Lecture 13: Introduction to Site Characterization; Different methods and experiments; Geotechnical properties; Site classification and worldwide code recommendation and Steps involved in site characterization

Topics

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- 3-D Subsurface Modeling of Geotechnical Data Using GIS
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- Comparison of seismic site classification

Keywords: *Site Characterization, Methods, Experiments, site class*

Topic 1

Introduction to Site Characterization, Site characterization data and Need for Site Characterization

- Estimation of geotechnical site characterization and assessment of site response during earthquakes is one of the crucial phases of seismic microzonation, which includes ground shaking intensity and amplification.

- Site characterization provides the basic soil index property and engineering properties, which are determined based on in-depth exploration to identify and evaluate a potential hazard.

- Site characterization involves investigation (laboratory and field), data collection, interpretation of data and finally representing in terms of maps. Geotechnical site characterization is usually done using experimental investigations of standard penetration test, cone penetration test, Multichannel analysis of surface wave testing (shear wave velocity survey) and numerical methods.

- Standard penetration test and Multichannel analysis of surface methods are widely used methods for site characterization. Site characterization is carried out with the following objective:

    1. Measuring and the interpretation of soil properties
    2. Complementing and Extending the land cover mapping;
    3. Developing soil maps of a region; and
    4. Providing information or input for computer modeling of site response.

- A general site characterization should describe:

    1. The site
    2. Provide Geotechnical, Geological and hydro-geological/ground water data
    3. Characterize the aquifer or permeable characteristics
    4. Describe the condition and strength of the soil
    5. Give a risk assessment and reveal the presence and distribution of any contaminants.
    6. It must give detailed information about the mechanical and geometrical parameters of the subsurface
    7. The effects of the proposed project on its environment and an investigation of existing structures or lifelines below the subsurface.
    8. Site description and location
    9. Climatic conditions

- This data can thus be used to select a site, design the foundation and earthworks, and study the effects of the earthquake. How a soil deposit responds during an earthquake depending on the frequency of the base motion and the geometry and material properties of the soil above the bedrock.

- The geometries and material properties of soil are directly or indirectly quantified and represented by many researchers as a part of seismic microzonation. Seismic site classifications are widely used to quantify site effects and spectral acceleration.
- A geographic distribution of site class based on V_S^{30} is useful for future zonation studies because the amplification factors are defined as a function of V_S^{30}, such that the conditions of the ground on the site shaking can be taken into account (Kockar et al., 2010).
- Need for Site Characterization - There are hazards and uncertainties in the ground, as a result of natural and manmade processes, that may jeopardize a project and its environment if they are not adequately understood and mitigated.
- An appropriate site characterization will maximize economy by reducing, to an acceptable level, the uncertainties and risk that site conditions pose to a project. Site characterization also plays an important role in safety assessments and identification of potential environmental effects.
- Site characterization involves the determination of the nature and behaviour of all aspects of a site and its environment that could significantly influence, or be influenced by, a project.
- The basic purpose of site characterization is to provide sufficient, reliable information of the site conditions to permit good decisions to be made during assessment, design and construction phases of a project.
- Site Characterization should include an evaluation of subsurface features, sub surface material types, subsurface material properties and buried/hollow structures to determine whether the site is safe against earthquake effects.
- Site characterization involves determining information on previous and current land use, topography and surface features, hydrogeology, hydrology, meteorology, geology, seismology, geotechnical aspects, environmental aspects and other factors.
- **How to do site characterization** - There are mainly three methods used for site characterization (Table 13.1)

    1. Geological and Geomorphologic methods
    2. Geotechnical Methods
    3. Geophysical methods

- A site characterization for seismic microzonation using geological, geo-technical, and geophysical data can be conducted. Commonly used geotechnical tests include standard penetration tests (SPT), dilatometer tests (DMT), pressure meter tests, and seismic cone penetration tests (SCPT), of which SPT is the most widely used in many countries because of the availability of existing data.

- Many geo-physical methods for seismic site characterization have been attempted but the methods commonly used are Spectral Analysis of Surface Waves (SASW) and Multi-channel Analysis of Surface Waves (MASW).

- One of the important parameter to be considered in geotechnical studies is the scale of geotechnical data collection. The proposed scale for geotechnical data collection for different levels of seismic microzonation studies are listed below.

For Level I

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

For Level II

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

For Level III

1. Homogeneous sub-surface – 0.5 km x 0.5km to 2 km x 2km
2. Heterogeneous Sub-surface – 0.1 km x 0.1 km to 0.5 km x 0.5k

- Steps involved in site characterization - The important steps involved in Site characterization are

1. Base map Preparation
2. Available data collection
3. Experimental study
4. Data Analysis
5. Estimation of Equivalent shear wave velocity
6. Site classification
7. Mapping

Table 13.1: Comparison, advantages and limitations of methods of Site Characterization

Description	**Geology and Geomorphology**	**Geotechnical Methods**		**Geophysical methods**	
		SPT	**SCPT**	**SASW**	**MASW**
Strain	-	Large	Large	Small	Small
Drilling	-	Essential	Essential	No	No
Cost	Low	High	High	Low	Medium
Time	Long	Long	Medium	Short	Short
Quality of data	Poor	Good	Very Good	Fair	Very Good
Detection of variability of soil deposits	Poor	Good	Very Good	Good	Very Good
Suitable soil type	All	Non Gravel	Non Gravel	All	All
Depth of information suitable for Microzonation	Poor	Good	Fair – Vs is available up to 20m	Good	Very Good
Measurement of dynamic properties	Poor	Fair	Good	Good	Very Good
Success full cases used	Small	Large	Medium	Medium –Large	Very Large

- **Base map preparation** – a base map is one of the important ingredients of the seismic microzonation studies; a preparation of which requires a special consideration. Over the last four decades Geographical Information systems (GIS) have emerged as the predominant medium for graphic representation of geospatial data, including geotechnical, geologic and hydrologic information.

- The base map includes several layers of information such as outer and administrative boundaries, Contours, Highways, Major roads, Minor roads, Streets, Rail roads, Water bodies, Drains, Landmarks and Borehole locations.

- Preparation of base map data requires familiarity with GIS data formats and spatial references (i.e. datums and coordinate systems/projections) so that all base map layers properly co-register and have adequate resolution.

- For base map data obtained from different sources, preparation may, for example, involve projecting datasets to a common spatial reference, defining a spatial reference for data that lack such as, clipping to an area of interest and/or preparing derivative base map layers, such as those that portray slope or topography in shaded relief.

- Steps involved

    1. Define region of interest for current project
    2. Decide which datasets are valuable to your project
    3. Identify sources of datasets and download to local computer
        a. Topographic
        b. Thematic
        c. Imagery
    4. Identify and georeference any additional non-digital sources of spatial data
        a. Compile tabular data with x,y location information into a spreadsheet and add to GIS application
        b. Scan paper map products
            i. Clip scanned image to area of interest and save as compatible file format (tiff, jpeg)
            ii. Georeference scanned image
    5. Confirm that all datasets are in the same coordinate system, projection and datum.
        a. If coordinate system is undefined, find original coordinate system and establish spatial reference
        b. If not all the same, project to a common coordinate system and datum
    6. Prepare topographic derivatives such as slope or hill shade layers
    7. Clip all spatial data to a common project area extent

- **Available data collection** - The major contributions for the microzonation studies are the probabilistic assessment of the regional earthquake hazard, interpretation of the microtremor records, and interpretation of the available geological and geotechnical data based on a grid approach

- The compilation of the available Geological and Geotechnical data and additional subsurface explorations are carried out to supplement the available data.

- Evaluation and analysis of the available geotechnical data is done to determine the necessary parameters for conducting the microzonation with respect to different parameters.

- For the identification of the local soil conditions, an approach was chosen by taking available existing data into account.

- Data are available from different sources, with varying degree of information on the site investigations being conducted, reliability and quality of the derived data.

- Plausibility check of the available data is essential prior to carrying out the microzonation procedure; direct use of this kind of data from such a variety of different sources might lead to an unrealistic scenario, and might not be comparable or even withstand a subsequent confirmation of this approach in terms of the hypothetical boreholes.

- Nonetheless data from different sources should be taken into account if the quality appears to be acceptable so that it is possible to benefit from an independent view of the soil conditions in overall terms and the reliability of a single site investigation in particular.

Topic 2

3-D Subsurface Modeling of Geotechnical Data Using GIS

- Geographical information system (GIS) based subsurface model is developed which helps in data management, develop geostatistical functions, 3-dimensional (3-D) visualization of subsurface with geo-processing capability and future scope for web based subsurface mapping tool. The three major outcomes are:

    1. Development of digitized map of the area with layers of information
    2. Development of GIS database for collating and synthesizing geotechnical data available with different sources
    3. Development of 3-dimensional view of subsoil strata presenting various geotechnical properties such as location details, physical properties, grain size distribution, Atterberg limits, SPT ‘N’ values and strength properties for soil and rock along with depth in appropriate format.

- The 3-D subsurface model with geotechnical data has been generated with base map. The boreholes are represented as 3-dimensional objects projecting below the base map layer up to the available borehole depth, geotechnical properties are represented as layers at 0.5m intervals with SPT ‘N’ values (Fig 13.1).

- Each borehole is attached with geotechnical data versus depth. Also scanned image files of borelogs and properties table are attached to the location in plan. The data consists of visual soil classification, standard penetration test results, ground water level, time during which test has been carried out, and, other physical and engineering properties of soil.

- From this 3-D, geotechnical model, geotechnical information on any borehole at any depth can be obtained at every 0.5m interval by clicking at that level (donut).The model provides two options to view the data at each borehole

1. Visualize the soil character as colored layer with depth information along with properties in excel format and
2. Bore logs and properties as an image file.

A

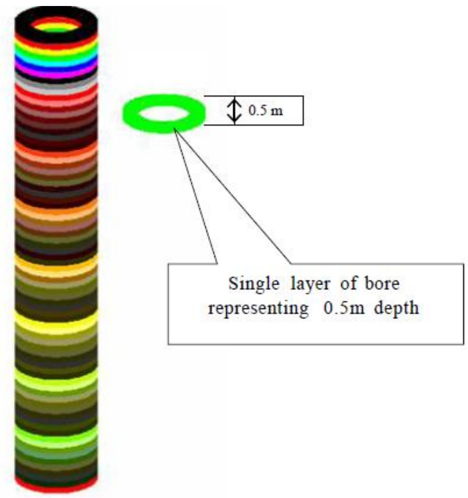


Fig 13.1: Typical Borehole in Three Dimensional View

Topic 3

Site characterization using Geological data

- Earthquake damage is commonly controlled by three interacting factors- source and path characteristics, local geological and geotechnical conditions and type of the structures. Obviously, all of this would require analysis and presentation of a large amount of geological, seismological and geotechnical data.

- The initial geological site characterization must focus on locating and quantifying the most fractured zones, for it is these highly fractured areas that will most significantly affect the evaluation of the site.

- The response of a soil deposit is dependent upon the frequency of the base motion and the geometry and material properties of the soil layer above the bedrock. Seismic microzonation is the process of assessment of the source & path characteristics and local geological & geotechnical characteristics to provide a basis for estimating and mapping a potential damage to buildings, in other words it is the quantification of hazard.

- Seismic microzonation would start with the assessment of the local geological formations and with the mapping of the surface geology based on available information, site surveys, site investigations, and soil explorations. The purpose is to determine the boundaries and the characteristics of the geological formation and to prepare a geology map at a scale of 1:5000 or larger.

- This map clearly indicates the geological formations and their variation however, it is important, as pointed out by Willis et al., (2000), to base the site classification on measured characteristics of geologic units taking into consideration the possible variations in each unit. The deviations from the mean values obtained for each geological unit may exceed the permissible limits to justify its use for assessing the effects of local soil conditions.

- Wills and Silva (1998) suggested using average shear wave velocity in the upper 30 m as one parameter to characterize the geological units while also 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.

Topic 4

Variation of rock depth or soil overburden thickness

- Spatial variability of the bed/hard rock with reference to ground surface is vital for many applications in geosciences. Rock depth in a site is very useful parameter to the geotechnical earthquake engineers to find their basic requirement of hard strata and ground motion at rock level.

- In the ground response analysis, Peak Ground Acceleration (PGA) and response spectrum for the particular site is evaluated at the rock depth levels and further on at the ground level considering local site effects. This is an essential step to evaluate site amplification and liquefaction hazards of a site and further to estimate induced forces on the structures.

- In ground response analysis, the response of the soil deposit is determined from the motion at the bed rock level. In all these problems, it is essential to evaluate the depth of the hard rock from the ground level.

- With an objective of predicting the spatial variability of the reduced level of the bed/hard rock in Bangalore, an attempt has been made to develop models based on Ordinary Kriging technique, Artificial Neural Network (ANN) and Support Vector Machine (SVM). It is also aimed at comparing the performance of these developed models for the available data in Bangalore.

- The kriging method was developed during the 1960s and 1970s and has been acknowledged as a good spatial interpolator (Matheron 1963; Isaaks and Srivastava 1989; Davis 2002). The most important features of this method are
    1. The unbiased estimate of results,
    2. The minimum estimation error, and
    3. Uncertainty evaluation of interpolation data points.

