Geo-Technical Engineering

(22404)

SECOND YEAR DIPLOMA

Maharashtra State Board of Technical Education (MSBTE)

Semester IV - Civil Engineering Groups (CE/CR/CS)

Strictly as per new revised syllabus of 'I' Scheme w.e.f. academic year 2018-2019

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Syllabus

Unit	Unit Outcomes (UOs) (in cognitive domain)	Topics and Sub-topics
Unit – I Overview of geology and geotechnical engineering	1a Describe the salient features of given type of rocks.	1.1 Introduction of geology, different branches of geology, importance of geology for civil engineering structure and composition of earth.
Refer chapter 1	1b Identify the given type of rocks.	1.2 Introduction of petrology, definition of a rock, classification based on their genesis (mode of origin), formation, classification and engineering uses of igneous, sedimentary and metamorphic rocks.
	1c Suggest the type of soil for the given situation.	1.3 IS definition of soil, Importance of soil in Civil Engineering as construction material in Civil Engineering Structures, as foundation bed for structures.
	1d Describe the applications of Geo-technical Engineering for the construction of the given civil structure.	1.4 Field application of geotechnical engineering for foundation design, pavement design, design of earth retaining structures, design of earthen dam.
Unit - II Physical and Index Properties of Soil Refer chapter 2	2a Use the relevant IS code for determining the given physical properties of Soil with justification.	2.1 Soil is a three phase system, water content, determination of water content by oven drying method as per IS code, void ratio, porosity and degree of saturation, density index, unit weight of soil mass - bulk unit weight, dry unit weight, unit weight of solids, saturated unit weight, submerged unit weight, determination of bulk unit weight and dry unit weight by core cutter method and sand replacement method as per IS code, specific gravity, determination of specific gravity by pycnometer.
	2b Calculate Atterberg's limits of Consistency for the given data.	2.2 Consistency of soil, stages of consistency, Atterberg's limits of consistency viz. Liquid limit, plastic limit and shrinkage limit, plasticity index, determination of liquid limit, plastic limit and shrinkage limit as per IS code.
	2c Interprete Atterberg's limits of Consistency for the given data.	2.3 Particles size distribution, mechanical sieve analysis as per IS codeparticle size distribution curve, effective diameter of soil, Uniformity coefficient and coefficient of curvature, well graded and uniformly graded soils, particle size. classification of soils, I.S. classification of soil.
	2d Classify the given soil sample as per IS provision	
	2e Interpret the particle size distribution curve for the given data.	

Unit	Unit Outcomes (UOs) (in cognitive domain)	Topics and Sub-topics
Unit - III Permeability and Shear Strength of Soil **Refer chapter 3**	3a Identify the factors affecting the permeability of given type of soil sample. 3b Compute the coefficient of permeability for a given soil sample data. 3c Compute the shear strength of soil sample for the given data. 3d Interpret the shear failure of soil sample for the given data. 3e Use the application of flow net in the given situation.	3.1 Definition of permeability, Darcy's law of permeability, coefficient of permeability, factors affecting permeability, determination of coefficient of permeability by constant head and falling head permeability tests, simple problem to determine coefficient of permeability Seepage through earthen structures, seepage velocity, seepage pressure, phreatic line, flow lines, application of flow net, (No numerical problems) 3.2 Shear failure of soil, field situation of shear failure, concept of shear strength of soil, components of shearing resistance of soil-cohesion, internal friction. Mohr-coulomb failure theory. Strength envelope, strength Equation for purely cohesive and cohesion less soils. Direct shear test and vane shear test laboratory methods
Unit - IV Bearing Capacity of Soil **Refer chapter 4**	4a Suggest the soil parameters to determine bearing capacity of given soil sample with justification 4b Suggest the method to determine bearing capacity of the soil for the given strata with justification. 4c Choose the relevant type of foundation using Rankine formula for the given situation. 4d Corelate the effect of water table on bearing capacity of soil for the given data.	4.1 Bearing capacity and theory of earth pressure : Concept of bearing capacity, ultimate bearing capacity, safe bearing capacity and allowable bearing pressure, Introduction to Terzaghi's analysis and assumptions made, effect of water table on bearing capacity. 4.2 Field methods for determination of bearing capacity – Plate load test and standard penetration test. Test procedures as Per IS:1888 & IS:2131 4.3 Definition of earth pressure, active earth pressure and passive earth pressure for no surcharge condition, coefficient of earth pressure, Rankine's theory and assumptions made for non-cohesive Soils.

Unit	Unit Outcomes (UOs) (in cognitive domain)	Topics and Sub-topics
Unit - V ompaction and abilization of soil efer chapter 5	5a Describe the process of compaction and stabilization in the given situation.	5.1 Concept of compaction, purpose of compaction, field situations where compaction is required, Standard proctor test, test procedure as per IScode, Compaction curve, optimum moisture content, maximum dry density, Zero air voids line, Modified proctor test, factors affecting compaction, field methods of compaction rolling, ramming and vibration and Suitability of various compaction equipments smooth wheel roller, sheep foot roller, pneumatic tyred roller, rammer and vibrator, difference between compaction and consolidation.
	5b Suggest the relevant compacting equipment for the given type soil sample with justification.	5.2 Concept of soil stabilization, necessity of soil stabilization, different methods of soil stabilization – echanical soil stabilization, lime stabilization, cement stabilization, bitumen stabilization, fly-ash stabilization California bearing ratio, C.B.R. test, meaning of C.B.R value.
	5c Choose the relevant method of soil stabilization for the given situation with justification.	5.3 Necessity of site investigation and sub-soil exploration, types of exploration, criteria for deciding the location and number of test pits and bores. Field identification of soil-dry strength test, dilatancy test and toughness test.
	5d Compute the CBR value for given data of soil sample.	
	5e Interpret the value of CBR with reference to IS provisions.	

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CHAPTER 1

Overview of Geology and Geotechnical Engineering

Syllabus

1.1 Introduction of geology, different branches of geology, importance of geology for civil engineering structure and composition of earth.

1.2 Introduction of petrology, definition of a rock, classification based on their genesis (mode of origin), formation, classification and engineering uses of igneous, sedimentary and metamorphic rocks.

1.3 IS definition of soil, Importance of soil in Civil Engineering as construction material in Civil Engineering Structures, as foundation bed for structures

1.4 Field application of geotechnical engineering for foundation design, pavement design, design of earth retaining structures, design of earthen dam.

Syllabus Topic : Introduction of Geology

1.1 Introduction

➔ **(MSBTE – S-14, S-15, S-16)**

Q. Define 'Geology'. *(S-14, S-15, S-16)*

☞ ***Definition of Geology : Geology** (from Greek: geo "earth" and logos, "science") is the science of study of the solid matter that constitutes the Earth.*

– Encompassing such things as rocks, soil, gemstones, geology studies, the composition, structure, physical properties, history, and the processes that shape Earth's components.

– Geologists have established the age of the Earth at about 4.6 billion (4.6×10^9) years, and have determined that the Earth's lithosphere, which includes the crust, is fragmented into tectonic plates that move over a rheic upper mantle (asthenosphere) via processes that are collectively referred to as plate tectonics.

– Geologists help locate and manage the Earth's natural resources, such as petroleum and coal as well as metals such as iron, copper and uranium.

– Additional economic interests include gemstones and many minerals such as asbestos, perlite, mica, phosphates, zeolites, clay, pumice, quartz and silica as well as elements such as sulphur, chlorine and helium.

Syllabus Topic : Different Branches of Geology, Introduction of Petrology

1.2 Branches of Geology

➔ **(MSBTE – S-16, W-16, S-18)**

Q. State branches of Geology. *(S-16)*

Q. Define petrology. *(W-16)*

Q. Define the following branches of Geology:

(i) Stratigraphy

(ii) Rock Mechanics *(S-18)*

Geology is sub-divided into the following branches :

Branches of Geology

- 1. Physical geology
- 2. Geomorphology
- 3. Mineralogy
- 4. Petrology
- 5. Structural geology
- 6. Stratigraphy
- 7. Palaeontology
- 8. Historical geology
- 9. Applied geology
  - (a) Engineering geology
  - (b) Mining geology
  - (c) Economic geology
  - (d) Petroleum geology
- 10. Hydrology
  - (a) Indian Geology
  - (b) Photo-Geology
- 13. Relationship of Geology with Other Branches of Science
  - (a) Geo-Chemistry
  - (b) Geo-Physics
  - (c) Geo-Hydrology
  - (d) Rock Mechanics

Fig C1.1 : Branches of Geology

→ **1) Physical geology**

It deals with the origin, development and ultimate fate of various surface features of the Earth and also its structures. It is the study of role played by the physical agents like wind, rain, running water, ice, volcanism and earthquakes.

→ **2) Geomorphology**

It deals with the land surface features of the Earth and encompasses their development, structure, modifications in space and time. It includes the study of development and disposition of mountains, plains, plateaus, valleys and basins and associated land forms.

→ **3) Mineralogy**

It deals with study of formation, occurrence, aggregation, properties and uses of minerals.

→ **4) Petrology**

It deals with study of various types of rocks, their mode of occurrence, their composition, textures, structures, geological and geographical distribution.

→ **5) Structural geology**

It deals with study of morphology, classification, causes of development and all other aspects related to the structural features of the rock surface. The structural features are folds, faults and joints.

→ **6) Stratigraphy**

It deals with study and interpretation of stratified rocks, and with the identification, description, sequence, both vertical and horizontal, mapping and correlation of stratigraphic rock units.

→ **7) Palaeontology**

It deals with study of fossils of ancient life forms and their evolution.

→ **8) Historical geology**

It deals with the study of past history of the earth as deciphered from the rocks. It includes Palaeogeography, Palaeotology and Stratigraphy. It gives the pictures of the land and seas, the climate and life of early times upon the earth.

→ **9) Applied geology**

The knowledge of geology is utilized for the benefit of man for various purposes in various ways and can be sub divided as under :

→ **a) Engineering geology**

- It deals with the study and use of geology in civil engineering.

It is the application of the geologic principles to engineering practice for the purpose of assuring that the geologic factors affecting the location, design, construction, operation and maintenance of engineering works are properly addressed

→ **b) Mining geology**

- It deals with the application of geology in mining engineering.
- It consists of the extractions of mineral resources from the Earth. Some resources of economic interests include gemstones, metals, and many minerals such as asbestos, perlite, mica, phosphates, zeolites, clay, pumice, quartz, and silica, as well as elements such as sulfur, chlorine, and helium.

→ **c) Economic geology**

It deals with the study of those minerals and rocks and materials exist in the earth which can be used for the benefit of the man. It includes non ores like petroleum, coal, building stones, natural gases and ores of all metals like iron, copper and uranium as well as minerals for refractories, abrasives, insulations and chemicals.

→ **d) Petroleum geology**

Petroleum geologists study locations of the subsurface of the Earth which can contain extractable hydrocarbons, especially petroleum and natural gas. Because many of these reservoirs are found in sedimentary basins, they study the formation of these basins as well as their sedimentary and tectonic evolution and the present-day positions of the rock units.

→ **10) Hydrology**

As a branch of geology, it deals with the studies of both quality and quantity of water that are present in the rocks in different states (Conditions). Moreover, it includes :

→ **a) Indian Geology**

As a branch of geology, it deals with the study of our motherland in connection with the coal/petroleum, physiographic, stratigraphy and economic mineral of India.

→ **b) Photo-Geology**

As a branch of geology deals with the study of aerial photographs.

→ **11) Relationship of Geology with Other Branches of Science**

In order to carry out civil engineering projects safely and successfully, geology should be related to the other branches of bordering sciences as described as follows

→ **(a) Geo-Chemistry**

As a branch of science, it deals with geology in such a way that it concerns with the abundance and distribution of various elements and compounds in the earth.

→ **(b) Geo-Physics**

As a branch of science, it is related with geology in such a way that it concerns with the constitution of the earth and the nature of the physical forces operating on within the earth.

→ **(c) Geo-Hydrology**

As a branch of science, it is related with geology in setting of ground water. In other words, Geohydrology is an "interaction between Geology and Hydrology".

→ **(d) Rock Mechanics**

As a branch of science, it is related with geology in dealing with the behaviour of rocks that is subjected to static and dynamic loads (force fields).

Syllabus Topic : Importance of Geology for Civil Engineering

1.3 Importance of Geology In Civil Engineering

→ **(MSBTE – S-15, S-16, W-17)**

Q. State any one importance of geology in civil engineering. *(S-15)*

Q. State objectives of geotechnical engineering. ***(S-16)***

Q. State any four importance of geology. ***(W-17)***

The role of geology in civil engineering may be briefly outlined as follows :

- Geology provides a systematic knowledge of construction materials, their structure and properties.
- The knowledge of Erosion, Transportation and Deposition by surface water helps in soil conservation, river control, coastal and harbor works.
- The knowledge about the nature of the rocks is very necessary in tunneling, constructing roads and in determining the stability of cuts and slopes. Thus, geology helps in civil engineering.
- The foundation problems of dams, bridges and buildings are directly related with geology of the area where they are to be built.
- The knowledge of ground water is necessary in connection with excavation works, water supply, irrigation and many other purposes.
- Geological maps and sections help considerably in planning many engineering projects.
- If the geological features like faults, joints, beds, folds, solution channels are found, they have to be suitably treated. Hence, the stability of the structure is greatly increased.
- Pre-geological survey of the area concerned reduces the cost of engineering work.

Syllabus Topic : Structure and Composition of Earth

1.3.1 Internal Structure and Composition of Earth

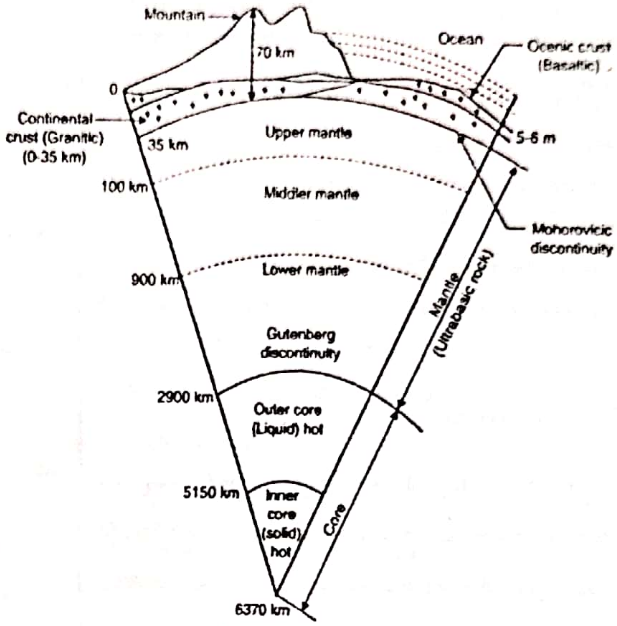


Fig. 1.3.1 : Internal structure and composition of earth

Syllabus Topic : Definition of a Rock

1.4 Rocks

→ **(MSBTE – W-16)**

Q. Define rock. ***(W-16)***

Definition of Rocks : *Rocks are defined in Geology as aggregates of the minerals.*

- Rocks of all kinds decompose or weather at the Earth's surface. Weathering breaks rocks into smaller fragments such as gravel, sand and clay.
- At the same time, rainwater may dissolve some of the rock. Streams, wind, glaciers, and gravity then erode the weathered particles, carry them downhill, and deposit them at lower elevations.
- All such particles, formed by weathering and then eroded, transported, and deposited in layers, are called sediment. The

sand on a beach and mud on a mud flat are examples of sediment that accumulated by these processes.

- A sedimentary rock forms when sediment becomes cemented or compacted into solid rock. When the beach sand is cemented, it becomes sandstone; the mud becomes shale. Sedimentary rocks make up less than 5 percent of the Earth's crust.

Syllabus Topic : Classification Based on their Genesis (Mode of Origin)

1.4.1 Types of Rocks

→ (MSBTE – S-14, S-15, S-16, S-17, W-17)

Q. State classification of rocks based on their genesis. (S-14, S-15, S-16, S-17, W-17)

Geologists group rocks into three categories on the basis of how they form : **Igneous rocks**, **Sedimentary rocks** and **Metamorphic rocks**.

- Geologists group rocks into three categories on the basis of how they form
  - 1. Igneous rocks
  - 2. Sedimentary rocks
  - 3. Metamorphic rocks

Fig. C1.2 : Classification of Rocks

- Igneous rocks formed at various depth below earth surface the rock granite and basalt comes under Igneous type of rock are used as building stones.
- Metamorphic rocks i.e. Marble is used for facing concrete or masonry exterior and interior walls and floor.
- The sedimentary rocks possess hard water and oil holding capacity hence can be used as reservoir rock which is suitable for dam and water retaining structure. In construction of roads houses, tunnels canals sedimentary rocks are used.

1.5 Igneous Rock

→ (MSBTE – W-14)

Q. State any two engineering uses of igneous rock. (W-14)

- An igneous rock forms when magma solidifies. About 95 percent of the Earth's crust consists of igneous rock and metamorphosed igneous rock.
- Although much of this igneous foundation is buried by a relatively thin layer of sedimentary rock, igneous rocks are conspicuous because they make up some of the world's most spectacular mountains. Igneous rocks are composed principally of silicate minerals.

Uses

(i) These are used for decorative work for flooring and walls.

(ii) These are also used as aggregate for concrete.

Syllabus Topic : Classification and Engineering uses of Igneous Rocks

1.5.1 Classification of Igneous Rocks

- Classification of Igneous Rocks
  - 1. Classification based on depth of formation
    - i) Extrusive Rocks
    - ii) Intrusive Rocks
      - a) Plutonic Rocks
      - b) Hypabyssal Rocks
  - 2. Classification based on Silica (SiO_2) Content
    - (i) acidic
    - (ii) intermediate
    - (iii) basic and
    - (iv) ultrabasic
  - 3. Classification based on Silica saturation
    - (i) Silica Under saturated Rocks
    - (ii) Silica Oversaturated Rocks
    - (iii) Silica Saturated Rocks
  - 4. Classification based on Colour of Minerals
    - (i) Leucocratic rocks
    - (ii) Mesocratic rocks
    - (iii) Melanocratic rocks
    - (iv) Hypermelanic rocks

Fig. C1.3 : Classification of Igneous Rocks

1) Classification based on depth of formation

Igneous rocks are classified into following two types :

i) Extrusive Rocks

Magma can either rise all the way through the crust to erupt onto the Earth's surface, or it can solidify within the crust. An extrusive igneous rock forms when magma erupts and solidifies on the Earth's surface. Because extrusive rocks are so commonly associated with volcanoes, they are also called volcanic rocks after Vulcan, the Greek god of fire. The texture of these rocks are the fine grained texture.

ii) Intrusive Rocks

An intrusive igneous rock forms when magma crystallized within the crust. Depending upon the depth of formation, the intrusive rocks are classified into plutonic rocks and hypabyssal Rocks.

a) Plutonic Rocks

- The rocks formed at greater depth are called as plutonic rocks after Pluto, the Greek god of the underworld. The texture of these rocks are coarse grained texture.
- For examples : Granites, Gabbros rocks.

b) Hypabyssal Rocks

- The rocks formed at depth close to the crust are called as hypabyssal rocks.
- Hypabyssal igneous rocks are formed at a depth in between the plutonic and volcanic rocks. These are less common than plutonic or volcanic rocks and do often form dikes, sills, laccoliths, Dolarites rocks.
- The texture of these rocks are the finer than plutonic and coarse grained than volcanic rock.

2) Classification based on Silica (SiO_2) Content

On the basis of the percentage of silica present in the rock, igneous rocks are classified as :

(i) Acidic (ii) Intermediate

(iii) Basic (iv) Ultrabasic.

(i)	Acid (or Felsic)	Silica > 66 wt. %	example : Granites, Quartz, Rhyolite
(ii)	Intermediate	Silica 52-66 wt%	example : Diorite
(iii)	Basic (or mafic)	Silica 45-52 wt%	example : Basalt
(iv)	Ultrabasic (or Ultramafic)	Silica < 45 wt %	example : Peridotite

This terminology is based on the onetime idea that rocks with a high % SiO_2 were precipitated from waters with a high concentration of hyrdosilicic acid H_4SiO_4.

3) Classification based on Silica saturation

- If a magma is oversaturated with respect to Silica then a silica mineral, such as quartz, cristobalite, tridymite, or coesite, should precipitate from the magma, and be present in the rock.
- On the other hand, if magma is under saturated with respect to silica, then a silica mineral should not precipitate from the magma, and thus should not be present in the rock.
- The silica saturation concept can thus be used to divide rocks in silica under saturated, silica saturated, and silica oversaturated rocks. The first and last of these terms are most easily seen.

(i) Silica Under saturated Rocks

In these rocks we should find minerals that, in general, do not occur with quartz. Such minerals are :

Nepheline : $NaAlSiO_4$

Leucite : $KAlSi_2O_6$

Sodalite : $3NaAlSiO_4.NaCl$

Noscan : $6NaAlSiO_4.Na_2SO_4$

Hauyne : $6NaAlSiO_4.(Na_2,Ca)SO_4$

Perovskite : $CaTiO_3$

Melanite : $Ca_2Fe + 3Si_3O_{12}$

Melilite : $(Ca, Na)_2(Mg, Fe + 2, Al, Si)_3O_7$

→ **(ii) Silica Oversaturated Rocks**

These rocks can be identified as possibly any rock that does not contain one of the minerals in the above list. Silica oversaturated rocks will contain normative quartz.

→ **(iii) Silica Saturated Rocks**

- These are rocks that contain just enough silica that quartz does not appear, and just enough silica that one of the silica under saturated minerals does not appear.
- These rocks contain olivine, or hypersthene + olivine, but no quartz, no nepheline, and no leucite.

→ **4) Classification based on Colour of Minerals**

The colour index of a rock is an expression of the percentage of **mafic minerals** (those with lots of ferromagnesium minerals) that it contains.

Four categories have been distinguished :

(i) **Leucocratic rocks :** which contain less than 30% dark minerals

(ii) **Mesocratic rocks :** which contain between 30 and 60% dark minerals

(iii) **Melanocratic rocks :** which contain between 60 and 90% dark minerals

(iv) **Hypermelanic rocks :** which contain over 90% dark minerals

Usually, acidic rocks are leucocratic, whereas basic and ultrabasic rocks are melanocratic and hypermelanic, respectively.

In general, the following shows the classification of igneous rocks.

1.6 Formation of Sedimentary Rocks

- Rivers, oceans, winds and rain runoff all have the ability to carry the particles washed off of eroding rocks, such material called detritus consists of fragments of rocks and minerals.
- When the energy of the transporting current is not strong enough to carry these particles, the particles drop out in the process of sedimentation. This type of sedimentary deposition is referred to as Clastic sedimentation. Another type of sedimentary deposition occurs when material is dissolved in water, and chemically precipitates from the water. This type of sedimentation is referred to as chemical sedimentation.
- A third process can occur, wherein living organisms extract ions dissolved in water to make such things as shells and bones. This type of sedimentation is called biogenic sedimentation.
- Thus, there are three major types of sedimentary rocks : Clastic sedimentary rocks, chemical sedimentary rocks and biogenic sedimentary rocks.

1.6.1 Engineering Importance of Sedimentary Rocks

- Sedimentary rocks are deposited in strata that form a structure called bedding.
- The study of sedimentary rocks and rock strata provides information about the subsurface that is useful for civil engineering, for example in the construction of roads, houses, tunnels canals or other constructions.
- Sedimentary rocks are also important sources of natural resources like coal, fossil fuels, drinking water or ores.
- The sedimentary rocks are possesses good water and oil holding capacity due to which these rocks can be used ad

reservoir rocks i.e. these are suitable for the dam and other water supply schemes.

- Chemical sedimentary rocks have a non-clastic texture, consisting entirely of crystals. To describe such a texture only the average size of the crystals and the fabric are necessary.

Syllabus Topic : Classification and Engineering uses of Sedimentary Rocks

1.6.2 Classification of Sedimentary Rocks

Sedimentary rocks are classified into three groups depending upon their mode of formation. These groups are clastic, chemical precipitate and biochemical (or biogenic).

Classification of Clastic Sedimentary Rocks

Clastic sedimentary particles are classified in terms of size as tabulated in Table 1.6.1.

Table 1.6.1 : Classification of Clastic Sedimentary Rocks

Name of Particle	Size Range	Loose Sediment	Consolidated Rock
Boulder	> 256 mm	Gravel	Conglomerate or Breccia (depends on rounding)
Cobble	64 - 256 mm	Gravel	
Pebble	2 - 64 mm	Gravel	
Sand	1/16 - 2mm	Sand	Sandstone
Silt	1/256 - 1/16 mm	Silt	Siltstone
Clay	< 1/256 mm	Clay	Claystone, mudstone and shale

Based on the presence of the sediment of particular grain size, the clastic rocks are further sub divided into the following three types :

Classification of Clastic Sedimentary Rocks
- (i) Rudaceous rocks
- (ii) Arenaceous rocks
- (iii) Organic rocks
- (iv) Argillaceous rocks

Fig. C1.4 : Classification of Clastic Sedimentary Rocks

(i) Rudaceous rocks

Definition of Rudaceous Rocks : The rocks which include all the coarsed grained of more than 2mm size heterogeneous composition are called as rudaceous rocks. These are also called as rudites or phephites.

- Breccia and Conglomerates are the important rocks.

(ii) Arenaceous rocks

Definition of Arenaceous Rocks : The rocks which include all the sediment of size between 1/16 – 2 mm are called as arenaceous rocks. These are also called as arenites or psamites.

- Sandstone, Greywackes and Arkoses are the important rocks.
- These shells accumulate on the ocean floor in great quantities to form sedimentary rocks.
- Examples : Shell Limestone.

(iii) Organic rocks

Definition of Organic Rocks : The sedimentary rocks formed due to accumulation and decomposition of vegetation matter are called as organic rocks.

- Examples : Peat, Lignite, Coal.

(iv) Argillaceous rocks

Definition of Argillaceous Rocks : The rocks which include all the finest sediment of size less than 1/16 mm are called as argillaceous rocks. These are also called as lutites or pelites.

- Silt, Clay, Shales and Mudstones are the important rocks.

1.6.2.1 Classification of Chemically Formed Sedimentary Rocks

On the basis of the chemical composition of sedimentary rocks, these are rocks are sub divided into following types :

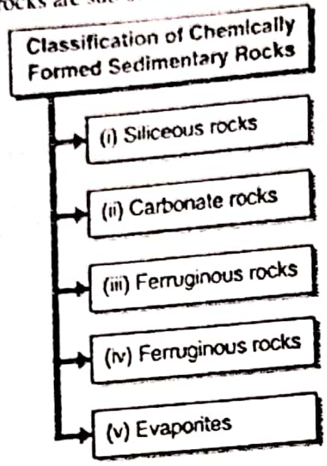


Fig. C1.5 : Classification of Chemically Formed Sedimentary Rocks

→ **(i) Siliceous rocks**

- These sedimentary rocks are contained the silica as chief constituent.
- Example : Flint, Chert, Jasper.

→ **(ii) Carbonate rocks**

- These sedimentary rocks are precipitated from carbonate rich water under different conditions controlled by the concentration of carbon dioxide.
- Example : Carbonate deposits of iron, limestone, dolomites.

→ **(iii) Ferruginous rocks**

- These sedimentary rocks are formed by precipitation of iron oxides.
- Example : Hematite, Pyrite, Iron stone.

→ **(iv) Ferruginous rocks**

- These sedimentary rocks are precipitated from sea water rich water rich in phosphoric acid.
- Example : Limestone, Shale.

→ **(v) Evaporites**

- These sedimentary rocks are formed by the evaporation of saline lakes.
- Example : Gypsum, Anhydrite, Rock salt, Borates.

1.6.2.2 Classification of Organically Formed Sedimentary Rocks

On the basis of the composition of remains of animals or plants, these are rocks are sub divided into following types :

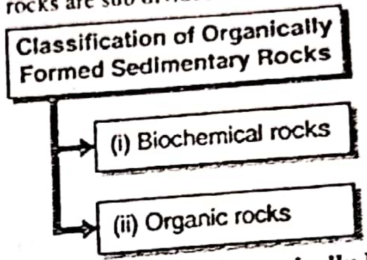


Fig. C1.6 : Classification of Organically Formed Sedimentary Rocks

→ **(i) Biochemical rocks**

- The biochemical sediments are produced when plants and animals living under water, extract from it dissolved mineral matter, usually calcite, to form shells.
- These shells accumulate on the ocean floor in great quantities to form sedimentary rocks.

e.g. Shelt Lime Stone.

→ **(ii) Organic rocks**

> ☞ ***Definition of Organic rocks : The sedimentary rocks formed due to accumulation and decomposition of vegetation matter are called as organic rocks.***

e.g. Peat, Lignite, Coal.

Syllabus Topic : Classification and Engineering uses of Metamorphic Rocks

1.7 Introduction of Metamorphic Rocks

→ **(MSBTE – S-18)**

> **Q. Give the most common classification of the Metamorphic Rocks based on the basis of foliation** *(S-18)*

A metamorphic rock forms when any preexisting rock is altered by heating, increased pressure, or tectonic deformation.

- Tectonic processes can depress the Earth's surface to form a basin that may be hundreds of kilometers in diameter and thousands of meters deep.
- Sediment accumulates in the depression, burying the lowermost layers to great depths.
- When a rock is buried, its temperature and pressure increase, causing changes in both the minerals and the texture of the rock.
- These changes are called metamorphism, and the rock formed by these processes is a metamorphic rock.
- Metamorphism also occurs when magma heats nearby rock, or when tectonic forces deform rocks, Schist, gneiss, and marble are common metamorphic rocks.
- Most common classification is based on the basis of foliation and the rocks are classified into following two types :

Classification is based on the basis of foliation and the rocks
- 1. Foliated Rocks
- 2. Non Foliated Rocks

Fig. C1.7 : Classification based on the basis of foliation

1.7.1 Foliated Rocks

If a rock is foliated, its name is determined by the type of foliation present and the dominant minerals for example: a kyanite schist.

1.7.2 Non Foliated Rocks

If a rock is not foliated, its name is derived from its chemical composition. A quartz-rich rock such sandstone, for example, is called a quartzite when it has been metamorphosed. A metamorphosed limestone is called a marble.

1.8 Origin of Soil

→ (MSBTE – W-14, W-16)

Q. Explain the formation process of soil. State various types of soils available in India. *(W-14, W-16)*

Soil formation mainly takes place due to mechanical disintegration or chemical decomposition of rocks whenever rock get exposed to atmosphere, it is acted by various weathering agencies and it gets disintegrates or decomposed in to small particles and then it is converted into soil.

The common types of soil available in India along with gravel, sand, silt, clay, organe, peat are.

☞ Types of soil available in India

1. Residual soil

   (a) Red soil (b) Laterite soil

Black cotton soil

2. Transported soil

   (a) Colluvial soil (b) Alluvial soil

   (c) Glacial soil (d) Lacustrine soil

   (e) Eolian soil

G7/Hyd/HEN/N5T

Syllabus Topic : IS definition of soil

1.8.1 I.S. Definition of Soil

→ (MSBTE – S-08, W-09, W-10, S-11, W-12, S-14, W-14, W-15, S-16)

Q. Define soil. *(W-10, W-12)*
Q. Define soil as per IS 2809-1972. *(S-11)*
Q. State IS code definition of 'soil'. *(S-14, W-15)*
Q. Define soil as per I.S. *(S-08, W-09, W-14, S-16)*

- The word 'soil' has different meanings according to different context. Hence there exist various definitions of soil.

- For example, an agriculturist and an engineer will view soil according to their own particular requirement. Thus, soil has various definitions as given below

> ☞ ***Definition of Layman's :*** *The upper surface of earth or the earth crust containing loose material, with any one or mixture of clays and gravel, pebbles etc.*
>
> ☞ ***Definition of Agriculturists :*** *Part of earth surface which supports, sustains and nourishes plants.*

☞ **Meaning as per Webster Dictionary : Soil (n)**

1. Finely divided rock mixed with decayed vegetable or animal matter, constituting that portion of the earth in which plants grows.
2. The ground in general, native land, country.
3. A mixture of lampblack, glue and water used in plumbing.

> ☞ ***Definition of Civil engineering :*** *Mixture of minerals and rock, derived from chemical and mechanical weathering of rock.*

☞ **As per Indian standards 2809-1972 I.S. Definition**

> ☞ ***Definition of Soil :*** *Soil is defined as the sediment or other unconsolidated accumulation of solid particles produced by physical and chemical disintegration of rock.*

1.8.2 Importance of Soil in Civil Engineering

→ **(MSBTE – S-09, W-13, W-17)**

> **Q.** State the importance of soil in Civil Engineering. *(S-09)*
>
> **Q.** State any four importance of soil in Civil Engineering Structures. *(W-13, W-17)*

The importance of soil in civil engineering is soil can be used as :

- Importance of soil
  - 1. As construction material in civil engineering
  - 2. As foundation material

Fig. C1.8 : Importance of soil

- Soil is a very useful material in civil engineering. Its easy availability and low cost make it a very versatile and widely used material.
- Thus importance of soil is manifold. It can be used in many different ways.
- The various factors which define the importance of soil are discussed here.

Syllabus Topic : Importance of Soil in Civil Engineering as Construction Material in Civil Engineering Structures

1.8.3 As Construction Material in Civil Engineering Structures

→ **(MSBTE – S-15, S-18)**

> **Q.** State importance of soil as construction material in civil engineering. *(S-15)*
>
> **Q.** State any for applications of soil as construction material in civil engineering. *(S-18)*

- Construction Material In Civil Engineering Structures
  - 1. Clay
  - 2. Pervious and impervious soil
  - 3. For WBM water bound macadam roads
  - 4. Canals and embankments
  - 5. Retaining walls

Fig. C1.9 : Construction Material in Civil Engineering Structures

→ **1. Clay**

The clayey soil is generally used in the formation of clay bricks with proper addition of required amount of water to make it. It is cheap and can be utilised for various construction operations.

→ 2. Pervious and impervious soil

A part from making bricks clay is main ingredient of earthen dam the whole hearting zone is constructed with clay with optimum moisture content so as to form impervious layer similarly murrum is used to cover the hearing zone.

→ 3. For WBM water bound macadam roads

For many road work soil is used as a binder. The soil is compacted at optimum moisture content and thus gets filled in the gap of stone and ballast and act as a binders.

→ 4. Canals and embankments

After construction of canals soil is particularly used as filling material.

→ 5. Retaining walls

In retaining wall soil is used as filling material. The type of soil used may vary depending upon its necessity and required parameters.

(6) A part from this soil can be used for other purposes mainly.

(i) River protection work.

(ii) Soil cement mixture used as sub grade.

(iii) Soil is also used to hold water due its impermeable nature (particularly clayey soil).

(iv) Soil is also used as filter.

Syllabus Topic : Importance of Soil in Civil Engineering as Foundation Bed for Structures

1.8.4 As Foundation Bed for Structures

→ **(MSBTE – S-17, S-18)**

Q. Explain the use of soil as foundation material. *(S-17)*

Q. State any for applications of soil as foundation material. *(S-18)*

- Soil, at various levels is known as soft soil, murum, hard murum, rock etc.
- Generally the foundation of any structure will go below the ground level upto hard murum or rock
- Following structures can be cited as having soil as their ultimate supporting structure :
  1. Bridge pier foundations
  2. Building foundations
  3. Earth dams
  4. Railways
  5. Runways of airports, roads, etc.
- In fact, ultimately the foundation of any structure on this earth will have to rest on soil.
- The foundation of any structure rest on soil thus, it acts as a bed for structure. Soil cement mixture used as sab grade pervious and impervious soil is used in earthen dam. Thus soil helps to give necessary support to various structures stated above.

Syllabus Topic : Field application of Geotechnical Engineering for Foundation Design, Pavement Design, Design of Earth Retaining Structures, Design of Earthen Dam

1.9 Field Applications of Geo-technical Engineering

→ **(MSBTE – S-08, W-08, S-10, W-10, S-11, W-11, S-14, W-14, S-15, W-15, S-16, W-16, S-17, W-17, S-18)**

Q. State any two civil engineering situations where knowledge of geo-technical engineering is required. *(W-08, W-11)*

Q. State any four field application of geo-technical engineering. *(S-08, S-10, S-14, S-16, W-16, S-17, W-17, S-18)*

> **Q.** Give any six reasons why the knowledge of Geotechnical Engineering is important for Civil Engineering field. *(W-10)*
>
> **Q.** Explain field applications of geotechnical engineering. *(S-11)*
>
> **Q.** Explain any four field applications of geotechnical engineering knowledge. *(W-14)*
>
> **Q.** State field application of geotechnical engineering. *(S-15, W-15)*

- Geo-technical Engineering also called sometimes soil mechanics, can be applied in various situations where the properties of soil and its behaviour under the action of forces or the passage of water through soil is of importance.
- The various applications of Geo-technical engineering are as follows :

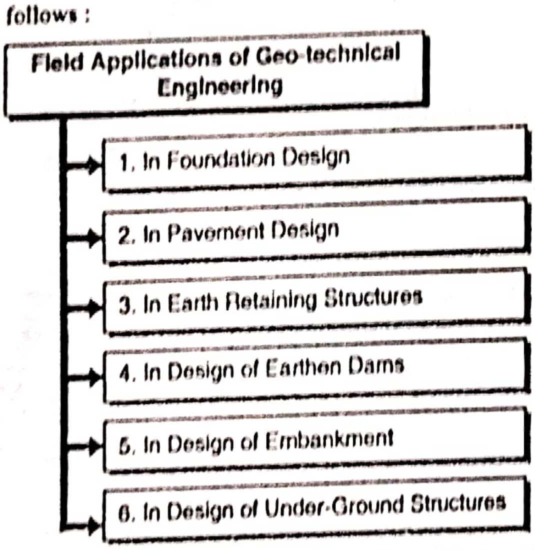


Fig. C1.10 : Field Applications of Geo-technical Engineering

1.9.1 Field Application in Foundation Design

→ **(MSBTE – W-16)**

> **Q.** State importance of soil as a foundation bed for structures in civil engineering. *(W-16)*

- The property of soil which is considered mainly in foundation design is its **Bearing capacity**. Bearing capacity is the maximum compressive stress that a soil will be able to carry.
- The area of the foundation will depend upon the bearing capacity. By increasing the area of foundation, the intensity of load will reduce.
- If this reduced intensity of load i.e. compressive stress is within limit of the bearing capacity, the foundation will be safe. It will not settle down.
- Thus, in foundation design, knowledge of geo-technical engineering is essential.
- Soil is ultimate foundation material which support any structure and proper functioning of any structure.
- It will depend on the success of foundation. Good will be the foundation more will be the life of structure.
- It is therefore necessary to know the stress pattern in the soil below the foundation.
- The probable settlement if any in the foundation effects of vibrations, ground water table, its shrinkage, swelling etc. should be known.

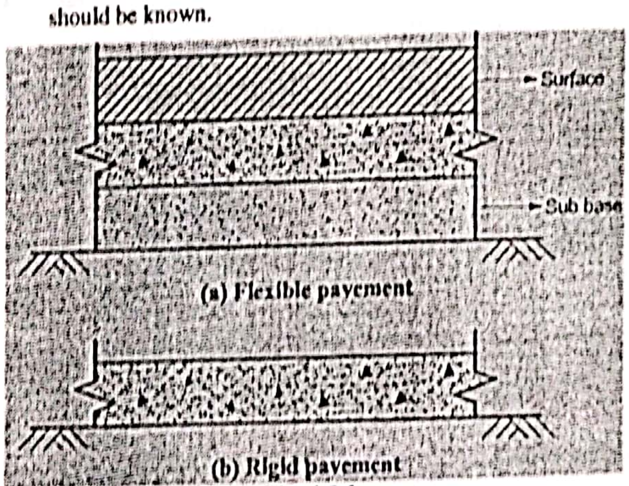


Fig. 1.9.1

1.9.2 In Pavement Design

- In pavement design, due to seepage of water below the pavement, the permeability of soil is the important property to be considered in the application.
- Also compaction and consolidation of soil comes into picture in pavement design.

- Depending upon subsoil, pavement can be flexible or rigid and it directly attached with the properties of subsoil.
- Therefore while designing the pavement by considering various points related with soil like effects of frost or thaws. Its suitability as a construction material for constructing highways, railways etc. should be decided.
- The thickness of pavement depends upon the type of soil strength etc.

1.9.3 In Design of Earth Retaining Structures

- The active and passive earth pressures and the safe and critical heights of an open cut form the important properties in the design of earth retaining wall.
- Permeability of the soil retained behind the wall is also important for the drainage of the retained earth.
- The knowledge of active earth and passive earth pressure, safe and critical heights of an open cut density and moisture content is important for the design of earth retaining structures.

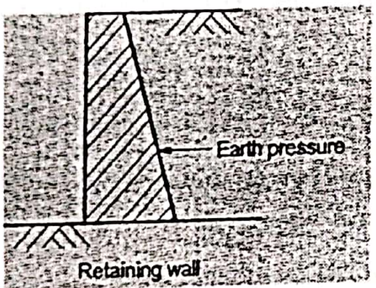


Fig. 1.9.2

1.9.4 In Design of Earthen Dams

- Earth dams requires huge amount of both pervious and impervious soils.
- The design of earth dams is a very sophisticated process involving various properties of soil. Thus, the importance of soil mechanics in various field situations is evident from the above examples.
- In earthen dam soil is the only construction material used, hence construction of earth dam requires through knowledge of soil mechanics.
- So all the properties should be known, the optimum water content at which maximum density would be obtained, all the characteristics like consolidation, stability of slopes etc. should be taken into account.

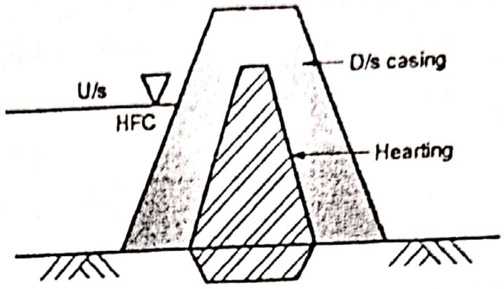
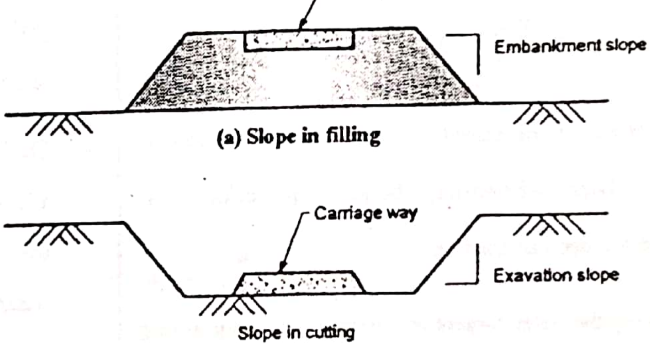


Fig. 1.9.3 : Earth dam

1.9.5 Design of Embankment or Cutting

- In embankment or in cutting the knowledge of shearing strength, angle of repose and frictional coefficient is essential.
- Thus from above example it is clear that the soil mechanics is important in various field of constructions.



(b) Slope in cutting and filling
Fig. 1.9.4

1.9.6 In Design of Under Ground Structures

While designing the underground structures such as tunnels conduits power houses the role of soil is important as load is transferred through the soil, therefore stress distribution should be taken into account.

Chapter Ends

□□□

CHAPTER 2

Physical Index Properties of Soil

Syllabus

2.1 Soil as a three phase system, water content, determination of water content by oven drying method as per IS code, void ratio, porosity and degree of saturation, density index, unit weight of soil mass – bulk unit weight, dry unit weight, unit weight of solids, saturated unit weight, submerged unit weight, determination of bulk unit weight and dry unit weight by core cutter method and sand replacement method as per IS code, specific gravity, determination of specific gravity by pycnometer.

2.2 Consistency of soil, stages of consistency, Atterberg's limits of consistency viz. Liquid limit, plastic limit and shrinkage limit, plasticity index, determination of liquid limit, plastic limit and shrinkage limit as per IS code.

2.3 Particle size distribution, mechanical sieve analysis as per IS code particle size distribution curve, effective diameter of soil, Uniformity coefficient and coefficient of curvature, well graded and uniformly graded soils, particle size, Classification of soils, I.S. classification of soil.

Introduction

Soil is a complex engineering material with properties different than other conventional materials. This is because soil has a microstructure made up of particles and spaces between the particles. These spaces may be filled by air or water. As such the properties of soil tend to be complex. Other engineering materials are mostly solid, sometimes liquid or gaseous but majority of them consist of only one phase of matter. In contrast to this soil has all the three phases of material together.

Syllabus Topic : Soil as a Three Phase System

2.1 Soil as a Three Phase System

→ (MSBTE-W-08, W-10, S-11, W-11, W-12, S-13, W-14, S-16, S-17, W-17, S-18)

Q. Explain fully dried soil and partially saturated soil with the help of phase diagram. *(W-08)*

Q. Draw a phase diagram for saturated and dry soil. *(W-10, S-11, S-16)*

Q. Explain soil as three phase system with labeled sketch. *(W-11, W-12, S-13, S-17, W-17)*

Q. Draw three phase diagram for fully saturated soil. *(W-14, S-16, S-17)*

Q. Draw three phase diagram for dry condition with neat labelled diagrams and explain all the notations used thereinl. *(S-18)*

Q. Soil is called as three phase system, why ? Explain with a neat sketch with the meanings of all notations used therein. *(S-18)*

- In general, the soil mass is a three phase system composed of solid, liquid and gaseous matter. The solid phase comprises of mineral, organic matter or both.
- The mineral portion consists of particles of different sizes and shapes.

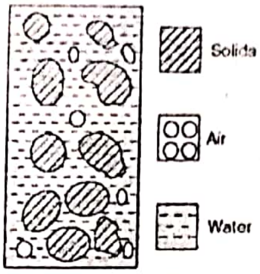
- The organic fraction is the plant and animal residue which may be present in various stages of decomposition.
- The solids enclose the open spaces termed voids.
- The liquid phase is generally water and fills partly or wholly the voids.
- The gaseous phase, usually air, occupies the voids not filled by water.
- The relative volume wise and weight wise proportions of the three phases in a soil mass are important factors influencing its physical properties.
- It is therefore necessary to study them. Though the three phases present cannot be separated as shown in the Fig. 2.1.1, for better understanding they are shown as occupying separate spaces.

> ☞ ***Definition of phase diagram :** The diagrammatic representation of the different phases present in a soil mass is called the phase diagram.*

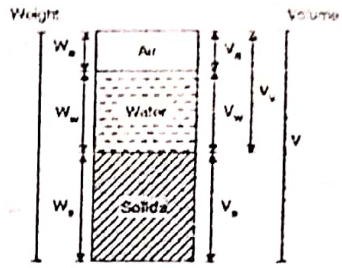
- Fig. 2.1.1(a) shows a three phase system which is present in a partially saturated soil.
- When it is **fully saturated** i.e. all the voids are filled with water and thus gaseous phase is absent and the soil becomes a two-phase system. **(Fig. 2.1.1(d))**.
- Also, if the soil becomes totally dry and consists of only solid and gaseous phases. In this case also, it is a two phase system. This is shown in Fig. 2.1.1(c).
- From saturated condition i.e. Fig. 2.1.1(d) it shows two phase system the voids are fully filled only with water and hence.

$$V_v = V_w$$

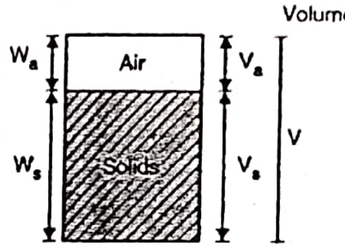
$$\therefore \text{Total weight } W = W_s + W_w$$



(a) Natural soil

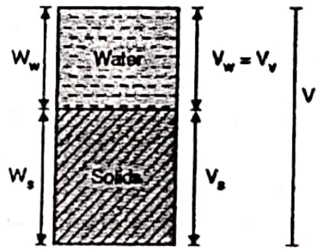


(b) Partially saturated condition



(c) Dry condition

Fig. 2.1.1



W = Total weight of soilmass
W_w = weight of water
W_s = weight of solid
V_w = volume of water
V_v = volume of voids
V_s = Total volume of soilmass
V_a = volume of air

(d) Saturated condition

Fig. 2.1.1 : Phase diagram

From 2.1.1(b)

$$\text{Total weight } W = W_s + W_w$$

$$\text{Total volume } V = V_s + V_a + V_w$$

$\because$ V_a is small it is neglected

$$\therefore V = V_s + V_w$$

For dry condition : Fig. 2.1.1(c)

$$W = W_s$$

$$\therefore V = V_s + V_a$$

For saturated condition : Fig. 2.1.1(d)

$$W = W_s + W_w$$

$$\therefore V = V_s + V_w$$

2.2 Primary Properties

Some of the important properties of soil are discussed below :

Syllabus Topic : Water Content

2.2.1 Water Content (w)

→ (MSBTE- S-10, S-11, S-13, S-14, W-14, S-15, S-16, W-16, W-17)

Q. Define : Water content.
(S-10, S-11, S-13, S-14, W-14, S-16, W-17)
Q. State any four methods to find water content of soil sample. *(S-15, W-16)*

- It is also called moisture content.

☞ ***Definition of Water Content :*** *It is defined as the ratio of the weight of water to the weight of solids in a given soil-mass.*

$$\text{Water content} = \frac{\text{Weight water}}{\text{Weight of solids}}$$

$$\%\ \text{water content} = \frac{\text{Weight water}}{\text{Weight of solids}} \times 100$$

$$= \frac{W_w}{W_s} \times 100 \qquad [0 \le w < \infty]$$

- Water content is expressed as a percentage, though in formulae giving relationships between quantities, it is expressed as a decimal fraction.
- In laboratory the water content is determined by oven drying method.

The various method to find water content are :

(1) Pychnometer method

(2) Oven drying method

(3) Sand bath method

(4) Calcium carbide method

(5) Alcohol method

Syllabus Topic : Determination of Water Content by Oven Drying Method as per IS Code

2.2.2 Determination of Water Content by Oven-drying Method as per IS Code IS-2809-1972

→ (MSBTE- S-09, W-09, S-11, W-11, W-12, S-17, W-17)

Q. Explain the determination of water content of soil by oven drying method. *(S-09, W-12)*
Q. Write step by step procedure for determination of water content by oven drying method as per I.S. code. *(W-09, W-11, S-17, W-17)*
Q. How do you determine water content of soil ? Explain. *(S-11)*

- This is the commonly adopted and simplest method for determination of water content of a soil sample in the laboratory.
- The method basically consists of drying a weighed moist sample of soil, in an oven at a controlled temperature for a period of twenty four hours, after which the dry weight of the sample is taken.
- The drying of soil is recommended at a temperature of 105°–110° C as temperatures higher than 110° C may break the crystalline structure of clay particles and result in loss of water of crystallisation, thus giving wrong results.
- A lower temperature of 60°C is recommended for highly organic soils at 110° C, oxidation of organic matter may take place.
- Sand and gravels require less time to dry, i.e. 4 to 6 hours, but routine laboratory procedure is for drying for twenty four hours at 105° C – 110° C.
- A clean, non-corrodible container is weighed within 0.01 gm accuracy.
- About 30 - 40 g of moist soil sample is placed in it and weighed accurately.
- It is then placed in the oven for drying at 110° C for 24 hours.

After 24 hours, it is taken out, allowed to cool and ...
again

The water content is then calculated as shown below ...

W_1 = Weight of container

W_2 = Weight of container with moist ...

W_3 = Weight of container with dried ...

$$\text{Weight of water } W_w = W_2 - W_3$$

$$\text{Weight of solids } W_s = W_3 - W_1$$

$$\text{Water content } w = \frac{W_2 - W_3}{W_3 - W_1} \times 100\%$$

Sr. No.	Size of particles more than 90% passing through	Mini... quantit...
1.	425 µ – IS sieve	2
2.	2 mm	5
3.	4.75 mm	2
4.	10 mm	3
5.	20 mm	[illegible]
6.	40 mm	1

The above table gives the minimum quantity of s...
taken for test.

3 Index Properties of Soil

As a three phase material soil has some spec... ese are called the index properties of soil. These ... ow :

Syllabus Topic : Void Ratio

3.1 Void Ratio (e)

→ (MSBTE- S-09, S-11, W-1... S-15, S-16, W-...

Q. Define : Void ratio.
(S-09, S-11, W-12, W-13, W-14, S-15... S-17, W-17)

- After 24 hours, it is taken out, allowed to cool and weighed again.

