CHAPTER 9

SOIL EXPLORATION

9.1 INTRODUCTION

The stability of the foundation of a building, a bridge, an embankment or any other structure built on soil depends on the strength and compressibility characteristics of the subsoil. The field and laboratory investigations required to obtain the essential information on the subsoil is called *Soil Exploration or Soil Investigation.* Soil exploration happens to be one of the most important parts of Foundation Engineering and at the same time the most neglected part of it. Terzaghi in 1951 (Bjerrum, et al., 1960) had rightly remarked, that *"Building foundations have always been treated as step children".* His remarks are relevant even today. The success or failure of a foundation depends essentially on the reliability of the various soil parameters obtained from the field investigation and laboratory testing, and used as an input into the design of foundations. *Sophisticated theories alone will not give a safe and sound design.*

Soil exploration is a *must* in the present age for the design of foundations of any project. The extent of the exploration depends upon the magnitude and importance of the project. Projects such as buildings, power plants, fertilizer plants, bridges etc., are localized in areal extent. The area occupied by such projects may vary from a few square meters to many square kilometers. Transmission lines, railway lines, roads and other such projects extend along a narrow path. The length of such projects may be several kilometers. Each project has to be treated as per its requirements. The principle of soil exploration remains the same for all the projects but the program and methodology may vary from project to project.

The elements of soil exploration depend mostly on the importance and magnitude of the project, but generally should provide the following:

1. Information to determine the type of foundation required such as a shallow or deep foundation.
2. Necessary information with regards to the strength and compressibility characteristics of the subsoil to allow the Design Consultant to make recommendations on the safe bearing pressure or pile load capacity.

Soil exploration involves broadly the following:

1. Planning of a program for soil exploration.
2. Collection of disturbed and undisturbed soil or rock samples from the holes drilled in the field. The number and depths of holes depend upon the project.
3. Conducting all the necessary *in-situ* tests for obtaining the strength and compressibility characteristics of the soil or rock directly or indirectly.
4. Study of ground-water conditions and collection of water samples for chemical analysis.
5. Geophysical exploration, if required.
6. Conducting all the necessary tests on the samples of soil /rock and water collected.
7. Preparation of drawings, charts, etc.
8. Analysis of the data collected.
9. Preparation of report.

9.2 BORING OF HOLES

Auger Method

Hand Operated Augers

Auger boring is the simplest of the methods. Hand operated or power driven augers may be used. Two types of hand operated augers are in use as shown in Fig. 9.1

The depths of the holes are normally limited to a maximum of 10 m by this method. These augers are generally suitable for all types of soil above the water table but suitable only in clayey soil below the water table (except for the limitations given below). A string of drill rods is used for advancing the boring. The diameters of the holes normally vary from 10 to 20 cm. Hand operated augers are not suitable in very stiff to hard clay nor in granular soils below the water table. Hand augering is not practicable in dense sand nor in sand mixed with gravel even if the strata lies above the water table.

Power Driven Augers

In many countries the use of power driven continuous flight augers is the most popular method of soil exploration for boring holes. The flights act as a screw conveyor to bring the soil to the surface.

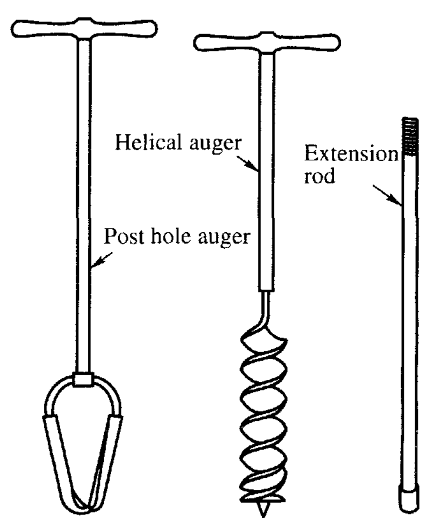


Figure 9.1 Hand augers

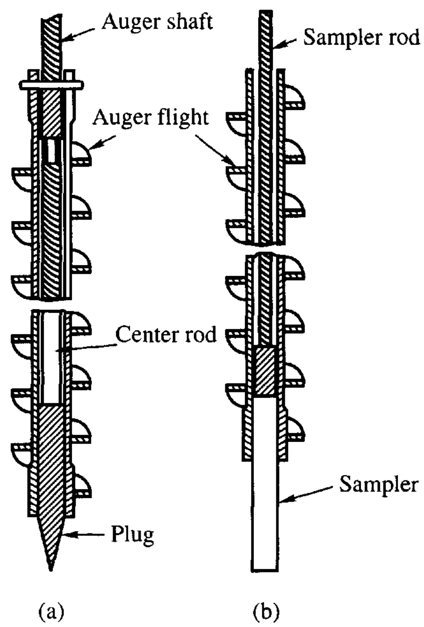


Figure 9.2 Hollow-stem auger
(a) Plugged while advancing the auger, and (b) plug removed and sampler inserted to sample soil below auger

This method may be used in all types of soil including sandy soils below the water table but is not suitable if the soil is mixed with gravel, cobbles etc. The central stem of the auger flight may be hollow or solid. A hollow stem is sometimes preferred since standard penetration tests or sampling may be done through the stem without lifting the auger from its position in the hole. Besides, the flight of augers serves the purpose of casing the hole. The hollow stem can be plugged while advancing the bore and the plug can be removed while taking samples or conducting standard penetration tests (to be described) as shown in Fig. 9.2. The drilling rig can be mounted on a truck or a tractor. Holes may be drilled by this method rapidly to depths of 60 m or more.

Wash Boring

Wash boring is commonly used for boring holes. Soil exploration below the ground water table is usually very difficult to perform by means of pits or auger-holes. Wash boring in such cases is a very convenient method provided the soil is either sand, silt or clay. The method is not suitable if the soil is mixed with gravel or boulders.

Figure 9.3 shows the assembly for a wash boring. To start with, the hole is advanced a short depth by auger and then a casing pipe is pushed to prevent the sides from caving in. The hole is then continued by the use of a chopping bit fixed at the end of a string of hollow drill rods. A stream of water under pressure is forced through the rod and the bit into the hole, which loosens the soil as the water flows up around the pipe. The loosened soil in suspension in water is discharged into a tub. The soil in suspension settles down in the tub and the clean water flows into a sump which is reused for circulation. The motive power for a wash boring is either mechanical or man power. The bit which is hollow is screwed to a string of hollow drill rods supported on a tripod by a rope or steel cable passing over a pulley and operated by a winch fixed on one of the legs of the tripod.

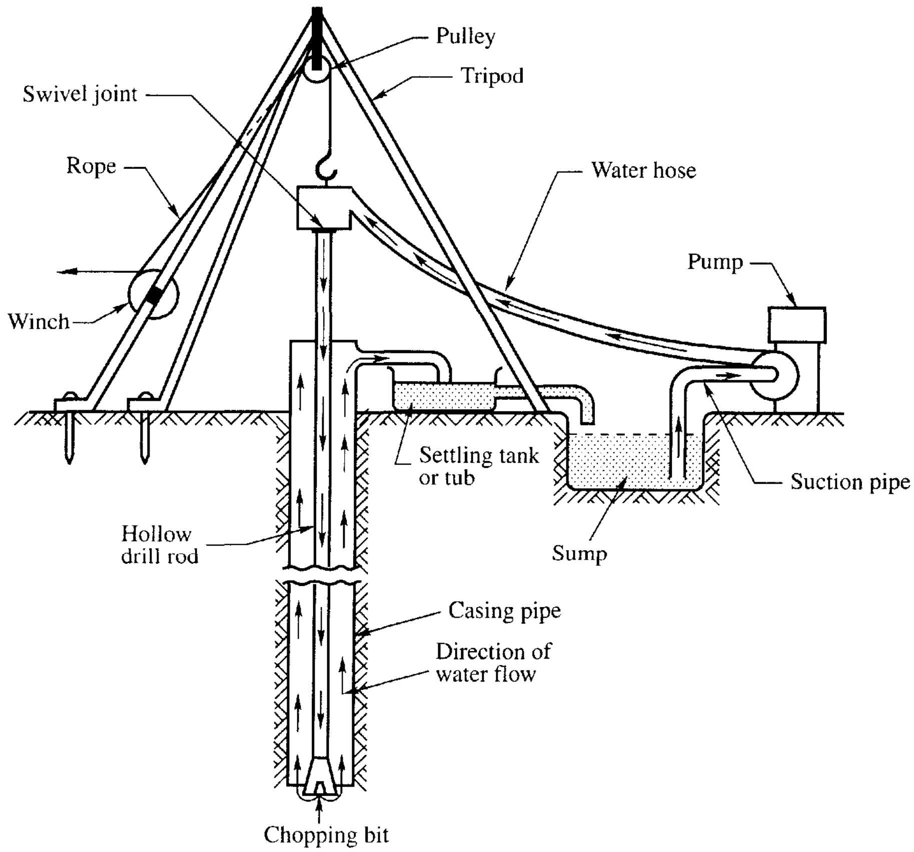


Figure 9.3 Wash boring

The purpose of wash boring is to drill holes only and not to make use of the disturbed washed materials for analysis. Whenever an undisturbed sample is required at a particular depth, the boring is stopped, and the chopping bit is replaced by a sampler. The sampler is pushed into the soil at the bottom of the hole and the sample is withdrawn.

Rotary Drilling

In the rotary drilling method a cutter bit or a core barrel with a coring bit attached to the end of a string of drill rods is rotated by a power rig. The rotation of the cutting bit shears or chips the material penetrated and the material is washed out of the hole by a stream of water just as in the case of a wash boring. Rotary drilling is used primarily for penetrating the overburden between the levels of which samples are required. Coring bits, on the other hand, cut an annular hole around an intact core which enters the barrel and is retrieved. Thus the core barrel is used primarily in rocky strata to get rock samples.

As the rods with the attached bit or barrel are rotated, a downward pressure is applied to the drill string to obtain penetration, and drilling fluid under pressure is introduced into the bottom of the hole through the hollow drill rods and the passages in the bit or barrel. The drilling fluid serves the dual function of cooling the bit as it enters the hole and removing the cuttings from the bottom of the hole as it returns to the surface in the annular space between the drill rods and the walls of the hole. In an uncased hole, the drilling fluid also serves to support the walls of the hole. When boring

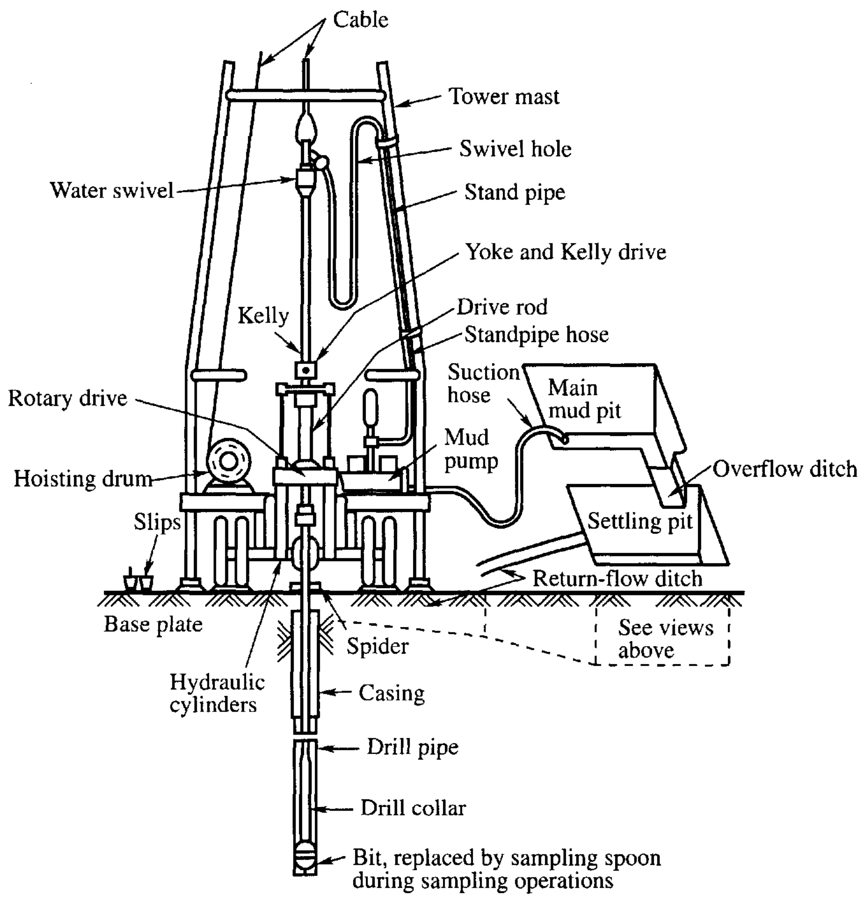


Figure 9.4 Rotary drilling rig (After Hvorslev, 1949)

in soil, the drilling bit is removed and replaced by a sampler when sampling is required, but in rocky strata the coring bit is used to obtain continuous rock samples. The rotary drilling rig of the type given in Fig. 9.4 can also be used for wash boring and auger boring.

Coring Bits

Three basic categories of bits are in use. They are diamond, carbide insert, and saw tooth. Diamond coring bits may be of the surface set or diamond impregnated type. Diamond coring bits are the most versatile of all the coring bits since they produce high quality cores in rock materials ranging from soft to extremely hard. Carbide insert bits use tungsten carbide in lieu of diamonds. Bits of this type are used to core soft to medium hard rock. They are less expensive than diamond bits but the rate of drilling is slower than with diamond bits. In saw-tooth bits, the cutting edge comprises a series of teeth. The teeth are faced and tipped with a hard metal alloy such as tungsten carbide to provide wear resistance and thereby increase the life of the bit. These bits are less expensive but normally used to core overburden soil and very soft rocks only.

Percussion Drilling

Percussion drilling is another method of drilling holes. Possibly this is the only method for drilling in river deposits mixed with hard boulders of the quartzitic type. In this method a heavy drilling bit is alternatively raised and dropped in such a manner that it powders the underlying materials which form a slurry with water and are removed as the boring advances.

9.3 SAMPLING IN SOIL

Soils met in nature are heterogeneous in character with a mixture of sand, silt and clay in different proportions. In water deposits, there are distinct layers of sand, silt and clay of varying thicknesses and alternating with depth. We can bring all the deposits of soil under two distinct groups for the purpose of study, namely, coarse grained and fine grained soils. Soils with particles of size coarser than 0.075 mm are brought under the category of coarse grained and those finer than 0.075 mm under fine grained soils. Sandy soil falls in the group of coarse grained, and silt and clay soils in the fine grained group. A satisfactory design of a foundation depends upon the accuracy with which the various soil parameters required for the design are obtained. The accuracy of the soil parameters depends upon the accuracy with which representative soil samples are obtained from the field.

Disturbed Samples

Auger samples may be used to identify soil strata and for field classification tests, but are not useful for laboratory tests. The cuttings or chopping from wash borings are of little value except for indicating changes in stratification to the boring supervisor. The material brought up with the drilling mud is contaminated and usually unsuitable even for identification.

