TH:4

GEOTECHNICAL ENGINEERING

CIVIL ENGINEERING
(SECOND YEAR)
(3rd Semester)

Prepared by
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CEPC207 TH:4 - GEOTECHNICAL ENGINEERING

L	T	P	Total Marks: 100	Course Code: CEPC 209
3	0	0		
Total Contact Hours				**Theory Assessment**
Theory : 45 Hrs				End Term Exam 70
				Progressive Assessment : 30
Pre Requisite : Nil				
Credit 3				Category of Course : PC

RATIONALE

The knowledge and skills of Geo-Technical Engineering help the Practicing Civil Engineers in Civil Engineering Construction Works, especially in the design and construction of building foundation.

LEARNING OUTCOME

After completion of this course, the students will be able to

- Determine physical and index properties and classification of soil
- Estimate permeability and shear strength of soil
- Identify the load bearing capacity of soil
- Explain various soil stabilization and compaction methods
- Use IS codes for different soil testing

COURSE CONTENT DETAILS

UNIT NO.	CONTENT	TIME ALLOTTED (HRS.)
UNIT-I	**Overview of Geology and Geotechnical Engineering** • Introduction of Geology, Branches of Geology, Importance of Geology for civil engineering structure and composition of earth, • Definition of a rock: Classification based on their genesis (mode of origin), formation. Classification and engineering uses of igneous, sedimentary and metamorphic rocks. (Concepts only) • Importance of soil as construction material in Civil engineering structures and as foundation bed for structures. (Concepts only) • Field application of geotechnical engineering for foundation design, pavement design, design of earth retaining structures, design of earthen dam. (Concepts only)	5

UNIT-II	**Physical and Index Properties of Soil** • Soil as a three phase system, water content, determination of water content by oven drying method as per BIS code, void ratio, porosity and degree of saturation, density index.,air Content,Percentage of air voids,Relation between the parameters. • 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 and sand replacement method, Determination of specific gravity by pycnometer. • Consistency of soil, Atterberg limits of consistency: Liquid limit, plastic limit and shrinkage limit. Plasticity index. • Particle size distribution test and plotting of curve, Determination of effective diameter of soil, well graded and uniformly graded soils, BIS classification of soil.	**8**
UNIT-III	**Permeability and Seepage** • 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 tests, simple problems to determine coefficient of permeability. Seepage through earthen structures, seepage velocity, seepage pressure, phreatic line, flow lines, application of flow net, (Concepts only No numerical problems).Effective stress,quick Sand	**8**
UNIT-IV	**Compaction,Consolidation and stabilization of soil** Concept of compaction, Standard and Modified proctor test as per IS code, Plotting of Compaction curve for determining: Optimum moisture content (OMC), maximum dry density (MDD), Zero air voids line. Factors affecting compaction, field methods of compaction – rolling, ramming and vibration. • Consoildation,Difference between compaction and consolidation.Terzaghi's Model analogy of compression/springs showing the process of consolidation,Field implications • Concept of soil stabilization, necessity of soil stabilization, different methods of soil stabiliza tion. California bearing ratio (CBR) test - Meaning and Utilization in Pavement Construction • Necessity of site investigation and soil exploration: Types of exploration, criteria for decidingthe location and number of test pits and bores. Field identification of soil – dry • strength test, dilatancy test and toughness test.	**8**

UNIT-V	**Shear Strength of Soil** • Shear failure of soil-General,local and punching shear, 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, triaxial and vane shear test laboratory methods.	**8**
UNIT-VI	**Bearing Capacity of Soil and Foundation** • **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, effect of water table on bearing capacity.** • **Field methods for determination of bearing capacity – Plate load and Standard Penetration Test. Test procedures as per IS:1888 & IS:2131.** **Definition of earth pressure, Active and Passive earth pressure for no surcharge condition, coefficient of earth pressure, Rankine's theory and assumptions made for non- cohesive Soils.** ● **Type of foundations-shallow,deep foundation**	**8**
		45

SUGGESTED LEARNING RESOURCES:

1. Punmia, B.C., Soil Mechanics and Foundation Engineering, Laxmi Publication, Delhi.
2. Murthy, V.N.S., A text book of soil mechanics and foundation Engineering, CBS Publishers & Distributors Pvt. Ltd., New Delhi.
3. Ramamurthy, T.N. & Sitharam,T.G., Geotechnical Engineering(Soil Mechanics), S Chand andCompany LTD., New Delhi.
4. Raj, P. Purushothama, Soil Mechanics and Foundation Engineering, Pearson India, New Delhi.
5. Kasamalkar, B. J., Geotechnical Engineering, Pune Vidyarthi Griha Prakashan, Pune.
6. Arora K R, Soil Mechanics and Foundation Engineering, Standard Publisher.

UNIT-I Overview of Geology and Geotechnical Engineering

1.1. Geology

The word "Geology" is made up of two parts: "geo," meaning Earth, and "logy," meaning study. So, geology is simply the **study of the Earth**. It looks at the Earth's **composition**, **origin**, and **structure**. Geology helps explain how the Earth's surface has changed over millions of years.

1.1.1. Branches of Geology

1.1.1.1. Physical Geology

Physical geology is the study of the forces that shape and change the Earth's crust. These forces can be **internal** (from inside the Earth) or **external** (from things like wind, water, or ice). It is also known as **dynamic geology** because it focuses on the Earth's changing nature.

1.1.1.2. Petrology

Petrology is the study of **rocks**. It looks at what rocks are made of, their texture, how they are arranged, and how they form. It also studies where rocks are found and how different conditions affect them.

1.1.1.3. Mineralogy

Mineralogy is the study of **minerals**. It includes looking at the structure, chemical properties, and physical features of minerals. This branch also studies how minerals form and change in nature.

1.1.1.4. Structural Geology

Structural geology deals with how **rocks deform**. This includes understanding the shapes and structures of rocks caused by forces that push, pull, or twist them. It helps geologists study how rocks change over time and how they create features like mountains or valleys.

1.1.1.5. Stratigraphy

Stratigraphy is the study of **rock layers** (called strata). It helps geologists understand the order in which rocks formed and how they relate to each other in time.

1.1.1.6. Paleontology

Paleontology is the study of **fossils** (the remains of plants and animals) from the past. It helps us learn about life that existed millions of years ago by studying fossilized plants, animals, and even tiny microorganisms found in rocks.

1.1.1.7. Economic Geology

Economic geology focuses on finding and using **natural resources** like minerals, metals, and fuels. It looks at how these resources are formed and where they can be found for mining and other uses.

1.1.1.8. Engineering Geology

Engineering geology is about using **geological knowledge** to solve engineering problems. It helps engineers make sure that their projects (like buildings, bridges, or dams) are safe, cost-effective, and well-suited to the land they are built on.

1.1.2 Importance of Geology in Civil Engineering

What is Engineering Geology?

Engineering geology is the study of **rocks and soils** that are used in construction. It helps civil engineers understand the ground where a project will be built.

Why is geology important for civil engineers?

- Helps in **planning safe and strong structures**.
- Helps avoid **construction problems** by studying the ground before building.
- Saves **time and money** by avoiding surprises during construction.
- Rocks and soils **carry the weight** of buildings, bridges, dams, and roads.

Where is geology used in civil engineering?

- **Dams, tunnels, bridges, railways**, and other large structures.
- Choosing **good construction materials** like sand, gravel, and rocks.
- Reading **geological maps** to know the type of soil or rock in an area.
- Understanding natural features like **faults, joints, and folds** that can affect safety.

Before construction starts:

- **Geological investigation** must be done.
- Site problems should be **recorded** and shared with engineers.
- Solutions must be applied to design **safe and stable structures**.
- Civil engineers and geologists should **work together** for success.

1.2 Rocks

The **Earth's crust** (outer layer of Earth) is made of different rocks. Rocks are used in construction as **building material** and **foundation support**.
Rocks are classified based on how they were formed:

1.2.1 Igneous Rocks

Formed from **cooling of molten magma** (hot fluid from inside the Earth).

- **Intrusive Igneous Rocks**: Cooled **inside** the Earth (e.g., Granite). Slow cooling = **large crystals**.
- **Extrusive Igneous Rocks**: Cooled **on the surface** (e.g., Basalt). Fast cooling = **fine crystals or glassy** texture.

Common Igneous Rocks and Uses:

- **Granite**: Hard, durable. Used for **foundations, tiles, countertops**.
- **Charnockite**: Like granite. Used in **temples and historical buildings**.
- **Gabbro (Black Granite)**: Used in **roads and pavements**.
- **Dolerite**: Very hard. Used in **road layers and concrete**.
- **Basalt**: Fine-grained, used in **road base, railway ballast**.

Fig. 1. 1 Examples of igneous rocks

1.2.2 Sedimentary Rocks

Formed by **deposition and compression** of rock particles or chemical materials.

Examples and Uses:

- **Sandstone**: Strong and used in **foundations** and buildings.
- **Shale**: Soft rock, used in making **bricks, tiles, cement**.
- **Conglomerate**: Formed from rounded stones stuck together. Used as **filler** or **decorative stone**.

Fig. 1. 1 Examples of Sedimentary rocks

1.2.3 Metamorphic Rocks

Formed from other rocks (igneous or sedimentary) under **high heat and pressure**.

Examples and Uses:

- **Gneiss**: Very hard. Good for **foundations**.
- **Khondalite**: Used in **ancient Indian temples**.
- **Granulite**: Strong, used as **floor tiles and decorative stone**.
- **Schist**: Easily split. Used for **walls and decoration**.
- **Slate**: Splits into flat sheets. Used in **roofs, fences, and walls**.
- **Quartzite**: Very hard. Excellent **foundation material**.
- **Marble**: Decorative stone used in **buildings and statues**.
- **Laterite**: Soft when wet. Used for **bricks and foundation**.

Fig. 1. 1 Examples of Metamorphic rocks

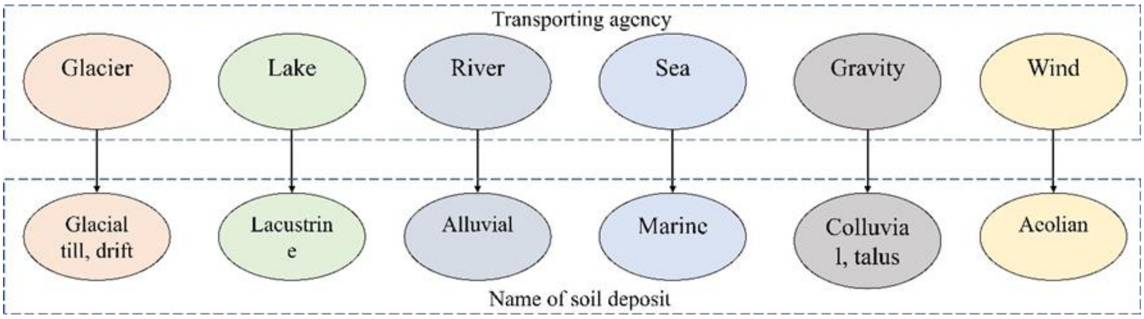
1.3 Soil

What is soil?

Soil is made from **broken rocks** and contains **air, water, and organic matter**.
It supports structures like **buildings, roads, and dams**.

Types of Soil:

- **Residual Soil**: Stays near where it was formed.
- **Transported Soil**: Moved by **water, wind, glaciers, or gravity**.



Engineering Properties of Soil:

- **Strength**
- **Permeability (water flow)**
- **Compaction**
- **Plasticity**
- **Shrink-swell behavior**

1.3.1 Applications of Soil in Construction

1. Foundations

- Transfer building load to the soil.
- Two types:
  - **Shallow Foundations** (near the surface)
  - **Deep Foundations** (piles, for deep soils)

2. Pavements

- **Flexible Pavements**: Layers of soil, gravel, and bitumen.
- **Rigid Pavements**: Concrete slabs over soil and gravel.

3. Retaining Structures

- Walls that **hold back soil** on slopes or different levels.
- Made of concrete, stone, or sheet piles.

4. Earthen Dams

- Dams made using soil.
- Types: **Homogeneous, Zoned, Diaphragm**.
- Require careful design for **slope stability** and **seepage control**.

Unit Summary (Simple)

- Geology is very important in civil engineering.
- Rocks and soil are **main materials** in construction.
- **Understanding geology** helps to build safe, strong, and long-lasting structures.
- Different rocks and soils have **different uses** depending on their properties.
- Soil and rock support all civil engineering structures like buildings, roads, and dams.

EXERCISES

Multiple Choice Questions

1. Soil transported by gravity is called:
   d) Colluvial
   d) Alluvial
   d) Aeolian
   d) Lacustrine
2. Based on its origin, schist is classified as:
   d) Igneous rocks
   d) Sedimentary rocks
   d) Metamorphic rocks
3. The branch of geology that deals with the layers of soil is known as:
   d) Engineering geology
   d) Palaeontology
   d) Stratigraphy
   d) Structural geology
4. Which of the following rocks are least suitable as a foundation material
   d) Granite
   d) Shale
   d) Gneiss
   d) Quartzite

Short and Long Answer Type Questions

1. What are intrusive and extrusive igneous rocks?
2. What is metamorphism?
3. What is meant by a retaining structure?
4. What are the different branches of geology?
5. Describe the engineering applications of different igneous rocks.
6. How will geological knowledge help in civil engineering projects?
7. Explain the classification of rocks based on their origin, with examples.
8. What are the major roles of soil in construction projects? Explain with examples.
9. Draw the figure of a zoned earthen dam and mark the parts.

UNIT-II

Physical and Index Properties of Soil

1.1. Soil as a three-phase system

Soil is a mixture of solids, air, and water, which constitutes the three phases of soil. The solid phase exists as particulate material of varying sizes, and the void spaces in between are filled with either air or water or both. When all the voids are filled with air, the soil is said to be in dry condition, and when water occupies all the void spaces, the soil is said to be in a fully saturated condition.

1.1.1. Three phase diagrams

Three phase diagram is the graphical representation of different phases in soil. The dry state, partially saturated state, and fully saturated state can be represented graphically as shown in Fig.

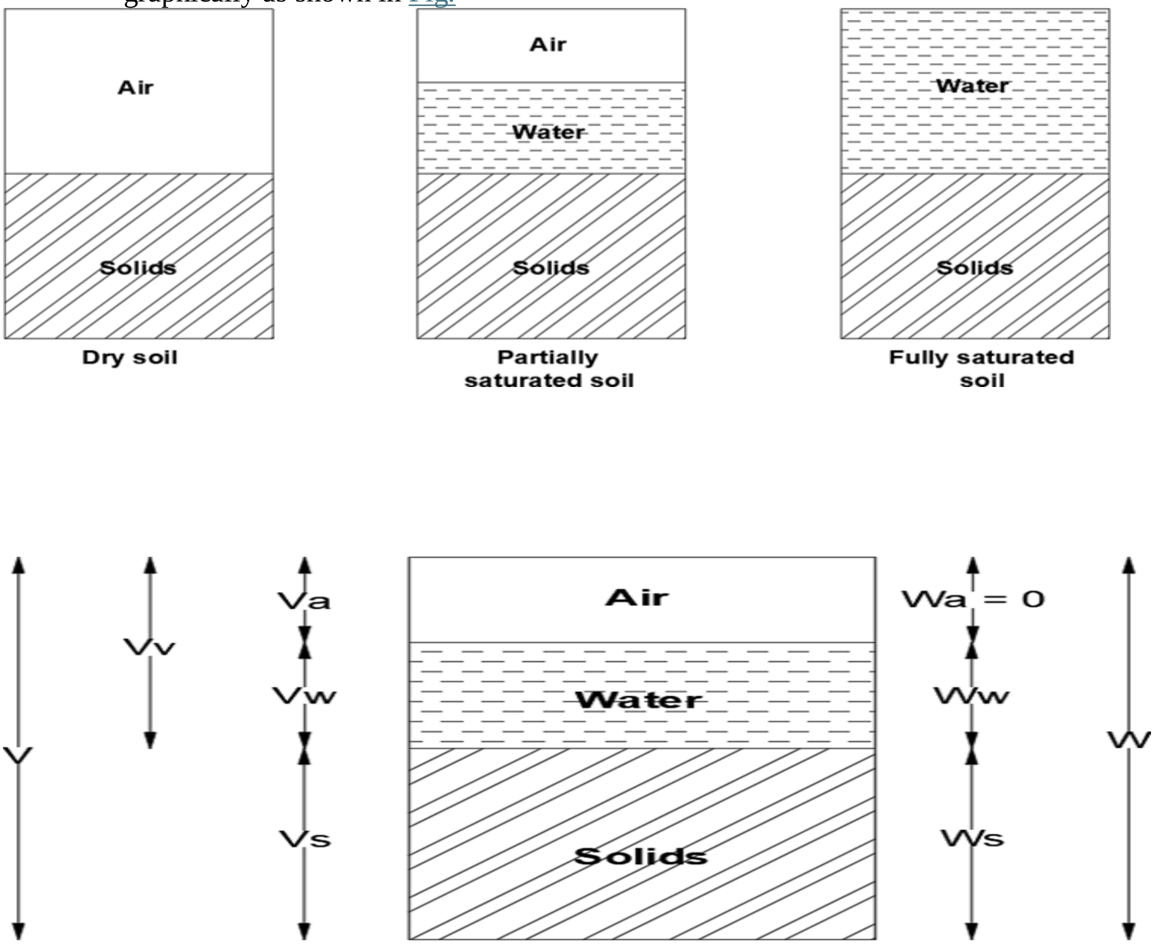


Fig. 2. 2 Three-phase diagram

The total volume of the soil sample always remains constant, but the weight of the sample varies according to the amount of water in void spaces. The different volumetric relationships in the three-phase system are as follows:

1.1.1.1. *Void ratio (e)*

Void ratio is the ratio of the volume of voids to the volume of solids in a soil mass.

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

1.1.1.2. *Porosity (n)*

Porosity is the ratio of the volume of voids to the total volume of soil mass.

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

1.1.1.3. *Degree of saturation (S)*

Degree of saturation is the ratio of the volume of water to the volume of voids in the soil mass.

$$S = \frac{V_W}{V_V}$$

1.1.1.4. *Percent air voids (n_a)*

Percent air voids is defined as the ratio of the volume of air to the total volume of soil mass.

□□ = □□
□

1.1.1.5. *Air content (a_c)*

Air content is the ratio of the volume of air to the volume of voids in the soil mass.

$$ac = \frac{V_a}{V_V}$$

1.1.2. Water content

Water content or moisture content of a soil mass is defined as the ratio of the weight of water to the weight of solids in the soil mass.

$$w = \frac{W_w}{W_s}$$

The water content of the soil is a critical parameter that controls the engineering behaviour of soil. The following are some methods used to determine the water content:

- Oven drying method
- Pycnometer method
- Torsion balance method
- Sand bath method
- Calcium carbide method
- Alcohol method
- Rapid moisture meter

Oven drying is the most widely followed laboratory method for the determination of water content. This is highly accurate but requires a duration of 24 hours to get the results. In the field, water content is determined by rapid moisture meter, sand bath method, alcohol method, or calcium carbide method.

1.1.3. Unit weight of soil

The unit weight, also known as the ‘weight density, of a soil refers to its weight per cubic metre and is typically expressed as kilonewtons per cubic metre (kN/m^3), or tons per cubic metre (t/m^3). Unit weight is a crucial parameter required for geotechnical design and volume estimation of earthworks. The water content, particle composition, and degree of compaction all affect the unit weight. Based on the water content and submergence, different unit weights of soil are defined as follows:

1.1.3.1. Bulk unit weight (γ)

Bulk unit weight is the total weight of soil per unit volume, and is given by:

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

1.1.3.2. Dry unit weight (γ_d)

Dry unit weight is the weight of solids per unit weight of soil.

$$\gamma d = \frac{W_s}{V}$$

1.1.3.3. Saturated unit weight (γ_{sat})

When the soil is fully saturated, the bulk unit weight is termed as saturated unit weight.

$$\gamma sat = \frac{W_{sat}}{V}$$

1.1.3.4. *Submerged unit weight (γ_{sub})*

If the soil exists below the groundwater table, it is said to be in submerged condition. The unit weight in this condition will be the buoyant weight per unit volume of soil.

$$\gamma sub = \frac{W_{sub}}{V}$$

This value is also equal to the difference between the saturated unit weight of soil and the unit weight of water.

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

The unit weight of water at 4°C is 1 g /ml or 9.81 kN/m^3.

1.1.3.5. *Unit weight of soil solids*

Unit weight of soil solids is the ratio of the weight of solids to the volume of solids in a soil mass.

$$\gamma s = \frac{W_s}{V_s}$$

In laboratory and field conditions, the bulk unit weight is measured using the following approaches:

- Water displacement method
- Submerged weight method
- Core cutter method
- Sand replacement method
- Water balloon method

Core cutter is a field method, suitable for soft and fine-grained soils, while the sand replacement is the most widely followed approach, for all soil types. The water balloon method is also suitable for all soil types. Once the bulk unit weight is determined, dry unit weight can be calculated using the following equation:

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

1.1.4. Specific gravity

In general, specific gravity is termed as the ratio of unit weight of any material to the unit weight of water. In case of soil, two values of specific gravity are used, which are mass specific gravity and true specific gravity.

1.1.4.1. Mass or bulk specific gravity (G_m)

Mass specific gravity is defined as the ratio of bulk unit weight of the soil to the unit weight of water, given by:

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

1.1.4.2. True or absolute specific gravity or specific gravity of soil solids (G)

True specific gravity is the ratio of the unit weight of soil solids to the unit weight of water.

