Indian Standard
METHOD OF LOAD TEST ON SOILS

(Second Revision)

0. FOREWORD

0.1 Visual examination of the soil exposed in suitably located trial pits at the site, combined with the already established data for different types of soils is commonly used for deciding on the safe bearing capacity. While this procedure may be adequate for light or less important structures under normal conditions, relevant laboratory tests or field tests are essential in the case of unusual soil types and for all heavy and important structures. This standard covers plate load test method for determination of ultimate bearing capacity of soil in place which assumes that soil strata is reasonably uniform. The load test included in the standard is also used to find modulus subgrade reaction useful in the design of raft foundation and in the design of pavements.

0.2 Plate load test, though useful in obtaining the necessary information about the soil with particular reference to design of foundation has some limitations. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger than the test plates, the settlement and shear resistance will depend on the properties of a much thicker stratum. Moreover this method does not give the ultimate settlements particularly in case of cohesive soils. Thus the results of the test are likely to be misleading, if the character of the soil changes at shallow depths, which is not uncommon. A satisfactory load test should, therefore, include adequate soil exploration (*see* IS 1892 : 1979) with due attention being paid to any weaker stratum below the level of the footing.

0.3 Another limitations is the concerning of the effect of size of foundation. For clayey soils, the bearing capacity (from shear consideration) for a larger foundation is almost the same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation. Thus tests with smaller size plate tend to give conservative values in dense sandy soils. It may, therefore, be necessary to test with plates of at least three sizes and the bearing capacity results extrapolated for the size of the actual foundation (minimum dimensions in the case of rectangular footings).

0.4 This standard was first published in 1962 and subsequently revised in 1971. In the present revision, the use of apparatus has been generalized and also specific sizes of plates have been mentioned for the different types of soils, besides incorporating zero correction which was present in 1971 version and prescribing log log scale for cohesionless and partially cohesive soils.

1. SCOPE

1.1 This standard lays down the method for conducting load test for estimation of bearing capacity of soils and its settlement.

2. TERMINOLOGY

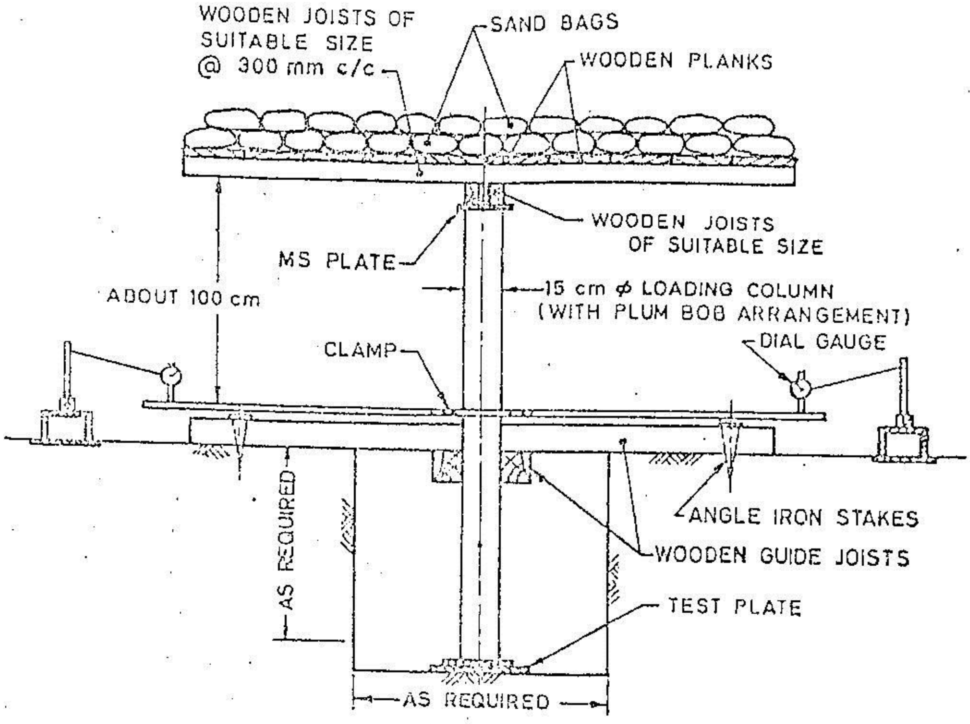
2.1 For the purpose of this standard, the definitions given in IS 2809 : 1972 and IS 6403 : 1981 shall apply.

3. APPARATUS

3.1 Loading platform truss of sufficient size and properly designed members so as to estimate load reaction for conducting the test shall be used. The typical set up used for gravity loading is given in Fig. 1, for reaction loading in Fig. 2 and for loading truss in Fig. 3.

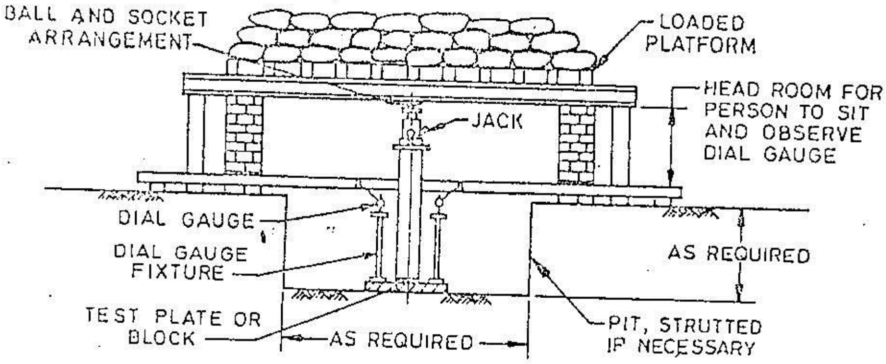
3.2 Hydraulic jack of required capacity with properly calibrated load measuring device, such as pressure gauge, electronic laod cell, or proving ring shall be used.

3.3 **Bearing Plates** — Circular or square bearing plates of mild steel, not less than 25 mm in thickness and varying in size from 300 to 750 mm with chequered or grooved bottom (*see* Fig. 4), provided with handles for convenient setting and centre marked. As an alternative, cast *in-situ* or precast concrete blocks may be used with depths not less than two-thirds the width.



NOTE — Clamp could also be at lower level.

FIG. 1 TYPICAL SET UP FOR GRAVITY LOADING PLATFORM



NOTE — Dial gauge fixture may be on the form clamp also.

FIG. 2 TYPICAL SET UP FOR REACTION LOADING PLATFORM

3.4 Settlement Recording Device — Dial gauges with 25 mm travel, capable of measuring settlement to an accuracy of 0.01 mm.

3.5 Datum Beam or Rod — Beam or rod of sufficient strength capable of maintaining straightness when fitted on two independent supports fitted with arms or magnetic bases for holding dial gauges.

3.6 Miscellaneous Apparatus — A ball and socket arrangement, loading columns, steel shims, wooden blocks, collar, reaction girder with cradles for independent fitting to the reaction platform as necessary to the particular set up.

4. PROCEDURE

4.1 Selection of Location — The locations for load test shall be based on exploratory borings, and unless otherwise desired, shall be conducted at an elevation of the proposed foundation level under

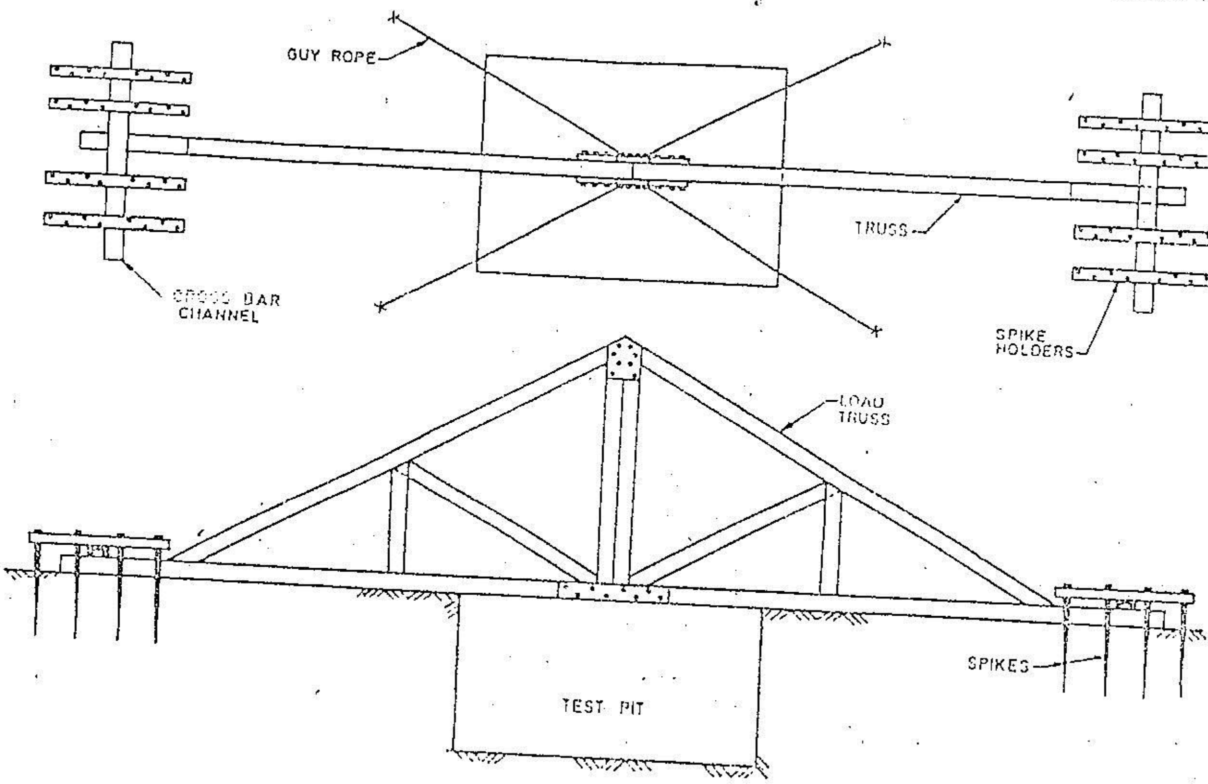
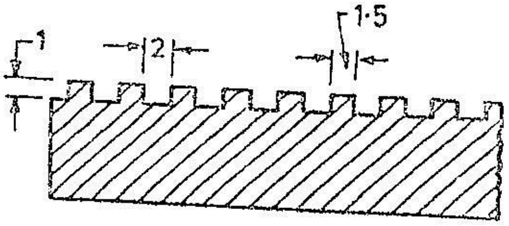


FIG. 3 TYPICAL SET UP FOR LOADING TRUSS



All dimensions in millimetres.

FIG. 4 DETAILS OF CHEQUERS OR GROOVES

the worst estimated conditions. In case the water table is within the depth equal to the width of the test plate, the test shall be conducted at water table level. In case water table is higher than the test level, it shall be lowered to the test level and maintained by pumping through a sump, away from the test plate, however, for the soils like cohesionless silt and fine sand which cannot be drained by pumping from the sump, the test level shall also be water table level.

4.2 Test Pit — The pits, usually at the foundation level, having in general normally of width equal to five times the test plate or block, shall have a carefully levelled and cleaned bottom at the foundation level; protected against disturbance or changes in natural formation.

4.3 Dead Load — The dead load of all equipment used, such as ball and socket, steel plate, loading column, jack, etc, shall be recorded prior to application of load increment.

4.4 Size and Shape of Plate — Except in case of road problems and circular footings, square plates may be adopted. For clayey and silty soils and for loose to medium dense sandy soils with $N < 15$, a 450 mm square plate or concrete blocks shall be used. In the case of dense sandy or gravelly soils $(15 < N < 30)$ three plates of sizes 300 mm to 750 mm shall be used depending upon practical considerations of reaction loading and maximum grain size. The side of the plate shall be at least four times the maximum size of the soil particles present at the test location.

NOTE — N is the standard penetration resistance value determined in accordance with IS 2131 : 1981.

4.5 Test Arrangement

4.5.1 The loading platform shall be supported by suitable means at least 2.5 m from the test area with a height of 1 m or more above the bottom of the pit to provide sufficient working space. No support of loading platform should be located within a distance of 3.5 times size of test plate from its centre.

4.5.2 The test plate shall be placed over a fine sand layer of maximum thickness 5 mm, so that the centre of plate coincides with the centre of reaction girder/beam, with the help of a plumb and bob and horizontally levelled by a spirit level to avoid eccentric loading. The hydraulic jack should be centrally placed over the plate with the loading column in between the jack and reaction beam so as to transfer load to the plate. A ball and socket arrangement shall be inserted to keep the direction of the load vertical throughout the test. A minimum seating pressure of 70 g/cm² shall be applied and removed before starting the load test.

4.5.3 The two supports of the reference beam or datum rod shall be placed over firm ground, fixed with minimum two dial gauges resting at diametrically opposite ends of the plates. The dial gauges shall be so arranged that settlement is measured continuously without any resetting in between.

4.6 **Load Increments** — Apply the load to soil in cumulative equal increments up to 1 kg/cm² or one-fifth of the estimated ultimate bearing capacity, whichever is less. The load is applied without impact, fluctuation or eccentricity and in case of hydraulic jack load is measured over the pressure gauge, attached to the pumping unit kept over the pit, away from the testing plate through extending pressure pipes.

4.7 **Settlement and Observation** — Settlements should be observed for each increment of load after an interval of 1, 2.25, 4, 6.25, 9, 16 and 25 minutes and thereafter at hourly intervals to the nearest 0.02 mm. In case of clayey soils, the 'time settlement' curve shall be plotted at each load stage and load shall be increased to the next stage either when the curve indicates that the settlement has exceeded 70 to 80 percent of the probable ultimate settlement at that stage or at the end of 24 hour period. For soils other than clayey soils, each load increment shall be kept for not less than one hour or up to a time when the rate of settlement gets appreciably reduced to a value of 0.02 mm/min. The next increment of load shall then be applied and the observations repeated. The test shall be continued till a settlement of 25 mm under normal circumstances or 50 mm in special cases, such as dense gravel, gravel and sand mixture is obtained or till failure occurs, whichever is earlier. Alternatively, where settlement does not reach 25 mm, the test should be continued to at least two times the estimated design pressure. If needed, rebound observations may be taken while releasing the load.

5. DETERMINATION OF ULTIMATE BEARING CAPACITY/SAFE BEARING PRESSURE/SETTLEMENT

5.1 **Shape of the Load/Settlement Curve** — A load settlement curve shall be plotted out to arithmetic scale. From this load settlement curve, the zero correction which is given by the intersection of the early straight lines or the nearly straight line part of the curves with zero deadline shall be determined and subtracted from the settlement readings to allow for the perfect seating of the bearing plate and other causes.

5.1.1 Four typical curves are shown in Fig. 5. Curve *A* is typical for loose to medium cohesionless soil; it is straight line in the earlier stages but flattens out after some time, but there is no clear point of failure. Curve *B* is for cohesive soil, it may not be quite straight in the early part and leans towards settlement axis as the settlement increases. For partially cohesive soils, curve *C* possessing the characteristics of both the curves *A* and *B* is obtained while curve *D* is purely for dense cohesionless soils.

5.2 From the corrected load settlement curves, no difficulty should be experienced in arriving at the ultimate bearing capacity in case of dense cohesionless soils or cohesive soils (*see* Fig. 5, curves *D* and *B*) as the failure is well defined. But in the case of curves *A* and *C*, where yield point is not well defined, settlements shall be plotted as abcissa against corresponding load intensities as ordinate, both to logarithmic scales (*see* Fig. 6), which give two straight lines, the intersection of which shall be considered as yield value of soil.

5.3 From Fig. 5, the safe bearing pressure for medium and dense sands could be read,

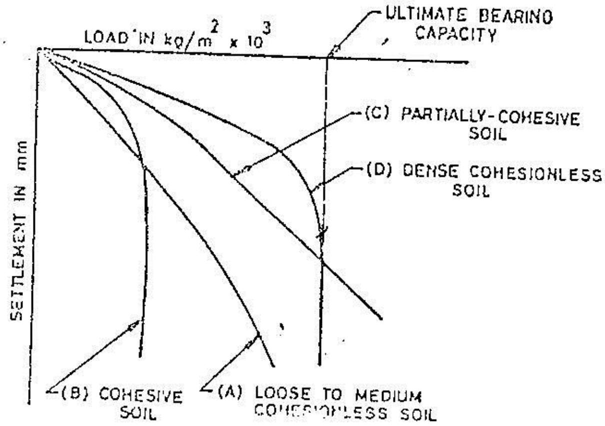


FIG. 5 LOAD SETTLEMENT CURVES

2

Rocks

2·1. Rocks

Since the details of Rock properties are available in geologists's literature and their language, it is necessary that a rock engineer should have an information about the rock qualities, as observed by the geologist, to evaluate numerical conclusions. It is necessary to know about formation, qualities properties and defects of rocks. Hence, in this chapter these topics will be dealt with in brief so that a rock engineer may be acquainted with rocks fully.

Geologists define rocks as naturally occuring aggregates of minerals or mass of mineral matters constituting an essential part of the earth's crust.

2·2. Minerals

A mineral is a natural inorganic substance of a definite structure and chemical composition such as Calcite, Quartz, Chlorite, Hematite etc.

Minerals are generally identified by their colour, streak, hardness, cleavage and fracture, tenacity, crystal form, specific gravity and lusture.

If a mineral is rubbed over an unglased porcelain or China plate, a coloured or white streak of minute mineral particles is left on the plate. The colour of the streak is compared with a standard colour chart available, which gives an idea of characteristics of the mineral. If struck with a sharp object, a mineral breaks along a definite plane which is parallel to a crystal face and is know as cleavage plane. It is usually a smooth surface and appears to be polished. The cleavage plane also helps in identifying a mineral.

If a sharp blow is given to a mineral block it breaks into irregular surfaces known as fracture. Along this failure planes, fresh mineral particles can be observed by microscope or simply by eye, depeding on the size of the grains, and they can thus be identified.

The ability of a mineral to withstand crushing, shearing or bending is its tenacity. They are described as brittle, malleable, flexible and elastic. If it can be powdered easily it is brittle ; malleable when it can be hammered into thin sheets. If it bents but does not return to its original shape after bending when the force is released then it is known as flexible, where as it will be known as elastic if it returns to its original shape after the removal of a bending force.

Every mineral has a definite crystal form bounded by several or many crystal faces and belongs to a definite crystallographic system. The crystal forms may be Monoclinic, Hexagonal, Rhombohedral etc.

By keeping the mineral in hand, (just for preliminary identifications) its heavyness will give an idea of the specific gravity of the mineral. Feeling of more heaviness will represent more specific gravity of the mineral. However measurement of specific gravity can be done in the laboratory, and an exact value can be known for proper identification of mineral.

Most minerals exhibit a certain characteristic appearance (luster) under reflected light. The luster may be metallic, non-metallic or sub-metallic.

Hardness of a mineral is also a good scale for its identification. The hardness of a mineral is expressed by its number in Mohs scale of hardness. A mineral listed in a particular scale can scratch all minerals of smaller numbers.

2·3. Structure of Earth

Before discussing 'rocks' in details, a brief idea of the Earth structure is given for reference to the readers.

Due to high pressure and temperature at the Inner most part of the earth the minerals contain very high density. This inner most part of the earth crust is known as *Inner core* and consists of solidified heavy metals such as iron and nickel. It is estimated to be about 850 kilometres thick.

The inner core is surrounded by outer core which is estimated to be 2100 km thick. Composition of outer core is similar to that of inner core. However, the inner core is in more fluid state as compared to the outer core.

The outer core again is surrounded by a layer which is estimated to be 2900 km thick and is known as Mantle. The materials in mantle are about two to three times as dense as those at the surface of the earth. By sismic observations, it has been found that a major change or discontinuity occurs at the boundary between the Mantle and the outer Core.

Outside the Mantle, the portion of the Earth upto the surface is known as crust. The crust consists of two parts.

(*a*) Inner annulus also known as Sima or Basaltic Layer.

(*b*) Outer annulus also known as Sial or Granitic Layer.

Sima or Basaltic Layer is made up of dense, dark-coloured material which is rich in magnesia and it is similar to those which comes out of the Volcanoes.

Sial or Granitic is composed of less dense materials. It is rich in silica and alumina and has got similartiy in composition of rock granite.

The depth of crust which includes Basaltic as well as Granitic layer is about 40 kilometres in the continental areas, whereas the depth of Bastic layer, which forms the floor of ocean under the oceanic areas, is about 6 km. The prime constituent of basaltic and granitic layers is considered to be molten silicate material or magma. From time to time, the magma comes to the surface of the earth, in the form of lava, through the mouths of Volcanoes.

Although geologists often call 'rock' to all constituents of the earth crust, engineers devide the earth crusts into rocks and soils.

The hard and compact natural materials of the earth crust are known as rocks and their derivatives due to weathering etc. are termed as "soils".

As discussed earlier the subject which deals with the laws of hydraulics and mechanics applied to Rocks is known as Rock Mechanics whereas the subject dealing with laws of hydraulics and mechanics applied to "Soils" is known as "Soil Mechanics".

2·4. Rock for Engineers

A better definition of rock may now be given as "Granular aelotropic, heterogeneous technical substance which occurs naturally and which is composed of grains of varied poly-crystalline or non-crystalline materials which are cemented together either by "glue" or by mechanical bond, but ultimately by atomic, ionic and molecular bonds within the grains and glue and at every interface of the boundary".

Thus by "rock", an engineer means a firm and coherent substance which normally cannot be excavated by manual methods alone. Thus like any other material a rock is frequently assumed to be homogeneous and isotropic. But in most of the cases, it is not so.

A homogeneous substance is one in which a small element has the same property as that of the whole substance and a hetrogeneous substance has different properties of the elements within the body.

An isotropic material is one that has the same physical properties in every direction at any point whereas an anisotropic or aleotropic material is one in which certain property may vary in a particular direction at a point in the substance.

In a small scale, a Rock may be homogeneous and isotropic but when considered at a large scale it may not be so. Therefore, for large engineering projects these difficulties are overcome with introduction of finite element method techniques and solving the problems with the help of computers.

2·5. Rock as a Construction Material

Rock is used as a construction material in two ways.

(*a*) Rocks with which constructions are done.

(*b*) Rocks over which constructions are done.

For the first category, rocks are brought from quarry etc. and used in place where construction is done.

For example :

(1) For making breakwaters and other such structures.

(2) For protective blanketing of earth dams and other earth works against erosion by water in the form of riprap.

(3) For putting as ballast to support railway sleepers.

(4) As a base and sub-base and top course for roads, and air-field runways.

(5) As a coarse aggregate for concrete.

(6) As facing stones for building, bridges and hydraulic structures to protect such structures from weathering and to improve their elevations.

(7) As building blocks for putting them in foundation as well superstructures of buildings.

A few examples for (*b*) category are :

(1) To support massive structures such as dams, weirs, multi-storeyed building etc.

(2) For construction of tunnels for vehicular transport and sometimes water transportation also.

(3) For making shafts.

2·6. Classification of Rocks

Some of the classification systems of rock are :

(*a*) By origin or genesis.

(*b*) Geological or lithological classifications.

(*c*) Engineering classification of intact rock.

(*d*) A combination of these.

2·6·1. By Origin or Genesis

Based on genesis or mode of origin rocks are grouped into three major groups.

Igneous rocks
Sedimentary rocks
Metamorphic rocks.

2·6·2. Igneous Rocks

Igneous Rocks are those rocks which are formed by the solidification of molten magma originating in the interior of the Earth. Magma is a "fluid" consisting of molten silicates water vapour and volatiles. If a rock is formed by the cooling and solidification of magma at depth (*i.e.* below the surface of earth) the rate of solidification is relatively slow and consequently a coarse grained structure is formed. If a rock is formed from the solidification of lava on the surface, (which mainly comes due to irruption of volvanoes) the rate of cooling is faster and rapid solidification takes place. In such cases, fine grained materials are formed.

Igneous rock which are formed at a depth, are known as plutonic igneous rocks and those formed from lava and formed mainly at the surface, are known as volcanic igneous rocks.

Classification of igneous rock depends upon the composition of the parent magma and its textural classification will depend upon the environment in which it solidifies.

Thus the chemical composition of magma rock is sub-divided into four parts :

(*i*) Acidic
(*ii*) Intermediate
(*iii*) Basic
(*iv*) Ultrabasic.

In an igneous rock if quartz and feldspar are predominant it will be acidic in composition whereas if ferromagnesian minerals are predominant the rock will be having basic composition.

The acidic igneous rocks are generally light coloured whereas basic one are dark coloured due to the presence of ferromagnesian minerals.

Textural subdivisions are done as :

(*i*) Coarse grained also known as Plutonic
(*ii*) Medium grained also known as Hypabyssal
(*iii*) Fine grained also known as Volcanic.

Table 2·1 gives an idea of classification of some importa nt Igneous Rocks as discussed by Krynine and Judd.

Table 2·1
Classification of Igneous Rocks

Structure	*Composition*			*Remarks*
	Acidic	*Intermediate*	*Basic*	
Coarse to medium grained (Plutonic)	Granite (L.C) Granodionite (L.C)	Syenite (L.C) Diorite (D.C)	Gabbro (D.C)	
Medium to Fine Grained (Hypabyssal)	Microgranite Quartz-porphyry (L.C) Micro granodiorite (L.C)	Microsyenite Microdiorite (D.C)	Dolerite (D.C)	L.C→Light Coloured D.C→Dark Coloured
Fine Grained and Glassy (Volcanic)	Rhyolite (L.C) Pitchstone Dacite (L.C)	Trachyte (L.C) Andesite (D.C)	Basalt (D.C)	

2·6·3. Sedimentary Rocks

Sedimentary Rocks are derived due to weathering and decomposition of earth crust or from any rock type.

When a rock or earth crust is weathered or decomposed and transported and redeposited, and subsequently consolidated and cemented partly or fully, then the new product formed is known as Sedimentary Rocks.

The size of weathered products and the degree of consolidation and cementation determine the strength of the sedimentary rocks.

The type of sedimentary deposit formed in any area depends upon :

(*a*) The nature of sediment carried into the area
(*b*) The physical environment within the area.

The material from which sedimentary deposits are formed are derived in the following ways :

(*i*) Mechanically
(*ii*) Organically
(*iii*) Chemically.

Mechanically. Weathering agents act on the surface a of pre-existing rocks which may be igneous, sedimentary or metomarphic. These weathered products latter become constituents of the new sedimentary rocks.

The weathering agents are :

(*a*) **Atmospheric agents**—Rain, frost, wind and temperature changes.
(*b*) **Gravity**—The rock disintegration during landslide.
(*c*) Rivers
(*d*) Seas
(*e*) Ice
(*f*) **Organic agents**—such as animals and plants.

Organically—Sediments are formed from the remains of plants and animals.

Chemically—Sediments are formed due to chemical decomposition.

Sedimentary Invironments.

Invironments which cause sedimentations are mainly :

(*a*) Continental,
(*b*) Intermediate,
(*c*) Marine.

Continental. In such cases the agents are rivers, lakes, glaciers, and wind. For example deposits transported and deposited by wind are Aeolian, deposited by rivers—Fluviatile and deposited by glaciers are Glacial.

Intermediate. Deposits formed in deltas are Deltaic and deposits formed in estauries of rivers are Estuarine.

Marine Environments. Deposits formed along coastlines—Shore, deposits formed under great depth of water—Abyssal.

The shape of grains in a sediment depend upon :

(*a*) The original shapes of the materials supplied by the disintegration of pre-existing rocks. For example—weatherings from granite will be sharp angular quartz and from feldspars it will be irregular mica flakes.

(*b*) The amount and nature of transport : The greater the distance of transportation the more will be the rounding of the grains. Wind transported sediments will be rounded of up to the maximum extent, water causes some degree of rounding whereas ice will cause the least rounding off.

Generally sedimentary rocks occurs in well defined beds. They are known as bedding planes or stratification.

Classification of sedimentary Rocks.

As a broad classification, sedimentary deposits are classified in two categories.

(*i*) Unconsolidated Sedimentary Deposits;
(*ii*) Consolidated Sedimentary Rocks ;

In fact due to further consolidation and cementation of sedimentary deposits sedimentary rocks are formed. To the engineer group (*i*) constitutes the engineering soils and group (*ii*) is the rock.

From an engineering view point, the most important sedimentary rock are sandstones, shales and limestones.

The size of grains in sandstones varies from fine grained type to coarse grained type. The structures are massive, horizontally bedded or cross-bedded. Sandstones are having good bearing capacity. However presence of some weak materials in the mass make it weaker.

The groups of sedimentary rocks formed by clay minerals are shales. A shale is laminated sedimentary rock frequently dark in colour and composed of mainly clay sized particles (Finer from 0·002 mm size particles). Sometimes small percentage of sand and silt may occur. It has got laminations which can easily be observed. But the shale has got variable hardness depending upon its consolidation. A good shale gives a clear ring if struck with a hammer.

Limestone. It is primarily composed of calcium carbonate and has got crystalline structure. The grains are fine. It may be considered as a good foundation material unless it has got cavities.

Another important type of rock which may be considered in the sedimentary category, is the Conglomerate. It is a rock composed of very coarse (6 mm to 75 mm) particles which are rounded off. Sometimes well graded particles with good percentage of fines are there in the materials. The well graded materials make such type of rock as good bearing surface. Some of important sedimentary rocks are given in Table 2·2.

Table 2·2
Classification of Sedimentary Rocks

Group	*Unconsolidated Sediments*	*Consolidated Rocks*
Rudaceous (Pebbly)	Gravels Fragmented rock	Conglomerates Breccias
Arenaceous (Sandy)	Sands	Sandstones Ferruginous sandstone—iron Cement : Seliceous sandstone—siliceous cement. Calcareous sandstone-Calcareous cement. Greywocke—composed of mixture of rock fragments.
	Silts	Siltstones—a consolidated rock of silt grade.
Argillaceous (Muddy)	Muds, Clays, Brick earth	Hard clays, shales, mudstones, fine clay, marl.
Calcareour (Limey)	Shell sand, Coral reef, $CaCO_3$ precipitate from solution	Lime stones Shelly limestones Siliceous limestone Ferruginous Limestone Chalk.
Carbonaceous Peat		Coals Lignite Anthracite Cannel
Ferruginous		Clay ironstone Blackband ironstone
Siliceous	Silica gel	Flint, Chert

2·6·4. Metamorphic Rocks

Due to high pressure, high temperatures and temperature gradings as well as high shearing stresses on existing igneous or sedimentry rock masses, under the earth crust, recrystallisation of rocks takes place and the resulting complete or incomplete recrystallised mass is known as metamorphic rocks. Due to recrystallisation a new rock mass evolves due to changed environments.

The process which brings out changes in the rocks is known as metamorphism.

The agents which are active for metamorphism are :

(*a*) Temperature
(*b*) Pressure
(*c*) Stress
(*d*) Chemically reactive substances.

These agents may act separately, or combined together depending upon conditions to bring the changes in the rock mass. During recrystallisation, the action of directed pressure or stress on the rock mass causes a change in the orientation of the mineral grains or crystals. The nature of metamorphic rocks not only depends upon a particular agent as listed above but it also depends on the extent to which they are in action to bring metamorphism.

The mineralogical composition of a metamorphic rock is dependent upon two factors.

(*i*) The initial composition of the rock
(*ii*) The degree of metamorphism (*i.e.* change) undergone by the rock mass.

Limestone which consists of calcium carbonate is changed into marble due to metamorphism and sandstone which contains silica is changed to quartzite.

A shale under low grade of metramorphism is changed to slate whereas under high grade of metramorphism, it is converted into phyllites.

Igneous rocks are formed under a condition of falling temperature and pressure and hence, the minerals crystallise in descending order of their melting points. In case of metamorphic rocks, there is commonly a rise in temperature which brings out new reactions depending upon the degree of temperature and all the constituent minerals are formed simultaneously when cooling takes place. Then the minerals which occur in a metamorphic rock are known as anti-stress minerals if they have been formed under uniform pressure whereas they will be known as a stress-minerals when formed under a directed pressure or stress *i.e.* when pressure or stress in a particular direction was more than the other directions. Anti-stress minerals are generally equidimensional whereas stress minerals are flaky or laminar.

Some of the important metamorphic rocks which are used by engineers are marble, slate, schist, Gneiss, Quartzite, Hornfels etc.

Marble results from recrystallization of lime stones. Slate is formed by metamorphism of fine grained sedimentary rocks. Cleavage planes are highly developed.

Schist results from mica, chlorite, talk and hornblende minerals. They are of medium to coarse grained size.

Gneiss results from metamorphism of igneous or sedimentary rocks. Minerals are arranged in irregular bands. Common minerals are feldspar, quartz, mica or hornblende. Quartzite is formed due to recrystallization of sandstones under heat and pressure Dominant mineral is quartz.

Hornfels are formed by thermal metamorphism of muddy rocks, such as shales. Minerals present are feldspar, biotite and quartz.

Table 2·3

Classification of Metamorphic Rocks

Structure and Texture	*Composition*	*Rock name*
Massive Banded, consisting of alternating lenses	Various tabular prismatic and granular minerals (frequently elongated)	Gneiss
Granular, consisting mostly of equidimensional grains.	Calcite, dolomite quartz in small particles	Marble or quartzite
Foliated or platy	Various tabular and/or prismatic minerals (generally elongated)	Schist, slate phyllite

2·7·1. Geological or Lithological Classification

The lithology of a rock is the study of its physical character. It includes the study of mineralogical composition, texture, colour, physical appearance etc. It helps in the selection of a particular rock for engineering purpose. Generally engineers are mainly concerned with strength properties of the rock material. Hence if an engineer is conversant with the lithological or geological classification of rocks he can select the rocks primarily for his purpose. For example if he has to use a building block, he will be using marble where external exposure is required, whereas he will use granite when the rock has to be used as foundation blocks.

When he refers the name of granite, limestone, sandstone, mica shist, quartzite he gets an idea of the physical property of the rock and he ascertains whether a particular rock will be suitable for a specific purpose.

