000)

Liquefaction Potential Index

The methods described in the previous sections measures the liquefaction potential at specified depths (the depth selected in the present study was 3 m and 6m). However by using the Liquefaction Potential Index (LPI, proposed by Iwasaki et al., 1982) this short coming can be overcome. In this method it is assumed that the severity of liquefaction should be proportional to the thickness of the liquefied layer, proximity of the liquefied layer to the surface, and the factor of safety of the liquefied layer.

The LPI, which was proposed by Iwasaki et al. (1982), is given by:

$$LPI = \int_0^{20} F_Z W(z) dz \tag{5.34}$$

Where z – depth below ground surface in metres; $W(z)$ – weightage factor for depth ($W(z) = 10 - 0.5z$ for $z \le 20$ and $W(z) = 0$ for $z > 20$) and F_Z – function of factor of safety against liquefaction. By assuming a threshold value of FS as 1.2, the value of F_Z can be calculated as (Sonmez, 2003):

$$\begin{aligned} F_Z &= 0 \; for \; FS \ge 1.2 \\ F_Z &= 1 - FS \; for \; FS < 0.95 \\ F_Z &= 2 \times 10^6 e^{-18.427 FS} \; for \; 1.2 > FS > 0.95 \end{aligned} \tag{5.35}$$

The prediction by the liquefaction potential index is different than that made by the simplified procedure of Seed and Idriss (1971). According to Toprak and Holzer (2003), the simplified procedure predicts what will happen to a soil element whereas LPI considers the performance of the whole soil column and the consequences of liquefaction at the ground surface.

5.4 Static Liquefaction and Concepts of Steady State Behavior

Based on the discussions in the previous sections, the readers should not come to the conclusion that only cyclic loading will cause liquefaction. Instead static loading can also cause liquefaction in case of loose saturated sand. Under appropriate static loading condition, saturated sand mass may suffer a rapid drop in its shear resistance to a minimum value, the steady state strength, which remains constant with further strain. Thus, there are two important aspects in the undrained behaviour of sands: the triggering condition of the collapse and the undrained residual shear strength.

Published data reports the existence of three types of stress-strain behaviour of saturated sands in undrained static triaxial tests namely (i) strain hardening behaviour, (ii) steady state behaviour, and (iii) quasi-steady state behaviour (see, Figure 5.15). The stain hardening behaviour usually observed in dense sands, the shear resistance always increases with axial strain. In the steady state behaviour, the shear stress of a sand sample rapidly decreases once its collapse is triggered (liquefaction), and then remains constant with further axial strain. The constant shear stress at the steady state is considered as the undrained residual strength, or the so-called steady state strength of the sample. For a given sand, the steady state strength is only a function of void ratio, independent of sample preparation methods, stress path, strain rate and drainage conditions. Furthermore, the steady state strengths from undrained static loading as well as from undrained cyclic loading are the same for a given void ratio (Castro *et al.*, 1982; Alarcon-Guzman *et al.*, 1988).

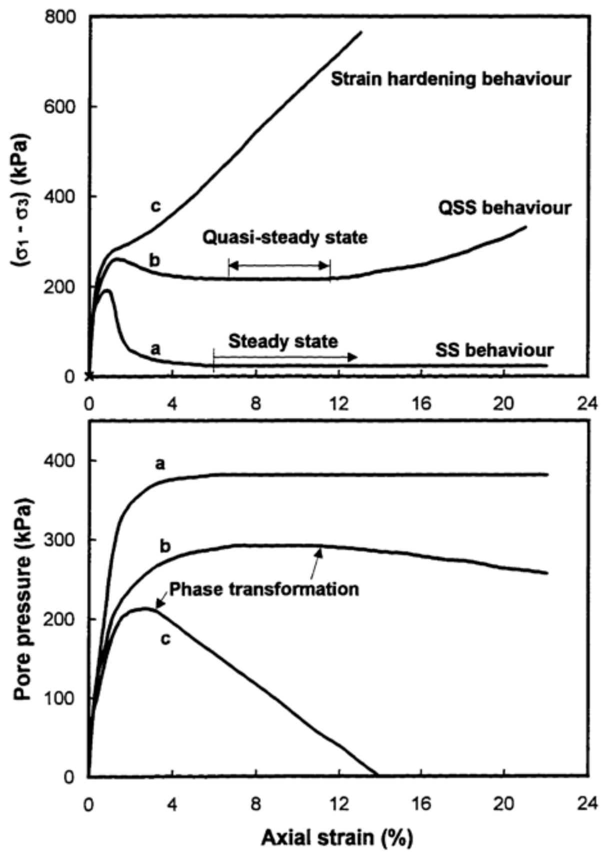

Figure 5.15: Undrained behaviour of sands in triaxial tests (Zhang, 1997)

In the quasi-steady state behaviour, the shear stress rapidly decreases after a peak value and then drops to a minimum value; however it remains at this minimum value for a short period and then increases with increasing strain (limited liquefaction). In other words no unique ultimate shear resistance exists in this type of behaviour (Zhang, 1997). In this behaviour, the pore pressures initially increases to a peak value, thereafter it starts decreasing and it continuously decreases until the end of the test. Ishihara et al. (1975) called this transition state from increase to decrease in pore pressure as "phase transformation" and the stress conditions at the starting of this stage is called "initial phase transformation point". Alarcon-Guzman et al. (1988) called the transitory steady state in the stress-strain curve as the "quasi-steady state". Unlike the effective stresses at steady state which are only a function of the void ratio, the effective stresses at the quasi-steady state appear also to be affected by some other factors such as initial confining pressure, initial fabric of a sample, etc. (Ishihara, 1993; McRoberts and Sladen, 1992).

5.5 Liquefaction Induced Ground Subsidence and Lateral Displacements

Soon after the earthquake the soil gets densified and this will appear at the ground surface in the form of settlement. The settlement of the dry sand will be complete once the earthquake is over. However in the case of saturated soils, the duration of settlement will be more and it will continue till the excess pore pressure, generated due to the earthquake, is fully dissipated. This dissipation of pore water pressure depends on density of sand, induced shear strain and the excess pore water pressure developed during the earthquake. The empirical curves developed by Tokimatsu and Seed (1987) for prediction of ground subsidence after liquefaction is given in Figure 5.16. These curves give the volumetric strain values as a function of corrected SPT and CSR values.

Ishihara and Yoshimine (1992) have developed experimental charts (Figure 5.17) for prediction of liquefaction induced settlement. In this work the settlement was modeled as a function of factor of safety against liquefaction, corrected SPT and CPT values, and the CSR value.

The lateral displacement caused due to liquefaction has been a major liquefaction related topic due to its huge impact on the buried structures. Lots of research work is going on in this topic due to its impact on the buried under ground structures. Severe damages were observed at Turnagain Height area of Anchorage (Alaska earthquake, 1964), Niigata (1964), Noshiro City (During Japan Sea (Nipponkai-chubu) earthquake of 1983), Dagupan City (Luzon earthquake, 1990), Kobe (1995) etc. Bartlett and Youd (1992, 1995) proposed a relation to estimate the lateral ground displacement. This relation was further modified by incorporating more data by Youd et. al. (2002) and the relation suggested by them are given below. The relation for free face conditions is:

$$\log D_H = -16.713 + 1.532M - 1.406\log R^* - 0.012R + 0.592\log W + 0.540\log T_{15} \\ +3.413\log(100 - F_{15}) - 0.795\log(D50_{15} + 0.1\,mm) \qquad (5.36)$$

Where D_H - estimated lateral ground displacement (in m); M - moment magnitude of the earthquake; R - nearest horizontal or map distance from the site to the seismic source (in km); R^* - modified source to site distance (in km); T_{15} - cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than 15 (in m); F_{15} - average fines content (fraction of sediment sample passing a No. 200 sieve) for granular materials included within T_{15}; $D50_{15}$ - average mean grain size for granular materials within T_{15} (in mm); S – ground slope (in percentage); W - free-face ratio defined as the height (H) of the free face divided by the distance (L) from the base of the free face to the point in question (in percentage).

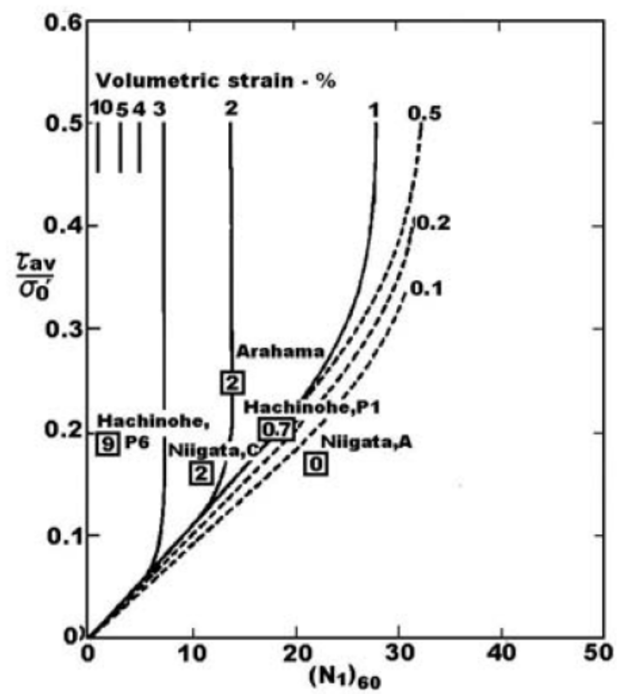

Figure 5.16: Liquefaction induced volume contraction (Tokimatsu and Seed, 1987)

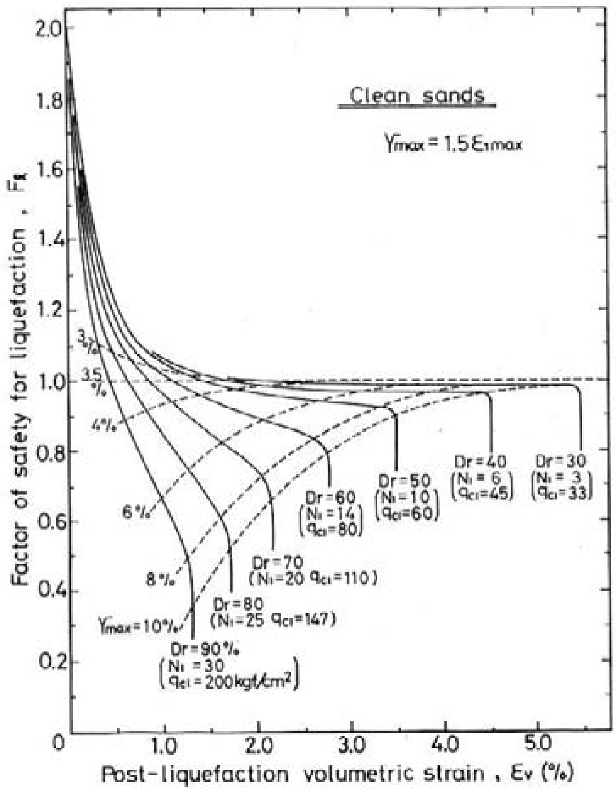

Figure 5.17: Post liquefaction volumetric strain (Ishihara and Yoshimine, 1992)

Liquefaction induced Lateral Displacement

The relation for gently sloping ground is:

$$\begin{aligned}\log D_H &= -16.213 + 1.532M - 1.406\log R^* - 0.012R + 0.338\log S + 0.540\log T_{15} \\ &+3.413\log(100 - F_{15}) - 0.795\log(D50_{15} + 0.1\,mm)\end{aligned} \quad (5.37)$$

Where S – ground slope parameter and other variables are same as that in the previous equation.

5.6 Liquefaction of Cohesive Soils

The liquefaction susceptibility of clayey soils are less and it was thought that clays will never liquefy. The results of the cyclic shear test conducted with constant strain amplitude (Ohara and Matsuda, 1988) have shown that the excess pore pressure increases with loading cycles. Till now there is no evidence of liquefaction of clay during earthquakes. The undrained cyclic triaxial test in undisturbed clay samples (Adachi et al., 1995) has also shown an increase in strain amplitude with increase in number of loading cycles. However the behavior of the clay was similar to that of dense sands and this study has also concluded that the liquefaction susceptibility of clays will be very less.

In the case of clayey sand, when the clay content is less, the clay bridges the sands (Osipov et al., 2005). During the cyclic loading this bridge between the sand will get destroyed and the soil will liquefy. However when the clay content is more, the clay will fill the intergranular voids and in this state it will offer high resistance against liquefaction. during seismic loading it has been observed that there has been softening of clay due to the cyclic loading. In some cases the amplitude of strain exceeds 5 – 10% (Towhata, 2008) and this satisfies the criteria for liquefaction of sand. However, since the laboratory tests have shown that the behavior of clay under cyclic loading is more similar to dense sand and the effective stress was never reduced to zero, this softening of clay cannot be termed as liquefaction (Towhata, 2008).

5.7 Evaluation of Factor of Safety Against Liquefaction (Case study for Bangalore)

Factor of safety against liquefaction can be evaluated from the methods based on deterministic as well as probabilistic analysis. In either of these methods, liquefaction expected to occur during the earthquake if the cyclic stress ratio caused by the earthquake is higher than the cyclic resistance ratio of in situ soil. The factor of safety against liquefaction is defined as follows:

$$FS = \left(\frac{CRR}{CSR}\right) \tag{5.38}$$

The CRR values can be obtained from both laboratory and field tests as explained in detail in the section 5.3. Procedures for the calculation of factor safety based on the deterministic and probability approaches are illustrated using a case study in the following subsections.