The water content is then calculated as shown below :

Let
W_1 = Weight of container

W_2 = Weight of container with moist sample

W_3 = Weight of container with dried sample

Weight of water $W_w = W_2 - W_3$

Weight of solids $W_s = W_3 - W_1$

Water content $w = \dfrac{W_2 - W_3}{W_3 - W_1} \times 100\%$

Sr. No.	Size of particles more than 90% passing through	Minimum quantity (gm)
1.	425 μ – IS sieve	25
2.	2 mm	50
3.	4.75 mm	200
4.	10 mm	300
5.	20 mm	500
6.	40 mm	1000

The above table gives the minimum quantity of soil sample to be taken for test.

2.3 Index Properties of Soil

As a three phase material soil has some special properties. These are called the index properties of soil. These are discussed below :

Syllabus Topic : Void Ratio

2.3.1 Void Ratio (e)

→ (MSBTE- S-09, S-11, W-12, W-13, W-14, S-15, S-16, W-16, S-17,W-17)

Q. Define : Void ratio. *(S-09, S-11, W-12, W-13, W-14, S-15, S-16, W-16, S-17,W-17)*

Definition of void ratio : The ratio of volume of voids ($V_v = V_a + V_w$) to the total volume (V) of solids is called as void ratio.

$$\therefore \quad e = \frac{V_a + V_w}{V_s}$$

$$e = \frac{V_v}{V_s}$$

- The void ratio is generally expressed as a decimal fraction.

Syllabus Topic : Porosity

2.3.2 Porosity (n)

→ (MSBTE - W-08, S-10, W-11, W-12, S-13, W-13, S-15, S-16, W-16, S-17, W-17)

Q. Define : Porosity. *(W-08, S-10, W-11, W-12, S-13, W-13, S-15, S-16, W-16, S-17, W-17)*

Definition : The ratio of volume of voids (V_v) to the volume of soil (V) is called as Porosity (n).

$$\therefore \quad n = \frac{V_v}{V} \times 100$$

2.3.3 Relation between Porosity and Void Ratio

$$e = \frac{V_v}{V_s} = \frac{V_v}{V - V_v} = \frac{V_v / V}{\left(\frac{V - V_v}{V}\right)}$$

$$\therefore \quad e = \frac{V_v / V}{1 - \frac{V_v}{V}}$$

$$\therefore \quad e = \frac{n}{1 - n}$$

$$n = \frac{V_v}{V} = \frac{V_v}{V_v + V_s}$$

Dividing numerator and denominator by V_s,

$$n = \frac{V_v / V_s}{\left(\frac{V_v + V_s}{V_s}\right)}$$

$$\therefore \quad n = \frac{V_v / V_s}{\frac{V_v}{V_s} + 1}$$

$$\therefore \quad n = \frac{e}{e + 1}$$

Subtracting n from 1,

or $$1 - n = 1 - \frac{e}{e + 1}$$

$$1 - n = \frac{e + 1 - e}{e + 1}$$

$$\therefore (1 - n)(e + 1) = 1$$

Syllabus Topic : Degree of Saturation

2.3.4 Degree of Saturation (S)

→ (MSBTE - W-08, S-09, S-12, S-13, W-15, S-16, W-16)

Q. Define : Degree of saturation. *(W-08, S-09, S-12, S-13, W-15, S-16, W-16)*

Definition of Degree of Saturation : *The degree of saturation, S is defined as the ratio of volume of water V_w to the volume of voids V_v. Thus,*

$$S = \frac{V_w}{V_v} = \frac{V_w}{V_a + V_w}$$

- The degree of saturation is usually expressed as a percentage. As it indicates the portion of voids filled with water, for a fully saturated soil, $V_a = 0$, and S = 1 or 100%.
- Similarly for a completely dry soil, $V_w = 0$ and S = 0 or zero percent. For a general case of partially saturated soil, the degree of saturation varies between 0 percent and 100 percent.

Syllabus Topic : Density Index

2.3.5 Density Index (I_D)

→ (MSBTE - S-08, W-09, S-13)

Q. Define : Density Index. *(S-08, W-09, S-13)*

- The term density index is not applicable to all types of soil. It is generally applied to cohesion less soils such as sand or silt.

Definition of Density index : *It is defined as the measure of comparative denseness of soil with respect to its loosest and densest state.*

- Mathematically it is given as,

$$\text{Density Index} = I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

Where, e_{max} = Maximum void ratio in loosest state of soil

e_{min} = Minimum void ratio in densest state of soil

e = Void ratio of soil in natural state

When soil is in loosest state,

$$e = e_{max} \quad \therefore I_D = 0$$

When soil is in densest state,

$$e = e_{min} \quad \therefore I_D = 1.$$

In natural state I_D lies between 0 and 1.

Density index	Compaction state
0 – 15	Very loose
15 – 35	Loose
35 – 65	Medium dense
65 – 85	Dense
85 – 100	Very dense

2.3.6 Density (ρ)

Definition : *Density (ρ) of soil mass is defined as the mass (M) per unit volume of soil.*

$$\rho = \frac{M}{V}$$

Where, M = Mass of soil

V = Total volume of soil

It is expressed in term of Kg/m^3, gm/cm^3

Syllabus Topic : Unit weight of Soil Mass – Bulk Unit Weight, Dry Unit Weight, Unit Weight of Solids, Saturated Unit Weight, Submerged Unit Weight

2.4 Unit Weight of Soil Mass

- In case of soil, different unit weights are defined instead of a unique value of unit weight.
- This is due to considering different weights such as weight of only soil solids, weight of soil solids and water etc. and also due to different volumes. Below, these unit weights are defined.
- It is defined as the weight of unit volume of a soil

$$\gamma = \frac{W}{V}$$

W = Weight of soil

V = Total volume of soil and it is expressed kN/m^3

2.4.1 Bulk Unit Weight (γ)

> ☞ ***Definition of Bulk Unit Weight :*** *It is defined as the total weight of the soil mass per unit total volume of soil mass. It is also called 'In-situ unit weight'.*

Thus, $$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a}$$

2.4.2 Dry Unit Weight (γ_d)

→ (MSBTE - W-14 , S-17)

> Q. Define dry unit weight of soil. (W-14, S-17)

> ☞ ***Definition of Dry Unit Weight :*** *It is defined as the weight of soil solids (W_s) per unit total volume of the soil mass (V).*

Thus, $$\gamma_d = \frac{W_s}{V}$$

2.4.3 Unit Weight of Soil Solids (γ_s)

> ☞ ***Definition of Unit Weight of Soil Solids :*** *It is defined as the weight of soil (W_s) solids per unit total volume of the soil solids (V_s).*

Thus, $$\gamma_s = \frac{W_s}{V_s}$$

2.4.4 Saturated Unit Weight (γ_{sat})

→ (MSBTE - S-17)

> Q. Define : Saturated unit weight. (S-17)

> ☞ ***Definition of Saturated Unit Weight :*** *It is defined as the weight of fully saturated soil (W_{sat}) per unit volume of that fully saturated soil mass (V).*

$$\gamma_{sat} = \frac{W_{sat}}{V_s}$$

- Where W_{sat} is the weight of the soil mass when it is fully saturated.

2.4.5 Submerged Unit Weight (γ_{sub})

→ (MSBTE - S-12)

> Q. Define : Submerged unit weight. (S-12)

- The soil particles below the water table behave as if they are immersed in water. When any solid is immersed in water, its weight is reduced due to buoyancy.
- Hence the submerged unit weight is defined as the submerged weight of soil solids per unit volume of the soil mass.

Thus, $$\gamma_{sub} = \frac{(W_s)_{sub}}{(V_s + V_v)} = \frac{(W_s)_{sub}}{V}$$

It can be shown as,

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

- Where γ_w is the unit weight of water and it is generally taken as 9.81 kN/m^3 or approximately 10 kN/m^3.

2.4.6 Air Content

→ (MSBTE - W-15)

> Q. Define : Density index. (W-15)

> ☞ ***Definition of Air Content :*** *It is the ratio of volume of air (V_a) to volume of voids (V_v).*

$$a_c = \frac{V_a}{V_v} \times 1w$$

- It is expressed as percentage.

If $a_c = 0$ % soil is saturated.

If $a_c = 100$% soil is dry.

2.4.7 Relation between G, w, S and e

→ (MSBTE - S-08, W-08, S-09)

> Q. Derive relation between e, S and w, G. (S-08)
> Q. Derive the following equation; eS = wG. (W-08)
> Q. Derive the relation between void ratio, water content, degree of saturation and specific gravity. (S-09)

$$w = \frac{W_w}{W_s} = \frac{V_w \cdot \gamma_w}{V_s \cdot \gamma_s}$$

$$= \frac{1}{G} \cdot \frac{V_w}{V_s} \qquad \left(\because G = \frac{\gamma_s}{\gamma_w}\right)$$

$$= \frac{1}{G} \cdot \frac{V_w}{V_s} \cdot \frac{V_v}{V_v} = \frac{1}{G} \cdot \frac{V_w}{V_v} \cdot \frac{V_v}{V_s}$$

$$= \frac{1}{G} \cdot s \cdot e \quad \ldots(1)$$

$$w = \frac{s \cdot e}{G}$$

2.4.8 Relation between γ_d, γ, and w

→ (MSBTE – W-09, S-11)

> Q. Derive relation between γ_d, γ and W. *(W-09)*
>
> Q. Derive the following equation : $\rho_b = \frac{(G + e.Sr)}{1 + e} \rho_w$ *(S-11)*

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{\gamma_s \cdot V_s + \gamma_w \cdot V_w}{V_s + V_v}$$

$$= \frac{\gamma_w \cdot G \cdot V_s + \gamma_w \cdot V_w}{V_s + V_v} \quad (\because \gamma_s = \gamma_w \cdot G)$$

Dividing throughout D and N by V_s

$$\gamma = \frac{\gamma_w\left(G \cdot \frac{V_s}{V_s} + \frac{V_w}{V_s}\right)}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}$$

$$= \frac{\gamma_w\left(G + \frac{V_w}{V_v} \cdot \frac{V_v}{V_s}\right)}{1 + e}$$

$$\gamma = \frac{\gamma_w (G + s \cdot e)}{1 + e} \quad \ldots(2)$$

The above equation can be written in another form replacing s by w using Equation (1).

$$\gamma = \frac{\gamma_w\left(G + \frac{wG}{\cancel{s}} \cdot \cancel{s}\right)}{1 + e}$$

$$\gamma = \frac{\gamma_w \cdot G (1 + w)}{1 + e} \quad \ldots(3)$$

☞ **Particular cases**

For dry soil : $\gamma = \gamma_d$ and $w = 0$

$$\gamma_d = \frac{\gamma_w \cdot G}{\gamma_d} \quad \ldots(4)$$

Also
$$1 + e = \frac{\gamma_w \cdot G}{\gamma_d}$$

$$e = \frac{\gamma_w \cdot G}{\gamma_d} - 1$$

Compare Equation (3) and (4)

$$\gamma_d = \frac{\gamma}{1 + w} \quad \ldots(5)$$

For fully saturated soil : $\gamma = \gamma_{sat}$ and $s = 1$

∴ Equation (2) becomes

$$\gamma_{sat} = \gamma_w \left[\frac{G + e}{1 + e}\right] \quad \ldots(6)$$

For submerged soil :

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$\therefore \gamma_{sub} = \gamma_w \left[\frac{G + e}{1 + e}\right] - \gamma_w = \gamma_w \left[\frac{G + e}{1 + e} - 1\right]$$

$$= \gamma_w \left[\frac{G + e - 1 - e}{1 + e}\right]$$

$$\gamma_{sub} = \gamma_w \left[\frac{G - 1}{1 + e}\right] \quad \ldots(7)$$

2.4.9 Relation between γ and γ_s

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s\left(1 + \frac{W_w}{W_s}\right)}{V_s\left(1 + \frac{V_v}{V_s}\right)}$$

$$\gamma = \gamma_s \left[\frac{1 + w}{1 + e}\right] \quad \ldots(8)$$

2.5 Determination of Unit Weight

In laboratory, the in-situ or bulk unit weight is determined. At the same time water content is also determined. From these, other unit weights are determined.

Syllabus Topic : Determination of Bulk Unit Weight and Dry Unit Weight by Core Cutter Method as per IS Code

2.5.1 Determination of Bulk Unit Weight and Dry Unit Weight by Core Cutter Method as Per IS Code : IS – 2720, part 29-1975

→ (MSBTE - W-08, S-10, S-12, W-12, S-13, W-13, S-14, W-15, S-17, W-17,S-18)

> Q. Explain the procedure for determination of dry density by core-cutter method. *(W-08, W-12, W-17,S-18)*
>
> Q. Explain the laboratory procedure to determine bulk unit weight and dry unit weight of soild by core cutter method. *(S-10, W-13)*
>
> Q. Explain, determination of bulk unit weight and dry unit weight by cone cutter. *(S-12)*

> Q. Explain how to determine bulk unit weight and dry unit weight of soil. *(S-13)*
> Q. Enlist two methods used to determine bulk density and dry density. *(S-14)*
> Q. Explain the method of determination in-situ density with a core-cutter. *(W-15, S-17)*
> Q. Explain core-cutter method. *(S-17)*

- The core cutter method consists of driving a core-cutter of known volume usually 1000 cc, into the soil after placing it on a cleaned soil surface.
- The core cutter is usually provided with a 25 mm high dolly.
- The driving of core-cutter is accomplished by hitting the dolly mounted on the core cutter with the rammer.
- The cutter filled with the soil is removed by cutting under it with a knife or sharp edge. The extra soil is trimmed off. The cutter with the soil is weighed.
- The volume of the core-cutter is calculated from the inner dimensions. The weight of empty core-cutter is also taken. Then the in-situ unit weight is determined by dividing the weight of soil in the core-cutter by the volume of the core-cutter.
- For determining the dry unit weight, a piece of the soil is taken and its water content, w, is determined by oven drying method as given is section 2.2.2.

Procedure

1. Measure inside dimension of core cutter and calculate volume.
2. Weight core cutter without dolly. (W_1)
3. Clean top of soil on site and level it place dolly on top of the core cutter and drive in to the soil with help and rammer until about 1 to 1.5 cm of dolly remain above surface.
4. Dig out container which is containing soil from ground. Remove dolly with help of straight edge trim flat and the end of cutter.
5. Weight cutter full with soil and find out water content (W_2).
6. Repeat procedure 2 to 3 times for getting average unit weight.

Then the dry unit weight can be found from the bulk unit weight as follows :

Bulk unit weight $= \gamma = \dfrac{W}{V}$

Where, W = Weight of soil

Dry unit weight $= \gamma_d = \dfrac{\gamma}{1+w}$

M = Mass of soil

V = Volume of core cutter

w = Water content

Where w is the water content expressed in decimal fraction.

The core-cutter is shown in Fig. 2.5.1.

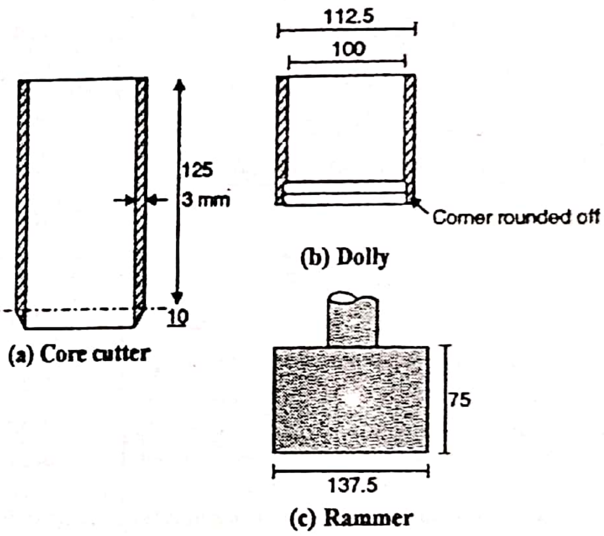


Fig. 2.5.1 : Core-cutter apparatus

Syllabus Topic : Determination of Bulk Unit Weight and Dry Unit Weight by Core Cutter Method and Sand Replacement Method as per IS Code

2.5.2 Determination of Bulk Unit Weight and Dry Unit Weight by Sand Replacement Method : IS 2720, Part 28-1974

→ **(MSBTE – S-08, S-15, S-18)**

> Q. Write step by step procedure for determination of field density by sand replacement method. *(S-08)*
> Q. Explain with neat sketch stepwise procedure to determine bulk density by sand replacement method. *(S-15)*

Q. Draw neat labelled sketch to explain stepwise procedure to determine bulk density by sand replacement method. (S-13)

- The core-cutter method cannot be used for loose soils or non-cohesive soils. Under these conditions, the sand replacement method is better suited.
- It consists of making a hole in the ground. The excavated soil is weighed and the volume of the hole is found out by filling it with standard sand, whose unit weight is known.
- Knowing the weight of excavated soil and the volume of the hole, the bulk or in-situ unit weight can easily be calculated.
- In this method, the site is cleaned and a square tray with a central hole in it is placed on the cleaned surface.
- A hole of diameter equal to the diameter of the hole in the tray and depth about 10 to 15 cm, is made and the soil is scooped out.
- The collected soil is weighed. Next, a sand bottle about two thirds full of clean, dry sand is weighed and placed centrally over the hole.
- The tap is opened and sand allowed to run to fill the excavated hole and the cone. When no further flow of sand takes place, the tap is closed, and the bottle with the remaining sand is weighed.
- The bottle is then kept on a level surface and the weight of sand filling the cone is noted. From this the weight of sand filling only the hole is calculated.
- The volume of the cone is known. From the weight of sand filling only the cone and the volume of cone, unit weight of sand is calculated.
- The weight of sand filling only the hole divided by this unit weight will give the volume of the sand filling the hole.
- In other words, it gives accurate volume of the irregular, excavated hole. Dividing the weight of excavated soil by the volume of the hole will give the bulk unit weight γ of soil.

The water content 'w' of the excavated soil is determined by oven drying method as given in section 2.2.2. Then, dry unit weight is calculated by,

$$\gamma_d = \frac{\gamma}{1+w}$$

This is shown in Fig. 2.5.2.

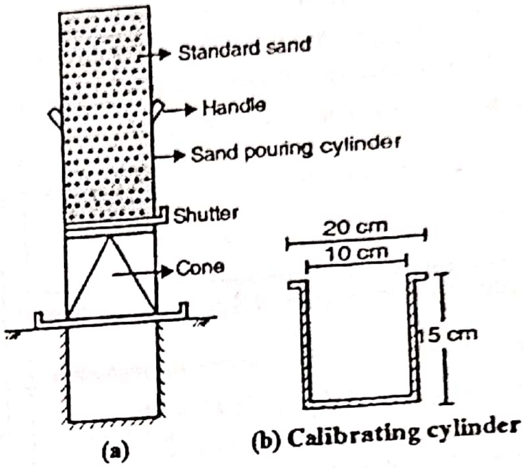


Fig. 2.5.2 : Sand replacement method

Syllabus Topic : Specific Gravity

2.6 Specific Gravity (G)

→ (MSBTE – W-10, W-11, S-12)

Q. Define specific gravity of soil. (W-10)
Q. Define : Specify gravity. (W-11, S-12)

☞ ***Definition of Specific Gravity :*** *It is defined as the ratio of unit weight of soil solids, i.e. γ_s to the unit weight of water γ_w at 4°C at normal pressure.*

$$G = \frac{\gamma_s}{\gamma_w}$$

γ_s = Unit weight of soil solid,

γ_w = Unit weight of water

It is a pure member and does not have any units.

☞ **Absolute specific gravity**

☞ ***Definition of Absolute Specific Gravity :*** *It is the ratio of weight absolute solids as to the weight of equal volume of water.*

$$G_a = \frac{[\gamma_s]a}{\gamma_w}$$

Where, G_s - Absolute specific gravity

$(\gamma_s)a$ - Unit weight of absolute solids

γ_w - Unit weight of equal volume of water

- **Bulk specific gravity (G_m)**

All soil solids have both permeable impermeable voids in them, permeable voids may get filled with water. If we want to determine true volume of solids, internal voids of soil particles are excluded. Thus specific gravity obtained is called absolute or true specific gravity.

$$G_m = \frac{\gamma}{\gamma_w}$$

Syllabus Topic : Determination of Specific Gravity by Pycnometer

2.6.1 Determination of Specific Gravity by Pycnometer : IS 2720 Part 3, Section 1 – 1980, Reaffirmed 1987

→ (MSBTE – S-09, S-10, W-10, W-11, S-12, W-12, S-16, S-17)

> Q. Explain the laboratory method of determination of specific gravity by pycnometer method. (S-09, S-10, W-12, S-16)
> Q. How is it determined in Laboratory? (W-10)
> Q. Write step by step procedure for determination of specific gravity by pycnometer bottle. (W-11)
> Q. Describe Specific gravity determination by pycnometer. (S-12, S-17)

- A pycnometer is a bottle with a lid in the form of inverted cone with a hole on the top. An empty dried pycnometer is weighed, say W_1.

- Next about 300 g of soil dried in the oven and cooled in the desiccators is placed in the pycnometer and the pycnometer is weighed again say W_2.

- The pycnometer is again filled with water and its weight is noted, W_3. Finally, the pycnometer is emptied completely, cleaned and dried and weighed after filling it completely with water, W_4.

Now weight of dry soil $= W_s = W_2 - W_1$

Weight of equal volume of water

$$= (W_2 - W_1) - (W_3 - W_4)$$

Specific gravity $= G$

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)}$$

$$G = \frac{W_2 - W_1}{W_2 - W_1 - W_3 + W_4}$$

The pycnometer is shown in Fig. 2.6.1.

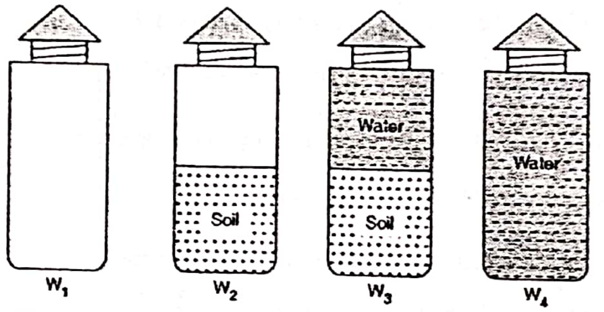


Fig. 2.6.1 : Specific gravity by pycnometer

Syllabus Topic : Consistency of Soil

2.7 Consistency of Soil

> ***Definition of consistency of soil : The ease with which a soil paste can be moulded is known as the consistency of soil.***

- It is also the state of liquidity, which the soil can achieve at particular moisture content.

- The soil, especially a clayey soil, behaves like a solid, plastic or liquid depending upon its water content.

- This behaviour of change of phase with water content is known as consistency.

Syllabus Topic : Stages of Consistency

2.7.1 Stages of Consistency

- Solid, plastic and liquid are the three stages of consistency. With increase in water content, the soil passes from solid to plastic and then from plastic to liquid state.

- If we heat a liquid paste of soil to reduce its water content, it will decrease in volume. This is known as shrinkage of soil.
- The volume will decrease upto a certain point after which the volume of the soil mass will remain constant. These are the different stages of consistency.

Syllabus Topic : Atterberg's Limits of Consistency viz. Liquid limit, Plastic limit and Shrinkage limit

2.7.2 Atterberg's Limits of Consistency

→ (MSBTE – S-08, W-08, S-09, S-10, W-10, S-11, S-12, W-12. S-14, S15, W-18)

Q. State and define Atterberg's limits of consistency. *(S-08, S-10, S-11)*
Q. State and define Atterberg's limits of consistency of soil. *(W-08, S-09)*
Q. What are Atterberg's limits ? Explain in brief. *(W-10)*
Q. Define consistency units. Show them on graph. *(S-12, W-12)*
Q. Define : Atterberg's limit of consistency. *(S-14, S-15)*
Q. Explain Atterberg's limit of consistency. *(S-18)*

Consistency limits

Definition of consistency limits : The percentage moisture content at which the soil passes from one state to another are called as consistency limits. These consistency limits are called as Atterbery's limits.

- The soil may be very stiff, plastic or flowing or liquid state depending on the water content. Atterberg in 1911 determine the boundary water contents at which the soil changes from one state to the other.
- Of course, these boundary water contents will have different values for different soils. These are known as Atterberg's limits.
- Later in 1932, A Casagrande presented Atterberg's work along with improved testing procedures.
- If water content is plotted against total volume of soil mass we get an idea of Atterberg's consistency limits. (Fig. 2.7.1).
- Let us consider a completely dry soil mass. Its volume will remain constant till the water content is increased upto saturation point. This is shown in part A-B of the graph.
- If water content is still increased, the total volume of the soil mass increases and the soil becomes semi-solid like putty or a thick paste. But if it is pressed with finger and released, it comes back to original state.
- A mark is not formed. It is not yet plastic. This is shown by graph B-C. Upon further increase of water content, the soil passes from semi-solid to plastic state.
- If it is pressed with finger, it retains the mark. It becomes mouldable to any shape. But it is still not liquid. This is shown on the graph as C-D.
- If water content is further increased, the soil will pass into liquid state, like flowing mud. It will flow under gravity and assume the shape of the container. This is shown on the graph as D-E and further.
- These states are the different phases of soil as they change with the water content.
- These phases can be considered in the reverse way also.
- Let us imagine that a very liquid mud is slowly heated in a can to reduce the water content. As the water content reduces, the total volume of soil mass will go on reducing and at a certain point, the soil passes from liquid to plastic phase.
- On further heating, the soil mass will shrink further and pass from the plastic to semi solid phase.
- When the soil reaches the point when it is just saturated with water, on further heating, its volume will not reduce any more, and it will pass into solid state. If we go on further heating, it will become more and more dry but its volume will not reduce.
- Thus, the water content below which the liquid soil passes in plastic phase is called the liquid limit.
- The water content below which the soil passes from plastic to semi-solid phase is called the plastic limit.
- The water content below which the plastic soil becomes semi solid is called the plastic limit.

- The water content below which the soil does not shrink further upon reduction of water content is called the shrinkage limit.
- Thus there are 4 phases of solid : solid, semi-solid, plastic and liquid.
- There are three limits separating these phases : liquid limit, plastic limit and shrinkage limit.
- Liquid limit is the water content separating the liquid and plastic phase.
- Plastic limit is the water content separating the plastic and semi-solid phase.
- Shrinkage limit is the water content separating the semi-solid and solid phase.

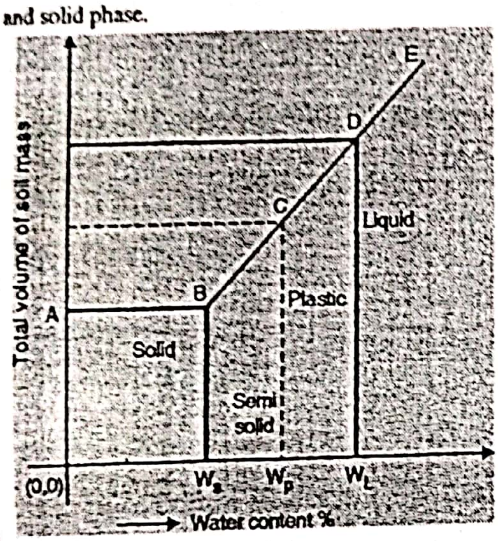


Fig. 2.7.1

2.7.3 Liquid Limit (w_L)

→ (MSBTE- W-09, W-11, S-13, W-14, S-16, S-17)

Q. Define : Liquid limit. (W-09, W-11, S-13, W-14, S-16, S-17)

- It is the water content at which the plastic soil changes into liquid soil, if water content is increased. Thus it is the minimum water content required for liquid state of soil.
- It can also be defined as the maximum water content upto which the soil will remain in a liquid state.
- The liquid limit can be determined by any standard experimental procedure.
- According to Casagrande's procedure, liquid limit is defined as the water content at which a groove cut in a part of soil cut by a grooving tool of standard dimensions will flow together for a distance of 1.25 cm under the impact of 25 blows in a standard liquid limit device.

2.7.4 Plastic Limit (W_p)

→ (MSBTE - W-09, W-11, S-13, S-16, S-17)

Q. Define : Plastic limit. (W-09, W-11, S-13, S-16, S-17)

- It is the minimum water content at which soil remains plastic, without being semi-solid or non plastic.
- It can also be defined as the maximum water content upto which the soil remains semisolid without becoming plastic.

☞ ***Definition of Plastic Limit : In procedural terms, it is defined as the water content at which a soil will just begin to crumble when rolled into a thread of approximately 3 mm diameter.***

2.7.5 Shrinkage Limit (W_s)

→ (MSBTE- W-09, W-11, S-17)

Q. Define : Shrinkage limit. (W-09, W-11, S-17)

☞ ***Definition of Shrinkage Limit : It is the water content after which further decrease in water content does not cause any decrease in the volume of the soil mass.***

- In other words, it is the minimum water content at which a given soil will be fully saturated.

Syllabus Topic : Plasticity Index

2.7.6 Plasticity Index (I_p)

→ (MSBTE- W-09, S-12, S-13, W-16, S-17)

Q. Define : Plasticity index. (W-09, S-12, S-13, W-16, S-17)

☞ ***Definition of Plasticity Index : It is the range of water content over which a soil exhibits plasticity. It is the numerical difference between the liquid limit and plastic limit. Thus,***

$$I_P = W_L - W_P$$

- Two other indices viz. liquidity index I_L and consistency index I_C are also defined as follows :

2.7.7 Consistency Limit

→ (MSBTE- W-11)

Q. Define : Consistency limit. *(W-11)*

Definition of Consistency Limit : *The water content at which the soil passes from one state to the next is called as consistency limit.*

$$I_L = \frac{W - W_P}{I_P}$$

2.7.8 Consistency Index

Definition of Consistency Index : *It is the ratio of liquid limit (W_L) – natural water content (W) to plasticity index I_P.*

$$\therefore \quad I_C = \frac{W_L - W}{I_P}$$

2.8 Experimental Determination of Consistency Limits

→ (MSBTE- W-10, S-11, W-12, S-13)

Q. How Liquid Limit for soil is determined in laboratory ? *(W-10)*

Q. Sketch and explain liquid limit test on soil. *(S-11)*

Q. Explin determination of liquid limit. *(W-12)*

Q. Write the procedure of determination of L.L. by Casagrande's apparatus with figure. Show the nature of curve and how to obtain L.L. from graph. *(S-13)*

The consistency limits can be determined by laboratory procedures as given below :

Syllabus Topic : Determination of Liquid Limit as per IS Code

2.8.1 Determination of Liquid Limit : IS-2720, Part 5-1985

→ (MSBTE- W-13, W-14, W-17)

Q. Write step by step procedure to determine liquid limit of soil by Casagrande's liquid limit apparatus. *(W-13)*

Q. Explain the experimental procedure to determine liquid limit of soil using suitable sketches. *(W-14,W-17)*

- In the laboratory, standard apparatus known as Casagrande's apparatus, along with the standard tools is used to determine the liquid limit. The apparatus is shown in Fig. 2.8.1.
- It consists of a brass cup mounted on a hard rubber base. The cup can be raised and made to fall on the hard rubber base.
- The distance of the fall can be regulated by screws.
- The fall is made 1 cm before the start of the test.
- There are two types of grooving tools as shown in the Fig. 2.8.1, to be used for different types of soils. For more sandy soils, ASTM tool is used and for clayey soils, Casagrande tool or spatula is used.
- About 120 gm of air dried soil passing through IS sieve of 425 micron is taken and mixed with water such that the soil attains a putty like consistency.
- A portion of the paste is placed in the cup and is levelled so as to have a maximum depth of 1 cm.
- A groove is cut in the soil placed in the cup by using appropriate grooving tool.
- In cutting the groove, the tool is drawn through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.
- The handle is rotated at the rate of 2 revolutions per second and the number of blows required to close the groove for a distance of 1.25 cm is noted.
- The groove should close by flow and not by slippage of soil.
- Then about 10 gm of soil near the groove is taken to determine its water content. By altering the water content of the soil and repeating the procedure, 4 to 5 readings of water content in the range to 10 to 40 blows are obtained.

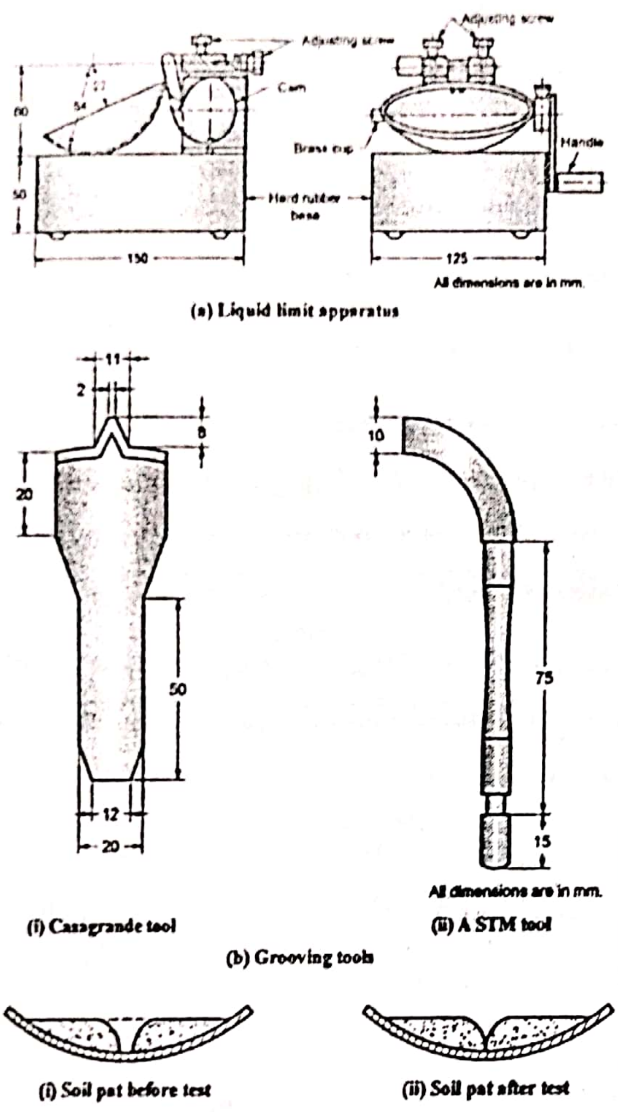


Fig. 2.8.1 : Casagrande apparatus

- A graph is then plotted between number of blows on a logarithmic scale and water content on a natural scale as shown in Fig. 2.8.2.

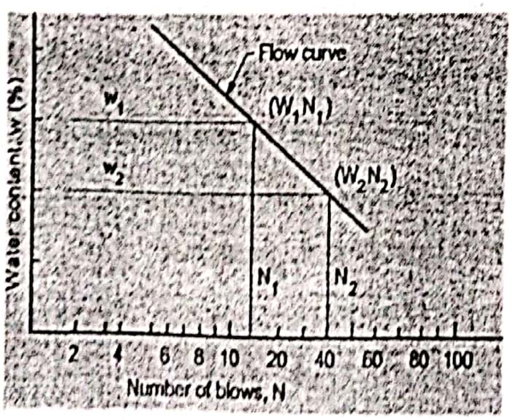


Fig. 2.8.2 : Flow curve

- It will be seen that the semi-logarithmic plot is a straight line whose equation is,

$$W_1 - W_2 = I_f \log_{10}\left(\frac{N_2}{N_1}\right)$$

- Where W_1 and W_2 are water contents corresponding to blows N_1 and N_2. I_f is the slope of the line and it is called the flow index. The line is called the flow curve.
- The liquid limit is then determined by reading the water content corresponding to 25 blows, on the flow curve.

Syllabus Topic : Determination of Plastic limit as per IS code

2.8.2 Determination of Plastic Limit : IS–2720, Part 5 – 1985

→ **(MSBTE- W-12, S-13, W-14, S-15)**

> **Q.** Explain determination of plastic limit of soil. ***(W-12, S-13)***
>
> **Q.** Explain the determination of plastic limit of given soil sample. ***(W-14)***
>
> **Q.** Explain in steps laboratory method to determine plastic limit of soil sample as per IS 2720. ***(S-15)***

- About 15 gm of air dried soil passing through IS sieve 425 micron is taken for plastic limit determination and is mixed with a sufficient quantity of water which would enable the soil mass to become plastic enough to be easily shaped into a ball.
- A portion of the ball is taken and rolled on a glass plate with the palm of the hand into a thread of a uniform diameter.
- When a diameter of 3mm is reached, the soil is remoulded into a ball.
- The process of making the thread and remoulding is continued till the sample at a diameter of 3 mm just starts crumbling.

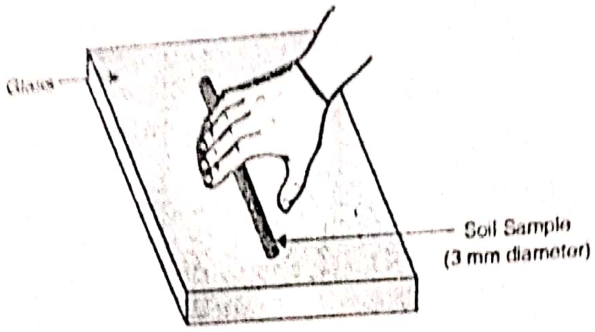
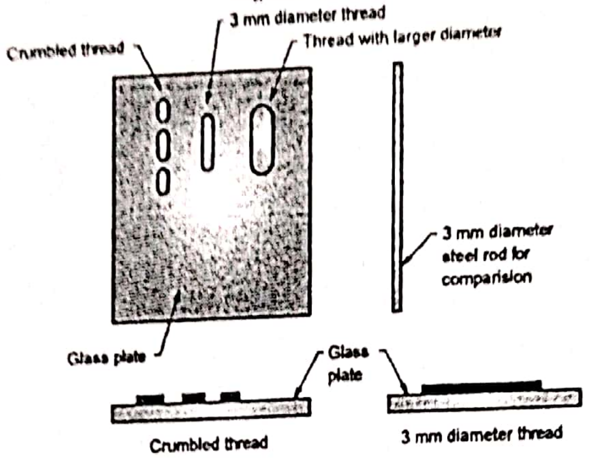


Fig. 2.8.3(a)



(b)

Fig. 2.8.3 : Plastic limit determination

- Some of the crumbled pieces are then taken for water content determination by oven drying method.
- The test is repeated twice more with fresh samples and the average of the three values is taken as the plastic limit.
- A steel rod of 3 mm diameter kept by the side of the glass plate helps in comparing the thread diameter easily.
- The test is shown in Fig. 2.8.3.

Syllabus Topic : Determination of Shrinkage Limit as per IS code

2.8.3 Determination of Shrinkage Limit : IS – 2720, Part 6- 1972, Reaffirmed 1975

- About 30 gm of soil passing 425 micron sieve is taken in an evaporating dish.
- The soil is mixed with sufficient quantity of water to bring the soil to a consistency that it may flow.
- The soil mixture is placed in the shrinkage dish in three equal quantities so as to fill the dish. The excess soil is removed and the dish is weighed with the soil.
- The soil pat is allowed to dry till the colour changes from dark to light.
- The dish is placed in the oven at 110° C till its weight becomes constant.
- The shrinkage dish is weighed with dry sample and the volume of dry pat of soil is measured by displacement of mercury.
- The shrinkage limit is calculated as shown below :
- Fig. 2.8.4 shows the arrangement for shrinkage limit test.
- Fig. 2.8.5 shows the phase diagrams.

Referring to Fig. 2.8.2.

- Fig. 2.8.5(a) represents the soil sample in plastic state which fills the container of known volume V_1 and weighs W_1.
- As the sample is gradually dried, the water content at a certain stage becomes equal to the shrinkage limit as in Fig. 2.8.5(b).
- At this point the volume is decreased to V_2, the corresponding weight being W_2. The sample is in perfectly saturated state.
- Beyond shrinkage limit, sample continues to dry and its weight continue to reduce. But its volume remains V_2 as now the decrease in weight is due to loss of water from the voids.
- When there is no further loss in weight, the sample is completely dry and its weight is W_S while volume is V_2, as weight of air is negligible. See Fig. 2.8.5(c)
- The shrinkage limit is the water content at situation of Fig. 2.8.5 (b).

$$w_s = \frac{W_w}{W_s} \times 100$$

$$W_w = W_1 - W_s - (V_1 - V_2)\gamma_w$$

Scanned with CamScanner

$$\therefore w_s = \frac{(W_1 - W_s) - (V_1 - V_2)\gamma_w}{W_s} \times 100$$

Thus by measuring the starting weight and volume and the dry weight and volume, shrinkage limit w_s can be calculated.

Here γ_w is the limit weight of water.

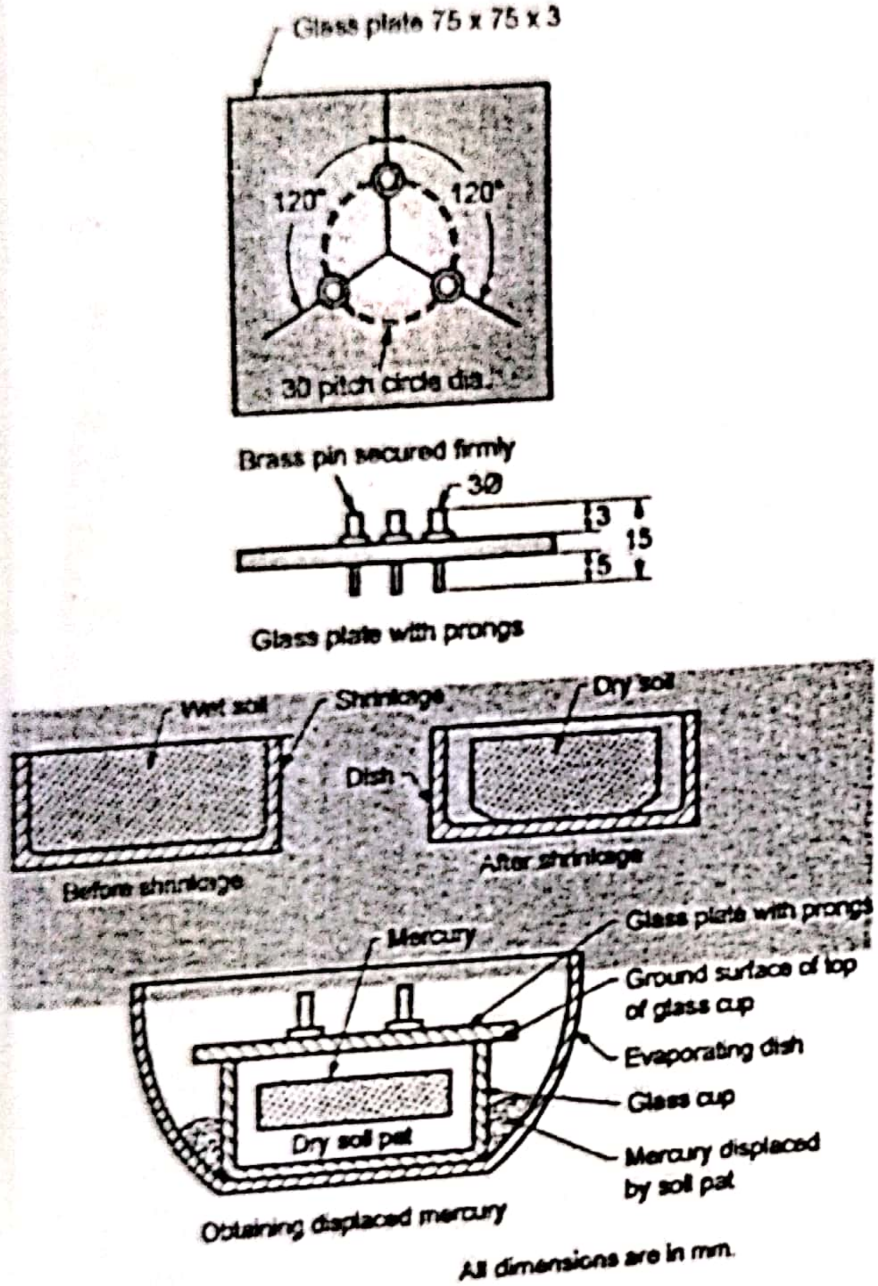
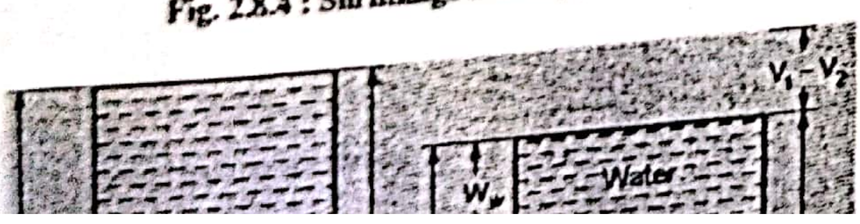
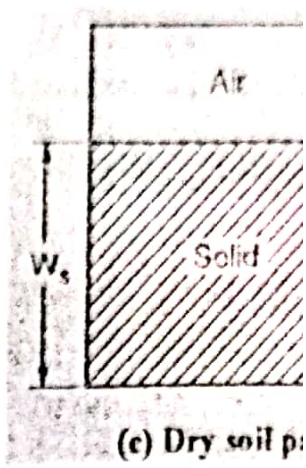


Fig. 2.8.4 : Shrinkage limit apparatus





(c) Dry soil p

Fig. 2.8.5 : Calculation of sh

2.8.4 Differentiation betwe and Shrinkage Limit

Q. Differentiate between Plastic Limit. (W-10)		
No.	Plastic limit	
1.	It is the water content at which soil will just begin to crumble when rolled into a thread of approximately 3 mm diameter.	It cont decr does decr mass
2.	It is water content separating liquid and plastic phase.	It sepa solid

Syllabus Topic : Particle Si

2.9 Particle Size Distrib

- In the combined analysis, dry soil is first analysed by sieving and then the very fine soil is analysed by hydrometer or pipette method by mixing it with water.

Syllabus Topic : Mechanical Sieve Analysis as per IS Code

2.9.1 Mechanical Sieve Analysis as per IS Code : IS 2720, Part 4 – 1985

→ (MSBTE- S-11, S-12, W-12, W-13, S-14, W-14, W-16, W-17, S-18)

> Q. Why do we carry out mechanical analysis of soil. *(S-11)*
> Q. Explain laboratory procedure for mechanical sieve analysis of soil. *(S-12, W-12)*
> Q. Write the procedure of mechanical Sieve analysis for fine grained soil. *(W-13)*
> Q. A 5 kg soil sample is given to you, explain how will you carryout mechanical sieve analysis as per IS code method. *(S-14)*
> Q. Explain the procedure of mechanical sieve analysis for grading of soil using sketches. *(W-14)*
> Q. Explain the mechanical sieve analysis of soil. *(W-16, W-17, S-18)*

- The size of particle other than cube or sphere cannot be universal which is not defined by single linear dimension and particle size depends upon dimension and hence to determine size of particle mechanical analysis is carried out.
- Sieves are wire screens having square opening. According to IS 460-1962 (Revised) the sieve number is the mesh width expressed in mm for large sizes and in microns for small sizes.
- The 4.75 mm sieve separates the soil into 2 parts. The fraction larger than 4.75 mm is called the coarse fraction. The fraction less than 4.75 mm are called as **fine fraction**.
- This fraction is analysed by the following series of sieves : 100 mm, 63 mm, 20 mm, 10 mm and 4.75 mm sieves.
- The sieves are arranged in descending sizes from top to bottom. A weighed, dry soil sample is put onto the top sieve.
- Generally 1000 gm sample is analysed. The top sieve is covered with lid and the bottom sieve has a pan below it.
- The entire assembly is then shaken either manually with careful up-down and circular motion or in a machine known as sieve shaker. Usually ten minutes of shaking is sufficient.
- The soil retained on each sieve is then weighed and the weight is recorded. The soil collected in the pan is subjected to further analysis.
- For this a series of 4.75 mm, 2.4 mm, 1.2 mm, 600μ (μ means microns), 425μ, 300μ, 150μ and 75μ sieves is used.
- These sieves are arranged and the sample from the pan is shaken for another ten minutes. The weight of soil retained on each sieve is then recorded.
- The soil particles passing through the 75μ sieve are collected in the pan.
- If the amount is significant, it is mixed with water for hydrometer or pipette analysis.
- A table is prepared, in which "percentage finer" and "cumulative percentage finer" particles corresponding to each sieve size are worked out and plotted on a semi-log graph paper.

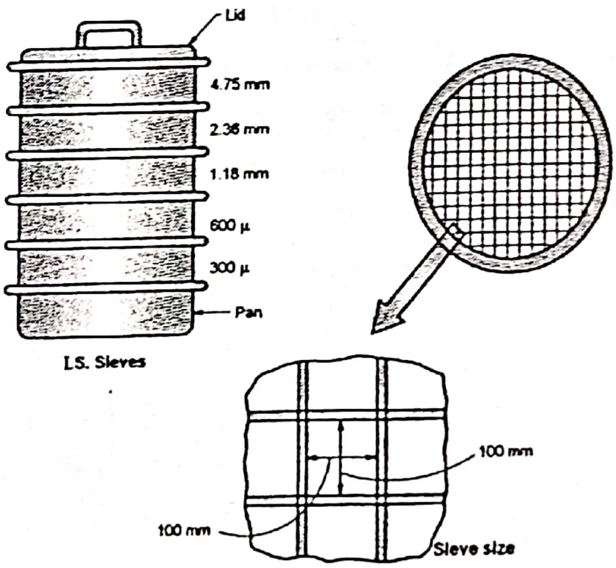


Fig. 2.9.1 : Sieve analysis

- This graph gives particle size distribution curve, which determines the characteristics of the soil. The sieves are shown in Fig. 2.9.1.

Syllabus Topic : Particle Size Distribution Curve

2.9.2 Particle Size Distribution Curve

→ (MSBTE- S-10, S-13, W-14, W-17)

Q. What is particle size distribution curve. *(S-10, W-14, W-17)*

Q. Write the significance of particle size distribution curve and show typical curves for well graded, poorly graded and uniformly graded soils. *(S-13)*

- As said earlier, the cumulative percentage of a particular size of particles passing through that size of sieve opening is worked out.
- It is plotted against the size in log scale. It is drawn on semi-log graph paper, because the particle size may range from a few microns to a few hundred mm.
- Hence, on ordinary scale, a very long graph paper would be required which would be impractical.
- Thus "cumulative percentage finer than" or "cumulative percentage passing through" is plotted on Y-axis in natural scale and the corresponding sieve sizes are plotted on X-axis in logarithmic scale.
- **The resulting graph is known as particle size distribution curve.** This curve forms one of the important index properties of the soil. Fig. 2.9.2 shows typical curves for well graded, uniformly graded and gap graded soils.

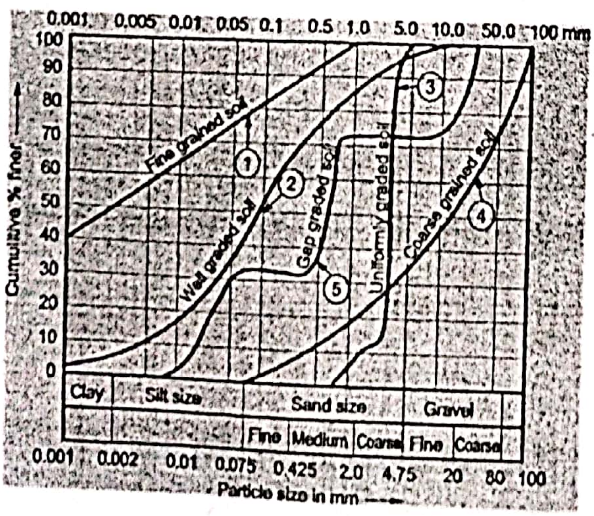


Fig. 2.9.2 : Particle size distribution curves

2.9.2.1 Use of Particle Size Analysis

→ (MSBTE- S-10, S-11, W-11, S-13)

Q. State the use of particle size analysis of soil. *(S-10, S-11, S-13)*

Q. State any four uses of particle size distribution curve. *(W-11)*

The size of particles helps in determining the properties of soil like permeability which is greating released size of grains. Therefore it is important in predicting behaviour of soil and finding its suitability in various construction activities.

Particle size distribution curve is used for :

(i) Determining properties of soil.

(ii) It is also helps in determining behaviour of soil.

(iii) To determine suitability of soil.

(iv) To determine correct measure / methods according necessity.

Syllabus Topic : Effective Diameter of Soil

2.9.3 Effective Diameter of Soil

→ (MSBTE- W-10, S-12, S-13, W-13, S-15, W-15)

Q. What is meant by effective diameter of soil ? *(S-13)*

Q. Define Effective diameter. *(W-13)*

Q. Define : Effective size. *(W-10, S-12, S-15)*

Q. Define with formula D_{10}, D_{30} and D_{60} wit grain size distribution. *(W-15)*

- Since soil contains particles of many different diameters, it is necessary to choose some diameter as representative of the soil.
- This diameter is called the effective diameter for a soil type.

