Geotechnical Engineering-II Sessional

CE 0732-4102

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Preface

Geotechnical Engineering is the specialty of Civil Engineering which deals with the property and behavior of soil and rock in engineering purposes. To obtain different properties of soil, laboratory tests are performed on collected disturbed and undisturbed soil samples, while field tests are performed on sub-soil at in-situ condition following mainly standard ASTM methods. In Bangladesh mainly wash boring method is adopted to make holes which are known as Bore Holes (BH) and from these holes, disturbed samples are collected from different layers by mainly split spoon sampler as well as undisturbed samples are collected by thin-walled tube/Shelby tube during soil exploration/investigation programme at the site. This Lab manual mainly deals with the common and standard laboratory tests on different types of soil. Field identification tests of soil and laboratory tests like- grain size analysis by sieve and hydrometer tests, specific gravity test, moisture content determination test, organic content determination test, Atterberg limits test, compaction test, relative density test and direct shear test may be performed on collected disturbed soil samples, whereas permeability test, unconfined compression test, tri-axial shear test and consolidation test will be performed on collected undisturbed soil samples according to ASTM (American Standards for Testing Materials) methods.

The authors are highly indebted to their colleagues for their constant support and guidance during the course of preparing this manual. In addition, concepts were taken from principles of geotechnical engineering book by BM Das and AUST manual, while the pictures were collected from the internet.

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Chapter 1

Soil Exploration

1.1 Introduction

1.1 Introduction

The object of site investigation is to obtain reliable, specific and detailed information about the soil/rock and groundwater conditions at a site for enabling engineers in the safe and economic design and execution of engineering works. To meet this objective investigation should be carried out to the required depth and horizontal extent in the region likely to be affected by the proposed constructions. The investigation should yield precise information about the following:

i. Order of occurrence and extent of soil/rock strata.
ii. Nature and engineering properties of the soil/rock strata.
iii. Location of groundwater table and its fluctuation.

Depth of investigation, in general, is decided based on the intensity of structured loading and the type of foundation contemplated. This depth up to which the increase in stress due to structural loading causes shear failure or excessive settlement of foundation is known as significant depth. This depth of investigation is generally taken as the depth of pressure bulb of intensity 0.1q where ‘q’ is the intensity of loading at the base of foundation. IS 1892 provides the following guidelines for depth of exploration for different types of foundations.

Table 1.1: Depth of exploration (IS: 1892-1979)

Sl no.	Type of foundation	Depth of exploration
1	Isolated spread footings or raft or adjacent footings with clear spacing equal or greater than four times the width	One and half times the width
2	Adjacent footings with clear spacing less than twice the width	One and half times the length

3	Adjacent rows of footings i. With clear spacing between rows less than twice the width ii. With clear spacing between rows greater than twice the width iii. With clear spacing between rows greater than four times the width	 Four and half times the width Three times the width One and half times the width
4	Pile and well foundations	One and half times the width of structure from bearing level (toe of pile or bottom of well)
5	Road cuts	Equal to the bottom width of the cut
6	Fill	Two meters below the ground level or equal to the height of the fill, whichever is greater

The member and spacing of borings/test pits depends on the type and size of foundations and extent of variation in soil conditions. IS 1892 makes the following recommendations:

i. For a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in each corner and one in the centre should be adequate
ii. For smaller and less important buildings even one bore hole or trail pit in the centre will suffice.
iii. For very large areas covering industrial and residential colonies, the geotechnical nature of the terrain will help in deciding the number of bore holes or trail pits.
iv. Cone penetration tests may be performed at every 50 m by dividing the area in a grid pattern and number of bore holes or trail pits decided by examining the variation in penetration curves. The cone penetration tests may not be possible at sites having gravelly or boulderous strata. In such cases geophysical methods may be suitable.

1.2 Boring of Holes

Making or drilling bore holes into the ground with a view to obtaining soil or rock samples from specified or known depths is called ‘boring’. The common methods of advancing bore holes are described below.

1.2.1 Auger Method

Soil auger’ is a device that is useful for advancing a bore hole into the ground. Augers may be hand-operated or power-driven; the former are used for relatively small depths (less than 3 to 5 m), while the latter are used for greater depths. The soil auger is advanced by rotating it while pressing it into the soil at the same time. It is used primarily in soils in which the bore hole can be kept dry and unsupported. As soon as the auger gets filled with soil, it is taken out and the soil sample collected.

1.2.1.1 Hand Operated Augers

The term boring refers to making or drilling holes into the ground for the purpose of obtaining samples or conducting insitu tests. 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 1.1. 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. 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 auguring is not practicable in dense sand nor in sand mixed with gravel even if the strata lie above the water table.

1.2.1.2 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.

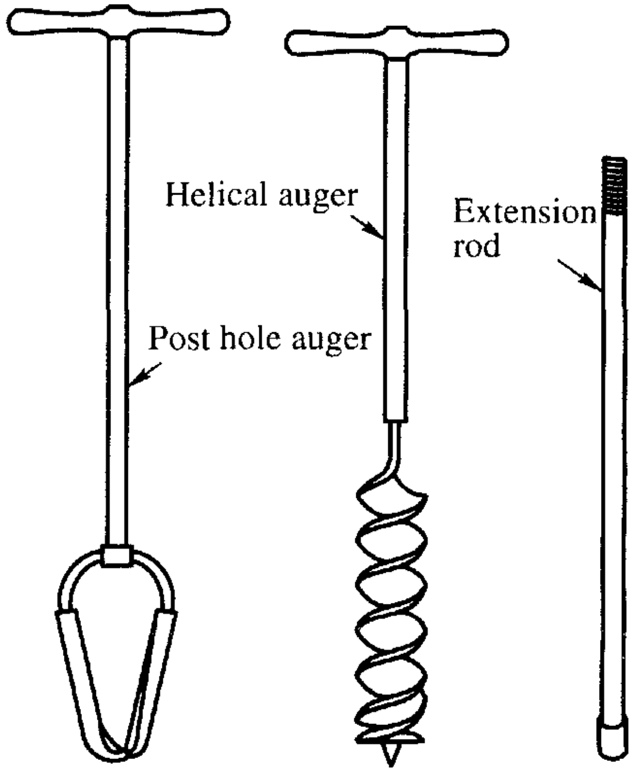


Figure 1.1 Hand augers

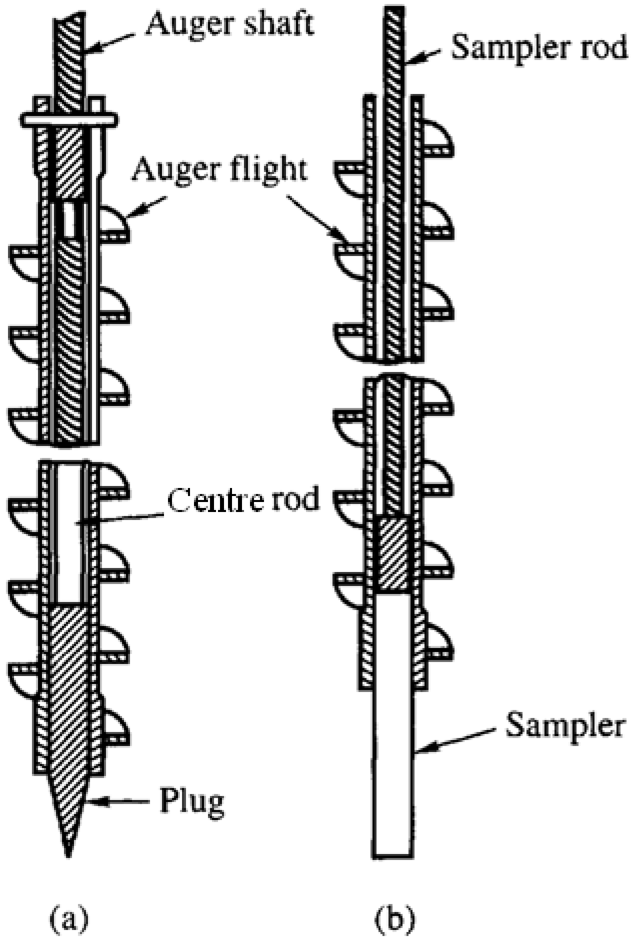


Figure 1.2 Hollow-stem auger

(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 as shown in Fig 1.2. 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.