For proper identification and classification of a soil, representative samples are required at frequent intervals along the bore hole. Representative samples can usually be obtained by driving or pushing into the strata in a bore hole an open-ended sampling spoon called a split spoon sampler (Fig. 9.5) which is used for conducting standard penetration tests (refer Sect. 9.5). It is made up of a driving shoe and a barrel. The barrel is split longitudinally into two halves with a coupling at the upper end for connection to the drill rods. The dimensions of the split spoon are given in Fig. 9.5. In a test the sampler is driven into the soil a measured distance. After a sample is taken, the cutting shoe and the coupling are unscrewed and the two halves of the barrel separated to expose the material. Experience indicates that samples recovered by this device are likely to be highly disturbed and as such can only be used as disturbed samples. The samples so obtained are stored in glass or plastic jars or bags, referenced and sent to the laboratory for testing. If spoon samples are to be transported to the laboratory without examination in the field, the barrel is often cored out to hold a cylindrical thin-walled tube known as a *liner*. After a sample has been obtained, the liner and the sample it contains are removed from the spoon and the ends are sealed with caps or with metal discs and wax. Samples of cohesionless soils below the water table cannot be retained in conventional sampling spoons without the addition of a *spring core catcher.*

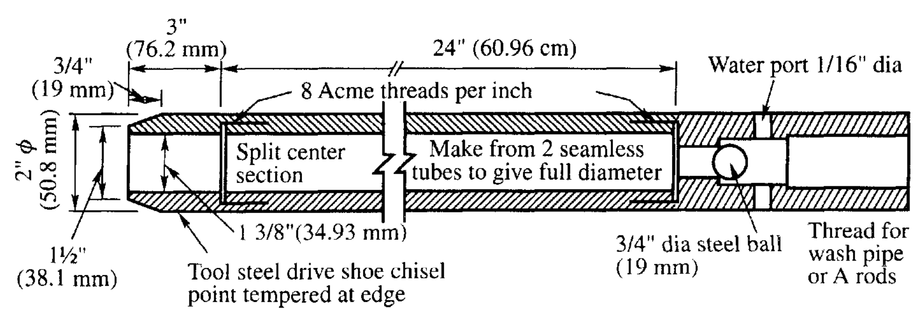


Figure 9.5 Split barrel sampler for standard penetration test

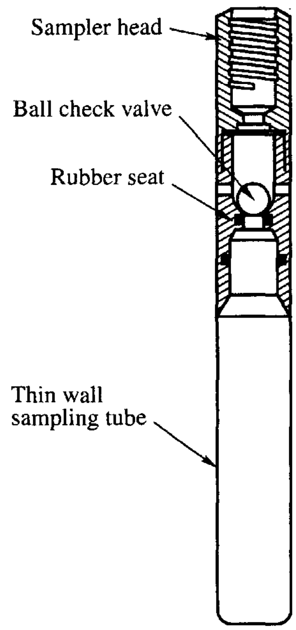


Figure 9.6 Thin wall Shelby tube sampler

Sampler

Many types of samplers are in use for extracting the so called undisturbed samples. Only two types of samplers are described here. They are,

1. Open drive sampler,
2. Piston sampler.

Open Drive Sampler

The wall thickness of the open drive sampler used for sampling may be thin or thick according to the soil conditions met in the field. The samplers are made of seamless steel pipes. A thin-walled tube sampler is called as shelby tube sampler (Fig. 9.6), consists of a thin wall metal tube connected to a sampler head. The sampler head contains a ball check valve and ports which permit the escape of water or air from the sample tube as the sample enters it. The thin wall tube, which is normally formed from 1/16 to 1/8 inch metal, is drawn in at the lower end and is reamed so that the inside diameter of the cutting edge is 0.5 to 1.5 percent less than that of the inside diameter of the tube. The exact percentage is governed by the size and wall thickness of the tube. The wall thickness is governed by the *area ratio*, A_r, which is defined as

$$A_r = \frac{d_o^2 - d_i^2}{d_i^2} \times 100 \text{ percent,} \tag{9.1}$$

where, d_i = inside diameter,

d_o = outside diameter.

A_r is a measure of the volume of the soil displacement to the volume of the collected sample. Well-designed sampling tubes have an area ratio of about 10 percent. However, the area ratio may have to be much more than 10 percent when samples are to be taken in very stiff to hard clay soils mixed with stones to prevent the edges of the sampling tubes from distortion during sampling.

Sample Extraction

The thin-wall tube sampler is primarily used for sampling in soft to medium stiff cohesive soils. The wall thickness has to be increased if sampling is to be done in very stiff to hard strata. For best results it is better to push the sampler statically into the strata. Samplers are driven into the strata where pushing is not possible or practicable. The procedure of sampling involves attaching a string of drill rods to the sampler tube adapter and lowering the sampler to rest on the bottom of the bore hole which was cleaned of loose materials in advance. The sampler is then pushed or driven into the soil. Over driving or pushing should be avoided. After the sampler is pushed to the required depth, the soil at the bottom of the sampler is sheared off by giving a twist to the drill rod at the top. The sampling tube is taken out of the bore hole and the tube is separated from the sampler head. The top and bottom of the sample are either sealed with molten wax or capped to prevent evaporation of moisture. The sampling tubes are suitably referenced for later identification.

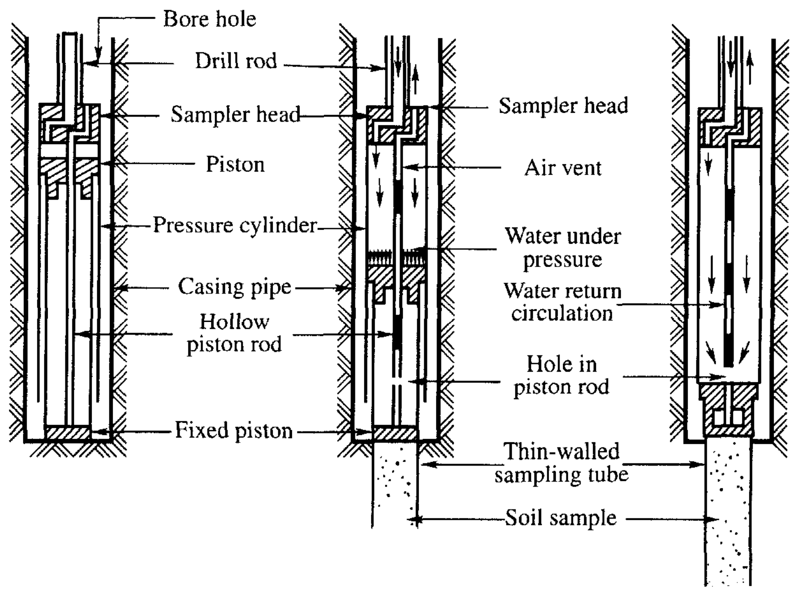


Figure 9.7 Osterberg Piston Sampler (a) Sampler is set in drilled hole. (b) Sample tube is pushed hydraulically into the soil. (c) Pressure is released through hole in piston rod.

Piston Sampler (After Osterberg 1952)

To improve the quality of samples and to increase the recovery of soft or slightly cohesive soils, a *piston sampler* is normally used. Such a sampler consists of a thin walled tube fitted with a piston that closes the end of the sampling tube until the apparatus is lowered to the bottom of the bore hole (Fig. 9.7(a)). The sampling tube is pushed into the soil hydraulically by keeping the piston stationary (Fig. 9.7(b)). The presence of the piston prevents the soft soils from squeezing rapidly into the tube and thus eliminates most of the distortion of the sample. The piston also helps to increase the length of sample that can be recovered by creating a slight vacuum that tends to retain the sample if the top of the column of soil begins to separate from the piston. During the withdrawal of the sampler, the piston also prevents water pressure from acting on the top of the sample and thus increases the chances of recovery. The design of piston samplers has been refined to the extent that it is sometimes possible to take undisturbed samples of sand from below the water table. However, piston sampling is relatively a costly procedure and may be adopted only where its use is justified.

Example 9.1

The following dimensions are given for a shelby tube sampler:

External diameter = 51 mm
Internal diameter = 48 mm
Determine the area ratio

Solution

Per Eq (9.1) the area ratio A_r is

$$A_r = \frac{d_o^2 - d_i^2}{d_i^2} = \frac{51^2 - 48^2}{48^2} = 0.129 = 12.9\%$$

Example 9.2

75 mm is the external diameter of a sampling tube. If the area ratio required is 20%, determine the thickness of the sampling tube. In what type of clay would such a high area ratio be required?

Solution

$$A_r = \frac{75^2 - d_i^2}{d_i^2} = 0.20$$

Solving $\quad d_i = 68.465$ mm.

The wall thickness $\quad = \dfrac{75.0 - 68.465}{2} = 3.267$ mm

When samples are to be taken in very stiff to hard clay soils mixed with stones, sampling tubes with high area ratios are required.

9.4 ROCK CORE SAMPLING

Rock coring is the process in which a sampler consisting of a tube (core barrel) with a cutting bit at its lower end cuts an annular hole in a rock mass, thereby creating a cylinder or core of rock which is recovered in the core barrel. Rock cores are normally obtained by rotary drilling.

The primary purpose of core drilling is to obtain intact samples. The behavior of a rock mass is affected by the presence of fractures in the rock. The size and spacing of fractures, the degree of weathering of fractures, and the presence of soil within the fractures are critical items. Figure 9.8 gives a schematic diagram of core barrels with coring bits at the bottom. As discussed earlier, the cutting element may consist of diamonds, tungsten carbide inserts or chilled shot. The core barrel may consist of a single tube or a double tube. Samples taken in a single tube barrel are likely to experience

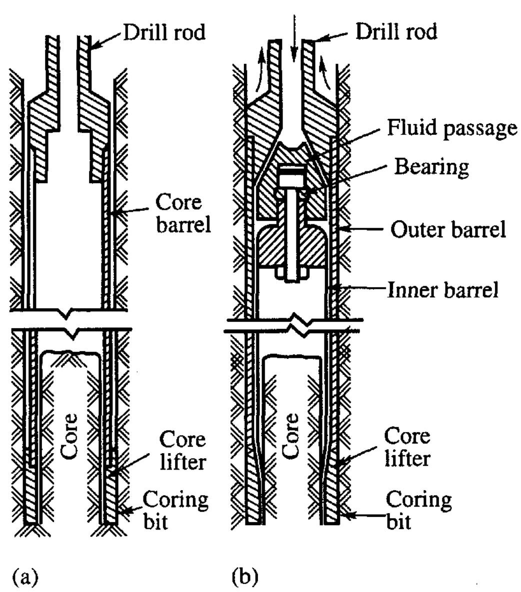


Figure 9.8 Schematic diagram of core barrels (a) Single tube. (b) Double tube.

considerable disturbance due to torsion, swelling and contamination by the drilling fluid, but these disadvantages are not there if the coring is conducted in hard, intact, rocky strata. However, if a double tube barrel is used, the core is protected from the circulating fluid. Most core barrels are capable of retaining cores up to a length of 2 m. Single barrel is used in Calyx drilling. Standard rock cores range from about 1¼ inches to nearly 6 inches in diameter. The more common sizes are given in Table 9.1.

The *recovery ratio* R_r, defined as the percentage ratio between the length of the core recovered and the length of the core drilled on a given run, is related to the quality of rock encountered in boring, but it is also influenced by the drilling technique and the type and size of core barrel used. Generally the use of a double tube barrel results in higher recovery ratios than can be obtained with single tube barrels. A better estimate of in-situ rock quality is obtained by a modified core recovery ratio known as the *Rock Quality Designation* (RQD) which is expressed as

$$RQD = \frac{\overline{L}_a}{L_t} \tag{9.2}$$

where, $\overline{L}_a$ = total length of intact hard and sound pieces of core of length greater than 4 in. arranged in its proper position,

L_t = total length of drilling.

Breaks obviously caused by drilling are ignored. The diameter of the core should preferably be not less than $2^1/_8$ inches. Table 9.2 gives the rock quality description as related to RQD.

Table 9.1 Standard sizes of core barrels, drill rods, and compatible casing (Peck et al., 1974)

Core Barrel			Drill Rod		Casing		
Symbol	Hole dia (in)	Core dia (in)	Symbol	Outside dia (in)	Symbol	Outside dia (in)	Inside dia (in)
EWX, EWM	$1\frac{1}{2}$	$1^3/_{16}$	E	$1^{15}/_{16}$	–	–	–
AWX, AWM	$1^{15}/_{16}$	$1^3/_{16}$	A	$1^5/_8$	EX	$1^{13}/_{16}$	$1\frac{1}{2}$
BWX, BWM	$2^3/_8$	$1^5/_8$	B	$1^7/_8$	AX	$2^1/_4$	$1^{29}/_{32}$
NWX, NWM	3	$2^1/_8$	N	$2^3/_8$	BX	$2^7/_8$	$2^3/_8$
$2^3/_4 \times 3^7/_8$	$3^7/_8$	$2^{11}/_{16}$	–	–	NX	$3^1/_2$	3

Note: Symbol X indicates single barrel, M indicates double barrel.

Table 9.2 Relation of RQD and *in-situ* Rock Quality (Peck et al., 1974)

RQD %	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

9.5 STANDARD PENETRATION TEST

The SPT is the most commonly used *in situ* test in a bore hole in the USA. The test is made by making use of a split spoon sampler shown in Fig. 9.7. The method has been standardized as ASTM D-1586 (1997) with periodic revision since 1958. The method of carrying out this test is as follows:

1. The split spoon sampler is connected to a string of drill rods and is lowered into the bottom of the bore hole which was drilled and cleaned in advance.
2. The sampler is driven into the soil strata to a maximum depth of 18 in by making use of a 140 lb weight falling freely from a height of 30 in on to an anvil fixed on the top of drill rod. The weight is guided to fall along a guide rod. The weight is raised and allowed to fall by means of a manila rope, one end tied to the weight and the other end passing over a pulley on to a hand operated winch or a motor driven cathead.
3. The number of blows required to penetrate each of the successive 6 in depths is counted to produce a total penetration of 18 in.
4. To avoid seating errors, the blows required for the first 6 in of penetration are not taken into account; those required to increase the penetration from 6 in to 18 in constitute the *N*-value.

As per some codes of practice if the *N*-value exceeds 100, it is termed as refusal, and the test is stopped even if the total penetration falls short of the last 300 mm depth of penetration. Standardization of refusal at 100 blows allows all the drilling organizations to standardize costs so that higher blows if required may be eliminated to prevent the excessive wear and tear of the equipment. The SPT is conducted normally at 2.5 to 5 ft intervals. The intervals may be increased at greater depths if necessary.