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

Specific gravity is determined in the laboratory using the following approaches:

- Density bottle method
- Pycnometer method
- Measuring flask method
- Gas jar method
- Shrinkage limit method

1.1.5. Inter relationships

Among the discussed parameters, water content, unit weight and specific gravity are determined directly in the laboratory, and the other parameters are derived using inter-relationships between the parameters. These relationships are listed below in

Table 2. 1 Inter-relationships

Sl. No.	Relationship
1	$n = \frac{e}{1+e}$
2	$e = \frac{n}{1-n} = \frac{G - G_m}{G_m - S}$
3	$n_a = n.a_c = 1 - \frac{(1+wG)\gamma_d}{G\gamma_w}$
4	$S + a_c = 1$
5	$eS = wG$
6	$\gamma = \frac{(G+eS)\gamma_w}{1+e} = \frac{(1+w)G\gamma_w}{1+e}$
7	$\gamma_d = \frac{\gamma}{1+w} = \frac{G\gamma_w}{1+e} = \frac{(1-n_a)G\gamma_w}{1+wG}$
8	$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$
9	$\gamma_{sub} = \frac{(G-1)\gamma_w}{1+e}$

1.2. Particle size distribution

Soil particles are mechanically separated based on their particle size. The particles of size greater than 75 µm, are called coarse grained soil, and their separation is carried out using sieve analysis. The particles with size less than 75 µm are called fine grained soils, and their particle size distribution is carried out using sedimentation.

Based on the size, soils are classified into gravels (>4.75 mm), sand (4.75 mm > particle size > 75

1.2.1. Laboratory analysis

Sieve analysis is carried out by arranging sieves of different sizes in decreasing order (largest one on top), and by shaking (manually, or using a mechanical shaker). The particles retained on each sieve is then measured to plot the grain size distribution curve. The sieve analysis is done separately for particles greater than 4.75 mm and those in between 4.75 mm and 75 µm.

For fine grained soil, sedimentation analysis is carried out based on Stoke's law, either using a pipette, or using a hydrometer.

When the soil mass consists of both coarse- and fine-grained particles, first the particles are washed to separate the fine particles attached to the larger ones. Sieve analysis is then carried out for the larger particles, and sedimentation for fine particles, and the final particle size distribution is obtained from the combined analysis. This process is called wet sieve analysis.

1.2.2. Particle size distribution curve

The particle size distribution curve is the graphical representation of particle sizes in a soil mass. It is a plot between the percentage of soil mass finer than a given size on y-axis, and the particle size on log scale in x-axis.

The distribution of particles of different sizes in a soil mass is called grading, and it can be determined from the particle size distribution curve. The particle size distribution curves for different soils are plotted below in Fig. 2. 3

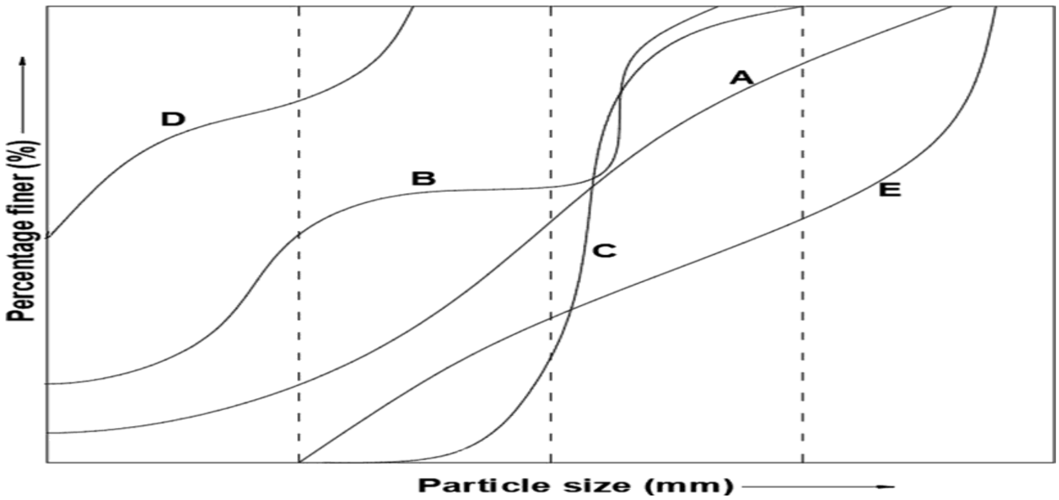


Fig. 2. 3 Particle size distribution curves

The curve A is a flat S curve, which implies that the mass consists of particles of different size in good proportion. Such soils are call well graded soil. In such soils, the space between larger particles will be occupied by smaller particles, and the void space will be minimum. Curve with intermediate flat portions as in B represents gap graded soil, where particles of intermediate size are missing. Very steep S curves like C represents uniformly graded soil, in which particles are of similar size. Curves like D on the left upper side of the graph represents fine particles, and as the curve shifts right, it indicates that the particles are of larger size.

Two important coefficients are calculated from the particle size distribution curve, which are known as the uniformity coefficient (C_u), and coefficient of curvature (C_c). While C_u expresses the uniformity of the soil, the general shape is described by C_c. These coefficients can be calculated from the plot as:

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

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

where D_{60} is the particle size such that 60 % of the soil is finer than this size, and similarly D_{30} and D_{10} are the particle sizes such that 30 % and 10 % of the particles are finer than this size respectively. D_{10} is also called the effective size. Higher values of C_u indicate that the particle sizes are largely varying.

Gravels are considered well graded when they have a C_u value greater than 4, and a C_c value between 1 and 3.

In the case of sand, C_u greater than 6 and C_c between 1 and 3 are considered as the criteria for well graded soil. If any of these criteria are not satisfied, the soil is considered to be poorly graded.

1.2.3. Relative density

Relative density is another important index property of coarse-grained soils. It is also known as density index. This parameter indicates how the soil will behave under loads. Higher relative density indicates that the soil mass is dense and can take heavy loads. The relative density can be calculated using the following expression, either using void ration, or by using dry unit weight:

$$D_r or\ I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \left[\frac{(\gamma_d)_{max}}{\gamma_d}\right]\left[\frac{\gamma_d - (\gamma_d)_{min}}{(\gamma_d)_{max} - (\gamma_d)_{min}}\right]$$

where e_{max} and $(\gamma_d)_{min}$ corresponds to the void ratio and dry density at the loosest state of soil,

e_{min} and $(\gamma_d)_{max}$ corresponds to the void ratio and dry density at the densest state, and e and γ_d

are the void ratio and dry density at the natural state of soil.

1.3. Consistency of soil

In the case of coarse-grained soil, the particles are separate, and particle size distribution and relative density can provide indications on the engineering behaviour. When it comes to fine- grained soil, the particles often stick together due to cohesion, and the properties are highly influenced by the moisture content. The ease at which soil can be deformed is known as consistency. The same soil can exist in solid state or can behave like a liquid with variation in water content. This range from liquid solid was divided into four distinct states by Swedish Engineer Atterberg in 1911. The four stages are separated by three moisture contents, which are known as the Atterberg limits or consistency limits (Fig. 2. 4).

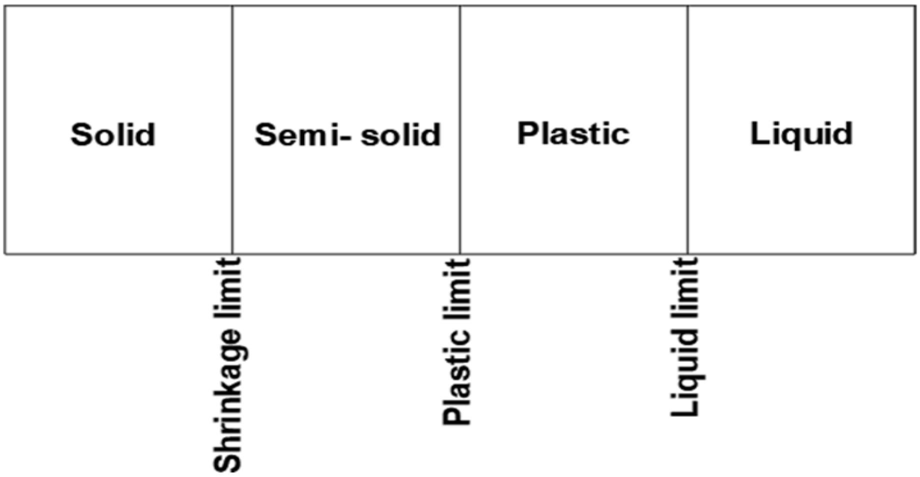


Fig. 2. 4. Atterberg limits

1.3.1. Atterberg limits

1.3.1.1. *Liquid limit (w_l)*

Liquid limit is the water content at which soil changes from plastic to liquid state. It is defined as the water content at which a soil pat in the standard liquid limit apparatus cut by a groove of standard dimensions will flow together for a distance of 12 mm under impact of 25 blows of standard height.

1.3.1.2. *Plastic limit (w_p)*

Plastic limit is defined as the minimum water content at which soil just begin to crumble when rolled into a thread of approximately 3 mm in diameter.

1.3.1.3. *Shrinkage limit (w_s)*

Shrinkage limit is the maximum water content at which a reduction in water content will not cause a decrease in the soil volume.

1.3.1.4. *Plasticity index (I_P)*

Plasticity index is the range of water content over which the soil remains in the plastic state. It can be calculated as the difference between liquid limit and plastic limit.

$$I_P = w_l - w_p$$

1.3.1.5. *Shrinkage index (I_s)*

The shrinkage index is the numerical difference between plastic limit and shrinkage limit.

$$I_s = w_p - w_s$$

1.3.1.6. *Liquidity index (I_l)*

Liquidity index is the ratio of the difference between natural water content and the plastic limit, to the plasticity index.

$$I_l = \frac{w - w_p}{w_l - w_p}$$

If the value of I_l is greater than 1, the soil is in liquid state, and if the value is less than zero, soil is in semi solid state. Any value between 0 and 1 indicates that the soil is in plastic state.

1.3.1.7. *Consistency index (I_c)*

Consistency index is defined as the ratio of the difference between liquid limit and the natural water content to the plasticity index of soil.

$$I_c = \frac{w_l - w}{w_l - w_p}$$

If the value of consistency index is greater than 1, it means that the soil is in semi-solid state, and if the value is less than 0, the soil is in liquid state.

The consistency of soil in the field can be stated based on the values of I_l and I_c as listed in Table 2. 2

Table 2. 2 Consistency classification

I_c	I_l	Consistency
1.00 to 0.75	0.00 to 0.25	Stiff
0.75 to 0.50	0.25 to 0.50	Medium stiff
0.50 to 0.25	0.50 to 0.75	Soft
0.25 to 0.00	0.75 to 1.00	Very soft

1.3.1.8. *Flow index (I_f)*

The flow index is the slope of the flow curve drawn between the number of blows (log scale) on x axis and the water content along y axis in Cassagrande’s method of liquid limit determination (Fig. 2. 5).

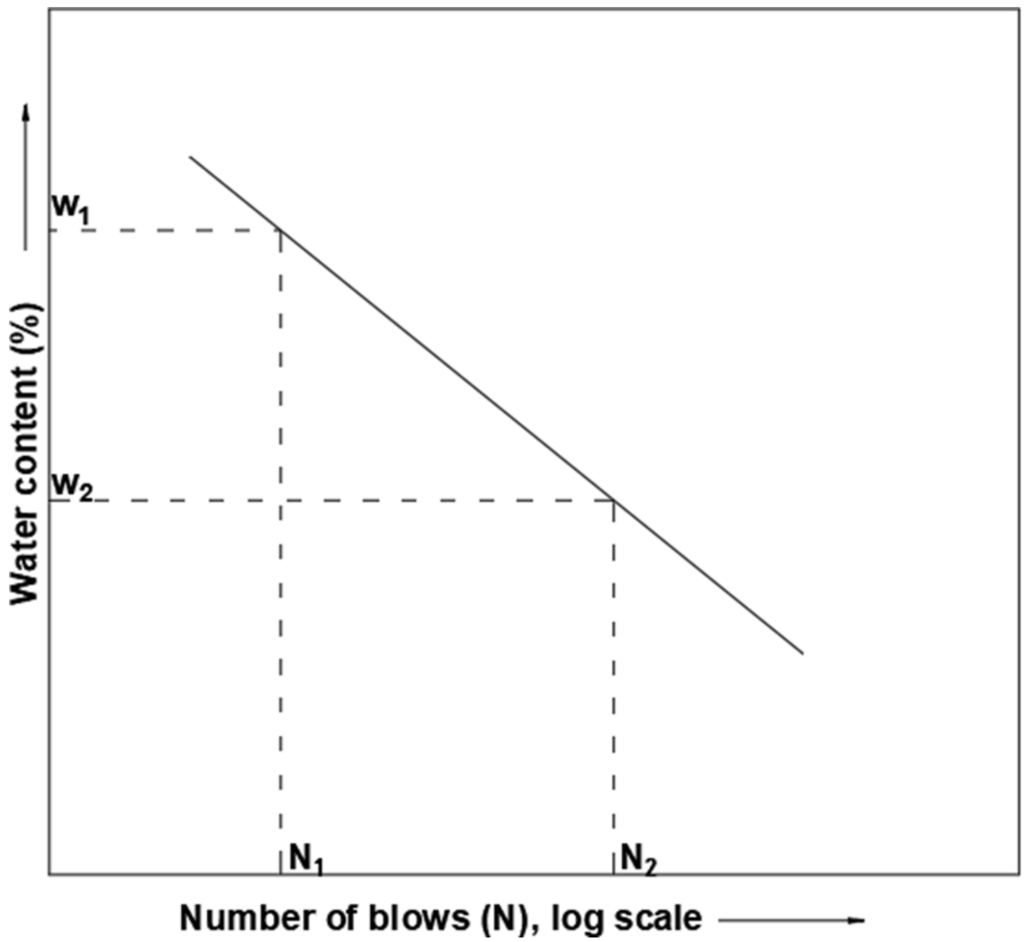


Fig. 2. 5

Flow curve Mathematically, I_f can be

calculated as:

$$I_f = \frac{w_1 - w_2}{\log_{10}\left(N_2 / N_1\right)} \text{H}$$

1.3.1.9.*Toughness index (I_t)*

Toughness index can be defined as the ratio of plasticity index and flow index.

$$I_t = \frac{I_p}{I_f}$$

1.3.1.10. Sensitivity

In the case of clayey soil, the strength properties are highly related to the structure of clay, which is the orientation and arrangement of particles. When samples are remoulded, it affects the strength of soil. Sensitivity (S_t) is defined as the ratio of unconfined compressive strength of soil in undisturbed condition to that in

remoulded condition, without any change in the water content.
Unconfined compressive strength (q_u) is obtained by providing axial compressive load to a cylindrical soil sample, which is laterally unsupported. The test procedure will be discussed in the further sections.

$$S_t = \frac{(q_u)_{undisturbed}}{(q_u)_{remoulded}}$$

Clays are considered to be sensitive if the value of sensitivity is greater than 4.

1.3.1.11. Activity

Minerals present in clay also have a very significant role on the engineering properties of soil. Activity is an indirect index of effect of clay minerals in a soil sample, using its plasticity index and amount of clay present in the sample, as mentioned in the following equation:

$$A = \frac{I_p}{F}$$

where *F* is the percentage of clay fraction in the soil sample. A clayey sample is considered to be active when the value of activity is greater than 1.25.

1.3.1.12. Thixotropy

Thixotropy is the change due to touch. Soil loses its strength while remoulding due to rearrangement of particles and disturbance caused to water molecules. Some of these changes can be reversed with time. When a remoulded soil sample stays without loss of water, it regains some part its strength, and this process of regaining strength with time after remoulding is known as thixotropy.

1.4. Classification of soils – IS classification system

Soil classification categorises different types of soils into groups according to their engineering behaviour. IS 1498:1971 (Reaffirmed in 2007) is the Indian Standard that deals with the classification and identification of soils for general engineering purposes. Based on this classification system, soils are broadly classified into 3, coarse grained soil, fine grained soils and highly organic soils and other miscellaneous materials. When more than 50 % of the material (by

weight) is retained on a 75 µm sieve, soil is called coarse grained soil, and if more than 50 % of the material (by weight) is passing through 75 µm sieve, the soil is classified as fine grained.

Organic soil and other miscellaneous soil materials consist of large percentages of organic matter such as decomposed vegetation, and peat. In addition, soils containing other materials like cinders, shells and non-soil materials are also grouped under this category.

Coarse grained soils can be further classified into gravels and sand, and the classification is shown in Fig. 2. 6.

The first letter indicates the particle size which is present in the maximum quantity. For coarse grained soils, it is always G or S, representing gravel or sand respectively. The second letter represent whether the soil is well graded or poorly graded, if the soil has no significant fine fraction. When fine fraction is present, the second letter represents either clay (C) or Silt (M), based on the plasticity chart (Fig. 2. 7), or dual symbols can be used. GW is termed as well graded gravel, and GC is termed as clayey gravel.

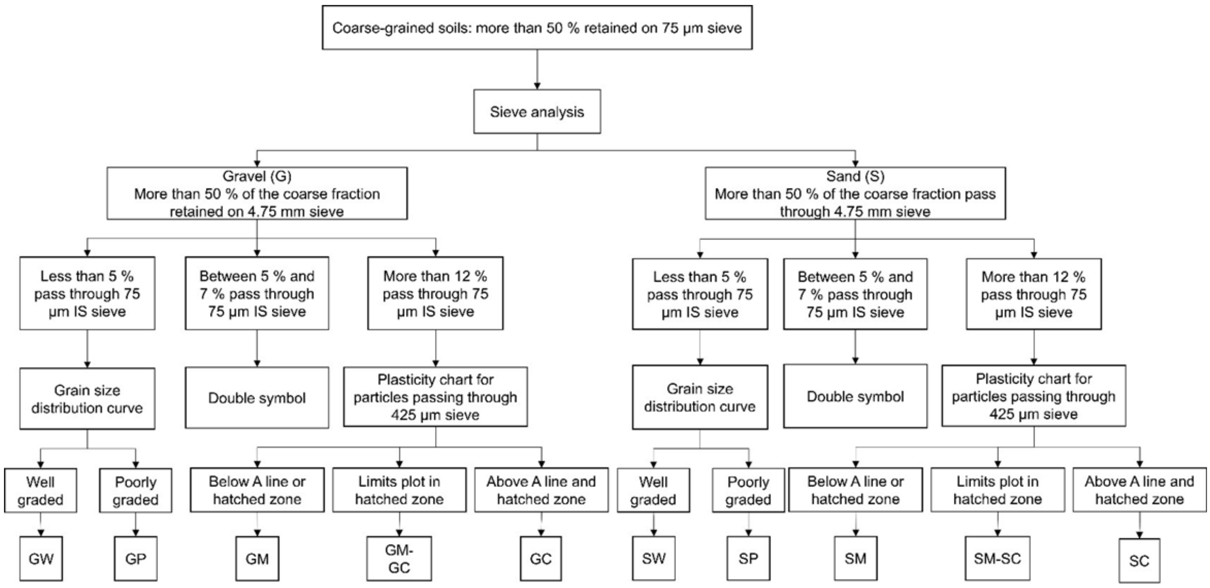


Fig. 2. 6 Flow chart for classification of coarse-grained soils

In the case of fine grained soils, plasticity chart plays a key role in the classification. The plasticity chart as per IS classification is shown in Fig. 2. 7.

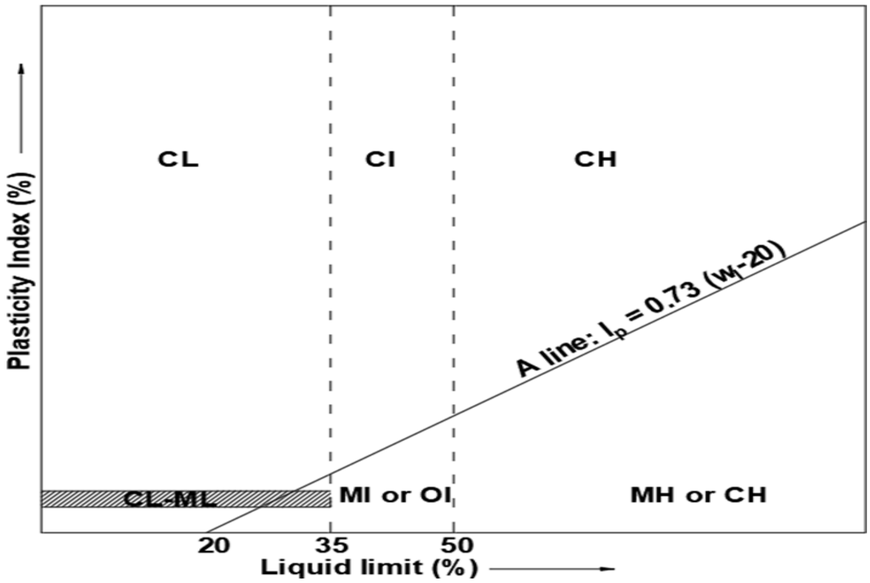


Fig. 2. 7 Plasticity chart and IS soil classification.

Similar to coarse grained soils, the first letter in the case of fine-grained soil also represent the predominant soil type. Unlike the coarse-grained classification, here the first letter is decided based on the liquid limit and plasticity index, which decided whether the soil lies above or below the A line on plasticity chart. The second letter in naming is decided based on the liquid limit value, and the symbols L, I and H represents low compressible, intermediate compressible, and highly compressible soils respectively. The flow chart for classification of fine-grained soils as per IS code is given in Fig. 2. 8.