***Definition of Effective Size :** The size D_{10} is called as effective size or effective diameter of soil it represents a size in mm such that 10% of particles are finer than that size.*

- The diameter D_{10} which represents the size for which the given soil contains 10% particles finer than this size. Note that D_{10} size does not mean 10 mm size particles.

- D_{10} will vary for different soils. Suppose a soil contains 10% particles by weight which is smaller than 63 mm, then D_{10} for that soil is 63 mm. This is the method of designating different particle sizes e.g. D_{50} means that size for which the total smaller particles in the given soil are 50%. D_{50} does not mean 50 mm particles.

Syllabus Topic : Uniformity Coefficient

2.9.4 Uniformity Coefficient (C_u)

→ (MSBTE- W-10, S-12, W-13, S-14, S-15)

> Q. Define : Uniformity co-efficient.
> (W-10, S-12, W-13, S-14, S-15)

- It is defined as the ratio of D_{60} size to D_{10} size for a given soil.

$$C_u = \frac{D_{60}}{D_{10}}$$

e.g. D_{60} size for a soil is 2.48 mm and D_{10} is 1.24 mm, then C_u is,

$$C_u = \frac{2.48}{1.24} = 2$$

- It the soil is uniformly graded the coefficient C_u is nearly unity.

Syllabus Topic : Coefficient of Curvature

2.9.5 Coefficient of Curvature (C_c)

→ (MSBTE- W-10, W-13, S-14, S-15)

> Q. Define : Coefficient of Curvature.
> (W-10, W-13, S-14, S-15)

It represents the shape of the particle size distribution curve. It is given by,

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$$

C_c = 1 to 3 for well graded soil

C_c > 4 for well graded gravel

C_c > 6 for well graded sand.

Syllabus Topic : Well Graded and Uniformly Graded Soils

2.9.6 Well-Graded and Uniformly Graded Soils

→ (MSBTE- W-08, W-13, S-14, S-15)

> Q. What do you mean by well-graded and uniformly graded soil. Give the values of coefficient of curvature for each of them. *(W-08)*
> Q. Define (i) Well graded soil (ii) Poorly graded soil (ii) Gap graded soil. *(W-13)*
> Q. Explain well graded soil. *(S-14)*
> Q. Well graded and uniformly graded soil with the help of particle size distribution curve. *(S-15)*

> ☞ ***Definition of well-graded soil :*** *If a soil contains grains of all sizes in significant amount, then it is called a well-graded soil.*

- Such a soil will give a particle size distribution curve which is S-shaped. See Fig. 2.9.2 graph.
- A well graded soil is considered structurally superior as it can be compacted to a very high density. Because of presence of all sizes, voids between bigger particles are filled with smaller particles.
- Voids between smaller particles are filled with still smaller particles and so on. Hence it will give a high compacted density.
- A **uniformly graded soil** contains particles almost of only one size. See graph B in Fig. 2.9.2. Because of this, the compacted density that can be achieved is much lower for this type of soil. Uniformly graded soil is also called poorly graded soil. The value of Cu is nearly unity and for well graded soil it is in between 1 - 3.
- Another type called **gap graded soil** contains not a single particle of a particular size or some particular sizes. Such a soil is called gap graded soil. See graph C, in Fig. 2.9.2.

Syllabus Topic : Particle Size, Classification of Soils

2.10 Particle Size Classification of Soils

→ (MSBTE- S-09, W-09)

> **Q.** Explain particle size classification of soil. *(S-09)*
> **Q.** State and explain particle size classification. *(W-09)*

The classification of soils according to the predominant or effective sizes is called the particle size classification. The various terms used in this classification are as follows :

1. Clay : less than 2 μ
2. Silt : 2 μ to 75 μ
3. Sand : 75 μ to 4.75 mm
4. Gravel : 4.75 mm to 80 mm
2. Pebbles : 80 mm to 300 mm
6. Boulders : More than 300 mm

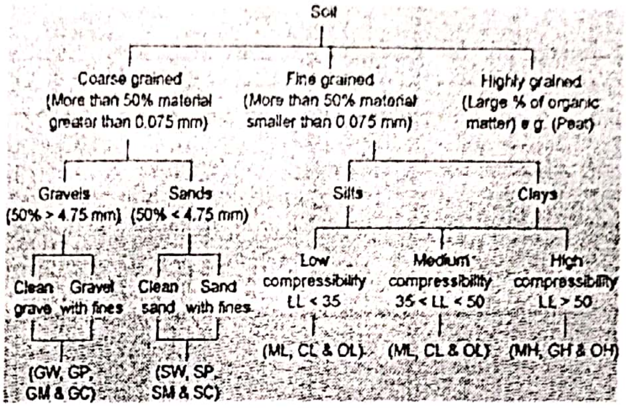
Syllabus Topic : I.S. Classification of Soil

2.10.1 IS Classification of Soil

→ (MSBTE- S-08, W-09, W-10, W-11, S-12, W-12, W-13, S-16, W-16, S-17)

> **Q.** What do you mean by coarse grained soil fine grained soil ? State their sub-divisions as per ISCS. *(S-08, S-16)*
> **Q.** State and explain IS classification of soil. *(W-09, S-17)*
> **Q.** What does GW and SP stands for ? *(W-10)*
> **Q.** Draw Indian Standard Plasticity Chart and classify soil accordingly. *(W-10, W-16)*
> **Q.** Classify the soil into subdivisions according to indian standard code. *(W-11)*
> **Q.** Discuss IS classification of soil as per 1498. (Show chart of it.) *(S-12, W-12, W-16)*
> **Q.** State its acceptance criteria for soft murum and hard murum. *(S-12)*
> **Q.** Explain Casagrandes plasticity chart with figure and how the soil classification is done according to the chart. *(W-13)*
> **Q.** State meaning of following symbols. GW, GC and SP, SM. *(S-16)*

– The Indian Standard Soil Classification System (ISSCS) classifies the soil with the help of a prefix indicating the particle size and a suffix indicating grading or plasticity of the soil. The different suffixes are give in Table 2.10.1.



Where,
G → Gravel,
M → silt
O → Organic silt
W → Well graded
P → Poorly graded
I → Medium compressibility
L → Low compressibility
H → High compressibility
S → Sand
C → Clay

Hence,
GW → Well graded gravel
GM → Well graded silt
SW → Well graded sand
SP → Poorly graded land

Table 2.10.1 : Suffixes and prefixes

Soil type	Prefix	Subgroup	Suffix
Gravel	G	Well graded	W

Soil type	Prefix	Subgroup	Suffix
Sand	S	Poorly graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
Organic	O	$w_L < 35$ per cent	L
		$35 < w_L < 50$	I
Peat	Pt	$50 < w_L$	H

- A plasticity chart is used as a help in classification. It is the plot of liquid limit against plasticity index. Fixed lines are printed on the chart, dividing it in to different areas.
- Liquid limit W_L and plasticity index Ip is determined for the given soil and the point is plotted on the plasticity chart.

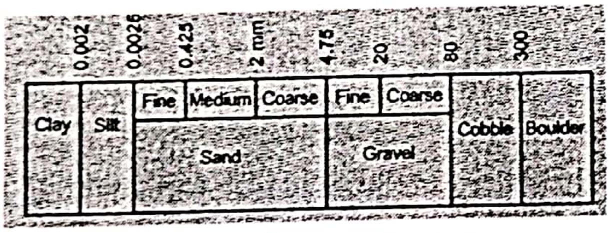


Fig. 2.10.1 : I.S. classification of soil

- If it falls in a particular region, it is classified as that soil type. Plasticity chart is shown in Fig. 2.10.1.

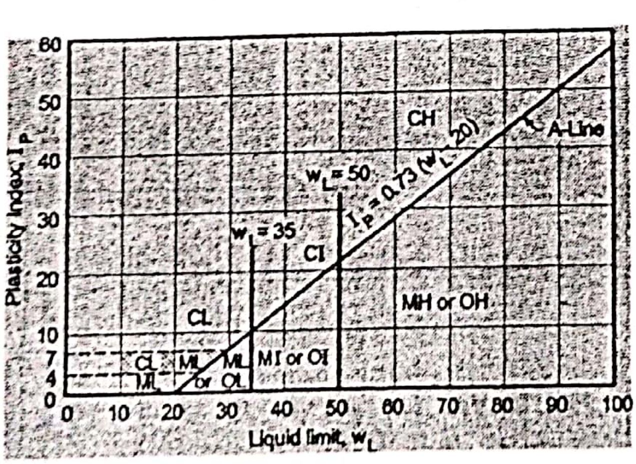


Fig. 2.10.2 : Plasticity chart

- Acceptance criteria for soft murum is in between 4-8 and for hard murum it is in between 0-4.

2.10.2 Plasticity Chart

A Casagrande devised a chart useful for identifying and classifying fine grained soils. The basic of classification is the relationship between liquid limit $[W_L]$ and the plasticity index $[I_P]$. A line called 'A-line' is drawn diagonally across the chart. The area above the A-line represents inorganic clay and that below A-line represents slit and organic soils.

The equation of A-line is : $I_p = 0.73\ [W_L - 20]$

Majority of Indian black cotton soils lie along a band above the A-line. The plot of some of the black soils is also found to lie below the A-line. Care should be taken in classification of such soils.

Using plasticity chart soil are classified as follows :

(i) For silt and clays L.L. less than and equal to 50% soil are classified as ML, CL, OL

(ii) For silt and clays L.L. greater than 50% soil are classified as MH, CH, OH.

2.11 Solved Problems

Ex. 2.11.1

A soil has bulk unit wt. of 3.02 gm/cc when water content was 15%. Calculate water content of soil partially dries to a density of 2.89 gm/cm^3. Calculate specific gravity of soil.

Soln. :

Given : $\gamma = 3.02$ gm/cc, $\gamma' = 2.89$ gm/cm^3,

$w = 15\ \% = 0.15,\ w' = ?,\ G = ?$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{3.02}{1+0.15} = \frac{3.02}{1.15} = 2.62 \text{ gm/cc}$$

$$\gamma_d = \frac{\gamma'}{1+w'} \Rightarrow 2.62 = \frac{2.89}{1+w}$$

$$\therefore \quad 1 + w = \frac{2.89}{2.62}$$

$$w = 1.103 - 1 = 0.10\ \%$$

$$\therefore \quad G_m = \frac{\gamma}{\gamma_w} = \frac{3.02}{9.81}$$

$$G_m = 3.02$$ ...Ans.

Ex. 2.11.2

A soil sample has dry density of soil 1.75 g/cm^3. What is its dry unit weight. Find bulk density if unit weight is 19.8 kN/m^3.

Soln. :

Given : $\rho_d = 1.75$ g/cm^3 , $\rho = ?$

$$\gamma_d = \rho_d \times g$$

$$= 1.75 \times 9.81 = 17.17 \text{ kN/m}^3$$

$$\rho = \frac{\gamma}{g} = \frac{19.80}{9.81}$$

$$\rho = 2.02 \text{ g/cm}^3 \quad \text{...Ans.}$$

Ex. 2.11.3

Calculate void ratio and dry density of a soil sample having porosity 35% and specific gravity 2.67.

Soln. :

$$e = \frac{n}{1-n} = \frac{0.35}{1-0.35}$$

$$e = 0.538$$

$$\gamma_d = \frac{\gamma_w \cdot G}{1+e} = \frac{1 \times 2.67}{1+0.538}$$

$$\gamma_d = 1.735 \text{ gm/cc} \quad \text{...Ans.}$$

Ex. 2.11.4

A soil sample has porosity of 25% and specific gravity of 2.67. Calculate void ratio and dry density.

Soln. :

$$e = \frac{n}{1-n} = \frac{0.25}{1-0.25}$$

$$e = 0.33$$

$$\gamma_d = \frac{\gamma_w \cdot G}{1+e} = \frac{1 \times 2.67}{1+0.33}$$

$$\gamma_d = 2.007 \text{ gm/cc.} \quad \text{...Ans.}$$

Ex. 2.11.5

The bulk density of a soil is 1.6 gm/cc. If the specific gravity of soil solids is 2.7 determine the void ratio with the assumption that the soil is perfectly dry. What will be the void ratio if the sample is assumed to have a water content of 17% ?

Soln. :

Given : $\gamma = \gamma_d = 1.6$ gm/cc, $e = ?$, $G = 2.7$, $e = ?$ if $w = 17\%$

$$\gamma_d = \frac{\gamma_w \cdot G}{1+e}$$

$$1.6 = \frac{1 \times 2.7}{1+e}$$

$$e = \frac{2.7}{1.6} - 1$$

$$e = 0.69 \approx 0.7$$

Using $$\gamma_d = \frac{\gamma}{1+w}$$

$$1.6 = \frac{\gamma}{1+0.17}$$

$$\gamma = 1.17 \times 1.6 = 1.872 \text{ gm/cc}$$

$$e = \frac{\gamma_w \cdot G(1+w)}{\gamma} - 1$$

Using $$\gamma = \frac{\gamma_w \cdot G \cdot (1+w)}{1+e} = \frac{\gamma_w \cdot G \cdot (1+w)}{\gamma} - 1$$

$$= \frac{1 \times 2.7(1+0.17)}{1.872} - 1 = 1.68 - 1$$

$$e = 0.687 \approx 0.7 \quad \text{...Ans.}$$

Ex. 2.11.6

The density of soil sample is 2000 kg/m^3 and its water content is 16%. Determine its dry density, void ratio porosity and degree of saturation.

Assume G = 2.7, γ_w = 10 kN/m^3.

Soln. :

Given : $\gamma = 2000$ kg/m^3 = 2 gm/cc, $\gamma_d = ?$

$w = 16\% = 0.16$, $e = ?$ $G = 2.7$, $n = ?$

$\gamma_w = 10$ kN/m^3 = 1 gm/cc, $\delta = ?$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{2}{1+0.16} = 1.724 \text{ gm/cc}$$

Also, $$\gamma_d = \frac{\gamma_w \cdot G}{1+e}$$

$$1.724 = \frac{1 \times 2.7}{1+e}$$

$$e = \frac{2.7}{1.724} - 1$$

$$e = 0.566$$

$$n = \frac{e}{1+e} = \frac{0.566}{1+0.566} = 0.3614$$

$$n = 36.14\ \%$$

$$G \cdot w = S.e.$$

$$S = \frac{G \cdot w}{e} = \frac{2.7 \times 0.16}{0.566} = 0.7632$$

$$S = 76.32\ \% \quad \text{...Ans.}$$

Ex. 2.11.7

The porosity of soil sample is 38% and specific gravity of solid is 2.7. Calculate [a] Void ratio, [b] Dry unit weight.

Soln. :

Given : $n = 38\,\% = 0.38,\ e = ?,\ G = 2.7,\ \gamma_d = ?$

$$e = \frac{n}{1-n} = \frac{0.38}{1-0.38}$$

$$e = 0.61$$

$$\gamma_d = \frac{\gamma_w \cdot G}{1+e} = \frac{1\times 2.7}{1+0.61}$$

$$\gamma_d = 1.67\ \text{gm/cm}^3 \qquad \text{...Ans.}$$

Ex. 2.11.8 S-17

A saturated clayey sample weighing 1540 gm, weight 1100 gm after over drying. If its dry density is 1350 kg/m^3. Determine its content, void ratio and specific gravity. Assume G = 2.70 and γ_w = 10 kN/m^3.

Soln. :

Given : $W = 1540,\ W = ?,\ W_s = 1100\ \text{gm},\ e = ?,$

$\gamma_d = 1350\ \text{kg/m}^3 = 1.35\ \text{gm/cc},\ G_m = ?$

$$\gamma_d = \frac{W_s}{V} = \frac{1100}{1.35} = 814.81\ \text{cc}$$

$$\gamma = \frac{W}{V} = \frac{1540}{814.81} = 1.890\ \text{gm/cc}$$

Now, $$\gamma_d = \frac{\gamma}{1+W}$$

$$1.35 = \frac{1.890}{1+W}$$

$$W = \frac{1.890}{1.35} - 1 = 0.4 = 40\ \%$$

OR $$W_w = W - W_s = 1540 - 1100 = 440\ \text{gm}$$

Ex. 2.11.9 S-18

Calculate void ratio, porosity and (mass of bulk density 1.76 specifi and water content 30 %.

Soln. :

Given : $\gamma = 1.76\ \text{gm/cc},\ e = ?,$

$W = 30\% = 0.30,\ \delta = ?$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{1.7}{1+(}$$

$$\gamma_d = 1.353\ \text{gm/cc}$$

$$\gamma_d = \frac{G\cdot\gamma_w}{1+e}$$

$$1.353 = \frac{2.7\times 1}{1+e}$$

$$e = \frac{2.7}{1.353} - 1 = 0$$

$$e = 99\ \%$$

$$e = \frac{n}{1-n}$$

$$0.99 = \frac{n}{1-n}$$

$$0.99 - 0.99\,n = n$$

$$0.99 = 1.99\,n$$

$$n = 0.49$$

$$n = 49\ \%$$

Ex. 2.11.10

A soil sample has porosity of 35% Calculate its void ratio, dry density saturation is 50 % and 100 %.

$$\gamma_d = \frac{G \cdot \gamma_w}{1+e} = \frac{2.67 \times 1}{1+0.538}$$

$$\gamma_d = 1.736 \text{ gm/cc}$$

a) when s = 50 % = 0.5

$$\gamma = \frac{(G + e \cdot s)\gamma_w}{1+e} = \frac{(2.6 + 0.538 \times 0.5) \times 1}{1+0.538}$$

$$\gamma = 1.86 \text{ gm/cc}$$

b) when s = 100 % = 1 i.e. fully saturated

$$\gamma = \frac{(G + e \cdot s)\gamma_w}{1+e} = \frac{(2.67 + 0.538 \times 1) \times 1}{1+0.538}$$

$$\gamma = 2.086 \text{ gm/cc}$$ ...Ans.

Ex. 2.11.11

An undisturbed soil sample has volume of 100 cc. It's weight is 190 gms. After drying in oven weight reduced to 160 gms. If specific gravity is 2.68, find water content, void ratio, degree of saturation.

Soln. :

Given : V = 100 cc, w = ?, W = 190 gm, e = ?,

W_s = 160 gm, s = ?, G = 2.68

$$W = \frac{W_w}{W_s} = \frac{30}{160}$$

$$= 0.1875 = 18.75 \%$$

$$\gamma = \frac{W}{V} = \frac{190}{100}$$

$$\gamma = 1.90 \text{ gm/cc}$$

$$\gamma_d = \frac{W_s}{V} = \frac{160}{100} = 1.60 \text{ gm/cc}$$

$$\gamma_d = \frac{G\gamma_w}{1+e}$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 1}{1.60} - 1$$

$$e = 0.675$$

$$S = \frac{W \cdot G}{e} = \frac{0.1875 \times 2.68}{0.675}$$

$$S = 0.744 = 74.4 \%$$ ...Ans.

Ex. 2.11.12

The actual void ratio of sand is 0.6. Here e_{max} = 1.0 and e_{min} = 0.4. Find the relative density of sand.

Soln. :

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{1.0 - 0.6}{1.0 - 0.4}$$

$$= \frac{0.4}{0.6} = 0.6666$$

$$I_D = 66.66\%$$ ...Ans.

Ex. 2.11.13

Porosity of soil sample is 35% and specific gravity is 2.7. Calculate void ratio dry density, saturated density and submerged density.

Soln. :

Given : n = 35 % = 0.35, e = ?, G = 2.7,

γ_d = ?, γ_{sat} = ?, γ_{sub} = ?

$$e = \frac{n}{1-n}$$

$$e = \frac{0.35}{1-0.35}$$

$$e = 0.538$$

$$\gamma_d = \frac{G \cdot \gamma_w}{1+e} = \frac{2.7 \times 1}{1+0.538}$$

$$\gamma_d = 1.756 \text{ gm/cc} = \frac{(2.7+0.538)}{(1+0.538)} \times 1$$

$$\gamma_{sat} = \frac{[G+e]}{[1+e]} \cdot \gamma_w = \frac{(2.7+0.538)}{(1+0.538)} \times 1$$

$$\gamma_{sat} = 2.105 \text{ gm/cc}$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$= 2.105 - 1$$

$$\gamma_{sub} = 1.105 \text{ gm/cc}$$ ...Ans.

Ex. 2.11.14 S-11

A soil sample has a porosity of 42% and specific gravity of solids is 2.70. Determine :

(a) Void ratio

(b) Dry density

(c) Unit weight if soil was 40% saturated.

Soln. :

Given : η = 42 % = 0.42, e = ?, G = 2.70,

γ_d = ?, S = 40 %, γ = ?

when s = 40%

$$e = \frac{n}{1-n} = \frac{0.42}{1-0.42}$$

$$e = 0.724$$

$$\gamma_d = \frac{G \cdot \gamma_w}{1+e} = \frac{2.70 \times 1}{1+0.724}$$

$$\gamma_d = 1.566 \text{ gm/cc}$$

$$\gamma = \frac{(G + s \cdot e)\gamma_w}{(1+e)} = \frac{(2.7 + 0.4 \times 0.724)}{(1+0.724)} \times 1$$

$$\gamma = 1.734 \text{ gm/cc} \quad \text{...Ans.}$$

Ex. 2.11.15

Find porosity of soil whose voids ratio is 1.24.

Soln. : We know the equations,

i) $$\text{Porosity} = \frac{\text{Volume of voids}}{\text{Total volume}} \times 100$$

$$\text{i.e. } n = \frac{V_v}{V} \times 100$$

ii) $$\text{Voids ratio} = \frac{\text{Volume of voids}}{\text{Volume of solids}}$$

$$\text{i.e. } e = \frac{V_v}{V_s}$$

We also know the inter-relations between porosity (n) and voids ratio (e) as given below.

$$e = \frac{n}{1-n} \text{ and } n = \frac{e}{1+e}$$

we get,

$$n = \frac{e}{1+e} = \frac{1.24}{1+1.24} = 0.5536$$

$$\text{i.e. } n = 52.36\ \% \quad \text{...Ans.}$$

i.e. 52.36 % pores are there in the soil.

Ex. 2.11.16

The porosity of a certain soil is 58%, find the void ratio for the soil.

Soln. :

Using the equation $e = \frac{n}{1-n}$

We get,

$$e = \frac{0.58}{1-0.58} = \frac{0.58}{0.42} = 1.38 \quad \text{...Ans.}$$

i.e. the soil have 1.38 time more voids than solids.

Ex. 2.11.17

The porosity of a soil sample is 38 percent and specific gravity of the soil is 2.78. Find voids ratio, dry density, saturated density and submerged density of the soil.

Soln. :

$$\text{We know voids ratio } e = \frac{n}{1-n} = \frac{0.38}{1-0.38}$$

$$\text{i.e. } e = 0.631 \quad \text{...Ans.}$$

$$\text{Dry density} = \gamma_d = \frac{G \cdot \gamma_w}{1+e} = \frac{2.78 \times 1}{1+0.613}$$

$$\text{i.e. } \gamma_d = 1.723 \text{ gm/cm}^3 \quad \text{...Ans.}$$

$$\text{Saturated density} = \frac{G \cdot \gamma_w}{(1+e)} = \frac{(2.78 + 0.613)1}{(1+0.613)}$$

$$\text{i.e. } \gamma_s = 2.013 \text{ gm/cm}^3 \quad \text{...Ans.}$$

$$\text{Submerged density} = \gamma' = \gamma_{sat} - \gamma_w$$

$$\text{i.e. } \gamma' = 2.013 - 1 = 1.103 \text{ gm/cm}^3 \quad \text{...Ans.}$$

Note γ_w = unit weight of water = 1 gm/cm^3

Ex. 2.11.18

An undisturbed soil sample has volume of 60 cm and weight 110 grams. After oven drying its dry weight was found to be 95 grams. Find the water content voids ratio, porosity and degree of saturation for the soil if G = 2.6.

Soln. :

$$\text{Water content} = \frac{W_2 - W_1}{W_1} \times 100 = \frac{110-95}{95} \times 100$$

$$\text{i.e. } W = 12.79\ \% = 0.1579 \quad \text{...Ans.}$$

$$\text{bulk density} = \gamma = \frac{W}{V} = \frac{110}{60} = 1.83 \text{ gm/cm}^3 \quad \text{...Ans.}$$

$$\text{dry density} = \gamma_d = \frac{\gamma}{1+w} = \frac{1.83}{1+0.1579} = 1.58 \text{ gm/cm}^3$$

$$\text{Also dry density} = \gamma_d = \frac{wd}{v} = \frac{95}{60} = 1.58 \text{ gm/cm}^3 \quad \text{...Ans.}$$

$$\text{Voids ratio} = e = \left(\frac{G \cdot \gamma_w}{\gamma_d} - 1\right)$$

$$e = \left(\frac{2.6 \times 1}{1.58} - 1\right) = 0.645 \quad \text{...Ans.}$$

$$\text{Porosity } n = \frac{e}{1+e} = \frac{0.645}{1+0.645} = 0.3921$$

$$\text{i.e. } S = \frac{0.1579 \times 2.6}{0.645} = 0.636 = 0.64$$

$$\text{Degree of saturation} = s = \frac{Wg}{e} \quad \text{...(by se = WG)}$$

i.e. $S = \frac{0.1579 \times 2.6}{0.645} = 0.6365$

i.e. $S = 63.65\ \%$ ...Ans.

Ex. 2.11.19

The volume of a soil sample having natural water content of 42% is 26 cm^3 and its weight is 45 grams. Calculate dry density, voids ratio, porosity and degree of saturation for the soil sample if G = 2.76.

Soln. :

Given :

$$\text{Bulk density} = \gamma = \frac{W}{V} = \frac{45}{26} = 1.73 \text{ gm/cc} \quad \text{...Ans.}$$

$$\text{dry density} = \gamma_d = \frac{\gamma}{1+w} = \frac{1.73}{1+0.42}$$

i.e. $\gamma_d = 1.218$ gm/cc ...Ans.

$$\text{Voids ratio} = e = \left(\frac{G \cdot \gamma_w}{\gamma_d} - 1\right)$$

$$\text{i.e. } e = \left(\frac{2.76 \times 1}{1.218} - 1\right) = 1.266 \quad \text{...Ans.}$$

$$\text{Porosity } n = \frac{e}{1+e} = \frac{1.266}{1+1.266} = 0.5587$$

i.e. $n = 52.87\ \%$ ...Ans.

$$\text{Degree of saturation} = S = \frac{Wg}{e} \text{ (from se = WG)}$$

$$\text{i.e. } S = \frac{0.42 \times 2.76}{1.266} = 0.9156$$

i.e. $S = 91.56$ ...Ans.

Ex. 2.11.20

The porosity of soil sample is 35 percent and the specific gravity of its particles is 2.7. Calculate its voids ratio, dry density, saturated density and submerged density.

Soln. :

$$e = \frac{n}{1-n} = \frac{0.35}{1-0.35} = 0.538$$

$$\gamma_d = \frac{G_{\gamma w}}{1+e} = \frac{2.7}{1.538} = 1.755 \text{ g/cm}^3$$

$(\gamma_w = 1 \text{ g/ml})$

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{2.7+0.538}{1.538} = 2.105 \text{ g/cm}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 2.105 - 1 = 1.105 \text{ g/cm}^3$$

Alternately :

$$\gamma_d = (1-n)\, G_{\gamma w} = 0.65 \times 2.7 = 1.755 \text{ g/cm}^3$$

$$\gamma_{sat} = (1-n)\, G_{\gamma w} + n_{\gamma w}$$

$$= 1.755 + 0.35 = 2.105 \text{ g/cm}^3$$

Ex. 2.11.21

A natural soil deposit has a bulk density of 1.88 g/cm^3 and water content of 5 percent. Calculate the amount of water required to be added to 1 cubic metre of soil to raise the water content to 15 percent. Assume the voids ratio to remain constant. What will then degree of saturation? Assume G = 2.67.

Soln. :

Given :

$\gamma = 1.88$ gm/cm^3 and W = 5 %

$$\therefore \gamma_d = \frac{\gamma}{1+w} = \frac{1.88}{1+0.05} = 1.79 \text{ g/cm}^3$$

Earlier, when W = 5 %

$$W = 0.05 = \frac{W_w}{W_d}$$

Taking one cubic metre of soil, $V = (100)^3 \text{ cm}^3$

$$\therefore W_d = \gamma_d v = 1.79 \times 10^6 \text{ gms.}$$

$$\therefore W_w = 0.05 \times W_d = 0.05 \times 1.79 \times 10^6$$

$$= 8.95 \times 10^4 \text{ gms.}$$

$$\therefore V_w = \frac{W_w}{\gamma_w} = 8.95 \times 10^4 \text{ cm}^2$$

Later, when W = 15%,

$$W_w = W_{Wd} = 0.15 \times 1.79 \times 10^6$$

$$\therefore \quad V_w = 26.85 \times 10^4 \text{ cm}^3 = 26.85 \times 10^4$$

Hence, additional water required to raise the water content from 5% to 15%.

$$= 26.85 \times 10^4 - 8.95 \times 10^4$$

$$= 17.9 \times 10^4 \text{cm}^3 = 179 \text{ liters.}$$

Voids ratio, $e = \dfrac{G\gamma_w}{\gamma_d} - 1 = \dfrac{2.67 \times 10}{1.79} - 1 = 0.49.$

After the water has been added, e remains the same

$$S_r = \frac{WG}{e} = \frac{0.15 \times 2.67}{0.49}$$

$$= 0.817 \times 100 = 81.7\% \qquad \text{...Ans.}$$

Ex. 2.11.22

A soil sample has a porosity of 40%. The specific gravity of solid is 2·70 calculate :

(a) Void ratio

(b) Dry density

(c) Unit weight

Soln. :

Given : $n = 40\% = 0.4;\ G = 2.70.$

(a) We have,

$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$$

(b) $$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7 \times 1}{1+0.667} = 1.62$$

(c) $$e = \frac{wG}{Sr} \qquad (\text{Taking } \gamma_w = 1\ \text{g/cm}^3)$$

$$w = \frac{eS_r}{G} = \frac{0.667 \times 0.5}{2.70} = 0.124$$

$$\gamma_d = 1.62 \text{ (as before)}$$

$$\gamma = \gamma_d(1+W) = 1.62 \times 1.124 = 1.82\ \text{gm/cm}^3$$

Check : $$\gamma = \gamma_w \frac{(G - eS_r)}{1+e}$$

$$= \frac{1\,(2.70 + 0.667 \times 0.5)}{1+0.667} = 1.82$$

(d) When the soil is fully saturated,

$$e = W_{sat}\,6$$

$$\therefore\ W_{sat} = \frac{e}{G} = \frac{0.667}{2.70} = 0.248$$

$$\gamma_{sat} = \gamma_d(1 + W_{sat})$$

$$= 1.62 \times 1.248 = 2.02\ \text{gm/cm}^3$$

Alternately, $\gamma_{sat} = G\gamma_w(1-n) + \gamma_{wn}$

$$= 2.7(1-0.4) + 1 \times 0.4$$

$$= 1.62 + 0.4 = 2.02\ \text{gm/cm}^3$$

Ex. 2.11.23

The volume of a clay sample having a natural water content of 40 % is 22.6 cm³ and its wet weight is 43.50 g. Calculate the degree of saturation of the sample, if G = 2.72.

Soln. : $$\gamma = \frac{W}{V} = \frac{43.5}{25.6} = 1.7\ \text{g/cm}^3$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{43.5}{25.6} = 1.7\ \text{g/cm}^3$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.75}{1.214} - 1 = 1.26$$

$$S_r = \frac{WG}{e} = \frac{0.4 \times 2.75}{1.26}$$

$$= 0.874 = 87.4\ \%. \qquad \text{...Ans.}$$

Ex. 2.11.24

The inside weight of an undisturbed soil sample of volume 60 cm³ is found to be 100 gms on overdrying the weight got reduced to 82.5 gms. If the specific gravity is 2.7. What were the water content and degree of saturation of undisturbed sample.

Soln. :

$$W = \frac{W_w}{W_d} = \frac{100 - 85.5}{85.5} = 0.17,$$

$$\gamma = \frac{W}{V} = \frac{100}{60} = 1.67$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{1.67}{1+0.17} = 1.425\ \text{gm/cm}^3$$

$$\gamma_d = \frac{W_d}{\gamma} = \frac{85.5}{60} = 1.425\ \text{gm/cm}^3$$

$$e = \frac{G\gamma w}{\gamma d} - 1 = \frac{2.7 \times 1}{1.425} - 1 = 0.895$$

$$S_r = \frac{WG}{e} = \frac{0.17 \times 2.7}{0.895}$$

$$= 0.513 = 51.3\ \% \qquad \text{...Ans.}$$

Ex. 2.11.25

A fully saturated sample of soil has volume 20 cc and weight 35 gms. After drying in oven, its weight is 25 gms. Determine

A) % water content

B) Degree of saturation.

Soln. :

Given : V = 20 cc

$$W_w = 35 \text{ gms. (Wet weight)}$$

$$W_d = 25 \text{ gms. (Dry weight)}$$

To find : w, S

i) % water content

$$w = \frac{W_w - W_d}{W_d} \times 100$$

$$= \frac{35 - 25}{25} \times 100$$

$$\therefore w = 40\%$$

ii) Sample is fully saturated, hence volume of voids is equal to volume of water.

$$\therefore S = \frac{V_w}{V_v} \times 100$$

$$\therefore S = 100\%$$

$$\because V_w = V_v$$

Ex. 2.11.26

A fully saturated sample of soil has a volume of 20 cc and weight 36 gms. After drying in the oven, its weight is 24.0 gms. With the help of phase diagram, determine :

(i) Water content

(ii) Void ratio

(iii) Porosity

(iv) Degree of sat

(v) Saturated unit weight

Soln. :

To find : w, e, n, S, γ_{sat}

For fully saturated soil $V_v = V_w$

$$\text{But } \gamma_w = \frac{W_w}{V_v} \text{ where } \gamma_w = 1 \text{ gm/cc}$$

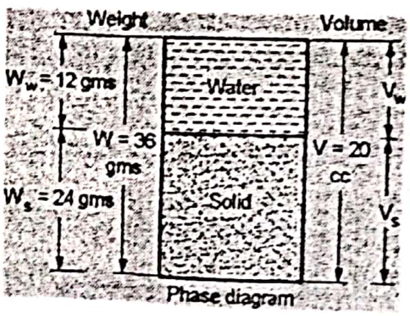


Fig. P.2.11.26 : Phase Diagram

$$\therefore 1 = \frac{W_w}{V_w}$$

$$\therefore W_w = V_w$$

$$\therefore V_w = 12 \text{ cc}$$

$$\therefore W_w = 12 \text{ gms.}$$

$$\therefore V_s = V - V_w$$

$$= 20 - 12 = 8 \text{ cc}$$

$$\therefore V_s = 8 \text{ cc}$$

i) Water content :

$$w = \frac{W_w}{W_s} \times 100 = \frac{12}{24} \times 100$$

$$\therefore w = 50\%$$

ii) Void ratio :

$$e = \frac{V_v}{V_s} = \frac{12}{8}$$

$$e = 1.5$$

iii) Porosity :

$$n = \frac{V_v}{V} \times 100 = \frac{12}{20} \times 100$$

$$n = 60\%$$

iv) Degree of saturation :

$$S = \frac{V_w}{V_v} \times 100 = \frac{12}{12} \times 100$$

$$S = 100\%$$

v) Saturated unit weight :

$$\gamma_{sat} = \frac{W_{sat}}{V} = \frac{36}{20}$$

$$\gamma_{sat} = 1.8 \text{ gm/cc}$$

Ex. 2.11.27 S-08, S-14

An undisturbed sample of soil has a volume of 100 cm^3 and mass of 190 gm on oven drying for 24 hrs the mass is reduced to 160 gm. If the specific gravity of soil grains is 2.68, determine the water content, voids ratio and degree of saturation of the soil.

Soln. :

$$\text{Bulk density : } \gamma = \frac{W}{V} = \frac{190}{100}$$

$$\gamma = 1.9 \text{ gm/cc}$$

Dry density : $\gamma_d = \frac{W_s}{V} = \frac{160}{100} = 1.6$ gm/cc

Now $\gamma_d = \frac{G \cdot \gamma_w}{1+e} \Rightarrow 1.6 = \frac{2.68 \times 1}{(1+e)}$

$\therefore \ e = 0.675$

but $\gamma_d = \frac{\gamma}{1+w} \Rightarrow 1.6 = \frac{1.9}{1+w}$

$\therefore \ w = 18.75\%$

Also $w = \frac{s.e.}{G} \Rightarrow 0.1875 = \frac{s \times 0.675}{2.68}$

$s = 0.744$

Ex. 2.11.28 **S-08, S-10**

Specific gravity test was conducted in the laboratory by pycnothod and recorded observations below. Find average specific gravity of soil.

Determination No.	1	2	3
1) Mass of pycnometer (gm)	652	652	652
2) Mass of pycnometer + soil (gm)	918	950	933
3) Mass of pyc. + soil + water (gm)	1640	1658	1649
4) Mass of pyc. + water (gm)	1470	1470	1470

Soln. :

Determination 1

Given : $W_1 = 652$ gm

$W_2 = 918$ gm

$W_3 = 1640$ gm

$W_4 = 1470$ gm

Specific gravity, $G = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}$

$= \frac{(918 - 652)}{(918 - 652) - (1640 - 1470)}$

$= \frac{266}{[266 - 170]}$

$\therefore \ G = 2.77$

Determination 2

Given : $W_1 = 652$ gm

$W_2 = 950$ gm

$W_3 = 1658$ gm

$W_4 = 1470$ gm

$G = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}$

$= \frac{(950 - 652)}{(950 - 652) - (1658 - 1476)}$

$G = \frac{298}{[298 - 188]}$

$\therefore \ G = 2.709$

Determination 3

Given : $W_1 = 652$ gm

$W_2 = 933$ gm

$W_3 = 1649$ gm

$W_4 = 1470$ gm

$G = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}$

$= \frac{(953 - 652)}{(933 - 652) - (1649 - 1470)}$

$\therefore \ G = \frac{281}{[281 - 179]}$

$\therefore \ G = 2.75$

$\therefore$ Average $G = \frac{2.77 + 2.709 + 2.75}{3}$

$\therefore \ G = 2.743$ ...Ans.

Ex. 2.11.29 **W-08, S-09**

Calculate coefficient of uniformity and coefficient of curvature, if $D_{10} = 0.18$ mm, $D_{30} = 0.37$ mm and $D_{60} = 0.71$ mm, were found from sieve analysis of soil sample.

Soln. :

Coefficient of curvature is obtained by,

$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(0.37)^2}{(0.18 \times 1.07)}$

$\therefore \ C_c = 0.711$ ...Ans.

This indicates that soil sample is not a well graded soil.

Ex. 2.11.30 **S-05, S-13, W-13, S-17**

Given $D_{10} = 150\mu$, $D_{30} = 4.75$ mm, and $D_{60} = 20$ mm. Find coefficient of curvature for soil particle.

Soln. :

Coefficient of curvature,

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(4.75)^2}{(0.15 \times 20)}$$

$$\therefore \quad C_c = 7.52 \quad \text{...Ans.}$$

This indicates a uniformly graded sandy soil.

Ex. 2.11.31 W-07

Find coefficient of uniformity C_u and co-efficient of curvature C_c for a soil particle if D_{10} = 0.15 mm, D_{30} = 0.75 mm and D_{60} = 2 mm.

Soln. :

Coefficient of uniformity,

$$C_u = \frac{D_{60}}{D_{10}} = \frac{2}{0.15}$$

$$\therefore \quad C_u = 13.33 \quad \text{...Ans.}$$

It is sandy soil.

Coefficient of curvature,

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(0.75)^2}{(0.15 \times 2)}$$

$$C_c = 1.87\dot{5} \quad \text{...Ans.}$$

Well graded sandy soil

Ex. 2.11.32 W-09

Field density test by core cutter method was conducted on field and recorded observations as given below :

(i) mass of empty core cutter = 900 gm

(ii) mass of soil + cutter = 3010 gm

(iii) volume of core cutter = 1178 cc

(iv) moisture content = 14.7%

Determine bulk density and dry density in kN/m^3.

Soln. :

$$\text{Bulk density} = \frac{(m_2 - m_1)}{v} = \frac{(3010 - 900)}{1178}$$

$$= 1.79 \text{ kN/m}^3$$

$$\text{Dry density} = \frac{\text{Bulk density}}{1 + w} = \frac{1.79}{1t}$$

$$= 0.114 \text{ kN/m}^3 \quad \text{...Ans.}$$

Ex. 2.11.33 W-09

A soil sample has a porosity of 34% and specific gravity 2.68. Calculate its voids ratio and dry density.

Soln. :

Porosity $\eta = 0.34$

Specific gravity $= 2.68$

$$e = \frac{n}{1-n} = \frac{0.34}{1 - 0.34} = 0.52$$

$$\gamma_d = \frac{\gamma_w \times G}{1 + e} = \frac{1 \times 2.68}{1 + 0.52}$$

$$= 1.76 \text{ gm/cc.} \quad \text{...Ans.}$$

Ex. 2.11.34 W-09

A sieve analysis test was conducted in the laboratory and from particle size distribution curve following observations recorded. Calculate C_u and C_c and classify the given soil.

(i) D_{10} = 0.30 mm

(ii) D_{30} = 1.10 mm

(iii) D_{60} = 2.0 mm

Soln. :

Given :

(i) $D_{10} = 0.30$ mm

(ii) $D_{30} = 1.10$ mm

(iii) $D_{60} = 2.0$ mm

$$C_u = \frac{D_{60}}{D_{10}} = \frac{2.0}{0.30} = 6.67$$

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{1.10^2}{0.3 \times 2.0} = 2.02$$

Ex. 2.11.35 W-08, S-09

A soil sample of volume 160 CC, weights 304 gms, when partially saturated. It weights 269.28 gms when fully dry. Specific gravity of soil is 2.64. Determine porosity, void ratio, water content and degree of saturation.

Soln. :

Given : V = 160 CC ; W = 304 gm

W_s = 269.28 gm ; G = 2.64

$$r = \frac{W}{V} = \frac{304}{160} = 1.90 \text{ gm/cc}$$

$$\gamma = \frac{Ws}{V} = \frac{269.28}{160} = 1.683 \text{ gm/cc}$$

$$\gamma_d = \frac{\gamma}{1+w}$$

$$1.683 = \frac{1.90}{1+w}$$

$$w = \frac{1.90}{1.683} - 1 = 0.1289 = 12.89\%$$

$$\gamma_d = \frac{G\cdot\gamma_w}{1+e}$$

$$1.683 = \frac{2.64\times 1}{1+e}$$

$$e = \frac{2.64}{1.683} - 1 = 0.569$$

$$n = \frac{e}{1+e} = \frac{0.569}{1+0.569} = 0.363$$

$$S = \frac{w\cdot G}{e}$$

$$= \frac{0.1289\times 2.64}{0.569} = 0.598 = 59.80\%$$

Ex. 2.11.36 W-19

A saturated soil sample has a water content of 49 percent. If specific gravity of solids is 2.7, compute the void ratio porosity and the saturated density of soil sample.

Soln. : Given data : $G = 2.7$, $\eta = 49\% = 0.49$

To find : i) Void ratio (e)

ii) Dry density (γ_d)

iii) Saturated density (γ_s)

i) void ratio $e = \frac{n}{1-n} = \frac{0.49}{1-0.49} = 0.96$

ii) Dry density $(\gamma_d) = \frac{G\cdot\gamma_w}{1+e} = \frac{2.7\times 1}{1+0.96} = 1.37 \text{ gm/cm}^3$

iii) Saturated density $\gamma_s = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.7+0.96)\times 1}{(1+0.96)}$

$$= 1.86 \text{ gm/cm}^3$$

Ex. 2.11.37 S-11

An undisturbed sample of soil has a volume of 100 cm^3 and mass of 190 gm on oven drying the mass is reduced to 160 gm. If the specific gravity of soil grains is 2.70, determine the water content, void ratio and degree of saturation.

Soln. : Given data :

$V = 100$ cc $\quad G = 2.70$

$W = 190$ gm $\quad W_s = 160$ gm

$$w = \frac{W_w}{W_s} = \frac{30}{160} = 18.75\%$$

$$\gamma = \frac{W}{V} = \frac{190}{100} = 1.90 \text{ gm/cc}$$

$$\gamma_d = \frac{W_s}{V} = \frac{160}{100} = 1.60 \text{ gm/cc}$$

$$\gamma_d = \frac{G\cdot\gamma_w}{(1+e)}$$

$$e = \frac{G\cdot\gamma_w}{\gamma_d} - 1 = \frac{2.7\times 1}{1.60} - 1$$

$$e = 0.6875$$

$$S = \frac{w\cdot G}{e} = \frac{0.1875\times 2.7}{0.6875} = 0.7363$$

$$S = 73.63\%$$

Ex. 2.11.38 S-10

The porosity of soil sample is 27% and specific gravity of solids is 2.7. Calculate :

(i) Void ratio (ii) Dry density.

Soln. :

$$\eta = 27\% \quad G = 2.67, \; e = ? \; \gamma_d = ?$$

$$e = \frac{n}{1-n} = \frac{0.27}{1-0.27} = 0.37$$

$$\gamma_d = \frac{\gamma_w\cdot G}{1+e}$$

$$= \frac{1\times 2.7}{1+0.27} = 2.13 \text{ gm/cm}$$

Ex. 2.11.39 S-10; S-12

Calculate coefficient of uniformity and coefficient of curvature for a soil sample for which $D_{10} = 0.430$ mm, $D_{30} = 0.790$ mm, $D_{60} = 1.300$ mm.

Soln. :

Given : $D_{10} = 0.430 \quad D_{60} = 1.300 \quad D_{30} = 0.790$

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.300}{0.430} = 3.02$$

Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{D_{10}\times D_{60}} = \frac{(0.790)^2}{0.43\times 1.3} = 1.16$$

Soil is well graded soil.

Ex. 2.11.40 [illegible]

Calculate coefficient of uniformity and coefficient of curvature for a soil sample for which D_{10} = 0.0019 mm, D_{30} = 0.030 mm, D_{60} = 0.49 mm.

Soln. :

Given : $D_{10} = 0.0019$ $D_{60} = 0.49$ $D_{30} = 0.030$

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.49}{0.0019} = 257.8 = 0.257$$

Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(0.030)^2}{0.0019 \times 0.49} = 0.96$$

Ex. 2.11.41 [illegible]

The unit weight of soil sample is 21 kN/m³ and its water content is 18%. Determine its dry unit weight, void ratio porosity and degree of saturation. Assume specific gravity G = 2.72.

Soln. :

$\gamma = 21$ kN/m³, w = 18 %, G = 2.75, γ_d = ?

n = ?, e = ?, s = ?

(i) Dry unit w+ :

$$V_d = \frac{\gamma}{1+w} = \frac{21}{1+0.18} = 17.7966 \text{ kN/m}^3$$

(ii) void ratio :

$$e = \frac{G \cdot v}{V_d} - 1 = \frac{2.75 \times 9.87}{17.1966} - 1 = 0.516$$

$$= 51.6 \%$$

(iii) Porosity :

$$n = \frac{e}{1+e} = \frac{0.516}{1+0.516} = 34.03 \%$$

(iv) Degree of saturation :

$$e.s = w \cdot G$$

$$0.516 \times s = 0.18 \times 2.75$$

$$s = 0.959$$

$$\therefore s = 92.9 \%$$

Ex. 2.11.42 [illegible]

A sieve analysis test was conducted in laboratory and from particle size distribution curve following observations recorded. Calculate co efficient of curvature and co efficient of uniformity for following given data.

(i) D_{10} = 0.32 mm

(ii) D_{30} = 1.25 mm

(iii) D_{60} = 1.98 mm

Soln. :

D_{10} = 0.32 mm, D_{30} = 1.25 mm, D_{60} = 1.98 mm

∴ Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(1.25)^2}{0.32 \times 1.98}$$

$$= 2.46$$

Coefficient of uniformed

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.98}{0.32} = 6.1875$$

Ex. 2.11.43 [illegible]

What is relative density of sand. Calculate relative density for following data of soil sample.

(i) The actual void ratio of sand is 0.62.

(ii) e_{max} = 1.0 and e_{min} = 0.45

Soln. :

e = 0.65, e_{max} = 1.0, e_{min} = 0.45

$$\Sigma d = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$= \frac{1.0 - 0.65}{0.65 - 0.45} = 1.75$$

Ex. 2.11.44 S-12

A fully saturated soil sample has a volume of 25 cc and weight 35 gms. After drying in the oven, its weight becomes 25 gms. With the help of phase diagram determine.

(i) Water content (ii) Voids ratio (iii) Porosity

Soln. :

To find : w, e, n

For fully saturated soil $V_v = V_w$

But $\gamma_w = \dfrac{W_w}{V_w}$

Where $\gamma_w = 1$ gm/cc

$$\therefore 1 = \frac{W_w}{V_w}$$

$$\therefore W_w = V_w$$

$$\therefore V_w = 10 \text{ cc}$$

$$\therefore W_w = 10 \text{ gms}$$

$$\therefore V_s = V - V_w = 25 - 10 = 15 \text{ cc}$$

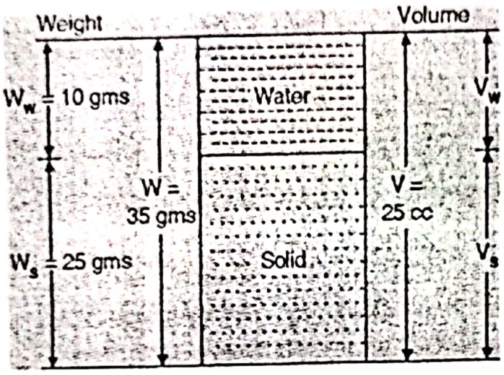


Fig. P.2.11.44

(i) Water content

$$W = \frac{W_w}{W_s} \times 100 = \frac{10}{25} \times 100 = 40\%$$

(ii) Voids ratio

$$e = \frac{V_v}{V_s} = \frac{10}{15} = 0.66$$

(iii) Porosity

$$n = \frac{V_v}{V} \times 100 = \frac{10}{25} \times 100 = 40\%$$

Ex. 2.11.45 S-12

Find coefficient of uniformity Cu and coefficient of curvature C_c for a soil particles of $D_{10} = 0.2$ mm, $D_{30} = 0.8$ mm, $D_{60} = 2$ mm. Also classify and grade the soil.

Soln. :

Coefficient of uniformity $(C_u) = ?$

Coefficient of curvature $(C_c) = ?$

$$D_{10} = 0.2 \text{ mm}$$

$$D_{30} = 0.8 \text{ mm}$$

$$D_{60} = 2 \text{ mm}$$

$$\text{Coefficient of uniformity } C_u = \frac{D_{60}}{D_{10}} = \frac{2}{0.2} = 10$$

Soil is not uniformly graded

$$\text{Coefficient of curvature } C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{0.8^2}{0.2 \times 2} = \frac{0.64}{0.4}$$

$$\therefore C_c = 1.6$$

Soil is well graded sandy soil.

Ex. 2.11.46 S-13, W-13

Specific gravity test was conducted in laboratory by pycnometer method and the recorded observations are as below. Find the specific gravity of soil.

1.	Mass of pycnometer	480 gm
2.	Mass of pycnometer + soil	726.8 gm
3.	Mass of pycnometer + soil + water	1667.6 gm
4.	Mass of pycnometer + full of water	1512.5 gm

Soln. :

Given : $W_1 = 480$ gm

$$W_2 = 726.8 \text{ gm}$$

$$W_3 = 1667.6 \text{ gm}$$

$$W_4 = 1512.5 \text{ gm}$$

$$\text{Specific gravity, } G = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}$$

$$= \frac{(726.8 - 480)}{(726.8 - 480) - (1667.6 - 1512.5)}$$

$$\therefore G = 2.69$$

Ex. 2.11.47 S-14, W-16

The following observations were recorded in a liquid – limit test carried out in Casagrande's apparatus. Determine the liquid limit. The weight of container W_3 is 5 gm.

