

PART VI
Appendix D**Methods of Soil Exploration, Sampling and Groundwater Measurements****D.1 Methods of Soil Exploration**

The detailed methods of soil investigation usually include collecting undisturbed samples and or performing field tests. Listed below are some of the common methods of subsoil exploration.

D.1.1 Open Trial Pits

In this method trial pits are excavated exposing the subsoil thoroughly. Undisturbed samples are taken from intact sides and bottom of the trial pits. This is suitable for all types of formation but for cuts which cannot stand below water table, proper bracing shall be provided. This method is normally used for shallow depths (up to 3 m). Test pits are usually prepared by hand excavation which allows access for a full observation and description of the soil profile. Hand-cut samples known as block or chunk samples can be obtained from the test pit. In stiff clays it provides fairly accurate idea of the depth of open excavations or vertical cuts. It also provide better picture of the patchy ground where the soil lies in pockets

D.1.2 Auger Boring

Augers, hand or power operated, are rotated and forced into soil. Augers are withdrawn and emptied when full. Soil cuttings obtained are used to interpret stratification and soil type. The method is unsatisfactory for cohesionless soils above or below ground water.

D.1.3 Shell and Auger Boring

Manual or mechanized rigs are used for vertical boring. The tools consist of auger for soft to stiff clays, shells for very stiff to hard clays, shells or sand pumps attached to sectional boring rods for sandy strata. Hand operated auger can be used up to depths of 18 m to 21 m in stiff clay. Mechanically operated tools are required for higher depths. An auger is used for boring holes to a depth of about 6 m in soft soil. Soft to stiff clays are removed by a cylindrical auger. Hard clays and cohesionless deposits are removed by a shell. The soil recovered contains all constituent. Bailers are be used to remove soil cuttings. Care is taken while withdrawing the shell to avoid sand boiling. Special care is taken to minimize soil disturbance below borehole. The advantages of shell and auger boring are as follows:

- (i) Easy to identify the soil
- (ii) Easy to note changes in strata
- (iii) Soil profile and depths undisturbed sampling can be determined with greater accuracy
- (iv) Boring in partially saturated materials above the ground water level
- (v) Determination of ground water level is relatively easy

D.1.4 Wash Boring

In this method, soil is loosened by chopping and cutting by impact and twisting action of a lightweight bit. Soil is removed from the borehole by a stream of water or drilling mud from lower end of the wash pipe which is worked up and down or rotated into the borehole. The water or mud flow carries the soil through the annular space between the wash pipe and casing and is overflowed at ground level. The soil in suspension is allowed to settle in a pond or tank and the fluid is recirculated as required. The soil brought to surface by the wash water can be used for identification purposes but is not representative of the character and consistency of the material penetrated and the flushing water may disturb the surrounding ground. Subsoil can be identified thoroughly if field tests (viz. Standard Penetration Test) are performed and or undisturbed samples are collected frequently.

D.1.5 Sounding/Probing

A number of sounding methods are available. The most common is the Standard Penetration Test (SPT). The SPT test is specified in ASTM D1586 and ASTM D6066. Other methods include procedures like Cone Penetration Test (CPT) and Dynamic Probing (DP). Sounding/probing may be done in conjunction with inhole tests such as "Field Vane Shear Test in Cohesive Soil", (ASTM D2573), bore-hole shear (Iowa Bore-hole Shear) Test, Flat Dilatometer Test (DMT) or "Prebored Pressuremeter Testing in Soils", (ASTM D4719).

D.1.6 Geophysical Methods

Geophysical survey techniques are based on determining variations in physical properties, such as electrical conductivity (resistivity), variation in density (gravimetric), magnetic susceptibility (magnetic) or velocity of sonic waves (seismic). Anomalies such as near surface disturbance (often known as noise) are common in urban environment and may limit the usefulness of geophysics in these areas. Moreover, a geophysical anomaly does not always match an engineering or geological boundary, and often there is a transition zone at a boundary. These may lead to a margin of uncertainty.

D.1.7 Percussion Boring

In percussion drilling method borehole is advanced by chopping action of a heavy bit driven by power. Drilling is performed raising and dropping a heavy drilling bit. The borehole is generally kept dry. Water is added at the bottom of the borehole during chopping action, if ground water is not already struck. Slurry formed at the bottom of hole is removed by bailer or sand pump. Casing may be needed. This method is Simple to operate and suitable for drilling bore holes in deposits of gravels and boulders. However, using this method, determination of the changes in the soil strata is very difficult. The method is relatively slow and not economical for boring diameter less than 100 mm

D.1.8 Rotary Drilling

In rotary drilling, borehole is advanced by power rotation of drilling bit. The particles from inside is removed by circulating fluids which may be water, bentonite slurry or mud slurry in a manner similar to that in wash boring. Casing may or may not be needed during drilling. This method is particularly suitable for very hard formation and rock. Rate of progress is fast. Difficult to detect changes in strata

D.2 Choice of Method

The choice of a method of soil exploration depends upon:

- (i) The topography, type of ground to be investigated and ground water conditions;
- (ii) The type of building envisaged and technical requirements;
- (iii) Amount of existing information;
- (iv) Expected variability of soil;
- (v) External constraints such as availability of plant, access, cost and time.

The technical requirements of the investigation rather than cost should be the overriding factor in the selection of exploration method. In clayey soils, borings are suitable for deep exploration and pit for shallow exploration. In sandy soils special equipment are needed for taking representative samples below the water table. Ground investigation is normally done by boreholes, but where only shallow depths are to be investigated, and where ground water problems are not envisaged, trial pits may prove more versatile and economical. Boreholes may be necessary on waterlogged sites where it is impracticable to excavate trial pits without dewatering.

Safety aspects must be considered when selecting and carrying out exploration. Precautions relating to safety, health and welfare of workmen, hazards from underground services, contaminated ground and inspection pits or shafts shall be undertaken. Overhead power lines are a hazard if ground investigation rigs are to operate in the vicinity.

D.3 Sample Disturbance and Sampling Methods

The major purposes of retrieving soil sample from ground are as follows:

- (i) Inspection of the material and to describe its fabric
- (ii) To classify the material by index tests
- (iii) Determination of mechanical properties, such as, stress-strain-strength-stiffness, compressibility and expansibility, and permeability

Samples shall contain all the mineral constituents of the strata from which they have been taken. They shall not be contaminated by any material from other strata or from additives used during the sampling procedure. For index tests, the sample should be intact in terms of its constituents, and changes in fabric and stress state are acceptable. To measure most mechanical properties the sample needs to be undisturbed and representative. Material submitted for testing should be representative of the mass which will be affected by construction and the level of sample disturbance should be acceptable.

The physical process of obtaining samples has been recognized as a prime cause of sample disturbance. The main causes of sampling disturbance are as follows:

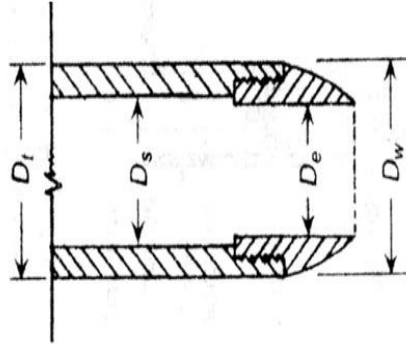
- (i) Disturbance of the soil to be sampled before the beginning of sampling as a result of poor drilling operation leading to swelling, compaction, base heave, piping and caving
- (ii) Mechanical distortion during the penetration of the sampling tube into the soil. This is referred as tube penetration disturbance
- (iii) Mechanical distortion and suction effects during the retrieval of the sampling tube.
- (iv) Release of total in situ stresses.
- (v) Disturbance of the soil during transportation, storage and sample preparation.

The first cause can be reduced by sampling with properly cleaned boreholes advanced by using bentonite slurry. The second and third causes are directly associated with sampler design and can be controlled to certain extent. The fourth cause is unavoidable even though its effects may be different depending on the depth of sampling and soil properties. The fifth cause can be reduced by storing samples for minimum time in controlled atmosphere and careful handling of samples during transportation and preparation.

The design of a sampler is one of the most important factors that should be considered for quality sampling. The amount of disturbance varies considerably depending upon the dimensions of the sampler and the precise geometry of the cutting shoe of the sampler. Hvorslev (1949) discussed at length the importance of the design of a sampler and introduced the concepts of area ratio, inside and outside clearance ratio and cutting edge taper angle in controlling sampling disturbance. Terms used to define geometry of cutting shoe of a tube sampler is shown in Figure 6.D.1.

Increasing area ratio gives increased soil disturbance and remoulding. The penetration resistance of the sampler and the possibility of the entrance of excess soil also increase with increasing area ratio. For soft clays, area ratio is kept to a minimum by employing thin-walled tubes. For composite samplers, the area ratio, however⁶, is considerably higher. In these cases, sample disturbance is reduced by tapering the outside of the sampler tube very gradually from a sharp cutting edge (Hvorslev, 1949, recommended a

maximum 10°), so that the full wall thickness is far removed from the point where the sample enters the tube. The combined requirements for area ratio and cutting edge taper angle to cause low degree of disturbance and the optimum length to diameter ratios for clays of different sensitivities were proposed by ISSMFE's sub-committee on Problems and Practices of Soil Sampling.



$$\text{Area ratio, } Ar = \frac{D_w^2 - D_e^2}{D_e^2}$$

$$\text{Inside Clearance ratio, } ICR = \frac{D_s - D_e}{D_e}$$

$$\text{Outside Clearance Ratio, } OCR = \frac{D_w - D_t}{D_t}$$

Figure 6.D.1 Dimensions of a tube sampler and terms used to define geometry of cutting

Tube sampler characteristics suggested by ISSMFE (1965) is presented in Table 6.D.1. Inside wall friction is one of the principal causes of disturbance of the sample (Hvorslev, 1949). One of the methods of reducing or eliminating wall friction between the soil and sampler is to provide inside clearance by making the diameter of the cutting edge, D_e , slightly smaller than the inside diameter of the sampler tube, D_s . Inside clearance gives the soil sample room for some swelling and lateral strain due to horizontal stress reduction. Although neither of these types of behaviour is desirable, they are less undesirable than the consequences of adhesion between the soil and the inside of the sampler tube (Clayton et al, 1982).