- The main goal of kriging is to predict the unknown properties from the knowledge of semi-variogram. Semi-variogram is the analytical tool used to evaluate and quantify the degree of spatial autocorrelation. The semi-variogram is an appreciation of the dispersion of the parameters, which equates to the variance and also gives an autocorrelation distance that represents the radius of influence of a measurement made at a given point.

- Further, it provides the type of variability that indicates how values fluctuate in space. A new method for cross-validation analysis of developed models has been also proposed and validated. The cross-validation of the model has been done based on the examination of residuals.

Topic 5

Geotechnical Explorations for Site Characterization

- Scope of the investigation 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 varies from 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 for all the significant factors that influence the seismic hazards. This objective is achieved only through proper planning of in situ and laboratory testing.

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

    1. 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.
    2. 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
    3. Define groundwater table conditions considering seasonal variations
    4. Perform in situ testing and procure samples for laboratory testing

- Various geotechnical tests including both in situ and laboratory tests are available for seismic microzonation which are discussed at length in the coming section. However, it is important to mention that such geotechnical investigations are suitable for depths upto 50 to 60 m, beyond these depths, undisturbed sampling becomes difficult.

Topic 6

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) (Fig. 13.1):

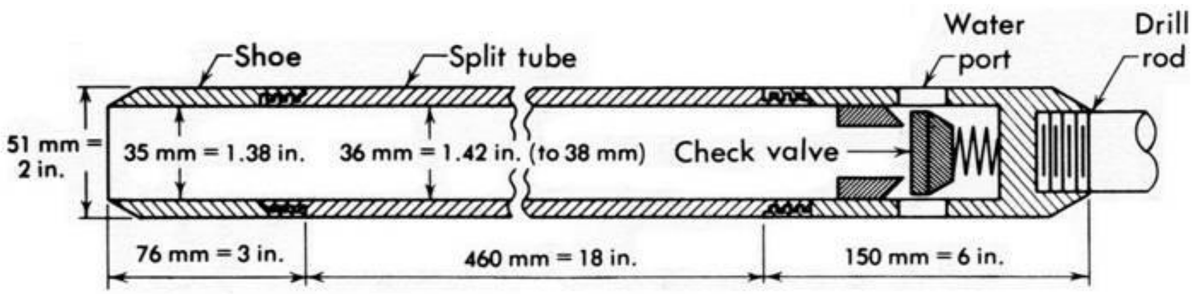


Fig. 13.2: A Standard Penetrometer

- SPT uses a thick-walled sample tube, with an outside diameter of 51 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm.

- The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm up to a depth of 450 mm is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value".

- In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground.

- 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:
  1. 50 blows are required for any 150 mm penetration
  2. 100 blows are required for any 300 mm penetration
  3. 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.

Topic 7

Corrections Applied for SPT "N" Values

- The SPT data collected is field 'N' values with out applying any corrections. Usually for engineering use of site response studies and liquefaction analysis the SPT "N" values has to be corrected with various corrections and a seismic borelog has to be obtained.

- The seismic bore log contains information about depth, observed SPT 'N' values, density of soil, total stress, effective stress, fines content, correction factors for observed "N" values, and corrected "N" value.

- The 'N' values measured in the field using Standard penetration test procedure have been corrected for various corrections, such as:

1. Overburden Pressure (C_N),
2. Hammer energy (C_E),
3. Borehole diameter (C_B),
4. presence or absence of liner (C_S),
5. Rod length (C_R) and
6. fines content (C_{fines})

- Corrected 'N' value i.e., $(N_1)_{60}$ is obtained using the following equation:

$$(N_1)_{60} = N \times (C_N \times C_E \times C_B \times C_S \times C_R \tag{13.1}$$

- **Correction for Overburden Pressure** - The effective use of SPT blow count for seismic study requires the effects of soil density and effective confining stress on penetration resistance to be separated. Consequently, Seed et al (1975) included the normalization of penetration resistance in sand to an equivalent σ'_w of one atmosphere as part of the semi empirical procedure.

- SPT N-values recorded in the field increases with increasing effective overburden stress; hence overburden stress correction factor is applied (Seed and Idriss 1982). This factor is commonly calculated from equation developed by Liao and Whitman (1986).

- However Kayen et al. (1992) suggested the following equation, which limits the maximum C_N value to 1.7 and provides a better fit to the original curve specified by Seed and Idriss (1982):

$$C_N = 2.2/(1.2 + \sigma'_{vo} / P_a) \tag{13.2}$$

- Where, σ'_w = effective overburden pressure, Pa = 100 kPa, and C_N should not exceed a value of 1.7. This empirical overburden correction factor is also recommended by Youd et al (2001). For high pressures (300kPa), which are generally below the depth for which the simplified procedure has been verified, C_N should be estimated by other means (Youd et al, 2001).

- **Correction for hammer energy ratio** - Another important factor which affects the SPT 'N' value is the energy transferred from the falling hammer to the SPT sampler. The energy ratio (E_R) delivered to the sampler depends on the type of hammer, anvil, lifting mechanism and the method of hammer release. Approximate correction factors to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 13.2 (Robertson and Wride 1998).

Table 13.2: Hammer correction factors (Robertson and Wride 1998)

Type of Hammers	Notation	Range of correction
Donut Hammer	C_E	0.5-1.0

Safety Hammer	C_E	0.7-1.2
Automatic-trip Donut Hammer	C_E	0.8-1.3

- Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor C_{ER} has been observed as noted in the table. Even when procedures are carefully monitored to confirm the established standards some variation in C_E may occur because of minor variations in testing procedures.

- Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. The workshop participants of NCEER 1996 & 1998 (Youd et al, 2001) recommend measurement of the hammer energy frequently at each site where the SPT is used.

- Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a C_E value. Use of good-quality testing equipment and carefully controlled testing procedures will generally yield more consistent energy ratios.

- For Liquefaction calculation Yilmaz and Bagci (2006) had taken the C_E value as 0.7 for SPT hammer energy donut type for soil liquefaction susceptibility and hazard mapping in Kutahya, Turkey. Similar kind of hammer is used for soil investigations; hence the value of 0.7 is taken for C_E.

- **Other correction factors** - The other correction factors adopted such as correction for borehole diameter, rod length and sampling methods modified from Skempton (1986) and listed by Robertson and Wride (1998) are presented in Table 13.2.

- Correction for borehole diameter (C_B) is used as 1.05 for 150 mm borehole diameter, Rod length (C_R) is taken from the Table 13.3, based on the rod length the presence or absence of liner (CS) is taken as 1.0 for standard sampler.

- The corrected "N" Value $(N_1)_{60}$ is further corrected for fines content based on the revised boundary curves derived by Idriss and Boulanger (2004) for cohesionless soils as described below:

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \tag{13.3}$$

$$\Delta(N_1)_{60} = \exp\left[1.63 + \frac{9.7}{FC+0.001} - \left(\frac{15.7}{FC+0.001}\right)^2\right] \tag{13.4}$$

FC = percent fines content (percent dry weight finer than 0.074mm).

Table 13.3: Correction factors for Borehole Diameter (C_B), Rod Length (C_R) and Sampling Method (C_S)

Factor	Equipment Variable	Notation	Correction
Borehole Dia.	65-115mm	C_B	1.00
Borehole Dia.	150mm	C_B	1.05
Borehole Dia.	200mm	C_B	1.15
Rod Length	<3m	C_R	0.75
Rod Length	3-4m	C_R	0.80
Rod Length	4-6m	C_R	0.85
Rod Length	6-10m	C_R	0.95
Rod Length	10-30m	C_R	1.00
Sampling method	Standard samplers	C_S	1.00
Sampling method	Sampler without liners	C_S	1.1-1.3

Topic 8

Interpretation of SPT N_{30}

- The following factors can affect the SPT results:
    1. nature of the drilling fluid in the borehole,
    2. diameter of the borehole,
    3. The configuration of the sampling spoon and the frequency of delivery of the hammer blow.

- Therefore, it should be noted that drilling and stabilisation of the borehole must be carried out with care. The measured N-value (blows/0.3 m) is the so-called standard penetration resistance of the soil. The penetration resistance is influenced by the stress conditions at the depth of the test.

- The resistance (N_{30}) has been correlated with the relative density of granular soils. Sand and gravel can be classified as shown in Table 13.4.

Table 13.4: Classification of sand and gravel

SPT N Value	Relative density	Classification
0-4	0-15	Very loose
4-10	15-35	Loose
10-30	35-65	Medium dense
30-50	65-85	Dense
>50	85-100	Very dense

- The sources of some of the common errors while carrying out SPT tests are listed in Table 13.5 (Kulhawy and Mayne, 1990).

Table 13.5: 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 ungreased 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

Topic 9

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 (IP). The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction.

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

- Permeability characteristics of the soils can be determined using falling head or fixed head Permeameter as per BIS: 2720 (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:
    1. Unconfined Compressive Strength of rock samples [BIS:9143-1979]
    2. Dynamic Modulus of rock core specimen, [BIS:10782-1983]
    3. Modulus of Elasticity, Poisson's Ratio, in uniaxial compression [BIS:9221-1979]
    4. 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 XII) – 1981. Triaxial Shear tests comprise UU, CU (Consolidated Undrained test with and without Pore Water Pressure Measurement) or CD (consolidated drained) tests.

Topic 10

Site Classification using SPT data

- GIS database for collating and synthesizing geotechnical data available with different sources and 3-dimensional view of soil stratum presenting various geotechnical parameters with depth in appropriate format should be developed.

- In the context of prediction of reduced level of rock (called as "engineering rock depth" corresponding to about SPT "N" >100) in the subsurface and their spatial variability evaluated using Artificial Neural Network (ANN).

- Observed SPT 'N' values are corrected by applying necessary corrections, which can be used for engineering studies such as site response and liquefaction analysis.

- The site characterization is attempted using geotechnical bore log data and standard penetration test "N" values. The Standard Penetration Test (SPT) is one of the oldest, most popular, and commonly used in situ test for exploration in soil mechanics and foundation engineering because the equipment and test procedures are simple.

- The Standard Penetration Test (SPT) is particularly useful for seismic site characterizations, site response, and liquefaction studies towards seismic microzonation. In most cases the site specific response analysis, shear wave velocity, and shear modulus (Gmax) of layers are estimated using relationships based on the SPT N values (Anbazhagan and Sitharam, 2010).

Topic 11

Cone Penetration Test (CPT)

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

- **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) (Fig. 13.3).

- 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.

- **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).

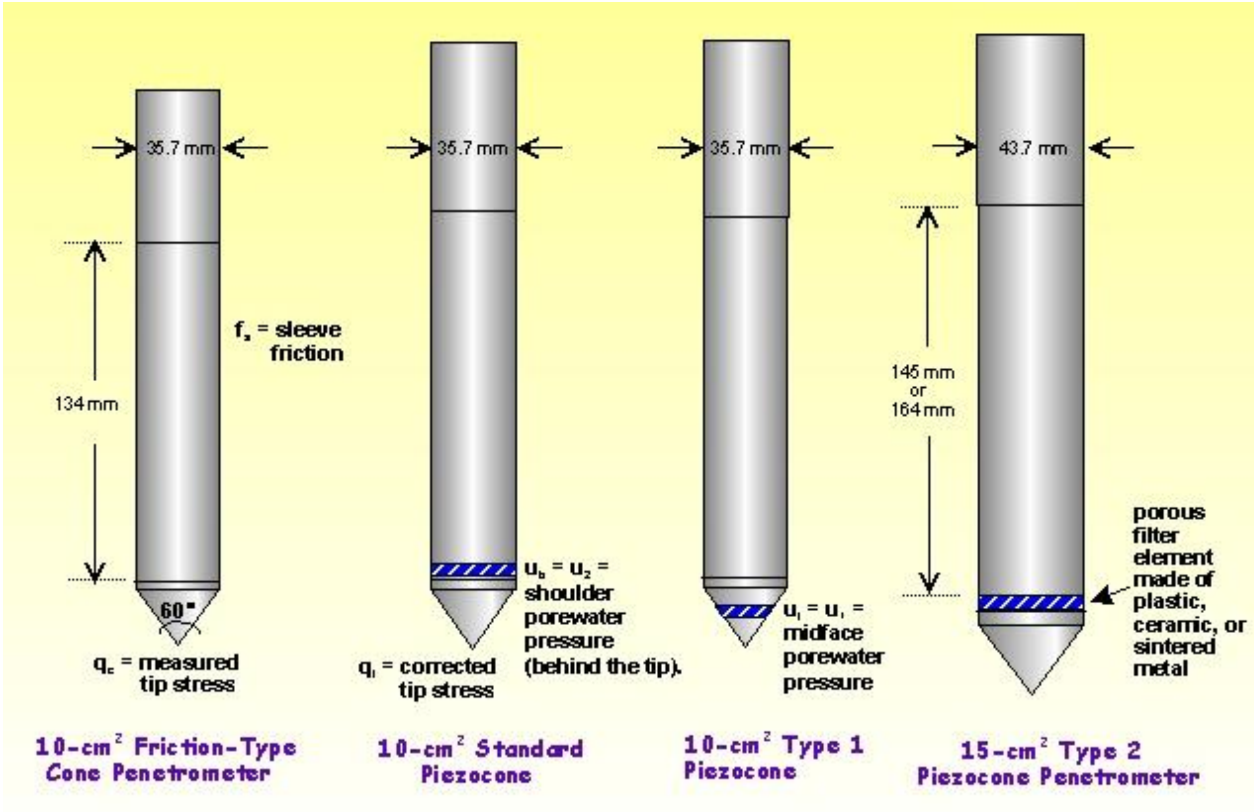


Fig. 13.3: Different types of Cones used in CPT test (http://geosystems.ce.gatech.edu/Faculty/Mayne/Research/devices/cpt.htm)

- **Seismic Cone Penetration Test (SCPT) -** The seismic cone penetration test uses a standard cone penetrometer with two geophones. One set of geophones is located behind the friction jacket and the other set is located one meter above the first set (Fig. 13.4).