Deterministic Approach

One of the common methods used is based on the simplified procedure proposed by Youd et al. (2001). This method was used for the liquefaction potential evaluation of Bangalore. In this method, the earthquake induced loading is expressed in terms of cyclic shear stress (CSR) and this is compared with the liquefaction resistance (CRR) of the soil. Steps involved in the calculation of liquefaction hazard are shown in Figure 5.18.

Peak Ground Acceleration

Estimation of factor of safety against liquefaction of soil layer requires the ground level peak acceleration due to an earthquake. About 58 MASW tests have been carried to study the amplification potential of Bangalore. Surface level peak ground acceleration (PGA) values are estimated from the one dimensional ground response analysis carried using soil data and estimated peak values of bed rock motions. These PGA values were used to estimate the cyclic stress ratio (CSR).

The earthquake loading can be evaluated 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:

$$CSR = 0.65\frac{a_{\max}}{g}\frac{\sigma_{vo}}{\sigma'_{vo}}\frac{r_d}{MSF} \tag{5.39}$$

In this equation, $a_{\max}$ is peak ground acceleration, g is the acceleration due gravity, σ_{vo} and σ_{vo}' are the total and effective vertical stresses and r_d is the stress reduction factor. For the calculation of stress reduction factor can be done suing the method suggested by Cetin and Seed (2004) which is explained in the previous section.

Magnitude Scaling Factor (MSF)

The magnitude-scaling factor used in the present study is the revised Idriss scaling factor for the magnitude other than 7.5 is given below:

$$MSF = \left[\frac{10^{2.24}}{M_W{}^{2.56}}\right] \tag{5.40}$$

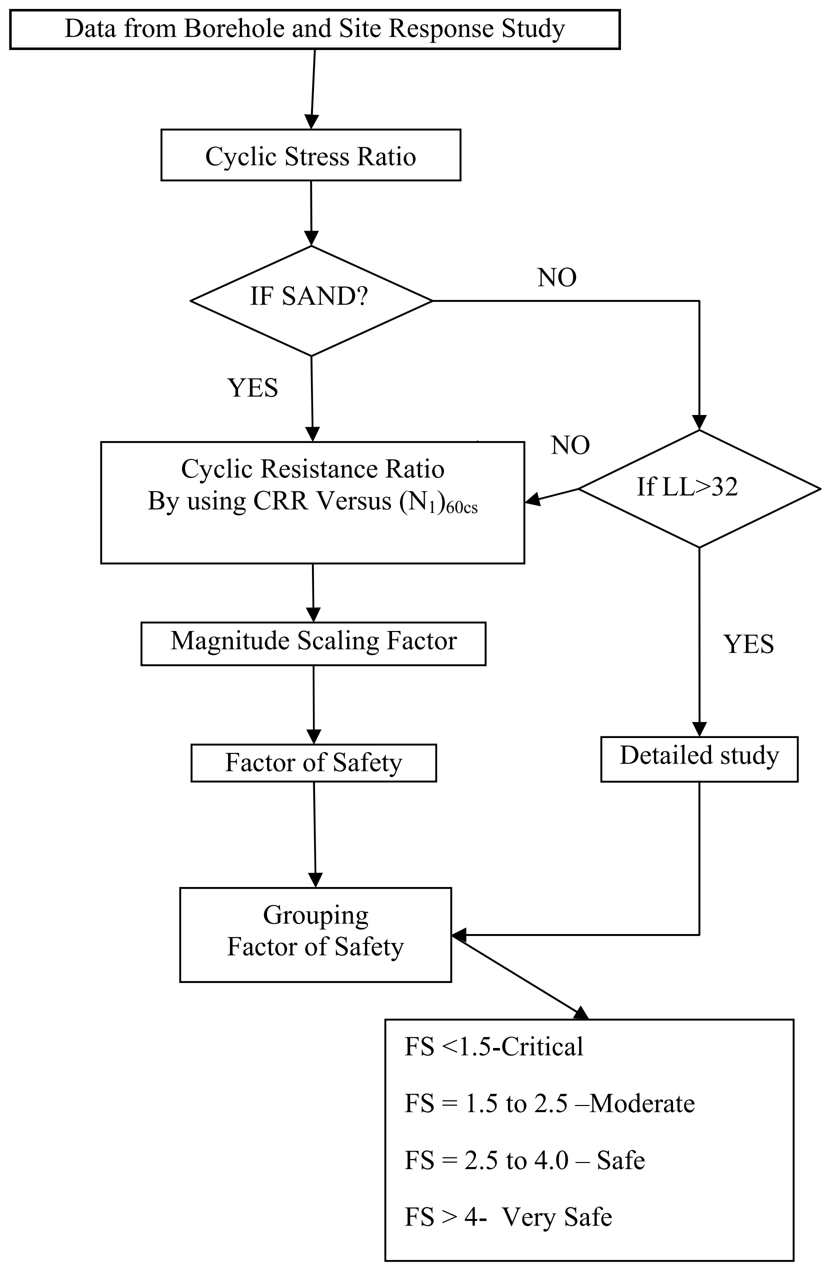

Figure 5.18: Flow chart for liquefaction hazard assessment

Factor of Safety Calculation

After applying necessary corrections to SPT 'N' values an excel spread sheet was developed for the factor of safety calculations. The factor of safety for each layer of soil was calculated by considering corresponding "N" values (FS = CRR/CSR). Typical liquefaction analysis is shown in Table 5.1. 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)/ 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 the strength loss in plastic silts or clays during cyclic or seismic loading.

Table 5.1: Typical liquefaction analysis for a borehole (M_W – 5.1, Anbazhagan, 2007)

Depth (m)	Corrected N value $(N_1)_{60cs}$	σ_{vo} kN/m^2	σ_{vo}' kN/m^2	r_d	CSR	FC %	Liquid Limit	CRR	FS
1.50	4	30.00	30.00	0.99	0.08	46.2	0	0.08	1.00
3.20	3	64.00	47.32	0.98	0.11	40.9	0	0.08	0.72
4.20	21	84.00	74.19	0.97	0.09	53.3	26	0.22	2.44
5.20	20	104.00	94.19	0.96	0.09	53.1	31	0.21	2.33
7.00	44	140.00	122.34	0.95	0.09	57.1	25	19.54	NL
8.50	102	170.00	155.29	0.93	0.09	59.2	27	NL	NL

NL – Non Liquefiable

Typical Calculations for Evaluation of *CRR* based on *V*s for Delhi*:*

Shear wave velocity (V_s), of late, is increasingly being used as a field index of liquefaction resistance as an alternative to *SPT* based approach (Andrus and Stokoe, 2000). Andrus and Stokoe (1997) proposed the following equation for evaluating *CRR* based on V_s.

$$CRR = 0.022\left(\frac{V_{s1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{s1*} - V_{s1}} - \frac{1}{V_{s1*}}\right) \tag{5.41}$$

Where V_{s1} is overburden stress corrected shear wave velocity and calculated as follows:

$$V_{s1} = V_s\left(\frac{P_a}{\sigma'_{v0}}\right)^{0.25} \tag{5.42}$$

P_a is atmospheric pressure approximated by 100 *kPa*

σ'_{v0} is the initial effective overburden stress in *kPa*. $V_{s1}{}^*$ is the limiting upper value of V_{s1} for liquefaction occurrence. Values of $V_{s1}{}^*$ were assumed to vary linearly from 200 *m/s* for soils with fines content of 35% to 215 *m/s* for soils with fines content of 5% or less. *a, b* are curve fitting parameters and considered as 0.022 and 2.8 respectively.

Evaluation of Estimation of Zone of Liquefaction

The variation of CSR with depth using simplified approach as well as from wave propagation analysis is shown in Figure 5.19. It is worth noting that CSR estimated from simplified approach is on the conservative side. The zone of liquefaction estimated based on *SPT,* shear wave velocity and laboratory tests at sand and silty sand sites is also shown in Figure 5.19.

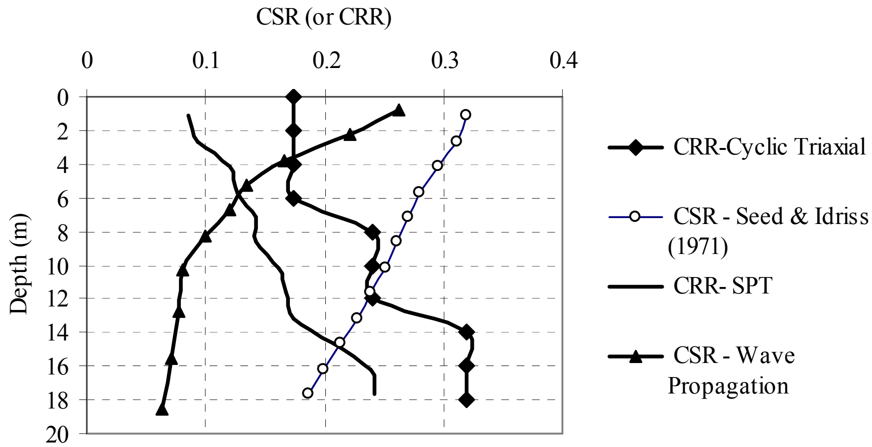

Figure 5.19: Zone of Liquefaction at typical sand site in Delhi

Liquefaction Hazard Map

The liquefaction hazard maps were prepared by considering the minimum factor of safety obtained from each bore log. Figure 5.20 shows the spatial variation of factor of safety against liquefaction (FS) for Bangalore city (magnitude – 5.1). The factor of safety against liquefaction locations is more than unity except in northwestern part (area in and around Kurubahalli) and southern part (in and around Venkatapura, Jakasandra, Sri Narashimaraj colony, Basaveshwara nagar and Kethamaranahalli) of Bangalore.

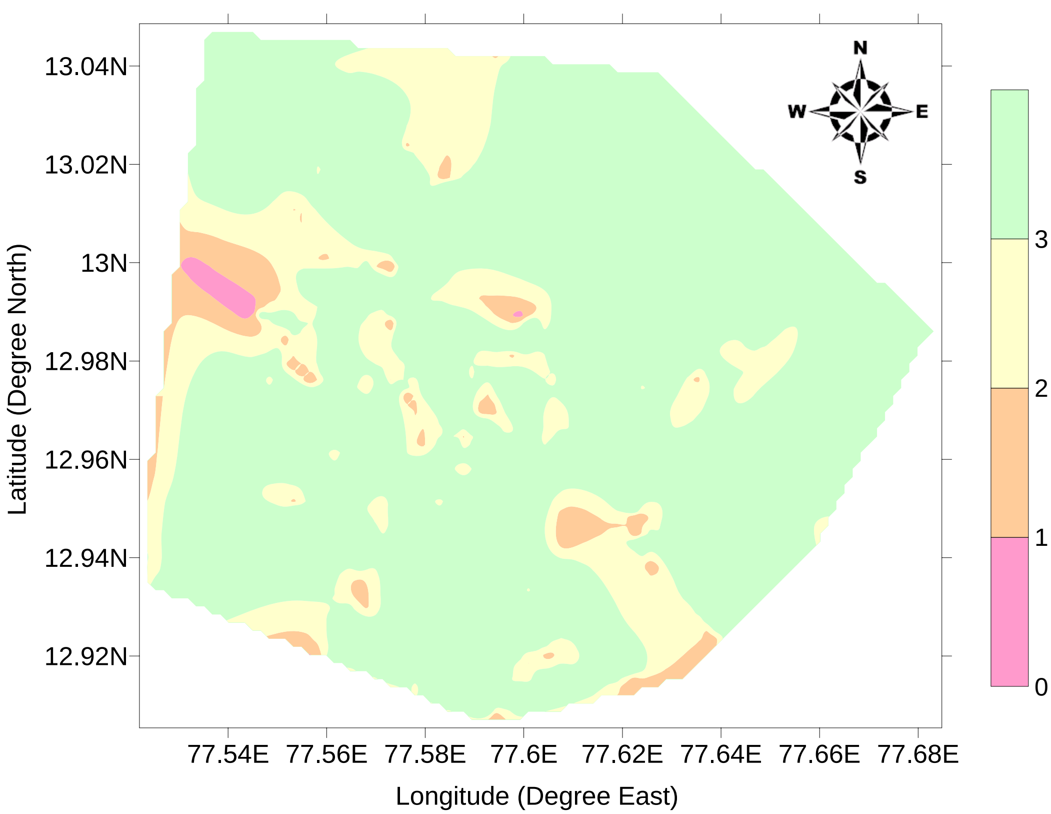

Figure 5.20: Distribution of factor of safety against liquefaction

Probabilistic Performance Based Approach

The variation of factor of safety against liquefaction and the annual frequency of exceedance for Bangalore were evaluated based on the probabilistic performance based methods given in the previous sections (Kramer and Mayfield, 2007) for a depths of 3 m from the ground surface. The range of ground acceleration values (deaggregated with respect to magnitude) were divided into very small intervals at lower acceleration ranges and as the acceleration values increases, the intervals were also increased. Such a division was adopted to account for the variation in annual frequency of exceedance more accurately, because at lower acceleration values the variation of annual frequency of exceedance will be more and at higher acceleration values it will be less.