Table 6.D.1: Tube Sampler Characteristics (after ISSMFE, 1965)

Area ratio/cutting-edge taper*	
Leading edge	
60° taper angle:	
• up to a thickness of 0.3 mm for cohesive soils • up to a thickness of D_{10} in granular soils	
Cutting shoe	
Area ratio (%)	Taper angle (degrees)
5	15
10	12
20	9
40	5
80	4
Inside clearance ratio/length-to-diameter (L/D) ratio ⁺	

Soil type	Greatest permissible L/D ratio
Clay (sensitivity > 30)	20
Clay (sensitivity 5-30)	12
Clay (sensitivity <5)	10
Loose frictional soil	12
Medium loose frictional soil	6

* Suggested for samplers of about 75 mm diameter.

+ For samplers with smooth and clean inside surfaces, and with an inside clearance ratio of 0.5-1.0%.

Inside clearance should be large enough to allow partial swelling and lateral stress reduction but it should not allow excessive soil swelling or loss of the sample when withdrawing from the sampling tube. Hvorslev (1949) suggests an inside clearance ratio of 0.75 to 1.5% for long samplers and 0 to 0.5% for very short samplers. Kallstenius (1958) on the basis of Swedish clays sampled by six different piston samplers and also recommends that a sampler have a moderate inside clearance. The clearance reduces the wall friction and probably counteracts to a certain extent the disturbance from displacement of soil caused by the edge and sampler wall during the driving operation. If the inside clearance and the edge angle are moderate, the above positive effects outweigh the disturbance caused by deformation when the sample tends to fill the clearance. In order to reduce outside wall friction, samplers are often provided with outside clearance. An outside clearance ratio of a few per cent may decrease the penetration resistance of samplers in cohesive soils. Although outside clearance increases the area ratio, a clearance of 2 to 3% can be advantageous in clay (Hvorslev, 1949).

The recommended criteria for high quality sampling areas follow:

- (i) Area ratio : less than 10%
- (ii) ICR = 0 to 0.5 % for very short samplers; ICR = 0.75 to 1.5 % for long samplers
- (iii) OCR : not more than 2 to 3%

For 50 to 75 mm diameter samplers:

- (i) L/D_i ratio: not more than 5 to 10 for loose to dense cohesionless soils (D_i = Internal diameter of sampling tube)
- (ii) L/D_i ratio: not more than 10 to 20 for very soft to stiff cohesive soils

Samples retrieved from can be classified into the following types:

- (i) Representative
- (ii) Non-representative
- (iii) Undisturbed
- (iv) Disturbed

A representative sample is one in which the fabric and structure of the soil in situ is fully represented. A non-representative sample is which does not the grading, fabric and structure of the soil in situ.

An undisturbed sample is one, which represents as practicable the true in situ structure and moisture content of the soil. Truly, undisturbed samples cannot be taken from boreholes, and in practice there are only differing levels of disturbed samples. Material may be secured in open tube samplers for clays except of firm or of stiff consistency. For softer clays stationary piston samplers of low area ratio, shall be used and careful boring and sample preservation technique shall be employed. The minimum diameter of undisturbed sample shall be 40 mm with minimum length/diameter ratio of 3.

A disturbed sample is one which preserves the grading of the in situ soil but in which the soil structure is significantly damaged or completely destroyed and the moisture content also changed considerably from the in situ value

Sample quality is dependent on type of soil being sampled, type and condition of equipment and the skill with which it is used. The weaker material is the most significant in an investigation, and is usually difficult to secure in an undisturbed condition. It is rarely possible to sample granular (non-cohesive) materials in undisturbed condition, unless special techniques are used. Granular soil conditions are usually assessed by in-situ tests and confirmed by disturbed samples which permit classification and grading analysis and visual inspection. Cohesive soils may be tested both in-situ and in laboratory on undisturbed or relatively undisturbed samples.

The sample and/or test locations must be such that all changes of stratum are recorded. A number of extra samples and test results are usually required to assess variation of the properties of a stratum with depth. The record of all borings shall include the following information:

- (a) Size of casing (if used),
- (b) Number of blows per 300 mm required to drive the sampling spoon,
- (c) The elevation of the ground surface referred to an established datum,
- (d) Location and depth of boring and its relation to the proposed construction,
- (e) Elevation at which samples were taken,

- (f) Elevation of the boundaries of soil strata,
- (g) Description of the soil strata encountered and any particular unusual or special condition such as loss of water in the earth and rock strata, presence of boulders, cavities and obstructions, use of special type of samplers, traps, etc., and,
- (h) The level of ground water together with a description of how and when it was observed.

All abandoned and unsuccessful attempts of borings or drillings shall also be reported. In complex formations, details of sampling are necessary and, therefore, separate holes may be employed purely for sampling or testing, termed as “double hole sampling”.

Care shall be taken in protecting, handling, labelling and subsequently transporting the samples, so that samples can be received in a fit state for examination and testing, and can be correctly recognized as coming from a specific trial pit or boring.

Soil samples for laboratory tests can be divided in five quality classes with respect to the soil properties that are assumed to remain unchanged during sampling and handling, transportation and storage. The classes are described in Table 6.D.2. Class 1 and Class 2 samples listed in Table G-2 are generally referred to as 'undisturbed' while Classes 3, 4, and 5 as 'disturbed' samples. The amount of sample generally required for testing purposes is given in Table 6.D.3.

Table 6.D.2: Categories of Soil Samples Based on Quality

Quality	Recommended use of Sample
Class 1	Stratigraphy, Stratification, Atterberg limits, particle size, moisture content, organic content, unit weight, relative density/density index, porosity, permeability, shear strength and compressibility.
Class 2	Stratigraphy, Stratification, Atterberg limits, particle size, moisture content, organic content, unit weight, relative density/density index, porosity and permeability.
Class 3	Stratigraphy, Stratification, Atterberg limits, particle size, moisture content and organic content.
Class 4	Stratigraphy, Stratification, Atterberg limits, particle size and organic content.
Class 5	Stratigraphy and Stratification.

Table 6.D.3: Weight of Soil Sample Required for Laboratory Tests

Purpose of Sample	Type of Soil	Weight of Sample Required, kg.
Soil identification, natural moisture content test, mechanical analysis and index properties	Cohesive soil	1
	Sand and gravel	3
Chemical test	Cohesive soil	2
	Sand and gravel	3
Compaction test	Cohesive soil and sand	12.5
	Gravelly soil	25
Comprehensive examination of construction materials including stabilization	Cohesive soil and sand	25 to 50
	Gravelly soil	50 to 100

D.4 Ground Water Measurements in Soil

Groundwater investigations shall provide all relevant information on groundwater needed for geotechnical design and construction. Groundwater investigations should provide, when appropriate, information on:

- (i) the depth, thickness, extent and permeability of water-bearing strata in the ground
- (ii) the elevation of the groundwater surface or piezometric surface of aquifers and their variation over time and actual groundwater levels including possible extreme levels and their periods of recurrence
- (iii) the pore water pressure distribution
- (iv) the chemical composition and temperature of groundwater.

The information obtained should be sufficient to assess the following aspects, where relevant:

- (i) the scope for and nature of groundwater-lowering work
- (ii) possible harmful effects of the groundwater on excavations or on slopes (e.g. risk of hydraulic failure, excessive seepage pressure or erosion)
- (iii) any measures necessary to protect the structure (e.g. waterproofing, drainage and measures against aggressive water)
- (iv) the effects of groundwater lowering, desiccation, impounding etc. on the surroundings
- (v) the capacity of the ground to absorb water injected during construction work

The determination of the groundwater table or pore water pressures in soils and rocks shall be made by installing open or closed groundwater measuring systems into the ground. The type of equipment to be used for groundwater measurements shall be selected according to the type and permeability of ground, the purpose of the measurements, the required observation time, the expected groundwater fluctuations and the response time of the equipment and ground.

There are two main methods for measuring the groundwater pressure; open systems and closed systems. In open systems the piezometric groundwater head is measured by an observation well, usually provided with an open pipe. In closed systems the groundwater pressure at the selected point is directly measured by a pressure transducer. Open systems are best suited for soils and rock with a relatively high permeability (aquifers and aquitards), e.g. sand and gravel or highly fissured rock. With soils of low permeability they may lead to erroneous interpretations, due to the time lag for filling and emptying the pressure pipe. The use of filter tips connected to a small diameter hose in open systems, decreases the time lag. Closed systems can be used in all types of soil. They should be used in very low permeability soils. Closed systems are also recommended when dealing with high artesian water pressure. When short term variations or fast pore water fluctuations are to be monitored, continuous recording shall be used by means of transducers and data loggers.

In cases where open water is situated within or close to the investigation area, the water level shall be considered in the interpretation of the groundwater measurements. The water level in wells, the occurrence of springs and artesian water shall also be noted.

The number, location and depth of the measuring stations shall be chosen considering the purpose of the measurements, the topography, the stratigraphy and the soil conditions, especially the permeability of the ground or identified aquifers.