Number of blows	40	30	20	15	10
Wet weight W_1 (gm)	30.67	32.20	31.30	32.75	30.05
Dry weight W_2 (gm)	22.00	23.00	22.35	23.26	21.44

Soln. :

Number of blows	40	30	20	15	10
Wet weight (W_1 in gm)	30.67	32.20	31.30	32.75	30.05

Dry weight W_2 in gm	22.00	23.00	22.35	23.26	21.44
Water content in %	28.26	28.57	28.59	28.97	28.65

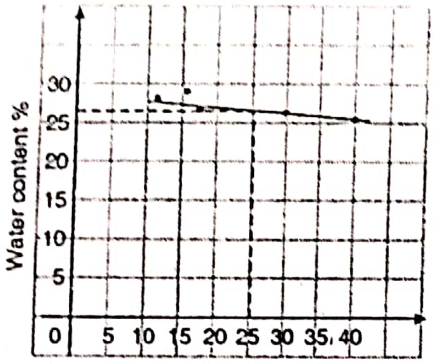


Fig. P.2.11.47 : Number of blows

Please plot above on semilog graph paper and observe corresponding value of 25 number of blows. That will give liquid limit. Here it is near to 26%.

Ex. 2.11.48 W-14

Calculate coefficient of uniformity and coefficient of curvature if D_{10} = 0.43 mm, D_{30} = 1.78 mm and D_{60} = 2.39 mm.

Soln. :

Find coefficient of curvature for soil particle.

1. coefficient of curvature for soil particle

$$\text{Coefficient of curvature} = C_c = \frac{(D_{30})^2}{(D_{10}) \times (D_{60})}$$

$$\text{Coefficient of curvature} = C_c = \frac{(1.78)^2}{(0.43) \times (2.39)} = 3.08$$

2. Coefficient of uniformity

$$\text{Coefficient of uniformity} : \frac{(D_{60})}{(D_{30})} = \frac{2.39}{1.78} = 1.343$$

Ex. 2.11.49 W-15

In a specific gravity test the following data is obtained at room temperature of 30°C.

(i) Mass of pycnometer = 680 gm

(ii) Mass of pycnometer + water = 720.26 gm

(iii) Mass of pycnometer + soil solids + water = 750.36 gm

(iv) Mass of pycnometer + soil solids = 727.25 gm

Determine the specific gravity of soil solids.

Soln. :

Given : $W_1 = 680$, $W_2 = 720.26$, $W_3 = 750.36$, $W_4 = 727.25$

$$\text{Specific gravity, G} = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)}$$

$$= \frac{720.26 - 680}{(720.26 - 680) - (750.36 - 727.25)}$$

$$G = \frac{40.26}{(40.26) - (23.11)} = 2.34$$

Ex. 2.11.50 W-15

The field density of sandy soil was found to be 1800 kg/m³ at a water content of 10%. If the void ratio in the loosest and densest states were 0.75 and 0.47 respectively, determine the value of density index. Also comment about the degree of compaction based on the value of density index.

Soln. :

Given Data :

(i) Field density of sandy soil = 1800 kg/m³ = 1.8 gm/cc

(ii) Water content = 10%

(iii) Void ratio at loosest state = 0.75

(iv) Void ratio at densest t state = 0.47

$$\text{Density index} = I_D = [e_{max} - e] / [e_{max} - e_{min}]$$

$$\gamma_d = \left[\frac{1.8}{(1 + 0.10)}\right] = 1.64$$

$$\gamma_d = \left[\frac{(G \cdot \gamma_w)}{(1 + e)}\right]$$

... use this equation for finding, 'e' natural

$$e = \left[\frac{(G \cdot \gamma_w)}{\gamma_d}\right] - 1$$

$$= \left[\frac{(2.67 \times 1)}{1.64}\right] - 1 \quad \text{...Assume, G = 2.67}$$

$$e = 0.62$$

When soil is loosed state

$e = e_{max}$ ... Hence $I_D = 0$

$$\text{Density index} = I_D = \frac{[e_{max} - e]}{[e_{max} - e_{min}]};$$

$$\text{Density index} = I_D = \frac{[0.75 - 0.62]}{[0.75 - 0.47]} = \frac{0.13}{0.28} = 0.46$$

Comment about the degree of compaction based on the value of density index as the value density index = I_D = 0.46, lies in between 0.35 to 0.62.Hence, the degree of compaction is medium dense.

Ex. 2.11.51 W-16, S-18

The density of a soil sample is 2000 kg/m^3 and its water content is 18%. Determine its dry density, voids ratio, porosity and degree of saturation. Assume G = 2.72, Vw = 10 kN/m^3.

Soln. :

$\gamma = 2000\ kg/m^3 = 2\ gm/cc,\ \gamma_d = ?,\ w = 18\% = 0.18$

$e = ?,\ G = 2.7,\ n = ?,\ \gamma_w = 10\ kN/m^3 = 1\ gm/cc,\ \delta = ?$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{2}{1+0.18} = \frac{2}{1.18} = 1.695\ gm/cc$$

Also,

$$\gamma_d = \frac{\gamma_w G}{1+e} = 1.695 = \frac{1 \times 2.7}{1+e}$$

$$e = \frac{2.7}{1.695} - 1 = 0.593 \qquad \therefore e = 0.593$$

$$n = \frac{e}{1+e} = \frac{0.593}{1+0.593} = 0.372$$

$$n = 37.22\ \%$$

$$G \cdot w = S \cdot e$$

$$S = \frac{G \cdot w}{e} = \frac{2.7 \times 0.18}{0.593} = 0.8195$$

$$S = 81.95\ \%$$

Ex. 2.11.52 W-15

A liquid limit test by Casagrande apparatus gave the following results :

Number of blows	15	21	38	51
Moisture content in %	74.6	68.4	66.7	52.60

Plot the flow curve and find the liquid limit.

Soln. :

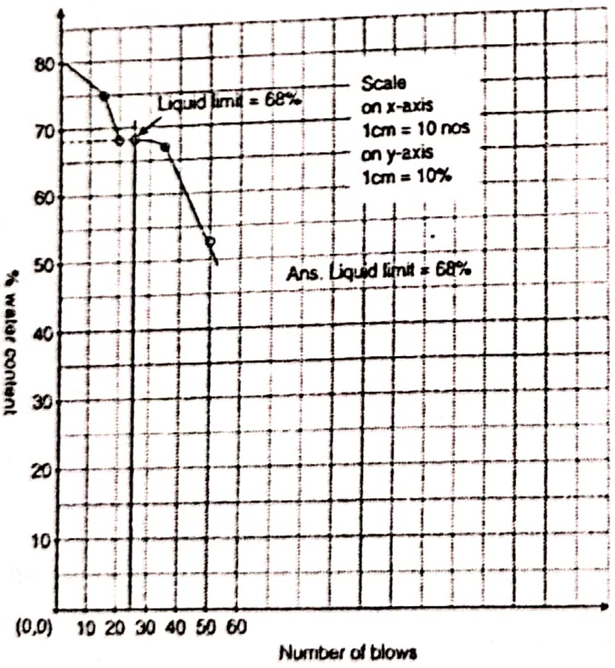


Fig. P.2.11.52

Liquid limit = 68 %

Ex. 2.11.53 S-17

Calculate shrinkage limit for a given soil sample from the following data :

(i) Mass of empty container w_1 = 13 gm

(ii) Mass of container with wet soil w_2 = 43 gm

(iii) Mass of container with dry soil w_3 = 32.3 gm

(iv) Vol. of wet soil v_1 = 20.7 cm^3

(v) Vol. of dry soil Pat v_2 = 10.3 cm^3

Soln. :

Given :

Mass of empty container w_1 = 13 gm

Mass of container with wet soil w_2 = 43 gm

Mass of container with dry soil w_3 = 32.3 gm

$$\text{Vol. of wet soil } v_1 = 20.7\ cm^3$$

$$\text{Volt of dry soil pat } v_2 = 10.3\ cm^3$$

Find shrinkage limits W_S = ?

$$\text{Mass of wet soil} = M = w_2 - w_1$$

$$= 43 - 13 = 30\ gm$$

$$\text{Mass of dry soil} = M = w_3 - w_1$$

$$= 32.3 - 13 = 19.3\ gm$$

By formula $W_S = \{[(M - M_d) - (V - V_d)\gamma w] / M_d\} \times 100$

$$W_S = 0.01554 \times 100$$

$$W_S = 1.554\%$$

Ex. 2.11.54 W-17

A saturated clayey soil weighing 1600 gms weighs 1200 gms after oven drying. If its dry density in 1350 kg/m^3, determine its water content, void ratio, porosity and degree of saturation. Assume G = 2.50 and γ_w = 12 KN/m^3

Soln. :

Given : $W = 1600,,\ W_s = 1200\ gm,$

$$\gamma_d = 1350\ kg/m^3 = 1.35\ gm/cc\ ,$$

Find : $W = ?,\ e = ?,\ G_m = ?,\ \eta = ?$

$$\gamma_d = \frac{W_s}{V} = \frac{1200}{1.35}$$

$$= 888.88\ cc$$

$$\gamma = \frac{W_s}{V} = \frac{1600}{888.88}$$

$= 1.800$ gm/cc

Now, $\gamma_d = \frac{\gamma}{1+W}$

$1.35 = \frac{1.800}{1+W}$

$W = \frac{1.800}{1.35} - 1 = 0.33 = 33\%$

OR

$W_w = W - W_s = 1600 - 1200 = 400$ gm

$W = \frac{W_w}{W_s} \times 100 = \frac{400}{1200} \times 100$

$W = 33.33\%$ ...Ans.

Using $\gamma_d = \frac{G \cdot \gamma_w}{1+e}$

$1.35 = \frac{2.50 \times 12}{1+e}$

$e = \frac{2.50 \times 12}{1.35} - 1$

$e = 1.22$ ...Ans.

$G_m = \frac{\gamma}{\gamma_w} = \frac{1.800}{12}$

$G_m = 0.15$ gm/cc ...Ans.

we know that,

$\eta = \frac{1.22}{1.22+1}$

$= 0.5495 \times 100$

$\eta = 54.95\%$ ...Ans.

S or $S_r = S_r = \frac{2.50 \times 33.33}{1.22} = 0.6829$

$S_r = 68.29\ \%$ ...Ans.

Chapter Ends

□□□

UNIT III

Chapter 3 Permeability and Shear Strength of Soil

Syllabus

3.1 Definition of permeability, Darcy's law of permeability, coefficient of permeability, factors affecting permeability, determination of coefficient of permeability by constant head and falling head permeability tests, simple problems to determine coefficient of permeability. Seepage through earthen structures, seepage velocity, seepage pressure, phreatic line, flow lines, application of flow net,(No numerical problems).

3.2 Shear failure of soil, field situation of shear failure, concept of shear strength of soil, components of shearing resistance of soil – cohesion, internal friction. Mohr-coulomb failure theory, Strength envelope, strength Equation for purely cohesive and cohesion less soils. Direct shear test and vane shear test –laboratory methods.

☞ **Introduction**

As soil is made up of separate particles, the voids or spaces between them are interconnected and it can be viewed as a complex network of irregular tubes. If the voids are larger and more interconnected, the resistance to flow of water is much less than that offered by smaller voids. Thus, even though almost all soils are **pervious** to water, they exhibit different degrees of perviousness. This gives rise to the quantitative idea of **permeability** of soil, which is nothing but the degree of perviousness. This is much important with respect to various structures like dams, there foundations and its settlement.

Syllabus Topic : Definition of Permeability

3.1 Definition of Permeability

→ (MSBTE - W-10, S-11, W-11, W-15, S-16, W-16, S-17)

Q. Define Permeability.
(W-10, W-11, W-15, S-16, W-16, S-17)

Q. Define permeability of soil. (S-11)

☞ ***Definition of Permeability :*** *Permeability of soil is defined as the property of a soil, which permits the seepage of fluids through the interconnected voids under gravity.*

- Quantitatively, permeability can be defined as the speed at which water flows through soil under unit head at unit hydraulic gradient.

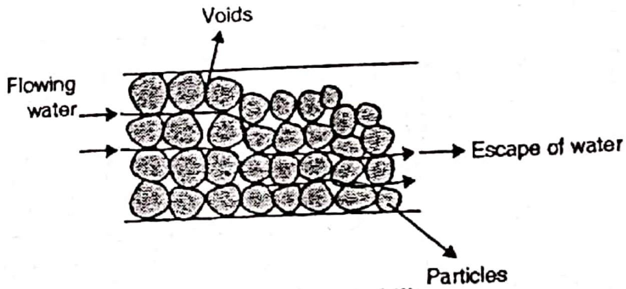


Fig. 3.1.1 : Permeability

3.2 Theory of Permeability

- Much of the important work for analysis of the phenomenon of permeability was done by Darcy.

- He proposed a model where the hydraulic gradient is the main factor affecting permeability.

Syllabus Topic : Darcy's law of permeability

3.2.1 Darcy's Law of Permeability

→ **(MSBTE – S-08, S-09, W-09, S-10, W-10, S-12, W-12, S-13, S-14**

> Q. State Darcy's law of permeability. *(S-08, S-09, W-09, S-10, S-16)*
> Q. State Darcy's law. *(W-10, W-12, S-13)*
> Q. State Darcy's law and explain with formula permeability of soil. *(S-12)*
> Q. Explain Darcy's law of permeability. *(S-14)*

- The flow of water can be of two types. Laminar flow and turbulent flow.
- The flow through the pores of soil due to their small size is almost laminar.
- Assuming the flow to be laminar, Darcy's law can be applied to it. Darcy's law states that for laminar flow the velocity of flow, v is proportional to the hydraulic gradient i.

$$v \propto i \quad \text{or} \quad v = K i$$

K - Constant of proportionality / Darcy's coefficient of permeability

- This equation is known as Darcy's law. The constant of proportionality is known as coefficient of permeability.

Syllabus Topic : Coefficient of Permeability

3.2.2 Coefficient of Permeability

→ **(MSBTE - S-08, W-08, W-15, W-16)**

> Q. Define coefficient of permeability. *(S-08, W-15, W-16)*
> Q. Define coefficient of permeability and state its unit. *(W-08)*

- Consider a soil mass as shown in Fig. 3.2.1. Let A be the cross sectional area of the soil mass, and let L be the length.
- Let there be level difference of water Δh between the water levels at both sides of the soil mass. Then

$$i = \frac{\Delta h}{L}$$

- The discharge through the soil is given by.

$$q = v A = K i A = \text{constant}$$

$$\because \quad i = \frac{\nabla h}{L}$$

Thus discharge is given by,

$$q = A \times v$$

$$q = k i A.$$

$$\because \quad V = ki$$

Where,

q = discharge per - unit time

A = total c/s are of soil mass

$i = \frac{\Delta h}{L}$ = hydraulic gradient

K = Darcy's coefficient of permeability

v = Velocity of flour

L = Length of soil sample

Δh = Differential head of water $(h_1 - h_2)$

☞ **Laminar flow**

> ☞ ***Definition of Laminar flow :*** *The flow in which water particles are moving in a definite path or moving parallel without crossing the path of other particles is called as laminar flow.*

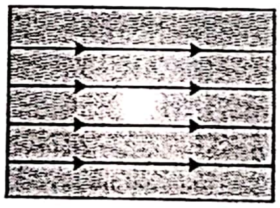


Fig. 3.2.1(a) : Laminar flow

☞ **Turbulent flow**

> ☞ ***Definition of Turbulent flow :*** *The flow in which the particle of water does not follow a definite path while flowing and may cross the parts of each other is called as turbulent flow.*

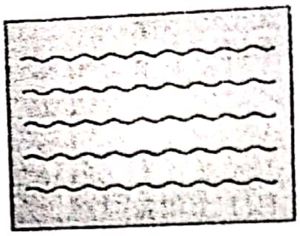


Fig. 3.2.1(b) : Turbulent flow

$$q = K \frac{\Delta h}{L} A = \text{Constant.}$$

- The constant 'K' is known as the coefficient of permeability or simply permeability.
- The coefficient of permeability K can be defined as the velocity of flow under a unit hydraulic gradient through a soil.
- It is a property of the soil. Its units are the same as velocity, i.e. m/s or cm/s.

$$K = \frac{V}{i}$$

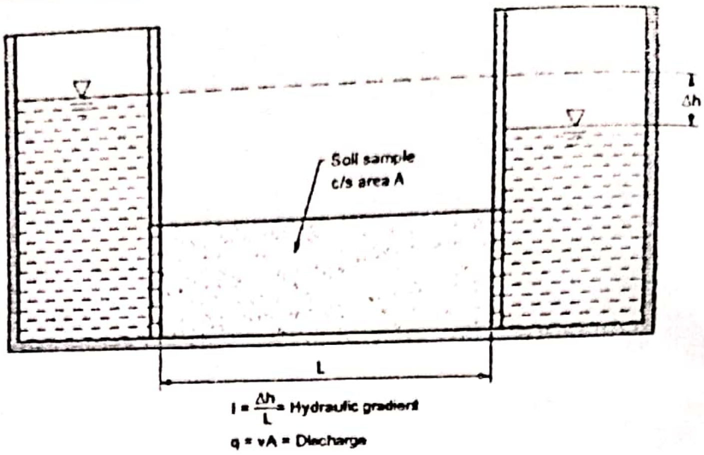


Fig. 3.2.1(c) : Darcy's experiment

3.2.3 Typical Values of Coefficient of Permeability for Different Soils

→ (MSBTE - W-10)

Q. Mention the typical values of k for different soils. (W-10)

- As the coefficient of permeability is the property of the soil, it changes with the type of soil.
- Following Table 3.2.1 shows values of coefficient of permeability for different soil types. It should be noted that if a soil is compacted its coefficient of permeability will change.

Table 3.2.1 : Values of 'K' for different soils

Sr. No.	Soil type	Soil classification	Grain size	Coefficient of permeability K in cm/sec.
1	Clean gravel poorly graded	GP	4.75 mm and above	Greater than 10^{-2}
2	Clean gravel well graded	GW	4.75 mm and above	10^{-2} to 10^{-3}
3	Clean sand poorly graded	SP	4.75 mm to 0.075 mm	10^{-3} to 5×10^{-4}
4	Clean sand well graded	SW	4.75 mm to 0.075 mm	5×10^{-4} to 10^{-5}
5	Fine sand with silt	SM	0.2 mm to 0.075 mm	2×10^{-5} to 10^{-5}
6	Silts or silty sandy clays	ML, MH, CL, ML, SC	0.0 mm to 0.075 mm	5×10^{-6} to 10^{-7}
7	Clays	CL, CI, CH	0.2 mm to 0.075 mm	10^{-8} or lesser

Syllabus Topic : Factors affecting Permeability

3.3 Factors Affecting Permeability

→ (MSBTE - S-08, S-09, S-10, S-12, S-13, S-14, W-14, W-15, S-16, S-17)

> Q. State and explain factors affecting permeability of soil. *(S-08, 09, 10, 12, 13, W-08, 09, 10, 11,12, S-15, S-16)*
>
> Q. Explain briefly effect of the following parameters on permeability of soil.
> (i) grain size (ii) void ratio *(S-12)*
>
> Q. Explain various factors affecting permeability. *(S-14, W-14)*
>
> Q. State the factors affecting permeability and explain in brief. *(W-15)*
>
> Q. Enlist factors affecting permeability. *(S-16)*
>
> Q. State factors affecting permeability. *(S-17)*

Factors Affecting Permeability

- 1. Grain size
- 2. Properties of pore fluid
- 3. Temperature
- 4. Void ratio
- 5. Stratification of soil
- 6. Entrapped air and organic impurities
- 7. Adsorbed water
- 8. Degree of saturation
- 9. Shape of particles
- 10. Structure of soil mass

Fig. C3.1 : Factors Affecting Permeability

Permeability depends on many factors. Following are the main factors that affect permeability :

→ 1. Grain size

- Grain size of the soil, or the effective size D_{10} is one of the factors which affect permeability.
- Allen Hazen gave the relation

$$K = 100 (D_{10})^2$$

Where, K = Coefficient of permeability in cm/s and D_{10} is the effective grain size of the soil.

- The permeability of coarse grained soil is more than that of fine grained soil.

→ 2. Properties of pore fluid

Permeability is directly proportional to the unit weight of pore fluid and inversely proportional to the viscosity of the pore fluid.

→ 3. Temperature

Since viscosity of the pore fluid decreases with the temperature, permeability increases with temperature, as unit weight of the pore fluid does not change much with change in temperature.

→ 4. Void ratio

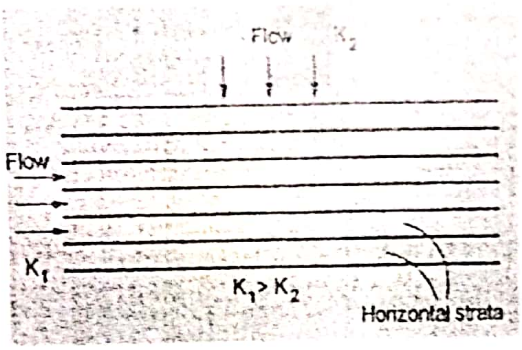
Increase in void ratio increases the area available for flow, hence the permeability increases for critical conditions.

$$K \propto \frac{e^3}{1+e}$$

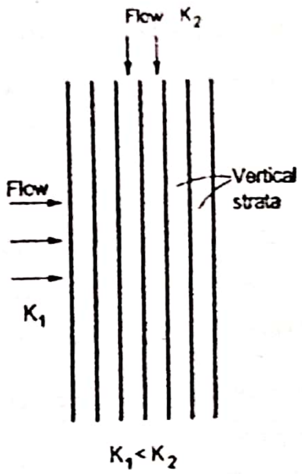
or $\quad K \propto e^2$

→ 5. Stratification of soil

- Stratified soils are those, which are formed by layer upon layer of earth or dust deposited upon one another.
- If the flow is parallel to the layers or stratification, the permeability is maximum while the flow in perpendicular direction to the stratification occurs with minimum permeability.
- This is shown in Fig. 3.3.1.



Horizontal strata



Vertical strata

Fig. 3.3.1 : Stratified soil

→ **6. Entrapped air and organic impurities**

Organic impurities and entrapped air obstruct the flow and coefficient of permeability is reduced due to their presence.

→ **7. Adsorbed water**

- Adsorbed water means a thin, microscopic film of water surrounding individual soil grains.
- This water is not free to move and hence reduces the effective pore space and thus decreases the coefficient of permeability.

→ **8. Degree of saturation**

Thee permeability of partially saturated soil is less than that of fully saturated soil.

→ **9. Shape of particles**

Permeability is inversely proportional to the specific surface e.g. the angular particles have more specific surface area as compare to rounded particles.

There fore the soil having angular particles is less permeable than soil of rounded particles.

→ **10. Structure of soil mass**

For same void ratio the permeability is more for flocculant structure as compare dispended structure.

3.4 Determination of Permeability

- Mainly, two laboratory tests are used to determine the coefficient of permeability. Which test is suitable depends on the type of soil.
- Soils with low permeability and those with high permeability require different tests.
- This is discussed below :

Syllabus Topic : Determination of Coefficient of Permeability by Constant Head Tests

3.4.1 Determination of Coefficient of Permeability by Constant Head Permeability Test IS 2720 – Part 17, 1986

→ **(MSBTE – W-08, S-10, W-10, W-12, S-12, W-13, W-14)**

Q. Differentiate between constant head method and falling head method of permeability. *(S-10)*

Q. Explain briefly about a test for permeability of soil. *(W-10)*

Q. Explain determination of coefficient of permeability by constant head method. *(W-12)*

Q. Explain how will you determine coefficient of permeability by constant head method. *(W-08)*

Q. Explain the procedure for determination of constant head test for finding out coefficient of permeability of soil. *(W-13, W-14)*

- It is based on measuring the volume of water flowing through a soil sample of known cross sectional area A and length L in time t under a constant head h of water.
- The arrangement is shown in Fig. 3.4.1.

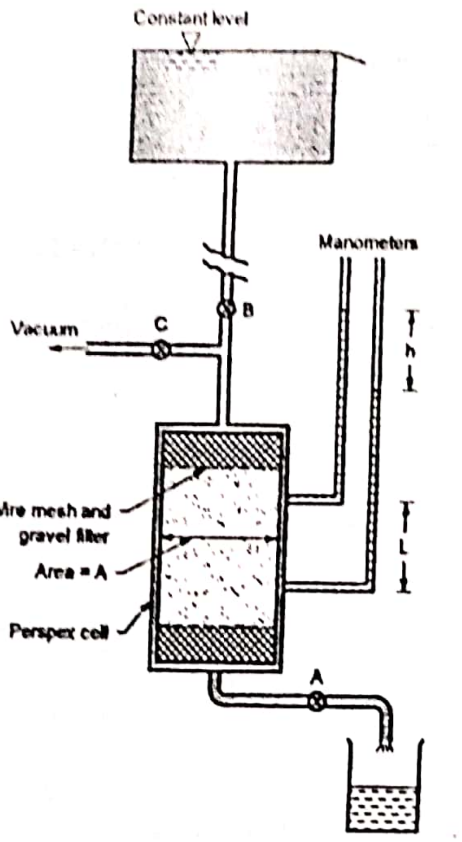


Fig. 3.4.1 : Constant head permeability test

- The soil sample is enclosed in a perspex cylindrical tube. A number of manometer points are provided of the side of the cylinder. These can be used in pairs.
- One such pair is shown in the Fig. 3.4.1.
- Water is allowed to flow through the soil at a constant head 'h' as indicated by the level difference in manometer tubes.
- The quantity of water or volume of water V collected in time t is measured, from which discharge q is calculated. The coefficient of permeability is then calculated as,

$$K = \frac{q}{iA} = \frac{VL}{Aht}$$

Where, $q = \frac{V}{t}$ = discharge in m^3/s

V = volume of water collected in m^3 in time t sec.

A = cross sectional area of specimen in m^2.

h = level difference between the manometer tubes in m.

L = distance between manometer points in m.

- Vacuum is used at the start of the test to ensure that all entrapped air from soil sample is removed keeping valves A and B closed.
- Then the valve C is closed. Valve A is used to control the flow so that a constant head flow is achieved as indicated by constant level difference 'h' between the manometer tubes.
- This test can be used for highly pervious soils where falling head test cannot be used due to very rapid fall in head, which cannot be measured.
- This test cannot be used for soil with very little permeability or highly impervious soils like clay as rate of collection of water may be very low, sometimes even days, so that the rate of evaporation may be higher.
- This will give wrong results as much of the volume of collected water will evaporate.

Syllabus Topic : Determination of Coefficient of Permeability by Falling Head Permeability Tests

3.4.2 Falling Head Permeability Test : IS 2720 – Part 17, 1986

→ **(MSBTE - W-09, S-13, S-16, S-18)**

Q. Write step by step procedure for determination of permeability of soil by falling head permeability test. Explain with neat sketch. ***(W-09, S-18)***

Q. With sketch explain how to determine coefficient of permeability by falling head test. ***(S-13)***

Q. Write step by step procedure to determine coefficient of permeability of fine grained soil by falling head method in laboratory. ***(S-16)***

- This test is used to determine the permeability of fine-grained soils with very low permeability.
- Due to low permeability, the collected water will evaporate and will lead to wrong results in constant head test. Hence, this test is used.
- The arrangement is shown in Fig. 3.4.2.
- A cylinder containing the soil sample is placed on a base, i.e. a perforated disc, fitted with a fine gauze. The cylinder has a rubber stopper at top in which a graduated stand pipe is fitted.

- The test is conducted by filling the standpipe with de-aired water. It is allowed to flow through the soil sample.
- During the test, the water level in the stand pipe continuously drops and the height of the water in stand pipe is measured at several time intervals and it is then recorded. Any one pair of measurements can be used to calculate the coefficient of permeability from the following formula :

$$K = \frac{aL}{A(t_2 - t_1)} \log_e \frac{h_1}{h_2}$$

Or
$$K = 2.303 \frac{aL}{A(t_2 - t_1)} \log_{10} \frac{h_1}{h_2}$$

Where, a = area of c/s of stand pipe.

A = area of soil sample.

L = length of soil specimen.

and h_1, h_2 = heights of water measured in the stand pipe at time t_1 and t_2.

If $t_2 - t_1 = t$ then,

$$K = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$

Several pairs of readings are used in this way and average of 4 – 5 values gives the average coefficient of permeability.

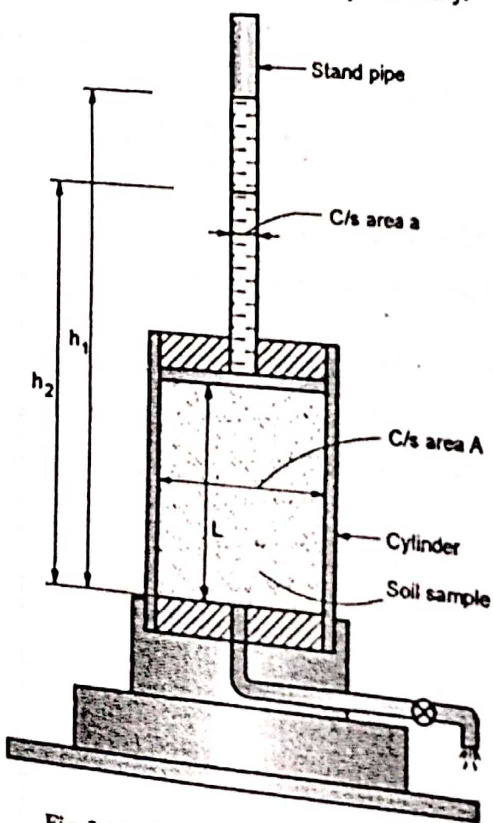


Fig. 3.4.2 : Falling head permeability test

Difference between constant head and falling head method

Constant head	Falling head
1. It is suitable for coarse grained soil.	1. It is suitable fine grained soil.
2. Water level is in constant head chamber is constant throughout experiment.	2. Water level in stand pipe falls.
3. Coefficient of permeability is $K = \frac{Q}{t}\frac{L}{b}\frac{1}{A}$	3. Coefficient of pearmeabillity is $K = 2.3 \frac{QL}{At} \log_{10} \frac{h_1}{h_2}$
4. Water collected through soil mass is more.	4. Water collected through soil mass is very less.

Syllabus Topic : Simple Problems to Determine Coefficient of Permeability.

3.4.3 Simple Problems to Determine Coefficient of Permeability

Ex. 3.4.1 S-18

In a falling head permeability test on a sample 12.2 cm high and 44.41 cm² in cross-section area, the water level in stand pipe of 3.25 mm internal diameter dropped from a height of 75 cm through 24.7 cm in 15 minutes. Find the coefficient of permeability.

Soln. :

Given : $a = \frac{\pi}{4} \times (0.625)^2 = 0.307 \text{ cm}^2.$

$t_2 - t_1 = t = 15 \text{ minutes} = 15 \times 60 = 900 \text{ sec.}$

$h_1 = 75 \text{ cm}, \; h_2 = (75 - 24.7) = 50.3$

$$\therefore \; K = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$

$$K = 2.303 \times \frac{0.307 \times 12.2}{44.41 \times 900} \log_{10}\left(\frac{75}{50.3}\right)$$

$$K = 3.74 \times 10^{-5} \text{ cm/s}$$

Ex. 3.4.2

A soil sample was tested in a constant head permeameter. Diameter of sample was 4 cm and length was 15 cm. Under a head of 20 cm, discharge was found to be 75 cc in 10 minutes under a head of 20 cm. Find the coefficient of permeability.

Soln.:

Given :

$$A = \frac{\pi}{4} \times 4^2 = 12.56 \text{ cm}^2, \quad L = 15 \text{ cm.}$$

$$t = 10 \text{ minutes} = 10 \times 60 = 600 \text{ sec.}$$

$$V = 75 \text{ cc} = 75 \text{ cm}^3, h = 20 \text{ cm.}$$

$$K = \frac{VL}{Aht} = \frac{75 \times 15}{12.56 \times 20 \times 600} = 7 \times 10^{-3} \text{ cm/s.}$$

Ex. 3.4.3

A soil sample 8 cm in diameter and 14 cm long was tested in a variable head parmeameter. The initial head of water in the burette was 50cm and it dropped through 20cm in 200 seconds. If diameter of burette is $\frac{1}{4}$th of diameter of soil sample, find the coefficient of permeability.

Soln.:

Given :

$$\frac{a}{A} = \frac{1}{4} = 0.25, \; L = 14 \text{ cm}, \frac{d_a}{d_A} = \frac{1}{4} = 0.25,$$

$$\frac{a}{a_A} = \frac{\pi \times 0.25^2}{4} = 0.04 \text{ a}, \; t = 200 \text{ sec.} = t_2 - t_1.$$

$$h_1 = 50 \text{ cm}, h_2 = 50 - 20 = 30 \text{ cm}$$

$$K = 2.3 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$

$$K = 2.3 \left(\frac{a}{A}\right) \frac{L}{t} \log_{10} \frac{h_1}{h_2}$$

$$K = 2.3 \times 0.049 \times \frac{14}{200} \log \frac{50}{30}$$

$$K = 1.75 \times 10^{-3} \text{ cm/s.}$$

Ex. 3.4.4

In a falling head test, head dropped from 150 cm to 75 cm in 30 min. what may be the head at 15 min?

Soln. :

Given : $h_1 = 150$ cm. $h_2 = 150 - 75 = 75$ cm.

$$t_1 = 30 \text{ min} = 30 \times 60 = 1800 \text{ sec.}$$

$$t_2 = 15 \text{ min.} = 900 \text{ sec.}$$

$$K = \text{constant}$$

$$\therefore \frac{aL}{At_1} \log \frac{h_1}{h_2} = \frac{aL}{At_2} \log \frac{h_1}{h_3}$$

$$\therefore \frac{1}{t_1} \log \frac{h_1}{h_2} = \frac{1}{t_2} \log \frac{h_1}{h_3}$$

$$\therefore \frac{1}{1800} \log \frac{150}{75} = \frac{1}{900} \log \frac{150}{h_3}$$

$$\therefore \frac{1}{2} \log \frac{150}{75} = \log \frac{150}{h_3}$$

$$\therefore \log \left(\frac{150}{75}\right)^{1/2} = \log \left(\frac{150}{h_3}\right)$$

$$\therefore \left(\frac{150}{75}\right)^{1/2} = \left(\frac{150}{h_3}\right)$$

$$\therefore h_3 = 150 \times \left(\frac{75}{150}\right)^{1/2}$$

$$h_3 = 103.06 \text{ cm}$$

Ex. 3.4.5

A sample of soil is 10 cm high and 50 cm² in cross section area. Water flows through the soil under a constant head of 100 cm. Water collected in 9 minutes is 460 cc. Calculate K.

Soln. :

Given : $Q = 460 \text{ cm}^3, \; A = 50 \text{ cm}^2, \quad L = 10 \text{ cm.}$

$$h = 100 \text{ cm}, \; t = 9 \times 60 = 540 \text{ sec.}$$

$$K = \frac{QL}{Aht}$$

$$\therefore K = \frac{460 \times 10}{50 \times 100 \times 540}$$

$$\therefore K = 1.7 \times 10^{-3} \text{ cm/sec.}$$

Ex. 3.4.6

A soil sample was tested in a constant head permeameter. The length of the sample was 20 cm. The diameter of the permeameter used was 5 cm. Under a head of 40 cm, the discharge was found to be 150 cc in 10 mts. Calculate the coefficient of permeability.

Soln. : Coefficient permeability is given by,

$$K = \frac{Q}{t} \cdot \frac{1}{A} \cdot \frac{L}{h}$$

Where,

$$Q = 150 \text{ cc}$$

$$t = 10 \times 60 = 600 \text{ sec.}$$

$$A = \frac{\pi}{4}(5)^2 = 19.63$$

$$L = 20 \text{ cm}$$

$$h = 40 \text{ cm.}$$

$$\therefore K = \frac{150}{600} \times \frac{1}{19.63} \times \frac{20}{40}$$

$$\therefore K = 3.37 \times 10^{-3} \text{ cm / sec}$$

Ex. 3.4.7

A sample of soil 10 cm length was tested in a variable head permeameter. The initial head of water in the burette was found to be 36 cm and it was observed to drop through 12 cm in 900 sec. If the diameter of burette is 0.1 times of that of the soil sample, then, calculate the coefficient of permeability.

Soln. :

Given : $l = 10$ cm, $h_1 = 36$ cm, $h_2 = 36 - 12 = 24$ cm, $t = 900$ sec.

Let, D = diameter of soil sample

$$d = 0.1 D$$

$$\therefore A = \frac{\pi}{4}D^2, \text{ and } a = \frac{\pi}{4}(0.1D)^2 = 0.01\frac{\pi}{4}D^2$$

Coefficient of permeability is given by

$$K = 2.303 \frac{a\,l}{A\,t} \log_{10}\left[\frac{h_1}{h_2}\right]$$

$$K = 2.303 \times \frac{0.01\frac{\pi}{4}D^2 \times 10}{\frac{\pi}{4}D^2 \times 900} \log_{10}\left[\frac{36}{24}\right]$$

$$\therefore K = 4.5 \times 10^{-5} \text{ cm/s}$$

Ex. 3.4.8 S-08

A soil sample 10 cm in diameter and 15 cm long was tested in a variable permeameter. The initial head of water in the burette was 90 cm and it dropped through 25 cm in 25 minutes. Diameter of burette is 19 mm. Find the coefficient of permeability.

Soln. :

Given :

$$l = 15 \text{ cm.}$$

$$h_1 = 90 \text{ cm.}$$

$$h_2 = (90 - 25) = 65 \text{ cm.}$$

$$t = 25 \times 60 = 1500 \text{ sec.,}$$

$$D = 10 \text{ cm.}$$

$$d = 19 \text{ mm} = 1.9 \text{ cm.}$$

C/S area of burette, $a = \frac{\pi}{4}d^2 = \frac{\pi}{4}(1.9)^2 = 2.835 \text{ cm}^2$

C/S area of burette, $A = \frac{\pi}{4}D^2$

$$= \frac{\pi}{4}(10)^2 = 78.54 \text{ cm}^2$$

But, coefficient of permeability by falling head is,

$$K = 2.303 \frac{a\,l}{At} \log_{10}\left[\frac{h_1}{h_2}\right]$$

$$= 2.303 \times \frac{2.835 \times 15}{78.54 \times 1500} \log_{10}\left[\frac{90}{65}\right]$$

$$\therefore K = 1.175 \times 10^{-4} \text{ cm/sec.}$$

Ex. 3.4.9 S-10

A constant head permeameter gives a discharge of 200 ml in 115 seconds under a constant head of 400 mm. Determine the permeability if the soil sample was 250 mm long and 3000 mm² in area.

Soln. :

$$Q = 200 \text{ ml}$$

$$t = 115 \text{ seconds}$$

$$h = 400 \text{ mm} = 200 \times 10^3 \text{ mm}^3$$

$$L = 250 \text{ mm}$$

$$A = 3000 \text{ mm}^2 \quad K = ?$$

$$K = \frac{Q}{t} \times \frac{1}{A} \times \frac{L}{h}$$

$$= \frac{200 \times 10^3}{115} \times \frac{1}{3000} \times \frac{250}{400}$$

$$= 0.362 \text{ mm/sec.}$$

Ex. 3.4.10 S-11

In a constant head permeameter diameter of a soil sample was 4 cm and length was 14 cm under a constant head of 25 cm. The discharge was found to be 80 cc in 10 minutes. Calculate coefficient of permeability.

Soln. :

Given : diameter of sample = 4 cm

length of sample, $L = 14$ cm

constant head, $h = 25$ cm

Quantity of water, $Q = 80$ cc

time period, $t = 10 \text{ min} = 600$ sec.

c/s area of soil sample,

$$A = \frac{\pi}{4}(4)^2 = 12.56 \text{ cm}^2$$

We have coefficient of permeability given by

$$K = \frac{Q}{t} \times \frac{1}{A} \times \frac{L}{h}$$

$$= \frac{80}{600} \times \frac{1}{12.56} \times \frac{14}{25} = 5.94 \times 10^{-3} \text{ cm/sec.}$$

Ex. 3.4.11

Calculate coefficient of permeability in the falling head test when following data is recorded.

(a) Area of stand pipe = 0.53 cm²

(b) Length of specimen = 15 cm

(c) c/s area of soil sample = 78.5 cm²

(d) Time of test = 100 minute

(e) Fall of head = 80 cm to 25 cm

Soln. :

Given : With usual notations

$$a = 0.53 \text{ cm}^2$$

$$A = 78.5 \text{ cm}^2$$

$$L = 15 \text{ cm}$$

$$h_1 = 80 \text{ cm}$$

$$h_2 = 80 - 25 = 55 \text{ cm}$$

$$t = 100 \text{ min.} = 6000 \text{ sec.}$$

∴ Coefficient of permeability

$$K = 2.303 \times \frac{a \times L}{A \times t} \times \log_{10}\left(\frac{h_1}{h_2}\right)$$

$$= 2.303 \times \frac{0.53 \times 15}{78.5 \times 6000} \times \log_{10}\left(\frac{80}{25}\right)$$

$$= 1.96 \times 10^{-5} \text{ cm/s}$$

Ex. 3.4.12 W-08, S-09

A soil sample having 10 cm diameter, 15 cm long was tested under variable head permeameter. The initial water head was 45 cm. The water head was found to be dropped through 30 cm in 195 sec. The burette diameter was found to be 1.9 cm. Calculate the coefficient of permeability in meters/day.

Soln. :

$$D = 10 \text{ cm}, \quad L = 15 \text{ cm}, \quad h_1 = 45 \text{ cm}$$

$$h_2 = 45 - 30 = 15 \text{ cm}$$

$$t = 195 \text{ sec.} \quad d = 1.9 \text{ cm}$$

$$K = ?$$

$$A = \frac{\pi D^2}{4} = \frac{\pi}{4} \times 10^2 = 78.54 \text{ cm}^2$$

$$a = \pi d^2/4 = \pi/4 \times 1.9^2 = 2.835 \text{ cm}^2$$

$$K = 2.3 \frac{a \cdot L}{A \cdot t} \log_{10}\left(\frac{h_1}{h_2}\right)$$

$$= 2.3 \times \frac{2.835 \times 13}{78.54 \times 195} \log_{10}\left(\frac{13}{15}\right)$$

$$= 3.047 \times 10^{-3} \text{ cm/s}$$

$$K = \frac{3.047 \times 10^{-3} \times \frac{1}{100}}{\frac{1}{60} \times \frac{1}{60} \times \frac{1}{24}} = 2.63 \text{ m/day.}$$

Ex. 3.4.13 [illegible]

A constant head permeameter gives discharge of 305 ml in 270 seconds under a constant head of 870 mm, determine the permeability if the soil sample was 120 mm long and 78.5 cm^2 in area.

Soln. :

$Q = 305 \times 10^3 \text{ mm}^3$ $\quad h = 870$ mm

$L = 120$ mm $\quad A = 78.5 \text{ cm}^2 = 7850 \text{ mm}^2$

$t = 270$ seconds

$$k = \frac{Q}{t} \times \frac{1}{A} \times \frac{L}{h}$$

$$= \frac{305 \times 10^3}{270} \times \frac{1}{78.5 \times 100} \times \frac{120}{870}$$

$$k = 0.01985 \text{ mm/sec}$$

Ex. 3.4.14 S-12

A soil sample was tested in a constant head permeameter. The length of sample was 25 cm. The diameter of permeameter used was 6 cm. Under a head of 50 cm the discharge was found to be 150 cc in 15 min. Find coefficient of permeability.

$$= \frac{3750}{1271700} = 0.($$

$$K = 2.94 \times 10^{-3}$$

Ex. 3.4.16 W-12

The falling head permeabilit
sample of 4 cms diameter a
from 1.0 m to 0.40 m at 20
area of the stand pipe was 1
permeability.

Soln. :

$a = 1 \text{ cm}^2$

$t = 20$ mi

$h_1 = 1.0$ m

$h_2 = 0.40$ m

$D = 4$ cm

$L = 18$ cm

$A = \frac{\pi}{4} \times 4^2$

(Coefficient of permeability

$k = 2.3 \times \frac{al}{A}$

$= 2.3 \times$

$= 1.093 \times$

$$Q = 450 \text{ cm}$$

$$k = \frac{Q}{t} * \frac{1}{A} * \frac{L}{h}$$

$$k = \frac{450}{540} * \frac{1}{50} * \frac{10}{80}$$

$$k = 2.08 * 10^{-3} \text{ cm/sec.}$$

Ex. 3.4.17

In a falling head permeability test on a sample 12 cm height and 44.41cm2 in cross sectional area. The water level in stand pipe of 3.25 mm internal diameter dropped from a height of 75cm to 24.7 cm in 15 minutes. Find the coefficient of permeability.

Soln. :

$$T_2 - T_1 = 15 \text{ minutes} = 15 \times 60 = 900 \text{ sec.}$$

$$H_1 = 75 \text{ cm}, H_2 = (75 - 24.7) = 50.3 \text{ cm}$$

$$\alpha = \frac{\pi}{4} \times 0.625^2 = 0.307 \text{ cm}^2$$

$$K = 2.303 \frac{\alpha L}{At} \log_{10} \frac{h_1}{h_2}$$

$$K = 2.303 \frac{0.307x \times 12}{44.1x \times 900} \log_{10} \frac{75}{50.3}$$

$$K = 3.683 \times 10^{-5} \text{ cm/s}$$

Ex. 3.4.18

A soil sample 10 cm in diameter and 15 cm long is tested in falling head permeameter. The initial head was 45 cm, which was dropped to 25 cm in 12 minutes. The diameter of burette pipe was 0.5 cm. Find coefficient of permeability in metre/day.

Soln. :

Given : $D = 10$ cm, $L = 15$ cm, $h_1 = 45$ cm, $h_2 = 25$ cm, $t = 12$ minutes i.e. $= 720$ sec, $d = 0.5$ cm Find K in m/day.

$$A = (\pi/4) \times d^2 = (\pi/4) \times 10^2 = 78.54 \text{ cm}^2$$

$$a = (\pi/4) \times d^2 = (\pi/4) \times 0.5^2 = 0.196 \text{ cm}^2$$

$$K = 2.3 \frac{a}{A} \cdot \frac{L}{t} \cdot \log_{10} [h_1/h_2]$$

$$K = 2.3 \frac{0.196}{78.54} \cdot \frac{15}{720} \cdot \log_{10} [45/25]$$

$$K = 3.05 \times 10^{-5} \text{ cm/sec.}$$

$$K = 3.05 \times 10^{-5} \times (1/100) = 0.026 \text{ m/day}$$

Ex. 3.4.19

A soil sample was tested in constant head permeameter. Diameter of sample was 4 cm, length was 15 cm. Under constant head of 20 cm. Discharge was found to be 75 cc in 10 minutes. Find coefficient of permeability.

Soln. :

Find coefficient of permeability = ?

Diameter of sample = 4 cm hence $A = \pi/4\ (4)^2 = 12.56 \text{ cm}^2$

Length of sample = 15 cm

Constant head = 20 cm

Quantity of discharge = 75 cc

Time period = 10 minutes = $10 \times 60 = 600$ seconds

$$K = (Q/t) \times (1/A) \times (L/h)$$

$$K = [75/600] \times [1/12.56] \times [15/20]$$

$$K = 0.125 \times 0.0796 \times 0.75$$

$$K = 7.46 \times 10^{-3} \text{ cm/sec.}$$

Ex. 3.4.20

A permeameter ($A = 3000 \text{ mm}^2$, $L = 200$ mm) gave discharge of 25 ml in 20 minutes under a constant head of 1 m. Determine the value of co-efficient of permeability.

Soln. :

$A = 3000 \text{ mm}^2$, Length of sample = 200 mm,

constant head = 1000 mm

Quaintly of discharge = 25 ml = $25 \times 1000 = 25000 \text{ mm}^3$

Time period = 20 minutes = $20 \times 1000 = 1200$ seconds

$$K = \left(\frac{Q}{t}\right) \times \left(\frac{1}{A}\right) \times \left(\frac{L}{h}\right)$$

$$K = [25 \times 1000/1200] \times [1/3000] \times [200/1000]$$

$$= 20.83 \times 0.00033 \times 0.20 = 0.0014 \text{ mm/sec.}$$

Ex. 3.4.21

A constant head permeameter gives discharge of 350 ml in 270 seconds under a constant head of 1050 mm. Determine coefficient of permeability in m/day, if the soil sample was 150 mm long and 78.50 cm^2 in cross section area.

Soln. :

$A = 78.50\ cm^2 = 7850\ mm^2$, Length of sample = 150 mm

Constant head = 1050 mm, Quantity of discharge = 350 ml

$= 350000\ mm^3$, Time period = 270 second

$$k = (G/t) \times (1/A) \times (L/h)$$

$$= \left(\frac{350 \times 1000}{270}\right) \times \left(\frac{1}{7850}\right) \times \left(\frac{150}{1050}\right)$$

$$= 1293.29 \times 0.000127 \times 0.143$$

$$k = 0.0235\ mm/sec.$$

Ex. 3.4.22 S-16

A constant head permeameter gives discharge of 350 ml in 270 seconds under a constant head of 1050 mm. Determine coefficient of permeability in m/day, if the soil sample was 150 mm long and 78.50 cm^2 in cross section area.

Soln. :

$Q = 350\ ml$

$t = 270$ seconds

$h = 1050mm = 200 \times 10^3\ mm^3$

$L = 250\ mm$

$A = 78.50\ mm^2$ $K = ?$

$$K = \frac{Q}{t} \times \frac{1}{A} \times \frac{L}{h}$$

$$= \frac{350 \times 10^3}{270} \times \frac{1}{78.5 \times 100} \times \frac{250}{1050}$$

$$K = 0.0393\ mm/sec.$$

Ex. 3.4.23 S-17

In a falling head permeability test on a sample 15 cm high and 45 cm2 in cross-section area, the water level in stand pipe of 8 mm internal diameter dropped from a height of 75 cm to 25 cm in 15 minutes. Find the co-efficient of permeability.

Soln. :

Given :

$L = 15cm$, $A = 45\ cm^2$, $d = 8mm$, $h_1 = 75cm$, $h_2 = 25\ cm$,

$t = 15\ min = 900\ sec.$

To find:- $k = ?$

$$a = \pi/4 \times d^2 = \pi/4 \times 0.8^2 = 0.50$$

$$K = 2.3\frac{a}{A} \cdot \frac{L}{t} \cdot \log_{10}[h_1/h_2]$$

$$K = 2.3\frac{0.5}{45} \cdot \frac{15}{900} \cdot \log_{10}[75/25]$$

$$K = 2.034 \times 10^{-4}\ cm/s$$

Syllabus Topic : Seepage through Earthen Structures

3.5 Seepage through Earthen Structures

→ (MSBTE - W-09)

Q. State any four engineering problems where study of seepage of water is required. (W-09)

☞ ***Definition of seepage :*** *If the soil is permeable, water will flow through it. This flow of water through earthen structures is called seepage.*

- Seepage can be considered a two dimensional flow. Where velocity varies in both horizontal and vertical directions.
- Whereas Darcy's equation considers one dimensional flow, for analysis of seepage, Laplace's equation is used.

According to Darcy's equation the velocity of flow of water through soil is,

$$v = Ki$$

And the discharge is,

$$q = vA$$

- Here the velocity v is called the fictitious or superficial velocity of flow, because the actual flow is through the area of voids and not through the entire cross sectional area.

- Thus, according to Darcy's law, permeability can be defined as the superficial velocity of flow under unit hydraulic gradient.
- The various engineering problems wherestudy of seepage of water is required are :

1. Where any embankment is constructed on a soil having low coefficient of permeability.
2. During construction of retaining wall.
3. During construction of earthen dam.
4. Losses from irrigation canals.
5. Draining of water logged agricultural land.
6. Rate of settlement of structure.
7. Safety of hydraulic structure against piping.

Syllabus Topic : Seepage Velocity

3.5.1 Seepage Velocity

→ **(MSBTE - S-09, S-11)**

Q. Define Seepage velocity. (S-09, S-11)

- As stated above, the velocity obtained by Darcy's equation is superficial velocity.

Definition of seepage velocity : The actual velocity of flow flowing in the voids is called the seepage velocity, v_s.

Hence, $q = vA = v_s A_v$

- Where A_v is the area of voids across the cross section.

$\therefore$ $$v = v_s \frac{A_v}{A}$$

Since,

$\therefore$ $$\frac{A_v}{A} = \frac{V_v}{V} = n$$

Where, n = Porosity

$\therefore$ $$v = nv_s = \left(\frac{e}{e+1}\right) \times v_s$$

- Since n is always less than one, seepage velocity is always greater than the superficial or discharge velocity.

Syllabus Topic : Seepage Pressure

3.5.2 Seepage Pressure

→ **(MSBTE - S-11, W-11)**

Q. What is seepage pressure ? (S-11, W-11)

Definition of Seepage Pressure : It is the additional pressure caused by frictional force acting on the surface of particles.