1.2.1.3Wash 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 sand, silt, or clay. The method is not suitable if the soil is mixed with gravel or boulders. Fig 1.3 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 and 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. 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.

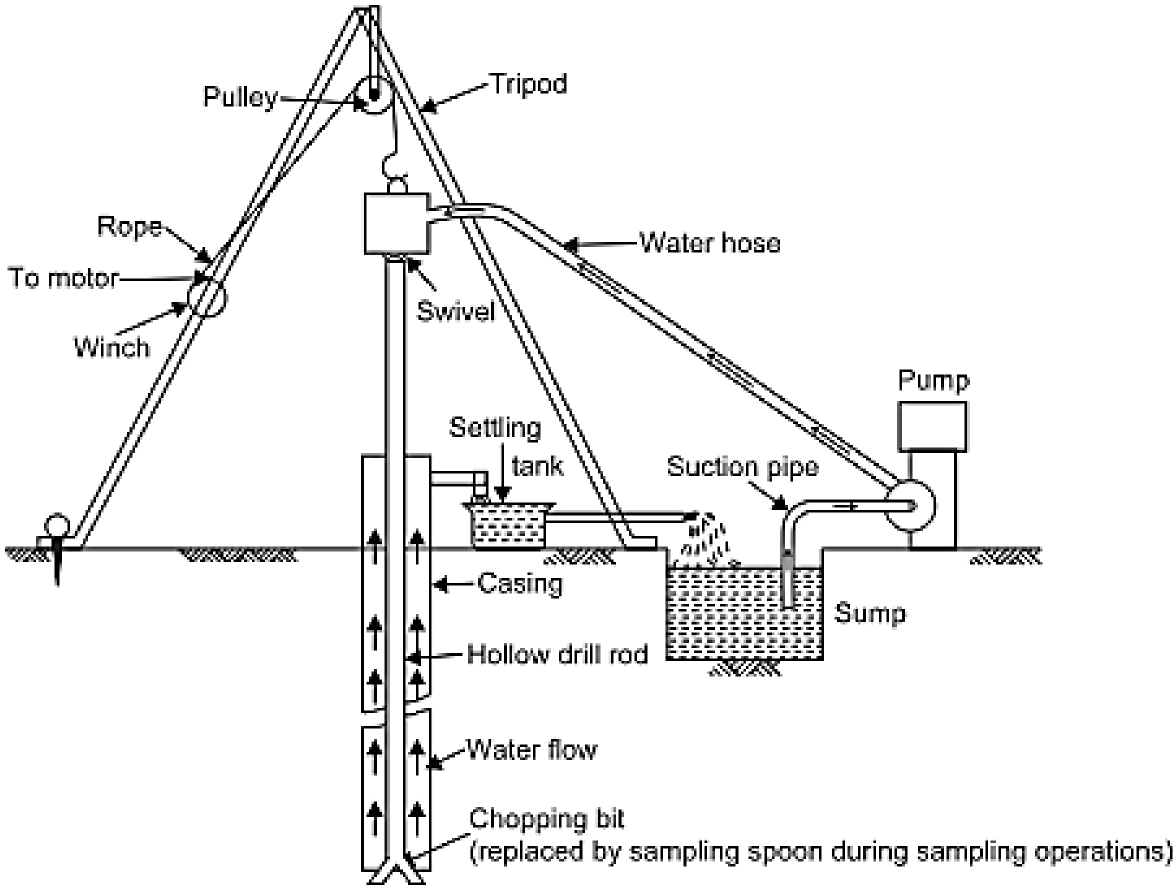


Fig 1.3: Wash boring

1.2.1.4 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 wash boring. Rotary drilling is used primarily for penetrating the overburden between the levels at 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. This 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 through 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 in soil, the drill 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.

1.2.1.5 Coring Bits

The three basic categories of coring bits in use are diamond, carbide insert, and saw tooth. Diamond coring bits may be of the surface set or diamond impregnated type. The most versatile of all coring bits are the diamond coring bits. This is because 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 such type are used to core soft to medium hard rock. Even though they are less expensive than diamond bits, the rate of drilling is slower than with diamond bits. The cutting edge comprises a series of teeth in saw tooth bits. The teeth are faced and tipped with a hard metal alloy such as tungsten carbide in order 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.

1.3Sampling of soils

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 forthe design are obtained. The accuracy of the soil parameters depends upon the accuracy with which representative soil samples are obtained from the field.

1.4Disturbed Samples

Auger samples may be used to identify soil strata and for field classifications 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. 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 1.4. 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 cohesion less soils below the water table cannot be retained in conventional sampling spoons without the addition of a spring core catcher.

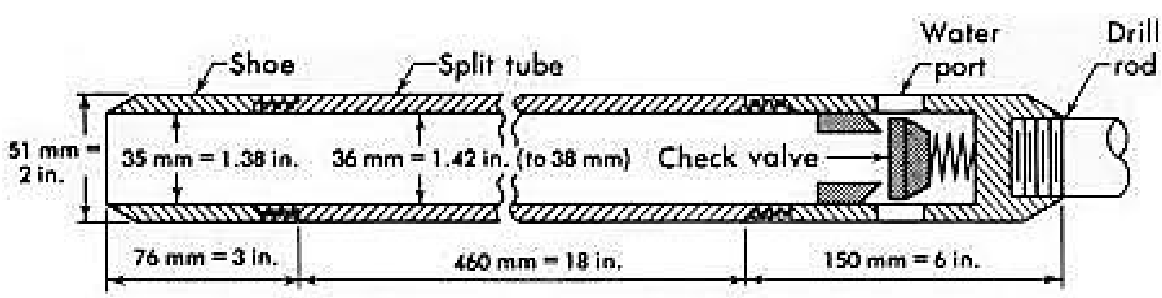


Fig: 1.4: Split spoon sampler

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.

1.4.1Open Drive Sampler

The wall thickness of the open drive samplerused 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 also called as Shelby tube sampler (Fig. 1.5), consists of a thin wall metal tube connected to a sampler head. The sampler head contains a ball check valve and ports which allows the escape of air or water 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 for this is governed by the size and wall thickness of the tube.

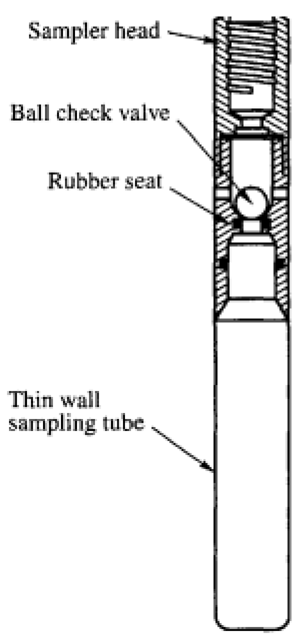


Fig. 1.5: Shelby tube sampler

The wall thickness is governed by the area ratio, A_r , which is defined as

$$A_r = \frac{d_o^{\,2} - d_i^{\,2}}{d_i^{\,2}}$$

d_o= outside diameter

d_i= inside diameter

A_r is a measure of the volume of the soil displacement to the volume of the collected sample well designed sampling tubes has an area ratio of about 10 percent. However, the area ratio may have to be much more than 10percent 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.

1.5Standard Penetration Test (SPT)

The SPT is the most commonly used in situ test in a bore hole. The test is made by making use of a split spoon sampler shown in Fig1.4. The method has been standardized as ASTM D-1586 in USA and IS 2131 in India. 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 has been drilled and cleaned in advance.
2. The sampler is driven into the soil strata to a maximum depth of 450 mm by making use of a 65 kg weight falling freely from a height of 75 cm 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 150 mm depths is counted to produce a total penetration of 450 mm.
4. To avoid seating errors, the blows required for the first 150 mm of penetration are not taken into account; blows required to only increase the penetration from 150 mm to 450 mm 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 1.5 m interval or at the change of stratum. The intervals may be increased at greater depths if necessary.