For monitoring projects e.g. groundwater lowering, excavations, fillings and tunnels, the location shall be chosen with respect to the expected changes to be monitored. For reference purposes, measurement of the natural fluctuations in ground water should be made, if possible, outside the area affected by the actual project. The number and frequency of readings and the length of the measuring period for a given project shall be planned considering the purpose of the measurements and the stabilization period.

If it is intended to assess groundwater fluctuations, measurements shall be taken at intervals smaller than the natural fluctuations to be characterized and over a long period of time.

During the drilling process, the observation of the water level at the end of the day and the start of the following day (before the drilling is resumed) is a good indication of the groundwater conditions and should be recorded. Any sudden inflow or loss of water during drilling should also be recorded, since it can provide additional useful information during the first phases of site investigations, some of the boreholes may be equipped with open perforated pipes protected with filters. The water level readings obtained during the following days yield a preliminary indication of groundwater conditions.

The evaluation of groundwater measurements shall take into account the geological and geotechnical conditions of the site, the accuracy of individual measurements, the fluctuations of pore water pressures with time, the duration of the observation period, the season of measurements and the climatic conditions during and prior to that period. The evaluated results of groundwater measurements shall comprise the observed maximum and minimum elevations of the water table, or pore pressures and the corresponding measuring period. If applicable, upper and lower bounds for both extreme and normal circumstances shall be derived from the measured values, by adding or subtracting the expected fluctuations or a reduced part of them, to the respective extreme or normal circumstances. The frequent lack of reliable data for extended periods of time of this type of measurements will necessitate the derived values being a cautious estimate based on the limited available information. The need for making further measurements or installing additional measuring stations should be assessed during the field investigations and in the geotechnical investigation report.

D.5 Standard Penetration Test

The Standard Penetration Test (SPT) is widely used to determine the in-situ properties of soil. The test is especially suited for cohesionless soils as the correlation between the SPT value and ϕ is now well established. The standard test methods are provided in ASTM D156 and ASTM D6066.

The test consists of driving a split spoon sampler, Figure 6.D.2, into soil through a borehole of 55 to 100 mm in diameter at the desired depth. It is done by a hammer weighing 65 kg (140 lb) dropping onto a drill rod from a height of 750 mm (30 inch). The number of blows N required to produce a penetration of 300 mm (12 inches) is regarded as the penetration resistance. To avoid seating errors, the blows for the first 150 mm (6 inches) of penetration are not taken into account; those required to increase the penetration from 150 mm to 450 mm constitute the N -value. A demonstration of Standard Penetration Test is shown in Figure 6.D.3.

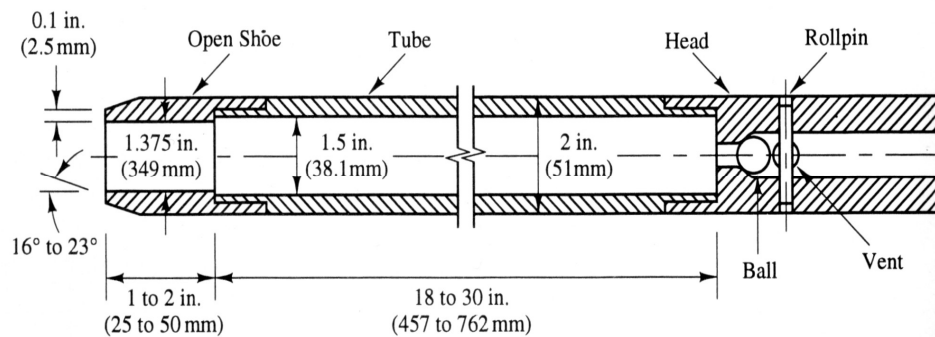


Figure 6.D.2 Schematic diagram of split spoon sampler

It is important to point out that several factors contribute to the variation of the standard penetration number N at a given depth for similar soil profiles. Among these factors are the SPT hammer efficiency, borehole diameter, sampling method, rod length, water table and overburden pressure important. The most two common types of SPT hammers used in the field are the safety hammer and donut hammer. They are usually dropped using a rope with two wraps around a pulley. The hammers are shown in Figure 6.D.4.

Usually SPT is conducted at every 1.5 m or 2 m depth or at the change of stratum. In hard formations, the testing is discontinued if N value is found to be over 100 and it is termed refusal.

D.5.1 Corrections/Standardization of SPT Value for Field Procedures for all Soil Types

On the basis of field observations, it appears reasonable to standardize the field SPT number as a function of the input driving energy and its dissipation around the sampler around the surrounding soil. The variations in testing procedures may be at least partially compensated by converting the measured N to N_{60} as follows.

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} \quad (6.D.1)$$

Where,

N_{60} = Corrected SPT N-value for field procedures

E_H = Hammer efficiency (Table 6.D.4)

C_B = Borehole diameter correction (Table 6.D.5)

C_S = Sampler correction (Table 6.D.5)

C_R = Rod length correction (Table 6.D.5)

N = Measured SPT N-value in field

This correction is to be done irrespective of the type of soil encountered.

Table 6.D.4: SPT Hammer Efficiencies

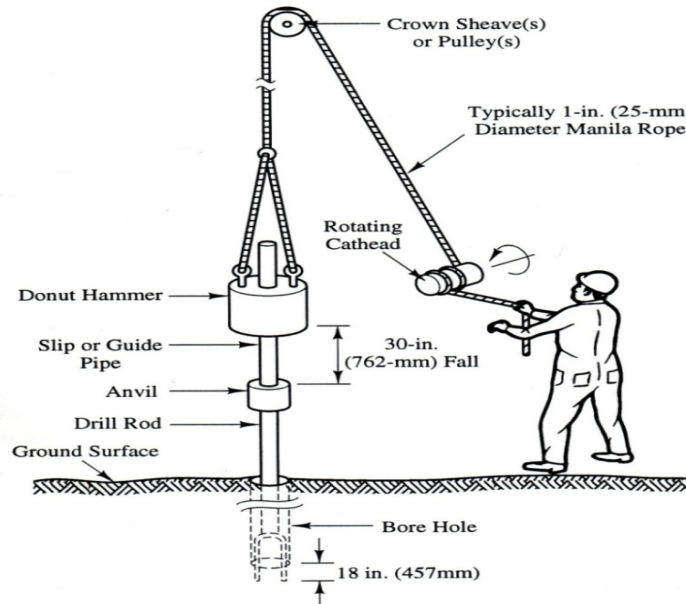
Hammer Type	Hammer Mechanism	Release	Efficiency, E_H
Automatic	Trip		0.70
Donut	Hand dropped		0.60
Donut	Cathead + 2 turns		0.50
Safety	Cathead + 2turns		0.55-0.60
Drop/ Pin	Hand dropped		0.45

Table 6.D.5: Borehole, Sampler and Rod Correction Factors

Factor	Equipment Variables	Correction Factor
Borehole Diameter Factor, C_B	65 mm – 115 mm	1.00
	150 mm	1.05
	200 mm	1.15
Sampling Method Factor, C_S	Standard sampler	1.00
	Sampler without liner	1.20
Rod Length Factor, C_R	3 m – 4 m	0.75
	4 m – 6 m	0.85
	6 m – 10 m	0.95
	> 10 m	1.00

D.5.2 Corrections of SPT Value for Overburden Pressure for all Types of Cohesionless Soils

In cohesionless soils, the overburden pressure affects the penetration resistance. For SPT made at shallow levels, the values are usually too low. At a greater depth, the same soil at the same density index would give higher penetration resistance. It was only as late as in 1957 that Gibbs & Holtz (1957) suggested that corrections should be made for field SPT values for depth.

**Figure 6.D.3 Demonstrations of standard penetration test**

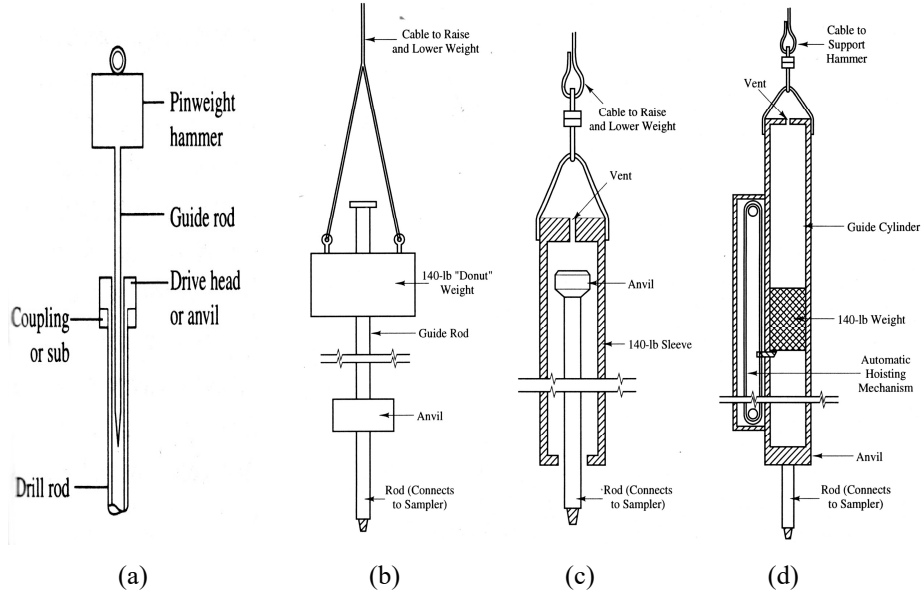


Figure 6.D.4 Configurations of various SPT hammers; (a) Pin (Drop) hammer; (b) Donut hammer; (c) Safety hammer; (d) Automatic hammer

As the correction factor came to be considered only after 1957, all empirical data published before 1957 like those by Terzaghi is for uncorrected values of SPT. Since then a number of investigators have suggested overburden correction. Gibbs & Holtz took standard pressure of 280 kN/m² (corresponding to a depth of 14 m) and duly made overburden correction for other overburdens. Thornburn suggested a standard pressure of 138 kN/m² (corresponding to a depth of 7 m). Finally, Peck et. al. (1974) suggested a standard pressure of 100 kN/m² (Equivalent to 1 tsf or 1 kg/cm² overburden corresponding to a depth of 5 m). As such, all field SPT values are to be corrected by the correction factor given by them as:

$$C_N = 0.77 \log \frac{(2000)}{\sigma'_0} \quad (6.D.2)$$

Where, σ'_0 is the effective overburden pressure. Thus,

$$(N_1)_{60} = C_N \times N_{60} \quad (6.D.3)$$

While using the correction factor given by Equations 6.D.2 and 6.D.3, the corrected N_{60} value is termed as $(N_1)_{60}$ as this correction corresponds to an overburden pressure of 1 ton/ft² or 1 kg/cm². The maximum value of correction factor C_N is 2.