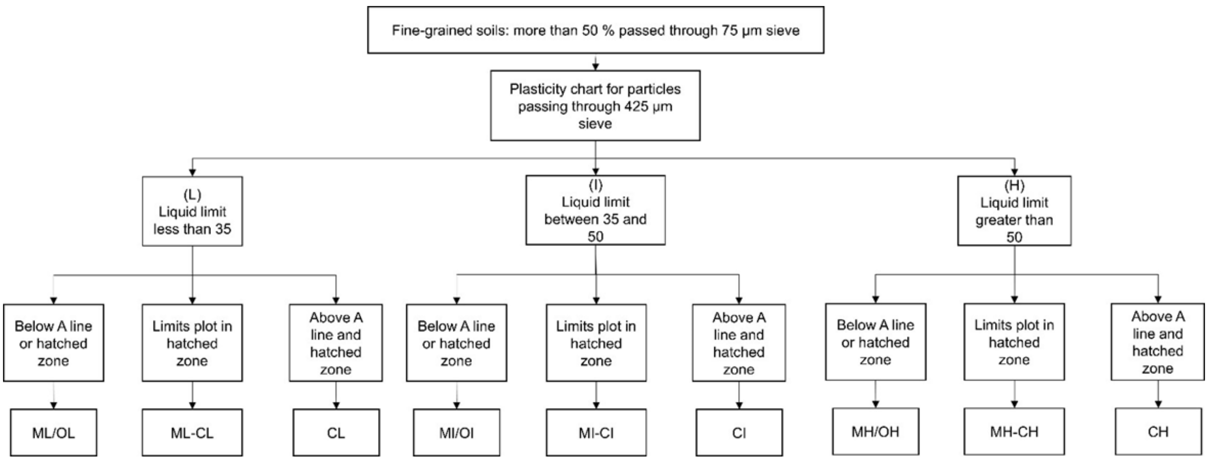


Fig. 2. 8 Flow chart for classification of fine-grained soils

UNIT SUMMARY

The unit discusses the physical and index properties of soils. Starting from the three-phase system, the volumetric and weight volume relationships are discussed in detail. Further, the index properties and soil classification are explained, to help the student understand how soil should be categorised for engineering application, based on Indian Standards.

EXERCISES

Multiple Choice Questions

1. The ratio of the volume of voids to the total volume of soil is-
    d) Voids ratio
    d) Degree of saturation
    d) Porosity
    d) Air content
2. The soil which plots above the 'A' line in the plasticity chart are-
    d) Silts
    d) Clays
    d) Sands
    d) Organic soils
3. At shrinkage limit, the soil is-
    d) Saturated
    d) Partially saturated
    d) Dry
    d) None of the above
4. A saturated soil sample has a bulk unit weight of 19 kN/m^3 and a specific gravity of 2.68. Determine the water content of soil?
    d) 0.166
    d) 0.3
    d) 0.143
    d) 0.277

4) Numerical Examples:

1. A partially saturated soil sample collected from an earth fill has a natural moisture content of 23 % and a unit weight of 19.6 kN/m^3. If the specific gravity of the soil is 2.65, calculate
    a. Void ratio
    b. Degree of saturation
    c. Saturated unit weight

2. A soil sample in dry state weighs 400 g and has a volume of 250 cm^3. Calculate the shrinkage limit and void ratio of the sample if the specific gravity is 2.67.

Exercises:

1. The liquid limit and plastic limit of a soil sample are 55 % and 25 % respectively. If the natural water content of the soil is 32%, calculate the consistency index and the liquidity index? **(I_c =0.6, I_l= 0.24)**
2. A soil sample has a bulk unit weight of 19.88 kN/m^3 at a moisture content of 16 %. Calculate the moisture content of the soil if the unit weight becomes 18.42 kN/m^3 after drying, while the voids ratio remains unchanged. **(w=7.48%)**
3. A clayey soil sample has liquid limit 51% and plastic limit 33 %. (a) In what state of consistency is this soil at a moisture content of 44 %? (b) What is the plasticity index of the soil?
   (c) The void ratio of this soil at the minimum volume reached on shrinkage, is 0.78. What is the shrinkage limit, if its grain specific gravity is 2.61? **((a) Plastic state (b) I_p= 18% (c) w_s=**
   29.88 %)
4. A clay sample with a specific gravity of 2.68 has void ratio of 0.50 in the dry condition. Determine the shrinkage limit of this clay? **(w_s=18.65%)**
5. The liquid limit and plastic limit of a clayey soil sample are 60% and 35%, respectively. From a particle size distribution curve, it was observed that the sample consists of 60% of particles smaller than 0.002 mm. What is the activity of soil? **(A=0.41)**

Short and Long Answer Type Questions

10) Explain Atterberg limits for soil and their necessity?
10) Define the terms specific gravity and density index.
10) Differentiate between sieve analysis and sedimentation analysis.
10) What is the function of plasticity chart in soil classification?
10) What is the significance of effective size of particle in sieve analysis.
10) Explain in detail the procedure for determination of grain size distribution of a soil sample containing both coarse and fine particles.
10) Write down the procedure for determining specific gravity of a given soil in the laboratory by using a pycnometer.
10) What is meant by the term “index properties of soils”? What is their importance?
10) What do you understand by the consistency of soil? How is it determined?
10) What do you understand by the three-phase system of soil? Explain with a neat sketch.

UNIT-III

Permeability and Seepage

1.1. Permeability of soil

Permeability is an important engineering property of soils. It is defined as the property of a soil which allows the flow of water (or any other fluid) through its interconnecting pores. Determination of permeability is critical in solving number of engineering problems, settlements in foundations, yield of wells, and seepage through and below the earthen structures. The hydraulic stability of a soil mass is controlled by its permeability.

1.1.1. Darcy's law of permeability

Among different soil types, large sized particles such as gravels are highly permeable while fine particles like clays are least permeable. When the flow of water through soils laminar, it is governed by Darcy's Law (1956). According to Darcy's law, velocity of laminar and continuous flow in a homogeneous saturated soil, the is proportional to the hydraulic gradient, and is given by:

$$v = ki$$

where, v - Velocity of flow,

k - Coefficient of permeability

i - hydraulic gradient, given by $i = \frac{h}{L}$

L - length of the sample and

h - head causing flow.

The law is formulated based on the following assumptions:

- The soil is fully saturated
- The flow is laminar
- The flow is steady and continuous
- The total cross-sectional area of soil is considered

The volume of flow per unit time or discharge (q) is obtained by multiplying v by the total cross- sectional area of both solids and voids (A).

$$q = vA = kiA$$

1.1.2. Coefficient of permeability

Coefficient of permeability is defined as the average velocity of flow through the total cross- sectional area of soil under unit hydraulic gradient. The coefficient of permeability (k) has the dimensions of velocity, and is given by the equation:

$$k = C\left(\frac{\gamma_w}{\mu}\right)\left(\frac{e^3}{1+e}\right)D^2$$

where C is a constant depending upon the shape of conduit, γ_w is the unit weight of water, μ is the coefficient of viscosity, e is the void ratio, and D is the diameter of the hypothetical spherical grains assumed. The coefficient of permeability depends upon the following factors:

- Particle size: As evident from Eq. (3.3), coefficient of permeability is proportional to the square of the diameter of particle. Coarse particles are highly permeable, while fine particles are less permeable.
- Shape of particles: The particle shape decides the specific surface. If the void ratio remains the same, angular particles are less permeable than rounded particles.
- Structure of soil mass: The coefficient C considers the shape of flow conduit, which depends on the structure of soil mass. Flocculated soil structure is more permeable than dispersed structure, at the same void ratio.
- Properties of the pore fluid (water): As mentioned in Eq. (3.3), coefficient of permeability is directly proportional to the unit weight of water and is inversely proportional to its viscosity coefficient. When temperature increases, the permeability increases due to decrease in viscosity.
- Adsorbed water: Adsorbed water is usually observed in fine grained soils, and they are immobile under the influence of gravity. This water causes obstruction to the movement of fluid through pores, and thus decreases permeability.
- Void ratio: From Eq. (3.3), it can be understood that the coefficient of permeability of a soil sample is proportional to $\frac{e}{1+e}$. However, the permeability of a soil at a given void ratio does not have any relationship with that of other soils with the same ratio. Even though clays have the maximum void ratio, they are the least permeable, due to the small size of void passage.
- Degree of saturation: When the soil is not fully saturated, the voids are partially occupied by air. The entrapped air blocks the passage of water and reduces permeability.
- Impurities in water: Any impure particle in water has the tendency to block the conduit of flow, and thus reducing the permeability.

1.1.3. Determination of coefficient of permeability

Coefficient of permeability can be determined using both laboratory and field tests. In the laboratory, different methods are used for the determination of coefficient of permeability of fine grained and coarse-grained soils.

1.1.3.1. Constant head permeability test

The coefficient of permeability of a relatively more permeable soil (coarse grained) can be determined in laboratory by the constant head permeability test where a reasonable discharge can be collected in time interval (Fig. 3. 1). The coefficient of permeability (k) is computed using the equation.

$$k = \frac{Q}{t} . \frac{L}{h} . \frac{1}{A}$$

where, Q - the quantity of flow in a time interval t

L - length of the sample,

h - head causing flow,

A - Total cross-sectional area of the sample

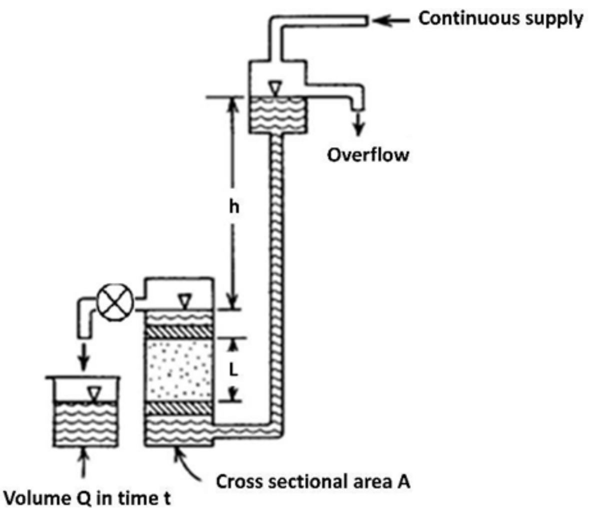


Fig. 3. 1 Constant permeability test

1.1.3.2. Falling head permeability test or variable head permeability test

For relatively less permeable soils (fine grained), the quantity of water collected in the graduated jar of the constant-head permeability test is very small and cannot be measured accurately (Fig. 3. 2). In case of such soils, the variable head permeability test is used. The value of k is computed using the equation,

$$\text{Permeability Falling Head (K)} = 2.303 \times \frac{a(\mathrm{L})}{A(tf - ti)} \times \log_{10}\left(\frac{h1}{h2}\right)$$

where, a - Cross-sectional area of the standpipe,

L – Length of the sample,

A -Total cross-sectional area of the sample,

h_1 - Head at time t_1,

h_2 - Head at time t_2,

$t = t_2 - t_1$, the time interval during which the head reduces from h_1 to h_2.

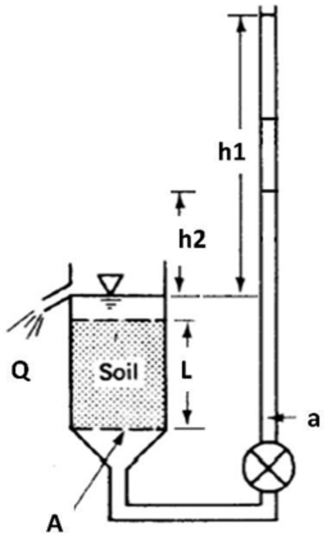


Fig. 3. 2 Falling head permeability test

Preparing soil samples in laboratory with the same particle arrangement and density is very difficult and hence the results of field permeability tests are more suitable for practical applications. Field tests given in-situ values of permeability with minimum disturbance.

1.1.3.3. Field Tests

The field tests may be in the form of pumping out-tests or pumping- in tests wherein the water is pumped out or pumped into the drilled wells. The pumping-in tests give the value of the coefficient of permeability of stratum close to the hole and is economical whereas the pumping- out tests give the value for a large area around the hole and gives more reliable values.

1.1.4. Permeability of stratified deposits

Soil deposits in field are generally stratified. Their bedding planes may be horizontal, inclined or vertical. In such cases, each layer may be assumed to be homogeneous and isotropic and with a separate value of coefficient of permeability. The average permeability of the whole deposit can be calculated based on the direction of the bedding planes, but also depends upon the direction of flow of water through the soil mass.

1.1.4.1. Average permeability parallel to bedding planes

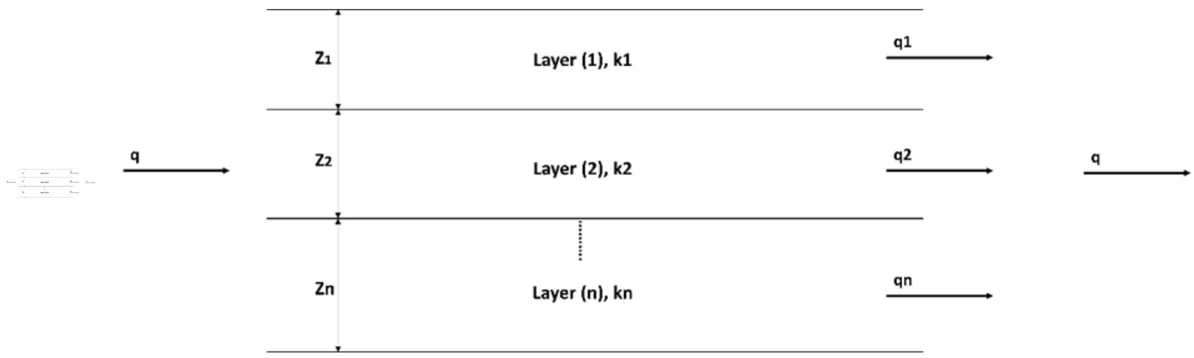


Fig. 3. 3 Flow parallel to bedding planes

$$kX = \frac{k_1Z_1 + k_2Z_2 + k_3Z_3 + \ldots + k_nZ_n}{Z_1 + Z_2 + Z_3 \ldots + Z_n}$$

where, k_X = Average permeability of the soil deposit parallel to the Bedding planes.

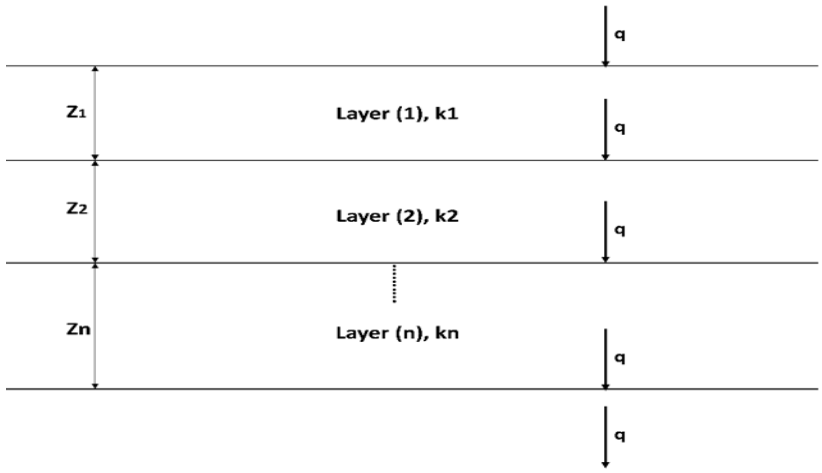
$Z_1 + Z_2 + Z_3 + \cdots + Z_n$ - Thickness of individual layers

$k_1, k_2, k_3, \ldots, k_n$ - Coefficient of permeability of the individual layers,

$q_1, q_2, q_3, \ldots, q_n$ - Discharge through the individual layers.

1.1.4.2. Average permeability perpendicular to bedding planes

Fig. 3. 4 Flow perpendicular to bedding planes



where, k_y - Average permeability of the soil deposit perpendicular to the bedding planes,

$Z_1 + Z_2 + Z_3 + \cdots + Z_n$ - Thickness of individual layers,

$i_1, i_2, i_3, \ldots, i_n$ - Hydraulic gradient for each stratum,

$k_1, k_2, k_3, \ldots, k_n$ - Coefficient of permeabilities of the individual layers.

1.1.5. Seepage analysis

Seepage is the flow of water under gravitational forces through a permeable medium. The flow is generally laminar and takes place from a point of high head to a point of low head. During this process of seepage, the path that is followed by a water particle is marked as a flow line. The lines connect points of equal head on various flow lines are known as equipotential lines. Both these lines intersect each other at right angles, and a small area covered by a pair of flow lines and equipotential lines is called a field.

The flow lines and equipotential lines together form a flow net (Fig. 3. 5) that provides a pictorial representation of the path taken by water particles and the pressure variation along the flow path.

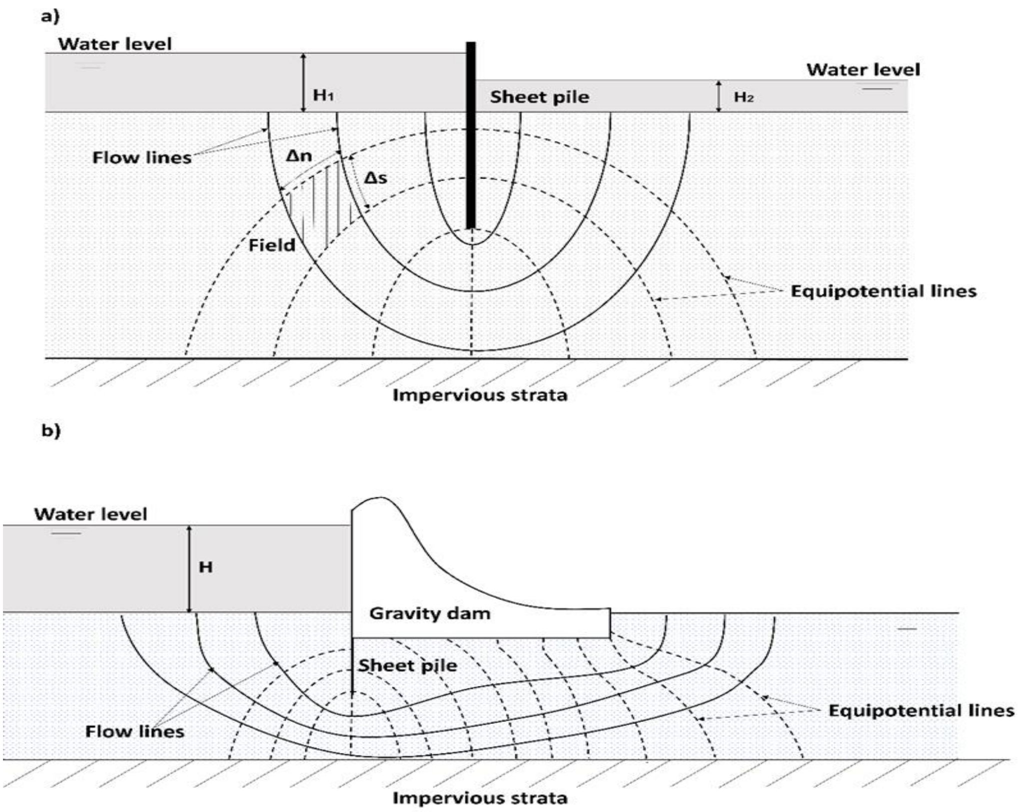


Fig. 3. 5 Flow nets. a) below a sheet pile, and b) below a gravity dam Flow nets are constructed with the following assumptions, using Laplace equation:

- The flow is two dimensional
- Water and soil are incompressible
- Soil is isotropic and homogeneous
- The soil is fully saturated
- The flow is steady
- Darcy's law is valid.

1.a.i.1. Properties of flow net

The properties of flow net can be summarized as under:

- Every intersection between a flow line and an equipotential line should be at right angles.

- The discharge (Δq) between any two adjacent flow lines is constant.
- The drop of head (Δh) between the two adjacent equipotential lines is constant.
- The ratio of the length and width of each field ($\Delta s/\Delta n$) is constant.
- Smaller the dimension of the field, greater will be the hydraulic gradient and velocity of flow through it.
- In a homogeneous soil, every transition in the shape of the curves is smooth, being either elliptical or parabolic in shape.

1.a.i.2. Applications or uses of flow net

A flow net can be used for the following purposes:

- Determination of seepage flow (Discharge)

$$q = k.h.\frac{N_f}{N_d}$$

where, q - Discharge through a flow net,
k - Coefficient of permeability,
h - Total hydraulic head causing flow,
N_f - Number of flow channels, and
N_d - Number of equipotential drops.

The above expression is valid for isotropic soils per unit length of structure perpendicular to the plane of section and should be multiplied by the length of structure to get the total discharge.

- Determination of hydraulic gradient

The average value of hydraulic gradient for any flow field is given by

$$i = \frac{\Delta h}{\Delta s}$$

where, Δs - the length of flow field.

At the exit, the length Δs is minimum and the hydraulic gradient is generally maximum, and velocity is also maximum.

- Determination of total head

The total head (h) at any point (P) is given by

$$h_p = h - nx\frac{h}{N_d}$$

where, n - the number of equipotential drops upto point P.

