GEOTECHNICAL ENGINEERING

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SYLLABUS

1

Introduction

1.1 Soil and Soil Engineering

1.2 Scope of Soil Mechanics

1.3 Origin and formation of soil

2

Preliminary Definitions and Relationship

2.1 Soil as a three Phase system.

2.2 Water Content, Density, Specific gravity, Voids ratio, Porosity, Percentage of air voids, air content, degree of saturation, density Index, Bulk/Saturated/dry/submerged density, Interrelationship of various soil parameters

3

Index Properties of Soil

3.1 Water Content

3.2 Specific Gravity

3.3 Particle size distribution: Sieve analysis, wet mechanical analysis, particle size distribution curve and its uses

3.4 Consistency of Soils, Atterberg's Limits, Plasticity Index, Consistency Index, Liquidity Index

4

Classification of Soil

4.1 General

4.2 I.S. Classification, Plasticity chart

5

Permeability and Seepage

5.1 Concept of Permeability, Darcy’s Law, Co-efficient of Permeability,

5.2 Factors affecting Permeability.

5.3 Constant head permeability and falling head permeability Test.

5.4 Seepage pressure, effective stress, phenomenon of quick sand

6

Compaction and Consolidation

6.1 Compaction: Compaction, Light and heavy compaction Test, Optimum Moisture

Content of Soil, Maximum dry density, Zero air void line, Factors affecting Compaction, Field compaction methods and their suitability

6.2 Consolidation: Consolidation, distinction between compaction and consolidation.

Terzaghi‘s model analogy of compression/ springs showing the process of consolidation – field implications

7

Shear Strength

7.1 Concept of shear strength, Mohr- Coulomb failure theory, Cohesion, Angle of internal friction, strength envelope for different type of soil, Measurement of shear strength;- Direct shear test, triaxial shear test, unconfined compression test and vane-shear test

8

Earth Pressure on Retaining Structures

8.1 Active earth pressure, Passive earth pressure, Earth pressure at rest.

8.2 Use of Rankine's formula for the following cases (cohesion-less soil only)

(i) Backfill with no surcharge, (ii) backfill with uniform surcharge

9

Foundation Engineering

9.1 Functions of foundations, shallow and deep foundation, different type of shallow and deep foundations with sketches. Types of failure (General shear, Local shear & punching shear)

9.2 Bearing capacity of soil, bearing capacity of soils using Terzaghi's formulae & IS Code formulae for strip, Circular and square footings, Effect water table on bearing capacity of soil

9.3 Plate load test and standard penetration test

Chapter 1: Introduction

1.1 Soil and Soil Engineering

Definition of soil: The term 'soil' in soil engineering is defined as unconsolidated material, composed of soil particles, produced by the disintegration of rocks. The voids space between the particles may contain air, water or both. The soil particles may contain organic matter.

Definition of soil mechanics:

- The term 'soil mechanics' was coined by Dr. Karl Terzaghi in 1925, who is also known as the father of soil mechanics.
- According to Terzaghi, soil mechanics is the application of the laws of mechanics & hydraulics to engineering problems dealing with sediments & other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rook, regardless of whether or not they contain an admixture of organic constitute.
- **Soil mechanics is therefore, a branch of mechanics which deals with the action of forces on soil and with the flow of water in soil.**

Definition of soil engineering:

- Soil engineering is an applied science dealing with the applications of principals of soil mechanics to practical problems.
- It has a much wider scope than soil mechanics, as it deals with all engineering problems related with soils. It includes site investigations, design and construction of foundations, earth-retaining structures and earth structures.

Definition of geotechnical engineering:

- Geotechnical is a broader term which includes soil engineering, rock mechanics and geology.
- Sometimes geotechnical engineering is used synonymously with soil engineering.

1.2 Scope of Soil Mechanics:

Soil Mechanics is a basic subject and its scope will never end because all structures are built on soil and for buildings and structures to perform well and for a long time, soil tests should be done so that to know about the properties of soil and its characteristics.

1.3 Origin and formation of soil:

- In a broad sense, soil may be thought of an incidental material in vast geological cycle which has been going on continuously for millions of years of geological time.

- The geological cycle consists of 3 phases, Erosion, Transportation and deposition & Earth Movement.

a) **Erosion Phase:**

- The cycle starts with erosional phase in which there is degradation of exposed rock by weathering process.
- The weathering process may be
  i. Physical weathering
  ii. Chemical weathering

i. Physical weathering :

- The physical weathering process may be
  - Erosion of rock caused by the action of wind, water, glaciers.
  - Disintegration caused by alternate freezing and thawing in cracks in the rock.
- The resulting soil particles retain the same composition as that of parent rock.
- Their shape can be indicated by terms such as angular, rounded, flat and elongated.
- Gravel and sand fall into this group.

ii. Chemical weathering:

- The chemical process results in changes in the mineral form of parent rock due to the action of water.
- Chemical weathering results in the formation of group of crystalline particles of colloidal size ($< 2\mu$) known as clay mineral.
- If the products of rock weathering are still located at the place where they originated, they are called residual soil.

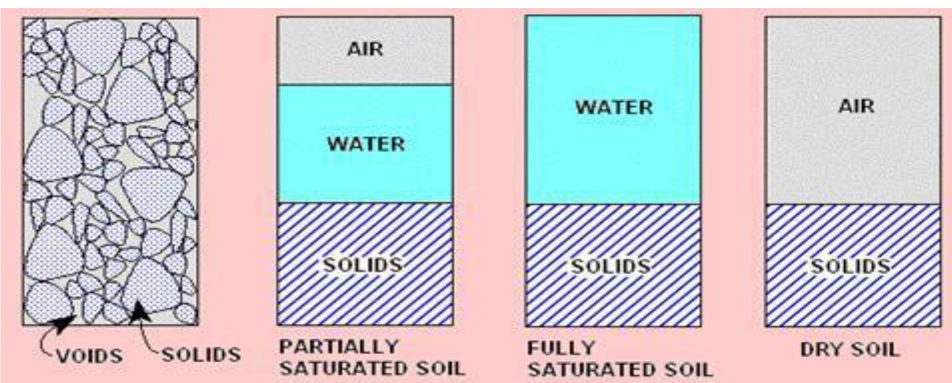
b) **Transportation/Deposition:**

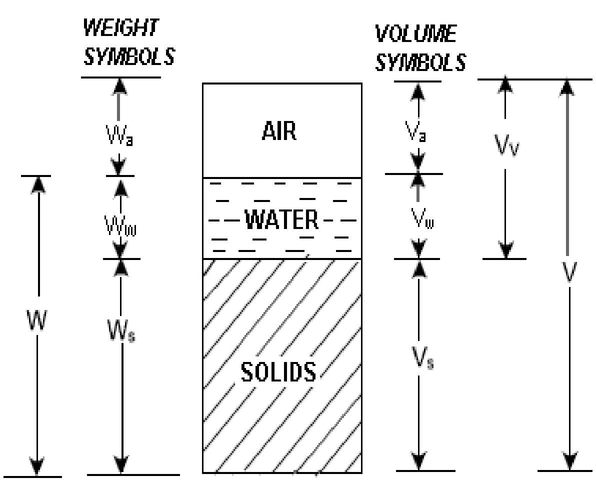
- In the second phase, the fragmented material is transported by agent such as wind, water or ice to new locations.
- Soil transported from their origin by wind, water, ice or any other agency and has been deposited is called transported soil. They have generally small grain size, large amount of pores.
- According to the transporting agency, soils are classified as:
  - Alluvial deposit – deposited by river water.
  - Lacustrine deposit – deposited by still water like lakes.
  - Marine deposit – deposited by sea water.
  - Aeolian deposit – transported by wind.
  - Glacial deposit – transported by ice.
  - Colluvial soil – deposited by gravity. (e.g. talus)

Chapter 2: Preliminary Definitions and Relationship

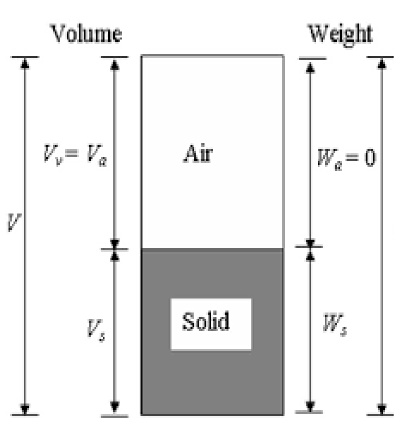
2.1 Soil as a three Phase system.

- A soil mass consists of solid particles which form a porous structure. The voids in the soil mass may be filled with air, with water or partly with air and partly with water. The three constituents are blended together to form a complex material.
- However, for convenience, all the solid particles are segregated and placed in the lower layer of the three phase diagram.
- Likewise, water and air particles are placed separately, as shown in figure below.
- The 3-phase diagram is also known as **Block diagram**.
- It may be noted that the 3-constituents cannot be actually segregated, as shown.
- A 3-phase diagram is an artifice used for easy understanding and convenience in calculation.
- Soil can be either two-phase or three-phase composition.
- Fully saturated soil and fully dry soil are two phase system.
- Partially saturated soil are three phase system.

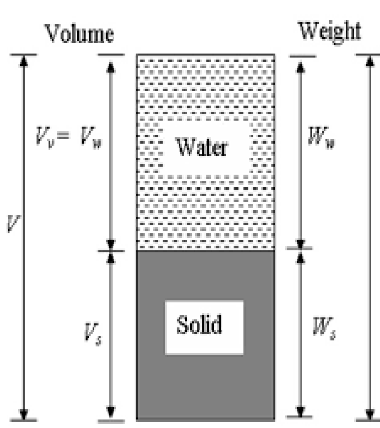




Three phase system



Two phase system



Two phase system

Where

V_a= Volume of air W_a = Weight of air = 0

V_w = Volume of water W_w = Weight of water

V = Total volume of Soil mass W_s = Weight of soil solid

V_v = Volume of voids   W = Total weight of soil mass

V_s = Volume of Solid W_{sat} = Saturated weight of soil mass

2.2 Important definitions:

1. Water content (w):

- Water content or moisture content of a soil is defined as the ratio of weight of water to weight of solids (dry weight) of the soil mass.

$$w = \frac{Ww}{Ws} \text{ x } 100 \;; \;\; w \geq 0$$

- It is denoted by the letter symbol w and is commonly expressed as percentage i.e. 20%, 50% etc.
- The minimum value of water content is 0.
- There is no upper limit for water content.
- Generally fine grained soil have higher water content as compared to coarse grained soil.

2. Void ratio (e):

- Void ratio (e) is the ratio of the volume of voids (Vv) to the volume of soil solids (Vs), and is expressed as a decimal.

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

- There is no upper limit of void ratio in soil suspension.
- Void ratio of fine grained soil are generally higher than those of coarse grained soil.
- Size of void in coarse grained soil are generally larger than that in fine grained soil.

3. Porosity (n):

- Porosity (n) is the ratio of the volume of voids to the total volume of soil (V), and is expressed as a percentage.

$$n = \frac{V_V}{V} x\,100$$

- $V = V_V + V_S$, $V_V = V_W + V_a$
- The porosity of soil cannot exceed 100% hence it has a upper limit of 100% or 1.
- Both porosity and void ratio are measured of denseness or looseness of soil.

Note:

Total volume V is a variable quantity. But, since solids are incompressible, V_s remain invariant in the total volume V of the soil.

4. Degree of saturation (S):

- Degree of saturation of soil mass is defined as the ratio of volume of water in the voids to volume of voids.
- $S = \frac{V_w}{V_v}$, $0 \leq S \leq 100$
- For a fully saturated soil mass $V_v = V_{w,}$ hence for the saturated soil mass S = 100% .
- For fully dry soil $V_w = 0$, hence for a fully dry soil mass s = 0%
- For partially saturated soil mass degree of saturation of soil mass varies between 0 – 100%, which is most common condition in nature.

5. Percentage Air voids (n_a):

- Percentage air voids (n_a) is the ratio of the volume of air to the total volume.
- $n_a = \frac{V_a}{V} \times 100$

6. Air content (a_c):

- Air content (a_c) is the ratio of the volume of air (Va) to the volume of voids V_v.

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

- $n_a = \frac{V_a}{V} = \frac{V_a \times Vv}{V \times Vv} = \frac{Vv}{V} \times \frac{V_a}{Vv} = n \times a_c$

  $\boxed{n_a = n \times a_c}$

7. Bulk unit weight (γ_t/γ):

- Bulk unit weight of a soil mass is defined as the weight per unit volume.
- $\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{Vw + Va + Vs}$
- It is generally expressed as KN/m^3, N/m^3, kgf/ KN/cm^3

8. Unit weight of Solid (γ_s):

- It is defined as the ratio of weight of solids to weight of volume of solids. It is also called **absolute unit weight** of soil mass.
- $\gamma_s = \frac{W_s}{V_s}$

9. Unit weight of water (γ_w):

- $\gamma_w = \frac{W_w}{V_w}$
- The value of γ_w changes with temperature but usually we take γ_w as 9.81 kN/m^3 which is at 4^oC.

10. Dry unit weight (γ_d):

- It is defined as weight of soil solid (or weight of dry soil) per total volume of soil.
- Unit is KN/m^3, N/m^3, kgf/ KN/cm^3
- $\gamma_d = \frac{W_s}{V}$
- Dry unit weight is used as a measure of denseness of soil. A high value of dry unit weight indicates that more solids are packed in unit volume of soil hence a more compact soil.

11. Saturated unit weight (γ_{sat}):

- It is defined as the bulk unit weight of soil mass in saturated condition.
- $\gamma_{sat} = \frac{\text{Wt. of Saturated soil}}{\text{Volume of soil}}$

12. Submerged unit weight (γ_{sub} or γ'):

- It is defined as the weight of submerged soil per total volume of soil mass. When the soil exists below the ground water table, the soil mass is said to be in submerged condition.

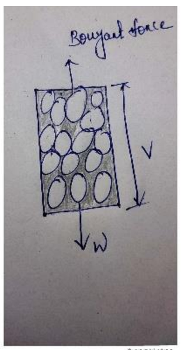
- $\gamma_{sub} = \frac{\text{Wt. of Submerged soil}}{\text{Volume of soil}}$
- $\gamma' = \gamma_{sat} - \gamma_w$

Note:

Bulk unit weight = Saturated unit weight , (when soil is fully saturated)

Bulk unit weight = Dry unit weight, (when soil is completely in dry condition)

- When the soil exist below the ground water table, two forces are act on it.
- One is weight of soil, acting vertically downward and 2nd is buoyant force/weight acting vertically upward as shown in fig. below.



Buoyant weight/force = Submerged weight of soil

$\gamma' = \gamma_{sat} - \gamma_w$

$\gamma' = \frac{\text{Wt. of Submerged soil}}{\text{Volume of soil}}$

Wt. of submerged soil = total weight of soil sample – weight of water displaced by the sample = Buoyant weight

$$= W - \gamma_w \times V$$

$$\gamma' = \frac{W - \gamma_w \times V}{V} = \frac{W}{V} - \frac{\gamma_w \times V}{V} = \gamma_{sat} - \gamma_w$$

13. Specific gravity of solids (G/G_s):

- It is defined as the ratio of the unit weight of solids (absolute unit weight of soil) to unit weight of water.
- $G_s = \frac{\gamma_s}{\gamma_w}$
- It is a unit less quantity.

- This is also known as Absolute specific gravity or Grain specific gravity.

14. Mass specific gravity (G_m):

- It is the ratio between the bulk unit weight of soil to unit weight of water
- $G_m = \frac{\gamma_t}{\gamma_w}$

5. Relative Density (D_r):

- $D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$
- e_{max} & e_{min} represents the soil in very dense and loose conditions respectively. e is the void ratio of natural soil.

Inter-Relations:

1. e.s = G.w

$$w = \frac{W_W}{W_s} = \frac{\gamma_W . V_W}{G_s . \gamma_W . V_s} = \frac{V_W}{G_s . V_s} = \frac{S . V_v}{G_s . V_s} = \frac{S . e}{G_s}$$

2. $W_s = \frac{W}{1+w}$
3. $V_s = \frac{V}{1+e}$
4. $e = \frac{n}{1-n}$
5. $n = \frac{e}{1+e}$
6. $\gamma_t = \frac{G+e.s}{1+e} \times \gamma_w$
7. $\gamma_{sat} = \frac{G+e}{1+e} \times \gamma_w$ (as S = 1)
8. $\gamma_d = \frac{G}{1+e} \times \gamma_w$ (as S = 0)
9. $\gamma_d = \frac{\gamma_t}{1+w}$

Chapter 3: Index Properties of Soil

- In this chapter, we shall describe the methods of determining those properties of soil which are used in their identification and classification.
- These include the determination of
    i. Water content (w)
    ii. Specific gravity (G)
    iii. Particle size distribution
    iv. Consistency limits
    v. In-situ density
    vi. Density index
- These properties are known as index properties.

i. Water content (w):

- Water content of soil is an important soil parameter which significantly influences the behavior of soil, particularly cohesive soils.
- Water content and unit weight changes during transportation and storage. Hence it is important to determine it before carrying out any other tests.
- Water content determination is also important because some physical state properties are calculated using water content following the practical measurements of others e.g. dry unit weight from bulk unit weight.
- The water content of a soil sample can be determined by any one of the following methods:
    a) Oven drying method
    b) Torsion balance method
    c) Pycnometer method
    d) Sand Bath method
    e) Alcohol method
    f) Calcium carbide method
    g) Radiation method

a) Oven drying method:

- The oven drying method is a standard laboratory method. This is very accurate method.
- The soil sample is taken in a small, non-corrodible, air tight container.

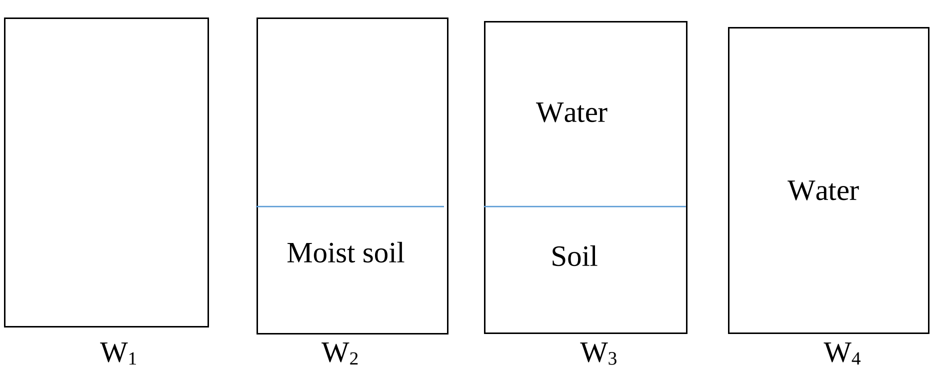
- The soil sample in container is then dried at temperature of 105-110^{o}C for 24 hour in laboratory. Above 110 ^{o}C, water of crystallization may lost.
- Water crystallization is the water in the molecular structure.
- For soil containing significant amount of organic matter, a temperature of 60^{o}C to 80^{o}C is recommended.
- If W_1 = weight of container, W_2 = wt. of container + wt. of moist soil and W_3 = wt. of container + wt. of dry soil, then water content is given by:

$$w = \frac{Ww}{Ws} = \frac{W2-W3}{W3-w1}$$

b) Pycnometer method:

- A pycnometer is a glass jar of about 1 liter capacity and fitted with a brass conical cap means of a screw-type cover. The cap has a small hole of 6mm dia. At its apex.

- The pycnometer meter method for determination of water content can be used only if the specific gravity of solid (G) particle is known.
- First the weight of empty pycnometer is determined (W_1) in the dry condition. Then the sample of moist soil, is placed in the pycnometer and its weight with the soil is determined (W_2).
- The remaining volume of the pycnometer is then gradually filled with distilled water or kerosene. The weight of pycnometer, soil and water is obtained (W_3).
- Lastly the bottle is emptied, thoroughly cleaned and filled with distilled water or kerosene, and its weight taken (W_4).



- $W_w = W_2 - W_1 - W_S$

$$w = \frac{W_W}{W_S} = \frac{W_2 - W_1 - W_S}{W_S}$$

$$W_4 - W_1 = W_3 - W_1 - W_S + \frac{W_s\,\gamma_w}{G_s\,\gamma_w}$$

$$W_S = [(W_3 - W_4) \times G_S] / G_S - 1$$

$$w = [(W_2 - W_1) / [\{(W_3 - W_4).G_S\} / (G_S - 1)]] - 1$$

$$w = [\frac{W_2 - W_1}{W_3 - W_4} \times \frac{(Gs - 1)}{Gs} - 1]$$

- Tish method is more suitable for cohesion less soil as in case cohesive soil removal of entrapped air is difficult.

ii) Specific gravity of soil particles:

- We know $G = \frac{\gamma_s}{\gamma_w} = \frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}$
- If $V_s = V_w$, $G = \frac{W_s}{W_w} = \frac{M_s \times g}{M_w \times g} = \frac{M_s}{M_w}$
- Therefore Sp. Gravity of a soil particle is the ratio between mass of soil solid to mass of equivalent volume of water at same temperature.