Standardization of SPT

The validity of the SPT has been the subject of study and research by many authors for the last many years. The basic conclusion is that the best results are difficult to reproduce. Some of the important factors that affect reproducibility are

1. Variation in the height of fall of the drop weight (hammer) during the test
2. The number of turns of rope around the cathead, and the condition of the manila rope
3. Length and diameter of drill rod
5. Diameter of bore hole
6. Overburden pressure

There are many more factors that hamper reproducibility of results. Normally corrections used to be applied for a quick condition in the hole bottom due to rapid withdrawal of the auger. ASTM 1586 has stipulated standards to avoid such a quick condition. Discrepancies in the input driving energy and its dissipation around the sampler into the surrounding soil are the principal factors for the wide range in *N* values. The theoretical input energy may be expressed as

$$E_{in} = Wh \tag{9.3}$$

where W = weight or mass of the hammer

h = height of fall

Investigation has revealed (Kovacs and Salomone, 1982) that the actual energy transferred to the driving head and then to the sampler ranged from about 30 to 80 percent. It has been suggested that the SPT be standardized to some energy ratio R_e keeping in mind the data collected so far from the existing SPT. Bowles (1996) suggests that the observed SPT value N be reduced to a standard blow count corresponding to 70 percent of standard energy. Terzaghi, et al., (1996) suggest 60 percent. The standard energy ratio may be expressed as

$$R_{es} = \frac{\text{Actual hammer energy to sampler}, E_a}{\text{Input energy}, E_{in}} \tag{9.4}$$

Corrections to the Observed SPT Value

Three types of corrections are normally applied to the observed N values. They are:

1. Hammer efficiency correction
2. Drillrod, sampler and borehole corrections
3. Correction due to overburden pressure

1. Hammer Efficiency Correction, E_h

Different types of hammers are in use for driving the drill rods. Two types are normally used in USA. They are (Bowles, 1996)

1. Donut with two turns of manila rope on the cathead with a hammer efficiency $E_h = 0.45$.
2. Safety with two turns of manila rope on the cathead with a hammer efficiency as follows:

   Rope-pulley or cathead = 0.7 to 0.8;

   Trip or automatic hammer = 0.8 to 1.0.

2. Drill Rod, Sampler and Borehole Corrections

Correction factors are used for correcting the effects of length of drill rods, use of split spoon sampler with or without liner, and size of bore holes. The various correction factors are (Bowles, 1996).

a) Drill rod length correction factor C_d

Length (m)	Correction factor (C_d)
> 10 m	1.0
4–10 m	0.85–0.95
< 4.0 m	0.75

b) Sampler correction factor, C_s

Without liner $C_s = 1.00$

With liner,

Dense sand, clay = 0.80

Loose sand = 0.90

c) Bore hole diameter correction factor, C_b

Bore hole diameter	Correction factor, C_b
60-120 mm	1.0
150 mm	1.05
200 mm	1.15

3. Correction Factor for Overburden Pressure in Granular Soils, C_N

The C_N as per Liao and Whitman (1986) is

$$C_N = \left[\frac{95.76}{\rho'_o}\right]^{1/2} \tag{9.5}$$

where, ρ'_o =effective overburden pressure in kN/m^2

There are a number of empirical relations proposed for C_N. However, the most commonly used relationship is the one given by Eq. (9.5).

N_{cor} may be expressed as

$$N_{cor} = C_N N E_h C_d C_s C_b \tag{9.6}$$

N_{cor} is related to the standard energy ratio used by the designer. N_{cor} may be expressed as N_{70} or N_{60} according to the designer's choice.

In Eq (9.6) $C_N N$ is the corrected value for overburden pressure only. The value of C_N as per Eq. (9.5) is applicable for granular soils only, whereas $C_N = 1$ for cohesive soils for all depths.

Example 9.3

The observed standard penetration test value in a deposit of fully submerged sand was 45 at a depth of 6.5 m. The average effective unit weight of the soil is 9.69 kN/m^3. The other data given are (a) hammer efficiency = 0.8, (b) drill rod length correction factor = 0.9, and (c) borehole correction factor = 1.05. Determine the corrected SPT value for standard energy (a) R_{es} = 60 percent, and (b) R_{es} = 70%.

Solution

Per Eq (9.6), the equation for N_{60} may be written as

(i) $N_{60} = C_N N E_h C_d C_s C_b$

where N = observed SPT value

C_N = overburden correction

Per Eq (9.5) we have

$$C_N = \frac{95.76}{p'_o}^{1/2}$$

where p'_0 = effective overburden pressure

$= 6.5 \times 9.69 = 63$ kN/m^2

Substituting for p'_0,

$$C_N = \frac{95.76}{60}^{1/2} = 1.233$$

Substituting the known values, the corrected N_{60} is

$N_{60} = 1.233 \times 45 \times 0.8 \times 0.9 \times 1.05 = 42$

For 70 percent standard energy

$$N_{70} = 42 \times \frac{0.6}{0.7} = 36$$

9.6 SPT VALUES RELATED TO RELATIVE DENSITY OF COHESIONLESS SOILS

Although the SPT is not considered as a refined and completely reliable method of investigation, the N_{cor} values give useful information with regard to consistency of cohesive soils and relative density of cohesionless soils. The correlation between N_{cor} values and relative density of granular soils suggested by Peck, et al., (1974) is given in Table 9.3.

Before using Table 9.3 the observed N value has to be corrected for standard energy and overburden pressure. The correlations given in Table 9.3 are just a guide and may vary according to the fineness of the sand.

Meyerhof (1956) suggested the following approximate equations for computing the angle of friction ϕ from the known value of D_r.

For granular soil with fine sand and more than 5 percent silt,

$$\phi^\circ = 25 + 0.15D_r \tag{9.7}$$

For granular soils with fine sand and less than 5 percent silt,

$$\phi^\circ = 30 + 0.15D_r \tag{9.8}$$

where D_r is expressed in percent.

9.7 SPT VALUES RELATED TO CONSISTENCY OF CLAY SOIL

Peck et al., (1974) have given for saturated cohesive soils, correlations between N_{cor} value and consistency. This correlation is quite useful but has to be used according to the soil conditions met in the field. Table 9.4 gives the correlations.

The N_{cor} value to be used in Table 9.4 is the blow count corrected for standard energy ratio R_{es}. The present practice is to relate q_u with N_{cor} as follows,

$$q_u = \bar{k}N_{cor} \text{ kPa} \tag{9.9}$$

Table 9.3 N_{cor} and ϕ Related to Relative Density

N_{cor}	Compactness	Relative density, D_r (%)	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	36-41
>50	Very Dense	>85	> 41

Table 9.4 Relation Between N_{cor} and q_u

Consistency	N_{cor}	q_u, kPa
Very soft	0-2	<25
Soft	2-4	25-50
Medium	4-8	50-100
Stiff	8-15	100-200
Very Stiff	15-30	200-400
Hard	>30	>400

where q_u is the unconfined compressive strength.

or $$\bar{k} = \frac{q_u}{N_{cor}} \text{ kPa} \tag{9.10}$$

where, $\bar{k}$ is the proportionality factor. A value of $\bar{k}$ = 12 has been recommended by Bowles (1996).

Example 9.4

For the corrected N values in Ex 9.3, determine the (a) relative density, and (b) the angle of friction. Assume the percent of fines in the deposit is less than 5%.

Solution

Per Table 9.3 the relative density and ϕ are

For $N_{60} = 42$, $D_r = 77\%$, $\phi = 39°$

For $N_{70} = 36$, $D_r = 71\%$, $\phi = 37.5°$

Per Eq (9.8)

For $D_r = 77\%$, $\phi = 30 + 0.15 \times 77 = 41.5°$

For $D_r = 71\%$, $\phi = 30 + 0.15 \times 71 = 40.7°$

Example 9.5

For the corrected values of N given in Ex 9.4, determine the unconfined compressive strength q_u in a clay deposit.

Solution

(a) From Table 9.4

$$\left.\begin{array}{l}\text{For } N_{60} = 42 \\ \text{For } N_{70} = 36\end{array}\right\} q_u > 400 \text{ kPa - The soil is of a hard consistency.}$$

(b) Per Eq (9.9)

$q_u = \bar{k} N_{cor}$, where $\bar{k} = 12$ (Bowles, 1996)

For $N_{60} = 42$, $q_u = 12 \times 42 = 504$ kPa

For $N_{70} = 36$, $q_u = 12 \times 36 = 432$ kPa

Example 9.6

Refer to Example 9.3. Determine the corrected SPT value for R_{es} = 100 percent, and the corresponding values of D_r and ϕ. Assume the percent of fine sand in the deposit is less than 5%.

Solution

From Example 9.3, $N_{60} = 42$

$$\text{Hence } N_{100} = 2 \times \frac{0.6}{1.0} \approx 25$$

From Table 9.3, $\phi = 34.5°$ and $D_r = 57.5\%$

From Eq. (9.8) for $D_r = 57.5\%$, $\phi = 30 + 0.15 \times 57.5 = 38.6°$.

9.8 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. 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:

1. General introduction of the electric penetrometer providing more precise measurements, and improvements in the equipment allowing deeper penetration.
2. 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.
3. The addition of other simultaneous measurements to the standard friction penetrometer such as pore pressure and soil temperature.

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 (Schmertmann 1975) and provides useful information in particular when no bore hole 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,

1. The Mechanical Type,
2. The Electrical Type.

Mechanical Type

The Begemann Friction Cone Mechanical type penetrometer is shown in Fig. 9.9. It consists of a 60° cone with a base diameter of 35.6 mm (sectional area 10 cm^2). A sounding rod is screwed to the base. Additional rods of one meter 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 the dimensions in Fig. 9.9 are in mm.

Jacking System

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 Fig. 9.10. Bourden gauges are provided in the driving mechanism for measuring the pressures exerted by the cone and the 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 upthrust. For ground based rigs screw anchors are provided to take up the reaction thrust.

Operation of Penetrometer

The sequence of operation of the penetrometer shown in Fig. 9.11 is explained as follows:

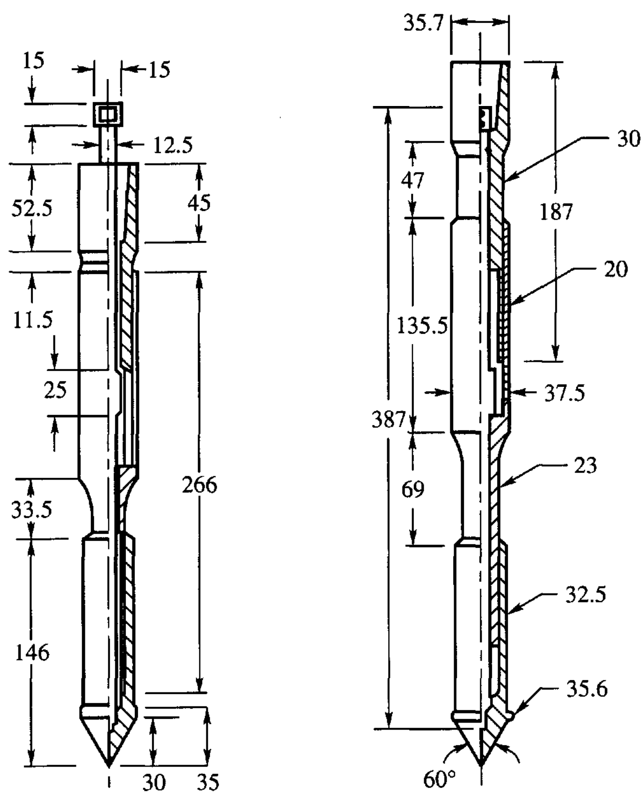
Position 1 The cone and friction jacket assembly in a collapsed position.

Position 2 The cone is pushed down by the inner sounding rods to a depth a 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_t. 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 above 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.



Note: All dimensions are in mm.

Figure 9.9 Begemann friction-cone mechanical type penetrometer

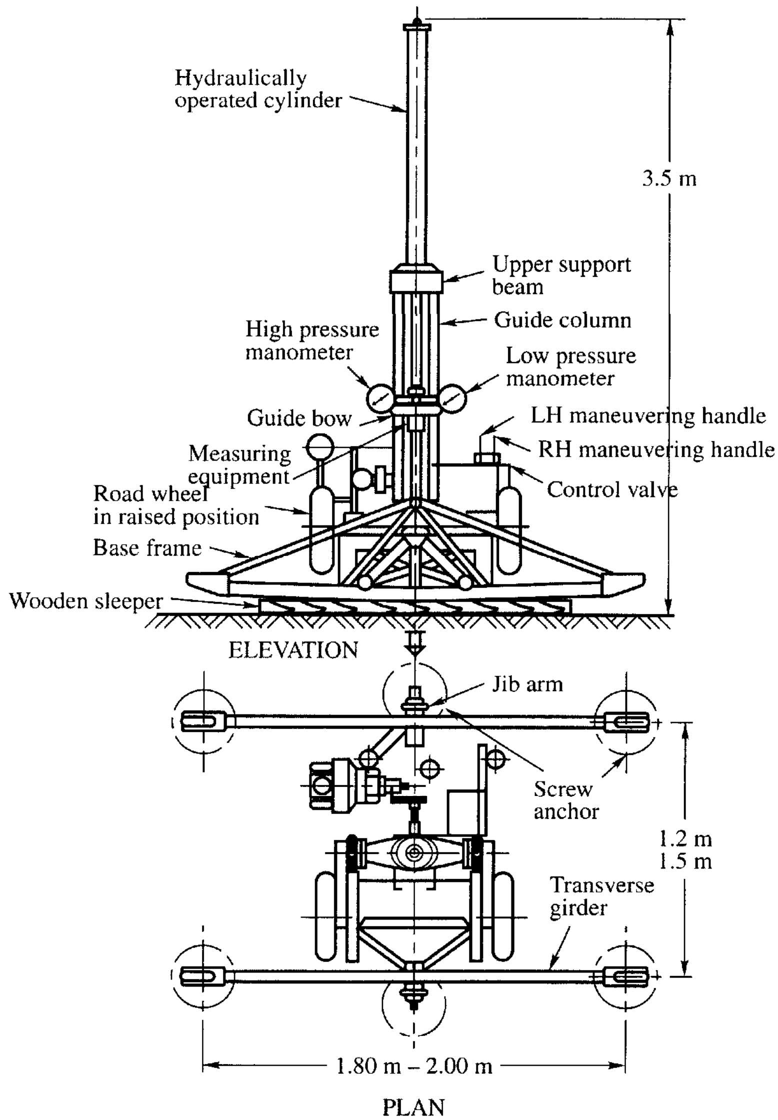


Fig. 9.10 Static cone penetration testing equipment

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 9.12 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) connect the penetrometer with the recording equipment at the surface which produces graphs of resistance versus depth.