Hence, the seepage pressure at any point (P) is obtained by simply multiplying total head at P (h_p) with the unit weight of water (γ_w) and this pressure acts in the direction of flow.

- Determination of pressure head

The pressure head at any point is equal to the total head minus the elevation head. The downstream water level is generally (usually) taken as datum.

1.1.6. Seepage pressure

As the water flows through a soil, it exerts a force on the soil. The force acts in the direction of Bow in the case of isotropic soils. The force is known as the drag force or seepage force. The pressure induced in the soil is termed seepage pressure.

The seepage force (J) is given by

$$J = \gamma_w h A$$

The seepage force per unit volume (j) is expressed as

$$j = \gamma_w h A = h\,\gamma$$

$$j = \frac{\gamma_w h A}{AL} = \frac{h}{L}\gamma_w$$

The seepage pressure (p_s) is the seepage force per unit area.

$$p_s = \frac{J}{A} = \frac{\gamma_w h A}{A}$$

The seepage pressure (p_s) can be expressed in terms of the hydraulic gradient.

$$p_s = \gamma_w h = \gamma_w \frac{h}{L} L$$

$$p_s = i\gamma_w L$$

L - Length of the sample,

A - Cross-sectional area of the sample,

h - Hydraulic head,
γ_w – Unit weight of water.

1.1.7. Concept of effective stress

In a loaded soil mass which is below water, there are two types of stresses that act within soil mass: effective stress, and neutral stress or pore water pressure.

1.1.7.1. *Effective Stress*

It is also known as inter-granular pressure. It is transferred to soil grains through their point of contact of the interconnected particles of a soil and is represented by $\bar{\sigma}$ or σ'.

1.1.7.2. *Pore water pressure*

It is transmitted to the soil base through the pore water and is represented by u.

The effective stress cannot be measured directly in the laboratory. It is deduced from total stress and pore water pressure.

$$\bar{\sigma} \; or \; \sigma' = \sigma - u \qquad (3.16)$$

where, $\bar{\sigma} \; or \; \sigma'$ - effective stress,

σ - total stress,

u - pore water pressure.

In fully saturated condition, the vertical downward effective stress acting on soil of Z depth is $\gamma_{sub}Z$. When the upward seepage pressure becomes equal to this value, cohesionless soil losses its shear strength. This condition in saturated cohesionless soil is called quicksand condition or boiling condition. The critical gradient at quicksand condition is given by:

$$i_c = \frac{\gamma_{sub}}{\gamma_w} = \frac{G-1}{1+e}$$

where γ_{sub} is the submerged unit weight of soil, γ_w is the unit weight of water, G is the specific gravity of soil, and e is the void ratio of soil.

1.1.8. Seepage velocity

The velocity of water through the pore spaces in fluid is called seepage velocity (V_s). This velocity is always greater than or equal to the discharge velocity, due to the reduction in cross sectional area. It can be calculated as follows:

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

where V is the discharge velocity and n is the porosity of soil.

1.1.9. Seepage through earthen structures

When an earthen dam is constructed, water seeps through the dam body. For a homogenous earthen dam with an impervious foundation, the impermeable boundary is a flow line which forms the lower boundary of the flow net. The topmost flow line is known as phreatic line (Fig. 3. 6) or seepage line, and the pressure acting on this line is equivalent to atmospheric pressure. The soil is unsaturated above the phreatic line and saturated below the phreatic line. As the pressure head is zero on the phreatic line, the total head is equal to the elevation head.

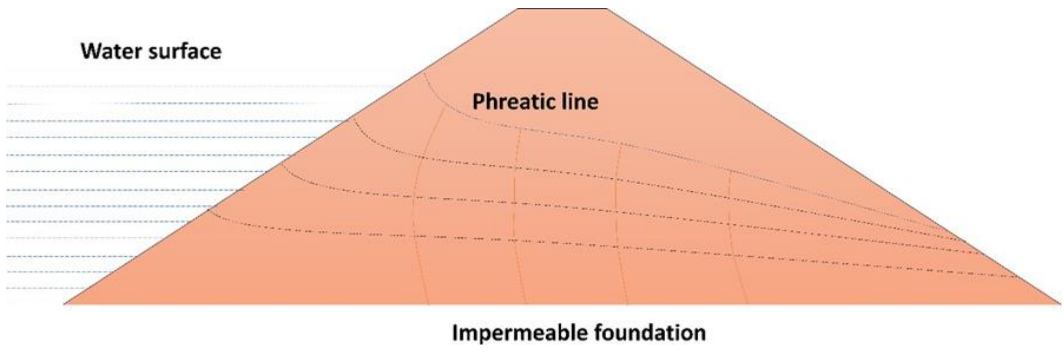


Fig. 3. 6 Phreatic line through an earthen dam

Once the phreatic line has been located, the flow net can be drawn, and the discharge can be computed using the equation

$$q = k.h.\frac{N_f}{N_d}$$

It is also computed using the equation, $q = ks$

where, s is the focal distance given by:

$s = \sqrt{d^2 + h^2} - d$

where $d = bottom\ width\ of\ the\ dam - length\ of\ the\ filter - 0.7\ b$

where $b = n\ h$

Upstream slope is given by 1: *n (V:H)* and h is the water depth.

If a provision for drainage is provided at the downstream side, the seepage path and thus the phreatic line of the dam will get modified as shown in Fig. 3.7.

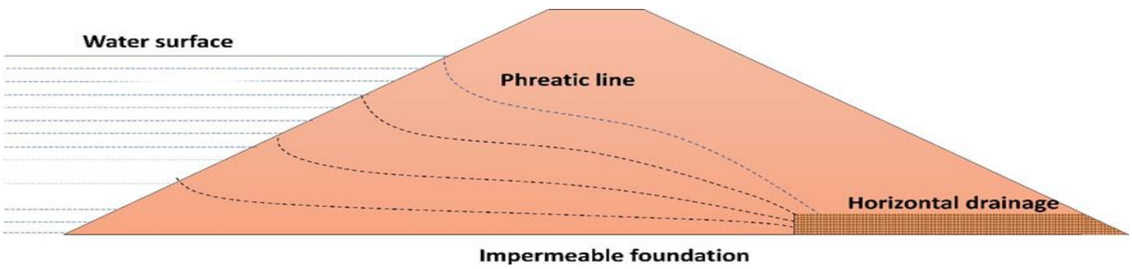


Fig. 3. 7 Phreatic line through an earthen dam with horizontal filte

1.1.9.1. Piping

Hydraulic structures, such as weirs and dams, built on pervious foundations sometimes fail formation of a pipe-shaped channel in its foundation or body (Fig. 3. 8). This type of failure is called piping failure. It occurs when water flowing through the foundation has a very high exit gradient and it carries with it soil particles. The factor of safety against piping is defined as the ratio of critical hydraulic gradient to the actual exit gradient, and the value should be greater than 4 to avoid piping.

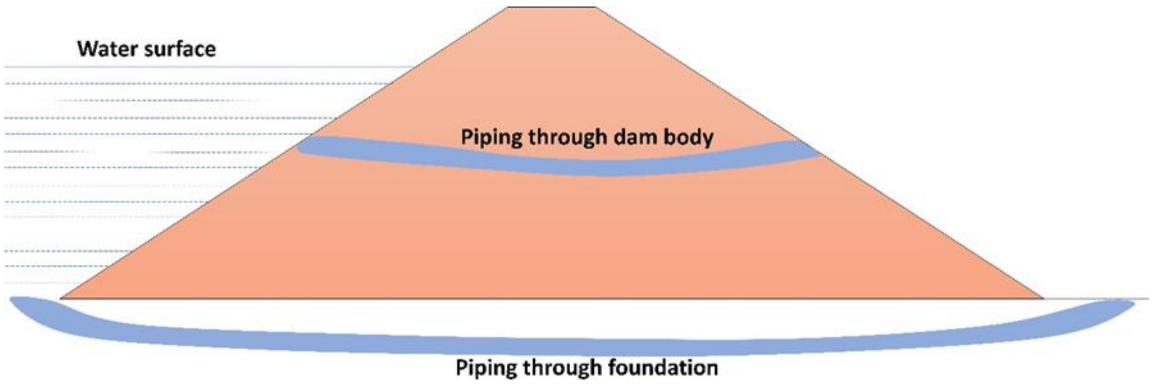


Fig. 3. 8 Piping through dam body

and foundation The following measures are usually adopted to prevent piping failures.

- Increasing the path of percolation

The length of the path of percolation can be increased by increasing the base width of the hydraulic structure, by providing vertical cut off walls below the hydraulic structure or by providing an upstream impervious blanket, as shown in Fig. 3. 9

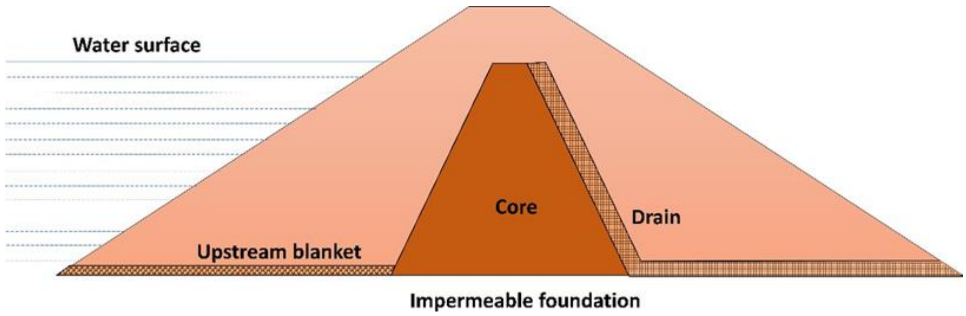


Fig. 3. 9 Measures to control piping

- Impervious core

The quantity of seepage is reduced by providing an impervious core (Fig. 3. 9).

- Providing Drainage Filter or loaded filter

The drainage filter may be horizontal or in the form of a rock toe. It may also be in the form of a chimney drain, as shown in Fig. 3. 9. A loaded filter consists of coarse-grained particles such as sands and gravels. It is provided in order to increase the downward seepage force without a rise in the upward seepage force.

Flow net

A Flow net is a graphical representation of flow of water through a soil mass. It is a curvilinear net formed by the combination of flow lines and equipotential lines. Properties and application of flow net are explained in this article.

Flow lines

Flow lines represent the path of flow along which the water will seep through the soil. Equipotential lines are formed by connecting the points of equal total head.

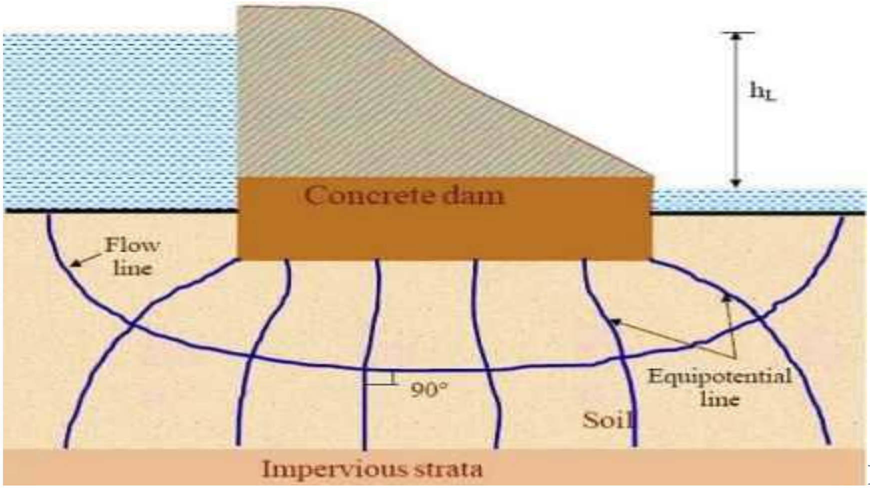


Fig 1: Flow Net

Properties of Flow Net

Properties of flow net are as follows:

- The angle of intersection between each flow line and an equipotential line must be 90^o which means they should be orthogonal to each other.
- Two flow lines or two equipotential lines can never cross each other.
- Equal quantity of seepage occurs in each flow channel. A flow channel is a space between two flow lines.
- Head loss is the same between two adjacent potential lines.
- Flow nets are drawn based on the boundary conditions only. They are independent of the permeability of soil and the head causing flow.
- The space formed between two flow lines and two equipotential lines is called a flow field. It should be in a square form.
- Either flow lines or equipotential lines are smoothly drawn curves.

Applications of Flow Net

Flow net is useful to determine the following parameters in seepage analysis of soil :

1. Rate of Seepage loss
2. Seepage Pressure
3. Uplift Pressure
4. Exit Gradient

1. Rate of Seepage loss (Q)

Using flow net, the rate of seepage loss or seepage quantity can be determined using the below expression :

$$Q = k.H.\frac{N_f}{N_d}.\frac{\Delta B}{\Delta L}$$

Where,

k = coefficient of permeability

H = Head causing flow

N_f = Number of flow lines

N_d = Number of Equipotential lines

ΔB = Width of the flow filed

ΔL = Length of the flow field

2. Seepage Pressure (P_s)

Seepage pressure at any point is determined by using the below mentioned formula :

$$P_s = \gamma_w.h$$

Where,

γ_w = Unit weight of water

h = Hydraulic potential after “n” potential drops. It can be expressed as :

$$h = H - n.\Delta H$$

$$\Delta H = \frac{H}{N_d}$$

Where,

ΔH = Potential drop or Drop in head between two adjacent equipotential lines.

3. Uplift Pressure (P_u)

The uplift pressure at any point within the soil mass can be found using the undermentioned formula. It is also called as hydrostatic pressure.

$$P_u = \gamma_w . h_w$$

Where,

γ_w = Unit weight of water

h_w = Piezometric head or pressure head = total head – elevation head

$$h_w = h \pm z$$

4. Exit Gradient (i_{exit})

The exit gradient is the hydraulic gradient at the downstream end of flow line where seepage water from the soil mass joins with free water at the downstream. Exit gradient can be expressed as :

$$i_{exit} = \frac{\Delta H}{\Delta L}$$

Where,

ΔL = Length of the flow field

ΔH = Potential drop or Drop in head between two adjacent equipotential lines.

Quicksand Condition

Quicksand condition is the floatation of particles of cohesionless soil, like fine gravel and sand, due to vertical upward seepage flow. As sand boiling occurs, the bearing capacity and shear strength of the cohesionless soil decrease and the agitations of soil particles become apparent.

Quicksand condition is not a type of soil but a flow condition that occurs in cohesionless soils. Practically, boiling condition may occur when excavations are made below the water table and water is pumped out from the excavation pit to keep the area free from water.

How Quicksand Condition Occurs?

Quicksand condition occurs when seepage pressure, which acts in the upward direction, overcomes the downward direction pressure due to submerged weight of soil, and the sand grains are forced apart. The result is that the soil has no capability to support a load.

The soil that experiences quicksand condition would lose shear strength and bearing capacity. The shear strength of cohesionless soil depends on the effective stress. The shear strength is given by:

$$shear\ strength\ (s) = \sigma' \tan \emptyset \qquad \textit{Equation 1}$$

Where:

σ': effective stress.

$\emptyset$: angle of shearing resistance.

The effective stress is given by the following expression:

$$\sigma' = \sigma - u \qquad \textit{Equation 2}$$

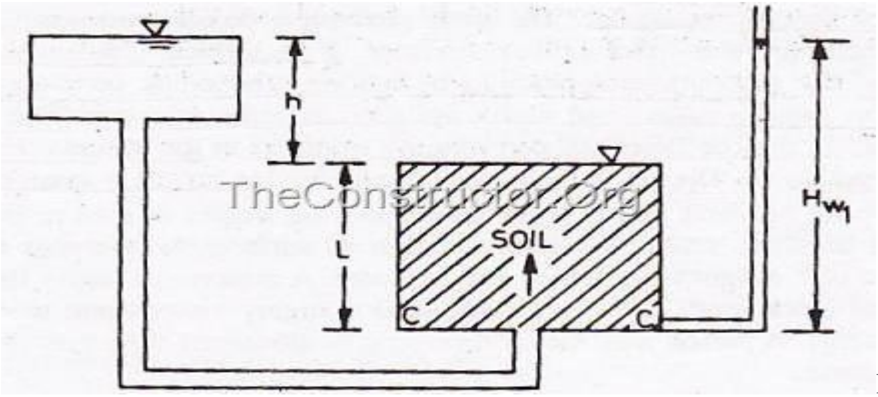


Fig. 1: Quick Sand Condition

UNIT-IV

Compaction, Consolidation and stabilization of soil

5.1 COMPACTION

What is Compaction?

Compaction is the process of making soil denser by pressing the soil particles closer together using mechanical equipment like rollers, rammers, or vibrators.

- It reduces the amount of **air (voids)** in the soil.
- It increases the **strength**, **stability**, and **load-bearing capacity** of the soil.
- It reduces **compressibility** and **permeability** (how easily water can pass through).

Proctor's Discovery (1933)

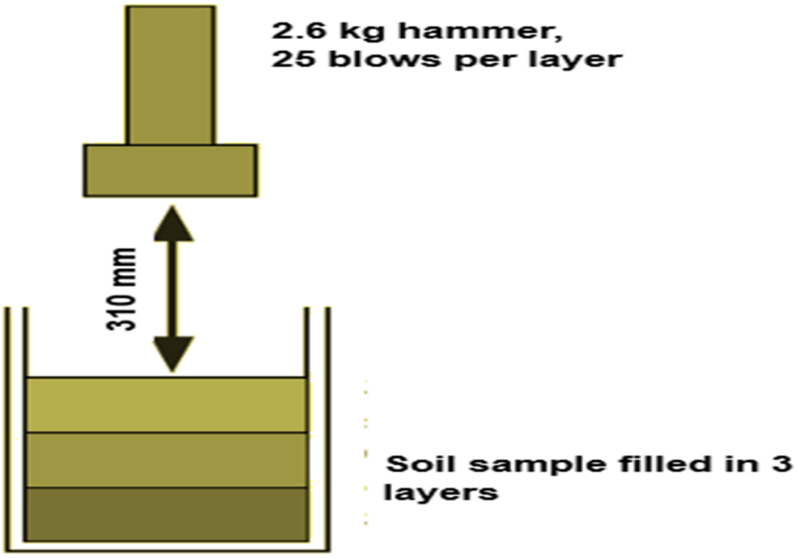
- Engineer R.R. **Proctor** discovered that **soil compacts best at a certain amount of water**.
- If we add water and compact the soil in layers, we can find the **maximum dry density (MDD)** and the **optimum moisture content (OMC)**.
- These are determined in the lab using **compaction tests**.

5.1.1 LIGHT COMPACTION (Standard Proctor Test)

This test is done as per **IS 2720 (Part VII)**.

Procedure:

- A cylindrical **metal mould** is used with a base plate and collar.
- Soil is compacted in **3 layers**, each layer gets **25 blows** using a **2.6 kg rammer** dropped from **310 mm** height.
- After compacting, excess soil is trimmed and a sample is taken to find the **water content**.



Formulas used:

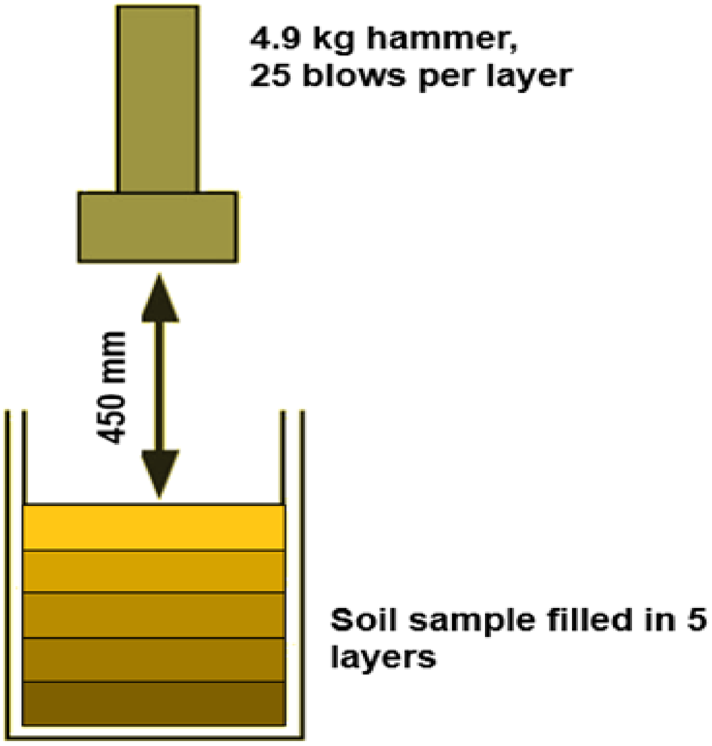
- **Bulk Density (ρ)** = Mass of soil / Volume of mould
- **Dry Density (ρd)** = Bulk density / (1 + w)
  Where **w** = water content

5.1.2 HEAVY COMPACTION (Modified Proctor Test)

This test is used for areas that need higher compaction, like **airfields and highways**. Described in **IS 2720 (Part VIII)**.

Procedure:

- Same mould is used, but soil is compacted in **5 layers**.
- Each layer gets **25 blows** from a **4.9 kg rammer** dropped from **450 mm** height.
- This test gives **higher density** and needs **less water** compared to light compaction.



5.1.3 COMPACTION CURVE

- Soil is compacted at **different water contents**.
- Dry density is calculated for each trial.
- A graph is plotted between:
    - **Water Content (X-axis)**
    - **Dry Density (Y-axis)**
- This is called the **compaction curve**.