To ascertain the engineering properties of rocks it is necessary to know the following rock properties which can be ascertained by visual examinations to make a preliminary inference about the suitability of a particular rock for a particular purpose. For example, granites, lime stones and other rocks may exist in a very hard and strong state to a completely decomposed and weak state. Hence if simply rock type is described the informations may not be sufficient. Hence in order to describe the rock fully for a particular engineering purpose it is necessary to describe following properties :

(*i*) Texture

(*ii*) Structure

(*iii*) Composition

(*iv*) Colour

(*v*) Grain size.

2·7·2. Texture

Rock materials may be of any of the following textural group.

(*i*) Crystalline

(*ii*) Indurated

(*iii*) Crystalline-indurated

(*iv*) Compact

(*v*) Cemented.

Crystalline rock materials are composed of visible interlocking crystals or crystal grains. When scratched by the blade of a penknife, particles do not come out of the rock mass. If particles come out due to scratching, the rock will not be taken in crystalline group.

Indurated rock materials are those in which interlocking crystals and crystal grains are not visible by naked eye. Grains are fine. But the rock is strong as particles do not come out of the rock mass when scratched by the edge of a knife.

Crystalline-indurated rock materials fall between crystalline and indurated rock materials. Its individual crystal grains or crystal aggregates are finer than crystalline structure but coarser than indurated. Rocks of this type of structure are hard because the grains do not come out when scratched by the edge of a knife.

In compact rock materials, the particles are held together purely by tightness for grain packing. Grains are finer. Particles or powder come of the rock mass when scratched by the edge of a knife.

Cemented rock materials are medium to coarse-grained rock having grain to grain bonding by some cementing materials. Grains are visible to the naked eye and particles come out from the rock mass even when scratched with a finger nail.

2·7·3. Structure

Structures refers to placing of various textures within the rock material. It also refers to fractures or any preferred mineral orientation within the rock mass.

The various types of structures are as follows :

(*a*) Homogeneous

(*b*) Lineated

(*c*) Intact-foliated

(*d*) Fracture foliated.

Homogeneous. If the grains and crystals are having random orientation the structure will be called homogeneous. By visual examinations only the homogeneous structures in a rock mass can be ascertained.

Lineated. If the mineral particles are having a preferred orientation in a particular linear direction/directions the structure will be known as lineated.

Intact-foliated. When the minerals in the rock mass are having a preferred orientation of a planer nature.

Fracture-foliated. When the planer structure is having closed or incipient fracture such as bedding planes or cleavage planes.

Generally the lineated structures pose problems because properties of rock mass is not the same in all directions in such cases. The mass is known as aelotropic or non-isotropic.

2·7·4. Composition

Presence of calcite is of prime importance when considering mechanical and physical characteristics of rock mass. The important sub-divisions are :

(*a*) Noncalcareous

(*b*) Part-calcareous

(*c*) Calcareous.

Non-calcareous. Rock materials are those in which calcium carbonate is absent.

Part-calcareous. The rock contains mainly non-calcareous materials. The calcareous material is present as a bond between the grains.

Calcareous. The rock materials which are mainly composed of calcite.

2·7·5. Colour

If the rock is of basic nature, it will be of dark colour whereas acidic rocks are of light colour. Light-coloured rocks are generally feldspathic whereas dark coloured rocks generally contain ferro-magnesiun minerals. Calcarrous rocks, which contain impure materials, are dark in colour whereas pure calcareous rocks are light.

2·7·6. Grain Size

Sometimes classification of rocks is done on the basis of their grain sizes. In such cases origin or type of rock is not so important.

The rock material is classified in three groups—Coarse, Medium and Fine grained.

Coarse-grained—When the particles are larger than 2 mm in diameter.

Medium grained—When particles size lies between 2 mm and 0·1 mm.

Fine grained—Particles of less than 0·1 mm size and invisible to the natural eye.

2·8. Engineering Classification

The problems of Rock mechanics are mainly associated with intact rocks. Hence it is necessary to classify the rock based on insitu properties. The behaviour of rock mass subjected to change in stress due to change in physical conditions due to construction of some superstructures or excavation of tunnels etc. is governed by mechanical properties of the intact rock mass and number and nature of geological discontinuities present in the mass. Hence, in rock classification these two items are taken separately. In addition to these, rocks are also classified sometimes according to their degree of weathering. Because this also gives an important information about the load bearing capacity of the rock mass. While making classification as per degree of weathering rocks are classified as grade I, II, III etc.

Table 2·4 gives the classification based on degree of weathering of the rock mass as suggeted by Geological Society of London.

Table 2·4

Classification based on degree of weathering of rock mass

Term	Description	Grade
Fresh	No visible sign of material weathering.	IA
Faintly weathered	Discolouration on major discontinuity surfaces.	IB
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolured by weathering and may be somewhat weaker than in its fresh condition.	II
Moderately weathersd	Less than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a contineous frame work or as core stones.	III
Highly weathered	More than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

The basis of engineering classification of rocks is uniaxial compressive strength and modulus of elasticity. Based on uniaxial compressive strength the rock is classified as Class A, B, C, D and E. The compressive strength value is based on the results of the specimen having length diameter ratio of atleast 2. It may be observed from the table that strength of different classes follow a geometric progression. Deer and Miller (1966) has given Table-2·5, for classification of rocks in different categories.

Table 2·5

Engineering Classification of Intact Rock (After Deer and Miller, 1966)

Class	Description	Uniaxial Compressive Strength kg/cm^2	Rock Material
A	Very High Strength	Over 2250	Quartzite, diabase, basalts, Majority of igneous rocks, Strong metamorphic rocks.
B	High Strength	1125—2250	Weakly cemented sand stones, hard shales, majority of lime stones, dolomites
C	Medium Strength	562·5—1125	Many shales, porous sandstones, and limestones, schistose varieties of metamorphic rocks.
D	Low Strength	281·25—562·5	Porous low-density rocks, friable sandstone, truff.
E	Very low Strength	Less than 281 25	Clay shales, weathered and chemically altered rocks of any lithology.

Table 2·6 shows the engineering classification of intact rock on the basis of modulus ratio,

$$M_R = E_{t\,50}/\sigma_{ult}.$$

where E_{t50} = tangent modulus at 50% ultimate compressive strength of rock and

σ_{ult} = unaixial ultimate compressive strength.

Table 2·6

Engineering Classification of Intact Rock (After Deer and Miller, 1966)

Class	Description	Modulus ratio
H	High	More than 500
M	Average (medium)	200—500
L	Low	Less than 200

On the basis of above two tables engineering classification is done like *AM*, *BH*, *CM*. etc. (*e.g. CM* means medium strength and average modulus ratio).

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Classification based on degree of weathering of rock mass

Term	Description	Grade
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3

Defects in Rock Mass

3·1. Discontinuities and Defects in Rock Mass

When the foundation of an engineering structure will be put on a rock mass an information regarding the type of rock, such as igneous, sedimentary or metamorphic, is not sufficient. The rock may be an igneous one, but it may consist of a lot of discontinuities in the mass which may make the rock unable to withstand high stresses due to the superstructure.

Therefore, for proper design of engineering structures 'on' and "in" rock, it is necessary to have a good information about the existing and probable discontinuities or weaknesses inside the rock mass. These discontinuities and weaknesses may be in the following forms.

Fractures, Cracks and haircracks, Fissures, Bedding planes and laminations, Stratification, Joints, Faults, Folds and cavities.

In their 'in-situ' conditions rock masses consist of discontinuities in any of the above forms in varying degrees. Sometimes these fractures split the rock mass into blocks or units of rock materials. The size of blocks or units depends on discontinuities existing. Thus the rock mass has a predetermined structure prior to commencement of an engineering work. In engineering design it is necessary to specify the materials, such as brick, which will be used in forming the framework of the building. The engineer has to consider the strength and quality of the brick and accordingly he will design the framework such as foundations, and different bays with suitable columns. Similarly in case of rock masses we have to consider :

(*a*) the nature of the rock material present and

(*b*) th: sub surface structural design depending upon the presence of structural faults present in the rock mass.

3·2. Cause of Defects in Rock Mass

Although the earth seems to be stable at its surface, but the earth's crust is in a state of continuous unrest which leads to gradual or sudden changes in its structure and configuration. The earthquakes generally confirms the existence of such forces. The forces within the crust which modify the structure of rocks bring out generally two types of crusted movement.

(*a*) Epeirogenic or continent-building movements.

(*b*) Orogenic or mountain building movements.

The epeirogenic or continental building movements are continuous but slow. In such cases, portions of continents or ocean

floor are subjected to slow continuous movements. Due to such movements there may be upheaval or subsidence. Deformations in such cases may be uniform. The orogenic or mountain-building movements are associated with immense disturbing forces. Such movements are of more severe type, but affect the earth periodically. Due to sudden and severe disturbing forces the geological structures formed are of complicated nature.

Due to eperogenic or orogenic movements the resulting structures in sedimentary rocks are generally of simple nature due to their bedding planes. But in case of igneous or metamorphic rocks it is not so. In such cases complex structures are formed and their study is rather difficult.

3·3. Strike and Dip

Unequal forces acting on the crust cause unequal uplift or subsidence. Due to unequal uplift or subsidence, the original horizontal sedimentary beds get tilted. These tilted beds slope in some direction and subtend an angle with the horizontal plane.

The direction of the line along which an inclined bed meets a horizontal plane is known as the strike of the bed. It is described as $N(°)E$, $N(°)W$ or $S(°)E$, $S(°)W$ which means a particular angle of deviation in the direction of East or West from North or South direction. For example $N20°E$ means that strike makes an angle of 20° eastwards from the North direction. This has been shown in Fig. 3·1. Similarly $S30°E$ mean thats trike is 30° eastwards of south direction.

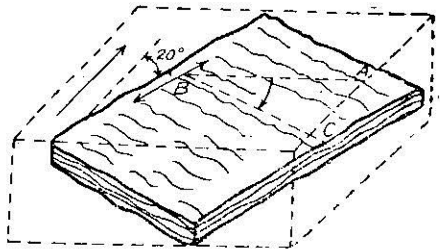


Fig. 3·1.

Dip indicates the maximum slope of a particular inclined plane. The significance of the term dip is evident with reference to the angle ABC, which the inclined plane BC makes with horizon BA, as shown in Fig. 3·2. Dip is described as (°)N, (°)S, (°)SE etc. For example 20°S means a dip in southerly direction and 25°SE means a dip of 25° in South-east direction. While describing dip the true dip angle should be measured. A true dip means the angle made with horizontal by a line which lies in the inclined plane and is perpendicular to the strike. In Fig. 3·1 angle ABC is a true dip of the inclined bedding plane. Strikes and dips are used to describe joints.

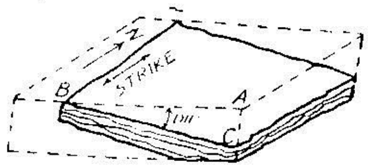


Fig. 3·2.

3·4. Bedding Planes, Stratification and other Defects

In addition to Fractures, Cracks, Fissures, Joints and Faults which indicate discontinuities of the rock mass, there are some weaknesses existing inside the rock mass which are named as Bedding planes, Laminations, Stratifications, Cleavage Planes and Partings.

Bedding planes are the planes which separate the sedimentary and stratified rocks in different layers.

Lamination of a rock strata also constitutes a discontinuity in the rock mass, and it is a mechanical weakness of rock en masse.

Stratification is layering of geological materials. There are different types of materials in the stratifications and these are the planes which helps the rock material to be separated along beding planes.

Cleavage is the property of rock mass to split into thin, parallel sheels. The cleavage planes are the planes in rocks along which cleavage takes place.

A parting is a thin layer of deposited and altered weak materials, such as carboneceous or organic, which exhist as separating beds in sedimentary or metamorphic rocks.

Separation is a break between beds along bedding planes.

3·5. Joints

The tensile and compressive stresses which act within the rock are produced due to decrease in volume *i.e.*, shrinkage of the rock mass. These decrease in volume are caused due to :

(*a*) Drop in temperature

(*b*) Loss of moisture

(*c*) Drop in temperature as well as loss of moisture.

Due to tensile and compressive stresses in the rock mass regular and irregular cracks or discontinuities are developed in the rock mass.

Any break in a rock mass irrespective of its size is termed as *fracture*.

Minor fractures are designated as *cracks* and *fissures*.

Cracks along which the fractured rock masses appear to have suffered no relative displacement are known as *joints*.

Joints occur in all types of rocks, *i.e.*, igneous, sedimentary and metamorphic.

In sedimentary rocks generally there are two system of mutualy perpendicular joints both perpendicular to bedding planes.

In igneous rocks there are three regular sets of joints. They are :

(*i*) Flat laying joints

(*ii*) *Q* joints or cross joints

(*iii*) *S* joints or longitudinal joints.

Flat-lying joints are approximately horizontal and are parallel to the bedding planes.

Q joints or cross joints are almost perpendicular to the flow lines.

S joints or longitudinal joints, dip steeply and strikes parallel to the flow lines if projected to a plane surface.

The three joints are shown in Fig. 3·3.

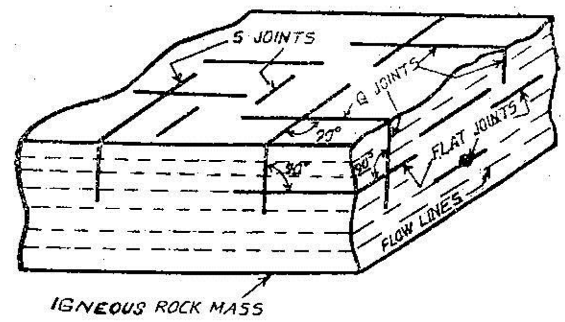


Fig. 3·3. Types of joints.

Joints seldom occur alone. Generally a number of more or less parallel joints occur together in the form of a joint set. Two or more joint-sets together constitute a joint system. A joint can be open or closed. A closed joint is one whose walls are in contact. Closed joints may be invisible. Along such surfaces there is no resistance against separation. Hence sometimes they pose serious problems if they are not detected by a proper investigation. When the structures are constructed and stresses act along such joints, sliding occurs and if not delt with properly it may endanger the structure. When a rock mass consists of many joints, the rock mass is divided into individual blocks to form a three-dimensional network. In such cases the rock mass may be considered as aggregate of uncemented,

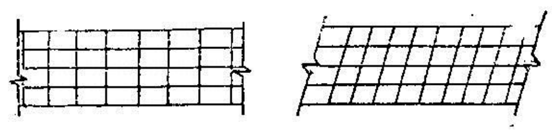


Fig. 3 4 (*a*)

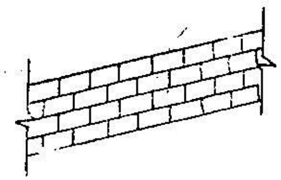


Fig. 3 4 (*b*)

cohesionless cuboid blocks. These may be compared with closely fitted blocks in a dry masonry wall. If the blocks are properly imbricated, shear strength will be more in the rock mass against the forces trying to dislocate the structure, Fig. 3 4 (*b*) whereas shear strength will be lessif imbrication will be poor as shown in the Fig. 3·4 (*a*).

3·6. Faults

When there is a displacement on each side of a fracture in the rock mass along the fracture plane, then the plane is classified as a fault. The displacement may be horizontal, vertical or both. In wide contrast with joints, faults are well defined cracks.

When several faults occur in close proximity and are parallel to each other, then the resulting zone of broken rock is called a shear or fault zone. As faults occur along well defined planes, their description in space is defined in terms of their dip and strike. In Fig. 3·5

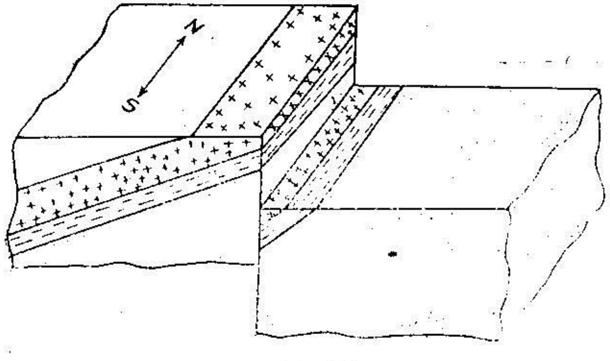
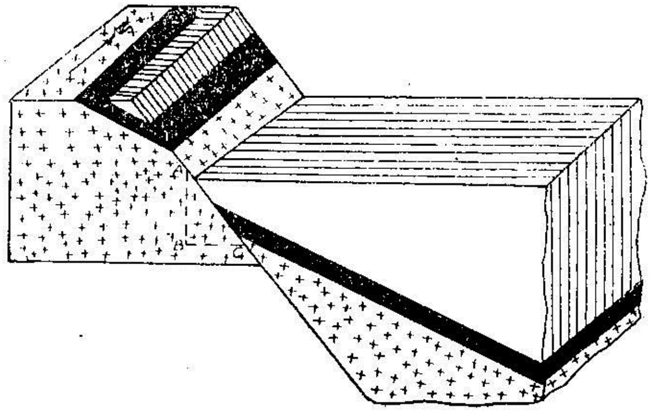


Fig. 3·5



Dip $\angle ACB = 50^\circ$
Hode $\angle CAB = 40^\circ$
Fig. 3·6

the strike of the fault is north-south and the fault is vertical. In Fig. 3·6 the fault is an inclined one and its dip is 50° towards east. The hade of a fault is the angle subtended by the fault plane to any vertical plane, striking in the same direction. Head and dip of a fault are thus complementary to each other.

The total displacement of the block due to a fault is known as its net slip. Movement of the block due to fault may be along the

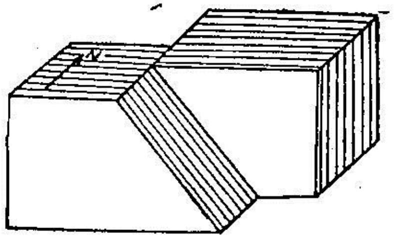


Fig. 3·7

strike or along the dip. In Fig. 3·7, it is a strike slip and in Fig. 3·8 it is a dip slip.

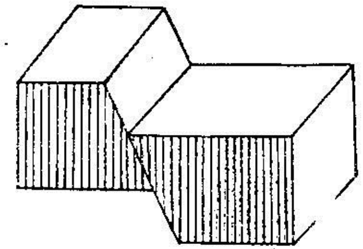
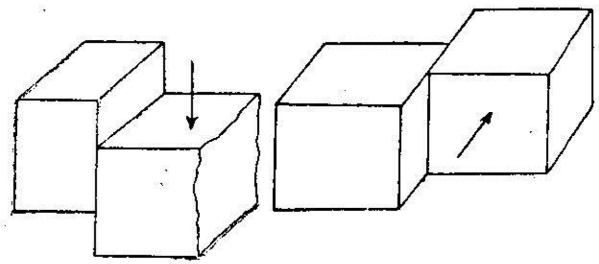


Fig. 3·8

Thus, slip along the true dip of the faulting plane is known as a dip slip whereas the slip parallel to the strike of the fault is known as a strike slip.

Depending upon the forces acting inside the crust the fault may be horizontal, or vertical as shown in Fig. 3·9 or it may be inclined



(*b*) Vertical Fault (*a*) Horizontal Fault
Fig. 3·9

as shown in Fig. 3·8. Since the fault is generally caused due to shear failure inclined faults are more common. In otherwords it can be said that since the shear failure occurs along an inclined plane, faults occurring in nature are generally inclined. Thus the two blocks on each side of the inclined shear plane appear to rest on the other. The block which seem to support the other block is known as a foot wall side and to other one being supported on foot wall side is known as a hanging wall side as shown in Fig. 3·10.

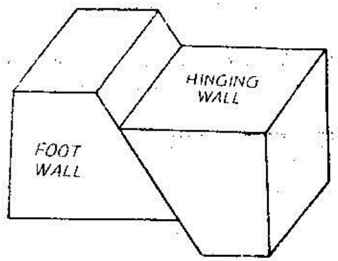


Fig. 3·10

Types of Faults

Considering the relative positions of footwall side and handing wall sides, the fault is classified in two types.

(*a*) Normal fault.

(*b*) Reverse fault.

When the hanging wall side appears to have moved relatively downwards in comparison with the adjoining footwall side, it will be called a normal fault. In a normal fault, the dip of the fault plane is usually more than 45°. In normal faults, the fault is caused mainly by vertical pressure as shown in Fig. 3·11.

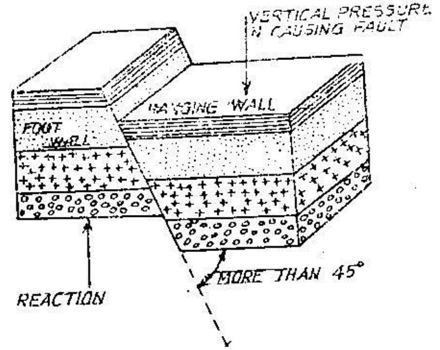


Fig. 3·11

When the footwall side appears to have shifted downwards in comparison with the adjoining hanging wall side, the fault is known as reverse fault. In such cases, the hanging wall side appears to have risen and dip of the fault plane is usually less than 45°. The reverse fault is usually caused by horizontal thrust as shown in Fig. 3·12.

When an engineering project has been taken up and a fault has been detected below the foundation, it is necessary to know whether

the fault, which has occurred, may have movements in future. In fact it is very difficult to predict such phenomena. However, for

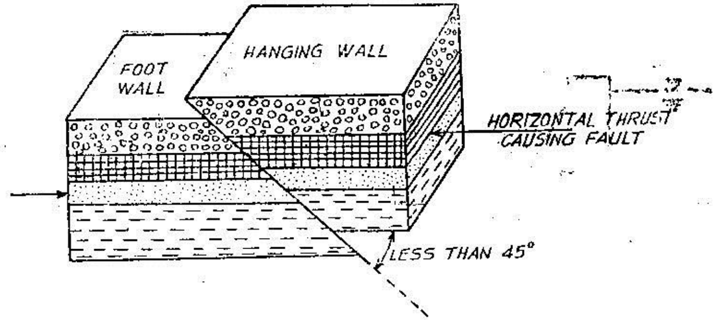


Fig. 3·12

precaution against such hazards, generally the faults are termed as Active and Inactive faults. Active faults are those in which the movements have occurred during the recorded history of mankind and in such cases further movements can be expected at any time. Inactive faults are also called as passive faults. When there is no recorded history of movement along the fault, it is known as an inactive fault. Such faults are generally assumed to be of static condition in future.

For important civil engineering works, such as the foundations of a bridge piers and masonary dam, thorough geophysical investigation is most important for an useful life of the project. For example, if a bridge foundation is put over a fault, as shown in Fig. 3·13,

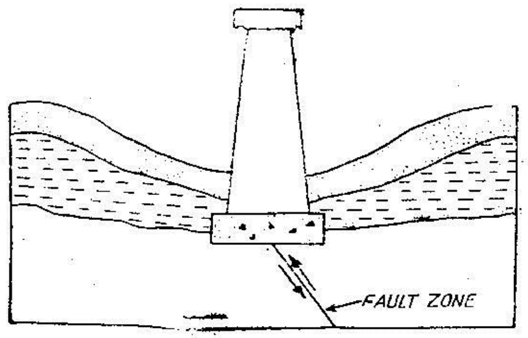
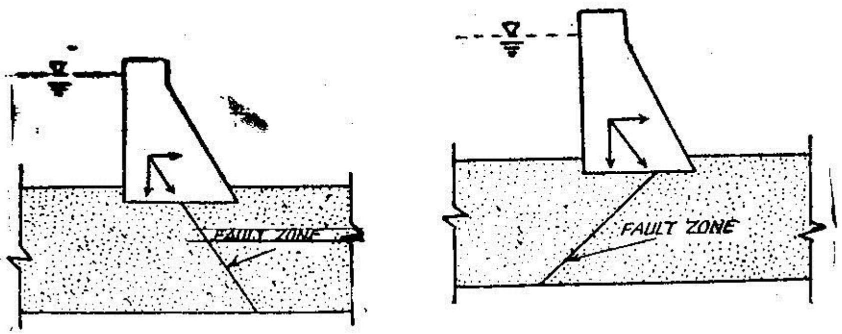


Fig. 3·13

there is a chance of sinking of the bridge pier due to the superstructure load because there might be sliding along the faulted zone.

Similarly, if a masonry dam is put on a faulted zone as shown in Fig. 3·14 (*a*), there is a chance of danger to the structure as sliding might take place due to a resultant force *R* as shown in the figure.



(*a*) (*b*)

Fig. 3·14.

In such cases foundation bed is improved by grouting but if the location of the fault zone is as shown in Fig. 3·14 (*b*) danger to the structure may not be there.

Hence, with the above two examples, it is clear that location of faults or a fault zone is necessary before starting a project. But search for fault is not always effective. Sometimes, they are discovered during the construction. Faults might be burried deeply. If the foundation excavation has been started and faults containing gauge and bracciated rock is met with, then it is better to abondon the site from economic point of view. Because excessive excavation may be required to remove the fault or fault zone which may continue up to a great depth, or a ground improvement may be necessary. However, a proper decision depends upon other factors also.

3·7. Folds

Folds are wavy undulations which are developed in the country-rocks when the region is subjected to high stresses. The wave like form is made up of a series of alternate crests and troughs.

3·7·1. Modes of Folds

There are three fundamental modes of folding.

(1) Concentric folding

(2) Cleavage

(3) Flow.

Concentric Folding means that all internal movements are parallel to the bedding plane. It is fundamentally an elastic bending of an originally horizontal sheet, with development of parallel concentric shear planes in the flanks of the folds. The deformation may

be called elastic-viscous, and is common in the upper part of the earth's crust, but it is not restricted to the upper part only. Concen-

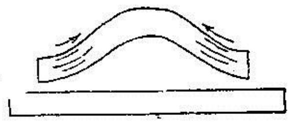


Fig. 3·15 (*a*) concentric Fold.

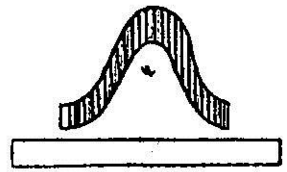


Fig. 3·15 (*b*) Elevage Fold.

tric folding is also known as parallel folding or distance-true folding, because the thickness of any concentrically folded bed remains unchanged.

Cleavage is a process by which all internal movement is along shear-planes which do not change their position during the process of folding. Normally they are perpendicular to the deformative stress. The process is a dilatation in a vertical direction and a compression in a lateral direction. Generally, the deformation is elastic-viscous where the elastic properties have not yet lost their influence, but it is almost near the plastic range. Cleavage is common in the lower region of *orogenic* belts [Fig. 3·15 (*b*)].

Flow is a kind of distortion in which fixed orientation between shear-planes and stress direction is lost. The internal movement is not oriented in any direction. It can take place in any direction with the result that it can no longer be represented adequately in a section. It is common in weak rocks like salt or in other rocks at very high confining pressures or high temperature The deformation of rock in such a case is plastic [Fig. 3·15 (*c*)].

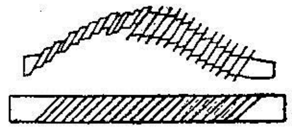
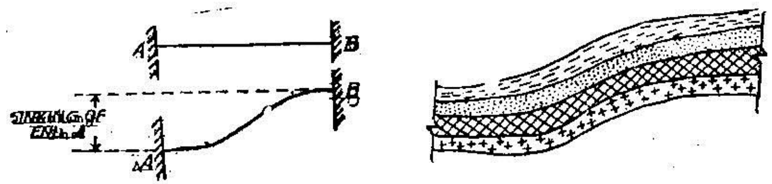


Fig. 3·15 (*c*) Oblique Shear Fold.

Combinations of cleavage and concentric folding are very common.

3·7·2. Types of Folds

The simplest kind of a fold is the monocline. It can be compared with a flexible beam firmly embedded at two ends. If the



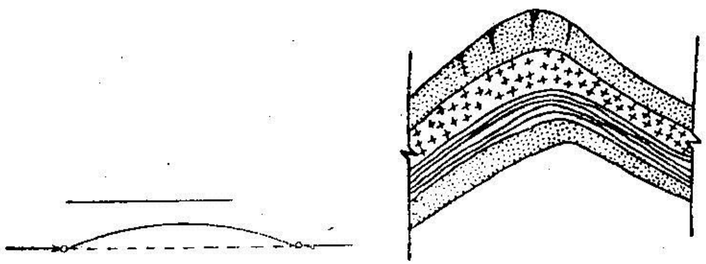
(*a*) (*b*)

Fig. 3·16.

support sinks a point of inflection is formed as shown in Fig. 3·16.

Monoclinal fold may be compared with the deflected shape of the beam. Such folds are formed due to sinking of a part of the basin. Such types of folds are also termed as bedding folds.

When the fold appears in the wave form having crests and troughs, anticlines and synclines are formed. Anticlines are crests and synclines are trough of the wave form folds. In case of very large folds, consecutive crests or troughs may be hundreds of kilometres apart. The smallest folds may have several crests of troughs within a span of a few metres. The portion of rock beds laying between any crest and the adjacent trough is known as the limb of the fold. Such type of folds are known as buckling folds.



(a) (b)

Fig. 3·17

The mechanism of buckling fold can be compared with the buckling of a column as shown in Fig. 3·17 and 3·18.

An over loaded column may deflect as in Figs. 3·17 (*a*) or 3·18 (*a*). The vertical force acting on the column as shown in Figs. 3·17 (*a*) and 3·18 (*a*) can be compared with horizontal stresses acting on the rock mass inside the crust. When deflection is as shown in Fig. 3·17(*a*) Anticlines may be formed, and when deflection is an shown in Fig. 3·18 (*a*), a set of anticlines and synclines are formed. Some other theories also have been given to explain the phenomena of fold. According to this theory a change in the shape of mass is due to action of shearing stresses and not due to a compressive force. The horizontal shears may be compared with the horizontal couple, and this horizontal couple has to be counter balanced by a vertical couple. The vertical stresses are caused due to upward acting force of the vertical couple, and this causes wrapping in the upward direction which is called anticline and similarly the downward acting force of the couple will cause wrapping downward, and thus, synclines are formed. Due to folding, there will be stretching at the top of anticlines and bottom of synclines. At the top of anticlines there is no resistance to such

Fig. 3.18 (*a*)

stretching whereas at the bottom of synclines there is a resistance to stretching due to lower strata. Hence, more tension cracks develop

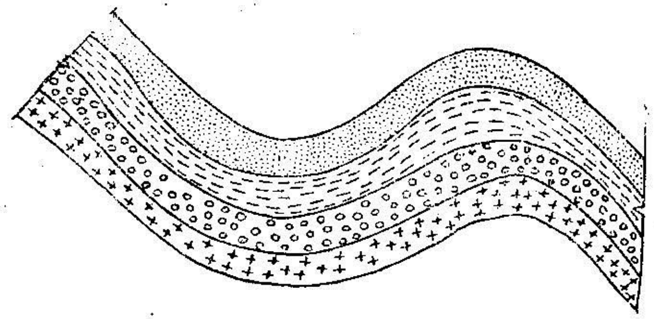


Fig. 3 18 (*b*)

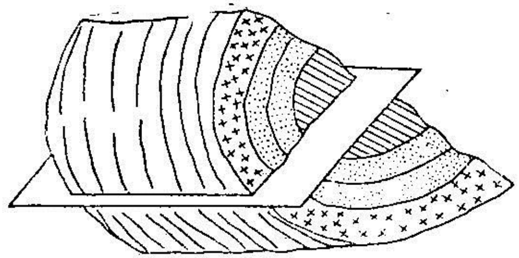


Fig. 3 19 (*a*)

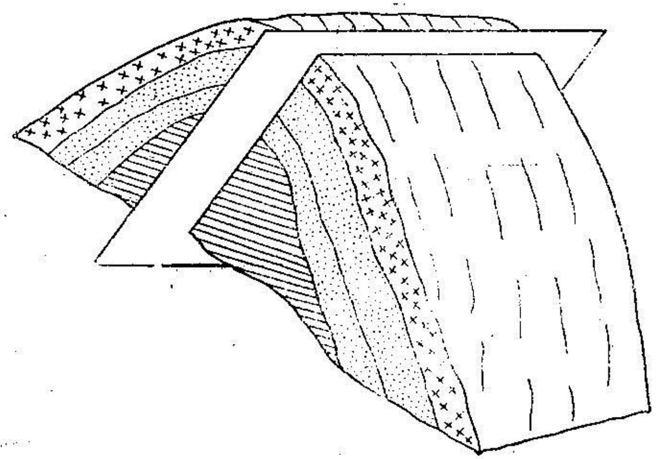


Fig. 3·19 (*b*)

at the top of anticlines. The extent of cracks depends upon nature of folding, type of rock and forces causing folding.

When the axial plane of the fold is vertical and both the limbs have the same amount of dip the fold is known as symmetrical. If the limbs have unequal dips the fold is described as asymmetrical.

3·7·3. Importance of Folds for Engineering Projects

Folds do not cause serious problems in case of foundations for buildings. But in case of dam foundations, reservoir locations and

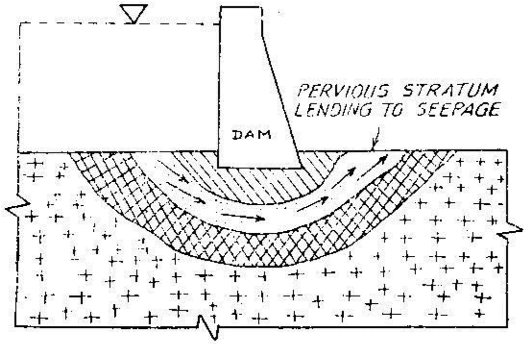


Fig. 3·20

tunnel locations, significances of folds are there. When a monocline containing pervious strata dips down stream, there will be excessive seepage below the dam from the upstream side to the down stream side as shown in Fig. 3·20. However, there is no such danger if the monocline dips upstream as shown in Fig. 3·21.