Curves showing the variation of factor of safety against mean annual rate of exceedance at a depth of 3 m for four different locations in Bangalore are shown in Figure 5.21. The main advantages of these curves are, the factor of safety against liquefaction for any given return period can be obtained directly. In a similar way, the curves between the corrected *N* values required to prevent liquefaction and annual frequency of exceedance are presented in Figure 5.22. The $(N_1)_{60,cs}$ required to prevent liquefaction for any specified return period can be obtained from these curves. If the corrected *N* value (obtained from the site investigation) at the site is less than the value obtained from this curve, then the site is vulnerable to liquefaction for that return period and vice versa. Since the variation of PGA values at the four locations in Bangalore were very less, the curves in Figure 5.22 are packed together.

The contour curves showing the spatial variation of factors of safety against liquefaction and $(N_1)_{60,cs}$ required to prevent liquefaction for a return period of 475 years at a depth of 3 m are shown in Figures. 5.23 & 5.24. The factor of safety range of 0 – 1 indicate that these locations are highly vulnerable to liquefaction; the range of 1 – 2 are moderately vulnerable and the factor of safety higher than 2 indicate that these locations are safe against liquefaction.

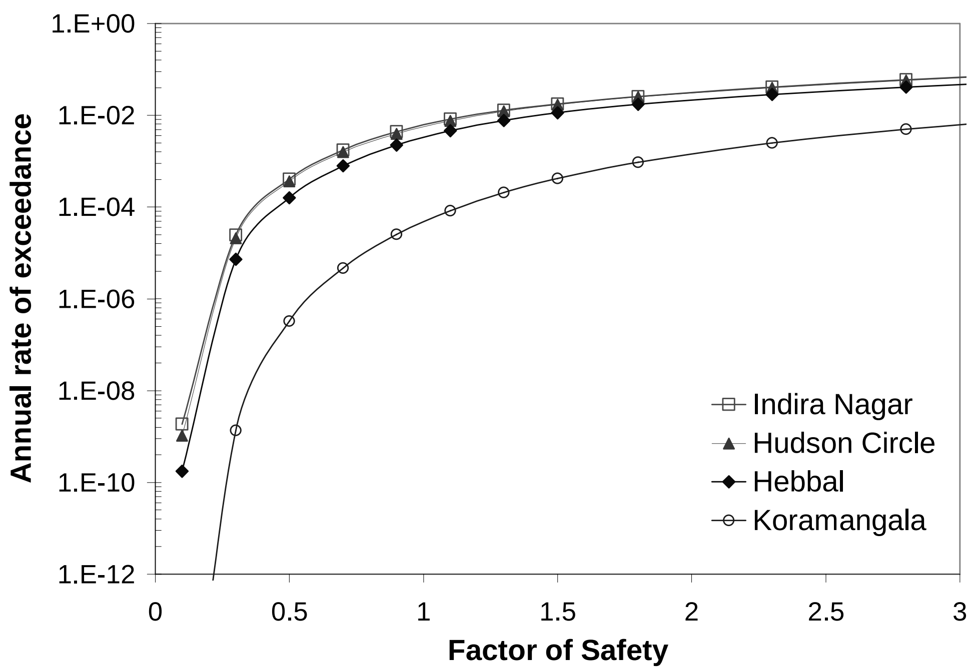

Figure 5.21: Factor of safety against liquefaction Vs annual rate of exceedance of liquefaction based on SPT data (at 3m depth)

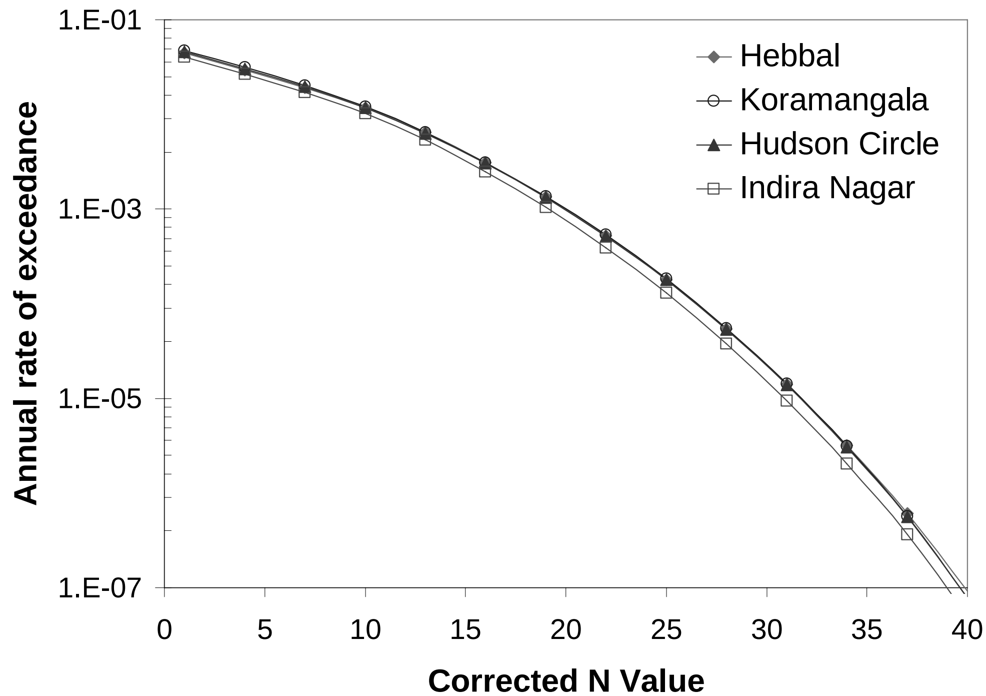

Figure 5.22: $(N_1)_{60,cs}$ required to prevent liquefaction Vs annual rate of exceedance of liquefaction based on SPT data (at 3m depth)

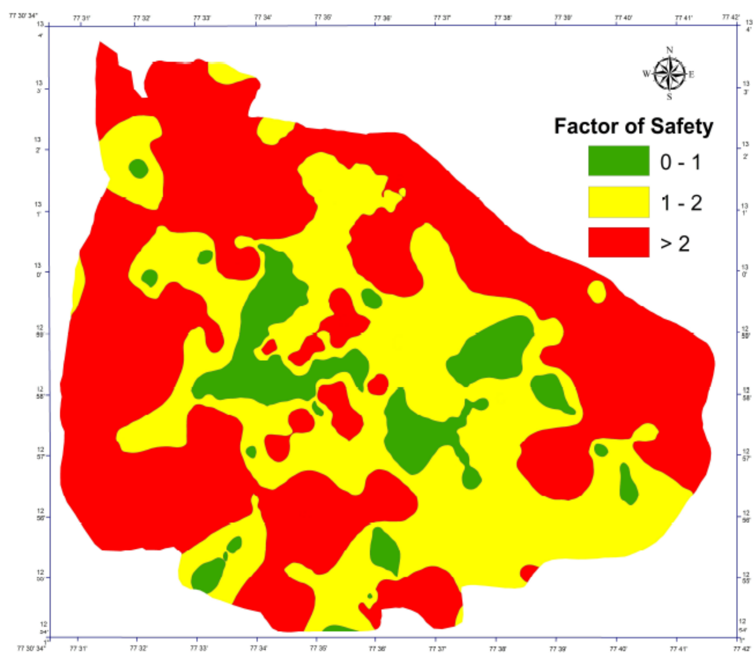

Figure 5.23: Factor of safety against liquefaction for a return period of 475 years at 3 m depth

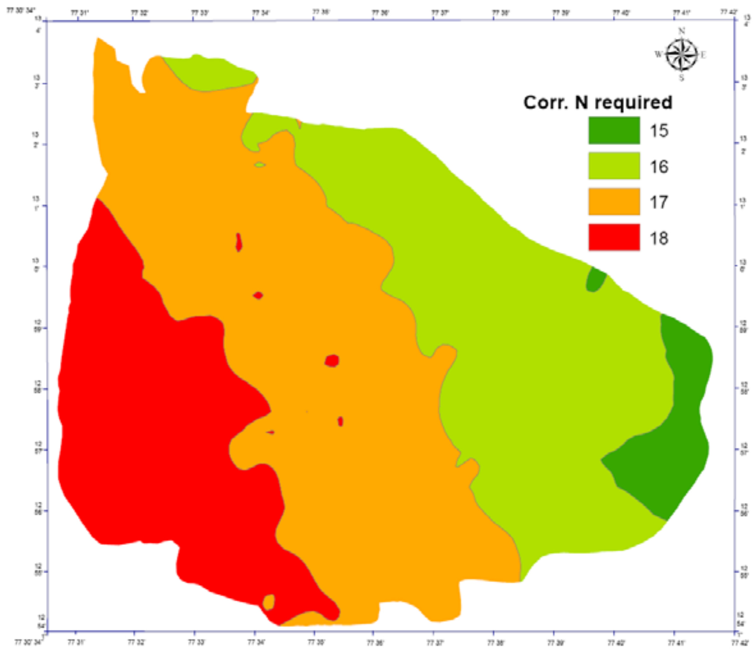

Figure 5.24: $(N_1)_{60,CS}$ required to prevent liquefaction for a return period of 475 years at 3 m depth

5.8 Liquefaction Mitigation Steps

If the liquefaction potential evaluation of a particular site gives an unacceptable level of risk, then mitigation steps need to be undertaken to reduce the risk. These remediation measures need to be done on a case to case basis, depending on the local site conditions. The effect of liquefaction on structures will also differ, it may cause excessive differential settlement or in some cases the buried structures may become buoyant. Even though it is beyond the scope of this document, some of the general measures are discussed in this section.

Some of the mitigation steps include removal and replacement, dewatering, in-situ soil improvement, modification of site geometry, increasing the relative density of soils (for sandy soils), drains for dissipation of excess pore pressure. A summary of methods which can be used of the liquefaction potential mitigation is given in table 5.2: (Ferritto, 1997).

Table 5.2: Liquefaction Remediation Measures

Method	Principle	Suitable soil condition / Types	Maximum effective depth of treatment	Relative cost
1) Blasting	Shock waves and vibrations cause limited liquefaction , displacement, remolding, and	Saturated, clean sands; partly saturated sands and silts after flooding.	>40 m	Low

Method	Principle	Suitable soil condition / Types	Maximum effective depth of treatment	Relative cost
	settlement to higher density.			
2) Vibratory Probe a) Terraprobe b) Vibrorods c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3- 4 m depth); > 30 m sometimes; vibrowing 40 m.	Moderate
3) Vibrocompaction a) Vibrofloat b) Vibro-Composer system.	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	> 20 m	Low to moderate
4) Compaction Piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	> 20 m	Moderate to high
5) Heavy tamping (dynamic compaction)	Repeated application of high intensity impacts at surface.	Cohesionless soils best, other types can also be improved.	30 m (possibly deeper)	Low
6) Displacement (compaction grout)	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
7) Surcharge/ buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining	Can be placed on any soil surface.	Dependent on size of surcharge/ buttress	Moderate if vertical drains are used

Method	Principle	Suitable soil condition / Types	Maximum effective depth of treatment	Relative cost
	pressures in the foundation.			
8) Drains a) Gravel b) Sand c) Wick d) Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/ wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand > 30 m; depth limited by vibratory equipment; wick, > 45 m	Moderate to high
9) Particulate grouting	Penetration grouting- fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout Methods
10) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser.	Unlimited	High
11) Pressure injected lime	Penetration grouting- fill soil pores with lime	Medium to coarse sand and gravel.	Unlimited	Low
12) Electrokinetic injection	Stabilizing chemical moved into and fills soil pores by electro-osmosis or colloids in to pores by electro-phoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive
13) Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
14) Mix-in-place	Lime, cement or asphalt introduced	Sand, silts, clays,	> 20 m (60 m	High

Method	Principle	Suitable soil condition / Types	Maximum effective depth of treatment	Relative cost
piles and walls	through rotating auger or special in-place mixer.	all soft or loose inorganic soils.	obtained in Japan)	
15) In-situ vitrification	Melts soils in place to create an obsidian-like vitreous material.	All soils and rock.	>30 m	Moderate
16) Vibro-replacement stone and sand columns a) Grouted b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	> 30 m (limited by vibratory equipment)	Moderate
17) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high

5.9 Conclusions

This chapter discusses in detail about the liquefaction phenomena and the methods for assessing the liquefaction susceptibility and potential. Various procedures based on insitu tests and laboratory tests are explained in detail. In addition to this two cases studies of Bangalore, based on two different methods are also discussed here. This will be helpful in understanding the evaluating the liquefaction potential in a better way. The important recommendations are

- It will be better to assess the liquefaction potential using the SPT or CPT data.
- It will be better to calculate the factor of safety against liquefaction (FS_L) based on probabilistic performance based approach.
- If the required computational facilities are not available then the FS_L calculations can be done using an Excel spread sheet.
- The method suggested by Idriss and Boulanger (2006) can be used for the evaluation of CRR based on SPT values.
- The methods suggested by Robertson and Wride (1998) can be used for the evaluation of CRR values using CPT data.