Corrections to the Observed SPT Value

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

1) Drill rod, sampler and borehole corrections
2) Correction due to overburden pressure
3) Hammer Efficiency Correction

1.5.1 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	**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, $C_s = 0.80$

Loose sand, $C_s = 0.90$

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

1.5.2Correction Factor for Overburden Pressure in Granular Soils, C_N

The $C_N$as per Liao and Whitman (1986) is

$$C_N = \left[\frac{95.76}{\rho'_0}\right]^{\frac{1}{2}}$$

……Eq 1.1

where, ρ'_0 = 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 1.1

N_{cor} may be expressed as

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

……Eq 1.2

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 1.2 $C_N N$ is the corrected value for overburden pressure only. The value of C_N as per Eq 1.1 is applicable for granular sols only, whereas C_N = 1 for cohesive soils for all depths.

1.5.3 Hammer Efficiency Correction

Different types of hammers are in use for driving the drill rods. Two types are normally used worldwide. 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, E_h= 0.7 to 0.8;

   Trip or automatic hammer, E_h = 0.8 to 1.0.

Table 1.2: 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 1.3: 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.

1.6Cone Penetration Test (CPT)

The static cone penetration test normally called the Dutch cone penetration test (CPT) 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 the adequate sample quality in marine environment.
3) The addition of other simultaneous measurements to the standard cone penetrometersuch as soil temperature and pore pressure.

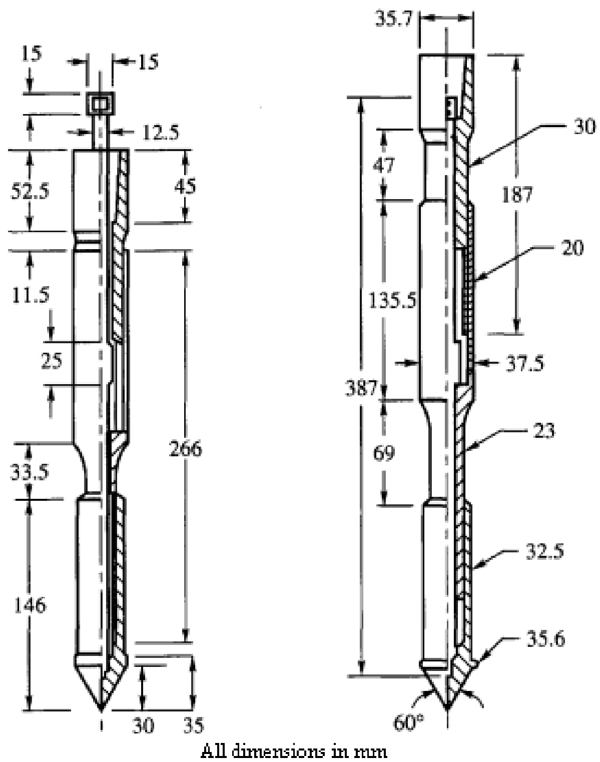


Fig 1.6: Standard cone Penetrometer

1.7Operation of Penetrometer

The sequence of operation of the penetrometer shown in Fig 1.7. is explained as follows:

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

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

Position 3: The sounding rod is pushed further to a depth b. This pushes the friction jacket and the cone assembly together; the force is Qt.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.

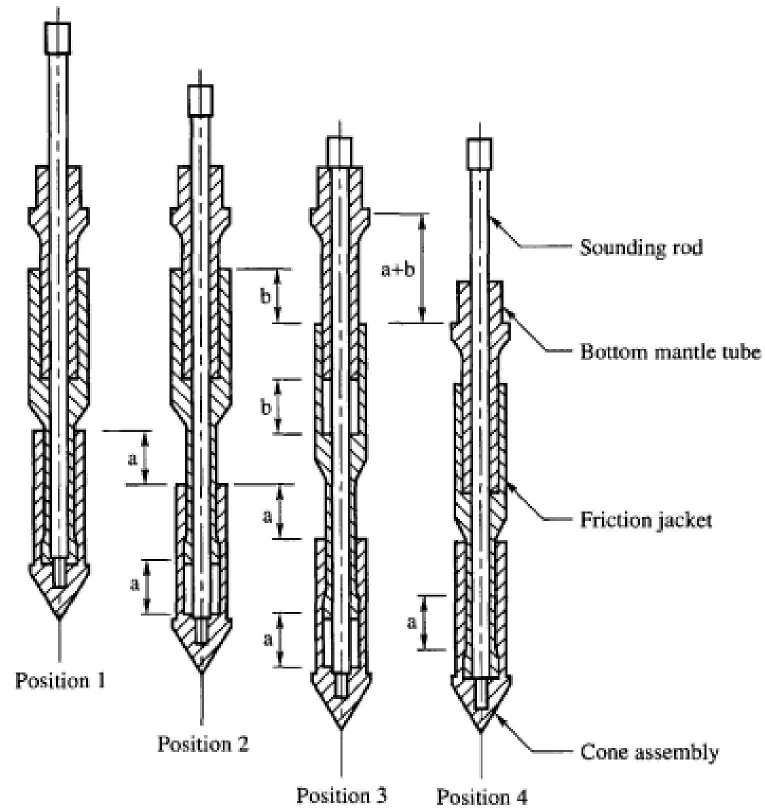


Fig 1.7: Operation of cone Penetrometer

Table 1.4: Soil classification based on friction ratio R_f (Sanglerat, 1972)

***Rf* (%)**	**Type of soil**
0 – 0.5	Loose gravel fill
0.5 – 2	Sands or gravels
2 - 5	Clay sand mixtures and silts
> 5	Clay, peats etc

1.8Correlation between SPT and CPT

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

$q_c = 0.4\ N\ MN/m^2$

His findings are given in Table 1.5

Table 1.5: 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	φ (°)
Very loose	< 0.2	< 4	< 2.0	< 30
Loose	0.2 – 0.4	4 - 10	2 – 5	30 - 35
Medium dense	0.4 – 0.6	10 – 30	5 – 10	35- 40
Dense	0.6 – 0.8	30 – 50	10 – 20	40 - 45
Very dense	0.8 – 1.0	>50	> 20	>45

1.9 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 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 cannot replace bore holes. Two methods of exploration which are sometimes useful are discussed briefly in this section. They are

1. Seismic Refraction Method,

2. Electrical Resistivity Method.

1.9.1Seismic Refraction Method

The seismic refraction method is based on the fact that seismic waves have different velocities in different types of soils (or rocks). 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 i.e., longitudinal waves are observed. These waves are classified as either direct, reflected or refracted waves. 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, a 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. 1.8(a), point A is the source of seismic impulse. The points D_1, D_2 ….. D_8represent the locations of the geophones or detectors which are installed in a straight line. The spacings of the geophones depend 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 around 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. 1.8(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_2 , 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.

The general types of soils or rocks can be determined from 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

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

…….Eq 1.3a

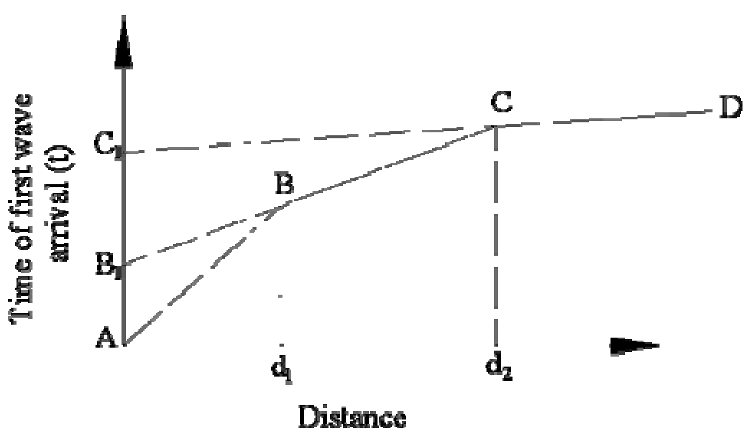
The thickness of the second layer (H_2) is obtained from