- Seepage pressure causes 'drag' of the water in the opposite direction of flow. Seepage pressure force per unit volume is given by $F = i\gamma_w$.
- The effective pressure due to seepage is given by,

$$\sigma_{eff} = L(\gamma_{sub} + i\gamma_w)$$

If flow is in direction of gravity i.e. downwards

and $$\sigma_{eff} = L(\gamma_{sub} - i\gamma_w)$$

- If flow is in upward direction. L is the length across which flow occurs.
- The pressure exerted by water on the soil through which it percolates called as seepage pressure (P_s) and it is given by,

$$P_s = h\gamma_w$$

$$P_s = \frac{h}{L} \times L\gamma_w \qquad h - \text{hydraulic head}$$

$$P_s = iL\gamma_w \qquad L - \text{length over which head lost}$$

$$i - \text{hydraulic gradient}$$

$$\because \frac{h}{L} = i \qquad \gamma_w - \text{unit weight of water}$$

seepage force is given by,

$$f_s = P_s \cdot A = i \times L \times \gamma_w A$$

A - total c/s of soil mass

The seepage force per unit volume is given by,

$$f_s = \frac{iLA\gamma_w}{LA}$$

$$f_s = i\gamma_w$$

Syllabus Topic : Phreatic Line

3.5.3 Phreatic Line

→ (MSBTE - S-09, W-11, S-13, S-14, S-15. S-16, S-17, W-17)

Q. Define Phreatic lines. *(S-09, W-11, S-13, S-16, S-17)*

Q. Discuss briefly equipotential lines and phreatic lines with neat sketches. *(S-12)*

Q. Define Phreatic line with sketch. *(W-13)*

Q. Define preatic line. *(S-14)*

Q. Explain with neat sketch phreatic line in earthen dam with pressure head at different point and show construction points of this line. *(S-15)*

Q. Explain phreatic line in earthen dam with a sketch. *(W-17)*

☞ ***Definition of Phreatic Line :*** *When flow of water occurs through soil, the top surface of the flow zone is called the phreatic surface, and in section, the top line of flow zone is called the phreatic line.*

- In earth retaining structures, the flow due to seepage will cause the phreatic line.
- All the points on the phreatic line have equal seepage pressure and it can be taken as atmospheric pressure. Fig. 3.5.1 shows phreatic lines in different types of earth dams.
- Phreatic or seepage line is the line within a dam section below which there is positive hydrostatic pressure exists in dam and it is atmospheric on it.

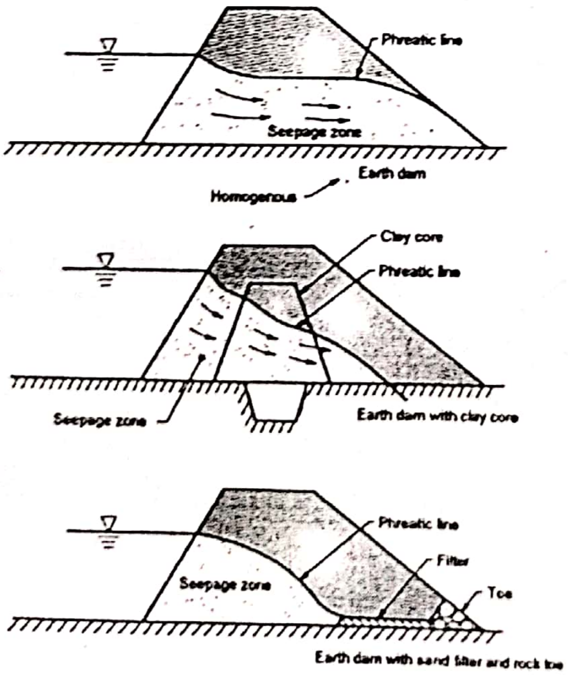


Fig. 3.5.1 : Phreatic lines in earth dams

Syllabus Topic : Flow Lines

3.5.4 Flow Lines and Equipotential Lines

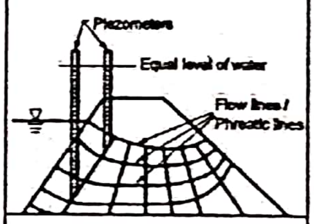
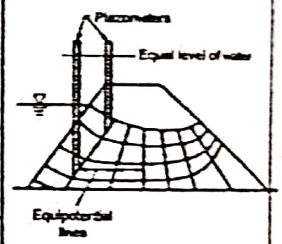
→ (MSBTE - S-09, W-10, S-12)

Q. Define Flow lines, Equipotential lines. *(S-09, W-11)*

Q. Differentiate between phreatic line and equipotential line. *(W-10)*

Q. Discuss briefly equipotential lines and phreatic lines with neat sketches. *(S-12)*

☞ **Difference between Phreatic line and Equipotential line**

Sr. No.	Phreatic line	Equipotential line
1.	It is line within dam section below which there is positive hydrostatic pressure.	Equipotential lines are lines joining points of equal pressure along flow line.
2.		

☞ ***Definition of Flow lines :*** *When water flows through soil in laminar flow conditions, the paths along which the layers of water flow are called flow lines.*

- Equipotential lines are the lines joining points of equal pressure along the flow lines. These can also be defined as lines along which the points have equal head of water.
- If piezometers or glass tubes are inserted along an equipotential line, water will rise to the same height in the tubes.

Equipotential lines are always perpendicular to the flow lines.

3.6 Graphical Representation of Seepage

Analysis of seepage phenomenon is simplified by using the graphical solutions. One of the most used graphical tool in "Flow Net".

3.6.1 Flow - Net

→ (MSBTE - S-08, S-10, W-10, W-11, S-14, W-15, S-16, W-17)

Q. Define flow net. (S-08, S-10, W-11, W-15, S-15)
Q. What is Flow net ? (W-10)
Q. Define flow net. Draw neat sketch of flow net. (S-14)
Q. Explain with sketch flow net. (W-17)

☞ *Definition of Flow Net : The grid, mesh or net formed by the intersection of equipotential lines and flow lines is called flow-net.*

☞ *Definition of Flow Channel : The portion of a flow net between two adjacent flow lines is called a flow channel.*

- Every section of a flow channel between two successive equipotential lines is called a field. This is illustrated in Fig. 3.6.1.

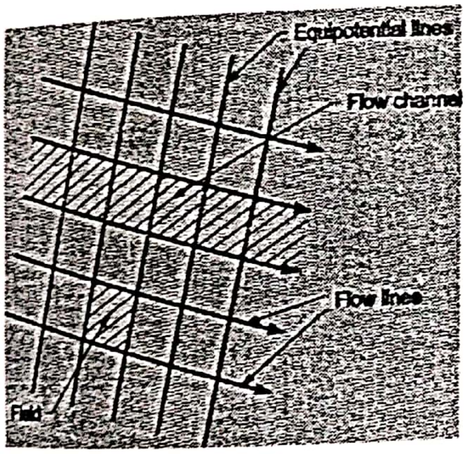


Fig. 3.6.1 : Flow-net

3.6.2 Characteristics of Flow-net

→ (MSBTE - S-09, W-09, W-10, S-11, W-12, S-13, W-13, S-15, W-15, S-16, S-17, S-18)

Q. State the characteristics of a flow net. (S-09, W-10, S-11, W-12, S-13, W-13, S-15, S-16, S-17, S-18)
Q. State any four characteristics of flow net and draw neat sketch of flow net. (W-09)
Q. State properties of flow net. (W-15)

Following are the characteristics of flow-net, which are useful in construction of flow-nets and in calculations in solving seepage problems :

1. In a flow-net, flow lines and equipotential lines intersect each other at right angles.
2. The quantity of water flowing through each flow channel is the same.
3. The drop of head, or the potential drop between any two successive equipotential lines is the same.
4. The fields are approximately squares.
5. The flow net is representative of the flow pattern and dissipation of the hydraulic head see Fig. 3.6.2.

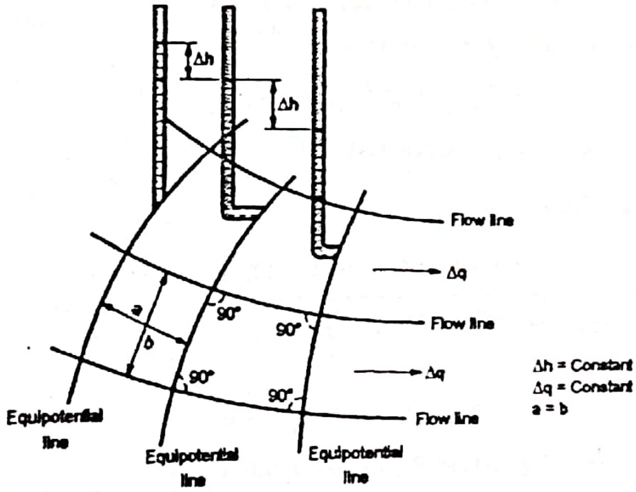


Fig. 3.6.2 : Part of flow-net illustrating characteristics

Syllabus Topic : Application of Flow Net

3.6.3 Applications of Flow-net

Q. State any four application of flow net. (W-11)

☞ **Uses of flow net**

A flow net chart is used for following purposes :

(1) Determination of discharge.

(2) Determination of total head.

(3) Determination of pressure head.

(4) Determination of hydraulic gradient.

(5) For calculating quantity of seepage.

(6) For stability of earthen dam.

(7) For analysis of flow phenomenon.

(8) For stability analysis.

(1) **Determination of discharge**

From darcy's law,

$$q = K.H\frac{n_f}{n_d}$$

q – Discharge due seepage

Where n_f – number of flow channel

n_d – number of potential drop

q – discharge due to seepage

K – coefficient of pearmeability

H – Head of water causing flow.

(2) **Determination of total head**

The total head at any point can be determined by considering loss of head from one equipotential line to next equipotential line i.e. H / n_d and is given by,

$$H_t = (H - n)\frac{H}{n_d}$$

n – number of equipotential drop

H_t – total head

(3) **Determination of pressure head**

The pressure at any point is given as the total head minus elevation head.

$$H_p = H_t - (-H_e)$$

$$\therefore \quad H_p = H_t + H_e$$

H_p – pressure head

H_t – total head

H_e – elevation head

(4) **Determination of hydraulic gradient**

The average hydraulic gradient for any flow field is given by,

$$i = \frac{\Delta h}{L}$$

ΔH – equipotential drop in the field, L – length of field

Introduction

- Shear force is the force applied along or parallel to a surface or cross-section, instead of being applied perpendicular to the cross-section.
- Shear stress is this force divided by the area on which it acts. And the capacity to withstand the maximum shear stress without occurrence of failure is the shear strength of the material.
- Failure of soil rarely occurs due to tension, as, if the soil is pulled from both directions, particles will separate and it will immediately collapse. So that tensile load can hardly be applied to soil.
- Due to compression, the particles become more and more closely packed, densification increases strength increase and thus, compression failure of soil is also rare.
- That leaves us with shear failure, which is the most common mode of failure for any soil. Hence in foundation design, in earth road design, in earth dams, in retaining structures, in open canal design, in design of spillway apron shear strength of soil is important.

3.7 Theories of Failure

Various theories about failure of soil are proposed by different investigators. But the most common theory is the Mohr-Coulomb-Theory.

Syllabus Topic : Shear Failure of Soil

3.7.1 Shear Failure of Soil

- When there is load on a soil, shear stress with different intensities will develop along different planes. A pair of planes perpendicular to each other will be the planes with maximum shear.

- If this maximum applied shear stress is more than the shear strength, failure will occur along one of the two planes of maximum shear.
- An entire wedge of soil slides over another when shear failure of soil occurs.

Syllabus Topic : Field Situation of Shear Failure

3.7.2 Field Situation of Shear Failure

→ (MSBTE - S-08, W-09, S-10, W-10, S-13, W-13, S-16, W-17)

> Q. State field situations of shear failure. *(S-08, W-09, W-10, S-16, W-17)*
> Q. State any two field situations of shear failure of soil. *(S-10, S-13)*
> Q. State any four field situations of shear failure. *(W-13)*

The field situations where shear failure can occur are given below :

(1) Upstream slope of earthen dam, especially during sudden draw down.

(2) Earth behind retaining wall, especially **"surcharge"**.

(3) Under foundations along planes of maximum shear.

(4) Subgrades of road etc.

Syllabus Topic : Concept of Shear Strength of Soil

3.8 Concept of Shear Strength of Soil

→ (MSBTE S-08, W-09, W-10, S-11, S-12, W-12, S-16)

> Q. Define shear strength. *(S-08, W-09, W-10, S-12, W-12, S-16)*
> Q. State two factors on which the shear strength of soil depends upon. *(S-11)*
> Q. Explain concept of shear strength of soil. *(W-12)*

- In most of the problems in soil mechanics such as those concerning the foundations of structures, earthwork engineering etc., the soil mass has to withstand shearing stresses, which are unlike in nature than the compressive stresses.
- Shearing stresses tend to displace part of the soil mass relative to rest of the soil mass. Shear strength of a soil is the capacity of the soil to resist shearing stress.
- It can be defined as the maximum value of shear stress that can be mobilized within a soil mass.
- If this value is equalled by the shear stress on any plane or surface at a point, failure will occur in the soil because of movement of a portion of the soil mass along that plane or surface.
- The soil is then said to have failed in shear.

→ (MSBTE - W-11)

> Q. State any four factors which affect the shear strength of soil. *(W-11)*

- The shear strength depends upon :

1)	Type of soil	2)	Water content
3)	Cohesion	4)	Internal friction
5)	Compaction.		

Syllabus Topic : Components of Shearing Resistance of Soil – Cohesion, Internal Friction

3.9 Components of Shearing Resistance of Soil – Cohesion and Internal Friction

→ (MSBTE – W-08,S-13)

> Q. State and define components of shearing resistance of soil. *(W-08, S-13)*

The shearing resistance of soil can be viewed as composed of two parts :

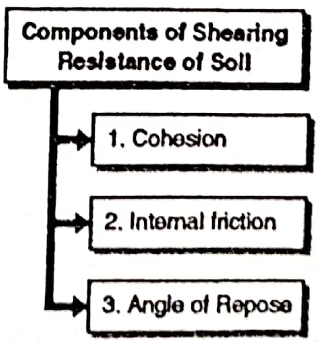


Fig. C3.2 : Components of Shearing Resistance of Soil

→ 1. Cohesion

→ (MSBTE - S-09, W-11, S-12)

Q. Define Cohesion. *(S-09, W-11, S-12)*

- The force of attraction between the soil particles is called the cohesion. Cohesion means attraction between similar particles.
- The particles of soil at contact points have bonds of attraction with each other. The sum total of these small forces of attraction will cause cohesion of soil.
- In some soils this force of cohesion is more prominent. These are called cohesive soils. e.g clay and organic soil.
- In some soils, the force of cohesion is negligible. These are called cohesionless soils e.g. sand, murum etc.

→ 2. **Internal friction**

→ (MSBTE – W-10,W-11,S-12)

Q. Define Internal friction. *(W-10, W-11, S-12)*

- When the particles move against one another, a force of friction comes into play in the opposite direction of motion.
- These small forces of friction resisting the motion of particles against one another add up to give a force, which resists the movement of soil. This force is called the internal friction of soil.
- Soils having microscopic particles often have zero or negligible internal friction e.g. clay, some soils have maximum or complete shear resistance due to internal friction e.g. sand, murum etc.
- Shear resistance of most of the soils is made up partly from cohesion and partly from internal friction.

→ 3. **Angle of Repose**

→ (MSBTE - S-12)

Q. Define : angle of repose. *(S-12)*

It is the angle made by soil mass with inclined surface when it just start sliding in down ward direction due to self weight.

Syllabus Topic : Mohr-Coulomb Failure Theory

3.10 Mohr Coulomb Failure Theory

→ (MSBTE – W-08, S-09, W-09, W-10, S-11, W-12, S-14,S-16, S-17, W-17)

Q. State the shear strength equation and draw shear envelope for :
(i) C - soil, (ii) ϕ - soil
(iii) C - ϕ soil *(W-08, W-09)*

Q. State Mohr-Coulomb's equation. Draw shear failure envelope for : *(S-09)*
(i) C - soil (ii) ϕ - soil (iii) C - ϕ soil

Q. Explain in brief the Mohr-Coulomb failure theory. *(W-10)*

Q. Describe Mohr's theory of shear failure of soil with sketch. *(S-11, S-17)*

Q. Draw Mohr's envelope for cohesive and cohesionless soil. *(W-12)*

Q. Show graphically and state shear strength equation for cohesive and cohesion less soils. *(S-14)*

Q. Draw strength envelope for : (i) C-soil (ii) f-soil and (iii) C-f soil *(S-16)*

Q. Explain with sketch Mohr – Coulomb failure theory. *(W-17)*

- Mainly shear failure in soil occurs by slippage of particle due to shear stress.
- Shear stress at failure normally depends upon normal stresses on the potential failure plane.
- According to Mohr soil failure is a function of normal stress applied on the soil.

i.e. $\tau_f = f(\sigma)$ ...(1)

τ_f = shear stress

σ = normal stress

The shear stress at failure is called as shear strength

∴ Equation (1) becomes,

$\tau_f = s = f(\sigma)$

s = shear strength.

A curve as shown is obtained when normal and shear stress corresponding to failure are plotted are called as strength envelope.

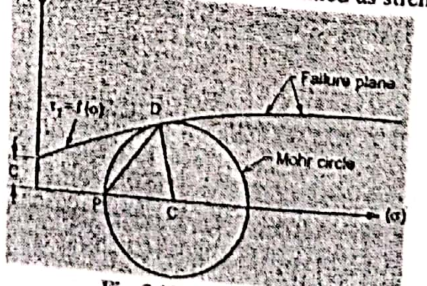


Fig. 3.10.1: Mohr. Theory

Coulomb later modified Mohr's equations and gave the following equation.

$$s = c + \sigma \tan \phi$$

$\therefore$ Thus Mohr envelope is replaced by straight Line

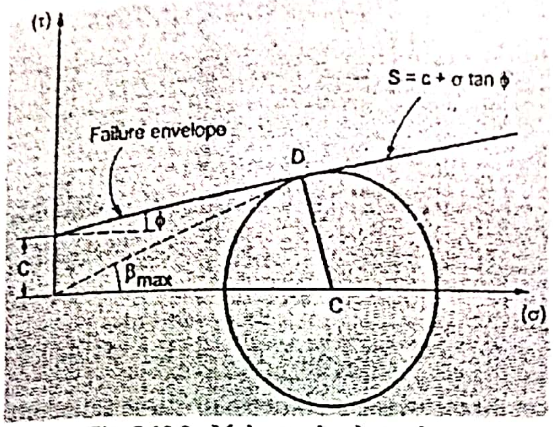


Fig. 3.10.2 : Mohr. coulomb envelope

- We can say that c is intercept on $\bar{c}$ axis and ϕ is angle made by envelope with σ axis.
- The component 'c' of shear strength is called as cohesion which is independent of normal stress which hold soil particle together and d-represent angle of internal friction.

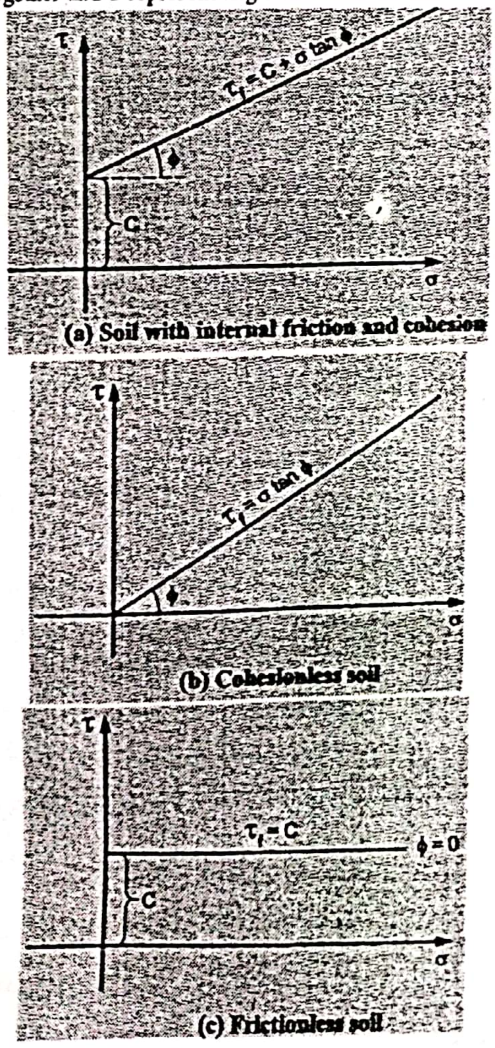


Fig. 3.10.3 : Mohr-Coulomb theory

- Failure occurs when Mohr circle touches failure envelope or one can say at a maximum obliquin (β_{max}) in which resultant touches Mohr circle and when σ and $\bar{c}$ gives a critical combination failure occurs.

Incase where $c = 0$ the graph starts from origin and equation becomes.

$$s = \sigma \tan \phi$$

- Incase where $\phi = 0$ i.e. friction less or fully cohesive soil the line will be horizontal.
- Later it will be found that c and ϕ depends upon number of factor like water content drainage conditions and lesting condition.
- Thus terzaghi shows that the effective stress control the shear strength and hence the equation gets modified

$$\therefore \quad s = c' + \bar{\sigma} \tan \phi'$$

c' = Effective cohesion intercept

ϕ' = Effective angle of shearing resistance

$\bar{\sigma}$ = Effective stress

This above equation is called as Mohr. Coulomb equation for shear strength of soil.

Syllabus Topic : Strength Envelope

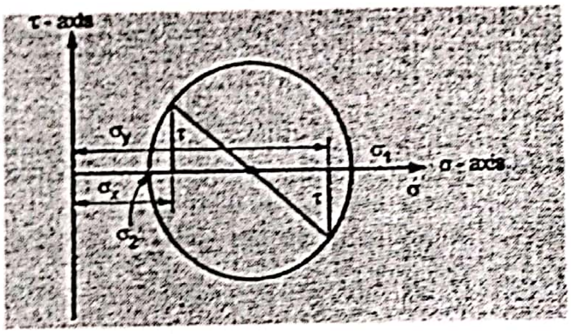
3.10.1 Strength Envelope

→ **(MSBTE - S-11, W-12)**

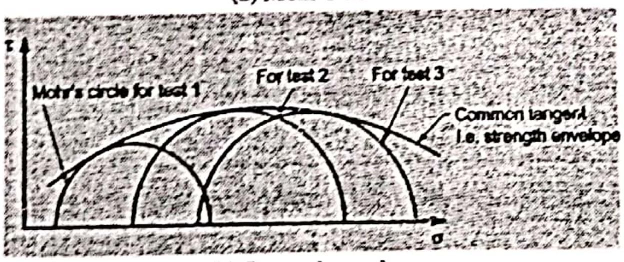
Q. Draw shear strength envelope for non-cohesive soil. (S-11, W-12)

- As soil is a complex material, it is very difficult to give a criterion of failure and also to define the plane of failure.
- For a given value of τ_f and σ, a Mohr's circle can be drawn by finding the stress, σ_x and σ_y.

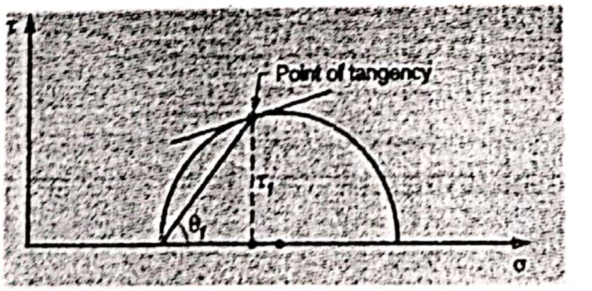
- This is shown in Fig. 3.10.4(a). For a series of tests on the same soil, different Mohr's circles can be drawn. This is shown in Fig. 3.10.4(b).
- If a common tangent is drawn to all the Mohr's circles, it will represent the values of τ as a function of σ, at failure condition.
- This common tangent is known as the strength envelope of the soil. See Fig. 3.10.4(b).
- Further, if a perpendicular is drawn from the point of tangency, it will give the failure shear stress corresponding to a particular normal stress.
- The angle shown as θ_f will give the angle of the failure plane.
- There will be two mutually perpendicular failure planes.



(a) Mohr's circle



(b) Strength envelope



(c) Failure plane and failure stress

Fig. 3.10.4 : Strength Envelope

Revised Mohr Coulomb equation

- The study and research shown the parameter c and ϕ in equation are not the fundamental properties of soil as it was assumed by Coulomb these parameters depend upon number of factors.
- Such as water content, drainage conditions, their testing and current practice is that to consider these as the mathematical parameters which represents failure conditions for a particular soil under given condition and that is why c and ϕ are now called as cohesion intercept and angle of shearing resistance. This indicates the intercept and slope of failure envelope respectively.

Thus equation can be written as :

$$S = c' + \bar{\sigma} \tan \phi'$$

- Where c' and ϕ' are the cohesion intercept and angle of shearing resistance in terms of effective stresses, this above equation is called as Revised Mohr Columb's equation. For shear strength of soil.

Syllabus Topic : Strength Equation for Purely Cohesive and Cohesionless Soils

3.10.2 Strength Equation

→ (MSBTE - S-14)

Q. Show graphically and state shear strength equation for cohesive and cohesion less soils. (S-14)

- The strength equation for a soil gives the relation between a normal stress applied on a soil and the corresponding shear stress at failure i.e. the shear strength of the soil.
- As discussed earlier, the strength equation is,

$$\tau_f = C + \sigma \tan \phi$$

Limitations of Mohr coulomb theory

(1) The angle of failure plane found is not correct.

(2) For some clayey soil as there is no fixed relation in between normal and shear stresses and on the plane of failure it can not be used for such soil.

(3) It neglects the effect of intermediate principle stresses.

(4) It gives curved failure envelope by straight line which may not give correct result.

3.11 Purely Cohesive and Cohesionless Soils

→ (MSBTE – W-08, S-09, W-14, W-17)

Q. Define : (i) Cohesive soil (ii) Cohesionless soil. (W-08)

Q. What do you mean by cohesive and cohesionless soils ? (S-09)

Q. Draw shear strength envelope for purely cohesive and cohesionless soil with its equations. (W-14, W-17)

☞ ***Definition of purely cohesive soils :*** *In some soils such as clays, organic clays, peat, etc. there is almost zero frictional resistance between the grains. These soils are called **purely cohesive soils.** i.e. ($\phi = 0$).*

– Whatever shear strength these soils possess is due to the cohesion. Thus, for purely cohesive or frictionless soils,

$$\tau_f = C$$

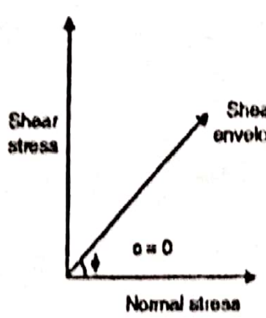
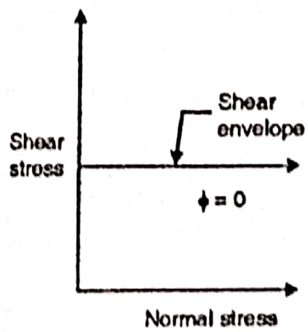
And the graph will be as shown in Fig. 3.11.1.

☞ ***Definition of cohesionless soils :*** *In some other soils, such as completely dry, clean sand, the cohesion is zero and whatever resistance to shear is present is completely due to friction. Such soils are called cohesionless soils. For cohesionless soils, ($c' = 0$).*

$$\tau_f = \sigma \tan \phi$$

and the graph will be as shown in Fig. 3.11.2.

☞ **Difference between cohesionless soil and purely cohesive soil**

Sr. No.	Cohesionless soil	Purely cohesive soil
1.	In cohesionless soil there is no cohesion or attraction between individual particles.	There exists inter attraction.
2.	In cohesionless soil the particles have small specific surface due to their size and shape.	In purely cohesive soil the specific surface of particles is large because of their size and shape.
3.	Behaviour of particles is governed by mass or gravity forces. e.g. sand and gravel.	Behaviour of particles is governed by the surface force. e.g. saturated clays and silt.
4.	 Fig. 3.11.1	 Fig. 3.11.2

3.12 Shear Strength Determination

➔ (MSBTE - S-10, W-11)

> Q. Enlist atleast two laboratory tests to determine shear strength of soil. *(S-10, W-11)*

Various tests are available for determination of shear strength, either in the field or in the laboratory. These are discussed below :

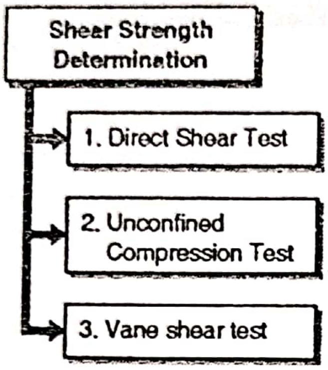


Fig. C3.3 : Shear Strength Determination

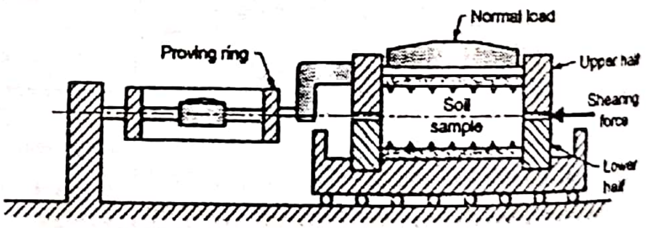
Syllabus Topic : Direct Shear Test – Laboratory Methods

3.12.1 Laboratory Determination of Shear Strength of Soil – Direct Shear Test : IS 2720 - Part 39/sec 1-1977- Reaffirmed 1987

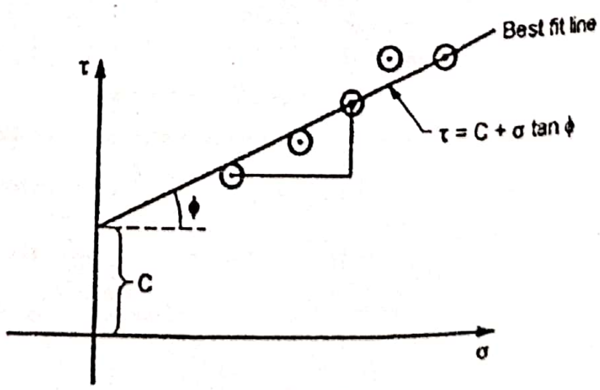
➔ **(MSBTE - S-09, W-10, S-11, W-12, W-14, W-17)**

> Q. Explain laboratory method of determination of shear strength by Direct Shear Test. *(S-09, W-12)*
> Q. Explain Box Shear Test in brief. *(W-10)*
> Q. Explain with figure laboratory determination of shear strength of soil with direct shear test. *(S-11)*
> Q. Explain direct shear test carried out on given soil sample. *(W-14)*
> Q. Describe direct shear test with a neat sketch. *(W-15)*
> Q. State the advantages of Direct Shear Test and its any two limitations. *(W-17)*

- This is the oldest shear test method in use and still most common because of its simplicity. It is also known as shear box test.
- The soil specimen is confined in a metal box that is split horizontally.
- If the specimen is fully or partially saturated, perforated metal plates and porous stones are placed above and below the specimen for drainage.
- If the specimen is dry, solid metal plates are used.
- A pressure pad is placed on top and the entire box is placed in a trolley.
- The upper half of the box is fixed to a support through a proving ring and the lower half of the box is pushed at a constant rate of strain.
- A vertical load is applied on the pressure pad. At time of failure, shear stress is measured by the proving ring.
- Then the test is repeated for another sample of the same soil, for a different vertical load on the pressure pad.
- 4-5 repetitions are made for 4-5 different normal loads.
- By dividing normal load and corresponding shear load at failure by the internal horizontal area of the shear box, the normal and shear stress values can be obtained.
- These values are plotted and a best fit straight line through these points gives the strength envelope. From this C and ϕ can be determined.
- The arrangement is shown in Fig. 3.12.1.



(a) Test arrangement



(b) Graph

Fig. 3.12.1 : Direct shear test

- Thus, the direct shear test is the most commonly used test as it has following advantages.

Advantages

→ (MSBTE – W-09, S-11, S-13, W-13, S-15, S-18)

> **Q. State advantages of Direct Shear test.**
> ***(W-09, S-11, S-13, W-13, S-15, S-18)***

1. Test is simple and fast.
2. Drainage is quick due to less thickness of sample.

Disadvantages

→ (MSBTE - W-09, S-11, S-13, W-13, S-15, S-18)

> **Q. State disadvantages of Direct Shear test.**
> ***(W-09, S-11, S-13, W-13, S-15, S-18)***

1. Failure of soil specimen is always along a horizontal plane, which may not be very realistic.
2. If any large soil particles or stones etc. are present at failure plane, it will give wrong results.
3. Actual field condition is not simulated in the set up.
4. Measurement of pore pressure is not possible.

3.12.2 Unconfined Compression Test : IS 2720 Part 10 – 1973

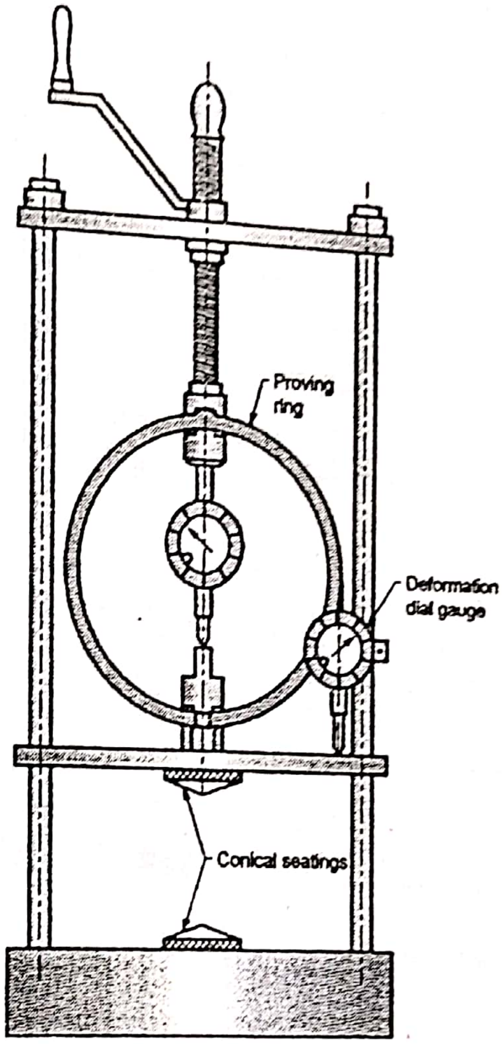
→ (MSBTE - S-13)

> **Q. Write the procedure for unconfined compression test for determining shear strength of cohesive soils with sketch. *(S-13)***

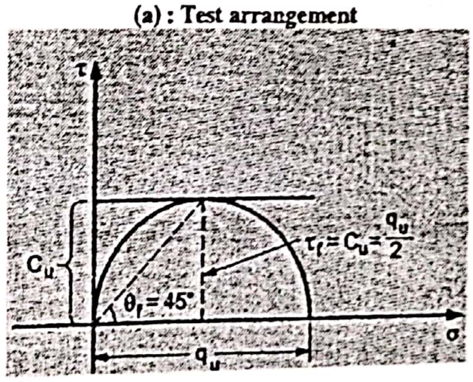
- This test is especially useful for homogenous cohesive clayey soils. It cannot be used for dry, non-cohesive, loose soils.
- In this test, a cylindrical soil specimen is fitted between two plates, with slight conical projections for firm grip on the specimen, and a compressive axial stress is applied on it till it fails.
- Since the test is quick, water is not allowed to drain out from the specimen, hence it is an undrained test.
- If q_u is the stress at failure, then the shear strength is given as,

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

- The arrangement is shown in Fig. 3.12.2. The Mohr's circle in this case is drawn and the various parameters are shown.
- Internal friction is assumed to be zero as this test can be used only for purely cohesive soils.



(a) : Test arrangement



(b) Mohr's circle

Fig. 3.12.2 : Unconfined compression test

Advantages of Unconfined Compression Test

(1) It is quick and convenient for measuring shear strength.

(2) It is useful for homogeneous cohesive clayey soil.

(3) It is suitable for measuring unconsolidated, undrained shear strength of saturated class.

(4) It is used to measure in situ strength and very useful for field test.

Syllabus Topic : Vane Shear Test – Laboratory Methods

3.12.3 Vane Shear Test : IS 2720

→ (MSBTE - S-08, S-12, W-16)

> Q. Write step by step procedure for determination of shear strength of soil by vane shear test. Give formula for shear strength. *(S-08)*
>
> Q. Explain vane shear test (IS : 2720) to determine shear strength. *(S-12)*
>
> Q. Explain vane shear test to determine shear strength of soil specimen in laboratory with neat sketch. *(W-16)*

- This is a test preferred when shear strength of soil in the undisturbed state is required.
- For some soils, it is very difficult to get undisturbed sample.
- In that case, the field vane shear test is very useful.
- In this test, a shaft to which 4 vanes are welded is slowly penetrated in the ground (in an undisturbed large sample brought to the laboratory) and by turning the shaft slowly, the torque required for failure of soil is measured.
- At the time of failure, a cylindrical portion of soil is sheared off from rest of the soil mass.

By using following formula, shear strength τ_f can be found out.

$$\tau_f = \frac{T}{\pi\left(\frac{d^2H}{2} + \frac{d^3}{6}\right)}$$

When the vanes penetrate well below the top surface of the soil, or,

$$\tau_f = \frac{T}{\pi\left(\frac{d^2H}{2} + \frac{d^3}{12}\right)}$$

When the vanes are flush with the top surface of soil.

Where,

τ_f = shear strength,

T = torque applied,

H = height of vanes,

d = diameter of circle formed on rotating vanes.

The arrangement is shown in Fig. 3.12.3.

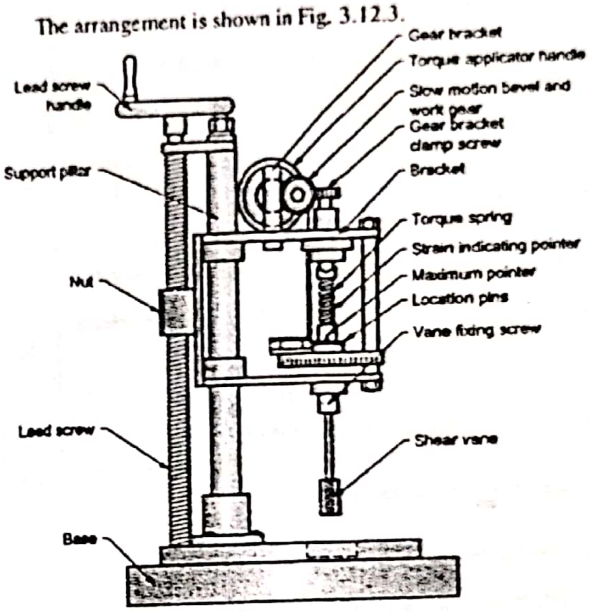


Fig. 3.12.3 : Vane shear apparatus

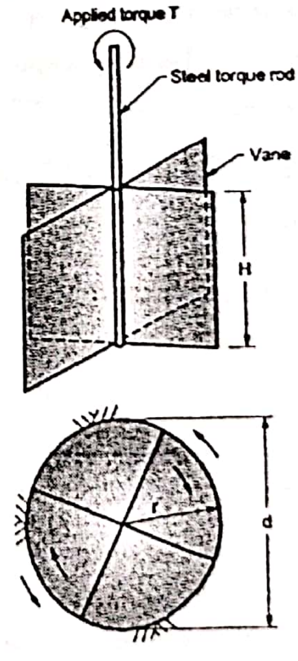


Fig. 3.12.4 : Vane shear test

☞ Experimental procedure

1. Prepare two or three specimens of the soil sample of dimensions of at least 33.5 mm diameter and 75 mm length in specimen. (L/D ratio 2 or 3).
2. Mount the specimen container with the specimen on the base of the vane shear apparatus. If the specimen container is closed at one end, it should be provided with a hole of about 1 mm diameter at the bottom.
3. Gently lower the shear vanes into the specimen to their full length without disturbing the soil specimen. The top of the vanes should be atleast 10 mm below the top of the specimen. Note the readings of the angle of twist.
4. Rotate the vanes at an uniform rate say 0.1°/s by suitable operating the torque application handle until the specimen fails.
5. Note the final reading of the angle of twist.
6. Find the value of blade height in cm.
7. Find the value of blade width in cm.

3.12.4 Plotting Strength Envelope

- In all the shear tests, plotting of strength envelopes is done with the help of Mohr's circle. Two or more samples must be tested for drawing the strength envelope.
- Only in unconfined test, only one Mohr's circle is drawn and a horizontal tangent to it gives strength envelope.
- In case of other tests, two or more Mohr's circles corresponding to the results are drawn aud a best-fit common tangent to these circles, gives the strength envelope. See Fig. 3.12.2 (a) and 3.12.2 (b).

3.12.5 Determining Shear Strength Parameters of Soil

- The cohesion 'C' measured in N/mm^2 and the angle ϕ are called the parameters of the shear strength τ_f measured in N/mm^2.
- As shown in Fig. 3.12.1 (b) after plotting any test results, the shear strength parameters C and ϕ can be directly measured from the graph.
- In case of direct and triaxial tests (triaxial test is not in the syllabus), C and ϕ can be determined without plotting the values simply by solving simultaneous equations generated by putting different values of σ and τ_f in the equation $\tau_f = C + \sigma \tan \phi$.

☞ **Advantages of Vane Shear Test**

(1) It can used to determine sensitivity of soil.

(2) It is used for determination of underdrained shear strength of non fissured fully saturated clay.

(3) It is simple and quick.

3.13 Factors Affecting Shear Strength

→ **(MSBTE - W-16)**

Q. State any four factors which affect shear strength of soil. (W-16)

Cohesion less soil	Cohesive soil
(1) Gradation	(1) Plastic index
(2) Shape of particle	(2) Clay content
(3) Pressure	(3) Drainage condition
(4) Denseness	(4) Pressure
(5) Moisture	

3.13.1 For Cohesionless Soil

(1) Gradation : Exhibit greater strength incase of well graded sand.

(2) Shape : Max. angular and sharp edge particle more will be the strength

(3) Pressure : With increase in confining pressure shear strength increases.

(4) Denseness : More is denseness more is shear strength.

(5) Moisture : IFS and is saturated apparent cohesion is destroyed.

3.13.2 For Cohesive Soil

(1) Plasticity index : Value of ϕ decreases with increase in plasticity index.

(2) Clay content : As clay content increases angle of shearing resistance decreases.

(3) Drainage condition : Less strength if drainage is not proper.

(4) Pressure : Shear strength of clay increases with increase in confining strength.

3.13.3 Drainage Conditions For Various Shear Tests in Laboratory

→ (MSBTE - W-10)

Q. State the drainage conditions adopted for various shear tests in laboratory. (W-10)

The various drainage condition adopted for various shear test in laboratory are

1) Unconsolidated undrained test.

2) Consolidated undrained test.

3) Drained test.

3.14 Examples

Ex. 3.14.1

In an unconfined compression test, the specimen has a diameter of 5 cm. It fails at load of 105 N. Find the shear strength of soil.

Soln. :

$$q_u = \text{Stress at failure} = \frac{\text{Failure load}}{\text{Area}}$$

$$\text{Area} = \frac{\pi}{4} \times 50^2 = 1963.495 \text{ mm}^2$$

$$q_u = \frac{105}{1963.495} = 0.0534 \text{ N/mm}^2$$

$$\tau_f = \frac{q_u}{2} = 0.0267 \text{ N/mm}^2$$

$$\tau_f = 0.0267 \text{ N/mm}^2$$

Ex. 3.14.2

In a shear box test, the following observations were recorded at the failure of soil specimens :

Normal stress N/mm^2	1.0	1.5	2.0	2.5
Shear stress N/mm^2.	0.9	1.15	1.4	1.65

Find C and ϕ for the soil.

Soln. :

Putting the first two values in the equation,

$$\tau_f = C + \sigma \tan \phi$$

We get, $$0.9 = C + 1 \times \tan \phi \quad ...(1)$$

and $$1.15 = C + 1.5 \times \tan \phi \quad ...(2)$$

Subtracting Equation (1) from Equation (2),

$$0.25 = 0.5 \tan \phi$$

$$\therefore \quad \tan \phi = \frac{0.25}{0.5} = 0.5$$

$$\therefore \quad \phi = 26.56^0$$

Putting in Equation (1),

$$0.9 = C + 0.5$$

$$\therefore \quad C = 0.9 - 0.5 = 0.4 \text{ N/mm}^2$$

In the same way C and ϕ can be calculated for any other pair of values and then average can be taken.

Graphical solution

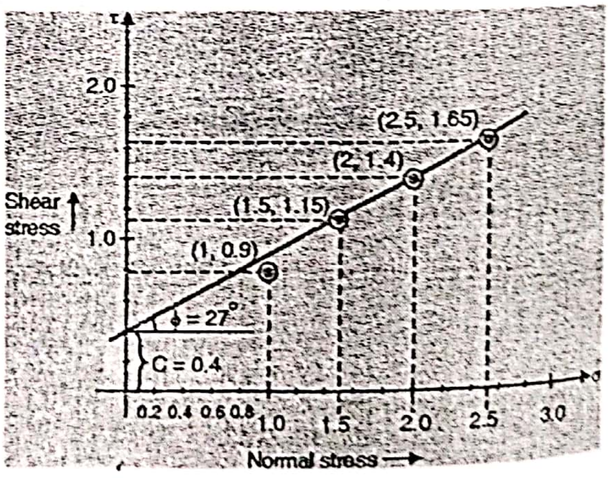


Fig. P. 3.14.2

By adopting a scale, say 1 cm = 0.2 N/mm². The four points are plotted on the graph paper. Then a line is passed through them. On this graph C and ϕ can directly be measured.

Ex. 3.14.3

Determine shear strength of soil sample tested by a vane of diameter 75 mm and height 75 mm. If torque applied is 30 Nm.

Soln. :

When it is not given whether the vanes are well below the ground surface or not, it is assumed that they are and the formula.

$$\tau_f = \frac{T}{\pi\left(\frac{d^2 H}{2}+\frac{d^3}{6}\right)}$$

Here, $T = 30\text{ Nm} = 30 \times 10^3$ N-mm.

$d = 75$ mm

$H = 75$ mm.

$$\therefore \tau_f = \frac{30 \times 10^3}{\pi\left(\frac{75^2 \times 75}{2}+\frac{75^3}{6}\right)}$$

$$\therefore \tau_f = \frac{30 \times 10^3}{\pi \times 281250}$$

$$\tau_f = 0.03395\text{ N/mm}^2$$

Ex.3.14.4

In the direct shear test the following observations were made

Normal load in N	50	100	150	200	250
Shear load in N	90	110	130	150	170

Size of shear box 60 mm x 60 mm. Plot the failure envelope for the soil and find the value of angle of shearing resistance and cohesion.

Soln. :

$$\text{Stress} = \frac{\text{Load}}{\text{Area}}$$

where area = 60×60 mm = 3600 mm²

Normal load in N.	50	100	150	200	250
Normal stress in N/mm² (σ)	0.014	0.028	0.042	0.056	0.069
Shear load in N.	90	110	130	150	170
Shear stress in N/mm² (S)	0.025	0.031	0.036	0.042	0.047

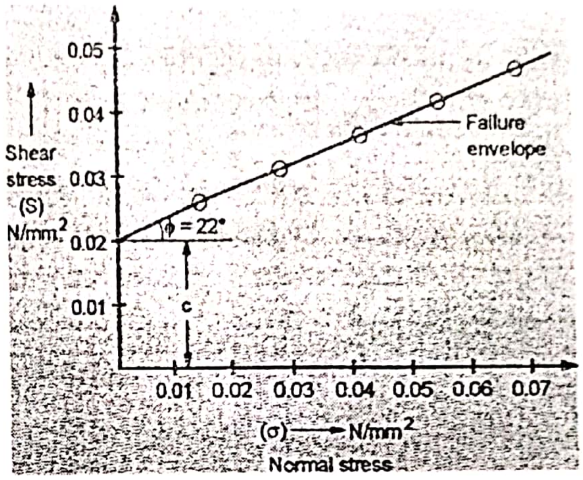


Fig. P. 3.14.4

(i) Cohesion (c) = 0.02 N/mm²

(ii) Angle of shearing resistance = ϕ = 22°

Ex. 3.14.5 S-08

Table below gives observations for normal stress and maximum shear stress for the specimens of sand clay tested in the shear box under undrained conditions. Plot the failure envelope for the soil and determine the values of shear parameters the values of shear parameters c and ϕ.

Normal stress (Kg / cm²)	0.5	1.0	1.5	2.0
Shear stress (Kg / cm²)	0.55	0.76	0.97	1.17

Soln. :

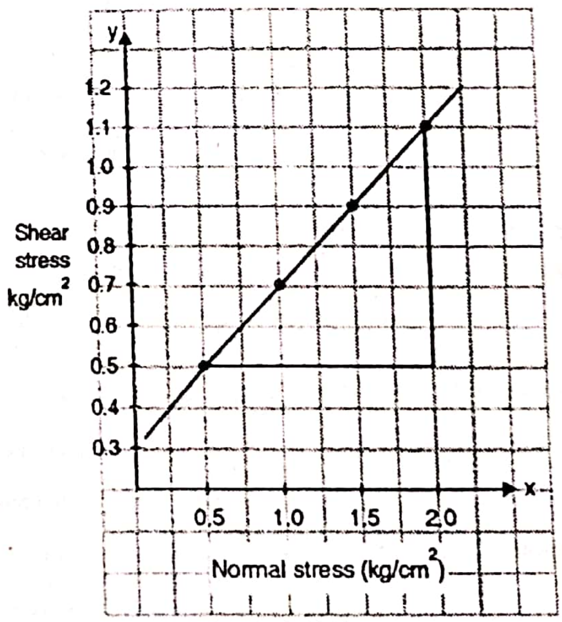


Fig. P. 3.14.5

$$m = \frac{y_2 - y_1}{x_2 - x_1} = \frac{1.17 - 0.55}{2.0 - 0.5} = 0.413$$

$$\phi = \tan^{-1}(m) = \tan^{-1}(0.413) = 22.45\ \%$$

$$s = c + \sigma_n \tan\phi$$

$$0.55 = c + 0.5 \times 0.413 = c + 0.206$$

$$c = 0.55 - 0.206 = 0.343\ N/mm^2$$

(i) Cohesion $C = 0.35\ kg/cm^2$

(ii) Angle of shearing resistance $\phi = 22.5°$

Ex. 3.14.6

Following readings were taken in a direct shear test on a soil sample.

Normal stress in N/mm^2	0.1	0.2	0.3	0.4
Shear stress at failure in N/mm^2	0.110	0.152	0.193	0.235

Determine the values of C and ϕ.

Soln. :

$$M = \frac{y_2 - y_1}{x_2 - x_1} = \frac{0.235 - 0.110}{0.4 - 0.1} = 0.4167$$

$$\phi = \tan^{-1} m$$

$$= \tan^{-1} 0.4167 = 22.62\ \%$$

$$s = C + \sigma_n \cdot \tan\phi$$

$$0.11 = C + 0.1 \times 0.4167$$

$$C = 0.068\ N/mm^2.$$

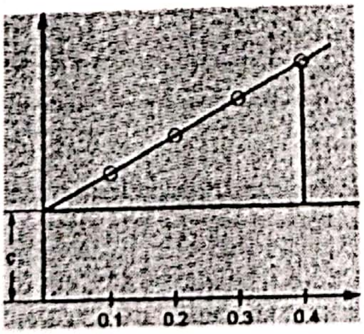


Fig. P. 3.14.6

Ex. 3.14.7

In a shear box test, following observations were recorded at the failure of soil specimen.

Normal stress kg/cm^2	1.0	1.50	2.50	3.50
Shearstress kg/cm^2	0.80	1.15	1.42	1.70

Find the value of C and ϕ.

Soln. : Putting the first two values in the equation,

$$\tau_f = C + \sigma \tan\phi$$

$$0.8 = C + 1 \times \tan\phi \qquad ...(1)$$

and

$$1.15 = C + 1.5 \times \tan\phi \qquad ...(2)$$

Subtracting Equation (1) from Equation (2).

$$0.35 = 0.5 \tan\phi$$

$$\therefore \tan\phi = \frac{0.35}{0.5} = 0.7$$

$$\therefore \phi = 34.99^0$$

Putting in Equation (1),

$$0.8 = C + 0.7$$

$$\therefore C = 0.8 - 0.7 = 0.1\ N/mm^2$$

In the same way C and ϕ can be calculated for any other pair of values and then average can be taken.