D.5.3 Corrections of SPT Value for Water Table (Dilatancy) in case of Fine Sand and Silty Sand

In addition to corrections of overburden, investigators suggested corrections of SPT-value for water table in the case of fine sand or silt below water table. Apparently, high N-values may be observed especially when observed value is higher than 15 due to dilatancy effect. In such cases, following correction is recommended (Terzaghi and Peck, 1948).

$$(N_1)_{60(CORR)} = 15 + \frac{1}{2}[(N_1)_{60} - 15] \quad (6.D.4)$$

Where, $(N_1)_{60(CORR)}$ is the corrected $(N_1)_{60}$ for water table. For coarse sand this correction is not required. In applying this correction, overburden correction is applied first and then this dilatancy correction is used.

D.5.4 SPT Value and Density Index Relations

Although, the SPT is not considered as a refined and a completely reliable method of investigation, it gives useful information with regard to relative density of cohesionless soil and consistency of cohesive soils. Terzaghi and Peck give the following correlation (Tables 6.D.6 and 6.D.7) between SPT value and other soil parameters.

Several investigators presented average relations between SPT value and peak angle of internal friction, ϕ . The following relations may be noted.

$$\phi^\circ = 27 + 0.3(N_1)_{60} \quad (6.D.5)$$

$$\phi^\circ = \sqrt{20(N_1)_{60}} + 15 \quad (6.D.6)$$

The relations are also presented in Figure 6.D.5. The $(N_1)_{60}$ values used in Equations 6.D.5 and 6.D.6 are corrected values of $(N_1)_{60}$

Table 6.D.6: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesionless Soil: Fairly reliable)

N-value	Soil Condition	Relative Density, D_r	Angle of internal friction, ϕ
0 - 4	Very loose	0 – 15%	< 28°
4 - 10	Loose	15 – 35%	28° – 30°
10 - 30	Medium	35 – 65%	30° – 36°
30 - 50	Dense	65 – 85%	36° – 42°
> 50	Very Dense	> 85%	> 42°

Table 6.D.7: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesive Soil)

N- value	Consistency	UC Strength (q_{uc}), kN/m ²
0 - 2	Very soft	< 25
2 - 4	Soft	25 - 50
4 - 8	Medium	50 - 100
8 - 15	Stiff	100 - 200
15 - 30	Very stiff	200 - 400
> 30	Hard	> 400

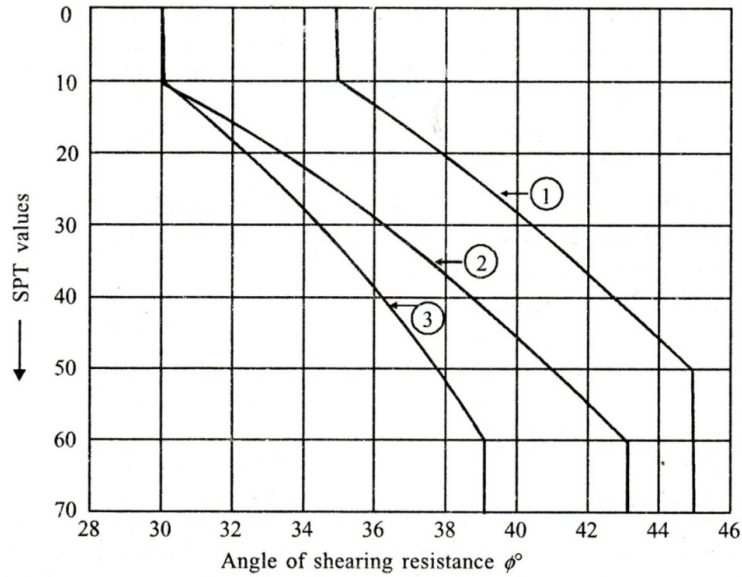


Figure 6.D.5 SPT- ϕ relations for granular soils; (1) Well graded sand and gravel; (2) Uniform fine sand (Average value); and (3) Silty Sand

The correlation for clays with SPT value is not fully established. Hence, vane shear test is recommended for more reliable information. Even though, SPT values are not considered as a good measure of the strength of clays, it is used extensively as a measure of the consistency of clays. The consistency is then related to its approximate strengths. They may be expressed as:

$$q_u \text{ (in kN per m}^2\text{)} = 12.5 N_{60} \quad (6.D.7)$$

Where, q_u is unconfined compressive strength. For highly plastic clays ($PI > 30$) the relation may be:

$$q_u \text{ (in kN per m}^2\text{)} = 10 N_{60} \quad (6.D.8)$$

The relations (q_u in kg/cm²) are also expressed in Figure 6.D.6.

D.6 Dynamic Cone Penetration Test

The dynamic cone penetration test can be considered only as a variation of SPT. Instead of the spoon sampler used for SPT, a special solid cone of 60° and 50 mm diameter is used as a penetrometer. This probe can be used either in the borehole as an SPT test or without a borehole as a continuous penetration test. The latter may be of the recoverable type or the expendable type. Experience shows that in most soils, the solid cone penetrometer test tends to give a slightly higher value than SPT. These tests are to be used along with SPT tests so that a correlation between the two can be worked out for each site under investigation. It is then used to determine the nature of deposits in other locations at the same site without putting an expensive borehole. As this test is very much cheaper than SPT tests in boreholes, a large number of dynamic cone penetration tests can be made at various locations at nominal cost along with SPT tests.

The blow count for every 100 mm penetration due to a 65 kg weight falling through 750 mm is taken. The total blows for one foot (300 mm) penetration is the dynamic cone value N_{dc} . To save the equipment from damage, driving may be stopped when the number of blows exceeds 35 for 100 mm penetration. Typical cone used dynamic cone penetration test is shown in Figure 6.D.7.

Dynamic cone test can be used with or without bentonite (mud) slurry. But when depth of investigation is more than 6 m, use of bentonite or mud slurry is recommended as otherwise friction on the rods would be tremendous. Data from dynamic cone test is plotted as a curve of penetration resistance, N_{dc} number of blows per 30 cm of penetration, versus depth. The N_c values from dynamic cone penetration tests needed to be corrected for overburden pressure in cohesionless soil like N-values of SPT.

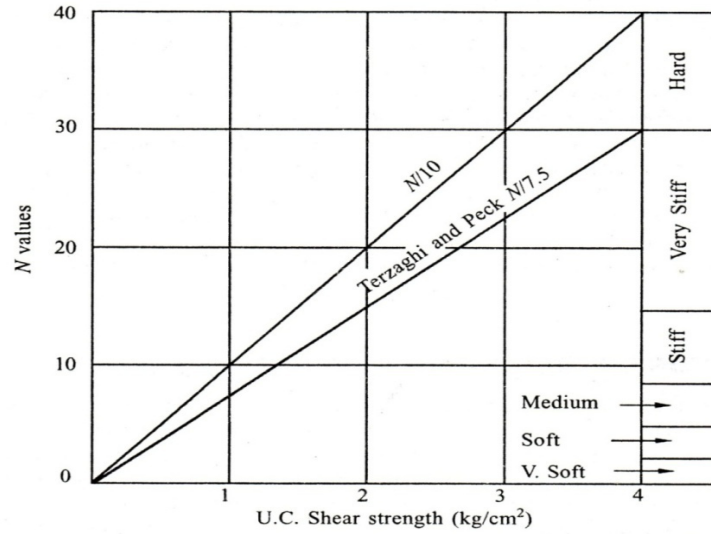


Figure 6.D.6 Relation between SPT and unconfined compressive strength (q_u) of clays

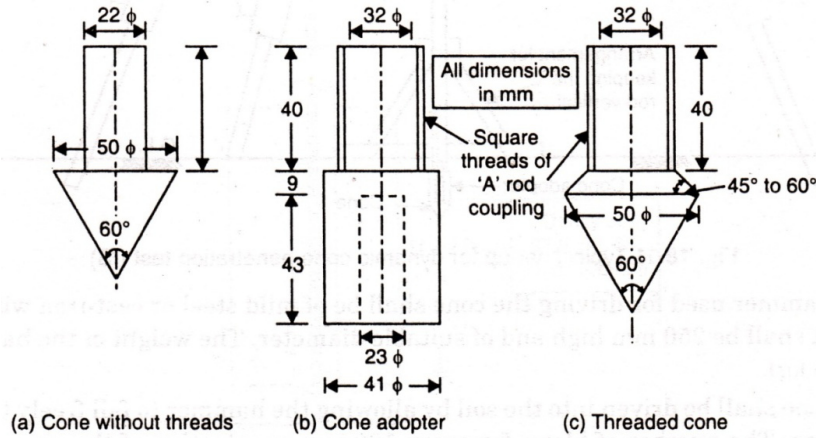


Figure 6.D.7 Typical cone details for dynamic cone penetration test

D.7 Static Cone Penetration Test (Cpt)

The static cone penetration test normally called the Dutch cone penetration test (CPT). It has gained acceptance rapidly in many countries. The method was introduced nearly 50 years ago. The test can be now performed using both mechanical and electrical cones. These test procedures can be found in ASTM D3441 and ASTM D5778 respectively. This test is widely used in Holland, Belgium, Britain, Indonesia, Malaysia, Singapore, West Indies etc. It is finding increasing use in India.