6 Summary

India has experienced most disastrous earthquakes in the recent past. The earthquakes can neither be predicted nor be prevented. However, the severity of the damages can be minimized by proper land use planning and safe construction practices. Seismic microzonation provides the required information for the effective mitigation of seismic hazards. As discussed in detailed in the current report, local site effects play major role on the severity of damages observed during earthquake shaking. Local soils modify the bed rock motions significantly depending upon their geotechnical characteristics, local topography, and hydrogeological site conditions. Earthquake associated disasters such as occurrence of liquefaction, lateral spreading, sand boils, landslides and etc, are also depend upon the geotechnical characteristics of the local soils. Hence, it is very important part of the seismic microzonation studies to carry proper geotechnical/geophysical investigations. Since, there are no clear guidelines, earlier seismic microzonation studies in India followed different approaches. The procedures are often modified or simplified and these are not adequately reflected in the final product as discussed. Here in this report, review of various techniques available for geotechnical/geophysical investigations for the purpose of seismic microzonation is provided. Detailed methodologies to characterize local soils are provided in this report. General guidelines for carrying out these investigations are also laid out.

In this concluding chapter, overall summary of various important topics discussed in the earlier chapters is presented. In addition, summary of guidelines for carrying out geotechnical and geophysical investigations for the purpose of Seismic Microzonation are presented. Figure 6.1 shows the flow chart of the implementation methodology for seismic microzonation.

6.1 Overview of the Topics Covered in the Report

Planning of site investigations (geophysical and geotechnical investigations) depends upon the scope of the seismic microzonation (eg. its scale, procedures adopted, etc) to be carried. Hence, proper planning of site investigation often require to understand the underlying principles of seismic microzonation and is methodology. A General overview of the methodology for seismic microzonation is presented in the Chapter 1. This chapter also offers brief description of various components of the microzonation. It outlines the required geotechnical and geophysical investigations and their methodologies. It also lists the major cities which have high seismic risk (seismic zone III and above) with population over half a million.

Chapter 2 presents general procedure for carrying out site investigations for the purpose of seismic microzonation. It also provides guidelines for the proper planning of subsurface explorations. It discusses importance of the geotechnical and geophysical methods for microzonation and their suitability. Recommendations are given for selecting appropriate scales for microzonation studies based on vulnerability and population density of a region, in this chapter. Suitable scales for data collection using geotechnical and geophysical tests are also suggested here, based on the vulnerability of a region and heterogeneity in the soil subsurface.

Chapter 3 presents different in situ tests available for site characterization and the for the evaluation of site effects. The details of different low strain and high strain tests are also listed in this chapter. It also provides procedures for the evaluation of dynamic properties from the laboratory as well as in situ tests.

Local site conditions play significant role on the amplification of seismic waves and the resulted earthquake disasters. In depth discussions are made in Chapter 4, on the various local site conditions which influence the ground shaking. Different available methods for the assessing the local site conditions are presented in the Chapter 4. Various codal provisions for site classifications are also discussed here.

Various parameters which influence the liquefaction susceptibility are presented in the Chapter 5. It provides detailed procedures for evaluation of liquefaction potential based on field and laboratory tests. This chapter also explains various liquefaction phenomenon associated with earthquakes and the methods to quantify.

The details of different geotechnical and geophysical studies carried for seismic microzonation in the India are presented in Annexure II. Problems associated with the current practices are highlighted here. The main issues in microzonation works and the codal provisions related to geotechnical and geophysical testing are also explained in this.

6.2 Guidelines for Geotechnical/Geophysical Investigations for the purpose of Seismic Microzonation

First step in the seismic microzonation is to select a grade/level of seismic microzonation to be carried out. Selection of appropriate grade of the study depends upon the extent of the area of investigation, vulnerability and population density of a region. Guidelines for the selection of appropriate grade of seismic microzonation for a given region are outlined in the Section 1.3.1.

Next step in the process of seismic microzonation of a region is to either obtain expected bed rock motions or estimate ground motion parameters. Many ground response analysis methods require time history of the ground motion at bed rock. This required information will be generated/provided by the PSHA group.

After obtaining the bed rock motions at a particular place, the next important step is to characterize the local soils and then carry ground response analysis to assess local site effects including site amplifications, liquefaction hazards, landslides, tsunami etc. The details of the landslide and Tsunami hazard assessment are not covered in the document. Different steps involved in this process are listed below

- First step is to select appropriate scale for geotechnical data collection. The selection of scale depends upon the heterogeneity in the subsurface and the level of microzonation adopted. Guidelines for the selection of appropriate scale are provided in the Sections 1.3.1 and 2.3.
- Next step is to carry geotechnical/geophysical investigations to characterize the local soils. General procedures for planning of geotechnical/geophysical investigations are described in the Section 2.6. General guidelines are also given over the required number of boreholes, their locations and other tests in the Section 2.8.
- The routine suggested field tests for site characterization are SPT, CPT and MASW. As carrying out geotechnical field tests are expensive, it is not possible to entirely depend upon geotechnical tests (SPT, CPT) as discussed in the Section 2.4.1 & 2.4.2. Hence, it is required to depend upon geophysical tests(MASW) and use geotechnical tests for the purpose of cross verification.

- Carry at least one MASW test in each of the grid. Carryout geotechnical tests such as SPT or CPT, by drilling boreholes in selective grids as discussed in the Section 2.8. Collect the soil samples for laboratory testing. Verify the subsurface profiles with the results of MASW tests.
- It is also advised to carry out any of the direct geophysical tests such as crosshole, or up or down hole, or SCPT to obtain direct shear wave measurements of the subsurface layers, at the above selected grid points. Use these results for cross checking of the results from MASW, an indirect method based on surface wave measurements. Highlight the differences. If the variations are very large, it is suggested to investigate the reasons for such high disparities.
- Establish the topography and subsurface geomorphology (bedrock depth and geometry of subsurface soil layers) of the region from the geotechnical and geophysical tests, along with water table conditions.
- The recommended method for site characterization is based on V_s^{30} values. If the shear wave velocity values are not available, then these values need to be obtained based on the correlations between SPT, CPT with shear wave velocities.
- The site classes need to delineate based on the NEHRP site classification scheme.
- The list of standard correlations suggested by different researchers for different parts of the world and for different soil types are presented in table 4.6. The appropriate correlation (based on the geographical region and soil type) should be used.
- It is advisable to cross check the values obtained from these correlations with some of the data available for some other parts of the study region.
- Low strain soil tests such as resonant column or bender element tests discussed in Chapter – 3, may be conducted for evaluating the dynamic properties such as damping and shear modulus.
- The insitu shear modulus (maximum shear modulus) can be obtained either from the field tests or from low strain laboratory tests. Alternatively, it can also be assessed using index properties.
- Determination of strain dependent modulus (modulus reduction curve) and damping parameters, characterizing the soil properties of any site under consideration may also be carried out using laboratory tests. Resonant column or bender element tests are required to determine dynamic properties at low strains, and cyclic triaxial tests are required to obtain these dynamic properties at large strains. Details of the procedures available for the evaluation of the dynamic properties are discussed in the Section 3.6. In case of non-availability of the suitable testing apparatus, expertise personal or funding, suitable modulus reduction and damping curves can be assumed from the standard curves based on the soil characterization. Details are discussed in the Section 4.6.4.
- Based on the dynamic characterization and the estimated input bed rock motions (will be given by PSHA group), ground response analysis is to be carried out to obtain surface ground motions and then response spectra using the methods explained in the Sections 3.8 and 4.6. SHAKE is the most commonly used computer program for carrying one dimensional site specific ground response analysis for level grounds. However, selection of one, two or three dimensional

ground models for the analysis depends upon the terrain and heterogeneity of the subsurface soils.

- Site classification can be done based on V_s^{30} values and using estimated values of bed rock motion parameters at a site (will be given by PSHA group), the peak ground accelerations and Sa (response spectra) values can be obtained based on various codal provisions as explained in the Chapter 3.
- After assessing the soil/site response to the given input bed rock motions, it is required to carry further investigations to assess liquefaction potential at the site of interest (see Chapter 5).
- Field tests conducted to obtain dynamic properties (site characteristics), can be utilized to assess liquefaction resistance of the soils as explained in the Section 5.3.2. Instead of this additional laboratory tests may be conducted to evaluate liquefaction resistance as detailed in the Section 5.3.1.

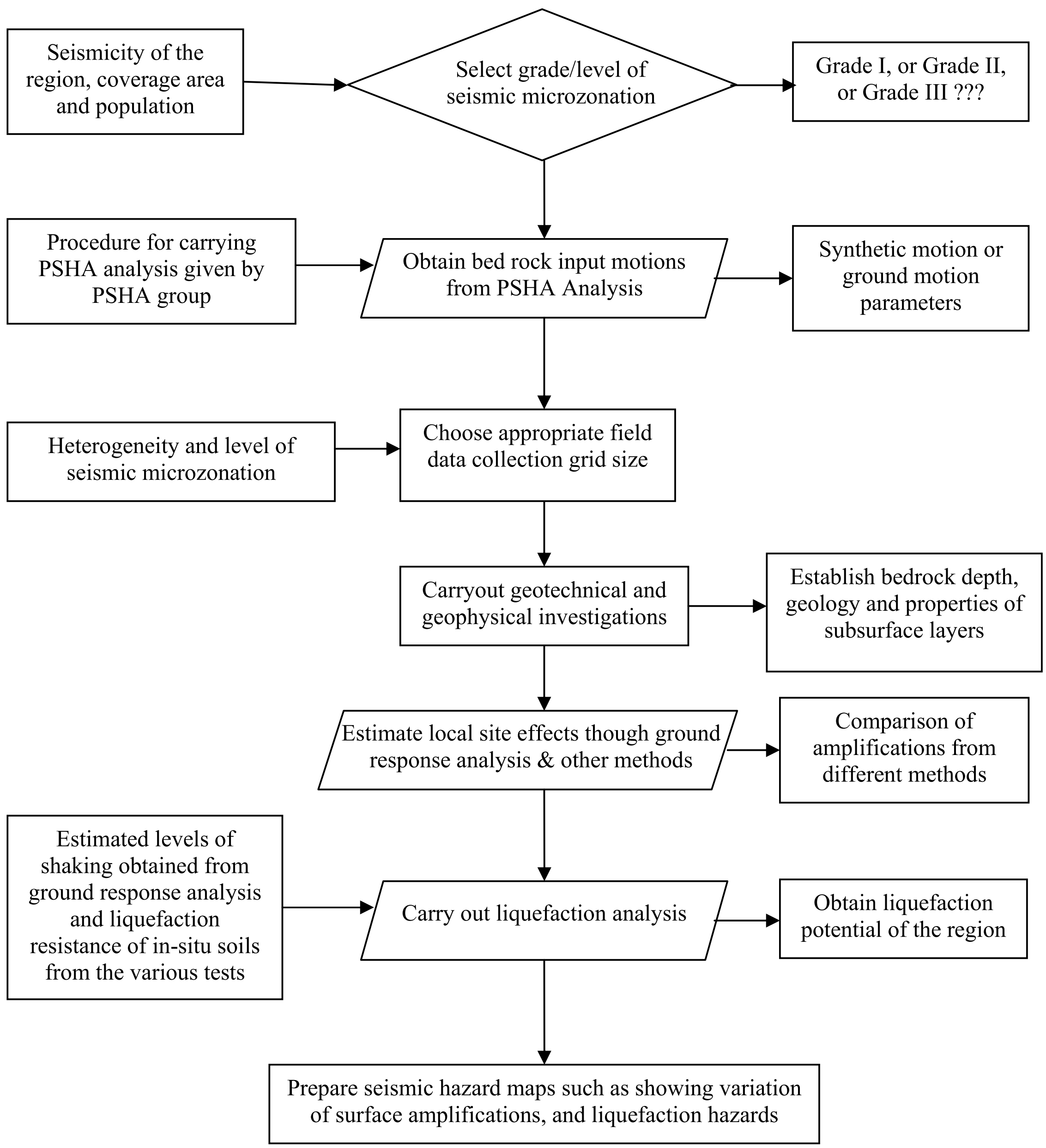

Figure 6.1: Flow chart for the implementation of methodology for seismic microzonation

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Annexure I
Relevant IS code numbers

A1.1 Relevant IS codes

IS 14243 : Part 1 : 1995 Guidelines for selection and development of site for building in hill areas Part 1 Microzonation of urban centres.

IS 15681 : 2006 Geological exploration by geophysical method (Seismic Refraction)

IS 1893 : Part 1 : 2002 Criteria for Earthquake Resistant Design of Structures - Part 1 : General Provisions and Buildings

IS 2720 (Part XI) : 1993 (Reaffirmed 2002) Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure

IS 2720 (Part XII) – 1981 Methods of test for soils; Determination of Shear Strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure.

IS 2810 : 1979 Glossary of terms relating to soil dynamics

IS 4968 : Part 1 : 1976 Method for subsurface sounding for soils: Part 1 Dynamic method using 50 mm cone without bentonite slurry

IS 4968 : Part 3 : 1976 Method for subsurface sounding for soils: Part 3 Static cone penetration test

IS 4968 : Part II : 1976 Method for Subsurface Sounding for Soils - Part II : Dynamic Method Using Cone and Bentonite Slurry

IS 5249 : 1992 Method of test for determination of dynamic properties of soil

Annexure II

Case Studies of Geotechnical/Geophysical Investigations Carried in India for Seismic Microzonation

A2.1 Introduction

In the last three decades, large earthquakes have caused massive loss of lives and extensive physical destruction throughout the world (Armenia, 1988; Iran, 1990; US, 1994; Japan, 1995; Turkey, 1999; Taiwan, 1999, India 2001, Sumatra 2004, Pakistan, 2005). India has been facing threat from earthquakes since ancient times. In India, the recent destructive earthquakes are Killari (1993), Jabalpur (1997), Bhuj (2001), Sumatra (2004) and Indo-Pakistan (2005). Very preliminary process of reducing the effects of earthquake is by assessing the hazard itself. Seismic microzonation is first and fore most step to minimize seismic related damages and loss of lives.