Graphical solution

By adopting a scale, say 1 cm = 0.2 N/mm^2. The four points are plotted on the graph paper. Then a line is passed through them. On this graph C and ϕ can directly be measured.

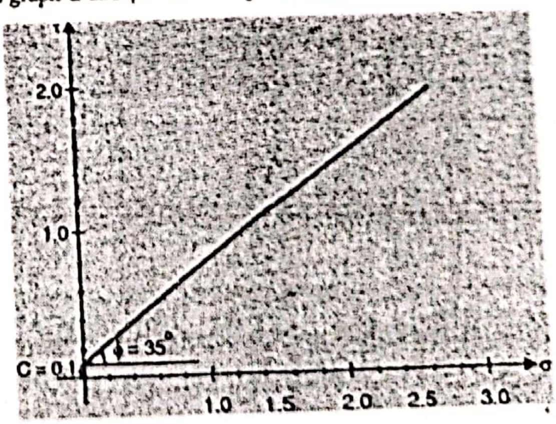


Fig. P. 3.14.7

Ex. 3.14.8

A sample of dry sand tested in direct shear test gives failure shear stress of 100 kN/m² at a normal stress of 200 kN/m². Determine the law of shear strength. $s = C + \sigma_n \cdot \tan\phi$.

Soln. :

In case of dry sand i.e. cohesionless soil C = 0

$$\therefore \quad 100 = 0 + 200 \tan\phi$$

$$\therefore \quad \tan\phi = \frac{100}{200}$$

$$\phi = \tan^{-1}\left(\frac{1}{2}\right) = 26.56°$$

$\therefore$ Law of shear strength is,

$$s = \sigma_n + \tan 26.56°$$

Ex. 3.14.9

Determine the shear strength of a sensitive clay if a shear vane 75 mm diameter × 115 mm gave maximum torque of 45 N/m.

Soln. :

$$\text{diameter, } d = 75 \text{ mm}$$

$$\text{height of vane, } H = 115 \text{ mm}$$

$$\text{torque, } T = 45 \text{ N.m} = 4500 \text{ N.mm}$$

We have undrained shear strength

$$C_u = \frac{T}{\pi d^2 \left[\frac{H}{2} + \frac{d}{6}\right]}$$

$$= \frac{45000}{\pi \times 75^2 \left[\frac{115}{2} + \frac{75}{6}\right]} = 0.036 \text{ N/mm}^2$$

$$= 0.36 \text{ kg/cm}^2$$

Ex. 3.14.10

A cylinder of soil fails under an axial vertical stress 140 kN/m², when it is laterally unconfined. The failure plane extends on an angle of 48° with the horizontal. Calculate the values of cohesion and the angle of internal friction of the soil.

Soln. :

$$\alpha = 45 + \frac{\phi_u}{2}$$

$$\therefore \quad \phi_u = 2(48 - 45°) = 6°$$

$$\tan\alpha = \tan\left(45° + \frac{\phi_u}{2}\right) = \tan 48 = 1.1106$$

Sample is unconfined $\sigma_3 = 0$

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2C\tan\alpha$$

$$140 = 2C \tan 48$$

$$C = 63.06 \text{ kN/m}^2$$

Ex. 3.14.11 W-11

In an unconfined compression test, the specimen has a diameter of 5 cm. it fails at load of 110N. Find the shear strength of soil.

Soln. :

$$q_u = \text{stress at failure} = \frac{\text{Filure load}}{\text{Area}}$$

$$\text{Area } A = \frac{\pi}{4} \times 50^2 = 1963.495 \text{ mm}^2$$

$$\therefore \quad q_u = \frac{110}{1963.495} = 0.056 \text{ N/mm}^2$$

$$I_f = \frac{94}{2} = 0.056 = 0.028 \text{ N/mm}^2$$

Ex. 3.14.12 S-12, S-14

In direct shear test the following observations were made :

Normal stress σ N/cm²	2	4	6	8	10
Shear stress S N/cm²	3.6	4.4	5.2	6.0	6.8

Size of shear box 5 × 5 cm. Plot failure envelope for soil. And find the value of angle of shearing resistance and cohesion.

Soln. :

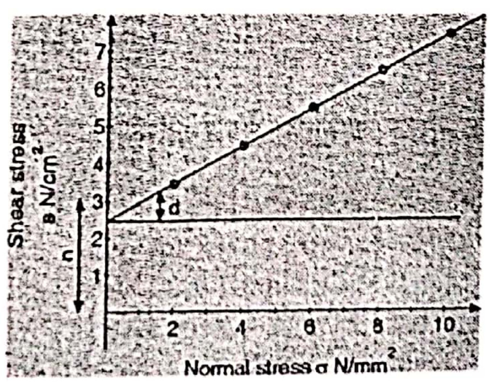


Fig. P. 3.14.12

) Angle of shearing resistance $\phi = 22°$

$$m = \frac{y_2 - y_1}{x_2 - x_1} = \frac{6.8 - 3.6}{10 - 2} = 0.4$$

$$\phi = \tan^{-1}(m) = \tan^{-1}(0.4) = 21.80 \%$$

$$\therefore \phi = 21.80 \%$$

i) Cohesion

$$s = c + \sigma_n \tan\phi$$

$$3.6 = c + 2 \times 0.4$$

$$\text{Cohesion (c)} = 2.8 \text{ N/mm}^2$$

x. 3.14.13 S-13

etermine shear strength of soil sample tested by a vane of ameter 75 mm and height 75 mm. if torque applied is 0 mm.

oln. :

Assuming unit of T = 40 N-mm or 40 N-m.

$$\text{hear strength } C_u = \frac{T}{\pi d^2\left[\frac{H}{2} + \frac{d}{6}\right]}$$

$$\text{Where, Torque } T = \frac{40 \text{ N-m}}{\text{Assuming}} = 40000 \text{ N-mm}$$

Diameter of vane, d = 75 mm

Height of vane, H = 75 mm

$$C_u = \frac{40000}{3.14\,(75)^2\left[\frac{75}{2} + \frac{75}{6}\right]}$$

$$= \frac{40000}{5625 \times 50} = \frac{40000}{281250}$$

$$C_u = 0.142 \text{ N/mm}^2$$

If $T = 40$ N-mm then

$$C_u = 1.59 \times 10^{-4} \text{ N/mm}^2$$

Ex. 3.14.14 W-15

At the bottom of a bore hole, a vane 120 mm long and 80 mm in diameter was pressed into soft clay. Torque was applied and gradually increased to 57 N-m, when failure took place estimate the shear strength of clay.

Soln. :

Given : H = 120 mm, D = 80 mm, T = 57,

Nm = 57000N-mm

$$\text{Share strength } C_u = \frac{T}{\pi d^2\left(\frac{H}{2} + \frac{d}{2}\right)}$$

$$= \frac{57000}{\pi\, 80^2\left(\frac{120}{2} + \frac{80}{2}\right)} = \mathbf{0.0283 \text{ N/mm}^2}$$

Chapter Ends

□□□

CHAPTER 4

Bearing Capacity of Soil

Syllabus

4.1 Bearing capacity and theory of earth pressure: Concept of bearing capacity, ultimate bearing capacity, safe bearing capacity and allowable bearing pressure, Introduction to Terzaghi's analysis and assumptions made, effect of water table on bearing capacity.

4.2 Field methods for determination of bearing capacity – Plate load test and standard penetration test. Test procedures as Per IS: 1888 & IS: 2131.

4.3 Definition of earth pressure, active earth pressure and passive earth pressure for no surcharge condition, coefficient of earth pressure, Rankine's theory and assumptions made for non-cohesive Soils.

Introduction

The most widespread use of soil is as a substratum over which the foundation of a structure and thus ultimately the structure rests. The soil at that depth must be strong enough to support this entire load. The load of foundations is mainly compressive. But shear comes into play due to lateral forces on soil. The strength for resisting the compressive stress is called the bearing capacity of soil.

4.1 Bearing Capacity and its Importance

→ (MSBTE – W-11)

Q. Define : Bearing Capacity of soil. (W-11)

Bearing Capacity

Definition of Bearing Capacity : It is defined as the maximum average intensity in of applied pressured that loaded area can carry before it fails shear.

The knowledge of bearing capacity plays an important role in the following aspects of civil engineering :

Bearing Capacity and its Importance

1. Foundation design
2. Consolidation and compaction of earth
3. Earth and gravity dams
4. Retaining walls
5. Tunnels etc

Fig. C4.1 : Bearing Capacity and its Importance

→ 1. Foundation design

- Entire load of any structure ultimately comes on the foundation. The foundation then transmits this load on the soil in contact with the base of the foundation.
- Thus the area of base of foundation in contact with the soil ultimately decides the intensity of the load per unit area. Any soil has some fixed capacity of tolerating the intensity of load per unit area. This capacity mainly constitutes the bearing capacity of soil.

- In foundation design, the load of the structure does not change but the area of base of foundation required, so that the load intensity is less than that the soil can tolerate, can be calculated. This is done along with a factor of safety. Thus, bearing capacity is important in foundation design.

→ **2. Consolidation and compaction of earth**

- In many civil engineering situations, the properties of soil or earth either change or are modified and enhanced. If the properties change naturally, it is known as consolidation and if we deliberately change and enhance the properties, it is known as compaction.
- In both cases, the density of soil changes. This change in density leads to increase in the shear strength, decrease in permeability and increase in the bearing capacity.
- Thus, bearing capacity is important in determining various factors and parameters while undertaking the process of compaction or estimating the amount of consolidation and hence the settlement of the structure which will occur in the future.

→ **3. Earth and gravity dams**

- Earth dams are entirely made up of soil and hence require detailed knowledge of soil to be used for its construction as well as the soil on which it will stand.
- Earth dams are mainly of two types: homogeneous and non-homogeneous. In non-homogeneous type, the core is made up of highly compacted impervious soil. The compaction involves the knowledge of bearing capacity.
- In case of gravity dams, the uplift pressure below the dam and the total load intensity of the dam coming over the soil in contact with the foundation base necessitate the knowledge of bearing capacity of soil.

→ **4. Retaining walls**

- Retaining walls form a special area of soil mechanics, as the design of retaining wall involves the mobilisation of soil strength under different conditions of limiting equilibrium. Thus active, passive and neutral earth pressure comes into picture.
- These are lateral pressures whereas the bearing capacity is essentially a vertical compressive stress.
- Still, in the analysis of active, passive or neutral earth pressures, the bearing capacity is one of the important factors.
- In Rankine's theory of earth pressures also, the bearing capacity plays an important role.

→ **5. Tunnels, etc**

Tunnels, road subgrades, stability of open cuts, docks and harbour construction, in short all aspects of civil engineering where excavation of soil, safety and use of soil as the ultimate substratum supporting the load are required, bearing capacity plays a very important role.

Syllabus Topic : Concept of Bearing Capacity

4.1.1 Concept of Bearing Capacity

- The bearing capacity forms the main criteria in the design of foundation by conventional methods.
- The stresses induced in the soil should be within limit.
- The stresses are mainly shear stress and bearing stress or compressive stress.
- Lateral movement of soil below the foundation and shear failure can be one cause of failure.
- The other criteria are excessive settlement and uneven settlement. Thus bearing capacity plays a very important role in design of all types of foundations.
- The bearing capacity is qualified in different values as given below :

Syllabus Topic : Ultimate Bearing Capacity

4.1.2 Ultimate Bearing Capacity (q_u)

→ **(MSBTE – S-08, W-08, W-12, S-13, W-13, W-14, W-15, S-16)**

Q. Define ultimate bearing capacity. *(S-08, W-08, W-12, S-13, W-13, W-14, W-15, S-16)*

> ☞ ***Definition of Ultimate bearing capacity :*** *Maximum unit pressure that any soil can bear without rupture in shear or without excessive settlement of the supported structure is called the* ***Ultimate bearing capacity of the soil.***

- It is also called Ultimate bearing value or Ultimate bearing pressure. When only "bearing capacity" is mentioned, it is understood to mean "ultimate bearing capacity."

Syllabus Topic : Safe Bearing Capacity

4.1.3 Safe Bearing Capacity (q_s)

→ **(MSBTE - W-08, W-09, W-10, S-11, W-12, S-13, W-13, W-14, W-15, S-16, S-17)**

> **Q.** Define safe bearing capacity. *(W-08, W-09, W-10, W-12, S-13, W-13, W-14, W-15, S-16, S-17)*
> **Q.** What is meant by safe bearing capacity of soil ? Explain. *(S-11)*

> ☞ ***Definition of Safe bearing capacity :*** *Safe bearing capacity is defined as the maximum pressure which the soil can tolerate without any risk of failure.*

$$q_s = \frac{\text{ultimate bearing capacity}}{\text{factor of safety}}$$

- The safe bearing capacity is the value of bearing capacity used in design of foundations.
- It is obtained from the ultimate bearing capacity, by dividing it by a suitable factor of safety. Since the estimation is very complex, quite high factor of safety is used in foundation design. The factor of safety is generally 3 to 5.

☞ **Safe bearing pressure**

> ☞ ***Definition of Safe Bearing Pressure :*** *It is the net loading intensity, including the surcharge on the foundation at which neither the soil fails due to shear rupture nor is there excessive settlement of the foundation.*

Syllabus Topic : Allowable Bearing Pressure

4.1.4 Allowable Bearing Pressure or Bearing Capacity (q_a)

→ **(MSBTE - S-08, W-09, S-17)**

> **Q.** Define allowable bearing capacity. *(S-08, W-09, S-17)*

> ☞ ***Definition of Allowable Bearing Pressure :*** *It is defined as the net loading intensity, including the surcharge on the foundation at which neither the soil fails due to shear rupture nor is there excessive settlement of the foundation.*

- But whatever is the allowable bearing pressure for a soil, it is not taken for design purposes.
- To account for uncertainty and unknown factors, always a sufficient factor of safety is used.

4.2 Theories about Bearing Capacity

- Different theories are developed for the analysis of soil behaviour at the time of failure due to the load coming through the foundations.
- Out of these, Terzaghi's analysis is the most commonly used, because it is simple, realistic and is found to give good results and safe designs. This section deals with Terzaghi's analysis.

Syllabus Topic : Introduction to Terzaghi's Analysis

4.2.1 Terzaghi's Analysis

- When the footing sinks into soil, a triangular wedge **abd** sinks in ground without any lateral movement due to the friction between foundation bottom and soil. Thus wedge abd acts as an integral part of foundation. It is designated as zone I, i.e. **Zone of no shear.**
- Due to the sinking wedge abd, wedges bde and ade are pushed out radially outwards at an angle ϕ. These wedges constitute the **Zone of radial shear.**
- The wedges of zone II rise up and due to these, the wedges **bef** and **aeg** are pushed out in the lateral direction. These wedges form the zone III i.e. Zone of linear shear. See Fig. 4.2.1.

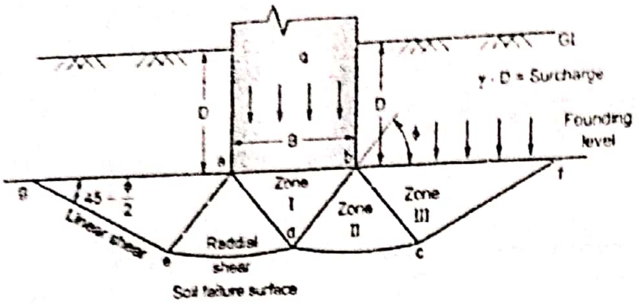


Fig. 4.2.1 : Terzaghi's theory

– At equilibrium, the force causing the sinking or downward movement and the force resisting this movement must balance.

$$\therefore \quad q_u = C N_c + \gamma D N_q + 0.5 \gamma_B N_\gamma$$

Where, C = cohesion.

N_c, N_q, N_r are the bearing capacity factors corresponding to angle of friction.

For a square footing

$$q_u = 1.2 C N_c + \gamma D N_q + 0.4 \gamma_B N_\gamma$$

For a circular footing.

$$q_u = 1.2 C N_c + \gamma D N_q + 0.3 \gamma_B N_\gamma$$

Syllabus Topic : Assumptions Made in Terzaghi's Analysis

4.2.2 Assumptions Made in Terzaghi's Analysis

→ (MSBTE - W-08, S-09, W-09, S-10, W-10, W-12, S-13, W-13, S-14, W-14, W-15, S-17, W-17)

Q. State the assumptions made in Terzaghi's analysis of bearing capacity of soil. *(W-08, S-09, W-10, W-12, S-13, W-13, S-14, W-14)*

Q. State any four assumption made in Terzaghi's analysis. *(W-09, S-10, W-11)*

Q. State any four assumptions of Terzaghi's bearing capacity theory. *(S-14)*

Q. List assumptions made by Terzaghi's analysis for soils bearing capacity. *(S-17)*

Q. Enlist assumptions of Terzaghi's bearing capacity theory. *(W-17)*

☞ **Following assumptions are made in the derivation of Terzaghi's theory**

1. The footing has a large L/B ratio and hence a two dimensional or plane strain condition is envisaged.
2. The base of the footing is rough.
3. The base of the footing is laid at a shallow depth, i.e. D ≤ B.
4. The shearing resistance of the soil between the surface and the depth D is neglected. The footing is considered as a uniform footing with surcharge γ × D.
5. General shear failure is assumed to take place and the soil volume is unchanged prior to failure.
6. The shear strength of soil is governed by Mohr-coulomb equation.

4.3 Effect of Different Factors on Bearing Capacity

→ (MSBTE - W-09, S-12, S-13, W-13, S-15, S-18)

Q. State and explain factors affecting bearing capacity. (any four) *(W-09, S-12, S-13, W-13, S-15, S-18)*

Various factors affect the bearing capacity. Out of these, the presence of water table is an important factor. Terzaghi's equation is modified for the effect of water table. Other factors on which the bearing capacity depends are :

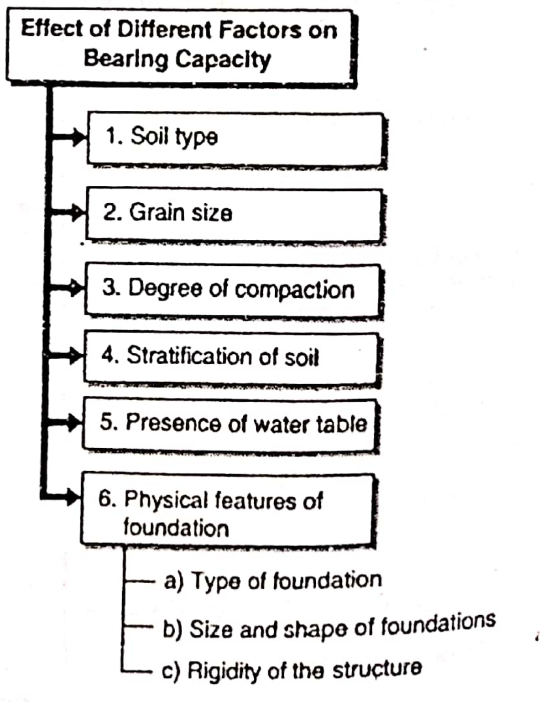


Fig. C4.2 : Effect of Different Factors on Bearing Capacity

1. Soil type

- Different soil types have different mechanisms of resisting the load coming on them. Mainly two types of mechanisms are activated when load comes on any soil.
- The internal friction or cohesion. Generally, any ordinary soil resists the load by a combination of internal friction and cohesion.
- Thus, the bearing capacity is different for purely cohesive soils where internal friction is zero, and for purely frictional soil such as dry sand where cohesion is zero.
- It is clear from this that the soil type and its values of cohesion 'c' and internal friction or angle of internal friction 'ϕ' will play an important role in the bearing capacity.
- The type of soil according to its mineral composition is also important. Organic soils have very little bearing capacity. Laterite type soils have medium bearing capacity. Soils derived from basalt granite, felspar, quartz etc. have more bearing capacity than laterite soils.
- In general, soils obtained by weathering of igneous rocks have more bearing capacity than soils derived from sedimentary rocks.

2. Grain size

- Grain size is not very important in cohesive soils. The most important factor in the bearing capacity of a cohesive soil is the cohesion of the soil.
- In case of soil with a combination of internal friction or a completely non-cohesive and frictional soil, the bearing capacity generally decreases as the grain size increases.
- Thus non-cohesive fine grained soils have more bearing capacity. But this is not always true. The real situation is dependent on the grain size distribution.
- A soil which is well graded will have more bearing capacity even if it is coarse-grained in general aspects.

3. Degree of compaction

- The soil can be compacted before the laying of foundation it. Due to compaction the main effect is the increase in density and drainage of water from soil voids.
- The secondary effects of increase in density and decrease of water content is to increase the shear strength as well as to increase the bearing capacity of the soil.
- Thus, more the amount of compaction, more will be the bearing capacity upto the limit of maximum dry density (MDD) which can be achieved for a particular soil.
- After MDD, additional compaction will not increase the bearing capacity.

4. Stratification of soil

- In case of stratified soil, the direction of stratification is very important. If the stratification is perpendicular to the direction of load coming on the soil, the bearing capacity is maximum.
- If the stratification is exactly parallel to the load coming over the soil, the bearing capacity is minimum.
- For all other cases, the bearing capacity is intermediate. See Fig. 4.3.1.

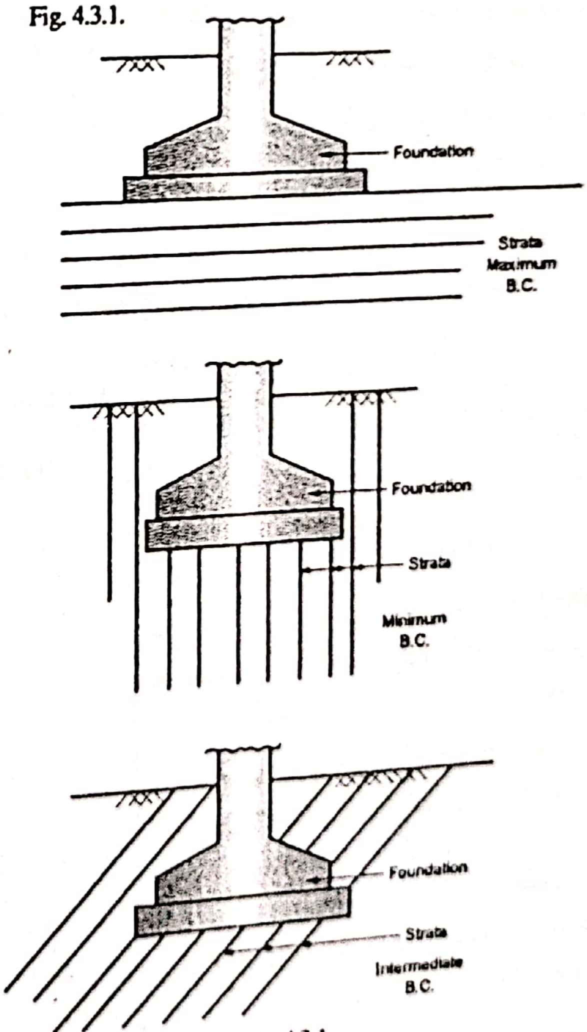


Fig. 4.3.1

→ 5. Presence of water table

- In general, water table at shallow depths is considered bad for foundation.
- The bearing capacity for non granular soils decreases with the presence of water table. Higher the water table, lesser is the bearing capacity.
- In case of purely cohesive soils, bearing capacity is not much affected by the presence or absence of water table.

→ 6. **Physical features of foundation**

Physical features of the foundation to be considered are as follows :

→ (a) **Type of foundation**

Bearing capacity of soil for shallow foundations is less than that of deep foundations.

→ (b) **Size and shape of foundations**

- In general square or rectangular foundation gives more bearing capacity than a circular foundation.
- The bearing capacity increases with the width of footing on sandy soils or granular soils. The bearing capacity also increases with depth in granular soils. In case of purely cohesive soils, width or depth do not have much effect of bearing capacity.

→ (c) **Rigidity of the structure**

More rigid structures require stronger base for foundation. Hence the more rigid structure, less will be the bearing capacity of the soil.

Syllabus Topic : Effect of Water Table on Bearing Capacity

4.3.1 Effect of Water Table on Bearing Capacity

→ **(MSBTE – S-08, S-09, W-09, S-10, W-10, S-11, S-12, W-12, S-14, W-14, W-16, S-17)**

Q. Explain the effects of water-table on bearing capacity of soil. ***(S-09, W-09, S-10, S-12, W-12, S-14, W-14, W-16, S-17)***

Q. State effect of water table on bearing capacity when
(i) Water table is well below foundation base.
(ii) Water table is above foundation base.
(S-08, W-10)

Q. What are the effects of water table on bearing capacity of soil ? ***(S-11)***

- It was observed that the rise of water table from below the foundation resulted in a decrease in the bearing capacity in granular soil because of the decrease in effective unit weight of soil due to the submerged condition.
- If the water table reaches the ground level, rising from a depth equal to or greater than the width of footing, then the bearing capacity can be reduced by 50% or more.
- In case of purely cohesive soils, the bearing capacity is not affected to that extent by the presence of water table.

For cohesionless soils,

$$q_u = \gamma D N_q + 0.5 \gamma_B N_r$$

When water table rises upto base of footing,

$$q_u = C N_C + \gamma D N_q + 0.5 \gamma_{sub B} N_r$$

When water table is at the ground level,

$$q_u = \gamma_\omega D + \gamma_{sub D} N_q + 0.5 \gamma_{sub B} N_r$$

$$\text{As } \gamma_{sub B} = \frac{1}{2} \gamma_B .$$

The value of q_u reduces by nearly half, when water table rises upto the ground surface.

4.3.2 Methods of Improving Bearing Capacity

→ **(MSBTE – W-11, S-12, W-13)**

Q. State and explain the methods of improving the bearing capacity of soil? ***(W-11)***

Q. Mention methods of improving bearing capacity. ***(S-12, W-13)***

The various methods adopted to improve bearing pressure soil are :

Methods of Improving Bearing Capacity

- 1. Compaction of soil
- 2. Replacement of poor soil
- 3. Drainage arrangement
- 4. By increasing depth and width of foundation
- 5. By use of chemicals
- 6. Grouting

Fig. C4.3 : Methods of Improving Bearing Capacity

→ **(1) Compaction of soil**

- By adopting various methods of compaction the open space or gaps between soil gets reduced and thus are less liable to displacement and thus increases density as well as shear strength of soil mass ultimately increases bearing capacity.

→ **(2) Replacement of poor soil**

- The poor soil is removed completely from the site and is replaced by soil having good qualities and thus increases the bearing capacity.

→ **(3) Drainage arrangement**

- Water is key factor which affect the bearing capacity therefore the areas where ground water table is high bearing capacity can be increases by making power drainage arrangement.

→ **(4) By increasing depth and width of foundation**

- As we proceed in downward direction denseness increases this result into increased bearing capacity along the depth.
- Similarly bearing capacity is directly proportional to width of foundation. Greater the width more will the bearing capacity as the bearing area is more.

→ **(5) By use of chemicals**

The various chemicals are used to make soil dense. These are either mixed with soil or by injecting them with solution these forms get like structure and thus improves bearing capacity e.g. Sodium silicate, calcium chloride.

→ **(6) Grouting**

If soil below foundation contains cracks, cavities or voids. It is injected under pressure with cement grout. So as to form soild mass thus increases the bearing capacity.

4.4 Field Methods for Determination of Bearing Capacity

- Determination of bearing capacity in situ is more realistic and makes better practical sense.
- Hence various field methods are devised which have to be used on the actual site where ultimately the foundation and the structure coming over it will be built. Below two field methods are discussed :

Syllabus Topic : Field Methods for Determination of Bearing Capacity – Plate Load Test, Test procedures as Per IS: 1888

4.4.1 Plate Load Test : Procedure as Per IS : 1888

→ **(MSBTE – S-08, W-08, S-09, W-10, S-11,W-11, S-12, W-12, S-13, W-13, W-14,S-15,W-15, W-16,S-17,W-17,S-18)**

Q. Draw a neat labeled sketch of plate load test set up for determination of field bearing capacity. *(S-08, S-09, W-11)*

Q. Draw a neat sketch of plate load test (Reaction by truss method). Also draw nature of load settlement curve in plate load test. *(W-08, S-11, S-13)*

Q. Explain in brief about Plate Load Test and its limitations. *(W-10, S-12)*

Q. Explain plate load test as per IS 1884. *(W-12)*

Q. Explain plate load test with figure. *(W-13)*

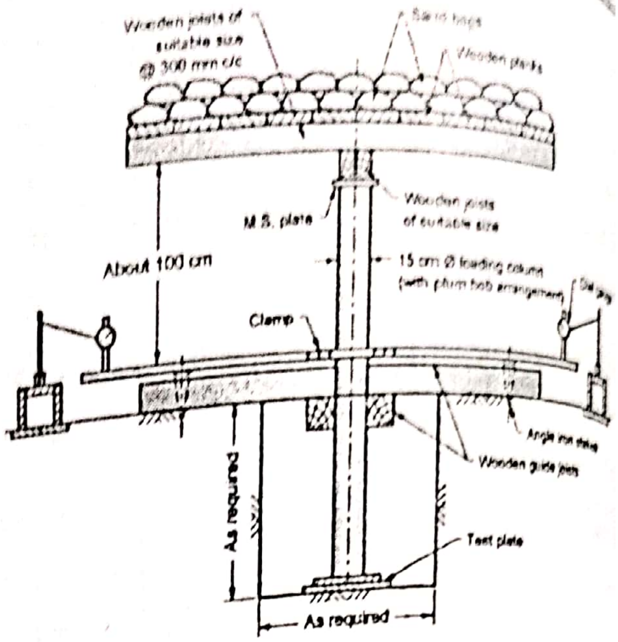
Q. Draw the experimental set up of plate load test using gravity loading. *(W-14, S-16)*

Q. Explain with neat sketch plate load test as per IS 1888 with two limitations of this test. *(S-15)*

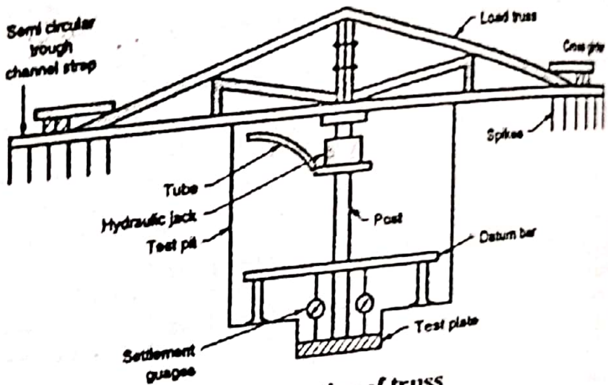
Q. Draw neat sketch of plate load test. *(W-15, S-17)*

> Q. Explain plate load test and draw a load settlement curve. (W-16)
>
> Q. Explain the step-by-step procedure for determination of plate load test with neat sketches. (W-17)
>
> Q. Explain with neat sketch plate load test as per IS 1888 by (i) Gravity loading PLAN (ii) Gravity loading SECTION (iii) Graph to show limitations of plate load test. (any two) (S-18)

1. The site where testing is to be done is selected.
2. A test pit, at least 5 times the diameter or width of the plate, and upto the depth of proposed foundation level, is dug.
3. The plate is seated firmly at the centre of the pit.
4. The dead load of all the equipment ball and socket, steel plate, loading column, jackets – is recorded before applying the load increments.
5. A minimum seating pressure of 70 gm/cm^2 is applied and removed before starting the load test.
6. A minimum load is applied to soil, in cumulative increment upto 1kg/cm^2 or $\frac{1}{5}^{th}$ of the estimated ultimate bearing capacity, which ever is lower.
7. The settlement is observed after each load increment at 1, 2.25, 4, 6.25, 9, 16, 30 minutes and thereafter at hourly intervals, and it is recorded.
8. The recording is stopped when the increase in settlement is only 0.02 mm.
9. The procedure is repeated after every increment in load.
10. The observations are plotted on a log-log scale. The settlement in mm is plotted on X-axis and load in kg/m^2 is plotted on Y-axis.
11. From this plot, the ultimate bearing capacity is determined as shown in Fig. 4.4.1(c). The plate load test setup is shown in Fig. 4.4.1(a), for gravity type of loading. The load increment can either be applied through gravity method or by reaction of truss method. The loading setup for reaction of truss method is shown in Fig. 4.4.1 (b).



(a) Gravity loading



(b) Reaction of truss

Fig. 4.4.1

Limitations of plate load test

The limitations of plate load test are :

Limitations of plate load test
1. Size effect
2. Time effect
3. Layer effect

Fig. C4.4 : Limitations of plate load test

1. Size effect

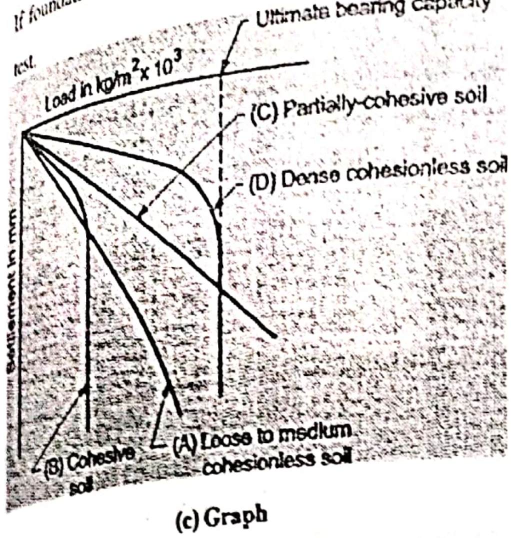
The actual settlement may vary from the plate weather same pressure is applied.

2. Time effect

As duration of test is small it does not give the ultimate settlement with respect long time.

3. Layer effect

If foundation is large accurate result cannot be obtained by test.



(c) Graph

Fig. 4.4.1 : Plate load test

Syllabus Topic : Field Methods for Determination of Bearing Capacity - Standard Penetration Test, Test procedures as Per IS: 2131

4.4.2 Standard Penetration Test : Procedure as Per IS : 2131

→ (MSBTE – S-12)

Q. Describe in brief standard penetration test to determine bearing capacity of soil. (S-12)

- This is the most extensively used penetrometer test in India and in U.S.A.
- The test employs split-spoon sampler, as shown in Fig. 4.4.2, which consists of a driving shoe, a split barrel of circular cross section which is longitudinally split into two parts and a coupling.
- IS : 2131-1981 gives the standard procedure for carrying out the test.
- It is reproduced below in brief :

1. The borehole is advanced to the required depth and the bottom is cleaned.
2. The split-spoon sampler, attached to standard drill rods of required length is lowered into the borehole and rested at the bottom.
3. The split-spoon sampler is driven into the soil for a distance of 450 mm by blows of a drop hammer of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required to penetrate every 75 mm is recorded while driving the sampler. The number of blows required for last 300 mm penetration is added together and recorded as N value at that particular depth of the borehole. The number of blows required for the first 150 mm penetration, called the seating drive is disregarded.
4. The split spoon sampler is then withdrawn and the soil sample in the split barrel is transported to the laboratory for testing.
5. This procedure is carried out at various depths in the borehole, usually, at every 0.75 m vertical interval.
6. The N values are corrected for :

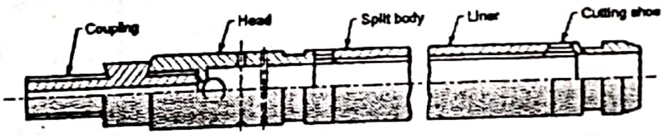
   (a) Overburden pressure and (b) Dilatancy of soil

   And then the settlement corresponding to the particular N-value and the width 'B' of proposed footing, is found out from standard charts.
7. This settlement value is then corrected as :

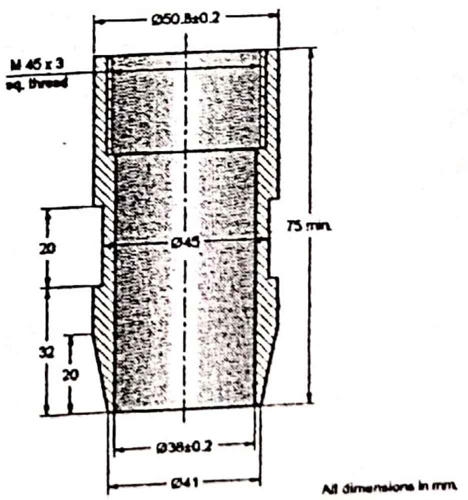
   $$\text{Actual settlement} = \frac{\text{Settlement from charts}}{\text{Correction factor}}$$

   The correction factor is given by correction factor $= 0.5 + 0.5\frac{Z_w}{B} \leq 1$

   Where Z_W is depth of water table measured from the base of the footing and B is the width of the proposed footing. From the settlement, the bearing capacity is calculated.



(a) Assembly of split spoon sampler



(b) Cutting shoe

Fig. 4.4.2

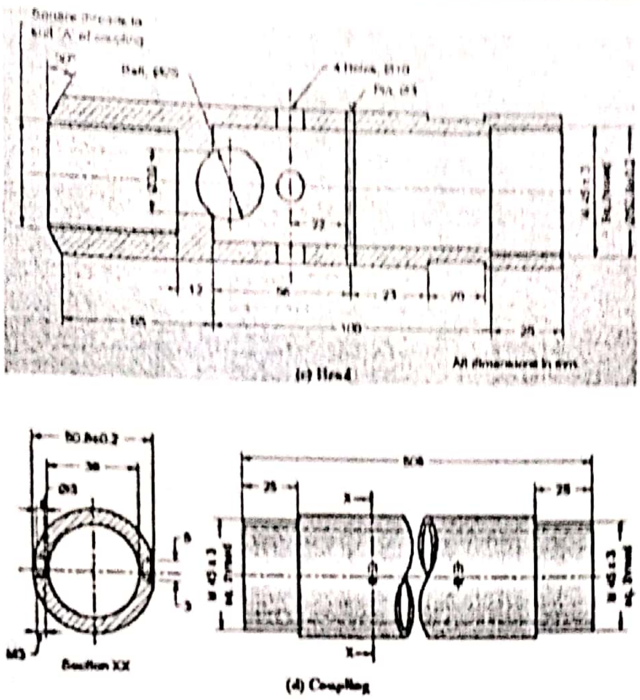


Fig. 4.4.2 : Split spoon sampler

4.5 Typical Value of Bearing Capacity from Building Code : IS : 1904

→ (MSBTE – S-10)

Q. Differentiate between cohesive and cohesionless soil. (S-10)

Typical values of bearing capacity for different types of soils are given below in Table 4.5.1.

Table 4.5.1 : Bearing capacity for various soils

Sr. No.	Cohesionless Soils		Cohesive soils	
	Description	SBC t/m²	Description	SBC t/m²
1.	Gravel, sand and gravel, compact and offering high resistance to penetration when excavated by tools.	45	Soft shale, hard or soft clay in deep bed, dry.	45
2.	Coarse sand, compact and dry.	45	Medium clay readily indented with a thumb nail.	25
3.	Medium sand, compact and dry.	25	Moist clay and sand clay mixtures indented with strong thumb pressure.	15
4.	Fine sand, silt (dry lumps easily pulvarised by finger).	15	Soft clay indented with moderate thumb pressure.	10
5.	Loose gravel or sand gravel mixture, loose coarse or medium sand dry.	25	Very soft clay which can be penetrated several inches with thumb.	5
6.	Fine sand, loose and dry.	10	B.C. soil or other shrinkable/ expansive clay in dry codition (50% saturation).	15

Table 4.5.1(a) : Typical value of bearing capacity from building code IS 1904

Sr. No.	Description	SBC
1.	Rock (hard) with of lamination and defect e.g. granite, trap	330 t/m²
2.	Laminated rock e.g. sand stone limestone	165 t/m²
3.	Residual deposits of shattered and broken bedrock and hard shale.	90 t/m²
4.	Softrock	45 t/m²

Syllabus Topic : Definition of Earth Pressure

4.6 Earth Pressure

→ (MSBTE – S-11, S-14, W-16, S-18)

Q. What are different types of earth pressure ? *(S-11)*

Q. Define earth pressure. *(S-14)*

Q. Define : *Earth pressure at rest (W-16)*

Q. Explain different types of earth pressure with the help of neat labelled sketches. *(S-18)*

- There are different types of earth pressures in the lateral direction according to the structure against which the soil is retained and according to the equilibrium conditions.
- These are : Earth pressure at rest, active earth pressure, passive earth pressure.

☞ ***Definition of Earth pressure at rest** : It is the earth pressure when the soil mass is not subjected to any lateral movement.*

Syllabus Topic : Active Earth Pressure and Passive Earth Pressure for No Surcharge Condition

4.6.1 Active Earth Pressure

→ (MSBTE – S-14)

Q. Enlist and define types of earth pressure. *(S-14)*

Active Earth Pressure

- a. Lateral earth pressure
- b. Active earth pressure
- c. Passive earth pressure
- d. Back fill
- e. Surcharge

Fig. C4.5 : Active Earth Pressure

→ **(a) Lateral earth pressure**

☞ ***Definition of Lateral earth pressure** : Soil in contact with any vertical or inclined face of structure exerts force on structure which is known as lateral earth pressure.*

→ **(b) Active earth pressure**

→ (MSBTE – S-08, W-08, S-10, W-10, S-11, W-11, S-12, W-13, W-14, S-15, W-16, S-17, W-17)

Q. Define Active earth pressure. *(S-08, W-08, S-10, S-11, W-13, W-16)*

Q. What is Active earth pressure. *(W-10)*

Q. Define active earth pressure with neat sketch. *(W-11, W-14, S-15, S-17, W-17)*

Q. Explain active earth pressure on soil. *(W-12)*

☞ ***Definition of Active Earth Pressures** : Active earth pressures defined as pressure exerted on retaining wall resulting from slight movement of wall away from filling.*

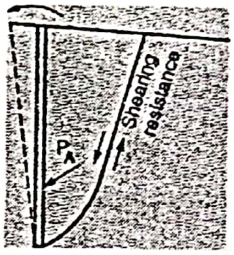


Fig. 4.6.1(a)

→ **(c) Passive earth pressure**

→ (MSBTE - S-08, W-08, W-10, W-11, W-12, W-13, W-14, S-15, W-16, S-17, W-17)

Q. Define : Passive earth pressure. *(S-08, W-08, W-11, W-13, W-16)*

Q. What is Passive earth pressure? *(W-10)*

Q. Define passive earth pressure with neat sketch. *(W-11, W-12, W-14, S-15, S-17, W-17)*

☞ ***Definition of Passive Earth Pressure** : Passive earth pressure is pressure when the movement of the retaining wall is such that the soil tends to compress horizontally.*

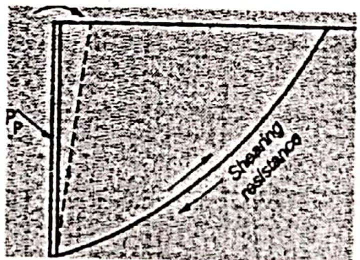


Fig. 4.6.1(b)

→ **(d) Back fill**

→ (MSBTE - S-10, W-11)

Q. Define Back fill. *(S-10, W-11)*

☞ ***Definition of Back fill :*** *Back fill is defined as the material retained by retaining wall or supported structure is called back fill.*

- (Refer Fig. 4.6.2)

→ **(c) Surcharge**

→ **(MSBTE -S-10)**

Q. Define surcharge. *(S-10)*

☞ ***Definition of Surcharge :*** *The back fill may have its top surface horizontal or inclined. This back fill lying above horizontal plane at elevation at top of wall is called as surcharge.*

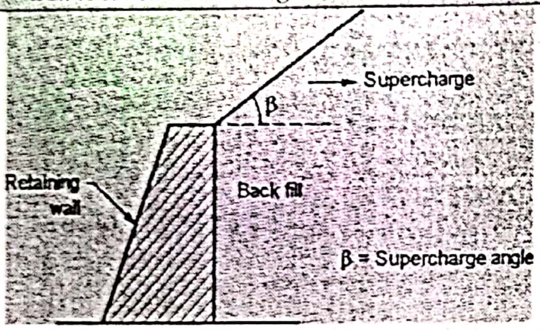


Fig. 4.6.2

☞ **Co-efficient of Earth Pressure**

→ **(MSBTE – W-15,W-16)**

Q. Define : Co-efficient of earth pressure *(W-15, W-16)*

☞ ***Definition of Co-efficient of Earth Pressure :*** *Co-efficient of earth pressure is the ratio of the horizontal stress to the vertical stress.*

Syllabus Topic : Rankine's Theory and Assumptions Made for Non-Cohesive Soils

4.6.2 Active Earth Pressure Rankine's Theory

→ **(MSBTE - S-12, W-12, S-13, S-14,S-15,W-15,S-18)**

Q. Explain Rankine's theory of earth pressure with assumptions made in it. *(S-12, W-12, S-13, S-18)*
Q. Explain Rankine's theory and assumptions made for cohesive soils. *(S-14)*
Q. State assumptions made by Rankines theory of earth pressure for non - cohesive soil. *(S-15)*
Q. State the four assumptions of Rankine's theory. *(W-15)*

☞ **Assumptions of the Rankine theory**

(1) The soil mass is semi infinite, homogeneous dry and cohesionless.

(2) The ground surface is plane which may be horizontal or inclined.

(3) The back of wall is vertical is smooth.

(4) The wall yields about the base thus satisfies deformation condition for plastic equilibrium.

(5) The soil element is in state of plastic equilibrium i.e. on verge of failure.

Active earth pressure for non cohesive soil. The following cases of cohesion less back fill are as follows.

Cases of cohesion less back fill
- 1. Dry back fill with no surcharge
- 2. Submerged back fill
- 3. Back fill with uniform surcharge

Fig. C4.6 : Cases of cohesion less back fill

→ **(1) Dry back fill with no surcharge**

The pressure distribution on retaining wall due to back fill is shown in Fig. 4.6.3.

The pressure distribution at base is,

$$P_a = k_a \gamma H$$

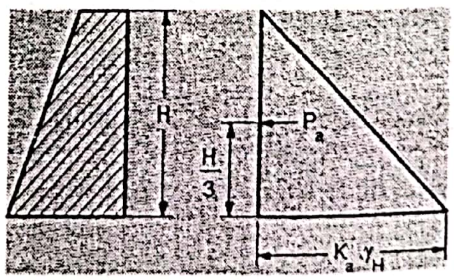


Fig. 4.6.3

Where, k_a = coefficient of active earth pressure

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

γ - unit weight of soil.

ϕ - angle of internal friction of soil.

Total active earth pressure paper unit length of wall.

$\therefore$ $P_a = \frac{1}{2} k_a \gamma H^2$ acting at H/3 above base of wall.

(2) Submerged back fill

In submerged back fill the land behind wall is saturated with ter and lateral pressure is having pressure due to,

(1) Submerged weight γ'

(2) Water

Therefore at base of retaining wall will be,

ssure, $P_a = k_a\gamma' H + \gamma_w H$

γ' = unit weight of submerge of soil.

γ_w = unit weight of water.

k_a = coefficient of active earth pressure.

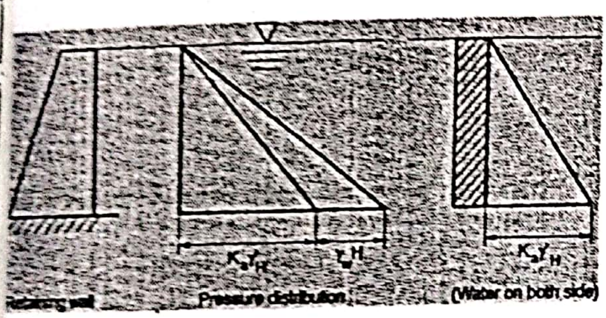


Fig. 4.6.4 : Submerged backfill

If water is present on both side of wall then water pressure ed not to be considered in that case lateral pressure is given by,

$$P_a = k_a\gamma' H$$

If back fill is partially filled and if back fill is most to a depth below then lateral pressure is given by,

$$P_a = k_a \cdot \gamma H_1 + K_a\gamma' H_2 + \gamma_w H_2$$

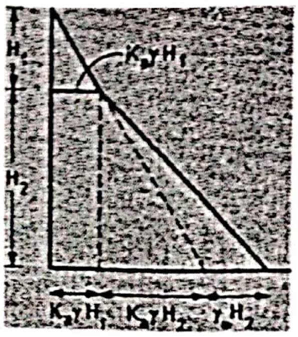


Fig. 4.6.5

(3) Back fill with uniform surcharge

Q. Describe the effect of uniform surcharge on active earth pressure of soil. *(S-11)*

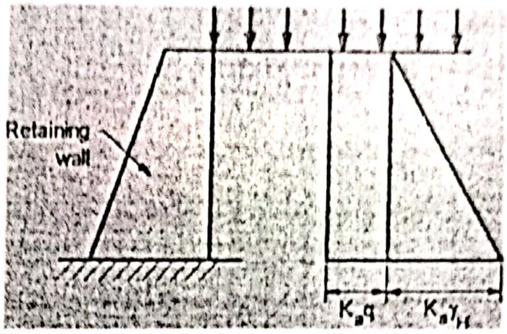


Fig. 4.6.6

If back fill is horizontal and carries a surcharge of uniform intensity as per unit area.

The pressure at base of retaining wall consist of pressure made of :

(1) Unit weight of back fill and

(2) Pressure due to uniform surcharge

$\therefore$ Pressure at base of retaining wall will be

$$P_a = k_a\gamma H + k_a q$$

k_a – coefficient of active earth pressure.

q – uniform surcharge intensity.

4.6.3 Structures Subjected to Earth Pressure in Field

→ (MSBTE – W-10,W-11)

Q. Name the field condition where active and passive earth pressure observed. *(W-10)*

Q. State any four field application where the structure is subjected to earth pressure. *(W-11)*

The main structures for which the consideration and analysis of earth pressure or the lateral pressure is important are as follows :

Structures Subjected to Earth Pressure in Field
- 1. Retaining walls
- 2. Walls of basements
- 3. Underground structures
- 4. Breakwaters and sea walls
- 5. Earth dams
- 6. Tunnel sides and ceilings of tunnels

Fig. C4.7 : Structures Subjected to Earth Pressure in Field

→ **1. Retaining walls**

Retaining walls are structures constructed for retaining soil of open cuts when the depth of cut is more than that required for stability of the slope or the vertical side of the open cut.

→ **2. Walls of basements**

- In principle, walls of basement are the same as retaining walls, as they retain the vertical earth cut against lateral movement.
- But the walls of basement have short length and these cannot be made trapezoidal in section.
- Hence buttress or bracing or keying is adopted in these walls for resisting the earth pressure.

→ **3. Underground structures**

- Underground structures are surrounded by all sides on soil, hence they experience lateral pressure.
- It must be remembered that if wall retains soil of same type on both sides, the effective force on the soil will be zero.
- Only when earth is retained on one side of wall, the lateral pressure comes into picture.

→ **4. Breakwaters and sea walls**

These structures, on their sea-ward side, absorb energy of the sea waves and tides but on their leeward sides, the lateral earth pressure is resisted by them.

→ **5. Earth dams**

- Lateral pressure of soil is not actually resisted by the earth dams, they rather resist the lateral hydraulic pressure.
- But for the stability of slope knowledge of internal soil pressure and its effect on slope stability is required.

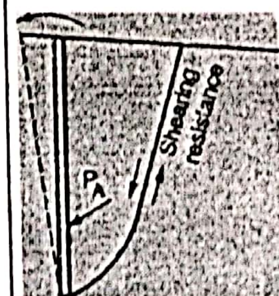
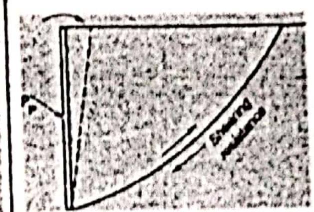
→ **6. Tunnel sides and ceilings of tunnels**

- This is one more situation where lateral earth pressure comes into picture indirectly.
- The stability of open cut and circular cut, the stresses and forces around a tunnel opening due to lateral forces is a very complex phenomenon and it requires thorough knowledge of earth pressure.

4.6.4 Difference between Active and Passive Earth Pressure

→ (MSBTE – S-12)

Q. Differentiate between active and passive earth pressure. (S-12)

Sr. No.	Active earth pressure	Passive earth pressure
1.	It is the lateral earth pressure against the retaining structure when the structure is just on the point of moving away from earth.	It is the lateral earth pressure when the retaining structure is just on the point of moving towards the backfill.
2.	Here the structure tends to move away from the soil.	Here the structure tends to move towards the soil.
3.	Very little movement is required to mobilize the active pressure.	Relatively much larger movement may be required to mobilize full passive resistance.
4.		