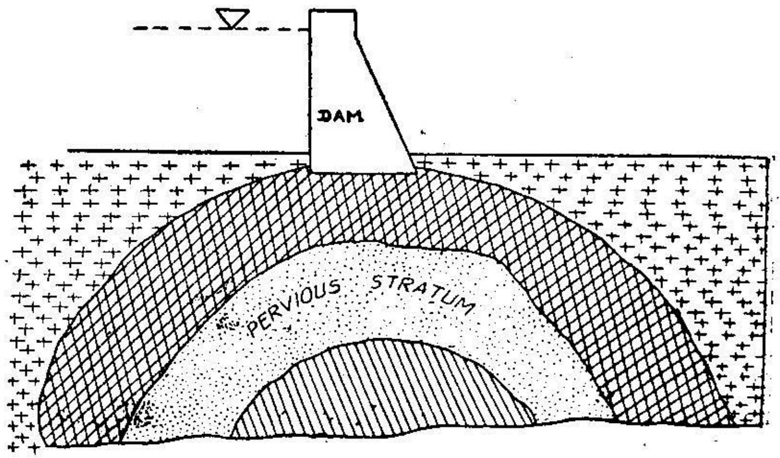


Fig. 3·21

If a tunnel passes through a cyncline, the maximum pressure is expected at the middle points of the tunnels and if pervious strata

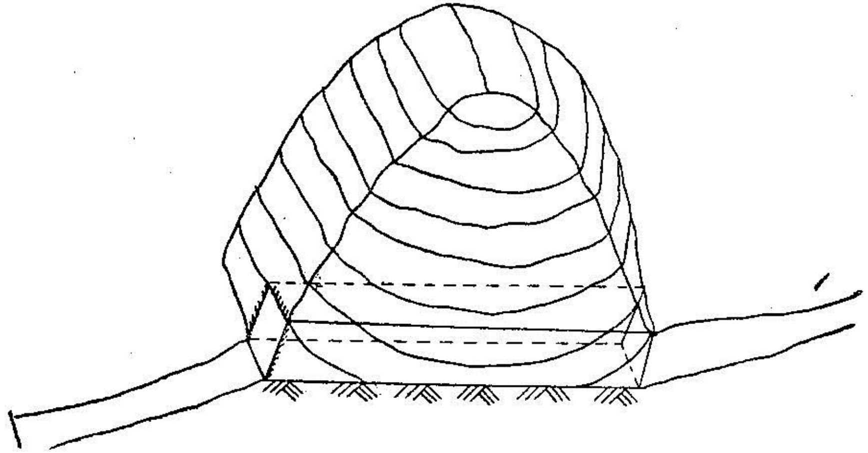
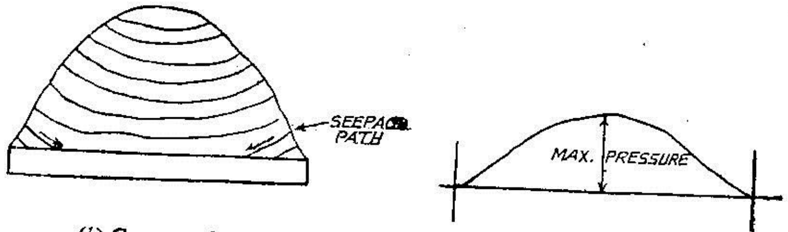


Fig. 3·22 (*a*) Tunnel Driven through a Cyncline.



(*b*) Cross-section. Fig. 3·22 (*c*) Pressure on Tunnel.

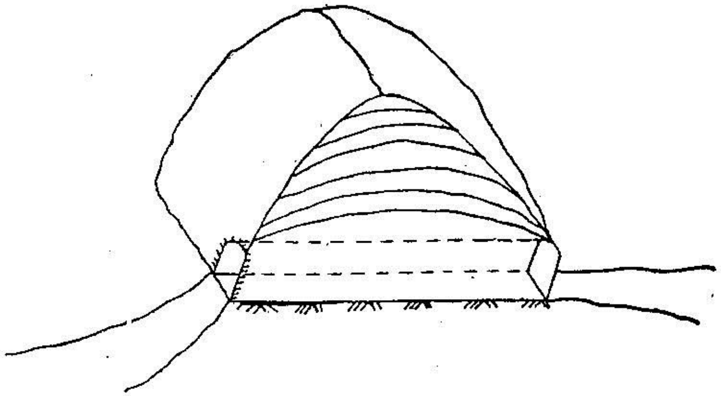


Fig. 3·23 (*a*) Tunnel Driven through Anticline.

exists in the layers, the seepage will be directed inside the tunnel which causes serious problems for the maintenance (Fig. 3·22) whereas, in case, an alignment passes through anticline, the pressure will be directed at the two ends due to an arching action, and maximum pressure will be at the two ends, and at mid-point the pressure will be minimum. Seepage due to pervious strata also will be directed at the two ends (Fig. 3·23).

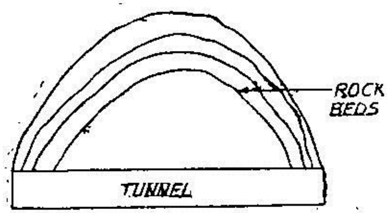


Fig. 3·23 (*b*) Cross-section.

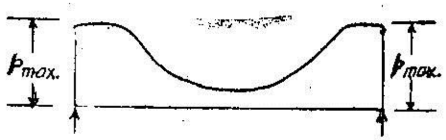


Fig. 3·23 (*c*) Pressure Distribution on Tunnel.

Thus, it is clear from the above examples that thorough information about folds is necessary for a successful performance of an engineering project.

4

Methods of Rock Exploration

(Geophysical Prospecting)

4·1. Introduction

Careful design of an engineering structure is not completed unless full information about environments of the strata, on which the structure is going to stand is available. Intelligent design for safe and economical construction requires thorough knowledge of subsurface conditions. Many useful engineering details are obtained from geological maps. Geological maps give information about the rock units directly underlying the project area. The characteristics of rocks are of prime importance in the selection of water retaining structures and dams. Conditions beneath the surface can often be correctly predicted by the three dimensional information given on geological maps. On geological maps rocks are identified by name and geological age. A formation is an individual bed or several beds of rock that extend over a large area, and that can be clearly differentiated from overlying or underlying beds because of a distinct difference in lithology, structure or age. Geological maps often carry one or more geological sections which describe the disposition of the various strata in depth along an arbitrary line usually marked on the map. These sections are somewhat hypothetical and is to be used with caution. They give a general strata condition and an exhaustive information can not be obtained with the study of geological map of an area. That is why for any important engineering project such as dams, multistoreyed buildings, or a massive structure, it is necessary to know the strata conditions in detail below the construction site. Some times minor modifications may serve the purpose and some times the site may be shifted if the strata condition is not favourable. Therefore after completion of the Civil Engineering Survey—that is survey of the surface ground—it is necessary to do survey of the substrata which may be said to the geological and rock mechanics survey. It is said that strong rocks such as granite are suitable for almost all engineering purposes, but it is not always true. Much depends on details of jointing, weathering, permeability, ground water conditions etc.

4·2. Object of Rock Exploration

While conducting geological or substrata survey following item of information are required to be described and measured.

(*i*) Homogeneous zones and their extent.

(*ii*) Rock type and their engineering properties.

(*iii*) Geological separations and their geomechanical significance.

(*iv*) Width of separations with details of fillings.

(*v*) Joint surfaces.

(*vi*) Determination of the degree of jointing, spacing of joints and their two and three dimensional extent.

(*vii*) Water conditions in respect to both rock material and joint and fault material.

Information of homogeneous zones and their extent gives us an idea of suitability of the strata as a supporting member of the foundation if the structure is to be built on or above the strata and some times they give an idea of materials available for a particular construction work if the structure is to be built with the material available along the strata. For example—extent of material available for a masonary dam.

Information about the Rock Type and their engineering properties helps the designer in selecting the type of foundations and limit of their dimensions. And details of joints, faults etc. caution the designer in selecting the dimensions of the foundations and to see if some treatment of sub-strata is required or not. Some times treatment of fractured sub-strata becomes absolutely necessary for successful life of the project.

If we summarise the objective of subsurface exploration then it can be said that the objective of sub surface exploration are to obtain quantitative data on the kinds, properties, amounts, distributions and structure of the material under and adjacent to a proposed structure.

4·3. Methods of Rock Exploration

Two groups of methods are available to obtain these data.

(*i*) By direct penetration of the materials.

(*ii*) By making certain physical measurements from the surface without direct penetration and then to interpret those data for ascertaining the above required information.

Before starting any engineering geological survey it is beneficial to study the available literature and official records on the geology and hydrogeology of the area. This helps to establish the extent of geological and hydrogeological exploration to be done on the construction site and correctly outlining a definite programme of the required survey work by determining the required number of exploration bore holes. The number of such bore holes and their depth depends upon the complicated nature (complexity) of the geological structures and suspectibility of the strata to settlement.

4·4. Rock Exploration by Direct Penetration of the Material

Instead of some indirect methods for ascertaining any property of a material it is always better to have some direct method. Direct penetration into the rock mass helps in direct observation of its structure with which its property can be inferred to a great reliability. Therefore, boring is the best method for rock or sub-surface exploration.

Borings are of two types

1. Core borings.
2. Large diameter calyx holes.

4·4·1. Core Borings

The primary purpose of core boring is to obtain samples of the materials penetrated and to get inference about the lower strata by direct observation and preliminary testing of the core samples. Core boring can be done in almost every type of rock and the samples are machined cylinders of the materials penetrated. Core holes are mostly vertical but in some exceptional cases inclined borings are also done.

The equipment commonly used are known as drilling rings and are run by variable speed machines. Drilling fluid is used inside the hole to progress further deep. Sometimes compressed air drilling is also done for greater depths. The bit makes an annular opening in the rock and as drilling continues the core barrel slides down over the central core of rock material. When the core barrel is raised a catcher holds the core in place and the core is raised with the core barrel.

Cutting tools commonly used are diamond bits and steel alloy toothed cutters.

Diamond bits cut even the hardest rocks but tend to clog in softer materials. Steel-alloy tooched bits are generally better adopted to shales and soft rocks and operates satisfactorily in dense and unweathered basalt. The diameters of cores with diamond or steel alloy bit range from 22 mm to 100 mm. Length of the core cut depends on core barrel.

4·4·2. Core Recovery

The success of core-drilling is measured by the percentage of core-recovered. Sometimes a good operator abtains between 90 to 100 percent recovery if the rock penetrated is sound. Bad rock conditions cause blocking of the core in the barrel and consequent grinding of the core. If the core recovery is not good it should be checked at the time of drilling how losses are occuring. From behaviour of the drill rig and character of the returning drilling fluid and cuttings, fracture systems in the rock mass very poor recovery is obtained. To get a correct picture of fracture etc. inside the rock mass sufficient number of borings are done and on the basis of core recovery from different bore holes inference is made.

4·4·3. Rock Quality Designation (R.Q.D.)

We have seen in the above para that good core recovery represents good quality of rock whereas poor core recovery represents poor rock strata. That weakness might be due to poor strength of the rock or due to presence of joints, fissures, faults etc. inside the rock mass. Of course this can be ascertained by observation of the fresh core being recovered.

Now-a-days designers, consulting engineers and geologists in several countries use the "R.Q.D." as yardstick for evaluating the quality of rock at site R.Q.D. stands for Rock Quality Designation.

When a drilling has been done for a particular depth the cores coming out of it are accounted for. Then total length of core recovery (that is the rock which has come up) is measured. Then ratio of total length of core recovery and length of core run *i.e.*, drilling done, is expressed as core recovery ratio. The ratio is expressed in percentage.

$$\therefore \quad \text{Core Recovery} = \frac{\text{Total length of core recovered}}{\text{Length of drilling done}}$$

In this procedure length of all the cores whatever be their length, are counted. But a new system was adopted in which the length of rock cores which are less than 10 cm are not counted and the core recovery ratio now expressed in percentage are known as Rock Quality Designation (R.Q.D.). In such a case the effect of rock weaknesses are taken into account. Because if the rock will be weak, then cores will not be of bigger length. At the same time due to joints, fracture etc. also the cores will be of lesser dimensions thus R.Q.D. gives a good assessment for the quality of rock. The cores might be broken during handling and drilling processes also. In such cases the surfaces may be having fresh irregular breaks. In such cases, fresh broken pieces are fitted together and counted as one piece provided they form the length of 10 cm or more. Natural joint surfaces can be identified looking at the broken core samples and generally in such cases pieces can not be fitted together to give an appearance of one piece. Per running metres the number of fractures may also be counted looking at the recovered core but the result may not be good. Hence R.Q.D. has been taken as yard stick to represent the quality of rock. Table 4·1 gives description of rock strength based on R.Q.D.

Fig. 4·1 explains determination of Rock Quality Designation (R.Q.D.) from the cores obtained from a bore hole.

Due to presence of bedding planes, some times correct inference is not obtained by this method for rock strata having sedimentary beds but for igneous and metamorphic rocks more exact inferance is made.

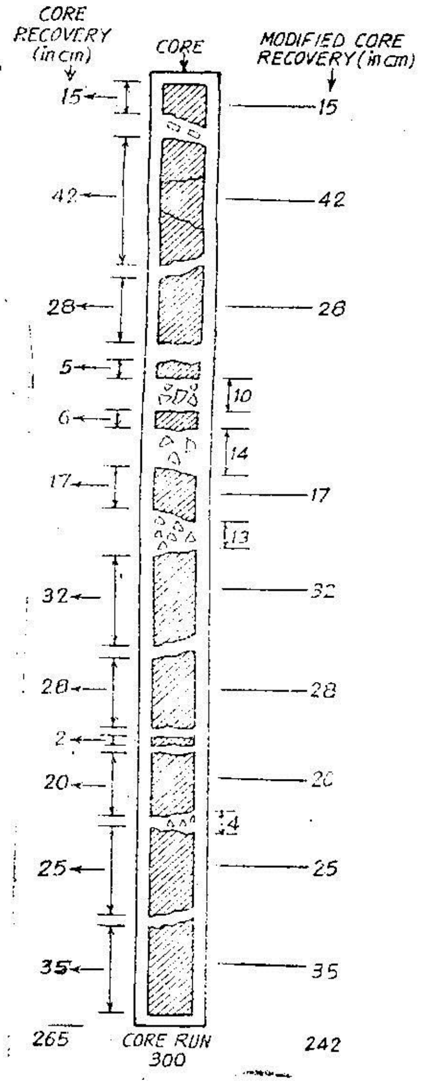


Fig. 4·1.

Total Core Recovery
$= 265$ cm

Total Core Run
$= 300$ cm

Modifien Core Recovery
$= 242$ cm

Core recovery
$= \frac{265}{300} = 88{\cdot}3\%$

R.Q.D.
$= \frac{242}{300} = 80{\cdot}7\%$

As per table 4·1 Rock Quality Description
= Good (For R.Q.D. = 87%)

Note –
Cores for 10, 14, 13 and 4 cm resp. have come in small pieces. Hence not counted for R.Q.D.

Table 4·1

R.Q.D (Rock Quality Designation) (%)	*Rock Quality Description*
0—25	Very poor
25—50	Poor
50—75	Fair
75—90	Good
90—100	Excellent

4·4·4. Fracture Frequency

For description of quality of rock some times fracture frequency is used. Natural discontinuities in fracture per foot or per metre run is described as fracture frequency.

4·5. Large Diameter Calyx Holes

Although this method is costly, yet it gives a correct picture of the strata condition inside the hole. By this method the rock can be examined in-situ. With the help of camera, photographs are taken inside the hole for different sub-surface strata and inferance is drawn with those photographs. Sometimes, electronic equipments are also used as sub-surface probe.

4·6. Logging of Cores

Logging of cores means keeping the rock cores (which come out of the bore holes) in porper sequence with detailed descriptions. Cores are of no value unless properly logged. In order to write a good log report the site incharge must understand drilling methods and comprehend the engineering purposes for which the work is to be undertaken. For example if bearing piles are to be located then determination of elevation of the depth of sound rock mav be sufficient. If investigation for a masonary dam is being done, then location of faults, joints and fractures are also necessary.

Percentage of core recovery is to be determined after each pull and effort to be made to ascertain the cause of losses of the core. If the drill stem drops suddenly and circulation of drilling fluid is lost temporarily it can be inferred that core loss may be due to an open cavity. If drilling stem drops at faster rate than the normal while circulation of drilling fluid is normal but it becomes muddy then core loss may be due to grinding or washing away of the core, which means that in the zone, there might be fracture filled with silt which was washed away due to drilling fluid. For good reporting and preparation of good boring log, it is necessary that reporter should always be present at the site and must be able to make proper interpretations. A typical example of a boring log has been shown in Table 4·2.

4·7. Geophysical Prospecting

Geophysical prospecting may be described as Geophysical exploration. This is a type of field investigation in which physical measurements are done at the ground surface to obtain information about sub-surface. In this method of exploration, physical measurements which are based on principles of physics are interpreted in terms of sub-surface geological structure and lighology. In fact, an engineer is satisfied with bore holes only from which he gets a physical verification of the substrata condition. But making many bore holes for a project may be a costly affair. In such a case, geophysical methods are adopted which gives an qualitative idea of the sub-surface condition at comparatively lower cost and then, bore holes are done at determined places to have a correct picture. Hence bore holes and Geophysical methods may be considered as complementary to each other for an engineering project.

In a broad sense, geological prospecting is done for the following three purposes.

(*a*) Determination of extent of mineral deposits or a construction material.

(*b*) Determination of rock structure and lighology.

(*c*) Detection of underground metal or pipe lines etc.

4·8. Methods of Geophysical Prospecting

The properties of rocks which are mostly used in geophysical prospecting are density, magnetic susceptibility, elasticity and electrical conductivity. The four principal methods of geophysical exploration are :

(*i*) Seismic, (*ii*) Electrical, (*iii*) Magnetic, (*iv*) Gravitational.

4·8·1. Seismic Methods

Of all the methods seismic methods is highly developed and most of the details are obtained correctly where it is employed. Rocks have different elastic properties and densities therefore elastic waves are propagated through them with different velocities. At the interfaces, the velocities are reflected and refracted because properties of the rocks change at the interface. An elastic wave generated by an impulse at the surface travels in side the rock mass but when an interface is met with, some part of it is reflected back to the surface and some after refraction travels through the lower rock mass. Travel time of the wave is observed at different points at the surface. And with the help of graph or other methods the depth of interface is obtained.

Seismic exploration is done by two methods :

(*i*) Reflection

(*ii*) Refraction.

CHAPTER 1

SUBSURFACE INVESTIGATION

1.1 INTRODUCTION

The primary objective, in civil engineering, of a subsoil investigation is to determine stratigraphy and pertinent physical properties of soils underlying the site so that a safe and an economical foundation may be designed.

Soil stratigraphy is most commonly determined by making borings, test pits etc. and collecting soil samples, disturbed and undisturbed, and carrying out necessary tests on these samples. Though boring is most widely used method of subsoil investigation, there are many other methods of subsoil investigation, and more common of these methods are discussed briefly in Section 1.2.

The characteristic of soils are generally variable and may change sharply within limited distances. Degree of thoroughness and completeness required of an investigation is linked with job requirements and availability of time and funds. In Sections 1.4 and 1.5 certain guidelines on extent of investigation desired are given.

Pertinent physical properties generally needed from an investigation are strength, compressibility and permeability. Often the chemical nature of subsoil and ground water may be desired to evaluate hazard of corrosion on the foundation structure. Table 1.1.1 lists the various soil properties that may be used in analyses and designs involving applications of principles of soil engineering.

Physical properties of soils may be evaluated from in-situ test and also from laboratory tests on undisturbed, disturbed and/or remoulded soil samples. In Section 1.7, certain methods for in-situ measurements of soil properties are very briefly described whereas Section 1.8 deals with measurements of soil properties in the laboratory. Requirements for obtaining, undisturbed samples of soil are given in Section 1.6.

It is imperative that the data obtained from field and laboratory investigation is presented in a systematic manner. Table 1.24.1 provides a very convenient format in furnishing data collected from a borehole and/or trial pit. Such a presentation not only helps in finding out discrepancies, if any, between different results obtained but is also very useful in assigning appropriate values to different soil parameters needed in the design.

To achieve the objective stated at the very beginning of this section, soil investigation may have to be carried out in stages. On initial broad determination of stratigraphy and physical properties, particular zone may be investigated in greater detail. Furthermore, it is desirable that information predicted from soil investigation carried prior to construction work is compared with information revealed say by excavation etc. during construction work. If there is significant variation between reality and prediction, then further investigation may be necessary to recheck the design considerations. It may also be necessary to carry out certain post-construction observations to ensure that assumptions made in design are satisfied. Observations for pore water pressure, settlement etc. after an embankment is constructed can be included in this category. Such post construction observations may not be explicit in the objective stated earlier but should form part of the general soil investigation.

TABLE 1.1.1. Soil Properties for Analysis and Design

Property	*Usual or Recommended Symbol*	*Units*[1]	*How Usually Obtained*	*Direct Application*
Volume Weight Characteristics[2]				
Moisture Content	w	D	From test	Classification and in volume weight relationships.
Density	γ	FL^{-3}	From test	Classification and pressure computation.
Porosity	n	D	Computed from volume weight relationships	Parameters used to represent relative volume of solids in given volume of soil.
Void ratio	e	D		
Specific Gravity	G_S	D	From test	Volume computations.
Plasticity Characteristics				
Liquid limit	w_L	D	From test	Classification and property correlationships.
Plastic limit	w_P	D	From test	" "
Plasticity Index	I_P	D	$I_P = w_L - w_P$	" "
Shrinkage limit	w_S	D	From test	Classification and computation of swell.
Shrinkage index	I_s	D	$I_P = w_L - w_S$	" "
Liquidity index	I_L	D	$I_L = \frac{w - w_P}{I_P}$	Study of field behaviour. Estimation of degree of pre-consolidation.
Consistency index	I_C	D	$I_C = \frac{w_L - w}{I_P}$	Study of field behaviour.
Activity	A	D	$A = \frac{I_P}{\%\text{ clay fraction}}$	Identification of clay mineral and computation of swelling potential and swelling pressure.
Gradation Characteristics				
Effective Diameter	D_{10}	L	From grain size curve	Classification, permeability, and/or filter design.
Per cent grain size	D_{15}	L	From grain size curve	" "
	D_{30}	L	"	" "
	D_{50}	L	"	" "
	D_{60}	L	"	" "
	D_{85}	L	"	" "
Uniformity coefficient	U_C	D	$U_c = D_{60}/D_{10}$	Classification and filter design.
Coefficient of curvature	C_C	D	$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$	
Clay size fraction	C	D	From grain size curve	Classification and property correlation.

TABLE 1.1.1 *(Contd.)*

Property	*Usual or Recommended Symbol*	*Units*[1]	*How Usually Obtained*	*Direct Application*
Drainage Characteristics				
Coefficient of permeability	k	LT^{-1}	From permeability or consolidation test.	Drainage, seepage and consolidation analysis.
Capillary head	hc	L	From test	Drainage and drawdown analysis.
Consolidation Characteristics				
Coefficient of compressibility	a_v	L^2F^{-1}	From e vs $\bar{\sigma}$ curve	Computation of settlement under loading.
Coefficiency of volume compressibility	m_v	L^2F^{-1}	$m_v = \frac{a_v}{1+e_0}$	" "
Compression index	Cc	D	From e vs $\log \bar{\sigma}$ curve	Computation of settlement under loading.
Recompression index	C_R	D	"	" "
Swell or expansion index	C_e	D	"	Computation of heave on release of overburden pressure.
Coefficient of consolidation	c_v	L^2T^{-1}	Time consolidation curve	Computation of time rate of settlement.
Coefficient of secondary compression	C_α	D	From semilog plot of time consolidation curve	Computation of secondary compression.
Preconsolidation pressure	σ_P	FL^{-2}	From e vs $\log \bar{\sigma}$ curve	Consolidation analysis; to analyse consolidation stage of subsoil.
Strength Characteristics				
Angle of internal friction	ϕ	D	From strength test	Analaysis of stability and load carrying capacity based on strength.
Cohesion intercept	c	FL^{-2}	Mohr envelope for total normal stress	
Angle of internal friction	ϕ', ϕ	D	From strength tests	Analysis of stability and load carrying capacity based on strength.
Cohesion intercept	$c', \bar{c}$	FL^{-2}	From Mohr envelope For effective normal stress	
Unconfined compression strength	q_u	FL^{-2}	From test	" "
In-situ shear strength	s	FL^{-2}	In-situ vane shear tests	" "
Blow count from SPT, continuous dynamic cone penetration tests	N, N_c	L^{-1}	From tests	Empirical relationship of strength and compressibility characteristics.
Cone resistance from static cone test	C_{kd}	FL^{-2}	From test	" "
Bearing capacity factors	$N_c, N_q, N_\gamma, N_c', N_q', N_\gamma'$	D	From ϕ value	Bearing capacity.
Sensitivity	S_t	D	$\frac{(q_u)\text{ undisturbed}}{(q_u)\text{ remoulded}}$	Estimating effect of disturbance of structure on strength.

TABLE 1.1.1. (Contd.)

Property	*Usual or Recommended Symbol*	*Units*[1]	*How Usually Obtained*	*Direct Application*
Modulus of elasticity	E	FL^{-2}	From tests	Computation of settlement for dynamics analysis.
Lateral earth pressure coefficients	K_a, K_p, K_o	D	From appropriate equations	Computation of lateral earth pressures.
Characteristics of Compacted Soils				
Maximum unit weight	γ_{dmax}	FL^{-3}	From test	Compaction control, computation of stresses, forces in stability analysis
Optimum moisture content	w_{omc}	D	From test	Compaction control, to estimate strength parameters.
Relative density	D_r	D	From test, empirical relations	
California bearing ratio	CBR	D	From test	Pavement design.

1. Units. F = force or weight = MLT^2; L = length; T = time; M = mass; D = dimensionless.
2. For complete list of volume-weight relationships refer Section 1.21.

Thus subsoil investigation in general may consist of the following four stages.

(*a*) Initial studies and explorations to determine soil stratification and characteristics required for design.

(*b*) Amplification, if necessary, of specific portions of the initial investigation to obtain more complete information, as desirable during the design phase.

(*c*) Verification of anticipated foundation conditions during construction so that changes may be made, if necessary, to ensure proper performance and control for assurance of compliance with design.

(*d*) Observation of structure and soil performance following construction.

Item (*a*) and (*c*) are to be considered essential. Items (*b*) and (*d*) may be limited or even eliminated, depending on the nature of the project.

1.2. METHODS OF SUBSURFACE INVESTIGATION

Table 1.2.1 summarizes the various subsoil exploration methods presently available.

1.3. GEOPHYSICAL EXPLORATION: SEISMIC REFRACTION METHOD

1.3.1 Introduction

Different methods of geophysical explorations as noted in Table 1.2.1, are seismic, electrical resistivity, magnetic, gravitational and sonic methods. Their applicability has also been briefly stated in Table 1.2.1.

Of the various geophysical methods, seismic and resistivity methods are more widely used.

Seismic methods of subsurface investigation are based on the principle that velocity of propagation of a wave or energy impulse introduced in to the ground depends on the properties of material through which impulse passes. Velocities vary greatly in different materials ranging from loose sedimentary deposits to solid rock. This enables prediction or strata type from the determination of velocity of propagation.

Two types of seismic waves viz **shear** or **S** waves and **longitudinal or compression** or **P** waves are transmitted through subsoil and rock. The velocities of these waves are related to properties of transmitting medium by the following equations:

TABLE 1.2.1. Subsoil Exploration Methods

Method	*Descriptions*	*Applicability*
(1) Aerial Photography		For intensive investigation in accessible area aerial photography is not essential for soil exploration. For in accessible and unfamiliar areas air photography may be adopted as an aid in planning for detailed exploration work.
(2) Geophysical Methods	They are grouped as (*a*) seismic (*b*) electrical (*c*) magnetic (*d*) gravitational and (*e*) sonic.	
(*a*) Seismic method	Shock or seismic waves are created by detonating small charges or by striking a rod or a plate near the surface. The radiating waves are picked up and time of travel from source recorded by detectors known as geophones or seismometers. In seismic method, either refracted or reflected waves are detected.	
(*i*) Refraction method[1]	In this method, time of arrival of waves refracted at interfaces between different strata are recorded.	Used to determine depth to rock or depths of significantly differing soil strata. Can be used only when velocity of travel in lower layers is significantly greater than the upper ones. This method is usually limited to depths up to 30 m in a single stratum.
(*ii*) Reflection method	Here seismometers record the travel time of seismic waves reflected from interface between adjoining strata.	This method is usually adopted to determine depth of deep bed rocks. Generally applied for depths exceeding 600 m. At present this method is mainly used in offshore investigation.
(*iii*) Velocity sounding methods	In this method, seismic waves are generated. Their travel times and hence travel velocities in travelling through soil along the hole in down or up direction or across the holes are determined.	These methods are used for determining dynamic, elastic and shear modulus which enable to estimate coefficient of elastic uniform compression etc.
(*b*) Electrical resistivity method	In this method four metallic spikes to serve as electrodes are driven into the ground at equal intervals along a line. A known potential is then applied between the outermost electrodes and potential drop is measured between the innermost electrodes. Flow of electric current is also measured. This enables to estimate resistivity of stratum. From known resistivity of different strata, prediction can be made about the nature of the stratum.	Used to determine vertical as well as horizontal extent of soil strata at foundation site for large structures, such as dams. Depth of exploration is generally limited to about 30 m. Also used to obtain data for designing electrical grounding system.
(*c*) Magnetic method		Rarely used in civil engineering works.
(*d*) Gravitational method		Rarely used in civil engineering works.

TABLE 1.2.1. (Contd).

Method	*Descriptions*	*Applicability*
(*c*) Sonic method	In this method time of travel of sound waves reflected from certain boundaries between different strata are measured. From the knowledge of velocity of sound wave in different strata, depth to strata can be obtained.	Used to determine position of mud line and depth to hard stratum underlying mud. Use is currently limited to shallow depths.
(3) Test Pits, Trenches and Open Cuts	Test pits, trenches or any other type of open excavation can be carried out manually or by machines. The sides of open cuts need be provided with lateral support with the help of bracings whenever there is a danger of cave-ins.	Applicable in all soils; provide for visual examination of soil strata in their natural conditions; disturbed or undisturbed samples can be conveniently obtained at required depths. Use is usually limited to shallow depths of 3 to 5 m. For greater depths cost of open excavation and necessary side bracings becomes uneconomical.
(4) Borings	Principal boring types are: auger boring, wash boring, percussion drilling and rotary drilling.	Refer respective type of boring.
(*a*) Auger boring	Bore hole is advanced by hand or power operated auger with periodic removal of material. In some cases continuous auger may be used requiring only one removal. Casing is generally not used with auger boring.	Hand augers are used in soft to stiff cohesive soils, in sandy silty soils above water table. With hand auger, depth is usually limited to 6 m. Power driven augers can be used to great depths, even to 30 m, and used in almost all types of soils above water table. This method provides almost continuous disturbed samples. Undisturbed samples can be obtained at required depths by using proper samplers.
(*b*) Wash boring	Bore hole is advanced by chopping, twisting action of a light chopping bit and jetting action of a drilling fluid, usually water, under pressure. Changes in soil strata are indicated by changes in the rate of progress of boring, examination of outcoming slurry and cuttings in the slurry. Casings are used whenever necessary to prevent cave-ins.	Can be used in all types of soils except those containing boulders; can be used to great depths, adopted easily at inaccessible locations. Samples obtained are in highly disturbed and slurry form; undisturbed samples whenever needed can be obtained by use of proper samplers.
(*c*) Percussion drilling	Bore hole is advanced by chopping action of heavy bit driven by power. Water is added at the bottom of bore hole during chopping action, if the ground water is not already struck. Slurry formed at bottom of bore hole is removed by bailer or sand pump. Casing is generally required. Changes in strata are predicted from the rate of progress of boring and examination of slurry bailed out.	Can be used in all soil types including soils containing boulders. Can be used for great depths. Use is limited because of difficulty in determining strata changes as chopping action can cause considerable disturbance and because of high cost. May be used in combination with auger or wash borings when boulders are encountered. Undisturbed samples whenever needed can be obtained by use of proper samplers.

TABLE 1.2.1. (Contd).

Method	*Descriptions*	*Applicability*
(*d*) Rotary drilling	Bore hole is advanced by power rotation of drilling bit and removal of cuttings by circulating fluids which may be water, bentonite slurry or mud slurry. Whenever rock or boulders are encountered suitable bits viz. diamond studed bits or tun gesten carbide bits or steel bits with shots are to be used for drilling. Casing may or my not be needed during drilling. Changes in strata are indicated by change in rate of advancing of bore hole, action of drilling tools, examination of cuttings in drilling fluid.	Can be used in all types of soils and rocks, can be used to great depths, being increasingly used because of fast rate of progress in all soil types, but difficult to use at increasible locations because of heavy machinery. Undisturbed samples can be obtained at desired depths by using suitable samplers.
(5) Sounding and Probing	In this method some sounding device like split spoon sampler, a cone, or a rod is forced, statically or dynamically, into the soil. The energy consumed in penetration in to soil is an indication of consistency of soil. Standard penetration test[1], dynamic cone penetration test[1] and static cone penetration test[1] are commonly adopted standard tests.	Can be used in any type of soil and to any depth. Depth by static cone penetration is limited by the capacity of equipment. Presently 3 tonne and 10 tonne machines are in use. Standard penetration test is in a borehole.
(6) Load Tests (*a*) Vertical plate bearing test[1]	Vertical plate bearing test is very common. In this method a plate of 30 to 75 cm. square or circular shape is loaded in increments to obtain load settlement curve.	Applicable in sandy soils, murum, weathered rock. To be used with caution in clayey soil. Depth of influence is limited by virtue of limited plate size. In absence of probing to greater depth by other methods this test can be misleading. Used in estimating allowable bearing presure, coefficient of elastic uniform compression (when the test is cyclic) and modulus of sub-grade reaction.
(*b*) Pressure meter test[1]	Generally done in a bore hole. Cylindrical probe is inserted in a bore and subject to cylindrical pressure increments and deformations noted.	Useful in all soils particularly in bouldery strata, weathered rock etc. where it is difficult to obtain undisturbed samples for laboratory tests of difficult to rely on in-situ tests, like penetration tests.