The equipment consists essentially of a steel cone with an apex angle of 60° and overall base diameter of 35.7 mm giving a cross sectional area of 10 cm^2 . The cone is attached to rod which is in turn connected to other rods as necessary. These rods are protected by sleeves known as mantle tubes.

Immediately above the cone a friction jacket, of outside diameter greater than mantle tube, is fitted. The cone and the friction jacket in combination or separately are pushed into the ground by hydraulic cylinder of a machine of capacities presently varying from 20 kN to 100 kN. The necessary reaction is obtained by anchors and sometimes by surcharge loading.

One of the greatest values of the CPT consists of its function as a scale model pile test. Empirical correlations established over many years permit the calculation of pile bearing capacity directly from the CPT results without the use of conventional soil parameters.

The CPT has proved valuable for soil profiling as the soil type can be identified from the combined measurement of end resistance of cone and side friction on a jacket. The test lends itself to the derivation of normal soil properties such as density, friction angle and cohesion. Various theories have been developed for foundation design. The popularity of the CPT can be attributed to the following three important factors:

- (i) General introduction of the electric penetrometer providing more precise measurements, and improvements in the equipment allowing deeper penetration.
- (ii) The need for the penetrometer testing in-situ technique in offshore foundation investigations in view of the difficulties in achieving adequate sample quality in marine environment.
- (iii) The addition of other simultaneous measurements to the standard friction penetrometer such as pore pressure and soil temperature.

D.7.1 The Penetrometer

There are a variety of shapes and sizes of penetrometers being used. The one that is standard in most countries is the cone with an apex angle of 60° and a base area of 10 cm^2 . The sleeve (jacket) has become a standard item on the penetrometer for most applications. On the 10 cm^2 cone penetrometer, the friction sleeve should have an area of 150 cm^2 as per standard practice. The ratio of side friction and bearing resistance, the friction ratio, enables identification of the soil type and provides useful information in particular when no borehole data are available. Even when borings are made, the friction ratio supplies a check on the accuracy of the boring logs.

Two types of penetrometers are used which are based on the method used for measuring cone resistance and friction. They are: (i) The Mechanical Type; (ii) The Electrical Type.

D.7.2 Mechanical Penetrometer

The Begemann Friction Cone Mechanical type penetrometer is shown in Figure 6.D.8. It consists of a 60° cone with a base diameter of 35.6 mm (sectional area 10 cm²). A sounding rod is screwed to the base. Additional rods of one metre length each are used. These rods are screwed or attached together to bear against each other. The sounding rods move inside mantle tubes. The inside diameter of the mantle tube is just sufficient for the sounding rods to move freely whereas the outside diameter is equal to or less than the base diameter of the cone. All dimensions in Figure 6.D.8 is in mm.

The rigs used for pushing down the penetrometer consist basically of a hydraulic system. The thrust capacity for cone testing on land varies from 20 to 30 kN for hand operated rigs and 100 to 200 kN for mechanically operated rigs as shown in Figure 6.D.9.

Bourden gauges are provided in the driving mechanism for measuring the pressures exerted by the cone and friction jacket either individually or collectively during the operation. The rigs may be operated either on the ground or mounted on heavy duty trucks. In either case, the rig should take the necessary up thrust. For ground based rigs screw anchors are provided to take up the reaction thrust.

D.7.3 Operation of Mechanical Penetrometer

The sequence of operation of the penetrometer shown in Figure 6.D.10 is explained as follows:

Position 1: The cone and friction jacket assembly in a collapsed position.

Position 2: The cone is pushed down by the inner sounding rods to a depth until a collar engages the cone. The pressure gauge records the total force Q_c to the cone. Normally, $a = 40$ mm.

Position 3: The sounding rod is pushed further to a depth b . This pushes the friction jacket and the cone assembly together; the force is Q_f . Normally, $b=40$ mm.

Position 4: The outside mantle tube is pushed down a distance $(a + b)$ which brings the cone assembly and the friction jacket to position 1. The total movement $= a + b = 80$ mm.

The process of operation illustrated is continued until the proposed depth is reached. The cone is pushed at a standard rate of 20 mm per second. The mechanical penetrometer has its advantage as it is simple to operate and the cost of maintenance is low. The quality of the work depends on the skill of the operator. The depth of CPT is measured by recording the length of the sounding rods that have been pushed into the ground.

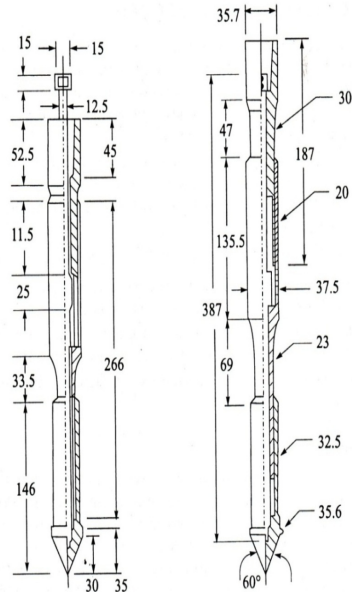


Figure 6.D.8 Friction cone mechanical type penetrometer jacking system (all dimensions are in mm)

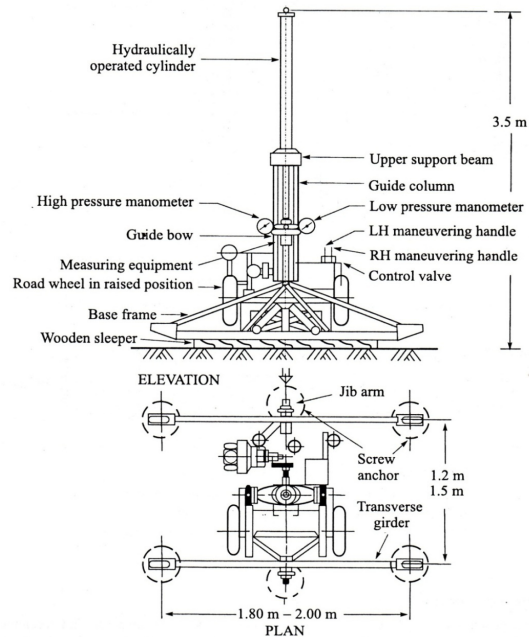


Figure 6.D.9 Assembly of cone penetration rig

D.7.4 The Electric Penetrometer

The electric penetrometer is an improvement over the mechanical one. Mechanical penetrometers operate incrementally whereas the electric penetrometer is advanced continuously.

Figure 6.D.11 shows an electric-static penetrometer with the friction sleeve just above the base of the cone. The sectional area of the cone and the surface area of the friction jacket remain the same as those of a mechanical type. The penetrometer has built in load cells that record separately the cone bearing and side friction. Strain gauges are mostly used for the load cells. The load cells have a normal capacity of 50 to 100 kN for end bearing and 7.5 to 15 kN for side friction, depending on the soils to be penetrated. An electric cable inserted through the push rods (mantle tube) connects the penetrometer with the recording equipment at the surface which produces graphs of resistance versus depth.

The electric penetrometer has many advantages. The repeatability of the cone test is very good. A continuous record of the penetration results reflects better the nature of the soil layers penetrated. However, electronic cone testing requires skilled operators and better maintenance. The electric penetrometer is indispensable for offshore soil investigation.

D.7.5 Operation of Electric Penetrometer

The electric penetrometer is pushed continuously at a standard rate of 20 mm per second. A continuous record of the bearing resistance q_c and frictional resistance f_s against depth is produced in the form of a graph at the surface in the recording unit.

D.7.6 Piezocone

A piezometer element included in the cone penetrometer is called a piezocone, Figure 6.D.12. There is now a growing use of the piezocone for measuring pore pressures at the tips of the cone. The porous element is mounted normally midway along the cone tip allowing pore water to enter the tip. An electric pressure transducer measures the pore pressure during the operation of the CPT. The pore pressure record provides a much more sensitive means to detect thin soil layers. This could be very important in determining consolidation rates in a clay soil within the sand seams.