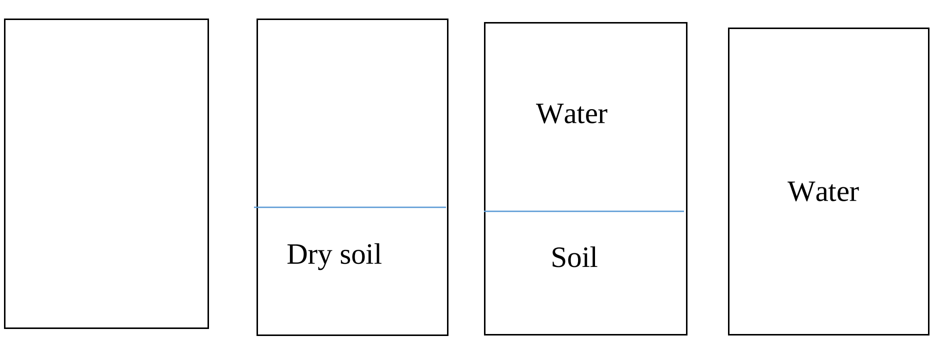
Determination of sp. Gravity:

Sp. gravity of soil is determined by 2- methods:

a) Density bottle method
b) Pycnometer method

Density bottle method is more accurate and suitable for all type of soil. But pycnometer method is suitable for coarse grained soil.

a) Density bottle method:



M_1 M_2 M_3 M_4

$$G = \frac{M_s}{M_w} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

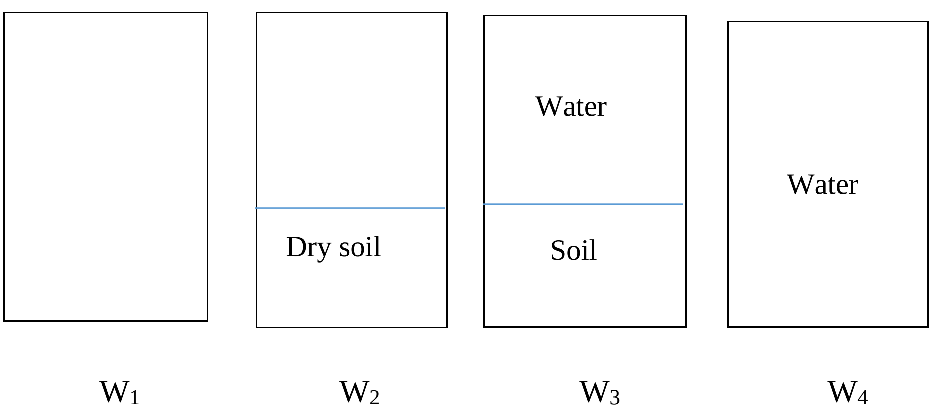
Procedure:

- A clean and empty density bottle is taken and weighted. Let the mass be M_1.
- Again the density bottle is poured with some dry soil whose sp. gravity is to determined and the whole assembly is weighted. Let the mass be M_2.
- Again the whole assembly is completely filled with water and weighted. Let the mass be M_3.
- Again the density bottle is cleaned and filled with water alone and weighted. Let the mass be M_4.
- Then, the sp. gravity $G = \frac{M_s}{M_w} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$
- When kerosene is used in case of water, then the sp. gravity of soil $G = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \times G_k$.

  Where G_k = sp. gravity of kerosene.
- Generally sp. gravity test is carried out on 27℃.
- Then sp. gravity of soil at t℃ = $G_{t^oC} = \frac{G_{27℃} \times \gamma \text{ of distilled water at } 27℃}{\gamma \text{ of distilled water at } t℃}$.

b) Pycnometer method:

- This method is same as pycnometer method of water content determination with the difference that here dry soil sample is taken instead of moist soil sample as was taken in water contents determination.



Sp. gravity $G = \frac{\text{wt. of solid}}{\text{wt. of equivalent volume of water}}$

Wt. of solid = $W_2 - W_1$

Wt. of equivalent volume of water = $(W_4 - W_1) - (W_3 - W_2)$

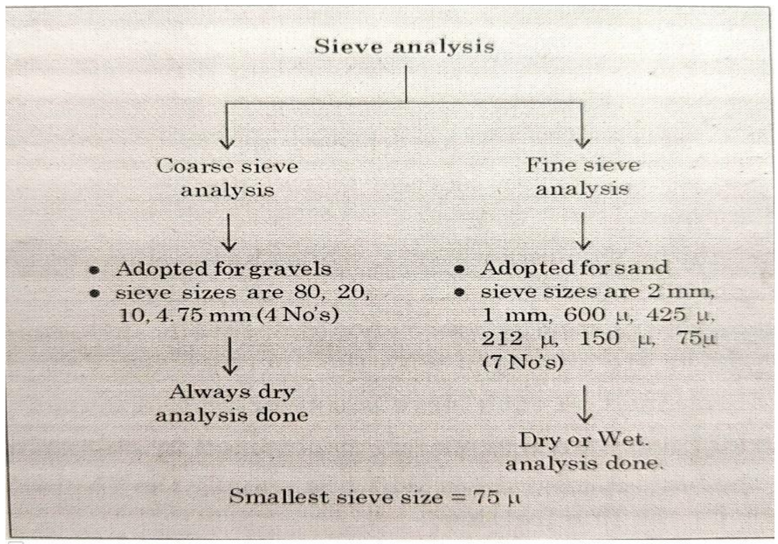
$$G = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}$$

- Sp. gravity values are generally reported at 27°C.

iv) Particle size distribution:

- The classification of soils according to their size is known as particle size distribution.
- It is done by 2-methods.
  a. Sieve analysis
  b. Sedimentation analysis for particles of size less than 75µ

Particle Size	Type of Soil		
> 300 mm	Boulder		
300 – 80 mm	Cobbles		
80 mm — 4.75 mm	gravel		Coarse Grained soil
4.75 mm — 2 mm	coarse sand	Sand	
2 mm — 0.475 mm	medium sand		
0.475 mm — 75 µ	fine sand		
75 µ — 2 µ	silt		Fine Grained soil
less then 2 µ	clay		



a) Sieve analysis:

- It is done for a soil which size lies more than 75µ.
- In this process sieves are places one over another in decreasing order of their aperture size.
- The sieve analysis is of 2 types
  i. Coarse sieve analysis; which consist of sieving of soil through 40mm, 20mm, 10mm and 4.75mm IS sieve.
  ii. Fine sieve analysis; which consist of sieving of soil though 2mm, 1mm, 600µ, 425µ, 212µ, 150µ, 75µ IS sieve.

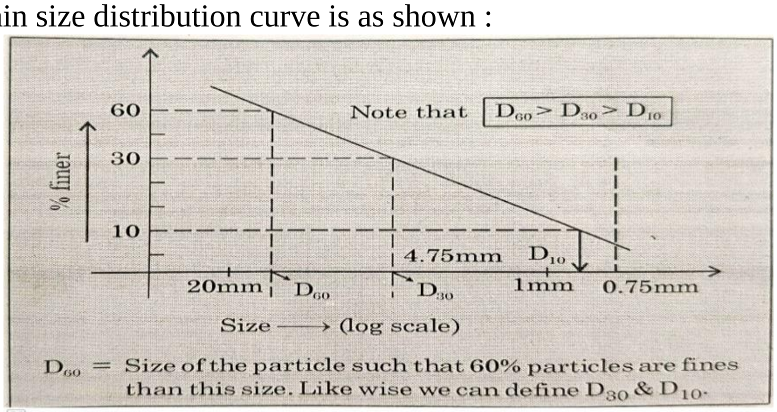
Procedure of sieve analysis:

- The soil sample is tested is dried, lumps are broken if necessary, and the sample is pass through the series of sieves by shaking.

- The fraction retained on and passing 2mm IS sieve are tested separately.
- An automatic sieve shaker, run by an electric motor, may be used; about 10 to 15 minutes of shaking is considered as adequate.
- Larger particles are caught on the upper sieves, while the smaller ones is filter through to be caught on one of the smaller underling sieves.
- The material retained on any particular sieve should naturally include that retained on the sieves on the top of it, since the sieves are arranged with the aperture size decreasing from top to bottom.
- The weight of the material retained on each sieve is converted to a percentage of the total sample.
- The percentage material finer than a sieve size is obtained by subtracting this from 100.
- The materials passing the bottom most sieve, which is usually 75µ sieve, is used to conduct the sedimentation analysis for the fine fraction.
- If the soil is clayey in nature the fine fraction cannot be easily passed through the 75µ sieve in dry condition.
- In such case, the material is to be washed through it with water, until the wash water is fairly clean.
- The material which passes through the sieve is obtained by evaporation. This is called wet sieve analysis, may be required in case of cohesive granular soil.
- The resulting data are conventionally presented as a grain size distribution curve plotted on semi log co-ordinates, where the sieve size is on a horizontal logarithmic scale, and the percentage by weight of size smaller than a particular sieve size is on the vertical arithmetic scale.
- Logarithmic scales for the particle diameter give a very convenient representation of the sizes because a wide range of particle diameter can be shown in a single plot.

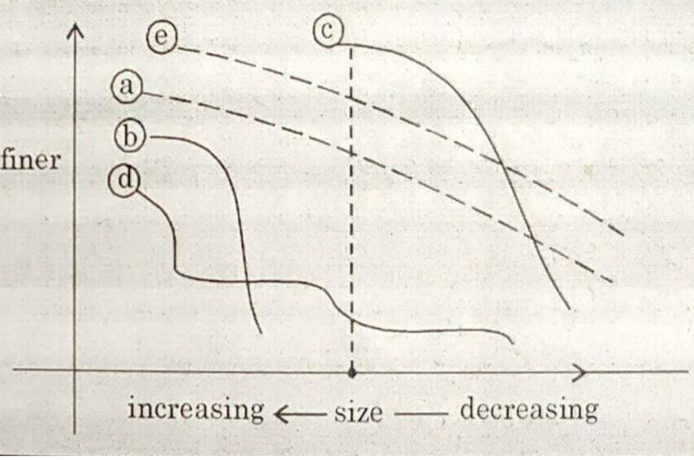
Sieve size	Total wt. of soil taken	Wt. retained on a particular sieve (gram)	Cumulative wt. retained	Cumulative %retained	% finer = 100 – cumulative %retained

- The resulting grain size distribution curve is as shown :



- Actually, sieve size is assumed to correspond to size of particle.
- The various types of curves obtained in sieve analysis are classified as follows:

Curve (a) is Well graded
Curve (b) is Poorly/uniformly graded coarse soil
Curve (c) is Poorly graded fine soil
Curve (d) is gap graded soil
Curve (a) and (e) are similarly graded soil.



- Well graded means soils of all sizes are present.
- Poorly graded/uniformly graded means soil of predominantly one size is only present.
- Gap graded means some of the soil particle size are missing.
- Position of the curve indicates **type of soil** whereas shape of the curve indicates **gradation**.
- As the slope of the curve decrease – gradation increase.
- D_{10} = effective size of Particle i.e. particle size which if present alone will cause the same effect as is caused by the soil.
- Grain size distribution curve is used to find out the following shape parameters.
    1. Co-efficient of uniformity (C_u) = $\frac{D_{60}}{D_{10}}$
    2. Co-efficient of curvature (C_c) = $\frac{D_{30}^2}{D_{60} \times D_{10}}$

If, $C_u = 1$ – Soil is perfectly uniformly graded. (Curve will be vertical)

$C_u > 4$ – well graded gravel

$C_u > 6$ – well graded sand

$1 \leq C_c \leq 3$ – well graded soil

- For well graded sand, $C_u > 6$, $1 \leq C_c \leq 3$
- For well graded gravel, $C_u > 4$, $1 \leq C_c \leq 3$

Larger the value of C_u, larger is the range of particles in soil.

Sedimentation analysis: -

- The soil particles less than 75µ size can be further analyzed for the distribution of various grain size of the order of silt and clay by 'sedimentation analysis' or 'wet analysis'.
- The soil fraction is kept in suspension in a liquid medium, usually water.
- The particles descend at velocities, related to their size, among other things.
- The analysis is based on **Stokes law.**
- As per this law, if a single sphere is allowed to fall in an infinite liquid medium without interference, its velocity first increase under the influence of gravity, but soon attain a constant value.
- This constant velocity, which is maintained indefinitely unless the boundary condition change, is known as the **terminal velocity.**
- Coarser particles tend to settle faster than finer ones.
- By Stokes law, the terminal velocity of the spherical particle is given by:
- V = $\frac{(\gamma_s - \gamma_l)D^2}{18\mu}$, where γ_s = unit weight of soil particles in N/m^3

  γ_l = unit weight of liquid in N/m^3, D – diameter of particles in meter, μ- dynamic viscosity in N.s/m^2. then V will be in m/s.
- We can also write V = $\frac{(G-1)gD^2}{18\vartheta}$, where $\vartheta -$ is kinematic viscosity in m^2/s and G- sp. gravity, $\vartheta = \frac{\mu}{\rho}$, unit- m^2/sec.
- Stocks law is considered valid for particle diameters ranging from 0.2 to 0.0002 mm.

Procedure of sedimentation analysis:

- About 50gm of dry soil sample passing through 75µ IS sieve is weighted accurately and is taken in a porcelain dish.
- About 50cc of dispersing agent is added to the soil sample.
- Then some quantity of distilled water is added to form a soil slurry and it is gently stirred using glass rod.
- The content of dish is then transferred to the cup of high-speed stirrer.
- Then the soil slurry is transferred to a 1000ml sedimentation jar.
- Then some water is added to the sedimentation jar, to make it 1000ml.
- Then the mouth of jar is close tightly with the pam of hand and jar is inverted several times to ensure uniform distribution of soil particles throughout the jar.
- Then jar is placed in a level surface and stope watch is started simultaneously.

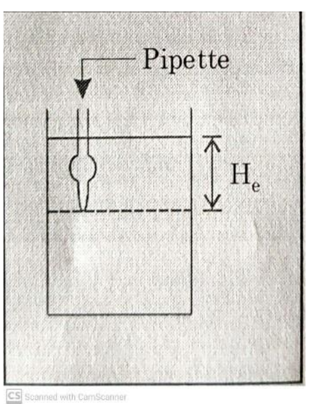
- At the end of different time intervals, t minutes (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.) the size D mm of largest particle still in suspension at depth H_e cm and % finer N is determined by using hydrometer method or pipette method.

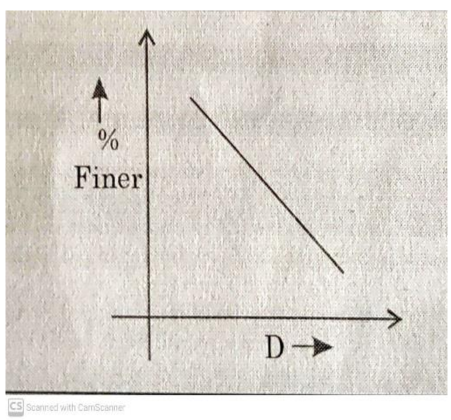
i. Pipette method:

- It is a laboratory method. A pipette, sedimentation jar, and a number of sampling bottles are necessary for the test.
- The method consists in drawing off 10ml samples of soil suspension by means of the sampling pipette from a standard depth of 10cm (i.e. H_e = 1cm) at various time intervals after the start of sedimentation (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.).
- The pipette should be inserted about 20 seconds prior to the chosen instant and the process of the sucking should not be take more than 20 seconds.
- Each of the samples taken is transferred to a sampling bottle and dried in an oven. The concentration of all particles are present in a particular volume at time t = 0.
- If after time 't' sample is taken out in pipette from height H_e then all particles having settling velocity greater than $\frac{He}{t}$ will have settle below height H_e.
- At this height at time 't' sample collected will have the same concentration of particles of settling velocity less than $\frac{He}{t}$ as was there in the original soil suspension.
- If settling velocity of particle size D = $\frac{He}{t}$, then

  % finer than D = $\left(\frac{W_D}{\frac{W_d}{V}} \times 100\right)$%, where W_D = wt. of soil particles finer than size Dmm per ml, still is suspension at depth H_e at elapsed time interval t min., W_d = wt. of soil particles taken for sedimentation analysis, V = vol. of soil suspension.

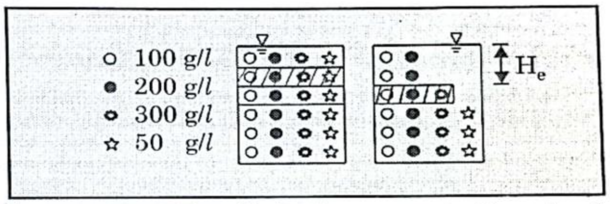
  Now, $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{He}{t}$, find 'D' from this formula corresponding to various time t and plot grain size distribution curve.





Notes: In pipette method, sample s collected from height H_e at various time intervals i.e. H_e is fixed.

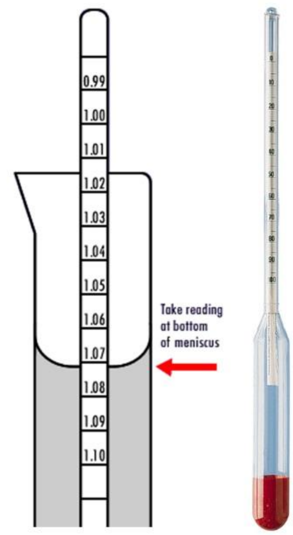
Note: • Let there be 4-types of particles in the sample. Let in the original soil suspension before the start of sedimentation the concentration of various particles be as shown below



- If particle has settling velocity greater than H_e/t, then concentration of soil suspension collected at height H_e will be 600 g/l.

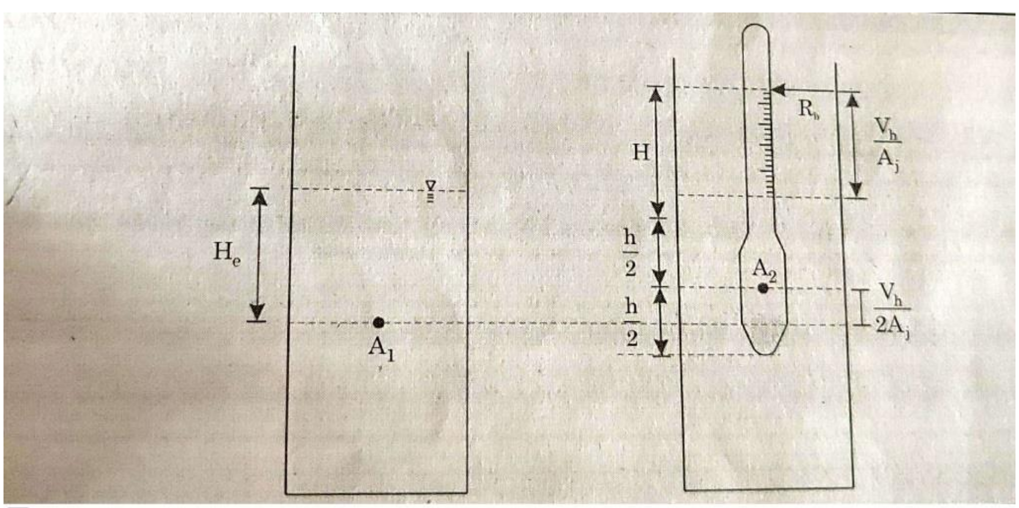
$$\text{Thus, \% finer than size} \star = \frac{\text{concentration of sample collected at time t from height } H_e}{\text{concentration of soil from original soil suspension}} \times 100 = \frac{600}{650} \times 100$$

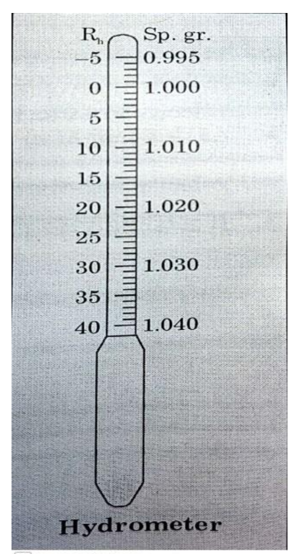
- If particle ☆ has size D, then % finer than size D is $\frac{600}{650} \times 100$, where D is given by $\frac{H_e}{t} = \frac{(\gamma_s - \gamma_w)D^2}{18\mu}$



ii) Hydrometer method:

- The hydrometer method differs from the pipette analysis in that the weights of solids per ml in the suspension at a chosen depth at a chosen instant of time are obtained indirectly by reading the sp. gravity of the soil suspension with the aid of a hydrometer.
- **Hydrometer is the device which is use to measure the sp. gravity of liquids.**
- We have already discussed that % of finer than D = $\frac{wt. of\ the\ solid\ per\ cc\ at\ depth\ H_e\ at\ time\ t}{wt. of\ solid\ per\ cc\ in\ original\ soil\ suspension}$
- Where D is given by $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{He}{t}$





- Thus, point at A_1 occupies the position A_2. A_2 is the center of volume of hydrometer.

  V_h = Volume of hydrometer

  A_j = Area of jar

  h = length of bulb in hydrometer
- Volume of bulb can be thought of as the total volume of hydrometer.
- Hydrometer measures density of soil suspension at depth H_e which is given by:

  $H_e = H + \frac{h}{2} + \frac{V_h}{2A_j} - \frac{V_h}{A_j}$
- H will correspond to hydrometer reading R_h. Thus, H_e will be related to R_h.

Procedure of hydrometer method:

- The hydrometer in inserted in the soil suspension prepared in 1000ml sedimentation jar at the end of different time interval, t min. (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.) and the reading of the hydrometer i.e. R_h is noted.
- The reading of hydrometer R_h is corrected by using suitable correction. Then from the calibration chart the effective depth H_e cm. is found corresponding to hydrometer reading R_h.
- The size D mm. of largest particles still in suspension at depth H_e cm. at any elapsed time interval t min. is computed by using following equations.
- % finer than D = $\left(\frac{W_D}{\frac{W_d}{V}} \times 100 \right)$%, $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{\text{He}}{t}$

  where W_D = wt. of soil particles finer than size Dmm per ml, still is suspension at depth H_e at elapsed time interval t min., W_d = wt. of soil particles taken for sedimentation analysis, V = vol. of soil suspension.