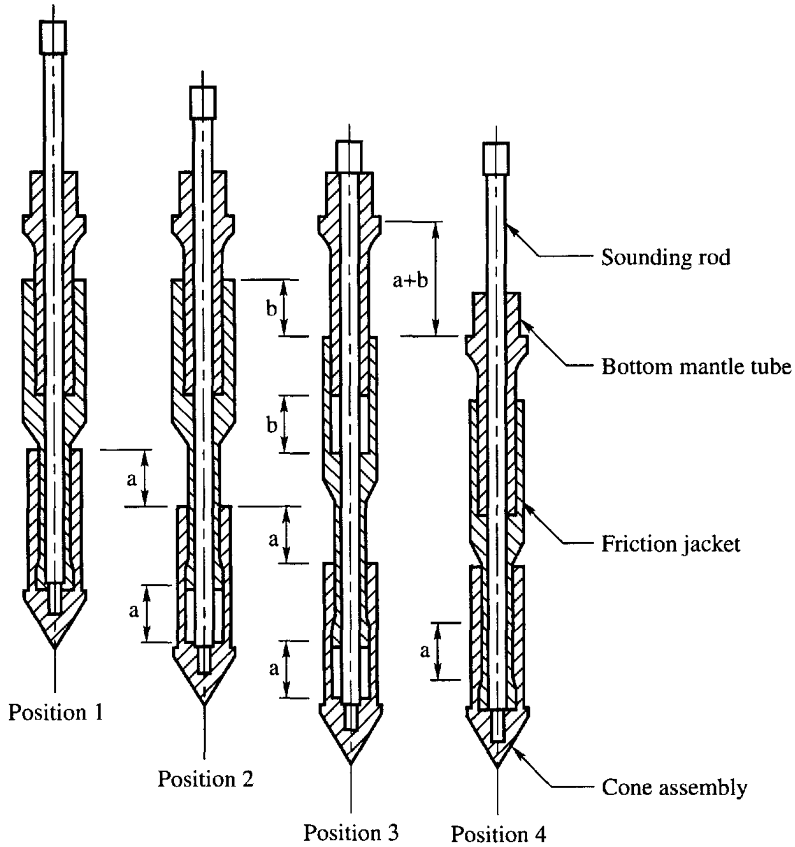


Figure 9.11 Four positions of the sounding apparatus with friction jacket

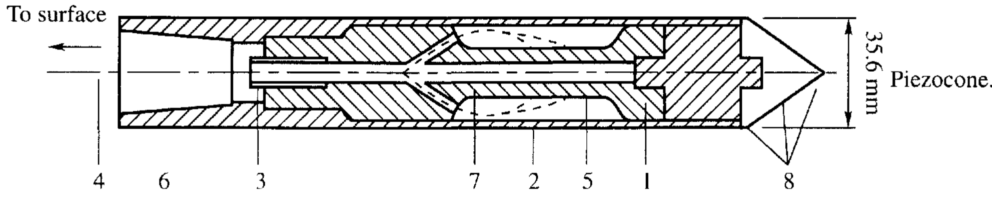
The electric penetrometer has many advantages. The repeatability of the cone test is very good. A continuous record of the penetration results reflects better the nature of the soil layers penetrated. However, electronic cone testing requires skilled operators and better maintenance. The electric penetrometer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

Operation of 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.

Piezocone

A piezometer element included in the cone penetrometer is called a *piezocone* (Fig. 9.13). 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 within the sand seams.



1. Load cell
2. Friction sleeve
3. Water proof bushing
4. Cable
5. Strain gases
6. Connection with rods
7. Inclinometer
8. Porous stone (piezometer)

Figure 9.12 An-electric-static cone penetrometer

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.

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.

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 of the cone.

$$q_c = \frac{Q_c}{A_c}. \tag{9.11}$$

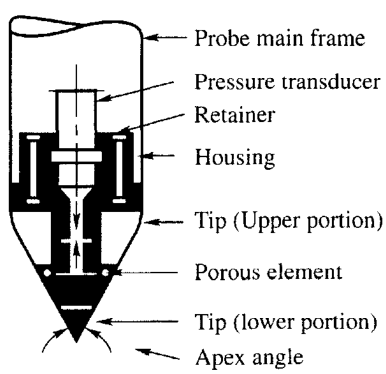


Figure 9.13 Details of 60°/10 cm^2 piezocone

In the same way, the local side friction f_c is

$$f_c = \frac{Q_f}{A_f}, \tag{9.12}$$

where, $Q_f = Q_t - Q_c$ = force required to push the friction jacket,

Q_t = the 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.

Friction Ratio, R_f

Friction ratio, R_f is expressed as

$$R_f = \frac{f_c}{q_c}, \tag{9.13}$$

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 (Fig. 9.16).

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 (Fig. 9.14) which may be used to estimate D_r based on q_c and effective

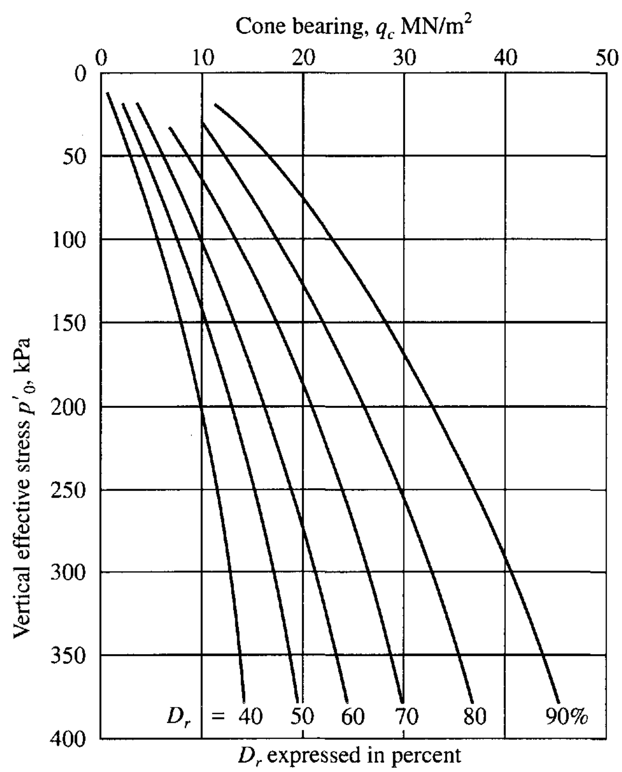


Figure 9.14 Relationship between relative density D_r and penetration resistance q_c for uncemented quartz sands (Robertson and Campanella, 1983a)

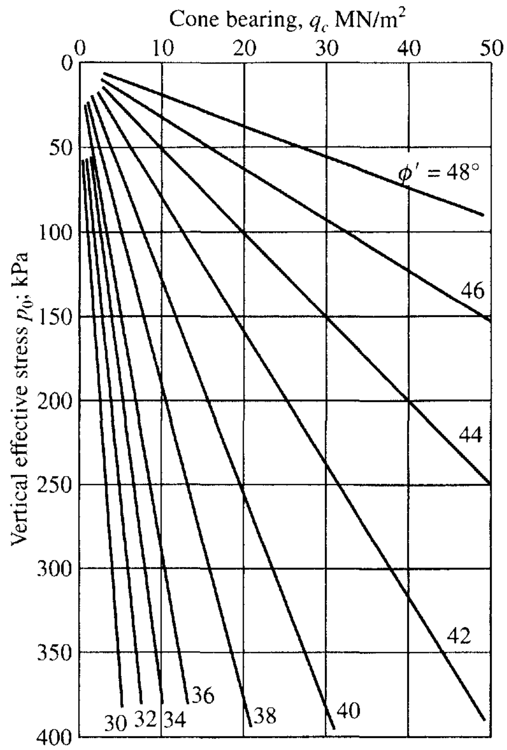


Figure 9.15 Relationship between cone point resistance q_c and angle of internal friction ϕ for uncemented quartz sands (Robertson and Campanella, 1983b)

overburden pressure. These curves are supposed to be applicable for normally consolidated clean sand. Fig. 9.15 gives the relationship between q_c and ϕ (Robertson and Campanella, 1983b).

Relationship Between q_c and Undrained Shear Strength, c_u of Clay

The cone penetration resistance q_c and c_u may be related as

$$q_c = N_k c_u + p_0 \quad \text{or} \quad c_u = \frac{q_c - p_o}{N_k} \tag{9.14}$$

where, N_k = cone factor,

$p_o = \gamma z$ = overburden pressure.

Lunne 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:

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

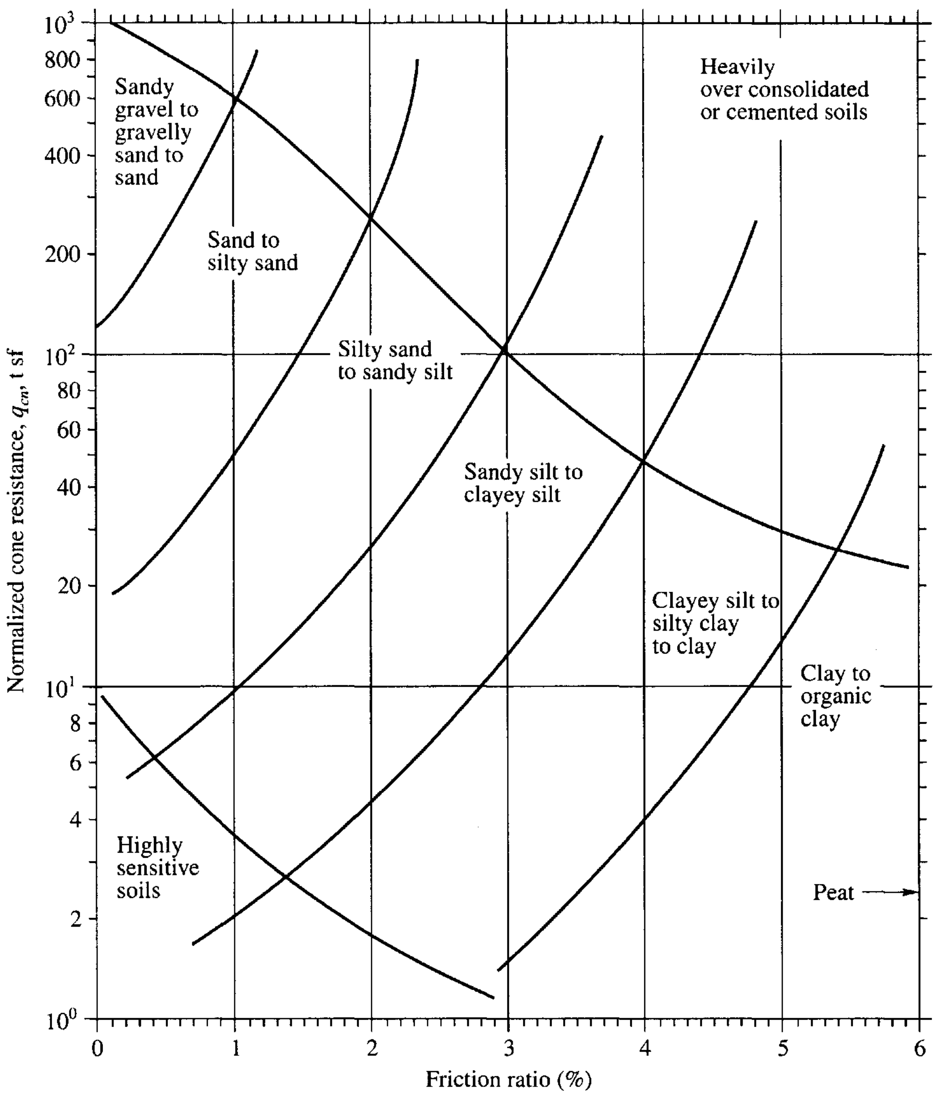


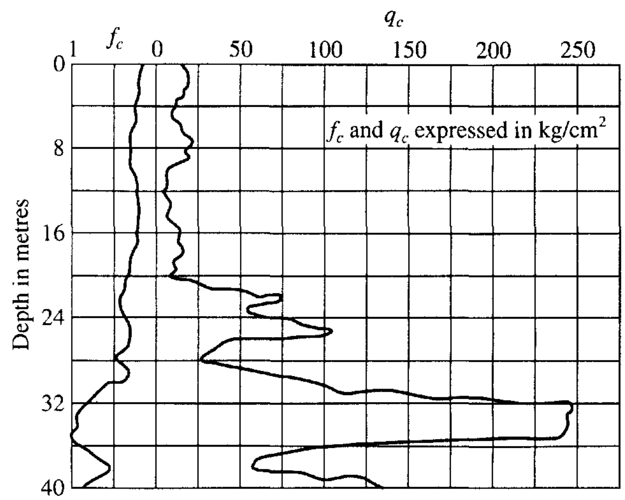
Figure 9.16 A simplified classification chart (Douglas, 1984)

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

Soil Classification

One of the basic uses of CPT is to identify and classify soils. A CPT-Soil Behavior Type Prediction System has been developed by Douglas and Olsen (1981) using an electric-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.



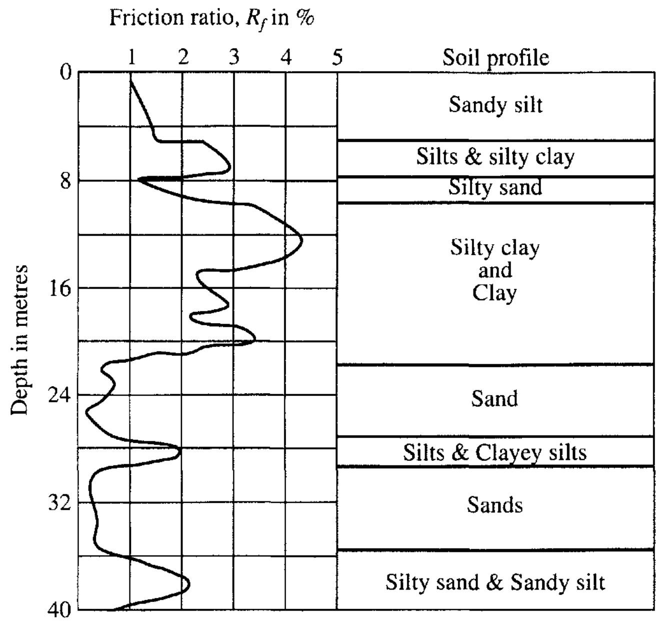


Figure 9.17 A typical sounding log

Douglas (1984) presented a simplified classification chart shown in Fig. 9.16. His chart uses cone resistance normalized (q_{cn}) for overburden pressure using the equation

$$q_{cn} = q_c (1 - 1.25 \log p'_o) \tag{9.15}$$

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

In conclusion, CPT data provide 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 Fig. 9.17.

Table 9.5 Soil classification based on friction ratio R_f (Sanglerat, 1972)

R_f%	Type of soil
0–0.5	Loose gravel fill
0.5–2.0	Sands or gravels
2–5	Clay sand mixtures and silts
> 5	Clays, peats etc.

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 9.5

Correlation Between SPT and CPT

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

$$q_c = 0.4\, N \text{ MN/m}^2 \tag{9.16}$$

His findings are as given in Table 9.6.

Table 9.6 Approximate relationship between relative density of fine sand, the SPT, the static cone resistance and the angle of internal fraction (Meyerhof, 1965)

State of sand	D_r	N_{cor}	q_c (MPa)	$\phi°$
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

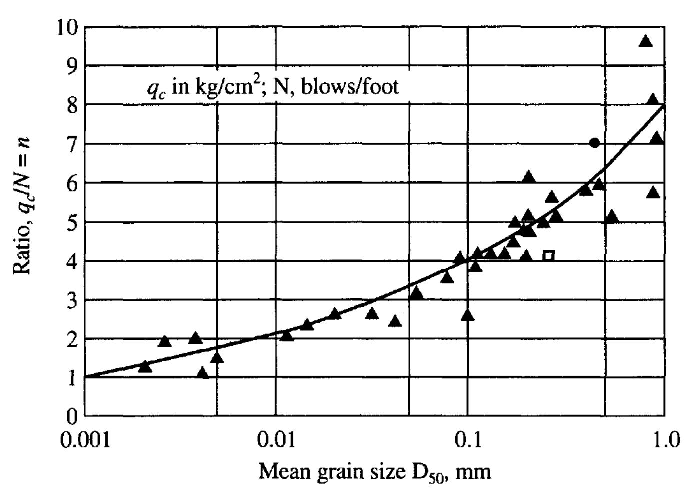


Figure 9.18 Relationship between q_c/N and mean grain size D_{50} (mm) (Robertson and Campanella, 1983a)

The lowest values of the angle of internal friction given in Table 9.6 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 the angles of internal friction which apply to well graded sand, may be increased by 5° for gravelly sand.