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

…….Eq 1.3b

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

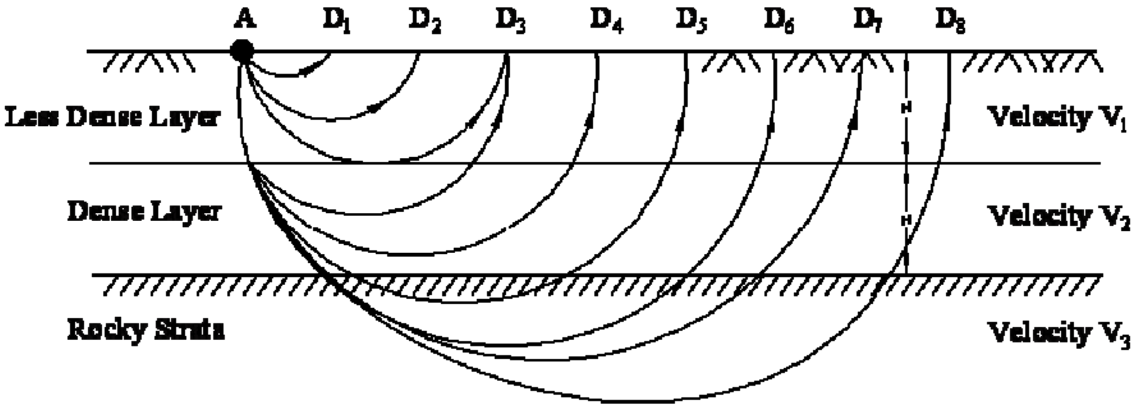
If the thickness of any stratum is not constant, average thickness is taken.



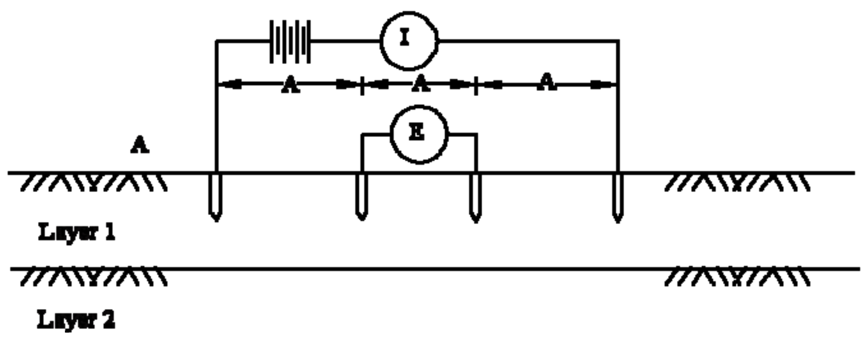
Slope of AB = $\frac{1}{V_1}$

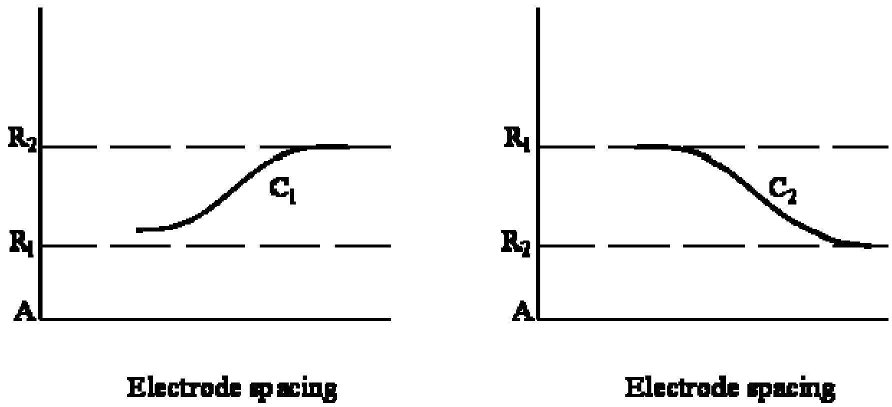
$$\text{Slope of BC} = \frac{1}{V_2}$$

$$\text{Slope of CD} = \frac{1}{V_3}$$



(a) Schematic Representation of seismic refraction method





(b) Schematic representation of electrical resistivity method

Fig 1.8: Geophysical methods

1.9.2Electrical Resistivity Method

This 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. 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. 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. 1.8(b). In this method a dc current of known magnitude is passed between the two outer (current) electrodes, thereby producing an electric field within the soil, whose pattern can be determined by the resistivities of the soils present within the field and the boundary conditions. By means of the inner electrodes the potential drop ‘E’ for the surface current flow lines is measured. The apparent resistivity ‘R’, is given by the equation

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

It is customary to express A in centimeters, E in volts, I in amperes, and R in 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 electrical 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 spacing, 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. 1.8(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 electrical 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 1.6.

Table 1.6: 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 - 4000

1.10 Soil 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 knowledgeable, 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 bore logs, and other field and laboratory test results
8. Drawings which include an index plan, a site-plan, test results plotted in the form of graphs and charts, soil profiles, etc.

1.11BoreholeLog

A borehole log is a record of information obtained from in situ tests and summary of laboratory tests on samples for a particular borehole. It includes description or classification of various soil / rock types at different depths with summary of essential properties including presence or otherwise of ground water table. A typical Borehole log is illustrated in Fig 1.9

Job No.	Date: 06-04-1984
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				Sample type	Remarks
				15 cm	15 cm	15 cm	N		
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

Fig 1.9: Typical borehole log

EXPERIMENT 2

DIRECT SHEAR TEST

Objective:

To determine the shearing strength of the soil using the direct shear apparatus.

Standards:

1. Indian Standards : IS: 2720 (Part-13)
2. ASTM: D-3080 (Granular soil)
3. AASHTO: T-236

Need and scope:

In many engineering problems such as design of foundation, retaining walls, slab bridges, pipes, sheet piling, the value of the angle of internal friction and cohesion of the soil involved are required for the design. Direct shear test is used to predict these parameters quickly.

Apparatus:

1. The shear box, grid plates, porous stones, base plates, and loading pad and water jacket shall conform to IS: 11229-19857.
    a. Shear box: Shear box of internal dimension 60 mm x 60 mm x 25 mm. Shear box, divided into two halves by a horizontal plane and fitted with locking and spacing screw.
    b. Base plate having cross grooves on its top surface
    c. Grid plates perforated (2 nos.)
    d. Porous stones 6 mm thick (2 nos.)
    e. Loading yoke, loading pad.
2. Loading frame (motor attached).
3. Dial gauge.
4. Proving ring.
5. Tamper.
6. Straight edge.
7. Balance.
8. Aluminum container.
9. Spatula.

Figure1: Direct shear test setup

Procedure:

1. Check the inner dimension of the soil container.
2. Put the parts of the soil container together.
3. Calculate the volume of the container. Weigh the container.
4. Place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil.
5. Weigh the soil container, the difference of these two is the weight of the soil. Calculate the density of the soil.
6. Make the surface of the soil plane.
7. Put the upper grating on stone and loading block on top of soil.
8. Measure the thickness of soil specimen.
9. Apply the desired normal load.
10. Remove the shear pin.
11. Attach the dial gauge which measures the change of volume.
12. Record the initial reading of the dial gauge and calibration values.

13. Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil.
14. Start the motor. Take the reading of the shear force and record the reading.
15. Take volume change readings till failure.
16. Add 5 kg normal stress 0.5 kg/cm2 and continue the experiment till failure
17. Record carefully all the readings. Set the dial gauges zero, before starting the experiment

Shearing stage

Rate of shearing ________ mm/min

Normal stress 0.5 kg/cm^2 L.C =....... P.R.C =.........

Date and Time	Displacement Dial Reading	Displacement, δ	Area Correction	Corrected Area	Proving dial reading	Shear Force	Shear Stress	Vertical Dial Reading	Vertical Dial Difference	Thickness of Specimen

Normal stress 1.0 kg/cm^2 L.C =....... P.R.C =........

Date and Time	Displacement Dial Reading	Displacement, δ	Area Correction	Corrected Area	Proving dial reading	Shear Force	Shear Stress	Vertical Dial Reading	Vertical Dial Difference	Thickness of Specimen

Normal stress 1.5 kg/cm^2 L.C=....... P.R.C=........

