Soil Exploration

19

19.1 INTRODUCTION

The knowledge of subsoil conditions at a site is a prerequisite for safe and economical design of substructure elements. The field and laboratory studies carried out for obtaining the necessary information about the subsoil characteristics including the position of ground water table, are termed as *soil exploration.*

A well planned and properly executed site investigation programme will provide information about the stratigraphy and physical properties of the soils at site including ground water table and its fluctuations. These may be supplemented with geological studies and geological surveys along with the normal topographical survey.

In view of the complexity of natural deposits, no single method of exploration is suited for all situations. The choice depends on the nature of subsoil, their extent, and the purpose of the exploration programme. In discussing the various methods of soil exploration, this chapter provides guidelines for selecting depth, location and method of soil exploration for general types of buildings.

19.2 METHODS OF EXPLORATION

The subsoil explorations are usually carried out in two stages, namely, preliminary and detailed.

Preliminary exploration consists of the geological study of the site and the site reconnaissance. During the site visit, the study of local topography, excavations, cuttings, drainage pattern and other natural features like streams, flood marks etc. will be useful. During preliminary investigations, geophysical methods and tests with cone penetrometers and sounding rods can be very useful.

Detailed investigation follows the preliminary investigation and is normally carried out to determine the nature, sequence and thickness of various subsoil layers, their lateral variation, their physical properties and the position of ground water table. Borings and detailed sampling are usually undertaken to obtain this information. Various *in situ* tests also form a part of the detailed investigation programme.

Detailed soil exploration, can be limited in scope where the subsoil layers are very erratic in distribution, the structure transmits light loads and is relatively unimportant and inexpensive or where a good record of subsoil details already exists or where sound rock is available at a shallow depth. Where the conditions are contrary to these, detailed soil exploration has to be quite extensive and elaborate.

19.3 METHODS OF BORING

Making and advancing of boreholes is known as *boring*. The four different methods commonly used for boring are discussed in the following subsections. The suitability of any particular method of boring depends mainly on the nature of soil, the position of water table, the ease and accuracy with which changes in soil and ground water conditions can be determined and the likely disturbance of soil samples that have to be taken.

Auger Boring

Boring by an auger is carried out by holding it vertically and pressing it down while the auger is rotated. The turning action cuts the soil which fills the annular space. Once the annular space is filled, the auger is withdrawn and cleaned. The cleaned auger is again inserted in the hole and the process repeated. Figure 19.1 shows two types of augers.

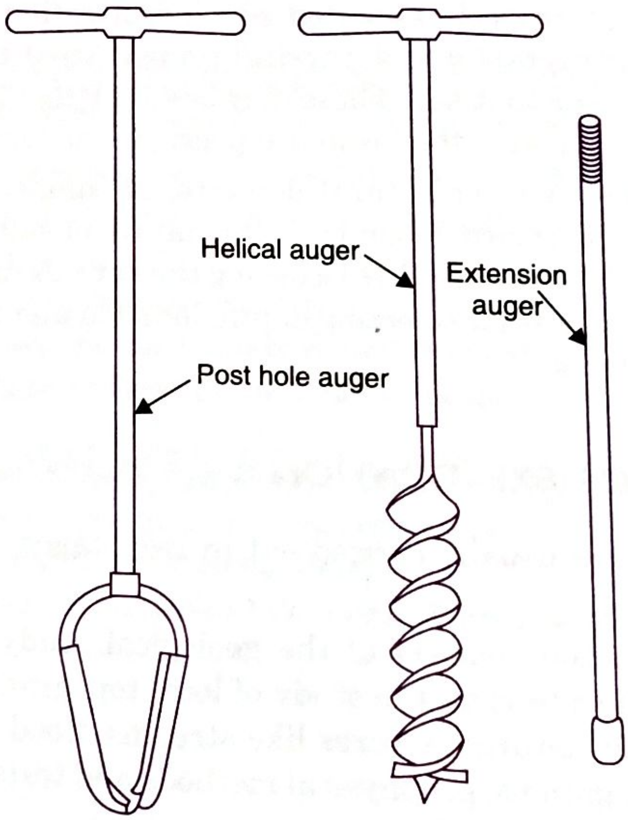


Fig. 19.1 Hand augers

Hand operated augers may be used for boring holes to a depth of 6 m in soft soil which can stand unsupported. However, if the sides of the hole are likely to cave in, casing pipe may be used to prevent the collapse of the sides of borehole. Power driven augers are used for greater boring depths or where hard or stiff soil strata are encountered.

Auger boring is convenient in the case of partially saturated sands, silts and medium to stiff cohesive soil. As far as possible, auger borings are kept dry. Samples obtained from the soil brought up by augers are severely disturbed and therefore are useful for identification purposes only. Auger borings are well suited for explorations for shallow foundations, highways and borrow pits, where the required depth of exploration is relatively small.

Shell and auger method is used widely in India. A shell (Fig. 19.2), also called a *sand bailer*, is a heavy duty pipe with a cutting edge. Different lengths and weights are used according to requirements. Sinker bars are sometimes used to add weight to the bailer. The shell is raised and let fall in a hole. The soil that is cut, enters the tube which is emptied when full. Boring is always started, to begin with, by augering and the shell is used when augering becomes difficult.

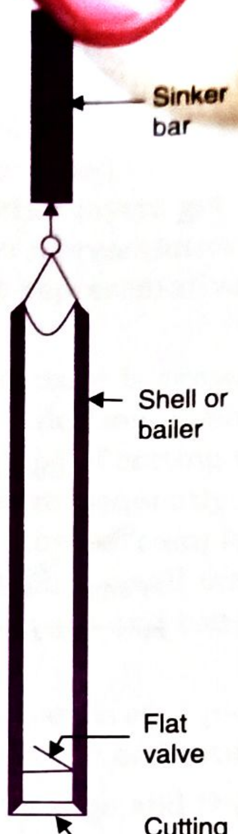


Fig 19.2 Shell with sinkerbar

Wash Boring

The method consists of driving a casing pipe usually through a heavy drop hammer supported on a tripod and pulley. Water is forced under pressure through a hollow drill rod which may be rotated or moved up and down inside the casing pipe. The lower end of the drill rod, fitted with a sharp cutting edge or chopping bit, cuts the soil. The soil thus cut gets mixed with water and floats up through the annular space between the casing pipe and the drill rod (Fig. 19.3). The slurry flowing out provides an indication of the soil type. The change in soil strata can be surmised from the rate of progress and the slurry flowing out.

In this method, heavier particles of different soil layers remain under suspension in the casing pipe and get mixed up. Because of this, the samples recovered from the wash water are of no value. Samples of the soil should be obtained through suitable samplers after the borehole has been cleaned.

Wash boring can be conveniently used even below water table in practically all types of soil except hard soil or rock.

Percussion Boring

Boring by percussion drilling is carried out by breaking up the formation by repeated blows of a heavy bit or a chisel inside a casing pipe. The borehole is usually kept dry, except for a limited quantity of water used to form the slurry of pulverized material. The pulverized slurry is bailed out using a bailer or sand pump. Unless the sides of borehole are likely to cave in, a casing pipe may not be necessary. Many a time, in percussion drilling, drill rods are replaced by cables.

This is the only method suitable for drilling boreholes in bouldery and gravelly strata.

Rotary Boring

Rotary boring or rotary drilling is useful if the soil is highly resistant to augering or wash boring. The method can also be used in case of sands and clay. In this method, boring is effected by the cutting action of a rotating bit which is kept in firm contact with the bottom of the hole. The bit is attached to the lower end of a hollow drill rod which is rotated by a suitable chuck. Drilling mud (usually bentonite solution with some admixtures) is continuously forced down the hollow drill rods. The mud returning upwards through the annular space between the drill rods and the side of the hole brings the cuttings to the surface. The method is also known as *mud rotary drilling*.

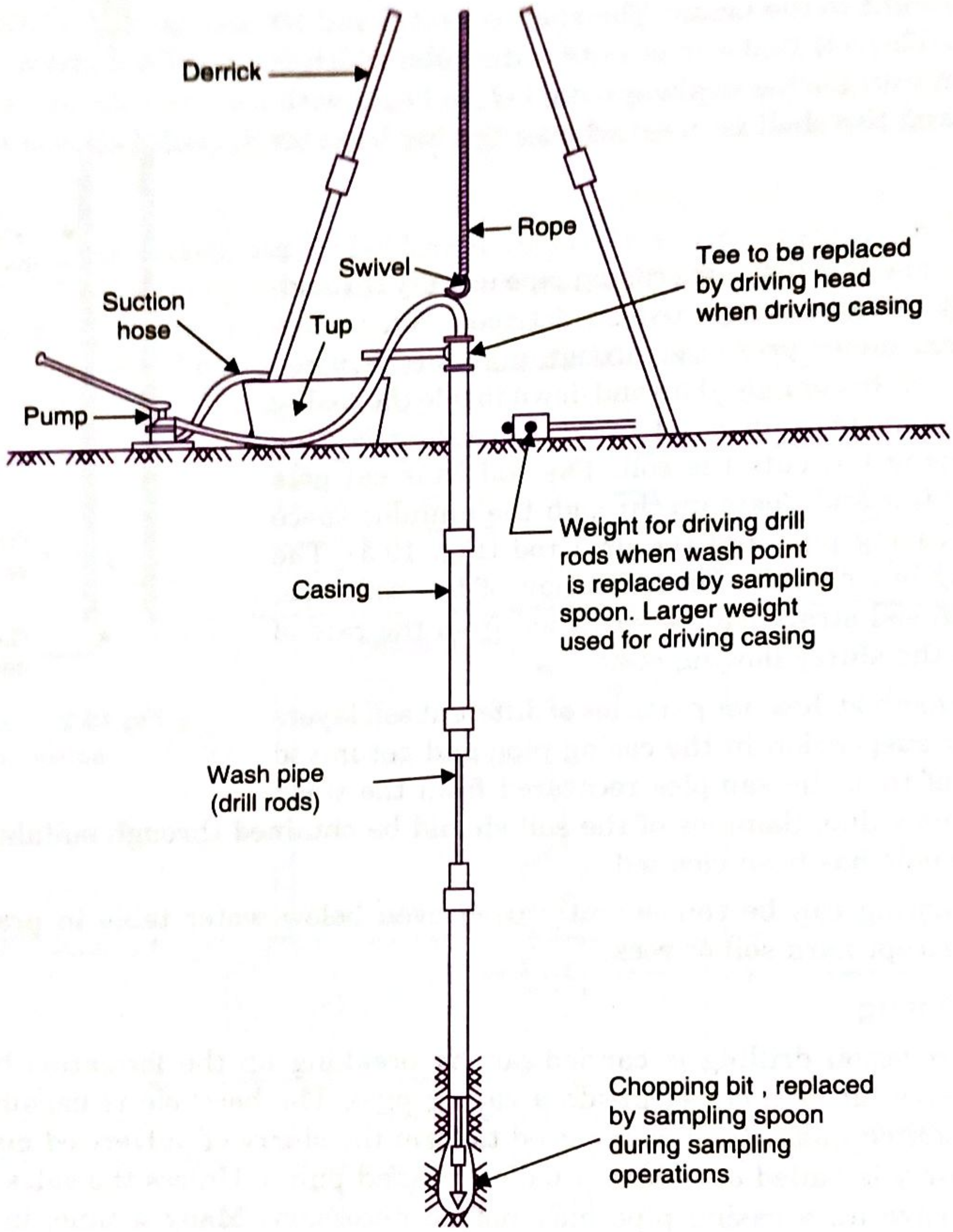


Fig. 19.3 Set-up for wash boring

Core barrels with diamond bits are also used in rotary drilling and enables the simultaneous obtaining of rock cores. The method is known as *core drilling*. For large diameter holes (over 150 mm), *shot drilling* is used. The system is different from other types of core drilling because the coarser cuttings do not return to the surface but are accumulated in a chip immediately above the bit. Chilled shot is used as an abrasive instead of the drilling head.

19.4 SOIL SAMPLES

Soil samples, in general, can be classified into two categories, namely, *disturbed samples* and *undisturbed samples*. Disturbed samples are those where the natural soil structure gets modified or destroyed during the sampling operation. With suitable precautions, the natural moisture content and the proportion of mineral constituents can be preserved. These are called *representative samples*, even though they are disturbed samples. Where, in addition to alteration in the original soil structure, soil from other layers get mixed up or the mineral constituents get altered, the samples are called *non-representative samples*. Representative samples are useful for identification tests but non-representative samples are virtually of no use.