- 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.

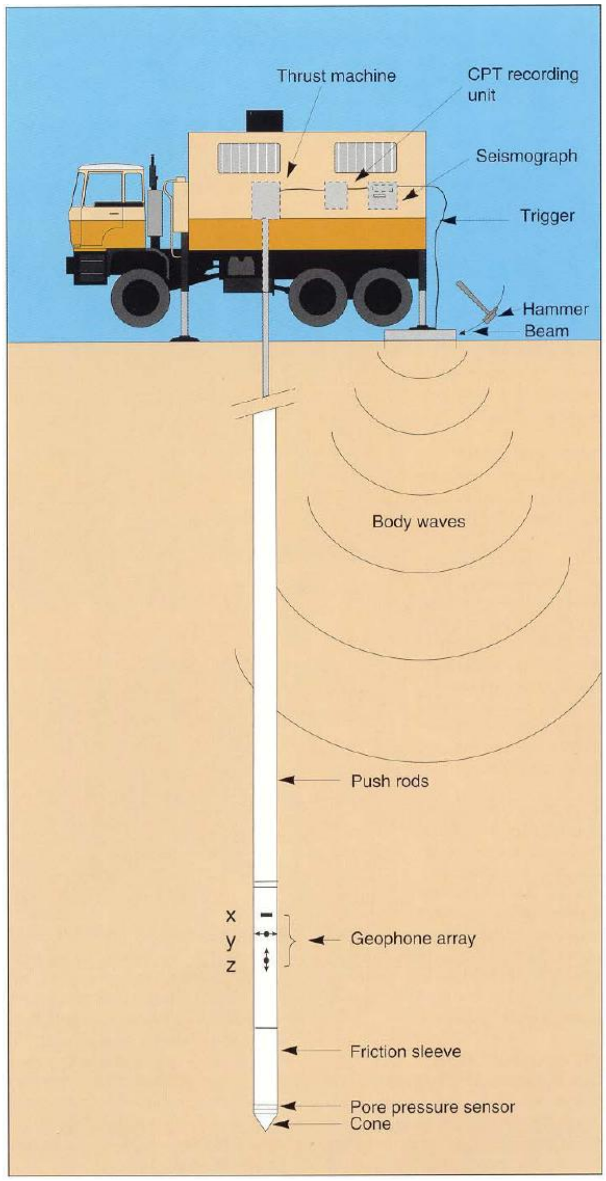


Fig. 13.4: Seismic Cone Penetration test (Fugro Company)

Topic 12

Site Characterization by Cone Penetration Testing

- Cone penetration testing (CPT) is a fast and reliable means of conducting site investigations for exploring soils and soft ground for support of embankments, retaining walls, pavement subgrade, bridge foundations etc. The CPT soundings can be used either as a replacement or complement to conventional rotary drilling and sampling methods.

- In CPT, an electronic steel probe is hydraulically pushed to collect continuous readings of point load, friction, and pore water pressures with typical depths up to 30 m (100 ft) or more reached in about 1 to 1 1⁄2 h.

- Data are logged directly to a field computer and can be used to evaluate the geostratigraphy, soil types, water table, and engineering parameters of the ground by the geotechnical engineer on-site, thereby offering quick and preliminary conclusions for design. With proper calibration, using full-scale load testing coupled with soil borings and lab- oratory testing, the CPT results can be used for final design parameters and analysis.
- In its simplest application, the cone penetrometer offers a quick, expedient, and economical way to profile the subsurface soil layering at a particular site. No drilling, soil samples, or spoils are generated; therefore, CPT is less disruptive from an environmental standpoint.

- The continuous nature of CPT readings permit clear delineations of various soil strata, their depths, thicknesses, and extent, perhaps better than conventional rotary drilling operations that use a standard drive sampler at 5-ft vertical intervals. Therefore, if it is expected that the subsurface conditions contain critical layers or soft zones that need detection and identification, CPT can locate and highlight these particular features.

Topic 13

Corrections to CPT

- For electric cones that record pore pressure, corrections can be made to account for unequal end area effects. Baligh et al. (1981) and Campanella et al (1982) proposed that the cone resistance, qc, could be corrected to a total cone resistance, qt, using the following expression:

$$q_t = q_c + (1 - a)u \tag{13.5}$$

where u is pore pressure measured between the cone tip and the friction sleeve and a is net area ratio. It is often assumed that the net area ratio is given by

$$a = \frac{d^2}{D^2} \tag{13.6}$$

where d is diameter of load cell support and D is diameter of cone. However, this provides only an approximation of the net area ratio, since additional friction forces are developed due to distortion of the water seal O-ring.

- Therefore, it is recommended that the net area ratio should always be determined ‘in a small caliion vessel (Battaglio and Mankcalco 1983; Campanella and Robertson 1988). A similar correction can also be applied to the sleeve friction (Iunne ez al. 1986; Konrad 1987). Konrad (1981) suggested the following expression for the total stress sleeve friction, ft:

$$f_t = f_s - (1 - \beta b)cu \tag{13.7}$$

$$\text{Where, } b = \frac{A_{st}}{A_{sb}},\ c = \frac{A_{sb}}{A_s},\ \beta = \frac{u_s}{u} \tag{13.8}$$

- A_{st} is end area of friction sleeve at top, A_{sb} is end area of friction sleeve at bottom, A_s, is outside surfacea area of friction sleeve, and u_s, is pore pressure at top of friction sleeve.

- However, to apply this correction, pore pressure data are required at both ends of the friction sleeve. Konrad (1987) showed that this correction could be more than 30% of the measured f_s, for some cones. However, the correction can be significantly reduced for cones with an equal end area friction sleeve (i.e., b=1.0).

- The corrections in cone resistance and sleeve friction are only important in soft clays and silts where high pore pressure and low cone resistance occur. The corrections are negligible in cohensionless soils where penetration is generally drained and cone resistance is generally large. The author believes that the correction to the sleeve friction is generally unnecessary provided the cone has an equal end area friction sleeve.

Topic 14

CPT Profile, Downhole Memphis

- By recording three continuous measurements vertically with depth, the CPT is an excellent tool for profiling strata changes, delineating the interfaces between soil layers, and detecting small lenses, inclusions, and stringers within the ground.

- The data presentation from a CPT sounding should include the tip, sleeve, and porewater readings plotted with depth in side-by-side graphs, as shown in Figure 13.5.

- The total cone tip resistance (qt) is always preferred over the raw measured value (qc). For SI units, the depth (z) is presented in meters (m), cone tip stress(qt) in either Pascal (MPa or kPa), and sleeve resistance (fs) and porewater pressure (um) in kPa.

- If the depth of the water table is known (Zw), it is convenient to show the hydrostatic pore water pressure (u_0), if the groundwater regime is understood to be an unconfined aquifer (no drawdown and no artesian conditions). In that case, the hydrostatic pressure can be calculated from:

$$u_0 = (Z - Zw).\gamma_w$$

- Where $\gamma_w = 9.8 kN/m^3$, in some CPT presentations, it is common to report the um reading in terms of equivalent height of water, calculated as the ratio of the measured pore water pressure divided by the unit weight of water.

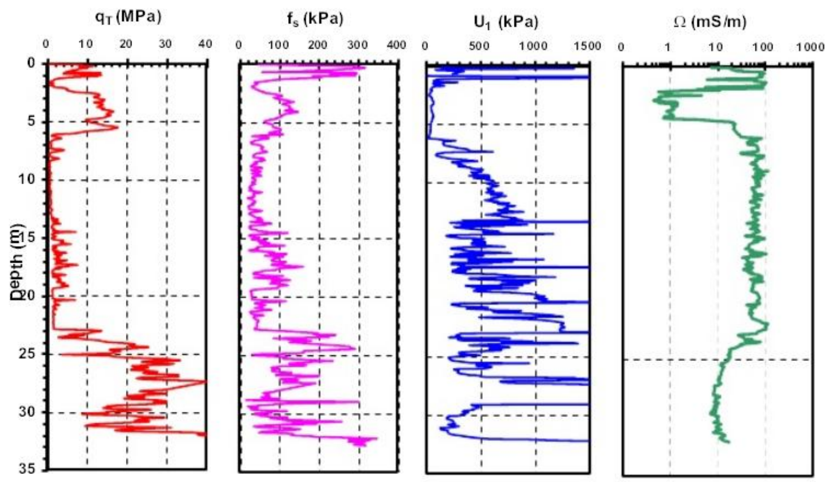


Fig 13.5: Example of Conductivity Piezocone Test at Mud Island, Memphis, Tennessee.

Topic 15

Comparison CPT and SPT, Downhole Memphis

- A direct comparison between CPT and SPT at the same location would be ideal. The pair shown in above Fig. 13.6 is approximately 230 m apart. both tests are in the geologic unit afbm (artificial fill over San Francisco Bay Mud).

- The stratigraphy is relatively consistent for the CPT and SPT with 2–3 m of sands and gravels (fill), over about 2 m of silts and clays, over 6–8 m of sands and silty sands. The CPT stratigraphy tends towards silts and clays whereas the SPT stratigraphy tends towards gravel; however, because of the separation distance, we cannot conclude if this is a bias of the Ic-based soil types for the CPT or the result of natural spatial variation in the geologic deposit.

- Both CPT and SPT identify potentially liquefiable material between 2 and 6 m, above and below the silt/clay layer. The primary difference between the methods is that the CPT also shows an additional liquefiable layer between 9 and 11 m, which is missed in the SPT data. The LPI for these locations are 12.7 for the CPT and 6.9 for the SPT.

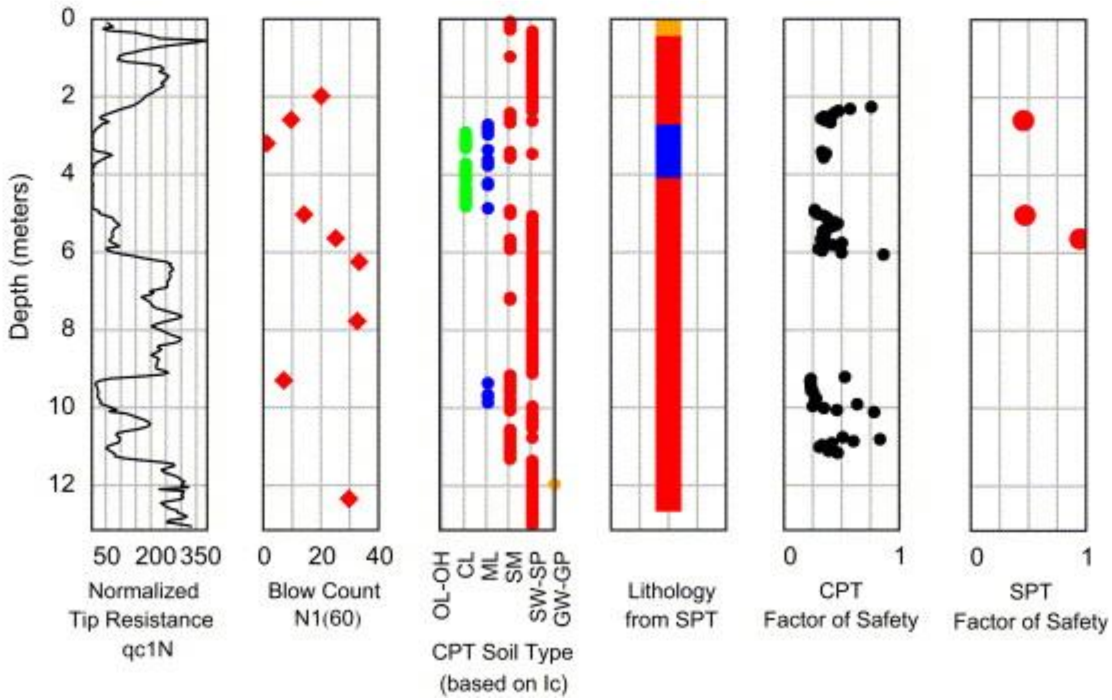


Fig 13.6: Comparison between CPT and SPT location, the distance between these locations is approximately 230 m and both are located in the surficial geology unit afbm (artificial fill over San Francisco Bay Mud). The characteristics compared are soil density (normalized tip resistance for the CPT and N-values for the SPT), stratigraphy, and the factor of safety against liquefaction.

Topic 16

CPT Soil Behavioral Classification

- A new soil behaviour type classification system has been presented using normalized cone penetration test parameters. The new charts represent a three-dimensional classification system incorporating allo three pieces of data from CPTu.

- The charts are global in nature and can be used to define soil behaviour type. Factors such as changes in stress history, in situ stresses, sentivity, stiffness, macrofabric, and void ration will also influence the classification.