Date and Time	Displacement Dial Reading	Displacement, δ	Area Correction	Corrected Area	Proving dial reading	Shear Force	Shear Stress	Vertical Dial Reading	Vertical Dial Difference	Thickness of Specimen

Plot shear stress- shear displacement curve and find:

a) Maximum shear stress, and
b) Corresponding shear displacement.

Proving Ring constant.......   Least count of the dial........

Calibration factor.......

Leverage factor........

Dimensions of shear box= 60 x 60 mm

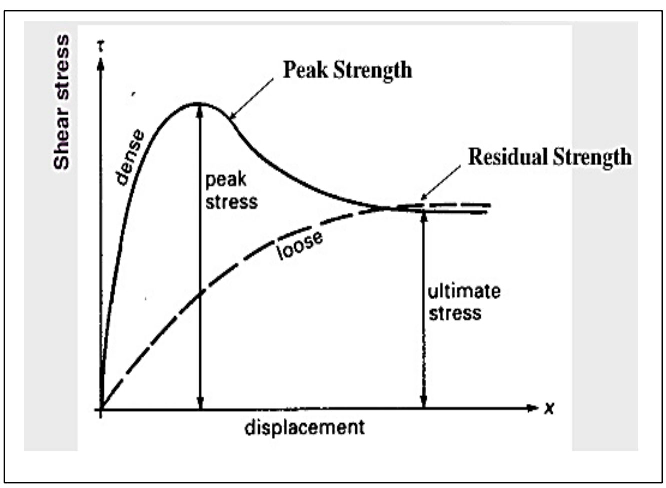
Empty weight of shear box........

Least count of dial gauge.........

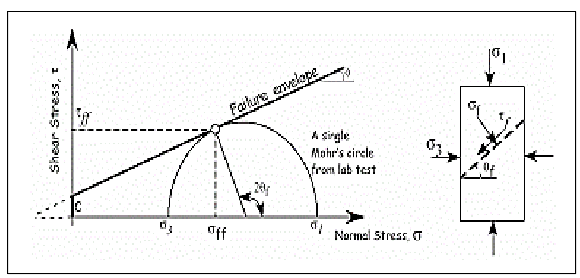
Normal Stress (kg/cm^2)	Shear Stress (kg/cm^2)			
	Proving Ring Reading (Division)	Proving Ring Constant	Shear Force (kg)	Shear Stress (kg/cm2)
0.5				
1				
1.5				

Plot shear normal stress displacement curve and find:

a) Cohesion intercept, and
b) Angle of shearing resistance.



Stress-strain plot



Mohr's circle for direct shear

Figure 1: Sample graphs from test data

Calculations:

1. Shear strength of soil

$$\tau_f = \sigma_n tan\emptyset + c$$

where,

τ_f = Shear strength of soil = shear stress at failure.
C = Cohesion intercepts.
σ_n = Total normal stress on the failure plane
$\emptyset$ = Angle of internal friction

2. Deformation (δ) = elapsed time× strain rate
3. Corrected Area

$$A = A_o\left(1 - \frac{\delta}{3}\right)$$

where,

A = Corrected area (cm^2)
A_o = Initial area of specimen (cm^2)
δ = Displacement

4. Normal load = normal weight added + weight of yoke
5. Normal stress

$$\sigma = \frac{\text{Normal load}}{\text{Initial area of specimen}}$$

6. Shear stress

$$\tau = \frac{\text{Shear load}}{\text{Corrected area}}$$

Result:

Angle of internal friction (Ø):

Cohesion (c):

General Remarks:

1. In the shear box test, the specimen is not failing along its weakest plane but along a predetermined or induced failure plane i.e. horizontal plane separating the two halves of the shear box. This is the main drawback of this test. Moreover, during loading, the state of stress

cannot be evaluated. It can be evaluated only at failure condition i.e Mohr's circle can be drawn at the failure condition only. Also failure is progressive.

2. Direct shear test is simple and faster to operate. As thinner specimens are used in shear box, they facilitate drainage of pore water from a saturated sample in less time. This test is also useful to study friction between two materials – one material in lower half of box and another material in the upper half of box.
3. The angle of shearing resistance of sands depends on state of compaction, coarseness of grains, particle shape and roughness of grain surface and grading. It varies between 28^o (uniformly graded sands with round grains in very loose state) to 46^o (well graded sand with angular grains in dense state).
4. The volume change in sandy soil is a complex phenomenon depending on gradation, particle shape, state and type of packing, orientation of principal planes, principal stress ratio, stress history, magnitude of minor principal stress, type of apparatus, test procedure, method of preparing specimen etc. In general loose sands expand and dense sands contract in volume on shearing. There is a void ratio at which either expansion contraction in volume takes place. This void ratio is called critical void ratio. Expansion or contraction can be inferred from the movement of vertical dial gauge during shearing.
5. The friction between sand particles is due to sliding and rolling friction and interlocking action.
6. The ultimate values of shear parameter for both loose sand and dense sand approximately attain the same value so, if angle of friction value is calculated at ultimate stage, slight disturbance in density during sampling and preparation of test specimens will not have much effect.

Questionaire:

EXPERIMENT - 3

UNCONFINED COMPRESSION TEST

Objective:

To determine the unconfined compressive strength of the given cohesive soil.

Standards:

1. Indian Standards : IS: 2720 (Part-10)
2. ASTM: D-2166
3. AASHTO: T-208

Need and scope of the experiment:

It is not always possible to conduct the bearing capacity test in the field. Sometimes it is cheaper to take the undisturbed soil sample and test its strength in the laboratory. Also to choose the best material for the embankment, one has to conduct strength tests on the samples selected. Under these conditions it is easy to perform the unconfined compression test on undisturbed and remoulded soil sample.

The unconfined compression test is inappropriate for dry sands or crumbly clays because the materials would fall apart without lateral confinement. A cylindrical soil specimen is subjected to gradually increasing axial stress until it fails. Since the test is quick, water is not allowed to drain out of the sample. Hence it is also called undrained or 'quick' test. Since the test produces only one mohr's circle (corresponding to $\sigma_3 = 0$), the test is applicable only to soils for which $\varphi_u = 0$, i.e, fully saturated, non-fissured clay.

For $\sigma_3 = 0$ $\quad \sigma_{1f} = 2c_u\sqrt{\frac{1+sin\emptyset_u}{1-sin\emptyset_u}}$

The subscript u is used since the test is an undrained test.

Since $\emptyset_u = 0$ $\quad \sigma_{1f} = 2c_u$

In the unconfined compression test, the major principal stress at failure, σ_{1f} is called the unconfined compressive strength and is usually denoted by the notation q_u.

Hence, $\quad q_u = 2c_u$

The undrained shear strength of saturated clay is expressed as,

$$\tau_f = c_u = \frac{q_u}{2}$$

Apparatus:

1. Loading frame of capacity of 2t, with constant rate of movement.
2. Proving ring of 0.01 kg sensitivity for soft soils; 0.05 kg for stiff soils.
3. Soil trimmer.
4. Split mould: 38 mm diameter, 76 mm long.
5. Frictionless end plates (Perspex plate with silicon grease coating).
6. Evaporating dish (Aluminum container).
7. Soil sample of 75 mm length.
8. Dial gauge (0.01 mm accuracy).
9. Balance of capacity 200 g and sensitivity to weigh 0.01 g.
10. Oven thermostatically controlled with interior of non-corroding material to maintain the temperature at the desired level.
11. Sample extractor and split sampler.
12. Vernier calipers

Figure 1: Experimental setup for unconfined compression test

Sample preparation:

1. **Specimen size:** The specimen for the test shall have a minimum diameter of 38 mm and the largest particle contained within the test specimen shall be smaller than 1/8 of the specimen diameter. If after completion of test on undisturbed sample, it is found that larger particles than permitted for the particular specimen size tested are present, it shall be noted in the report of test data under remarks. The height to diameter ratio shall be within 2 to 2.5.
   Measurements of height and diameter shall be made with vernier calipers or any other suitable measuring device to the nearest 0.1 mm.
2. **Undisturbed specimen**
    1. Note down the sample number, borehole number and the depth at which the sample was taken.
    2. Remove the protective cover (paraffin wax) from the sampling tube.
    3. Place the sampling tube extractor and push the plunger till a small length of sample moves out.
    4. Trim the projected sample using a wire saw.
    5. Again, push the plunger of the extractor until a 75 mm long sample comes out.
    6. Cutout this sample carefully and hold it on the split sampler so that it does not fall.
    7. Take about 10 to 15 g of soil from the tube for water content determination.
    8. Note the container number and take the net weight of the sample and the container.
    9. Measure the diameter at the top, middle, and the bottom of the sample and find the average and record the same.
    10. Measure the length of the sample and record.
    11. Find the weight of the sample and record.
3. **Remoulded sample**
    1. For the desired water content and the dry density, calculate the weight of the dry soil Ws required for preparing a specimen of 3.8 cm diameter and 7.5 cm long.
    2. Add required quantity of water W_w to this soil.