Undisturbed samples are those where the original soil structure is preserved and the material properties have not undergone any alteration or modification: Such samples are practically impossible to obtain as even the most efficient method of cutting of sample from the parent material involves a change in stress conditions and consequently, a change in soil structure. For all practical purposes, however, an undisturbed sample may be considered as one in which the material has been subjected to such a small disturbance that it is still suitable for all laboratory tests including shear strength and consolidation tests.

The extent of disturbance to the sample due to the sampler depends on three features of its design. These are: (*a*) cutting edge, (*b*) inside wall friction and (*c*) non-return valve.

The following ratios related to the dimensions of the cutting edge and the sampler (Fig. 19.4) are useful:

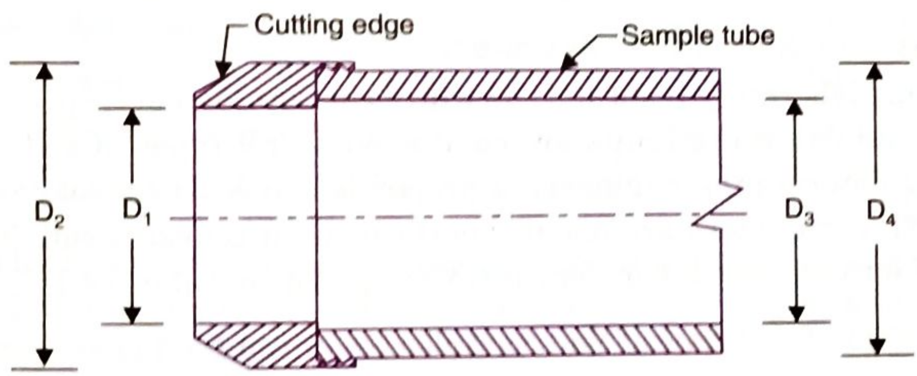


Fig. 19.4 Lower section of a sampler

(*a*) Inside clearance, $$C_i = \frac{D_3 - D_1}{D_1} \times 100 \text{ per cent} \quad \text{...(19.1)}$$

(*a*) Outside clearance, $$C_0 = \frac{D_2 - D_4}{D_4} \times 100 \text{ per cent} \quad \text{...(19.2)}$$

(*a*) Area ratio, $$A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100 \text{ per cent} \quad \text{...(19.3)}$$

where D_1 = inside diameter of cutting edge/drive shoe

D_2 = outside diameter of cutting edge/drive shoe

D_3 = inside diameter of sampling tube

D_4 = outside diameter of sampling tube

The inside clearance is meant to reduce friction between the soil sample and the sampler when the soil enters the tube, by allowing for elastic expansion. If the inside clearance is too large, there will be too much of lateral expansion. Outside clearance will help reduce friction while the sampler is being driven and when it is being withdrawn after the sample is collected.

IS: 1892-1979 recommends that the inside clearance, C_i should be from 1 to 3 per cent. The outside clearance should not be much greater than the inside clearance. Its value usually lies between 0 and 2 per cent. Also, the area ratio A_r should be kept as low as possible, consistent with the strength requirements of the drive shoe and sampling tube, so as to reduce sample disturbance. Its value should not be greater than about 20 per cent for stiff formations, whereas for soft sensitive clays, an area ratio of 10 per cent or less is preferred.

Another parameter which is an index of sample disturbance is the recovery ratio, L_r

$$\text{Recovery ratio} \quad L_r = \frac{\text{recovered length of the sample}}{\text{penetration length of the sampler}} \qquad \text{...(19.4)}$$

$L_r = 1$ indicates a good recovery. $L_r < 1$ indicates that the soil is compressed while $L_r > 1$ means that the soil has swelled.

Further, to reduce wall friction, the sampling tube should have a smooth finish and should be properly oiled before use. The non-return valve should have a large orifice to allow the air and water to escape quickly and easily when driving the sampler.

Types of Samples Required in the Laboratory

While routine laboratory tests such as water content, density, specific gravity, grain size distribution and Atterberg limits are conducted on all types of soil, cohesive or non cohesive, tests for determining engineering properties such as consolidation parameters and shear strength parameters are usually performed on cohesive soil. The types of soil samples required for different laboratory tests are given in Table 19.1.

Table 19.1 Types of Soil Samples Required for Laboratory Tests

Type of test	*Type of sample required*
Natural water content	Undisturbed or SPT sample
Density	Undisturbed
Specific gravity	Representative or undisturbed
Grain size distribution	Representative or undisturbed
Atterberg limits	Representative or undisturbed
Coefficient of permeability	Undisturbed
Consolidation parameters	Undisturbed
Shear strength parameters	Undisturbed

19.5 SOIL SAMPLERS AND SAMPLING

The commonly used samplers can be classified into three categories, namely, (*a*) open drive samplers, (*b*) piston samplers and (*c*) rotary samplers.

Open Drive Sampler

This sampler essentially consists of a seamless open-end steel tube with a cutting edge. The tube is connected through a head to the drill rod. The sampler head is provided with vents (ports) to permit water and air to escape during sampling and also a ball check-valve to retain the sample during the withdrawal of sampler.

The sampling tube may be *thick-walled* or *thin-walled*. Thick-walled samplers are used for obtaining disturbed but representative soil samples. They may be in the form of a solid tube or a split tube with or without a liner. Figure 19.5 (IS: 9640-1980) shows a split spoon sampler assembly. This split-spoon sampler is used in the standard penetration test. The sample is collected by the thick-walled sampler by the repeated blows of a falling weight.

Thin wall samplers are used for obtaining undisturbed samples. The area ratio is usually below 15 per cent. Thin-walled tubes are cold-drawn seamless tubes made out of brass, aluminium or any other suitable material having adequate strength, durability and resistance to corrosion. IS: 2132-1972 has laid down requirements for thin-walled sampling tubes (Table 19.2). The sampling tube for sampling of soil is pushed into the soil in a continuous rapid motion without impact or twisting.

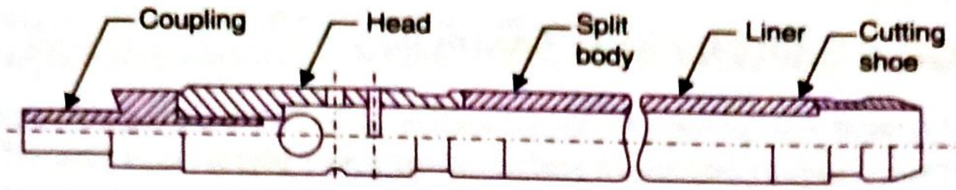
Table 19.2 Requirements of Sampling Tubes

Inside diameter, mm	38	70	100
Outside diameter, mm	40	74	106
Minimum effective length (that is, length available for soil sample), cm	30	45	45

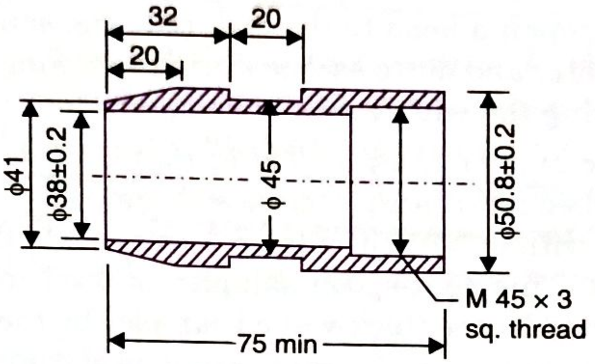
Note 1 The inside and outside diameters specified above give area ratios of 10.9, 11.8 and 12.4 per cent for the 38, 70 and 100 mm sampling tubes respectively.

Note 2 The three diameters recommended in Table 19.2 are indicated for purposes of reducing the number of sizes and fittings to be inventoried. Sampling tubes of intermediate or larger diameters may be used with the permission of the soil engineer-in-charge. Lengths of tubes shown are illustrative. Proper lengths may be determined to suit field conditions.

Open drive thin wall samplers are suitable for sampling all soil possessing some cohesion. They cannot be used in soils which are too hard or gravelly through which the sampler cannot penetrate and in soil which are too soft or too wet to be retained in the sampler.

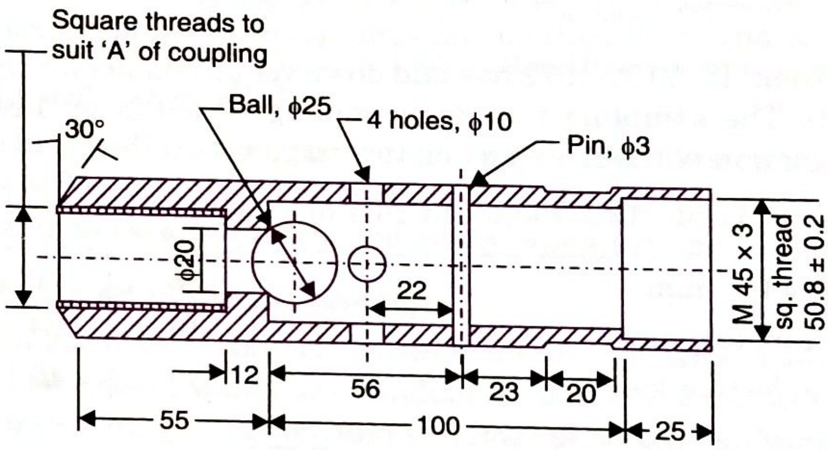


(a) Assembly of split spoon sampler



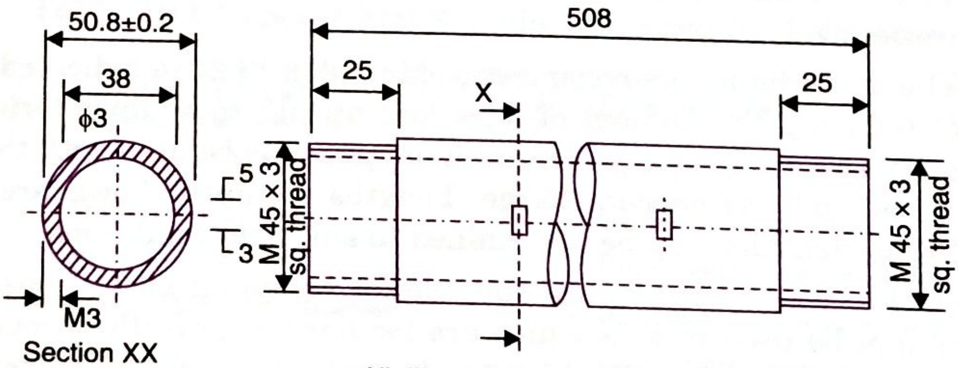
All dimensions in mm

(b) Cutting shoe



All dimensions in mm

(c) Head



All dimensions in mm

(d) Split body

Fig. 19.5 Split spoon sampler

Piston Sampler

A piston sampler consists of two separate parts: (*a*) sampler cylinder and (*b*) piston system. The piston which is actuated separately, fits tightly in the sampler cylinder.

During the driving and upto the start of the sampling, the bottom of the piston is maintained flush with the cutting edge of the sampler. At the proposed sampling depth, the piston is fixed in relation to the ground and the sampler cylinder is forced into the soil independently, cutting a sample out of the soil. As the sampler cylinder slides past the tight fitting piston during the sampling operation, a negative pressure develops above the sample which holds back the samples during withdrawal. After the sampler cylinder is pushed to the required depth, both the sampler cylinder and piston system are withdrawn with the sample inside the sampler cylinder.

Piston sampler is useful in sampling saturated sands and other soft and wet soil which cannot be sampled by open-drive samplers.

Rotary Sampler

A rotary sampler is a double-walled tube sampler with an inner removable liner. The outer tube or the rotating barrel is provided with a cutting bit. The bit cuts an annular ring when the barrel is rotated. The inner tube which is stationary, slides over the cylindrical sample cut by the outer rotating barrel. The sample is collected in the inner liner.