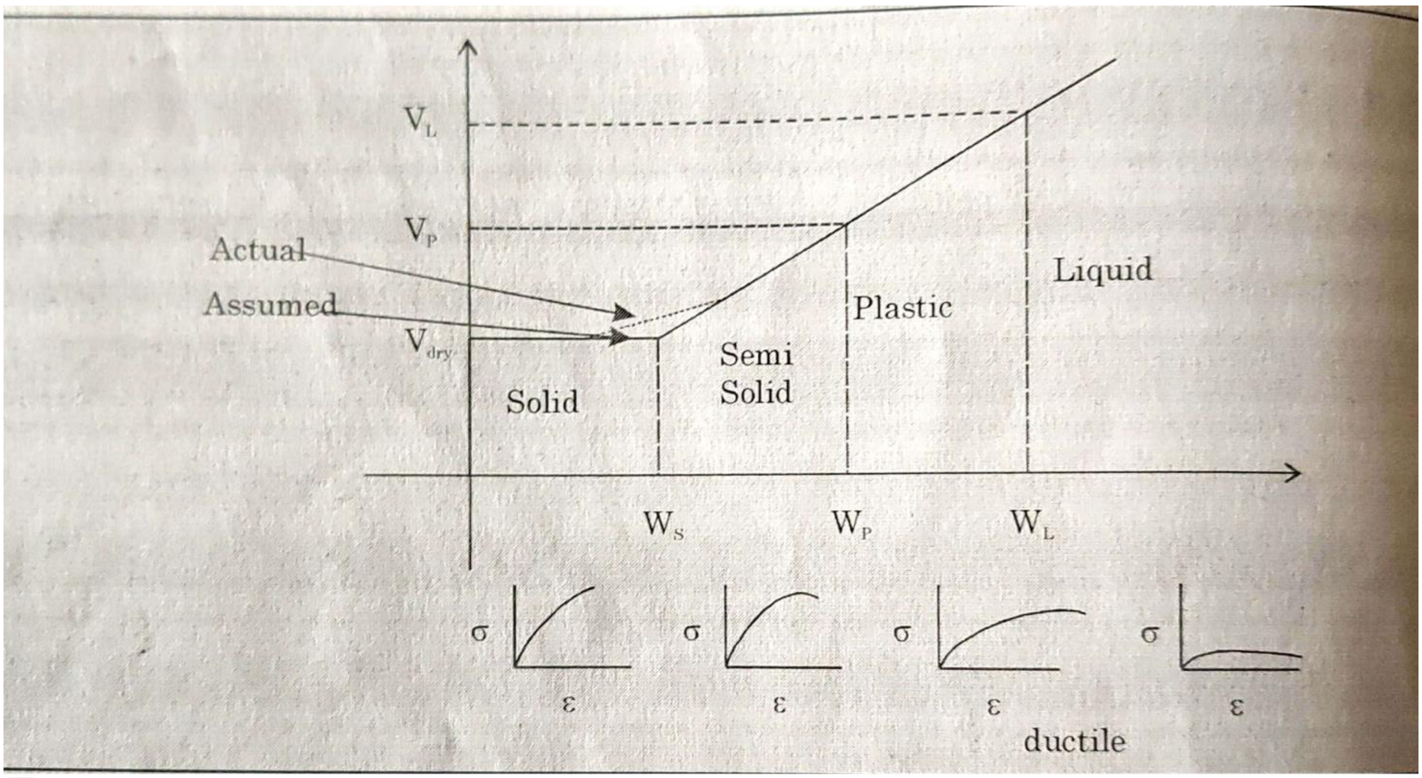
Limitation of Stocks' law:

- Stocks' law is applicable for spherical particles only. But the fine clay particles are not spherical in shape. So the concept of equivalent diameter may be used.
- It is assumed that every particle settles freely without interference, in an infinity liquid medium. The sedimentation analysis is conducted in a one-liter jar, the depth being finite; the walls of the jar could provide a source of interference to the free fall of particles near it. The fall of any particle is affected by the presence of adjacent particles; thus, the fall may not be really free. However it is assumed that the effect of these sources of interference is insignificant if suspension is prepared with about 50gm of soil per litter of water.

- The soil particles of soil specimen may have different values of specific gravity. But in computation avg. value of sp. gravity may use.

Consistency of soil:

- Consistency represents the relative ease with which a soil be deformed.
- This term is mainly used for clayey soil and is related to water content i.e. how with change in water content the consistency of soil changes.
- Atterberg classified the consistency in 4-stages. Behaviour of soil is different in different stages.
    1. Solid stage
    2. Semi solid stage
    3. Plastic stage
    4. Liquid stage



W_L = liquid limit water content

W_P = Plastic limit water content

W_S = Shrinkage limit water content

V_L = Volume of soil at liquid limit

V_P = Volume of soil at plastic limit

V_{dry} = Volume of soil at shrinkage limit

Chapter-3 ; Index properties of soil

Consistency of soils:-

→ Consistency represents the relative ease with which a soil be deformed.

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1. Solid stage
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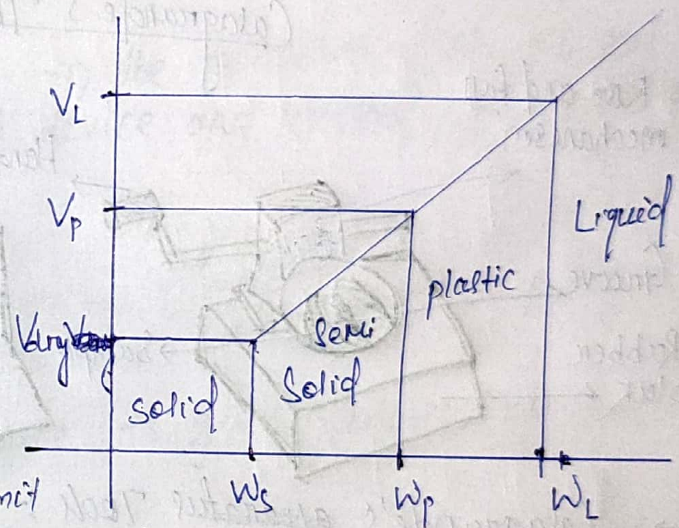


W_L = liquid limit water content
W_P = plastic limit water content
W_S = Shrinkage limit water content
V_L = Volume of soil at liquid limit
V_P = Volume of soil at plastic limit
V_{dry} = Volume of soil at shrinkage limit

From the above figure :- $\frac{V_L - V_P}{W_L - W_P} = \frac{V_P - V_{dry}}{W_P - W_S}$

Notes
Naturally existing soil has water content between W_L & W_P

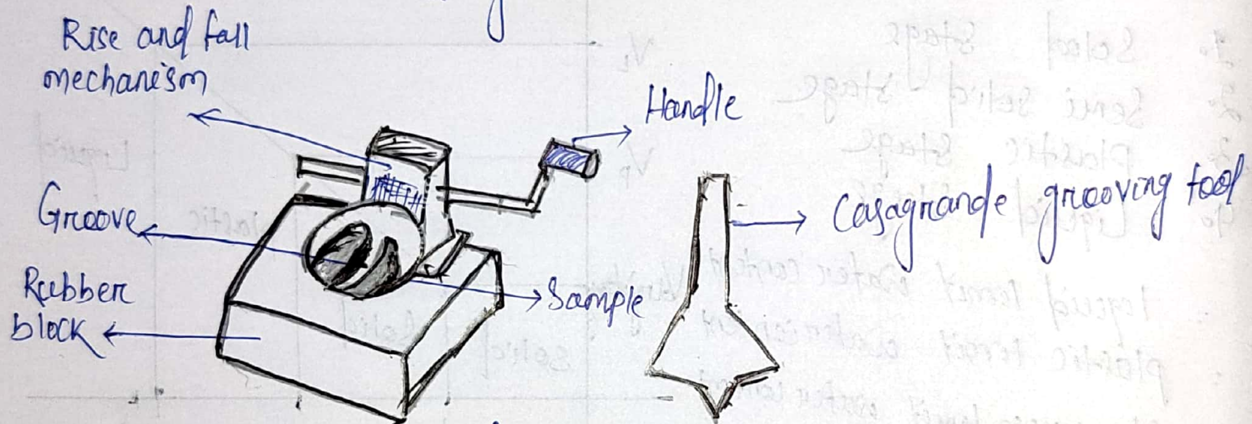
Liquid Limit

→ Min water content at which soil has a tendency to flow is called liquid limit water content.

→ 'or'

It is the minᵐ water content at which a groove of 2mm made with standard grooving tool is filled by 25 no. of blows in Casagrande's apparatus.

Casagrande's apparatus



→ Casagrande's ~~apparatus~~ Tools :-

Soil is taken and water is added and the soil put inside Casagrande's apparatus. A groove of 2mm size is cut and the apparatus is given blows over a rubber pad. No. of blows required to close the 2mm grove is noted. water content at which 25 blows closes the groove is called liquid ~~lotw~~ limit.

Determination of liquid limit :-

→ liquid limit can be determined using standard grooving tools and casagrandes apparatus.

→ The apparatus consists of a brass cup which can be raised and lowered to fall on a base by means of a cam operated by a handle. The height of fall of cup is adjusted by adjusting screw.

→ The Standard grooving tool is of 2mm wide at bottom, 13.6mm at top, 10mm depth.

Test procedure:-

(i) About 100gm. of soil sample passing through 425μ IS sieve is taken in a porceline dish.

(ii) Some quantity of distilled water is added to it and thoroughly mixed to form a soil paste of uniform colour.

(iii) The height of fall of cup of liquid limit device is adjusted to 1cm.

(iv) A portion of soil paste in the porceline dish is put in the cup of liquid limit device and levelled by means of spatula.

(v) Then a groove is made on the soil pat in the cup by using standard grooving tools.

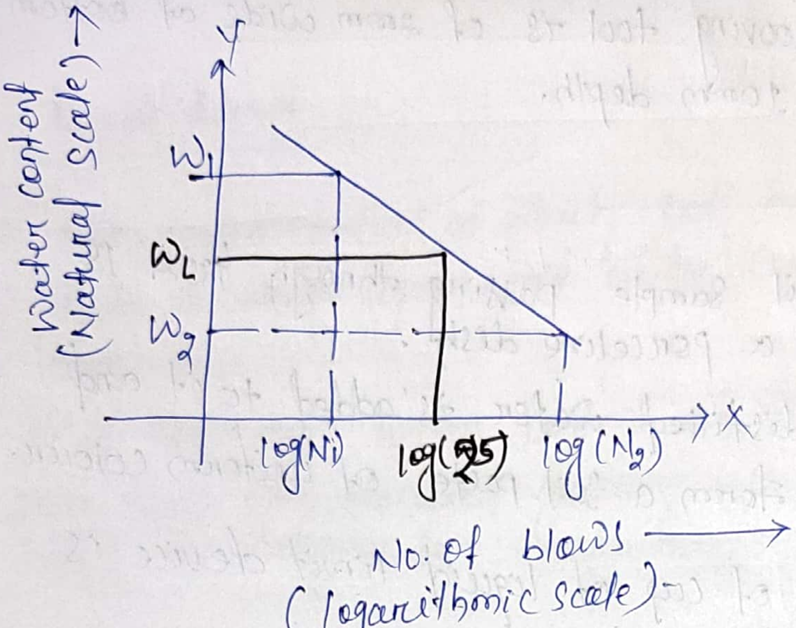
(vi) Then the liquid limit device is given blows by cam operated by handle.

(vii) The no. of blows required to close the groove is noted.

(viii) This procedure is repeated to the same soil sample by changing the water content and finding corresponding no. of blows.

(ix) Then a graph is plotted between water content and no. of blows by taking water content on ordinate (Y-axis) in natural scale against no. of blows as absisa in (X-axis) logarithmic scale.

(x) From the graph the water content corresponding to 25 no. of blows is found out which is known as liquid limit of soil.



→ Slope of this graph will give the flow index

i.e $I_f = \dfrac{W_1 - W_2}{\log(N_2) - \log(N_1)} = \dfrac{W_1 - W_2}{\log\left(\frac{N_2}{N_1}\right)}$

Plastic limit (W_P)

→ It is the min^m water content at which soil mass is in plastic stage, is called plastic limit water content.

'or'

It is the water content in which a soil mass can be rolled into a thread of 3mm dia. shows the sign of cracking.

Determination of plastic limits:-

Procedure:-

(i) About 30gm of soil sample is passing through 425μ of IS sieve is taken and some quantity of distilled water is added to it and thoroughly mixed to form a soil paste which can be rolled by the palm of hand.

(ii) A small portion of the ball is then rolled on a smooth plate in a thread of 3mm. dia and thread is looked for the sign of cracking.

(iii) If no cracks is seen, the thread is picked up and again rolled into a ball both palms to reduce water content.

(iv) The ball is then rolled to a thread of 3mm. dia.

(v) This steps are repeated until the thread shows sign of Cracking.

(vi) A portion of thread is taken for water content determination which gives the plastic limit.

Shrinkage limit (W_s)

→ It is the max^m water content at which further reduction in water content does not cause any reduction in the volume of soil sample is called shrinkage limit water content.

→ When water content is reduced below W_s, the particles are so closely spaced that volume reduction will not take place and the void space starts getting occupied by air instead of water.

→ It is the min^m water content at which soil is saturated.

Determination of shrinkage limit:-

Test procedure:-

(i) About 50gm. of soil sample passing through 425μ IS-sieve is taken in a porceline dish, distilled water is added to it and mixed thoroughly to form a soil paste of slightly flowing consistency.

(ii) The shrinkage dish (non-corrodable cup) of 45mm dia & 15mm height) is weighted after coating inner side with a thin layer of grece or oil.

(iii) The shrinkage cup will filled with the soil paste in 3 layers.

(iv) The surface of soil is levelled and outer side of cup is cleaned.

(v) The mass of wet soil pat with shrinkage cup is found and this is deducted from ~~mass~~ weight of shrinkage cup to get weight of wet soil pat (W_1)

(vi) The wet soil pat is allowed to dry in air for some time and then kept in a thermostatically controlled oven and dried for 24hrs. at 105°C - 110°C.

(vii) After oven drying the weight of oven dried soil (W_d/W_s) and vol. of oven dried sample (V_d) is found out. Then the shrinkage limit is given by;

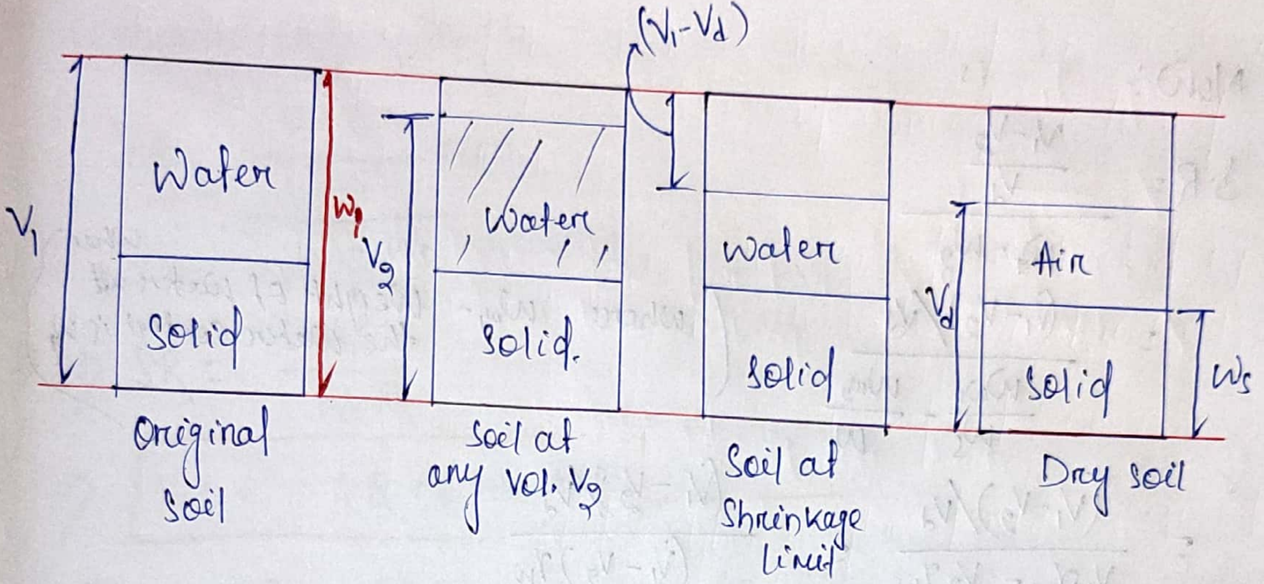
$$\boxed{W_s = \frac{(W_1 - W_s) - (V_1 - V_d)\gamma_w}{W_d}}$$

V_1 = Vol. of wet soil pat or original soil volume.

V_d = dry vol. of soil

W_1 = weight of wet soil pat or original wt. of soil.

W_d & W_s = dry wt. of soil.



Now, water content $= \frac{W_w}{W_s}$

→ At shrinkage limit, $W_w = W_1 - W_s - (V_1 - V_d)\gamma_w$

∴ $$\boxed{\text{Shrinkage limit } (w_s) = \frac{W_1 - W_s - (V_1 - V_d)\gamma_w}{W_s}}$$

Shrinkage limit can also determine by :-

$$\boxed{w_s = \frac{1}{(G_m)_{dry\ state}} - \frac{1}{G}}$$

$(G_m)_{dry state}$ - Mass sp. gravity at dry state.

Shrinkage Ratio (S.R.)

→ Shrinkage ratio is defined as vol. change in soil above shrinkage limit expressed as percentage of dry soil per unit change in water content above shrinkage limit.

$$\boxed{S.R = \frac{\left(\frac{V_1 - V_2}{V_d}\right) \times 100}{w_1 - w_2}}$$

w_1 = water content at vol. V_1
w_2 = " " at vol. V_2
V_d = Dry vol. of soil sample

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Now :-

$$S \cdot R = \frac{\frac{V_1 - V_2}{V_d}}{w_1 - w_2}$$

$$= \frac{(V_1 - V_2)/V_d}{\frac{W_{w_1}}{W_s} - \frac{W_{w_2}}{W_s}}$$ (where W_{w_1} - Weight of water when the water content is w_1)

$$= \frac{(V_1 - V_2)/V_d}{\frac{V_1 \gamma_w - V_2 \gamma_w}{W_s}} = \frac{(V_1 - V_2)/V_d}{\frac{(V_1 - V_2)\gamma_w}{W_s}}$$

$$\Rightarrow S \cdot R = \frac{(V_1 - V_2)}{V_d} * \frac{W_s}{(V_1 - V_2)\gamma_w}$$

$$= \frac{W_s}{V_d \gamma_w}$$

$\therefore$ $\boxed{S \cdot R = \frac{\gamma_d}{\gamma_w}}$

$$\boxed{S \cdot R = \frac{\text{Dry density}}{\text{Density of water}}}$$

Volumetric Shrinkage (V.S)

$$\boxed{V \cdot S = \frac{V_1 - V_d}{V_d} \times 100}$$

→ It is the reduction in vol. of soil mass expressed as %age of its dry vol.

$$\therefore S \cdot R = \frac{\frac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2}$$

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we also can write

$$S.R = \frac{\frac{V_1 - V_d}{V_d} \times 100}{W_1 - W_s}$$

$$\Rightarrow S.R = \frac{V.S}{W_1 - W_s}$$

$$\Rightarrow \boxed{V.S = S.R * (W_1 - W_s)}$$

where W_s = Shrinkage limit or Shrinkage limit water content

Indices:-

→ The different types of indices are :-

(i) plasticity Index (I_P)
(ii) Liquidity Index (I_L)
(iii) Consistency Index (I_C)
(iv) Flow Index (I_f)
(V) Density Index (I_d)
(Vi) Toughness Index (I_t)

(i) plasticity Index (I_P) :-

→ It is difference bet^n liquid limit (L.L) & plastic limit (P.L) of a Soil

I_P = L.L − P.L

$$\boxed{I_P = W_L - W_P}$$

(ii) Liquidity Index (I_L)

→ It is the ratio bet^n difference of natural water content & plastic limit to the plasticity index.

$$\boxed{I_L = \frac{W - W_P}{I_P}} \Rightarrow \boxed{I_L = \frac{W - W_P}{W_L - W_P}} \quad (\because I_P = W_L - W_P)$$

When $I_L = 0$

$\Rightarrow \frac{W - W_P}{I_P} = 0$

$\Rightarrow W = W_P$ ($\because$ natural water content is equal to plastic limit water content)

So the soil is in plastic state

When $I_L = 1$

$\Rightarrow \frac{W - W_P}{I_P} = 1$

$\Rightarrow W - W_P = W_L - W_P$

$\Rightarrow W = W_L$

So, the soil is in liquid state.

Consistency index (I_C) :-

$\Rightarrow$ It is the ratio betn difference of liquid limit & natural water content and plasticity index.

$$\boxed{I_C = \frac{W_L - W}{I_P}}$$

$$\boxed{I_C = 1 - I_L}$$ $\left(\because 1 - \frac{W - W_P}{W_L - W_P} = \frac{W_L - W}{I_P} = I_C\right)$

When $I_C = 0$

$\Rightarrow W_L = W$

so, the soil is in liquid state.