Important Points:

- **MDD**: Maximum Dry Density
- **OMC**: Optimum Moisture Content – the water content at which MDD is achieved.

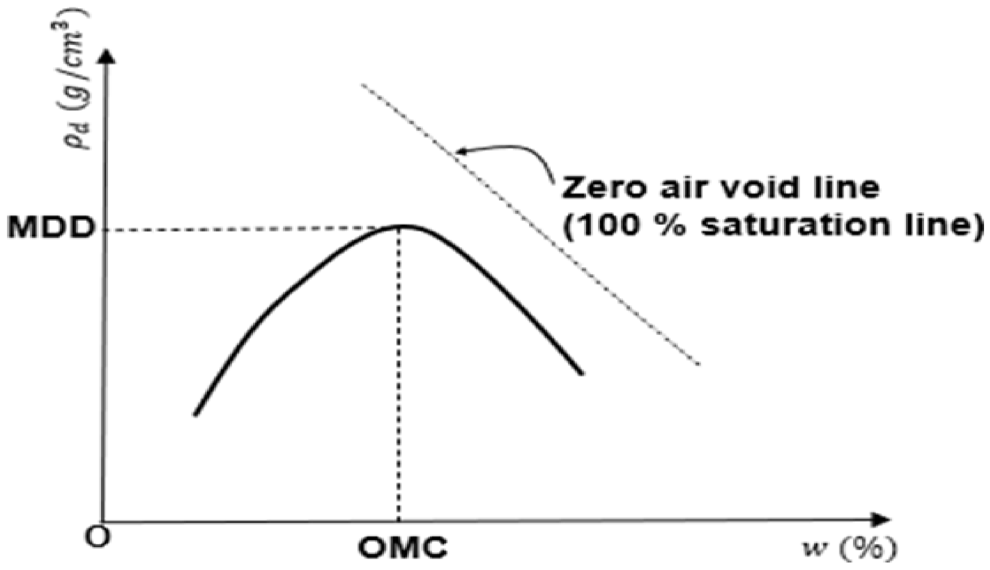


Fig. 5. 3 Compaction curve

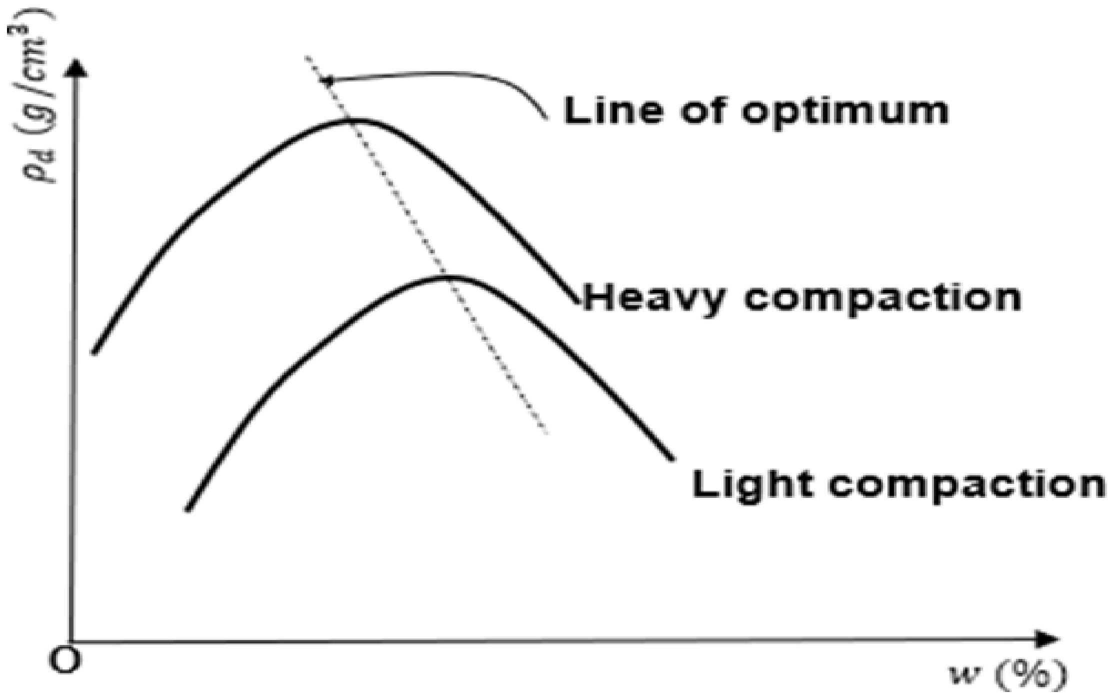
5.1.3.1 Zero Air Void Line

This is a **theoretical line** on the compaction graph. It shows the **maximum dry density possible if there is no air** in the soil (100% saturation).
Formula:

$$\rho_d = \frac{(1 - n_a)G\rho_w}{1 + wG}$$

Where:

- G = Specific gravity of soil
- ρw = Density of water
- w = Water content



Dry Side vs Wet Side of OMC

Property	Dry of OMC	Wet of OMC
Soil structure	Flocculent (clumpy)	Dispersed (spread)
Permeability	More	Less
Strength	More	Less
Swelling	More	Less
Shrinkage	Less	More
Compressibility	Low	High

- **Dry Side** is stronger and less compressible
- **Wet Side** is softer and more water-retaining

5.1.3.2 Compaction Curve for Sandy Soil

- In **sandy soils**, water does not help compaction much.
- A small amount of water can cause **bulking** (soil becomes fluffy).
- Best compaction is achieved when the sand is **fully saturated**.
- Sandy soils don't show a clear **OMC** like clay soils.

5.1.4 Factors Affecting Compaction

1. **Water Content**
   a. Dry soil doesn't compact well
   b. As water is added, soil becomes more flexible and compacts better
   c. After a certain point (OMC), more water reduces density
2. **Amount of Compaction (Energy)**
   a. More energy → Higher dry density
   b. OMC decreases slightly with more effort
3. **Method of Compaction**
   a. Rolling, kneading, vibrating or static pressure
   b. Each method gives different results depending on soil type
4. **Type of Soil**
   a. **Gravel and sand** compact more easily
   b. **Clay** needs more water and energy
5. **Use of Admixtures**
   a. Lime, cement, fly ash can improve compaction properties

Suitability of various compaction equipment

Compaction can be done at both shallow and deep depth. The compaction at shallow depths or surface compaction is widely carried out as a part of ground improvement and for pavements. Surface compaction can be carried out by three methods, rolling, ramming and vibration.

Roller

The compaction achieved by rollers depend upon their contact pressure, number of passes, layer thickness and speed of roller. While the compaction increases with contact pressure and number of passes, it decreases with an increase in layer thickness. The speed of roller should be optimized for the application required. Different types of rollers (Fig. 5. 6) are used for compaction as listed below:

- Smooth wheel roller
  - Compaction is achieved by application of pressure over the soil.
  - Suitable for coarse grained soil like gravel, crushed stone and sand etc.
  - Generally used in construction of road.
- Sheep foot roller:
  - Compaction is carried out by kneading action which provide comparatively strong bond between compacted layers of soil.
  - Suitable for cohesive soil.
  - Used in construction of earthen dam.
- Pneumatic tyred roller
  - Compaction is carried out by the combined action of pressure and needing.
  - Suitable for all types of soil but generally preferred for cohesive soil.
  - Used in construction of roadway, airfield and homogeneous dams.

Fig. 5. 6 Different types of rollers

Rammer

Rammers or tampers are used to compact the soil under the effect of impact. Rammers are generally preferred for cohesive soil. They are used for compacting soil in confined areas such as near to retaining walls, basement walls etc.

Vibrator

In this approach, vibrations are induced in soil during compaction. This method is best suited for compaction of sand. This method is widely used for compacting soil in confined areas and in construction of embankment of oil storage tanks.

Table 5. 3 Difference between compaction and consolidation

Compaction	Consolidation
It is almost instantaneous process.	It is time dependent process.
Soil is in unsaturated condition.	Soil is completely saturated.
Volume reduction due to expulsion of air from void spaces.	Volume reduction is due to expulsion of pore water.
Specified mechanical techniques are used in the process (like roller, rammer and vibrator)	Consolidation occurs on account of a load placed on soil.

5.2 SOIL STABILIZATION

Soil Stabilization means improving the soil to make it **stronger**, **more stable**, and **suitable for construction**.

Methods of Stabilization

1. Mechanical Stabilization

- Done by mixing different types of soil (sand, clay, etc.)
- Goal: Get a mix with the right **strength** and **binding**
- **Gravel** provides strength; **clay** gives binding and holds water

2. Cement Stabilization

- Adding **Portland cement** to soil makes it harder
- Works well for sandy or gravel soils
- Not all clay particles may bond well with cement

3. Lime Stabilization

- Used for **clayey soils**
- Lime makes clay less plastic and more workable
- Improves strength over time due to chemical reactions

4. Bitumen Stabilization

- Used for making roads waterproof
- Bitumen binds the soil particles and stops water entry
- Types: soil-bitumen, sand-bitumen, oiled earth

5. Chemical Stabilization

- Chemical sprays or binders (like polymers) used to temporarily hold loose soil
- Helps in controlling **dust** and **erosion**

6. Stabilization by Heating

- Heating clay soil to 400–600°C changes its structure
- Makes soil non-plastic and creates **hard soil aggregates**

7. Electrical Stabilization (Electro-osmosis)

- Used in clay soils
- An electric current removes water from the soil, making it stronger

8. Reinforced Earth & Geosynthetics

- **Geosynthetics** (like plastic sheets) are placed in soil to improve strength
- They act like reinforcement in soil
- Help with **drainage**, **filtration**, and **soil retention**

5.3 CALIFORNIA BEARING RATIO (CBR) TEST

What is the CBR Test?

The **CBR test** is a **penetration test** used to find the **strength of soil** used in **road and pavement** construction.
It helps engineers decide how thick the pavement layers should be.

- It is the **most common method** used to design **flexible pavements** (roads that bend slightly under loads).
- The standard procedure is given in **IS 2720 (Part 16) – 1987**.

CBR Value – Definition

The **CBR value** is defined as:

$$\text{CBR (\%)} = \frac{\text{Test Load}}{\text{Standard Load}} \times 100$$

- A **50 mm diameter plunger** is pushed into the soil at **1.25 mm per minute**.
- The **load** required to push the plunger into the soil is compared with the **load required for standard crushed stone**.
- The standard material (crushed stone) is assumed to have **CBR = 100%**.

Standard Load Values

Penetration (mm)	Standard Load (kg)
2.5	1370
5.0	2055

If the CBR value at **5 mm** is **consistently higher** than at 2.5 mm, the **5 mm value is used**.

Test Equipment

The following are used for the CBR test:

- Loading machine (at least **5000 kg** capacity)
- **50 mm diameter plunger**
- Cylindrical **metal mould**
- Extension collar and perforated base plate
- Circular spacer disc
- Weights for surcharge

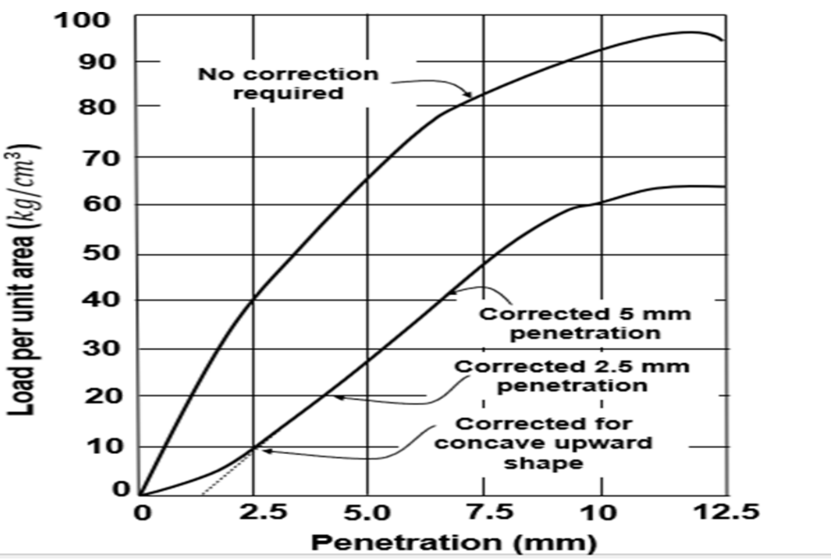


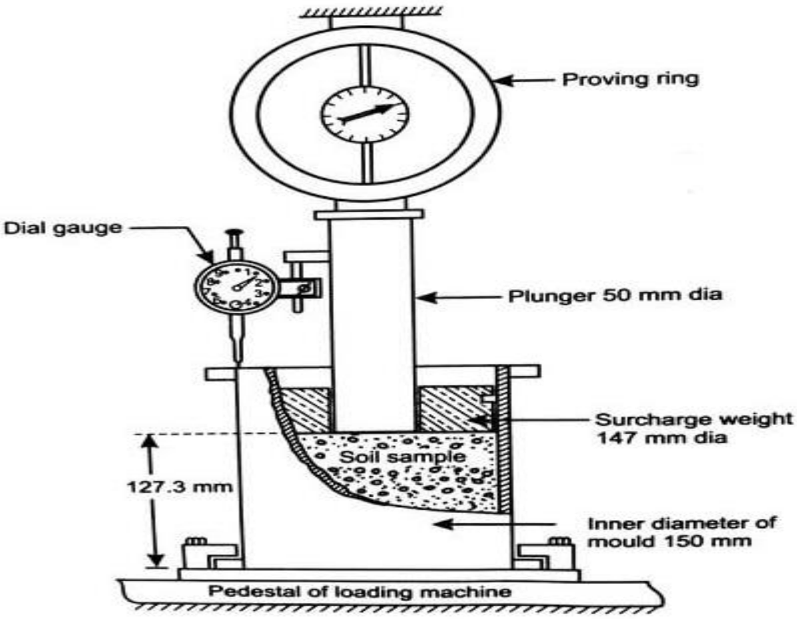
Fig. 5. 7 Load penetration curves for CBR

How the Test is Performed

1. **Soil Sample Preparation:**
   a. Soil is **compacted** in the mould using **heavy compaction** (5 layers, 56 blows per layer).
   b. Compaction is done at **Maximum Dry Density (MDD)** and **Optimum Moisture Content (OMC)**.
   c. Surface is levelled after removing the collar.
2. **Soaking (Optional but common for road tests):**
   a. Soaked samples are kept **immersed in water for 96 hours**.
   b. A **surcharge weight** is placed to simulate pavement pressure.
3. **Penetration Test:**
   a. The sample is placed in the **loading machine**.
   b. The **plunger is pushed** into the soil at a constant rate.
   c. Load readings are taken at the following **penetration depths**: 0.5, 1.0, 1.5, 2.0, **2.5**, 4.0, **5.0**, 7.5, 10, and 12.5 mm.
4. **Graph and Correction:**
   a. A **graph is plotted** between **load** (Y-axis) and **penetration** (X-axis).
   b. If the curve is not smooth (concave near the X-axis), it must be corrected (as per standard).

Use of CBR Values

- The CBR value is used as an **index of soil strength**.
- Helps in designing the **thickness of pavement layers** depending on traffic.
- Design charts and empirical curves are used along with the CBR values for pavement design.



5.4 SITE INVESTIGATIONS

Why Do Site Investigations?

Before starting any construction, it's important to **understand the soil** at the site.

- This is done through **field and lab tests**.
- It helps in choosing the right **foundation**, checking **bearing capacity**, and avoiding **future problems**.

What is Site Investigation?

- It means **studying the surface and subsurface conditions** of the land where construction will happen.
- It includes **soil exploration**, but also involves:
    - **Reconnaissance** (initial inspection)
    - Studying **maps**
    - **Aerial photographs** or satellite images

Useful online sources:

- **Bhuvan** (ISRO)
- **Bhukosh** (Geological Survey of India)
- **USGS Earth Explorer**

5.4.1 Preliminary Steps

These are steps taken before actual soil tests:

- **Reconnaissance** – walking around the site and noting topography, drainage, vegetation, etc.
- Study **topographical maps**, **geological maps**, and **remote sensing images**.
- Helps in planning the soil investigation better.

5.4.2 Soil Exploration

Soil exploration is done to:

1. Know the **type and depth** of soil layers
2. Find the **groundwater level**
3. Collect **soil samples** for lab tests
4. Measure **engineering properties** in the field
5. Assess **bearing capacity** and **settlement**

5.4.2.1 Direct Methods

- **Test Pits / Trial Pits / Trenches**
    - Open excavations made at the site
    - Good for **shallow depths** (up to 3–4 m)
    - Useful for **visual inspection**, sampling, and **field tests** (like plate load test)

5.4.2.2 Semi-Direct Methods – Boring

Boring means making holes in the ground to collect deeper samples. Methods include:

1. **Auger Boring** – for soft soils
2. **Wash Boring** – for loose and sandy soils
3. **Rotary Drilling** – for hard or rocky layers
4. **Shell and Auger Boring** – for deeper holes
5. **Percussion Drilling** – for very hard soils

5.4.2.3 Indirect Methods (Geophysical Tests)

These methods do **not collect samples** but provide **information about soil layers** by using electrical or seismic methods.

- Examples:
    - **Electrical resistivity**
    - **Seismic refraction**
    - **Ground Penetrating Radar (GPR)**
    - **Crosshole and Downhole testing**

These are often used **with other methods** for large projects.

5.4.2.4 Planning the Exploration

Planning includes:

- **Spacing of Borings**: Depends on type and size of structure
- **Depth of Borings**: Should reach the depth where soil is not affected by the building load

Recommended Boring Spacing (Table 5.5):

Project Type	Spacing (m)
Highways	300 – 600
Borrow pits	30 – 120
Earth Dams	30 – 60
Single-storey Factories	30 – 90
Multi-storey Buildings	15 – 30

Recommended Depth of Exploration (Table 5.6):

Foundation Type	Depth of Exploration
Isolated Footings	1.5 × Width of footing
Closely Spaced Footings	1.5 × Length of footing
Pile/Well Foundations	1.5 × width below bearing level
Road Cuts	Equal to depth of cut
Fills	2 m below ground or height of fill, whichever is greater

5.4.3 Field Identification of Soils

In the field, we can quickly **identify soils** using **simple tests**.

Coarse-grained vs Fine-grained Soils

- **Gravel and sand**: Particles can be **seen easily** with the eye
- **Silt and clay**: Look similar, need tests to identify

Simple Field Tests

Test Name	What it Detects	Description
Dispersion Test	Sand vs Silt vs Clay	Put soil in water. Sand settles fast. Silt takes time. Clay stays in water for long.
Dilatancy Test	Silt vs Clay	Shake wet soil in palm. Silt shines. Clay remains dull.
Dry Strength Test	Silt vs Clay	Dry a soil ball. Silt breaks easily. Clay is tough.
Toughness Test	Clay's Plasticity	Roll moist soil. Silt crumbles, Clay forms long thread.

UNIT SUMMARY

- **Compaction** improves soil by reducing air voids and increasing strength.
- **CBR Test** helps to measure soil strength and design pavement layers.
- **Site Investigation** tells us about the surface and subsurface soil conditions.
- **Soil Exploration** includes direct, semi-direct, and indirect methods.
- **Field Identification** uses simple tests to classify soil on-site.

Numerical Examples:

1. What is the theoretical maximum dry density of a soil sample having specific gravity 2.65 and water content 18 %?

2. A cohesive soil gives maximum dry density of 1.8 g/cm^3 at a water content of 17 %, during a proctor test. If the specific gravity of the soil is 2.7, what is the degree of saturation?

Exercises

4) A laboratory compaction test on soil having specific gravity equal to 2.68 gave a maximum dry density of 1.82 g/cc and a water content of 17% per cm. determine degree of saturation,

   air content and percentage air voids at maximum dry density. What would be theoretical maximum dry density corresponding to zero air voids at OMC.

4) The following are the results of a compaction test: Volume of mould = 1000ml. Mass of mould = 1000g Sp. Gravity of mold = 2.70

   Determine degree of saturation at maximum dry density?

Mass of mould + wet soil	2935	3096	3152	3124	3056
Water content	10	12.2	14.3	16.2	18.5

4) Work out theoretical maximum dry density for a soil sample having specific gravity of 2.72 and OMC = 16%. Also explain the difference in OMC value in case of proctor test and modified proctor test for cohesive soil and granular soil.
4) A cohesive soil yields a maximum dry density of 1.85 gm/cc at an OMC of 16% during a standard proctor test. If the value of G is 2.63, what is the degree of saturation?

Short and Long Answer Type Questions

10) Enlist the differences between compaction and consolidation.
10) Is compaction test significant in the case of sandy soils? Justify your answer.
10) What are the different methods used for surface compaction?
10) An earthen dam is proposed near your place. While compacting the core of the dam, what moisture content will you suggest? We of optimum or dry of optimum? Why?
10) What do you mean by optimum moisture content?
10) Define zero air void line.
10) Explain the objectives of soil exploration.
10) What is stabilization of soil? What are the different methods?
10) What do you mean by dry of optimum or wet of optimum?
10) What are the different stages involved in site investigation? Explain in detail.

UNIT-V Shear Strength of Soil

□ 1. What is Shear Strength of Soil?

Shear strength is the **resistance of soil** to shearing (sliding) forces.
It tells us how much **load a soil can bear** before it starts to **fail or deform**.

Shear strength of a soil is its maximum resistance to shear stresses just before the failure. Most engineering applications of soil, such as, stability of slopes, lateral pressure exerted by soil on retaining walls, and bearing capacity of soil requires knowledge of shear strength of soil.

