

GEOTECHNICAL ENGINEERING (23HPC0114)

LECTURE NOTES

III-B.TECH & I-SEM

Prepared by:

T. MADHAVI LATHA, Assistant Professor

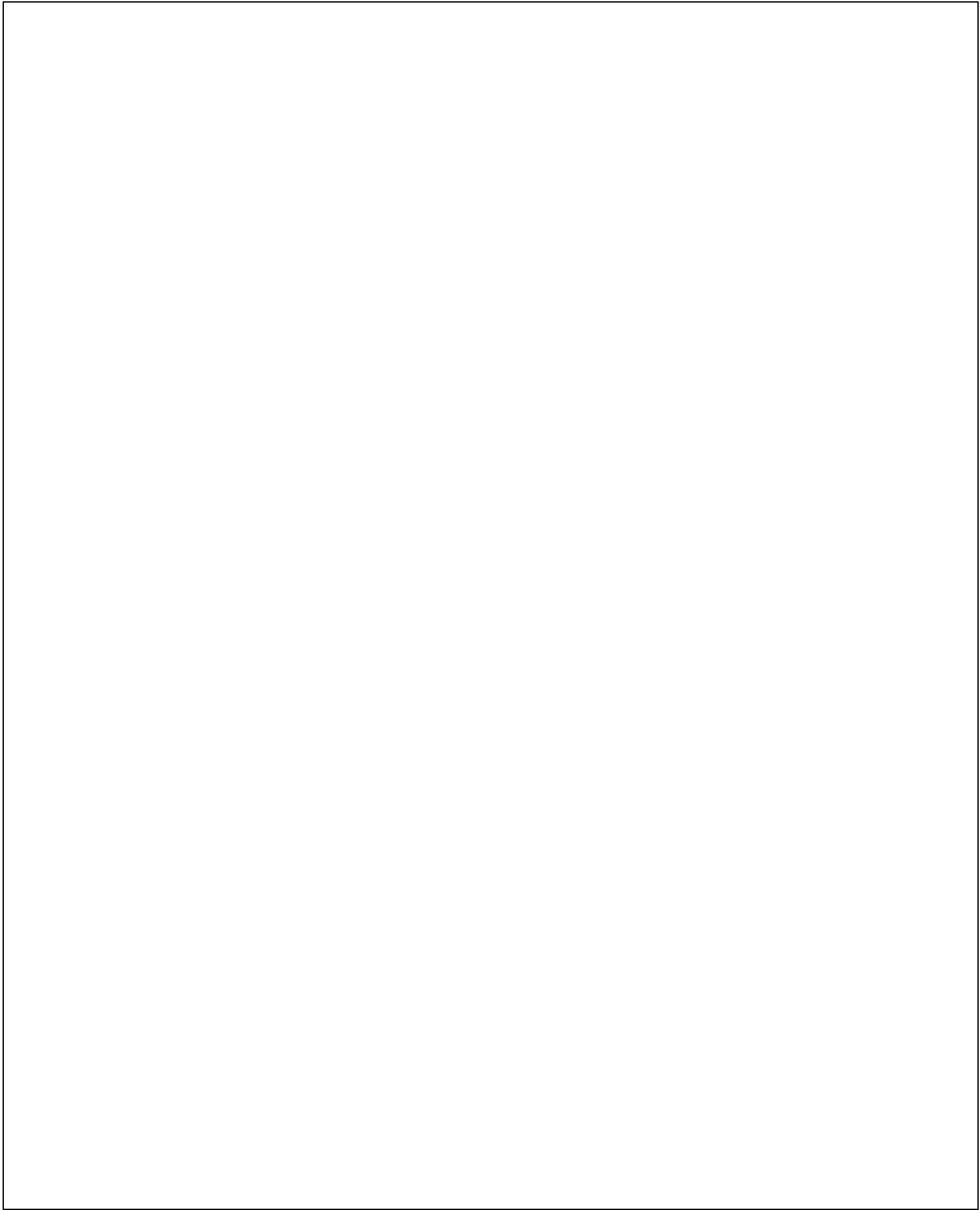
Department of Civil Engineering



**ANNAMACHARAYA INSTITUTE OF
TECHNOLOGY & SCIENCES
(AUTONOMOUS)**

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CIVIL ENGINEERING					
III B.Tech – I Semester					
Course Code	GEOTECHNICAL ENGINEERING	L	T	P	C
23HPC0114		3	0	0	3

Course Objectives:

The objectives of this course are to make the student to:

1. **Understand** the classification and compaction characteristics of different soil types and their engineering significance.
2. **Analyze** the concepts of effective stress, permeability, and seepage in soils and their impact on soil behavior.
3. **Apply** stress distribution theories and settlement computations to evaluate soil response under loads.
4. **Evaluate** shear strength properties of soil using various testing methods and their applications in geotechnical engineering.
5. **Assess** the stability of slopes using different analytical methods and suggest suitable slope protection measures.

Course Outcomes:

Upon successful completion of this course, students will be able to:

1. **Classify** soils based on their physical and index properties as per BIS and Unified classification systems.
2. **Analyze** soil permeability and seepage problems using Darcy's law and flow net concepts.
3. **Apply** stress distribution theories and settlement analysis to predict soil behavior under loading.
4. **Evaluate** shear strength of soils using experimental methods and interpret test results.
5. **Assess** slope stability and recommend suitable protection measures.

CO – PO Articulation Matrix:-														
Course Outcomes	PO 1	PO 2	PO 3	PO 4	PO 5	PO 6	PO 7	PO 8	PO 9	PO 10	PO 11	PO 12	PS O1	PS O2
CO -1	3	2	2	1	-	-	-	-	-	2	-	2	3	2
CO -2	3	3	3	2	2	-	-	-	-	-	-	1	3	2
CO -3	3	2	2	2	2	-	-	-	-	-	-	1	2	2
CO -4	3	3	3	2	2	-	-	-	-	-	-	1	3	3
CO -5	2	2	2	1	3	-	-	-	-	2	1	2	2	2

UNIT – I

INTRODUCTION: Soil Formation-Soil structure - Adsorbed water - Mass- Volume relationship — Relative density. Index Properties of Soils: Moisture Content, Specific Gravity, In-situ density, Grain size analysis — Sieve and Hydrometer methods — Consistency limits and indices -I.S. Classification of soils.

UNIT – II

PERMEABILITY: Soil Water — Capillary Rise — Flow of Water Through Soils - Darcy's

Law-Permeability Factors Affecting — Laboratory Determination of Coefficient of Permeability-Permeability of Layered Systems.

SEEPAGE THROUGH SOILS: Total, Neutral and Effective Stresses - Quick Sand Condition — Seepage Through Soils — Flow Nets : Characteristics and Uses.

UNIT – III

STRESS DISTRIBUTION IN SOILS: Boussinesq's and Westergaard's Theories For Point Loads and Areas of Different Shapes — Newmark's Influence Chart Compaction: Mechanism of Compaction — Factors Affecting — Effects Of Compaction on Soil Properties. — Field Compaction Equipment — Compaction Control.

UNIT – IV

CONSOLIDATION: Types of Compressibility -Primary Consolidation and Secondary Consolidation - Stress History of Clay; E-P And E-Log P Curves — Normally Consolidated Soil, Over Consolidated Soil and Under Consolidated Soil — Pre-Consolidation Pressure and Its Determination — Terzaghi's I-D Consolidation Theory — Coefficient of Consolidation: Square Root Time and Logarithm of Time Fitting Methods.

UNIT – V

SHEAR STRENGTH OF SOILS: Importance of Shear Strength- Mohr's- Coulomb Failure Theories — Types of Laboratory Tests for Strength Parameters Strength Tests Based on Drainage Conditions — Critical Void Ratio —Liquefaction

TEXT BOOKS:

1. Soil Mechanics and Foundation Engg by K.R.Arora, Standard Publishers and Distributors Delhi 7th edition 2009
2. Geotechnical Engineering by C.Venkataramiah, New Age International Pvt. Ltd, (2002).
- 3.

REFERENCE BOOKS:

1. Soil Mechanics and Foundation by B.C.Punmia, Ashok Kumar Jain and Arun Kumar Jain, Laxmi, publications Pvt.Ltd., New Delhi 17th edition 2017
2. Geotechnical Engineering by Iqbal H.Khan, PHI Publishers, 4th edition.
3. Basic and Applied Soil Mechanics by Gopal Ranjan & ASR Rao, New age International Pvt.Ltd, New Delhi 3rd edition 2016

Online Learning Resources:

<https://nptel.ac.in/courses/105101201>

<https://nptel.ac.in/courses/105105185>

docs

Soil Classification and Compaction

Definition of Soil

Acc to agriculturist, it is the loose material existing on the earth which can support the plant.

Acc to Geologist, the top most layer of earth crust on earth's mantle is defined as soil.

Acc to Engineer, it is a naturally occurring unconsolidated particulate material.

Father of Soil is
Karl Terzaghi's

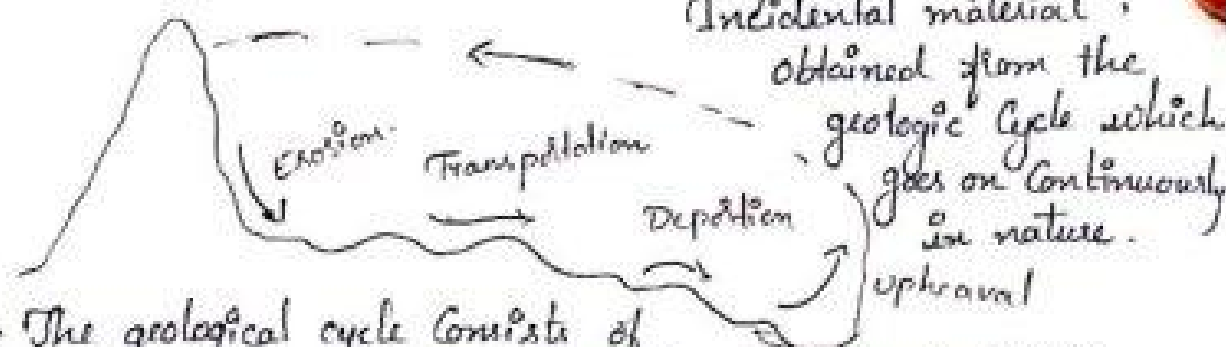
The unconsolidated and unconsolidated material existing on the earth, which can be used in the construction of material is said to be soil.

Origin of Soil

Soils are formed by weathering of rocks due to mechanical disintegration & chemical decomposition when a rock surface gets exposed to atmosphere for an appreciable time. It disintegrates & decomposes into small particles and thus the soils are formed.

Geologic Cycle

* Soil may be considered as an Incidental material.



- * The geological cycle consists of erosion, transportation, deposition & uplift of soil.
- * The products of erosion are picked up by agencies of transportation, such as water & wind, and are carried to new locations where they are deposited.
- * The shifting of material disturbs the equilibrium of forces on the earth.
- * If the soil stays at the place of its formation, just above the parent rock, it is known as Residual Soil or Secondary Soil.

* When the soil has been deposited at a place away from the place of its origin, it is called a Transported Soil.

Formation of Soil:-

As mentioned above, it is formed by either, * Chemical decomposition of Rocks or * Physical disintegration

→ When the soil has been deposited at a place away from the place of its origin, it is called a transported soil.

Formation of soil,

⇒ As mentioned above, soils are formed by either

(a) physical disintegration

(b) chemical decomposition of rocks

⇒ Physical Disintegration:- Physical disintegration

(a) mechanical weathering of rocks occurs due to the following physical processes:

① Temperature changes:- Different minerals of a rock have different coefficients of thermal expansion. Unequal expansion and contraction of these minerals occur due to temperature changes. When the stresses induced due to such changes are repeated many times, the particles get detached from the rocks and the soils are formed.

② wedging action of Ice:- Water in the pores and minute cracks of rocks gets frozen in very cold climates. As the volume of ice formed is more than that of water, expansion occurs. Rocks get broken into pieces when large stresses develop in the cracks due to wedging action of the ice formed.

③ spreading of roots of plants :- As the roots of trees and shrubs grow in the cracks and fissures of the rocks, forces act on the rock. The segments of the rock are forced apart and disintegration of rocks occurs.

④ Ablasion :- As water, wind and glaciers move over the surface of rock, abrasion and scouring takes place. It results in the formation of soil.

→ In all the processes of physical disintegration, there is no change in the chemical composition. The soil formed has the properties of the parent rock. coarse grained soils, such as gravel and sand, are formed by the process of physical disintegration.

⑤ Chemical decomposition :- when chemical decomposition or chemical weathering of rocks takes place, original rock minerals are transformed into new minerals by chemical reactions. The soils formed do not have the properties of the parent rock. The following chemical processes generally occur in nature.

① Hydration :- In hydration, ^{physical weathering} water combines with the rock minerals and results in the formation of new chemical ^{new minerals} ^{which are the cause of physical weathering}

compound. The chemical reaction causes a change in volume and decomposition of rock into small particles.

③ Carbonation:- It is a type of chemical decomposition in which carbon dioxide in the atmosphere combines with water to form carbonic acid. The carbonic acid reacts chemically with rocks and causes their decomposition.

④ Oxidation:- Oxidation occurs when oxygen ions combine with minerals in rocks. Oxidation results in decomposition of rocks. Oxidation of rocks is somewhat similar to rusting of steel.

⑤ Solution:- Some of the rock minerals form a solution with water when they get dissolved in water. Chemical reaction takes place in the solution and the soils are formed.

⑥ Hydrolysis:- It is a chemical process in which water gets dissociated into H^+ and OH^- ions. The hydrogen cations replace the metallic ions such as calcium, sodium and potassium in rock minerals. and soils are formed with a new chemical decomposition
→ chemical decomposition of rocks results in

formation of clay minerals. These clay minerals impart plastic properties to soils. clayey soils are formed by chemical decomposition.

water - H_2O - suspended soil
 river - H_2O - CO_2 & Ca^{2+} & Mg^{2+}
 sea - H_2O & $NaCl$

Transportation of soils :-

The soils formed at a place may be transported to other places by agents of transportation, such as water, wind, ice and gravity.