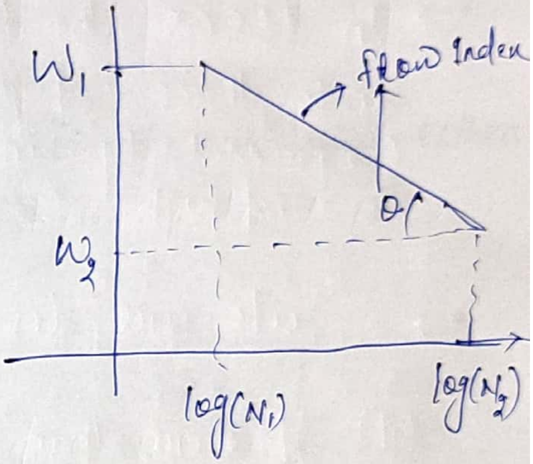
When $I_c = 1$, then $\frac{w_L - w}{I_p} = 1$

$\Rightarrow w_L - w = w_L - w_p$

$\Rightarrow w = w_p$

∴ So, the soil is in plastic state.

Flow Index (I_f) :-



→ It is the slop of the flow curve

$$I_f = \frac{w_1 - w_2}{\log(N_2) - \log(N_1)}$$

∴ $$\boxed{I_f = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}}$$

Where N_1 = No. of blows corresponding to water content w_1

N_2 = No. of blows corresponding to water content w_2.

Toughness Index (I_t) :-

→ It is the ratio betn plasticity index to flow index.

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

Density Index (I_d) :-

→ It is the ratio betn diffence of void ratio of a soil sample at it's loosest state (e_{max}) & natural (e) state to, difference of void ratio of that sample at its loosest state (e_{max}) & denser state (e_{min}).

$$\boxed{I_d = \frac{e_{max} - e}{e_{max} - e_{min}}}$$

Chapter-4 Classification of soil :-

4.1

→ A soil classification means sorting of soil into group, which will show similar behaviour.

→ Classification is done on the basis of grain size distribution and plasticity chart.

→ For classifications we use symbols, which are used either in suffix or prefix.

Soil type	Prefix	Subgroup	Suffix
Gravel	G	Well graded →	W
Sand	S	Poorly graded →	P
Clay	C	Clayey →	C
Silt	M	Silty →	M
Organic	O	liquid limit < 35 →	L (low compressibility)
peat	Pt	35 < Liquid limit < 50 →	I (intermediate ")
		liquid limit > 50 →	H (High ")

Ex:- GW → Well graded gravel
GM → Silty gravel
CH → Clay of high compressibility.

$\boxed{\text{Soil classification}}$

(1) Unified Soil Classification System (USCS)

(2) American Association of State highway and transport officials system. (AASHTO)

(3) Indian Standard Soil classification System (ISSCS)

Indian Standard Soil Classification System:-

→ Indian Standard Soil Classification System (ISSCS) adopted by Bureau of Indian Standards.

→ In ISSCS the fine grained soil are subdivided into three categories of low, medium and high Compressiblity instead of two categories of low and high compressibility in USC system.

→ Soils are divided into three broad devisions:-

(1) If more than 50% of soil retained on 75μ IS seive or above; the soil is essentially said to be Coarse-grained soil.

(2) If more than 50% of soil passes 75μ sieve then the soil is said as fine-grained soil.

(3) If the soil is highly organic and contains a large percentage of organic matter and particles of decomposed vegetation, it is kept in a separate category marked as (P_t)

→ In case of course grained soil if more than 50% of course fraction is passing 80mm sieve & retained on 4.75mm sieve, then the soil is essentially said to be gravel.

→ If more than 50% of course fraction is passing 4.75mm sieve and retained on 75μ sieve, then the soil is essentially said to be sand.

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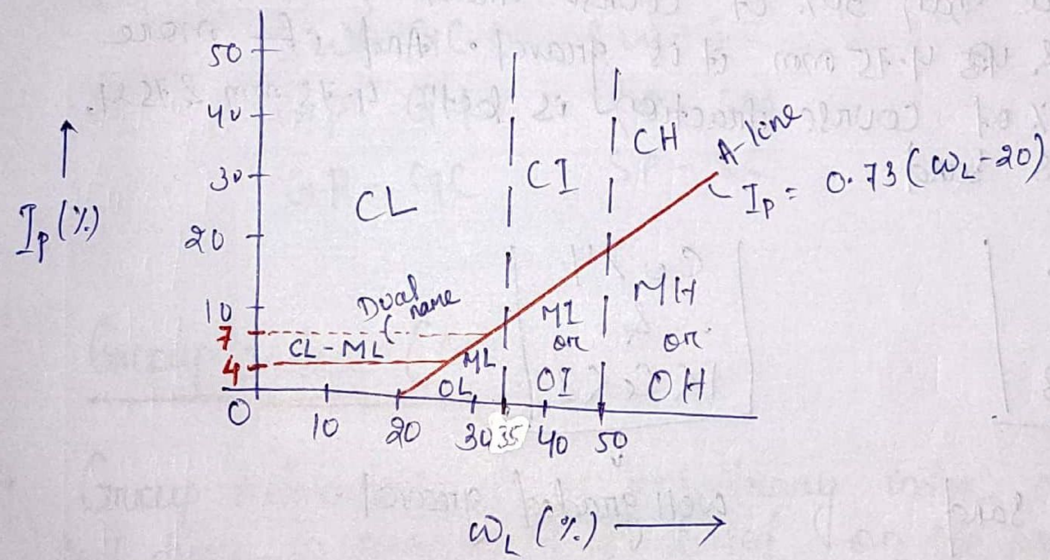
4.2 ~~classification~~

Plasticity Chart:-

→ It is based on the values of liquid limit (w_L) and plasticity index (I_P), is provided in ISSCS to aid classification.

→ Depending on the point in the chart, fine soils are divided into ~~dg~~ clay (C), silt (M) or Organic soils (O).



Classification of course grained soil:-

(a) When fines are less than 5% of weight of the total soil (Fine means → passing through 75 μ sieve)

(b) fines > 12% of weight

(c) fines betn 5 to 12%

(a) fines < 5%

→ The Various name gives as

GW → well graded gravel
GP → Poorly graded gravel
SW → well graded sand
SP → poorly graded sand.

→ If more than 50% of course fraction is betⁿ 80mm & 4.75 mm it is gravel. And if more than 50% of course fraction is betⁿ 4.75 mm & 75μ, then it is sand.

→ $\boxed{C_u > 6 \ \& \ 1 \le C_c < 3}$ → well graded sand

$\boxed{C_u > 4 \ \& \ 1 < C_c < 3}$ → well graded gravel.

(b) Fines > 12%

→ The Various name gives as

GM → silty gravel
GC → clayey gravel
SM → silty sand
SC → clayey sand.

→ In this case for the fines liquid & Plastic Limit test is done and I_P & W_L calculated and plot on plasticity chart.

→ If the points gets ploted above A-line, the fines are clayey in nature & if points get ploted below A-line fines are silty in nature.

(C) Fines are betn 5-12%

→ When fines are betn 5-12%, dowels symbols are used. And the various name gives as:-

GW-GM	SW-SM
GW-GC	SW-SC
GP-GM	SP-SM
GP-GC	SP-SC

Group Index (G.I)

→ Group index value is an arbitrary index assigned to the soil types in numerical eqn based on the percent fines, liquid limit & plasticity index.

→ The G.I value of soils vary in the range of 0 to 20

→ The higher the G.I value weaker the soil subgrade hence grater thickness of pavement required.

$$G.I = (0.2a + 0.005ac + 0.01bd)$$

$a = P - 35 \not> 40$ (expressed as whole no betn 0-40)

$b = P - 15 \not> 40$ (" " " 0-40)

$c = W_L - 40 \not> 20$ (" " " 0-20)

$d = I_p - 10 \not> 20$ (" " " 0-20)

P = %age finer (%age of soils passing from 0.075 mm sieve)

W_L = Liquid limit

I_P = Plasticity index = $(W_L - W_P)$

Chapter-5 permeability and seepage

Introduction

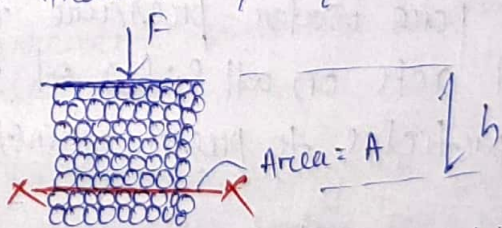
→ There are 2-types of interaction betn soil particles
(a) chemical interaction
(b) physical interaction

→ physical interaction is studied with help of effective stress; and effective stress concept was developed by Terzaghi and is applied to fully saturated soil and repeteds 3-types of stresses:

(a) Total stress
(b) pore water pressure
(c) Effective stress

(a) Total stress

→ Total stress (σ) on a plane with in a soil mass is the force per unit area of soil mass transmitted in normal direction across the plane.



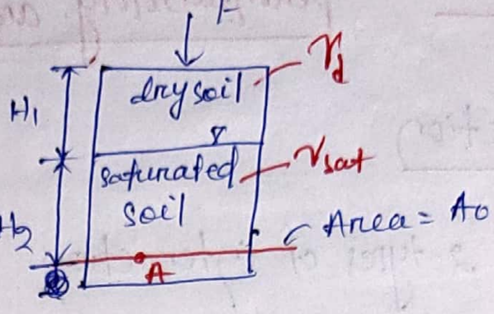
Total stress (σ) = $\frac{F}{A} + \frac{W}{A}$

where W = weight of the soil mass above the X-X plane in 'h' height.

$\Rightarrow \sigma = \frac{F}{A} + \frac{W * h}{A * h} = \frac{F}{A} + \left(\frac{W}{Vol}\right) * h$

$\Rightarrow \boxed{\sigma = \frac{F}{A} + \gamma * h}$

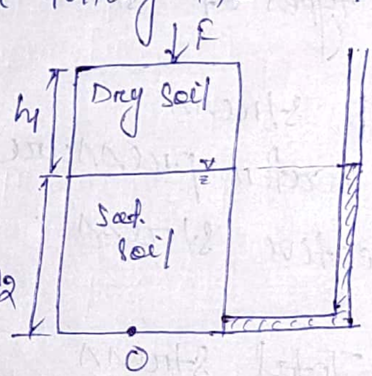
$$\boxed{\sigma_A = \frac{F}{A_0} + \gamma_d H_1 + \gamma_{sat} H_2}$$



→ Total stress is a physical parameter which can be measured by suitable arrangement, such as by pressure cell.

(b) pore water pressure (U)

→ It is the pressure of water filling the void space solid particles.



$$\boxed{U = \gamma_w h_2}$$

(A case when force 'F' has been acting on soil since long ago).

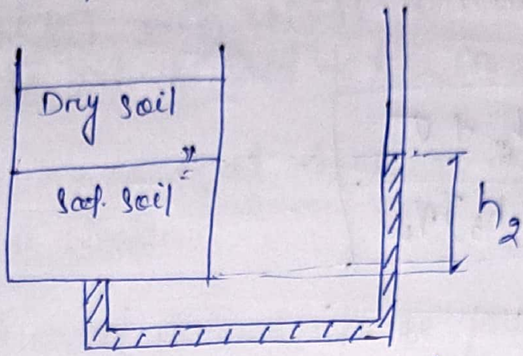
→ pore water pressure is also called as neutral stress because it acts on all sides of the particles, but does not cause particles to press against adjacent particles.

→ Pore water pressure at any point can be measured by inserting a stand pipe at the point under investigation and observing the height upto which the water rises in the stand pipe. (Thus it is a measurable quantity).

→ pore water pressure is measured using a piezometer or stand pipe.

Effect of External load on pore water pressure.

Case-I (pore water pressure before any loading is applied)



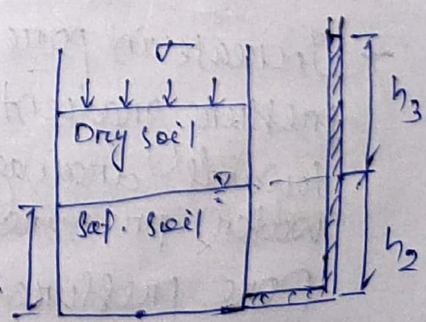
→ pore water pressure initially before the application of loading is constant at value governed by the position of water table ($\gamma_w h_2$). This initial pressure is called static pore water pressure. As shown in the figure above, the water level rises in the stand pipe upto the same level as ground water Table.

Case-II (pore water pressure immeadiatly after the application of external stress.)

→ As the external load is applied, soil particles immediately tries to occupy a new position closer together.

→ But as water and soil are incompressible and if soil is laterally confined, this rearrangement will not be possible without expulsion of pore water.

→ Thus as water is resisting the particle rearangement, pore water pressure is increased above the static ~~water~~ pore water pressure.



→ At $t=0$; i.e. just after application of load all the external load is taken up by pore water. This pore water pressure above static value is called excess pore water pressure; whose magnitude is equal to the external applied stress (σ).

Total pore water pressure = Static pore water pressure + Excess pore water pressure.

at $t=0$

$$\boxed{\begin{array}{l} U = \gamma_w h_2 + \sigma \\ \hline U = (h_2 + h_3)\gamma_w \end{array}}$$

where;

$$h_3 = \frac{\sigma}{\gamma_w}$$

→ Initially no part of the stress increment is taken up by the soil particles. This is possible for a fully saturated soil which is laterally confined such that volume changes occur entirely due to deformation of soil in vertical direction. (This cond^n will practically be met when thickness of soil layer is small compared to its area).

→ However, if lateral strain is possible than from the starting itself; some part of the stress increment is taken up by the soil particles and hence initial excess pore water pressure will be lesser than the increment of external load.

Case-III: (~~loading~~ long time after application of external load)

→ Increase in pore water pressure above static value causes pressure gradients and results in transient flow of pore water towards drainage face. This flow continues so long as the pore water pressure has not fallen down to ($h_2\gamma_w$) i.e. hydrostatic pore pressure

→ Reduction of excess pore water pressure as drainage takes place is called dissipation.

→ When the dissipation is complete, soil is said to be in drained condition.

→ prior to dissipation, with excess pore water pressure at its initial values, the soil is said to be in undrained condition.

→ Drained condition means that there is no stress induced pressure in the pore water.

→ Soil remains saturated throughout the process of dissipation.

→ Thus as pore water pressure dissipates, external forces are transferred to the soil grains, resulting into rearrangement of soil grains and increase in effective stress.

→ When all the excess pore water has dissipated, the effective stress increases by magnitude of stress increment.

→ When soil is subjected to reduction in total normal stress, soil does not rebound to the initial condition of reduced normal stress.

→ This is ~~due to~~ because particle rearrangement due to total stress increase is largely irreversible.

→ If the ~~soil~~ soil is having clay, there can be a significant expansion. As a result, pore water pressure will be reduced and excess pore water pressure will become negative.

→ Pore water pressure will gradually increase to its static value, flow taking place into the soil, effective normal stress decreases and volume increases. This process is called swelling.

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Effective Stress :-

→ Effective stress is defined as equal to the total stress (σ) minus the neutral stress or pore water pressure (U).

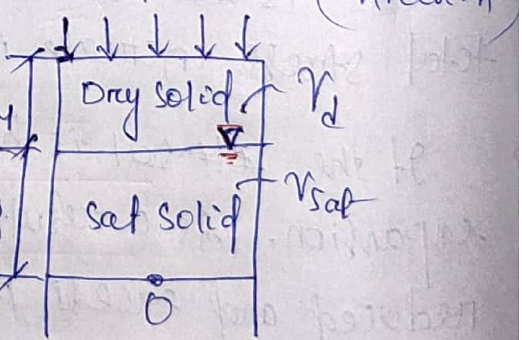
→ It is represented by symbol ($\bar{\sigma}$)

$$\bar{\sigma} = \sigma - U$$

→ Effective stress is the summation of forces transmited through the contact betn soil grains per total area of the soil mass.

→ Effective stress is not a physical parameter i.e. it can not be measure using any equipment. It can only be derive.

Calculation of effective stress :-



$$\bar{\sigma} = \sigma - U$$

$$= \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2 - \gamma_w h_2$$

$$\boxed{\bar{\sigma} = \frac{F}{A} + \gamma_d h_1 + \gamma_{sub} h_2}$$

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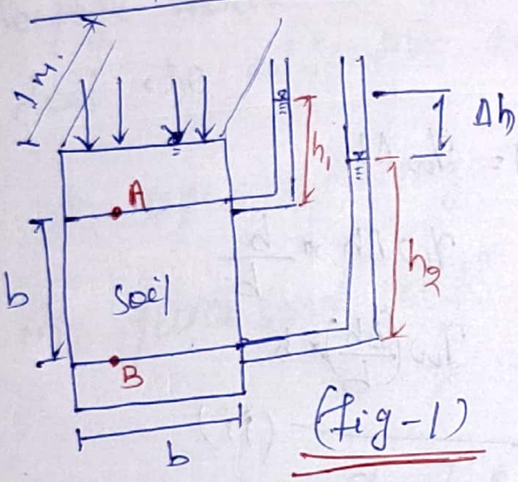
(5.4) Seepage pressure, effective stress, quick sand cond^n:-

Seepage:-

→ The process by which water flows through soil is called seepage.

(i) Downward seepage:-

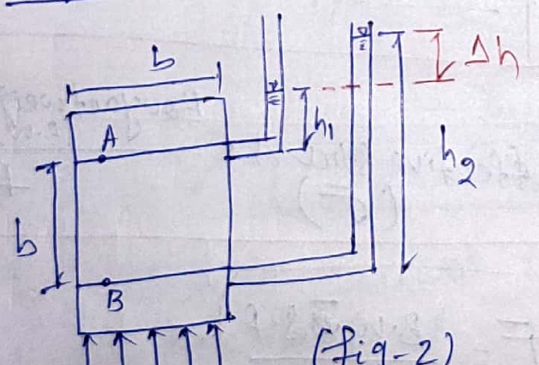


(fig-1)

Δh = loss in total head due to downward seepage in soil of length 'b'.

Crossectional area = $b * 1m.$
$= b\ m^2$

(ii) upward seepage:-



(fig-2)

Δh = loss in total head due to upward seepage in soil of length b.

→ Seepage force (S.F)

$S.F = \gamma_w *$ total head loss $*$ Area

$= \gamma_w * \Delta h * (b*1)$

$\therefore \boxed{S.F = \gamma_w \Delta h b}$

Seepage pressure (S.P) = $\frac{S.F}{Area} = \frac{\gamma_w \Delta h b}{(b*1)} = \gamma_w \Delta h$

$\therefore \boxed{S.P = \gamma_w \Delta h}$ (i)

Hydraulic gradient :- (i)

$$i = \frac{\text{head loss occured during seepage}}{\text{length over which head loss occured.}}$$

→ It is defined as head loss per length causing head loss.

$$\therefore \boxed{i = \frac{\Delta h}{b}}$$

Now seepage pressure (S.P) = $\gamma_w \Delta h$

$$\Rightarrow S.P = \gamma_w \Delta h * \frac{b}{b}$$

$$= \gamma_w \left(\frac{\Delta h}{b}\right) * b$$

$$\therefore \boxed{S.P = \gamma_w i b} \text{ ——— (ii)}$$

$$\therefore \boxed{S.P = \gamma_w \Delta h = \gamma_w i b}$$

For downward seepage ; Effective stress $(\bar{\sigma})$ = $\frac{\text{Bouyant weight (B.W) + Seepage force (S.F)}}{A}$

~~from fig~~ For upward seepage ; $\bar{\sigma} = \frac{B.W - S.F}{A}$

∴ from fig-1 & fig-2

B.W = $\gamma_{sub} * b * \text{Area}$; S.F = $(\gamma_w i b) * \text{Area}$

$$\therefore \bar{\sigma} = \frac{(\gamma_{sub} * b) * A \pm (\gamma_w i b) * A}{A}$$

$$\boxed{\bar{\sigma} = \gamma_{sub} b \pm \gamma_w i b}$$

Quick sand condⁿ:-

→ In case of upward seepage flow, if the upward seepage force becomes equal to the bouyant weight of soil the effective stress in the soil becomes zero.

→ In upward seepage flow $\bar{\sigma} = \gamma_{sub} b - \gamma_w i b$ —(1)

in quick sand condⁿ

⇒ bouyant weight = seepage force

$$\Rightarrow \gamma_{sub} b - \gamma_w i b = 0$$

$$\Rightarrow \gamma_{sub} b = \gamma_w i b$$

$$\Rightarrow \boxed{i_{cr} = \frac{\gamma_{sub}}{\gamma_w}}$$

→ The hydraulic gradient is also known as critical hydraulic gradient represented by a symbol i_{cr}.

$$\boxed{i_{cr} = \frac{\gamma_{sub}}{\gamma_w}}$$

→ When upward flow is taking place at critical hydraulic gradient, a soil such as sand looses all its shearing strength because effective stress become zero.

→ This condⁿ is called quick sand condⁿ or boiling ~~condⁿ~~ of sand because surface of sand looks as if it is boiling.

Note

→ quick sand condⁿ is not a type of sand, it is a hydraulic condⁿ

$$\therefore i_{cr} = \frac{\gamma_{sub}}{\gamma_w}$$

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

$$\Rightarrow i_{cr} = \frac{G-1}{1+e}\gamma_w \times \frac{1}{\gamma_w}$$

$$\therefore \boxed{i_{cr} = \frac{G-1}{1+e}}$$

(5.1) Concept of permeability, Darcy's Law, co-efficient of permeability :-

Permeability :-

→ permeability is the property of soil due to which water flow through the inter connected voids.