       $W_w = (W_S * W/100)$ g
    3. Mix the soil thoroughly with water.

4. Place the wet soil in a tight thick polythene bag in a humidity chamber and place the soil in a constant volume mould, having an internal height of 7.5 cm and internal diameter of 3.8 cm.
5. After 24 hours take the soil from the humidity chamber and place the soil in a constant volume mould, having an internal height of 7.5 cm and internal diameter of 3.8 cm.
6. Place the lubricated moulded with plungers in position in the load frame.
7. Apply the compressive load until the specimen is compacted to a height of 7.5 cm.
8. Eject the specimen from the constant volume mould.
9. Record the correct height, weight and diameter of the specimen.

Test procedure:

1. Place the sampling soil specimen at the desired water content and density in the large mould.
2. Push the sampling tube into the large mould and remove the sampling tube filled with the soil. For undisturbed samples, push the sampling tube into the clay sample.
3. Saturate the soil sample in the sampling tube by a suitable method.
4. Coat the split mould lightly with a thin layer of grease and weigh the mould.
5. Extrude the sample out of the sampling tube into the split mould, using the sample extractor and the knife.
6. Trim the two ends of the specimen in the split mould. Weigh the mould with the specimen.
7. Remove the specimen from the split mould by splitting the mould into two parts.
8. Measure the length and diameter of the specimen with vernier calipers.
9. Place the specimen on the bottom plate of the compression machine. Adjust the upper plate to make contact with the specimen.
10. Adjust the dial gauge and the proving ring gauge to zero.
11. Apply the compression load to cause an axial strain at the rate of ½ to 2% per minute.
12. Record the dial gauge reading, and the proving ring reading every thirty seconds upto a strain of 6%. The reading may be taken after every 60 seconds for a strain between 6%, 12% and every 2 minutes or so beyond 12%.
13. Continue the test until failure surfaces have clearly developed or until an axial strain of 20% is reached.
14. Measure the angle between the failure surface and the horizontal, if possible.

15. Take the sample from the failure zone of the specimen for the water content determination.
16. The values of compressive stress σ and strain ϵ obtained are plotted on a natural graph along Y-axis and X-axis respectively.
17. The maximum stress from this plot gives the value of the unconfined compressive strength (q_u).
18. In case no maximum occurs within 20 percent axial strain, the unconfined compressive strength shall be taken as the stress at 20 percent axial strain.

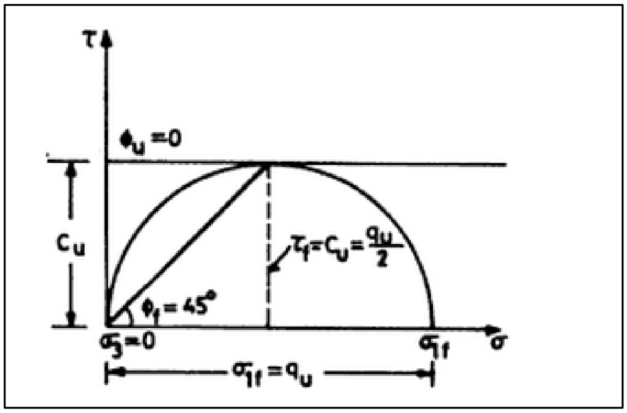


Figure 2: Mohr-Coulomb plot for unconfined compression test

Calculations:

1. Axial Strain (e):

$$\varepsilon = \frac{\text{Change in length } (\Delta L)}{\text{Original length of specimen } (L_O)}$$

2. Average Cross sectional area (A)

$$A = \frac{A_o}{1 - \varepsilon}$$

Where, A_o is the original cross-sectional area of the specimen

3. Compressive stress (σc)

$$\sigma_c = \frac{P}{A}$$

Where, P is compressive force.

A is average cross sectional area.

Observations:

Specific gravity (G_S) = Bulk density = kN/m^3

Water content=   Degree of saturation = %

Diameter (D_o) of the sample = cm   Area of cross-section = cm^2

Initial length (L_o) of the sample = cm

Sample 1: Proving ring number/constant:

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain e	Corrected area A (cm^2)	Proving ring dial reading	Axial force P (kg)	Compressive stress (kg/cm^2)

Sample 2:

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain e	Corrected area A (cm^2)	Proving ring dial reading	Axial force P (kg)	Compressive stress (kg/cm^2)

Sample 3:

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain e	Corrected area A (cm^2)	Proving ring dial reading	Axial force P (kg)	Compressive stress (kg/cm^2)

Result:

Unconfined compressive strength (q_u):

Undrained shear strength (τ_f):

Failure pattern:

Water content in specimen at testing:

Sensitivity = (q_u for undisturbed sample)/(q_u for remoulded sample):

Safety and precautions:

1. The specimen shall be handled carefully to prevent disturbance, change in cross section, or loss of water.
2. The specimen shall be of uniform circular cross-section with ends perpendicular to the axis of the specimen.
3. Where the prevention of the possible development of appreciable capillary forces is required, the specimens shall be sealed with rubber membranes, thin plastic coatings, or with coating or grease or sprayed plastic immediately after preparation and during the entire testing cycle.
4. Representative sample cuttings taken from the tested specimen shall be used for the determination of water content.

Questionnaire:

1. What is the difference between uniaxial compression and unconfined compression?
2. Is it suitable to conduct the above test for all types of soil?

EXPERIMENT - 4

TRIAXIAL SHEAR TEST

Objective:

To determine shear strength parameters of the given soil sample by conducting unconsolidated undrained (UU) triaxial shear test.

Standards:

1. Indian Standards : IS: 2720 (Part-11)
2. ASTM: D-2850
3. AASHTO: T-234

Theory:

The triaxial compression test, introduced by Casagrande and Terzaghi in 1936, is by far the most popular and extensively used shearing strength test, both for field application and for purposes of research. As the name itself suggests, the soil specimen is subjected to three compressive stresses in mutually perpendicular directions, one of the three stresses being increased until the specimen fails in shear. Usually a cylindrical specimen with a height equal to twice its diameter is used. The desired three-dimensional stress system is achieved by an initial application of all-round fluid pressure or confining pressure through water. While this confining pressure is kept constant throughout the test, axial or vertical loading is increased gradually and at a uniform rate. The axial stress thus constitutes the major principal stress and the confining pressure acts in the other two principal directions, the intermediate and minor principal stresses being equal to the confining pressure.

The apparatus consists of a lucite or perspex cylindrical cell, called 'triaxial cell' with appropriate arrangements for an inlet of cell fluid and application of pressure by means of a compressor, outlet of pore water from the specimen if it is desired to permit drainage which otherwise may serve as pore pressure connection and axial loading through a piston and loading cap.

The soil sample is placed inside a rubber sheath ,which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

The triaxial compression test consists of two stages:

(i) **First stage:** In this, a soil sample is set in the triaxial cell and confining pressure is then applied.

(ii) **Second stage:** In this, additional axial stress (also called deviator stress) is applied which induces shear stresses in the sample. The axial stress is continuously increased until the sample fails.