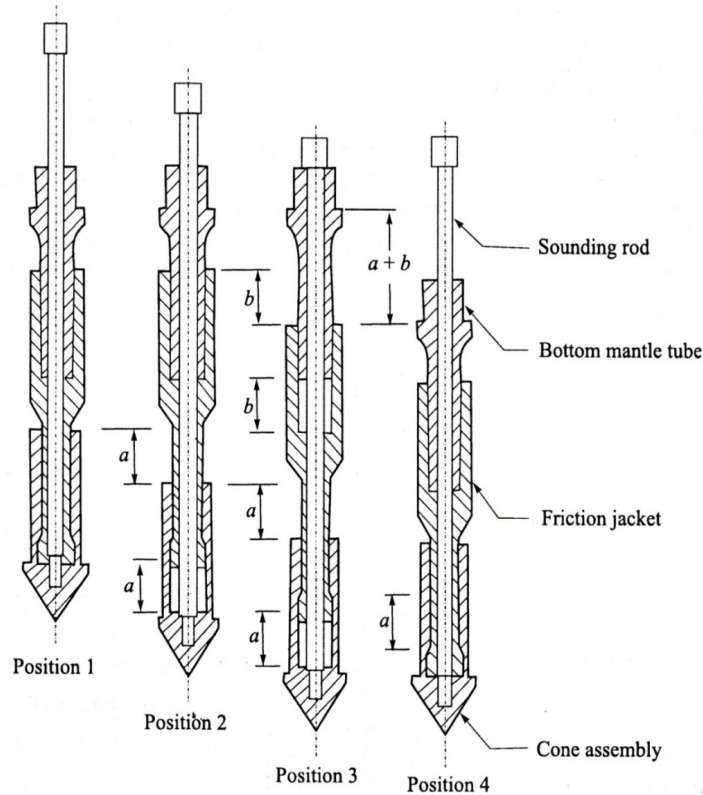


Figure 6.D.10 Steps in cone penetration testing

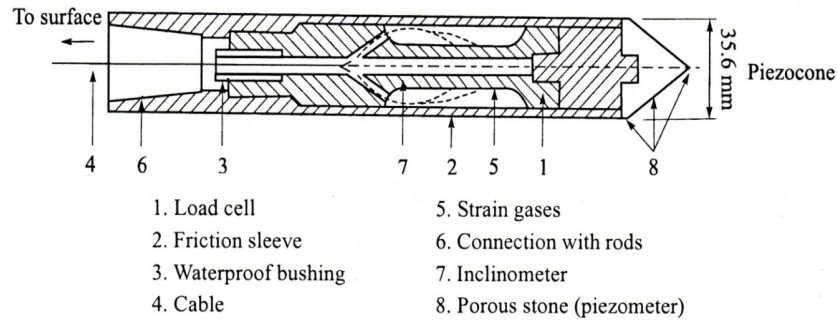


Figure 6.D.11 Electrical static cone penetrometer

D.7.7 Temperature Cone

The temperature of a soil is required at certain localities to provide information about environmental changes. The temperature gradient with depth may offer possibilities to calculate the heat conductivity of the soil. Measurement of the temperature during CPT is possible by incorporating a temperature sensor in the electric penetrometer. Temperature measurements have been made in permafrost, under blast furnaces, beneath undercooled tanks, along marine pipe lines, etc.

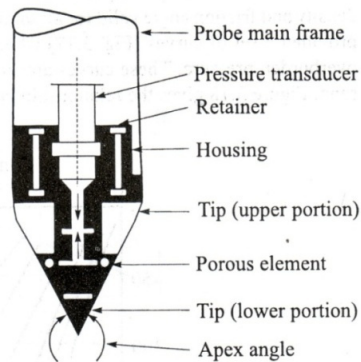


Figure 6.D.12 Details of piezocone

D.7.8 Effect of Rate of Penetration

Several studies have been made to determine the effect of the rate of penetration on cone bearing and side friction. Although the values tend to decrease for slower rates, the general conclusion is that the influence is insignificant for speeds between 10 and 30 mm per second. The standard rate of penetration has been generally accepted as 20 mm per second.

D.7.9 Cone Resistance q_c and Local Side Friction f_c

Cone penetration resistance q_c is obtained by dividing the total force Q_c acting on the cone by the base area A_c .

$$q_c = \frac{Q_c}{A_c} \quad (6.D.9)$$

In the same way, the local side friction f_c is:

$$f_c = \frac{Q_f}{A_f} \quad (6.D.10)$$

Where, $Q_f = Q_r - Q_c$ = force required for pushing the friction jacket

Q_t = total force required to push the cone and friction jacket together in the case of a mechanical penetrometer,

A_f = surface area of the friction jacket.

D.7.10 Friction Ratio, R_f

Friction ratio, R_f is expressed as:

$$R_f = \frac{f_c}{q_c} \quad (6.D.11)$$

Where, f_c and q_c are measured at the same depth. R_f is expressed as a percentage.

Friction ratio is an important parameter for classifying soil.

D.7.11 Relationship between q_c , Relative Density D_r and Friction Angle ϕ for Sand

Research carried out by many indicates that a unique relationship between cone resistance, relative density and friction angle valid for all sands does not exist. Robertson and Campanella (1983a) have provided a set of curves (Figure 6.D.13) which may be used to estimate D_r based on q_c and effective overburden pressure. These curves are supposed to be applicable for normally consolidated clean sand. Figure 6.D.14 gives the relationship between q_c and ϕ (Robertson and Campanella, 1983b).

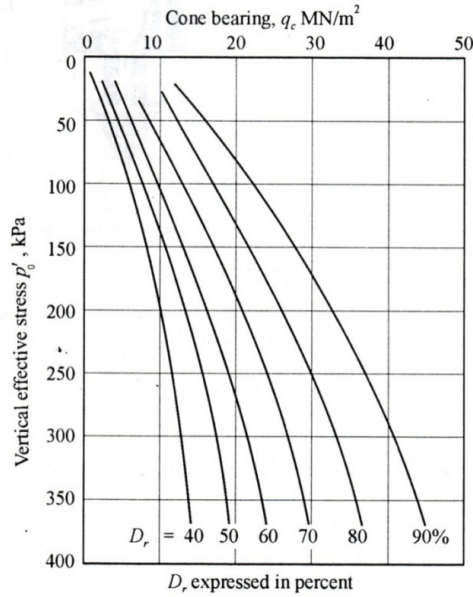


Figure 6.D.13 Relationship between relative density, D_r , and cone resistance, q_c for quartz sand (Robertson and Campanella, 1983a)

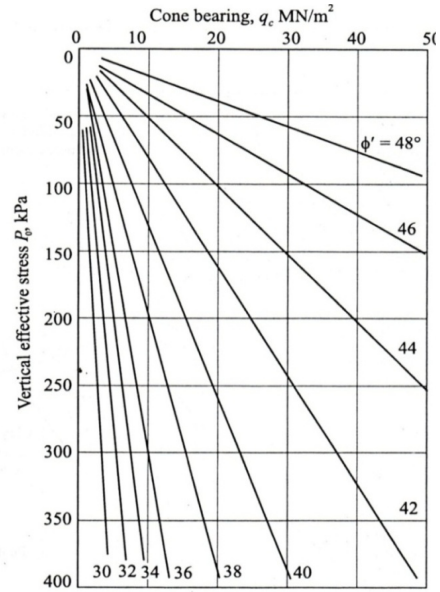


Figure 6.D.14 Relationship between cone resistance, q_c , and relative density, D_r , for quartz sand (Robertson and Campanella, 1983b)

D.7.12 Relationship between q_c and Undrained Shear Strength, c_u of Clay Soil

The cone penetration resistance q_c and c_u may be related as:

$$q_c = N_k c_u + p_o \text{ or, } c_u = \frac{q_c - p_o}{N_k} \quad (6.D.12)$$

Where, N_k = cone factor,

$p_o = \gamma Z$ = overburden pressure.

Lune and Kelven (1981), investigated the value of the cone factor N_k for both normally consolidated and overconsolidated clays. The values of N_k as obtained are given below in Table 6.D.7.

Table 6.D.7: Soil Type and q_c Value

Type of Clay	Cone Factor, N_k
Normally Consolidated	10 to 19
Over Consolidated:	
At shallow depth	15 to 20
At deep depth	12 to 18

Possibly a value of 20 for N_k for both types of clay may be satisfactory. Sanglerat (1972) recommends the same value for all cases where an overburden correction is of negligible value.

D.7.13 Soil Classification based on CPT Results

One of the basic uses of CPT is to identify and classify soils. A CPT-Soil Behaviour Type Prediction System has been developed by Douglas and Olsen (1981) using an electric and friction cone penetrometer. The classification is based on the friction ratio f_c/q_c . The ratio f_c/q_c varies greatly depending on whether it applies to clays or sands. Their findings have been confirmed by hundreds of tests.

For clay soils, it has been found that the friction ratio decreases with increasing liquidity index, I_L . Therefore, the friction ratio is an indicator of the soil type penetrated. It permits approximate identification of soil type though no samples are recovered.

Douglas (1984), presented a simplified classification chart shown in Figure 6.D.15. His chart uses cone resistance normalized (q_{cn}) for overburden pressure using the equation:

$$q_{cn} = q_c (1 - 1.25 p'_o) \quad 6.D.13)$$

Where, p'_o = effective overburden pressure in tsf, and q_c = cone resistance in tsf.

The CPT data provides a repeatable index of the aggregate behavior of in-situ soil. The CPT classification method provides a better picture of overall subsurface conditions than is available with most other methods of exploration. A typical sounding log is given in Figure 6.D.16.

The friction ratio R_f varies greatly with the type of soil. The variation of R_f for the various types of soils is generally of the order given in Table 6.D.8.

Table 6.D.8: Soil Classification Based on Friction Ratio

Friction Ratio, R_f (%)	Type of Soil
0.0 - 0.5	Loose gravel fill
0.5 - 2.0	Sands and gravels
2.0 - 5.0	Clay sand mixtures and silts
> 5.0	Clays, peats etc.

Meyerhof (1965), presented comparative data between SPT and CPT. For fine or silty medium loose to medium dense sands, he presents the correlation as:

$$q_c = 0.4N \quad (\text{in MN/m}^2) \quad (6.D.14)$$

Meyerhof's findings are as given in Table 6.D.9. The lowest values of the angle of ϕ given in Table 6.D.9 are conservative estimates for uniform, clean sand and they should be reduced by at least 5° for clayey sand. These values, as well as the upper values of ϕ that apply to well graded sand, may be increased by 5° for gravelly sand.