Rotary samplers are useful for sampling in firm to hard cohesive soil and particularly in rocks. The rock quality can be estimated from the core recovery ratio termed as *rock quality designation, RQD* (Peck, *et. al.*, 1974). The ratio of the total length of core recovered to the length of sampler advanced on a given run, expressed as per cent, is the value of *RQD*. While determining the length of core recovered, only those pieces of core that are atleast 100 mm long, hard and sound, are considered. Breaks caused by drilling should be ignored. The diameter of the core should preferably be not less than 54 mm. Deere (1963) has classified the rock quality on the basis of *RQD* (Table 19.3). From *RQD*, it is also possible to assess the *in situ* (field) modulus of elasticity (E_f) and compressive strength of the rock mass (q_f) based on the corresponding values (E_t and q_l) obtained by laboratory tests on rock cores.

Table 19.3 Relation between *RQD* and *in situ* Rock Quality

RQD	***Rock quality***	E_f/E_l **or** q_f/q_l
<25	very poor	0.15
25–50	poor	0.20
50–75	fair	0.25
75–90	good	0.30–0.70
90–100	excellent	0.70–1.0

Block or Chunk Samples

Block or chunk samples can be obtained from open excavations like test pits, shafts, etc. For chunk sampling, it is necessary that the soil has a trace of cohesion.

During excavation, a block of soil about 40 cm × 40 cm in plan, is left undisturbed. An undisturbed block of about 30 cm × 30 cm × 30 cm or any convenient size is usually trimmed with a flat knife or trowel. An open ended box is then slided over the trimmed block. The space between the side of the box and the sample is filled with dry sand. The end of the box is sealed with paraffin wax.

Chunk samples are not suitable, if these are to be transported to long distances because these are likely to be disturbed during transit.

Undisturbed samples may also be obtained by means of a sampling tube of 100 mm internal diameter, with a cutting edge. The soil surrounding the outside of the tube should be carefully removed while the tube is being pushed in.

19.6 NUMBER AND DISPOSITION OF TRIAL PITS AND BORINGS

The purpose of soil exploration, as discussed earlier, is to provide the designer with complete information about the subsoil layers at the site. Thus the disposition and the number of trial pits and borings should be such as to reveal any major changes in thickness, depth or properties of the strata at the site and immediate surroundings. The number and spacing of boreholes or trial pits would depend upon the extent of site, uniformity of strata, nature of the structure and the loading diagram. Many a time — particularly in a site of large area—the geological study of the site will be useful in deciding the number of boreholes or trial pits.

As a guideline, IS: 1892-1979 recommends that for a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in the centre and one at each corner will be adequate. For smaller and less important buildings, even one borehole or trial pit in the centre will be adequate. For larger areas, it may be useful to perform sounding tests/cone penetration tests at a spacing of 50 m to 100 m by dividing the area in a grid pattern. Thereafter, the number of boreholes or trial pits is decided by examining the variation in penetration resistance.

19.7 DEPTH OF EXPLORATION

The depth of exploration in any soil investigation should extend through any unsuitable foundation material and include all soil strata which are likely to undergo settlement due to the load of the structure. This depth, called the *'significant depth of exploration'* depends on the type of structure, size, shape and disposition of loaded areas, intensity of loading, soil profile and soil properties. As a general rule, unless bed rock is encountered, boring should be carried out to such a depth that the net increase in soil stress due to the load of the structure is less than 10 per cent of the average contact pressure due to the load of the structure for less than 5 per cent of the effective stress in the soil at that depth due to overburden (Task Committee Report, 1972). In the case of square loaded areas, the isobar of 10 per cent intensity of loading at foundation level extents to a depth of about twice the width of foundation below the base of foundation. It is, therefore useful to investigate the subsoil to a depth of atleast twice the width of the anticipated largest size foundation below the base of foundation.

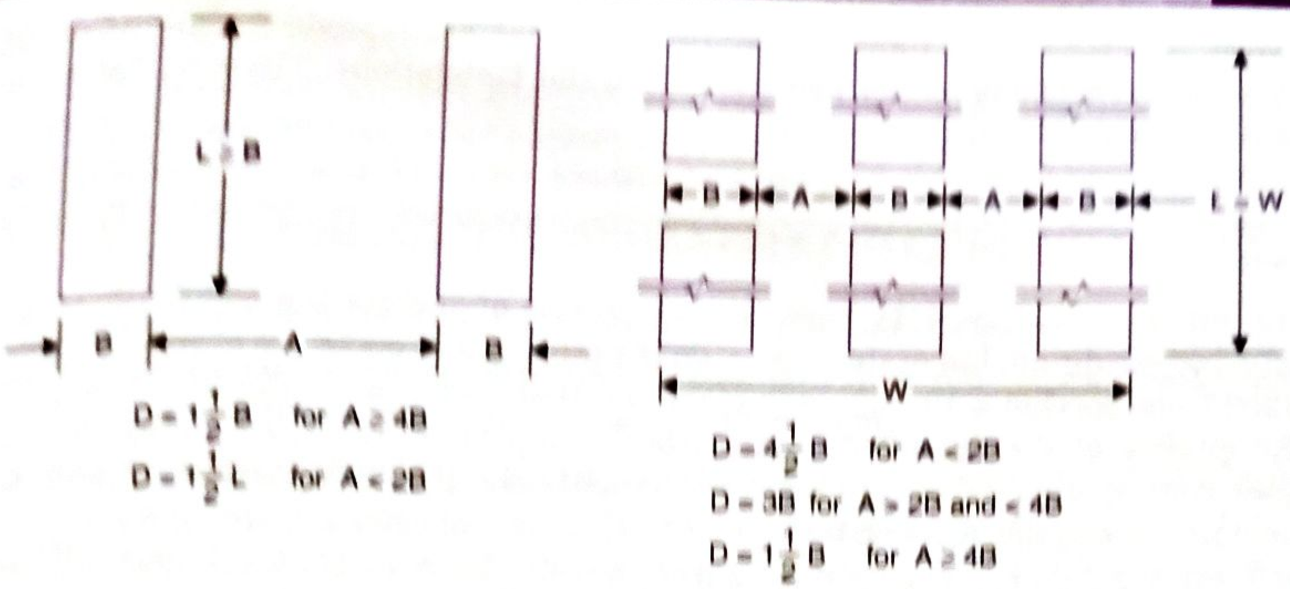


Fig. 19.6 Depth of exploration for building foundation

Table 19.4 shows the depth of exploration as recommended by IS: 1892-1979.

Table 19.4 Depth of Exploration

Sl. No.	*Type of foundation*	*Depth of exploration (D)*
(*i*)	Isolated spread footing or raft	One and a half times the width *B* (Fig. 19.6)
(*ii*)	Adjacent footings with clear spacing less than twice the width	One and a half times the length (*L*) of the footing (Fig. 19.6)
(*iii*)	Adjacent rows of footings	(Fig. 19.6)
(*iv*)	Pile and well foundation	To a depth of one and a half times the width of structure from the bearing level (toe of pile or bottom of well)
(*v*)	(*a*) Road cuts	Equal to the bottom with of the cut
	(*b*) Fill	Two metres below ground level or equal to the height of the fill whichever is greater

19.8 GROUND WATER OBSERVATIONS

The location and the probable fluctuations of ground water level are important for foundation work, deep excavations and water-logged areas. The position of ground water table can be estimated through observations of open wells at the site or in the vicinity. Boreholes can also be used for recording water levels but since water levels in boreholes may take time to reach an equilibrium state, observations should be made 12 to 24 hours after boring and compared with water levels in the wells existing in that area. In cased boreholes, at the end of drilling, the casing is pulled up about 30 cm and the water level recorded after 24 hours. The seasonal variation in ground water levels can be ascertained by piezometers which can be installed suitably and observations taken at intervals.

Many a time, the ground water and soil may contain some constituents in amounts sufficient to cause damage to the foundations. In such situations, chemical analysis of soil and water is necessary. In industrial areas, industrial waste products dumped at the site may also be responsible for corrosive action in the foundations. These wastes should be analysed chemically.

19.9 FIELD TESTS *VIS-A-VIS* LABORATORY TESTS

The *in situ* tests in the field have the advantage of testing the soil in their natural, undisturbed condition. Laboratory tests, on the other hand, make use of small size samples obtained from boreholes through samplers and therefore the reliability of these depends on the quality of the so called 'undisturbed' samples. Further, obtaining undisturbed samples from non cohesive, granular soil is not easy, if not impossible. Therefore, it is common practice to rely more on laboratory tests where cohesive soil are concerned. Further, in such soil, the field tests being short duration tests, fail to yield meaningful consolidation settlement data in any case. Where the subsoil strata are essentially non cohesive in character, the bias is most definitely towards field tests. The data from field tests is used in empirical, but time-tested correlations to predict settlement of foundations. The field tests commonly used in subsurface investigation are:

1. Penetration test
2. Pressuremeter test
3. Vane shear test
4. Plate load test
5. Geophysical methods

The plate load test has already been discussed in Section 15.6 and the vane shear test in Section 10.5. The vane shear test is perhaps the only field test of significance for soft, sensitive clay deposits. Under such soil conditions, undisturbed sampling is almost impossible and the vane shear test provides a satisfactory solution to the problem of estimating the undrained shear strength and sensitivity. For estimation of settlement, however, one has to use the laboratory consolidation tests.

The penetrometer tests, pressuremeter test and the geophysical methods are described in the following sections.

19.10 PENETROMETER TESTS

There are three penetrometer tests in common use. These are:

(*a*) Standard penetration test (*SPT*)

(*b*) Dynamic cone penetration test (*DCPT*)

(*c*) Static cone penetration test (*CPT*)

Of these, the standard penetration test is carried out in a borehole while the *DCPT* and *CPT* are carried out without a borehole. All the three tests measure the resistance of the soil strata to penetration by a penetrometer. Useful empirical correlations between penetration resistance and soil properties are available for use in foundation design.

Standard Penetration Test (*SPT*)

This is the most extensively used penetrometer test in India and many other countries like the USA. The test employs a split-spoon sampler (Fig. 19.5) which consists of a driving shoe, a split-barrel of circular cross-section which is longitudinally split into two parts and a coupling. IS: 2131-1981 gives the standard procedure for carrying out the test. It is briefly stated below:

(*a*) The borehole is advanced to the required depth and the bottom cleaned.

(*b*) The split-spoon sampler, attached to standard drill rods of required length is lowered into the borehole and rested at the bottom.

(*c*) The split-spoon sampler is driven into the soil for a distance of 450 mm by blows of a drop hammer (monkey) of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required to penetrate every 150 mm is recorded while driving the sampler. The number of blows required for the last 300 mm of penetration is added together and recorded as the *N value* at that particular depth of the borehole. The number of blows required to effect the first 150 mm of penetration, called the *seating drive,* is disregarded.

(*d*) The split-spoon sampler is then withdrawn and is detached from the drill rods. The split-barrel is disconnected from the cutting shoe and the coupling. The soil sample collected inside the split barrel is carefully collected so as to preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the *SPT*, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory.

The *SPT* is carried out at every 0.75 m vertical intervals in a borehole. This can be increased to 1.50 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the *N* value can be recorded for the first 300 mm penetration. The boring log shows *refusal* and the test is halted if,

(*i*) 50 blows are required for any 150 mm penetration

(*ii*) 100 blows are required for 300 penetration

(*iii*) 10 successive blows produce no advance

The standard penetration test is anything but standard, for a variety of reasons. Some of the pitfalls can be avoided if proper precautions are taken. Some of these precautions are as follows:

(*a*) The drill rods should be of standard specification and should not be in bent condition.

(*b*) The split spoon sampler must be in good condition and the cutting shoe must be free from wear and tear.

(*c*) The drop hammer must be of the right weight and the fall should be free, frictionless and vertical.

(*d*) The height of fall must be exactly 750 mm. Any change from this will seriously affect the *N* value.

(e) The bottom of the borehole must be properly cleaned before the test is carried out. If this is not done, the test gets carried out in the loose, disturbed soil and not in the undisturbed soil.

(f) When a casing is used in borehole, it should be ensured that the casing is driven just short of the level at which the *SPT* is to be carried out. Otherwise, the test gets carried out in a soil plug enclosed at the bottom of the casing.