Figure 9.18 shows a correlation presented by Robertson and Campanella (1983) between the ratio of q_c/N and mean grain size, D_{50}. It can be seen from the figure 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 judgment while selecting a value for n for a particular type of soil.

Example 9.7

If a deposit at a site happens to be a saturated overconsolidated clay with a value of q_c = 8.8 MN/m^2, determine the unconfined compressive strength of clay given p_0 = 127 kN/m^2

Solution

Per Eq (9.14)

$$c_u = \frac{q_c - p_o}{N_k} \quad \text{or} \quad q_u = \frac{2(q_c - p_o)}{N_k}$$

Assume N_k = 20. Substituting the known values and simplifying

$$q_u = \frac{2(8800-127)}{20} = 867 \text{ kN/m}^2$$

If we neglect the overburden pressure p_0

$$q_u = \frac{2\times 8800}{20} = 880 \text{ kN/m}^2$$

It is clear that, the value of q_u is little affected by neglecting the overburden pressure in Eq. (9.14)

Example 9.8

Static cone penetration tests were carried out at a site by using an electric-friction cone penetrometer. The following data were obtained at a depth of 12.5 m.

cone resistance q_c = 19.152 MN/m^2 (200 tsf)

friction ratio $R_f = \dfrac{f_c}{q_c} = 1.3$

Classify the soil as per Fig. 9.16. Assume γ(effective) = 16.5 kN/m^3.

Solution

The values of q_c = 19.152 × 10^3 kN/m^2 and R_f = 1.3. From Eq. (9.15)

$$q_{cn} = 200\times \left(1-1.25\log \frac{16.5\times 12.5}{100}\right) = 121 \text{ tsf}$$

The soil is sand to silty sand (Fig. 9.16) for q_{cn} = 121 tsf and R_f = 1.3.

Example 9.9

Static cone penetration test at a site at depth of 30 ft revealed the following

Cone resistance q_c = 125 tsf

Friction ratio R_f = 1.3%

The average effective unit weight of the soil is 115 psf. Classify the soil per Fig. 9.16.

Solution

The effective overburden pressure $p'_0 = 30 \times 115 = 3450 \text{ lb/ft}^2 = 1.725$ tsf

From Eq (9.15)

$q_{cn} = 125\,(1 - 1.25 \log 1.725) = 88$ tsf

$R_f = 1.3\%$

From Fig. 9.16, the soil is classified as sand to silty sand for $q_{cn} = 88$ tsf and $R_f = 1.3\%$

Example 9.10

The static cone penetration resistance at a site at 10 m depth is 2.5 MN/m^2. The friction ratio obtained from the test is 4.25%. If the unit weight of the soil is 18.5 kN/m^3, what type of soil exists at the site.

Solution

$q_c = 2.5 \times 1000 \text{ kN/m}^2 = 2500 \text{ kN/m}^2 = 26.1$ tsf

$p'_0 = 10 \times 18.5 = 185 \text{ kN/m}^2 = 1.93$ tsf

$q_{cn} = 26.1\,(1 - 1.25 \log 1.93) = 16.8$ tsf

$R_f = 4.25$ %

From Fig 9.16, the soil is classified as clayey silt to silty clay to clay

9.9 PRESSUREMETER

A pressuremeter test is an *in-situ* stress-strain test performed on the walls of a bore hole using a cylindrical probe that can be inflated radially. The pressuremeter, which was first conceived, designed, constructed and used by Menard (1957) of France, has been in use since 1957. The test results are used either directly or indirectly for the design of foundations. The Menard test has been adopted as ASTM Test Designation 4719. The instrument as conceived by Menard consists of three independent chambers stacked one above the other (Fig. 9.19) with inflatable user membranes held together at top and bottom by steel discs with a rigid hollow tube at the center. The top and bottom chambers protect the middle chamber from the end effects caused by the finite length of the apparatus, and these are known as guard cells. The middle chamber with the end cells together is called the *Probe*. The pressuremeter consists of three parts, namely, the probe, the control unit and the tubing.

The Pressuremeter Test

The pressuremeter test involves the following:

1. Drilling of a hole
2. Lowering the probe into the hole and clamping it at the desired elevation
3. Conducting the test

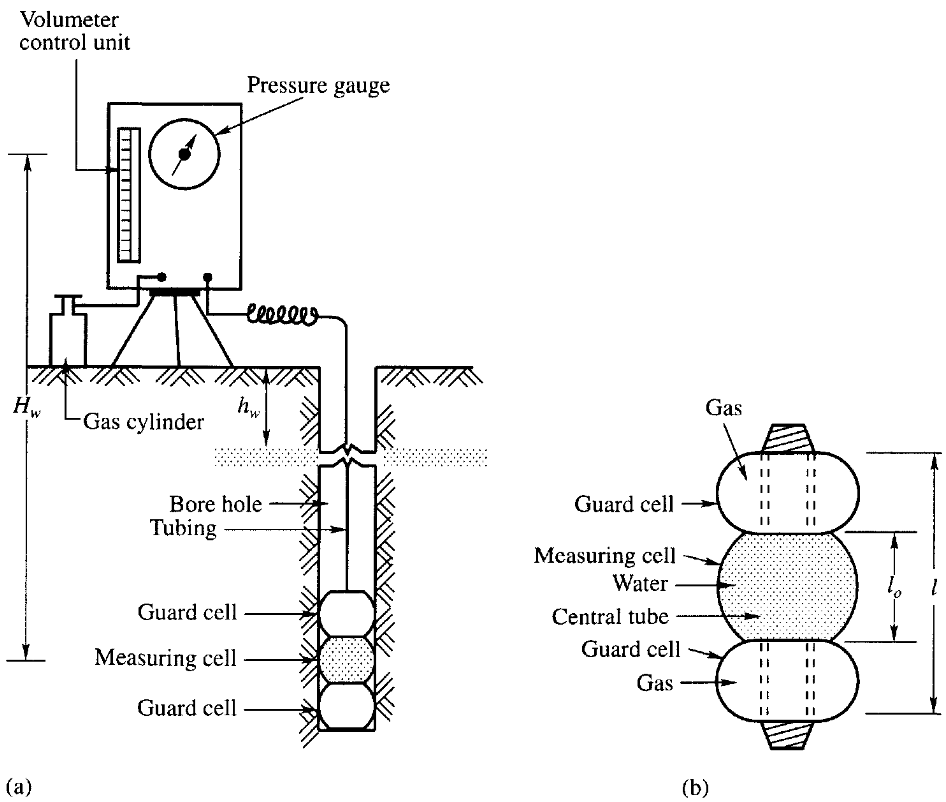


Figure 9.19 Components of Menard pressuremeter

Drilling and Positioning of Probe

A Menard pressure test is carried out in a hole drilled in advance. The drilling of the hole is completed using a suitable drilling rig which disturbs the soil the least. The diameter of the bore hole, D_h, in which the test is to be conducted shall satisfy the condition

$$1.03D_p < D_h < 1.20D_p \tag{9.17}$$

where D_p is the diameter of the probe under the deflated condition.

Typical sizes of the probe and bore hole are given in Table 9.7.

The probe is lowered down the hole soon after boring to the desired elevation and held in position by a clamping device. Pressuremeter tests are usually carried out at 1 m intervals in all the bore holes.

Conducting the Test

With the probe in position in the bore hole, the test is started by opening the valves in the control unit for admitting water and gas (or water) to the measuring cell and the guard cells respectively. The pressure in the guard cells is normally kept equal to the pressure in the measuring cell. The pressures to the soil through the measuring cell are applied by any one of the following methods:

1. Equal pressure increment method.
2. Equal volume increment method.

Table 9.7 Typical sizes of probe and bore hole for pressuremeter test

Hole dia. designation	Probe dia. (mm)	l_0 (cm)	l (cm)	Bore hole dia. Nominal (mm)	Bore hole dia. Max. (mm)
AX	44	36	66	46	52
BX	58	21	42	60	66
NX	70	25	50	72	84

Note: l_0 = length of measuring cell; l = length of probe.

If pressure is applied by the first method, each equal increment of pressure is held constant for a fixed length of time, usually one minute. Volume readings are made after one minute elapsed time. Normally ten equal increments of pressure are applied for a soil to reach the limit pressure, p_l.

If pressure is applied by the second method, the volume of the probe shall be increased in increments equal to 5 percent of the nominal volume of the probe (in the deflated condition) and held constant for 30 seconds. Pressure readings are taken after 30 seconds of elapsed time.

Steps in both the methods are continued until the maximum probe volume to be used in the test is reached. The test may continue at each position from 10 to 15 minutes. This means that the test is essentially an undrained test in clay soils and a drained test in a freely draining material.

Typical Test Result

First a typical curve based on the observed readings in the field may be plotted. The plot is made of the volume of the water read at the volumeter in the control unit, v, as abscissa for each increment of pressure, p, as ordinate. The curve is a result of the test conducted on the basis of equal increments of pressure and each pressure held constant for a period of one minute. This curve is a raw curve which requires some corrections. The pressuremeter has, therefore, to be calibrated before it is used in design. A pressuremeter has to be calibrated for

1. Pressure loss, p_c,
2. Volume loss v_c,
3. Difference in hydrostatic pressure head H_w.

Corrected Plot of Pressure-Volume Curve

A typical corrected plot of the pressure-volume curve is given in Fig. 9.20. The characteristic parts of this curve are:

1. The initial part of the curve OA. This curve is a result of pushing the yielded wall of the hole back to the original position. At point A, the at-rest condition is supposed to have been restored. The expansion of the cavity is considered only from point A. v_0 is the volume of water required to be injected over and above the volume V_c of the probe under the deflated condition. If V_0 is the total volume of the cavity at point A, we can write

$$V_0 = V_c + v_0 \tag{9.18}$$

   where v_o is the abscissa of point A. The horizontal pressure at point A is represented as p_{om}.
2. The second part of the curve is AB. This is supposed to be a straight line portion of the curve and may represent the elastic range. Since AB gives an impression of an elastic range, it is called the *pseudo-elastic* phase of the test. Point A is considered to be the start of the pressuremeter test in most theories. Point B marks the end of the straight line portion of the curve. The coordinates of point B are p_f and v_f, where p_f is known as the *creep pressure*.

3. Curve BC marks the final phase. The plastic phase is supposed to start from point B, and the curve becomes asymptotic at point C at a large deformation of the cavity. The *limit pressure*, p_l, is usually defined as the pressure that is required to double the initial volume of the cavity. It occurs at a volume such that

$$v_l - v_0 = V_0 = V_c + v_0 \tag{9.19}$$

$$\text{or} \quad v_l = V_c + 2v_0 \tag{9.20}$$

v_0 is normally limited to about 300 cm^3 for probes used in AX and BX holes. The initial volume of these probes is on the order of 535 cm^3. This means that $(V_c + 2v_0)$ is on the order of 1135 cm^3. These values may vary according to the design of the pressuremeter.

The reservoir capacity in the control unit should be of the order of 1135 cm^3. In case the reservoir capacity is limited and p_l is not reached within its limit, the test, has to be stopped at that level. In such a case, the limit value, p_l, has to be extrapolated.

At-Rest Horizontal Pressure

The at-rest total horizontal pressure, p_{oh}, at any depth, z, under the *in-situ* condition before drilling a hole may be expressed as

$$p_{oh} = (\gamma z - u)K_0 + u, \tag{9.21}$$

where u = pore pressure at depth z,

γ = gross unit weight of the soil,

K_0 = coefficient of earth pressure for the at-rest condition.

The values of γ and K_0 are generally assumed taking into account the type and condition of the soil. The pore pressure under the hydrostatic condition is

$$u = \gamma_w (z - h_w), \tag{9.22}$$

where γ_w = unit weight of water,

h_w = depth of water table from the ground surface.

As per Fig. 9.20, p_{om} is the pressure which corresponds to the volume v_0 at the start of the straight line portion of the curve. Since it has been found that it is very difficult to determine accurately p_{om}, p_{oh} may not be equal to p_{om}. As such, p_{om} bears no relation to what the true earth pressure at-rest is. In Eq. (9.21) K_0 has to be assumed and its accuracy is doubtful. In such circumstances it is not possible to calculate p_{oh}. However, p_{om} can be used for calculating the pressuremeter modulus E_m. The experience of many investigators is that a self-boring pressuremeter gives reliable values for p_{oh}.

The Pressuremeter Modulus E_m

Since the curve between points A and B in Fig. 9.20 is approximately a straight line, the soil in this region may be assumed to behave as a more or less elastic material. The equation for the pressuremeter modulus may be expressed as

$$E_p = 2G_s(1+\mu) = 2(1+\mu)V_m \frac{\Delta p}{\Delta v} \tag{9.23}$$

where G_s is the shear modulus.

If V_m is the volume at mid point (Fig 9.20), we may write,

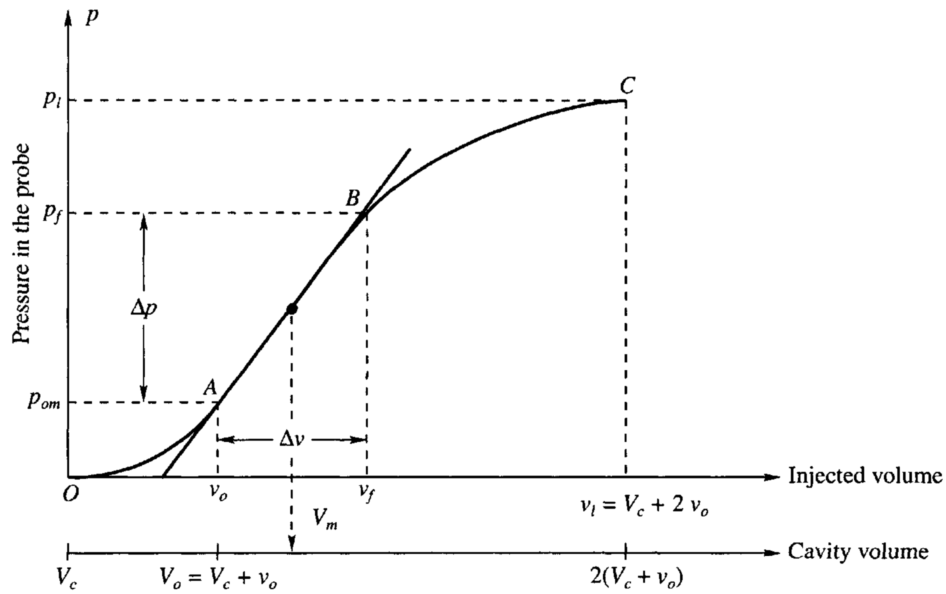


Figure 9.20 A typical corrected pressuremeter curve

$$V = V_m = V_c + \frac{v_0 + v_f}{2}, \tag{9.24}$$

where V_c is the volume of the deflated portion of the measuring cell at zero volume reading on the volumeter in the control unit.