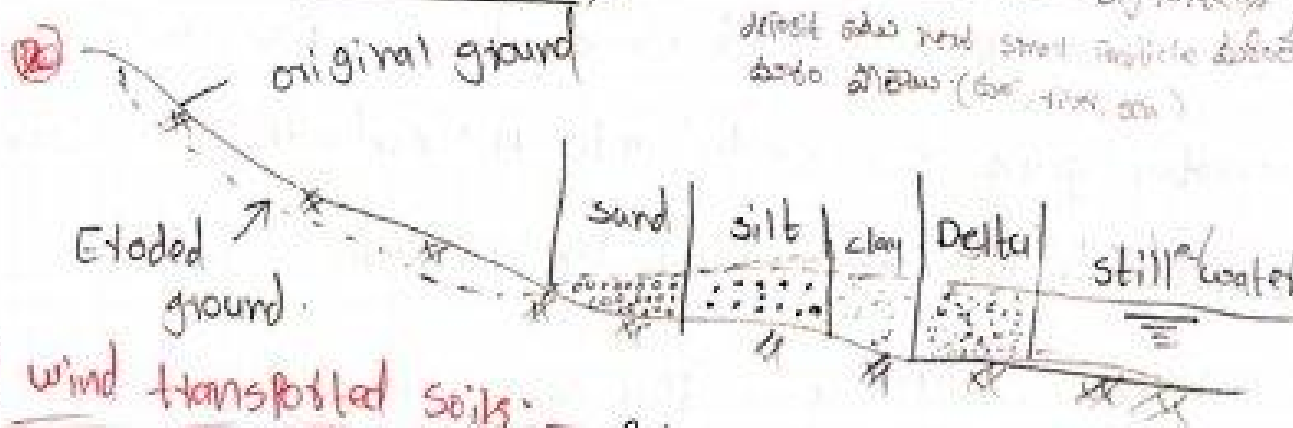
① water transported soils :-

Important agent for water
 fast R / low G3 water soil of
 rolling - H_2O & CO_2
 H_2O & $NaCl$

Flowing water is one of the most important agents of transportation of soils. Swift running water carries a large quantity of soil either in suspension or by rolling along the bed.

→ All type of soils carried and deposited by water are known as alluvial deposits.

water H_2O & CO_2 Big particles
 deposit also need small particles
 H_2O & $NaCl$ (low, river, sea)



② wind transported soils :-

Ice formed by the → soil particles are transported by winds. Glaciers are large masses of

→ The finer particles are carried far away from the place of the formation. — wind — ^{అంతరికీ విస్తరించే} - sand dunes ^{అంతరికీ విస్తరించే}

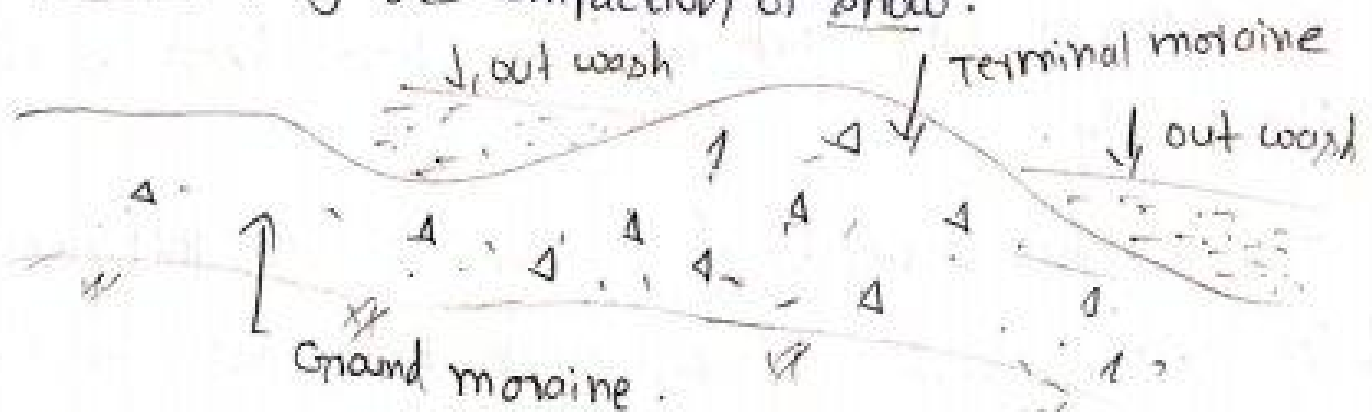
→ soils deposited by wind are known as aeolian deposits.

→ Large sand dunes are formed by wind. [sea with sandy beaches.]

→ Loess is a silt deposit made by wind. These deposits have low density and high compressibility.

→ The bearing capacity of such soils is very low.

③ Glacier deposited soils:- Glaciers are large masses of ice formed by the compaction of snow.



→ Deposits directly made by melting of glaciers are called till. ^{అంతరికీ విస్తరించే}

→ During their advancement, glaciers transport soil. At the terminus, a melting glacier drops the material in the form of ridges, known as terminal moraine.

→ The land which was once covered by glaciers and on which till has been deposited after melting is called ground moraine. The soil carried by the melting water from the

front of a glacier is termed out wash.

→ deposits of glacial till are generally well graded and can be compacted to a high dry density. These have generally high shearing strength.

④ Gravity-deposited soils:— soils can be transported through short distances under the action of gravity.

→ Rock fragments and soil masses collected at the foot of the cliffs or steep slopes had fallen from higher elevation under the action of the gravitational force.

→ Colluvial soils, such as talus, have been deposited by the gravity.

→ Talus consists of irregular, coarse particles. It is a good source of broken rock pieces and coarse-grained soils for many engineering works.

⑤ soils transported by combined action:—

→ Sometimes, two or more agents of transportation act jointly and transport the soil.

→ For example, a soil particle may fall under gravity and may be carried by wind to a far off place.

→ It might be picked up again by flowing water and deposited.

Major soil deposits of India

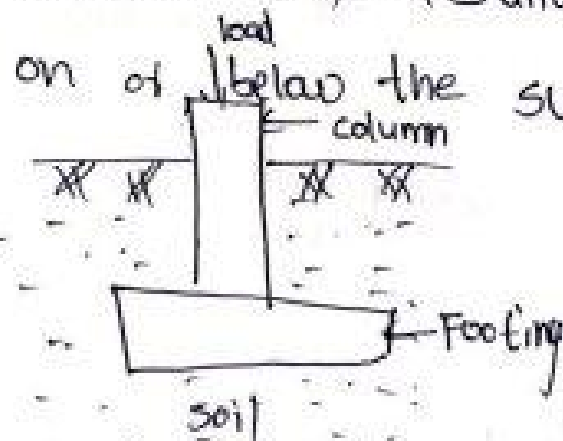
The soil deposits of India may be classified in the following five major groups, based on the climatic conditions, topography and geology of their formation.

- (1) marine deposits
- (2) Laterites and Lateritic soils
- (3) Black cotton soil
- (4) Alluvial soil
- (5) Desert soil

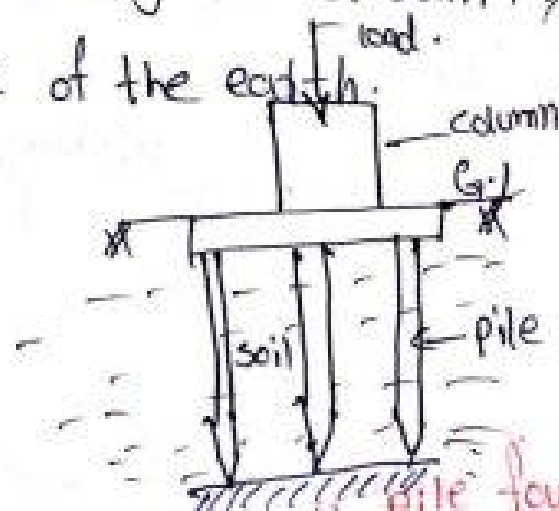
Scope of Soil Engineering :-

Soil engineering has vast application in the construction of various civil engineering works. Some of the important applications are as under.

(1) Foundations :- Every civil engineering structure, whether it is a building, a bridge, or a dam, is founded on or below the surface of the earth.

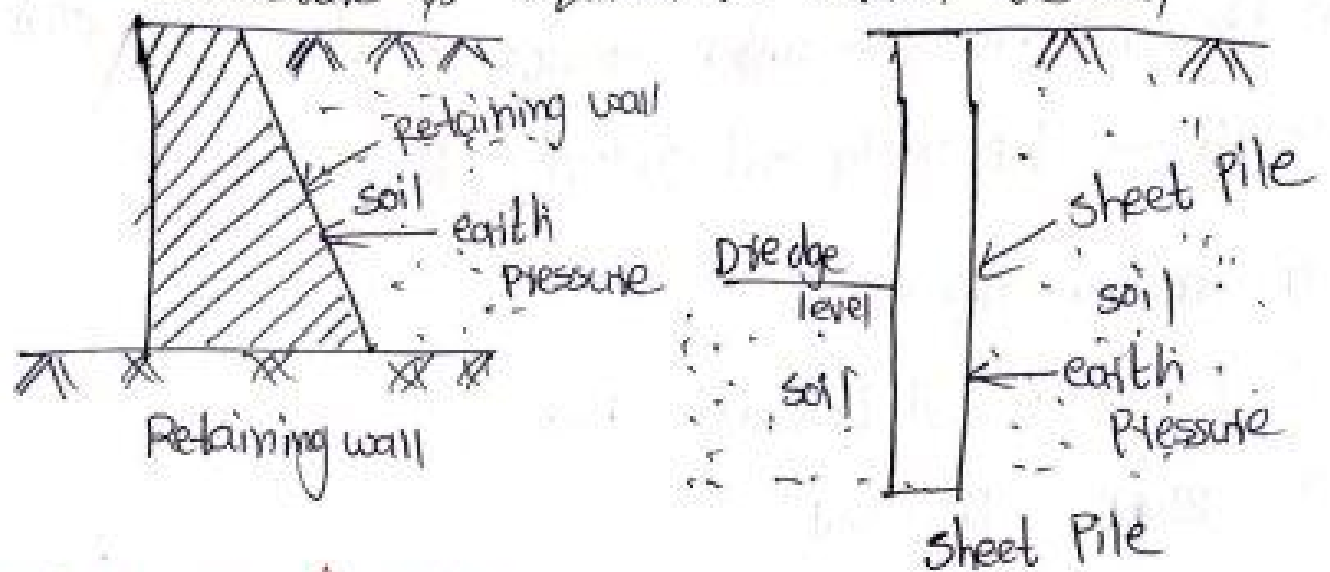


(a) Shallow foundation



(b) Pile foundation

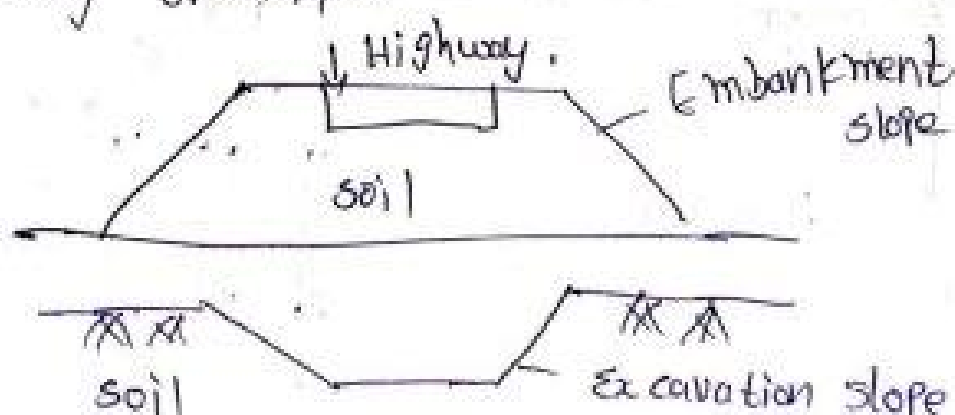
③ Retaining structures:- when sufficient space is not available for a mass of soil to spread and form a safe slope, a structure is required to retain the soil.



③ stability of slopes:- If soil surface is not horizontal, there is component of weight of the soil ~~which~~ which tends to move it downward and thus causes instability of slope.

The slopes may be natural (a) man-made the fig shows slopes in filling and cutting.

The soil engll provides the methods for checking the stability of slopes.

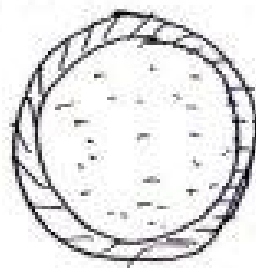


(4) underground structures :- The design and construction of underground structures, such as tunnels, shafts, and conduits, require evaluation of forces exerted by the soil on these structures.

→ These forces are discussed in soil engineering.

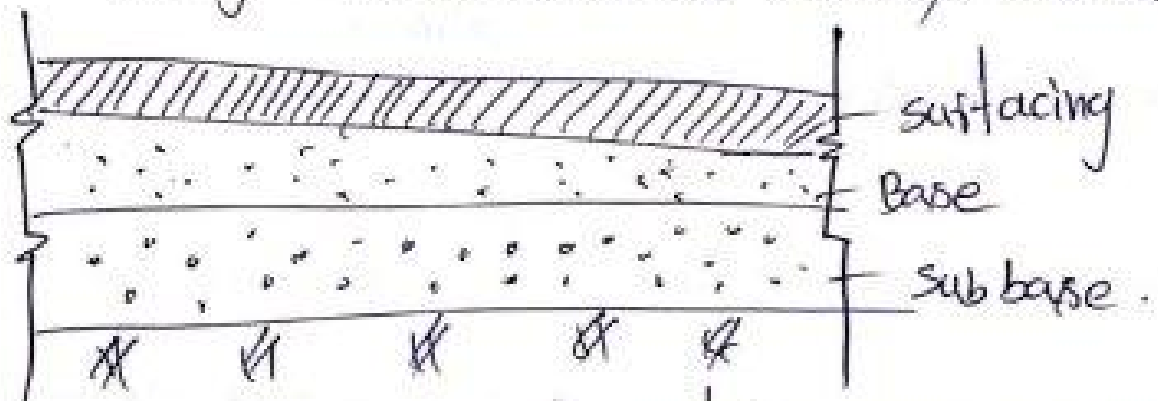


Tunnel



conduit.