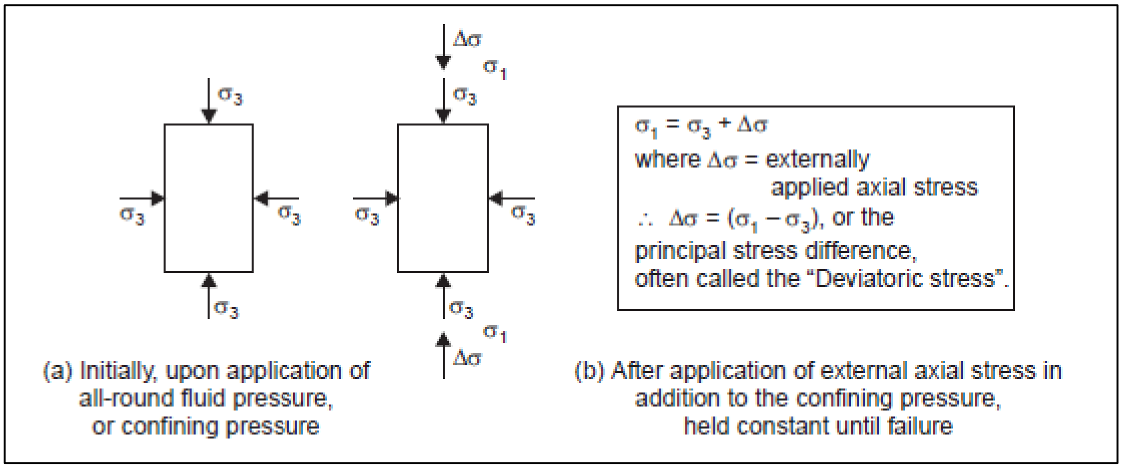
During both the stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

Test types:

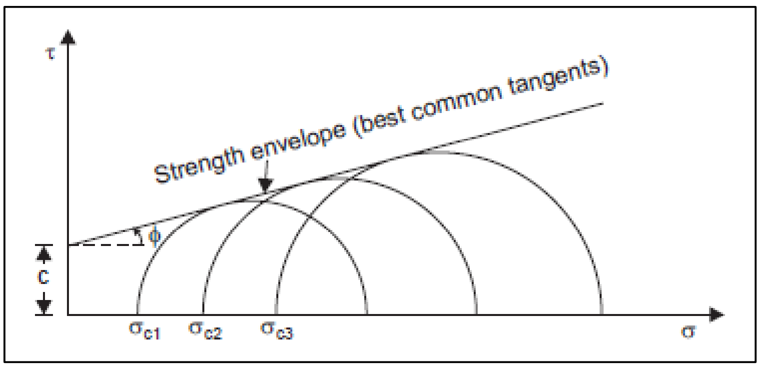
There are several test variations, and those used mostly in practice are:

1. **UU (unconsolidated undrained) test:** In this, cell pressure is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage.
2. **CU (consolidated undrained) test:** In this, drainage is allowed during cell pressure application. Then without allowing further drainage, deviator stress is increased keeping cell pressure constant.
3. **CD (consolidated drained) test:** This is similar to CU test except that as deviator stress is increased, drainage is permitted. The rate of loading must be slow enough to ensure no excess pore water pressure develops.

In the UU test, if pore water pressure is measured, the test is designated by $\overline{UU}$.
In the CU test, if pore water pressure is measured in the second stage, the test is symbolized as $\overline{CU}$.



Stresses on soil element under triaxial testing



Mohr's circle plot for triaxial compression test

Apparatus:

1. Triaxial testing machine complete with triaxial cell
2. Water pressure unit with hand pump
3. Proving ring
4. Dial gauge
5. Rubber membranes
6. Membrane stretcher
7. Sample trimming apparatus
8. Bins for moisture content determinations
9. Balance and box of weights
10. Drying oven

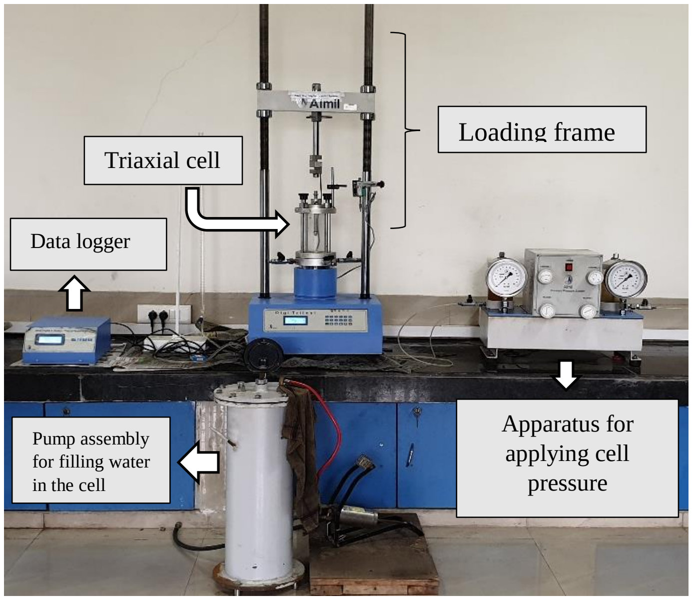


Figure 1: Triaxial test setput

Sample preparation:

Undisturbed specimens:

The object of the specimen preparation is to produce cylindrical specimens of height twice the specimen diameter with plane ends normal to the axis and with the minimum change of the soil structure and moisture content. The method of preparation will depend on whether the sample is received in the laboratory in a tube or as a block sample.

Remoulded samples:

Remoulded samples prepared at the desired moisture and density by static and dynamic methods of compaction or by any other suitable method, where necessary.

Experimental procedure:

1. Trim the soil specimen (prepared from the sampling tube of an undisturbed sample tube using universal extractor frame or from a compacted soil specimen as per standard proctors method, at optimum moisture content or any other moisture content to suite the field situations).
2. Using the trimming apparatus if necessary the trimmed specimen should be 76.2 mm long and 38.1 mm in diameter. The diameter and the length are measured at not less than 3 places and the average values are used for computation.
3. Note the weight of the specimen (W1).

4. The specimen is then enclosed in a 38.1 mm diameter and about 100 mm long rubber membrane, using the membrane stretcher. Spreading back the ends of the membrane over the ends of the stretcher and applying suction between the stretcher and the rubber membranes does by inhalation.
5. The membrane and stretcher are then easily slide over the specimen, the suction is released and membrane is unrolled from the ends of the stretcher.
6. Use non-porous stones on either side of the specimen as neither any pressure is to be measured nor any drainage of air or water is allowed.
7. Remove the porous cylinder from its base removing the bottom fly nuts.
8. The pedestal at the centre of the base of the cylinder on which the specimen is to be placed is cleaned and a 38.1 mm diameter rubber O-ring is rolled over to its bottom.
9. The specimen along with the non-porous plate on either side is centrally placed over the pedestal and the bottom edge of the machine covering the specimen is sealed against the pedestal by rolling back the O-ring over the membrane.
10. The cap is placed over the top plate of the specimen and the top of the rubber membrane is sealed against the cap by carefully rolling over it another O-ring. This arrangement of rubber O-ring forms the effective seal between the specimen with the membrane and the water under pressure.
11. The specimen is checked for its verticality and co-axiality with the cylinder chamber.
12. The chamber along with the loading plunger is carefully placed over its base without disturbing the soil specimen and taking care to see that the plunger rests on the cap of the specimen centrally.
13. The loading frame is then adjusted so that it just touches the plunger top by naked eye. The chamber is then rotated if necessary such that the dial gauge, recording compression, rests centrally over the top of the screw which can be locked at any level and which is attached to the top of the cylinder chamber carrying the specimen.
14. The cylinder is then attached to the base plate tightly by means of tightening the nuts.
15. The valve to drain out the chamber and the valve to drain out the air and water from the sample are closed and the air lock nut at the top of the cylinder is kept open to facilitate the

exit of air as water enters the chamber through another valve which connects the chamber to the water storage cylinder subjected to a pressure by a compressor or by any other means.