(g) When the test is carried out in a sandy soil below the water table, it must be ensured that the water level in the borehole is always maintained slightly above the ground water level. If the water level in the borehole is lower than the ground water level, 'quick' condition may develop in the soil and very low *N* values may be recorded.

In spite of all these imperfections, *SPT* is still extensively used because the test is simple and relatively economical. It is the only test that provides representative soil samples both for visual inspection in the field and for natural moisture content and classification tests in the laboratory. Because of its wide usage, a number of time-tested correlations between *N* value and soil parameters are available, mainly for cohesionless soil. Even design charts for shallow foundations resting on cohesionless soil have been developed on the basis of *N* values. These have already been discussed in Chapter 15. The used of *N* values for cohesive soil is limited, since the compressibility of such soil is not reflected by *N* values.

SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts. IS: 2131-1981 recommends that the field value of *N* be corrected for two effects, namely, (*a*) effect of overburden pressure, and (*b*) effect of dilatancy.

(*a*) *Correction for overburden pressure*

Several investigators have found that the penetration resistance or the *N* value in a granular soil is influenced by the overburden pressure. If two granular soil possessing the same relative density but having different confining pressures are tested, the one with a higher confining pressure gives a higher *N* value. Since the confining pressure (which is directly proportional to the overburden pressure) increases with depth, the *N* values at shallow depths are underestimated and the *N* values at larger depths are overestimated. Hence, if no correction is applied to recorded *N* values, the relative densities at shallow depths will be underestimated and at higher dephts, they will be overestimated. To allow for this, *N* values recorded from field tests at different effective overburden pressures are corrected to a standard effective overburden pressure.

The corrected *N* value is given by

$$N' = C_N N \qquad \text{...(19.5)}$$

in which N' = corrected value of observed N

C_N = correction factor for overburden pressure

The correction proposed by Peck, Hanson and Thornburn (1974) is given by the equation

$$C_N = 0.77 \log_{10} \frac{20}{\bar{p}} \qquad \text{...(19.6 a)}$$

where $\bar{p}$ = effective overburden pressure at the depth at which N value is measured, in kg/cm². Figure 19.7 gives the variation of C_N with $\bar{p}$. For $\bar{p} < 0.25$ kg/cm², C_N should be obtained only from Fig. 19.7 and not from Eq. 19.6(a) which gives high values of C_N for such a condition. It can be seen from Eq. 19.6(a) or Fig. 19.7 that C_N will be equal to 1.0 for $\bar{p} = 1$ kg/cm². At greater depths, that is, when $\bar{p} > 1$ kg/cm², the observed field values of N need to be decreased, while at shallow depths ($\bar{p} < 1$ kg/cm²), the observed values need an upward correction, with a maximum limit of 2 for the C_N value.

If the overburden pressure is in the units of kN/m²,

$$C_N = 0.77 \log_{10} \frac{2000}{\bar{p}} \qquad \text{...(19.6 } b\text{)}$$

Another overburden correction that is commonly used is due to Bazaraa (1967). It is given by

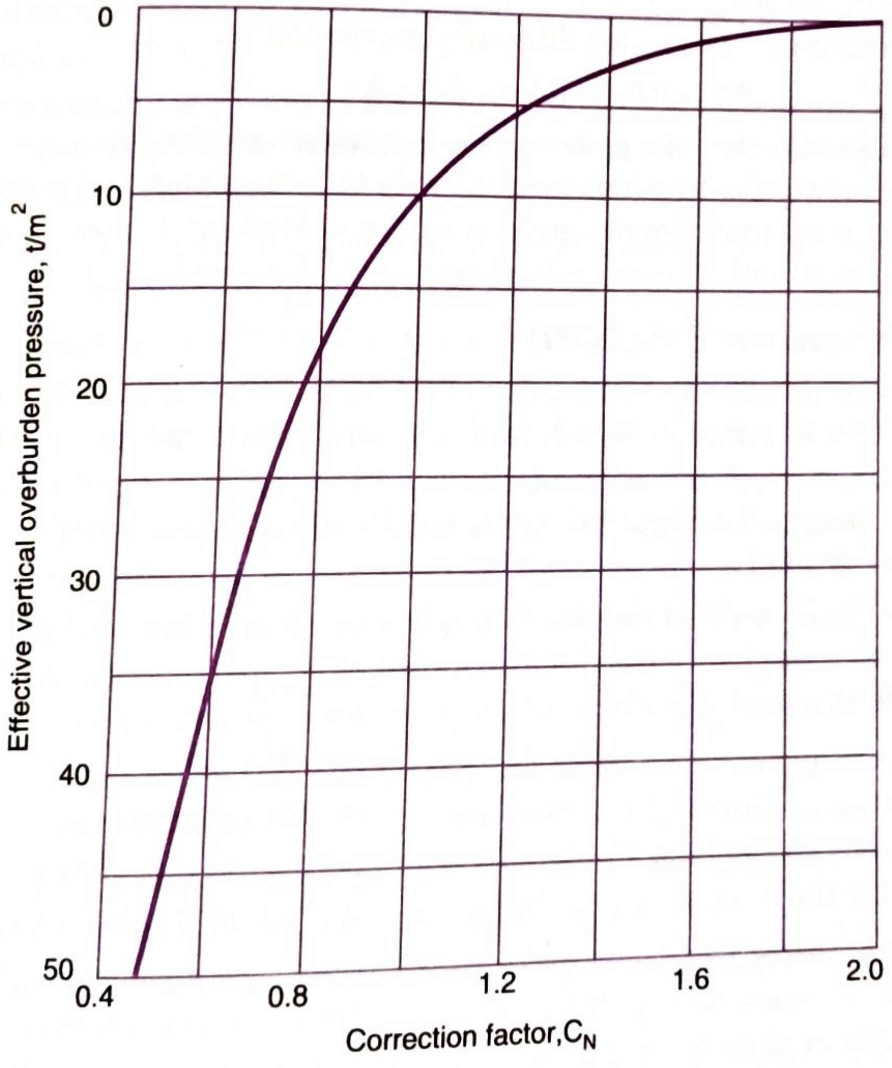


Fig. 19.7 Correction factor C_N due to overburden

$$C_N = \frac{4}{1 + 4\bar{p}} \quad \text{if } \bar{p} \le 0.75 \text{ kg/cm}^2 \qquad \text{...(19.7 } a\text{)}$$

and

$$C_N = \frac{4}{3.15 + 4\bar{p}} \quad \text{if } \bar{p} > 0.75 \text{ kg/cm}^2 \qquad \text{...(19.7 } b\text{)}$$

If $\bar{p}$ is in kN/m², the corresponding equations are :

$$C_N = \frac{4}{1 + 0.04\bar{p}} \quad \text{if } \bar{p} \le 75 \text{ kN/m}^2 \qquad ...(19.8\ a)$$

and

$$C_N = \frac{4}{3.25 + 0.01\bar{p}} \quad \text{if } \bar{p} > 75 \text{ kN/m}^2 \qquad ...(19.8\ b)$$

(*b*) *Correction for dilatancy*

Dilatancy correction is to be applied when N' obtained after overburden correction, exceeds 15 in *saturated fine sands and silts*. IS: 2131–1981 incorporates the Terzaghi and Peck recommended dilatancy correction (when $N' > 15$) using the equation

$$N'' = 15 + 0.5\,(N' - 15) \qquad ...(19.9\ a)$$

where N'' = final corrected value to be used in design charts.

If $N' \le 15, N'' = N'$

Bazaraa's recommendation for dilatancy correction is,

$$N'' = 0.6\,N' \quad \text{for} \quad N' > 15. \qquad ...(19.9\ b)$$

$N' > 15$ is an indication of a dense sand. In such a soil, the fast rate of application of shear through the blows of a drop hammer, is likely to induce negative pore water pressure in a saturated fine sand under undrained condition of loading. Consequently, a transient increase in shear resistance will occur, leading to a *SPT* value higher than the actual one.

Dynamic Cone Penetration Test (*DCPT*)

In this test, a cone which has an apex angle of 60° and attached to drill rods is driven into the soil by blows of a hammer of 65 kg, falling freely from a height of 750 mm. The blow count for every 100 mm penetration of the cone is continuously recorded. The cone is driven till refusal or upto the required depth and the drill rods are withdrawn, leaving the cone behind in the ground.

The number of blows required for 300 mm penetration is noted as the dynamic cone resistance, N_{cd}. The test gives a continuous record of N_{cd} with depth. No sample, however, can be obtained in this test.

Dynamic cone penetration tests are performed either by using a 50 mm diameter cone without bentonite slurry (IS: 4968—Part I—1976) or a 65 mm diameter cone with bentonite slurry (IS: 4968—Part II—1976). When bentonite slurry is used, the set-up has an arrangement for the circulation of slurry so that friction on the drill rod is eliminated.

The dynamic cone test is a quick test and helps to cover a large area under investigation rather economically. It helps in identifying the uniformity or the variability of the subsoil profile at the site and unearths local soft pockets, if any. It can also establish the position of rock stratum, when required. The test is much less expensive and much quicker than the *SPT*. If the tests are carried out close to a few boreholes, the data from *DCPT* can be compared with the *SPT* data and correlation between the two established for the particular site conditions. The correlation can then be used to obtain N values from N_{cd} values.

Some approximate correlations between N_{cd} and N, applicable for medium to fine sand are given below:

When a 50 mm diameter cone is used,

$N_{cd} = 1.5\ N$ for depths upto 3 m ...(19.10 *a*)

$N_{cd} = 1.75\ N$ for depths from 3 m to 6 m ...(19.10 *b*)

$N_{cd} = 2.0\ N$ for depths greater than 6 m ...(19.10 *c*)

The Central Building Research Institute, Roorkee, has developed the following correlations between N_{cbr} and N when a 65 mm dia. cone is used without bentonite slurry.

$N_{cbr} = 1.5\ N$ for depths upto 4 m ...(19.11 *a*)

$N_{cbr} = 1.75\ N$ for depths from 4 m to 9 m ...(19.11 *b*)

$N_{cbr} = 2.0\ N$ depths greater than 9 m ...(19.11 *c*)

Static Cone Penetration Test (*CPT*)

The static cone penetration test, simply called the cone penetration test (*CPT*), is a simple test that is now widely used in place of *SPT*, particularly for soft clays and silts and fine to medium sand deposits. The test was developed in Holland and is, therefore, also known as the Dutch cone test.

The test assembly is shown in Fig. 19.8. The penetrometer that is commonly used is a cone with an apex angle of 60° and a base area of 10 cm². The sequence of operations of the penetrometer is as follows:

1. *Position 1:* The cone and the friction jacket assembly is in a stationary position.
2. *Position 2:* The cone is pushed into the soil by the inner sounding rod to a depth a, at a steady rate of 20 mm/s, till a collar engages the cone. The tip resistance q_c called the *cone or point resistance*, can be calculated by the force Q_c read on a pressure gauge. $q_c = Q_c/A_c$ where A_c is the base area. Normally, $a = 40$ mm.
3. *Position 3:* The sounding rod is pushed further to a depth b. This has the effect of pushing the friction jacket and the cone assembly together. The total force Q_t required for this is again read on the pressure guage. The force required to push the friction jacket along, Q_f is then obtained as $Q_t - Q_c$. The side or the skin friction f_s is equal to $\dfrac{Q_f}{A_f}$ where A_f is the surface area of the friction jacket. The value of b is normally 40 mm.
4. *Position 4:* The outside mantle tube is pushed down to a distance $(a + b)$, bringing the cone and the friction jacket to position 1.

The procedure illustrated above is continued till the proposed depth of sounding is reached.

CPT gives a continuous record of variation of both cone resistance and friction resistance with depth. Unlike the *SPT* and the *DCPT*, this test measures the static resistance of the soil. *CPT*, however, does not yield any sample. The test is also unsuitable in gravels and very dense sands owing to the difficulty experienced in pushing the cone and the anchorage system.