- A guide to the influence some of these variables have on the classification has been included on the charts. Occasionally soil will fall within different zones on each chart. In these cases the rate and manner in which the excess pore pressures dissipate during a pause in the penetration can significantly aid in the classification.

- Some of the most comprehensive recent work on soil classification using electric cone penetrometer data was presented by Douglas and Olsen (1981). One important distinction made by them was that CPT classification charts cannot be expected to provide accurate predictions of soil type based on grain size distribution but can provide a guide to soil behaviour type.

- The CPT data provide a repeatable index of the aggregate behavior of the in-situ soil in the immediate area of the probe. An example of a soil classification chart for electric CPT data is shown in Fig. 13.7 and details are given in Table 13.6

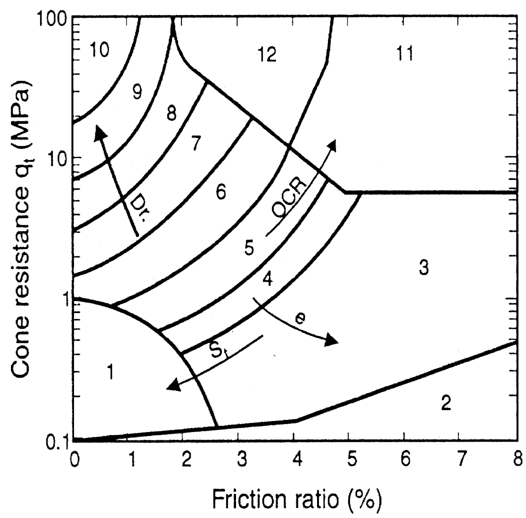


Fig 13.7: Simplified soil behavior type classification for standard electric friction cone (Robertson et al. 1986)

Table 13.6: Soil Behavior Type from CPT Classification Index, Ic (after Jefferies and Davies, 1993)

Soil Classification	Zone Number*	Range of CPT Index I_c Values
Organic Clay soils	2	I_c>3.22
Clays	3	2.82< I_c>3.22
Silt Mixtures	4	2.54< I_c>2.82
Sand Mixtures	5	1.90< I_c>2.54
Sands	6	1.25< I_c>1.90
Gravelly Sands	7	I_c<1.25

*Notes: Zone number as per Robertson SBT (1990)

Topic 17

CPT Tests to Evaluate Seismic Ground Hazards

- A series of cone penetration tests (CPTs) are conducted for quantifying seismic hazards, obtaining geotechnical soil properties, and conducting studies at liquefaction sites. The seismic piezocone provides four independent measurements for delineating the stratigraphy, liquefaction potential, and site amplification parameters.

- At the same location, two independent assessments of soil liquefaction susceptibility can be made using both the normalized tip resistance (qc_{1N}) and shear wave velocity (Vs_1). In lieu of traditional deterministic approaches, the CPT data can be processed using probability curves to assess the level and likelihood of future liquefaction occurrence.

- The cone penetrometer system used in these tests included an anchored truck-mounted hydraulic rig with field computer data acquisition and three geophysics-type penetrometers (5-, 10-, and 15-ton capacity). Each penetrometer consists of a 60° angled apex at the tip instrumented to measure five independent readings: tip resistance (q_c), sleeve friction (f_s), vertical inclination (i), penetration porewater pressure (either midface u_1 or shoulder u_2), and downhole shear wave velocity (V_s). Shear waves are recorded at 1-m depth intervals, whereas the other readings are obtained at a constant logging rate, generally set between 1 and 5 cm/s.

- The tip resistance (q_c) is a point stress related to the soil strength and the reading must be corrected for porewater pressure effects on unequal areas, especially in clays and silts. The corrected value is termed q_T. The sleeve resistance relates to the interface friction between the penetrometer and soil. Magnitudes of porewater pressure depend upon the permeability of the medium and the shoulder filter element (or u_2 position) is required for the tip correction.

- The tip resistance (q_T), sleeve friction (f_s) and pore pressure (u_2) are used together to characterize the subsurface layering, soil behavioral type, and strength properties. Particularly important in seismic investigations, a cyclic stress-based analysis of liquefaction-prone sediments is available using the q_T data.

- The seismic piezocone test ($SCPT_u$) includes both penetration readings and down hole geophysical measurements in the same sounding, thus optimizing data collection at a given location.

- In the test procedure, the shear waves are generated by striking a horizontal steel plank that is coupled to the ground under an outrigger. The downhole geophone is oriented parallel to the plank to detect vertically propagating, horizontally polarized shear waves. From the measured wave train at each depth, a pseudo-interval shear wave velocity (V_s) is determined as the difference in travel distance between any two successive events divided by the difference in travel times.

- The travel times are determined in two ways: (1) by visually inspecting the recorded wave traces and subjectively identifying the first arrival, and (2) by a rigorous post-processing technique known as cross-correlation to determine the time shift between the entire wave trains from successive paired records.

Topic 18

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 aimed at the following information
- Depth of the bedrock
- Very small strain stiffness of the ground
- To study variability of soils

- 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.

Topic 19

Surface Wave Methods

- Many geophysical methods are attempted for seismic site characterization, but widely used methods are Spectral Analysis of Surface Waves (SASW) and Multichannel Analysis of Surface Waves (MASW). SASW and MASW are surface wave methods widely used for many civil and earth science applications

- Historically, most of the surface wave applications have followed three fundamental steps:

  1. Acquisition
  2. Dispersion Analysis
  3. Seeking the layered-earth model (Vs, Vp, h, r, etc.)

- The main topics of development in recent history have been field procedures (data acquisition) and data processing (dispersion and inversion analyses). Early pioneering work in surface waves goes back to 1950s when the steady state method was first used by Van der Pol (1951) and Jones (1955).

- At this time, it was based on the fundamental-mode (M0)-only Rayleigh wave assumption and all other types of waves higher modes, body waves, etc. were ignored. This method then evolved later to be more-commonly called Continuous Surface Wave (CSW) method (Matthews et al., 1996).

- In the meantime, the soil site inversion theory was refined by Tokimatsu et al. (1991). Since the very early stage of the surface wave application, pavement was found to be more complex than soil (Sezawa, 1938; Press and Dobrin, 1956), with a special type of guided wave called leaky waves that required a complex-domain approach in solving wave equations (Jones, 1962; Vidale, 1964).

- A modern computer approach was introduced later by Martincek (1994), but it still produced limited results. 20th century when Jones (1961) and other investigators used small vibrators as wave experienced a boom in the mid-

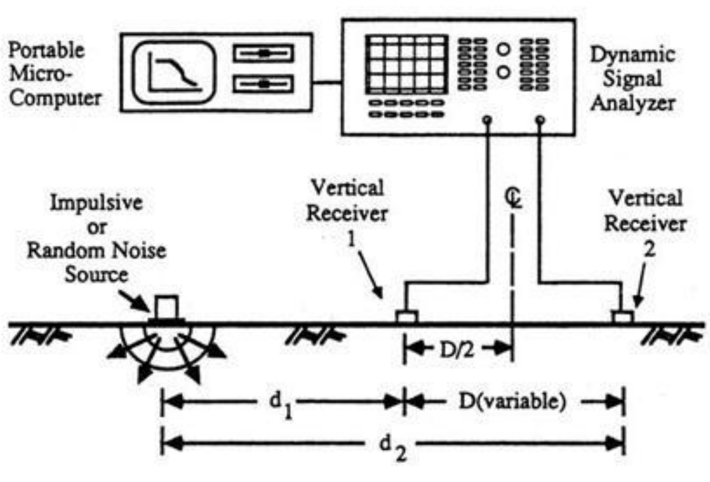
1980s when digital computers became popular. A brief coverage of this historical development can be found in the 2005 special issue of JEEG (Journal of Environmental and Engineering Geophysics) on the surface wave method. Another historical overview can be found in Park and Ryden (2007).

Topic 20

Two-Receiver Approach (The SASW Method)

- In early 80s, a two-receiver approach was introduced by investigators at the University of Texas (UT), Austin, that was based on the Fast Fourier Transform (FFT) analysis of phase spectra of surface waves generated by using an impulsive source like the sledge hammer (Figure 13.8). It then became widely used among geotechnical engineers and researchers. This method was called Spectral Analysis of Surface Waves (SASW) (Heisey et al., 1982).

- The fundamental-mode (M_0)-only Rayleigh wave assumption was used during the early stages. Simultaneous multi-frequency (not mono-frequency) generation from the impact seismic source and then separation by FFT during the subsequent data processing stage greatly improved overall efficiency of the method in comparison to earlier methods such as the continuous surface wave (CSW) method. Since then, significant research has been conducted at UT-Austin (Nazarian et al., 1983; Rix et al., 1991; Al-Hunaidi, 1992; Gucunski and Woods, 1992; Aouad, 1993; Stokoe et al., 1994; Fonquinos, 1995; Ganji et al., 1998) and a more complete list of the publications on SASW up to early 1990s can be found in “Annotated bibliography on SASW” by Hiltunen and Gucunski (1994). The overall procedure of SASW is as follows.



From Rix et al. (1991)

Figure 13.8: Schematic representation of overall procedure of the SASW method

a. Field setup with different separations (D's),
b. Data processing for phase velocity (Vph): Vph=2*pi*f / dp (dp=phase difference, f=frequency, pi=3.14159265)), and
c. Wavelength (L) filtering criteria—compact dispersion curve

- Earlier research of SASW method was focused on ways to enhance accuracy of the fundamental-mode (M_0) Rayleigh-wave dispersion curve through field procedure and data processing efforts. Then soon came the speculation about the possibility of the curve "being more than M_0" and subsequently higher modes (HM's) were included in the studies (Roesset et al., 1990; Rix et al., 1991; Tokimatsu et al., 1992; Stokoe et al., 1994). In consequence, the concept of "apparent (or effective)" dispersion-curve (Gucunski and Woods, 1992; Williams and Gucunski, 1995) was introduced that accounts for the possible mixture of multiple influences rather than M0 alone (Fig. 13.9).

- Once multiple modes were recognized and included, the field approach and data processing techniques attempted to account for the multiple-mode possibilities. Pavement investigation by SASW was regarded quite challenging, especially for base layers, and the possibility of multi-modal superimposition was speculated as being responsible for this. Reported difficulties with SASW fit into the following three main categories:

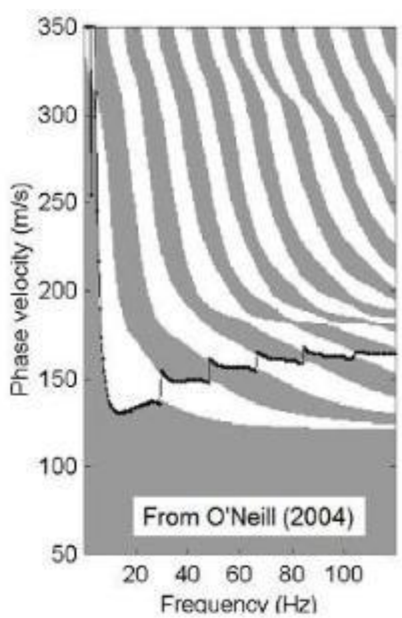


Figure 13.9: The apparent dispersion concept in the SASW method.

1. Higher modes (HM's) inclusion that was previously underestimated,
2. Inclusion of other types of waves (body, reflected and scattered surface waves, etc.) (Sheu et al., 1988; Hiltunen and Woods, 1990;

Foti, 2000) that was also underestimated or not considered at all, and
3. Data processing, for example, phase unwrapping (Al-Hunaidi, 1992) during the phase-spectrum analysis to construct a dispersion curve.

Topic 21

Multichannel Approach (MASW)

- In early 2000s, the MASW (Multichannel Analysis of Surface Waves) method came into popular use among the geotechnical engineers. The term “MASW” originated from the publication made on Geophysics by Park et al. (1999).

- The project actually started in mid-90s at the Kansas Geological Survey (KGS) by geophysicists who had been utilizing the seismic reflection method—long used in the oil industry to image the interior of the earth for depths of several kilometers. Called the high-resolution reflection method, it was used to image very shallow depths of engineering interest (e.g., 100 m or less).

- It was in the mid-90s when KGS started a project to utilize surface waves. Knowing the advantages with the multichannel method proven throughout almost half-century of its history for exploration of natural resources, their goal was a multichannel method to utilize surface waves mainly for the purpose of geotechnical engineering projects.

- From the extensive studies performed by SASW investigators, they acknowledged that surface wave properties must be more complex than previously assumed or speculated, and that the two-receiver approach had clearly reached its limitation to handle the complexity.

- Based on the normal notion that the number of channels used in seismic exploration can directly determine resolving power of the method, they utilized diverse techniques already available after a long history of seismic data analysis (Telford et al., 1976; Robinson and Treitel, 1980; Yilmaz, 1987) and also developed new strategies in field and data processing to detail surface wave propagation properties and characterized key issues to bring out a routinely-useable seismic method.

- The first documented multichannel approach for surface-wave analysis goes back to early 80s when investigators in Netherlands used a 24-channel acquisition system to deduce shear-wave velocity structure of tidal flats by analyzing recorded surface waves (Gabriels et al., 1987). It first showed the scientific validity of the multi channel approach in surface wave dispersion analysis and, in this regard, the study can be regarded as a feasibility test of the approach for routine use in the future.