Microzonation has generally been recognized as the most accepted tool in seismic hazard assessment and risk evaluation and it is defined as the zonation with respect to ground motion characteristics taking into account source and site conditions [TC4-ISSMGE, 1999]. The role of geological and geotechnical data is becoming very important in the microzonation in particular to plan city urban infrastructure, which can recognize, control and prevent geological hazards (Bell et al., 1987; Legget, 1987; Hake, 1987; Rau, 1994; Dai et al., 1994, 2001; Van Rooy and Stiff, 2001).

As part of the national level microzonation programme, Department of Science and Technology, Govt. of India has initiated microzonation of 63 cities in India (Bansal and Vandana, 2007). 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, North East India, Guwahati, Bangalore, 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.

Soil is highly heterogeneous and also very complex. There are very large number of geotechnical and geophysical tests available for 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 the many factors starting from the purpose and the scope of the study, availability of resources (equipment and expertise personal), type of analysis to be carried, etc. Geotechnical and geophysical investigations require for seismic microzonation are different from the conventional site investigations. The extent of 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 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 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 leads to improper estimate of the seismic hazards. In this chapter, case studies of geotechnical and geophysical investigations carried for microzonation of various cities in India are discussed, highlighting the problems in the investigations.

A2.2 Current State of Macrozonation, Microzonation and Codal Provisions in India, and the Importance of Consideration of local site conditions

First seismic macrozonation map was prepared by Geological Survey of India (GSI) in 1935 after the 1934 Bihar-Nepal earthquake. After Indian Standards Institution (ISI) established in 1947 and renamed as Bureau of Indian Standards in 1986, it is the agency responsible for producing and publishing the seismic hazard maps and codes in India. BIS published the seismic macrozonation map of India in 1962 (BIS-1893, 1962) based on earthquake epicenters and the isoseismal map published by the GSI in 1935. The macro zoning was reviewed and included additional information of geology and tectonic features. Revised macrozonation map was published in 1966 (BIS-1893, 1966), this map developed with more weightage to the tectonic maps, that delineated the fault systems (Tandon, 1992; Walling and Mohanty, 2009). Soon after 1967 Koyna earthquake, the zonation map underwent major revision in 1970 by including both geological and geophysical data (BIS-1893, 1970). In 1984 the regions of different seismogenic potential were identified on the basis of past earthquakes and the regional tectonic features, which was incorporated in zonation map and published in 1984 (BIS-1893, 1984). None of these macrozonation maps have considered the seismic hazard at different locations the return periods of the required design seismic coefficients.

The occurrence of the 1993 Latur earthquake (m_b - 6.3), 1997 Jabalpur (m_b - 6.0) and 2001 Bhuj (M_W - 7.6) in the lower seismic zone (according to BIS-1893(1984)) raised questions on the validity of the seismic zonation map of Peninsular India (Walling and Mohanty, 2009). This has emphasized the need to revise the seismic macrozonation map and in 2002. The new macrozonation map was developed with four seismic zones based on peak ground acceleration (PGA) values ranging from 0.1 g to 0.4 g (Walling and Mohanty, 2009). Besides the PGA, the expected maximum intensity of shaking in each zone was also estimated based on the Comprehensive Intensity Scale (CIS-64) (BIS-1893, 2002).

Modifications in the zonation map of India with the occurrence of significant earthquakes suggest the assessment of hazard on a regional scale is not consistent with the local variation (Walling and Mohanty, 2009). Many individuals have attempted to produce national and regional hazard maps based on seismic microzonation considering local variations (Tandon, 1956; Krishna, 1959; Guha, 1962; Gubin, 1968 and 1971; Basu and Nigam, 1978; Kaila and Rao, 1979; Khattri et al., 1984; Bhatia et al. 1999; Parvez et al. 2003; Sitharam and Anbazhagan, 2007; Jaiswal, and Sinha, 2007; Anbazhagan et al., 2009; Vipin et al., 2009). However, the seismic microzonation maps and regional hazard maps produced by researchers represent only the status of the present knowledge about the seismicity and the various methodologies available. Many of the studies lack in proper planning and sufficient number of geotechnical and geophysical investigations for site characterization, even though they are very important to bring out the effects of local variations in the subsurface site conditions. Hence, the current microzonation maps have to be used with caution for specific projects understanding their limitations.

Figure A2.1 shows the Earthquake distribution of the Indian subcontinent and location of cities with seismic microzonation study.

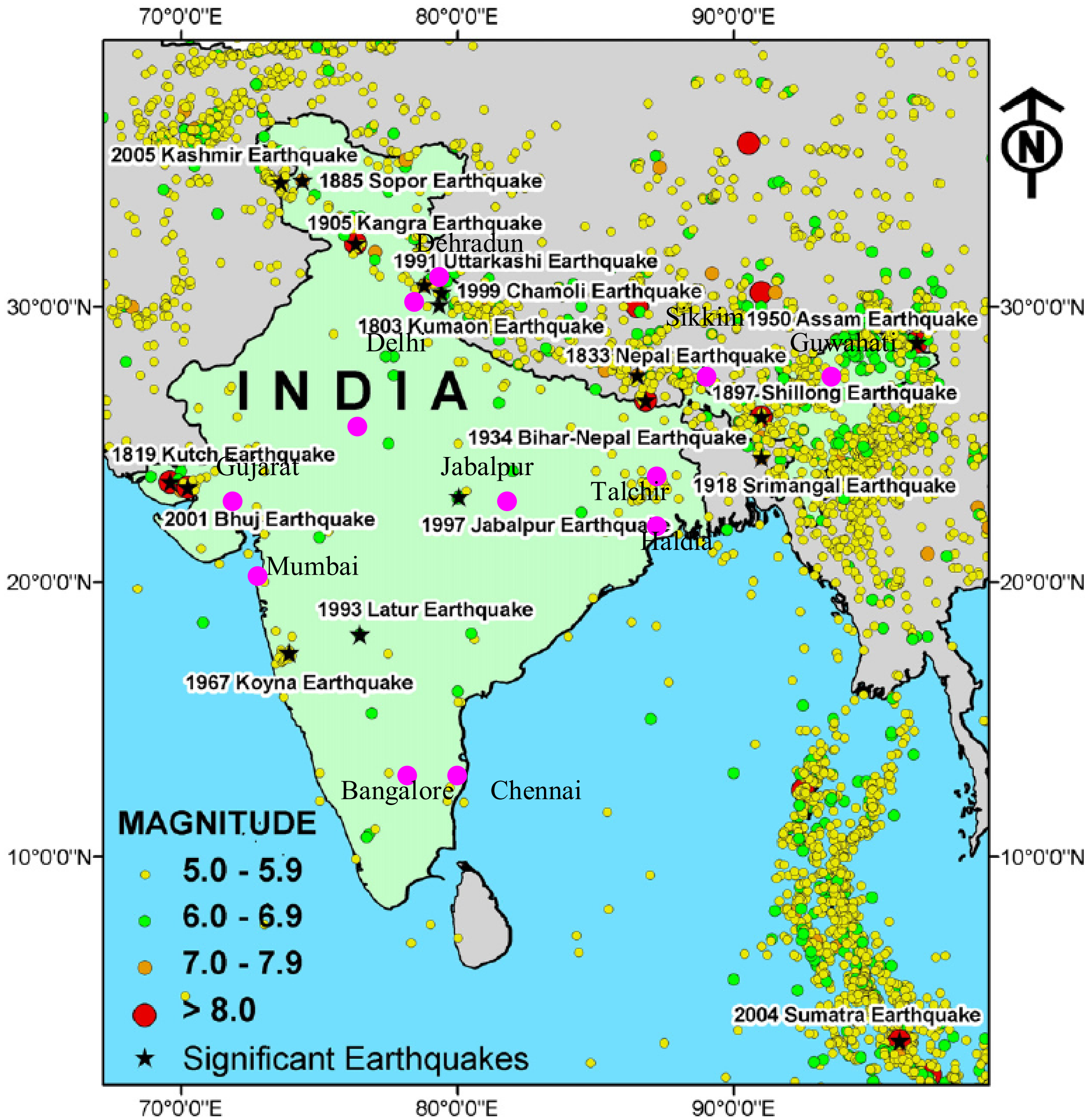

Figure A2.1: Earthquakes in India with microzonation cities (modified after Walling and Mohanty, 2009)

A2.3 Geotechnical and Geophysical Studies Carried for Seismic Microzonation in India

This section presents the summary of seismic hazard analysis and seismic microzonation of urban centers carried out in Indian sub continent, highlighting various issues related to geotechnical and geophysical studies. Except one or two cases, some of these microzonation exercises across India do not consider the detailed geotechnical investigation and further analyses based on these inputs. The quality of geotechnical investigation is also of questionable quality and thus the results are qualitative and not very useful in many cases. A survey presented here with some general comments at the end is to only highlight the status of seismic microzonation practices in India and this report does not certify the authenticity of the non-standard approaches adopted in any of these studies. Section A2.4 summarizes lessons learned from some of these exercises.

A2.3.1 Seismic Microzonation of Jabalpur Urban Area

The first microzonation work done in India was for the Jabalpur urban area. This work was carried out by the national nodal agencies like Geological survey of India, Central Region Nagpur, Indian Metrology Department New Delhi, National Geophysical Research Institute (NGRI), Hyderabad, Central Building Research Institute (CBRI), Roorkee and Government Engineering College, Jabalpur. Seismic 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). Nakamura technique was used to obtain predominant frequencies and then site amplifications. Even though, accuracy of estimation of natural frequencies by this technique is well established (Ohmachi *et al.* 1991, Field and Jacob 1993, Lachet *et al.* 1996, Fah *et al.* 1997), estimation of site amplification factors based on this technique is debatable (Lermo and Chavez-Garcia, 1994). 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 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 value at 30m depth (V_s^{30}). The predominant frequency and peak amplification maps were also prepared for Jabalpur. The final hazard map prepared for Jabalpur is shown in Figure A2.2.

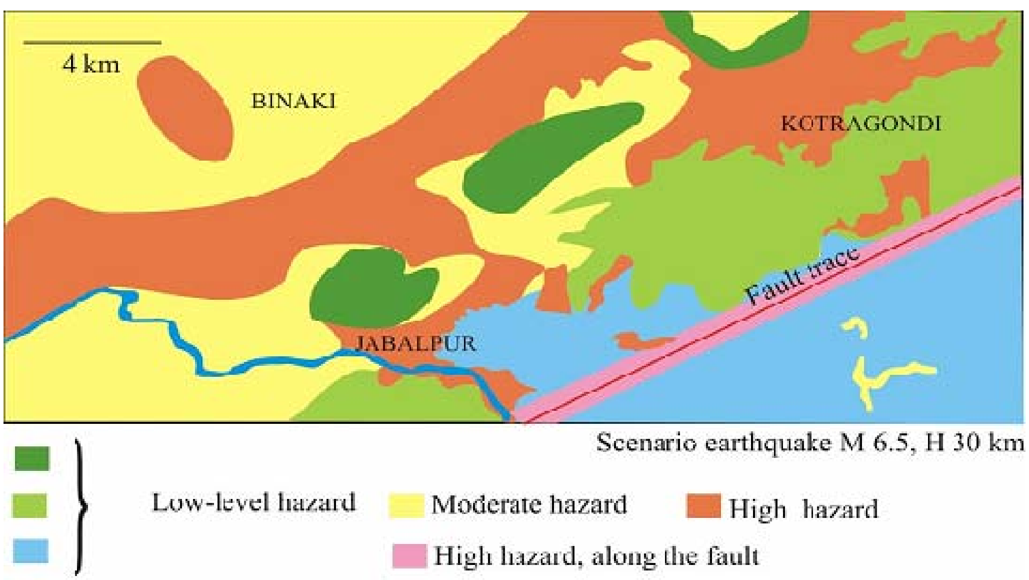

Figure A2.2: Final Hazard Map for Jabalpur

A2.3.2 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 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 Neelima Satyam (2005) by considering five seismic sources in Delhi region. A Geotechnical site characterization was carried out based on the borehole data, 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 Neelima Satyam, 2007). Mohanty et al (2007) prepared a first order seismic microzonation map of Delhi using five thematic layers viz., Peak Ground Acceleration (PGA), different soil types at 6 m depth, geology, groundwater fluctuation and bedrock depth, and integrating them using GIS platform (Figure A2.3).