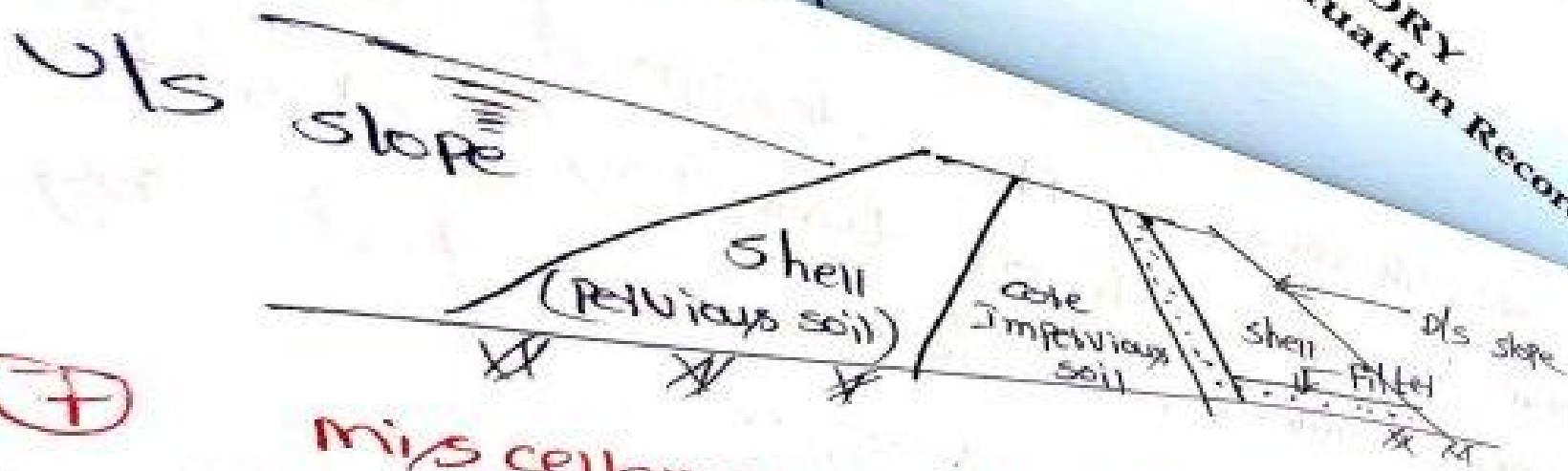
(5) Pavement Design :- A Pavement is a hard crust placed on soil (subgrade) for the purpose of providing a smooth and strong surface on which vehicles can move.



→ The pavements consists of surfacing such as a bitumen layer, base and subbase.

(6) Earth Dams :- Earth dams are huge structure in which soil is used as a construction material.

→ The earth dams are built for creating water repellents.



⊕

Miscellaneous soil problems

has sometimes to tackle miscellaneous problems related with soil.

properties of soil

Soil structure:-

The soil structure can be defined as the geometrical arrangement of particles of soil with respect to each other.

Soil cohesion:- A cohesive soil has an attraction b/w particles of the same type, origin and nature. Therefore, cohesive soils are a type of soil that stick to each other. Cohesive soils are the silts and clays, or fine grained soils.

Cohesionless soils:- Cohesionless soils are defined as any free-running type of soil, such as sand or gravel, whose strength depends on friction between particles. These are formed due to physical disintegration of rocks.

1.) Single Grained structure:-

The grains of cohesionless soils have less surface area and more gravitational force, when the grains pour some amount of soil on the

ground they will settle using gravitational force only. After attaining the final position each grain is in contact with the surrounding grains and the formed structure is called as single grained structure.

→ These structures are present in cohesionless soils like gravel and sand.

→ All the particles are spherical in nature then maximum void ratio $e = 0.90$ is occurred. When they are positioned in cubical manner it is called as loosest state of soil.

→ minimum void ratio $e = 0.35$ will occur if they are positioned in diagonal manner which is called densest state.

② Honey Comb structure :-

This type of structure is associated with silt deposits. When silt particles settle out of suspension in addition to gravitational forces the surface forces also play a significant role. When particles approach the lower terminus of suspension they will be

attracted by particles already deposited as well as the neighbouring particles leading to the formation of arches. The combination of a number of arches leads to the honeycomb structure as shown in fig 11. As the deposit has high void ratio, when disturbed as in pile driving, there will be large reduction in volume due to breakdown of structure.

③ Flocculated structure:-

clay particles are charged which have +ve charge on the edges and -ve charge on the face of the particles. when there is net attractive force b/w particles the +ve charged particles are attracted towards -ve charged faces. which result the formation of flocculated structures. These structure is present in clay particles, contains large surface area.

→ This type of soils has high shear strength, void ratio is high and water content will be maximum and compressability is very low.

(4) Dispersed structure: when clay soils are remoulded, their flocculated structure changes and particles change their orientation from edge to face orientation to face to face orientation. The kind of formed soil structure is called dispersed soil structure. Remolding reduces the shear strength of soil which reduces the net attractive forces b/w the particles. Hence, due to repulsion b/w them, the edge to face orientation turns into face to face orientation.

→ This type of structure is common in fresh water deposits.

Soil water. The engineering behaviour of soils is influenced by soil structure to varying degrees.

Soil structure :- → Geometrical arrangement of soil particles

→ It controls the behavior.

→ what is Geometrical arrangement → soil structure is a geometrical arrangement of soil particles in a soil mass.

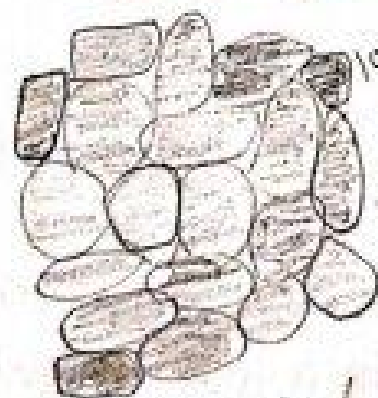
→ soil structure behavior or of soil difference on the soil structure.

→ soil structure behavior of the soil difference

Type of structure : main 5 soil structures

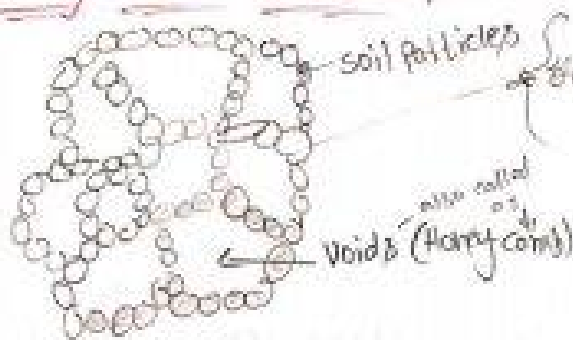
- (1) single grained structure → In the case of coarse-grained soil deposits
In Gravel & coarse sand
- (2) Honey-comb structure → In fine sand & silt deposits.
In the case of silt deposits.
- (3) Flocculated structure → In clays In the case of clay deposits
- (4) Dispersed structure → In Remoulded clays
- (5) Composite structure → In soil mix (clay + other soils)

(1) single grained structure :-



- Particles have good contact
- Particles settle independently of each other
- more shear strength (s.s)
- major force causing deposition is gravitational force
- Less void ratio → surface forces are too small
- contact particles are good to each other (s.s) are more circular, so less void ratio
- Particle diagonal shape of arrangement

- Silt deposits
 - The forces causing gravitational forces and surface forces also effect
 - The deposits has high void ratio
 - disturbance $\Rightarrow$ volume is decrease $\rightarrow$ can't resist vibrations
2. Honey comb structure

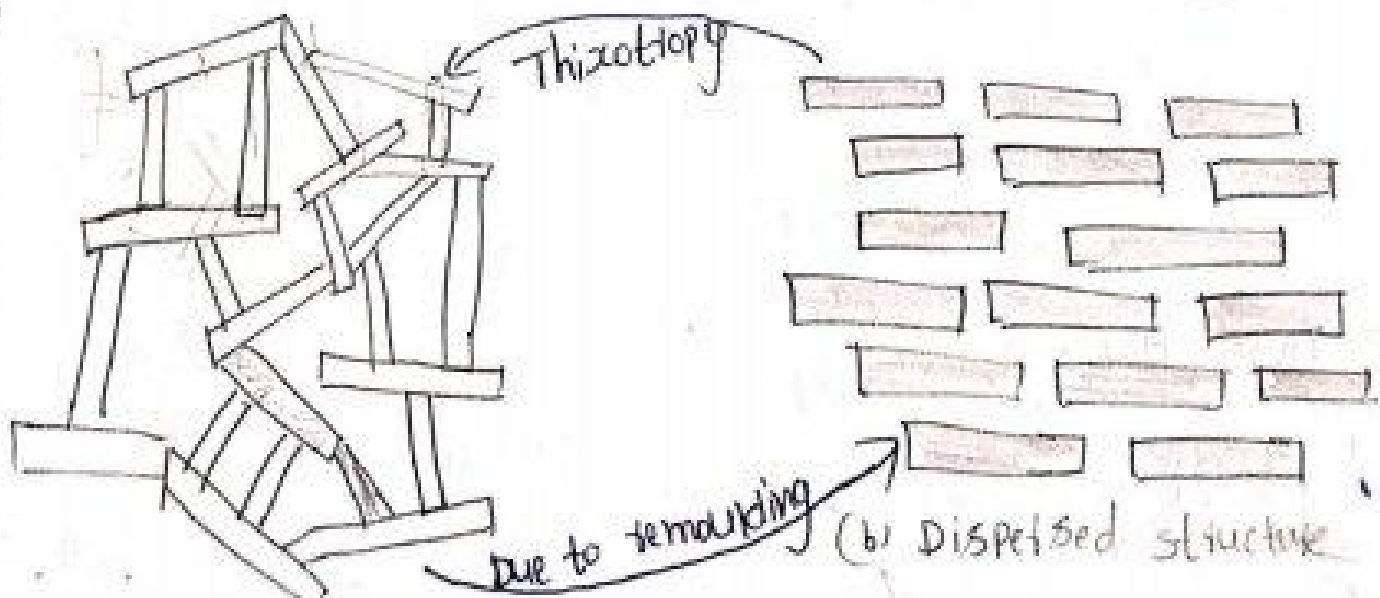


Honeycomb structure.

- soil's water add $\Rightarrow$ atmospheric pressure volume $\Rightarrow$ decrease $\Rightarrow$ structure is collapsed
- water add $\Rightarrow$ particles settle down side settle $\Rightarrow$ structure is collapsed
- $\Rightarrow$ vibrations $\Rightarrow$ create $\Rightarrow$ structure is collapsed

- If structure collapse on wetting (Volume decreases when water is adding)
- It is sensitive to vibrations. (Volume decreases due to vibrations)

3. Flocculated structure and Dispersed structure



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small distance

D.S

- Edge to face orientation
- Net ^{on} attraction ^{Edge to edge} (possible)
- Stable ^{net electrical forces} → high void ratio
- on sea bed
- more shear strength

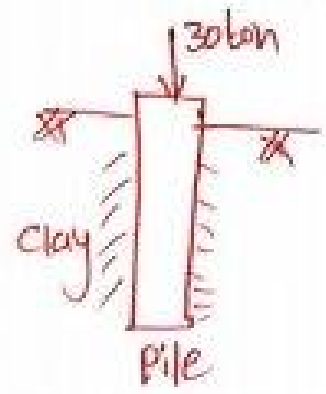
- Face to Face orientation
- Net repulsion ^(unstable) ^{net electrical forces}
- unstable
- on lake bed
- common in fresh water deposits
- Less shear strength
- high void ratio

→ Due to remoulding the strength of the clay decreases.

→ Thixotropy:- It is the phenomenon of regaining of strength with the passage of time under constant water content.

→ The Thixotropy property is significantly exhibited by Bentonite clay.

Ex



- In this clay after 6 months the strength of pile is > 35 ton.
- In sand no strength changes.
- Different soils to get diff. strengths.

Mineralogy

- Have ^{Rock minerals} no surface activity
- Present in gravel & sand

ex:- Quartz, mica, feldspar, etc.

- ^{clay minerals} Have surface activity (presence of some electro-chemical)
- Present in clays
- ex: Kaolinite, Illite, montmorillonite

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D.S

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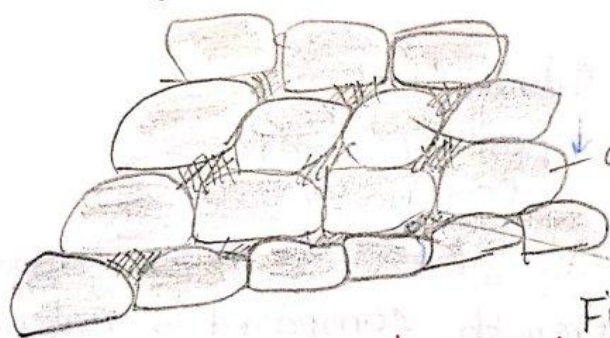
→ This type of structure is common in fresh water deposits.

→ structure of composite soils → mix of different soils.