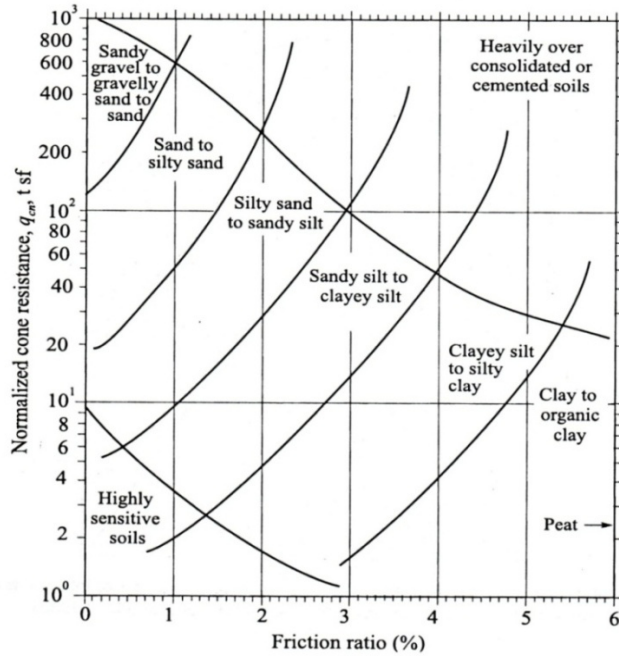


Figure 6.D.15 A simplified classification chart (after Douglas, 1984)

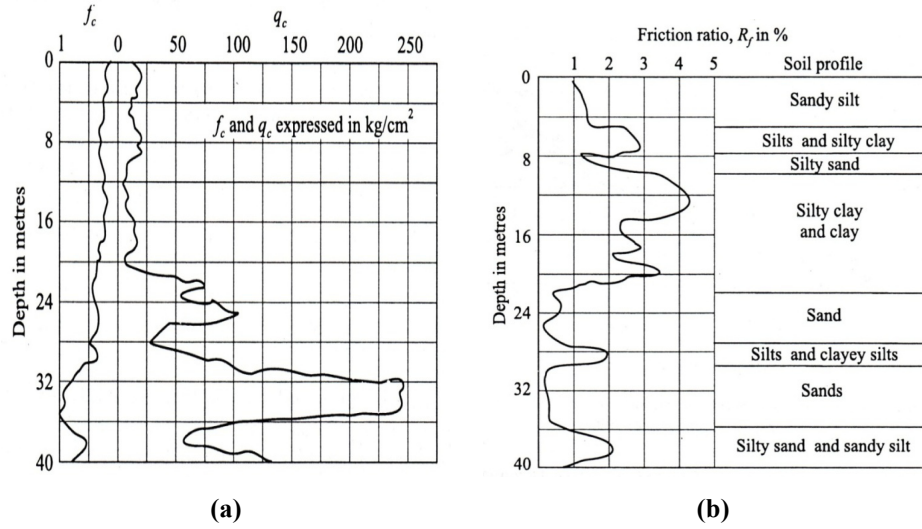


Figure 6.D.16 Typical Sounding Log; (a) Cone resistance with depth; (b) Friction ratio with depth

Table 6.D.9: Soil Classification Based on Friction Ratio

State of Sand	D_r	$(N_1)_{60}$	q_c (MPa)	ϕ^a
Very loose	< 0.2	< 4	< 2.0	< 30
Loose	0.2-0.4	4-10	2-4	30-35
Medium dense	0.4-0.6	10-30	4-12	35-40
Dense	0.6-0.8	30-50	12-20	40-45
Very dense	0.8-1.0	> 50	> 20	> 45

D.7.14 Correlation between SPT and CPT

Figure D.17 shows correlations presented by Robertson and Campanella (1983a), and Kuhawy and Mayne (1990) between the ratio of q_c/N and mean grain size, D_{50} . It can be seen from the Figure 6.D.17 that the ratio varies from 1 at $D_{50}=0.001$ mm to a maximum value of 8 at $D_{50}=1.0$ mm. The soil type also varies from clay to sand.

It is clear from the above discussions that the value of $n = q_c/N$ is not a constant for any particular soil. Designers must use their own judgement while selecting a value for n for a particular type of soil.

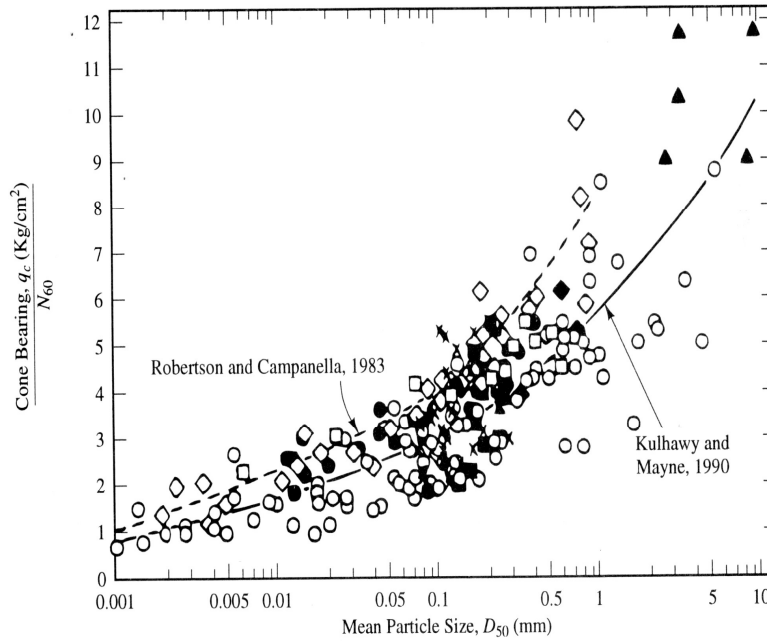


Figure 6.D.17 Relation between q_c/N and mean grain size D_{50}

D.8 Geophysical Methods of Exploration

Several types of geophysical exploration techniques permit a rapid evaluation of subsoil characteristics. These methods also allow rapid coverage of large areas and are less expensive than conventional exploration by drilling. However, in many cases, definitive interpretation of the results is difficult. For that reason, such techniques should be used for preliminary work only. Three types of geophysical exploration are common: the seismic refraction survey, cross-hole seismic survey and electrical resistivity survey.

D.8.1 Seismic Refraction Survey

This method is useful in obtaining preliminary information about the thickness of the layering of various soils and the depth to rock or hard soil at a site. The test method is described in ASTM D5777. Refraction surveys are conducted by impacting the surface, such as point A in Figure 6.D.18a, and observing the first arrival of the disturbance (stress waves) at several other points (e.g. B, C, D, ...). The impact can be created by a hammer blow or by a small explosive charge. The first arrival of disturbance waves at various points can be recorded by geophones.

The impact on the ground surface creates two types of stress wave: 'P' waves (or plane waves) and S waves (or shear waves). P waves travel faster than S waves; hence the first arrival of disturbance waves will be related to the velocities of the 'P' waves in various layers. The velocity of P waves in a medium is:

$$v = \sqrt{\frac{E_s (1 - \mu)}{\left(\frac{\gamma}{g}\right) (1 - \mu_s) (1 + \mu_s)}} \quad (6.D.15)$$

Where, E_s = modulus of elasticity of the medium

γ = unit weight of the medium

g = acceleration due to gravity

μ_s = Poisson's ratio of soil

To determine the velocity v of P waves in various layers and thicknesses of those layers, the following procedure is used.

The first arrival times $t_1, t_2, t_3 \dots$ at various distances $x_1, x_2, x_3 \dots$ from the point of impact is obtained. A graph of time t against distance x is plotted, as shown in Figure 6.D.18b. The slopes of the line ab, bc, cd , are determined.

$$\text{Slope of } ab = \frac{1}{v_1}$$

$$\text{Slope of } bc = \frac{1}{v_2}$$

$$\text{Slope of } cd = \frac{1}{v_3}$$

Here $v_1, v_2, v_3, \dots$ are the P-wave velocities in layers I, II, III, ... respectively (Figure 6.D.18a). The thickness of the top layer is determined as:

$$Z_I = \frac{x_c}{2} \sqrt{\frac{v_2^2 - v_1^2}{v_2^2 + v_1^2}} \quad (6.D.16)$$

The value of x_c can be obtained from Figure 6.D.17b. The thickness of the second layer can be determined from

$$z_2 = \frac{I}{2} \left[T_{i2} - 2Z_I \frac{\sqrt{v_3^2 - v_1^2}}{v_3 v_1} \right] \frac{v_3 v_2}{\sqrt{v_3^2 - v_2^2}} \quad (6.D.17)$$

Where, T_{i2} is the time intercept of the line cd (Figure 6.D.18a). Velocities of P-waves in various layers indicate the types of soil or rock that are present below the ground surface. They are listed in Table 6.D.10.

In analyzing the results of a refraction survey, two limitations need to be kept in mind.

- (i) The basic equations for the survey that is, Equations 6.D.16 and 6.D.17 are based on the assumption that the P-wave velocity $v_1 < v_2 < v_3 \dots$
- (ii) When a soil is saturated below the water table, the P-wave velocity may be deceptive. P-wave can travel with a velocity of about 1500 m/sec through water. For dry, loose soils the velocity may be well below. If the presence of groundwater has not been detected, the P-wave velocity may be erroneously interpreted to indicate a stronger material (sandstone). In general, geophysical interpretations should always be verified by the results obtained from borings.