→ Coarse grained soils are regarded as highly permeable; where as fine grained soil (clay) is ~~less~~ regarded as impermeable.

Darcy's law of permeability :

Statement :-

→ According to darcy's law, for laminar flow the velocity of liquid in soil is directly proportional to the "hydraulic gradient of soil".

$\therefore$ $V \propto i$

$\Rightarrow$ $\boxed{V = Ki}$ ——— (1)

where K is the constant of proportionality known as co-efficient of permeability.

$\therefore$ $V = Ki$

$i = \frac{h}{L} = \frac{\text{head loss}}{\text{length}}$

$\Rightarrow$ $V = K \cdot \frac{h}{L}$

If $L = 1$, $h = 1$

then $V = K$ or $K = V$

So, the co-efficient of permeability is equal to the velocity of flow of liquid in a soil having a unit head loss over unit length.

Discharge $(Q) = AV$ ($\because$ A = area of crossection of soil, V = velocity of liquid in soil)

$\Rightarrow Q = A(Ki)$

~~limitati~~ $\therefore$ $\boxed{Q = AKi}$

limitation:-

- Darcy's law is only applicable for laminar flow but in practice not all the flows are laminar.

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→ For laminar flow renyold's no. is less than 2000, But in wax experiment it is found that for validity of darcy's law renyold's no. should be less than 1.

(5.2) Factors affecting permeability:-

The different factors that affect permeability are:-

(i) Void ratio
(ii) properties of fluid
(iii) size of soil particles
(iv) Soil structure
(v) presence of adsorbed water
(vi) presence of foreign matter
(vii) Degree of saturation.

(i) Void ratio:-

→ If void ratio will be more than there will be more void spaces so, the permeability will be more. So the permeability is directly proportional to void ratio.

(ii) properties of fluid:-

→ permeability is directly proportional to the density/unit weight of liquid & inversely proportional to the viscosity of the fluid.

$$\therefore K \propto \frac{\gamma}{\mu}$$

μ = Viscosity of fluid.

$$\Rightarrow \frac{K_1}{K_2} = \frac{\gamma_1/\mu_1}{\gamma_2/\mu_2} \quad \therefore \boxed{\frac{K_1}{K_2} = \frac{\gamma_1 \mu_2}{\gamma_2 \mu_1}}$$

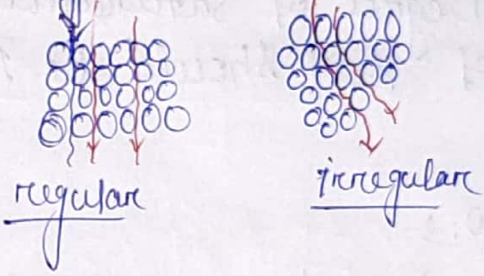
(iii) Size of the soil particle :-

→ The permeability of soil is directly proportional to square of size of soil particles.

$$\boxed{K = C * (D_{10})^2}$$

(iv) Soil structure :-

→ permeability will be more if the soils are arranged in regular manner than irregular manner.



(v) presence of adsorbed water :-

→ Due to presence of adsorbed water the permeability of soil decreases, as the adsorbed water decreased the air voids.

(vi) presence of foreign matter :-

→ Due to presence of foreign matter the void spaces get filled so, the permeability of soil decreases.

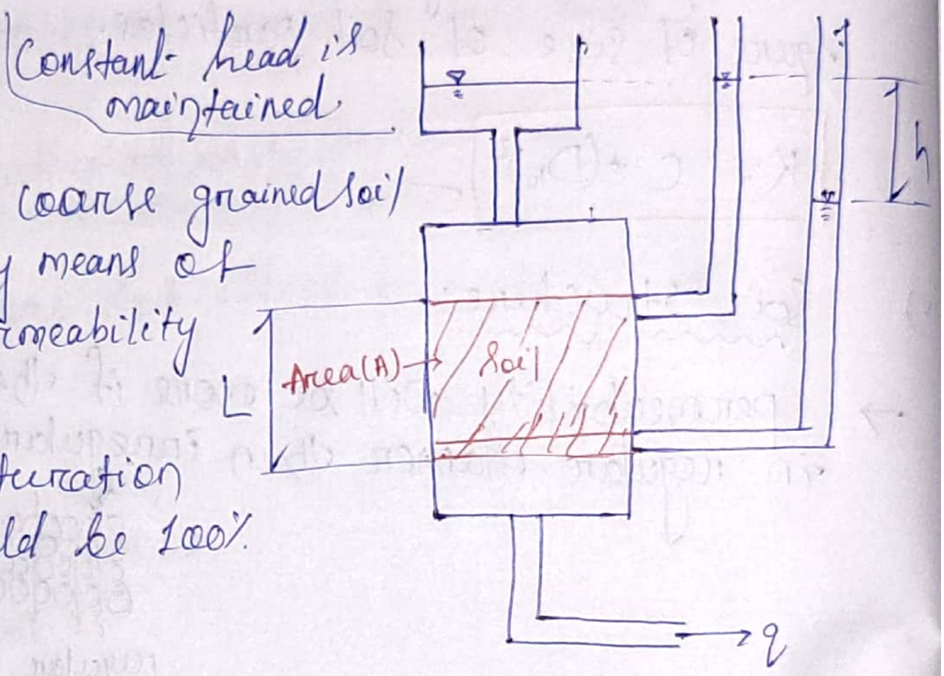
(vii) Degree of saturation :-

→ The permeability of partially saturated soil is less than that of fully saturated soil.

(5.3) Constant head permeability & falling head permeability

→ Coefficient of permeability for coarse grained soil is determined by means of Constant - head permeability test.

→ Degree of saturation of soil should be 100%.



Now; :-

consider a cylindrical soil specimen of length(L) and cross-sectional area(A) is subjected to Constant head permeability test.

Let h = head difference / head loss in length 'L'

k = Co-efficient of permeability of soil

We know Discharge $(Q) = AV$

$= Aki$

$\Rightarrow Q = Ak\frac{h}{L}$

$\Rightarrow \boxed{k = \frac{QL}{Ah}}$

L = length of soil speciment.

Now $K = \frac{QL}{Ah}$

$\therefore Q = \frac{Vol.}{time} = \frac{V}{t}$

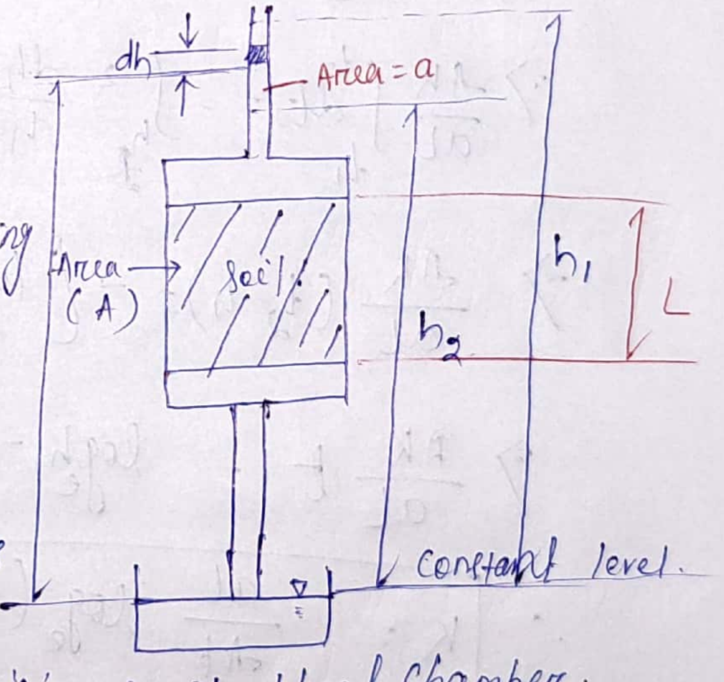
$\Rightarrow \boxed{K = \frac{VL}{tAh}}$

Falling head permeability test:-

→ For fine soils, falling head / variable head method is used.

→ Consider a soil specimen of length (L) and crossection area (A), subjected to falling head permeability test.

a = crossection area of stand pipe.



→ Here, the head of soil falls from h_1 at time instant 't_1' to h_2 at time instant 't_2' - Constant head chamber.

→ Let us consider the instant at which the head is 'h'. For very small time dt, the head fall by height dh. Let the discharge through the sample be Q & discharge in stand pipe be 'q'

$\therefore q = -\left(\frac{a \cdot dh}{dt}\right)$ —(i) $\left(\because q = \frac{Vol}{Time}\right)$

$Q = KiA$ — (ii)

$\therefore$ discharge in stand pipe = discharge in soil

$\Rightarrow -\left(\frac{a \, dh}{dt}\right) = KiA$

$\Rightarrow -\frac{a\,dh}{dt} = K \frac{h}{L} A$

$\Rightarrow a\,dh = -AK \frac{h}{L} dt$

$\Rightarrow \frac{AK\,dt}{aL} = \frac{-dh}{h}$

$\Rightarrow \frac{AK}{aL} \int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2} \frac{dh}{h}$

$\Rightarrow \frac{AK}{aL}(t_2 - t_1) = -(\log_e h_2 - \log_e h_1)$

$\Rightarrow \frac{AK}{aL} t = \log_e h_1 - \log_e h_2$ $(\because t_2 - t_1 = t)$

$$\therefore \boxed{K = \frac{aL}{At} \log_e \left(\frac{h_1}{h_2}\right)}$$

$$\boxed{K = \frac{2.303\,aL}{At} \log_{10} \left(\frac{h_1}{h_2}\right)}$$

where h_1 = initial head

h_2 = final head

t = time interval = $t_2 - t_1$

L = length of soil

(6) Compaction and consolidation :-

(6.1) Compaction :-

→ Compaction of soil is the process of increasing the unit weight of soil by forcing the soil solids into a dense state & reducing the air voids.

→ Compaction leads to increase in shear strength and helps improve the stability and bearing capacity of soil. It also reduces the compressibility and permeability of soil.

→ This is achived by applying static or dynamic loads to the soil.

→ Compaction is measured quantitatively in terms of dry unit wt. (γ_d) of the soil.

Laboratory test for compaction :-

→ There are 2-types of lab test used for compaction of soil.

(i) Standard proctor test / Light compaction test :-
(ii) Modified proctor test / Heavy compaction test :-

(i) Standard proctor test / Light Compaction test :-

Aim of the experiment :- To determine the optimum moisture content (OMC) of soil mass.

Apparatus required:-

(i) Cylindrical metal mould; ~~with~~ with detachable base plate having internal dia. of 10.16 cm, height of 11.86 cm & internal vol. of 945 cc.

(ii) Collar of height 5 cm.

(iii) Rammer of mass 2.5 kg with a height of free fall of 30.48 cm.

Procedure:-

(i) About 3 kg of dry soil passing through 4.75 mm IS sieve is taken in a tray.

(ii) Then some quantity of water is added to the dry soil. The amount of water is taken as 4% for coarse grained soil & 8% for fine grained soil for initial trial.

(iii) The computed quantity of water is added to soil in the tray and mixed thoroughly by means of hand.

(iv) Then the mass of mould with base plate is found and is taken as M_1.

(v) Then the mould is filled with some quantity of wet soil in the tray and compacted with 25 uniformly distributed blows on the surface using the standard rammer. The compacted soil should be about 1/3 of height of mould.

(vi) Then the surface of compacted soil is scratched with a knife to ensure bond with the next layer.

(vii) Then another layer (2nd layer) of soil is placed over the 1st layer in the same procedure prescribed above.

(viii) Care should be taken to see that the soil mass does not protrude more than 6mm into the collar fitted
(extend out of space)
above the cylinder.

(ix) Then the collar is removed from the top of the soil and the excess soil above the cylinder is wiped out by using a knife.

(x) Then the cylinder with wet soil mass and detachable base plate is weighted, let the mass be M_2.

(xi) Then from given vol. of cylinder the dry density of soil mass is found out as follows;

$$\text{Density } (\rho) = \frac{M}{V} = \frac{M_2 - M_1}{V_{cylinder}}$$

Then $\boxed{\rho_d = \frac{\rho}{1+w}}$

where w = water content of soil mass = % of water added to dry soil.

(xii) This procedure is repeated to the no. of soil samples by changing the value of water content.

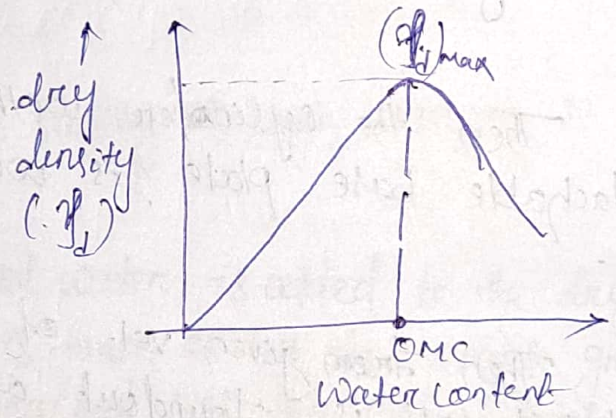
(xiii) for each sample the dry density is found out for the corresponding water content.

(xiv) Then a graph is plotted in betn water content as abscissa (x-axis) & dry density as Ordinate (Y-axis)

(xv) From the graph the optimum moisture content of soil is found out.

Optimum moisture content:- (OMC)

→ The water content corresponding to max^m dry density is called as optimum moisture content.



Conclusion:-

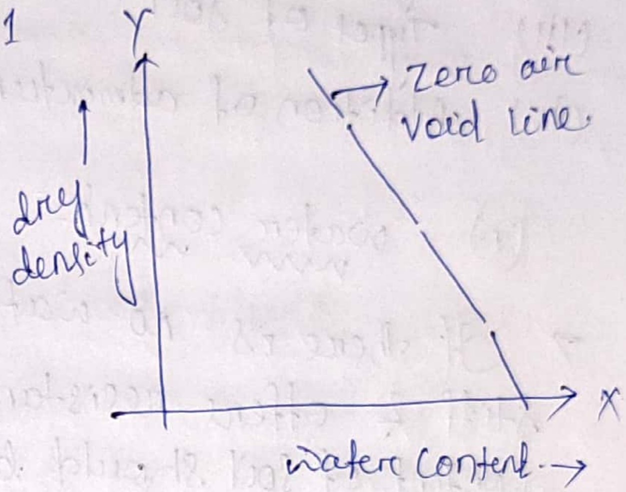
→ From the test it is concluded that initially the dry density increases with increase in water content. Then the dry density attends its max^m value. Then the dry density decreases with increases in water content.

→ The water content corresponding to max^m dry density is called as optimum moisture content (OMC).

Zero air void line :-

→ The relationship betn water content & dry density of fully saturated soil is called as zero air void line.

→ for zero air void line, $S_r = 1$



$$\therefore \gamma_d = \frac{G\gamma_w}{1+e} \quad \text{———} (i)$$

$$eS = Gw$$

$$\Rightarrow e = Gw \; (\because S = 1) \quad \text{———} (ii)$$

from (i) & (ii)

$$\boxed{\gamma_d = \frac{G\gamma_w}{1+Gw}}$$

This is theoritical max^m dry density, which can not be achive partically on the field because what ever the type of compaction their are some amount of air void always present in the compacted soil.

(ii) Modified proctor test / Heavy compaction test :-

→ weight of hammer = 4.54 kg
→ Height of fall = 18" = 457.2mm
→ volume of mould = 944 cc
→ Compacted on 5 layers with 25 blows in each layer.

procedure is same as light compaction test.

Factor affecting compaction:-

(i) Water content
(ii) Type of compaction
(iii) Types of soil
(iv) Addition of admixtures.

(i) Water content:-

→ If there is no water in soil then, the soil becomes stiff & offers resistance to compaction. So, the water content of soil should be adjusted to OMC for quick compaction.

(ii) Types of ~~soil~~ compaction:-

→ Compaction will be quick in dynamic compaction than static compaction.

(iii) Type of soil:-

→ The coarse grained soil will be compacted easily and attempts maxm dry density than fine grained soil.

(iv) Addition of admixtures:

→ The soil will be compacted rapidly by the use of certain admixtures like flyash or terrazyme

Field compaction methods :-

In field, compaction is done by 2 methods

(a) Roller
(b) Rammer.

Rammer :-

→ It consists of a wooden shaft with an iron shoe attached with the bottom of shaft.

→ The wt. of iron shoe varries from 2-5 kg.

→ It is used for compaction of soil in confined areas like bridge, trench filling etc.

Roller :-

There are 4 types of roller,

(a) Smooth wheeled roller
(b) pnumatic tyred roller
(c) Sheep foot roller
(d) Vibratory Compactor or Vibratory roller.

Smooth wheeled roller :-

→ It consists of 3 wheels, 2 large wheels, in rear and one small wheel in front side.

→ It's total mass varries betn 2-15 mega gram
(1 Mega gram = 1000 kg)

→ It is used ~~from~~ for compaction of granular base course in road.

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Pnumatic tyreed roller:-

→ It consist of 9-11 wheels
→ It's total mass varries from 5-200 megagram
→ It uses compressed air to develope required pressure.
→ It is used for compaction of cohessive as well as non-cohessive soil.

Sheep foot roller:-

→ It is similar to smooth wheeled rolled with some projections present on the surface of roller.
→ Brick ballast or water is used to increase the wt. of drum.
→ It is used for compaction of cohessive soil.

Vibrator roller or vibrator compactor:-

→ It is of 2 types.
(i) vibrator roller
(ii) vibrator compactor
→ If vibration is induced on roller it is called vibrator roller.
→ If vibration is induced in plate then it is called vibratory compactor.
→ It is available in both smooth wheel type & pnumatic tyreed wheel type

→ It is used for compaction of granular base coarse which thickness varaies from 0 to 30cm.

Consolidation :-

→ Consolidation is a process of removal of water voids from soil without replacement of water voids by air voids.

(or)

→ Consolidation is the gradual reduction in the volume of fully saturated soil of low permeability due to dessipation of excess pore water pressure set up due to increase in total stress or otherwise.

→ The total settlement of soil is expressed as three components

(i) Immediate Settlement = S_{im}

(ii) primery consolidation

(iii) Secondary Consolidation.

$$\boxed{S_t = S_{imm} + S_{pri} + S_{2^\circ}}$$

Immediate settlement :

→ If soil is initially partially saturated, expulsion of air as well as compression of pore air may take place with the application of external load which is called Initial Compression. It is a immidiate phenomenon.

1° Consolidation :-

→ 1° consolidation occurs due to expulsion of excess pore water pressure generated due to increase in total stress.

→ It is a time dependent phenomenon.

2° consolidation :-

→ 2° consolidation occurs after the end of 1° consolidation.

→ It occurs at constant effective stress.

→ 2° consolidation is thought to be due to gradual readjustment of clay particles into a more stable configuration following the structural disturbance caused by the decrease in void ratio.

Distinction betn compaction and consolidation :-

Compaction	Consolidation
(i) It is the process of removal of air voids	(i) It is the process of removal of water voids.
(ii) It is a rapid process	(ii) It is a gradual process.
(iii) It can be carried out by laboratory	(iii) It ~~can be~~ is done in field.
(iv) In compaction curve the dry density increases initially with increase in water content. Then attain maxm value at OMC. Then the dry density decreases with increase in water content.	(iv) In consolidation curve the void ratio decreases with increase in pressure.
(v) It is carried out in dry or partially saturated soil.	(v) It is carried out in ~~s~~ fully saturated soil.

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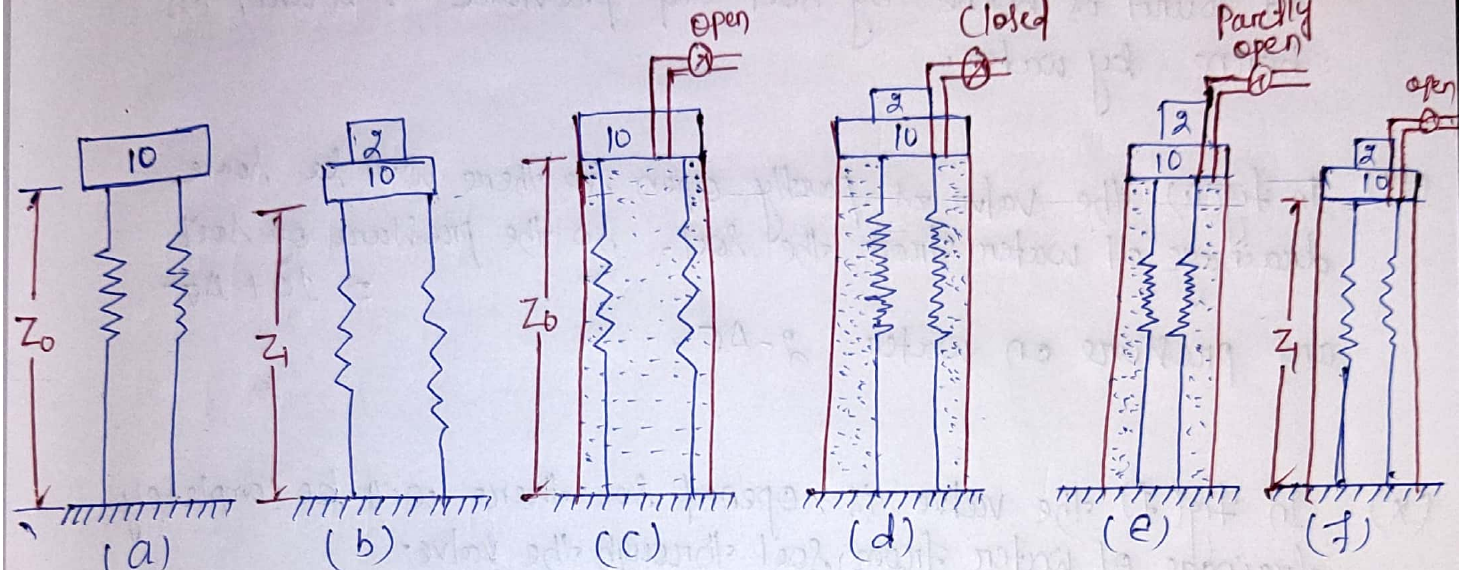
Terzaghi's model analogy of compression/ Springs showing the process of consolidation :- (Spring analogy)



(i) fig shows a saturated soil mass taken in a container consist of soil particles forming of skeleton of soil and voids filled with water.