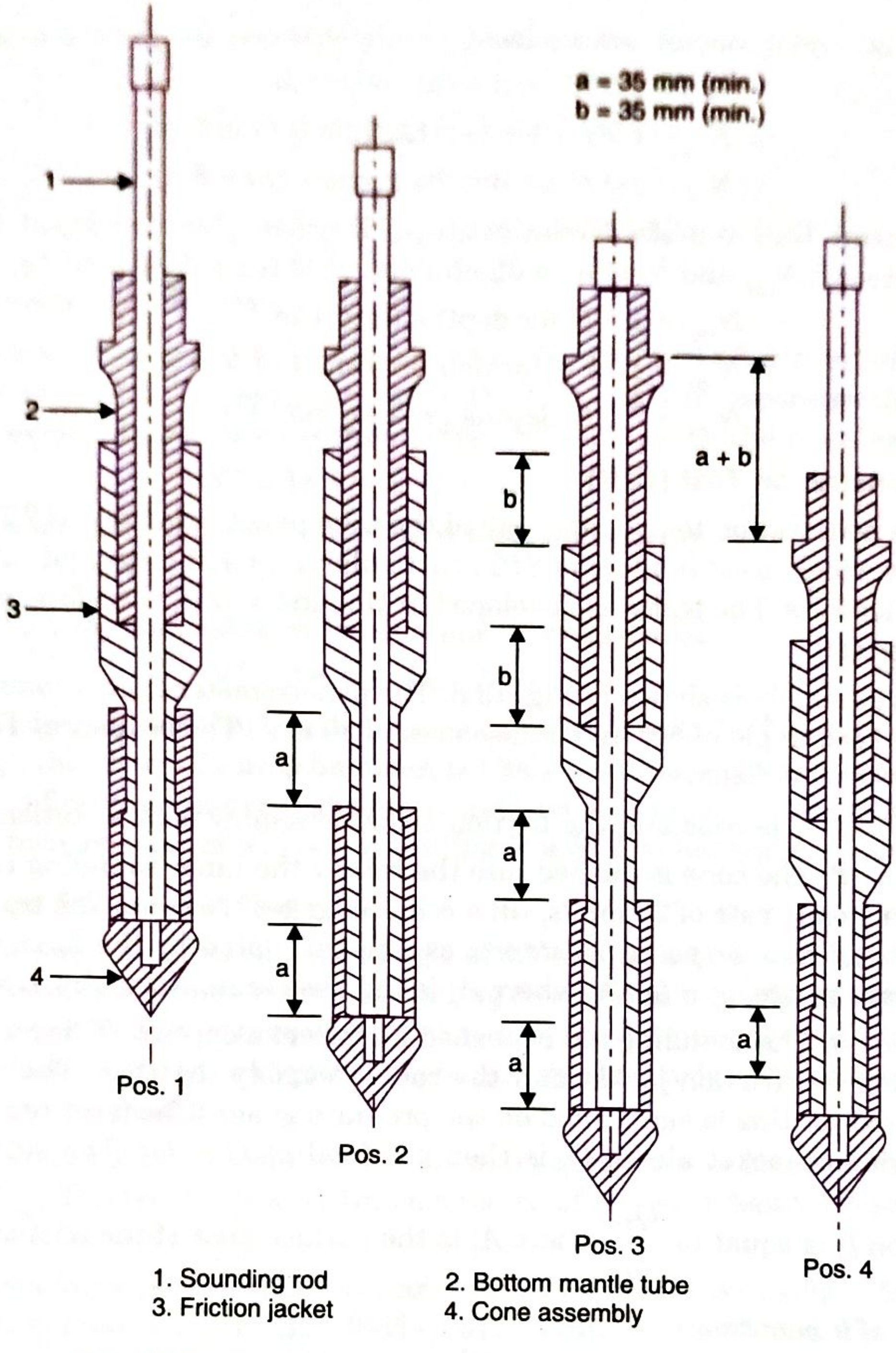


Fig. 19.8 Operation sequence of the sounding apparatus

Data from *CPT* is often used to estimate the point bearing resistance and skin friction resistance of a pile foundation. In granular soil, correlations have been established between q_c and N, Table 19.5 shows the correlations.

Table 19.5 Correlations between q_c and N

	Type of Soil	q_c/N (q_c *in kg/cm*^{2})
(*a*)	Sandy gravels and gravels	8 to 10
(*b*)	Coarse sand	5 to 10
(*c*)	Clean, fine to medium sands and slightly silty sand	3 to 4
(*d*)	Silts sandy silts, slightly cohesive silt-sand mixtures	2

Meyerhof's (1965) correlation between q_c and N for fine or silty medium-loose to medium-dense sands is expressed as:

$$q_c\ (\text{kg/cm}^2) = 4N \quad ...(19.12\ a)$$

or

$$q_c(MPa) = 0.4\ N \quad ...(19.12\ b)$$

Application of *CPT* data to design of foundations on granular soil is explained in Chapter 15.

The correlation between the q_c value and the undrained shear strength c_u of a clay may be stated as:

$$q_c = N_k c_u + \sigma_o \quad ...(19.13)$$

where N_k = cone factor

σ_o = total overburden pressure

Lunne and Kelvin (1981) gave the values of cone factor N_k for both normally consolidated and over-consolidated clay as shown in Table 19.6

Table 19.6 Values of Cone Factor, N_k

Type of clay	*Cone factor* N_k
Normally consolidated	11 to 19
Overconsolidated	
At shallow depths	15 to 20
At large depths	12 to 18

A median value of 20 can be used for N_k for all types of clay. Sanglerat (1972) also recommends the same value for all cases where the overburden correction is very small. In most of the cases, the overburden pressure in Eq. 19.13 is ignored.

19.11 PRESSUREMETER TEST (PMT)

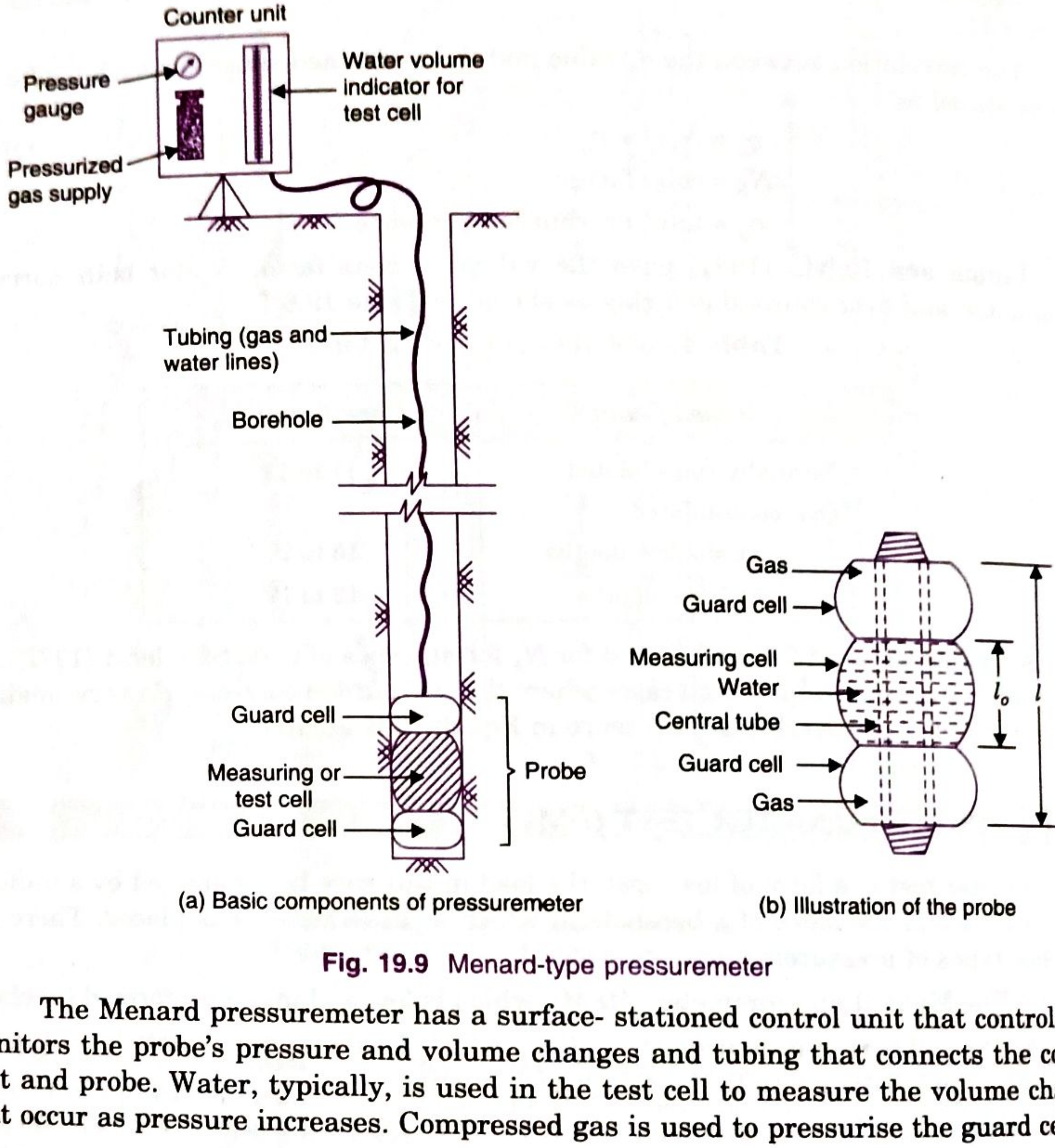
Pressuremeter test is a form of load test, the load in this case being applied by a uniform radial pressure to the sides of a borehole in which a pressuremeter is placed. There are two basic types of pressuremeters:

(*a*) The Menard pressuremeter (*MPM*), which is lowered into a preformed borehole.

(*b*) The self-boring pressuremeter (*SBP*), which forms its own borehole and, thus, causes much less disturbance to the soil prior to testing.

In both cases, the pressuremeter test consists of applying known stresses to the soil and measuring the resulting soil deformation. The interpretation of pressuremeter test data, therefore, does not rely on empirical correlations unlike the *SPT* and the *CPT*. The test results are used either directly or indirectly for the design of foundations.

Louis Menard of France is credited with the concept and development of the pressuremeter (*MPM*), which is named after him. It is a cylindrical apparatus whose volume can be increased by expanding in the lateral direction only. The Menard pressuremeter probe is an in-line three cell probe (Fig. 19.9 *a*). The middle part is the test or measuring cell; the two end cells are guard cells. During the test, each cell would be independently inflated to the same pressure. The guard cells protect the test cell from end effects that would develop in a single cell device and thus permit the test cell to expand only radially outward from the probe's longitudinal axis. The three independent cell chambers are stacked one above the other with inflatable rubber membranes held together at top and bottom by steel discs with a rigid hollow tube at the centre (Fig. 19.9 *b*).



(a) Basic components of pressuremeter (b) Illustration of the probe

Fig. 19.9 Menard-type pressuremeter

The Menard pressuremeter has a surface- stationed control unit that controls and monitors the probe's pressure and volume changes and tubing that connects the control unit and probe. Water, typically, is used in the test cell to measure the volume changes that occur as pressure increases. Compressed gas is used to pressurise the guard cells.

Test Procedure

The test is conducted in a predrilled borehole normally at intervals of 1 m. A borehole that is sufficiently (but not over about 10 per cent) oversized is used. Table 19.7 gives details of typical dimensions of probes and boreholes. With the probe in position, the control unit is used to admit water and gas to the test cell and guard cells respectively, to keep them at equal pressure.

Table 19.7 Typical Dimensions of Probe and Borehole

Hole dia. designation	Dia. of probe (mm)	l_0 (cm)	l (cm)	Borehole dia nominal	Borehole dia maximum
AX	44	36	66	46	52
BX	58	21	42	60	66
NX	70	25	50	72	84

The pressure of water in the measuring cell is increased in increments until the soil fails. Usually, failure is considered to have been reached when the total expanded volume of the test zone reaches twice the volume of the original cavity. Each increment of pressure is held for a fixed length of time, typically one minute, and the related volume change readings are noted. Normally, ten equal increments of pressure are applied to the soil in order to reach the limit pressure, p_l. A plot of pressure *versus* change in volume is made to obtain parameters necessary for foundation design. A typical test plot is shown in Fig. 19.10. This is a corrected pressuremeter curve after making the necessary corrections for pressure and volume losses in the system.