Comparison of Vs Measurements Reported by Various Agencies for Delhi

A 1st level of microzonation map of Delhi was brought out by Joshi et al. (2006), which mainly focuses on geological criteria and Quaternary litho-fill sediments. They have divided NCT, Delhi region into nine major units and suggested the possible hazard for each unit based on the information from geology, Quaternary litho units, ground water conditions, and frequency and amplifications based on ambient noise Nakamura technique etc. In order to validate the classification given by Joshi et al. (2006), the Earthquake Risk Evaluation Center (EREC) had requested Wadia Institute of Himalayan Geology, Dehradun to carry out shear wave velocity investigations in these major nine units. The shear wave velocity profiles were carried out in different individual sites as the representative of major nine units. The site response analysis carried out by Mundepi et al. (2010) shows fundamental resonance frequency of the order of 0.45 Hz to 3.5 Hz. The low frequency is very well corroborated with sites showing shear wave velocity less than < 180 m/s. On the other hand Suhalpur and Jasola sites showing harmonic shear wave velocity (Vs_{30}) more than >180 m/s for the 30 m soil column also corroborates with the frequency range of 1.5 Hz to 2 Hz. The same is true for hard rock sites showing fundamental frequency of 3.5 Hz or more. However, there are some difference of opinion in the shear wave velocity map driven by Satyam & Rao (2008) who classified Delhi region into three different zones Zone - A (Vs^{30} > 350 m/s), Zone - B (Vs^{30} 250-350 m/s) and Zone - C (Vs^{30} < 250 m/s). Whereas the results from Mahajan et al. (2011), based on ten representative sites, shows again three classes as per NEHRP classification - class - A (Vs_{30} > 760 m/s), class - D (Vs_{30} 180-360 m/s) and class - E (Vs_{30} <180 m/s). Further the shear wave velocity derived by Satyam & Rao (2008) for JNU campus was 380 m/s at a depth of 2 m and 480 m/s at a depth of 30 m whereas results from Mahajan et al. (2011) shows 700-800 m/s (2 m depth) and 1500 m/s at a depth of 30 m.

A2.3.3 Seismic Hazard and Microzonation Atlas of the Sikkim Himalaya

Seismic hazard and microzonation atlas of the Sikkim-Himalaya was prepared based on deterministic method at a scale of 1:50,000 (Nath, 2006) which is lesser than the usual scale of study recommended for seismic microzonation studies. Site response studies were carried out using the receiver function techniques and generalized inversion technique, considering the strong motion data.

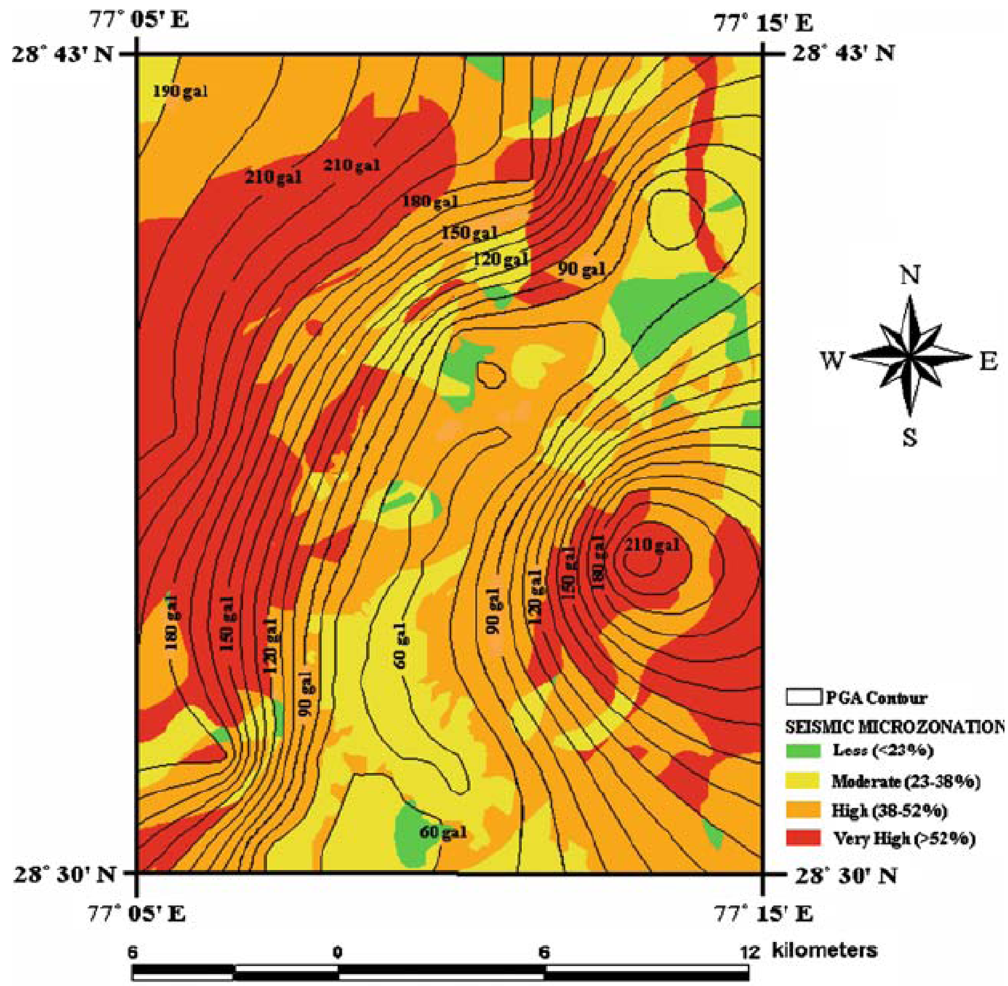

Figure A2.3: Final hazard map of Delhi (after Mohanty et al., 2007)

Site classification is done based on composition, grain size and lithology. It is conventional to consider measured shear wave velocity measurements and calculated V_{s30} for characterization of the site, which is missing in this study. Seismic microzonation map for the study area was presented in the form of geohazard map and quasi-probabilistic seismic microzonation index map. The geohazard map was prepared by integrating the weights and ratings of soil, surface geology, rock outcrop and landslides. Probabilistic seismic microzonation index map was prepared by integrating the weights and ratings of site response, peak ground acceleration, soil, rock outcrop and landslides. Figure A2.4 shows the microzonation map of Sikkim (Nath, 2007).

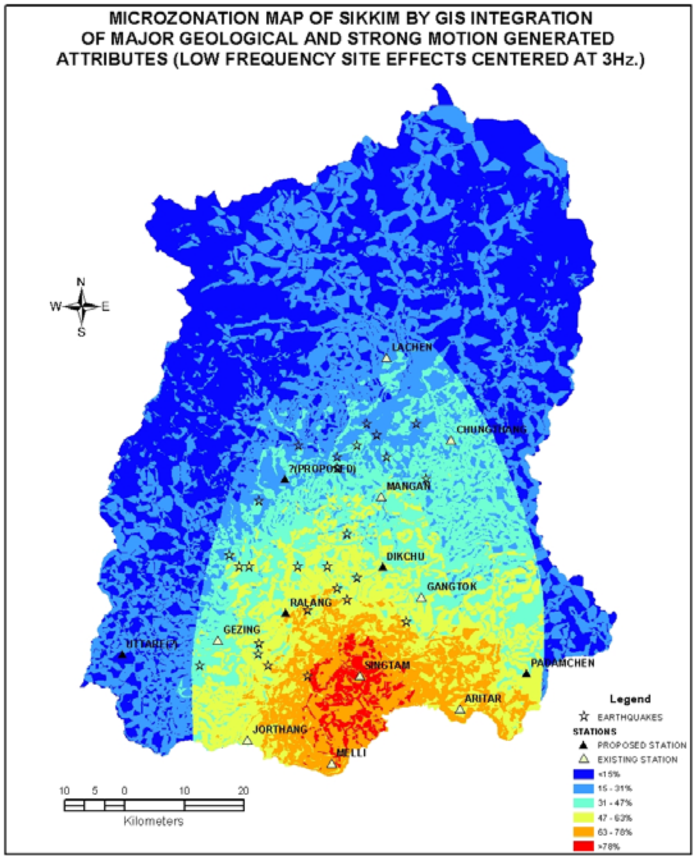

Figure A2.4: Microzonation Map of Sikkim (after Nath, 2007)

A2.3.4 Seismic Microzonation of Guwahati

The first level microzonation map of Guwahati was prepared based on amplification of ground motion, slope of exposed rocks, shape and constituents of overburden material inferred from geophysical surveys (Baranwal et al., 2005). In that work the soil profile was categorized in terms of their susceptibility to amplification. In this study, shear wave velocity profiles and V_{s30} values were obtained indirectly from SPT data using correlations between SPT and V_s. The common practice is to obtain shear wave velocity profiles from the in-situ testing. However, SPT data can be used for cross checking the V_s profiles using the correlations. Microzonation of Guwahati (Nath, 2007) contains important maps based on different categories like, geology and geomorphology, seismotectonics, soil characteristics, pre-dominant frequencies, peak ground acceleration, seismic hazard, demography and preliminary risk. Details and maps related to liquefaction hazard, a very important seismic hazard was not given due consideration. The total area was divided into five zones based on the hazard index values and most of the residential area fall in the moderate zone. The hazard index map of Guwahati (Nath, 2007) is shown in Figure A2.5.

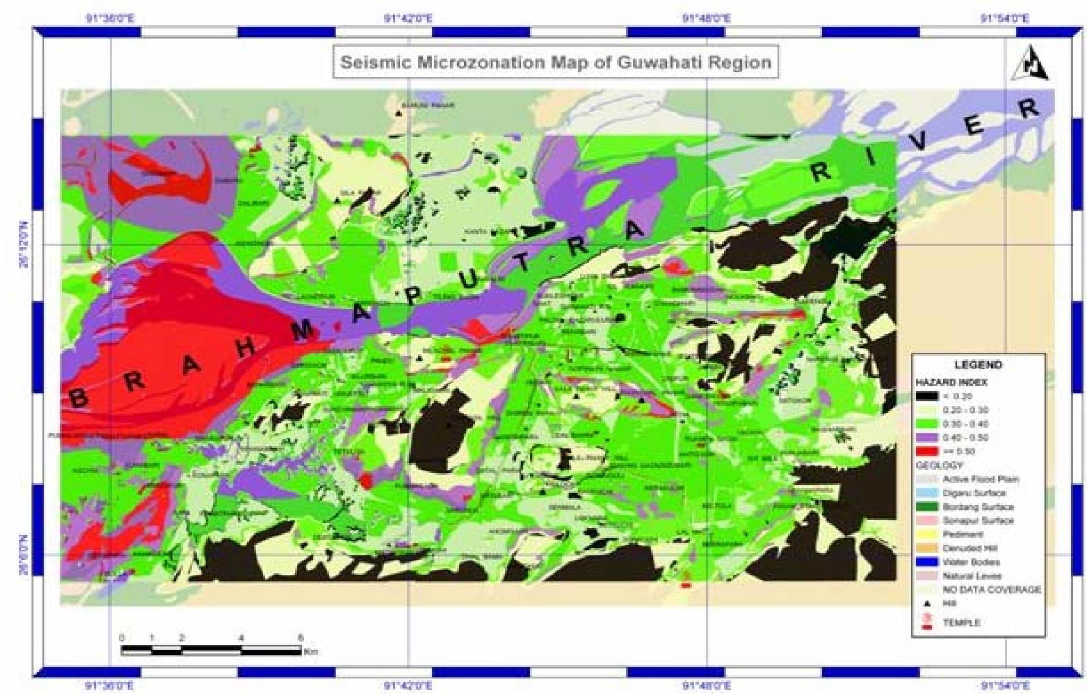

Figure A2.5: Hazard index map of Guwahati (after Nath, 2007)

A2.3.5 Seismic Microzonation of Dehradun

The Seismic microzonation of Dehradun was carried out by three researchers Anusuya Barua (2005), Rajiv Ranjan (2005) and Brijesh Gulati (2006). This study was done based on geology and geomorphology of Doon valley in regional scale and subsurface strata at local scale for Dehradun city. The database generated for the seismic microzonation contains information on lithology, tectonic features, landforms and associated neotectonic activity in regional scale. The site characterization of Dehradun was done based on the V_s^{30} values obtained from the MASW survey and the spectral acceleration values were developed for the surface level (Figure A2.6; Rajiv Ranjan, 2005). The earthquake risk assessment (ERA) of buildings in Dehradun was done using HAZUS program for Dehradun (Brijesh Gulati, 2006).

A2.3.6 Seismic Microzonation of Haldia

First order seismic microzonation of Haldia was presented by Mohanty and Walling (2008). The final map was generated considering three themes viz. Peak Ground Acceleration (PGA), predominant frequency and elevation map. PGA map was prepared by considering tectonic framework, past seismicity and five seismic source zones around Haldia. The five seismic source zones are 1) Arakan-Yoma Zone (AYZ), 2) Himalayan Zone (HZ), 3) Shillong Plateau Zone (SPZ), 4) Bay of Bengal Zone (BBZ) and 5) Shield Zone (SZ). The ground motion in terms of PGA from the five source zones was estimated using the attenuation relationship of Toro et al. (1997). The site specific predominant frequency is estimated by employing the Horizontal to Vertical Spectral Ratio (HVSR) technique of Nakamura (1989).