- Then, using uncorrelated Vibroseis data, Park et al. (1999) highlighted the effectiveness of the approach by detailing advantages with multichannel acquisition and processing concepts most appropriate for the geotechnical engineering applications. A subsequent boom in surface wave applications using the MASW method for various types of geotechnical engineering projects has been observed worldwide since that time. There were a few other applications of multichannel approach to aid oil-exploration reflection surveys (Al-Husseini et al., 1981; Mari, 1984).

Topic 22

What is MASW?

- First introduced in GEOPHYSICS (1999), the multichannel analysis of surface waves (MASW) method is one of the seismic survey methods evaluating the elastic condition (stiffness) of the ground for geotechnical engineering purposes. MASW first measures seismic surface waves generated from various types of seismic sources such as sledge hammer analyzes the propagation velocities of those surface waves, and then finally deduces shear-wave velocity (Vs) variations below the surveyed area that is most responsible for the analyzed propagation velocity pattern of surface waves.

- Shear-wave velocity (Vs) is one of the elastic constants and closely related to Young's modulus. Under most circumstances, Vs is a direct indicator of the ground strength (stiffness) and is therefore commonly used to derive load-bearing capacity. After a relatively simple procedure, final Vs information is provided in 1-D, 2-D, and 3-D formats.

- **Advantages of the MASW Method**

    1. Unlike the shear-wave survey method that tries to measure directly the shear-wave velocities which is notoriously difficult because of difficulties in maintaining favorable signal-to-noise ratio (S/N) during both data acquisition and processing stages. MASW is one of the easiest seismic methods that provide highly favorable and competent results.
    2. Data acquisition is significantly more tolerant in parameter selection than any other seismic methods because of the highest signal-to-noise ratio (S/N) easily achieved. This most favorable S/N is due to the fact that seismic surface waves are the strongest seismic waves generated that can travel much longer distance than body waves without suffering from noise contamination (See Figure 13.10)
    3. Because of an increased ability to discriminate useful signal from

harmful noise, the MASW method assures an increased resolution when extracting signal in the midst of noise that can be anything from natural or cultural activities (wind, thunder, traffic, etc.) to other types of inherent seismic waves generated simultaneously (higher-mode surface waves, body waves, bounced waves, etc.)

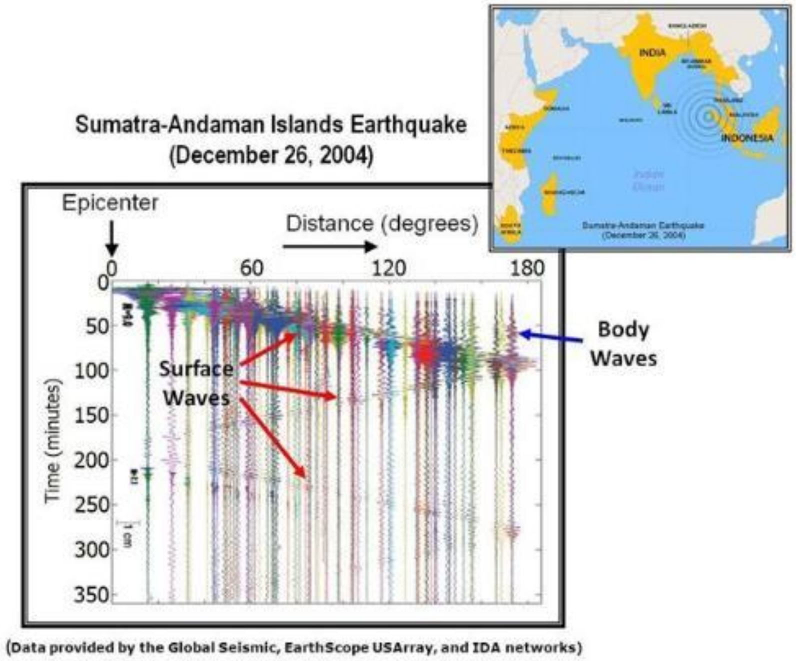


Figure 13.10: Earthquake recording around the earth showing strong surface waves

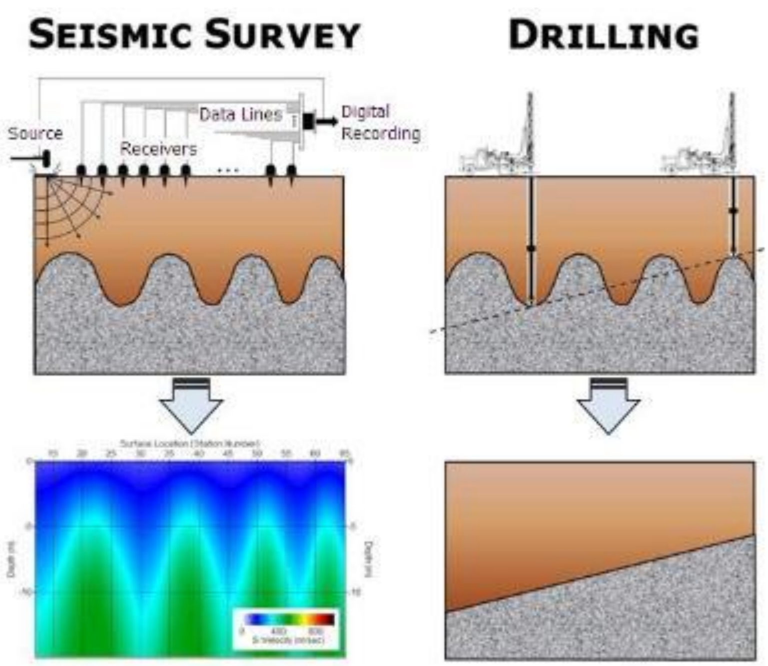


Figure 13.11: Comparison of seismic survey and conventional drilling

4. In consequence, overall field procedure for data acquisition and subsequent data-processing step becomes highly effective and tolerant, rendering a non-expert method (Fig 13.11).
5. The multichannel seismic concept is analogous to resolution in digital imaging technology (Figure 13.12). As the higher number of bits available, a broader color resolution is achieved, whereas the higher image resolution is achieved as more pixels are used to capture the image. The concept of number of channels plays similar roles to those by the bit and pixel concepts in delineating the subsurface information.

Figure 13.12: An analogy of the seismic multichannel approach to digital imaging concepts of number of bits and pixels

- **Overall Procedure of MASW Survey -** The common procedure for (1-D, 2-D, and 3-D) MASW surveys usually consist of three steps (Figure 13.13)

1. Data Acquisition - acquiring multichannel field records (commonly called shot gathers in conventional seismic exploration)
2. Dispersion Analysis - extracting dispersion curves (one from each record)
3. Inversion - back-calculating shear-wave velocity (Vs) variation with depth (called 1-D Vs profile) that gives theoretical dispersion curves closest to the extracted curves (one 1-D Vs profile from each curve).

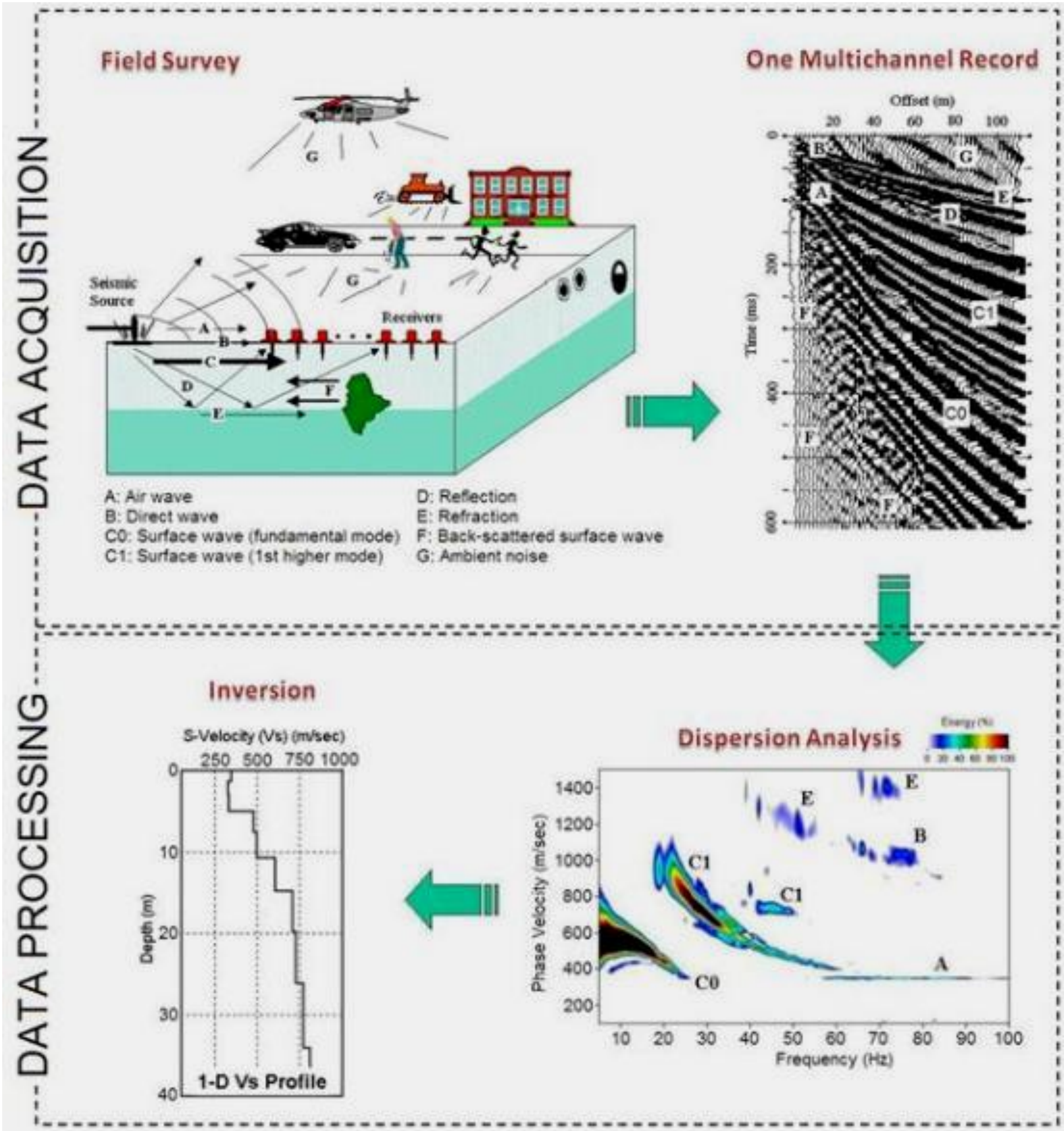


Figure 13.13: Common procedure for MASW surveys for 1-D, 2-D, and 3-D Vs mapping.

Topic 23

Downhole Shear Wave Velocity

- Steps involved in finding out the downhole shear wave velocity are:
    1. Anchoring System
    2. Automated Source
    3. Polarized Wave

4. Downhole Vs

- In the down-hole test, an impulse source is located on the ground surface adjacent to the borehole. A single receiver that can be moved to different depths, or a string of multiple receivers at predetermined depths, is fixed against the walls of the borehole, and a single triggering receiver is located at the energy source.

- All receivers are connected to a high speed recording system so that their output can be measured as a function of time. The objective of the downhole test is to measure the travel times of the p and/or s-waves from the energy source to the receivers.

- By properly locating the receiver positions, a plot of travel time versus depth can be generated. The slope of the travel-time curve at any depth represents the wave propagation velocity at that depth.

- With an SH-wave source, the down-hole test measures the velocity of waves similar to those that carry most seismic energy to the ground surface. Because the waves must travel through all materials between the impulse source and receivers, the down-hole test allows detection of layers that can be hidden in seismic refraction surveys.

- Potential difficulties with down-hole tests and their interpretation can result from disturbance of the soil during drilling of the borehole, casing and borehole fluid effects, insufficient or excessively large impulse sources, background noise effects.

- The effects of material and radiation damping on wave forms can make identification of s-wave arrivals difficult at depths greater than 30-60 m.

Topic 24

Automated Seismic Source

- To improve upon the downhole testing program, an automatic seismic source was developed for use in seismic piezocone testing. A new source, named the AutoSeis, was initially tested at the national geotechnical experimentation site in Spring Villa, Alabama and compared to available crosshole data to assess its ability to meet the primary and secondary goals.

- Later testing was conducted at two test sites in the Mid-America earthquake region near Memphis. With reliable shear waves generated to a depth of 20 m, the first iteration of the AutoSeis has proven successful and has provided the necessary information for the design of an improved version.

- In order to improve upon the downhole testing program and accuracy of the geophysical results, the decision was made to develop an automatic seismic source for use in seismic piezocone testing. It was determined that the current source, which consists of a 2.3 kg sledgehammer and steel beam, although mechanically adequate, could be improved to increase both consistency and productivity.

- This new source, named the AutoSeis, would have to meet certain design and performance criteria. The AutoSeis would have to be small, portable, and easy to use. It would also need to generate shear waves that are more uniform and repeatable. The first iteration of the AutoSeis source and control box can be seen in Figure 13.14.