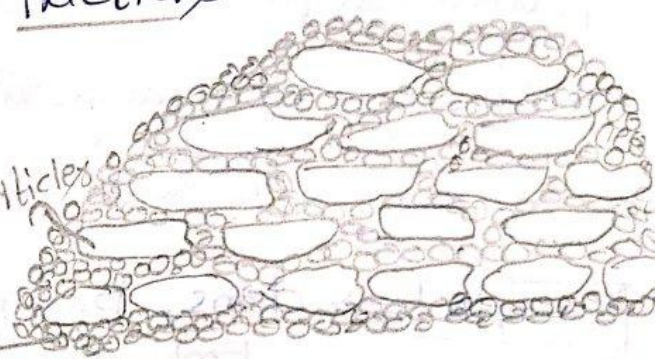
The following two types of soil structure are recognized in the case of composite soils.

- (1) coarse-grained skeleton structure
- (2) cohesive matrix structure.

→ Their formations depend on the relative proportions of coarse-grained and fine-grained fractions.



(a) Coarse grained skeleton structure



(b) Cohesive matrix structure

→ The coarse grained fraction is greater in proportion compared to fine grained fraction [c.g.p. ↑, f.g.p. ↓] (amount high, small)

→ The c.g.p. from the skeleton with particle to particle contact

→ The voids between these particles will be occupied by fine-grained particles.

→ less compressible in this structure.

→ The fine-grained fraction is more in proportion compared to coarse grained fraction. (f.g.p. ↑, c.g.p. ↓)

→ In this case the coarse-grained particles will be embedded in fine grained fraction.

→ particle to particle contact more compressible.

Importance of topic

→ soils are naturally, complex multi phase materials.

→ until we understand the composition of a soil mass, we will not be able estimate how it will behave under loads and how we can use it as a construction material.

1. origin of soils

$$[G_L = \overset{\text{Soil mech}}{S_m} + R_m + \overset{\text{Porewater form (const) composition}}{tE} + \overset{\text{Composition (s)}}{G_L}]$$

Soil:— It is a naturally occurring unconsolidated Particulated material.

→ Dr. Karl Terzaghi's [Father of soil mechanics]

origin of soils due to

↓
Weathering of rocks

↓
Results in inorganic soils

Ex:- Gravel, sand, silt, clay.

↓
Decomposition of organic matter.

↓
Results in "organic soil" or "Cumulose soil"

Ex:- Humus, Peat, muck

Phase diagrams

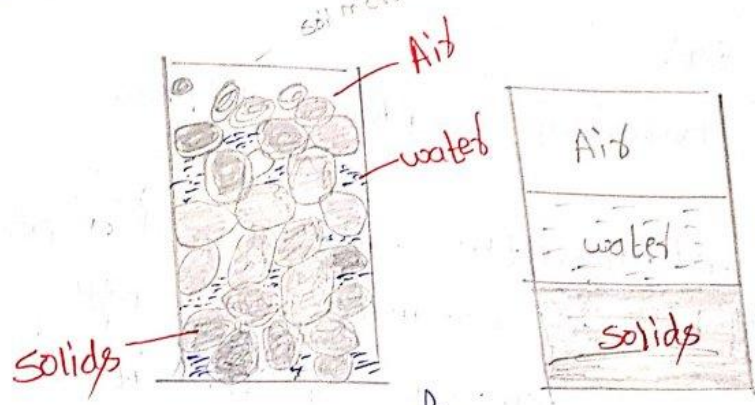
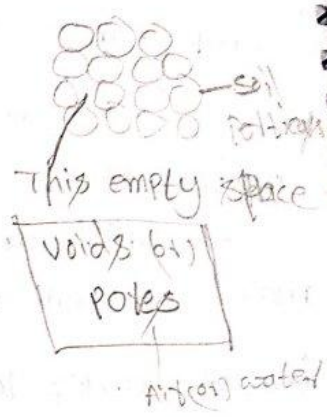


Fig-1



① 3-Phase Diagram

→ This diagram represents 3 components of soil -
 → Represents soil.
 → Voids consist of water and air.

- $W_w =$
- $W_s =$
- $W_a =$
- $V_a =$
- $V_w =$
- $V_s =$
- $V_v =$
- $V =$
- $W =$

Disassemble complex systems into parts and then study each part and its relationship to the whole.

- soil mass is a three phase system
- soil consists of 3 components - ① solids
 ② water
 ③ Air.

→ The diagram which represents these components of soil in separate spaces is called phase diagram.

→ There exists three types of phase diagrams as follows -

- ① 3-Phase Diagram (Partially saturated soil) - ex no. of voids = 10, 4 - water, 6 - Air.
- ② 2-Phase Diagram (Perfectly dry soil) - no. of voids = 10 - Air.
- ③ 2-Phase Diagram (Fully saturated soil) - no. of voids = 10 - water.

① 3- Phase Diagram [Partially saturated soil] :-

→ This diagram represents all the 3 components of soil - solids, water, air.

→ Represents partially saturated soil.

⇒ Voids contain both air and water.

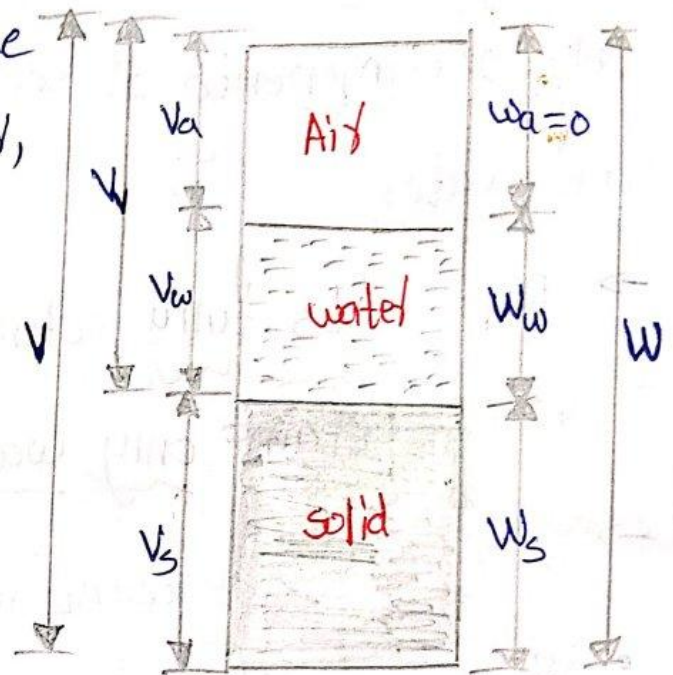


fig-2

Note
 $w_a = 0$ in soil Air weight is negligible

- W_w = weight of water
- W_s = weight of solids
- W_a = weight of air = 0
- V_a = volume of air
- V_w = volume of water.
- V_s = volume of solids
- V_v = volume of voids = $V_w + V_a$
- V = Total volume of soil mass = $V_s + V_v$
- W = Total ~~weight~~ ^{weight} of soil mass = $W_s + W_w$

$$V = V_v + V_s$$

$$V_v = V_a + V_w$$

$$V = V_a + V_w + V_s$$

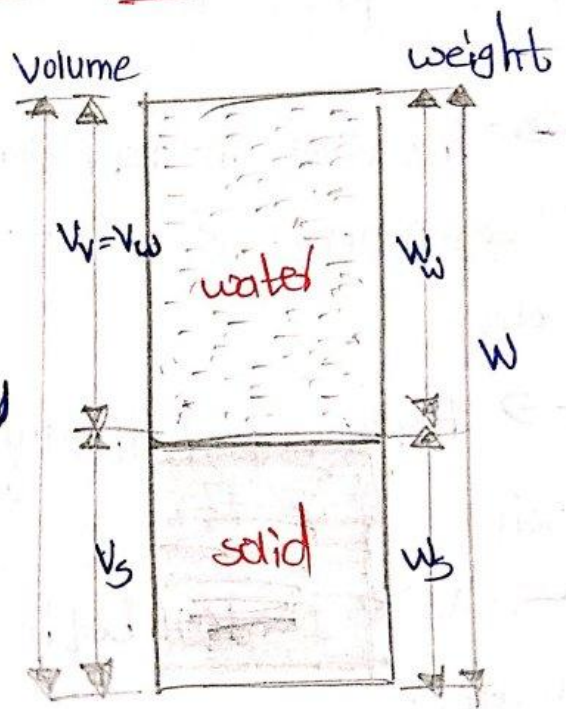
② 2- phase diagram [Fully saturated soil]

→ This diagram represents only 2 components of soil - solids and water.

→ Represents fully saturated soil

→ voids contain only water.

→ Air is not present in voids.



• W_w = weight of water

• W_s = weight of solids

• V_w = volume of water

• V_s = volume of solids

• V_v = volume of voids = V_w [$V_v = V_w$]

• V = Total volume of soil mass = $V_s + V_w$

• W = Total weight of soil mass = $W_s + W_w$

$$V = V_v + V_s$$

$$V_v = V_w$$

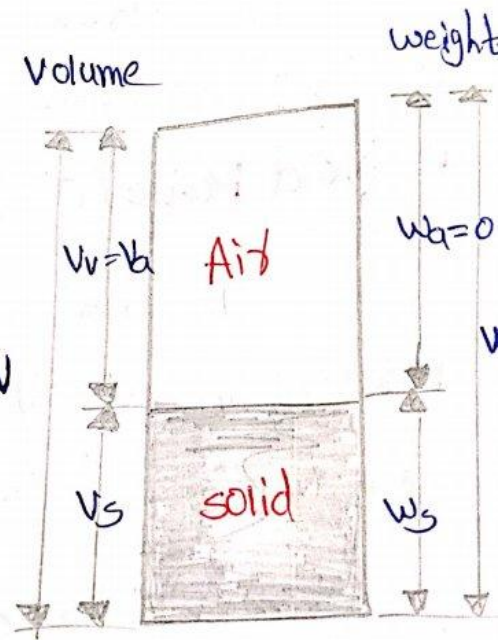
③ 2-Phase Diagram [Perfectly dry soil]

→ This diagram represents only 2 components of soil - solids and air

→ Represents Perfectly dry soil

→ Voids contain only air

→ water is not present in voids



- W_a = weight of air = 0

- W_s = weight of solids

- V_a = volume of air

- V_s = volume of solids

- V_v = volume of voids = V_a

- V = Total volume of soil mass = $V_s + V_a$

- W = Total weight of soil mass = W_s

$$\left. \begin{aligned} V &= V_a + V_s \\ W &= W_a + W_s \\ \boxed{W &= W_s} \end{aligned} \right\}$$

VOLUMETRIC RELATIONSHIPS:-

The following five volumetric relationships are widely used in soil engineering.

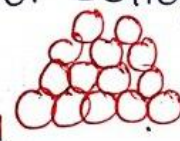
① Void Ratio (e):-

Defined as the ratio of volume of voids present in a soil mass to the volume of solids in the soil mass.

(a)



$e_{min} = 0.91$
Cubical array
(loosest)



$e_{min} = 0.35$
Diagonal
Packing
(densest)

It is defined as the ratio of the volume of voids to the volume of solids.

$$e = V_v / V_s$$

no units

(or)

Void ratio =

$\frac{\text{Volume of voids}}{\text{Volume of solids}}$

- * Expressed as a fraction or decimal
- * Most reliable value as compared to Porosity
- * $e > 0$, always positive $V = V_a + V_w + V_s$
- * can never be 0 ($V_v = 0$ can't be zero)
- * It can be greater than (or) equals to 1
[If $V_v = 0$, $e = 0$ This is not soil, only rock]
- * Therefore, we have $0 < e$ with no upper limit.
- * For coarse grained soils, the void ratio is generally smaller than that for fine grained soils.

se widely

(2) Porosity (n):- Defined as the ratio of total volume of Voids Present in a soil mass to the total volume of soil mass.

$$n = \left(\frac{V_v}{V} \right) \times 100\% \quad \text{Percentage} \quad \frac{\text{Volume of voids}}{\text{Total volume of soil}}$$

* Expressed as Percentage (%)

* $n > 0$, always positive

* can never be 0 ($n \neq 0$) ($n \neq 100\%$)

* can never be greater than or equal to 100%.

* Therefore, we have $0 < n < 100$ (0 to 100 middle of soil mass but not exact 100%)

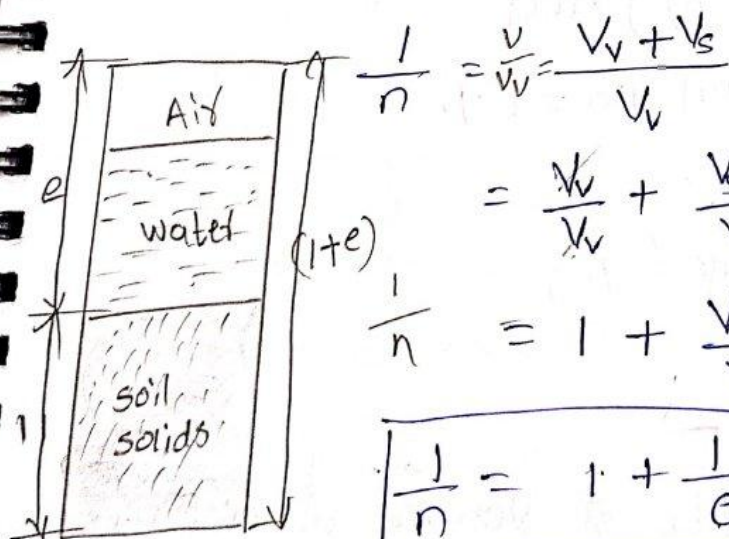
* Both Porosity and void ratio are measures of the denseness (or looseness)

* It is more convenient to use void ratio than porosity.