The abscissa of the plot indicates the volume of water injected and read at the control unit, v; the ordinate is the applied pressure at each increment, p. The following features of the curve are to be noted:

1. The initial part OA indicates the process of pushing the sides of the borehole back to their position before boring was made. The volume expansion of cavity is considered to begin only from A. If v_0 is the injected volume of water upto point A and V_c is the deflated volume of probe (cavity volume) before the start of the test, the volume of cavity at point A is equal to $V_c + v_0 \cdot p_0$ is the pressure corresponding to volume v_0.
2. The second part AB is a straight line which is considered as the *pseudoelastic stage* for the soil. Point B marks the end of this stage and the pressure corresponding to point B is p_f, which is known as the *creep pressure*. The corresponding volume is v_f.
3. The final part BC of the curve is the *plastic phase* of the soil deformation. The asymptotic value of pressure corresponding to point C is known as the *limit pressure*, p_l. This is conventionally taken as the pressure corresponding to an increase in volume (V_m) approximately equal to v_0. A common arbitrary value is V_m = 700 cm^3.

An equation for Menard's modulus (pressuremeter modulus), E_m is equal to 2.66 G_m where G_m is the Menard shear modulus for the pressuremeter curve between v_0 and v_f, i.e.,

$$G_m = \left(V_c + \frac{v_0 + v_f}{2} \frac{\Delta p}{\Delta v}\right)$$

or

$$E_m = 2.66\left(V_c + \frac{v_0 + v_f}{2}\right)\left(\frac{p_f - p_0}{v_f - v_0}\right) \quad \text{...(19.14)}$$

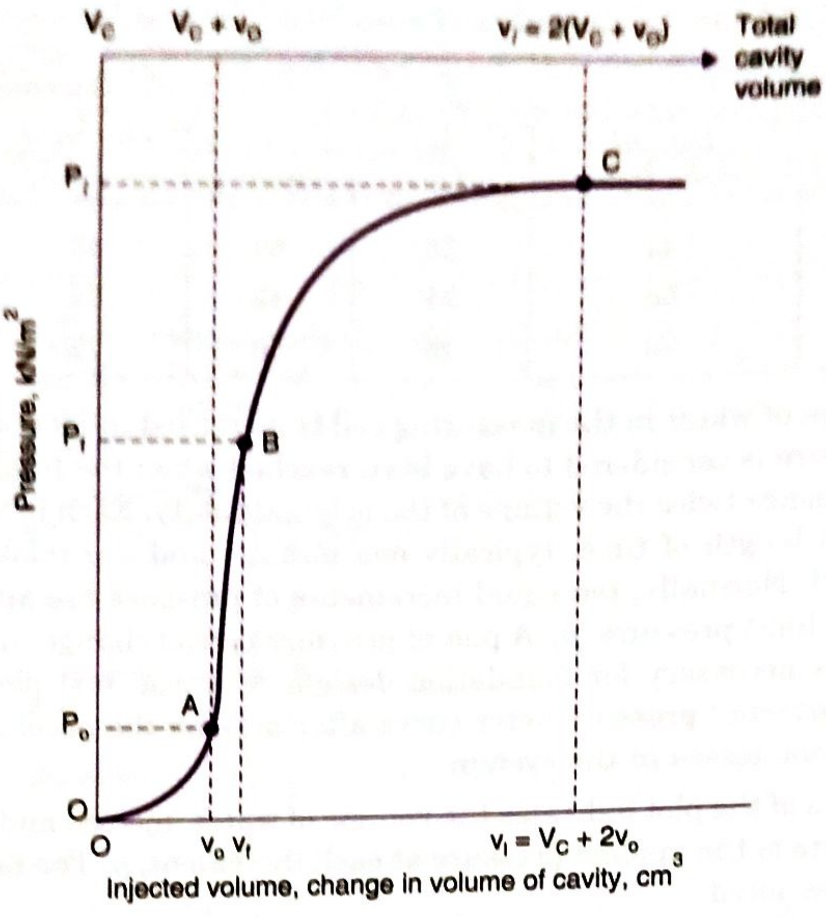


Fig. 19.10 Pressuremeter test results

Respective pressure and volume values are obtained from the pressuremeter curve. Units of E_m are kN/m².

E_m can be calculated for the soil at any depth below the ground surface.

Menard (1975) gave a relationship between Young's modulus E (or E_d) and E_m in the form:

$$E = \frac{E_m}{\alpha} \quad ...(19.15)$$

where α is called a *rheological factor*. Table 19.8 gives the value of α which depends on the soil type and the ratio $E_m/(p_l - \sigma_h)$, in which σ_h = at rest horizontal soil pressure at the planned foundation depth (depth of test).

Table 19.8 Rheological (Creep Deformation) Factor, α

Soil type	$E_m/(p_l - \sigma_h)$ α	α
Clay, normally consolidated	9 to 16	0.67
Clay, overconsolidated	>16	1.00
Silt, normally consolidated	8 to 14	0.50
Silt, overconsolidated	>14	0.67
Sand	7 to 12	0.33
Sand compact	>12	0.50

In a clay soil, the undrained shear strength c_u of the soil may be determined from the equation

$$c_u = \frac{p_l}{9} \qquad \text{...(19.16)}$$

The ultimate bearing capacity, q_{ult} for the soil zone supporting a shallow foundation has been related to the limit pressure by theoretical and empirical considerations, thus :

$$q_{ult} = \bar{\sigma}_v + k_{bc}\,(p_l - \sigma_h) \qquad \text{...(19.17)}$$

where q_{ult} = ultimate bearing capacity of the soil in kN/m^2

$\bar{\sigma}_v$ = effective vertical stress at the proposed level of the foundation in kN/m^2

σ_h = at rest horizontal soil pressure at the foundation depth or $K_0\,\bar{\sigma}_v$ in kN/m^2

k_{bc} = bearing capacity factor for the foundation shape and embedment and soil type (from Table 19.9).

Table 19.9 Values of k_{bc} for Pressure Bearing Capacity Equation

Values of k_{bc}. Strip Foundations									
Soil Type →	***Sand***			***Silt***			***Clay***		
↓ p_l (kN/m^2) D/B ratio →	***0***	***2***	***4***	***0***	***2***	***4***	***0***	***2***	***4***
500	0.8	1.5	1.8	0.8	1.40	1.60	0.8	1.3	1.5
1000	0.8	1.7	2.2	0.8	1.45	1.75	0.8	1.4	1.7
3000	0.8	2.0	2.6	0.8	1.65	2.00	0.8	1.5	1.8
6000	0.8	2.3	3.0						
Values of k_{bc}. Square and Circular Foundations									
500	0.8	2.2	3.0	0.8	2.10	2.45	0.8	1.9	
1000	9.8	3.0	4.0	0.8	2.25	2.75	0.8	2.2	2.5
3000	0.8	3.7	4.8	0.8	2.45	3.10	0.8	2.4	2.9
6000	0.8	3.9	5.4						

The settlement, ΔH of a shallow foundation located on a homogeneous soil can be determined from the equation

$$\Delta H = \frac{q_{des}}{9E_m}\left(2B_0\left(\lambda_d \frac{B}{B_0}\right)^{\alpha} + \alpha\,\lambda_c\,B\right)F_d \qquad \text{...(19.18)}$$

where q_{des} = foundation design bearing pressure *minus* the vertical pressure of the soil overburden adjacent to the foundation base

B_0 = a reference dimension, equal to 0.6 m

B = width or diameter of the foundation, provided that B is equal to or greater than B_0

α = rheological or creep deformation factor, which depends on the soil type and the ratio $E_m/(p_l - \sigma_h)$ (Table 19.8)

λ_c, λ_d = shape factors that are based on length-to-width ratio of the foundation (Table 19.10)

E_m = Menard pressuremeter modulus

F_d = depth factor, equal to 1 if the foundation depth is greater than B, equal to 1.20 for a foundation at the ground surface, equal to 1.10 for a foundation depth equal to one-half B.

Table 19.10 Shape Factors, λ

L/B ratio →	*Circular*	*Square*	*2*	*5*	*20*
λ_c	1	1.10	1.20	1.40	1.50
λ_d	1	1.12	1.53	2.14	2.65

Some of the problems associated with the Menard pressuremeter are:

1. Disturbance caused to the sides of the boreholes by the drilling process can vitiate the results.
2. Expansion of the soil due to the release in the *in situ* pressure in the borehole.
3. Diameter of the borehole being too large or too small compared to the uninflated condition of the probe.

The *self-boring pressuremeter* works on the same principle as the Menard pressuremeter but with one major difference. During insertion, the soil displaced by the equipment is slurried by a cutter and flushed to the surface with a drilling fluid (Fig. 19.11). The hole is thus cut by the pressuremeter itself, in such a way that no radial strains take place before the pressure membrane is expanded against the soil. In fact, the cell pressure required to enforce this zero radial strain condition can be used to calculate the *in situ* lateral total stress. The cell is expanded by water pressure and a direct measurement of the radial displacement is made at the midheight of the cell by measuring the strains in a piece of spring steel always in contact with the expanding membrane. A pore pressure transducer is mounted on the cell wall so that excess pore water pressure generated in the soil may be measured. Thus, total and effective stresses as well as the radial strains are all known.

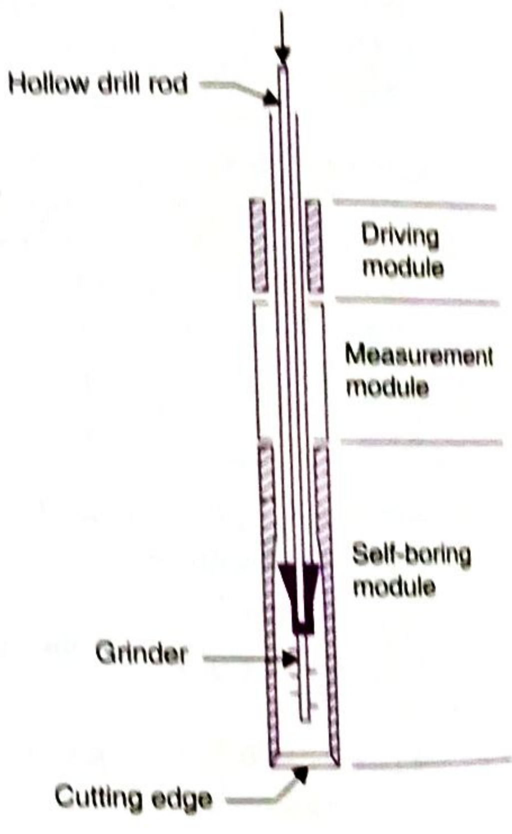


Fig. 19.11 Self-boring pressuremeter

The pressuremeter seems to have best applications in the same soil that are suitable for the *CPT*, namely, relatively fine-grained, sedimentary deposits.

19.12 GEOPHYSICAL METHODS

The two commonly used geophysical methods in civil engineering are the seismic refraction method and the electrical resistivity method. These are described in the following paragraphs:

Seismic Refraction Method

The method is based on the fact that seismic waves have different velocities in different types of soil or rock. Further, the waves are refracted when they cross the boundary between different types of soil. The method enables the determination of the general soil types and the approximate depth of boundaries of strata or the bedrock.