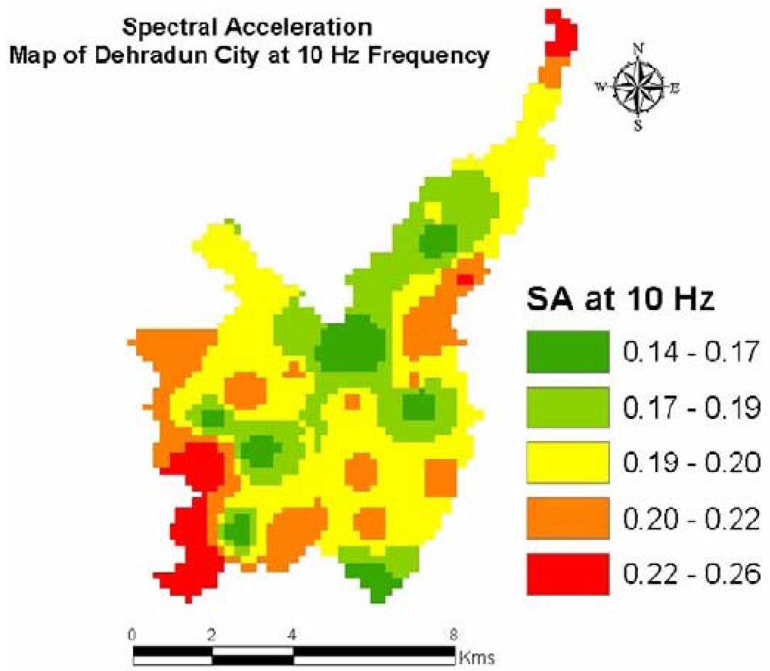

Figure A2.6: Spectral acceleration map of Dehradun (after Rajiv Ranjan, 2005)

Haldia overlies the Bengal delta and the elevation of the region will give an estimate of the thickness of the alluvium. The elevation map of Haldia was obtained by analyzing the Shuttle Radar Topography Mission (STRM) data accessed from the Global Land Cover Facility (GLCF) site (http://www.landcover.org) (Mohanty and Walling, 2008). Figure A2.7 shows the first order seismic microzonation of Haldia using Analytical Hierarchy Process (AHP) on the Geographic Information System (GIS).

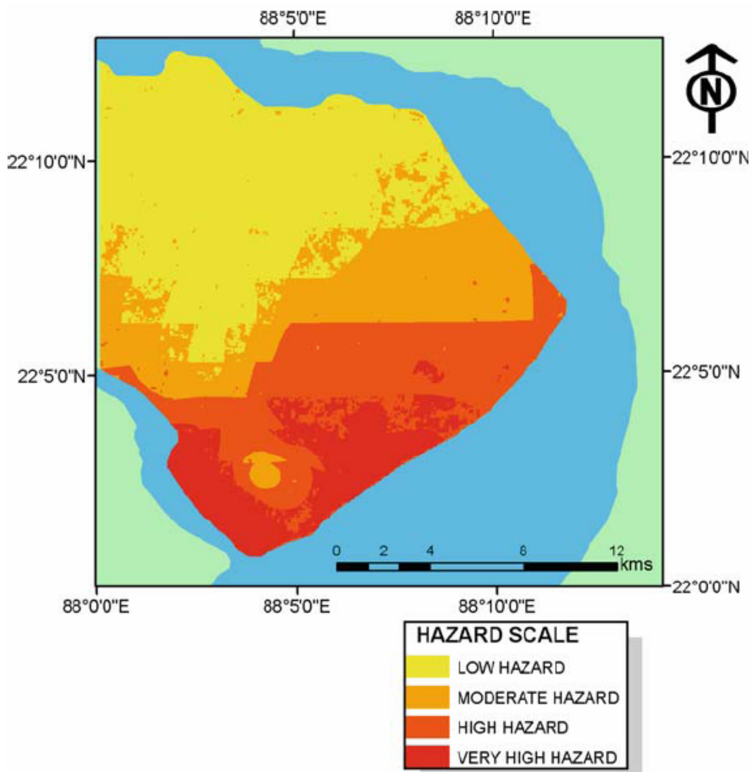

Figure A2.7: Seismic microzonation map of Haldia (after Mohanty and Walling, 2008)

A2.3.7 Seismic Microzonation of Talchir Basin

The Talchir Basin (84°19'E to 85°30'E and 20°44'N to 21°20'N) is in the state of Orissa, India. The Talchir Basin is an isolated basin surrounded by Precambrian rocks on all sides. The major tectonic structure of Talchir area is the North Orissa Boundary Fault (Mohanty et al. 2009). Mohanty et al (2009) simulated the multiphase synthetic seismograms for the Talchir Basin along eight profiles based on a hybrid method, which combines two computational techniques, a modal summation and a finite difference model**.** A scenario earthquake of magnitude of 6 and a focal depth of 5km along the North Orissa Boundary Fault (NOBF) was used to simulate peak acceleration (a_{max}) along each profile. The response spectra ratio (RSR) as a function of frequency was computed for the eight profiles, authors have found that higher amplification is associated with the thicker sedimentary cover. Talchir Basin is subdivided into three zones, zone 1 (low RSR level: 1.6–1.9), zone 2 (intermediate RSR level: 2.0–2.8) and zone 3 (high RSR level: 2.9–5.2) based on the RSR values for considered earthquake scenarios. Mohanty et al (2009) has highlighted that the proposed seismic zonation of Talchir Basin corresponds to a one degree variation of macroseismic intensity. Figure A2.8 shows the three zones of hazard of Talchir Basin based on the RSR values.

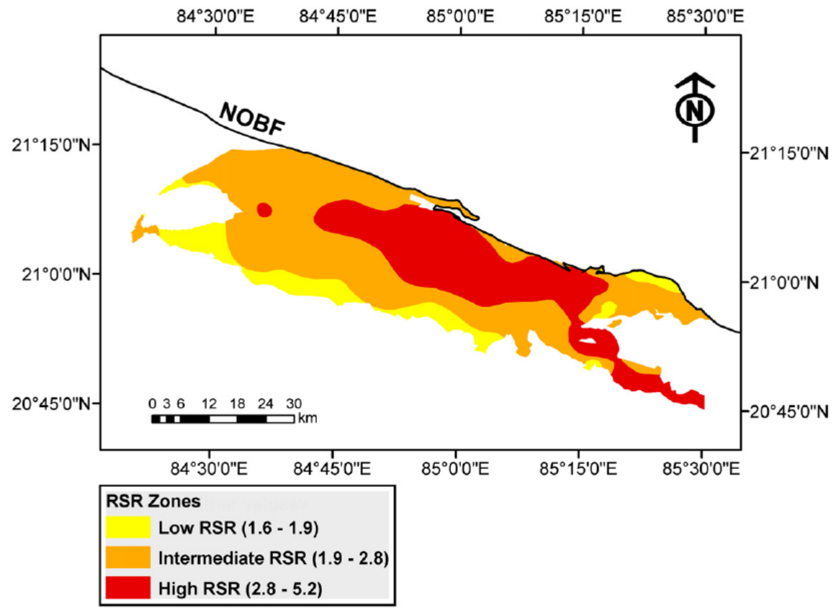

Figure A2.8: RSR values based three zones of hazard of Talchir Basin (after Mohanty et al., 2009)

A2.3.8 Seismic Hazard Estimation for Mumbai City

The seismic hazard of Mumbai city was estimated by Raghu Kanth and Iyengar (2006) based on probabilistic seismic hazard analysis. The influence of local site conditions was accounted by providing design spectra for different NEHRP site classes viz. A, B, C and D. They highlighted that the results presented can be directly used to create a microzonation map for Mumbai.

A2.3.9 Ongoing seismic Microzonation Studies of Other Cities in India

The Department of Science & Technology, presently Ministry of Earth Sciences (MoES), New Delhi has initiated the seismic microzonation of Bhuj, Kachchh, Ahmedabad, Chennai and Kochi regions. A brief description of these projects are given below.

An extensive study to identify and classify the seismic sources in the Kachchh was done by Mohanty (2006). Based on the analysis of the geological map of the region a new empirical seismic attenuation model for Kachchh was developed and the PGA values were calculated using PSHA method. The process of preparing a detailed microzonation map is going on and this will include the liquefaction potential of the regions.

The preliminary results of site-effects and shear wave velocity structures of sub-surface soil in Ahmedabad were presented by Parvez and Madhukar (2006). They highlighted that most of the sites are having the fundamental resonance frequency of 0.6Hz and rest of them is having frequency of 2 to 6Hz.

The seismic hazard and site response study of Chennai city was done by Suganthi and Boominathan (2006). They have concluded from the ground response analysis that significant amplification is present only in the low range of frequencies.

Center for Earth Science Studies (CESS) is carrying out the seismic microzonation of Kochi city. In this work the site response is measured using ambient noise (microtremor).

This will be related to the available information on geology, geomorphology, lineament patterns, soil type/ lithology, structural features etc. in the region.

A2.3.10 Seismic Microzonation of Bangalore

Seismic microzonation of Bangalore was carried out by considering two major attributes 1) Geomorphological attributes and 2) Seismological attributes. The usual geomorphological attributes considered in seismic microzonation mapping are 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 of the ground (EL). The seismological attributes have been generated based on detailed studies of seismic hazard analysis, site response studies and liquefaction analysis. From the detailed study different earthquake hazard parameters were generated and they are as follows:

1. Peak ground acceleration (PGA) at rock level generated based on synthetic ground motions considering MCE based on DSHA.
2. PGA at rock level at 10 % probability of exceedance in 50 years based on PSHA.
3. Amplification factor based on ground response analysis using SHAKE2000.
4. Predominant frequency based on site response and experimental studies.
5. Factor of safety against Liquefaction potential

Based on the above attributes, two types of hazard index map were generated. One is the deterministic seismic microzonation map (DSM), which is the 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.

Deterministic seismic microzonation map is the hazard index map for worst scenario earthquake. One of the important factors, PGA, is estimated from synthetic ground motions, which were generated based on MCE of 5.1 in moment magnitude for the closest vulnerable source of Mandya – Channapatna - Bangalore lineament (Sitharam and Anbazhagan 2008). Hazard index values were estimated based on normalized weights and ranks through the integration of all themes using the following equation:

$$DSM = \begin{pmatrix} \mathrm{DPGA}_W \mathrm{DPGA}_r + AF_W AF_r + ST_W ST_r + SS_W SS_r + \\ FS_W FS_r + PF_W PF_r + EL_W EL_r + DR_w DR_r + GG_w GG_r \end{pmatrix} / \sum W \qquad (7.1)$$

Based on these estimated values, the deterministic seismic microzonation map has been generated. Figure A2.9 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 groups, <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 at the western part of Bangalore. Eastern part of city is having a lower hazard when compare to other areas. Western and southern part has mixed hazard and northern part is having moderate hazard.

Similar to DSM hazard index calculation, probabilistic hazard index has been estimated. In this the PGA values were 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 relation. Based on probabilistic hazard index values probabilistic seismic microzonation map (PSM) has been generated. Probabilistic hazard index values were estimated based on normalized weights and ranks through the integration of all themes using the following equation:

$$PSM = \begin{pmatrix} \mathrm{PPGA}_W \mathrm{PPGA}_r + AF_W AF_r + ST_W ST_r + SS_W SS_r + \\ FS_W FS_r + PF_W PF_r + EL_W EL_r + DR_w DR_r + GG_w GG_r \end{pmatrix} \Big/ \sum W \qquad (7.2)$$

Figure A2.10 shows the probabilistic seismic microzonation map based on hazard index. Probabilistic hazard index values vary from 0.10 to 0.66 and have been divided into four groups such as < 0.1, 0.10 - 0.15, 0.15 - 0.30, 0.3 - 0.45, 0.45 - 0.6 and 0.6 to 0.66. These values are lesser than that of deterministic hazard index. The maximum hazard is attached to the seismic hazard index greater than 0.6 at south western part of Bangalore. Southern part of Bangalore is having moderate to maximum hazard when compare to the northern part.

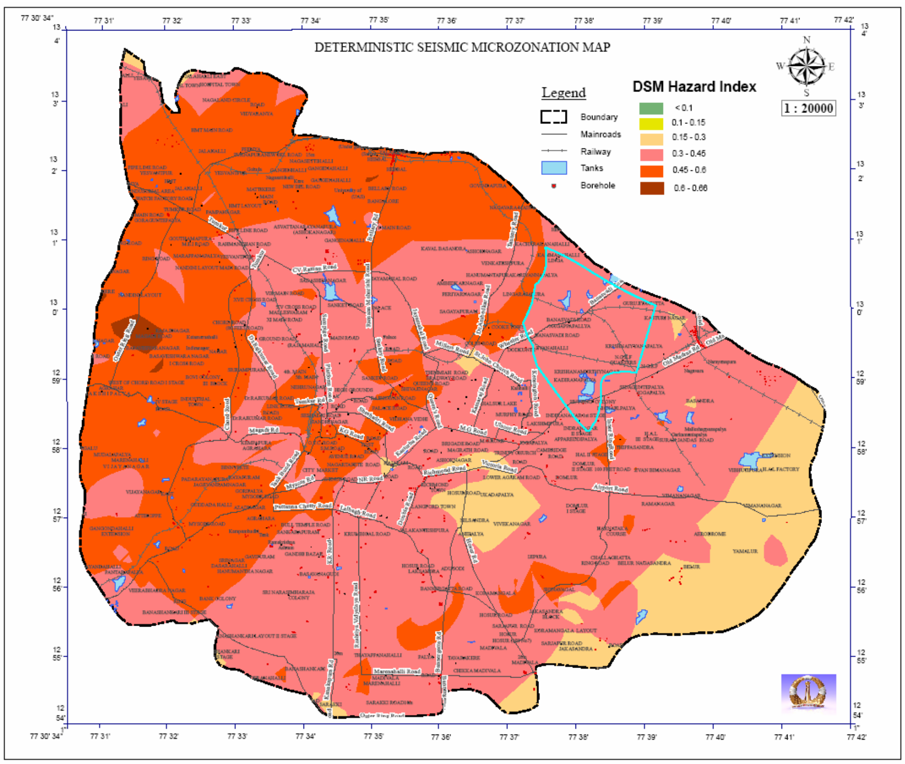

Figure A2.9: Deterministic seismic microzonation map (after Sitharam and Anbazhagan, 2008).