Relation Between Porosity & Void ratio:- none measure in soil dense or loose and void ratio measure is better.

$$n = \frac{V_v}{V} \Rightarrow \frac{1}{n} = \frac{V}{V_v}$$

$$[\because V = V_s + V_v]$$



soil elements in terms of e.

$$\frac{1}{n} = \frac{V}{V_v} = \frac{V_s + V_v}{V_v}$$

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

$$\frac{1}{n} = 1 + \frac{V_s}{V_v}$$

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

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

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

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

$$\therefore \frac{1}{e} = \frac{V_s}{V_v}$$

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

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

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

③ Degree of saturation: - (S) or (S_r)

Defined as the ratio of volume of water present in the voids to the total volume of voids in a given soil mass.

$$S = (V_w / V_v) \times 100 \%$$

- Expressed as Percentage (%).
- For perfectly dry soil, $S = 0\%$.
- For fully saturated soil, $S = 100\%$.
- For partially saturated soil, $0 < S < 100$

$$V_v = V_a; (V_w = 0)$$

$$V_v = V_w; (V_a = 0)$$

$$V_v = V_w + V_a \\ V_v \neq V_w$$

④ Air Percentage Air Voids (n_a): -

Defined as the ratio of volume of air present in the voids to the total volume of given soil mass

$$n_a = (V_a / V) \times 100$$

- For fully saturated soil, $n_a = 0\%$.
- It can never be 100%.
- Expressed as Percentage (%).

$$V_v = V_w$$

⑤ Air content (a_c): -

Defined as the ratio of volume of air present in the voids to the total volume of voids in a given soil mass.

$$a_c = (V_a / V_v) \times 100$$

- Expressed as Percentage (%)
- For fully saturated soil; $a_c = 0\%$.
- For perfectly dry soil, $a_c = 100\%$.
- For partially saturated soil, $0 < a_c < 100\%$.

$$\left. \begin{array}{l} V_v = V_w \\ V_a = 0 \\ V_v = V_a \\ V_w = 0 \end{array} \right\}$$

$\therefore$ Both air content and the percentage of voids are zero; when the soil is saturated ($V_a = 0$).

⇒ Inter Relationship between the % air voids and the air content can be obtained:-

$$n_a = \frac{V_a}{V} = \frac{V_a}{V} \times \frac{V_v}{V_v} = \left[\frac{V_a}{V_v} \right] \times \left[\frac{V_v}{V} \right]$$

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

$$\left[\because a_c = \frac{V_a}{V_v} \right]$$

$$\left[\because n = \frac{V_v}{V} \right]$$

⇒ Relation b/w weight Ratios :-

(i) water content (w) :-

defined as the ratio of weight of water present in a soil mass with respect to solids to the weight of solids in the soil mass.

$$\boxed{w = (W_w / W_s) \times 100}$$

→ Represents the quantity of water present in a soil mass with respect to solids.

→ For perfectly dry soil, $w = 0\%$.

→ It can be greater than 100% . (also 100 above in this value)

∴ weight of solids = weight of dry soil

$$w_s = w_d$$

It is also expressed as $w = \frac{\text{weight of water}}{\text{weight of dry soil}}$

$$w = \frac{w_w}{w_d}$$

(7) unit weight:—(γ) It is the ratio of weight of soil to the volume of soil

$$\text{unit weight} = \frac{\text{weight of soil}}{\text{Volume of soil}}$$

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

(a) Bulk unit weight (γ) It is the ratio of total weight of soil to the total volume of soil.

$$\gamma = w/v$$

$$\text{Bulk unit weight} = \frac{\text{Total weight of soil}}{\text{volume of soil}}$$

(b) dry unit weight (γ_d) It is the ratio of weight of solids to the total volume of soil.

$$\text{dry unit weight} = \frac{\text{weight of solids}}{\text{Total Volume of soil}}$$

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

$\therefore$ It is also defined as

$$\text{dry unit weight} = \frac{\text{weight of dry soil}}{\text{Total Volume of soil}}$$

$$\gamma_d = \frac{W_d}{V}$$

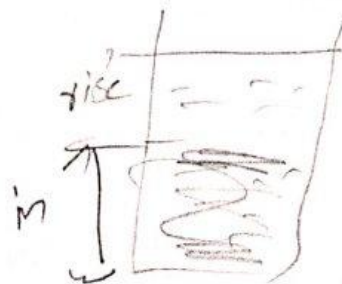
(c) saturated unit weight $:- (\gamma_{sat})$ It is the ratio of saturated weight of the soil to the Total Volume of soil.

$$\text{saturated unit weight} = \frac{\text{saturated weight of soil}}{\text{Total Volume of soil}}$$

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

(d) submerged unit weight $(\gamma_{sub} \text{ or } \gamma')$ $:-$ It is the ratio of weight of submerged soil to the total Volume of soil.

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



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

$\gamma_w =$ unit weight of water.

(e) unit weight of solids (γ_s) :- It is the ratio of weight of solids to the volume of solids.

$$\gamma_s = \frac{\text{weight of solids}}{\text{Volume of solids}}$$

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

mass no change in any location for example bottle
weight changed in particle locations (or) planets
depend on gravitational force.

Density of soil (ρ) It is the ratio of mass per unit

Volume

[density means the mass per unit volume & not weight]

(a) Bulk density, $\rho = \frac{\text{mass of soil}}{\text{Volume of soil}} = \frac{M}{V}$

(b) Dry density, $\rho_d = \frac{\text{mass of dry soil}}{\text{Volume of soil}} = \frac{M_d}{V}$

(c) saturated density, $\rho_{sat} = \frac{\text{mass of saturated soil}}{\text{Volume of soil}}$

$$= \frac{M_{sat}}{V}$$

(d) submerged density, ρ_{sub} (or) ρ' = mass of submerged soil
 $= \frac{M_{sub}}{V}$

Relation i

$$\rho_w =$$

∴ S.I. unit

Bulk

(e) Density

⇒ Spec

(1) Spec

the for

Particle

water

more

of

part

Relation is $\rho_{sub} = \rho_{sat} - \rho_w$

ρ_w = density of water.

= 1 gm/cm^3 in CGS
constant

= 1000 kg/m^3 in M.K.S

$\therefore$ S.I system (International system of units),

Bulk unit weight $\gamma = 1000 \times 9.81 = 9810 \text{ N/m}^3$

= 9.81 kN/m^3

② density of solids :-
$$\frac{\text{mass of solids}}{\text{volume of solids}} = \frac{m_s}{V_s}$$

$\Rightarrow$ specific gravity :- $\left[\begin{array}{l} \text{any substance mass density / reference} \\ \text{substance mass density (if water is reference)} \end{array} \right]$

① specific gravity of soil particles (or solids) (G_s or G_s) :-

① specific gravity of soil particles is defined as the ratio of the weight of a given volume of soil

particles to the weight of an equivalent volume of pure water at a stated ^{4°C} (different temp change doesn't stand put change) temperature. ② It is denoted by G_s (or)

more usually G_r . It can also be defined as the ratio of unit weight of soil particles to the unit weight of pure water at stated temperature. (4°C)

$$G_s = \frac{\text{weight of solids of certain volume}}{\text{weight of water of same volume.}}$$

$$= \frac{\text{mass of solids of certain volume}}{\text{mass of water of same volume}}$$

$$(or) = \frac{W_s/V_s}{W_w/V_w} = \frac{V_s}{V_w} (or) \frac{\rho_s}{\rho_w}$$

specific gravity of substance is defined as the ratio of the density of (or) that substance to the density of reference letter G_s substance (water). It is denoted by the $G_s = \frac{\text{density of material}}{\text{density of water at } 4^\circ C}$ $\rho_m = \text{unit less}$

$G_s > 1$ = heavier than water
 $G_s < 1$ = lighter than water.

Typical values of specific gravity of soil particles

soil type	value of G_s
sand	2.64 - 2.67
silt	2.68 - 2.70
clay	2.70 - 2.80
soil containing mica(ion)	2.85 - 2.90
organic soils	1.26 - 2.20

Specific

Soil mass
 to the unit
 → It is
 → It has n

Specific Function

① Relati

Hence

$$\frac{1}{G} = \frac{\gamma_w}{\gamma}$$

tain volume

volume

of the
reference
by the
ess-

② Specific gravity of soil mass:- specific gravity of

soil mass is the ratio of bulk unit weight of soil mass to the unit weight of pure water as a stated temperature

→ It is denoted by G_m

→ It has no units. weight of soil of certain volume

weight of water of same volume

$$G_m = \frac{W/V}{W_w/V_w} = \frac{\gamma}{\gamma_w} \quad (\text{or}) \quad \frac{\rho}{\rho_w}$$

[Specific Gravity of soil is determined by "Pycnometer method"]

Functional Relations:-

① Relationship between e, G, W, S

$$\text{water content } w = \frac{W_w}{W_s} = \frac{\text{weight of water}}{\text{weight of solids}}$$

$$\therefore \text{unit weight of water } \gamma_w = \frac{W_w}{V_w}$$

$$W_w = \gamma_w \times V_w$$

$$\therefore \text{unit weight of solids } \gamma_s = \frac{W_s}{V_s}$$

$$W_s = \gamma_s \times V_s$$

$$\text{Hence } w = \frac{W_w}{W_s} = \frac{\gamma_w \times V_w}{\gamma_s \times V_s} \Rightarrow \frac{1}{G} \cdot \frac{V_w}{V_s} = \frac{1}{G} \cdot \frac{S V_w}{V_s}$$

$$\frac{1}{G} = \frac{\gamma_w}{\gamma_s} \quad \boxed{G = \frac{\gamma_s}{\gamma_w}} \quad \boxed{S = \frac{V_w}{V_s} \rightarrow V_w = S \times V_s}$$

$$w = \frac{1}{e} \times \frac{S \cdot V_v}{V_s}$$

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

$$w = \frac{S}{e} \times e$$

$$\left[e = \frac{V_v}{V_s} \right]$$

$$w e = S e$$

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

2(a)
R15-2017
Relation between γ , e , G , S , γ_w , w :-

$$\text{Unit weight, } \gamma = \frac{w}{V}$$

$$= \frac{w_s + w_w + w_a}{V_s + V_v}$$

$$\therefore w_a = 0 = w_s + w_w =$$

$$\frac{V_s + V_v}{V_s + V_v}$$

$$\frac{w_s + (w_w \times \frac{w_s}{w_s})}{V_s + (V_v \times \frac{w_s}{w_s})} = w_s \left(1 + \frac{w_w}{w_s} \right)$$

$$\frac{V_s \left(1 + \frac{V_v}{V_s} \right)}{V_s}$$

$$\gamma = \frac{w_s}{V_s} \cdot \left(1 + \frac{w_w}{w_s} \right) \left(1 + \frac{V_v}{V_s} \right)$$

$$\rightarrow \text{Unit weight } \gamma = \frac{w}{V}$$

$$\rightarrow \text{Void ratio } (e) = \frac{V_v}{V_s}$$

$$w_s + w_w \rightarrow S \cdot G \cdot \gamma_w = \frac{\gamma_s}{S \cdot G}$$

$$\rightarrow \text{Degree of saturation } (S) = \frac{V_w}{V_v}$$

$$\rightarrow \text{Unit weight of water } (\gamma_w)$$

$$\rightarrow \text{water content } w = \frac{w_w}{w_s}$$

$$\rightarrow \text{unit weight of solids } (\gamma_s) = \frac{w_s}{V_s}$$

(3)

Relatio

$$\gamma_{sat} =$$

$$\gamma =$$

when $S =$

Degree

For 1.

$$s = \frac{V_s}{V} \frac{(1+w)}{(1+e)}$$

$$\therefore G = \frac{s}{w}$$

$$V_s = G \cdot V_w$$

$$s = \frac{G \cdot V_w (1+w)}{(1+e)}$$

$$= \frac{V_w (G + Gw)}{(1+e)}$$

$$s = \frac{V_w [G + eG]}{1+e}$$

$$\therefore wG = se$$

(3) Relation between s , G , e , w

$$s_{\text{sat}} = \text{saturated unit weight} = \frac{\text{weight of saturated soil}}{\text{Volume of soil}}$$

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

$$s = \text{unit weight} = \frac{W}{V}$$

when soil is fully saturated $w = w_{\text{sat}}$

$$s = s_{\text{sat}}$$

Degree of saturation, $S = \frac{V_w}{V_v}$

For fully saturated soil, $S = 1$

$$\therefore [V_v = V_w] \cdot V_v = 0$$

when $S=1$, $S = S_{sat}$

we know that $S = \left[\frac{G + eS}{(1+e)} \right] \times S_w$

$$S_{sat} = \left(\frac{G + e_{max}}{1+e} \right) S_w$$

$$S_{sat} = \frac{(G + e)}{(1+e)} S_w$$

(4) Relation between S_d , G , e , S_w :-

S_d for dry soil degree of saturation = 0

when the soil is dry S will become S_d

$$S = S_d \text{ when } S=0 \text{ (dry soil)}$$

$$S = \frac{(G + eS)}{1+e} S_w$$

$$S_d = \frac{w}{V}$$

$$= \frac{(G + e(0))}{1+e} S_w$$

$$S_d = \frac{G}{1+e} S_w$$

⑤ Relation between γ_{sub} , G , e , γ_w

$$\gamma_{sub} = \gamma_{sat} - \gamma_w \quad \text{unit weight of water.}$$

The above relations

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

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

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

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

⑥ Relation between γ_d , γ , w

$$\text{Bulk unit weight, } \gamma = \frac{W}{V} = \frac{w_s + w_w}{V} \times \frac{w_s}{w_s}$$

γ_d = dry unit weight

w = water content

$$= \frac{w_s}{V} \left(1 + \frac{w_w}{w_s} \right)$$

$$\therefore \frac{w_s}{V} = \gamma_d, \quad \frac{w_w}{w_s} = w \quad \gamma = \gamma_d (1+w)$$