1. These tests are discussed later in this chapter.

$$\upsilon_S = \left[\frac{G}{\rho}\right]^{0.5} \qquad \ldots(1.3.1)$$

$$\upsilon_P = \left[\frac{1}{\rho}\left(K + \frac{4G}{3}\right)\right]^{0.5} \qquad \ldots(1.3.2)$$

where, υ_s = velocity of shear wave
υ_p = velocity of compression wave

G = shear modulus of material
K = bulk modulus of material
ρ = bulk mass density of material = $\frac{\gamma}{g}$
γ = bulk (and not effective) density of material
g = acceleration due to gravity

From Eqs. 1.3.1 and 1.3.2 it is clear that v_P is greater than v_S

The seismic method of subsurface investigation consists essentially of introducing seismic energy into the ground to be investigated and measuring the time taken for the energy to travel along a defined path through the ground to a seismic detector located at a known distance from the energy source. The energy travel time between source and detector is measured to an ~~accuracy of 1 millisecond or better,~~ by a seismic recorder

The principal seismic techniques in common use are reflection method and refraction method. The seismic reflection method, which uses measurements of travel time of the reflected wave, is often used to explore deep geological structure, but is little used for shallow site investigation work on land. At present this method is mainly used in offshore investigation. A form of reflection method that is widely used for the exploration of sites under water is known as the *sparker method*. Here the energy source is the discharge of an electric spark under water, and a hydrophone is used to pick-up the reflected waves from the sea or river bed and from the boundaries between the layers of the under-lying sediments. The resulting continuous seismic record is known as an *echogram*. Care must be taken to distinguish between primary and multiple reflections on the echogram; for instance the second reflection from the sea or river bed must not be mistaken for the bed rock profile.

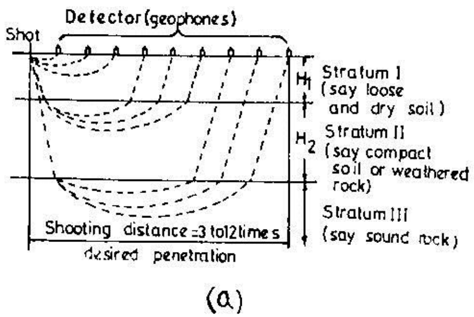
1.3.2. Refraction Method

Refraction method is mainly used in onshore investigation. Fig. 1.3.1 (*a*) explains the principle of refraction survey. A seismic shock is created at shot point. The energy source may be hammer blow or small explosion at or small distance below ground surface. Dropping weight or sledge hammer blow on a steel plate embedded in the ground is used when depth of penetration *i.e.* depth of investigation desired is limited to 10 m. Explosives may not be suitable in highly developed areas. In such cases trailer mounted dropping weight may be used.

Seismic waves, generated at shot point by drop weight or explosion, travelling through the ground are detected by a number of detectors, called geophones, fixed at the surface in a line, at increasing distances from the shot point [Fig. 1.3.1 (*a*)]. The distance between the shot point and the furthest geophone is generally three to twelve times the depth of penetration desired. The geophones are connected to the recording instruments by electric wiring. In one kind of apparatus the signal from each geophone is indicated by a separate galvanometer, and all are recorded simultaneously on photographic paper.

Fig. 1.3.2 is an illustrative field layout for seismic refraction method.

The time taken by the first signal, due either to the direct or the refracted wave, to reach eight geophone is measured and plotted against the distance of the geophone from the source [Fig. 1.3.1 (*b*)]. The gradient



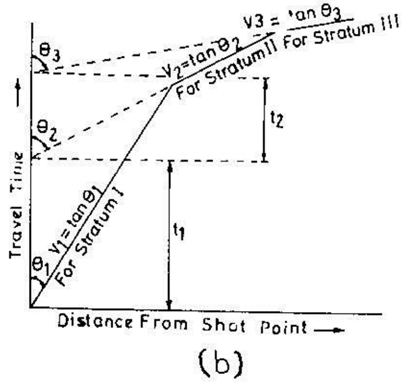


Fig. 1.3.1

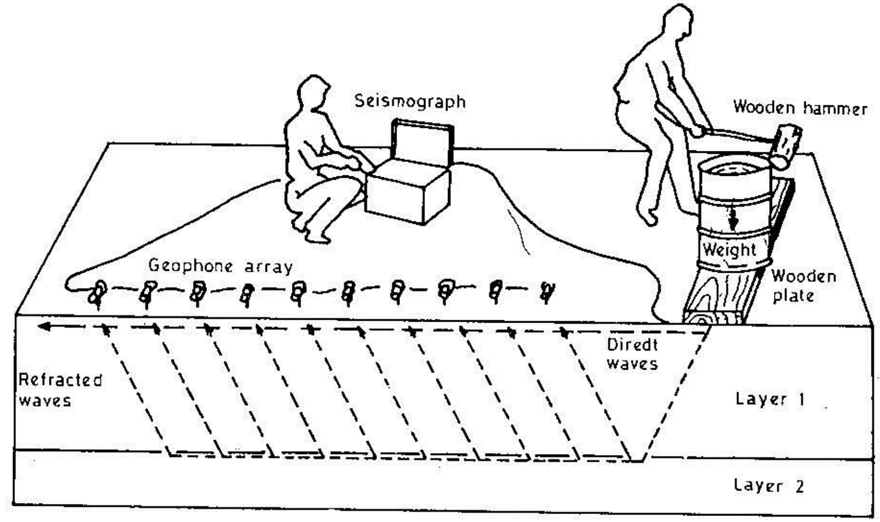


Fig. 1.3.2. Schematic field layout of the surface seismic refraction method showing the path of direct and refracted seismic waves in a two-layer system.

of the travel time graph so obtained gives the seismic wave velocity of the ground. A change of material with depth is indicated by a change in gradient, only if accompanied by an increase in velocity, and thicknesses of strata can be calculated as follows:

$$H_1 = \frac{t_1}{2} \frac{v_1 v_2}{\sqrt{v_2^2 - v_1^2}} \qquad ...(1.3.3)$$

$$H_2 = \frac{t_2}{2} \frac{v_2 v_3}{\sqrt{v_3^2 - v_2^2}} \text{ etc.} \qquad ...(1.3.4)$$

where H_1, H_2, v_1, v_2, t_1, t_2 etc. are illustrated in Fig. 1.3.1. v_1, v_2...refer to velocities of compression waves in strata I, II respectively.

The velocities of the beds give an indication of differences in their physical nature. Approximate ranges of values are given in Table 1.3.1. The table gives only a general indication of likely values, and in practice borehole data from the site of seismic investigation are needed to allow interpretation of the velocity data. Loose dry soils will give velocites close to 330 m/sec.—the wave velocity in air, and loose saturated soils will have velocities close to 1450 m/sec.—the value for water. Values range up to 5000 m/sec for sedimentar rocks and may exceed 6500 m/sec for some igneous rocks. Differences in velocity can be caused by differences in the weathering and jointing of a rock, the value decreasing as the rock becomes more weathered and jointed towards the surface.

Brown and Robertshow (1953) have given the following equation relating velocity v_p of compression wave to E, the modulus of elasticity.

E (kg/cm²) $= 0.001134 v_p^{2.34}$, where v_p is in m/sec. ...(1.3.5 *a*)

E (kN/m²) $= 0.111 v_p^{2.34}$ where v_p is in m/sec. ...(1.3.5 *b*)

Deere et al (1967) have empirically related v_p to rock quality designation (*RQD*) by the following equation:

$$RQD(\%) \approx \left(\frac{v_P - \text{field}}{v_P - \text{lab}}\right)^2 \times 100 \quad ...(1.3.6)$$

This relationship is used in practice to estimate the rippability of sub-surface rock formations.

TABLE 1.3.1. Approximate Ranges of Seismic (Compressional) Wave Velocity of Soils and Rocks

(A) After Abem (1972)

Material	*Seismic Wave Velocity, m/sec*
Soils	
Above water table	300–1300
Below water table	
Coarse sand	1000–1900
Clay	1250–1500
Gravel	1500–1900
Rocks	
Shale	2000–2500
Chalk	1750–3000
Limestones and sandstones	3075–5200
Quartzite	4350–5200
Gneiss	4150–5800
Igneous rocks	3150–6500

(B) After Hvorslev (1949)

Air	330
Water	1450
Losse and dry soils	150–1000
Clay and wet soils	750–1900
Coarse and compact soils	900–2600
Sandstone and cemented soils	900–4300
Shale—marl	1800–5300
Limestone—chalk	2100–6400
Metamorphic rock	3000–6600
Volcanic rock	3000–6900
Sound plutonic rock	4000–7600
Jointed granite	2300–4600
Weathered rocks	600–3000

1.3.3. Limitations of Seismic Refraction Method

The seismic refraction method gives reasonably reliable results provided the following three major conditions are satisfied.

(*i*) The seismic velocity of successively deeper subsurface layers increases with depth below ground surface.

(*ii*) The velocity to thickness ratio of each subsurface layer is less than a certain critical value in relation to the overlying and underlying layers.

(*iii*) The seismic velocities of subsurface layers are effectively constant over the length of the geophone spread.

If condition (*i*) is not fulfilled, and a low velocity layer is present, critical refraction does not occur. The seismic energy is refracted towards the normal to the interface and passes down wards into a higher velocity layers before being returned to the surface by critical refraction. Thus the low velocity layer, called a **blind zone**, is not detected, and consequently the calculated depths of the deeper refractors will be greater than the true depths.

If condition (*ii*) is not satisfied, a thin layer will not be detected because the energy refracted by it will be masked by energy refracted by the underlying higher velocity layer. This is known as the **hidden layer effect**, and results in the calculated depths of deeper refractors being less than the true depths.

Condition (*iii*) is probably fulfilled less frequently than the other two, particularly in case of superficial deposits and is probably the most frequent cause of depth determination errors in the seismic refraction records.

In addition to the above, there are two other limitations. The accuracy of depth measurements by the seismic refraction method is not greater than about ± 0.75 m and decreases with increasing depth, and results for depths shallower than 1.5 to 3 m are not reliable. Though the method is particularly suited to measurements of depth to bedrock, the accuracy is limited to about ± 10 per cent. Further, if the strata are found to have dip, the dip can only be found out by reversing shot point and geophones positions. Hence it is always advisible to adopt such reverse sequence also.

These limitations indicate importance of boreholes to control the interpretation of the geophysical data, not only for correlation of seismic velocities with geological strata, but to check whether condition mentioned above are satisfied.

1.3.4. Electrical Resistivity Method (IS 3043)

Electrical resistivity method is used to find the earth's resistance to electric flow. The knowledge of this resistance is necessary. In all problems involving the earth as the return conductor, such as in studies of grounding, stray current conduction and corrosion, inductive interference and lighning protection. The earth electrical resistance varies over a very wide range between 1 to 10,000 metre-ohms. Hence it is necessary to measure this value in locality in consideration.

In electrical resistivity method, a direct or a low frequency alternating current is transmitted through a pair of electrodes driven into the earth's surface. The voltages arising due to the flow are measured between another pair of electrodes.

There are several arrangement of electrodes (all involving four electrodes) viz Wenner array, the Schlumberger array, pole-dipole array or dipole-dipole array. However Wenner method is most widely used in Indian practice and elsewhere and only this method is discussed here.

In Wenner method, four electrodes are driven into the earth along a straight line at equal intervals. A current I is passed through the two outer electrodes and the earth as shown in Fig. 1.3.3 and the voltage difference V observed between the two inner electrodes. The following equation is used to estimate electrical resistivity, (assuming one layer system and uniform resistance)

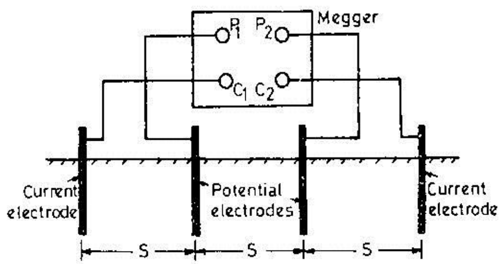


Fig. 1.3.3. Wenner Method for Electrical Resistivity Measurement

$$\rho = \frac{4\pi \frac{V}{I}}{1 + \frac{2s}{\sqrt{s^2 + 4l^2}} - \frac{2s}{\sqrt{4s^2 + 4l^2}}} \quad ...(1.3.7)$$

where ρ = resistivity of soil in ohm-metre

s = distance between two successive electrodes in metres

v = voltage difference between the two inner electrodes in volts.

I = current flowing through the two outer electrodes in amperes, and

l = depth of burial of electrodes in metres.

If the depth of burial of the electrodes in the ground l is negligible compared to spacing s between the electrodes then Eq. 1.3.7 reduces to

$$\rho = \frac{2\pi s V}{I} \quad ...(1.3.8)$$

or

$$\rho = 2\pi s R \quad ...(1.3.9)$$

where $R = \frac{V}{I}$ = megger reading in ohms.

Typical values of resistivity of certain rock and soil types are given in Tables 1.3.2 and 1.3.3.

There are however, many factors which affect resistivity value viz moisture content, salt content, depth of burial of probe and diameter.

Resistivity is much susceptible to moisture changes. At low moisture content resistivity decreases rapidly even with slight increase in moisture content and beyond 20 percent moisture content the decrease is very slow as may be noted from Fig. 1.3.4. Hence ground (earth) resistance is to be preferably measured during dry season.

The soil resistivity decreases with increase in salt content of the moisture in the soil. The salt content is expressed in percent of the contained moisture. It may be noted that the curve flattens off at about 5 percent salt content and a further increase in salt gives but little decrease in the soil resistivity. The effect of salt will be different for different kind of soil and for various moisture contents but the curve in Fig. 1.3.5 will convey an idea of how the soil condivity can be improved by addition of salt.

TABLE 1.3.2. Resistivity of Certain Rock and Soil Types

Rock type	*Resistivity (Ω-m)*
Highly weathered and saturated gneiss/granite	< 40
Weathered and saturated gneiss/granite	40–80
Weathered but less saturated	80–170
Unweathered granite/gneiss with water filled joints	170–400
Massive rock	≥400
Vesicular basalts saturated with water	100–150
Highly weathered basalt saturated with water	5–10
Gravelly sands with fresh water	100
Shale and clay	1
Sea water	0.3
Brackish water	1

TABLE 1.3.3. Earth Resistivity Values of Different Materials

Earth Resistivity Meter-Ohms	*Quarternary*	*Cretaceous Tertiary Quarternary*	*Carboniferous Triassic*	*Cambrian Ordovician Devonian*	*Pre-Cambrian and Combinat with Cambrian*
1 Sea Water					
10 Unusually Low 30 Very Low		Loan Clay Chalk	Chalk Trap Diabase Shale Limestone Sandstone		
100 Low 300 Medium 1000 High				Shale Limestone Sandstone	
3000 Very High 10,000 Unusually High	Coarse Sand and Gravel in Surface Layers			Dolomite	Sandstone Quartyite Slate Granite Gneisses

Fig. 1.3.6. indicates influence of driven depth and diameter of electrode on soil resistivity value. As the driven depth increases, resistivity recorded decreases. It also may be noted that as diameter of rod increases, the soil resistivity recorded decreases.

During the execution of electrical resistivity test by Wenner method, spacing is gradually increased along the line. In general adopted practice, the spacing between electrodes is maintained as 1 m, 5 m, 10 m and 15 m. It is generally assumed that the effective depth of investigation is equal to spacing(s) between electrodes. If soil resistivity does not change with depth, the resistivity estimated with spacing of electrodes 1 m, 5 m, 10 m or 15 m should be identical. In case soil consists of two layers within the effective depth of investigation, these values of resistance ρ_1 and ρ_2 of two layers can be easily be determined as illustrated in Fig. 1.3.7.

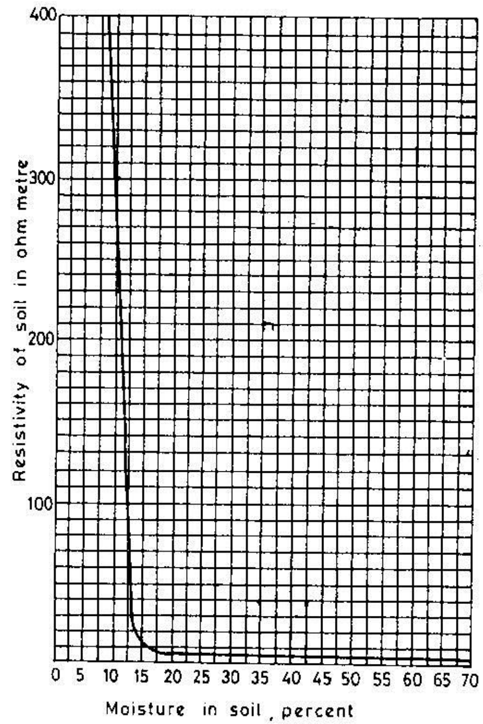


Fig. 1.3.4. Variation of soil resistivity with moisture content.

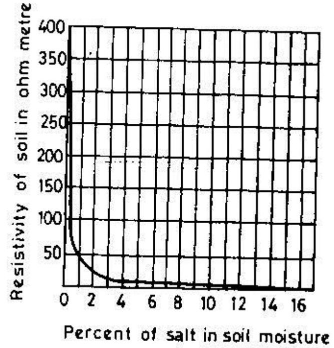


Fig. 1.3.5. Variation of Soil Resistivity with Salt (NaCl) Content.

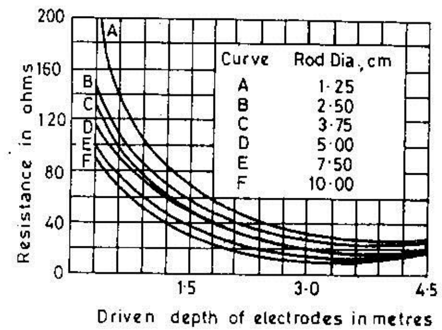
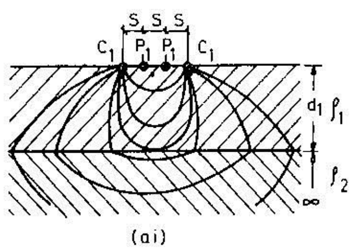
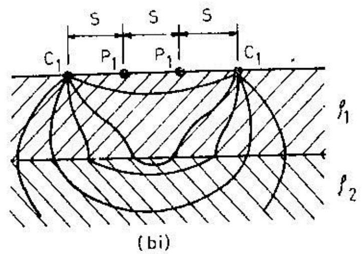


Fig. 1.3.6. Theoretical variation of resistance with driven depth for rod elelctrodes of various diameters with earth resistivity of 100 OHM-Metre (Assumed Uniform).

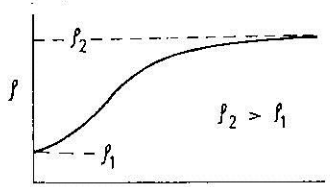
In Table 1.3.4. representative values of soil resistivity in various parts of India as given in IS 3043–1966 are furnished.



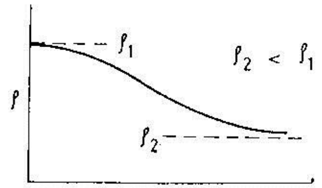
(ai)



(bi)



(aii) Determination of δ_1 & δ_2 for (ai) condition



(bii) Determination of δ_1 & δ_2 for (bi) condition

Fig. 1.3.7. Yokogawa Electric Works Ltd

Table 1.3.4. Representative Values of Soil Resistivity in Various Parts of India

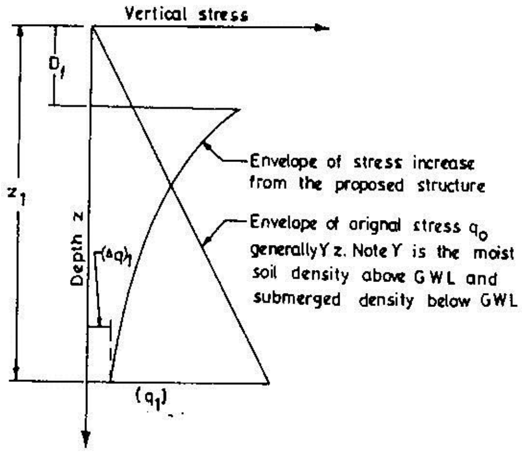
Sl. No.	*Locality*	*Type of Soil*	*Order of Resistivity in ohm-metre*	*Remarks*
1.	Kakarapar, Surat Distt, Gujarat	Clayey black soil	6–23	Underlying bedrock-Deccan trap
2.	Taptee Valley	Alluvium	6–24	do
3.	Narmada Valley	Alluvium	4–11	Underlying bedrock-sand-stones, shale & limestones, Decan trap & gneisses
4.	Purna Valley (Deogaon)	Agricultural	3–6	Underlying bedrock-Deccan trap
5.	Dhond, Bombay	Alluvium	6–40	do
6.	Bijapur Distt, Mysore State	a) Black cotton soil	2–10	do
		b) Moorn	10–50	do
7.	Garimenapenta, Nellore Distt, Andhra Pradesh	Alluvium (highly clayey)	2	Underlying bedrock-gneisses
8.	Kartee	a) Alluvium	3-5	Underlying bedrock-sand-stone, trap or gneisses
		b) Alluvium	9-21	
9.	Delhi			
	a) Najafgarh	a) Alluvium (dry sandy soil)	75–170	do
		b) Loamy to clayey soil	38–50	do
		c) Alluvium (saline)	1.5–9	do
	b) Chhatarpur	Dry soil	36–109	Underlying bedrock-quartzites

1.4. DEPTH OF INVESTIGATION

The depth of investigation depends on the size and type of proposed structure and character and sequence of subsurface strata. In general the depth of investigation shall be such so as to expose any stratum that would adversely affect the performance of the proposed structure.

1.4.1. Thumb Rules

(*i*) Unless bedrock is encountered first, the investigation shall be carried to the point at which the vertical stress due to proposed structure is equal to or less than 10 per cent of original effective stress at the point before the structure is constructed. The depth of investigation may be increased to the point, where increase in stress due to structure is only 5 percent or less of the original stress when compressible strata of soft to medium softness is encountered extending to the recommended depth of investigation. (Fig. 1.4.1 fully illusrates this rule). In absence of structural loadings, the above ruling may be simplified and stated as follows.



Note: D_f = Depth of foundation
z_1 = Recommended depth of investigation
$(\Delta q)_1$ = vertical stress increase from structure at depth z_1
$(q_0)_1$ = original vertical stress at depth z_1.
Requirement: $(\Delta_q)_1 \leq 10\% \, (q_0)_1$, generally
$(\Delta_q)_1 \approx 5\% \, (q_0)_1$, when compressible strata extend to depth of investigation

Fig. 1.4.1. Depth of investigation.

(*ii*) (*a*) It is good practice to have one boring carried the bedrock or at least to a level well below the anticipated level of influence of the building.

(*b*) For light structures, insensitive to settlement, the borings should be extended to a depth equal to 4 times the probable footing width but to not less than 6 m below the lowest part of the foundation.

(*c*) For more heavily loaded structures such as multi-storey structures and for framed structures at least 50 per cent of the borings should be extended to a depth not less than 15 m below the lowest part of the foundation.

(*d*) Where bedrock is encountered it should be proved by boring to a minimum depth of 3 m.

1.5. SPACING AND NUMBER OF FIELD TESTS

Number and relative positions of various field tests such as boreholes, static cone penetration tests, dynamic cone penetration tests, plate load tests, pressuremeter tests etc. depend upon

(*i*) Nature of soil profile — regular or erratic.
(*ii*) Type of structure — sensitive, insensitive etc.
(*iii*) Size or extent of the job.

Whether soil profile is regular or erratic can be known from the initial few boreholes. When the area to be investigated is large and/or when the plant layout is not finalised, a few widely spaced preliminary boreholes are recommended. These preliminary boreholes give rough ideas about the nature of sub-soil in the area and help in planning detailed investigation. For a job in which plant layout is practically finalised, one or other of the field tests, depending on merit, are spaced to fall in all the important plant units. These tests may be kept to the minimum initially and when warranted by erratic subsoil nature, they can be supplemented by additional field tests.

At all times, the investigation should be conducted considering the requirements and needs of the structure. For structures housing sensitive equipment such as atomic reactors etc., investigation has to be sufficiently extensive so as to reveal soil profile with great precision. However, for ordinary buildings investigation can be limited, to economise in money and time, and somewhat more conservative values can be considered for soil parameters needed in the design.

In case of buildings, certain guidelines are given in IS 1892 for selecting spacing and number of boreholes and or trial pits. IS 1892 considers that a single borehole at the centre of the plot is enough for small and less important buildings. For compact building site on about 0.4 hectare, it specifies five bores, one in each corner and one at the centre. Author considers it is advisible to have minimum three bores for a virgin site area. For very large site of industrial and residential colonies, it is advisible to have number of boreholes on a grid pattern.

In case of exploration for highways, the spacing of boreholes in general could be stated as one at every interval of 200 m. This spacing may be reduced to 50 m when the subsoil is highly erratic and may be increased to 500 m when the subsoil is very uniform.

When the exploration is to be carried out for earth and rockfill dams, guidelines given in IS 6995 would be beneficial.

Various field tests mentioned in the earlier sections are discussed in greater detail latter in this chapter.

1.6. UNDISTURBED SAMPLES

One of the aims of carrying out soil explorations is to obtain disturbed and undisturbed soil samples for relevant laboratory testing. The type of sampler and the method of sampling to be used for undisturbed sampling should be such as to yield the undisturbed sample with minimum disturbance. Some of the requirements for obtaining undisturbed samples are given in Table 1.6.1.

TABLE 1.6.1. Requirements for Obtaining Undisturbed Samples

Requirements	*Specification*
(1) **Requirements for samplers**	
(a) Diameter	Recommended minimum inside diameter of sampler is 75 mm. (*See Foot-note 1*)
(b) Length	Length of sampling should at least equal sum of intended length of sample plus 100 mm for allowing collection of disturbed material at the top and the length occupied by sample head and piston. (*See Footnote 2*).
(c) Area ratio*	The recommended area ratio for samplers up to around 100 mm. diameter is 10 per cent though presently higher area ratios are currently in use in Indian practices. Table 1.6.2 lists largest permissible area ratios for samplers upto around 100 mm. diameters in different countries of the world.
(d) Inside clearance ratio*	Inside clearance ratio should lie between 0.5 to 3 per cent for different types of samplers. Low value of clearance ratio is recommended for sampling in sands while clearance ratio upto 3 per cent may be needed in obtaining good sample recovery in certain clays. Table 1.6.4 may be followed while selecting samplers to suit soil type. (*See Footnote 3*).
(e) Cutting edge	Cutting edge should be machine prepared with outside taper not exceeding 20° (See Fig. 1.6.1). Blunt cutting edges should not be permitted. (*See Footnote 4*).
(f) Tubing	Tube should be seamless with no protrussions and should be of uniform cross section (*See Footnote 5*).
(2) **General requirements while sampling.** (*See Footnote 6*)	(1) Bottom of borehole should be cleaned thoroughly of loose material before taking undisturbed samples.

TABLE 1.6.1. (Contd.)

	(2) For sampling above ground water level, it is preferable to maintain borehole in dry condition.
	(4) Sampler should be pushed into soil for sampling. Driving of sampler should be permitted only when it cannot be pushed into the soil. Sampler should not be pushed or driven into the soil for its full length but should be stopped 50 to 80 mm before reaching the full length. Length of penetration should not exceed 5 to 10 times the diameter of sampler in cohesionless soil and 10 to 15 times the diameter of the sampler in cohesive soils. (*See Footnote 6*).
	(5) Sampling of soft or loose soil directly below a stiff or compact soil into same tube should be avoided. Hence penetration of sampling tube should be stopped when sudden decrease in penetration resistance is noticed.
	(6) Sampler should be separated from soil by turning one or two rounds and not by imparting sudden pull to the sampler.

* For details refer Fig. 1.6.1 on page 20.

Footnotes:

1. *Diameter of undisturbed soil samples (UDS) vary widely, generally between 50 mm to 150 mm. However presently widely adopted dimension is around 75 mm. (Refer Adachi, 1979, and Osterberg et al. 1979).*

   But author prefers diameter of 90 mm to 100 mm. This permits extracting 3 usual size samples from the same horizontal plane for triaxial testing.

   Sample disturbance decreases with increasing sample diameter (Fig. 1.6.2). For diameter less than 50 mm, sample disturbance could be significant and hence samples diameter should be greater than 50 mm. For many clays the best quality samples are obtained at a diameter of approximately 100 mm or more. For diameter greater than 100 mm, extra cost involved may be significantly more than reduction in sample disturbance. Further the possibility of losing a sample increases rapidly with increasing diameter because the weight of sampls increases in proportion to the square of the diameter while its adhesion to the wall of the tube increases only in proportion to diameter.

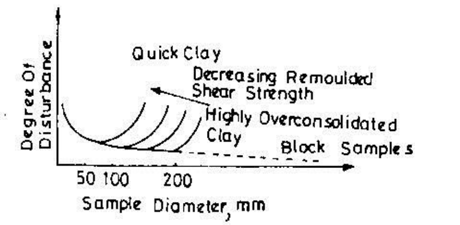


Fig. 1.6.2. Effect of sample diameter on degree of disturbance

2. *Length of UDS in practice generally vary between 300 to 1000 mm. It is advisible to limit the length. Author recommends length between 400 to 500 mm. If longer samples are collected, the practice of pushing the sample through the tube, during extraction for testing, disturbs the same. When UDS tubes are long, it is advisible to cut the tube and extract the samples. But this practive is hardly followed in India.*

   Sample quality is related to the length/diameter ratio of the sample as illustrated in Fig. 1.6.3. (Anderseen 1981). The optimum length/diameter ratios suggested for clays of different sensitivities, St, are:

St	*Length/diameter*
> 30	20
5 to 30	12
< 5	10

3. *Recent limited Japanese studies have indicated that it is preferably to have diameter of cutting edge same as inside diameter of tube i.e. inside clearance ratio as zero. (Adachi 1979).*

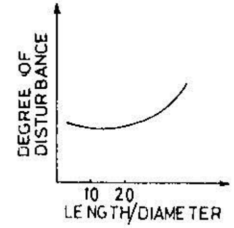


Fig. 1.6.3. Effect of length diameter ratio of sample on degree of disturbance

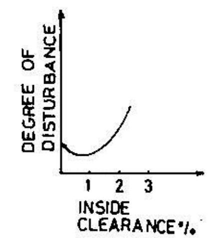


Fig. 1.6.4. Effect of inside clearance on degree of disturbance

Anderseen (1981) suggests an inside clearance ratio of 0.5 to 1 per cent for sampling to depths of 20 m in non-swelling clays as with higher clearance, the disturbance caused is significant as illusted in Fig. 1.6.4.

4. *Recent Japanese studies have proved conclusively that cutting edge angle and bluntness of cutting edge have significant effect on quality of UDS. The same studies have shown lesser importance of area ratio than so far thought off. Japanese standard now lays down that cutting edge angle should be limited to 6° ± 1° (Adachi 1979).*

Similar view as reflected in Fig. 1.6.5 has been expressed by Anderseen (1981). The effect of area ratio on sample disturbance is shown by Fig. 1.6.6. The combined requirements of area ratio and edge taper to cause low degree of disturbance are summerisea in Fig. 1.6.7 and Table 1.6.3.

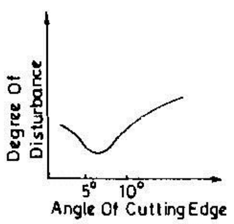


Fig. 1.6.5. Effect of angle of cutting edge on degree of disturbance

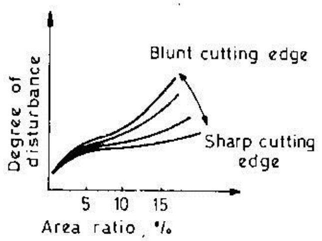


Fig. 1.6.6. Effect of area ratio on degree of disturbance.

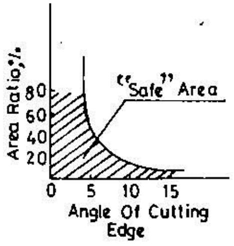


Fig. 1.6.7. Criterion for relationship between area ratio and angle of cutting edge to minimize disturbance.

5. *An important characteristic of sampling tube is roundness. This is defined as the difference in the outer diameter of the tube measured at any cross section and as per Japanese practice it should be less than 1.5 mm (for 76 mm UDS tube).*
6. *The sampling (penetration) speed must be constant and must not be too high or too slow. In clay, the friction increases if sampling is interrupted for more than a few seconds. If the penetration speed is reduced below 0.1 m per minute, adhesive forces build up which can cause severe disturbance as shown in Fig. 1.6.8.*
7. *The material used for sampler should be rigid, resistant to corrosion and machinable to a smooth surface. IS: 2132 recommends use of steel brass or aluminium. In USA use of a welded and drawn over the mandival (DOM) steel tube coated with lacquee or apoxy resin is recommended. Japanese standard recommends stainlss steel or brass tube for sampler. In Indian practice, steel tube is very widely used for samplers. Though aluminium is recommended in Indian standards, its use is hardly reported from anywhere in the word.*

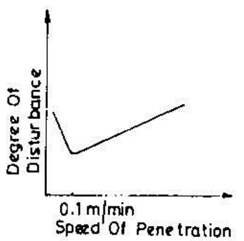


Fig. 1.6.8. Effect of sampler penetration rate on degree of disturbance.

8. *Inspite of adequate precautions, a certain degree of disturbance is unavoidable. Some of the methods evaluating sample disturbances are (Bhandari & Jain 1983): visual inspection, X Radiographic examination, recovery ratio, comparision of values obtained from insitu and laboratory tests for density, stress, strain deformation and permeability. Of these methods based on recovery ration is the simplest. Recovery ratio shall be noted for each sampling. In practice, a recovery ratio of 98% is satisfactory and anything less than 95% indicates inaccurate procedures and measurements during sampling or loss of sample and may be considered as a sign of possible disturbance.*
9. *Rotary method of boring is most widely used method of advancing a borehole. This method along with use of drilling mud to stabilize the side of the porehole is likely to give the least disturabnce to soils beneath the base of the borehole and permit the taking of near to truly undisturbed samples (Adachi, 1979; Posterberg et al. 1979; Mori et al. 1979; Cole, 1979).*

TABLE 1.6.2 Largest Area Ratios in Practice*

Country	*Largest Area Ratio %*
Denmark	15
Finland	15
France	15
India	20
Israel	4.5
Italy	12
Mexico	10
Japan	11
Norway	12
U.K.	10
U.S.A.	13
Yogoslavia	12

* From Bhandari and Jain (1983)

TABLE 1.6.3. Combination of Area Ratio and Edge Taper for Low Disturbance

Area Ratio, %	*Edge Taper Angle, Degrees*
6	15
10	12
20	9
40	5
80	5

TABLE 1.6.4. Inside Clearance Ratio for Samples for Undisturbed Sampling

Sl. No.	*Inside Clearance Ratio of Sampler*	*Soil Type*
1.	About 0.5 per cent	Sands, silts and soft clays.
2.	About 1.5 per cent	Stiff and hard clays below ground water level.
3.	About 3.0 per cent	Stiff expansive clays below ground water level.