(ii) Here the skeleton formed of soil particles can be assumed to be replaced by a no. of springs and water filling voids in soil mass by the water filling the cylinder.

(iii) The compressive stress is caused by load applied on piston placed on top of spring.

(iv) An outlet valve is provided to control the drainage of water from the cylinder.

(v) Let, Z_0 be the length of spring under a pressure of 10 unit, as shown in fig(a).

(vi) Let, the length of spring decreases to a depth of Z_1 due to increase of pressure by 2 unit as shown in fig(b).

(vii) In fig(c) a spring is filled with water but no drainage takes place as the entire pressure of 10 unit is taken by spring.

(viii) In fig (d) an additional pressure of 2 unit acts and the valve is closed, so no drainage will takes place. So pressure of 10 unit is borne by soil and pressure of 2 unit is borne by water.

(ix) In fig (e) the valve is partly open so there will be some drainage of water from the soil. So the pressure on soil $= 10 + \Delta\sigma$

and pressure on water $= 2 - \Delta\sigma$.

(x) In fig (f) the valve is opened so, there will be completely drainage of water from soil through the valve.
Hence pressure on soil $= 12$.
& pressure on water $= 0$

(9) Foundation Engineering:-

(9.1)
Introduction:-

→ Footings are generally the lowermost supporting part of the structure known as sub-structure and are the last structural elements through which load is transferred to foundation, comprising soil/rock.

→ The structural elements transfer the applied loads from one part of the building to the other. These are in turn transmitted to the foundation which transfers it to the underlying soil/rock.

Functions of foundations:-

(i) Reduction of load intensity:-
Foundations distribute the load of the superstructure to a larger area so that total intensity of the load does not exceed the safe bearing capacity of soil.

(ii) Even distribution of load:-
foundation distribute the non uniform load of the super structure evenly to the subsoil.

(iii) provision of level surface:-
foundations provide a levelled and hard surface over which a super-structure can be built.

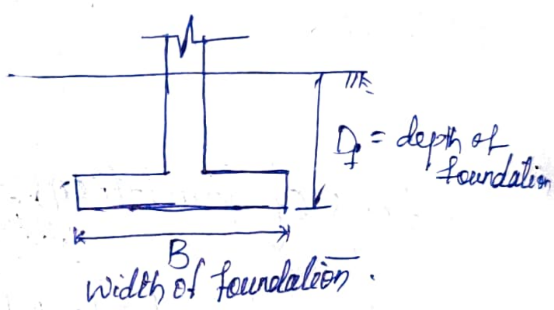
(iv) Lateral stability:-
It anchors the super-structure to the ground ~~to~~ thus imparting stability to the building.

(v) Safety against undermining:-
It provides safety against undermining or scouring due to burrowing animals & floodwater.

(vi) Protection against soil movements:-
Special measures prevent or minimise the distress (cracks) in superstructure, due to expansion or contraction of sub-soil.

Classification of foundation

As per Terzaghi :-



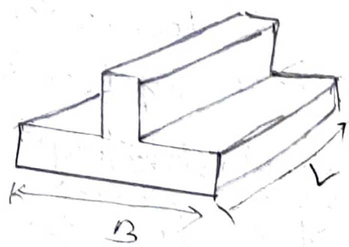
→ If $\frac{D_f}{B} \leq 1$ → The foundation is called shallow foundation.

→ Shallow foundation transfers the load at smaller depth e.g.:- combined footing, Raft foundation, Isolated footing

→ If $\left(\frac{D_f}{B}\right) > 15$ → The foundation is called deep foundation e.g., pile foundation.

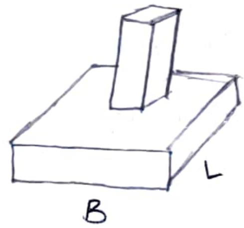
Types of footing:- (Shallow foundation):-

1. Strip footing :-
These are also known as wall footing to support wall. [If $L >> B$] → Strip footing



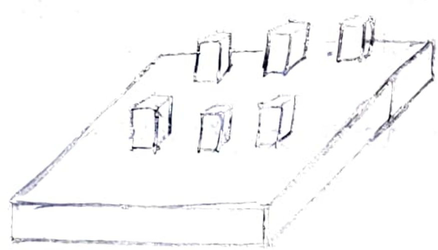
(2) Isolated footing:-

There are also known as spread footing. Isolated footing is used below the column.



3. Raft/Mat foundation:-

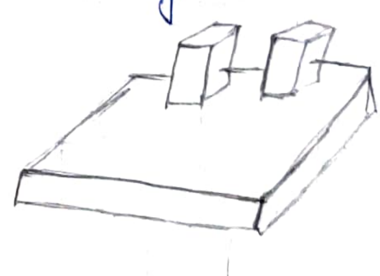
These type of foundations are large continous footing which support all collumns and walls of a structure and are constructed when soil is weak.



Raft foundation

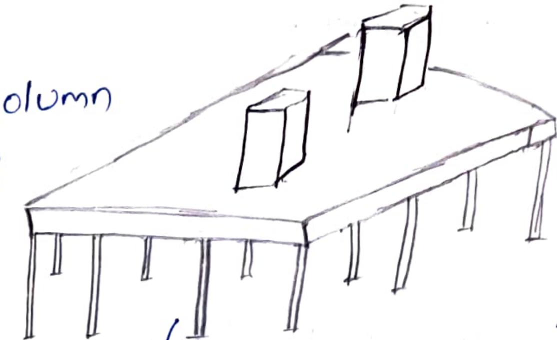
4) Combined footing:-

→ These footings are usually constructed due to space limitations and support two or more columns. They may be either rectangular or trapezoidal in shape.



Types of deep foundation:-

(1) Pile foundation:-



→ These are used to transmit heavey column loads to a group of pile joined at top by a pile cap.

→ These piles transmit the structural loads to the underlying soil through friction and bearing.

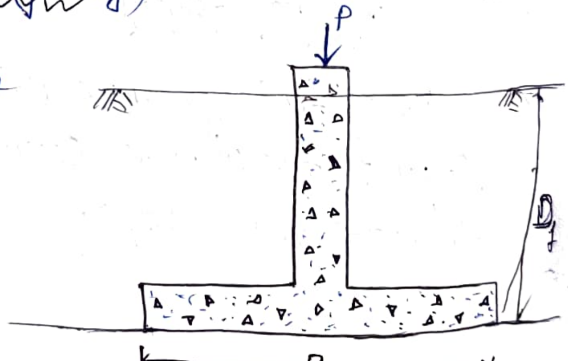
→ Such type of foundation system is usually adopted when the material below footing is too weak to support the structure and it becomes essential to transfer loads to better strata underlying weaker strata.

→ These foundations are very expensive.

Basic definitions:-

(1) Gross pressure or gross loading intensity (q_g)

→ It is the total pressure intensity at the base of footing.



$$\boxed{q_g = \frac{P}{B^2} + \gamma D_f}$$

(Square footing)

(2) Net pressure intensity (q_n)

→ It is generally the loading intensity at the base of footing in excess of the load intensity that the soil was originally subjected to that causes deformation in soil.

Hence, net pressure intensity $= \frac{P}{B^2}$

$$q_n = \left(\frac{P}{B^2} + \gamma D_f\right) - \gamma D_f \quad (\because \text{with backfill})$$

$$\therefore \boxed{q_n = q_g - \gamma D_f}$$

$$\boxed{q_n = \frac{P}{B^2} - \gamma D_f} \quad (\because \text{without back fill})$$

(3) Ultimate bearing capacity (q_u)

→ The maximum gross intensity of loading that the soil can support before it ~~fall~~ fails in shear is called ultimate bearing capacity (q_u)

(4) Net ultimate bearing capacity (q_{nu})

→ The max^m net intensity of loading at the base of the foundation that the soil can support before failing in shear.

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

(5) Net safe bearing capacity: (q_{ns})

→ It is the max^m net pressure a soil can carry safely without the risk of shear failure.

$$\boxed{q_{ns} = \frac{q_{nf}}{F.O.S}}$$

(6) Gross safe bearing capacity (q_{gs}) or Safe bearing capacity (q_s)

→ It is the max^m gross pressure soil can carry safely without the risk of shear failure.

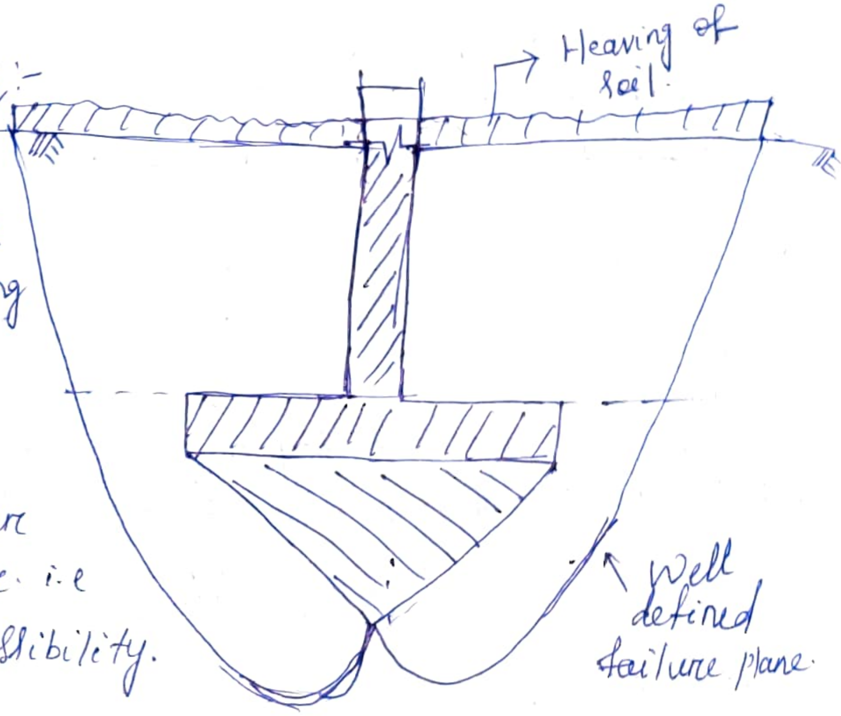
$$\boxed{q_s = q_{ns} + \gamma D_f}$$

Types of failure :-

There are 3 types of bearing capacity failures of soil :-

(1) General Shear failure
(2) Local Sheare failure
(3) punching Shear failure.

(1) General Shear failure :-



→ General Shear failure occurs in soil possessing Brittle type shear stress curve.

→ Dense Sand, Silt, over consolidated clay etc. i.e in soil of low compressibility.

→ In this case failure pattern is well defined and sudden shear failure is experienced with the bulging (Heaving) of ground surface adjacent to foundation at both sides.

→ As the state of plastic equilibrium is reached initially in the soil arround the edge of footing and gradually spread downward and outwards.

→ ultimately, the state of plastic equilibrium in fully developed throughout the soil and the failure surface is as shown in the above diagram.

→ Generally the general shear failure occure in the soil having relative density (D_r) > 70%.

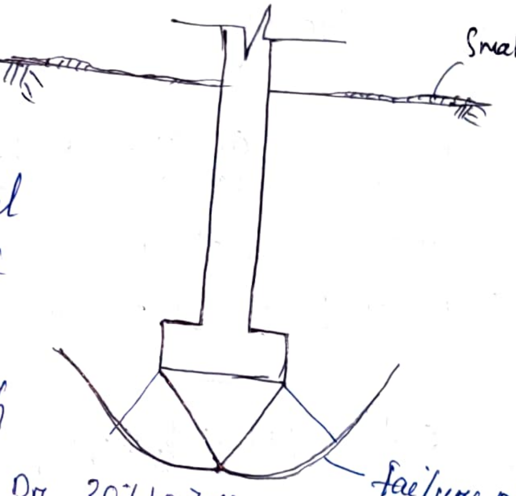
The characteristics of general shear failure are:-

(i) It has well defined failure surface reaching upto ground surface.

(ii) Failure is accompanied by tilting of footing.

(iii) This failures occures in soils of low compressibility.

(2) Local Shear failure:-



→ In this failure there is significant compression of soil under the footing and only partial development of state of plastic equilibrium is reached.

→ The failure surface donot reach the ground surface.

→ It occures in the soil having Dr 30% to 70%.

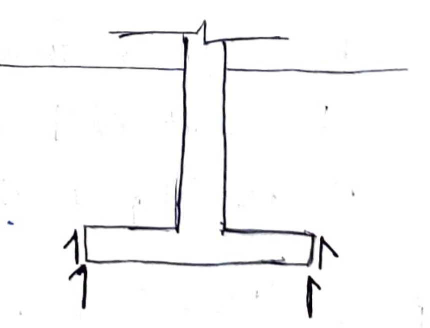
The characteristics of local shear failure are,

(i) The failure pattern is clearly defined only immediatly below the footing.

(ii) There is no tilting of footing.

(iii) It occures in soils of high compressibility.

(3) Punching Shear failure:-



→ This failure occures when there is relatively high compression of soil under the footing accompanied by shearing in vertical direction around the edges of footing.

→ The characteristics of punching shear failure are :-

(i) No failure pattern is observed.

(ii) No tilting of footing.

(iii) It occures in soil of low compressibility.

→ Generally occurs in very loose sand with relative density < 30%.

(9.2)

Terzaghi's theory of bearing Capacity :-

Assumption :-

- The soil is homogeneous & isotropic.
- The shear strength is represented by columb's eqn.
- The Strip footing has a rough base.
- Failure zones do not extend above the horizontal plane through the base of footing.

Terzaghi's analysis of bearing capacity :-

→ According to this theory the ultimate & Net ultimate bearing capacity of soil for strip footing is given by

$$\boxed{q_u = cN_c + qN_q + \tfrac{1}{2}B\gamma N_\gamma} \quad — \quad \boxed{q_{nu} = cN_c + q(N_q - 1) + \tfrac{1}{2}B\gamma N_\gamma}$$

where
cN_c = effect of cohesion
qN_q = Effect of over burden
$B\gamma N_\gamma$ = Effect of soil in the shearize zone

c = Cohesion.
$q = \gamma \times D_f$
D_f = depth of foundation.
B : width of Footing.
N_c, N_q, & N_γ are the bearing capacity factors.

(i) $N_c = (N_q - 1)\cot\phi$; (ii) $N_q = \dfrac{a^2}{2\cos^2(45 + \phi/2)}$

(iii) $N_\gamma = \tfrac{1}{2}\tan\phi\left[\dfrac{K_{p\gamma}}{\cos^2\phi} - 1\right]$

Where:-

$K_{p\gamma}$ = Passive earth pressure co-efficient

Where

$a = e^{(3\pi/4 - \phi/2).\tan\phi}$

Hence N_c, N_q & N_γ are the function of (ϕ) only. As K_p is also a function of ϕ. The value of bearing capacity factor increases as the value of ϕ increases.

In clayey soil:-

When $\phi = 0$,

$$\left.\begin{aligned} N_c &= 5.7 \\ N_q &= 1 \\ N_\gamma &= 0 \end{aligned}\right\} \text{i.e in clayey condition.}$$

Modification of Terzaghi's Bearing capacity equations for Various Types of footing:-

(1) Square footing:-

$$\boxed{q_{nu} = [1.3\,cN_c + q(N_q - 1) + 0.4B\gamma N_\gamma]}$$

(2) Circular footing:-

$$\boxed{q_{nu} = [1.3\,cN_c + q(N_q - 1) + 0.3B\gamma N_\gamma]}$$

(3) Rectangular footing:-

$$\boxed{q_{nu} = \left[\left(1 + 0.3\frac{B}{L}\right)cN_c + q(N_q - 1) + \left(1 - 0.2\frac{B}{L}\right)\left(\tfrac{1}{2}B\gamma N_\gamma\right)\right]}$$

→ The above formula is for general shear failure.

→ If the failure is local shear failure then the above formula should be modified by N_c', N_q' & N_γ'

$$\boxed{\tan\phi' = \frac{2}{3}\tan\phi}$$

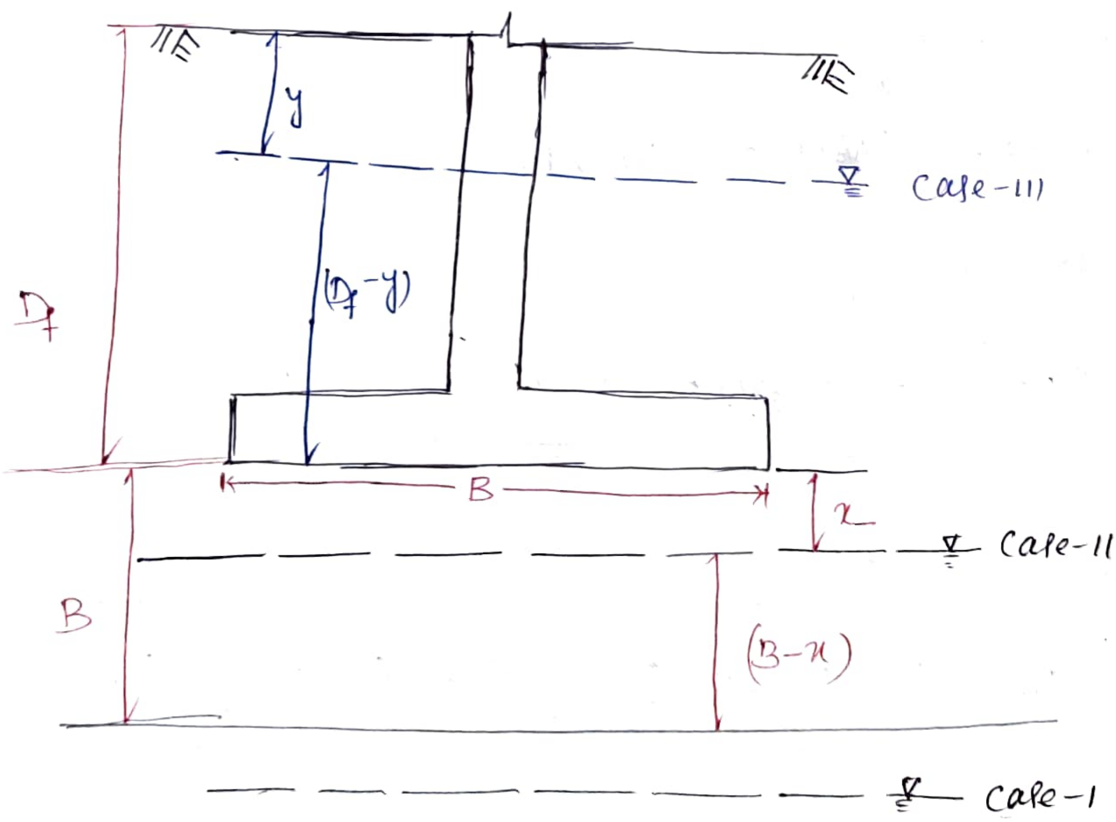
$$\boxed{c' = \frac{2}{3}c}$$

→ $$\boxed{q_{nu} = c'N_c' + qN_q' + \frac{1}{2}B\gamma N_\gamma'}$$

Note

for $\phi > 36°$; general shear failure is assumed &
for $\phi < 29°$; local shear failure is assumed.

Modification of Terzaghi's equation for the location of water table :-



We know that Terzaghi's equation for strip footing is

$$\boxed{q_{nu} = cN_c + q(N_q - 1) + \tfrac{1}{2} B \gamma N_\gamma}$$

→ Soil in the zone from (D_f) to $(D_f + B)$ is affected by the shear failure.

→ For drained condition testing effective stress approach is used.

Case-1 :- When water table is below depth $D = (D_f + B)$

$$\boxed{q_{nu} = cN_c + \gamma_t D_f (N_q - 1) + \tfrac{1}{2} B \gamma_t N_\gamma}$$

Where γ_t = Bulk unit wt.

Case-2 :- When water table is between depth $D = [D_f \text{ to } (D_f + B)]$

$$\boxed{q_{nu} = cN_c + \gamma_t D_f (N_q - 1) + \tfrac{1}{2} [x \cdot \gamma_t + (B - x) \gamma_{sub}] N_\gamma}$$

Where

$(x\gamma_t + (B - x)\gamma_{sub})$ is effective wt. of soil in the region from D_f to $(D_f + B)$ at depth $(D_f + B)$.

Case-3 :-

When water table in in zone from depth $(0 \text{ to } D_f)$

$$\boxed{q_{nu} = cN_c + [\gamma_t \cdot y + (D_f - y)\gamma_{sub}] \cdot (N_q - 1) + \tfrac{1}{2} B \gamma_{sub} N_\gamma}$$

Where $[\gamma_t y + (D_f - y)\gamma_{sub}]$ is the effective over burden wt. at the bottom of the foundation.