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

⑦ Relation between γ_d, G, n_a, w :-

$$n_a = \% \text{ air voids} = \frac{\text{Volume of air}}{\text{Total Volume}} = \frac{V_a}{V}$$

$$w = \text{water content} = \frac{w_w}{w_s}$$

$$\text{Total Volume } V = (V_s + V_w + V_a) \times \frac{V}{V}$$

$$\frac{V}{V} = \frac{V_s}{V} + \frac{V_w}{V} + \frac{V_a}{V}$$

$$1 = \frac{V_s}{V} + \frac{V_w}{V} + n_a$$

$$1 - n_a = \frac{V_s}{V} + \frac{V_w}{V}$$

$$\left[\begin{array}{l} \gamma_s = \frac{w_s}{V_s} \quad ; \quad \gamma_w = \frac{w_w}{V_w} \\ V_s = \frac{w_s}{\gamma_s} \quad \quad V_w = \frac{w_w}{\gamma_w} \end{array} \right]$$

$$1 - n_a = \frac{\frac{w_s}{\gamma_s}}{V} + \frac{\frac{w_w}{\gamma_w}}{V}$$

$$= \frac{w_s}{G \gamma_w} \times \frac{1}{V} + \frac{w \cdot w_s}{\gamma_w} \times \frac{1}{V}$$

$$1 - n_a = \frac{1}{G \gamma_w} \left(\frac{w_s}{V} \right) + \frac{w}{\gamma_w} \left(\frac{w_s}{V} \right)$$

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

$$\gamma_s = G \gamma_w$$

$$\therefore w = \frac{w_w}{w_s}$$

$$w_w = w \gamma_w$$

$$\therefore \gamma_d = \frac{w_s}{V}$$

$$(1-n_a) = \frac{s_d}{G \cdot s_w} + \frac{w s_d}{s_w} = \frac{s_d}{s_w} \left(\frac{1}{G} + w \right)$$

$$(1-n_a) = \frac{s_d}{s_w} \left(\frac{1}{G} + w \right) = \frac{s_d}{s_w} \left[\frac{1+wG}{G} \right]$$

$$s_d = \frac{(1-n_a) \times s_w \times G}{(1+wG)}$$

Functional Relationships :-

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

$$2. n = \frac{e}{1+e}$$

$$3. n_a = n \cdot a_c$$

$$4. e = \frac{wG}{S}$$

$$5. s = \frac{(G+e)s_w}{1+e}$$

Interms of density

$$\rho = \frac{(G+e)s_w}{1+e} \cdot \rho_w$$

$$6. s_{sat} = \frac{(G+e)s_w}{1+e}$$

$$\rho_{sat} = \frac{(G+e)s_w}{1+e} \cdot \rho_w$$

$$7. s_d = \frac{G s_w}{1+e}$$

$$\rho_d = \frac{G \rho_w}{1+e}$$

$$8. s_{sub} \text{ (or) } s' = \frac{(e-1)s_w}{1+e} \rightarrow p_{sub} = \frac{(e-1)}{1+e} p_w$$

$$9. s_d = \frac{s}{1+w} \quad \text{---} \quad p_d = \frac{p}{1+w}$$

$$10. s_d = \frac{(1-n_a)s_w \cdot G}{(1+we_g)} \quad \text{---} \quad p_d = \frac{(1-n_a)p_w \cdot G}{(1+we_g)}$$

Problem

① In a soil water and air occupied $1/3^{rd}$ and $1/6^{th}$ of soil. what is the void ratio of soil.

Ans

$$V_w = V/3$$

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

$$V_a = V/6$$

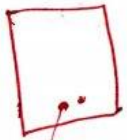
$$e = ?$$

$$V_v = V_w + V_a = \frac{V}{3} + \frac{V}{6} = \frac{3V}{6} = 0.5V$$

$$V_s = V - V_v = V - 0.5V = 0.5V$$

$$\therefore e = \frac{V_v}{V_s} = \frac{0.5V}{0.5V} = 1$$

② In a class room & particles are these. what is the void ratio?



Solids (n) = 0
Particles

$$e = \frac{V_v}{V_s} = \frac{(\text{---})}{0} = \infty$$

rice & stones
बबू बबू बबू

⇒ Soil description:- Soil is a complex natural body composed of minerals, organic matter, water, air, and living organisms. It forms at the Earth's surface and provides support for plant life, acting as a vital component for ecosystems. Soil is characterized by its horizons (layers) and its ability to support rooted plants.

Key aspects of soil description:-

1) Composition:- Soil consists of inorganic materials (like weathered rocks and minerals) and organic matter (decaying plant and animal material).

2) Texture:- Soil texture refers to the proportions of sand, silt, and clay particles, which influence water and nutrient retention.

3) Structure:- Soil structure describes how soil particles aggregate together, impacting porosity and water movement.

4) Colour:- Soil colour can indicate the presence of organic matter (darker colours) or mineral content (lighter colours).

Physical Properties:-

These include texture, structure, porosity, density, and temperature, all affecting soil's ability to support plant life.

Chemical Properties:- These include texture, ~~structure~~ soil chemistry, including pH and nutrient content, also influences plant growth.

Biological component:- soil is teeming with a vast array of living organisms from microbes to larger invertebrates, that contribute to nutrient cycling and soil health.

Terminology of Different types of soils

- ① Bentonite
- ② Black cotton soil
- ③ Boulders.
- ④ calcareous soils
- ⑤ caliche
- ⑥ clay
- ⑦ cobbles
- ⑧ Diatomaceous earth
- ⑨ Dispersive clay
- ⑩ Dune sand
- ⑪ Expansive clays

(13) Fills

(13) Gravel

(14) Hard Pans

(15) Humus

(16) Kalkar

(17) Laterites

(18) Loam.

(19) Loess

(20) marl

(21) morum

(22) muck

(23) Peat

(24) sand

(25) silt

(26) Till

(27) Top soils

(28) Tuft

(29) Tundra

(30) Varved clays.

⇒ soil Particle size and shape

The size of particles as explained earlier may range from gravel to the finest size possible.

There are different sizes

Coarse & Fine grained soils

CGS

(1) more than 50% > 75μ

(2) cohesionless soils

(3) Ex:- Gravel, sand

(4) non plastic soils

(5) Amount of water absorbed
Less

(6) Electrochemical forces between
particles are weak

FGS

(1) more than 50% < 75μ.

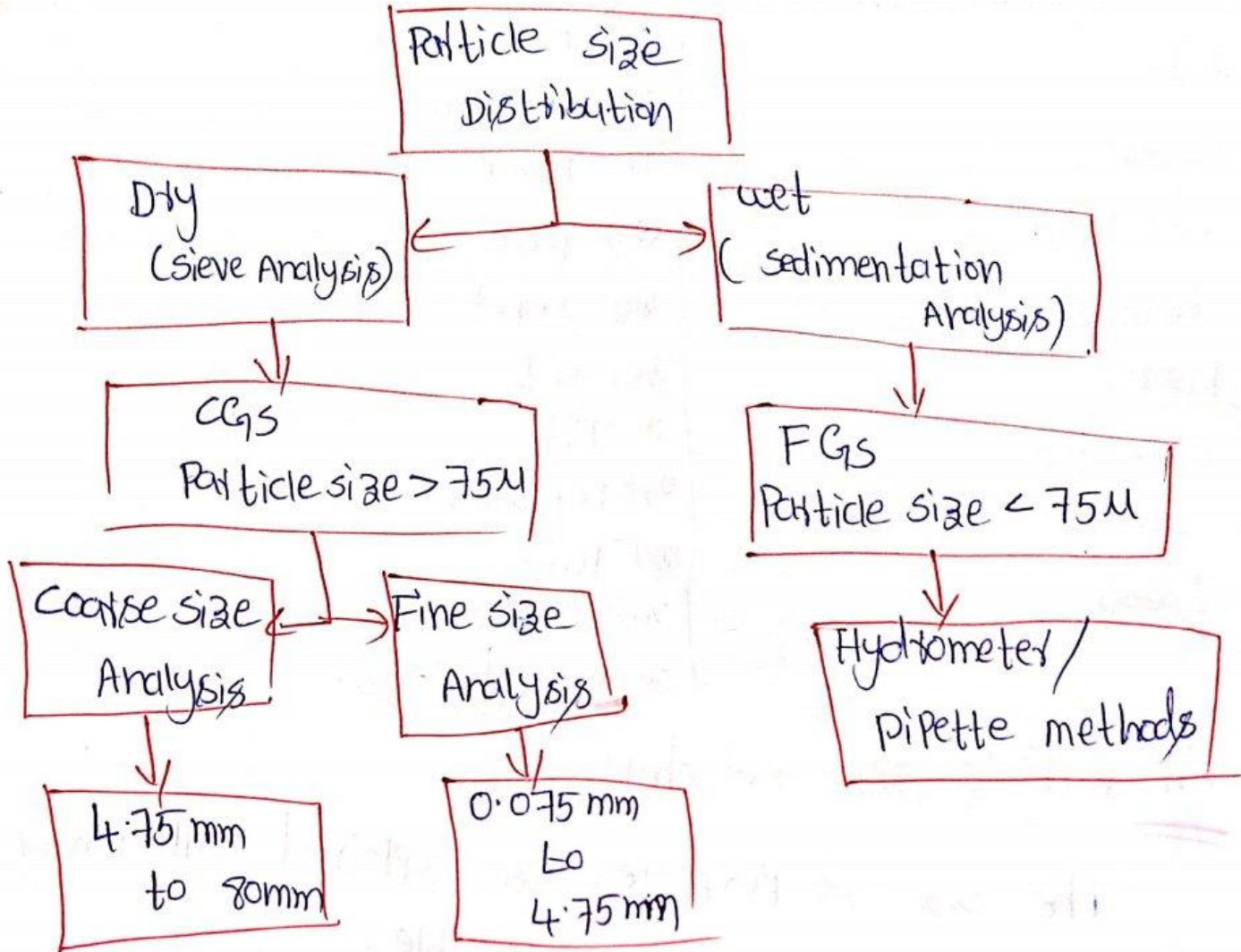
(2) cohesive soils

(3) Ex:- silt, clay.

(4) Plastic soils

(5) Amount of water absorbed
more.

(6) Electrochemical forces b/w
particles are strong



Index Properties
determination

⇒ Sieve Analysis :-

Objective :- To determine grain size distribution of soil

Equipment :- Sieves of all sizes 4.75 mm to 100 mm and 4.75 mm to 75 μ, weigh Balance with accuracy of 0.01 gm.

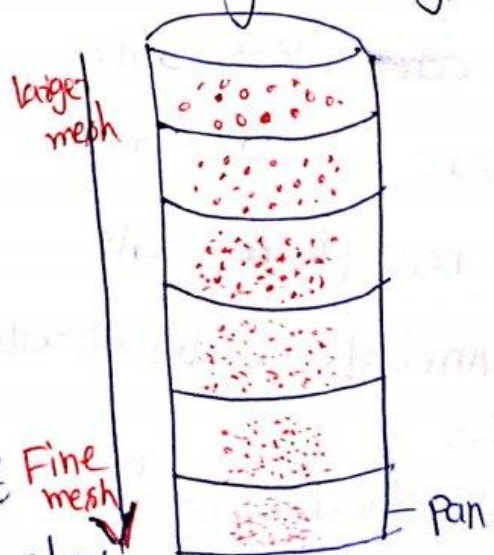
Reference : IS 2720 (Part-4)-1985

Procedure :-

1) Take 1000 gm of soil sample from disturbed representative sample.

2) conduct sieve analysis using a set of standard sieves as given in the data sheet

3) The sieving may be done either by hand or by mechanical



sieve for 10 mins.

4. weigh the material retained on the sieve.

5. The Percentage retained on each sieve is calculated on the basis of the total weight of the soil sample taken.

6. From these results the percentage passing through each of the sieves is calculated.

7. Draw the grain size curve for the soil in the semi logarithmic graph provided.

Observation:-

Sl. no	IS sieve	mass Retained (gms)	% mass retained	cumulative Retained	cumulative % passing
1	20mm				
2	10mm				
3	4.75mm				
4	2.0mm				
5	1.0mm				
6	600μm				
7	425μ				
8	300μ				
9	150μ				
10	75μ				

uniformity coefficient (C_u)

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

coefficient of curvature (C_c) or coefficient of gradation (G).

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

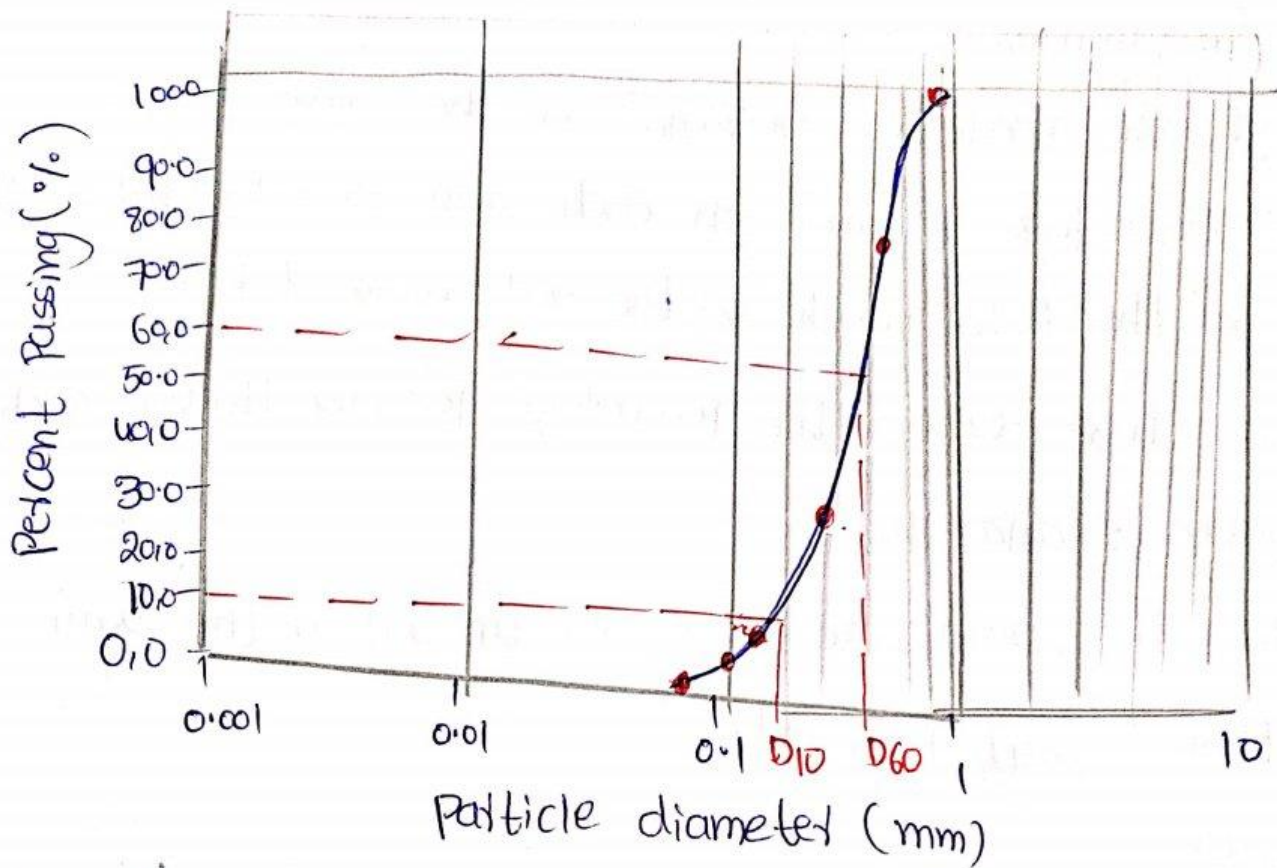
Particle size Distribution curve:-

D_{10} = Particle size such that 10% of them are fines

D_{30} = Particle size such that 30% of them are fines.