Samplers used for obtaining undisturbed samples are required to meet the requirements indicated in Table 1.6.1. Such samplers are known as **thin wall tube samplers.**

Thin wall tube samplers can essentially be divided into two groups *viz.*, thin wall tube sampler with stationary piston, and thin wall tube sampler without piston. There are number of varieties of piston sampler in use. Of these, **Swedish foil sampler** is used to recover long, continuous samples in soft soils free of gravel, sand layers or excessive shells. This sampler has thin metal strips housed inside a sampler head. These strips envelope the sample as it enters the tube thus minimizing the friction between the tube and the sample.

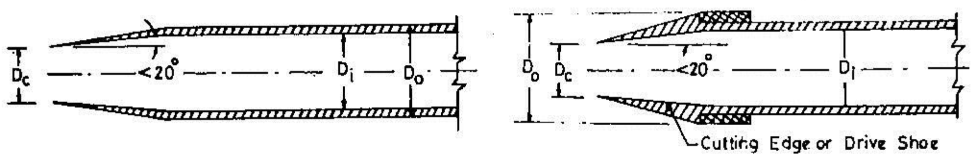
Other piston samplers can be grouped in one single category. The samplers in this category differ from one another mainly in the arrangement to lock the stationary piston during sampling. The choice for a particular type is generally based on convenience or availability. This samplers can be used in obtaining samples in soft to stiff clays, silts, silty and sandy clays.

Commonly used thin wall tube sampler of non piston type is the **Shelby tube** sampler. This sampler can be used in soft to hard clays, silts, silty and sandy clays. It is the most commonly used thin wall sampler for general civil engineering works.

In pits, trenches or in open cut excavation one can obtain undisturbed samples by hand carving. Properly executed **hand carved samples** will have least possible disturbance of any kind.

It should be noted that samples obtained from standard split spoon sampler used in SPT[1] cannot

1. Refers to standard penetration test.



D_o = outside diameter of sampling tube
D_c = inside diameter of cutting edge

$$A_r = \frac{D_0^2 - D_c^2}{D_c^2} 100 \quad ...(1.6.1)$$

(Area ratio)

D_i = inside diameter of sampling tube
$D_c < D_i < D_o$

$$I_r = \frac{D_i - D_c}{D_c} 100 \quad ...(1.6.2)$$

(Inside clearance ratio)

Fig. 1.6.1.

be treated as undisturbed sample. Area Ratio of this sampler is 112 per cent. However, if no undisturbed samples are collected and only SPT is run, such samples obtained in insensitive clays may be used to estimate in situ strength behaviour of the soil.

Undisturbed samples shall be collected at regular intervals, generally 2 to 3 metres apart, but for the following exceptions.

(*i*) In very soft to soft clays where samples may slip, it is preferable to avoid undisturbed samples and carry out vane shear tests.

(*ii*) If soil lacks cohesion, additional number of standard penetration tests should be preferred to collecting so called undisturbed samples.

(*iii*) In clay with *N* values of SPT of 30 and above, avoid sampling with Shelby tube and use *Denison* or *Pitcher* sampler. It is preferable to conduct additional SPT rather than collecting so called undisturbed samples. When *N* values of SPT is 50 or greater undisturbed sampling should be avoided and SPT adopted.

1.7. IN-SITU MEASUREMENTS OF SOIL PROPERTIES

In Table 1.7.1 some field tests commonly adopted to measure soil properties by in-situ testing are listed. Relevant methods of executing these tests and their applicability are also noted in the table.

TABLE 1.7.1 In-Situ Measurements of Soil Properties

Sl. No.	*Test*	*Recommended Procedure*	*Test Application and Other Relevant Remarks*
1.	**Penetration Tests**		
	(*a*) Standard penetration test	IS : 2131	Rountine procedure in test boring; empirical relations are used to predict strength and compressibility of soils. This test is best suited to cohesionless soils particularly sand.
	(*b*) Dynamic cone penetration test	IS : 4968 Part I and II	Has similar application as standard penetration test. Helps in reconnaisance survey of wide areas in a shorter time and lesser cost.
	(*c*) Static cone penetration test	IS : 4968 Part III	The test is useful in obtaining end-bearing resistance and skin frictional resistance at various depths or even continuously in soils. Useful both in cohesionless soils and cohesive soils which are not very dense or hard (limitation imposed by capacity of equipment) or do not contain or cobbles. Method is very fast and economical and involves no boring.

TABLE 1.7.1. (Contd.)

2.	**In-Situ Vane Shear Test**	IS : 4434	Best suited for determination of shear strength of soft and/or sensitive cohesive soils which are difficult to sample or are susceptible to sampling disturbances. Erratic results are obtained in soft soils containing shells, gravel or wood fragments. Yields undrained strength. Present equipment capacity limited to qu = 1.5 kg/cm^2 (147 kN/m^2).
3.	**Pressuremeter Test**	—	Menard pressuremeter is recently introduced in India. Helpful in obtaining in-situ stress strain characteristics of soils and rocks. Particularly useful in weathered rock and cohesionless soils in which undisturbed samples are difficult to obtain. Helps in estimating allowable bearing capacity based on strength and settlement.
4.	**Load Tests**		
	(*a*) Plate bearing test	IS : 1888	Test is useful in estimating allowable bearing pressure of shallow foundations except in saturated clays. If test is conduct as *cyclic* it is further useful in estimating elastic uniform compression or spring constant. The major limitation of the test is that the method assumes within the soils effective depths of test plate and actual foundation are similar. This should be ascertained by boring or other methods.
	(*b*) Pile load test Part IV	IS : 2911	To determine pile capacity. Cyclic test is useful in separating skin friction and end bearine and also in determining elastic resistance of pile.
5.	**Field Density Tests**		
	(*a*) Sand cone method	IS : 2720 Part XXVIII	Used in soil of any type but least accurate in clean, gravelly or rocky materials.
	(*b*) Rubber ballon method	IS : 2720 Part XXXIV	Used in compacted or firmly bonded soils. Not suitable for very soft soils or ot soils in which volume of hole cannot be maintained at a constant value under ballon pressure.
	(*c*) Core cutter method	IS : 2720 Part XXIX	Used in fine grained soils with some cohesion. This method has less chance of error and is more rapid than sand cone method in such soils.
	(*d*) Nuclear density test method	—	Necessary commercial equipment is available. Procedures are rapid, convenient, and particularly useful for determination in clean, coarse-grained soils. The results of individual tests may vary significantly from those obtained by conventional methods, but the averages of a large number of tests made by either methods are generally found to be in agreement.
6.	**Permeability Test**	IS : 5529	To determine in-situ permeability.
7.	**California Bearing Ratio Test**	IS : 2720 Part XXXI	Determination of *CBR* or subgrade reaction, for payment design.
8.	**Dynamic Tests for Dynamic Properties of Soils**	IS : 5249	For evaluation of in-situ dynamic properties *viz.* G (or E), coefficients c_u, c_T, c_ψ and damping coefficient of soils.

1.8. MEASUREMENT OF SOIL PROPERTIES IN THE LABORATORY

Table 1.8.1 lists various laboratory tests that may be conducted on soil samples. The table is by no means a complete listing of all the tests nor all these tests are always necessary on a given project. The particular tests to be conducted depend upon the nature of the project and in some cases it may be necessary to run

TABLE 1.8.1. Measurement of Soil Properties in the Laboratory

Test	*Recommended Procedure*	*Type of Soil Sample and other Relevant Remarks**
I. Index Property Tests		
1. Sample Preparation	IS : 2720 Part I	DS or UDS
2. Moisture Content	IS : 2720 Part II	DS (or UDS) with unaltered moisture content
3. Dry Unit Weight	Lambe[1], B.S.[3] 1377	UDS
4. Specific Gravity	IS : 2720 Part III	DS
5. Liquid Limit	IS : 2720 Part V	DS, Material passing IS seive No. 425 *i.e.* finer than 425 μ
6. Plastic Limit	IS : 2720 Part V	DS, material passing IS seive No. 425
7. Shrinkage Limit and Shrinkage Characteristics	IS : 2720 Part VI and XX	DS or UDS
8. Grain Size Analysis		
(*a*) Seive analysis (*b*) Hydrometer analysis (*c*) Pipette analysis	IS 2720 Part IV	DS
9. Relative Density of Cohesionless Soils	IS : 2720 Part XIV	DS or UDS
10. Soil Classification		
(*a*) Unified system	Lambe[1]	
(*b*) I.S. system	IS : 1498	
II. Mechanical Property Tests		
1. Permeability	IS : 2720 Part XVII	DS or UDS
2. Capillary Moisture Relationship	Lambe[1]	DS or UDS
3. Consolidation	IS : 2720 Part XV	UDS
4. Direct Shear	IS : 2720 Part XIII and XXXIX	DS or UDS
5. Unconfined Compression	IS : 2720 Part X	UDS
6. Triaxial Compression		
(a) UU test (b) CU test (c) CD test	IS : 2720 Part XI and XII Bishop and Henkel[2]	UDS
7. Vane Shear Test	IS : 2720 Part XXX	UDS
8. Standard Proctor Test (Heavy compaction test)	IS : 2720 Part VII	DS
9. Modified Proctor Test (Light compaction test)	IS : 2720 Part VIII	DS
10. Harvard Miniature Compaction Test		DS
11. Abbot Compaction Test		DS
12. Jodhpur Mini-Compactor Test		DS
13. California Bearing Ratio	IS : 2720 Part XVI	DS or UDS
14. Determination of Volume Change of Soils—Free Swell Test	IS : 2720 Part XL	DS or UDS
15. Determination of Swelling Pressure of Soils	IS : 2720 Part XL	DS or UDS
16. Test for Centrifuse Moisture Equivalent of Soils	IS : 2720 Part XIX	DS

TABLE 1.8.1. (Contd.)

Test	*Recommended Procedure*	*Type of Soil Sample and other Relevant Remarks**
17. Test for Sand Equivalent of Soils and Fine Aggregate		DS
18. Determination of Field Moisture Equivalent	IS : 2720 Part XVIII	UDS
III. Chemical Property Tests		
1. Base Exchange Capacity	IS : 2720 Part XXIV	DS
2. Silica Sesquioxide Ratio	IS : 2720 Part XXV	DS
3. Total Soluble Solids	IS : 2720 Part XXI	DS
4. Organic Metter	IS : 2720 Part XXII	DS
5. Sulphate Content	IS : 2720 Part XXVII	DS
6. pH.Value	IS : 2720 Part XXVI	DS
7. Chloride Content		DS
8. Total Soluble sulphates	IS : 2720 Part XVII	DS
9. Chemical analysis of water sample	IS : 3025	
IV. Stabilised Soils	IS : 4332 Parts I to X	DS or UDS

* DS = disturbed UDS = undisturbed
1. "Soil Testing for Engineers" by T.W. Lambe, John Wiley and Sons (1967).
2. "Measurement of Soil Porperties in Triaxial Tests" by A.W. Bishop and D.J. Henkel, Edward Arnold Ltd. (1964).
3. B.S.- British standards.

some special tests to understand specific behaviour of soils.

In Table 1.8.2, some of the common laboratory tests on rock samples are listed.

TABLE 1.8.2. Measurement of Properties of Rock Core in the Laboratory

Sl. No.	*Type of Test*	*Recommended Procedure*
1.	Unconfined compression test	IS 9143
2.	Point load index	IS 8764
3.	Uniaxial tensile strength	IS 10082
4.	Triaxial compression strength	ASTM -D2664
5.	Tensile strength in bending	
6.	Modulus of elasticity and poison's ration	IS 9221
7.	Preparation of rock specimen	IS 9179
8.	Water absorption	IS 1124
9.	Specific gravity	IS 1122
10.	Porosity	IS 1122
11.	Unit weight/density	—
12.	Moisture content	—
13.	Petrographic examination	IS 1125

1.9. STANDARD PENETRATION TEST, SPT

The standard penetration test is used extensively in soil exploration programmes.

The test consists of driving a standard split spoon, 50.8 mm outside diameter and 35 mm inside diameter, into soil under the blows of a drop weight (hammer) of 65 kg falling freely through 75 cm. The number of blows required for 30 cm of penetration of sampler in the soil is designated as N value and is termed as standard penetration blow count.

Attempts have been made to correlate N-values to relative density of cohesionless soils and unconfined compressive strength of cohesive soils. Attempts have also been made to correlate compressibility of soils to N-values. Such relations are utilised in arriving at allowable bearing pressure of soils. Such approaches can be and are subjects of criticisms. But past experiences indicate that standard penetration test can provide reliable and safe design criteria when used with *caution* keeping in mind the *limitations* of the test and its application.

In Table 1.9.1 the variables affecting the results of standard penetration test are listed. It is necessary that

TABLE 1.9.1. Variables Influencing the Results of SPT

Sl. No.	Variable	Measured N-Value: High	Measured N-Value: Low	Remedial Measures/Remarks
		Compared to Standard N-value		
1.	Weight of hammer	Usually high	But can be low also	Weigh the hammer during each *set* of tests. Maintain the weight to 65 kg.
2.	Height of drop	Usually high	Can be low also	Provide *guide* indicator so that height of drops is 75 cm.
3.	Free fall (hammer should drop freely through its entire height)	High		Release tension completely from the rope before allowing the weight to drop. Avoid using a wire line wound on a drum for raising and lowering the drop weight.
4.	Eccentricity of blows	High		Use guiding device such as a rod through the central hole of the drop weight.
5.	Cushion			Provide hardwood cushion block (in some drilling practices there is metal to metal contact at the stroke of the drive weight. The recommended practice is to eliminate such metal to metal contact by providing a hardwood cushion block).
6.	Rate of applications of blows			Yet to be standardised. (Such a standardization is difficult to achieve unless completely automatic drop weight is used in the test).
7.	Size of connecting rods			Use standard rods of size A* or bigger.
8.	Depth of boring	High (influence is negligible up to a depth of about 50 m)		Preferable to use other methods other than SPT for depths exceed 50 m, some energy will be lost in buckling of connecting rods. Experience has shown that this energy lost is negligible up to 50 m, with standard size rods, but is considerable when length is over 100 m).
9.	Drive shoe	High		Replace dented or detorned drive shoe.
10.	Couting blows	Can be high or low		Alterness on the part of driller is necessary.
11.	Sludge at the bottom of bore hole	Can be high or low (Usually low because sampler can penetrate easily in sludge. But when considerable sludge accumulates sometimes higher number of blows are needed as sampler may tend to wedge inthe soil confined in the casing).		Clean the hole. Also the practice of driving the sampler by 45 cm. and neglecting the number of blows needed for initial 15 cm and taking the number of blows needed for remaining 30 cm as N value is a good practice in this respect.

TABLE 1.9.1. (Contd.)

Sl. No.	Variable	Measured N-Value: High	Measured N-Value: Low	Remedial Measures/Remarks
		Compared to Standard N-value		
12.	Hydrostatic head			No net uplift head at the bottom of drill hole. (With excess uplift at the bottom of bore-hole N value, particularly in cohesionless soils can be very low, can be even zero.)
13.	Particle size			
	(*a*) With gravel size particle	High		Recommended practice is to take in minimum of the recorded N-values as the N-value for the entire formation.
	(*b*) Very fine sand or silt at or just above or below the phreatic surface or ground water	High		Personal judgement. Usually correction as suggested by Terzaghi and Peck (Eq. 1.10.1) is applied to the measured N-value.
14.	Overburden pressure.		Low in cohesion less soil.	Apply correction to N-value, in cohesionless soils as suggested by Gibbs and Holtz (1957) (Eq. 1.10.2). (Correction suggested by Gibbs and Holtz for over-burden is very widely adopted. Corrections suggested by some others are indicated in Section 2.19).

* Refer Table 1.18.1.

errors resulting from variation in these variables are kept to minimum and that the test is executed in a *standard manner*. Only then statistical correlations obtained can be more meaningfully applied.

1.10. CORRECTIONS TO MEASURED VALUES OF SPT

The N values of SPT as measured in field may need to be corrected. There are two types of corrections generally applied to measured N value. One type of correction is applied when SPT is conducted in fine or silty, saturated sand and when the recorded blow count is greater than 15. This correction as recommended by Terzaghi and Peck (1967) is as follows:

$$N = 15 + \frac{1}{2}(N' - 15) \qquad ...(1.10.1)$$

where N and N' are corrected and actual blow counts respectively.

The other type of correction is known as correction for overburden pressure. This correction is applied only to cohesionless soils (dry, moist or wet). The correction as suggested by Gibbs and Holtz (1957) and as widely adopted is as follows:

$$N = \frac{35N'}{\overline{\sigma}_0 + 7}, \text{ for } \overline{\sigma}_0 \le 28 \text{ t/m}^2$$

(Here $\overline{\sigma}_0$ is in t/m²) (1.10.2.*a*)

$$N = \frac{345N'}{\overline{\sigma}_0 + 69}, \text{ for } \overline{\sigma}_0 \le 276 \text{ kN/m}^2$$

(Here $\overline{\sigma}_0$ is in kN/m²) ...(1.10.2*b*)

Where N and N' are corrected and measured blow counts respectively and $\overline{\sigma}_0$ is the effective overburden pressure at the depth of SPT. (Correction for saturation effect, as per Eq. 1.10.1 is to be applied, where applicable, prior to overburden correction. Then N' in Eq. 1.10.2 is the N value from Eq. 1.10.1).

It is considered advisable to place an upper limit of 2 for overburden correction. With this limit, Eq. 1.10.2 becomes,

$$N = \frac{345N'}{\overline{\sigma}_0 + 7} \le 2N' \text{ for } \overline{\sigma}_0 \le 28 \text{ t/m}^2$$

(Here $\overline{\sigma}_0$ is in t/m²) ...(1.10.2c)

$$N = \frac{345N'}{\overline{\sigma}_0 + 69} \le 2N', \text{ for } \overline{\sigma}_0 \le 276 \text{ kN/m}^2$$

(Here $\overline{\sigma}_0$ is in kN/m²) (1.10.2d)

The overburden correction is graphically presented in Fig. 1.10.1 and Fig. 1.10.2 for the following respective two cases.

(*i*) Water table at great depth greater than 16.5 m below ground level with assumed moist density of soil as 1.7 t/m³ (16.7 kN/m³)

(*ii*) Water table at ground level and assumed submerged density of soil as 1.0 t/m³ (9.8 kN/m³)

1.11. N OF SPT AND CONSISTENCY OF SOILS

Consistency of a granular soil is indicated by its relative density and that of a cohesive soil by its unconfined compression strength.

Table 1.11.1, Fig. 1.11.1 and Fig. 1.11.2 represent empirical relationship currently in use in obtaining relative density of cohesionless soils from N values of SPT. From N values angle of internal friction, ϕ of soil can be obtained using Meyerhof's equation or directly from Table 1.11.1.

Relation in Table 1.11.1 and Fig. 1.11.1 are widely used in estimating relative density of cohesionless soils from N values SPT. Furthermore relationship

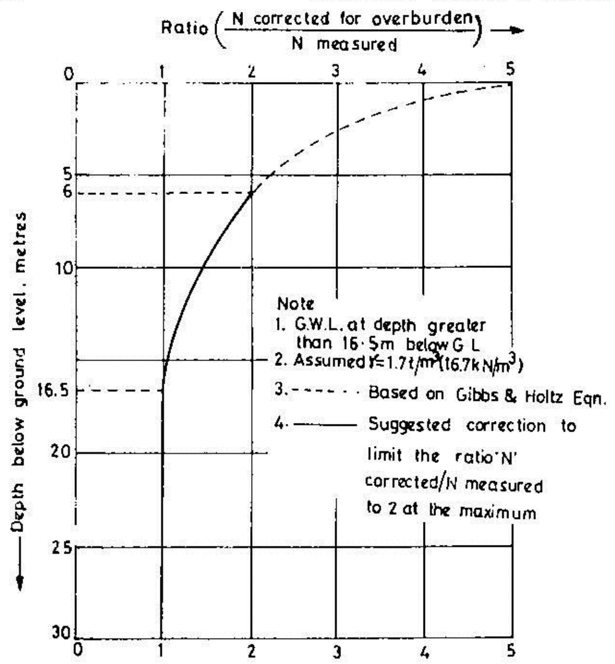


Fig. 1.10.1. Correction factor for overburden in dry or moist cohesionless soils.

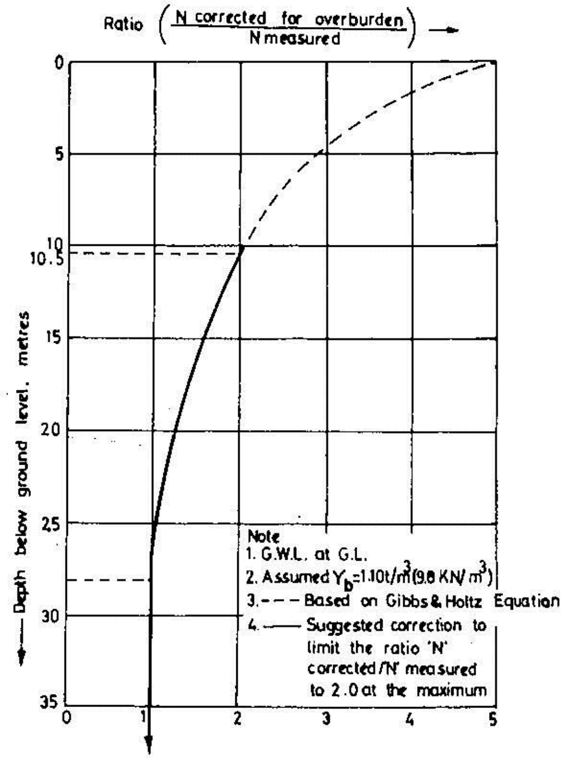


Fig. 1.10.2. Correction factor for overburden in submerged cohesionless soils.

in Fig. 1.11.1 is more commonly used in analysing liquefaction characteristics of soil under earthquakes.

In case of cohesive soils, the relationship between N values of SPT and consistency, as proposed by **Terzaghi** and **Peck**, is presented in Fig. 1.11.3.

Caution: It is absolutely necessary to realise that the relationships presented in Table 1.11.1 and Figs. 1.11.1 to 1.11.3 are oversimplification of the very complex relationships existing between blow count and consistency of soils. Hence these relationships are to be used with great caution and their use should be limited to the extent possible.

1.11.1. *N* of SPT Versus Relative Density of Cohesionless Soils—Skempton's Approach

After extensive literature survey, laboratory testing and methods of standard penetration tests as practised in different countries, skempton has suggested a rationalised approach for estimating relative density of sandy soils based on N values of SPT. This approach has been briefly described later

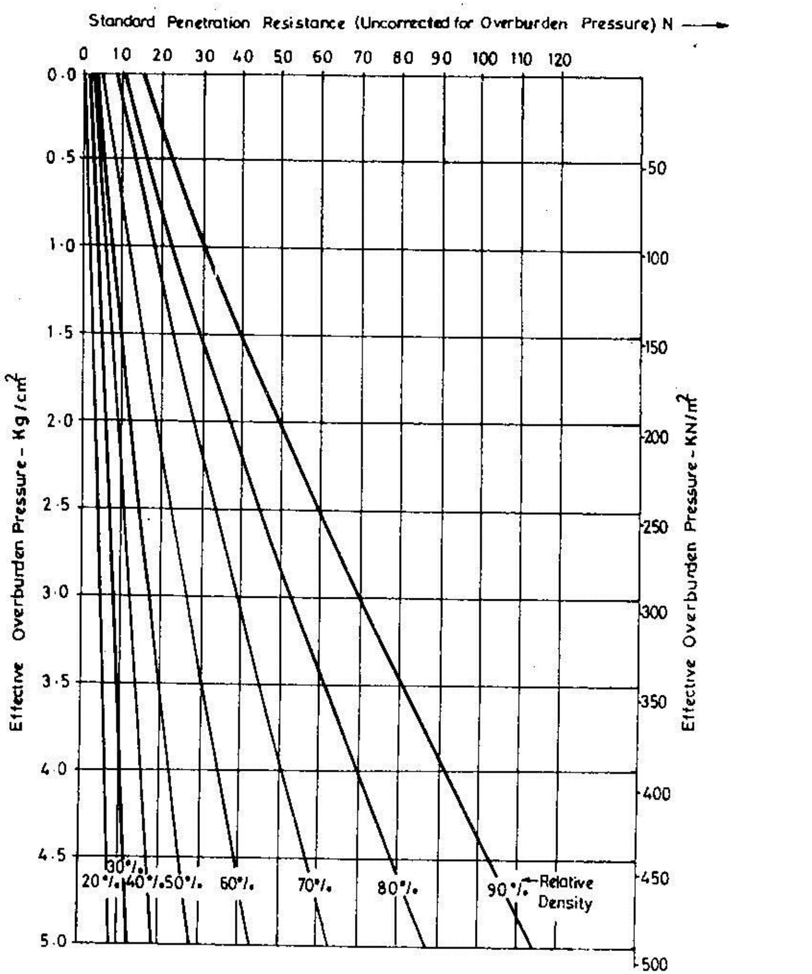


Fig. 1.11.1. Relationship between standard penetration resistance, relative density and effective overburden pressure for cohesionless soils (after Gibbs and Holtz).

on. However for full details readers are requested to his original paper "SPT procedures and the effects in sand of overburden pressure, relative density, particles size, ageing and overconsolidation" in Geotechnique journal, Vol XXXVI No. 3, Sept. 1986.

Skempton observed that measured N value of SPT is affected by many factors viz. energy transmitted at borehole bottom, type of SPT hammer, type of release of SPT hammer, length of connecting rods, borehole diameter, type of sampler, overburden pressure, ageing period in addition to the relative density of sandy subsoil.

In approach suggested by Skempton to estimate relative density of sands from measured values of SPT is described in simplified form as follows in following steps (1) to (4).

TABLE 1.11.1. Empirical Values of ϕ, D_r and Unit Weight of Granular Soils Based on the Standard Penetration Blow Count with Corrections for Saturation Effect and for Overburden, if any

Description		*Very Loose*		*Loose*		*Medium*		*Dense*		*Very Dense*	
Relative density, D_r, %	0		15		35		65		85		100
Standard penetration blow count, N			4		10		30		50		
Approximate angle of internal friction, ϕ*		25°–30°		27°–32°		30°–35°		35°–40°		38°–43°	
Approximate range of moist unit weight, t/m^3**		1.1–1.60		1.45–1.85		1.75–2.10		1.75–2.25		2.10–2.40	

* After Meyerhof, $\phi° = 25 + 0.15\,Dr$ with more than 5 per cent fines and $\phi° = 30 + 0.15\,Dr$ with less than 5 per cent fines.

** It should be noted that excavated material dumped from a truck will weigh 1.1 to 1.45 t/m^3.
Material must be quite dense and hard to weigh over 2.1 t/m^3. Values of 1.70 to 1.85 t/m^3 for non-saturated soils are common.
Relative density Dr of a cohesionless soils is defined as,

$$Dr = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where e_{max} = maximum void ratio *i.e.* the void ratio in the loosest state of packing of soil particles.
e_{min} = minimum void ratio *i.e.* the void ratio in the densest state of packing of soil particles.

Note: Relative density Dr may be expressed as per cent also.

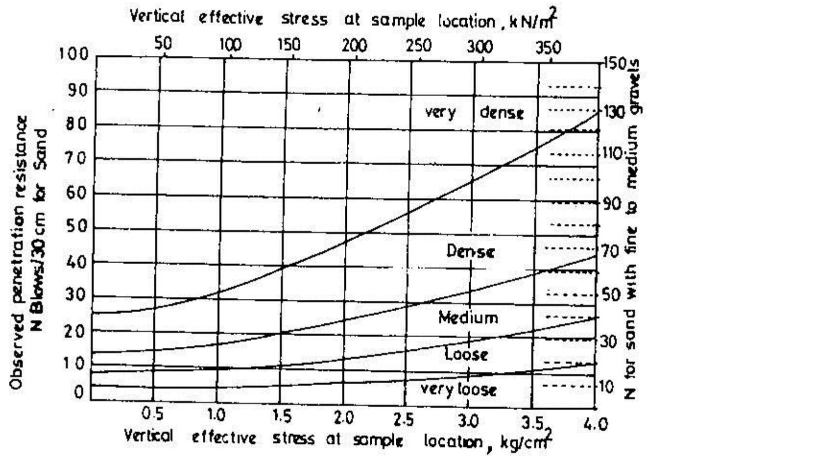


Fig. 1.11.2. (From Task Committee for Foundation Design Manual, Jour. ASCE, Soil Mech. Div. Vol. 98, No. SM 6, June 1972).

Step 1

Skemption observed that 100 percent of the energy imparted to the rod does not reach bottom of borehole. Hence skempton proposed that measured SPT value should be normalised to standard rod energy ratio (ER_r) and suggested Eq. 1.11.1.

$$N_{60} = N_m \frac{ER_r}{60} \qquad \text{...(1.11.1)}$$

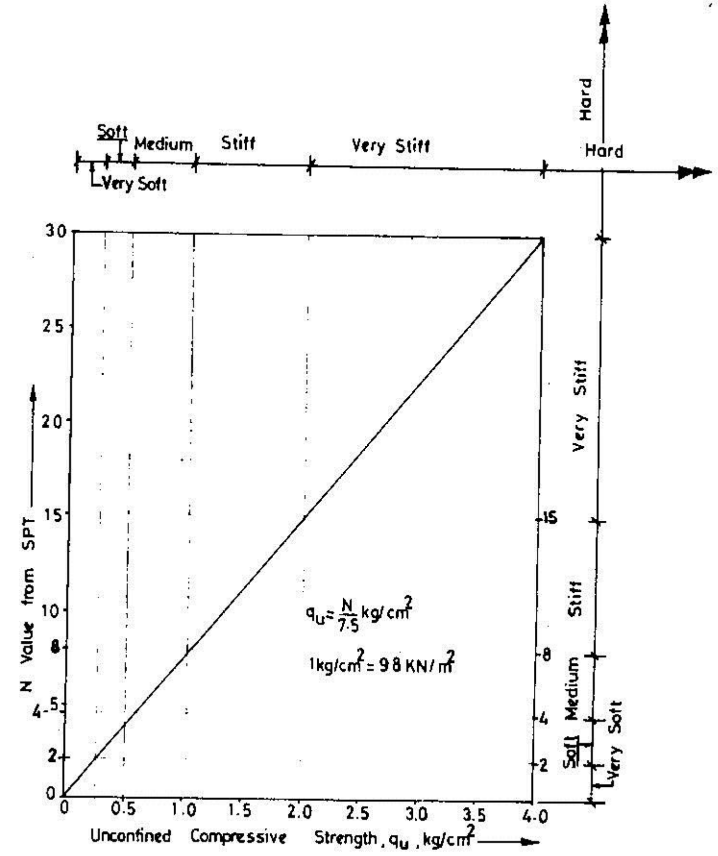


Fig. 1.11.3. N from SPT Vs consistency of cohesive soils.

where

N_m = measured SPT value

N_{60} = SPT value corresponding to rod energy ratio (ER_r) of 60 percent. This value was selected as for most of the SPT hammers used ratio of energy received at bottom of hole to that transmitted (i.e. ER_r) is 60 percent (Refer Table 1.11.2).

$\frac{ER_r}{60}$ = Ratio to be obtained from the Table 1.11.2 depending on type of SPT hammer and type of release of hammer.

Types of SPT hammers widely used are indicated in Fig. 1.11.4. In India widely used SPT hammer is "Donut" type. The type of "release" is generally "manual" or "two turns of rope". Hence based

TABLE 1.11.2. Summary of Rod Energy Ratios.

	Hammer	*Release*	ER_r : %	$ER_r/60$
Japan	Donut	Tombl	78	1.3
	Donut	2 Turns of Rope	65	1.1
China	Pilcon type	Trip	60	1.0
	Donut	Manual	55	0.9
USA	Safety	2 Turns of Rope	55	0.9
	Donut	2 Turns of Rope	45	0.75
UK	Pilcon, Dando, Old Standard	Trip	60	1.0
		2 Turns of Rope	50	0.8

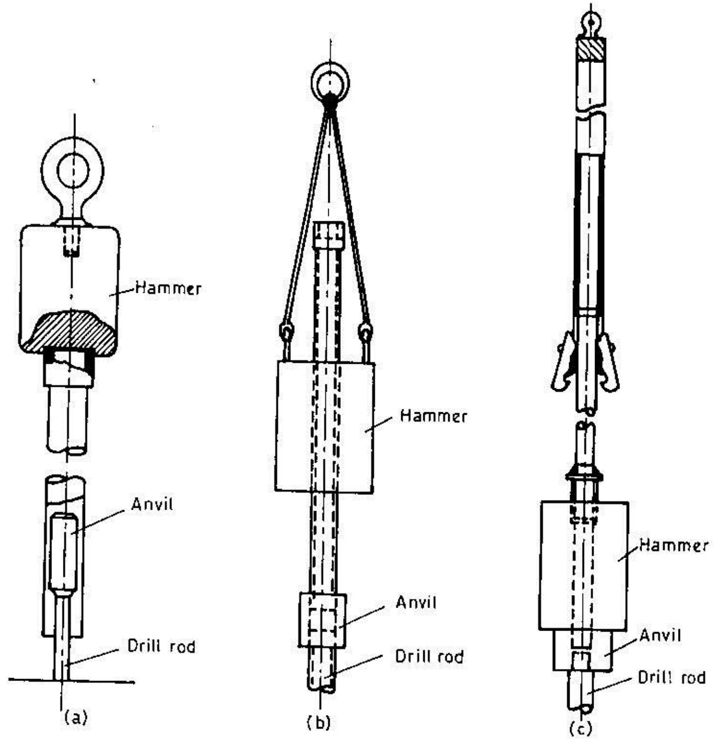


Fig. 1.11.4. SPT Hammers: [a] old standard; [b] donut; [c] trip.

on Table 1.11.2 for Indian practice $\frac{ER_r}{60}$ can be taken as 0.9 for manual release and 0.75 for two turns of rope release.