1.1.10. Components of shearing resistance

Shearing resistance is composed of

(ii) Cohesion, and

(ii) Friction,

Strength of cohesionless soils comes mostly from intergranular friction alone and cohesive soils from cohesion alone, while in other soils it comes from both cohesion and internal friction.

Cohesion is the force of attraction between the particles binding them together. Cohesion is present in clays and silts but is normally absent in sands and gravels. Internal friction is due to the inter-locking of particles. All soils except plastic undrained clay exhibit friction.

Types of shear failure of foundation soils

Depending on the stiffness of foundation soil and depth of foundation, the following are the modes of shear failure experienced by the foundation soil.

1. General shear failure (Fig.1(a))
2. Local shear failure (Fig.1(b))
3. Punching shear failure (Fig.1(c))

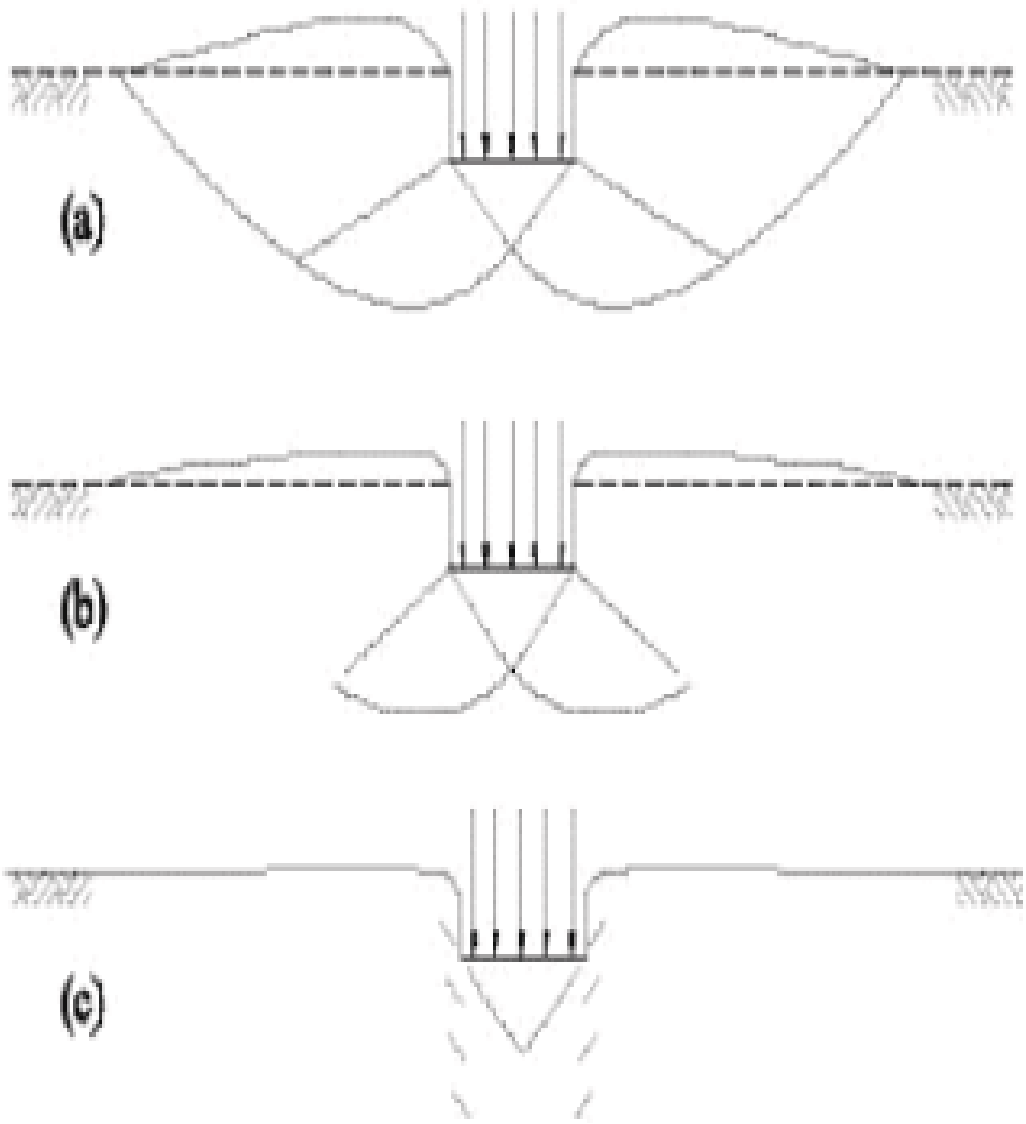


Fig.1: Shear failure in foundation soil

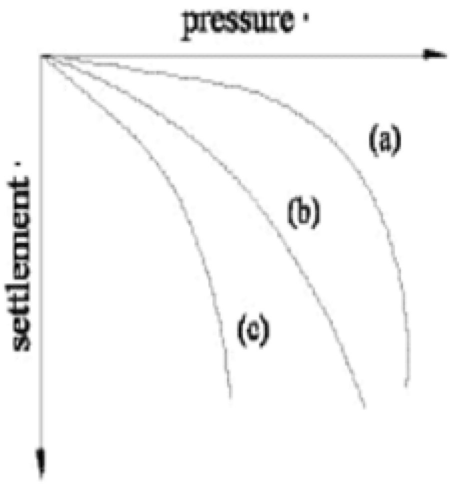


Fig: P – Δ Curve in different foundation soils

General Shear Failure

This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure.

1. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.
2. Dense or stiff soil that undergoes low compressibility experiences this failure.
3. Continuous bulging of shear mass adjacent to footing is visible.
4. Failure is accompanied by tilting of footing.
5. Failure is sudden and catastrophic with pronounced peak in $p-\Delta$ curve.
6. The length of disturbance beyond the edge of footing is large.
7. State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.
8. General shear failure is accompanied by low strain (<5%) in a soil with considerable ϕ (ϕ >36^{o}) and large N (N > 30) having high relative density (I_D> 70%).

Local Shear Failure

This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure.

1. A significant compression of soil below the footing and partial development of plastic equilibrium is observed.
2. Failure is not sudden and there is no tilting of footing.
3. Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.
4. Failure surface is not well defined.
5. Failure is characterized by considerable settlement.\
6. Well defined peak is absent in $p-\Delta$ curve.
7. Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low ϕ (ϕ <28o) and low N (N < 5) having low relative density (I_D> 20%).

Punching Shear Failure of foundation soils

This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure.

1. This type of failure occurs in a soil of very high compressibility.
2. Failure pattern is not observed.
3. Bulging of soil around the footing is absent.
4. Failure is characterized by very large settlement.
5. Continuous settlement with no increase in P is observed in $p-\Delta$ curve.

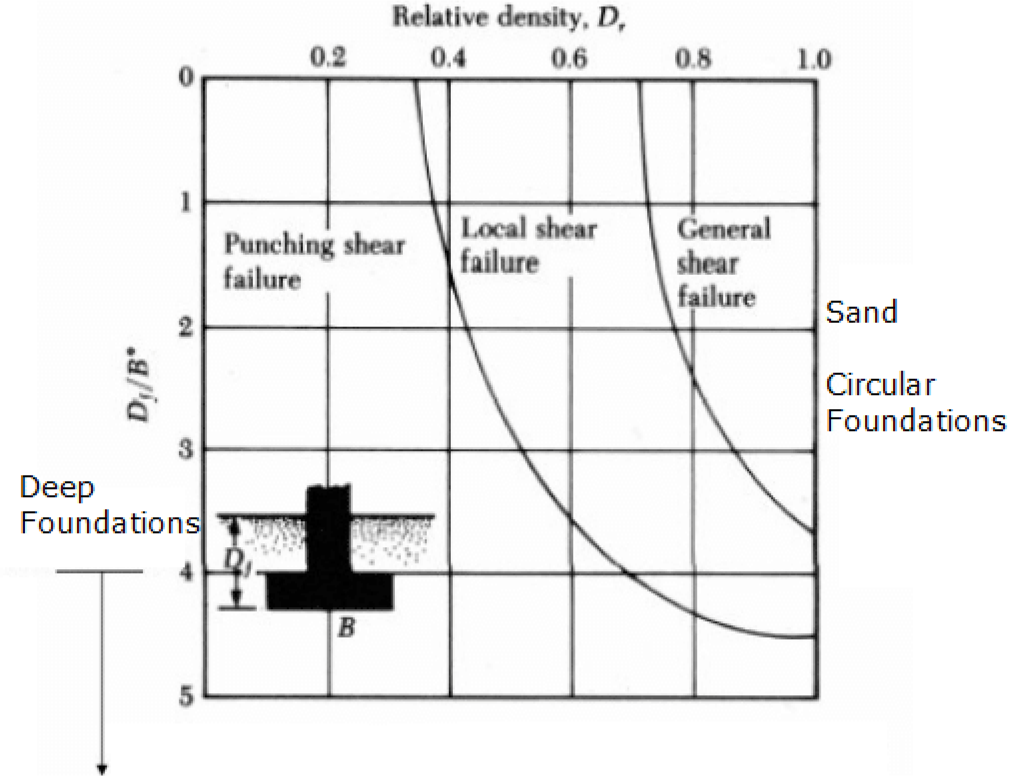


Fig. 2 presents the conditions for different failure modes in sandy soil carrying circular footing based on the contributions from Vesic (1963 & 1973)

1.a.1. Mohr-Coulomb failure theory

Mohr stated that shear failure of any soil happens with a combination of both shear and normal stresses. He also stated that the shear stress on failure plane is a function f normal stress acting along that plane. Later, Coulomb separated the shear strength of soil into two components:

(ii) Cohesion between the particles

(ii) Friction between the particles.

He proposed a straight-line law connecting shear strength and normal stress, as

$$\tau_f = c + \sigma tan\varphi$$

where. τ_f - Shear strength of soil,

σ - Normal stress on soil,

c - Cohesion

φ- Angle of internal friction.

The inclination of the failure envelope to the horizontal gives the angle of shearing resistance and its intercept on the vertical axis is equal to the cohesion. The equation of shear strength can be further simplified based on the type of soil as shown in Fig. 3. 10.

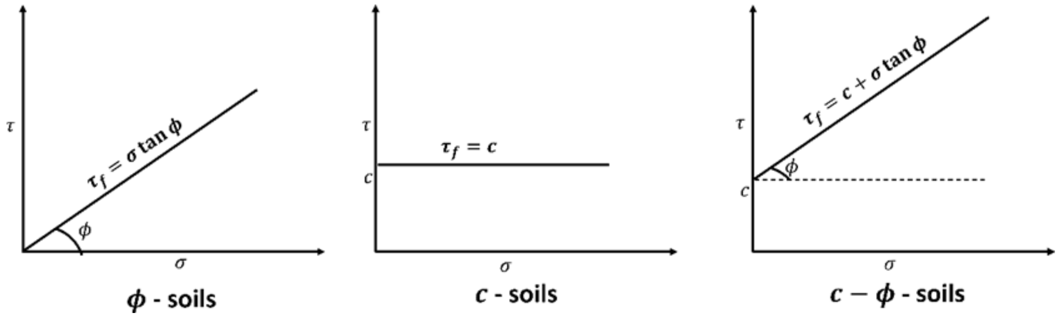


Fig. 3. 10 Failure envelopes for different types of soil

1.a.2. Modified Mohr-Coulomb failure theory

Terzaghi established that the strength of soil is controlled by effective stresses and not by total stresses.

In terms of effective stresses, Eq. (3.19) is written as

$$\tau_f = c' + \bar{\sigma} \tan \varphi'$$

where,

$\bar{\sigma}$ - Effective normal stress $= \sigma - \mu$

σ - total normal stress.

u - Pore water pressure,

c' - cohesion in terms of effective stresses and

and φ' - angle of shearing resistance in terms of effective stresses.

1.a.3. Determination of shear strength

The shear test must be conducted under appropriate drainage conditions that simulate the actual field case.

In shear tests, there are two stages. In the first stage normal stress (or confining pressure) is applied to the specimen. Later, in the second stage shear stress (or deviator stress) is applied to the specimen to shear it.

Depending upon the drainage conditions, there are three types of tests: Unconsolidated- Undrained (UU) tests, Consolidated-Undrained (CU) tests and Consolidated-Drained (CD) tests. The shear force is applied either by increasing the shear displacement at a given rate or by increasing the shear force at a given rate. Accordingly, the shear tests, are called either strain controlled, or stress controlled.

The following tests are used to measure the shear strength of soil.

I. Direct shear test
II. Triaxial test
III. Unconfined compression test
IV. Vane shear test

1.a.i.1. Direct shear test

Direct shear test can be conducted for any one of the three drainage conditions. A number of identical specimens are tested under different normal stresses. The shear stress required to cause failure is determined for each normal stress. The failure envelope is obtained by plotting and joining the points corresponding to shear strength at different normal stresses by a straight line (Fig. 3. 11).

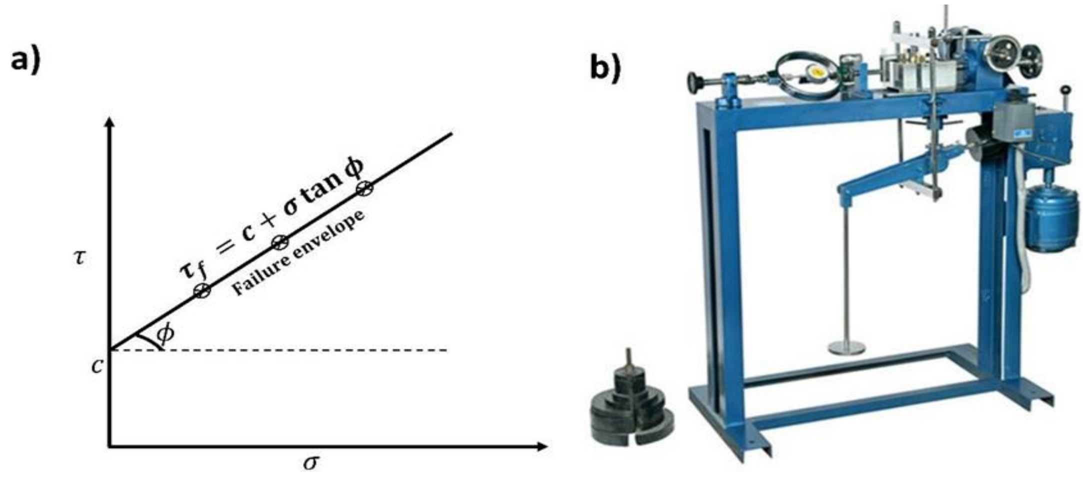


Fig. 3. 11 Direct shear test. a) Failure envelope and b) laboratory test set up

1.a.i.2. Vane shear test

This is a quick test, used either in the field or in the laboratory, to determine the undrained shear strength of cohesive soils. A vane shear test apparatus has four steel plates fixed at right angles to each other to a steel rod (Fig. 3. 12), and the vanes are pushed into soil and rotated at a constant speed (1 rpm). A calibrated torsion spring measures the resistance of soil to rotation, and the shear strength is determined by the following formula if both top and bottom end shear the soil.

Fig. 3. 12 Laboratory vane shear test setup

$$\tau_f = \frac{T}{\Pi D^2[(H/2) + (D/6)]}$$

If only bottom end take part in shearing, then

$$\tau_f = \frac{T}{\Pi D^2[(H/2) + (D/12)]}$$

where T - Maximum torque at failure.

H - Height of vanes,

D - Diameter of rotating blades.

The vane shear test can also be used to determine the sensitivity of the soil. After the initial test, the vane is rotated rapidly through several revolutions. The test is then caried out on the remoulded soil and the shear strength in remoulded state is determined. Thus,

$$Sensitivity\ (S_t) = \frac{(\tau_f)_{(undistributed)}}{(\tau_f)_{(remoulded)}}$$

UNIT SUMMARY

The unit discusses two critical engineering properties of soil, permeability and shear strength. Both are highly relevant in practical applications and should be determined specifically for each site, before starting construction activities. The factors affecting permeability, its laboratory determination and some practical applications are discussed, and for shear strength, the concept of shear strength and direct shear test and vane shear tests for determining shear strength ae discussed.

EXERCISES

Multiple Choice Questions

1. Constant head permeability test is suitable for:
   - d) Granular soils
   - d) Fine-grained soils
   - d) Cohesive soils
   - d) All the above
2. Which among the following is not an assumption while constructing flow nets?
   - d) The flow is two dimensional
   - d) Water and soil are incompressible
   - d) Soil is isotropic and homogeneous
   - d) The soil is partially saturated
3. Effective stress in soil is defined as the difference between total stress and ……………..
   - d) Normal stress
   - d) Shear stress
   - d) Pore water pressure
   - d) None of the above
4. Which among the following is not a widely followed shear test condition, based on drainage?
   - d) Unconsolidated-Undrained (UU) tests,
   - d) Consolidated-Undrained (CU) tests
   - d) Unconsolidated-Drained (UD) tests
   - d) Consolidated-Drained (CD) tests.

Numerical Examples:

1. A 20 cm long, 8 cm diameter coarse sand sample was tested in a constant head permeability test. After 15 mins of constant water flow under 1 m head, the volume of discharged water was found to be 1200 cc. Calculate the coefficient of permeability of soil.

2. The water level in a standpipe of 6 mm diameter reduced from 80 cm to 25 cm in a duration of 15 mins in a variable head permeability test. The soil sample had a height of 12 cm and a cross sectional area of 44.41 cm^2. Find the coefficient of permeability.

3. A silty sand soil has a cohesion of 25 kPa and an angle of internal friction of 25^o. Calculate the
   shear strength of the soil at 5 m below the ground surface, is the unit weight of soil in 18 kN/m^3 and water table is well below the layer of soil.

4. A soil sample with cohesion 20kPa was tested using a direct shear test with an applied normal stress of 200kPa. The sample failed at a shear stress of 150 kPa. Calculate the angle of internal friction of soil.

Exercises

1. A stratified soil deposit has 3 layers of soil. The top layer consists of 2 m deep soil with coefficient of permeability 5×10^{-4} cm/s and the second layer is 5 m thick and more permeable, with coefficient of permeability 2×10^{-2} cm/s. These layers are overlaid on top of a 2 m thick layer of coefficient of permeability 3×10^{-3} cm/s. Calculate the average coefficient of permeability in directions parallel to the bedding and perpendicular to the bedding. **($\boldsymbol{k_h = 1.1 \times 10^{-2} cm/s}$, $\boldsymbol{k_v = 1.9 \times 10^{-3}\ cm/s}$)**
2. The following are the observations from a direct shear test, during failure. Calculate the cohesion and angle of internal friction of the soil sample from the observations. **($\boldsymbol{c = 10\ kPa}$ and $\boldsymbol{\varphi = 30^o}$)**

Sample No.	Normal stress (kPa)	Shear stress (kPa)
1	20	21.55
2	40	33.09
3	60	44.64

3. A soil sample has been taken from a uniform deposit of dry sand and the unit weight was found to be 19 kN/m^3 and angle of internal friction was found to be 35^o. Calculate the shear strength of the soil on a horizontal plane, at 4m depth from the surface. **($\boldsymbol{\tau_f = 53.2\ kPa}$)**
4. A structure is proposed on the site mentioned in question 3, which will induce 60 kN/m^2 stress in the vertical direction and 70 kN/m^2 stress in the horizontal direction at 4 m depth. Will the increase in shear stress exceed the shear strength of soil on horizontal plane? **($\boldsymbol{\tau_f = 95.2\ kPa, hence\ stable}$)**
5. At a depth of 6 m below the ground surface at a site, a vane shear test gave a torque value of 6500 N-cm. The vane was 10 cm high and 6 cm across the blades. What will be the shear strength of the soil (in kN/m^2)? **($\boldsymbol{\tau_f = 95.8\ kN/m^2}$)**

UNIT – VI Bearing Capacity of Soil and Foundation

1.1. Bearing capacity of soil

The ultimate load of any structure is transferred to the soil through the foundations. Based the depth and width of foundations, they are primarily classified into shallow and deep foundations. The term bearing capacity is widely used in associated with shallow foundations, which are laid at a depth less than on equal to their widths. Shallow foundations are of different types; like isolated, strip and combined foundations. In this unit, we are discussing the bearing capacity theories related to strip foundations, that are used to provide a continuous, level strip of support to a linear structure such as a wall. The foundation transmits the load from superstructure to a larger area, and it should be designed in such a way that the settlements are always within the permissible limit, and the soil does not fail in shear. Bearing capacity is a term used to define the pressure that can be taken by the soil safely, without failure. There are different terminologies associated with the definition of bearing capacity.

1.1.1. Basic definitions

1.1.1.1. Ultimate bearing capacity (q_u)

Ultimate bearing capacity denotes the total pressure at the base of foundation at which shear failure occurs in soil.

1.1.1.2. Net ultimate bearing capacity (q_{nu})

It is the net increase in pressure due to the superstructure, at the base of foundation that leads to the shear failure of soil. It is the difference of ultimate bearing capacity and the overburden stress due to the soil above the base of foundation.

$$q_{nu} = q_u - \gamma D_f$$

where γ is the unit weight of soil and Df is the depth of foundation. The product γDf is the overburden pressure, which exists at the level of base of foundation, even before construction.