Suitable values for μ may be assumed in the above equation depending on the type of soil. For saturated clay soils μ is taken as equal to 0.5 and for freely draining soils, the value is less. Since G_s (shear modulus) is not very much affected by a small variation in μ, Menard proposed a constant value of 0.33 for μ. As such the resulting deformation modulus is called Menard's Modulus E_m. The equation for E_m reduces to

$$E_m = 2.66 V_m \frac{\Delta p}{\Delta v} \tag{9.25}$$

The following empirical relationship has been established from the results obtained from pressuremeter tests. Undrained shear strength c_u as a function of the limit pressure $\overline{p}_l$ may be expressed as

$$c_u = \frac{\overline{p}_l}{9} \tag{9.26}$$

where $\overline{p}_l = p_l - p_{oh}$ and p_{oh} = total horizontal earth pressure for the at rest condition.

Amar and Jézéquel (1972) have suggested another equation of the form

$$c_u = \frac{\overline{p}_l}{10} + 25 \text{ kPa} \tag{9.27}$$

where both $\overline{p}_l$ and c_u are in kPa.

Example 9.11

A pressuremeter test was carried out at a site at a depth of 7 m below the ground surface. The water table level was at a depth of 1.5 m. The average unit weight of saturated soil is 17.3 kN/m^3. The corrected pressuremeter curve is given in Fig. Ex. 9.11 and the depleted volume of the probe is V_c = 535 cm^3. Determine the following.

(a) The coefficient of earth pressure for the at-rest condition

(b) The Menard pressuremeter modulus E_m

(c) The undrained shear strength c_u. Assume that $p_{oh} = p_{om}$ in this case

Solution

From Fig. Ex 9.11, $p_{oh} = p_{om} = 105$ kPa

The effective overburden pressure is

$$p'_0 = 17.3 \times 7 - 5.5 \times 9.81 = 67.2 \text{ kPa}$$

The effective horizontal pressure is

$$p'_{0h} = 105 - 5.5 \times 9.81 = 51.0 \text{ kPa}$$

(a) From Eq (9.21)

$$K_0 = \frac{p'_{oh}}{p'_o} = \frac{51.0}{67.2} = 0.76$$

(b) From Eq (9.25)

$$E_m = 2.66\, V_m \frac{\Delta p}{\Delta v}$$

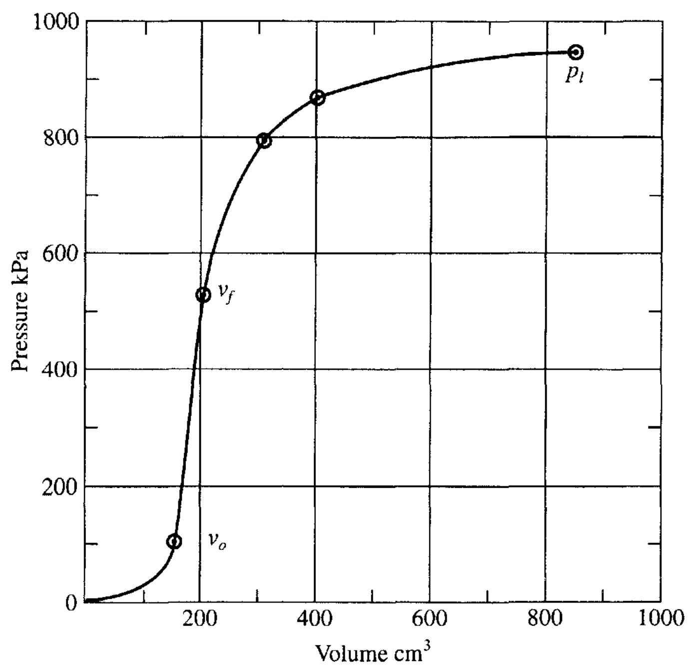


Figure Ex. 9.11

From Fig. Ex 9.11

$v_f = 200 \text{ cm}^3$ $\quad p_f = 530$ kPa

$v_o = 160 \text{ cm}^3$ $\quad p_{om} = 105$ kPa

From Eq (9.24) $V_m = 535 + \frac{200+160}{2} = 715 \text{cm}^3$

$$\frac{\Delta p}{\Delta v} = \frac{530-105}{200-160} = 10.625$$

Now $E_m = 2.66 \times 715 \times 10.625 = 20{,}208$ kPa

(c) From Eq (9.26)

$$c_u = \frac{\overline{p_l}}{9} = \frac{p_l - p_{oh}}{9}$$

From Fig Ex 9.11

$$\overline{p_l} = 950 - 105 = 845 \text{ kPa}$$

Therefore $c_u = \frac{845}{9} = 94$ kPa

From Eq (9.27)

$$c_u = \frac{\overline{p_l}}{10} + 25 = \frac{845}{10} + 25 = 109.5 \text{ kPa}$$

9.10 THE FLAT DILATOMETER TEST

The *flat dilatometer* is an *in-situ* testing device developed in Italy by Marchetti (1980). It is a penetration device that includes a lateral expansion arrangement after penetration. The test, therefore, combines many of the features contained in the cone penetration test and the pressuremeter test. This test has been extensively used for reliable, economical and rapid *in-situ* determination of geotechnical parameters. The flat plate dilatometer (Fig. 9.21) consists of a stainless steel blade with a flat circular expandable membrane of 60 mm diameter on one side of the stainless steel plate, a short distance above the sharpened tip. The size of the plate is 220 mm long, 95 mm wide and 14 mm thick. When at rest the external surface of the circular membrane is flush with the surrounding flat surface of the blade.

The probe is pushed to the required depth by making use of a rig used for a static cone penetrometer (Fig. 9.10). The probe is connected to a control box at ground level through a string of drill rods, electric wires for power supply and nylon tubing for the supply of nitrogen gas. Beneath the membrane is a measuring device which turns a buzzer off in the control box. The method of conducting the DMT is as follows:

1. The probe is positioned at the required level. Nitrogen gas is pumped into the probe. When the membrane is just flush with the side of the surface, a pressure reading is taken which is called the *lift-off* pressure. Approximate zero corrections are made. This pressure is called p_1.
2. The probe pressure is increased until the membrane expands by an amount $\Delta l = 1.1$ mm. The corrected pressure is p_2.

3. The next step is to decrease the pressure until the membrane returns to the lift off position. This corrected reading is p_3. This pressure is related to excess pore water pressure (Schmertmann, 1986).

The details of the calculation lead to the following equations.

1. Material index, $I_D = \dfrac{p_z - p_1}{p_2 - u}$ (9.28)

2. The lateral stress index, $K_D = \dfrac{p_1 - u}{p'_o}$ (9.29)

3. The dilatometer modulus, $E_D = 34.7(p_2 - p_1)$ kN/m^2 (9.30)

where, p'_o = effective overburden pressure = $\gamma' z$

u = pore water pressure equal to static water level pressure

γ' = effective unit weight of soil

z = depth of probe level from ground surface

The lateral stress index K_D is related to K_0 (the coefficient of earth pressure for the at-rest condition) and to OCR (overconsolidation ratio).

Marchetti (1980) has correlated several soil properties as follows

$$E_s = (1 - \mu^2)E_D \tag{9.31}$$

$$K_0 = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.6 \tag{9.32}$$

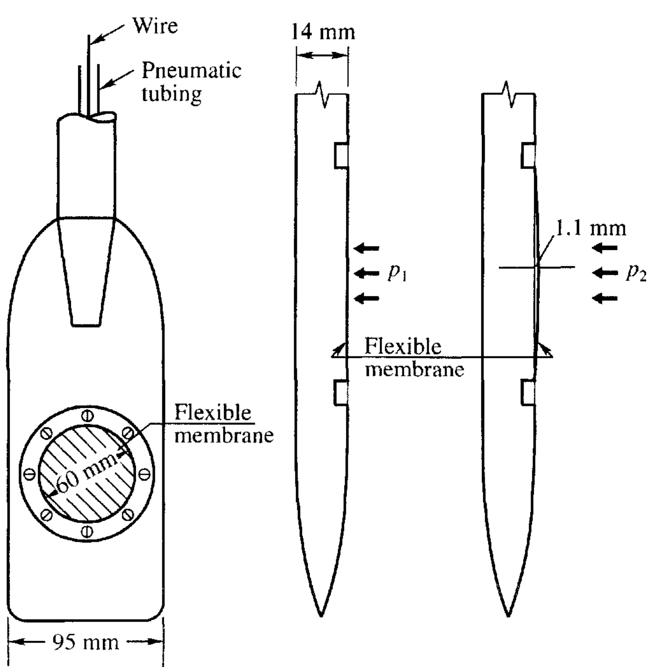


Figure 9.21 Illustration of a flat plate dilatometer (after Marchetti 1980)

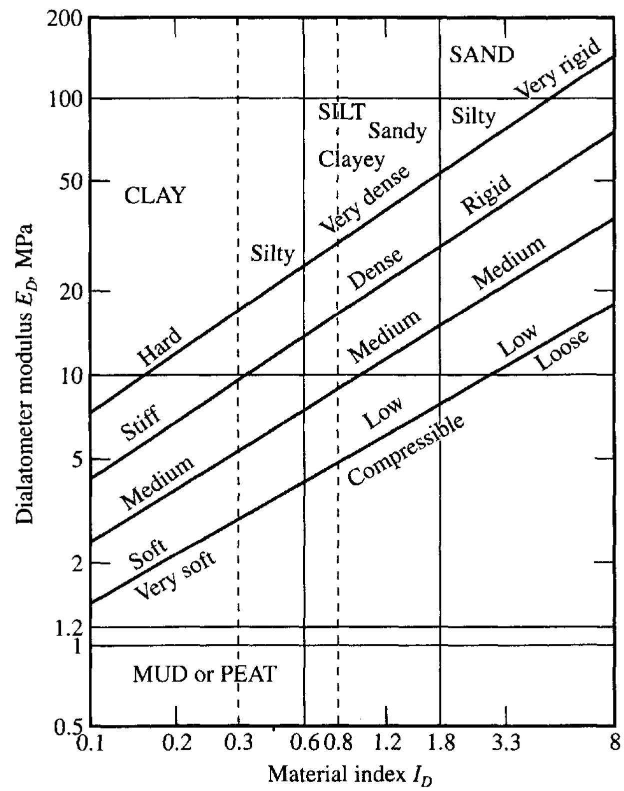


Figure 9.22 Soil profile based on dilatometer test (after Schmertmann, 1986)

$$OCR = (0.5K_D)^{1.6} \tag{9.33}$$

$$\left(\frac{c_u}{p'_o}\right)_{oc} = \left(\frac{c_u}{p'_o}\right)_{nc} \times (0.5K_D)^{1.25} \tag{9.34}$$

where E_s is the modulus of elasticity

The soil classification as developed by Schmertmann (1986) is given in Fig. 9.22. I_D is related with E_D in the development of the profile.

9.11 FIELD VANE SHEAR TEST (VST)

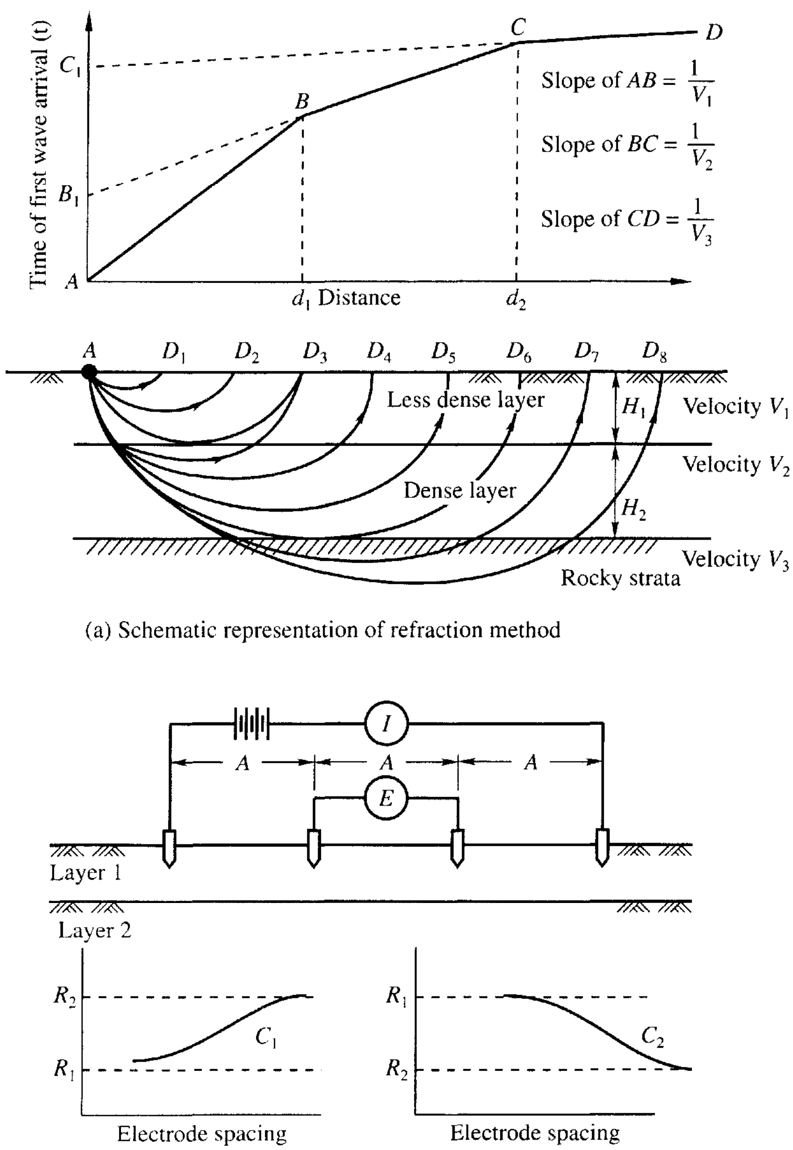
The vane shear test is one of the in-situ tests used for obtaining the undrained shear strength of soft sensitive clays. It is in deep beds of such material that the vane test is most valuable for the simple reason that there is at present no other method known by which the shear strength of these clays can be measured. The details of the VST have already been explained in Chapter 8.

9.12 FIELD PLATE LOAD TEST (PLT)

The field plate test is the oldest of the methods for determining either the bearing capacity or settlement of footings. The details of PLT are discussed under Shallow Foundations in Chapter 13.