Step 2

N_{60} as obtained from Eq. 1.11.1 needs to be corrected for rod lengths, type of sampler, borehole diameter in addition to overburden pressure etc. Eq. 1.11.2 can be used to obtain the corrected N_{60}, corrected for all the above factors except for overburden pressure.

$$(N_{60})\ \text{cor} = (C_1)\,(C_2)\,(C_3)\,N_{60} \quad \text{...(1.11.2)}$$

where

(N_{60}) cor = corrected N_{60} for rod length, type of sampler and borehole diameter.

C_1 = The correction for rod length and may be obtained from Fig. 1.11.5. It may be noted that correction factor C_1 is 1 when rod length is 10 m or more.

C_2 = The correction for type of sampler. SPT sampler may be provided with a inner liner (as in USA) or may not be provided with a liner (as in India). C_2 is taken as 1.0 for sampler with liner and 1.2 for sampler without liner. Hence for Indian conditions C_2 is generally 1.2.

C_3 = The correction for borehole diameter. Such a correction is to be applied only for sandy soils. For clayey soils, C_3 is always 1. In case of sandy soils, C_3 values may be obtained from Fig. 1.11.6. C_3 is taken as 1 for borehole diameter of 65 mm to 115 mm which is widely used diameter.

Step 3

SPT value measured is affected by the effective overburden pressure at the test depth. Hence it is now standardised practice to reduce N_{60} to $(N_1)_{60}$ where $(N_1)_{60}$ is N_{60} corresponding to effective overburden pressure of 1 kg/cm^2 (98.07 kPa). $(N_1)_{60}$ from N_{60} can be obtained from Eq. 1.11.3. Once $(N_1)_{60}$ is obtained, the relative density can be obtained from Table 1.11.3.

Table 1.11.3 is applicable for only normally consolidated natural deposits. In case, sand deposits are coarse or fine sands, the relative density of normally consolidated sands can be obtained from Fig. 1.11.8.

$$(N_1)_{60} = C_N\,N_{60}$$

where

C_N = Correction factor corresponding to actual overburden pressure. C_N can be obtained from Fig. 1.11.7. Thus $C_N > 1$ for $\bar{\sigma}_v <$ 1 kg/cm^2 and $C_N < 1$ for $\bar{\sigma}_v >$ 1 kg/cm^2.

Step 4

There is evidence that the resistance of sand to deformation is more for longer period of consolidation. This "ageing" effect is reflected in higher blow counts and appears to cause an increase in the parameter 'a' of Eq. 1.11.4:

$$\frac{N_{60}}{D_r^2} = a + b\sigma_v^1 \quad (1.11.4)$$

Typical values of 'a' for normally consolidated fine sands are found to range over a short range. 'a' is taken around 315 for laboratory tests (zero ageing period in years), as 40 for recent fill of 10 years age. For aging period of 100 years and more (natural deposits) its value is taken as 50. Hence ageing period by less than 10 years, a will lie between 35 to 40.

Overconsolidation increases the coefficient b of Eq. 1.11.3 by the following factor viz.

$$\frac{1 + 2K_0}{1 + 2K_{onc}}$$

where K_0 and Konc are respectively in-situ stress ratio for the overconsolidated and normally consolidated sands.

TABLE 1.11.3. Relative density V, $[N_1]_{60}$ for normally consolidated natural sand (Curve B of Fig. 1.11.8).

D_r =	0	15	35	50	65	85	100%
	Very loose	Loose	Medium		Dense	Very dense	
$(N_1)_{60}$ =	0	3	8	15	25	42	58

Rod length (m)	3.5	5.0	8.0	10.0	>10.0
Correction factor, C_1	0.75	0.85	0.95	1.00	1.00

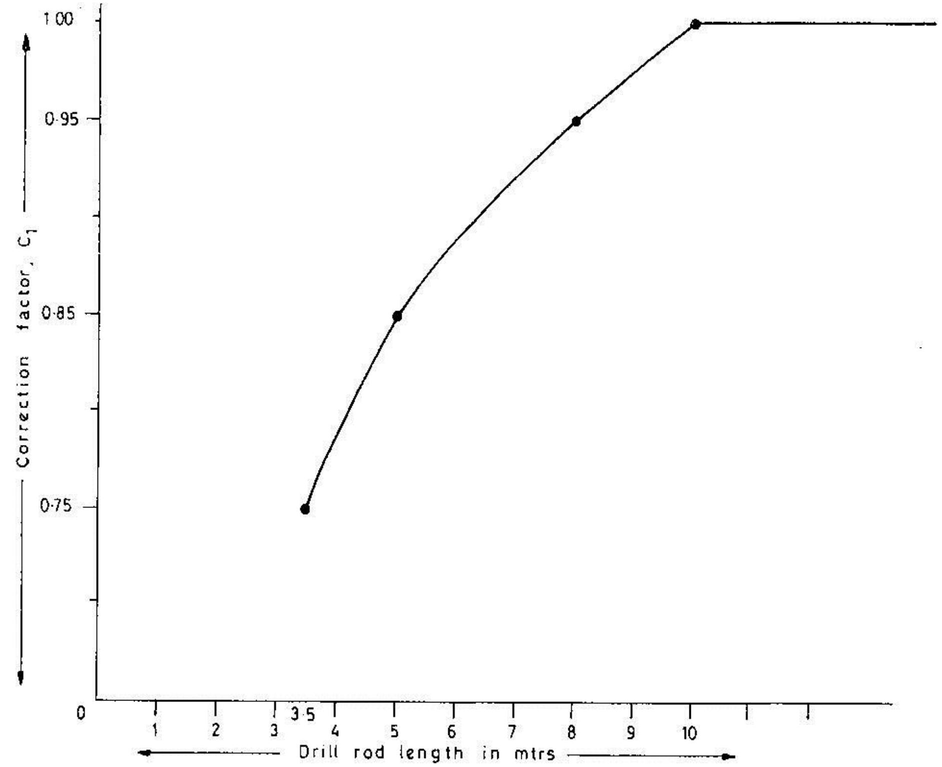


Fig. 1.11.5. Drill rod length V_s correction factor, C_1.

It is to be noted that the ratio a/b varies between 1.0 to 2.0 for normally consolidated sands. This ratio is taken as 1.0 for fine sands of medium density and 2.0 for dense coarse sand (of course both normally consolidated).

1.12. DYNAMIC CONE PENETRATION TEST

In this method a cone is driven into the ground in the same way as the SPT spoon is driven. But unlike in SPT, there is no preboring involved. There exists many varieties of penetrometers. Indian Standard Institution recommends 50 mm diameter and 65 mm diameter cones with apex angle of 60°. But very limited studies conducted to date indicate that 65 mm cone is preferable as it yields more consistent relationship with SPT values (Mohan et al. 1970). Dynamic cone test can be used with or without bentonite (mud) slurry. But when depth of investigation is more than 6 m, use of bentonite or mud slurry is recommended as otherwise friction on the rods would be tremendous. Data from dynamic cone test is plotted as a curve of penetration resistance, N_c number of blows per 30 cm of penetration, versus depth.

N_c values from dynamic cone penetration tests needed to be corrected for overburden pressure in cohesionless soil like N-values of SPT.

Borehole dia. (mm)	65	115	150	200
Correction factor, C_3	1.00	1.00	1.05	1.15

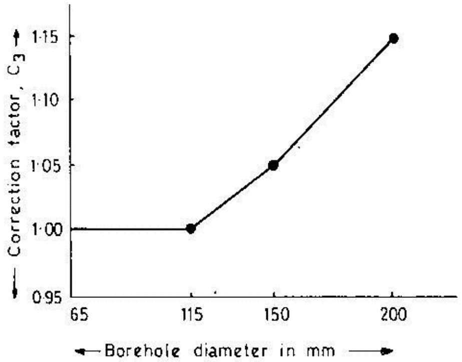


Fig. 1.11.6. Borehole diameter V, correction factor, C_3.

To judge the consistency of soils from N_c values, the general practice often used is to convert N_c into N values of SPT. The relationship commonly used is,

$$N_c = \frac{N}{C} \qquad \text{...(1.12.1)}$$

Where N_c and N are blow count values from dynamic cone and SPT for corresponding depths in the same subsoil,

and C = a constant (found generally to lie between 0.8 to 1.2 when bentonite is used with 65 mm dynamic cone test). In Eq. 1.12.1 it is always advisable that C value be fixed for each site by conducting SPT and cone tests nearby each other. When bentonite is not used with the test, following equations may be used for preliminary analysis;

$N_c = 1.5\,N$, for depths up to 3 metres ...(1.12.2 *a*)

and $N_c = 1.75\,N$, for depths between 3 and 6 metres ...(1.12.2 *b*)

1.12.1 Advantages of Dynamic Cone Penetration Test over Boring and SPT

(*i*) Dynamic cone penetration test is faster and more economical. The penetrometer is primarily useful in mapping of soil strata during the early stages of explorations when the number of borings is normally limited. During detailed investigation some geotechnical engineers may prefer to substitute a single bore hole by a number of dynamic cone tests and still maintain the cost and obtain more relevant information between the borings.

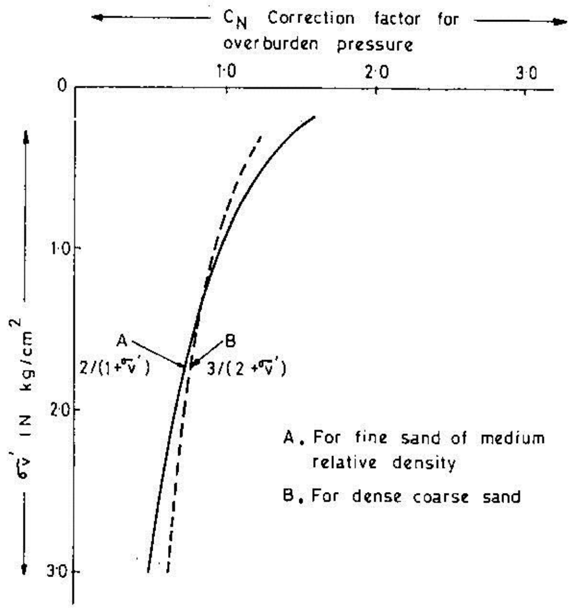


Fig. 1.11.7. C_N, correction factor for overburden pressure $V_s\ \bar{\sigma}_v$

(*ii*) It gives continuous penetration of strata penetrated. It often reveals the presence of strata which are not recovered or observed in sampling operations.

1.12.2. Limitations of Dynamic Cone Penetration Test

(*i*) The major limitation of the dynamic cone penetration test is that either no samples or only wash samples are obtained from it, therefore strata cannot be definitely be identified by soundings alone.

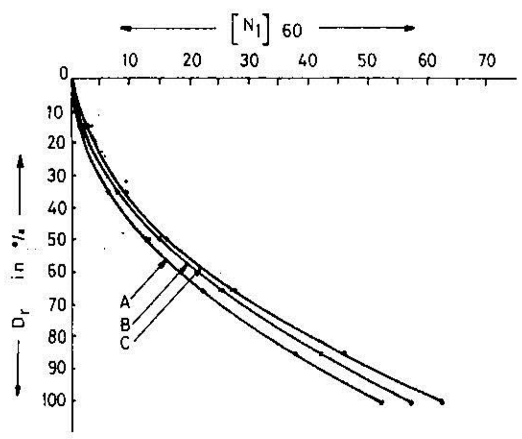
(*ii*) Presence of gravels/boulders within the soil strata can give misleading results. Consequently interpretation of results obtained from dynamic cone penetration test requires considerable experience, particularly in those areas in which correlations between the penetration resistance and engineering properties of soils penetrated are to be developed.

1.13. STATIC CONE PENETRATION TEST

This test is widely used in Holland, Belgium, Britain, Indonesia, Malaysia, Singapore, West Indies etc. It is finding increasing use in India.

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

Immediately above the cone a friction jacket, of outside diameter greater than mantle tube, is fitted. The cone and the friction jacket in combination or



A. Normally Consolidated Fine Sand.
B. Normally Consolidated Natural Sand.
C. Normally Consolidated Consolidated Coarse Sand.

Fig. 1.11.8. Relative density V_s normally consolidated sands.

separately are pushed into the ground by hydraulic cylinder of a machine of capacities presently varying from 2 tonnes to 10 tonnes (in India). The necessary reaction is obtained by anchors and some time by surcharge loading.

By pushing the cone alone and along with friction jacket values of cone resistance and skin friction are obtained with depth. Usually readings are obtained at interval of 0.5 m and graphs of cone resistance and skin friction are plotted against depth. The tests are usually conducted to refusal.

In India, at present, equipments of capacity upto 100 *kN*(10 *t*) are available whereas in other countries equipments even of capacity of 300 *kN* (30*t*) are available.

More advanced countries have started using electrical cone equipment, generally with heavy equipment (30 *t* capacity). With such an equipment continuous readings of cone resistance and side friction are obtained with depth at read out unit. This equipment can also be provided with sensor to measure pore-water pressure.

The advantages of the electric cone are:

(*a*) accuracy and repetibility of results particularly in weak soils.
(*b*) better delineation of data.
(*c*) faster overall speed of operation.
(*d*) more manageable data handling.
(*e*) possibility of incorporating pore pressure measurements or additional sensors in or above the tip.

Main limitations of such an equipment is its high cost and non-availibility of same in India.

Cone resistance values obtained, for given soil, depend on the following factors.

(*a*) Cone diameter. Bigger the cone diameter, smaller is the cone resistance. With a 50 mm diameter cone, cone resistance values of 14 percent less were obtained than with the more widely adopted standard diameter of 35.7 mm (cone angle considered 60°).

Variation due to this factor can be eliminated by standardising the cone. Fortunately in most countries including India cone diameter of 35.7 mm is used with

cone angle of 60°. With this the area of cone at its base works out to 10 cm^2.

(*b*) Rate of penetration. Rate of penetration is now standardised at 20 mm/sec with permissible variation of ± 5 mm/sec. Effect of such a variation is generally negligible in either sand or clay. However it is to be realised that even with standard rate of penetration liquefaction or near liquefaction can occur in very loose saturated sand giving abnormally low resistance. In clays higher rate of penetration gives lower penetration resistance as noted in Fig. 1.13.1.

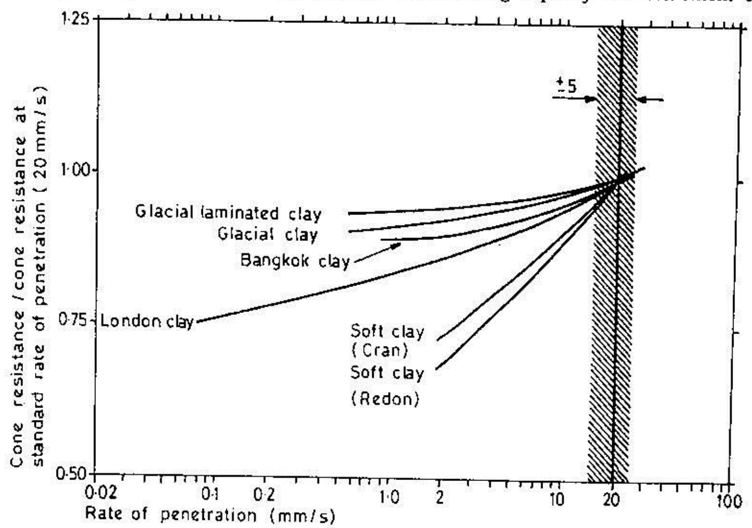
(*c*) Surface roughness of the cone. Cone resistance increases with increase in roughness of cone surface. A newly manufactured cone has a surface friction angle, δ, of about 0.5 ϕ, where ϕ is the angle of shearing resistance of soil. A 50 percent increase to 0.75 ϕ gives rise to an increase in generation resistance of about 50 per cent. Similarly sleeve friction increases with increase in surface friction of sleeve. Hence it is recommended that roughness of surfaces of cone and sleeves to be mainted at or less than 0.5 μm.

One of the important uses of penetration test is to delineate the soil profile. This can be done with greater accuracy with penetration testing than with conventional boring and sampling. However accuracy is not greater than that can be achieved with certain samplings such as Delft continuous sampler or by continuous sampling with a stationary piston sampler. Main advantages of sampling is that soil is available for visual inspection.

It is to be noted that cone resistance responds to soil changes within 5 to 10 top diameters above and below the cone, the distance increasing with soil stiffness. Hence there exists some imprecision in locating soil interfaces. A thin layer of sand within a clay stratum may not be detected if it is less than 100 mm thick. Similarly a clay layer within sand may not be detected if it is less than 150 to 250 mm thick.

A broad identification of soils can be obtained from the magnitude of the cone resistance, and more specifically from their friction ratios, R_s (i.e. the ratio of local side friction, l_s to cone resistance, q_c. This friction ration R_s is expressed as percent) at the same level by use of Fig. 1.13.2. A comprehensive scheme by Donglas and Olsen (1981) is given in Fig. 1.13.3 (*a*) and simplified version is given in Fig. 1.13.3 (*b*).

The results of static cone tests are correlated directly with bearing capacity and settlement of shallow



The effect of rate of penetration in clays (After Meigh, 1987)

Fig. 1.13.1. The effect of rate of penetration in clays (after meigh, 1987).

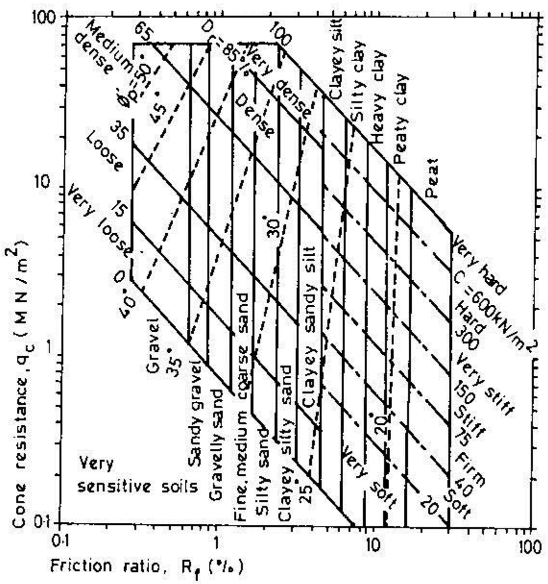


Fig. 1.13.2. Identification of soils using the Dutch mechanical friction sleeve penetrometer (from Searle, 1979).

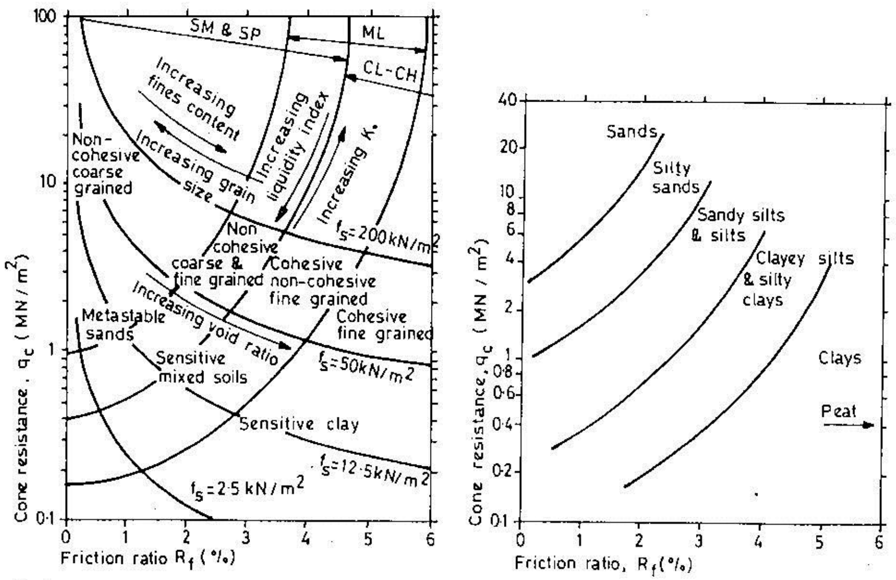


Fig. 1.13.3. Identification of soils using penetrometer. (*a*) Full scheme (after Douglas and Olson, 1981). (*b*) Working version (after Robertson and Campanella, 1983).

foundations and piles. These aspects are discussed in chapters 2 and 3 respectively. Empirical relationships relating cone resistance to relative density and angle of internal friction are also available as indicated in simplest form in Table 1.13.1. In Table 1.13.1, cone resistance values have not been corrected to overburden pressure. This is one of the major limitations of relationships presented in Table 1.13.1.

TABLE 1.13.1. Empirical Values of ϕ and D_r of Sands and Coarse Silts (Non-Plastic) Based on Cone Resistance of Static Cone Penetration Test

Cone Resistance C_{kd} kg/cm^2	*Relative Density D_r %*	*Angle of Internal Fricition Φ, Degrees*
<20	<20	25–30
20–40	20–40	30–55
40–120	40–60	35–40
120–200	60–80	40–45
>200	>80	>45

The relationship of Table 1.13.1 does not account for compressibility of sands. Based on laboratory tests, Jamiolkowsi et al. (1985) have recommeded relationship as in Fig. 1.13.4, which is applicable to uncemented normally consolidated quartz sands, with horrzontal in-situ stress ratio K_o close to 0.45. It may be noted that for a given relative density and overburden pressure, a sand of high compressibility has a lower q_c than a sand of low compressibility and this is reflected by two limiting parallel lines in Fig. 1.13.4.

As noted in Fig. 1.13.4, a correction for chamber size is to be applied while using the above figure. The correction for chamber size is made by dividing q_c by K_q where the correction factor K_q given by

$$K_q = 1 + \frac{0.2(D_r - 30)}{60} \qquad ...(1.13.1)$$

Because K_q is a function of D_r, it is necessary to determine D_r from Fig. 1.13.4 and Eq. 1.13.1 on a trial and error basis.

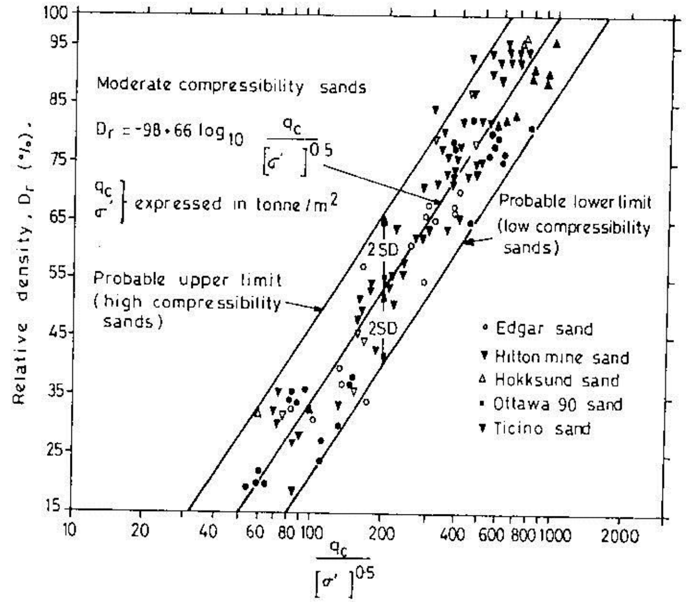


Fig. 1.13.4. Relationship between relative density and cone resistance of uncemented, normally-consolidated quartz sands (after Jamiolkowski et al., 1985) (Note: Correction required for chamber size).

If sand is over consolidated (OC), schmertmann (1975) has suggested a relationship between q_c for OC and q_c for NC sand of the form

$$q_c\,(OC)/q_c\,(NC) = 1 + \alpha\,[(OCR)^{\beta} - 1] \quad ...(1.13.2)$$

where α and β have the values 0.75 and 0.42 respectively. Baldi et al (1983) found that β varies with relative density as

$$\beta \approx 0.275 + 0.26\,D_r \quad ...(1.13.3)$$

and α varies approximately from 0.5 (for OCR = 2) to 0.25 (for OCR = 15), decreasing with increasing D_r.

Schmertmann (1978) proposed the relationship given in Fig. 1.13.5 to obtain peak effective angle of shearing resistance, ϕ' of free draining sands and this relationship is widely used.

A direct correlation between q_c and Φ' is shown in Fig. 1.13.6. It is derived from a bearing capacity theory developed by Durgunglu and Mitchell (1975), using a soil-to-cone friction angle equal to $0.5\,\phi'$ and a lateral earth pressure coefficient, $K_o = 1 - \sin\phi'$. The theory ignores the effects of soil compressibility.

Cone resistance values from static cone penetration tests have also been related to N values of SPT. In general the relationship can be presented as,

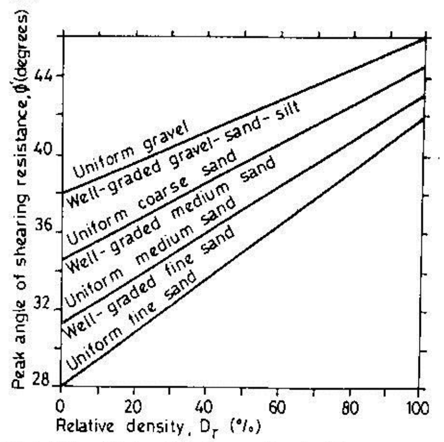


Fig. 1.13.5. Relationship between peak angle of shearing resistance and relative density of quartz sands (after Schmertmann, 1978).

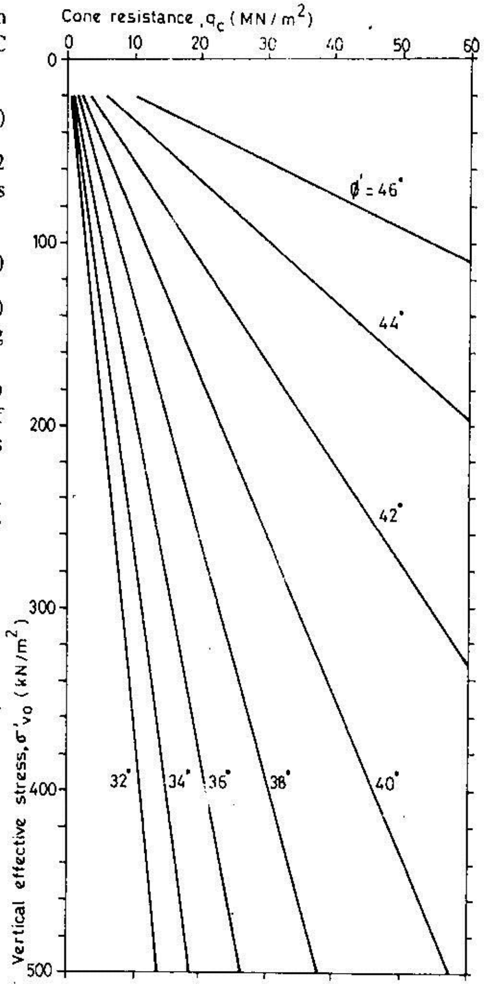


Fig. 1.13.6. Relationship between angle of shearing resistance and cone resistance for an uncemented: normally-consolidated quartz sand (after Durgunoglu and Mitchell, 1975).

$$N = \frac{C_{kd}}{C} \quad ...(1.13.4)$$

Where N = Blow count value of SPT, uncorrected for overburden pressure

C_{kd} = Cone resistance values in kg/cm²

C = An empirical constant to be established for each site. C is generally found to be ranging from 2 (for silts) to 6 (for gravels) as noted in Table 1.13.2.

The relationship given in Table 1.13.2 is the one proposed by schmertmann (1970). Based on further extensive data burland and Burbidge (1985) proposed the relationship given in Fig. 1.13.7.

Once C_{kd} is converted to N of SPT, these values can be used for estimate of relative density, angle of internal friction, allowable bearing pressure, pile capacity etc.

There is a considerable scatter in Fig. 1.13.7. Because of this uncertainty, direct use of static cone penetration results is preferable to conversion to N values of SPT.

1.13.1. Advantages of Static Cone Penetration Test over Boring and SPT

(*i*) Static cone penetration test is faster and much cheaper.

(*ii*) In cohesionless soils, particularly below the ground water table, SPT may yield misleading values as a result of loosening, sand blowing etc. In such strata, static cone penetration test is very beneficial.

(*iii*) Static cone penetration test gives practically a continuous resistance record of strata, which is generally not the case with boring.

(*iv*) Static cone penetration test gives values of skin friction with depth which is useful in estimating skin friction for piles.

TABLE 1.13.2. Relationship Between N of SPT and C_{kd} of Static Cone Penetration Test (From Schmertmann, 1970).

Soil Type	$C_{kd}/N(1)$	$C_{kd}/N(2)$
Silts, sandy silts, slightly cohesive silts-and mixture	0.2	2.0
Clean, fine to medium sands and slightly silty sands	0.3–0.4	3.5
Coarse sands and sands with little gravel	0.5–0.6	5.0
Sandy gravel and gravel	0.8–1.0	6.0

Note: C_{kd} in MPa in (1) and kg/cm² in (2).

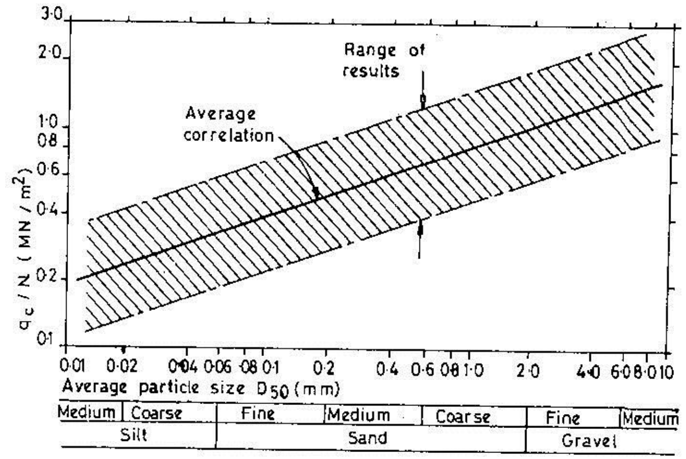


Fig. 1.13.7. Relationship between cone penetration test and standard penetration test (from Burland and Burbidge, 1985).

1.13.2. Limitations of Static Cone Penetration Test

(*i*) Test is unsuitable for gravelly soils.

(*ii*) Disturbed or undisturbed samples cannot be obtained with this test for vishal inspection etc.

(*iii*) Test depth is limited generally to 15–20 metres. Often depth penetrated would be less if smaller capacity machines are used or harder strata is obtained at shallow depths, or enough-anchorage is not available at shallow depths.

1.14. FIELD VANE SHEAR TEST

The vane test is used for determination of the undrained cohesion of clay, particularly very soft to medium stiff clay. Vane test is most valuable in sensitive clays wherein it is difficult to obtain truely undisturbed samples without disturbing their in-situ strength. The vane test is also useful in finding out sensitivity of subsoil by determining strength in undisturbed and remoulded states. The ratio of strength in undisturbed to remoulded state is known as sensitivity.

The test basically consists of inserting a four-bladed vane in the undisturbed/remoulded soil at required depth and rotating it from the surface through link rods to determine the torsional force required to cause a cylindrical surface to be sheared by the vane. This is then converted into unit shearing resistance in undrained condition, assuming failure along the cylindrical surface. It is to be noted that soil should in a saturated condition if the vane test is to be conducted, because, as mentioned above, cohesion in undrained condition is assumed to be obtained by this test.

Another major advantage of a vane shear test is that this test is executed in field conditions which are generally difficult to simulate in the laboratory, to obtain shear strength. For example, in laboratory tests lateral stresses, initial pore water pressure etc., are usually different than those existing in field.

1.15. FIELD PERMEABILITY TEST

In-situ permeability characteristics are often obtained by conducting permeability tests in boreholes. These tests are far cheaper and very rapid, though less accurate as compared to pumping tests. Two types of borehole permeability tests are in common use, namely the variable head and constant head methods.

Variable head test is recommended when the permeability of subsoil is low enough to permit accurate determination of the water level. This test may be run as a falling head test, the flow is from the hole to the surrounding soil. In such a test there is danger of clogging of soil pores by sediment in the test water used. Hence every care must be taken to use clean water for this test. Such eventuality does not exist in the rising head test, wherein water flows from the surrounding soil to the hole. However, there is possibility of the soil at the bottom of the hole being loosened if too great a gradient is imposed at the bottom of the hole. If the rising head test is used, it is recommended that the test should be followed by sounding of the base of the hole with drill rods to determine whether any heaving of the bottom has occurred. If significant heaving has occurred it indicates that falling head test should have been preferred for this stratum. In general, rising head test is preferable to falling head test.

Different methods of execution of variable head test and relevant formulae for estimating the coefficient of permeability are given in Fig. 1.15.1. From this figure, it may also be noted that tests can be run so as to obtain vertical permeability, horizontal permeability or mean permeability.

When the permeability of the subsoil is so high as to preclude accurate measurement of rising or falling head, a constant head test is used to predict the permeability of subsoil. In this method, water is added to the casing at a rate sufficient to maintain a constant water level at or near the top of the casing for a time period not less than 10 minutes. The water may be added by pouring from calibrated containers or by pumping through a water meter. Formulae for determination of the permeability by this method are given in Fig. 1.15.2.