1.1.1.3. Net safe bearing capacity (q_{ns})

It is the net pressure on soil that can be safely applied to the soil, without causing shear failure. This is calculated by considering a suitable factor of safety (FS), which is usually taken as 3.

$$q_{ns} = \frac{q_{nu}}{FS}$$

1.1.1.4. Gross safe bearing capacity (q_s)

It is the gross pressure that the soil can withstand, without shear failure. The value is obtained by adding the overburden stress to the net safe bearing capacity.

$$q_s = q_{ns} + \gamma D_f$$

1.1.1.5. Net safe settlement pressure (q_{np})

It is the net pressure that can be taken by the soil without exceeding the permissible settlement limits. The maximum allowable settlement depends upon the type of foundation.

1.1.1.6. Net allowable bearing pressure (q_{na})

It is the net bearing pressure used for design of foundations. As the soil should satisfy both shear failure and settlement criteria, the net allowable bearing pressure is the smaller of net safe bearing capacity and the net safe settlement pressure. This value is also known as the allowable soil pressure or allowable bearing capacity.

1.1.2. Terzaghi's bearing capacity theory

The bearing capacity theory proposed by Terzaghi in 1943 deals with strip footings, with the following assumptions:

- The base of footing is rough.
- The footing is laid at a shallow depth (less than or equal to the width).
- Shear strength of the soil above the base of footing is neglected.
- The loading of the footing is vertical and is uniformly distributed.
- Footing is long (L/B ratio is infinite, where L is the length and B is the breadth of the footing).
- The shear strength of soil is governed by Mohr-Coulomb criteria

The ultimate bearing capacity of a strip footing of width B placed at a depth D_f is given by the Terzaghi's equation as follows:

$$q_u = c'N_c + \gamma D_f N_q + 0.5B\gamma N_\gamma$$

where c' is the cohesion of the soil,
γ is the unit weight and N_c, N_q and N_γ are dimensionless numbers, known as Terzaghi's bearing capacity factors. These numbers depend upon the angle of internal friction of the soil, and the values are tabulated in Table 4. 1.

Table 4. 1 Terzaghi’s bearing capacity factors

φ'	General shear failure			Local shear failure		
	N_c	N_q	N_γ	N_c'	N_q'	N_γ'
0	5.7	1.0	0.0	5.7	1.0	0.0
5	7.3	1.6	0.5	6.7	1.4	0.2
10	9.6	2.7	1.2	8.0	1.9	0.5
15	12.9	4.4	2.5	9.7	2.7	0.9
20	17.7	7.4	5.0	11.8	3.9	1.7
25	25.1	12.7	9.7	14.8	5.6	3.2
30	37.2	22.5	19.7	19.0	8.3	5.7
35	57.8	41.4	42.4	25.2	12.6	10.1
40	95.7	81.3	100.4	34.9	20.5	18.8
45	172.3	173.3	297.5	51.2	35.1	37.7
50	347.5	415.1	1153.2	81.3	65.6	87.1

The value of bearing capacity factors also depends upon the type of shear failure. The shear failures are classified into three by Vesic (1973) as general shear failure, local shear failure and punching shear failure.

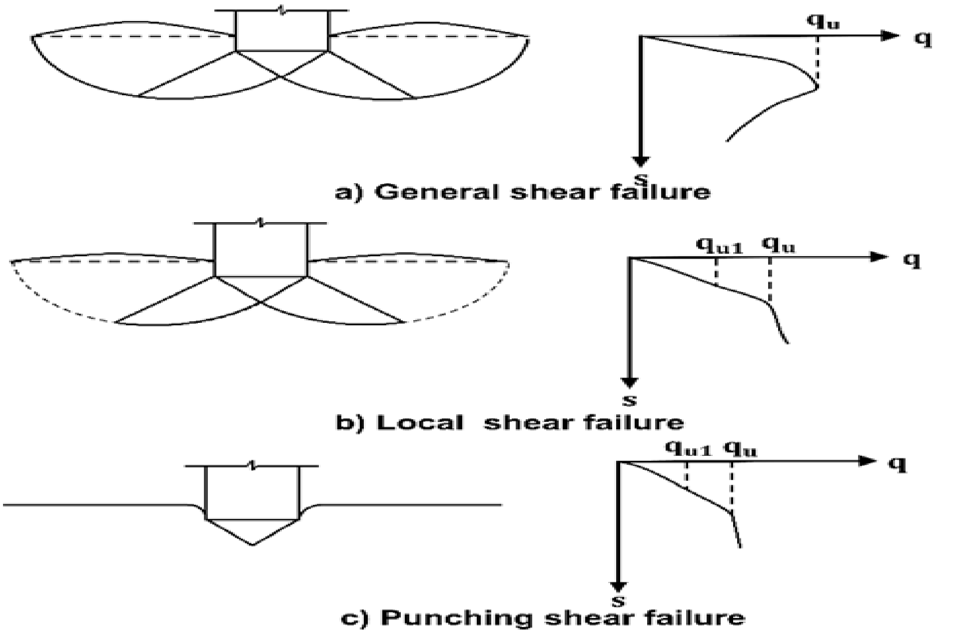


Fig. 4. 1 Types of shear failure

1.a.i.1. General shear failure

During general shear failure, the settlement below the footing increases suddenly at a stress of q_u, and the failure extend to the ground surface (Fig. 4. 1a). The failure always occurs with heave on both sides and occurs commonly in stiff clays and dense sands.

1.a.i.2. Local shear failure

As can be observed in the load per unit area (q) vs settlement (s) curve in Fig. 4. 1b, the movement starts at a load of q_{u1}, with sudden jerks in soil, and it gradually extends away from the foundation. For the failure surfaces to extend till the ground level, considerable movement is required. The load at this point is denoted as q_u, and when the load is exceeded beyond this, there is a substantial increase in settlement. Heaves are observed during local shear failure during large settlements. Local shear failure is commonly observed in medium dense sands and clays with medium consistency.

1.a.i.3. Punching shear failure

In the case of loose ands and soft clays, failure does not extend upto the ground surface. The jerk in foundation starts at a load of q_{u1} and the failure occurs at q_u. Beyond q_u, the load settlement curve is linear, and such failures are called punching shear failures. No heave is observed in this case.

1.1.3. Effect of water table on bearing capacity

Equation 4.4 for ultimate bearing capacity assumes that the water table is at great depth, and it has no effect on the bearing capacity of the foundation. The effect of water table is considered when it occurs at a depth between the ground level and a depth of $D_f + B$. In between these depths, two different conditions are considered as shown in Fig. 4. 2. Any variation water table between ground level and base of footing will have effects on both second and third terms in the right-hand side of Eq. 4.4, and if water table is located below the base of footing but above a depth of $D_f + B$, only the last term will be affected.

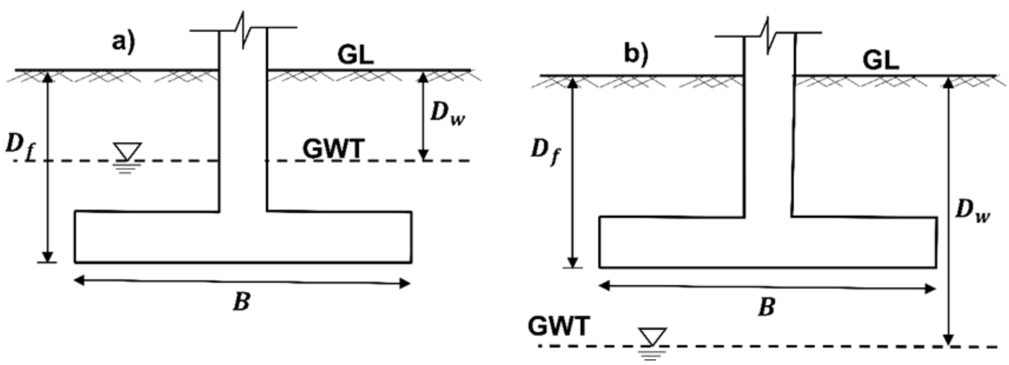


Fig. 4. 2 Depth of water table below foundation. a) Case 1 and b) Case II

1.1.3.1. Condition 1: Water table located above the base of footing

The soil below the water table is in submerged condition, and the second term in

Eq. 4.4 should be expressed in two parts. The first part considers the bulk unit weight of soil from ground surface to the depth of water table (D_w), and the second part considering submerged unit weight (γ') from the level of water table to the base of footing as shown in Eq. 4.5. The unit weight to be considered in the terms in the right-hand side of Eq. 4.4. is also the submerged unit weight.

$$\gamma D_f = \gamma D_w + \gamma'(D_f - D_w) \tag{4.5}$$

Thus Eq. 4.4 gets modified as:

$$q_u = c'N_c + [\gamma D_w + \gamma'(D_f - D_w)]N_q + 0.5B\gamma' N_\gamma \tag{4.6}$$

1.1.3.2. Condition 2: Water table located below the base of footing

In this case, the second term is not affected. The third term in the right-hand side of Eq. 4.4 gets modified into two parts, considering the submerged unit weight of soil below the water table.

$$B\gamma = \gamma(D_w - D_f) + \gamma'(D_f + B - D_w) \quad (4.7)$$

Eq. 4.4 is modified in this case as:

$$q_u = c'N_c + \gamma D_f N_q + 0.5[\gamma(D_w - D_f) + \gamma'(D_f + B - D_w)]N_\gamma \quad (4.8)$$

1.1.3.3. General expression

If $D_w = D_f$, i.e., if the water table is at the base of footing, Eq. 4.6 is same as Eq. 4.4, and if $D_w = D_f + B$, Eq. 4.8 is same as Eq. 4.4. Considering the variation along the depth is linear, two correction factors can be added to the second and third terms in the right-hand side of Eq. 4.4, to formulate a general expression given by:

$$q_u = c'N_c + \gamma D_f N_q W_q + 0.5B\gamma N_\gamma W_\gamma \quad (4.9)$$

where W_q is the water table correction for the second term to be used in Case 1 ($D_f \geq D_w$), given by:

$$W_q = 1 - 0.5\left(\frac{D_f - D_w}{D_f}\right)$$

$$W_\gamma = 0.5$$

and W_γ is the water table correction for the third term to be used during Case 2 ($D_w \geq D_f$), given by:

$$W_\gamma = 0.5 + 0.5\left(\frac{D_w - D_f}{D_f}\right)$$

Both W_q and W_γ varies between 0.5 and 1. Eq. 4.9 can also be used for isolated square and circular footings with minor modifications. In both the cases, the first term on right hand side of Eq. 4.9 gets multiplied by 1.2. In case of square footing, the third term gets multiplied by 0.4, while the coefficient for third term is 0.3 in the case of circular footings. The term B represents width of square footing and diameter in the case of circular footings.

1.1.4. Settlement of foundation

The allowable bearing pressure is decided based on both safe bearing capacity and settlement. The settlement in soil due to the external load should also be considered before deciding the dimensions for design. There are other causes also that may result in settlement, like underground erosion, landslides, frost and heave and vibrations, and if detected, suitable measures should be adopted to minimise these settlements as well.

The foundation settlement due to external loads has three different phases:

- Immediate or elastic settlement (Si): This occurs during or immediately after the construction of the structure. It is also known as the distortion settlement as it is due to distortions caused by the external load.
- Consolidation settlement (Sc): This settlement occurs due to gradual expulsion of water from the voids of the soil and is determined using Terzaghi's theory of consolidation (see Unit 5).
- Secondary Consolidation Settlement (Ss): This settlement occurs after completion of the primary consolidation. The amount of settlement in this case is very minor and is usually ignored.

The total settlement (S) is given by:

$$S = S_i + S_c + S_s$$

IS 1904 -1986 gives the safe values for maximum and differential settlements for different types of foundation, as listed in Table 4. 2.

Table 4. 2 Maximum and differential settlements (IS 1904 -1986)

Table 4. 2 Maximum and differential settlements (IS 1904 -1986)

	Sand and hard clay			Plastic clay		
	Max - Settlement	Diff - settlement	Angular Distortion	Max - settlement	Diff - settlement	Angular Distortion
(a) Isolated foundation						
(i) Steel structures	50 mm	0.0033 L	1/300	50 mm	0.0033 L	1/300
(ii) RCC structures	50 mm	0.0015 L	1/666	75 mm	0.0015 L	1/666
(b) Raft foundations						
(i) Steel structures	75 mm	0.0033 L	1/300	100 mm	0.0033 L	1/300
(ii) RCC structures	75 mm	0.002 L	1/500	100 mm	0.002 L	1/500

1.1.5. Field methods for determination of bearing capacity

IS 1904-1986 suggests that the safe bearing capacity of foundation should be calculated on the basis of field soil investigation results. For determining bearing capacity, plate load test is conducted in field as per IS 1888-1982.

1.1.5.1. Plate load test

To conduct a plate load test, the location should be selected based on the results of boring. Otherwise, it can also be conducted at an elevation of the proposed foundation in the worst estimated condition. If the depth of water table is within a depth equal to the width of the plate (B_P), the test shall be conducted at the level of water table. If water table is above the test level, it should be lowered to the test level by means of pumping out. The test pits are usually excavated with a width five times B_P, the size of the plate, to a depth equal to the depth of foundation (D_f). The test plate is usually a square of width varying from 300 mm to 750 mm, made of steel and is 25 mm thick. Occasionally, circular plates are also used. The dead load of all equipments including ball and socket, loading column, jack and steel plates should be noted before starting the test.

A central hole of the size $B_P \times B_P$, is excavated in the pit. The depth of the central hole (D_P) is obtained from the following relation:

$$\frac{D_P}{B_P} = \frac{D_f}{B_f}$$

where B_f is the width of foundation, and therefore,

$$D_P = (B_P/B_f) \times D_f$$

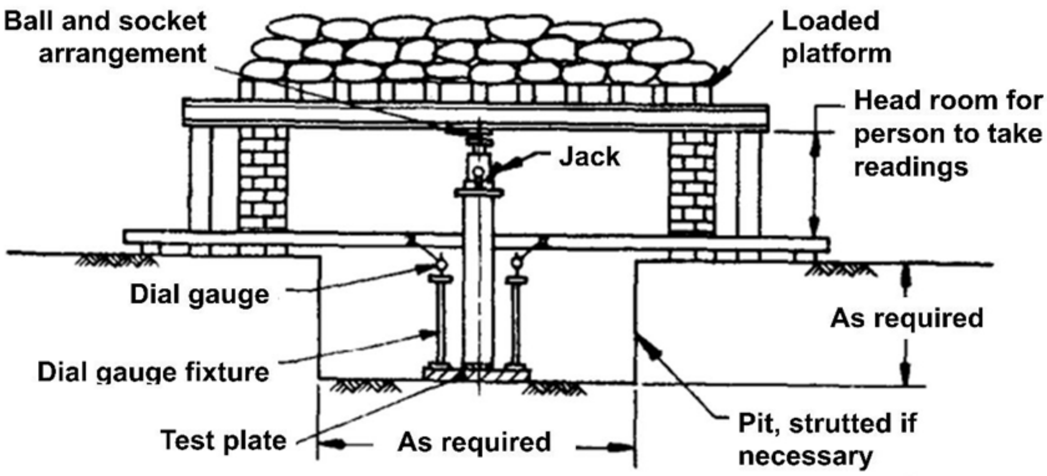


Fig. 4. 3 Typical setup for plate load test

Bearing Capacity of Soil

For conducting the plate load test, the plate is placed in the central hole and the load is applied by means of a hydraulic jack (Fig. 4. 3). Sometimes, trusses and reaction beam s are used instead of a loaded platform to take up the reaction. Wo dial gauges are required so that the settlement can be measured without any resetting in between. A seating load is first applied and released after some time. The load is then applied in increments of 1 kg/cm^2 or about 20% of the estimated ultimate bearing capacity, whichever is less. The settlement is recorded after 1, 2.25, 4, 6.25, 9, 16 and 25 minutes, and further at an interval of one hour to the nearest 0.02 mm. These hourly observations are continued for clayey soils until the rate of settlement is less than 0.2 mm/h. The test is conducted till twice the estimated design pressure or until failure or at least until the settlement of about 25 mm has occurred.

The ultimate load for the plate $q_u(p)$ is indicated by a break on the log-log plot between the load per unit area q and the settlement s. If the break is not well-defined, the ultimate load is taken as that corresponding to a settlement of one-fifth of the plate width (B_P). On the natural plot (Fig. 4. 4), $q_u(p)$ is obtained from the intersection of the tangents drawn on load-settlement curve.

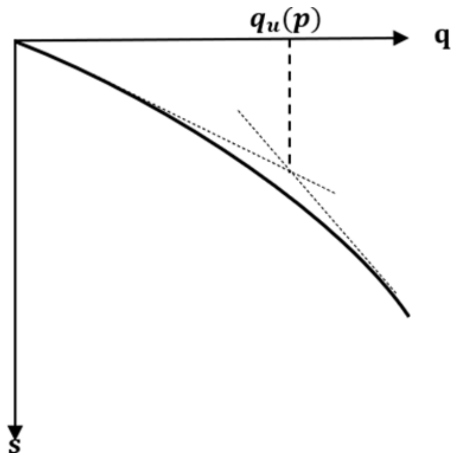


Fig. 4. 4 Load intensity vs settlement for plate load test

The following relations can be used for the calculation of ultimate bearing capacity of the proposed foundation $q_u(f)$:

For clayey soils,

$$q_u(f) = q_u(p) \tag{4.15}$$

For sandy soils,

$$q_u(f) = q_u(p) \times \frac{B_f}{B_P}$$

The plate load test can also be used to calculate the settlement for a given intensity of loading qo. The following relations can be used between the settlement of the plate (S_p) and that of the foundation (S_f) for the same load intensity:

For clayey soils,

$$S_f = S_P \times \frac{B_f}{B_P}$$

where S_p is the settlement of plate, obtained from the load intensity-settlement curve, corresponding to qo.

For sandy soils,

$$S_f = S_p \left[\frac{B_f (B_p + 0.3)}{B_p (B_f + 0.3)} \right]^2$$

In above equations, B_f is the width of foundation in metres and B_p is the width of the plate also in metres.

Even though the test is widely followed for determining the bearing capacity, plate load test has the following limitations:

- Size effect: The results of the plate load test reflect the strength and the settlement characteristics of the soil corresponding to the size of plate. The area where stress is induced in soil is much deeper for the actual foundation when compared to that of the plate and the test does not satisfactorily represent the actual conditions in non-homogeneous and anisotropic soils.
- Scale effect: The ultimate bearing capacity of clayey soils is does not depend on the size of the plate but for sandy soils, it increases with the size of the plate (Eq. 4.16). Hence it is advised to repeat the test with plates of different sizes in case of sandy soils.
- Time effect: A plate load test is conducted for a short duration when compared with the actual service period of a foundation. In case of clayey soils where consolidation settlement is important, the settlement obtained from the test is not satisfactory.
- Interpretation of failure load: The failure load is not well-defined, except in the case of a general shear failure. Hence the load interpreted from the graph highly depends upon the skill of the interpreter.

Bearing Capacity of Soil

- Reaction load: It is not practicable to provide a reaction of more than 250 kN. Hence, the test on a plate of size larger than 0.6 m width is difficult.
- Water table: The level of the water table affects the bearing capacity of the sandy soils. If the depth of water table is within a depth equal to the width of the plate (B_P), the test shall be conducted at the level of water table. If water table is above the test level, it should be lowered to the test level by means of pumping out.

1.1.5.2. Bearing capacity from standard penetration test

The bearing capacity of foundation can also be calculated using the results of standard penetration test (SPT). The test is conducted before starting the project, at the stage of site investigations, by drilling boreholes. The procedure can be followed as per IS 2131– 1981. Tests shall be made at every change in stratum or at intervals of not more than l.5 m whichever is less. The intervals be increased to 3 m if in between vane shear test is performed. The test set up consists of a drilling equipment, split spoon sampler and a drive weight assembly. The drive weight assembly shall consist of a driving head and a 63.5 kg weight with 75 cm free fall. The split spoon sampler resting on the bottom of borehole should be allowed to sink under its own weight; then the split spoon sampler shall be first seated 15 cm with the blows of the hammer falling through 75 cm. This first 15 cm is the seating drive. Thereafter, the split spoon sampler shall be further driven by 30 cm more. The number of blows required to affect each 15 cm of penetration shall be recorded. The total blows required for the second and third 15 cm of penetration shall be termed the penetration-resistance N.

The penetration resistance or the SPT N value indicates the resistance of soil to the penetrations induced by the hammer fall, and hence the value can be used for determining the bearing capacity of soils.

Method 1: The ultimate bearing, capacity of sandy soils may be determined using correlations between N and the value of angle of internal friction, φ as mentioned in

Table 4. 3. The average value of N between the base of the footing and the depth equal to 1.5 to

Table 4. 3 Correlation between N and φ

N	Denseness	φ
0 - 4	Very loose	$25^\circ - 32^\circ$
4 - 10	Loose	$27^\circ - 35^\circ$
10 - 30	Medium	$30^\circ - 40^\circ$
30 - 50	Dense	$35^\circ - 45^\circ$
> 50	Very dense	$> 45^\circ$

2.0 times the width of the foundation can be used, and the bearing capacity factors can be found. Table 4. 3 Correlation between N and φ

Method II: In this method, instead of using correlations between N and φ, the value of N is directly used for determining the bearing capacity. Teng (1962) gave the following equation for the net ultimate capacity of a strip footing:

$$q_{nu} = \frac{1}{6.0}\left[3N^2 B_f W_\gamma + 5(100 + N^2) D_f W_q\right]$$

Or $$q_{nu} = \left[0.5N^2 B_f W_\gamma + 0.83(100 + N^2) D_f W_q\right]$$

1.2. Earth pressure theories

While foundations transfer the vertical load to the underlying soil, earth retaining structures are required to resist the horizontal stress exerted by soil. Such structures are required when soil has to be retained at different elevations on both sides of a wall. In some cases, soil has to be supported at unsafe slopes, close to vertical, due to space constraints. Retaining structures can be used in such cases as well. Generally, the soil behind retaining structures is vertical, hindering the soil on higher elevation from sliding down. The soil retained is also known as the backfill.