D.8.2 Cross Hole Survey

The test method can be found in ASTM D4428-07. The velocity of shear waves created as a result of an impact to a given layer of soil can be effectively determined by the cross-hole seismic survey. The principle of this technique is illustrated in Figure 6.D.19. Two holes are drilled into the ground at a distance L apart. A vertical impulse is created at the bottom of one borehole by means of an impulse rod. The shear waves thus generated are recorded by means of a vertically sensitive transducer. The velocity of shear waves can be calculated as:

$$v_s = \frac{L}{t} \quad (6.D.18a)$$

Where, t is the travel time of the wave. The shear modulus G_s of the soil at the depth at which the test is done can be determined from the relation:

$$v_s = \sqrt{\frac{G_s}{(\gamma/g)}} \text{ or } G_s = \frac{v_s^2 \gamma}{g} \quad (6.D.18b)$$

Where, v_s = velocity of shear waves

γ = unit weight of soil

g = acceleration due to gravity

Shear modulus is useful in the design of foundations to support vibrating machinery and the like.

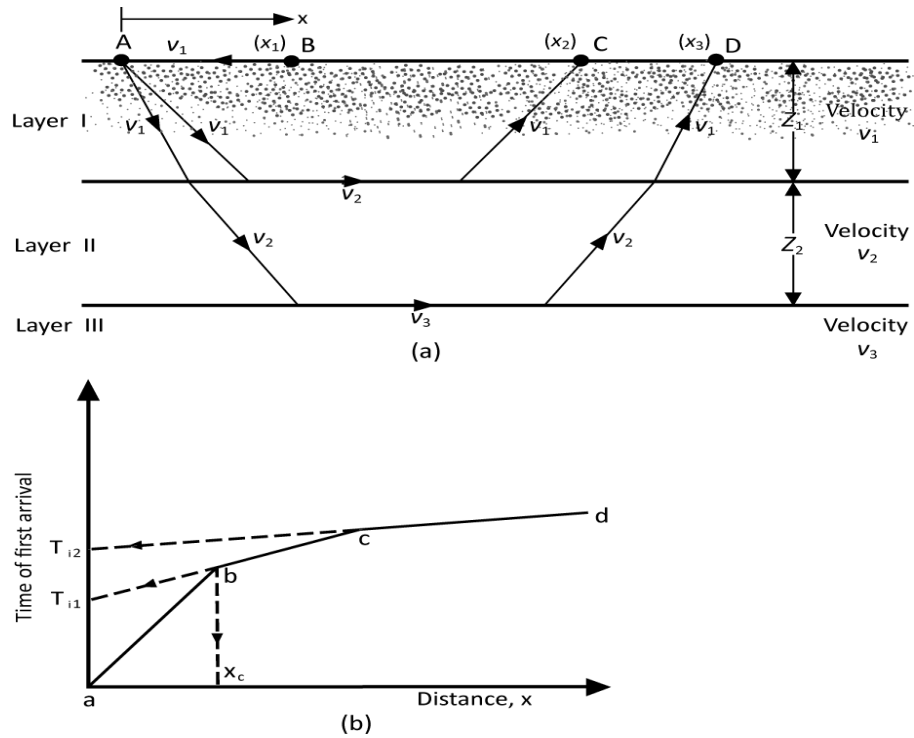


Figure 6.D.18 Seismic refraction survey

Table 6.D.10 : Range of P-Wave Velocity in Various Soils and Rocks

Type of soil or rock	P-wave velocity	
	M/sec	Ft/sec
Soil:		
Sand, dry silt, and fine grained top soil	200—1000	650—3300
Alluvium	500—2000	1650—6600
Compacted clays, clayey gravel and dense clayey sand	1000—2500	3300—8200
Loess	250—750	800—2450
Rock:		
Slate and shale	2500—5000	8200—16400
Sandstone	1500—5000	4900—16400
Granite	4000—6000	13100—19700
Sound limestone	5000—10000	16400—32800

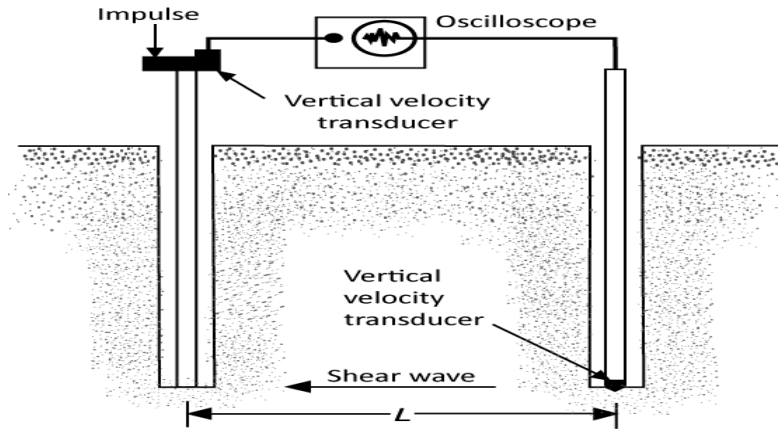


Figure 6.D.19 Cross-hole method of seismic survey

D.8.3 Resistivity Survey

The electrical resistivity of any conducting material having length L , cross sectional area A and electrical resistance R can be defined as :

$$\rho = \frac{RA}{L} \quad (6.D.19)$$

The unit of resistivity is ohm-metre. The resistivity of various soils depends primarily on their moisture content and also on the concentration of dissolve ions in them. The range of resistivity of various soils and rocks are given in Table 6.D.11.

The most common procedure of measuring the electrical resistivity of a soil profile makes use of four electrodes driven into the ground spaced equally along a straight line. The procedure is generally known as Wenner method. The two outside electrodes are used to send an electrical current I (using dc current) into the ground. The current is typically in the range of 50-100 milliamperes. The voltage drop V is measured between the two inside electrodes. The test arrangements are shown in Figure 6.D.20a. If the soil profile is homogeneous, its electrical resistivity is given by :

$$\rho = \frac{2\pi d V}{I} \quad (6.D.20)$$

In most cases, the soil profile may consists of various layers with different resistivities and Eq. 6.D.20 will yield the apparent resistivity. To obtain actual resistivity of various layers and their thicknesses, an empirical method may be used. It involves conducting test at various electrode spacing. Thus, the sum of the apparent resistivities $\Sigma\rho$ obtained is plotted against the spacing d as shown in Figure 6.D.20b. The plot thus obtained has relatively straight segments, the slopes of which give the resistivity of individual layers. The determination of thickness of layers is illustrated in Figure 6.D.20b. The resistivity survey is particularly useful in locationg gravel deposits within a fine grained soil.

Table 6.D.11 Electrical Resistivity of Various Types of Soils

Soil type	Resistivity (ohm-m)
Sand	500-1500
Clays, saturated silts	0-100
Clayey sand	200-500
Gravel	1500-4000
Weathered rock	1500-2500
Sound rock	> 5000

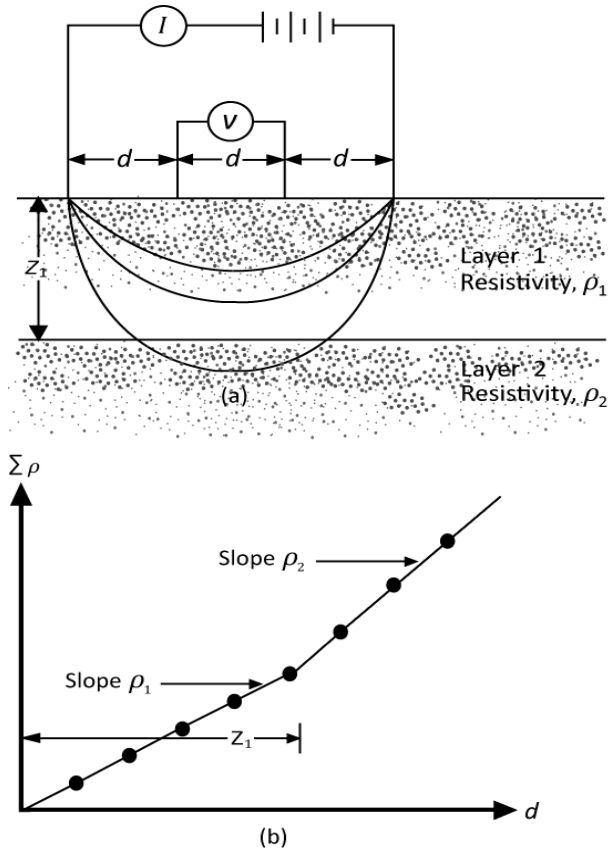


Figure 6.D.20 Electrical resistivity survey; (a) Wenner method of placing electrode; (b) Empirical method of determining resistivity and thickness of soil layer.

PART VI
Appendix E

Recommended Criteria for Identification and Classification of Expansive Soil

The following criteria may be adopted to identify and classify expansive soils:

- (1) Based on the values of plasticity index and shrinkage limit, United States Bureau of Reclamation (USBR) suggests the following classification criteria for expansive soil:

Plasticity Index	Shrinkage Limit	Degree of Expansion
>35	<10	Very High
25-41	6-12	High
15-28	8-18	Medium
<18	>13	Low

- (2) On the basis of previous data for linear shrinkage of Bangladesh soils, criteria for the degree of expansion proposed by Hossain (1983) is as follows:

Linear Shrinkage (%)	Degree of Expansion
> 14	High
10-14	Medium
0-10	Low

- (3) On the basis of the values of free swell, Indian standard (IS: 1948, 1970) recommends criteria of expansion is as follows:

Free Swell (%)	Degree of Expansion	Danger of Severity
<50	Low	Non-critical
50-100	Medium	Marginal
100-200	High	Critical
>200	Very High	Severe