Both the above tests are conducted in soils. In rock, the test is conducted as a packer test in which water

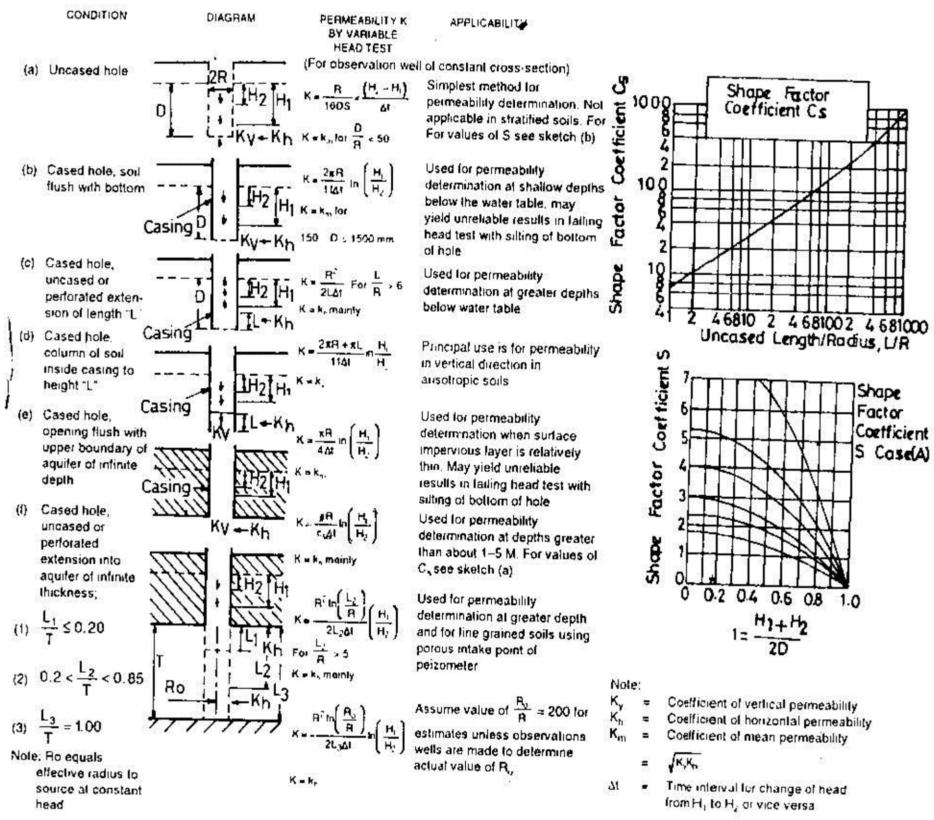


Fig. 1.15.1. Computation of permeability from variable head test in boreholes.

under pressure is forced into the rock through the walls of the borehole in rock. Hence this test also falls under constant head type. Such a test provides means of determining the apparent permeability of the rocks and yields information regarding its soundness. The information thus obtained is generally used in seepage studies and/or as a qualitative measure of the grouting required for impermeabilisation or strengthening of the rock. The formulae used to compute the permeability from packer or pressure test data are

$$k = \frac{Q}{2\pi LH} \ln \frac{L}{r}, \text{ for } L \geq 10\, r \quad \ldots(1.15.1a)$$

and $$k = \frac{Q}{2\pi LH} \sinh^{-1} \frac{L}{2r}, \text{ for } 10r > L \quad \ldots(1.15.1b)$$

where, k = coefficient of permeability

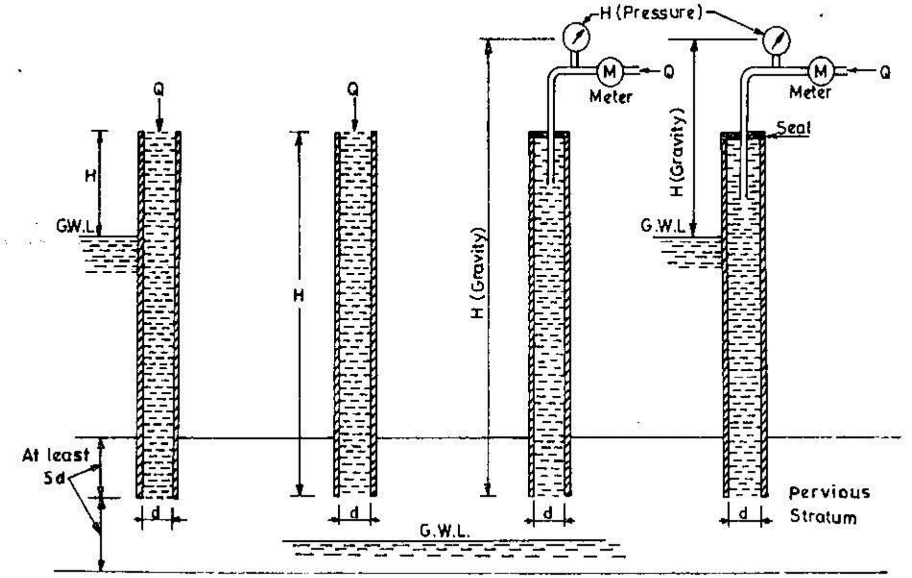
Q = constant rate of flow into the hole

L = length of test section

H = differential head of water on the test section

r = radius of the borehole

The above formulae in Eq. 1.15.1 provide only approximate values of k since they are based on several simplifying assumptions and do not take into account the flow of water from the test section back to the borehole.



Note : $k = \dfrac{Q}{2.75dH}$

where Q = constant rate of supply

H = difference in water level in pipe over GWL in (*a*)

= difference in water level in pipe over bottom of pipe in (*b*)

= H(gravity) + H(pressure) in (*c*) and (*d*)

d = diameter of pipe.

(*a*) & (*b*) Water poured manually at constant rate.

(*c*) & (*d*) Water added through meter at constant rate.

Fig. 1.15.2. Constant head permeability test.

Boreholes in which permeability tests in overburden or rock are to be performed should be drilled using only clear water as the drilling fluid till the last of the permeability tests in that hole is conducted. This prevents the forming of a mud cake on the walls of the hole or clogging of the pores.

1.16. PRESSUREMETER TEST

The pressuremeter described herein is the Menard Pressuremeter, manufactured in France. The typical set up is schematically given in Fig. 1.16.1.

The two main components of the set-up are a probe and a pressure-volumeter which are connented by plastic tubes through which water and gas (CO_2) are applied. The probe consists of three independent cells, two guard cells at either end and a central measuring cell. The central cell contains liquid under gas pressure whereas guard cells are pressurised under gas pressure only. The deformations are measured for the central cell only where conditions of plane strain are assumed to exist due to presence of guard cells.

The probes are available in three sizes to suit the standard borehole dimensions *AX*, *BX* and *NX*.

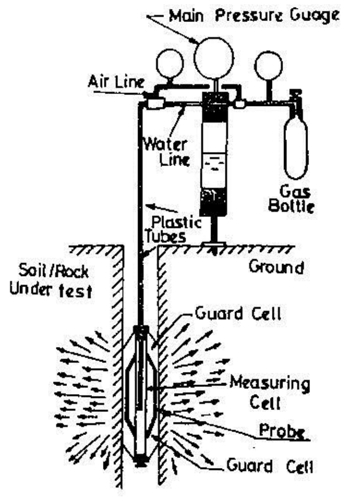


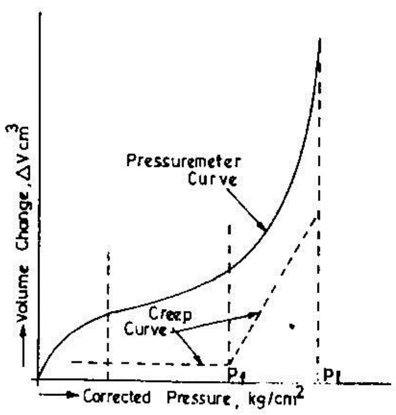
Fig. 1.16.1. Menard pressuremeter.

To conduct the test, the probe is inserted in a premade borehole of suitable dimension to required depth or the probe is driven to required depth. The probe is expanded by applying incremental pressures. The incremental pressure is maintained for varying period depending upon the type of test. For an undrained test, pressure is maintained for a minute or two. During this period, readings for volume change are taken. The probe pressure, applied in steps, is increased till *limit pressure* is reached or till required maximum cell pressure is obtained. Limit pressure corresponds to ultimate pressure the borehole could stand.

A typical stress-deformation curve (a pressuremeter curve) and a creep curve obtained from a pressuremeter test are illustrated in Fig. 1.16.2. From the curve it is evident that limit pressure refers to the maximum pressure which the soil or rock can sustain in a pressuremeter test. The limit pressure is related to cohesion and angle of internal friction as noted below.

For cohesive soils in undrained condition:

$$p_L = \sigma_{ho} + c_u\left[1 + \log_e \frac{E}{2c_u(1+\mu)}\right] \quad ...(1.16.1a)$$



Note: p_L = Limit pressure.

p_f = Creep pressure

Corrected Pressure = Pressure applied to probe—pressure utilised in expansion of the probe, which is to be obtained by calibration

To obtain pressuremeter curve, volume changes at 60 seconds are adopted and to obtain creep curve, volume changes between 30 and 60 seconds are considered.

Fig. 1.16.2. Typical pressuremeter curve and creep curve.

or $$p_L = \sigma_{ho} + c_u\left[1 + \log_e \frac{G}{c_u}\right] \quad ...(1.16.1b)$$

as $$E = 2G(1+\mu) \quad ...(1.16.1c)$$

Hence. $$c_u = \frac{p_L - \sigma_{ho}}{1 + \log_e \frac{G}{c_u}} \quad ...(1.16.1d)$$

or $$c_u = \frac{p_L - \sigma_{ho}}{N_p} \quad ...(1.16.1e)$$

where $$N_p = 1 + \log_e \frac{G}{c_u} \quad ...(1.16.1f)$$

where in Eq. 1.16.1,

p_L = limit pressure (It is reached theoretically when $\Delta V/V = 1$)

σ_{ho} = undisturbed in-situ lateral soil pressure

c_u = undrained cohesion

μ = poisson's ratio (Refer Tables 1.16.1 and 1.16.2).

E = deformation modulus in the elastic range.

N_P = pressuremeter constant.

N_P is found to range generally between 5.6 to 6.8 with an average value of 6.2 (Mair et al, 1987). N_P = 6.2 corresponds to G/c_u = 180. Range of ± 10%, gives the range of 5.6 to 6.8 to N_P. This corresponds to range of G/c_u values from about 90 to 330.

Shear modulus value can be obtained from pressuremeter curve. Since preboring is involved in MPM test (see below), G can best to obtained from small unload-reload cycles (Fig. 1.16.3).

The difference between limit pressure and undisturbed in-situ lateral pressure is known as net limit pressure p_L*. Thus

$$p_L{}^* = p_L - \sigma_{ho} \qquad ...(1.16.1g)$$

Baguetin et al (1978) has proposed relation between net limit pressure and consistency of clays and this is given in Table 1.16.1.

$$E = \frac{2\Delta p}{\Delta V}(1+\mu)V_0 \qquad ...(1.16.2)$$

where ΔV = Volume change, from initial volume V_o, corresponding to pressure change Δp in the elastic range.

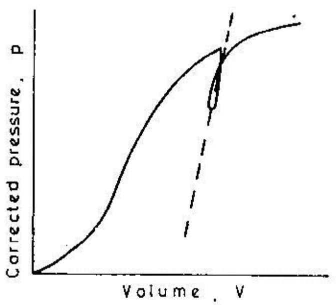


Fig. 1.16.3. Unload reload cycle for determining elastic modulus.

Eq. 1.16.2 applies to any type of soil, cohesive or cohesionless; weathered rock and also rock. Hence it is a generalised equation applicable to pressuremeter tests.

In Eq. 1.16.2, value of μ may be selected from Table 1.16.2 and/or Table 1.16.3.

For cohesionless soils:

$$p_L = p_0 N\phi^2 \qquad ...(1.16.3a)$$

wherein

$$N_\phi = \tan^2 45 + \frac{\phi}{2} \qquad ...(1.16.3b)$$

and ϕ = angle of internal resistance of soil.

TABLE 1.16.1. Net limit pressure versus consistency of clays.

p_l* (kN/m²)	*Description*	*Field Test*	*Undrained Shear strength (kN/m²)*
0 to 75	Very Soft	Penetrated by first, squeezes easily between fingers	< 20
75 to 150	Soft	Penetrated by finger, easily moulded.	20 to 40
150 to 350	Firm	Penetrated with difficulty, moulded by strong finger pressure	40 to 75
350 to 800	Stiff	Indented by strong finer pressure	75 to 150
800 to 1600	Very Stiff	Indented only slightly by finger pressure	>150
>1600	Hard	Cannot be indented by finger pressure, penetrated by finger-riail or pencil point	

TABLE 1.16.2. Typical Range of Values for Poisson's Ratio (from Wintercorn and Fang 1975; Bowles 1968)

Soil Type	*Poisson's Ratio*
Clay, saturated	0.50
Clay with sand and silt	0.30–0.42
Clay, unsaturated	0.35–0.40
Loess	0.44
Silt	0.30–0.35
Sandy soil.	0.15–0.25
Sand	0.30–0.35
Rock	0.10–0.40

TABLE 1.16.3. Typical Range of Values for Poisson's Ratio (from Lambe and Whitman, 1969)

Material	*Poisson's Ratio*
Soils	
Firm to stiff clays (undrained)	0.4–0.5
Loess	0.1–0.3
Silt	0.3–0.35
Fine Sand—medium dense	0.25
Sand—loose	0.2–0.35
Sand—dense	0.3–0.4
Rocks	
Sound, intact igneous and metamorphics	0.25–0.33
Sound, intack sandstone and limestone	0.25–0.33
Sound, intact shale	0.25–0.30
Other Materials	
Concrete	0.15–0.25
Ice	0.36
Steel	0.26–0.29

Cahloon (1970) gives a chart, given in Fig. 1.16.4 relating ϕ value to limit pressure and deformation modules of pressuremeter test.

Ramaswamy et al (1982) have attempted to establish corelationships between data from pressuremeter tests, standard penetration tests and static cone penetration tests. They conducted these 3 types of tests at three different sites with sandy subsoils. The results of their study are presented in Table 1.16.4.

From the Table 1.16.4 it is evident that no well established correlationships can be given even just to cover cohesionless soils. In any case it is evident that correlationship between limit pressure and cone resistance is more consistent than between limit pressure and blow count of standard penetration test.

Baguelin et al (1978) has proposed the relation given in Table 1.16.5 between net limit pressure and relative density of sands.

Settlement Estimates

Modulus of deformation obtained from pressuremeter test can be used in estimating foundation settlement. The settlement of a foundation, in general, consists of three components *viz.*

$$\delta = \delta_1 + \delta_2 + \delta_3 \qquad ...(1.16.4a)$$

where δ_1 = elastic settlement in relation to micro-deformation

δ_2 = settlement due to shear deformation without volume change

δ_3 = settlement due to volume-tric change (consolidation settlement)

Formulae for δ_1, δ_2, and δ_3 are (for B>60 cm),

$$\delta_1 = \frac{1+\mu}{6E_e} p_e B_0 \left(\frac{B}{B_0}\right)^{\alpha} \qquad ...(1.16.5)$$

$$\delta_2 = \frac{1+\mu}{6E}(p - p_e)B_0 \left(\lambda_2 \frac{B}{B_0}\right)^{\alpha} \qquad ...(1.16.6a)$$

$$\delta_3 = \frac{\alpha\, p_0 B \lambda_3}{9E} \qquad ...(1.16.7)$$

Where μ and E are as defined in Eq. 1.16.1 (Note: An approximate relationship between pressuremeter

TABLE 1.16.4. Correlations between Limit Pressure, SPT and Cone Resistance

Soil Type	*Location*	*Correlation Factor* $\frac{N}{p_L}$	$\frac{C_{kd}}{p_L}$	$\frac{C_{kd}}{N}$
Coarse to medium sand:	Changi, Singapore			
Loose state		10	7	0.7
Dense state		10	7	0.7
Calcerous sand:	Pulau Ayer Merbau Singapore			
Loose state		35–45	2.5–7	0.1
Dense state		10–35	7–12	0.5–0.7
Silty sand:	Ashuganj, Bangladesh			
Loose state		30	6–9	0.2–0.3
Dense state		30	6–9	0.2–0.3

Note: 1. Tests were conducted in natural state (loose) and on compaction by dynamic consolidation method. The latter represented the dense state.
2. In above table p_L and C_{kd} are expressed in *MPa*.

TABLE 1.16.5. Net Limit Pressure Versus Consistency of Sands

p_L (kN/m^2)	*Description*	*SPT N value*
0 to 200	Very loose	0 to 4
200 to 500	Loose	4 to 10
500 to 1500	Medium dense	10 to 30
1500 to 2500	Dense	30 to 50
>2500	Very dense	>50

modulus E and N values of SPT is given in Fig. 1.16.5) and

p = Pressure applied to soil/rock

p_e = Pressure corresponding to micro-deformation zone

E_e = Young's modulus corresponding to micro-deformation

B = Width (diameter) of foundation

B_o = Reference width (diameter) which is taken as 60 cm

λ_2, λ_3 = Shape factors obtained from Table 1.16.6

α = a rheological factor obtained from Table 1.16.7

TABLE 1.16.6. Shape Factors Versus Foundation Type

	Shape Factor λ_2	λ_3
Circular	1.0	1.0
Square	1.12	1.1
Rectangular; $\frac{L}{B} = 2$	1.53	1.2
$\frac{L}{B} = 3$	1.78	1.3
$\frac{L}{B} = 5$	2.14	1.4
$\frac{L}{B} = 10$	2.65	1.5

TABLE 1.16.7. Rheological Factor Versus Soil Type

Soil Type	*Suggested Value of Rheological Factor,* α
Peat	1
Sand and gravel	1/4
Sand	1/3
Silt	1/2
Clay, normally consolidated	2/3
Clay, over consolidated	1
Rock, highly weathered	2/3
Rock, others	1/2

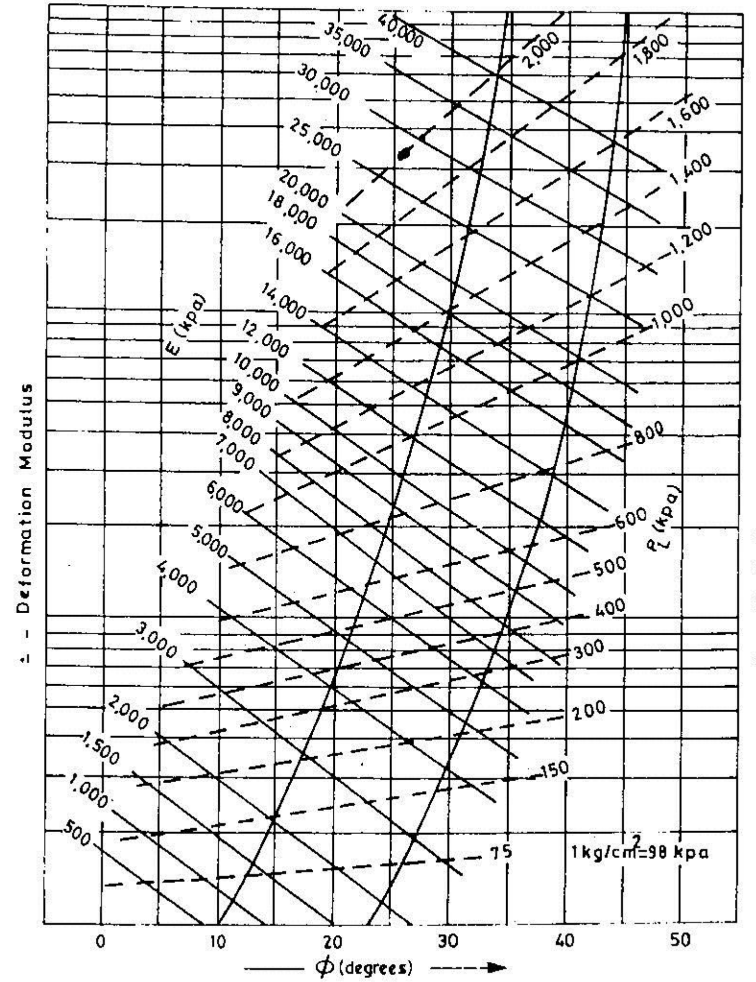


Fig. 1.16.4. Calhoon's chart relating ϕ with pressuremeter results (After Baguelin et al. 1978).

Settlement, δ_1 is usually very small and hence generally neglected.

Then $\delta = \delta_2 + \delta_3$ ...(1.16.4*b*)

Also p_e is small compared to p and hence it is also neglected. Then Eq. 1.16.6*a* becomes

$$\delta_2 = \frac{1+\mu}{6E} pB_0 \left[\lambda_2 \frac{B}{B_0}\right]^\alpha \quad ...(1.16.6b)$$

If foundation width (diameter) is less than 60 cm, then Eq. 1.16.6*b*, takes the form

$$\delta = \frac{1+\mu}{6E} pB\lambda_2^\alpha \quad ...(1.16.6c)$$

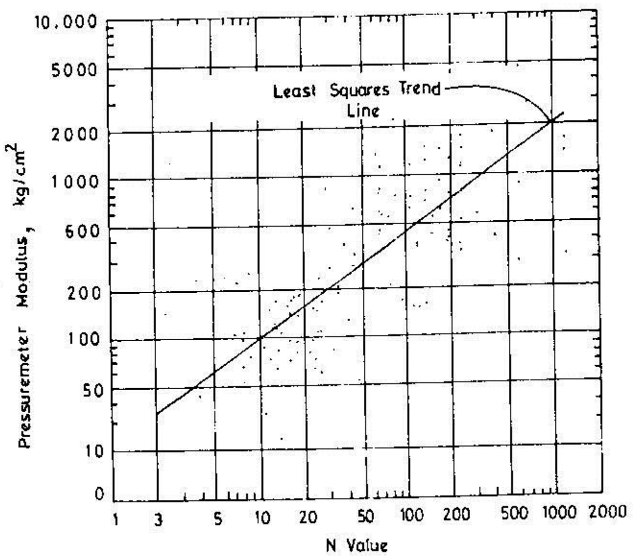


Fig. 1.16.5. Pressuremeter modulus versus *N* value data.

From the very wide experience accumulated in France, Canada including limited experience gained in India, certain empirical relationships can be given and certain typical values can be specified for different categories of soils as noted herein below.

The limit pressure is usually 1.5 to 2.2 times the creep pressure. For a given soil, this ratio is constant. In case of clays, this value is close to 1.5, may be generally between 1.6 to 1.8. In case of sand this value is 2.0 or higher. For dilatent sands this value can go upto 3 or 4. Baguelin et al (1978) have recommended the following relationship between *N* value of SPT and limit pressure

$$\frac{N}{p_L} = 2 \qquad \ldots(1.16.8)$$

where p_l = limit pressure in kg/cm².

In table 1.16.8 typical values of modulus of deformation and limit pressure for wide range of soils are given. Based on this, Table 1.16.9 is developed which gives the ratio of *E* to p_L for different soil types.

TABLE 1.16.8. Typical Values of E and p_L for Different Soil Types

Soil Type	*Modulus of Deformation E, kg/cm²*	*Limit pressure, kg/cm²*
Peat and very soft clay	2 to 15	0.2 to 1.5
Soft clay	5 to 30	0.5 to 3
Medium (firm) clay	30 to 80	3 to 8
Stiff clay	80 to 400	6 to 25
Marl	50 to 600	6 to 40
Loose silty sand	5 to 20	1 to 5
Silt	20 to 100	2 to 15
Sand and gravel	80 to 400	12 to 50
Sedimentary sand	75 to 400	10 to 50
Recent fill	5 to 50	0.5 to 3
Old fill	40 to 150	4 to 10

E as obtained from Eq. 1.16.2 refers to *E* obtained by applying pressure to a cylinder in radial directions. However often engineering structures transmit vertical loads to soil/rock as in a plate load test. From *E* of pressuremeter test E_{pl} corresponding to plate load test is obtainped as follows:

TABLE 1.16.9. Ratio of E to p_L for Different Soils

Soil Type	*E/p_L Value*
Very loose to loose sand	4 to 7
Compact to dense sand	7 to 10
Peat; soft to firm clay	8 to 10
Stiff to hard clay	10 to 20
Loess	12 to 15

$$E_{pl} = mE \qquad \ldots(1.16.9)$$

where *m* lies between 1.5 to 3 for most of soils. Suggested values of *m* are as follows,

m = 3, for sands and gravels
= 2, for silts
= 1.5, for clays
= 1.0, for peat and newly compacted soils

E_{pl} can also be used in estimating settlement as discussed in chapter 2.

Thus, pressuremeter tests can be utilised in estimating allowable bearing pressure based on strength (c, ϕ) or/and on settlement (δ).

In estimating allowable bearing pressure, a fact that bearing pressure shall be less than creep pressure with sufficient margin of safety is also kept in mind. Generally the recommended value of factor of safety over creep pressure is 1.5.

As already noted, pressuremeter described earlier referred to Menard Pressuremeter manufactured in France. However in India, Central Building Research Institute undertook study and developed a pressuremeter. This pressuremeter is manufactured under patent by Associated Instrument Manufacturers (India) Pvt. Ltd. and sold under the name *Subsoil Deformeter*. The main difference between this subsoil deformeter and Menard pressuremeter is that in deformeter measuring cell as well as guard cells are expanded by water only. However dimensions and specifications of the probe are the same, test procedures are the same and all theoretical bases, interpretation of tests and applications of results developed from pressuremeter tests apply in toto for the subsoil deformeter. The main limitation of the present deformeter is that it can be used for pressure upto 25 kg/cm² whereas Menard pressuremeter can be used for pressure upto 80 kg/cm² or higher. It is believed that further improvements will be made in deformeter shortly.

In general various pressuremeters in use today can be grouped into three categories viz. Menard type pressuremeter (MPM), self-boring pressuremeter (SBP) and push-in pressuremeter (PIP).

An MPM device is any type of pressuremeter where equipment is lowered into a pre-formed hole for testing. Two broad types of MPM device can be distinguished, differing in the means of applying the pressure to the membrane and in the method of measuring the response (strain) of the borehole. Menard type pressuremeter described earlier wherein water and gas are used for pressure application and an indirect measuring system is employed for strain measurements. The changes in radius of the borehole during expansion of the membrane is obtained by measuring the changes in the volume of the water-filled central cell. In the second type (type 2), the membrane is expanded under water, oil or gas pressure (i.e. only one type of fluid is used) and displacement of the borehole wall is more directly measured by feeler arms or displacement transduces inside the membrane. Naturally type 2 device, which measures displacement by electrical means, gives more accurate results than type 1 which measures volume changes with a liquid. Elastmeter pressuremeter manufactured by Oyo corporation of Japan is type 2 equipment and is available with Author's firm (refer Table 1.16.10).

In MPM pressuremeter, disturbances resulting from pre-boring of borehole are unavoidable. To overcome this limitation, self boring pressuremeter has been developed both in U.K. and France (Mair et al. 1987). Self-boring pressuremeter are all similar in principle in that they are, in essence miniature tunnelling machine which are steadily jacked into the ground. The soil displaced by the instrument enters the cutting head where it is broken into small pieces by a rotating cutter, and then flushed to the surface. The device developed in the U.K. is known as cambridge pressuremeter (camkometer), and the French instrument is known as the Pressiometre Autoforeur (PAF) or, more recently, Pressiometre Autoforeur pour Sol Raide ((PAFSOR).

With the increasing interest in offshore design and consequent need to obtain good measurements of the

TABLE 1.16.10. Principal Characteristics of Some Pressuremeters in Current Use

Device	Installation method (Test category)	Measurement systems: Pressure	Measurement systems: Deformation	Capacity: Maximum pressure (MN/m²)	Capacity: Maximum cavity strain (%)	Test control	Diameter (mm)	Length/diameter ratio
Menard[2] Pressuremeter (Type GB)	Installed in pre-formed test pocket in borehole (MPM)	Water Surface pressure gauge	Indirect volume measurement (Type 1)	10	27	Stress control	74[1]	6.5 (including guard cells)
Oyo (Elast-meter 200)	Installed in pre-formed test pocket in borehole (MPM)	Water, oil or gas: Surface pressure gauge	LVDT averaging displacement of two points (Type 2)	20	12	Stress control	70[1]	7.4
Deformeter	As above		(Type 2)	2.5		Stress control		
Self-boring pressuremeter (Camkometer)	Self-boring (SBP)	Gas: Diaphragm transducers in probe	Three independent strain gauged feeler arms (Type 2)	4	20	Strain or stress control	82	6.3
Cambridge Insitu high pressure dilatometer	Installed in pre-formed test pocket in borehole (MPM)	Oil: Diaphragm transducer in probe	Six independent strain gauged feeler arms (Type 2)	20	25	Stress control	74[1]	6.1
Push-in pressuremeter (developed at Building Research Establishment)	Pushed into base of borehole (PIP)	Oil: Diaphragm transducer in probe	Indirect volume measurement (Type 1)	3.5	10	Strain control	78	4.2

Notes:
(*1*) Diameters given here are for use with NX size holes (nominally 76 mm).
(*2*) 32-mm, 44-mm and 58-mm diameter probes are also available.

in-situ properties of stiff clays offshore, a pressuremeter was required which was robust and could be simply used with commercially available wire line drilling equipment. This resulted in the development in the U.K. of the push-in pressuremeter (PIP). This was originally developed by Building Research Institute of U.K. This is normally inserted by pushing either into an undersized pre-cored hole or into bottom of a borehole without any pre-coring. Most of the experience with PIP has been in offshore and even this has been relatively limited in comparision with MPM and SBP testing onshore.

Principal characteristics of some of pressuremeters in current use are given in Table 1.16.10.

Advantages of Pressuremeter Tests

(1) Pressuremeter tests eliminate the need for collection of undisturbed soil samples and testing them in a laboratory.

(2) In weathered rock wherein no undisturbed samples can be obtained or wherein no penetration test can be used to estimate allowable bearing pressure, pressuremeter test can be conveniently used.

Disadvantage of Pressuremeter Test

At present only limited number of agencies have this apparatus in India and its use is very much limited. Unless general geotechnical contractors have this apparatus, its use will be very much limited.

Pressuremeter testing is to be carried out by experienced persons and to be analysed by expert. Further the calibration of the instrument need to be done carefully particularly for type 1 of MPM equipment. Pressuremeter testing is almost worthless if proper calibrations of the instrument are not carried out.

1.17. VERTICAL PLATE BEARING TEST

Vertical plate bearing test was considered and is still considered as a model test for prototype foundations. However there are severe limitations to this test and this test alone cannot be a reliable indicator of allowable bearing capacity of soils. The major limitations are:

(*i*) Plate bearing test is of short duration, and hence consolidation settlement does not fully occur during this test. As such this test cannot be used to predict consolidation settlement. Hence from settlement consideration its use is restricted to sandy soils, and to partially saturated or rather highly unsaturated clayey soils.

(*ii*) If the soil is not homogeneous within the effective depth of prototype foundation, plate bearing test can give very misleading information. In Fig. 1.17.1, if the foundation is subjected to allowable bearing pressure *qa* as estimated from plate load test then foundation may undergo excessive settlement or failure because of the presence of very soft clay within its effective depth. This soft clay layer was not affected during the plate bearing test. Hence plate bearing test is not to be recommended or adopted in soils which are not homogeneous at least to a depth of 1.5 to 2 times the width of the prototype foundation.

There are inaccuracies introduced in extra-polating the results from small size plates to large size foundations. Methods of extrapolation are discussed in Chapter 2.

In addition to above limitations, a number of other minor limitations can be cited for plate bearing tests such as differences in environmental conditions for plate bearing tests and actual foundation loading *e.g.* rate and type of loading, initial stress conditions in soil etc.

The best method of load test for determining the bearing capacity of a foundation is to test a full sized foundation under design load long enough to observe all the settlements. This is rarely possible and hence often load tests on smaller size plates are carried out. However, it is stressed here that plate bearing test

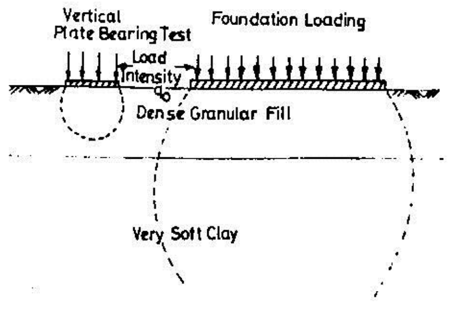


Fig. 1.17.1.

alone is not to be used in predicting the allowable bearing pressure for foundations.

The following guidelines are given in conducting plate bearing test.

(*a*) Pit for test shall be at least five times the size of the plate. The bottom of pit, if possible, shall coincide with the level of foundation.

(*b*) It is preferable to use 60 cm or higher size plate (square or circular).

(*c*) If the size of the base plate (bottom-most plate) is other than 30 cm (square or circular) use other plates, from the series, on the top of the base plate till top most plate is of 30 cm (square or circular). Note that all the plates of the series have to be square or have to be circular and of dimensions 30 cm, 45 cm, 60 cm, and 75 cm (square or circular).

(*d*) Do not forget to add the weight of plates, jack, and of other items, the weights of which may not be recorded by load measuring devices, to get the total load on the base plate.

(*e*) Total number of load increments shall not be less than 10. All increments of loading shall be equal and shall not exceed 10 t/m^2, unless specified. Table 1.17.1 may be used only as an aid in determining very approximately the final total Q_{ult} that can be taken by 60 cm square plate without shear failure in the subsoil, assuming the *N* value of SPT of the subsoil is known. This value helps in the selecting appropriate value for the load increment. If other size plates are used, change Q_{ult} proportional to B^2 for cohesive soils and proportional to B^3 for cohesionless soils. *B* refers to width of the test plate. In strong soils if Q_{ult} need not be reached then final load can be restricted to three times the design load of foundations.

(*f*) Do not run the test as confined but run it as unconfined. That is to say do not provide any embedment to the test plate.

(*g*) Do not try to saturate the soil, particularly cohesionless soils, in the test zone by adding external water. Such a practice is followed by some agencies hoping to simulate *submerged condition* in test zone. Submerged condition is difficult to produce and almost impossible in pervious soil. Attempt to saturate or submerge subsoil can have adverse effects. Hence it is strongly urged not to attempt to produce submergence.

TABLE 1.17.1. Approximate Ultimate Load for a Plate Load Test

N Value of SPT (corrected as per Eqs. 1.10.1 and 1.10.2 in cohesionless soils)	*Approximate value of Q_{ult}, tonnes*	
	Pure cohesive soils	*Pure cohesionless soils**
5	4	2
10	10	4
15	16	8
20	22	14
30	33	20
40	44	30
50	55	45

* **Assumption:** Ground water is not within the effective depth of testing. If ground water is at the level of the test plate, reduce the values in this column by 1/2.