9.13 GEOPHYSICAL EXPLORATION

The stratification of soils and rocks can be determined by geophysical methods of exploration which measure changes in certain physical characteristics of these materials, for example the magnetism, density, electrical resistivity, elasticity or a combination of these properties. However, the utility of these methods in the field of foundation engineering is very limited since the methods do not quantify the characteristics of the various substrata. Vital information on ground water conditions is usually lacking. Geophysical methods at best provide some missing information between widely spaced bore holes but they can not replace bore holes. Two methods of exploration which are some times useful are discussed briefly in this section. They are



(a) Schematic representation of refraction method

(b) Schematic representation of electrical resistivity method

Figure 9.23 Geophysical methods of exploration

1. Seismic Refraction Method,
2. Electrical Resistivity Method.

Seismic Refraction Method

The seismic refraction method is based on the fact that seismic waves have different velocities in different types of soils (or rock). The waves refract when they cross boundaries between different types of soils. If artificial impulses are produced either by detonation of explosives or mechanical blows with a heavy hammer at the ground surface or at shallow depth within a hole, these shocks generate three types of waves. In general, only compression waves (longitudinal waves) are observed. These waves are classified as either direct, reflected or refracted. Direct waves travel in approximately straight lines from the source of the impulse to the surface. Reflected or refracted waves undergo a change in direction when they encounter a boundary separating media of different seismic velocities. The *seismic refraction method* is more suited to shallow exploration for civil engineering purposes.

The method starts by inducing impact or shock waves into the soil at a particular location. The shock waves are picked up by geophones. In Fig. 9.23(a), point A is the source of seismic impulse. The points D_1 through D_8 represent the locations of the geophones or detectors which are installed in a straight line. The spacings of the geophones are dependent on the amount of detail required and the depth of the strata being investigated. In general, the spacing must be such that the distance from D_1 to D_8 is three to four times the depth to be investigated. The geophones are connected by cable to a central recording device. A series of detonations or impacts are produced and the arrival time of the first wave at each geophone position is recorded in turn. When the distance between source and geophone is short, the arrival time will be that of a direct wave. When the distance exceeds a certain value (depending on the thickness of the stratum), the refracted wave will be the first to be detected by the geophone. This is because the refracted wave, although longer than that of the direct wave, passes through a stratum of higher seismic velocity.

A typical plot of test results for a three layer system is given in Fig. 9.23(a) with the arrival time plotted against the distance source and geophone. As in the figure, if the source-geophone spacing is more than the distance d_1, which is the distance from the source to point B, the direct wave reaches the geophone in advance of the refracted wave and the time-distance relationship is represented by a straight line AB through the origin represented by A. If on the other hand, the source geophone distance is greater than d_1, the refracted waves arrive in advance of the direct waves and the time-distance relationship is represented by another straight line BC which will have a slope different from that of AB. The slopes of the lines AB and BC are represented by $1/V_1$, and $1/V_2$ respectively, where V_1 and V_2 are the velocities of the upper and lower strata respectively. Similarly, the slope of the third line CD is represented by $1/V_3$ in the third strata.

The general types of soil or rocks can be determined from a knowledge of these velocities. The depth H_1 of the top strata (provided the thickness of the stratum is constant) can be estimated from the formula

$$H_1 = \frac{d_1}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}} \tag{9.35a}$$

The thickness of the second layer (H_2) is obtained from

$$H_2 = 0.85 H_1 + \frac{d_2}{2}\sqrt{\frac{V_3 - V_2}{V_3 + V_2}} \tag{9.35b}$$

The procedure is continued if there are more than three layers.

If the thickness of any stratum is not constant, average thickness is taken.

Table 9.8 Range of seismic velocities in soils near the surface or at shallow depths (after Peck et al., 1974)

Material	Velocity ft/sec	Velocity m/sec
1. Dry silt, sand, loose gravel, loam, loose rock talus, and moist fine-grained top soil	600-2500	180-760
2. Compact till, indurated clays, compact clayey gravel, cemented sand and sand clay	2500-7500	760-2300
3. Rock, weathered, fractured or partly decomposed	2000-10,000	600-3000
4. Shale, sound	2500-11,000	760-3350
5. Sandstone, sound	5000-14,000	1500-4300
6. Limestone, chalk, sound	6000-20,000	1800-6000
7. Igneous rock, sound	12,000-20,000	3650-6000
8. Metamorphic rock, sound	10,000-16,000	3000-4900

The following equations may be used for determining the depths H_1 and H_2 in a three layer strata:

$$H_1 = \frac{t_1 V_1}{2\cos\alpha} \tag{9.36}$$

$$H_2 = \frac{t_2 V_2}{2\cos\beta} \tag{9.37}$$

where t_1 = AB_1, (Fig. 9.23a); the point B_1 is obtained on the vertical passing through A by extending the straight line CB,

t_2 = $(AC_1 - AB_1)$; AC_1 is the intercept on the vertical through A obtained by extending the straight line DC,

α = $\sin^{-1}(V_1/V_2)$,

β = $\sin^{-1}(V_2/V_3)$. (9.38)

α and β are the angles of refraction of the first and second stratum interfaces respectively.

The formulae used to estimate the depths from seismic refraction survey data are based on the following assumptions:

1. Each stratum is homogeneous and isotropic.
2. The boundaries between strata are either horizontal or inclined planes.
3. Each stratum is of sufficient thickness to reflect a change in velocity on a time-distance plot.
4. The velocity of wave propagation for each succeeding stratum increases with depth.

Table 9.8 gives typical seismic velocities in various materials. Detailed investigation procedures for refraction studies are presented by Jakosky (1950).

Electrical Resistivity Method

The method depends on differences in the electrical resistance of different soil (and rock) types. The flow of current through a soil is mainly due to electrolytic action and therefore depends on the concentration of dissolved salts in the pores. The mineral particles of soil are poor conductors of current. The resistivity of soil, therefore, decreases as both water content and concentration of salts increase. A dense clean sand above the water table, for example, would exhibit a high resistivity due to its low degree of saturation and virtual absence of dissolved salts. A saturated clay of high void ratio, on the other hand, would exhibit a low resistivity due to the relative abundance of pore water and the free ions in that water.

There are several methods by which the field resistivity measurements are made. The most popular of the methods is the Wenner Method.

Wenner Method

The Wenner arrangement consists of four equally spaced electrodes driven approximately 20 cm into the ground as shown in Fig. 9.23(b). In this method a *dc* current of known magnitude is passed between the two outer (current) electrodes, thereby producing within the soil an electric field, whose pattern is determined by the resistivities of the soils present within the field and the boundary conditions. The potential drop E for the surface current flow lines is measured by means of the inner electrodes. The apparent resistivity, R, is given by the equation

$$R = \frac{2\pi AE}{I} \tag{9.39}$$

It is customary to express A in centimeters, E in volts, I in amperes, and R ohm-cm. The apparent resistivity represents a weighted average of true resistivity to a depth A in a large volume of soil, the soil close to the surface being more heavily weighted than the soil at greater depths. The presence of a stratum of low resistivity forces the current to flow closer to the surface resulting in a higher voltage drop and hence a higher value of apparent resistivity. The opposite is true if a stratum of low resistivity lies below a stratum of high resistivity.

The method known as *sounding* is used when the variation of resistivity with depth is required. This enables rough estimates to be made of the types and depths of strata. A series of readings are taken, the (equal) spacing of the electrodes being increased for each successive reading. However, the center of the four electrodes remains at a fixed point. As the spacing is increased, the apparent resistivity is influenced by a greater depth of soil. If the resistivity increases with the increasing electrode spacings, it can be concluded that an underlying stratum of higher resistivity is beginning to influence the readings. If increased separation produces decreasing resistivity, on the other hand, a lower resistivity is beginning to influence the readings.

Apparent resistivity is plotted against spacing, preferably, on log paper. Characteristic curves for a two layer structure are shown in Fig. 9.23(b). For curve C_1, the resistivity of layer 1 is lower than that of 2; for curve C_2, layer 1 has a higher resistivity than that of layer 2. The curves become asymptotic to lines representing the true resistance R_1, and R_2 of the respective layers. Approximate layer thickness can be obtained by comparing the observed curves of resistivity versus electrode spacing with a set of standard curves.

The procedure known as *profiling* is used in the investigation of lateral variation of soil types. A series of readings is taken, the four electrodes being moved laterally as a unit for each successive reading; the electrode spacing remains constant for each reading of the series. Apparent resistivity is plotted against the center position of the four electrodes, to natural scale; such a plot can be used to locate the position of a soil of high or low resistivity. Contours of resistivity can be plotted over a given area.

The electrical method of exploration has been found to be not as reliable as the seismic method as the apparent resistivity of a particular soil or rock can vary over a wide range of values.

Representative values of resistivity are given in Table 9.9.

Table 9.9 Representative values of resistivity. The values are expressed in units of 10^3 ohm-cm (after Peck et al., 1974)

Material	Resistivity ohm-cm
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	150–4,000

Example 9.12

A seismic survey was carried out for a large project to determine the nature of the substrata. The results of the survey are given in Fig. Ex 9.12 in the form of a graph. Determine the depths of the strata.

Solution

Two methods may be used

1. Use of Eq (9.35)
2. Use of Eqs (9.36) and (9.37)

First we have to determine the velocities in each stratum (Fig. Ex. 9.12).

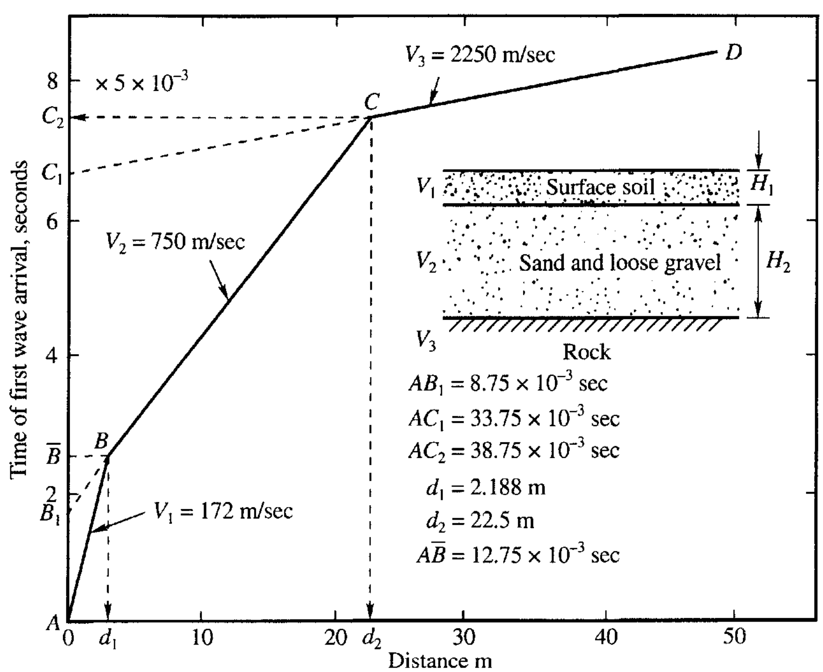


Figure Ex. 9.12

$$V_1 = \frac{\text{distance}}{A\overline{B}} = \frac{2.188}{12.75 \times 10^{-3}} = 172 \text{ m/sec}$$

$$V_2 = \frac{d_2}{AC_2 - AB_1} = \frac{22.5}{(7.75 - 1.75)5 \times 10^{-3}} = 750 \text{ m/sec}$$

In the same way, the velocity in the third stata can be determined. The velocity obtained is $V_3 = 2250$ m/sec

Method 1

From Eq (9.35 a), the thickness H_1 of the top layer is

$$H_1 = \frac{d_1}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}} = \frac{2.188}{2}\sqrt{\frac{750 - 172}{1000}} = 0.83 \text{ m}$$

From Eq (9.35b) the thickness H_2 is

$$H_2 = 0.85H_1 + \frac{d_2}{2}\sqrt{\frac{V_s - V_2}{V_s + V_2}}$$

$$H_2 = 0.85 \times 0.83 + \frac{22.5}{2}\sqrt{\frac{2250 - 750}{3000}}$$
$$= 0.71 + 7.955 = 8.67 \text{ m}$$

Method 2

From Eq (9.36)

$$H_1 = \frac{t_1 V_1}{2\cos\alpha}$$

$$t_1 = AB_1 = 1.75 \times 5 \times 10^{-3} \text{ sec(Fig. Ex.9.12)}$$

$$\alpha = \sin^{-1}\frac{V_1}{V_2} = \sin^{-1}\frac{172}{750} = 13.26°$$

$$\cos\alpha = 0.9733$$

$$H_1 = \frac{12.75 \times 10^{-3} \times 172}{2 \times 0.9737} = 1.13 \text{ m}$$

From Eq (9.37)

$$H_2 = \frac{t_2 V_2}{2\cos\beta}$$

$$t_2 = 5 \times 5 \times 10^{-3} \text{ sec}$$

$$\beta = \sin^{-1}\frac{750}{2250} = 19.47°; \cos\beta = 0.9428$$

$$H_2 = \frac{5 \times 5 \times 10^{-3} \times 750}{2 \times 0.9428} = 9.94 \text{m}$$

9.14 PLANNING OF SOIL EXPLORATION

The planner has to consider the following points before making a program:

1. Type, size and importance of the project.
2. Whether the site investigation is preliminary or detailed.

In the case of large projects, a preliminary investigation is normally required for the purpose of

1. Selecting a site and making a feasibility study of the project,
2. Making tentative designs and estimates of the cost of the project.

Preliminary site investigation needs only a few bore holes distributed suitably over the area for taking samples. The data obtained from the field and laboratory tests must be adequate to provide a fairly good idea of the strength characteristics of the subsoil for making preliminary drawings and design. In case a particular site is found unsuitable on the basis of the study, an alternate site may have to be chosen.

Once a site is chosen, a detailed soil investigation is undertaken. The planning of a soil investigation includes the following steps:

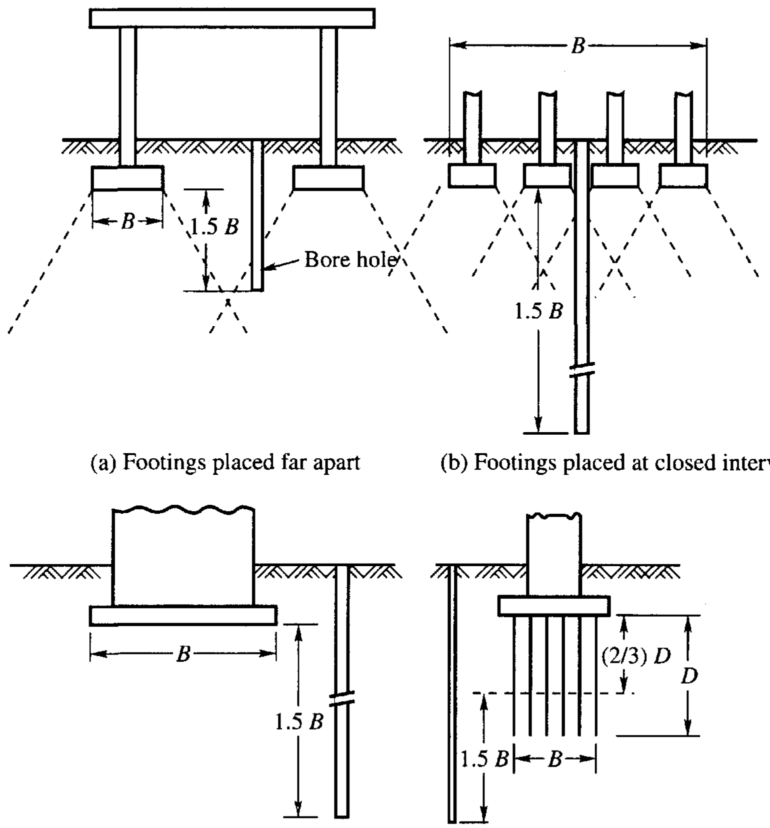
1. A detailed study of the geographical condition of the area which include
   (a) Collection of all the available information about the site, including the collection of existing topographical and geological maps,
   (b) General topographical features of the site,
   (c) Collection of the available hydraulic conditions, such as water table fluctuations, flooding of the site etc,
   (d) Access to the site.
2. Preparation of a layout plan of the project.
3. Preparation of a borehole layout plan which includes the depths and the number of bore holes suitably distributed over the area.
4. Marking on the layout plan any additional types of soil investigation.
5. Preparation of specifications and guidelines for the field execution of the various elements of soil investigation.
6. Preparation of specifications and guide lines for laboratory testing of the samples collected, presentation of field and laboratory test results, writing of report, etc.