The force acting on retaining structures are in the lateral direction, and the magnitude of this force increases along the depth. The force also depends upon the interaction between the wall and soil, including the friction between them. For simplicity, the retaining wall is assumed to be smooth, vertical and rigid in analysis. The lateral earth pressure is usually computed using the classical theories proposed by Coulomb (1773) and Rankine (1857).

1.2.1. Different types of lateral earth pressure

Depending upon the movement of the retaining wall with respect to the soil retained, lateral earth pressure is classified into three categories (Fig. 4. 5).

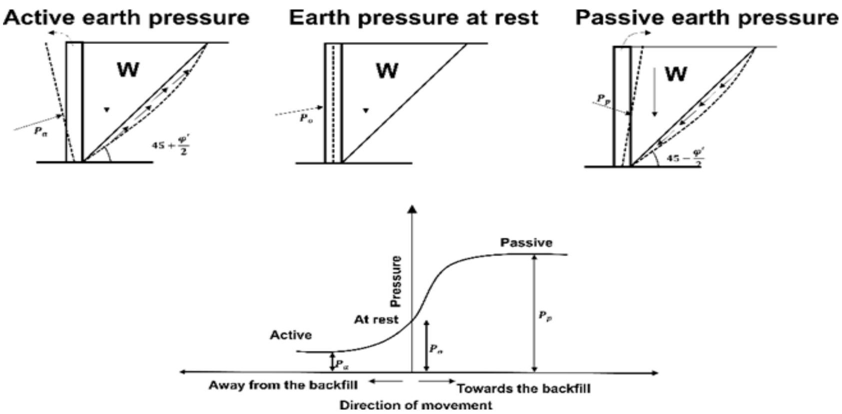


Fig. 4. 5 Relationship between lateral earth pressure and wall movement

1.2.1.1. At-rest pressure

The lateral earth pressure is called at-rest pressure when the retained soil is not subjected to any movement or lateral yielding. This occurs when the retaining wall is firmly fixed at its top without any provisions for rotation and lateral movement. Basement retaining walls and bridge abutments are examples for at rest pressure. This condition is also called the state of elastic equilibrium.

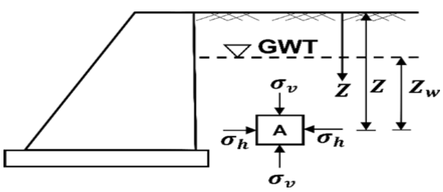


Fig. 4. 6 Earth pressure at rest

Fig. 4. 6 shows a retaining wall in which no movement takes place. The vertical effective stress at point A at a depth Z is given by

$$\bar{\bar{\sigma}}_{\bar{v}} = \gamma Z - \gamma_w Z_w$$

The coefficient of earth pressure at-rest (K_o), is equal to the ratio of the horizontal stress to the vertical stress given by:

$$K_o = \frac{\bar{\sigma}_h}{\bar{\sigma}_v}$$

The horizontal effective stress ($\bar{\sigma}_h$) can be obtained by multiplying K_o with $\bar{\sigma}_v$ as:

$$\bar{\sigma}_h = K_O\, \bar{\sigma}_v = K_O\,(\gamma Z - \gamma_w Z_w)$$

The coefficient of lateral pressure at rest (K_O) relates the effective stresses. This is because the stress induced by soil in different directions vary, while the hydrostatic stress exerted by pore pressure is same in all directions. Hence pore water pressure should not be multiplied with the coefficient. The total lateral pressure (p_h) is equal to the sum of the effective stress ($p_o = \bar{\sigma}_h$) and the pore water pressure (u).

Thus,

$p_h = p_o + u$

Therefore, the total lateral pressure at depth Z is,

$$p_h = K_O\,(\gamma Z - \gamma_w Z_w) + \gamma_w Z_w$$

When $Z = 0$, the value of p_h is 0 and it increases linearly till the bottom of the wall. Thus, the distribution of earth pressure is triangular along the depth of the wall (Fig. 4. 7).

If the water table is at a depth d, Eq. 4.25 can be modified for any depth Z, which is greater than d as:

$$\begin{aligned} p_h &= K_O\,[\gamma Z - \gamma_w(Z - d) + \gamma_w(Z - d)] \\ &= K_O\,\gamma d + K_O\,\gamma'(Z - d) + \gamma_w(Z - d) \end{aligned}$$

The pressure at the bottom of a wall of hight H can be calculated as:

$$p_h = K_O\,\gamma d + K_O\,\gamma'(H - d) + \gamma_w(H - d)$$

If the water table is at the ground surface the pressure at the bottom of the wall is given by, taking $d = 0$ in Eq. 4.26,

$$p_h = K_O \gamma' H + \gamma_w H$$

The total pressure acting per unit length of wall, or the resultant force (P) can be calculated by finding the area of triangles formed by both effective stress and pore water pressure.

$$P = \frac{1}{2}(K_O \gamma H^2 + \gamma_w H^2)$$

The point of application of the resultant pressure P is determined from the pressure distribution diagram. For triangular pressure distribution, it acts at height $H/3$ from the base.

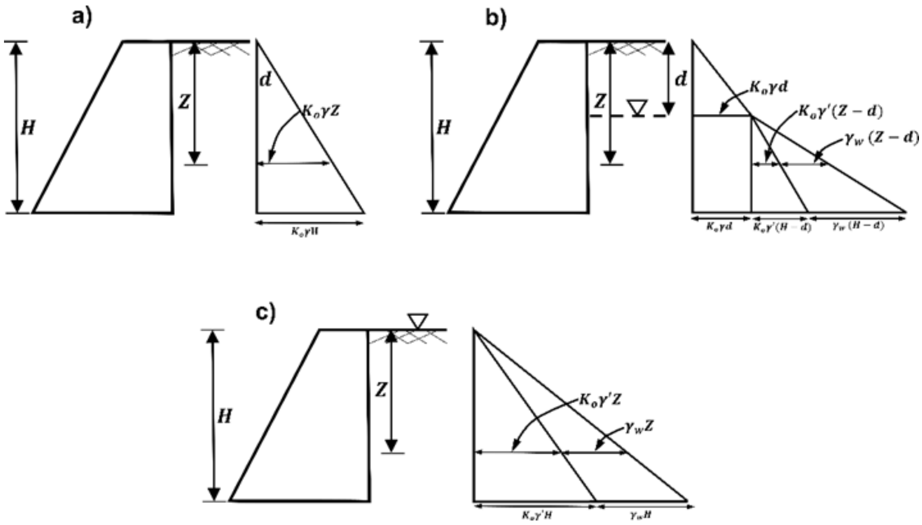


Fig. 4. 7 Distribution of lateral earth pressure. a) dry condition, b) water table above the bottom of the wall and c) water table at the top of backfill

1.2.1.2. Active pressure

A state of active pressure occurs when the wall moves away from the backfill. It is a state of plastic equilibrium where the backfill soil is on the verge of failure. The soil retained on higher elevation usually exerts pressure on the retaining wall and reaches the state of active earth pressure.

1.2.1.3. Passive pressure

A state of active pressure occurs when the wall moves towards the back fill. It is another extreme of the limiting equilibrium condition. The state of passive earth pressure exists in soil retained at lower elevation on one side of the backfill. Another example of the passive earth pressure is the pressure acting on an anchor block.

1.2.2. Rankine's earth pressure theory

Rankine (1857) considered the equilibrium of a soil element within a soil mass bounded by a plane surface, based on the following assumptions:

- The soil mass is homogeneous and semi-infinite.
- The soil is dry and cohesionless.
- The ground surface is plane, which may be horizontal or inclined.
- The back of the retaining wall is smooth and vertical.
- The soil element is in a state of plastic equilibrium, ie., at the verge of failure.

The coefficients of active and passive earth pressures can be derived using the concept of Mohrs circle as explained in the following sections.

1.a.i.4. Active Earth Pressure.

Fig. 4. 8 Active earth pressure

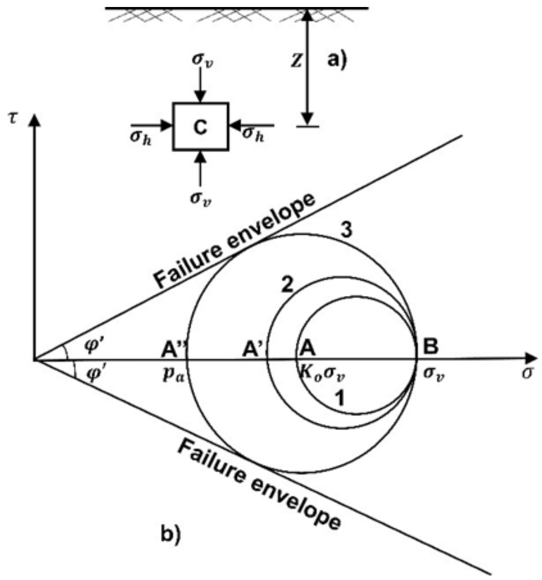
Consider an element of dry soil C, at a depth Z below a level soil surface (Fig. 4. 8a). Initially, the element is at-rest conditions (circle 1 in Fig. 4. 8b), and the horizontal pressure is given by

$$\sigma_h = K_0 \sigma_v$$

where σ_v ($\sigma_v = \gamma Z$)is the vertical stress at C, and K_0 is the coefficient of earth pressure at rest.

The stresses σ_v and σ_h are, respectively, the minor and major principal stresses, and are indicated by points A and B in the Mohr circle (Fig. 4. 8b).

When soil stretches horizontally, the vertical stress remains constant, but the horizontal stress is reduced. The point A shifts to position A' and the diameter of the Mohr circle increases. With further decrease in horizontal stress, the point A shifts to position A" when the Mohr circle touches the failure envelope, and the soil is at the verge of failure. This state is known as the Rankine active state of plastic equilibrium, and the horizontal stress at that state is the active pressure (P_a). the Mohr circle when active conditions are developed is plotted in Fig. 4. 9.



From the figure,

$$p_a = OE = OC - CE$$
$$CE = CD = OCsin\varphi',$$
$$p_a = OC - OCsin\varphi'$$
$$\sigma_v = OC + CB = OC + OCsin\varphi'$$
$$\frac{p_a}{\sigma_v} = \frac{(1 - sin\varphi')}{(1 + sin\varphi')}$$
$$p_a = \frac{(1 - sin\varphi')}{(1 + sin\varphi')}\sigma_v$$
$$p_a = K_a\gamma Z$$

where K_a, is a coefficient, known as the coefficient of active earth pressure. It is a function of the angle of shearing resistance (φ'), and is given by

$$K_a = \frac{(1 - sin\varphi')}{(1 + sin\varphi')} = tan^2\,[45^0 - \frac{\varphi'}{2}]$$

The pressure distribution is similar to one shown in Fig. 4. 7 in which K_a, is substituted for K_o.

1.a.i.5. Passive Earth pressure

The passive Rankine state of plastic equilibrium can be explained by considering the element of soil at a point at a depth of Z below the soil surface (Fig. 4. 10a).

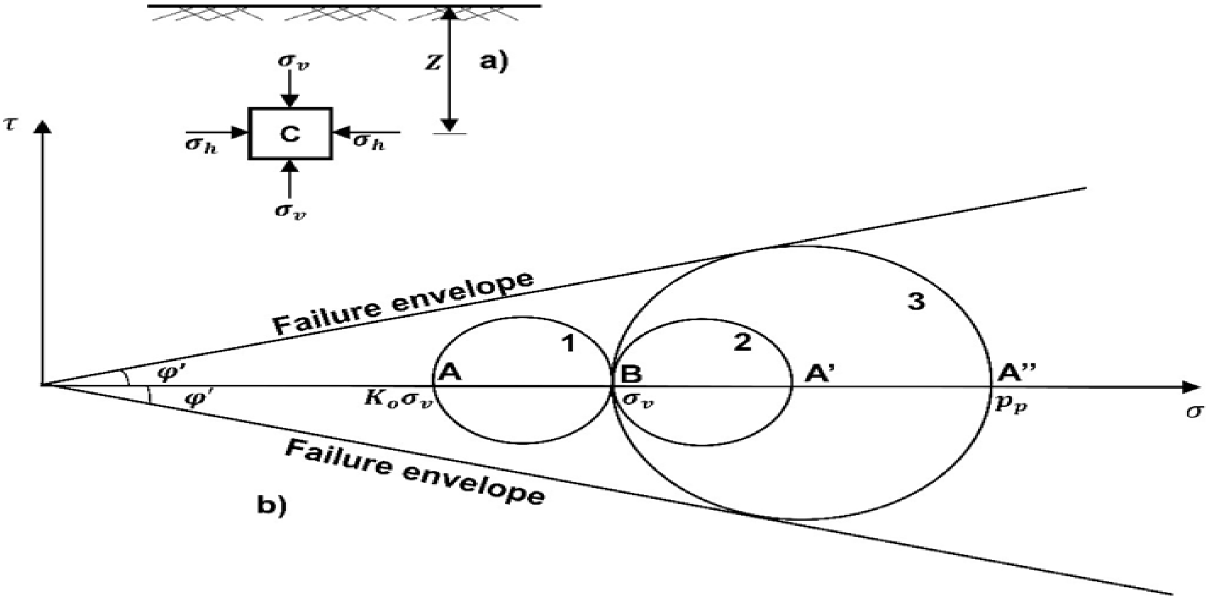
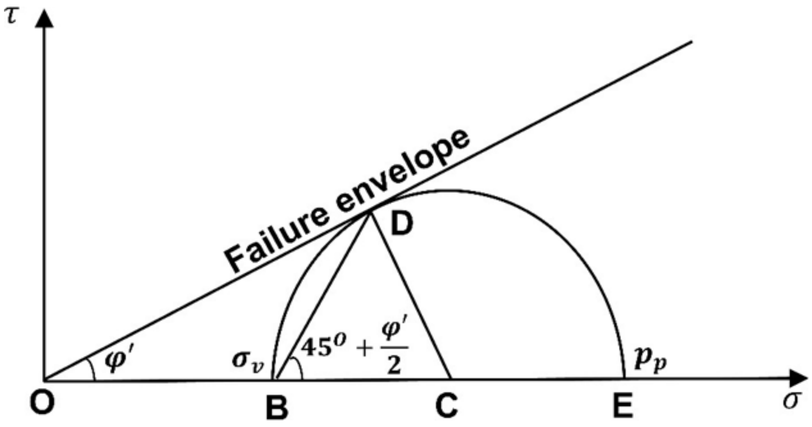


Fig. 4. 10 Passive earth pressure

As the soil is compressed laterally, the horizontal stress is increased, whereas the vertical stress remains constant. The Mohr circles in Fig. 4. 10b plots the variation from at rest to failure. At the verge of failure, the horizontal stress increases until it reaches a limiting value greater than the vertical stress, indicated by point A'' and the Mohr circle touches the failure envelope. Fig. 4. 11, shows the Mohr circle at failure.



From Fig. 4. 11,

Fig. 4. 11 Mohrs circle for passive earth pressure condition

$$p_p = OC + CE = OC + CD = OC + OC\, sin\varphi'$$

$$\sigma_v = OC - BC = OC - CD = OC - OC\, sin\varphi'$$

$$\frac{p_p}{\sigma_v} = \frac{(1 + sin\varphi')}{(1 - sin\varphi')}$$

$$p_p = \frac{(1 + sin\varphi')}{(1 - sin\varphi')}\sigma_v$$

$$p_p = K_p \gamma Z$$

where K_p is the coefficient of passive earth pressure, given by

$$K_p = \frac{(1 + sin\varphi')}{(1 - sin\varphi')} = tan^2\left[45^0 + \frac{\varphi'}{2}\right]$$

The coefficient of passive pressure (K_p) depends upon Φ'. The pressure distribution is similar to that shown in Fig. 4. 7, in which $K_p$is substituted for K_O

UNIT SUMMARY

The unit discusses two critical engineering applications of soil mechanics, the bearing capacity and the lateral earth pressure. In the first part, the calculation of bearing capacity and Terzaghi's theory are discussed and in the second part, different lateral earth pressure conditions are explained with Rankine's theory. Both the theories are important in designing geotechnical structures/

EXERCISES

Multiple Choice Questions

1. Which among the following is not an assumption in Terzaghi's bearing capacity theory?
    d) Footing is long
    d) Footing is laid at shallow depth
    d) Footing is in square shape
    d) The load distribution on footing is uniform
2. The increase in pressure due to the superstructure, at the base of foundation that leads to the shear failure of soil is known as:
    d) Ultimate bearing capacity
    d) Net ultimate bearing capacity
    d) Net safe bearing capacity
    d) Gross safe bearing capacity
3. SPT *N* value is the number of blows required by the falling hammer to penetrate
    d) First 15 cm
    d) First 30 cm
    d) Last 30 cm
    d) First 30 cm
4. Which among the following arrangements is correct in terms of magnitude of lateral earth pressure, when the vertical stress and other conditions remains constant?
    d) Active earth pressure > Passive earth pressure > Earth pressure at rest
    d) Active earth pressure > Earth pressure at rest > Passive earth pressure
    d) Passive earth pressure > Active earth pressure > Earth pressure at rest
    d) Passive earth pressure > Earth pressure at rest > Active earth pressure

Numerical Examples:

1. The strip footing below a wall is placed at ground level, with a width of 2 m. The soil is dry sand with a unit weight of 19 kN/m^3 and angle of internal friction 40^o. Calculate the ultimate bearing capacity of the footing.

2. Determine the ultimate bearing capacity of a strip footing 1.5 m wide, placed at a depth of 1
m. The soil has a cohesion of 15 kPa, angle of internal friction 35^o, unit weight
18 kN/m^3. Consider depth of water table at:
    a. Ground surface
    b. 1 m below the ground surface
    c. 10 m below the ground surface

3. A 6.3 m high vertical wall with smooth surface retains loose sand with bulk unit weight of 18 kN/m^3 and angle of internal friction 18^o. The backfill has the same height of wall and the surface of backfill is horizontal. Determine the total active thrust on the wall and its point of application, if the water table is well below the base of the wall.

Exercises

1. Two footings, one circular and the other square, are resting on the surface of a purely cohesionless soil. If both the foundations are having same base area, the ratio of their (circular to square) ultimate bearing capacities as per Terzaghi's theory is:
2. Determine the net safe bearing capacity for a square footing (2m x 2m) resting at a depth of 2m, when the water table is
   a. at ground surface.
   b. 1m below the ground surface.
   c. At the base of the footing

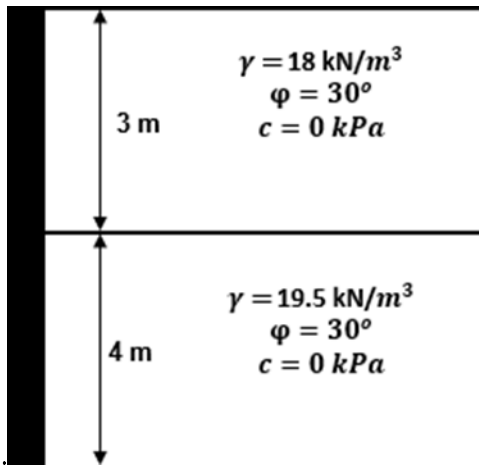
d. 5m below the ground surface.

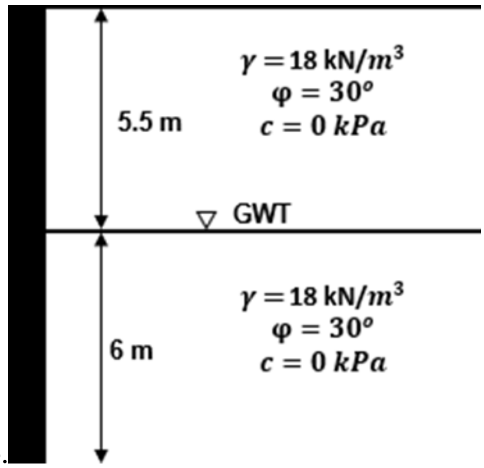
Use Terzaghi's equation.

The density of the soil is 18 kN/m^3, saturated density is 20 kN/m^3, c = 5 kN/m^2, φ = 40^o

3. For the retaining walls shown in figure, determine the active earth thrust per unit length of the wall. Also determine the location of the resultant force.

a.



b. 

Short and Long Answer Type Questions

9) What are the assumptions made in Terzaghi's bearing capacity theory?
9) Define the following terms:
   c) Ultimate bearing capacity
   c) Net safe bearing capacity
   c) Net allowable bearing pressure
9) What is meant by consolidation settlement? Is it observed in sandy soils? Justify our answer.
9) What is a standard penetration test?
9) What are the different categories of lateral earth pressure? Explain with a neat sketch.
9) Derive the equation for coefficient of active earth pressure using Rankine's theory, for cohesionless soils.
9) How is ultimate bearing capacity calculated using plate load test? Explain in detail.
9) What are the different types of shear failures that may occur in a foundation? Explain with neat sketches.
9) Write down the assumptions and limitations of Rankine's Earth Pressure theory.