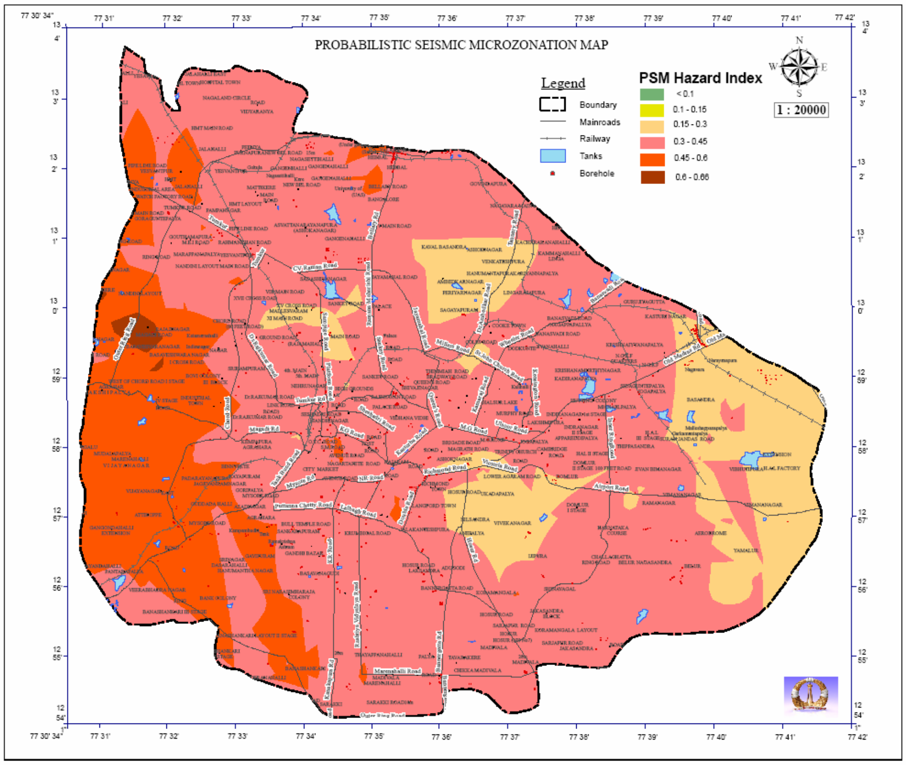

Figure A2.10: Probabilistic seismic microzonation map (after Sitharam and Anbazhagan, 2008).

A2.4 Lessons Learned from Various Microzonation Works in India

One of the main drawback of microzonation of Jabalpur was that the study has used a relatively old attenuation relation and the probabilistic seismic hazard analysis was also not done. The maps showing quantified values of the seismic hazard were also not presented and the importance given for geotechnical aspects were also very less. The microzonation of Sikkim-Himalaya was done considering various parameters and these values were integrated using a GIS platform. By doing such integration, the seismic hazard due to various effects were taken into account by giving proper weightage. However the geotechnical data collection done for this work was very less and the liquefaction hazard was also not evaluated. The microzonation of Dehradun has developed spectral acceleration maps only. The microzonation of Guwahati was done by considering various factors and various hazard maps were developed. The final hazard map was obtained by integrating these hazard maps in GIS platform. Site characterization is based on shear wave velocities estimated indirectly from SPT values, which require cross checking using actual measured shear wave velocities. The microzonation of Bangalore was done by considering various factors and these were integrated using GIS platform. The major improvement in this work was that the probabilistic and deterministic seismic hazard assessment was done separately and different maps were developed based on this. Moreover the geotechnical aspects were given due consideration and the liquefaction hazard was also evaluated.

A2.5 Issues Related to Seismology

Issues related to seismology are the preparation of regional seismotectonic map and estimation of seismic hazards:

a) Preparation of seismotectonic map for the study area.

Most of microzonation works completed in India does not have a complete seismotectonic map. It is essential that before starting the microzonation work a detailed seismotectonic map for the study area need to be prepared. A seismotectonic map has to be prepared by considering up to date seismicity, geological and seismotectonic details for a circular area of radius 300km (approximately 200miles) around the study area as per Regulatory Guide 1.165(1997). This map can be used for the detailed hazard estimation.

b) Seismic hazard estimation.

So far most of seismic microzonation maps published in India based on the deterministic seismic hazard analysis by considering scenario earthquake. But seismic microzonation hazard estimation need to consider the uncertainties involved in the earthquake occurrence and develop the hazard map for the required return periods. Hence special attention can be given to the probabilistic seismic hazard estimation, where the uncertainty is quantified and hazards are represented with required probability exceeded in particular years.

A2.6 Grade and Geology

The grade/level of the seismic microzonation maps can be desired based on scale of the study and method of estimating hazard parameters as suggested by the TC4 committee of ISSMGE (1993). Seismic microzonation studies in India show that, level/grade of map was not fixed properly. Geology can be considered for initial stage of the seismic microzonation mapping. Most of the seismic microzonation maps produced in India has given more importance to geology. But recently, it is proved that considering the geological units as the only criteria in seismic microzonation is not appropriate (Ansal et al., 2004). They also highlighted that geology map may be regarded as the basic information to plan detailed site investigations and to control the reliability of the results obtained by site characterizations and site response analyses.

A2.7 Geotechnical Issues

Another key issue in the seismic microzonation is the estimation of earthquake effects. On a broader scale the earthquake effects can be divided into two groups, site effects and induced effects. These effects are based on geotechnical properties and the behavior of subsurface materials during the earthquake. Hence more importance needs to be given to the geotechnical properties rather than geology. Case studies summarized above show that, geotechnical properties were not handled properly while assessing the site and induced effects. Site effects are combination of soil and topographical effects, which can modify the characteristics (amplitude, frequency content and duration) of the incoming wave field. Most of the Indian seismic microzonation studies show that the modification of waves is estimated using average shear wave velocity in the top 30m (V_s^{30}) irrespective of locations. This practice need to be completely reviewed, because V_s^{30} is not a standard parameter to reflect the site effects. Pitilakis (2004) shows the inability of the V_s^{30} for estimation of site amplification of soil layers. Particularly large amplifications of the deep incident wave field are practically absent when amplification are computed using the transfer ratio for shallower depths. The author has also showed that the use of V_s^{30} as a basis for site amplification is misleading in many cases. Hence it is necessary to use actual engineering rock depth (shear wave velocity more than 700 m/s) rather than V_s^{30} for amplification study. Another major issue is the estimation of induced effects such as liquefaction hazard and land slide hazard. Most of the case studies summarized above shows that liquefaction hazard was estimated using the old correlations and attenuation relation without much of local geotechnical knowledge. It is always recommended the liquefaction hazard has to be estimated based on recent developments in earthquake geotechnical engineering. Most of the land slide hazards in seismic microzonation studies in India were estimated based on geological data. However detailed geotechnical inputs are required for the precise mapping of landslide prone area during earthquake. There is a need of carrying out detailed geotechnical studies for seismic microzonation.

A2.8 Conclusions

This chapter describes various geotechnical and geophysical studies carried in India for the purpose of seismic microzonation. Pointed out the problems with the current practices in the geotechnical and geophysical investigations adopted for seismic microzonation. This chapter also highlights the problems in the interpretations of such studies in overall estimation of seismic hazards. This will also help in getting a broad picture of microzonation practices adopted in India. The main issues in the microzonation works and the codal provisions related lack geotechnical studies are also explained in this chapter.

Annexure III
Memorandum of Understanding (MoU) to be Signed Between NDMA and Indian Institute of Science, Bangalore

PREPARATION OF TECHNICAL DOCUMENT ON GEO-TECHNICAL/ GEOPHYSICAL INVESTIGATIONS FOR SEISMIC MICROZONATION STUDIES OF URBAN CENTRES IN INDIA.

Major parameters of the assignment VIZ. "Preparation of Technical Document on Teo-Technical/Geo-Physical Investigations for Seismic Microzonation Studies of Urban Centres in India", as finalized with NDMA, are given below:-

(i) The assignment will be executed by Indian Institute of Science, Bangalore, either through its own resources, or by engaging Professional Consultants as per the laid down procedures.

(ii) **Project Objectives:**

Phase-I:

i. The Working Group of Experts for Geotechnical Investigations (WGE-GT) shall prepare a Technical Document (TECH-DOC) giving all the relevant details about geotechnical issues for seismic microzonation in an elaborate manner. This Document should serve as a reference document for the geotechnical aspects of seismic microzonation work in the country.

The overall objective of these guidelines is to assist the design engineers/town planners to understand the general site conditions on the basis of the site classification.

ii. The WGE-GT shall review the global geotechnical practices adopted for various grades of seismic microzonation work and indicate their relative importance along with the respective limitations and advantages for various applications. Similarly, it shall review the similar works that have been done in India along with their usefulness for the intended purpose.

iii. The WGE-GT Technical Document shall address geotechnical inputs needed by structural engineers for design, retrofitting and construction work at a given site that may have liquefaction potential. In addition, to the geotechnical work needed for Disaster Management work at NDMA/town planners, the Technical Document will, therefore, include, the following (so that the Technical Document serves the purpose of an excellent Reference document for any detailed geotechnical investigation to be undertaken in the country): Design Response spectra for sites that are liquefiable

(a) Topographic effects

(b) Depth to bed rock (Vs+ 760-1500 m/s)

(c) Ground water

(d) Dynamic soil properties

(e) Role of N-values and borehole data

(f) Seismic slope instability/land slides

(g) Liquefaction quantification

(h) Procedures to decide the soil type (A/B/C/D/E/F) of a site to be used in the Probabilistic Seismic Hazard Analysis.

(i) Different grades/levels of maps (scale) based on geology/geomorphology/geophysical data/geotechnical data.

iv. The WGE-GT Technical Document shall provide the details of the methods to be adopted at the field and/or laboratory investigations along with the details of the corresponding equipment/instruments to be used and monitoring mechanism to be kept in place for ensuring the "quality" of the generated data shall be made available in respect of each of the aspects indicated above. Similarly, the methodology for interpretation of the data should also be spelt out for the various tests/investigations recommended above in para (i) and (iii).

Tech Doc. Shall also clearly spell out detailed "Site –Specific Investigations" required to be undertaken for any important projects. The methods of various tests required for seismic microzonation and those prescribed for "Site-Specific Investigations" shall be standardized and its implementation will be clearly described in the TECH-DOC so that the descriptions can be used as a standard reference document in the country. TECH DOC will also address the usage of geological and geomorphological maps (for prediction of ground motion using empirical relations between surface geology and sub-surface properties), recommend acceptable methods for delineating the surface level PSHM sites overlain with soft soil layers, of varied ages and composition, in seismically vulnerable cities (Zones III, IV V)) including those near river basins.

v. The Preparation of DPR is to be done in Phase-I instead of in Phase-II. A DPR will be prepared as per the details in the Technical Document which will include the detailed plan and the resources required, along with expected timelines, for completion for the various tasks proposed in Phase II.

Deliverable/Output: TECH DOC.

Phase II.

i. Pilot scale studies for seismic microzonation at two specific cities for validation of the recommended prescription of various tests in TECH DOC – Two likely sites identified (i) Some selected region within Mumbai metropolitan region and (ii) NOIDA City, Uttar Pradesh. The decision in respect of the two cities to be finalized in consultation with Steering Committee (Geo Physical Hazard).

ii. To carry out geotechnical investigations at some selected strong motion sites (of IIT Roorkee) in phases for validation of attenuation Relations used in PSHA work. First it will be done at those sites where earthquake records are available and in some sites having a large overburden to address issues of site effects in future. The details of geotechnical/geophysical investigations will be worked out and presented in the DPR.

Deliverable/Output : Validation of Attenuation Relations for different sites.

v. Time Frame:

The project is envisaged for duration of 1 ½ years from the date of release of fund to IISc, Bangalore.

vi. Activities:

(a) Collection of literature on seismic microzonation.

(b) Study of seismic microzonation case studies carried out in India and elsewhere.

(c) Preparation of Draft Technical Document on recommended procedure for geotechnical/geophysical testing related to seismic microzonation studies. The methods of various tests required for seismic microzonation will be standardized and its implementation will be clearly described in the TECHDOC so that the descriptions can be used as a standard reference document. Tech doc will also address the site specific cases and required geotechnical/geophysical testing along with detailed methodologies for the same.

(d) Conducting National Workshop to get the broad consensus/acceptability of the contents for the technical document after one year.

(e) Preparation of Final draft Report

(f) Preparation of Detailed project report for Phase II studies.

(g) Conducting National Workshop to disseminate the knowledge and handing over the deliverables to NDMA.

3. IISc Will organize quarterly review meetings at IISc. Or elsewhere (at respective IITs where the identified expert members belong to) and submit minutes of the meeting and progress reports to NDMA. NDMA will organize quarterly review meetings for physical and financial progress.

4. All plans, drawings, specifications, designs, reports, other documents and software prepared by IISc. under the Project, shall become and remain the property of NDMA.

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DIRECTOR(MITIGATION)	CHAIRMAN, CISTUP

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Indian Institute of Science Bangalore has been identified as the nodal agency for the project.

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