D_{60} = Particle size such that 60% of them are fines.

0.075 0.15 0.3 0.6 1.18 2.0 4.75 7.5 15 30 60 100



Uses of Particle size distribution curve:-

→ The Particle size distribution curve is extremely useful for coarse-grained soils. As the behavior of fine-grained soils (minus 75μ) depends upon the plasticity characteristic and not on the particle size, its use for fine-grained soils is limited.

1) The Particle size distribution curve is extremely useful for coarse-grained soils.

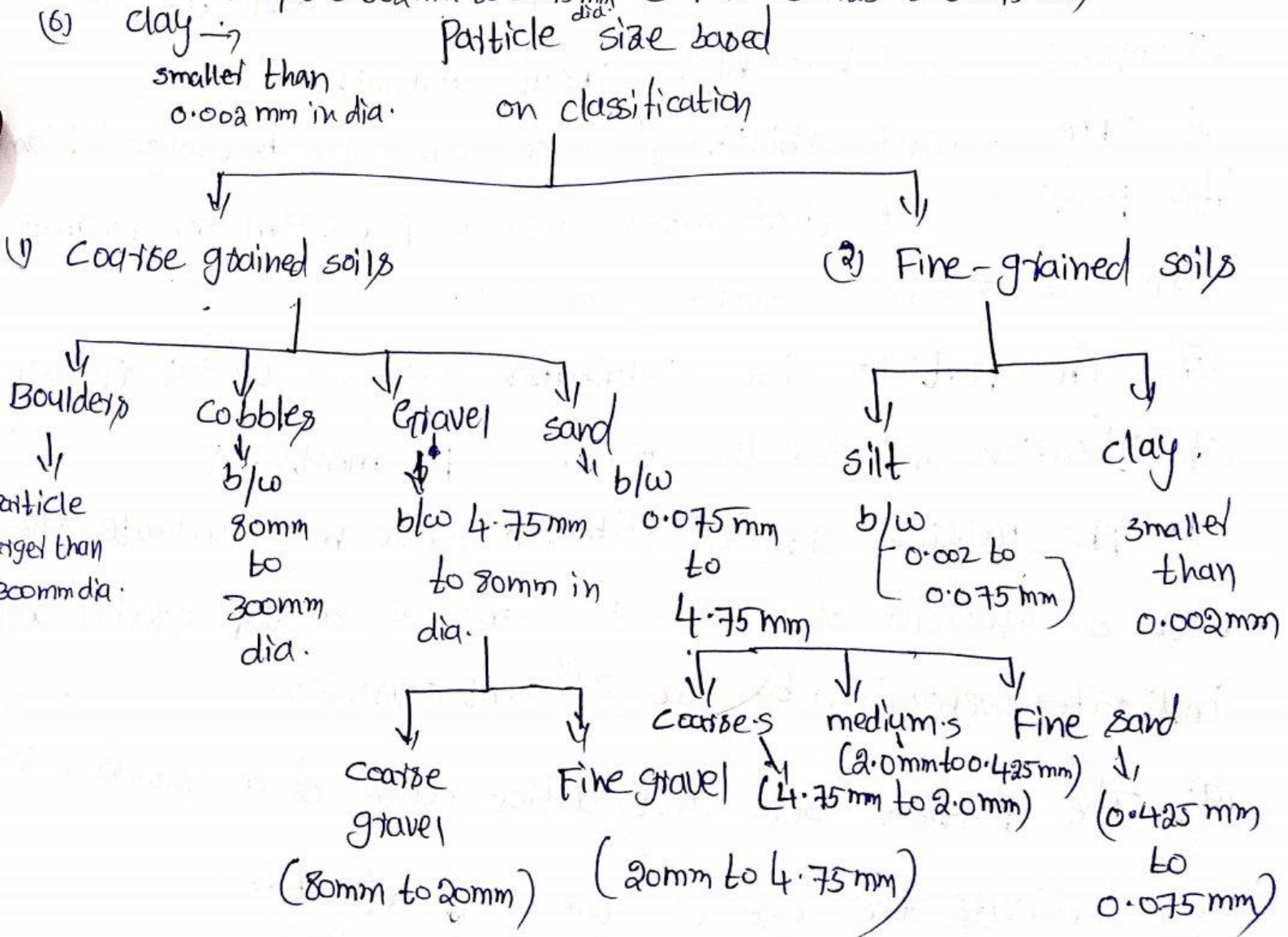
2) The coefficient of permeability of a coarse-grained soil depends to a large extent on the size of the particles. An approximate value of the coefficient of permeability can be determined from the particles size as discussed.

- ③ The Particle size is used to know the susceptibility of a soil to frost action.
- ④ The Particle size distribution curve is required for the design of drainage filters.
- ⑤ The Particle size distribution provides an index to the shear strength of the soil. Generally, a well-graded compacted sand has high shear strength.
- ⑥ The compressibility of a soil can also be judged from its Particle size distribution curve. A uniform soil is more compressible than a well-graded soil.
- ⑦ The Particle size distribution curve is useful in soil stabilisation and for the design of pavements.
- ⑧ The Particle size distribution curve may indicate the mode of deposition of a soil. For example, a gap-graded soil indicates deposition by two different agencies.
- ⑨ The Particle size distribution curve of a residual soil may indicate the age of the soil deposit.

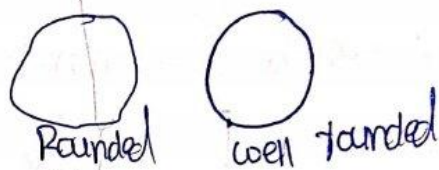
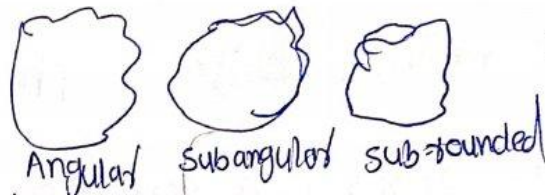
with increasing age, the average particle size decreases because of weathering. The Particle size distribution curve which is initially wavy becomes smooth and regular with age.

⇒ soils are classified by particle size into broad categories:

- 1) boulders - particles larger than 300mm in diameter
- 2) cobbles → particles b/w 80mm and 300 mm in diameter.
- 3) gravel - particle between 4.75 mm and 80mm in diameter. This is further divided into coarse gravel (80mm to 20mm) and Fine gravel (20mm to 4.75 mm).
- 4) sand - 0.075 to 4.75 mm - ① C.S - 4.75 to 2.0 mm ② M.S - 2.0 to 0.425
- 5) silt - b/w 0.002 mm to 0.075 mm ③ F.S - (0.425 to 0.075 mm)
- 6) clay → smaller than 0.002 mm in dia.



⇒ Particle shape:-



Important shapes of particles

① Angular →

② Rounded →

} In gravel, sand, silt.

③ flaky → → In clays

④ needle → → In corals deposits

→ The clay particle possitive electrical charge as shown below



① A soil sample is Partially saturated and its natural moisture content was 22% and Bulk density was 2 g/cm^3 . If specific gravity of solids is 2.65. Determine degree of saturation and void ratio.

Given ; water content $w = 22\% = 0.22$

Bulk density $\rho_d = 2 \text{ g/cm}^3$ (or) ρ

specific gravity $G_s = 2.65$

To find 1) degree of saturation (s)

$$\rho_d = \frac{G_s}{1+w}$$

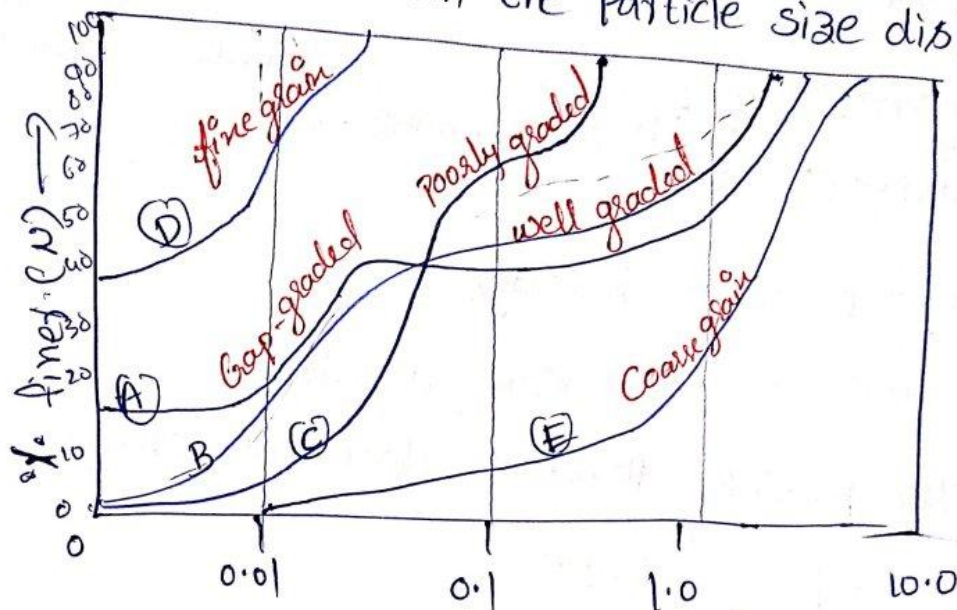
$$= \frac{2}{1+0.22}$$

$$= 1.639 \text{ g/cm}^3$$

$$\rho = \frac{\text{mass } m}{\text{Volume } V} \text{ g/cm}^3$$

Grading of soils:-

The distribution of particles of different sizes in a soil mass is called grading. The grading of soils can be determined from the particle size distribution curves.



① well-graded soil, Particle size D (mm)
A soil is well graded if it contains a good representation of all particle sizes, from coarse to fine, and the particles are well distributed.

- ① wide range of particles sizes
- ② Few voids between particles due to better packing.
- ③ High density and strength when compacted.
- ④ Good for construction and foundations.

→ smooth, upward-sloping curve (S) over a wide range.

② Poorly graded soil (~~uniformly graded~~)

A poorly graded soil consists mostly of particles of the same size, with little variation.

- narrow range of particle sizes
- poor packing efficiency, more voids
- lower strength and stability.

- Less suitable for compaction and load-bearing without modification
- steep (or) nearly vertical grain size distribution curve.
- Indicates a concentration of similar-sized particles.

(3) Gap-graded soil:- soil lacks certain particle sizes in its distribution - there's a "gap" (or) missing range.

- some sizes are completely absent
- can lead to segregation and uneven compaction
- may be intentional in certain concrete (or) asphalt mixes (e.g. to reduce shrinkage)
- not naturally ideal for engineering use unless purposely designed.
- Grain size distribution curve has a flat section (gap) where certain particles sizes are missing

[→ a curve situated to the right curve indicates a relatively coarse grained soil.
 → A curve situated higher up and to the left (D) indicates fine grained soils.]

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

D_{60} = Particle size such that 60% of the soil is finer than this size.

D_{10} = 10% finer

uniformity

Repet - 4 times

Soil Index Properties

What are the soil index properties - Index Properties are basically the properties which help you identify or classify the soil into a different group for the use in different engineering purposes.

→ Properties which facilitate identification and classification of soils for engineering purposes.

- 1) Identification
- 2) Classification
- 3) Study of engineering behaviour of soil

Types of Index Properties :-

→ All types of soils common optimum moisture content.

(1) water content (w) or

(2) specific gravity (G_s)

(3) unit weight (γ)

(4) Void Ratio (e)

(5) Porosity (n)

(6) Degree of Saturation (S_r)

For coarse grained soils (sand & gravel)

(7) Particle size Distribution (Gradation)

(8) Particle Shape (Visual inspection / microscopic analysis)

(9) Relative density or In situ density

For Fine grained soils (silt & clay)

(10) Atterberg limits or consistency limits

- (a) Liquid Limits
- (b) Plastic limit
- (c) shrinkage limit

(11) Atterberg Indices

- (a) Plasticity Index
- (b) Flow Index
- (c) Toughness Index
- (d) Consistency Index
- (e) Liquidity index.

① water content (or) optimum moisture content determination

→ water content determination is a quantitative measure of the wetness of a soil mass. It can be determined by several methods.