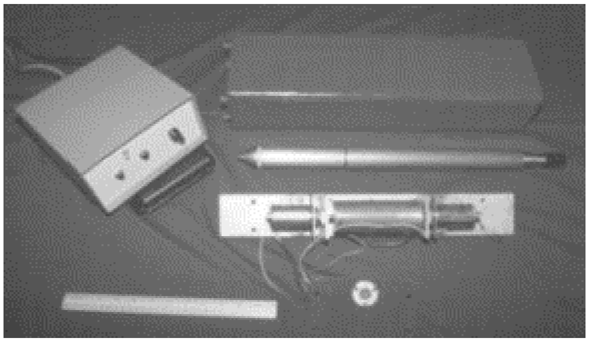


Fig 13.14: AutoSeis components, from left, control box, source frame, typical seismic cone, and internal chassis.

Topic 25

Downhole Shear Waves

- Shear-wave velocity profiles obtained from downhole surveys are routinely incorporated in site response modeling for earthquake hazard evaluation and structural design.

- A downhole seismic survey (also called a borehole velocity survey) is conducted by measuring the time for seismic waves generated by an impulsive source at the surface to travel to a sensor located at a sequence of depths in the borehole.

- The sensor consists of three geophones arranged in an X-Y-Z pattern. Two orthogonal horizontal geophones are used to detect shear-wave (S-wave) arrivals and a vertical geophone is used to detect compression-wave (P-wave) arrivals. At each measurement level, the sensor assembly is locked to the borehole wall using a clamping mechanism so that the geophones will couple with the seismic signals propagating in the earth.

- The downhole P wave velocity log is derived using either a 12- or 24-channel hydrophone array. This array is moved incrementally either up or down the borehole; a surface source (commonly a 12-gauge Buffalo gun fired in a shallow hole) is placed close to the borehole (3 to 6 m to one side, at 1 m depth).

- The spacing between hydrophones is fixed at 0.5 meters; hence incremental vertical moves of the array in the order of 1 m between source records will yield considerable redundancy of hydrophone locations. Travel-times between source and receivers are individually picked for each shot record. The data redundancy is used to obtain best estimates of interval velocities over short vertical intervals (Hunter et al., 1998). For this compilation plot of P wave velocities are given at intervals of 0.5 meters downhole. Usually 3 pt (over 1 m vertically) or 5 pt (over 2 m vertically) velocity fit results are shown in Figure 13.5.

- Compressional (P) wave velocities are strongly affected by the presence or absence of pore-water. Low velocities are exhibited above the water table and in areas of the borehole where gas exists in the pore space. Most normally consolidated water-saturated soils have velocities close to that of water (1480 m/s). Overconsolidation of water-saturated soils (with resulting reduction of porosity) is indicated by somewhat higher velocities (e.g. a compacted coarse-grained basal till can yield velocities of 2500-3500 m/s. Lithification to rock, or ice-bonding of soils, results in velocities which may range between 2500-5500 m/s. Empirical relationships between soil porosity and P wave velocities have been developed.

- The downhole S wave velocity log is derived using a single- or 3-pod well-locking geophone array. Each pod consists of 3 orthogonal 14 Hz geophones which can be locked against the side of the borehole with a motor-driven bow spring. The orientation of the single- or multi-pod array can be done from ground surface down to a maximum depth of 100 m. Commonly the array is moved vertically in increments of 1 meter. The seismic source is placed close to the borehole on ground surface; commonly a steel I-beam or wooden plank loaded by the front wheels of a light truck is struck horizontally to obtain polarized shear wave energy.

- The first arrival data from all three components is examined using commercial picking and display routines. Least squares fits of the data are computed and plotted; commonly a 3-pt fit is displayed.

- Shear wave velocities can be used to indicate the presence or absence of soft soils and resonant boundaries for earthquake hazards assessment and can be used to estimate liquefaction potential of non-cohesive soils. The values can also be used to estimate ultimate strength of cohesive soils, and to identify the presence of stress anisotropy associated with natural or man-made slopes.

Emperical relationships between shear wave velocity and soil porosity have been developed.

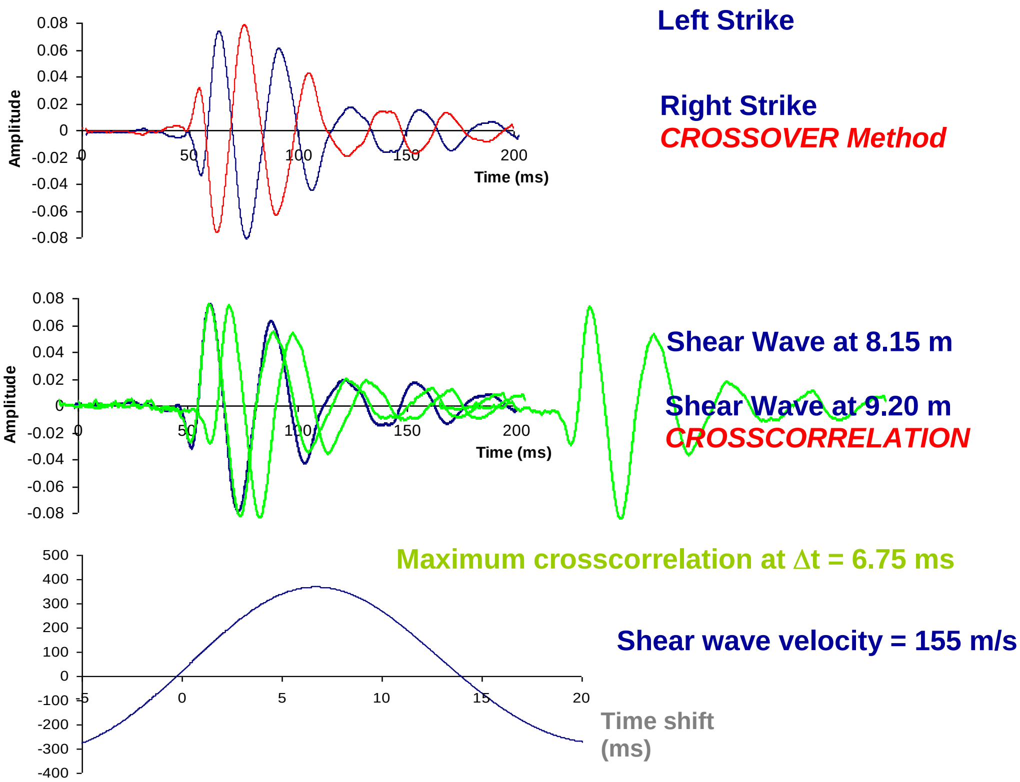


Fig 13.15: Downhole shear wave

Topic 26

Other Test used to Measure Shear wave velocity

- 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.

- **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.

1. **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.

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

3. **Suspension logging test:** This test is used to measure the wave propagation velocity and it is commonly used in petroleum industry. This is very effective at higher depths (up to 2 km).

4. **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.

5. **Seismic cross hole test:** In seismic cross hole test the wave velocities are measured using more than one bore hole (Fig. 13.16). In the simplest case two bore holes are used – one with an impulse source and the other with a receiver and both are kept at the same depth. The test is repeated at various depths to get the soil profile. Generation of body waves dependent upon source type the seismic wave generated could be P-, SV-, or SH body waves

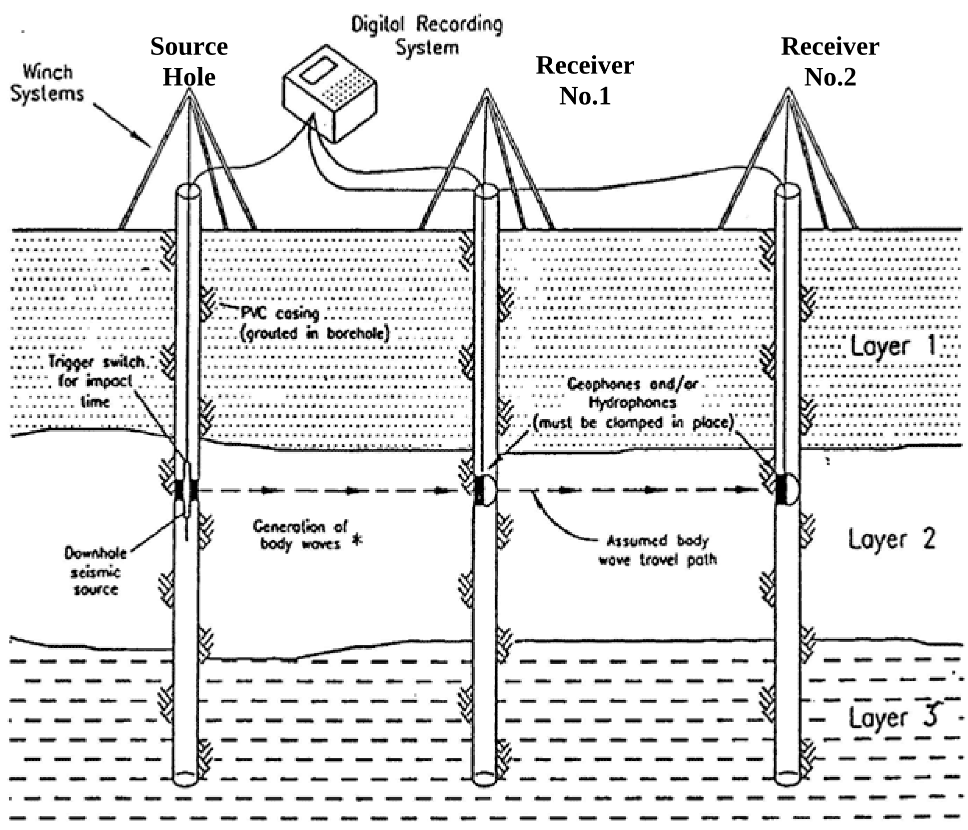


Fig. 13.16: Seismic cross hole test (Courtesy www.microgeo.com)

6. **Seismic Down hole (up hole) test**: This test is used to measure the travel time of seismic waves from source to receiver. It is performed using a single borehole. In seismic down hole test the receiver is kept at the ground surface and the impulse source is kept at different depths. The up hole test is done with receiver at the ground surface and the impulse source in the borehole. This test is not effective for depths greater than 30 to 60 m.

- **Electrical Resistivity Tests** - DC resistivity techniques, sometimes referred to as electrical resistivity, electrical resistivity imaging or vertical electric sounding, measure earth resistivity by driving a direct current (DC) signal into the ground and measuring the resulting potentials (voltages) created in the earth. From that data the electrical properties of the earth (the geoelectric section) can be derived and thereby the geologic properties inferred.

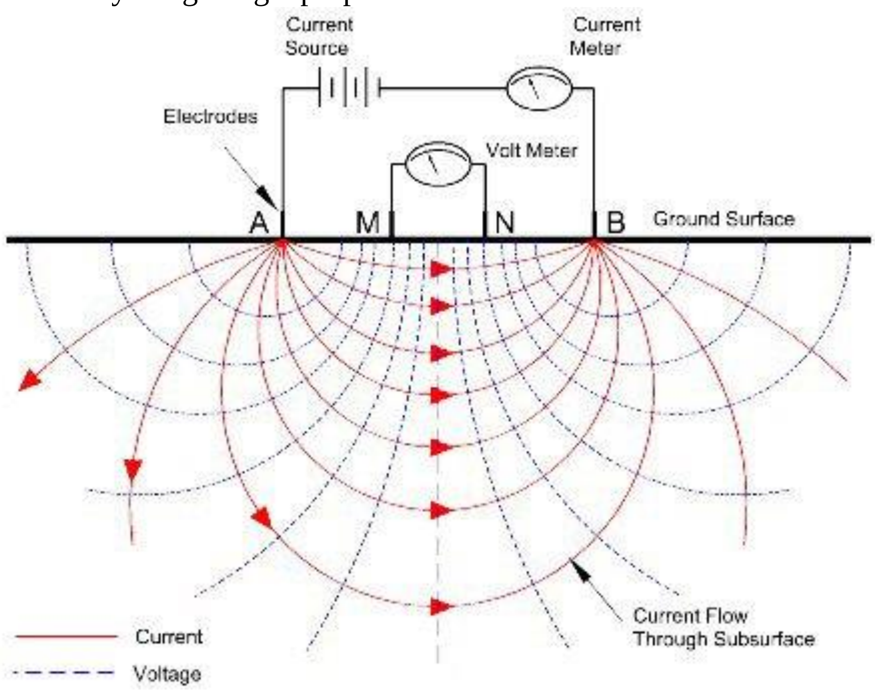


Fig 13.17: schematic diagram showing the basic principle of DC resistivity measurements.

- Two short metallic stakes (electrodes) are driven about 1 foot into the earth to apply the current to the ground. Two additional electrodes are used to measure the earth voltage (or electrical potential) generated by the current (Fig. 3.17). Depth of investigation is a function of the electrode spacing.

- The greater the spacing between the outer current electrodes, the deeper the electrical currents will flow in the earth, hence the greater the depth of exploration. The depth of investigation is generally 20% to 40% of the outer electrode spacing, depending on the earth resistivity structure.

- Instrument readings (current and voltage) are generally reduced to "apparent resistivity" values. The apparent resistivity is the resistivity of the homogeneous half-space which would produce the observed instrument response for a given electrode spacing. Apparent resistivity is a weighted average of soil resistivities over the depth of investigation. For soundings a log-log plot of apparent resistivity versus electrode separation is obtained. This is sometimes referred to as the "sounding curve."