16. The water storage cylinder is filled with water completely and its top is then closed by means of a valve. Necessary pressure is built up in the cylinder by working the hand pump and the pressure communicated to the cylinder where the specimen is placed, by opening the connecting valve.
17. The cylindrical chamber is allowed to be filled up completely which is indicated by the emergence of water through the air lock nut at the top of the chamber. Then the airlock nut is closed to develop necessary confining pressure by using compressor and the same is maintained constant.
18. If necessary, bring the loading plunger down until it is in contact with the specimen top cap by means of hand operated loading device. This is indicated by a spurt in the reading of the proving ring dial gauge.
19. For this position, adjust the deformation dial gauge reading to zero.
20. Record the initial reading of the proving ring and compression dial gauge.
21. The vertical load is applied to the specimen by starting the motor at the loading frame. The change in the proving ring dial gauge gives the measure of the applied load.
22. The deformation dial gauge gives the deformation in the soil specimen, which can be used to compute strain in the soil.
23. Take readings of proving ring dial gauge at 0.5, 1.0, 1.5, 2.0% (or any other smaller values) of strain and for every 1.0% strain thereafter up to failure or 20% strain whichever is earlier.
24. Throughout the test, make sure that the chamber, containing pressure is kept constant at the desirable value as indicated by the pressure gauge on the water cylinder. If necessary, the pressure can be made good for any possible losses by working the compressor.
25. After specimen has failed or 20% strain is recorded, as the case may be
    - (a) stop application of load
    - (b) disconnect the chamber from water storage cylinder by closing the linger valve
    - (c) open the air lock knob a little and
    - (d) open the valve to drain out the water in the cylinder.
26. After a few seconds open the airlock nut completely to facilitate quick draining out of water, by entry of air at top of the cylinder.

27. After the water is completely drained out, take out the cylinder from loading frame carefully, loosen the nuts and remove the Lucite cylinder from its base, without disturbing the sample.
28. Note the space of the failed specimen, angle of shear plane if any and dimensions of the specimen.
29. Wipe the rubber membrane dry and find its weight W2 that should be same as W1.
30. Remove the membrane from the specimen and take a representative specimen preferably from the sheared zone.
31. Repeat the test with three specimens of the same soil sample subjected to three different lateral pressures (confining) of 0.5, 1.0 and 1.5 kg/cm^2 (5, 10 and 15 psi or 50, 100 and 150 kpa)
32. A graph is drawn between the deviator stress and strain. The deviator stress is the difference between the stresses in axial and radial direction i.e. ($\sigma_1 - \sigma_3$) and is equal to the vertical stress P/A. σ_3 is the lateral confining pressure at any time, which is constant for a test.
33. From the plot, determine the second result at half the ultimate stress, which can be taken as modulus of elasticity.
34. The mohr's circle of stress to define the state of stress at failure is drawn for each sample. The circle has for its centre point ($\sigma_1 + \sigma_3$)/2 and the radius equal to ($\sigma_1 - \sigma_3$)/2.
35. An envelope, which approximates to a straight line, is drawn touching the circle.
36. The intercept made on Y-axis and the slope of the envelope gives the values of strength parameters of the soil C and φ respectively.

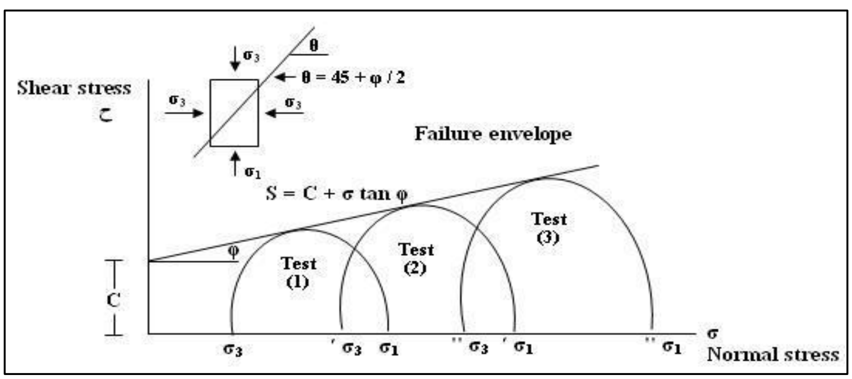


Figure 2: Mohr-coulomb plot for triaxial compression test

Calculations:

1) **Axial Strain (e):**

$$\varepsilon = \frac{\text{Change in length } (\Delta L)}{\text{Original length of specimen } (L_O)}$$

2) **Average Cross sectional area (A)**

$$A = \frac{A_o}{1 - \varepsilon}$$

where,

A_o is the original cross-sectional area of the specimen

3) **Deviator stress (σ_d)**

$$\sigma_d = \frac{P}{A}$$

where,

P is axial load

A is average cross sectional area

4) **Major Principal Stress (σ_1)**

$$\sigma_1 = \sigma_d + \sigma_3$$

where,

σ_d is deviator stress

σ_3 is cell pressure

5) **Correction to allow for the restraining effect of the rubber membrane:**

$$\text{Correction} = 4\text{M}\frac{(1 - \varepsilon)}{\text{D}}$$

where,

M is the compression modulus of the rubber membrane in kg/cm^2.

ε is the axial strain at the maximum principal stress.

D is initial diameter of the sample in cm.

The value of the correction calculated as above shall be deducted from the measured maximum principal stress difference to give the corrected value of the maximum principal stress.

Safety and precautions:

1. The most convenient method of recording the mode of failure is by means of sketch indicating the position of the failure planes. The angle of the failure plane to the horizontal may be recorded, if required. These records should be completed without undue delay to avoid loss of moisture from specimen.

2. Comparison with the recorded mass of the specimen before testing provides a check on the impermeability of the rubber membrane if water has been used as the operating fluid in the cell.
3. Precautions shall be taken to prevent adhesion between the soil and the extruder, for example, by interposing oiled paper discs or lightly oiling the face of the extruder.
4. The length, diameter and mass of the specimen shall be measured to an accuracy enabling the bulk density to be calculated to an accuracy of ± 0.1 percent.

Observation and recording:

The machine is set in motion (or if hand operated the hand wheel is turned at a constant rate) to give a rate of strain 2% per minute. The strain dial gauge reading is then taken and the corresponding proving ring reading is taken the corresponding proving ring chart. The load applied is known. The experiment is stopped at the strain dial gauge reading for 15% length of the sample or 15% strain.

Observations:

Sample No:

Date:

Location: Size of specimen:

Length: Proving ring constant:

Diameter: Initial area:

Initial Volume: Strain dial least count:

Sample 1: **Cell Pressure:**

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain ε	Corrected area A (cm^2)	Proving ring dial reading	Axial Load P (kgf)	Deviator stress (kg/cm^2)

Deviator stress at failure:

Sample 2: **Cell Pressure:**

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain ε	Corrected area A (cm^2)	Proving ring dial reading	Axial Load P (kgf)	Deviator stress (kg/cm^2)

Deviator stress at failure:

Sample 3: **Cell Pressure:**

Deformation dial gauge reading	Axial deformation ΔL (mm)	Axial strain ε	Corrected area A (cm^2)	Proving ring dial reading	Axial Load P (kgf)	Deviator stress (kg/cm^2)

Deviator stress at failure:

General remarks:

1. It is assumed that the volume of the sample remains constant and that the area of the sample increases uniformly as the length decreases. The calculation of the stress is based on this new area at failure, by direct calculation, using the proving ring constant and the new area of the sample. By constructing a chart relating strain readings, from the proving ring, directly to the corresponding stress.
2. The strain and corresponding stress is plotted with stress abscissa and curve is drawn. The maximum compressive stress at failure and the corresponding strain and cell pressure are found out.
3. The stress results of the series of triaxial tests at increasing cell pressure are plotted on a mohr stress diagram. In this diagram a semicircle is plotted with normal stress as abscissa shear stress as ordinate.
4. The condition of the failure of the sample is generally approximated to by a straight line drawn as a tangent to the circles, the equation of which is $\tau = c + \sigma \tan\phi$. The value of cohesion, c is read of the shear stress axis, where it is cut by the tangent to the mohr circles, and the angle of shearing resistance (ϕ) is angle between the tangent and a line parallel to the shear stress.

Questionnaire:

1. What is the stress path of the triaxial shear tests?
2. What is the basis of the sample size?