The planner can make an intelligent, practical and pragmatic plan if he is conversant with the various elements of soil investigation.

Depths and Number of Bore Holes

Depths of Bore Holes

The depth up to which bore holes should be driven is governed by the depth of soil affected by the foundation bearing pressures. The standard practice is to take the borings to a depth (called the significant depth) at which the excess vertical stress caused by a fully loaded foundation is of the order of 20 per cent or less of the net imposed vertical stress at the foundation base level. The depth the borehole as per this practice works out to about 1.5 times the least width of the foundation from the base level of the foundation as shown in Fig. 9.24(a). Where strip or pad footings are closely spaced which results in the overlapping of the stressed zones, the whole loaded area becomes in effect a raft foundation with correspondingly deep borings as shown in Fig. 9.24(b) and (c). In the case of pile or pier foundations the subsoil should be explored to the depths required to cover the soil lying even below the tips of piles (or pile groups) and piers which are affected by the loads transmitted to the deeper layers, Fig. 9.24(d). In case rock is encountered at shallow depths, foundations may have to rest on rocky strata. The boring should also explore the strength characteristics of rocky strata in such cases.



(a) Footings placed far apart (b) Footings placed at closed intervals

(c) Raft foundation (d) Pile foundation

Figure 9.24 Depth of bore holes

Number of Bore Holes

An adequate number of bore holes is needed to

1. Provide a reasonably accurate determination of the contours of the proposed bearing stratum,
2. Locate any soft pockets in the supporting soil which would adversely affect the safety and performance of the proposed design.

The number of bore holes which need to be driven on any particular site is a difficult problem which is closely linked with the relative cost of the investigation and the project for which it is undertaken. When the soil is homogeneous over the whole area, the number of bore holes could be limited, but if the soil condition is erratic, limiting the number would be counter productive.

9.15 EXECUTION OF SOIL EXPLORATION PROGRAM

The three limbs of a soil exploration are

1. Planning,
2. Execution,
3. Report writing.

All three limbs are equally important for a satisfactory solution of the problem. However, the execution of the soil exploration program acts as a bridge between planning and report writing, and as such occupies an important place. No amount of planning would help report writing, if the field and laboratory works are not executed with diligence and care. It is essential that the execution part should always be entrusted to well qualified, reliable and resourceful geotechnical consultants who will also be responsible for report writing.

Deployment of Personnel and Equipment

The geotechnical consultant should have well qualified and experienced engineers and supervisors who complete the work per the requirements. The firm should have the capacity to deploy an adequate number of rigs and personnel for satisfactory completion of the job on time.

BOREHOLE LOG

Job No.	Date: 6-4-84
Project: Farakka STPP	BH No.: 1
	GL: 64.3 m
Location: WB	WTL: 63.0 m
Boring Method: Shell & Auger	Supervisor: X
Dia. of BH 15 cm	

Soil Type		Level m	Depth m	SPT 15 cm	15 cm	15 cm	N	Sample type	Remarks
Yellowish stiff clay			1.0	4	6	8	14	D U	
		62.3							
Greyish sandy silt med. dense			3.3	7	10	16	26	D W	
		59.8							
Greyish silty sand dense			5.0	14	16	21	37	D	
			7.5	15	18	23	41	D U	
		56.3							
Blackish very stiff clay			9.0	9	10	14	24	D	
		53.3	11.0						

D = disturbed sample; U = undisturbed sample;
W = water sample; N = SPT value

Figure 9.25 A typical bore-hole log

Boring Logs

A detailed record of boring operations and other tests carried out in the field is an essential part of the field work. The bore hole log is made during the boring operation. The soil is classified based on the visual examination of the disturbed samples collected. A typical example of a bore hole log is given in Fig. 9.25. The log should include the difficulties faced during boring operations including the occurrence of sand boils, and the presence of artesian water conditions if any, etc.

In-situ Tests

The field work may also involve one or more of the in-situ tests discussed earlier. The record should give the details of the tests conducted with exceptional clarity.

Laboratory Testing

A preliminary examination of the nature and type of soil brought to the laboratory is very essential before deciding upon the type and number of laboratory tests. Normally the SPT samples are used for this purpose. First the SPT samples should be arranged bore wise and depth wise. Each of the samples should be examined visually. A chart should be made giving the bore hole numbers and the types of tests to be conducted on each sample depth wise. An experienced geotechnical engineer can do this job with diligence and care.

Once the types of tests are decided, the laboratory assistant should carry out the tests with all the care required for each of the tests. The test results should next be tabulated on a suitable format bore wise and the soil is classified according to standard practice. The geotechnical consultant should examine each of the tests before being tabulated. Unreliable test results should be discarded.

Graphs and Charts

All the necessary graphs and charts are to be made based on the field and laboratory test results. The charts and graphs should present a clear insight into the subsoil conditions over the whole area. The charts made should help the geotechnical consultant to make a decision about the type of foundation, the strength and compressibility characteristics of the subsoil etc.

9.16 REPORT

A report is the final document of the whole exercise of soil exploration. A report should be comprehensive, clear and to the point. Many can write reports, but only a very few can produce a good report. A report writer should be knowledgable, practical and pragmatic. No theory, books or codes of practice provide all the materials required to produce a good report. It is the experience of a number of years of dedicated service in the field which helps a geotechnical consultant make report writing an art. A good report should normally comprise the following:

1. A general description of the nature of the project and its importance.
2. A general description of the topographical features and hydraulic conditions of the site.
3. A brief description of the various field and laboratory tests carried out.
4. Analysis and discussion of the test results
5. Recommendations
6. Calculations for determining safe bearing pressures, pile loads, etc.
7. Tables containing borelogs, and other field and laboratory test results
8. Drawings which include an index plan, a site-plan, test results plotted in the form of charts and graphs, soil profiles, etc.

9.17 PROBLEMS

9.1 Compute the area ratio of a sampling tube given the outside diameter = 100 mm and inside diameter = 94 mm. In what types of soil can this tube be used for sampling?

9.2 A standard penetration test was carried out at a site. The soil profile is given in Fig. Prob. 9.2 with the penetration values. The average soil data are given for each layer. Compute the corrected values of N and plot showing the

(a) variation of observed values with depth

(b) variation of corrected values with depth for standard energy 60%

Assume: $E_h = 0.7$, $C_d = 0.9$, $C_s = 0.85$ and $C_b = 1.05$

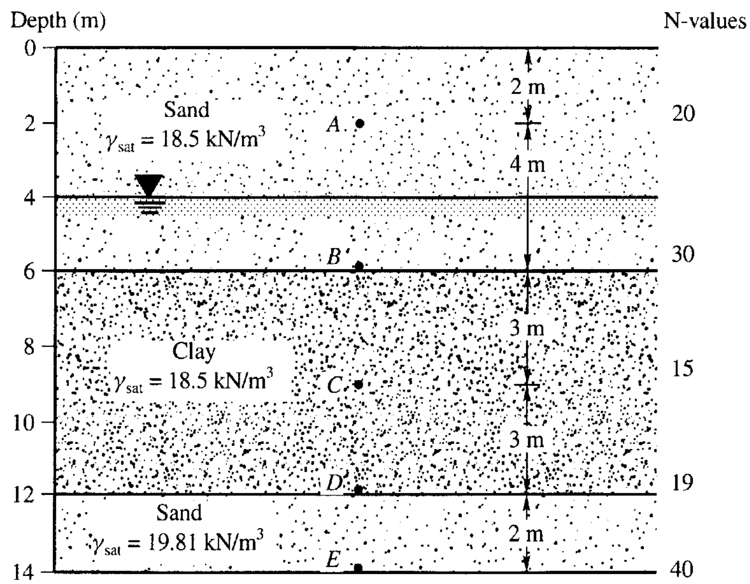


Figure Prob. 9.2

9.3 For the soil profile given in Fig. Prob 9.2, compute the corrected values of N for standard energy 70%.

9.4 For the soil profile given in Fig. Prob 9.2, estimate the average angle of friction for the sand layers based on the following:

(a) Table 9.3

(b) Eq (9.8) by assuming the profile contains less than 5% fines (D_r may be taken from Table 9.3)

Estimate the values of ϕ and D_r for 60 percent standard energy.

Assume: $N_{cor} = N_{60}$.

9.5 For the corrected values of N_{60} given in Prob 9.2, determine the unconfined compressive strengths of clay at points C and D in Fig Prob 9.2 by making use of Table 9.4 and Eq. (9.9). What is the consistency of the clay?

9.6 A static cone penetration test was carried out at a site using an electric-friction cone penetrometer. Fig. Prob 9.6 gives the soil profile and values of q_c obtained at various depths.

(a) Plot the variation of q_c with depth

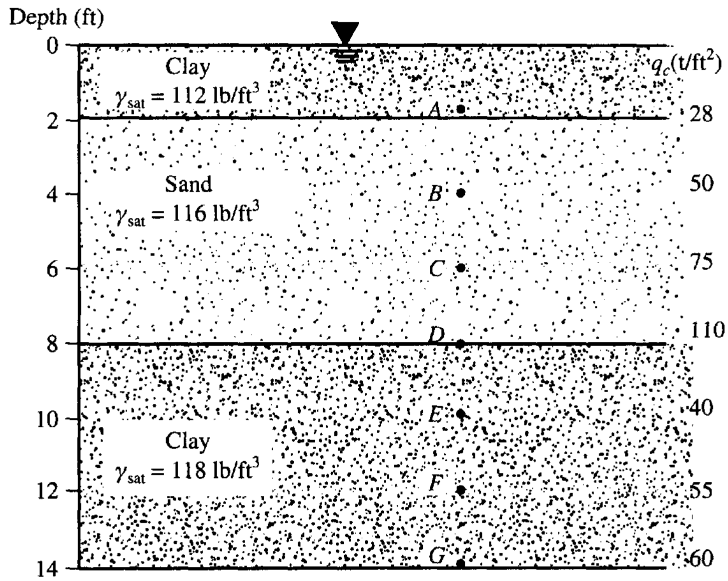


Figure Prob. 9.6

(b) Determine the relative density of the sand at the points marked in the figure by using Fig. 9.14.

(c) Determine the angle of internal friction of the sand at the points marked by using Fig. 9.15.

9.7 For the soil profile given in Fig. Prob 9.6, determine the unconfined compressive strength of the clay at the points marked in the figure using Eq (9.14).

9.8 A static cone penetration test carried out at a site at a depth of 50 ft gave the following results:

(a) cone resistance q_c = 250 t /ft^2

(b) average effective unit weight of the soil = 115 lb/ft^3

Classify the soil for friction ratios of 0.9 and 2.5 percent.

9.9 A static cone penetration test was carried out at a site using an electric-friction cone penetrometer. Classify the soil for the following data obtained from the site

q_c (MN/m^2)	Friction ratio R_f %
25	5
6.5	0.50
12.0	0.25
1.0	5.25

Assume in all the above cases that the effective overburden pressure is 50 kN/m^2.

9.10 Determine the relative density and the friction angle if the corrected SPT value N_{60} at a site is 30 from Eq (9.16) and Table 9.6. What are the values of D_r and ϕ for N_{70}?

9.11 Fig Prob 9.11 gives a corrected pressuremeter curve. The values of p_{om}, p_f and p_l and the corresponding volumes are marked on the curve. The test was conducted at a depth of 5 m below the ground surface. The average unit weight of the soil is 18.5 kN/m^3. Determine the following:

p_{om} = 200 kPa, v_o = 180 cm^3; p_f = 660 kPa; v_f = 220 cm^3;
p_l = 1100 kPa; v_l = 700 cm^3

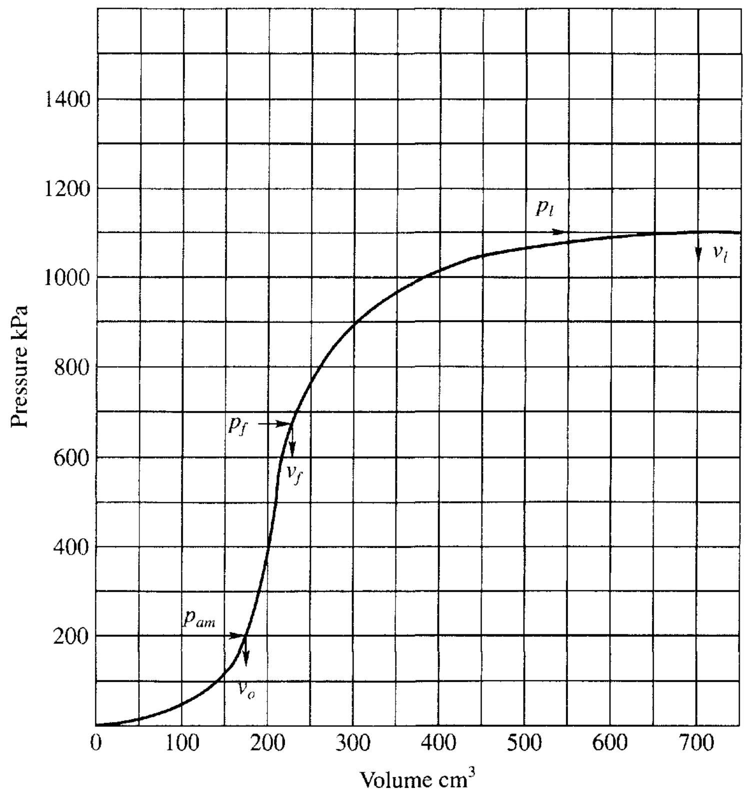


Figure Prob. 9.11

(a) The coefficient of earth pressure for the at-rest condition
(b) The Menard pressuremeter modulus
(c) The undrained shear strength c_u

9.12 A seismic refraction survey of an area gave the following data:

(i) Distance from impact point to geophone in m	15	30	60	80	100
(ii) Time of first wave arrival in sec	0.025	0.05	0.10	0.11	0.12

(a) Plot the time travel versus distance and determine velocities of the top and underlying layer of soil
(b) Determine the thickness of the top layer
(c) Using the seismic velocities evaluate the probable earth materials in the two layers