Shear Strength :-

(7.1)

→ Shear Strength is the capacity to resist shear stress.

→ Whenever a soil is loaded beyond its permissible limit then the shear failure of soil is occures.

Ex:- (i) Moment of wedge of soil behind the retaing wall.

(ii) Sinking of footing.

Shear Strength of Soil :-

→ The resistance of soil to shear deformation is called as shear strength of soil.

→ The shear strength of soil depends upon :-

(i) frictional resistance due to inter locking of soil particles.

(ii) Cohesion betn the soil particles.

Mohr - Columb failure theory :-

→ According to this theory, "the shear strength of soil depends upon normal stress" as follows

$$\boxed{\tau_f = c + \sigma \tan\phi}$$

where τ_f = Shear strength of soil

c = cohesion

σ = normal stress

ϕ = angle of shearing resistance or angle of internal friction.

→ c & ϕ are not inherent properties of soil. These are related to the type of test and condn under which the testing is done.

→ But after the development of effective stress concept, shear strength is express as

$$\boxed{\tau = c' + \bar{\sigma} \tan\phi'}$$

$\bar{\sigma}$ - effective stress

c' & ϕ' are effective stress shear strength parameter.

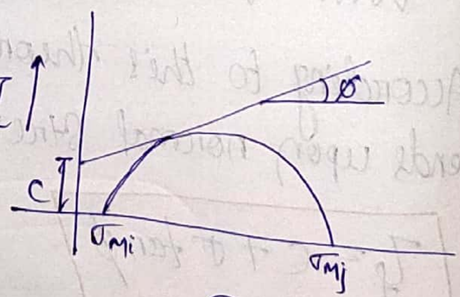
Cohesion :-

→ It is defined as the inter molecular force of attraction betn same particles.

~~True cohesion~~ :-

Angle of internal friction

→ Angle of internal friction is a physical property of earth materials or the slope of linear representation of the shear strength of earth material.

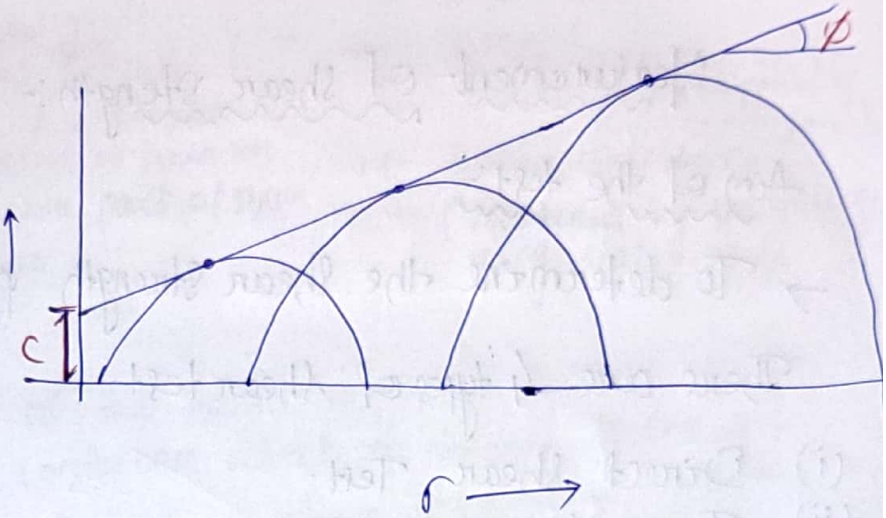


Strength envelope :-

→ Combining mohr-columb criteria it can be shown that if series of mohr circle is drawn corresponding to different test carried out on different specimen of soil upto failure, then common tangent to these circles represent mohr failure envelope or

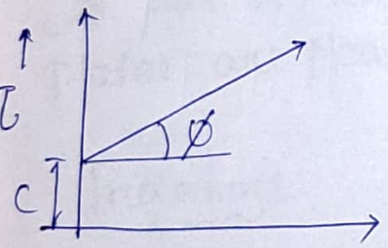
Strength envelop.

$\boxed{\tau = c + \sigma \tan\phi}$

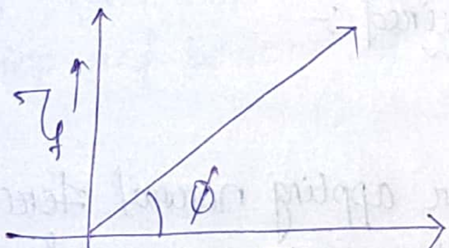


→ The relationship betⁿ shear strength and normal stress of soil particle is called strength envelop.

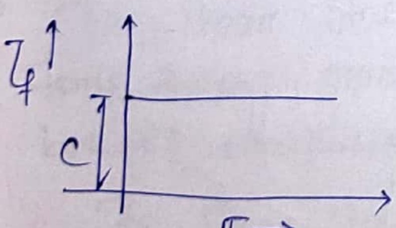
→ The strength envelop for different types of soil are as follows:-



σ → (Cohesive soil)
(C-φ soil)



σ →
(Non-cohesive soil) →
(C=0)



σ →
(Purely cohesive soil)
(φ=0)

Measurement of shear strength :-

Aim of the test :-

→ To determine the shear strength parameters (C & Ø).

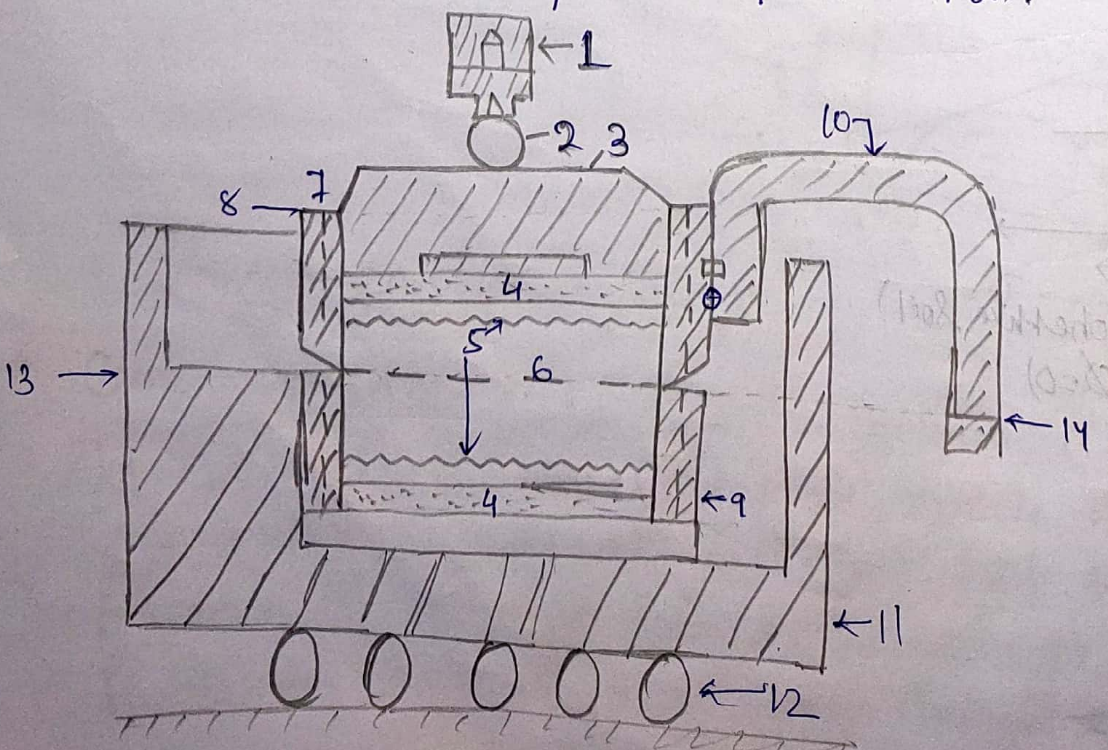
There are 4 types of shear test.

(i) Direct shear Test.
(ii) Triaxial compression test.
(iii) Unconfined compressive test.
(iv) Vane shear test.

(i) Direct shear test :-

Apparatus required :-

(i) Shear box.
(ii) Loading yolk for appling normal force.
(iii) Geared jack for appling shear force.
(iv) facilities for measuring shear stress.
(v) Shear measurment and vertical deformation.



1- loading yoke
2- Steel ball
3- loading pad
4- porous stones
5- metal grids
6- Soil specimen
7- pins to fix two halves of shear box
8- upper part of shear box
10- U-arm
11- Container for shear box
12- Rollers
13- Shear force (applied by jack)
14- Shear resistance. (measured by providing ring dial gauge)

→ The shear box consists of two halves, the lower halve is in contact with shear box container which is freely slides on rollers and to which the shear force is applied by means of geared Jack.

→ The soil specimen is placed on shear box such that it gets sheared on a horizontal plane exactly at its mid height.

→ A pair of metal grid plates and a pair of porous plates are placed on top and bottom of soil.

Procedure:-

→ A normal stress "σ" is applied on specimen and is kept constant through out the test.

→ The shear stress (τ) is caused by application of shear force through geared jack and is transmitted to the top of shear box is measured.

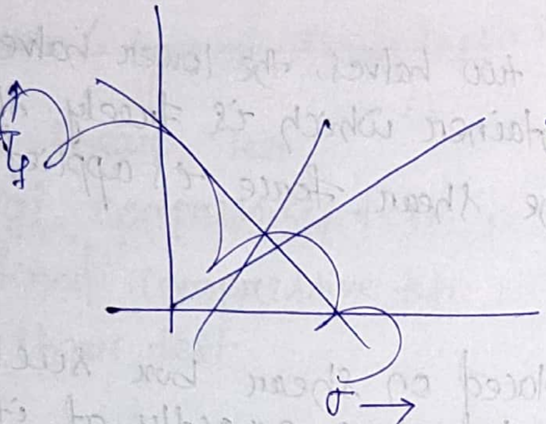
→ The shear stress is gradually increased until the specimen fails. The shear stress (τ_f) corresponding to failure of soil is noted.

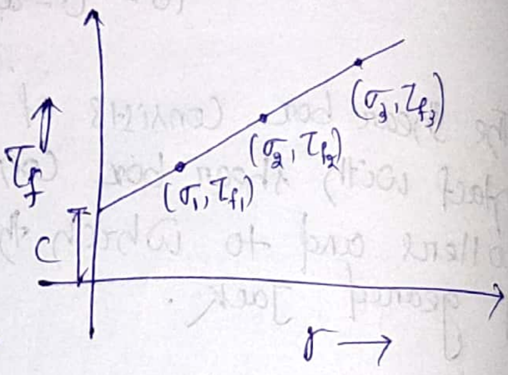
→ The value of τ_f and σ are noted.

→ This test is repeated on 3 specimens by changing the value of σ & the corresponding value of τ_f are noted.

→ Then a graph is plotted by taking σ on x-axis & τ_f on y-axis.

→ Then from the graph the value of c & ϕ are computed.





Advantages of direct shear test:-

→ It is simpler than triaxial compression test.

→ Since the thickness of sample is small, quick drainage of pore pressure is possible.

Disadvantages of direct shear test:-

→ The failure plane is predetermined i.e. the specimen is not allowed to fail along the weakest plane.

→ The shear stress is not uniformly distributed being more at the edge and lesser at the centre.

→ Measurement of pore pressure is not possible.

→ Shear displacement causes reduction in area under shear. Corrected area should be used in computing normal & shear stresses.

→

(ii) Triaxial Compression test:-

→ In this test the specimen is compressed by appling all the 3-type of principal stresses i.e $\sigma_1, \sigma_2, \sigma_3$.

→ In this test the soil specimen used in cylindrical in shape with length 2 to 2.5 times the diameter.

Apparatus required:-

(i) Triaxial cell

(ii) loading frame with accessories for appling gradually increasing axial load.

(iii) Provision for measuring axial force & axial displacement.

(iv) constant pressure system to apply and maintain constant cell pressure.

(v) pore pressure measuring apparatus.

(vi) vol. change gauge.

→ The triaxial cell consists of a high pressure cylindrical cell made up of transparent material.

→ An inlet for cell fluid and out-let for drainage of pore water is provided.

→ At the top an air release valve is provided to expale air from the soil.

procedure:-

→ The soil specimen is kept inside the triaxial cell with porous plate at top and bottom.

→ The loading cap is placed on the top of porous plate.

→ The specimen is enclosed in a rubber membrane to prevent its constant with cell fluid.

→ After filling the cell with cell fluid usually water the required cell pressure (σ_3) is applied by means of constant pressure system.

→ The additional axial force called the deviator force is applied through the plunger.

→ The test is continued and additional axial force is increased till the specimen fails.

→ The deviator stress corresponding to deviator force at failure is noted.

→ This test is repeated on 3 soil specimens by changing the value of σ_3 on each specimen and corresponding values for deviator stress for diff. soil specimens are noted.

→ Then from the deviator stress (σ_d) and cell pressure (σ_3) the value of σ_1 is computed for diff. soil specimen as follow; $\boxed{\sigma_1 = \sigma_d + \sigma_3}$

where

σ_d = Deviator stress = $\frac{F}{A_c}$ (A_c = Corrected C/A)

→ Then by using σ_1 & σ_3 Mohr's circle is drawn for each soil specimen.

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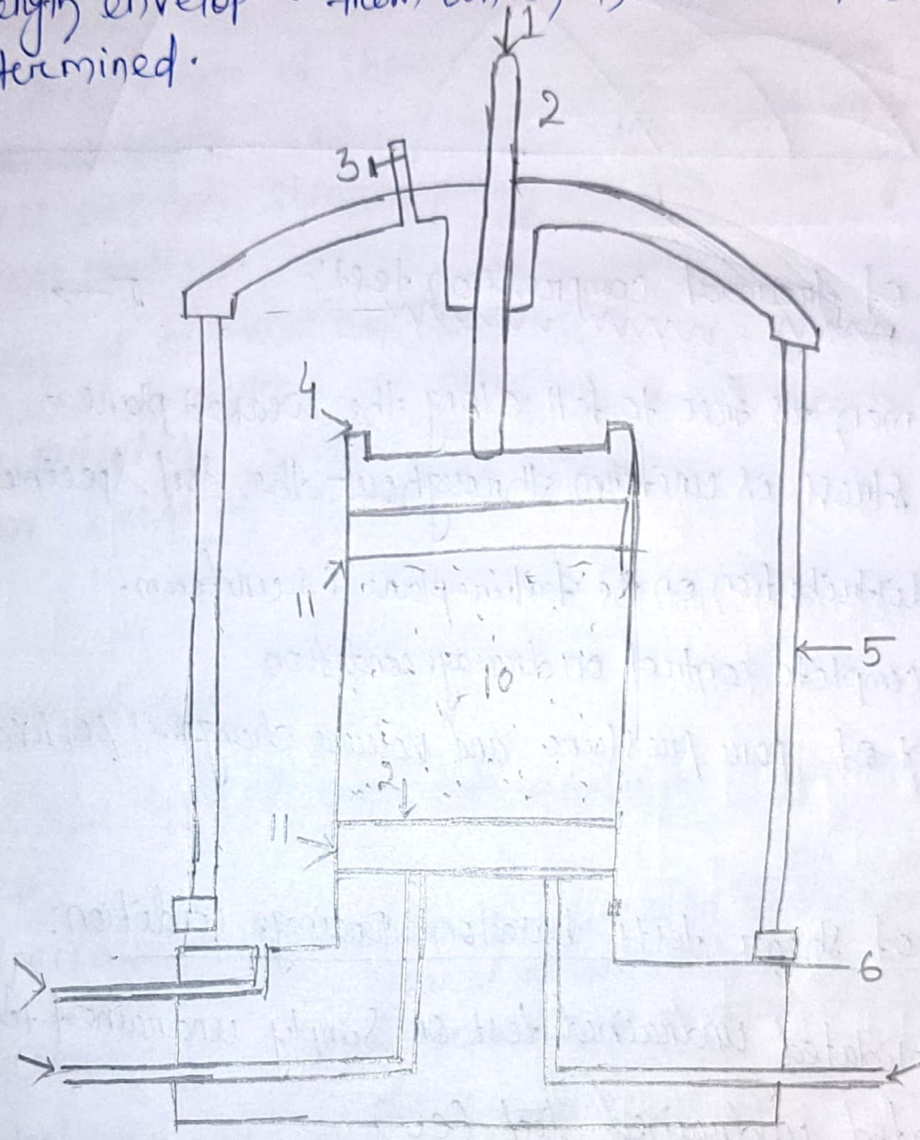
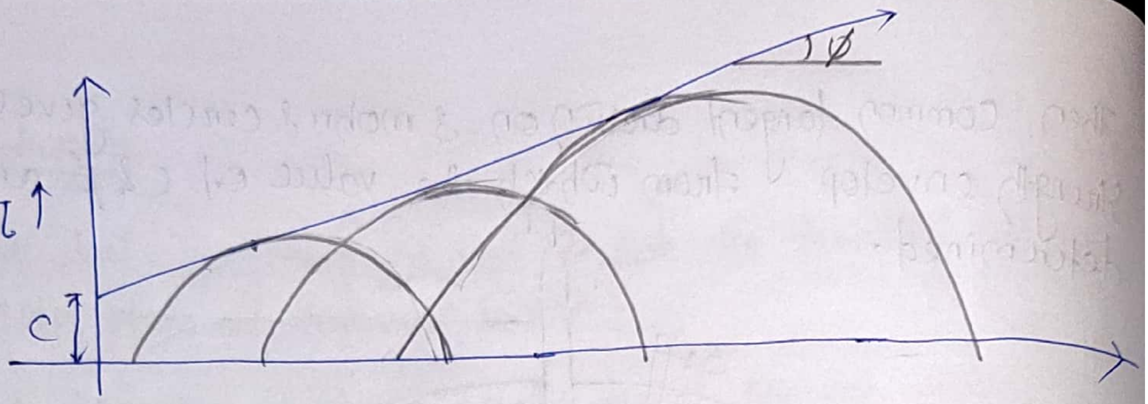
→ Then common tangent drawn on 3 mohr's circles gives the strength envelop from which the value of c & ∅ are determined.



1. Axial load (measure by proving ring) dial gauge
2. Loading ram
3. Air release valve
4. Top cap
5. perspex cylinder
6. Sealing ring
7. pore water outlet
8. Additional pore water outlet.
9. Cell fluid inlet.
10. Soil specimen (enclosed in rubber membrane with 'O' rings at the ends)
11. porous discr.



Advantages of triaxial compression test:-

→ The specimen is free to fail along the weakest plane.
→ The shear stress is uniform throughout the soil specimen.
→ The stress distribution on the failure plane is uniform.
→ There is complete control on drainage condition.
→ Measurement of pore pressure and volume change is possible.

Types of shear tests based on Drainage condition:-

(i) unconsolidated undrained test or simply undrained test (UU)
(ii) consolidated undrained test (CU)
(iii) Consolidated Drained test or simply drained test - (CD)

(i) undrained test :-

→ Drainage is not permitted throughout the test.
→ In case of triaxial compression test drainage is not permitted during the application of both cell pressure and deviator stress.
→ This test is also called quick test.

(ii) Consolidated undrained test:-

→ In this type of shear test the soil specimen is allowed to ~~consolidate~~ consolidate fully under initially applied stress and then sheared quickly without allowing dissipation of pore pressure.

→ In case of triaxial compression test the specimen is allowed to consolidate fully under applied cell pressure and then the pore water outlet is closed and the specimen subjected to increasing deviator stress at high rate of strain.

(iii) Drained test:-

→ In this type of shear test drainage is allowed throughout the test.

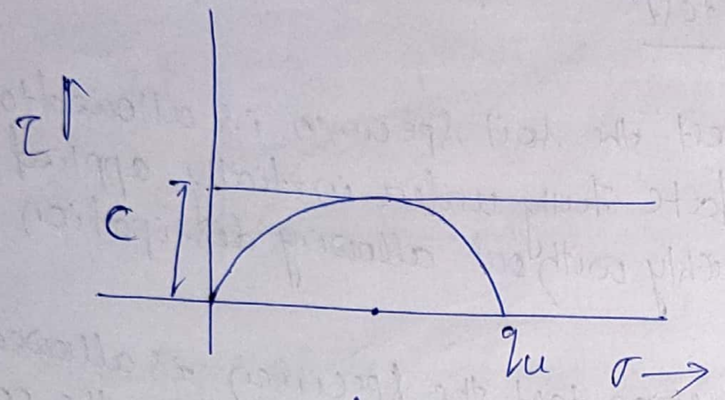
→ The specimen is allowed to consolidate fully under the applied initial stress and then sheared at low rate of strain giving sufficient time for the pore water to drain out at all stage.

→ The test may continue for several hours to several days.

Unconfined compression test:-

→ unconfined compression test can be regarded as a special case of triaxial compression test in which no lateral pressure or confining pressure is applied, so $\sigma_2 = \sigma_3 = 0$.

→ The soil specimen is cylindrical in shape with length about 2 to 2.5 times its diameter.



- The max^m axial stress a soil can carry safely without shear failure is called as unconfined compressive strength.

- It is denoted by 'q_u' and given by $\boxed{q_u = \frac{F}{A_c}}$

Where A_c = Corrected area of crossection.
F = unconfied axial load.

- It is generally used to findout the strength of purly cohessive soil.

- Here the axial stress on the soil specimen is increased till the specimen failss.

- The axial stress, corresponding to failure of soil is noted which is equal to $\sigma_d = q_u$.