The method consists of inducing impact (by striking a plate on the soil with a hammer) or generating shock by exploding a small charge at or near the ground surface. The radiating shock waves are recorded by a device called geophone which records the time of travel of the waves. The geophones are installed at suitable known distances on the ground in a line from the source of shock or the same is moved away from the geophone to produce shock waves at given intervals. Some of the waves, termed *direct* or *primary waves* travel directly from the shock-source along the ground surface in the direction of the geophones. Other waves travel in a downward direction at various angles to the horizontal and will be refracted if they pass into a stratum of different seismic velocity. If the underlying layer is denser, the refracted waves travel much faster. As the distance between the shock source and the geophone increases, the refracted waves reach the geophone earlier than the direct waves [Fig. 19.12(*a*)]. The arrival time is plotted against the distance between the source and the geophone. Fig. 19.12(*b*), shows a typical test plot. If the source-geophone spacing is less than *d* [Fig. 19.12(*b*)], the direct wave reaches the geophone earlier than the refracted

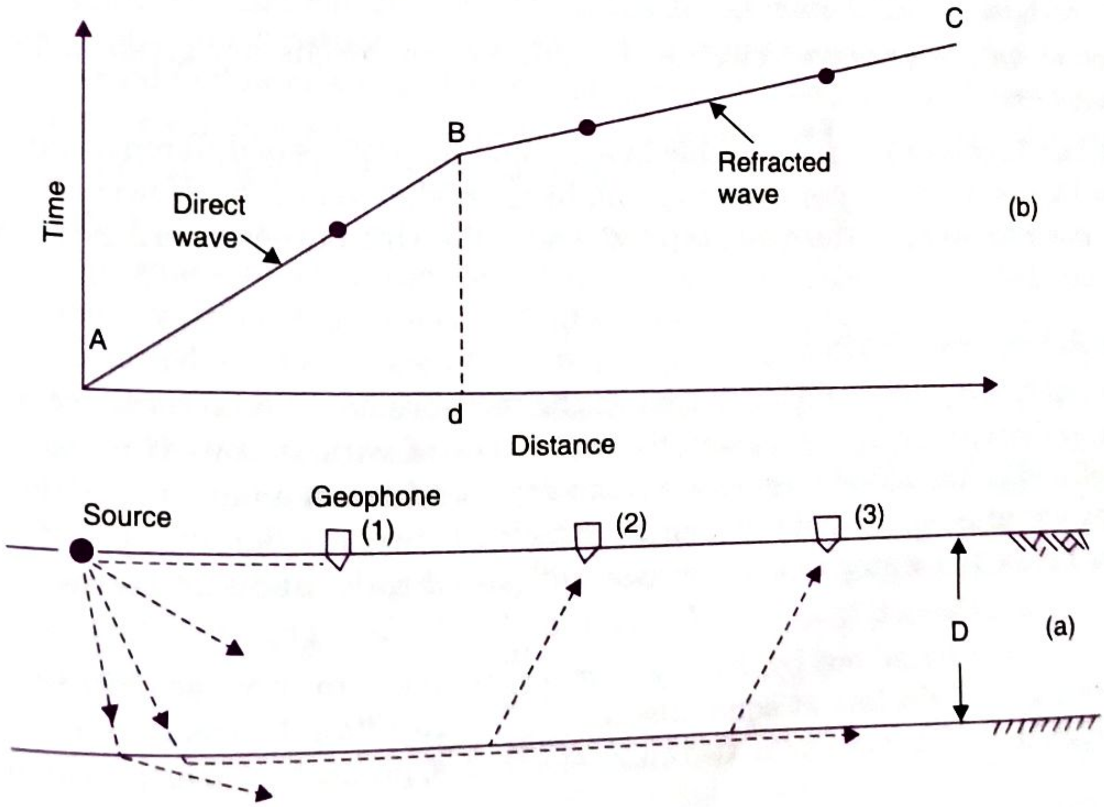


Fig. 19.12 Seismic refraction method

wave and the time-distance relationship is represented by a straight line *AB* through the origin. On the other hand, if the source-geophone spacing is greater than *d*, the refracted wave arrives earlier than the direct wave and the time-distance relationship is represented by a straight line *BC* at a different slope that of *AB*. The slopes of lines *AB* and *BC* are the seismic velocities v_1 and v_2 of the upper and lower stratum respectively. The general types of soil or rock can be determined from a knowledge of these velocities. The depth of the boundary between the two strata (assuming the thickness of the upper stratum as unvarying) can be estimated from the equation:

$$D = \frac{d}{2}\sqrt{\left(\frac{v_2 - v_1}{v_2 + v_1}\right)} \quad \text{...(19.19)}$$

Table 19.11 Wave Velocities in Different Materials

Material	*Wave velocity* (m/s)
Sand and top soil	180–365
Sandy clay	365–580
Gravel	490–790
Glacial till	550–2135
Rock talus	400–760
Water in loose materials	1400–1830
Shale	790–3350
Sandstone	915–2740
Granite	3050–6100
Limestone	1830–6100

Typical values of wave velocities in different materials are given in Table 19.11 (IS : 1892-1979).

The method is quick and reliable in establishing profiles of different strata provided the deeper layers have greater densities and hence higher velocities. However, the method cannot be used to identify the exact type of strata. For this purpose, borings and sampling are necessary.

Electrical Resistivity Method

The electrical resistivity method is based on the measurement and recording of changes in the mean resistivity or apparent specific resistance of various soil. The resistivity, ρ is usually defined as the resistance between the opposite faces of a unit cube of the material. Significant variations in resistivity can be detected between different types of soil strata; above and below the water table; between unfissured rocks and soil; between voids and soil/rock.

The test is carried out by driving four metal spikes to serve as electrodes into the ground along a straight line at equal distances [Fig. 19.13(*a*)]. Current (*I*) from a battery, flows through the soil between the two outer electrodes, producing an electrical field within the soil. The potential difference E between the two inner electrodes is then measured. The apparent resistivity, ρ is given by the equation

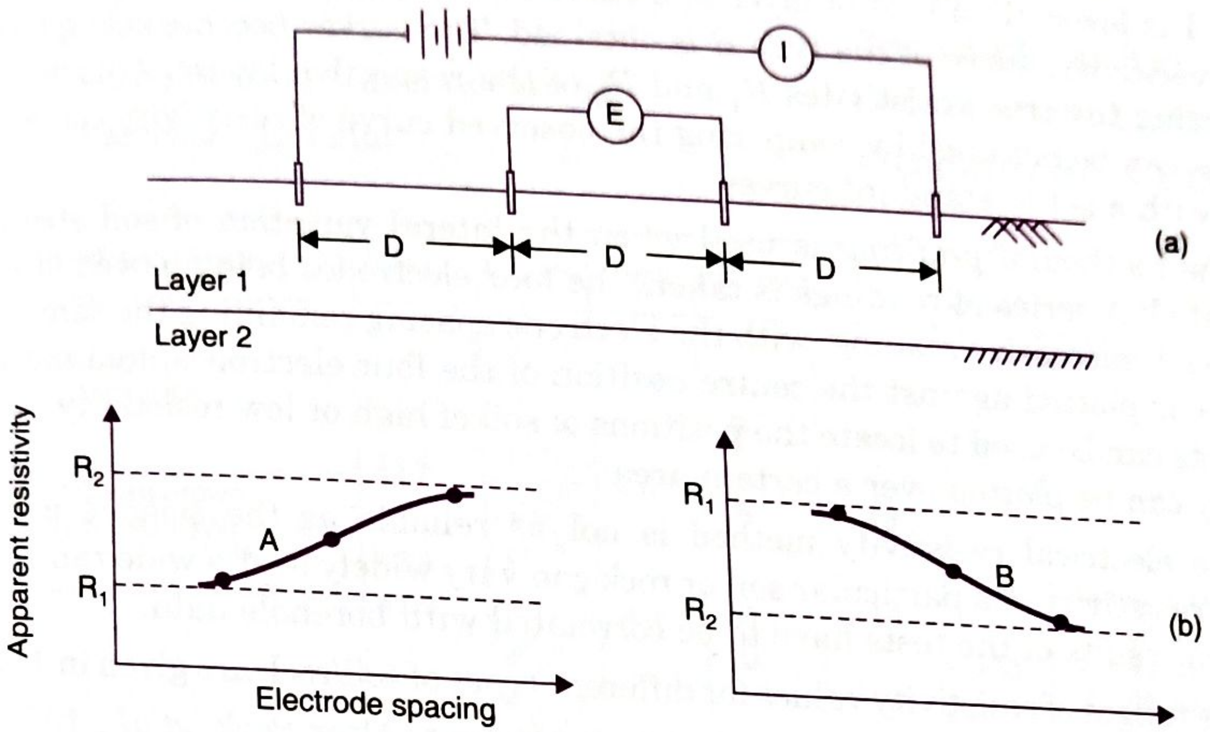


Fig. 19.13 Electrical resistivity method

$$\rho = \frac{2\pi DE}{I} \quad ...(19.20)$$

It is customary to express D in cm, E in volts, I in amperes and ρ in ohm-cm.

The apparent resistivity is the weighted average of true resistivity *upto a depth D* in a large volume of soil, the soil close to the surface being more heavily weighted than the soil at greater depths. If a stratum of low resistivity overlies a stratum of higher resistivity, the current is forced to flow closer to the ground surface, resulting in a higher voltage drop and hence a higher value of apparent resistivity. It would be the opposite if a stratum of high resistivity lies above a stratum of low resistivity.

The method of *sounding* is used when the variation of resistivity with depth is required. This provides rough estimates of the types and depths of strata. A series of readings is taken, with each successive reading corresponding to an increased spacing (equal) of the electrodes. The centre of the four electrodes, however, remains at a fixed point. When the spacing is increased, the apparent resistivity is influenced by a greater depth of soil. If the resistivity increases with increasing electrode spacing, the inference is that the underlying stratum with higher resistivity has influenced the value. On the other hand, if increased spacing produces decreasing resistivity, it can be concluded that the lower resistivity of the underlying stratum is beginning to influence the readings. The thickness of a stratum is larger if its influence is observed over a greater spacing of the electrodes and *vice versa*.

Apparent resistivity is plotted against electrode spacing, usually on a log-log graph. Figure 19.13(*b*) illustrates characteristic curves for a two layer structure. If the resistivity of layer 1 is lower than that of layer 2, a curve of the type *A* is obtained; if layer 1 has a higher resistivity, curve of the type *B* is obtained. The curves become asymptotic to lines representing the true resistivites R_1 and R_2 of the respective layers. Approximate layer thickness can be obtained by comparing the observed curve of resistivity *versus* electrode spacing with a set of standard curves.

The method of *profiling* is used when the lateral variation of soil strata is to be investigated. A series of readings is taken, the four electrodes being moved laterally as a unit for each successive reading, with the electrode spacing remaining the same. Apparent resistivity is plotted against the centre position of the four electrodes, to a natural scale. These plots can be used to locate the positions of soil of high or low resistivity. Contours of resistivity can be plotted over a certain area.

The electrical resistivity method is not as reliable as the seismic method. The apparent resistivity of a particular soil or rock can vary widely over a wide range of values. Hence, the results of the tests have to be correlated with borehole data.

The ranges of resistivity values for different types of soil/rock are given in Table 19.12.

Table 19.12 Representative Resistivity Values (After Peck, *et.al.*, 1974)

Material	*Resistivity (ohm–cm × 10^3)*
Clay and saturated silt	0–10
Sandy clay and wet silty sand	10–25
Clayey sand and saturated sand	25–50
Sand	50–150
Gravel	150–500
Weathered rock	100–200
Sound rock	150–4000

19.13 BOREHOLE LOGS

After the soil investigation has been completed and the results of laboratory tests become available, the ground conditions discovered in each borehole are summarised in the form of a borehole log. An example of such a log is illustrated in Fig. 19.14, but details can vary. The method of investigation and details of the equipment used should be stated on each log. The location, ground level and diameter of the hole should be specified. The names of the client and the project should be mentioned. Following this information, other data are presented, usually in a tabular form showing (*i*) the soil profile with elevations of different strata (*ii*) ground water level, (*iii*) termination level of the borehole, (*iv*) the depths or ranges of depth at which samples were taken or at which *in situ* tests were performed, (*v*) the type of soil samples, (*vi*) the results of important laboratory tests, and (*vii*) *N* values at the measured elevations.

Borehole Log

Location :	BH No. 1
Project :	Ground RL : 46.3 m
Boring method : Shell and auger	Date of Start : 1.1.1998
Diameter : 150 mm	Completed on : 4.1.1998

Description of start	R.L.	Legend	Depth	Samples	N	q_u kN/m^2	Remarks
Loose, light brown SAND (SP)				R	6		
Medium dense brown gravelly SAND (SW) ▽ W.T.	43.7		2.6	R	18		
	42.5						
Firm to stiff, yellowish-brown, CLAY of high palsticity (CH)	41.9		4.4	R	20		Water level : 1.1.98 42.8 m 2.1.98 42.5 m 3.1.98 42.5 m
				U		160	
				U		180	
				U		200	
				U		210	
Very dense, red, silty SAND (SM)	34.0		12.3	R	50		
				R	62		
					50 for 150 mm		Refusal
	30.0		16.3				Termination level of borehole 30.0 m

U : Undisturbed R : Representative sample

Fig. 19.14 Borehole log

The data from a number of boreholes at a site can be used to obtain a subsoil profile. A subsoil profile is a vertical section of the subsoil strata along a selected line of boreholes. One such subsoil profile along boreholes *BH*-1 to *BH*-4, covering a distance of 300 m, is shown in Fig. 19.15. It shows the boundaries of different strata along with their classification. It helps identify the subsoil profile at any intermediate locations which may not have been covered by boreholes. Boreholes are often advanced at the nodal points of a grid worked out for a site in which the locations of important structures are yet to be finalised. In such a situation, several subsurface profiles can be obtained along different lines of boreholes.