Ex. 4.6.1

Compute the intensities of active and passive earth pressure at depth of 8 m in dry cohesionless sand with an angle of internal friction of 30° and unit weight of 18 kN/m^3. What will be the intensities of active and passive earth pressures if the water level rises to the ground level? Take saturated unit weight of sand as 22 kN/m^3.

Soln. :

(a) Dry soil

Coefficient of active earth pressure, coefficient of passive earth pressure.

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

$$= \frac{1-\sin 30}{1+\sin 30} = \frac{1+\sin 30}{1-\sin 30}$$

$$= \frac{1}{3} = 3$$

Active earth pressure

$$\therefore \quad p_a = K_a \gamma H = \frac{1}{3} \times 18 \times 8$$

$$\therefore \quad p_a = 48 \text{ kN/m}^2$$

Passive earth pressure

$$p_p = K_p \gamma H = 3 \times 18 \times 8$$

$$p_p = 432 \text{ kN/m}^2$$

(b) Submerged soil

Submerged unit weight

$$\gamma' = \gamma_{sat} - \gamma_w = 22 - 10$$

$$\therefore \gamma' = 12 \text{ kN/m}^2$$

Active earth pressure,

$$p_a = K_a \gamma H + \gamma_w H = \left(\frac{1}{3} \times 12 \times 8\right) + (10 \times 8)$$

$$p_a = 112 \text{ kN/m}^2$$

Passive earth pressure

$$p_p = K_p \gamma' H + \gamma_w H = (3 \times 12 \times 8) + (10 \times 8)$$

$$p_p = 368 \text{ kN/m}^2$$

Ex. 4.6.2

A retaining wall 4 m high has a smooth vertical back. The back fill has a horizontal surface in level with the top of the wall. There is uniformly distributed surcharge load of 36 kN/m^2 intensity over the back fill. The density of back fill is 18 kN/m^3, its angle of shearing resistance is 30° and cohesion is zero. Determine magnitude and point of application of active pressure per metre length of the wall.

Soln. :

Coefficient of active earth pressure

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

Lateral pressure intensity due to surcharge is,

$$p_1 = k_a q = \frac{1}{3} \times 36$$

$$\therefore \quad p_1 = 12 \text{ kN/m}^2$$

Lateral pressure intensity due to backfill is,

$$p_2 = K_a \gamma H = \frac{1}{3} \times 18 \times 4$$

$$\therefore \quad p_2 = 24 \text{ kN/m}^2$$

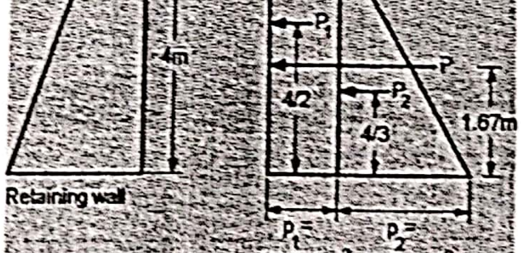


Fig. P. 4.6.2 : Pressure diagram

The resultant pressure per metre length of wall for surcharge and backfill are shown by P_1 and P_2.

$$\therefore \quad P_1 = p_1 \times H = 12 \times 4 \text{ and } P_2 = \frac{1}{2} \times p_2 \times H = \frac{1}{2} \times 24 \times 4$$

$P_1 = 48$ kN/m $\qquad \therefore P_2 = 48$ kN/m

Hence resultant pressure on wall is,

$$P = P_1 + P_2 = 48 + 48$$

$$\therefore \quad P = 96 \text{ kN/m}$$

The resultant P acts as,

$$P\bar{Z} = P_1 Z_1 + P_2 Z_2 \qquad \because \text{ Moments at base.}$$

$$\therefore \bar{Z} = \frac{(48 \times 2) + \left(48 \times \frac{4}{3}\right)}{96}$$

$$\therefore \bar{Z} = 1.67 \text{ m} \quad \text{above the base of wall.}$$

Ex. 4.6.3 S-03

A retaining wall of 5 m height, retains earth up to top. Water table is available at 2 m from top. Bulk density and saturated density of soil is 20 kN/m^2 and 18 kN/m^2 respectively. If angle of internal friction is 30°, Find intensity of active earth pressure at base and total active earth pressure.

Soln. :

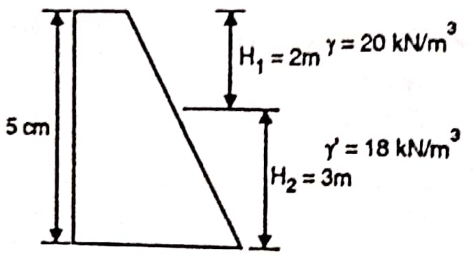


Fig. P. 4.6.3

$$\gamma = 20 \text{ kN/m}^3, \phi = 30$$

$$\gamma' = 18 \text{ kN/m}^3$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - 0.5}{1 + 0.5} = \frac{0.5}{1.5} = 1/3$$

Total active earth pressure at base,

$$P_a = K_a \gamma_1 H_1 + k_a \gamma_2 H_2 + \gamma_w H_2$$

$$= 1/3 \times 20 \times 2 + 1/3 \times 18 \times 3 + 10 \times 3$$

$$= 61.33 \text{ kN/m}^2$$

Ex. 4.6.4 S-12

Determine ultimate and net bearing capacity of a footing 2×2 m on a soil with density of 2000 kg/m3. $\phi = 15°$ C = 15 kN/m^3 if it is laid at depth of 1.8 m for $\phi = 15°$ Nc = 13, Nq = 4.5, NT = 2.5.

Soln. : Using Terzaghi's equation for ultimate bearing capacity of rectangular footing

$$q_u = 1.2 \times c \times N_c + \gamma_{D_f N_q} + 0.4 \gamma B N_\gamma$$

$$= (1.2 \times 15 \times 13) + (2 \times 9.8 \times 1.8 \times 4.5) + (0.4 \times 2 \times 9.8 \times 2 \times 2.5)$$

$$= 234 + 154.78 + 39.24$$

$$q_u = 432.02 \text{ kN/m}^2$$

Net bearing capacity

$$q_{net} = qu - \gamma df = 432.02 - (2 \times 9.8 \times 1.8)$$

$$= 396.74 \text{ kN/m}^2$$

Ex. 4.6.5 W-12

Compute the intensities of active and passive earth pressure at depth of 8 m in dry cohesionless sand with an angle of internal friction of 30° and unit at of 20 kN/m^3.

Soln. : Dry soil

Coefficient of active earth pressure :

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$$

Active earth pressure

$$\therefore P_a = k_a \gamma H = \frac{1}{3} \times 20 \times 8 = 6.66 \times 8$$

$$\therefore P_a = 53.28 \text{ kN/m}^2$$

Coefficient of passive earth pressure :

$$k_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1 + \sin 30}{1 - \sin 30} = 3$$

Passive earth pressure

$$P_p = k_p \gamma H = 3 \times 20 \times 8$$

$$= 480 \text{ kN/m}^2$$

Ex. 4.6.6 W-11

Calculate active earth pressure and passive earth pressure at depth of 9 m in dry cohesionless soil with an angle of internal friction of 30° and unit weight of 17 kN/m^3.

Soln. :

Coefficient of active earth pressure

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30}{1 + \sin 30} = 1/3$$

Active earth pressure

$$D_a = k_a \cdot \gamma \times H = \frac{1}{3} \times 17 \times 9$$

$$= 51 \text{ kN/m}^2$$

Coefficient of passive

$$\text{Earth pressure } k_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 30}{1-\sin 30}$$

Passive earth pressure

$$P_p = k_p \cdot \gamma \cdot H = 3 \times 17 \times 9 = 459 \text{ kN/m}^2$$

Ex. 4.6.7

A retaining wall 6 m high has a smooth vertical back. The backfill is horizontal. There is a uniformly distributed load of 3.6 t/m² over the back fill. Take $r = 1.8$ t/m³, $\phi = 30°$ and $c = 0$. Determine the magnitude of active earth pressure.

Soln. :

Coefficient of active earth pressure

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

Lateral pressure intensity due to surcharge is,

$$p_1 = k_a q = \frac{1}{3} \times 3.6 = 1.2 \text{ t/m}^2$$

Lateral pressure intensity due to backfill is,

$$p_2 = K_a \gamma H = \frac{1}{3} \times 1.8 \times 6 = 3.6 \text{ t/m}^2$$

The resultant pressure per metre length of wall for surcharge and backfill are shown by p_1 and p_2.

$$\therefore P_1 = p_1 \times H = 1.2 \times 6 \quad \text{and}$$

$$P_2 = \frac{1}{2} \times p_2 \times H = \frac{1}{2} \times 3.6 \times 6$$

$$P_1 = 7.2 \text{ t/m}$$

$$\therefore P_2 = 10.8 \text{ t/m}$$

Hence resultant pressure on wall is,

$$P = P_1 + P_2 = 7.2 + 10.8$$

$$\therefore P = 18 \text{ t/m}$$

Ex. 4.6.8

A retaining wall with a smooth vertical back of height 8 m support a cohesion less backfill of unit weight 14 kN/m³ and angle of shearing resistance of 30°. The surface of the soil is horizontal. Find total active pressure Paper linear meter of the wall by the Rankine's theory.

Soln. :

Given : Height of wall = 4.00 m

$\Upsilon = 14$ KN/m, $\phi = 30°$, P_a = at 4.00 m, $P_a = k_a \gamma H$.

$k_a = (1 - \sin\phi) / (1 + \sin\phi) = [1 - \sin 30/1 + \sin 30]$

$= [1/2] / [3/2] = 1/3$

$P_a = (1/3) * 14 * 4.00 = 37.33$ KN/m²

Total active pressure P_a unit length of wall

$= 1/2 * 37.33 * 8 * 1 = 149.33$ kN

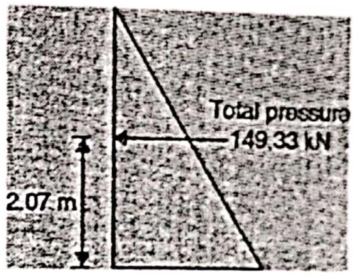


Fig. P. 4.6.8

Ex. 4.6.9

A retaining wall with a vertical back of height 8m supports a cohesionless backfill of unit weight 19 kN/M³ and angle of shearing resistance is 30°. The surface of soil is horizontal. Find the total active pressure Pa per linear metre of the wall by Rankine theory.

Soln. :

Height of wall = 8m

Unit weight (γ) = 19 kN/m³, $\phi = 30°$, P_a = ?

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30}$$

$$K_a = 0.3333$$

$$P_a = K_a \times \gamma \times H$$

$$= 0.3333 \times 19 \times 8 = 50.66 \text{ kN/M}^2$$

Total Active Pressrue Per meter length

$$= \frac{1}{2} \times P_a \times h \times l$$

$$= \frac{1}{2} \times 50.66 \times 8 \times 1 = 202.64 \text{ kN}$$

$$\text{It will act at } H/3 = \frac{8}{3} = 2.667 \text{ m}$$

Ex. 4.6.10

A strip footing 1.4 m wide is laid at a depth of 4.5 m in a purely cohesive soil having C = 144 kN/m² and bulk unit weight = 17.7 kN/m³; calculate ultimate bearing capacity by Terzaghi's analysis (N_c and N_q are 5.7 and 1 respectively)

Soln. :

Given : $\gamma = 17.7 \text{ kN/m}^3, C = 1444 \text{ kN/m}^2, B = 1.4 \text{ m}$

Depth $= D = 4.5$ m, $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

Ultimate bearing capacity is given by :

$$q_f = CN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

$$C = 141 \text{ kN/m}^3, \; N_c = 5.7, \; q = \gamma.$$

$$D_\gamma = 17.7 \times 4.5 = 79.65, N_q = 1, \; B = 1.40 \text{ m}$$

$$N_\gamma = 0$$

$$q_f = (144 \times 7) + 79.65 \times 1 + 0$$

$$= 820.80 + 79.65 = 900.45 \text{ kN/m}^2$$

Ex. 4.6.11

Compute the intensities of p_a and p_p at a depth of 8 m in dry cohesionless sand with $(\phi) = 30°$ and $\gamma = 19$ kN/m³.

Soln. :

Coefficient of active earth pressure :

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

Active earth pressure

$$\therefore \; p_a = k_a \gamma H = \frac{1}{3} \times 19 \times 8 = 8 \times 6.33$$

$$\therefore \; p_a = 50.64 \text{ kN/m}^3$$

Coefficient of passive earth pressure :

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 30}{1-\sin 30} = 3$$

Passive earth pressure

$$\therefore \; p_p = k_a \gamma H = 3 \times 19 \times 8$$

$$= 456 \text{ kN/m}^2$$

Ex. 4.6.12

Compute the intensity of active and passive earth pressure at depth 4.7 m in dry cohesionless sand with angle of internal friction of 28° and unit weight of 18 kN/m³. Also calculate total earth pressure and its line of action.

Soln. :

(1) Intensity of active pressure

$$P_a = k_a \cdot \gamma \cdot H$$

Coefficient of earth pressure

$$k_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$k_a = 0.36$$

$$P_a = 0.36 \times 18 \times 4.7 = 56.376 \text{ kN/m}^2$$

(2) Intensity of passive pressure

$$P_p = k_p \cdot \gamma \cdot H$$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

$$\therefore k_p = 2.76$$

$$P_p = 2.76 \times 18 \times 4.7$$

$$P_p = 432.216 \text{ kN/m}^2$$

(3) Total active pressure per unit length

$$P_a = \frac{1}{2} \cdot k_a \cdot \gamma H^2 \times l$$

$$= \frac{1}{2} \times 0.36 \times 18 \times 8.7^2 \times 1$$

$$P_a = 245.23$$

(4) Line of action the total pressure P_a is acting at

$H/3 = 8/3 = 2.9$ m from base.

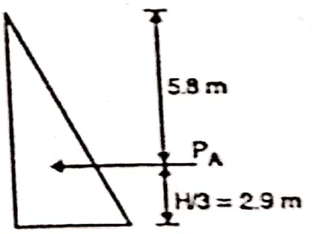


Fig. P.4.6.12 : Pressure distribution diagram

Ex. 4.6.13

A retaining wall with a vertical back of ht = 7.2 m supports cohesionless soil of dry unit wt 18.5 kN/m3 and angle of repose 27°, the surface of soil is horizontal. By Rankine's concept find the thrhst per m length of wall when the soil is absolutely dry.

Soln. :

Height of wall = 7.2m

Unit weight (γ) = 18.2 kN/m^3, $\phi = 27°$, $P_a = ?$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 27}{1 + \sin 27}$$

$$K_a = 0.376$$

$$P_a = K_a \times \gamma \times H$$

$$= 0.376 \times 18.2 \times 7.2 = 49.27 \text{ kN/M}^2$$

Total Active Pressrue Per meter length

$$= \frac{1}{2} \times P_a \times h \times l$$

$$= \frac{1}{2} \times 49.27 \times 7.2 \times 1 = 177.38 \text{ kN}$$

It will act at H/3 $= \frac{7.2}{3} = 2.4\text{m}$

Chapter Ends

□□□

UNIT V

CHAPTER 5

Compaction and Stabilization of Soil

Syllabus

5.1 Concept of compaction, purpose of compaction, field situations where compaction is required, Standard proctor test – test procedure as per IS code, Compaction curve, optimum moisture content, maximum dry density, Zero air voids line, Modified proctor test, factors affecting compaction, field methods of compaction – rolling, ramming and vibration and Suitability of various compaction equipments-smooth wheel roller, sheep foot roller, pneumatic tyred roller, Rammer and Vibrator, difference between compaction and consolidation.

5.2 Concept of soil stabilization, necessity of soil stabilization, different methods of soil stabilization – Mechanical soil stabilization, lime stabilization, cement stabilization, bitumen stabilization, fly-ash stabilization. California bearing ratio, C.B.R. test, meaning of C.B.R. value.

5.3 Necessity of site investigation and sub-soil exploration, types of exploration, criteria for deciding the location and number of test pits and bores. Field identification of soil – dry strength test, dilatancy test and toughness test.

Introduction

In case the bearing capacity is less than the required value at a site, there are two solutions. The first is to go at a greater depth for foundation. This involves expenditure and change in design. The other method is to improve the bearing capacity of soil by various procedures. This chapter deals with the modification of soil properties for better performance.

5.1 Compaction of Soils

- This is the simplest method of increasing the performance, characteristics, especially bearing capacity of soil.
- In this method, by the application of force or vibrations, the soil particles are more closely packed thereby increasing the density and hence bearing capacity of the soil.

Syllabus Topic : Concept of compaction

5.1.1 Concept of Compaction

→ (MSBTE – W-08, W-09, S-16)

Q. Define compaction. *(W-08, W-09, S-16)*

Definition of compaction : Increasing the density of soil by application of mechanical energy is called compaction.

- Compaction is also defined as the process where the density is increased by reducing air voids.
- It may involve modification of water content or gradation of soil or both.
- The theory of compaction was first developed by R.R. Proctor while building a dam in the USA.

- The principles of compaction developed by him were published in a series of articles in Engineering News record in 1933.

Syllabus Topic : Purpose of Compaction

5.1.2 Purpose of Compaction

→ (MSBTE – S-08, W-08, S-09, S-10, W-10, W-11, S-16,S-17,W-17)

> Q. State purpose of compaction. *(S-08, S-10, W-12, W-17)*
> Q. State objectives of compaction. *(W-08, W-11)*
> Q. State the necessity of compaction of soil. *(S-09, W-10)*
> Q. State purpose of compaction. *(S-16, S-17)*

Compaction of soil is undertaken for a number of purposes. These are listed below :

(1) To increase density and thereby shear strength and bearing capacity of soil, this is required in the case of slope stability improvement.

(2) To decrease the permeability of soil, this is required for earth dams.

(3) To reduce the settlement of structures after construction.

(4) To reduce danger of piping, this is required for seepage control of earth dam.

(5) To increase resistance towards erosion of soil by rain and other causes.

Syllabus Topic : Field Situations where Compaction is required

5.1.3 Field Situations Where Compaction Is Required

→ (MSBTE – S-08, S-10, W-10)

> Q. State field situations where compaction is required. *(S-08, S-10, W-10)*

As already stated above compaction is required in the following field situations :

(1) When sufficient bearing capacity is not available even at a greater depth.

(2) When the soil is totally cohesionless like dry sand.

(3) When the soil is highly pervious and is used in water retaining structures.

(4) In economical foundation design.

(5) In tunnels, roads etc.

(6) In special soil types.

(7) Under any situation where performance characteristics of soil are to be improved economically.

The values of maximum dry density can be obtained from following laboratory tests :

(a) Standard proctor's compaction test.

(b) Modified proctor's test.

(c) Harvard miniature compaction test.

(d) The dietert test.

(e) Abbot compaction test.

(f) Jodhpur mini-compactor test.

(g) U.S.B.R test.

(h) IS light compaction test.

(i) IS heavy compaction test.

5.2 Standard Proctor Test

→ (MSBTE – W-12)

> Q. Explain standard proctor test of soil. *(W-12)*

- When soil is compacted, it is generally compacted along with adding water. Optimum water content gives maximum dry density.

- Water content less than optimum value or more than optimum value will give less compacted density and consequently less dry density.
- To determine the optimum water content, the common test used is the standard proctor test.

Syllabus Topic : Standard Proctor Test – Test Procedure as per IS code

5.2.1 Standard Proctor Test Procedure as Per IS Code : IS : 2720

→ (MSBTE – W-11, S-13, W-13, W-14, S-18)

> Q. State the equipment used in standard proctor test and also state the used of this test. *(W-11)*
>
> Q. With the help of sketch explain standard proctor compaction test and show the nature of compaction curve. *(S-13)*
>
> Q. Explain the procedure of standard proctor compaction test to determine MDD and OMC. *(W-13, W-14, S-18)*

The stepwise procedure of standard proctor test is given below :

Preparation of specimen

(1) Take 16 kg of air-dried sample passing through 20 mm IS sieve.

(2) Apply water to bring water content to about 10 percent, less than the estimated optimum water content.

(3) Keep the soil in an air tight tin for about 20 hours to ensure thorough mixing of the water with the soil.

(4) Divide the sample in six equal parts.

Compaction test procedure

(1) Clean the mould and weigh it to nearest gram.

(2) Apply grease to inside of mould, base plate and collar.

(3) Assemble the mould and base plate together on the floor.

(4) Take one part of sample and fill the mould in 3 layers giving 25 blows to each layer with the 2.6 kg hammer dropping from 310 mm.

(5) Scratch with spatula each layer before putting in the next layer.

(6) Remove the collar and trim the compacted soil flush with the top of mould with a straight edge.

(7) Weigh the mould with the soil to nearest gram. Extract the soil from mould with the extruder.

(8) Cut the soil sample in the middle and take representative sample in an air tight container from middle of the cut surface.

(9) Determine the water content.

(10) Calculate bulk density.

(11) Calculate dry density using bulk density and water content values by using,

$$\gamma_d = \frac{\gamma}{1+w}$$

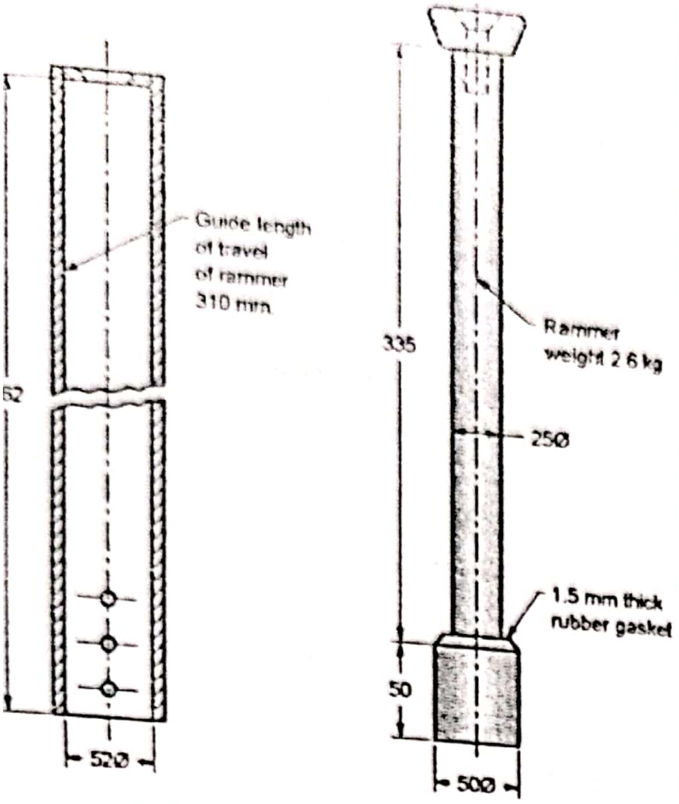
(12) Repeat steps 4 to 11 by taking 2 to 3% more water than preceding test.

(13) For all six samples, record the readings and plot moisture content against dry density.

(14) Find out dry density corresponding to the maximum point of the curve and corresponding moisture content.

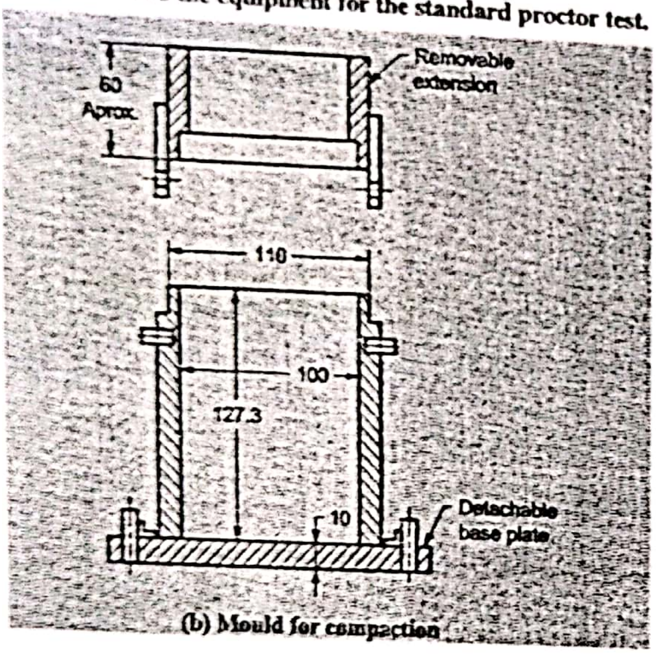
This dry density is known as maximum dry density (MDD) and the corresponding water content is known as the optimum moisture content (OMC).

> **Note :** This test is for light compaction. For heavy compaction Modified Proctor Test is used. In the modified test, the mould is filled in five equal layers instead of three, and each layer is given 25 blows of a hammer of 4.89 kg with a fall of 450 mm. The procedure in all other respects remains the same.



(a) Rammer for light compaction

Fig. 5.2.1 shows the equipment for the standard proctor test.



(b) Mould for compaction

Fig. 5.2.1 : Standard proctor test

Uses of the Test

a) To obtain OMC and MDD of a given soil sample.

b) To get relation between dry density and moisture content for a soil sample.

c) To achieve controlled compaction of soil in the field.

Syllabus Topic : Compaction Curve

5.2.2 Compaction Curve

- The plot of water content w on X-axis and dry unit weight γ_d on Y-axis is known as the compaction curve.
- The graph for light compaction gives the light compaction curve and for heavy compaction, it gives the heavy compaction curve.

Syllabus Topic : Optimum Moisture Content

5.2.3 Optimum Moisture Content

→ (MSBTE – W-10, S-11)

Q. Define OMC. How they are determined ? (W-10)

Q. Define O.M.C. and explain in detail. (S-11)

Definition of Optimum moisture content : *Optimum moisture content is defined as for each soil type, there exists a particular water content at which that soil can be compacted to maximum dense state.*

- This moisture content is known as "Optimum Moisture Content" or OMC. On the compaction curve for a soil, the water content corresponding to maximum dry density indicates the OMC for that particular soil.

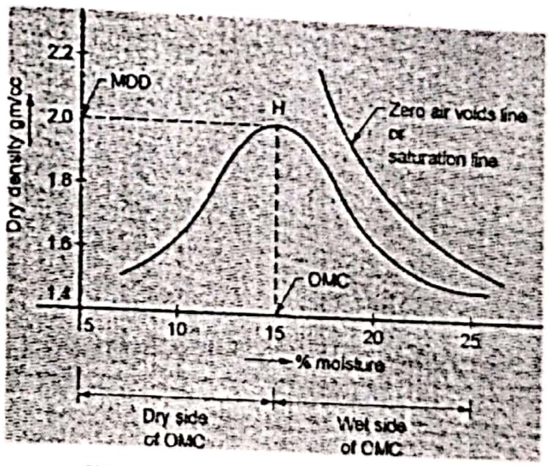


Fig. 5.2.2 : Graph for Proctor's Test

Syllabus Topic : Maximum Dry Density

5.2.4 Maximum Dry Density

→ (MSBTE – W-10, S-11)

Q. Define MDD. How they are determined ? (W-10)

Q. Define M.D.D. and explain in detail. (S-11)

☞ *Definition of Maximum dry density : When compaction achieved for a given effort is maximum at a optimum moisture content the corresponding density is called as MDD.*

- The crest of the compaction curve gives the value of maximum dry density or MDD. For different soils, the MDD will differ.
- MDD for light and heavy compaction also differs. This dry density gives a fair idea about the soil and the limit to which it can be compacted.
- If a perpendicular is drawn on X-axis from MDD value, it will give the optimum moisture content or OMC.

Syllabus Topic : Zero Air Voids Line

5.2.5 Zero Air Voids Line

→ (MSBTE – W-08, W-10, S-12, W-13, S-14, W-15, S-17)

Q. Draw a typical compaction curve and discuss the effect of moisture on dry density of soil. *(W-08)*

Q. What is 'zero air voids line'? *(W-10, W-13)*

Q. Define zero-air voids line? How is it drawn? State its significance. *(W-15)*

Q. Explain with sketch concept of zero air voids line. *(S-17)*

☞ *Definition of Zero air-voids line : If the soil is assumed to be 100% saturated and different dry densities are calculated for 100% saturation, then the resulting line on the compaction curve is called the 100% saturation line or the Zero air-voids line.*

Q. Draw the compaction cu
heavy compaction test,
energy applied in both the

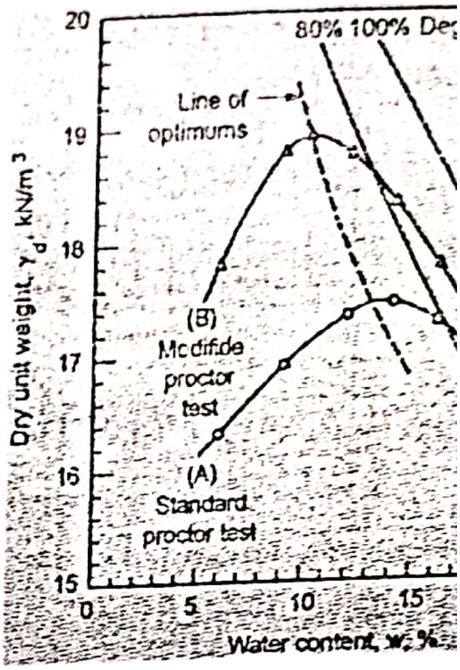


Fig. 5.2.3 : Comp

a) The compactive energy used f
1000 ml of soil or 595KJ/m3

b) In standard proctor test the weig

c) In Standard Proctor test, 3 layer
filled in the mould.

d) The vertical drop of hammer
310 mm.

Syllabus Topic : Modifi

5.3 Modified Proctor Part VIII

→ (MS

Q. Describe and differentiate st

- The test procedure is almost identical except the three factors, viz : number of layers, weight of hammer and the distance through which the hammer falls.
- A comparison of standard and modified tests is given below :

Sr. No.	Feature	Standard Proctor Test	Modified Proctor Test
1	Weight of hammer.	2.6 kg	4.89 kg
2	No. of layers in which soil is filled in the mould.	3	5
3	Vertical drop of hammer before striking the soil.	310 mm	450 mm

- Mould size in both tests is, 12.73 mm height and 100 mm diameter cylindrical mould and number of blows given in tests is 25.
- Modified Proctor test gives 5 to 10% higher value of dry density and 2 to 3% less OMC as compared to standard test provided that both tests are performed on the same soil.

Ex. 5.3.1

For certain soil, the following data was obtained while performing compaction test -

(a) Moisture content of soil sample in percentage = 10, 13, 16, 18, 20, 22 and 25.

(b) The corresponding wet density of soil sample in gm/cc = 1.55, 1.90, 2.30, 2.16, 1.95 and 1.6 respectively.

(c) Specific gravity of given soil = 2.7.

Find OMC and MDD for the soil and also plot the 100 % saturation line and 80 % saturation line for the soil.

Soln. : The detailed calculations for the soil sample no. I are shown below step by step. Calculations for soil sample.

(i) Moisture (water) content = $W = \frac{W_1 - W_2}{W_2} \times 100$

Given Data, W = 10%

In calculation W is taken as a fraction

$$W = 10\% = \frac{10}{100} = 0.10$$

(ii) Wet density of soil = $\gamma_{wet} = \frac{W_{wet}}{V_{wet}}$

$$\gamma_{wet} = 1.55 \text{ gm/cc} \quad \text{....(Given Data)}$$

(iii) Dry density of soil

$$\gamma_{dry} = \frac{\gamma_{wet}}{1+w} = \frac{\gamma_w}{1+w} = \frac{1.55}{1+0.10}$$

$$\gamma_{dry} = 1.41 \text{ gm/cc}$$

(iv) To draw 10% saturation line or 0% air voids line we use the equation.

$$\gamma_d = \frac{G \cdot \gamma_w}{1 + \frac{WG}{s}}$$

γ_w = Unit weight of water = 1 gm/cc

G = Specified gravity of soil = 2.7

W = Moisture content of the soil sample

= 0.10

S = Saturation = 100% = 1 in fraction form

Note : For any soil (S = 100 – A) or (A = 100 – S)

Where, S = Percentage saturation of soil sample and

A = Percentage air voids in the soil sample.

When air voids are zero we get saturation

$$S = 100 - A = 100 - 0 = 100\%$$

We get,

$$\gamma_d = \frac{G \cdot \gamma_w}{1 + \frac{WG}{s}} = \frac{2.7 \times 1}{1 + \frac{0.1 \times 2.7}{1}}$$

i.e. $\gamma_d = 2.13$ gm/cc

(v) To draw 80 % saturation line or 20 % air voids line we have to use the equation,

$$\gamma_d = \frac{G \cdot \gamma_w}{1 + \frac{WG}{S}}$$

Where, G = specified gravity of soil = 2.7 ..... given

γ_w = unit weight of water = 1 gm/cc

W = moisture content for the soil sample

= 0.10 i.e.

S = degree of saturation = 80 %

s = 0.80 in fraction form.

Note : For any soil sample, (S = 100 – A) or (A = 100 – S)

Where, S = percentage saturation of the soil sample

= degree of saturation.

and A = percentage air voids in the soil sample

When voids are 20 % we get saturation

$$S = 100 - A = 100 - 20 = 80\ \%$$

In calculations we uses in fraction form

$$\text{i.e. } s = 80\ \% = \frac{80}{100} = 0.80$$

$$\text{We get,} \quad \gamma_d = \frac{G \cdot \gamma_w}{1 + \frac{WG}{S}} = \frac{2.7 \times 1}{1 + \frac{0.10 \times 2.7}{0.8}}$$

$$\text{i.e. } \gamma_d = 2.02 \text{ gm/cc}$$

Similarly the results for other soil samples are obtained.

To find OMC and MDD of the soil, the results obtained in this example are plotted on a graph paper. On X-axis percentage moisture (W) is plotted and Y-axis dry density (γ dry) in gram/cc is plotted for each soil sample.

Thus we obtain seven points on the graph representing seven soil samples. These seven points are then joined by drawing a smooth curve. The highest point on the curve in the graph gives the required OMC and MDD from the plotted, graph we get :

(i) Percentage moisture is plotted on X-axis of the graph (W)

(ii) The corresponding dry density for each soil sample is calculated and plotted on Y-axis of the graph ($\gamma d'$).

(iii) Thus we obtain seven points on the graph representing seven soil samples.

(iv) If we join these points with a smooth curve, we obtain the required 100 % saturation line or 0 % air voids line for the given soil.

(v) This line shows the relation between moisture content and dry density of soil for a fixed saturation i.e. s = 100 % or Air voids = A = 0 %.

Same procedure is adopted to get 80 % saturation line or 20 % air voids line Graph shows all the three curves required.

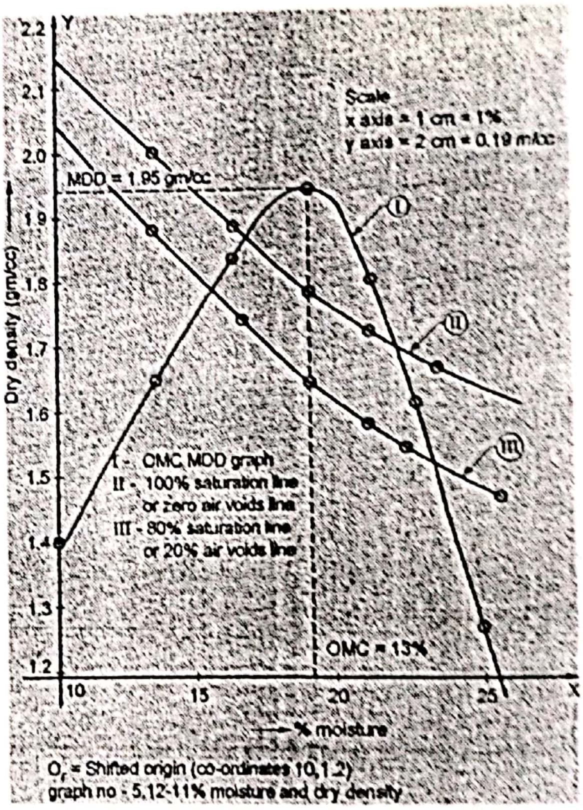


Fig. P. 5.3.1 : Moisture and Dry Density Graph

Ex. 5.3.2 [illegible]

The following observations were obtained using standard proctor test on a soft sample :soil.

Bulk density	1.6	1.95	2.15	1.80	1.70
Water content (%)	20	24	26	31	34

Using graph, determine maximum dry density and moisture content.

Soln. :

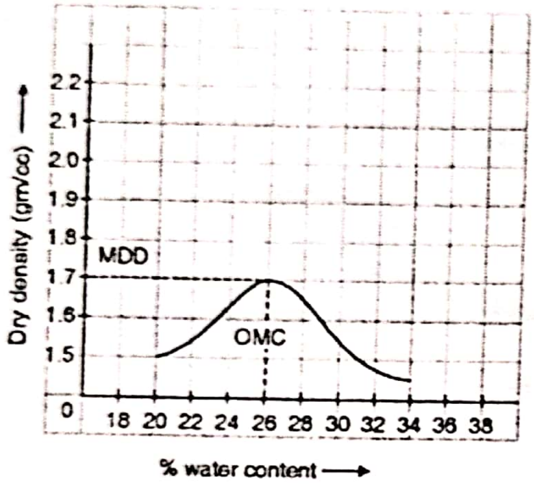


Fig. P. 5.3.2

Bulk density	1.6	1.95	2.15	1.80	1.70
Water content	20	24	26	31	34
Dry density	1.33	1.57	1.70	1.37	1.26

From the graph it can observed that optimum moisture content,

$$w = 26\ \%$$

Maximum bulk density $\gamma = 2.15$ gm/cc

Hence, maximum dry density γ_d,

$$\gamma_d = \frac{\gamma}{1+w} = \frac{2.15}{1+0.26}$$

$$\therefore \gamma_d = 1.706 \text{ gm/cc}$$

Ex. 5.3.3 [illegible]

The following are the results of standard compaction test performed on a sample of soil.

Water content (%)	5	10	15	20	25	30
Bulk density (gm / cc)	1.77	1.98	2.10	2.18	2.16	2.12

Plot the water content dry density curve and obtain the optimum water content and its maximum dry density.

Soln. :

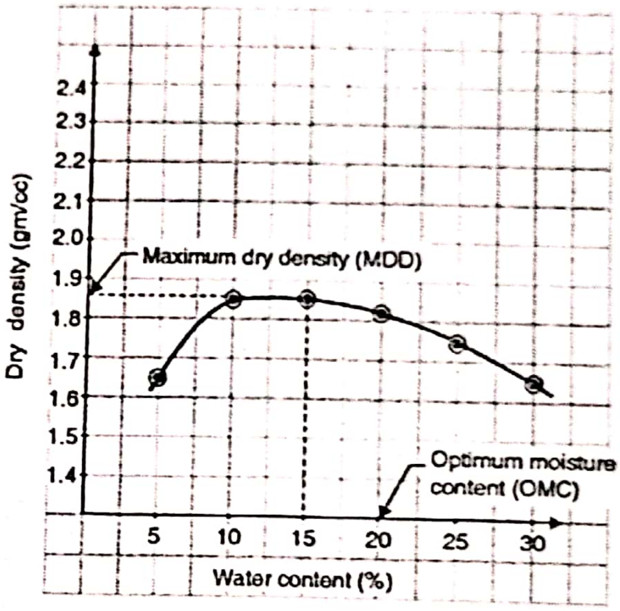


Fig. P. 5.3.3

Sample calculation

$$\gamma_d = \frac{1.77}{1+\frac{5}{100}} = \frac{1.77}{1.05} = 1.68$$

Water content (%)	5	10	15	20	25	30
Bulk density (gm / cc)	1.77	1.98	2.10	2.18	2.16	2.12
Dry density	1.68	1.8	1.82	1.81	1.73	1.63

Ex. 5.3.4 S-12, S-16

The following observations were made using standard Proctor test on a soil sample :

Ex. 5.3.2 [illegible]

The following observations were obtained using standard proctor test on a soft sample soil.

Bulk density	1.6	1.95	2.15	1.80	1.70
Water content (%)	20	24	26	31	34

Using graph, determine maximum dry density and moisture content.

Soln. :

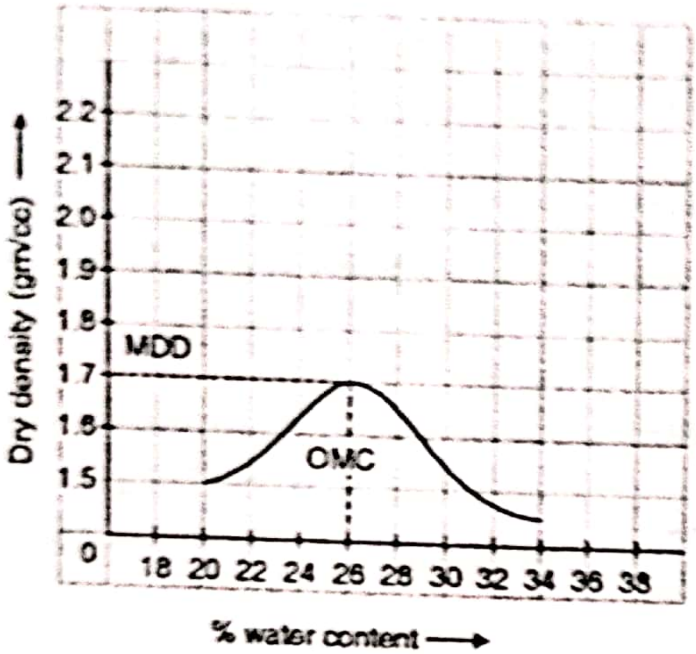


Fig. P. 5.3.2

Bulk density	1.6	1.95	2.15	1.80	1.70
Water content	20	24	26	31	34
Dry density	1.33	1.57	1.70	1.37	1.26

From the graph it can observed that optimum moisture content,

$$w = 26 \%$$

Ex. 5.3.3 [illegible]

The following are the results of stand

performed on a sample of soil.

Water content (%)	5	10	15
Bulk density (gm / cc)	1.77	1.96	2.10

Plot the water content dry density cu

optimum water content and its maximum

Soln. :

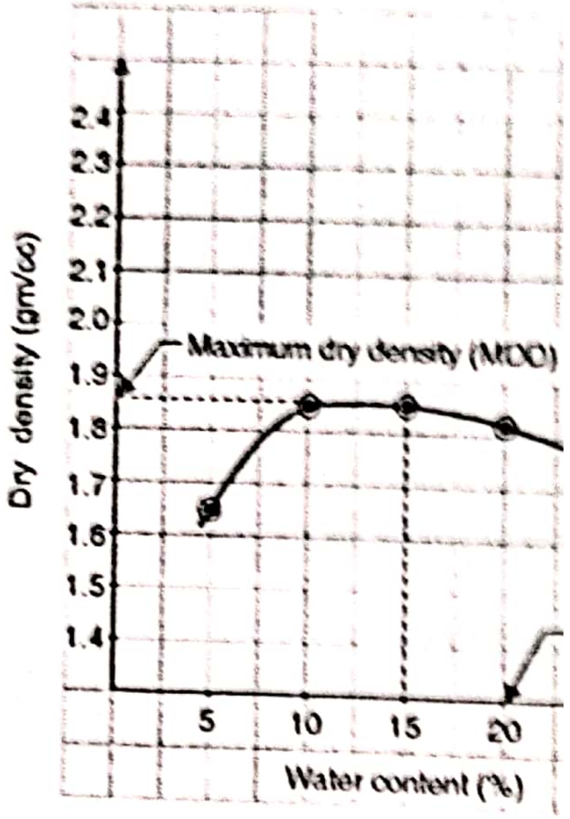


Fig. P. 5.3.3

Sample calculation

$$\gamma_d = \frac{1.77}{1 + \frac{5}{100}} = \frac{1.77}{1.05} =$$

Water content (%)	5	10	15

Bulk density in gm/cc	1.75	1.95	2.1	2.2	2.15	2.05
Water content in %	5	10	15	20	25	30

Determine OMC and MDD.

Soln.. :

Bulk density	1.75	1.95	2.1	2.2	2.15	2.05
Water content	5	10	15	20	25	30
Dry density	1.66	1.77	1.82	1.83	1.72	1.58

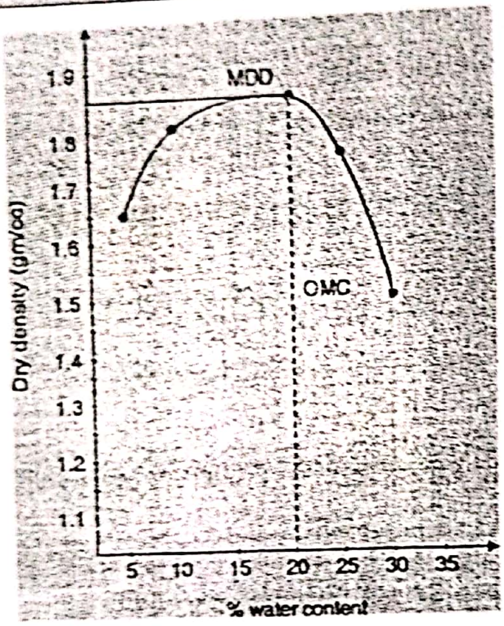


Fig. P. 5.3.4

$$W = 20\%$$

Maximum bulk density $\gamma = 2.2$

$$\text{Maximum dry density } \gamma_d = \frac{\gamma}{1+w} = \frac{2.2}{1+\left(\frac{20}{100}\right)}$$

$$= 1.83 \text{ gm/cc}$$

Ex. 5.3.5

The following are the results of standard compaction test performed on a sample of soil. Find OMC and MDD by plotting Graph.

Water content (%)	5	10	15	20	25
Bulk density (gm/cc)	1.6	1.82	1.90	1.84	1.75

Soln. :

Water content + (%)	5	10	15	20	25
Bulk density	1.6	1.82	1.90	1.84	1.75
Dry density	1524	1.655	1.652	1.533	1.4

$$\gamma_d = \frac{1.6}{1+\frac{5}{100}} = 1.524$$

From graph it can be observed that optimum moisture content

$$w = 10\%$$

Maximum buck density 1.82 gm/cc

∴ Minimum dry density

$$\gamma_d = \frac{\gamma}{1+w} = \frac{1.82}{1+0.1}$$

$$= 1.655 \text{ gm/cc}$$

Ex. 5.3.6

The following observations were made using S.P.T. on soil sample.

Bulk density gm/cc	1.65	1.95	2.1	2.2	2.15	2.05
Water content	5	10	16	22	25	30

Determine OMC and M.D.D.

Ans. :

Bulk density gm/cc	1.65	1.95	2.1	2.2	2.15	2.05
Water content	5	10	16	22	25	30
Dry density $\gamma_d = \frac{\gamma}{1+w}$	1.57	1.77	1.82	1.80	1.72	1.58

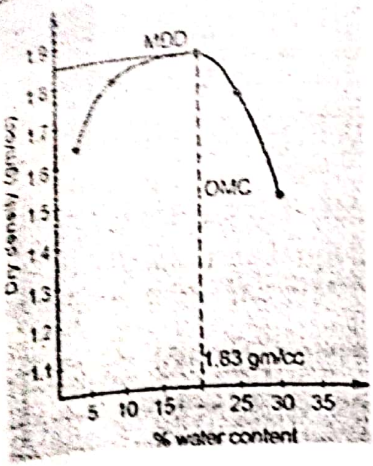


Fig. P. 5.3.6

...ph - MDD = 1.83 gm/cc

OMC = 18%

...um dry density

$$\gamma_d = \frac{\gamma}{1+w} = \frac{2.2}{1+\left(\frac{20}{100}\right)}$$

$$= 1.83 \text{ gm/cc}$$

...llabus Topic : Factors Affecting Compaction

... Factors Affecting Compaction

→ (MSBTE – W-08, S-09, W-09, W-10, S-11, W-11, S-12, W-12, W-13, S-15, S-17)

Explain any four factors affecting compaction of soil. *(W-08, S-11, W-11, W-13)*

State and explain any four factors affecting compaction. *(S-09, W-09, S-15)*

Q. State the factor affecting compaction. *(W-10, W-12, S-12)*

Q. State any four factors affecting compaction with their effect. *(S-17)*

Following factors affect the compaction of soil and the maximum dry density which can be achieved by compaction :

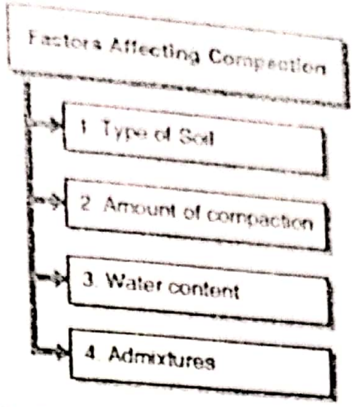


Fig C5.1 : Factors Affecting Compaction

→ **1. Type of Soil**

For the same compactive effort, a well graded coarse grained soil can be compacted to higher MDD than a uniformly graded soil. As the grain size decreases the OMC values goes on increasing and the MDD values goes on decreasing.

→ **2. Amount of compaction**

If the compactive effort is increased, MDD increases and OMC decreases. But the increase in MDD is not linear with increase in energy.

→ **3. Water content**

As is evident, if water content goes on increasing the maximum density of compacted soil goes on increasing upto a certain water content. If water content is further increased, the density goes on decreasing.

→ **4. Admixtures**

Various admixtures like lime, calcium chloride, aggregates in various proportions etc. are used to improve the compaction properties of soil. Lime can increase the dry density by about 5 to 10%.

Syllabus Topic : Field... Rolling, Ramm...

5.5 Field Metho...

→ (MSBTE – W-...

Q. What are the vario... sandy soil ? Explain ...

Q. Enlist four methods... factors affecting it. (...

Q. State various field c...

Q. State and explain m...

Q. Explain different m... *(W-17)*

- Field compaction i... equipments basica... methods of compa...

- These methods ... vibration.

Field...

Fig C5....

5.5.1 Rolling...

Q. Mention the... compaction ...

In this metho... due to their heavy ...

Syllabus Topic : Field Methods of Compaction – Rolling, Ramming and Vibration

5.5 Field Methods of Compaction

→ (MSBTE –W-10, S-12, W-12, S-13, W-13, W-17)

> **Q.** What are the various field methods of compacting sandy soil ? Explain about any one. *(W-10)*
>
> **Q.** Enlist four methods of compaction and mention four factors affecting it. *(W-12, S-13)*
>
> **Q.** State various field compaction methods. *(S-12)*
>
> **Q.** State and explain methods of compaction. *(W-13)*
>
> **Q.** Explain different methods of field compaction of soil. *(W-17)*

- Field compaction is carried out by various equipments. These equipments basically use three different procedures or methods of compaction.
- These methods are : Rolling, tamping or ramming and vibration.

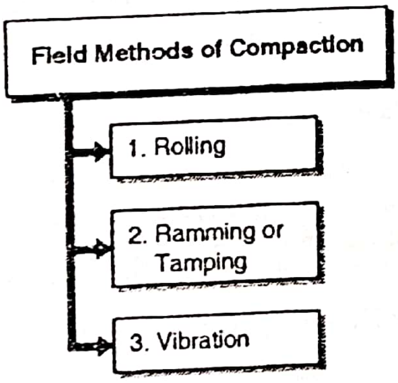


Fig C5.2 : Field Methods of Compaction

5.5.1 Rolling

→ (MSBTE – W-15)

> **Q.** Mention the suitability of the following method of compaction : (iii) Rolling *(W-15)*

In this method, rollers of different types pass over the soil and due to their heavy weight, the compaction takes place.

5.5.2 Ramming or Tamping

→ (MSBTE – W-15)

> **Q.** Mention the suitability of the following method of compaction : (i) Ramming (ii) Tamping *(W-15)*

- This method is very slow. In this method tampers or rammers are used to give repeated blows to the soil to compact it.
- Watering may be done during ramming.

5.5.3 Vibration

- In this method the soil is vibrated by equipment known as vibrators.
- Due to vibration, the soil particles are rearranged and more dense packing is possible. This increases the MDD of soil.