- The resistivity data is then used to create a hypothetical model of the earth and it's resistivity structure (geoelectric sections). Resistivity models are generally not unique; i.e., a large number of earth models can produce the same observed data or sounding curve. In general, resistivity methods determine the "conductance" of a given stratigraphic layer or unit. The conductance is the product of the resistivity and the thickness of a unit. Hence that layer could be thinner and more conductive or thicker and less conductive, and produce essentially the same results. Because of these constraints on the model, borehole data or assumed unit resistivities can greatly enhance the interpretation.

- The end product from a DC resistivity survey is generally a "geoelectric" cross section (model) showing thicknesses and resistivities of all the geoelectric units or layers. If borehole data or a conceptual geologic model is available, then a geologic identity can be assigned to the geoelectric units. A two-dimensional geoelectric section may be made up of a series of one-dimensional soundings joined together to form a two-dimensional section, or it may be a continual two-dimensional cross section. The type of section produced depends on the acquisition parameters and the type of processing applied to the data.

Topic 27

Ground Penetration Radar

Introduction

- Ground penetrating radar (GPR) is a geophysical technique to detect and identify structures, either natural or man-made, below the ground surface.GPR is a nondestructive method that produces a continuous cross-sectional profile or record of subsurface features, without drilling, probing, or digging.

- GPR profiles are used for evaluating the location and depth of buried objects and to investigate the presence and continuity of natural subsurface conditions and features. GPR is a nondestructive and environmentally safe method to detect, locate and map subsurface features. The fundamental principle of operation is the same as that used to detect aircraft overhead, but with GPR that antennas are moved over the surface rather than rotating about a fixed point.

- The radar technique was first proposed during the first decades of the 20th century but was not made truly, practically, functional until the military demands of World War II pushed the development forward. After the war commercial vehicle tracking radars for ships, airports etc. were quickly developed. Some experiments using pulsed radars for mapping of glaciers were also reported in the first decades of the century, but no really usable equipment similar to the GPR of today were available until the early 70s.

- We believe it's fair to say that up until the mid 70s all equipment as well as the services performed was related to scientific studies. In the early 70s the first commercial GPR equipment was introduced. These first GPR instruments were extremely expensive, large, analogue and difficult to operate. As with many other techniques the "digital revolution" changed the scene. During the 80s true digital, smaller and more efficient GPR units appeared on the market. The 90s was a decade when the personal computers as wells as miniaturized electronics changed the GPR products. From now on there were one-man systems as well as relatively easy to use systems available.

- From 2000 onwards people started to see dedicated systems, designed for special applications, with simplified man-to-machine interfaces. Improved and user-friendly software with semi-automatic processing of data has also helped to make ground penetrating radar become a technique usable for the technically skilled common man.

Radar Principal

- Radar is short for Radio Detection and Ranging, so it's quite clear what it is all about: detection of a target and determination of its distance from the radar antenna. In general radar systems determine not only the distance but also the direction or location of the target. Both conventional radars and GPR use the same principle of traveling and reflected electromagnetic waves although the ways the waves are generated and treated are completely different.

- A radar pulse is emitted by the transmitter antenna is partly reflected and partly transmitted when it meets with an electrical discontinuity in the ground, that is an interface at which there are a change in electromagnetic wave impedance or in other words a change in electrical properties. If the time for the pulse to go to the reflector and back again to the receiver antenna is measured, the location of the reflector in the ground can be decided, if the velocity of the pulse is known. It can be seen from Figure 13.18 clearly the interfaces as layers result in a layer in the radargram, whereas objects form so called hyperbolas.

- Theories of electromagnetic and seismic (elastic) wave propagation are similar in many respects. Both waves propagate with finite velocities that depend upon the material properties, and both are reflected and refracted based on local changes. The propagation of electromagnetic pulse depends upon dielectric properties of the material. This method is not suitable in high conductivity environments such as soils with saline water, as electromagnetic fields diffuse into the ground. An approximate value of electromagnetic parameters of typical soils and rocks are provided in the Table 13.7.

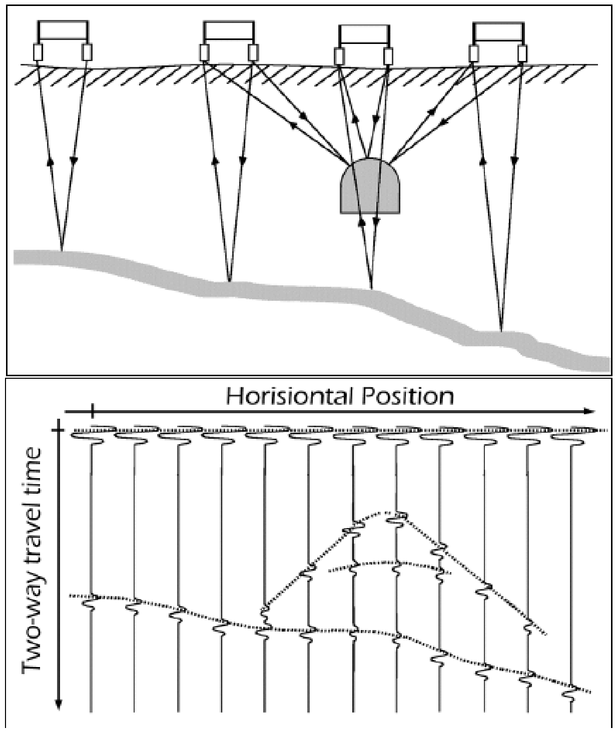


Figure 13.18: Basic principle of radar measurement

Table 13.7: Electromagnetic Parameters of Various Types of Rocks and Soils (Morey, 1974; Ulriksen, 1983)

Material	Conductivity, σ (mho/m or S/m)	Relative permittivity, K	Attenuation, α (dB/m)	Velocity, C (cm/ns)
Air	0	1	0	30
Fresh water	10-3	81	0.18	3.3
Sea water	4.0	81	103	3.3
Granite (dry)	10-8	5	10-5	13
Granite (wet)	10-3	7	0.6	11
Basalt (wet)	10-2	8	5.6	11
Shale (wet)	10-1	7	45	11
Sandstone(wet)	4 × 10-2	6	24	12
Limestone (wet)	2.5 × 10-2	8	14	11
Sandy soil (dry)	1.5 × 10-4	3	0.14	17
Sandy soil (wet)	7 × 10-3	25	2.3	6
Clayey soil (dry)	2.5 × 10-4	3	0.28	17
Clayey soil (wet)	5 × 10-2	15	20	7.8

Topic 28

Comparison of GPR from Other Non Destructive Methods

Seismic methods

- Comparisons are often done is between seismic methods and GPR. Now what is the difference between these two methods? The resulting data looks very similar There are however a few fundamental things which distinguish the two techniques from each other:

1. In seismic method, the reflection of the wave is caused by changes in the density of the material under investigation. In GPR the reflections are caused by changes in the electric properties, primarily in the dielectric constant ε.
2. Seismic methods require a very good physical contact between the receiving/transmitting elements and the ground.
3. Seismic method is at least 10 times more expensive than GPR per meter of profile.
4. In GPR the velocity of the media usually never change more than 50% and that would be a rather extreme case, on a certain site. In seismic velocity contrasts can be much larger. This is because variations in density are much stronger than variations in the dielectric constant.
5. GPR shows much more detail than seismic, in other words, the resolution for GPR is higher.
6. Seismic methods work very well in clay where GPR is almost useless. It also penetrates kilometers instead of meters as for GPR.

Ultrasonic methods

- Much of what was said about seismic is also true for ultrasonic since they are both acoustic methods, only the frequency differs. However the instrumentation is different and that makes a separate comparison valid:

1. In ultrasonics the frequency is often high enough to make mm resolution possible e.g. in medicine.
2. The contact with ground/media has to be so good that the sensors are often glued to the material one wants to investigate, or a contact gel is used.

Metal detectors/cover meters

- Cover meters are types of instruments used for detection of rebar in concrete and metal detectors is probably familiar to everyone. Obviously these instruments only detect metal, there are, however, a few more distinguishing characteristics:

1. Cover meters can never see through a wire mesh, anything below a first layer would be hidden. High-resolution radar sees trough a wire mesh and can detect both metallic and non-metallic objects under it.

2. Cover meters are based on an assumption of a certain diameter of the rebars. This means that if two rebars are too close the instrument will become very uncertain.
3. Metal detectors can be made very easy to use and also tuned to detect very small pieces of metal, smaller than GPR can resolve.

EM-locators

- EM-locators are used locate cables and metal pipes. They can be used both in active and passive mode, active meaning when the transmitter of the EM-locator is connected to the target. In practice the active mode is much preferred and people tend to not using the passive mode. When the active mode is possible, these instruments are easy to use and reliable. GPR is seldom used in these cases. Still there are many cases where GPR is more favorable:

1. GPR detects both metallic and non-metallic targets; an EM-detector is only capable of locating the metal ones.
2. The active mode requires the pipe, cable or tracer wire to be unbroken. For GPR this doesn't matter.
3. GPR can pinpoint many targets at one swat, when using an EM-locator you concentrate on one at a time, normally.

X-ray

- No need to argue, X-ray is probably the most revealing NDT method. However it is very expensive, not only due to the complex security measures necessary during its application it's also quite slow and cumbersome.

Topic 29

The Basic Radar System

- A radar system consist of

1. **A control unit**, which generates the control signals for the receiver and transmitter antenna electronics, keeps track of the distance along the profile and buffers data from receiver.
2. **Transmitter antenna** electronics, which generates impulses or steps, fed to the transmitting antenna.
3. **Receiver antenna** electronics, which takes care of the incoming signals for digitisation and storage.
4. **Transmitter and Receiver antenna elements**, bi-static. These can either be two separate units or mounted together in a box.
5. **A recording facility** to store and display data. Most often this would be a laptop or a dedicated monitor.
6. **An encoder to position and trigger the measurements**, most often a survey wheel or hip-chain, sometimes combined with GPS.

- **GPR used for -** As said before, only the imagination is setting the real limits for what can be done with GPR. In this section we're listing a few application areas in which GPR have been used previously. This list of assessments and analysis is not complete and new applications are added all the time.

Archaeology	*Borehole*	*Bridge deck*	*Building assessment*
Forensic	*Remote sensing*	*Geophysical*	*Rail track*
Contaminated soil	*Utility*	*Road*	*Asphalt thickness*
Trunk condition	*Snow thickness*	*Concrete*	*Moisture assessment*
Soil classification	*Earth dam*	*Tunnel detection*	*Fault study*

Topic 30

Grids for Site characterization

- Site characterization done to the areas are divided into cells by a grid system for estimating the effects of site conditions by assigning representative soil profiles at the centre of each grid.

- Site response analyses were conducted for each grid using the acceleration spectra compatible simulated earthquake time histories obtained for each grid separately based on the seismic hazard study.

- Pilot areas were divided into grids to evaluate the representative parameters for each grid by defining hypothetical boreholes located at the centre of the grids. A hypothetical borehole should be an idealized borehole, which will be the most representative for the soil conditions in the specific grid. In an ideal project, new site investigations might be conducted, almost in the centre of each of the grids.

- There are basically two reasons behind the grid approach adopted for evaluating and interpreting the effects of site conditions on the ground motion at the free field.
    1. To utilize all the available data in each grid in order to have more comprehensive and reliable information about the soil profile;
    2. To eliminate the effects of different distances among boreholes or site investigation points during the GIS mapping.

- The results obtained were mapped using GIS techniques by applying linear interpolation among the grid points, thus enabling a smooth transition of the selected parameters. Soft transition boundaries are preferred to show the variation of the mapped parameters.

Topic 31

Interpolation of non-filled grids and hypothetical boreholes

- The pilot areas were divided into cells by a grid system, if there are no boreholes or in-situ tests conducted in the outer grids, and then the boundaries of the both regions are modified as shown in the following maps for microzonation study to avoid the need for additional extrapolation that may not be very reliable.

- Interpolation of data to fill empty grid points has been carried out very carefully and was omitted for doubtful cases as well as for cases where extrapolation of borehole data would have been needed.

- A hypothetical borehole should be an idealized borehole, which will be the most representative for the soil conditions in the specific grid. In an ideal project, new site investigations might be conducted, almost in the centre of each of the grids.

- The hypothetical borehole was introduced for each grid as the soil profile, including the extension up to competent layer. In case of mixed layers, the softer layer has been chosen as the representative one.

Topic 32

Site Classification and 30m Concept

- Seismic hazard analysis gives seismic hazard parameters at rock level. But damages due to seismic activities depend upon the site specific properties subsurface materials within a km from surface. Site characterization is process of classifying region/ site considering average subsurface material properties. Classification of individual sites based on the properties is a more direct indicator of local site effects.

- Seismic site classification systems are inevitably reflected in modern seismic codal provisions to account for site effects. Recent modern seismic codes in America, Europe, Japan and worldwide [International building code (IBC 2009), Unified Building code (UBC 97), National Earthquake Hazards Reduction Program (NEHRP, BSSC, 2001) and Euro code 8, 2007] have produced numerous valuable information based on experimental and theoretical results.

- In engineering site investigation, 30 m is a typical depth of borings and detailed site characterizations. Therefore, most of the site-effect studies in earthquake ground motions are based on the properties in the upper 30 m.