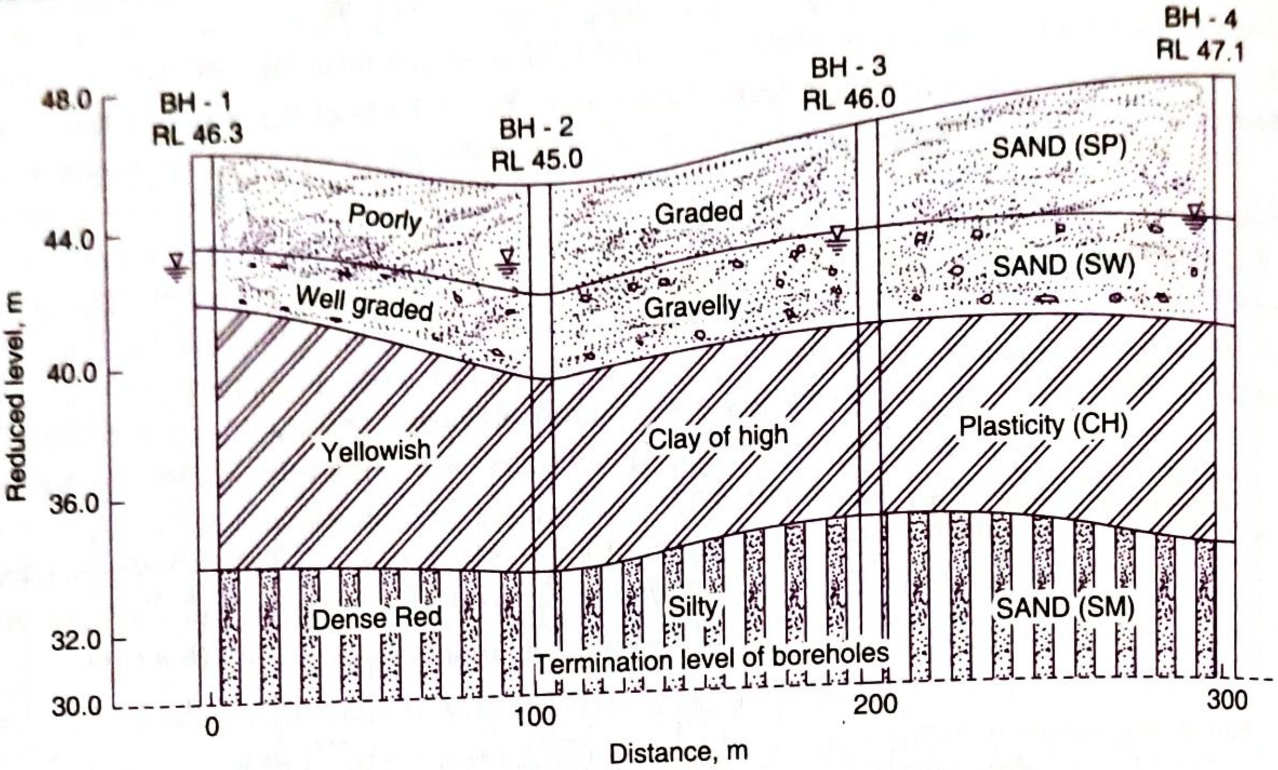


Fig. 19.15 Subsoil profile along *BH*-1 to *BH*-4

19.14 SITE INVESTIGATION REPORT

A site investigation report is usually the culmination of the investigation, exploration and testing programme, although interim reports may sometimes be required where long-term or ongoing observations are involved. The report is addressed to the client. It may be purely factual or may contain, if requested, advice and recommendations relating to design and construction and may also contain suggestions regarding post-construction monitoring.

Although individual reports vary in form, content and presentation, a typical report will normally include the following:

1. Introduction
2. Borehole logs
3. Field and laboratory test results
4. Analysis of data
5. Recommendations
6. References

In the introduction, the following points are included: (*i*) the nature of the project and the scope of the investigation, (*ii*) the location of the site and its description, including the geology of the site where necessary; (*iii*) the different tests carried out in the field and the laboratory, (*iv*) the site plan showing the locations of boreholes and other field tests like the *CPT*, *DCPT*, *MPT*, *PLT*, etc., and the locations of important structures, and (*v*) the manner of presentation of the report.

The borehole logs of different boreholes must contain all relevant information as discussed in Section 19.13.

Field and laboratory test results must be reported in the form of tables and test plots. Important test procedures must be included. References to Indian Standard Codes of Practice, where adopted, must be cited.

Analysis of test data must include (*i*) reference to borehole logs and subsoil profiles to determine soil stratification at important locations; (*ii*) R.Ls of natural water table and hard pan/refusal, if encountered within termination levels of boreholes, (*iii*) range of design soil parameters and their average values for the site or independent design parameters for carefully demarcated zones or locations, where necessary, and (*iv*) computations necessary for arriving at recommendations related to the type of foundation, the size and depth of foundation, and the allowable bearing pressure; and any other design decisions.

Recommendations must be presented clearly, concisely and pointwise to include all the design data obtained from the field and laboratory tests and the design decisions arrived at after analysing the data. The limitation of the investigation must be briefly stated. Any advice related to the construction of the structure and post-construction monitoring must also be included.

References and relevant literature extracts shall be included at the conclusion of the report.

EXAMPLES

Example 19.1: *Determine the area ratios for the following soil samplers and comment on the nature of samples obtained in each of the samplers.*

(a) Core cutter	*165 mm OD*	*150 mm 1D*
(b) Split barrel	*51 mm OD*	*35 mm 1D*
(c) Seamless tube (Shelby)	*51 mm OD*	*48 mm 1D*

Solution:

Area ratio,
$$A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100;\ D_2 = D_{outer};\ D_1 = D_{inner}$$

(*a*) *Core cutter*

$$A_r = \frac{165^2 - 150^2}{150^2} \times 100 = 21\ \%,$$

which is slightly more than 20%. Reasonably good undisturbed samples can be obtained.

(*b*) *Split barrel*

$$A_r = \frac{51^2 - 35^2}{35^2} \times 100 = 112.33\% >> 20\%$$

The samples will be disturbed in nature.

(c) Seamless tube

$$A_r = \frac{51^2 - 48^2}{48^2} \times 100 = 12.9\% < 20\%$$

Samples will be undisturbed, but in soft clays, area ratio must be preferably less than 10% for really undisturbed samples.

■ **Example 19.2:** *The field N value in a deposit of fully submerged fine sand was 40 at a depth of 6 m. The average saturated unit weight of the soil is 19 kN/m³. Calculate the corrected N value as per IS: 2131-1981.*

Solution: Since the soil is submerged fine sand, dilatancy correction is also to be applied in addition to the correction for overburden pressure.

$$\gamma' = \gamma_{sat} - \gamma_w = 19 - 9.8 = 9.2 \text{ kN/m}^2$$

Effective overburden pressure $\bar{p}$ at 6 m depth $= 9.2 \times 6 = 55.2$ kN/m²

(*a*) Correction for overburden pressure:

According to IS: 2131-1981 (Peck et. al. 1974),

$$C_N = 0.77 \log \frac{2000}{\bar{p}} = 0.77 \log \frac{2000}{55.2} = 1.2$$

$$N' = C_N N = 1.2 \times 40 = 48$$

(*b*) Correction for dilatancy effect:

$$N'' = 15 + 0.5\,(N' - 15)$$

$$= 15 + 0.5\,(48 - 15) = 31.5, \text{ say } 31.$$

If Bazaraa's overburden pressure correction factor C_N is used,

$$C_N = \frac{4}{1 + 0.04\bar{p}} = \frac{4}{1 + 0.04 \times 55.2} = 1.24, \text{ compared to } C_N = 1.2 \text{ of Peck et. al.}$$

■ **Example 19.3:** *For the corrected N value of 31 worked out in Ex. 19.2, estimate (a) the relative density and ϕ value using Table 15.7, and (b) the N value and bearing capacity factors N_q and N_γ using Peck et. al. (1974) plot of Fig. 15.10.*

Solution:

(*a*) For $N = 31$, from Table 15.7,

Relative density may be taken as 65% and ϕ value as 35°.

(*b*) For $N = 31$, from Fig. 15.10,

$\phi = 36.5°$ and $N_q = 38$; $N_\gamma = 47$.

■ **Example 19.4:** *The cone penetration resistance obtained in a clay soil in a CPT was 50 kg/cm². Determine the undrained strength of the clay. The total overburden pressure at the depth was 100 kN/m².*

Solution:

From Eq. 19.13, $$q_c = N_k c_u + \sigma_0$$

or $$c_u = \frac{q_c - \sigma_0}{N_k}$$

(*c*) *Seamless tube*

$$A_r = \frac{51^2 - 48^2}{48^2} \times 100 = 12.9\% << 20\%.$$

Samples will be undisturbed; but in soft clays, Area ratio must be preferably less than 10% for really undisturbed samples.

■ **Example 19.2:** *The field N value in a deposit of fully submerged fine sand was 40 at a depth of 6 m. The average saturated unit weight of the soil is 19 kN/m³. Calculate the corrected N value as per IS: 2131-1981.*

Solution: Since the soil is submerged fine sand, dilatancy correction is also to be applied in addition to the correction for overburden pressure.

$$\gamma' = \gamma_{sat} - \gamma_w = 19 - 9.8 = 9.2 \text{ kN/m}^2$$

Effective overburden pressure $\bar{p}$ at 6 m depth = 9.2 × 6 = 55.2 kN/m²

(*a*) Correction for overburden pressure:

According to IS: 2131-1981 (Peck *et. al.* 1974),

$$C_N = 0.77 \log \frac{2000}{\bar{p}} = 0.77 \log \frac{2000}{55.2} = 1.2$$

$$N' = C_N N = 1.2 \times 40 = 48$$

(*b*) Correction for dilatancy effect:

$$N'' = 15 + 0.5\,(N' - 15)$$

$$= 15 + 0.5\,(48 - 15) = 31.5, \text{ say } 31.$$

If Bazaraa's overburden pressure correction factor C_N is used,

$$C_N = \frac{4}{1 + 0.04\bar{p}} = \frac{4}{1 + 0.04 \times 55.2} = 1.24, \text{ compared to } C_N = 1.2 \text{ of Peck } et.\ al.$$

■ **Example 19.3:** *For the corrected N value of 31 worked out in Ex. 19.2, estimate (a) the relative density and ϕ value using Table 15.7, and (b) the N value and bearing capacity factors N_q and N_γ using Peck et. al. (1974) plot of Fig. 15.10.*

Solution:

(*a*) For $N = 31$, from Table 15.7,

Relative density may be taken as 65% and ϕ value as 35°.

(*b*) For $N = 31$, from Fig. 15.10,

$\phi \approx 36.5°$ and $N_q = 38$; $N_\gamma = 47$.

■ **Example 19.4:** *The cone penetration resistance obtained in a clay soil in a CPT was 50 kg/cm². Determine the undrained strength of the clay. The total overburden pressure at the depth was 100 kN/m².*

Solution:

From Eq. 19.13, $$q_c = N_k c_u + \sigma_0$$

or $$c_u = \frac{q_c - \sigma_0}{N_k}$$

Taking $N_s = 20,$

$$c_u = \frac{50 \times 100 - 100}{20} = 245 \text{ kN/m}^2$$

If the effect of overburden pressure is disregarded,

$$c_u = \frac{5000}{20} = 250 \text{ kN/m}^2$$

One can see that the influence of overburden pressure is not significant.

Example 19.5: *For a N value of 12 obtained in a clay soil, determine the consistency of the clay deposit and the approximate value of unconfined compressive strength using Table 15.8.*

Solution:

From Table 15.8, for $N = 12$,

Consistency of the clay deposit is *stiff* and the unconfined compressive strength, q_u is about 150 kN/m^2.

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