Syllabus Topic : Suitability of Various Compaction Equipments – Smooth Wheel Roller, Sheep Foot Roller, Pneumatic Tyred Roller, Rammer and Vibrator

5.5.4 Suitability of Various Compaction Equipment

→ (MSBTE – S-08, W-11,S-12, S-13, W-13, S-14, S-16, S-17, S-18)

> **Q.** State any four compacting equipments along with their suitability. *(S-08, W-11, S-12, S-13, S-17, S-18)*
>
> **Q.** Name the equipments used for compaction of soil. *(W-13)*
>
> **Q.** State various types of compaction equipments with their suitability. *(S-14)*
>
> **Q.** State suitability of following compaction equipments
> (i) Smooth wheel roller
> (ii) Sheep foot roller
> (iii) Rammer
> (iv) Vibrator *(S-16)*

For rolling, following equipments are used :

Equipments used for rolling

- 1. Smooth wheeled rollers
- 2. Pneumatic/ rubber tyred rollers
- 3. Sheep foot rollers
- 4. Rammers
- 5. Vibrator

Fig C5.3 : Equipments used for Rolling

→ 1. Smooth wheeled rollers

These have two rear wheels and one large drum in front. The weight is 10 to 16 tons and they are suitable for coarse-grained cohesionless soils.

→ 2. Pneumatic / rubber tyred rollers

These have 9 to 11 tyres on two axles in staggered fashion. These are suitable for both cohesive and non-cohesive soil.

→ 3. Sheep foot rollers

These rollers have a drum of diameter 0.9 to 1.8 m. This drum has projections of 23 cm length. These projections are called sheep-feet. The contact area of each foot is 35 to 90 cm^2. The weight is between 2 to 15 tons.

These are suitable only for fine-grained cohesive soils. Best suited for compaction of the core of an earth dam.

For ramming power rammers are used. These are suitable where access is difficult for rollers.

Vibration equipment consists of vibratory plates of 0.5 to 4.5 m coverage and 90 kg weight vibrating at 1600 cycles per minute. Vibratory plates are suitable for coarse grained soils with less cohesion.

→ 4. Rammers

- Rammers or tampers are mainly two types, hand operated and mechanical rammer. A hand operated rammer consists of a block of iron or stone about 3 to 5 kg in mass, attached to a wooden rod.
- The tamper is lifted for about 0.3 m and dropped on the soil to be compacted. A mechanical rammer is operated by compressed air or gasoline power. It is much heavier, about 30 to 50 kg. Ramming equipment's consists of three types: dropping weight type, internal combustion type and pneumatic type. Rammers or tampers are used to compact the soil.

Suitability: Suitable for all types of soil.

→ 5. Vibrator

- The vibrating equipment, mounted on screeds, plates or rollers are of two Types:

  a) Dropping weight type and

  b) Pulsating hydraulic type.
- By giving vibration to Soil, soil particles are packed together and compaction of soil is achieved.
- **Suitability:** Suitable for compacting granular soils. with no fines in layer up to 1 m thickness.

Syllabus Topic : California Bearing Ratio

5.6 California Bearing Ratio (CBR)

→ **(MSBTE – S-08, S-10, W-10, S-13, W-14, S-15, W-16, S-18)**

Q. Define California bearing ratio.
(S-08, S-10, W-14, S-15, W-16, S-18)

Q. What is CBR? (W-10, S-13)

Definition of California bearing ratio(CBR) : It is the ratio of the force per unit area required to penetrate a soil mass with a circular plunger of 50 mm diameter at the rate of 1.25 mm/minute to that required for corresponding penetration in a standard material.

- The ratio is usually determined for penetration of 2.5 mm and 5 mm.

Syllabus Topic : C.B.R. Test

5.6.1 CBR Test

→ (MSBTE - S-11, S-12, W-12, W-13, W-14, S-18)

Q. What is CBR value ? Draw CBR test setup and name the parts. *(S-11, W-13)*

Q. Explain CBR test on soil. *(S-12)*

Q. Explain determination of CBR of soil. *(W-12)*

Q. Explain CBR test with sketches. State is application of CBR test. *(W-14) (S-18)*

The test is performed in 3 steps. The stepwise procedure is given below and Fig. 5.6.1 shows the apparatus.

The stepwise procedure for CBR Test

1. Preparing a representative sample
2. Preparation of test specimen
3. Penetration test

Fig. C5.4 : The stepwise procedure for CBR Test

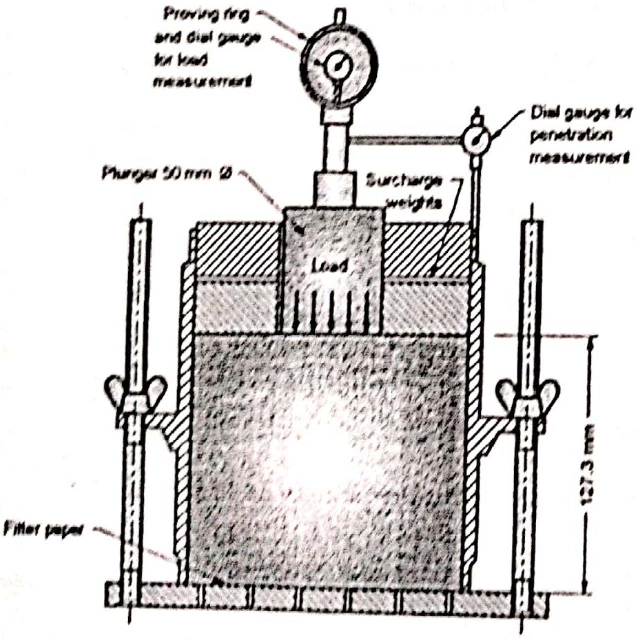


Fig. 5.6.1 : CBR test

→ 1. Representative sample preparation

(1) Sieve the soil received from the field using 19 mm IS sieve. The soil passing through 19 mm IS sieve is used for preparing specimen.

(2) Weigh the portion retained on 19 mm IS sieve and discard it.

(3) Take an equal amount of soil passing 19 mm IS sieve but retained on 4.75 mm IS sieve. Add this to the soil passing through 19 mm IS sieve in order to make an allowance for material larger than 19 mm. This gives us a representative soil sample. You should have about 5 kg of this sample.

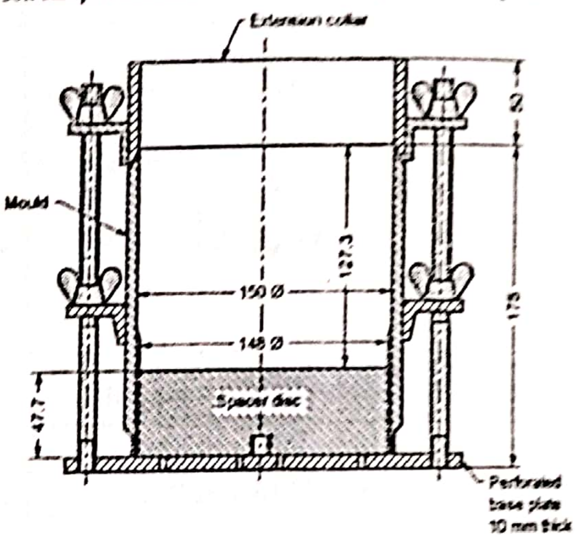


Fig. 5.6.2 : CBR test

→ **2. Preparation of specimen**

The specimen can be prepared at field density or maximum dry density for preparation a specimen at MDD. It is necessary to know the MDD and corresponding OMC of the soil sample. The procedure given here is for preparation a remoulded specimen by dynamic compaction using light compaction method :

(1) Take the exact calculated mass of soil from the representative sample, required to prepare specimen at MDD and add the calculated quantity of water to bring the sample to OMC. Mix it thoroughly. If the sample is found to contain appreciable amount of clay, it is better to keep mixed with water to mature 18 to 24 hours. This will ensure uniform distribution of water throughout the soil mass.

(2) Remix the soil before compacting. Make three equal parts.

(3) Place the spacer disc on the perforated base plate and keep a filter paper on the spacer disc.

(4) Weigh the empty mould and apply thin layer of oil on inside the mould. This will make it easier to remove the soil from the mould.

(5) Clamp the mould with the collar on the base plate.

(6) Fill one part of the mixed soil in the mould and compact by giving 56 blows uniformly using the rammer. Distribute the blows uniformly over the cross-section. Similarly compact the two more layers giving 56 blows to each layer. Use a mechanical rammer if available.

(7) Remove the extension collar. Trim carefully the compacted soil level with top of the mould. The compacted soil should not protrude by more than 5 mm above the top of the mould.

(8) Patch any hole developed during trimming the top surface of soil. Use small size material for that.

(9) Remove the mould and weigh it with the specimen. This will help in calculating the bulk and dry density of the specimen.

(10) Remove the spacer disc. Place a filter paper on the perforated base plate. Invert the mould and clamp it on the base plate with compacted soil surface in contact with the filter paper.

3. Penetration test

(1) Keep the mould on the loading machine. Place the annular surcharge weight on top of the soil.

(2) By operating the loading machine manually, bring the piston on the surface of the soil. The piston should be seated under a load of 4 kg. This establishes full contact between the surface of the specimen and the piston.

(3) Now place the additional surcharge weights over the annular weight.

(4) Adjust the plunger of the gauge, measuring the penetration of the piston over a rigid surface. Preferably on the wall of the mould. Set it to zero.

(5) Set the other dial gauge measuring the load also to zero.

(6) Adjust the loading machine to give the rate of penetration of 1.25 mm / minute and then start it.

(7) Record the load dial gauge reading at the penetration of 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5 and 10.0 mm.

(8) When the penetration reaches 12.5 mm stop the machine. If the maximum load occurs before the penetration of 12.5 mm, record the load and penetration.

(9) Raise the piston by manual operation and remove the mould.

(10) Repeat the penetration test as a check test on the reverse side of the specimen.

(11) Collect three samples of about 50 gm. Each in moisture cans, from the top 30 mm layer of the specimen and determine the mean moisture content.

Precautions during experiment

(1) Care should be taken to take representative sample for the test.

(2) The annular surcharge weight should be kept on the specimen in such a way that the plunger does touch the sides of the central hole of the weight.

(3) Before conducting the test check that rate of penetration is 1.25 mm/min.

Syllabus Topic : Meaning of C.B.R. Value

5.6.2 Significance of CBR Value

→ (MSBTE – S-08, S-10, W-11, W-12, S-13, S-15, W-16)

Q. State significance of CBR. (S-08, S-10, W-16)

Q. State any four necessities of soil stabilization. (W-11)

Q. Explain briefly significance of CBR. (W-12, S-13)

Q. State any four significance of CBR test. (S-15)

CBR test is useful in a number of field situations. The significance of CBR test is given below :

> ☞ ***Definition of CBR :*** *CBR is defined as the ratio of the test load to the standard load, expressed as percentage for given penetration of the plunger.*

CBR test is considered to be one of the most commonly used and widely accepted tests.

The CBR value shall be reported correct to the first decimal place.

(1) The CBR test can be used for the analysis of existing pavements, layer by layer in respect of their strength and load carrying capacity.

(2) CBR test also helps in identifying the cause of failure of load pavements.

(3) The CBR values are usually calculated for penetration of 2.5 mm and 5 mm.

(4) Generally, the CBR values at 2.5 mm penetration will be greater than that at 5 mm penetration and in such case the former is to be taken as the CBR values for design purposes.

(5) If the CBR values corresponding to a penetration of 5 mm exceed that for 2.5 mm, the test is repeated.

(6) If identical results follow, the CBR value corresponding to 5 mm penetration shall be taken for design purpose.

Table 5.6.1 : Standard load used in CBR test

Penetration depth in (mm)	Unit standard load (kg/cm^2)	Total standard load (kg)
2.5 mm	70	1370
5.0 mm	105	2055
7.5 mm	134	2630
10.0 mm	162	3180
12.5 mm	183	3600

Syllabus Topic : Difference between Compaction and Consolidation

5.7 Difference between Compaction and Consolidation

→ (MSBTE – W-08, S-09, W-09, S-10, W-10, S-11, W-11,S-12, W-12, S-13,S-14, W-14, S-15, S-17, W-17,S-18)

Q. Differentiate between compaction and consolidation. (S-09, 10, 11, 12, 13, W-08, 09, 10, 11, 12, S-15, S-16, W-17)

Q. Differentiate compaction and consolidation with respect to four points. (S-14, W-14, S-17,S-18)

- The gradual compression of a saturated soil under a steady load by expulsion of water from the pores is called consolidation. Consolidation and compaction must be clearly distinguished.
- Following is the difference between consolidation and compaction :

Sr. No.	Compaction	Consolidation
1.	Instant compression of soil under dynamic load is called compaction.	Gradual compression under a steady load is called consolidation.
2.	Takes place before building of structure.	Takes place after building of structure.
3.	Fast process	Very slow process.
4.	Carried out for improving soil property.	Occurs naturally due to load of structure. Does not improve soil property.
5.	Settlement is prevented due to compaction.	Settlement takes place due to consolidation.
6.	Artificial process.	Natural process.
7.	Pore water pressure not very important.	Pore water pressure very important.
8.	Does not go on indefinitely.	Goes on indefinitely.

5.8 Soil Stabilization

→ (MSBTE – S-08, W-08, W-09, S-12, W-13, W-14)

> Q. Define soil stabilization.
> (S-08, W-08, W-09, W-13, W-14)
>
> Q. State different objectives of soil stabilisation. (S-12)

> ☞ *Definition of soil stabilization : The term soil stabilization itself implies that soil is "unstable." Actually, the soil which is unsuitable for taking the load, fails due to shear, crushing or shows excessive settlement can be taken as "unstable soil".*

– Thus the term soil stabilization means improving the engineering performance of soil by artificial means. The process of compaction studied in the previous part of this chapter is one such means.

Syllabus Topic : Concept of Soil Stabilization

5.8.1 Concept of Soil Stabilization

→ (MSBTE – S-10)

> Q. Explain the concept of stabilization. (S-10)

– In soil stabilization, different methods are used to alter and improve the properties of a soil so that engineering performance can be improved.

– These methods mainly fall into three categories :

(1) Methods not using any admixtures.

(2) Methods using admixtures.

(3) A combination of (1) and (2).

– Drainage of soil to remove excess water and compaction are examples of first category. Addition of lime, cement etc. are examples of second category addition of missing fraction of soil and then compacting is the example of the third category.

Syllabus Topic : Necessity of Soil Stabilization

5.8.2 Necessity of Soil Stabilization

→ (MSBTE – S-08, W-08, S-09, W-09, S-10, W-11, W-13, W-14)

> Q. State of necessity of Soil Stabilization.
> (S-08, W-08, S-10, W-11, W-13)
>
> Q. State necessity of soil stabilization (any four points).
> (S-09, W-09)
>
> Q. State any three points of necessity of soil stabilization.
> (W-14)

– Many soils in natural state are very weak against shear load. Due to this their bearing capacity is also low, thus shear capacity is also low.

– The shear strength and bearing capacity can be improved by soil stabilization. Thus, the necessity of soil stabilization can be stated as :

(1) To increase shear strength.

(2) To improve the stability of slopes.

(3) To reduce construction cost by use of locally available material.

(4) For improving soil performance under heavy moving and impact loads in conditions such as sub-grade of roads, run ways in air ports etc.

(5) To increase strength against displacement and deformation.

(6) To increase density of soil.

(7) To reduce permeability e.g. core of earth dam.

(8) To reduce settlement of structures.

Syllabus Topic : Different methods of Soil Stabilization – Mechanical Soil Stabilization, Lime Stabilization, Cement Stabilization, Bitumen Stabilization, Fly-ash Stabilization

5.9 Different Methods of Soil Stabilization

→ (MSBTE – S-08, W-08, S-09, W-09, S-11, W-11, S-13, W-13, S-14, W-16, S-17, W-17, S-18)

> Q. State any four methods of soil stabilization and explain any one. (S-08, S-09, W-09)
>
> Q. List out the methods of soil stabilization and explain any one method. (W-08, W-11, S-13, W-13)

> **Q.** Enlist various methods of soil stabilization. *(S-11, S-14, S-18)*
>
> **Q.** State different methods of soil stabilization and explain any one. *(W-16, W-17)*
>
> **Q.** State any four methods of soil stabilization and write procedure of any one. *(S-17)*

Various methods of soil stabilization are in use from ancient times. Some of the important methods are discussed below :

Different Methods of Soil Stabilization

1. Mechanical stabilization
2. Soil-lime stabilization
3. Soil-cement stabilization
4. Soil-bitumen stabilization
   - 1) Sand bitumen
   - 2) Soil bitumen
   - 3) Waterproof mechanical stabilization
   - 4) Oiled earth
5. Complex stabilization with more than one stabilizer
6. Stabilization by heating
7. Stabilization by grouting
8. Stabilization by freezing
9. Stabilization by chemicals

Fig. C5.5 : Different Methods of Soil Stabilization

5.9.1 Mechanical Soil Stabilization

→ (MSBTE – S-15)

> **Q.** Explain mechanical soil stabilization. *(S-15)*

- Mechanical soil stabilization means stabilization without adding any chemicals or admixtures.
- It involves addition or removal of soil component. This may or may not be required. After this it involves compaction by mechanical means as discussed already under section 5.5.
- As we have seen in the index properties of soil and under particle size distribution curve, uniformly graded and gap graded soils are poor bases for compacting. Thus improving the grade of soil by adding missing fractions before compaction is most desirable.
- Sometimes, some undesirable constituents in soil like pockets of loose soil, calcium deposits, salt pans, etc. must be removed before applying compaction.
- For good compaction, the following requirements should be satisfied :

☞ For bases and sub-grades

Liquid limit $w_L \leq 25\%$

Plasticity index $I_P \leq 6$

☞ For surfaces

Liquid limit $w_L \leq 35\%$

Plasticity index I_P between 4 and 5.

- Compaction is very important in mechanical stabilization. It is already discussed in detail in earlier sections. However, it is briefly revised here.
- There are three methods of doing compaction in the field : Rolling, ramming or tamping and vibration.
- Rolling equipments are of the following types :

  (1) Smooth wheeled rollers

  (2) Pneumatic rollers

  (3) Sheep - foot rollers.
- Ramming equipments are of the following types :

  (1) Dropping weight type

  (2) Pneumatic

(3) Internal combustion engine type.

The vibrating equipments are of following types :

(1) Dropping weight type

(2) Pulsating hydraulic type.

Procedure of Mechanical Stabilization

Following steps may be adopted to obtain a stabilized road by mechanical soil stabilization method.

(i) Samples of locally available materials are collected, they are mixed in different proportions with different proportions with different moisture contents and several trial mixes of soil and aggregate are prepared. The dry density and strength of these trial mixes are found and any one best suitable trial mix is adopted for field work. This work is called as designing of mix proportions.

(ii) The required materials are collected and stacked along the sides of the proposed road. These materials are collected in required quantities according to proposed proportions of materials. Machinery or manual labour can be used for excavation, haulage and mixing etc. depending on the type and size of the work. Rollers are used for compaction of the mixed material in thickness of 15 cm.

(iii) The sub grade is prepared to required shape and size.

(iv) The designed proportions of materials are mixed. Water is added according to requirements. If excess moisture is present in the aggregates and soil then they are allowed to dry in sun before mixing.

(v) Wet mix of soil and aggregates is spread to the desired grade and thickness. It is then compacted by rollers. Rolling is started from the edges and is continued till the desired compaction is achieved.

(vi) Field control tests are performed before, during and just after compaction to find out dry density and moisture content. Modification if required is done in the field.

(vii) When the compacted layer hardens on drying, the stabilized road is opened to traffic.

Advantages of mechanical stabilization

(1) Basic properties are retained

(2) Economical

(3) Simple to carry out

(4) Rapid, construction can be immediately started

(5) Causes considerable improvement in soil properties.

Disadvantages of mechanical stabilization

(1) Effective only upto a certain extent.

(2) Not suitable for all soil types.

(3) Requires special equipment.

Factors affecting stabilize and strength of mechanical stabilization

The various factors affecting station and strength of mechanical stabilization are :

(1) Grading of materials.

(2) Compaction of materials.

(3) Presence of salts.

(4) Properties of soil and aggregate.

5.9.2 Lime Stabilization

→ (MSBTE –W-12, W-13)

Q. Explain lime stabilization of soil. (W-12, W-13)

- This type of stabilization is especially effective in case of plastic clays. Slaked lime (CaOH) is found very effective in such soils. This method is generally used for stabilizing road bases and subgrades.

- Lime is either used alone or in combination with fly ash, bitumen or cement.

- The percentage of lime required for coarse grained soils is 2% to 5% and 5% to 10% for plastic soils.
- The action of lime consists of two mechanisms :
  (1) It has cementing action and binds the soil particles together.
  (2) The nature of adsorbed moisture layer over the particles changes decreasing the plasticity index and thereby increasing strength.

Procedure of Soil-Lime Stabilization

The steps adopted for soil-lime stabilization are listed below.

(i) Several trial mixes are prepared and the amount of lime water and compaction required to get best soil-lime stabilization is determined. This can be called as mix design for soil-lime stabilization.

(ii) The surface is cleaned, excavated and sub-grade is prepared to required shape and size.

(iii) Soil to be stabilized is pulverised and mixed with lime and water at required quantities.

(iv) Keep the mix for twenty four hours for pre-conditioning it. Remix the soil and add more lime and water if required. Additive like pozzolana, fly ash, surkhi etc., may be added in required amount and the whole thing is mixed properly.

(v) The mix spread on subgrade to desired gradient and thickness and the proper compaction is carried out.

(vi) After compaction, moist curing is done and the mix is protected from drying out.

(vii) Wearing course is then applied above the soil-lime stabilized surface to withstand wearing.

(viii) Field control tests are performed to determine optimum moisture content and maximum dry density before and after compaction.

Advantages

(1) Cheap.

(2) Easy availability of lime.

(3) Expert supervision is not needed.

Factors affecting soil lime stabilisation the various factors affecting soil lime stabilization are :

(1) Type of soil

(2) Lime content

(3) Amount of compaction

(4) Curing

(5) Additives

Disadvantages

(1) No drastic alterations in properties.

(2) Suitable only for plastic clays.

5.9.3 Cement Stabilization

→ **(MSBTE - S-10, S-15)**

Q. Explain cement stabilization method of soil stabilization. *(S-10, S-15)*

- The soil stabilized with cement is known as soil cement stabilization.
- The cement action is the result of chemical reaction of the cement with silicon present in soil during the process of hydration. This happens in non-cohesive and coarse-grained soils.
- Very few particles for cement bond in case of fine grained cohesive soil.
- Almost all inorganic soils can be stabilized by this method. Black cotton soil is an exception. Organic matter and sodium sulphate, if present in soil, causes difficulties for cement stabilization.

- Well graded soils with fine fraction lesser than 50% and I_p less than 20 are the ones to give best results. The strength of soil cement increases with the cement content used.
- 5 to 15% cement of the dry weight of soil is required. Actual requirement is determined by trial and error.
- For tropical climate like India, the amount of cement which gives compressive strength of 25 to 30 kg/cm^2 is considered to be adequate.

The approximate amounts needed for different types of soils are :

Gravels :	5 to 10%
Sands :	7 to 12%
Silts :	12 to 15%
Clays :	12 to 20%

- If the soil-cement-water mixture is mixed thoroughly a better, soil cement is obtained. Mixing must be discontinued, when hydration start.
- The strength is acquired due to hydration reaction and hence curing by keeping moist should be done at least for 7 days.
- Some admixtures may also be used in cement stabilization, to reduce consumption of cement or to enhance stabilization.
- For clays and soils containing organic matter, lime and calcium carbonate is used along with cement. Sodium carbonate, sodium sulphate and fly ash are also used with cement for various different situations.
- The construction procedure for cement stabilization consists of following steps :

(1) Shaping sub-grade and scarifying the soil.

(2) Pulverizing the soil.

(3) Adding and mixing the cement and admixtures if required.

(4) Adding and mixing correct amount of water.

(5) Compacting

(6) Finishing

(7) Curing

(8) Adding finished surface.

There are three methods to carry out these operations :

(1) Mix in place method.

(2) Traveling plant method.

(3) Stationary plant method.

Procedure of Soil-Cement Stabilization

Following steps may be adopted to obtain a stabilized road by soil-cement stabilization.

(i) A number of trial mixes are prepared and tested and compaction require to get best soil-cement stabilization is determined. This may be called as mix design for soil-cement stabilization.

(ii) The road surface is cleaned, excavated and subgrade is prepared to required shape and size.

(iii) The soil to be stabilized is pulverised and dry mixed with cement.

(iv) Water is added and the mix of soil, cement and water is mixed properly.

(v) The mix is spread on subgrade to desired, grade, thickness and camber and it is compacted to required grade and camber.

(vi) Cure the compacted layer properly. Moist curing is done either by preventing the moisture to escape or by covering with moist soil.

(vii) Join the old work with new work. This should be made carefully because old soil-cement is brittle and the edges may get severly damaged.

(viii) Provide a wearing surface above this to protect the base course from abrasion.

Advantages

(1) Large increase in strength.

(2) Very useful for loose soils and non cohesive soils.

(3) Permanent solution.

Disadvantages

(1) Not useful for clays, organic soils and expansive soils.

(2) Costly.

(3) Complicated procedure.

Factors affecting soil cement stabilization.

The various factors that's affect soil cement stabilization are as follows :

(1) Type of Soil. (2) Quantity of Cement.

(3) Mixing of Ingredient. (4) Compaction.

(5) Curing. (6) Addition.

5.9.4 Bitumen Stabilization

→ **(MSBTE – W-10)**

Q. How Bitumen is useful in stabilization of soil ? Explain. (W-10)

- For pavement construction this method is the best.
- Tar and asphalt is used in this method.
- These are bituminous materials with very high viscosity and for proper mixing with soil, the viscosity is decreased by heating, emulsifying or by cut back process.
- When these bituminous materials are added, they impart cohesion and reduce water absorption. Thus either the binding action or water proofing action or both are responsible for increased strength.
- Bitumen stabilization is classified in following types :

→ **(1) Sand bitumen**

- When cohesionless soil like sand from beach, river or dunes is mixed with bitumen, it forms sand bitumen. Main action of bitumen is to bind the particles together. Sand should be free from clay or organic matter. The approximate percentages of different components are as follows :

Hot mix asphalt = 3 to 11%

Cut back = 4 to 10%

Emulsions = 5 to 10%

- Sometimes hydrated lime 1 to 2% is used as an admixture. These percentages are on the basis of dry weight of sand.

→ **(2) Soil bitumen**

In this type, the main function of bitumen is to reduce water absorption and making soil waterproof to preserve its natural cohesive strength.

→ **(3) Waterproof mechanical stabilization**

This is done by adding a small amount of bitumen, 1 to 3 % to already mechanically stabilized soil to make it waterproof.

→ **(4) Oiled earth**

Slow and medium curing road oils are spread on the ground surface to make it impervious to water and also to increase the abrasion resistance.

Procedure of Soil-bitumen Stabilization

The steps adopted to stabilize soil by soil-bitumen stabilization are listed below :

(i) Mix design of soil-bitumen mix is carried out and the exact quantity of soil, bitumen, water and compaction required for best soil-bitumen stabilization is determined.

(ii) The road surface is prepared to required shape and size.

(iii) Soil is pulverised, mixed with water and then it is mixed with cut back or emulsion bitumen.

(iv) The mix is spread on the sub-grade and compacted in 10 cm layers to desired grade and camber.

(v) Curing of the compacted layer is done and a suitable wearing surface is provided on top of it.

(vi) Field control tests are performed to determine moisture content, density, degree of compaction etc.

- **Advantages of bitumen stabilization**

(1) Very useful for roads, sub-grades and water proof structures.

(2) Durable for years.

(3) Cheaper than soil- cement.

- **Disadvantages of bitumen stabilization**

(1) Effective for only some soil types.

(2) Not useful for foundation.

(3) Water proofing rather than strength enhancement is achieved.

Factors affecting soil bitumen stabilization. The various factors affecting soil bitumen stabilization are as follows :

(1) Type of soil.

(2) Amount/Quantity of bitumen.

(3) Mixing.

(4) Compaction.

(5) Curing.

(6) Addition.

5.9.5 Fly ash Stabilization

- Fly-ash is a waste material formed due to combustion of mineral coal. It possesses a difficulty for disposal.
- When used in soil stabilization its action is twofold.
  1) It forms filler or pozzolana material.
  2) It hydrates forming cement like gel.
- Fly ash by itself is rarely used in soil stabilization.
- Rather it is used in combination with other methods as an admixture to enhance stabilization.

- **Advantages**

(1) Eco friendly as helps in recycling of fly ash.

(2) Simple.

(3) Cheap.

(4) Rapid.

- **Disadvantages**

(1) Cement be used as standalone method.

(2) Enhancement is not substantial.

Syllabus Topic : Site Investigation and Sub-Soil Exploration

5.10 Necessity and Importance

→ (MSBTE – W-10, S-17, W-17)

Q. What is soil exploration ? *(W-10, W-17)*

Q. Give necessity of soil exploration and site investigation. *(S-17)*

Q. State necessity of soil exploration. *(W-17)*

- **Introduction**

When any construction activity is to be started, the foundation design requires knowledge of conditions under the ground level to requires this we carry out site investigation and subsoil exploration at the beginning of any construction project or work.

- Site investigation and subsoil exploration can be considered as bases on which the foundation design rests.
- The soil exploration is technical investigation by which the necessary information regarding various soil properties are obtained which helps in designing safe and economical construction is called soil exploration.
- The purpose of soil exploration is to find out the nature and dimension of underlying soils. It also tells whether soil is gravel, Sand clay silt etc. It also helps in indicating depth and thickness of soil lager.

Syllabus Topic : Necessity of Site Investigation and Sub-Soil Exploration

5.10.1 Necessity of Site Investigations and Subsoil Explorations

→ **(MSBTE – S-08, W-08, S-09, W-10, S-11, W-11, S-12, S-17)**

> **Q.** State the necessity of site investigation. ***(S-08, S-09, W-11)***
>
> **Q.** What are the objectives of sub soil exploration ? ***(W-08)***
>
> **Q.** State the necessity of site investigation and sub-soil exploration. ***(W-10, S-17)***
>
> **Q.** Why soil investigation is necessary ? ***(S-11)***
>
> **Q.** State necessity of site investigation and explain any two. ***(S-12)***

Site investigations and subsoil explorations are necessary for followings purpose :

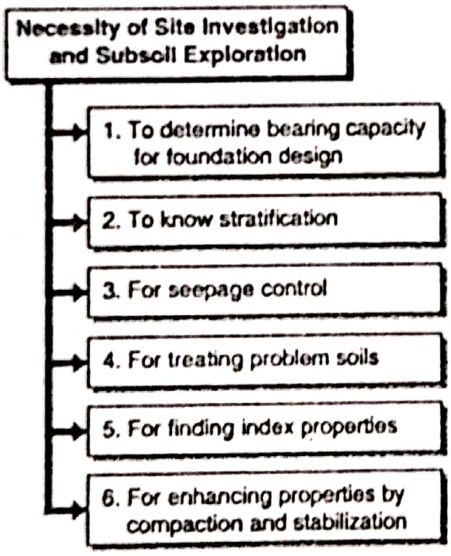


Fig. C5.6 : Necessity of Site Investigation and Subsoil Exploration

→ **1. To determine bearing capacity for foundation design**

- This is the most preliminary purpose of soil investigation. Bearing capacity goes on increasing with depth in most soils. As we go deeper and deeper, we encounter soil with more and more bearing strength.
- The bearing strength required for a particular structure must be available. Thus, the bearing capacity is estimated at a particular depth and then it can be actually tested by various tests available, of which plate load test is an important one.
- This bearing capacity is such a complex phenomenon, that after determining bearing capacity, a high factor of safety of 3 to 5 is still applied to that value.
- In view of all this, the site investigations are very important for determining the bearing capacity of soil.

→ **2. To know stratification**

- Soil contains different layers or strata below the surface. These strata are arranged in different ways. The strata may be continuous, displaced by a crack or fault, at different angles, separated by a dyke arranged in the form of folds etc.
- The arrangement of strata below the ground level is called stratification. Many properties of the soil like permeability or bearing capacity depend on the orientation and nature of the stratification.
- When we take boreholes at the foundation level, or when we dig bore holes or test-pits, this arrangement is clearly visible by the side of the bore holes or test-pits.
- By taking numerous bore holes, a geological map of the underground strata can be prepared. This will give a complete idea of the stratification of soil. Thus, investigations enable us to find the hidden features below the ground level.

→ **3. For seepage control**

- Seepage is another complex phenomenon in soil mechanics. Seepage is especially important in case of earth dams and weirs and aprons on permeable foundations.
- Soil which has same permeability in y direction, i.e. in gravitational direction, and in x direction, i.e. direction perpendicular to gravity is called isotropic soil. Soil which has different coefficients of permeability K_x and K_y is called anisotropic soil.

- The net effect can be considered as a two dimensional flow which can be analysed by Laplace equation and flow net technique. The effective flow will be the seepage flow.
- During detailed investigations of soil and subsoil, field tests will give realistic picture reinforced by laboratory tests conducted on soil samples, both disturbed and undisturbed, obtained from sub soil explorations.
- Thus for seepage control, the detailed investigations will give a fair basis for further analysis.

→ 4. For treating problem soils

- Problem soils are those which have uncommon behaviour when confronted with various parameters whether natural or artificial. Mainly two conditions come under this category :
- The first is called the quick sand condition. When water seeps through sand and comes to the surface, the head with which it comes out of the surface is called the exit gradient.
- Due to large increase in exit gradient, more than the critical gradient, effective submerged weight of sand may become zero due to upward force of seepage. Thus the sand comes to the surface continuously and appears to boil. This condition is called the quick sand condition and if it occurs below foundations, it will be dangerous.
- The other problem soil is known as the black cotton soil. The soil is not weak in bearing capacity, nor it is excessively permeable.
- The problem in black cotton soil is the disproportionate volume change which occurs upon change in moisture content.
- This soil swells to a very high degree, even upto double the original volume upon being saturated with moisture and shrinks equally largely when it loses the moisture.
- It may become even upto half its saturated volume when dried completely. Due to this property, the foundations resting on this type of soil will be subjected to large alternate movements during rainy season and summer.
- Hence special remedial measures must be taken when it is inevitable to build foundations on black cotton soil.
- This once again brings out the necessity of soil and subsoil investigations, as even small pockets of problem soils must be detected for safe foundations.

→ 5. For finding index properties

- Index properties of soil mean basic properties such as bulk density, specific gravity, void ratio, porosity, water content, degree of saturation etc. These properties come into play and affect the other more complex properties such as permeability, shear strength, compressibility, bearing capacity etc.
- Hence index properties form the important basic step of investigation. Most of the index properties are determined in the laboratory, but sometimes field tests are also deemed to be important.
- For collecting the samples for determination of index properties, the site investigation and the subsoil exploration is thus found to be necessary.

→ 6. For enhancing properties by compaction and stabilization

- Compaction is actually the simplest method of soil stabilization. The term stabilization is generally used when compaction is enhanced by some other method or when it is enhanced by adding some substance as an admixture.
- In any case, be it compaction or stabilization, the properties of soil such as Maximum Dry Density (MDD), Optimum Moisture Content (OMC), permeability drainage characteristics etc. must be determined to a fine degree.
- Doing such an investigation will achieve much desired effects to a large extent within the least expense.
- Even the decision of whether to go for compaction or not, whether simple compaction or stabilization is required etc. depend on investigations of soil and subsoil carried out in details to a meticulous degree of perfection.

Syllabus Topic : Types of Exploration

5.11 Types of Exploration

→ (MSBTE –S-10)

Q. State and explain in brief the types of exploration. (S-10)

The subsoil explorations are generally carried out in two stages : General or preliminary exploration and detailed exploration.

- **Types of Exploration**
  - 1. General Exploration
  - 2. Detailed Exploration

Fig. C5.7 : Types of Exploration

5.11.1 General Exploration

General or preliminary exploration consists of :

1. Geological study of the site and site reconnaissance.
2. Study of local topography, existing excavations, cuttings, and drainage patterns.
3. Study of other natural features like streams, lakes, hills etc.
4. Location of High Flood Level marks.
5. Depth and composition of soil strata.
6. Types of foundation will be selected depending upon the properties of soil.

5.11.2 Detailed Exploration

Detailed exploration follows the general exploration. It consists of :

1. Determining nature, thickness and sequence of various subsoil strata.
2. Position of ground water table.
3. Index properties of different layers.
4. Taking samples to the laboratory obtained from various depth below the ground level, with a view to conduct all the tests.
5. Conducting many field tests which will give lots of required information.
6. Shear strength.
7. Permeability.
8. Compressibility
9. Density.

5.12 Methods of Exploration

→ (MSBTE – W-08, S-11, W-11, S-12, W-12, S-13, W-13)

Q. Explain any two methods of soil investigation. (W-08)

Q. State the methods of soil exploration and explain any one in detail. (S-11, W-11, W-13)

Q. What are the types of soil exploration. (S-13)

Q. State any four methods of site investigations. (S-12, W-12)

There are mainly two procedures for exploration. These are **open excavation** and **boring.**

- **Methods of Exploration**
  - 1. Open Excavation
  - 2. Boring
    - 1. Augur boring
    - 2. Wash boring
    - 3. Percussion boring
    - 4. Rotary boring

Fig. C5.8 : Methods of Exploration

5.12.1 Open Excavation

- Open excavation is also called trial pit. Generally, a square or rectangular trial pit is dug, and by measuring the thickness of various layers on the open face of the pit, fairly well idea about subsurface geology can be formed.
- Soil excavated from trial pits is taken to the laboratory for testing. The location and number of trial pits is determined as per the requirement. Size is generally 1 m × 1 m or less.

- Depth is not fixed and depends upon the soil strata, water table etc.

5.12.2 Boring

→ (MSBTE – S-13,S-14)

Q. Explain augur boring. (S-13, S-14)

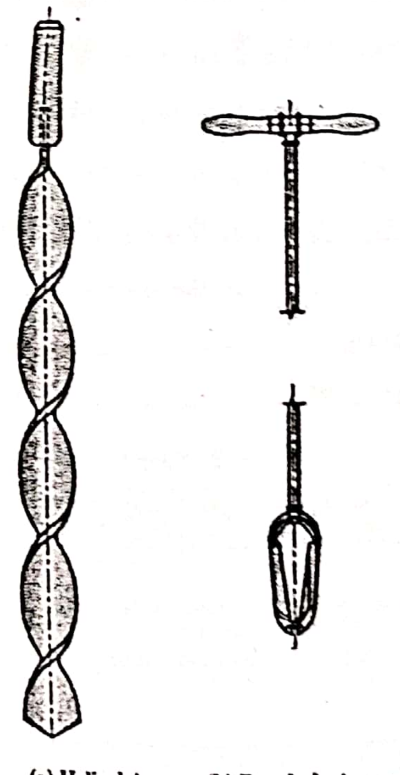
> ☞ ***Definition of Boring :*** *Marking and advancing a hole in the soil from the open surface is known as boring.*

There are four different methods of boring :

→ 1. Augur boring

- Boring by an augur is carried out by holding it vertically and pressing it down while it is rotated simultaneously.
- Due to the sharp edges, soil is cut and enters the annular space and due to spiral shape, rises up as the augur advances.
- After some depth the augur is brought up and cleaned, and then again inserted in the borehole. Fig. 5.12.1 shows two types of augurs.

Q. Draw neat sketches of hand augers used in sub-soil exploration. (S-14)



(a) Helical Augur (b) Post hole Augur

Fig. 5.12.1 : Augur boring

- Upto 6m depth hand operated augurs can be used. Augurs give severely disturbed samples. Augur boring is suitable for shallow foundations, roads etc.

→ 2. Wash boring

- The method consists of driving a casing pipe usually through a heavy drop hammer supported on a tripod and a pulley.
- Water is forced under pressure through a hollow drill rod which may be rotated or moved up and down inside the casing pipe.

Fig. 5.12.2 shows setup for wash boring.

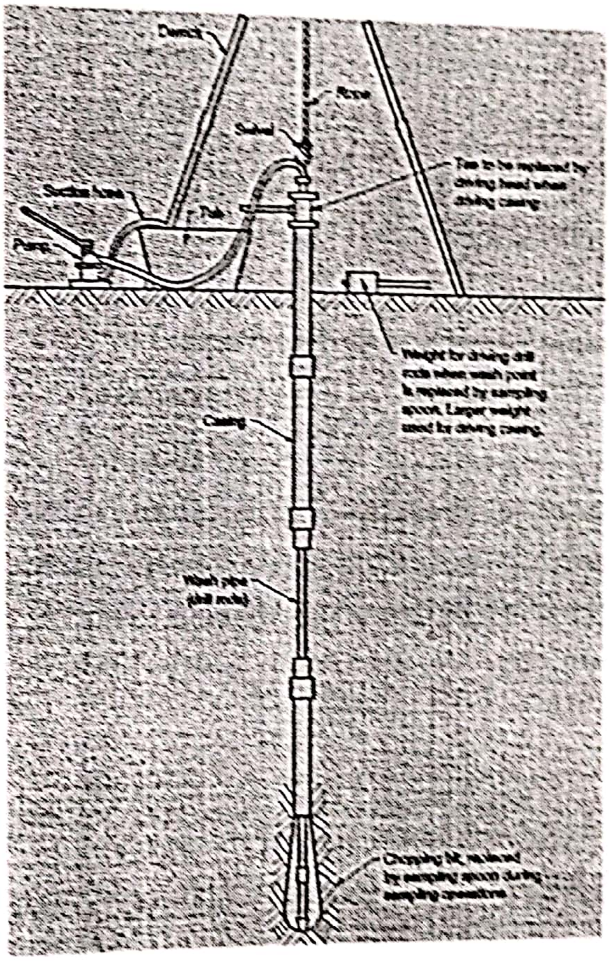


Fig. 5.12.2 : Wash boring

- The slurry flowing out can be analyzed for soil strata.
- Change in rate of progress and change in slurry characteristics indicates change of stratum.

As the particles from different depths may remain in suspension, analysis of slurry is of limited importance.

After sinking and cleaning the borehole, appropriate samplers must be used to collect samples from different depths.

The lower end of the drill rod fitted with a sharp cutting edge or diamond bit cuts the soil. The cut soil gets mixed with water and the slurry floats up.

3. **Percussion boring**

Percussion boring is carried out by repeated blows of a heavy bit or chisel inside a casing pipe.

The borehole is usually kept dry except for a limited quantity of water used to form the slurry of pulverized material.

The pulverized slurry is bailed out using a bailer or sand pump. A casing pipe is necessary only when the sides of borehole are likely to cave in.

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

➔ 4. **Rotary boring**

- In this method, boring is effected by cutting action of a rotating bit which is kept in firm contact with the soil.
- This method is also used to deepen boreholes already done by other methods.

5.12.3 Field Visit

- A field visit to an on the spot study is must for any site in estimation the experienced engineer 100 ks for or to pographic features, vegetation, streams, water levels in open well lakes and rivers.
- Performance of the neighbouring structures (cracks in the buildings, underscouring of foundations), rock exposures, flood marks, vibration sources etc. Such a visit coupled with inspection of a few test pits, is all that may be necessary as the site exploration for small unimportant project.

5.12.4 Library Study

This includes study of geological data and soil maps, toposheets, aerial photographs, hydrological data, flood records and history of seismic activity and such other available records.

5.12.5 Test Pits

- Trial pits, open shafts, quarries and tunnels and considered to give a reliable information of soil profile. The spot is directly accessible for visual examination, field testing and sampling. A test pit usually has the size of 1.2 m × 1.2 m.
- The depth of open excavation is limited, however, to about 6 m in clays. In non-cohesive soils, excavation below ground water table becomes very expensive, hence depth of open test pits is restricted to less than 3 m.
- Foundation trenches often serve as trial pits. Freshly excavated roadway cuts also serve similar purposes.
- Field density tests, plate load tests and collection of sample for routine laboratory testing are done in the test pits.

Syllabus Topic : Criteria for Deciding the Location and Number of Test Pits and Bores

5.13 Criteria for Deciding the Location and Number of Trial Pits and Bore Holes IS 1892 - 1979

➔ (MSBTE – S-08, W-09, W-10, S-12, S-13, W-15)

Q. State criteria for deciding the location and number of test pits and bores. *(S-08, W-09, S-12, W-15)*

Q. State the factors influencing location and type of soil exploration. *(W-10)*

Q. Discuss the criteria for deciding the location and number of test pits and bore holes for soil exploration. *(S-13)*

- The purpose of soil exploration as discussed earlier, is to provide the designer with complete data about subsoil strata at the site.

- Thus the location and number of trial pits and borings should be such as to reveal any major changes in thickness, depth or properties of different layers in and around the site.
- The number and spacing of bore holes depends upon :
  1. Extent of site
  2. Uniformity of strata
  3. Nature of structure and loading
  4. Geological study of the area.

1. **Number of Bores**

→ **(MSBTE –W-12)**

Q. Explain boring criteria for deciding the location and number of test pits for a cluster of buildings and Dam. ? *(W-12)*

- **IS 1892 – 1979 gives guidelines as follows :** Site area about 0.4 hectares, one bore hole or trial pit at center and one at each corner of plot.
- For smaller and less important buildings, even one trial pit or bore hole at the centre of plot is deemed to be sufficient.
- For larger areas, it may be useful to perform sounding tests or cone penetration tests at a spacing of 50 m to 100 m by dividing the area in a grid pattern. Thereafter, the number of bore holes or trial pits can be specified.

2. **Spacing of boring**

→ **(MSBTE – S-11)**

Q. Discuss the spacing and depth of borings for soil investigation for shallow and deep foundations. *(S-11)*

The spacing of borings should be such that

- For compact building sites covering an area of about 0.4 hectare, one bore hole or pit in each corner and one in centre may be adequate.
- For smaller and less important building, even one bore hole or test pit in center may suffice.
- Larger areas may be divided in a grid pattern and cone penetration may be performed at every 100m. On dam sites, it may be 50 m spacing. On road sites, the spacing along centre line may be about 100m.

3. **Depth of boring**

Exploration should be carried out to a depth up to which the increase in pressure due to structural loading is likely to cause settlement or shear failure.

Depth of boring depending :

(i) Type of structure

(ii) Weight of structure

(iii) Size and shape of structure

(iv) Disposition of loaded area

(v) Soil profile and its properties.

5.14 Soil Samples

Soil samples, in general, can be classified in two categories, namely, disturbed samples and undisturbed samples.

5.14.1 Disturbed Soil Samples

→ **(MSBTE – S-08, S-09, W-09, S-10, S-11, W-11,W-13)**

Q. What do you mean by disturbed samples *(S-08, S-09, W-09, S-10)*

Q. Explain distributed soil sample for testing. *(S-11, W-13)*

Q. Define disturbed soil sample. *(W-11)*

- These are samples where the natural soil structure gets modified or destroyed during the process of sampling. With suitable precautions the natural moisture content and proportion of mineral constituents can be preserved.
- These are called **representative samples** even if they are disturbed.

- Though it gets disturbed these samples contains the composition and the mineral contains of the soil and these disturbed sample can be used to determine the index properties of soil such as grain size, plasticity characteristic and specific gravity.

5.14.2 Undisturbed Samples

➔ (MSBTE – S-08, S-09, W-09, S-10, S-11, W-11, W-13)

> Q. What do you mean by undisturbed samples. *(S-08, S-09, W-09, S-10)*
>
> Q. Explain undisturbed soil sample for testing. *(S-11, W-13)*
>
> Q. Define undisturbed soil sample. *(W-11)*

- These are samples where the original soil structure is preserved and the material properties have not undergone any alteration or modification.
- Such samples are practically impossible to obtain as the act of cutting the sample will alter the stress conditions and the soil structure. For all practical purposes an undisturbed sample is considered as one in which the material has undergone very little change and it is still suitable for all laboratory tests including shear strength and consolidation tests.

Syllabus Topic : Field Identification of Soil – Dry Strength Test, Dilatancy Test and Toughness Test

5.15 Field Identification of Soil

➔ (MSBTE – W-08, S-09, W-09, S-10, W-10, W-13, S-15, W-15, S-16)

> Q. List out the field identification tests. Explain any two tests. *(S-09, W-09)*
>
> Q. Enlist the tests for field identification of soil and explain any one test. *(W-08, S-10, W-13, S-15)*
>
> Q. Explain in brief about the field tests done on soil for identification. *(W-10)*
>
> Q. Name any four field tests done on soil. *(W-10)*
>
> Q. Explain any two field identification tests for fine grained soils. *(W-15)*
>
> Q. State field identification tests on soil and explain any one. *(S-16)*

Field Identification of Soil

- 1. Dry Strength Test
- 2. Dilatancy Test
- 3. Toughness Test
- 4. Other Identification Test

Fig. C5.9 : Field Identification of Soil

- Many times, laboratory tests are neither feasible nor practicable.
- At such instances, if a name tag can be put to a soil, a knowledgeable soil engineer will be able to gain much data from simply identifying the soil type.
- Field tests and field identification is thus a great tool in soil engineering. Following important field tests are discussed here.
- All the tests are for fine grained soils. The soil is sieved on site through 425 micron sieve and the fraction passing through is taken for the tests.

5.15.1 Dry Strength Test

➔ (MSBTE – W-14)

> Q. Explain dry strength. *(W-14)*

- The prepared sample is completely dried in sun or by air drying. Its strength is tested by breaking lump between fingers.

> ☞ ***Definition of dry strength :*** *Resistance to breaking, termed as **dry strength** is a measure of the plasticity and is considerably influenced by the colloidal fraction content of the soil.*

If the dry sample can be easily powdered, it is said to have low dry strength, if considerable finger pressure is required to break the lump, it has **medium dry strength** and if the lump cannot be powered by fingers at all, it has a **high dry strength**.

Dry strength is characteristic of highly plastic clays. Typical inorganic silts have very less dry strength.

Silty fine sands and silts have the same low dry strength, but can be distinguished from each other by their feel during powdering of the dried sample.

Dry strength test is also known as **crushing resistance test.**

5.15.2 Dilatancy Test

→ **(MSBTE – W-10, W-13, W-14)**

Q. What is dilatancy of soil. *(W-10)*

Q. Explain Dilatancy test. *(W-13, W-14)*

- This is another simple test for fine fractions of soils.
- Dilatancy means reaction to shaking. About 5cc soil sample is taken and enough water is added to nearly saturate it.
- The pat of soil is placed in the open palm of the hand and shaken horizontally by striking vigorously against the other hand several times. The pat is then squeezed between the fingers.
- The appearance and disappearance of water with shaking and squeezing is called a positive reaction. This reaction is called quick if water appears and disappears rapidly, slow if water appears and disappears slowly and no reaction if water condition does not appear to change.
- The type of reaction is observed and recorded. Inorganic silts show a quick reaction where as clays show no reaction or slow reaction.

5.15.3 Toughness Test

- The soil sample used in dilatency test is dried by working and moulding till it reaches the consistency of putty.
- The time required to dry the sample is indicative of its plasticity. Further, the moisture content is reduced by rolling and re-rolling the soil into a thread of 3 mm diameter till it reaches the plastic limit.

> ***Definition of Toughness :** The resistance to moulding at plastic limit is called the toughness.*

- After the thread crumbles, the pieces are lumped together and the slight kneading action is continued until the lump also crumbles.
- If the lump can still be moulded slightly drier than plastic limit and if high pressure is required to roll the thread between the palms of the hand, the soil is said to have **high toughness.**
- **Medium toughness** is indicated by a medium thread and a lump formed of the threads slightly below the plastic limit will crumble while low **toughness** is indicated by a weak thread that breaks easily and cannot be lumped together when drier than plastic limit.

5.15.4 Other Identification Test

- If a dry or slightly moist lump of soil when cut or rubbed with considerable pressure with a knife blade produces a shiny surface, high plasticity is indicated dull surface indicate silt or clay of low plasticity.
- Wet gives greasy feel and does not wash off quickly where as slit will wash away easily. If dry, in a soil suspension in water of about 10 cm depth sand will settle with in half a minute and most of silt in about 5 to 60 min whereas clay size remain in suspension for atleast several hours for several dons

5.16 Empirical Correlation between Soil Properties and SPT Values

- The values of SPT are important as they have a direct correlation with soil properties.
- These correlations are only empirical and are not mathematically derived but they give very good results.

Following two tables show the empirical correlation between SPT values and soil properties for cohesionless and cohesive soils :

Table 5.16.1 : Cohesionless soils

N value	ϕ	Relative density (%)	Description
< 4	25 – 30	0	Very loose
4 – 10	27 – 32	15	Loose
10 – 30	30 – 35	65	Medium
30 – 50	35 – 40	85	Dense
> 50	38 – 43	100	Very dense

Table 5.16.2 : Cohesive soils

N value	Unconfined compressive strength (kg/cm^2)	Consistency
< 4	< 0.25	Very soft
2 – 4	0.25 – 0.50	Soft
4 – 8	0.50 – 1.00	Medium
8 – 16	1.00 – 2.00	Stiff
16 – 12	2.00 – 4.00	Very stiff
> 32	> 4.00	Hard

Chapter Ends

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