- For instance, Boore et al. (1993, 1994) based their regressions for ground motions on average shear velocity in the upper 30 m. Borcherdt (1992, 1994) and Martin and Dobry (1994) recommended that design of structures be based on these properties. The 30m average concept is widely used for the seismic Microzonation by considering experimental data from geotechnical and geophysical testing.

- The site classes are defined in terms of shear velocity to a depth of 30m, denoted by Vs^{30}, if no measurements of Vs to 30 m are feasible corresponding limits in terms of standard penetration resistance (*N*) and undrained shear strength (S_u) (Borcherdt, 1994).

- Site classification is followed in modern seismic codes to arrive design spectrum and some extent to represents site and induced effects of the site during earthquakes. This will also help to delineate sites for detailed analysis.

- The geotechnical parameters used to define the site class are based on the upper 100 ft (30 480 mm) of the site profile. Profiles containing distinctly different soil and rock layers shall be subdivided into those layers designated by a numbers that ranges from 1 to *n* at the bottom where there are a total of *n* distinct layers in the upper 100 ft (30 480 mm). The symbol/then refers to any one of the layers between 1 and *n*

- Where $\sum_{i=1}^{n} d_i$ is equal to 100 ft (30 480 mm)

- Shear wave velocity $$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \quad (13.9)$$

- Where v_{si} = the shear wave velocity in ft/sec or m/s
- di = the thickness of any layer (between o and 100 ft [30 480 mm])

- Standard Penetration Resistance $$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad (13.10)$$

- Where N_i and d_i in above equation are for cohesionless soil, cohesive soil, and rock layers.

- For cohesionless soil layers $$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad (13.11)$$

Where N_i and d_i in above equation are for cohesionless soil layers only and $\sum_{i=1}^{m} d_i = d_s$

d_s = the total thickness of cohesionless soil layers in the top 100 ft (30 480 mm)

m = The number of cohesionless soil layers in the top 100 feet (30 480 mm).

- N_i = The Standard Penetration Resistance determined in accordance with ASTM D 1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft. where refusal is met for rock layer, N_i shall be taken as 100 blows/ft

- Undrained Shear Strength $$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{S_{ui}}} \qquad (13.12)$$

  Where $\sum_{i=1}^{k} d_i = d_c$

  d_c = the total thickness of cohesive soil layers in the top 100 ft (30 480 mm)

  s_{ui} = The undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa), ASTM D 2166 or D 2850.

- $\overline{N}_{ch}$ for cohesionless soil layers (PI<20) in the top 100 ft (30 m) and average $\bar{s}_u$ for cohesive soil layers (PI>20) in the top 100 ft (30 m)

- PI = the plasticity index, ASTM D 4318.

Topic 33

Site Class Definitions – International Building Code

- Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Table 13.8. When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site.

Table 13.8: Site Class Definitions as per International Building Code (IBC, 2009)

Site Classification	Description	Average Properties in Top 30m		
		Shear wave velocity (m/s)	SPT N (blows/300mm)	Undrained Shear Strength s_u (kPa)
A	Hard Rock	>1500	NA	NA
B	Rock	750-1500	NA	NA
C	Very dense soil and Soft rock	360-750	>50	>100
D	Stiff soil	180-360	15-50	50-100
E	Soft Soil	<180	<15	<50
		Plus any profile with more than 3m of soil having the following characteristics: Plasticity Index, PI >20% Moisture content, w ≥ 40% Undrained Shear strength, Su <25kPa		
F	Any profile containing soils with one or more of the following characteristics Soil vulnerable to potential collapse under seismic loading e.g. liquefiable soils, quick and highly sensitive clay, collapsible weakly cemented soils. Peats and/or highly organic clays (H>8m of peat and/or highly organic clay) Very high plasticity clays (H>8m with PI>75%) Very thick soft/medium stiff clays (H>36m)			

Where the soil properties are not known in sufficient detail to determine the site calss in accordance with below, it shall be permitted to assume Site class D unless the authority having jurisdiction determines that site class E or F could apply at the site or in the event that Site Class E or F is established by geotechnical data

Site Class definitions. The Site Classes are defined as follows:

A Hard rock with measured shear wave velocity, $\bar{v}_s$ >5,000 ft/sec (1500 m/s)

B Rock with 2,500 ft/sec < $\bar{v}_s \le$ 5,000 ft/sec (760 m/s < $\bar{v}_s \le$ 1,500 m/s)

C Very dense soil and soft rock with 1,200 ft/sec < $\bar{v}_s \le$ 2,500 ft/sec (360 m/s < $\bar{v}_s \le$ 760 m/s) or with either $\bar{N}$ >50 or $\bar{s}_u$ > 2,000 psf (100 kPa)

D Stiff soil with 600 ft/sec ≤ $\bar{v}_s \le$ 1,200 ft/sec (180 m/s < $\bar{v}_s \le$ 360 m/s) or with either 15 ≤ $\bar{N}$ >50 or 1,000 psf ≤ $\bar{s}_u \le$ 2,000 psf (50 kPa ≤ $\bar{s}_u \le$100 kPa)

E A soil profile with $\bar{v}_s$ < 600 ft/sec (180 m/s) or with either $\bar{N}$ <15, $\bar{s}_u$ < 1,000 psf, or any profile with more than 10 ft (3 m) of soft clay defined as soil with *PI>20, w≥40* percent, and s_u<500 psf (25 kPa)

F Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils quick and highly sensitive clay, collapsible weekly cemented soils.
Exception: For structures having fundamental periods of vibration less than or equal to 0.5 second, site- specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site class May be determined in accordance with below section, assuming liquefaction does not occur.
2. Peat and /or highly organic clays (H> 10ft [3ft] of peat and/or highly organic clay, where H = thickness of soil)
3. Very high plasticity clays (H> 25ft [8m] with *PI*>75)
4. Very thick, soft/medium stiff clays (H> 120 ft [36 m]) with s_u<1,000 psf (50 kPa)

Steps for Classifying a Site

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as site class F and Conduct a site specific evaluation
Step 2: Check for the existence of a total thickness of soft clay> 10ft (3 m) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), $w \geq 40$ percent, and $PI > 20$. If these criteria are satisfied classify the site as Site Class E.
Step 3: Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ computed in all cases as specified in Sec.3.51

a. $\bar{v}_s$ for the top 100 ft (30 m) ($\bar{v}_s$ method)
b. $\bar{N}$ for the top 100 ft (30 m) ($\bar{N}$ method)
c. $\bar{N}_{ch}$ for cohesionless soil layers (PI<20) in the top 100 ft (30 m) and average $\bar{S}_u$ for cohesive soil layers (PI>20) in the top 100 ft (30 m) ($\bar{s}_u$ method)

Table 13.9: Site Classification as per NEHRP (BSSC, 2003)

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$ [a]
E	< 600 fps (<180 m/s)	<15	<1,000 psf (< 50 kPa)
D	600 to 1,200 fps (180 to 360 m/s)	15 to 50)	1,000 to 2,000 psf (50 to 100 kpa)
C	1,200 to 2, 500 fps (360 to 760 m/s)	> 50	> 2,000 psf (> 100 kPa)

[a] If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D)

- Assignment of Site Class B shall be based on the shear wave velocity for rock. For competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the site shall be assigned to Site Class C.

- Assignment of Site Class A shall be supported by either shear wave velocity measurements on site or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and Fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

- Site Classes A and B shall not be used where there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

Topic 34

Site Class Definitions – European Standard

- Further guidance concerning ground investigation and classification is given in EN 1998-5:2004, 4.2.

- The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake. The possibility of occurrence of such phenomena shall be investigated in accordance with EN 1998- 5:2004, Section 4.

- Depending on the importance class of the structure and the particular conditions of the project, ground investigations and/or geological studies should be performed to determine the seismic action.

Identification of ground types

- Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

- The site should be classified according to the value of the average shear wave velocity, $v_{s,30,}$ if this is available. Otherwise the value of N_{SPT} should be used.

- The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \qquad (13.13)$$

- where h_i and v_i denote the thickness (in metres) and shear-wave velocity (at a shear strain level of 10^{-5} or less) of the i-th formation or layer, in a total of N, existing in the top 30 m. The Table 13.10 gives the clear classification of the ground types based on Eurocode.

- For sites with ground conditions matching either one of the two special ground types S_1 or S_2, special studies for the definition of the seismic action are required. For these types, and particularly for S_2, the possibility of soil failure under the seismic action shall be taken into account.

- **NOTE:** Special attention should be paid if the deposit is of ground type S_1. Such soils typically have very low values of v_s, low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section **6**). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response spectrum on the thickness and v_s value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

Table 13.10: Ground types as per European Standard

Ground Type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT}	C_u (kPa)
A	Rock or other rock like geological formation, including at most 5m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand, gravel or stiff clay, at least several tens of meters 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 meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesion less soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70

E	A soil profile consisting of a surface alluvium layer with Vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with Vs > 800 m/s.			
S_1	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 to 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Topic 35

Comparison of seismic site classification

- Seismic ground response characteristics, defined generally as "site effects", are inevitably incorporated in modern seismic code provisions in many countries. However, the definitions of site classes in different codes are not consistent. The summary of site classes adopted in Table 13.11 shows the summary of site classes adopted in National Earthquake Hazards Reduction Program (NEHRP) (BSSC, 2001), International Building Code (IBC, 2009) or Uniform Building Code (UBC, 1997) and Eurocode 8 (2007). In order to avoid confusion of detailed specifications, only key information is given in Table 13.10 for direct comparison.

Table 13.11: Site classification system given in modern seismic codes

Site Class	Generalized Description	NEHRP (BSSC,2001)		IBC 2009/ UBC1997		Eurocode 8 (2007)$	
		N_{30}	Vs^{30}	N_{30}	Vs^{30}	N_{30}	Vs^{30}
A	Hard rock	N/A	>1500	N/A	>1524	N/A	N/A
B	Rock	N/A	760-1500	N/A	762-1524		>800
C	Very dense soil and soft rock	> 50	360-760	> 50	366-762	>50	360-800
D	Dense to medium soils	15-50	180-360	15-50	183-366	15 - 50	180 - 360

E	Medium to soft soil	< 15	< 180	< 15	< 183	<15	<180

N/A-Not applicable, * Not mention, $The site classes B, C, D and E in this table correspond to site classes A, B, C and D as per Eurocode 8

- It can be observed that site classification of IBC2006/UBC1997 and NEHRP are identical, which consider five different site classes together with one special site class (Site Class F) for very loose soil for which site specific study is recommended.

- Table 13.12 shows the summary of site classes adopted in National Earthquake Hazards Reduction Program (NEHRP) (BSSC, 2001), Australian Standards Part 4: Earthquake Actions in Australia (AS 1170.4, 2007), China Code for Seismic Design of Building (GB 50011, 2001) and Indian Code (BIS 1893, 2002).

- Australian Standards Part 4: Earthquake Actions in Australia (AS 1170.4, 2007), China Code for Seismic Design of Building (GB 50011, 2001) and Indian Code (BIS 1893, 2002). In order to avoid confusion of detailed specification, only key information is given in Table 13.11 for direct comparison. The soil types are mainly accounted by average SWV or SPT-N values.

- Australian Standard recommends five methods to classify a site; site class based on geotechnical details are placed higher order. General site classification of Australian Standard based on SWV and SPT N values are given in Table 13.11.

- A detailed site classification procedure recommended in Chinese Code GB 50011 (2001) is described in Chapter 4, Section 4.1.6 of the code. It also includes provision for fault within the site and liquefiable soil.

- Site classifications are based on 20 m equivalent SWV of soil (v_s^{20}) and thickness of site overlying layers. Site classification according to the Chinese code based on the description of subsurface materials is given in Table 13.11.

- There is no separate section for site classification that considers geotechnical characteristics of sites in the Indian code BIS 1893 (2002). But Section 6.3.5.2 of the code describes rough consideration of site conditions by specifying SPT-N values and type of foundation. Site classification in Indian code BIS 1893 (2002) are based on SPT-N values and given in Table 13.12.

Table 13.12: Comparison of seismic site classification in the Asia Pacific regions with international standards

Site Class	Generalized soil Description	NEHRP (BSSC,2001)		Australian Standards AS 1170.4, 2007		Chinese seismic Code GB 50011(2001)		Indian Standards BIS 1893 (2002)	
		N_{30}	Vs^{30}	N_{30}	Vs^{30}	N	Vs^{20}	N	Vs^{30}
A	Hard rock	N/A	>1500	*	>1500	*	*	*	*
B	Rock	N/A	760-1500	*	>360	*	>500	*	*
C	Very dense soil and soft rock	> 50	360-760	*	≤0.6s (surface to rock)	*	250-500	>30	*
D	Dense to medium soils	15-50	180-360	Soil with SPT N values of <6 for depth of <10m	>0.6s (surface to rock)	*	140-250	All the soil 10 to 30 or Sand with little fines N>15	*
E	Medium to soft soil	< 15	< 180	Soil with SPT N values of <6 for depth of >10m	More than 10m depth of Soil with Vs ≤150 or less	*	<140	<10	*

Lecture 13 in Introduction to Site Characterization; Different methods and experiments; Geotechnical properties; Site classification and worldwide code recommendation