- Then a mohr's circle is drawn by using σ_d.

- The tangent to the mohr's circle gives the strength envelop.

- From the strength envelop the value of c & ∅ are obtained.

Vane Shear Test:

→ Vane shear test is a quick test used to determine undrained shear strength of cohesive soil.

→ The equipment essentially consist of four high tensile steel plates called vanes which are welded to the bottom end of a steel rod called the torque rod with an arrangement to measure the torque and rotation.

→ A typical arrangement consists of a calibrated torsion spring attached to the top of torque rod which is rotated by a combination of worm gear and worm wheel.

→ The vane shear test can be conducted both in the laboratory and in field.

→ A typical laboratory set of vanes has 20mm height, diameter of 12mm across vanes with blade thickness of 0.5 to 1mm.

→ The field set of vanes usually is 100 to 200 mm in height, 50 to 100mm in diameter across vanes with blade thickness of 2.5mm.

→ To conduct the test the vanes are gently pushed into the soil and the torque rod is rotated at a uniform rate of usually 1° per min.

→ The torque T corresponding to angle of rotation θ at uniform interval are noted.

→ Torque T is plotted as ordinate (Y-axis) against angle of rotation θ as abscissa (X-axis).

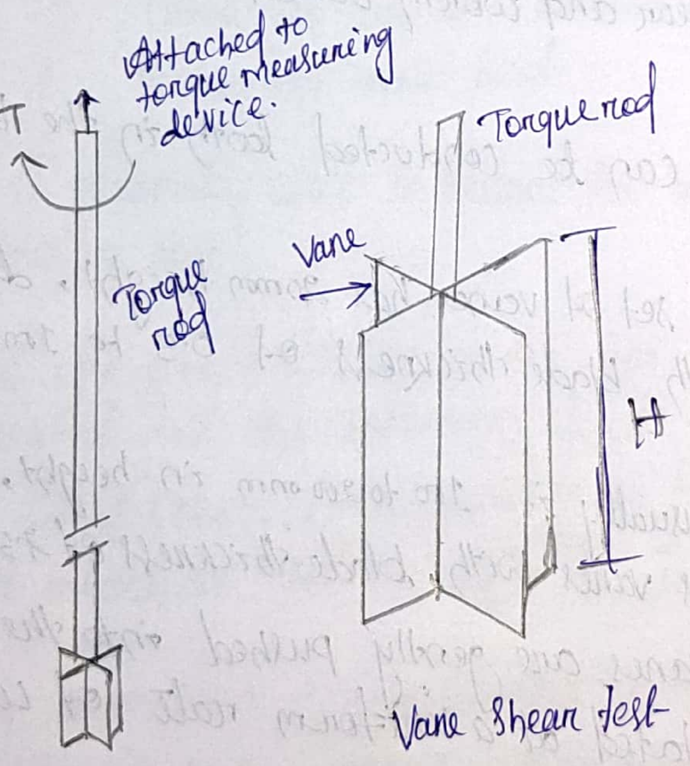
→ The torque T_f at failure is found and is used to calculated the shear strength τ_f :-

(i) $\boxed{T_f = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{6}\right]}$

When the vane is pushed with top end below the surface of soil so that both top and bottom ends take part in shearing.

(ii) $\boxed{T_f = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{12}\right]}$

When the vane is pushed inside the soil with its top end flush with surface of soil so that only bottom end take part in shearing.



Vane Shear test

~~Earth Pressure~~

Earth pressure on retaining structure:-

Introduction :-

→ Soil mass is stable when the slope of the surface of the soil mass is flatter than the safe slope. But at some places the space is limited, it is not possible to provide flat slope and the soil is to be retained at a slope steeper than the safe one.

→ Therefore to retain this soil mass in a stable state a retaining structure is provided to provide the lateral support to the soil mass.

→ In the design of these retaining structure it becomes imparative to know the magnitude and line of action of earth pressure, Where the earth pressure is the lateral force exerted by the soil on any structure retaining that soil.

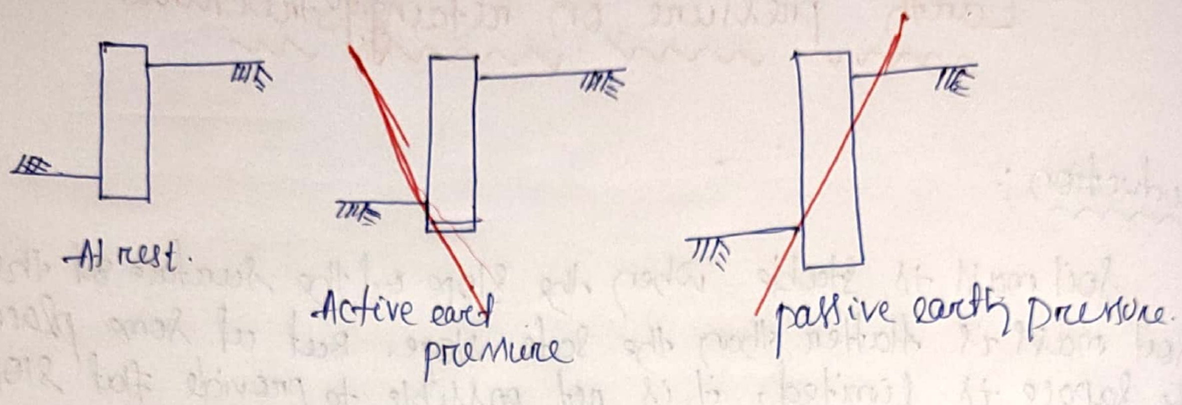
8.1 Retaining wall

→ A structure which is used to hold back a soil mass is called a retaining structure.

Types of lateral earth pressure:-

→ Lateral earth pressure can be devided into 3 categories, depending upon the movement of retaining wall with respect to back fill soil.

(1) Earth pressure At Rest:- Wall does not move at all.
(2) Active earth pressure - Wall moves away from the back fill soil.
(3) Passive Earth pressure - Wall moves towards the back-fill soil.



(1) Earth pressure at Rest :-

→ A soil element in its natural state at any depth 'z' below the ground surface is not subjected to any strain; the element in this condition is known as at rest condition.

→ $\boxed{\text{Earth pressure at rest } (P_0) = K_0 \sigma_z}$

K_0 = co-efficient of earth pressure at rest.
σ_z = Total stress or effective stress at the point on soil mass where we need to calculate earth pressure.

$K_0 = \frac{\mu}{1-\mu}$, μ - poisson's ratio.

$= 1 - \sin\phi$, ϕ - angle of internal friction.

(2) Active earth pressure :-

→ It is the pressure exerted by the backfill to the retaining wall.

→ Due to this pressure the retaining wall tends to move outword.

→ The intensity of active earth pressure :-

$\boxed{P_a = K_a \gamma H}$

Where, K_a = co-efficient of active earth pressure.
γ = unit wt. of backfill.
H = Height of retaing wall.

$$\boxed{K_a = \frac{1-\sin\phi}{1+\sin\phi}}$$

(3) Passive earth pressure:-

→ It is the pressure exereted by the retaingiing wall to the backfill.

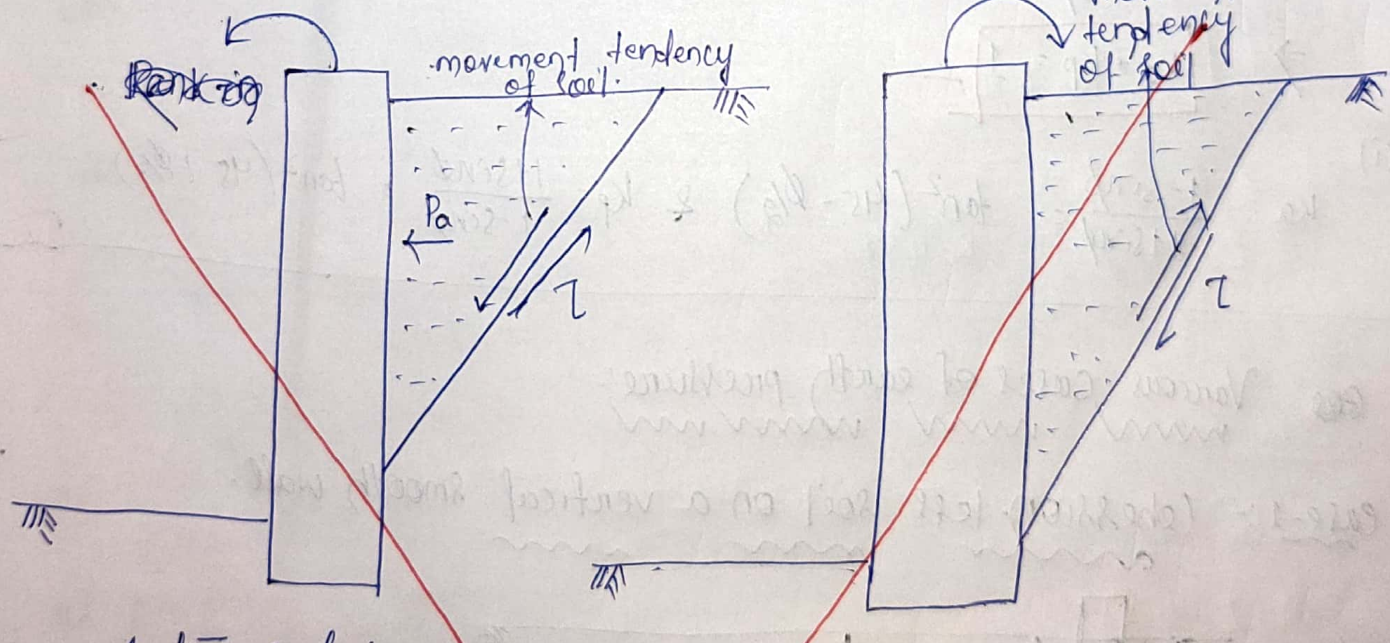
→ This pressure very rarely comes into play.

→ The intensity of passive earth pressure :-

$$\boxed{P_p = K_p \gamma . H}$$

Where K_p = co-efficient of passive earth pressure.

$$\boxed{K_p = \frac{1+\sin\phi}{1-\sin\phi}}$$



Active Earth pressure

Passive Earth pressure

Rankine's theory of earth pressure:-

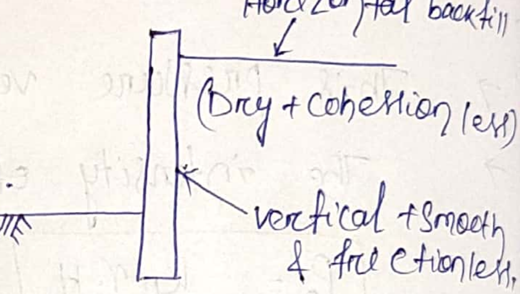
Assumptions:-

(i) Soil is semi-infinite, homogenous, isotropic, dry & cohesionless.

(ii) Soil is in the state of plastic condition at the time of active and passive pressure generation.

(iii) The backfill soil is horizontal.

(iv) Back of the wall is vertical and smooth.



Note

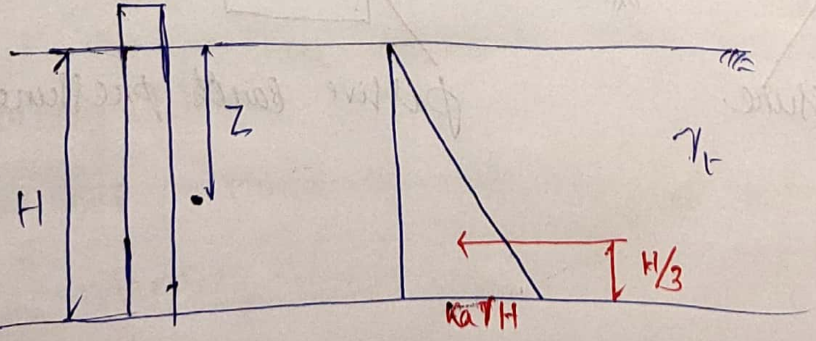
(i) $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ & $K_p = \frac{1+\sin\phi}{1-\sin\phi}$

$\Rightarrow \boxed{K_a * K_p = 1}$

(ii) $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45 - \phi/2)$ & $K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45 + \phi/2)$

Various cases of earth pressure:-

case-1:- Cohession-less soil on a vertical smooth wall.



at any depth Z = $P_a = K_a \gamma_t * Z$

at $Z=0$, at $Z=H$

$P_a = 0$ $\quad$ $P_a = K_a \gamma H$

Now; force or active thrust or total pressure due to active earth pressure per unit length of wall (F_a) = area of pressure prism.

i.e. $F_a = \frac{1}{2} * K_a \gamma H * H$ (acting at H/3 from base)

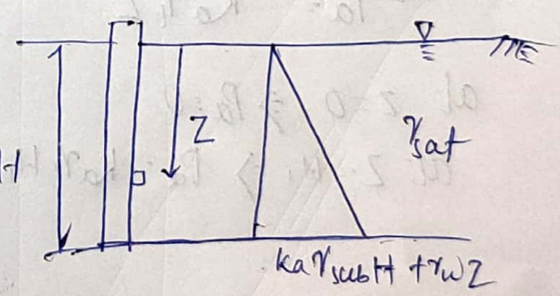
$$\boxed{F_a = \frac{1}{2} K_a \gamma H^2}$$

Similarly $F_p = \frac{1}{2} K_p \gamma H^2$, where F_p = total passive thrust

$F_o = \frac{1}{2} * K_o \gamma H^2$, where F_o = total pressure at rest.

Case-II:- Submerged cohesionless soil on vertical smooth wall:-

at any depth 'Z'



$P_a = K_a [\gamma_{sat} Z - \gamma_w Z] + \gamma_w Z$ ($\bar{\sigma}$ = total stress - pore water)

$\therefore P_a = K_a \gamma_{sub} Z + \gamma_w Z$

at $Z=0$

$\therefore P_a = 0$

at $Z = H$

$\therefore P_a = K_a \gamma_{sub} H + \gamma_w Z$

→ force or active thrust or total pressure due to active earth press. = $\frac{1}{2} * (K_a \gamma_{sub} H + \gamma_w H) * H$

$= \frac{1}{2} (K_a \gamma_{sub} H^2 + \gamma_w H^2)$ (acting at H/3 from base)

$$\therefore \boxed{F_a = \frac{1}{2} (K_a \gamma_{sub} H^2 + \gamma_w H^2)}$$

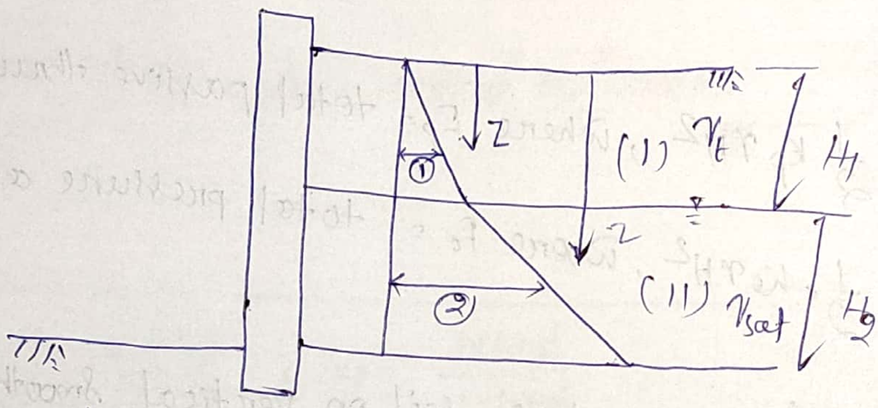
Scanned with CamScanner

Similarly :-

$$P_p = K_p \gamma_{sub} H + \gamma_w H$$

$$F_p = \frac{1}{2}(K_p \gamma_{sub} H^2 + \gamma_w H^2)$$

Case-III :- Partialy submerged cohession less soil on vertical smooth wall :-



Zone-I

$P_a = K_a \gamma_t Z$

at $Z=0 \Rightarrow P_a = 0$

at $Z = H_1 \Rightarrow P_a = K_a \gamma_t H_1$

Zone-II

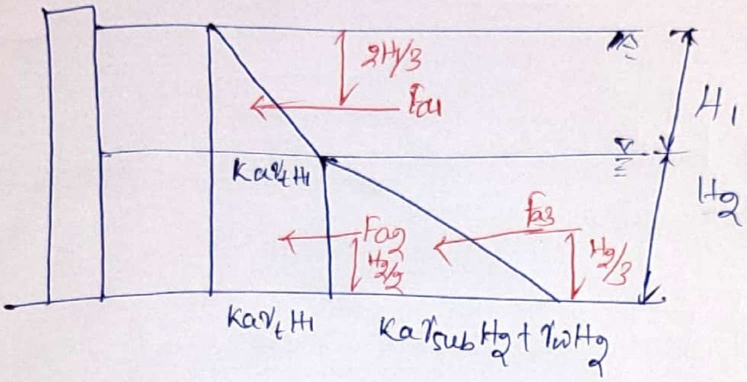
$$P_a = K_a \gamma_t H_1 + K_a \gamma_{sub} (Z - H_1) + \gamma_w (Z - H_1)$$

at $Z = H_1$

$P_a = K_a \gamma_t H_1$

at $Z = H_1 + H_2$

$\Rightarrow P_a = K_a \gamma_t H_1 + K_a \gamma_{sub} H_2 + \gamma_w H_2$.



$$F_a = F_{a1} + F_{a2} + F_{a3}$$

∴ point of application of Resultant force from bottom of base

$$= \frac{F_{a1}\left(H_1 + H_2 - \frac{2H_1}{3}\right) + F_{a2}\left(\frac{H_2}{2}\right) + F_{a3}\left(\frac{H_2}{3}\right)}{F_{a1} + F_{a2} + F_{a3}}$$

(viii) In fig(d) an additional pressure of 2 unit acts and the valve is closed, so no drainage will takes place. So pressure of 10 unit is borne by soil and pressure of 2 unit is borne by water.

(ix) In fig(e) the valve is partly open so there will be some drainage of water from the soil. So the pressure on soil $= 10 + \Delta\sigma$

and pressure on water $= 2 - \Delta\sigma$.

(x) In fig(f) the valve is opened so, there will be completely drainage of water from soil through the valve.

Hence pressure on soil $= 12$.

& pressure on water $= 0$

Field implications:-

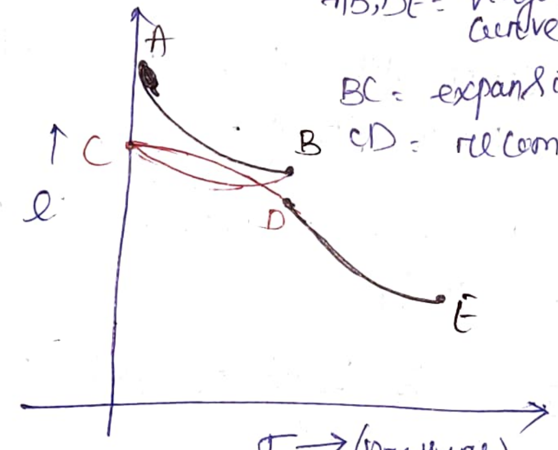
Pressure void ratio curve or Consolidation Curve:-

AB, DE = Virgin compression curve

BC = expansion curve

CD = recompression curve

→ The fig. shows the relationship betn void ratio & pressure of a remoulded soil specimen.



→ The curve AB is obtained by increasing the effective pressure on soil due to which the void ratio of soil decreases considerable.

→ At point B, the pressure is removed. So, in both point B & C the pressure of soil decreases and void ratio of soil increases; however the soil will not attened the original void ratio.

→ In both C & D the pressure increases and the void ratio decreases slowly.

→ The pressure at point D is nearly equal to pressure at point B.

→ Again, after point D the void ratio considerable decreases with an increase of pressure.

→ Here the curve AB and DE are called as virgin compression curve and BC called as expansion curve and CD as recompression curve.

Terzaghi's assumption for 1D consolidation:-

(i) The soil is homogeneous and fully saturated.

(ii) The soil particles and water both are incompressible.

(iii) Darcy's law for flow of water will be applicable during consolidation.

(iv) Co-efficient of permeability is constant during consolidation

(v) The load is applied in one direction only and deformation occures in the direction of load only.

(vi) The deformation is due to entirely decrease in vol.

(vii) The drainage of pore water occures only in one direction.

Important formulae:-

(1) $e = e_0 - C_c \log\left(\frac{\sigma'}{\sigma_0'}\right)$

(2) $e = e_0 - C_s \log\left(\frac{\sigma'}{\sigma_0'}\right)$

(3) $T_v = \frac{C_v t}{d^2}$

(4) $C_v = \frac{k}{m_v \gamma_w}$

(5) $T_v = \frac{\pi}{4}(U^2)$, if $U \leq 60\%$ U in decimal

(6) $T_v = 1.781 - 0.9332 \log(100-U)$ if for $U > 60\%$ U in %age

(7) for double drainage, $d = \frac{H}{2}$

(8) for single drainage, $d = H$

(9) $U = \frac{\Delta h}{\Delta H} \times 100$

$\therefore$ σ' = final effective stress

σ_0' = initial effective stress.

T_v = Time factor

C_v = Co-efficient of consolidation

t = time interval

d = depth of drainage
U = degree of consolidation
H = thickness of soil layer.
M_v = Co-efficient of vol. compressibility.

e_0 = initial void ratio
e = final void ratio.
C_c = compression index
C_s = swelling index.
Δh = settlement of any stage or time rate settlement.
ΔH = settlement at the end of consolidation.

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