(*h*) It is preferable, particularly in sandy soils, to lower the water table in the test zone to a depth at least 3*B* below the base of the plate when ground water is met within the effective depth of test.

(*i*) In mixed soils, it is preferable to do tests on two size plates. These tests shall be on similar soil stratum, at the same depth, shall be located as near to each other as possible but not as near as to overlap the zone of soil shear of one plate with that of the other. Hence it is recommended to keep the distance between two plates as 5 times the width of the larger plate.

1.18. ROCK CORING

Rock coring may be obtained by use of percussion or rotary drilling method. Current practice is to use rotary methods almost exclusively because of higher quality of samplers obtained. It is recommended that on geotechnical investigations only rotary drilling be permitted. Rock coring may be carried out by using diamond bits or tungsten carbide (T.C.) bits. T.C. bits are used in India by some of the investigating agencies mainly because these bits are far cheaper than diamond bits. For better core recovery in hard rock like basalt and granite, the use of diamond bits becomes essential.

The sizes of bit used in coring vary generally from *AX* to *NX* size though currently bigger sizes are also adopted. In India, *NX* size cores are commonly obtained. The details of standard drill tools are given in Table 1.18.1.

The bits are attached to core barrels through reamer shells. The core barrels currently in use are of three basic types, *viz*, the single tube, the double tube and the triple tube core barrels. The use of double tube core barrel in rotary rock drilling practice is well known. The inner tube of the double tube prevents washing away of core due to prolonged flow of water. Thus core recovery with double tube core barrel is generally better than that obtained with single tube core barrel particularly in jointed and weathered rock. But with the triple tube core barrel far better fore recovery may be obtained than with double tube core barrel particularly in highly weathered and highly friable rock. The triple tube core barrel assembly consists of the two tubes similar to the double tube core barrels. However another tube is provided inside the second one. This tube is a split tube, in case it is metallic. If it is platic then it is generally is single piece.

After cores are obtained in core barrel and barrel is withdrawn, the third inner tube is pushed out by a plunger operated under the water or air pressure. If the inner tube is plastic, entire tubing along with cores will be placed in core boxes. Such a plastic tube permits for good visual examination of cores in their relative position and this is one of the main advantages of triple tube core barrel.

At present double tube core barrels are widely used in rock drilling. Even in double tube core barrels, there are different types *viz X, G, W, T* and *M* series. *X, G, W* and *T* are similar type except that *T* series has thinner tube. *T* series core barrels are found more suitable in drilling hard and compact formations compared to *X, G* and *W* series.

M series core barrels differ in one important respect from other types *viz X, G, W & T*. In *M* series the inner tube extends into the core bit and core lifter is mounted inside the inner tube to protect cores of soft or friable formation. Thus in *M* series core lifter remains stationary as the inner tube even when bit rotates. In other types *viz X, W & G*, the core lifter rotates with the bit, subjecting

TABLE 1.18.1. Standard Sizes of Drill Tools[1]

Drill rod			*Casing Pipe*			*Core Barrel*			*Core Diameter in*		*Hole Diameter in*		*Designation*
Designation	*O.D. mm*	*O.D. inches*	*Designation*	*O.D. mm*	*O.D. inches*	*Designation*	*O.D. mm*	*O.D. inches*	*mm*	*inches*	*mm*	*inches*	
A	41.27	$1\frac{5}{8}$	AX	31.750	$2\frac{1}{4}$	AWX	47.625	1.875	28.575	$1\frac{1}{8}$	47.625	$1\frac{7}{8}$	AX
						AWM	47.625	1.875	28.575	$1\frac{1}{8}$	47.625	$1\frac{7}{8}$	AX
B	47.625	$1\frac{7}{8}$	BX	73.025	$2\frac{7}{8}$	BWX	59.563	2.345	41.275	$1\frac{5}{8}$	60.325	$2\frac{3}{8}$	BX
						BWM	59.563	2.345	41.275	$1\frac{5}{8}$	60.325	$2\frac{3}{8}$	BX
N	60.325	$2\frac{3}{8}$	NX	88.900	$3\frac{1}{2}$	NWX	75.311	2.965	53.975	$2\frac{1}{8}$	76.200	3	NX
						NWM	75.311	2.965	53.975	$2\frac{1}{8}$	76.200	3	NX
											99.000	4	HX

1. O.D. = Outside diameter. Standard dimensions are in inches. Equivalent in mm, to three decimal places, are given by conversion.

rock cores to some friction as they enter the core barrel.

The type of bit should suit type of core barrel *viz G* type bit is to be used with *G* type core barrel, *M* type bit with *M* type core barrel etc.

Bits in general may be tungsten carbide (T.C.) bit or diamond bit. T.C. bits are used in soft formations which correspond to hardness groups 1 and 2 on Mohr's hardness scale. These bits are also suitable for drilling through overburden and cleaning of drill holes. T.C. bits generally have octagonal, tungsten carbide inserts as cutting elements (Fig. 1.18.1) though obliquely positioned tungsten plates may be used in certain cases (Fig. 1.18.2). In practical terms, T.C. bits may be changed to diamond bits when speed of drilling progress becomes very slow and is of the order of 15 cm/hour or less. In any case it is to be noted that speed of drilling machine should be low, of the order of 0.5 m/sec when T.C. bits are used.

Diamond bits were first introduced in South Africa in early thirties. Initially hand set natural diamond bits were used but now synthetic diamond bits are extensively used particularly in USA, Canada, U.K.

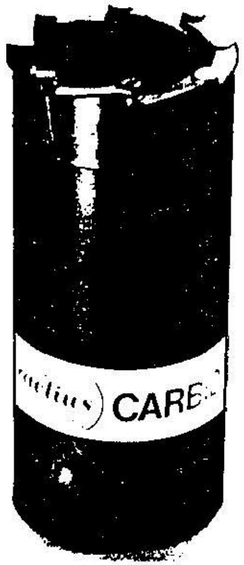


Fig. 1.18.1. Tungsten carbide drill bit with octagonal tungsten carbide inserts.

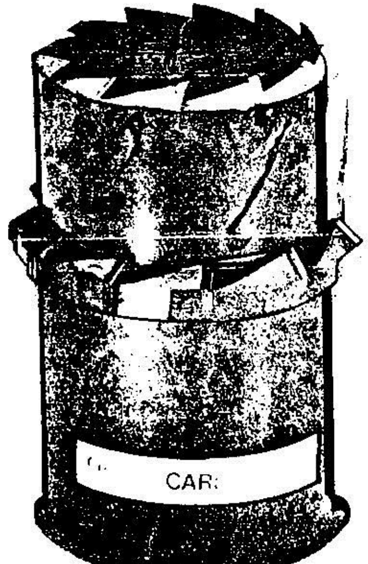


Fig. 1.18.2. Drill bit SK6L with obliquely positioned plates.

etc. In India still natural diamonds are used widely though synthetic bits are being introduced gradually.

Natural diamond bits may be "surface set" or "impregnated". Surface set diamond bits are used in soft to hard rock formations. Impregnated diamond bits are used in hard and also in fractured rock formations. In general impregnated bits are preferred in difficult formations.

Synthetic diamond bits are produced under severe controlled conditions. Hence their life in terms of metreage may be 3 to 4 times natural diamond bits. Hole deflection with synthetic diamond bits is less than natural diamond bits.

The drill bits are connected to core barrels through a reaming shell. The purpose of a reaming shell is to maintain the correct drill hole diameter, thoughout the total length of the hole. This enables a new replacement drill bit to be lowered to the bottom of the hole without incurring damage. A secondary function of the reaming shell is to stabilize the drill string, thereby protecting the lower end of the core barrel against excessive wear. It is always recommended that

a reaming shell be used when drilling in hard formations and/or when drilling deep into the rock. Reaming shell may be eliminated when drilling in soft formation and when length of drilling in rock is limited.

The reaming shell should always have a somewhat larger diameter than the drill bit. As soon as the outer diameter wears to that of the drill bit, the reaming shell should be replaced. Should the outer diameter of the reaming shell wears below that of bit, there is a risk that the hole becomes conical and can prevent a new drill bit from reaching the hole bottom. This can result in expensive reaming operation of the drill hole.

In general, core barrels are operated at speeds from 50 to 1750 r.p.m. Essentially, the harder the rock the faster the permissible speed. The ultimate factor determining the speed is the amount of vibration encountered as the speed is increased. The Acker Company of U.S.A. which manufactures drilling machines, recommends the following speed in good rocks : 800 to 1200 r.p.m. for *AX* size bits, up to 800 r.p.m. for *BX* size bits and up to 600 r.p.m. for *NX* size bits.

Downward pressure on the bit is determined on the basis of experience. Rod vibration or *chatter* generally indicates the need for a reduced bit pressure. Coring in soft rock also requires low bit pressure.

The pressure under which the drilling fluid, which invariably is water, should be introduced into the hole, should be minimum, consistent with adequate removal of cuttings from the hole and proper cooling of the bit.

Generally on drilling by 1.5 metres, core barrel is lifted. There are two devices commonly used to retain the core as the core barrel is lifted. These are split-ring core lifter and basket retainer. However split-ring core lifters are more widely used. On removing the core from the barrel, all pieces of core are put in a partitioned wooden box specially built for the purpose. In normal practice, to remove core barrel all drill rods need to be lifted. This could be a painful and time consuming operation particularly for deep drilling. This can be eliminated by using wire line technique.

In wire line drill technique, wire line drill rods are used and constitute the connection element between the core drill machine (1 of Fig. 1.18.3) and the wire line core barrel (4). The wire line drill rods transmit

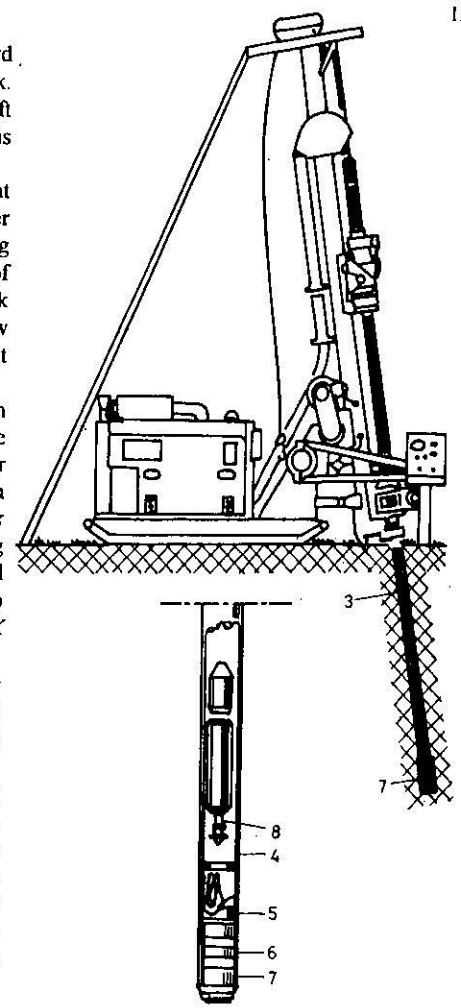


Fig. 1.18.3. Principle sketch of core drilling with a wire line core barrel.

torque and feeding force to the drill bit, which performs the cutting action.

The inner diameter of the wire line drill rod is sufficiently large to enable the inner core barrel (5) to pass freely through the drill string. For withdrawing the drilled rock core, the inner core barrel is withdrawn. When withdrawing the inner core barrel, the outer core barrel, bit and the drill string remain in the hole. The drill string need to be withdrawn when drill bits or outer core barrel etc need to be replaced.

The ratio of total length of rock pieces collected to length drilled, expressed as percentage, is known as core recovery. To obtain RQD, *Rock Quality Designation*, only those pieces of rock which are 101.6 mm (4 inches) and longer are measured for their total length. The above length divided by length drilled, expressed as percentage, is known as RQD. RQD value is always equal to or less than the value of core recovery.

Thus, core recovery in %

$$= \frac{\text{Length of core}}{\text{Length of run}} \times 100$$

$$\text{and RQD in \%} = \frac{\text{Length of core in pieces of 101.6 mm (4 inches and above}}{\text{Length of run}} \times 100$$

If the core is broken by handling or during drilling (*i.e.* the fracture surfaces are fresh, irregular breaks rather than natural joint surfaces), the fresh broken pieces are fitted together and counted as one piece. Some judgement is necessary in case of thinly bedded sedimentary rocks and foliated metamorphic rocks and the method is not so exact in these cases as in igneous rocks, thick bedded limestones, sand stones, etc. However, the system has been applied successfully even for shales, although it is necessary to log the cores immediately upon removing them from the core barrel before air slacking and cracking can begin.

The core recovery is an indication of soundness and degree of weathering of rock.

The following terms are used to describe the soundness of rock. Appropriate term may be selected to describe the rock based on following definitions.

(*i*) **Sound rock.** Rock which rings when struck with a pick or bar, does not disintegrate after exposure to air or water, breaks with a sharp, fresh fracture; in which cracks are unweathered and less than 3 mm wide and generally no closer than 1 m apart; core recovery with double tube core barrel with diamond bit is generally 85% or greater for each 1.5 m run.

(*ii*) **Medium rock.** Characteristics as for sound rock except that cracks may be 6 mm wide and slightly weathered, generally no closer than 60 cm; core recovery generally is 50% or more.

(*iii*) **Intermediate (between medium and soft) rock.** Rock which gives dull sound when struck with a pick or bar; does not disintegrate after exposure to air or water, in which broken pieces may show weathered faces; may contain fractured or weathered zones up to 25 mm wide, spaced no closer than 30 cm, and where core recovery generally is 35% or greater.

(*iv*) **Soft (or disintegrated) rock.** Any rock which slakes on exposure to air or water, or a rock which gives a dull sound when struck with a pick or bar and where core recovery generally is less than 35% but standard pentration resistance blow count is more than 50.

The above definitions are as per the Building Code Committee, Soil Mechanics and Foundation Division, New York City, Metropolitan Section ASCE. The Task Committee on Shallow Foundations of Soil Mechanics and Foundation Division, ASCE, recommends that the rock may be described based on weathering, hardness, joint bedding, foliation spacing and RQD. Tables 1.18.2, 1.18.3, 1.18.4 and 1.18.5 give respectively recommended descriptions of weathering, hardness, bedding and foliation, and RQD as suggested by the above mentioned Task Committee. However weathering classification as given in Table 1.18.6 is recommended by Indian Bureau of Standards (IS 4464–1985) and in British Standards (B.S 5930) and is widely used.

The Moh scale of hardness referred in footnote to Table 1.18.3 is a scale of hardness varying from 1 to 10, with 1 being the softest (mineral talcum) and 10 the hardest (mineral diamond). However it is a non lineal scale, i.e. hardness 10 does not mean that it is ten times harder than hardness of 1. A more appropriate scale for hardness is Knoop-scale. The Knoop scale is an absolute scale and is based on the compressive strength of the mineral and gives therefore a clearer comparison of hardness between the different minerals.

Diamond has a hardness of 8300 Knoop while boron nitride, the second known hardest material has a hardness of 4500 units. Quartz (which is often present in hard rock formations) has a hardness of 750 units and talcum 12 units. A comparision between the MOH scale and Knoop scale is given in Fig. 1.18.4.

TABLE 1.18.2. Descriptive Terms for Rock Weathering

Term	*Description*
Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer, if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show clay if open, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained and discolouration extends into rock upto 2.5 cm. Open joints contain clay. In granitoid rocks some occasionally feldsper crystals are dull and discoloured. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discolouration and weathering effects. In granitoid rocks most feldspars are dull and discoloured; some show clayey. Rock has dull sound under hammer and show significant loss of strength as compared with fresh rock.
Moderately severe	All rocks execpt quartz discoloured or stained. In granitoid rocks all feldspar show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes *clunk* when struck (Saprolite).
Severe	All rocks except quartz discoloured or stained. Rock *fabric* clear and evident but reduced in strength to strong soil. In granitoid rocks all feldspars kaolinised to some extent. Some fragments of strong rock usually left (Saprolite).
Very severe	All rocks except quartz discoloured or stained. Rock fabric discernible but mass effectively reduced to *soil* with only fragments of strong rock remaining.
Complete	Rock reduced to *soil*. Rock *fabric* not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

TABLE 1.18.3. Hardness of Rock*

Term	*Description*
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hard specimens requires several hard blows of geologists' pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves 6 mm deep can be excavated by hand blow of point of geologists' pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1.5 mm deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 2.5 mm maximum sizes by hard blows of the points of a geologists' pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several cms in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 25 mm or more in thickness can be broken by finger pressure. Can be scratched readily by finger nail.

* For engineering description of rock. Not to be confused with Moh's scale for minerals.

TABLE 1.18.4. Joint Bedding and Foliation Spacing in Rock*

Spacing	*Joints*	*Bedding and Foliation*
Less than 50 mm	Very close	Very thin
50 mm to 300 mm	Close	Thin
300 mm to 910 mm	Moderately close	Medium
910 mm to 1050 mm	Wide	Thick
More than 1050 mm	Very wide	Very thick

* Joint spacing refers to the distance normal to the plane of joints of a single system or *set* of joints which are parallel to each other or nearly so. The spacing of each *set* should be described, if possible to establish.

TABLE 1.18.5. Rock Quality Designation (RQD)*

RQD	*Diagnostic Description*
Exceeding 90%	Excellent
90–75%	Good
75–50%	Fair
50–25%	Poor
Less than 25%	Very poor

* RQD should always be given in percentile. Diagnostic description is intended for evaluating problems with tunnels or excavations in rocks.

The Associate Committee on the National Building Code of the National Research Council of Canada classifies rock with respect to strength as per Table 1.18.7.

As per the British Standards, BS 5930–1981 rock is classified, based on its strength, as per Table 1.18.8. IS 4464–1985 also adopts the same strength classification table. These characteristics serve as a framework for classifying rocks for engineering purposes.

TABLE 1.18.6. Weathering Classification of Rock

Item	*Description*	*Grade*
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	I
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally that in its fresh condition.	II
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as core stones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as core stones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

TABLE 1.18.7. Classification of Rock with Respect to Strength*

Rock Classification	*Description*
Rock of very high strength	Rock much stronger than concrete with a compressive strength greater than 2000 kg/cm^2.
Rock of high strength	Rock stronger than concrete with acompressive strength between 550 kg/cm^2 to 2200 kg/cm^2.
Rock of medium strength	Rock comparable to concrete with a compressive strength between 140 kg/cm^2 to 550 kg/cm^2.
Rock of low strength	Rock comparable to brick masonry with a compressive strength between 35 kg/ cm^2 to 140 kg/cm^3.
Rock of very low strength	Rock weaker than brick masonry with a compressive strength between 9 kg/ cm^2 to 35 kg/cm^2.

* *Note*: Rock with compressive strength less than 9 kg/cm^2 shoul be treated as *soil*. Similarly clay material that can be dug by hand with a shovel or pneumatic spade and also cemented soils and gravels in which cementing is discontinuous should be treated as soils though geologically these materials may correctly be referred to as rocks.

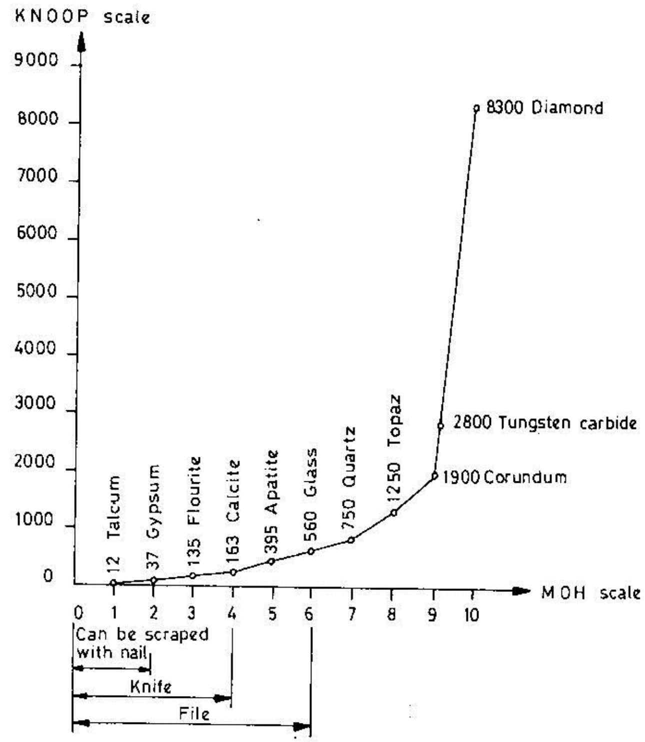


Fig. 1.18.4. Hardness comparison between the MOH scale and the KNOOP scale for different rock formations.

Table 1.18.8. Strength of rock material. A scale of strength, based on uniaxial compressive testing, is as follows

Term	*Compressive strength, MN/m^2*
Very weak	less than 1.25
Weak	1.25 to 5
Moderately weak	5 to 12.5
Moderately strong	12.5 to 50
Strong	50 to 100
Very strong	100 to 200
Extremely strong	greater than 200

Note: The strength of a rock material determined in the uni-axial compression test is dependent on the moisture content of the specimen, anisotropy and thetest procedure adopted.

1.19. GROUNDWATER INVESTIGATION

Parameters of importance in groundwater investigation are:

(*a*) The existence of groundwater; normal, perched or artesian.

(*b*) The exact level of the groundwater table and lower limit of perched groundwater.

(*c*) The thickness of strata and the piezometric level of artesian groundwater.

(*d*) The variation of these characteristics over the site and with time.

(*e*) The chemical composition of groundwater.

It is common practice to establish water table elevation at a site by making observations in boreholes and existing wells, if any. Observations in wells shall

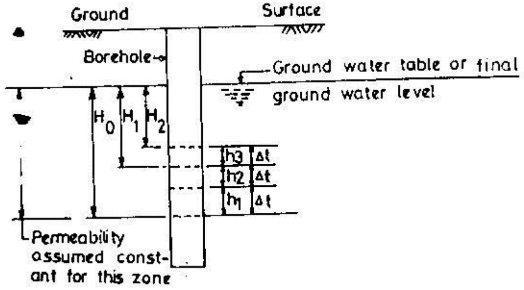


Fig. 1.19.1.

be preferred as they are more reliable. The level of water in a well can be taken as elevation of groundwater table.

In pervious soils and when bentonite or mud slurry process is not adopted during boring operation, elevation of groundwater table can be taken as the water level in the borehole 24 hours (or next day) of drilling operation. But in impervious soils or in pervious soils where bentonite or mud slurry process is adopted during boring, sufficient time some times as much as one week, shall be permitted for water level to obtain steady state. But if permeability of subsoil can be assumed fairly constant for the zone of interest, as shown in Fig. 1.19.1. the position of groundwater table can be predicted using Hvorslev method. In this method, readings for water level are taken at two or more equal intervals, Δt, of time. The water table is then established by using the following Eq. 1.19.1.

$$H_0 = \frac{h_1^2}{h_1 - h_2} \text{ or } H_1 = \frac{h_2^2}{h_1 - h_2} \text{ or } H_2 = \frac{h_3^2}{h_2 - h_3} \quad ...(1.19.1)$$

Wherein H_0, H_1, H_2, h_1, h_2, h_3, are as indicated in Fig. 1.19.1.

The depth to groundwater is usually measured by means of a chalked tape, a tape with a float, or an electric water level indicator. In the first method a short length of the lower end of a metal tape is chalked. The tape with a weight attached to its end is then lowered until the chalked section has passed slightly below the water surface. The depth to the water is then determined by subtracting the depth of penetration of the line into water, as measured by the water line in the chalked section, from the total depth from the ground surface. In the second method a tape with a float attached to its end is lowered until the float hits the water surface and it is just tout; the depth is then measured. Some floats are equipped with a whistle which sounds when the float hits the water surface. The electrical depth indicator consists of a weighted probe attached to the lower end of a length of electrical cable which is marked at intervals to indicate depth. When the probe reaches the water, a circuit is completed giving a whistle or a sudden deflection on a meter mounted on the cable reel, depending on the type of indicator used.

In case where direct observation for groundwater is not applicable and/or in cases where groundwater conditions are important for design, groundwater conditions should be investigated by the installation and observation of piezometers. In designing such installations, attention should be paid to stratigraphy (for location of piezometer tips) and the soil type (for selection of the type of piezometer).

Whenever groundwater is encountered at site, water samples, collected from bores/wells which are well spaced over the area in consideration, are collected for chemical analysis of subroil water. The following precautions need be taken while collecting groundwater samples.

(*i*) Avoid collecting water sample from a borehole in which bentonite/mud slurry has been used.

(*ii*) If possible water sample should be collected even before the addition of water to the borehole.

(*iii*) While collecting the water sample, it is essential to make sure that subsoil water is not contaminated by rain or surface water.

(*iv*) It is good practice to lower the water in the borehole by about half a metre by pumping and then allow it to rise to its original level and then collect the water sample.

(*v*) Water sample shall be stored in approved air tight and clean container. Container should be preferably of glass or of plastic.

(*vi*) Water samples shall be given for necessary laboratory testing at the earliest. Usually water samples are tested for *pH* value and contents of sulphates and chlorides.

1.20. PRESERVATION, SHIPMENT AND STORAGE OF SAMPLES

Soil samples, both disturbed and undisturbed, must be carefully sealed immediately after their collection.

In India undisturbed samples are generally preserved in sampling tubes in which they are retrieved. On extracting the sampling tube from borehole/trial pit etc., sample is trimmed on either side of the tube by about 2 to 3 cms and the ends of the sample tube are then filled with molten bees-wax added in increments. Use of paraffin alone, instead of wax, should be avoided because it becomes brittle and easily cracks, thereby breaking the seal. It is best to apply the wax at a temperature just above the congealing point. On completion of waxing, tubes are capped on either side by tightfitting plastic or galvanized steel caps and then bound on with adhesive tape.

Disturbed samples such as those obtained from drive samplers and those extracted from the top and bottom of the undisturbed samples should be placed in air-tight glass jars or in twin polythene bags.

All rock cores collected shall be placed in order of their occurrence in strong wooden boxes, suitably partitioned and provided with hinged wooden covers.

All samples shall be clearly labelled indicating job number, borehole/pit number, sample number, date of sampling, brief description of sample, type of sample, depth/elevation of sample etc. and in case of undisturbed samples, the top and bottom of samples shall also be clearly labelled. Each such label shall be pasted on the container and shall also be included in the container.

Samples stored temporarily at site prior to shipment to the testing laboratory shall be protected from direct sun/rain etc. Further samplers containing undisturbed samples should be covered with wet gunny bags to avoid loss of moisture.

Sample from site should be transported to the testing laboratory at the earliest. Prior to transporting, the samplers containing undisturbed samples shall be placed in wooden boxes. To minimise disturbance during shipment, saw dust or similar other resilient material shall be used during packing.

Once the samples are received at the laboratory, the relevant tests should be carried out at the earliest. However until testing, samplers containing undisturbed samples shall be preserved in a room maintained at high humidity such as around 90 to 95%. This prevents loss of moisture from the samples. In absence of such a room, these tubes shall be covered with wet gunny bags.

However, it is to be realised that if samples are stored in metallic tube (which is common practice for undisturbed samples collected from boreholes) for long time, rusting would take place. Generally because of this rusting increase in strength and decrease in compressibility would take place. Hence, it is advisable to test the samples as early as possible.

1.21 DISTURBANCES IN SOIL SAMPLES AND THEIR EFFECT ON PROPERTIES

The disturbances to which soil samples may be subjected can be grouped into following basic types viz.

(a) disturbances due to changes in stress conditions,
(b) disturbances due to changes in water content and void ratio,
(c) disturbances during sampling, preservation, shipment, and storage of samples and
(d) disturbances during testing of samples.

When sample is recovered from a depth, changes in stresses do occur. Generally there is decrease in surrounding stresses, resulting in decrease in strength. Hence, attempt should be made to re-produce during the testing, the stresses prevalent at original ground conditions.

Changes in water content and void ratio may take place during sampling or during testing in laboratory. Changes in void ratio can occur due to changes in surrounding stresses, disturbances during sampling, transportation, testing, etc. Increase in void ratio decreases the shear strength for given moisture content and vice versa.

Changes in moisture content can occur during storage in addition to those that occur during sampling process. Increase in water content decreases the shear strength and vice versa.

Disturbances that occur during sampling and also preservation, shipment and storage of samples are discussed in paras 1.6 and 1.20. Also guidelines to minimize these disturbances are given in these paras.

Significant disturbances can occur during testing. Sample should be extracted in the same direction as it enters the sampling tube to minimize effect of disturbances. Undisturbed samples should be cut by piano wire only and not by spatula etc.

1.22. PHASE RELATIONSHIPS AND RELATED FORMULAE

Various commonly used phase relationships are presented herein below. In Fig. 1.22.1, the volumes and weights of three phases of soil are so represented as to lend themselves for easy mathematical derivations of various relationships.

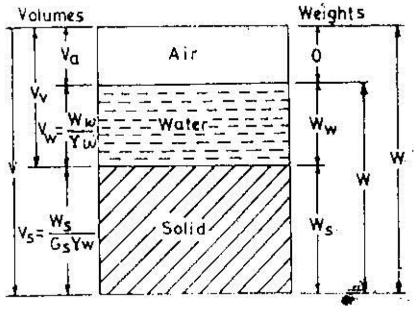


Fig. 1.22.1. Simplified sketch of "soil" for phase relationships

(i) Moisture content (w)

$$W,\% = \frac{\text{Weight of Water}}{\text{Weight of Solids}} \times 100$$

$$= \frac{W_w}{W_s} \times 100$$

(ii) Void ratio (e)

$$e = \frac{\text{Volume of voids}}{\text{Volume of solids}} = \frac{V_v}{V_s} = \frac{n}{100-n}$$

$$= \frac{G_s \cdot \gamma_w}{\gamma_d} - 1$$

$$= G_s \frac{w}{100}, \text{ for saturated soils.}$$

(iii) Porosity (n)

$$n,\% = \frac{\text{Volume of voids}}{\text{Total volume}} \times 100$$

$$= \frac{V_v}{V} \times 100$$

$$= \frac{e}{1+e} \times 100 = \left(1 - \frac{\gamma_d}{G_s \gamma_w}\right) \times 100$$

(iv) Wet (bulk) density of soil (γ)

$$\gamma = \frac{\text{Total weight of soil}}{\text{Total volume of soil}} = \frac{W}{V}$$

$$= \frac{G_s \gamma_w \left(1 + \frac{w}{100}\right)}{1+e}$$

$$= \frac{e\frac{S_r}{100} + G_r}{1+e} \gamma_w$$

$$= \frac{\gamma_w (G_w + e)}{1+e} = \gamma_{sat}, \text{ for saturated soil}$$

(v) Dry density of soil (γ_d)

$$\gamma_d = \frac{\text{Dry wt. of soil (i.e., wt. of solids)}}{\text{Total volume of soil}}$$

$$= \frac{W_s}{V}$$

$$= \frac{\gamma}{1 + \frac{w}{100}}$$

$$= \frac{G_s \gamma_w}{1+e} = \left(1 - \frac{n}{100}\right)(G_s \gamma_w)$$

(vi) Submerged (bouyant) density of soil (γ_b)

$$\gamma_b = \gamma_{sat} - \gamma_w$$

$$= \frac{G_s - 1}{1+e} \gamma_w = \left(1 - \frac{n}{100}\right)(G_s - 1)\gamma$$

(vii) Degree of saturation (S_r)

$$S_r,\% = \frac{\text{Volume of water}}{\text{Volume of voids}} \times 100$$

$$= \frac{V_w}{V_v} \times 100$$

$$= \frac{G_s \frac{w}{100}}{e} \times 100$$

(viii) Air voids (n_a)

$$n_a,\% = \frac{\text{Volume of air}}{\text{Total volume}} \times 100$$

$$= \frac{V_a}{V} \times 100$$

$$= \left[1 - \frac{\gamma_d}{\gamma_w}\left(\frac{1}{G_s} + \frac{w}{100}\right)\right] 100$$

(ix) Zero air void ratio density (γ_{zav}).

γ_{zav} = Dry density for zero air voids

$$= \frac{\gamma_w}{\frac{w}{100} + \frac{1}{G_s}}$$

(x) Plasticity Index (I_p)

I_p = Liquid limit − plastic limit

$= w_L - w_P$

(xi) Liquidity Index (I_L)

$$I_L,\% = \frac{w_L - w_n}{I_p} \times 100$$

where w_n = natural moisture content, %

(xii) Relative consistency or consistency index (I_C)

$$I_c,\% = \frac{W_L - W_n}{I_p} \times 100$$

(xiii) Activity (A)

$$A = \frac{I_P}{\text{\% Clay fraction}}$$

(xiv) Effective Size (D_{10})

D_{10} = particle size with 10% finer

(xv) Uniformity coefficient (U_C)

$$U_c = \frac{\text{Particle size with 60\% finer}}{\text{Particle size with 10\% finer}}$$

$$= \frac{D_{60}}{D_{10}}$$

(xvi) Coefficient of curvature (Cc)

$$C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$$

where D_{30} = Particle size with 30% finer.

1.23 ENGINEERING CLASSIFICATION OF SOILS

The purpose of soil classification is to present a systematic and satisfactory method of grouping soils so that soils of the same group have similar desired characteristics. The desired characteristics vary depending on the purpose for which the soil is used. Hence a number of classification methods are available. The classification adopted by geologists is different than that adopted by engineers. Moreover, even amongst engineers different classification systems are prevailing. The *AASHO* classification is widely used by different highway departments. The unified soil classification is popular with U.S. Army Corps of Engineers and also, with minor modifications, with U.S. Bureau of Reclamation. The above classification with some modifications has also been adopted by Indian Standards Institution.

The classification based on mineralogical composition or mode or origin of soils is known as geological classification. Classification based on *morphology* and *genesis* is known as pedological classification. The engineering classification is based on particle sizes with or without consistency limits.

Of the various classifications only a few, most commonly adopted ones, are discussed over here. In classifying the soil into different fractions *viz.*