① oven drying method:-

→ The oven drying method is a standard Laboratory method and very accurate method.

→ The soil sample is taken in a small airtight container and the mass of the sample and that of the container are obtained using an accurate weighing balance.

→ The soil sample in the container is then dried in an oven at a temperature of $110 \pm 5^\circ\text{C}$ for 24 hours.

→ If it is lower than $110^{\circ}\text{C} \pm 5^{\circ}\text{C}$ it may not cause complete evaporation. If the temperature is higher than $110^{\circ}\text{C} \pm 5^{\circ}\text{C}$ it may cause breaking down of the crystalline structure of the soil particles and loss of chemically bound structural water.

→ This temperature is not suitable for soils containing gypsum, other minerals and organic matter.

→ The drying period of 24 hours has been recommended for normal soils.

→ Water content of the soil sample is calculated from the following

$$\text{water content (w)} = \frac{m_w}{m_s} = \frac{m_2 - m_3}{m_3 - m_1} \times 100$$

* m_1 = mass of container

* m_2 = mass of container + wet soil

* m_3 = mass of container + dry soil.

(b) Torsion Balance method:-

→ The Torsion Balance method is a quick and accurate way to measure the moisture content (water content) of a soil sample.

→ main components

(1) Infra-red lamp - Heats and dries the soil sample

(2) Torsion balance - to weigh the sample accurately during drying

- The infra-red radiation is provided by a 250W Lamp built. (Lamp provides the heat.)
- The moisture meter (torsion balance) measures the weight loss as water evaporates.
- The balance is damped to reduce vibration and improve accuracy.
- Soil particle size for the test must be less than 2mm, and the test takes about 15 to 30 minutes.
- m' (moisture content on wet mass basis):

$$m' = \frac{M_w}{M} = \frac{M_w}{m_s + m_w}$$

* m_s = mass of dry soil

* M = Total wet mass of the sample

$$\Rightarrow \frac{1}{m'} = \frac{M}{m_w} = \frac{m_s + m_w}{m_w} = \frac{m_s}{m_w} + \frac{m_w}{m_w}$$

$$\frac{1}{m'} = \frac{m_s}{m_w} + 1 = \frac{1}{w} + 1 \quad \left[w = \frac{m_w}{m_s} \right]$$

$$\frac{1}{m'} = \frac{1}{w} + 1 \Rightarrow \boxed{w = \frac{m'}{1 - m'}}$$

If w and m' are expressed in %: $w = \frac{m'}{1 - m'} \times 100$

Conclusion:-

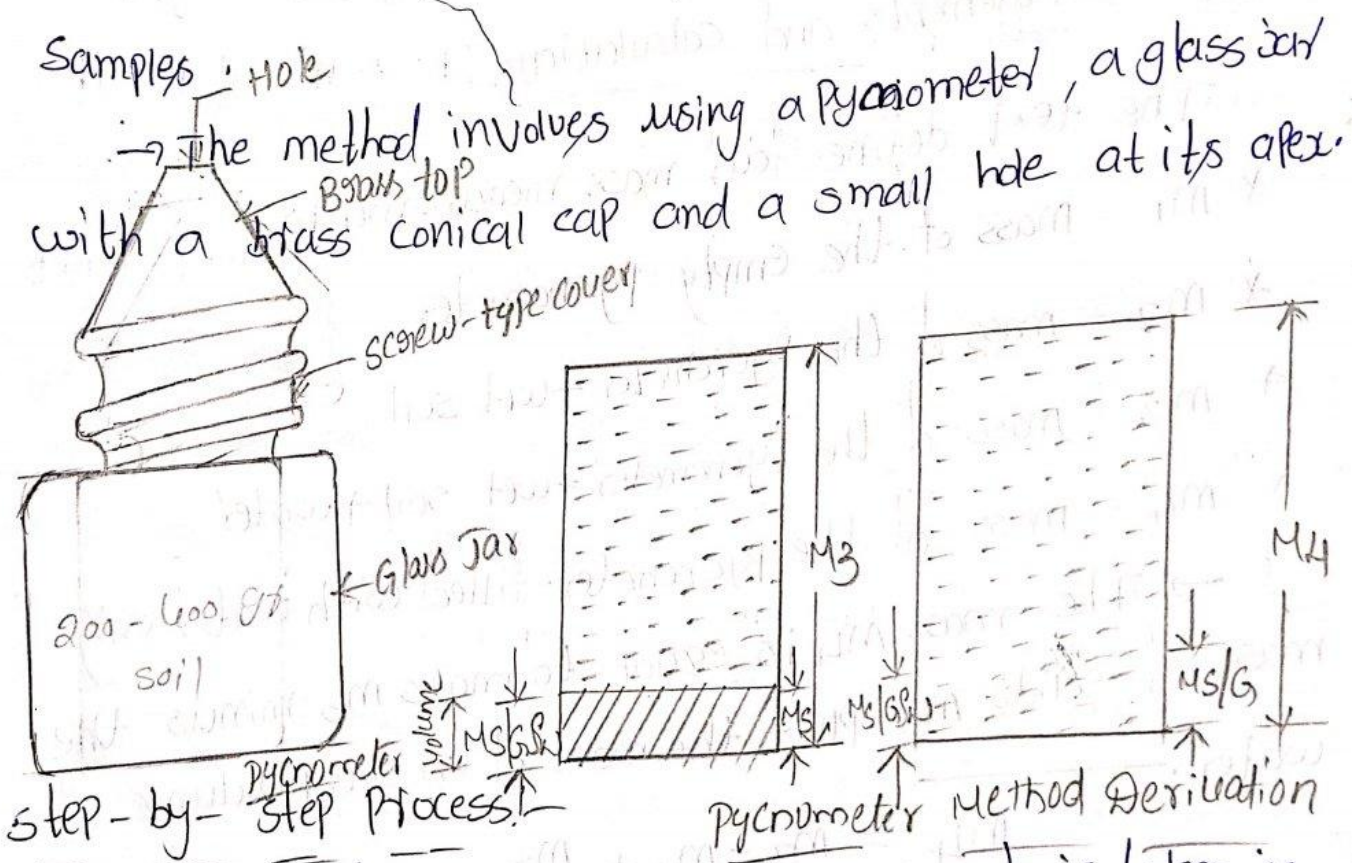
① These formulas are used to convert between moisture content based on wet weight and dry weight.

→ moisture content is crucial in assessing soil behavior (strength, compaction, etc--).

→ The torsion balance method is a fast and effective way to measure this moisture, especially useful for fine-grained soils.

(3) Pycnometer method:— (Coarse grain soil)

→ The pycnometer method is a laboratory technique used to measure the density and water content of soil



① A sample of wet soil (about 200-400g) is taken in the pycnometer and weighed.

② water is added to the soil in the pycnometer to make it about half full.

③ The contents are thoroughly mixed using a glass rod

- remove entrapped air.
- ④ more water is added, and the stirring process is continued until the pycnometer is filled flush with the hole in the conical cap.
- ⑤ The pycnometer is wiped dry and weighed.
- ⑥ The pycnometer is then completely emptied, washed thoroughly, and filled with water only.
- ⑦ The pycnometer is again wiped dry and weighed.

mass measurements and calculations.

The text defines four mass measurements.

- * m_1 - mass of the empty pycnometer
- * m_2 - mass of the pycnometer + wet soil
- * m_3 - mass of the pycnometer + wet soil + water
- * m_4 - mass of the pycnometer - filled with water only.

→ The mass m_4 is equal to mass m_3 minus the mass of solids m_s plus the mass of an equal volume of water.

$$m_4 = m_3 - m_s + \frac{m_s}{(G \rho_w)} \cdot \rho_w$$

$$m_4 = m_3 - m_s + \frac{m_s}{G}$$

$$= m_3 - m_s \left[1 - \frac{1}{G} \right]$$

when now, mass of wet soil = $m_2 - m_1$

$$m_s = (m_3 - m_4) \left[\frac{G}{G-1} \right]$$

$$\therefore \text{mass of water } m_w = (m_2 - m_1) - (m_3 - m_4) \left[\frac{G}{G-1} \right]$$

$$w = \frac{m_{w0}}{m_s} \times 100$$

$$w = \left[\frac{(m_2 - m_1)}{(m_3 - m_4)} \left[\frac{G_1 - 1}{G_1} \right] - 1 \right] \times 100$$

Conclusion:- The Pycnometer method is a precise technique for determining the water content and specific gravity of soil particles. By following the step by step process and using the derived equation, researchers and engineers can accurately measure these important soil properties.

④ Sand Bath method:- (sand, coarse grain soil)
(filled method)

→ Sand Bath method is a filled method for the determination of water content.

→ The method is rapid, but not very accurate.

→ A sand bath is a large, open vessel containing sand filled to a depth of 3cm (or) more.

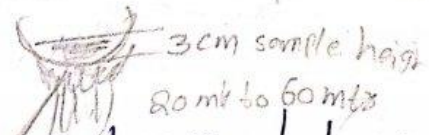
→ The soil sample is taken in a tray. The sample is crumbled and placed loosely in the tray. A few pieces of white paper are also placed on the sample.

→ The tray is weighed and the mass of wet sample is obtained. The tray is then placed on the sand bath.

→ The sand bath is heated over a stove. Drying takes about 20 to 60 minutes, depending upon the type of soil. and then weighed again.

→ The water content is determined $w = \frac{m_w}{m_s} = \frac{m_2 - m_3}{m_3 - m_1} \times 100$.

→ Precautions & considerations:-



- * The text emphasizes the importance of taking precautions to prevent overcooking (or) burning of the soil sample. (not exact value)
- * It also notes that the sand bath method is not suitable for soils with high clay (or) organic matter content.

⑤ Alcohol method:-

- Take a soil sample and mix it with methylated spirit.
- Ignite the mixture and allow it to burn until the water content is evaporated.
- weigh the sample again after the methylated spirit has burned away.
- calculate the water content by comparing the initial and final weights.

→ Precautions:- The Alcohol method is not recommended for soils with high calcium carbonate content.

Problems

① Following observations were made while determining water content of a soil sample by Pycnometer method.

weight of wet soil sample, $w = 2.308 \text{ N}$

weight of Pycnometer + soil + water, $w_1 = 30.930 \text{ N}$

weight of Pycnometer + water, $w_2 = 29.661 \text{ N}$

calculate water content of soil sample, if $G_s = 2.69$.

Soln

$$M_4 = \frac{M_3}{G} - M_s + \frac{M_s}{G}$$

$$M_3 = M_4 + M_s - \frac{M_s}{G}$$

$$W_1 = W_2 - M_s + \frac{M_s}{G}$$

$$W_1 = W_2 + M_s - \frac{W_s}{G}$$

$$W_1 = W_2 - \frac{W_s}{G} + W_s$$

W_s = weight of soil solids

$$W_s \left(1 - \frac{1}{G} \right) = W_1 - W_2$$

$$1 - \frac{1}{G}$$

$$\frac{G-1}{G}$$

$$W_s = (W_1 - W_2) \frac{G}{G-1}$$

$$= (30.930 - 29.661) \frac{2.69}{2.69-1} = 2.02 \text{ N}$$

→ weight of water present in soil sample

$$W_w = W - W_s = 2.308 - 2.02 = 0.288 \text{ N}$$

$$W = \frac{W_w}{W_s} = \frac{0.288}{2.02} = 0.1426 = 14.26\%$$

Q) In determining water content of a soil sample by Pycnometer method, following observations were made.

mass of Pycnometer, $m_1 = 803 \text{ g}$

mass of Pycnometer + wet sample, $m_2 = 1165 \text{ g}$

mass of Pycnometer + soil + water, $m_3 = 2008 \text{ g}$

mass of Pycnometer + water, $m_4 = 1802 \text{ g}$

calculate water content of soil sample, if $G = 2.65$

Solution

$$m_4 - \frac{m_s}{G} + m_s = m_3 \quad [m_s = \text{mass of soil solids}]$$

$$m_s \left(1 - \frac{1}{G} \right) = m_3 - m_4 \Rightarrow m_s = (m_3 - m_4) \left(\frac{G}{G-1} \right)$$

$$\Rightarrow m_w = (m - m_s)$$

where total mass of soil sample, $m = m_2 - m_1$

$$\begin{aligned}w &= \frac{m_w}{m_s} = \frac{(m_2 - m_1) - m_5}{m_s} \\&= \frac{(m_2 - m_1)}{m_s} - 1 = \left(\frac{m_2 - m_1}{m_3 - m_4} \right) \left(\frac{G - 1}{G} \right) - 1 \\&= \left(\frac{1165 - 803}{2008 - 1802} \right) \left(\frac{1.65}{2.65} \right) - 1 \times 100 \\&= 9.41 \%\end{aligned}$$

⑥ Calcium carbide method:-

→ The calcium carbide method determines soil water content by reacting water in the soil with calcium carbide ^[CaC₂] to produce acetylene gas (C₂H₂).

→ The reaction is: $\text{CaC}_2 + 2\text{H}_2\text{O} = \text{C}_2\text{H}_2 + \text{Ca(OH)}_2$

→ The water content is determined indirectly from the pressure of the acetylene gas, which is formed and measured using a moisture tester.

→ A wet soil sample is placed in a sealed container with calcium carbide.

→ No special preparation is needed for the soil sample; it is ground and pulverized, but cohesive and plastic soils require the addition of steel balls in the pressure vessels.

→ The test requires about 6 gm of soil

→ The pressure of the acetylene gas produced acts on the diaphragm of the moisture tester.

→ The quantity of gas is indicated on a pressure gauge, and from the calibrated scale of the pressure gauge, the water content (w) based on the total mass is determined.

→ The water content (w) based on the dry mass is determined using equation $w = \frac{m' - m}{m}$

→ Calcium carbide is highly susceptible to moisture absorption, so the lid of the container should be firmly fixed to prevent exposure to the atmosphere.

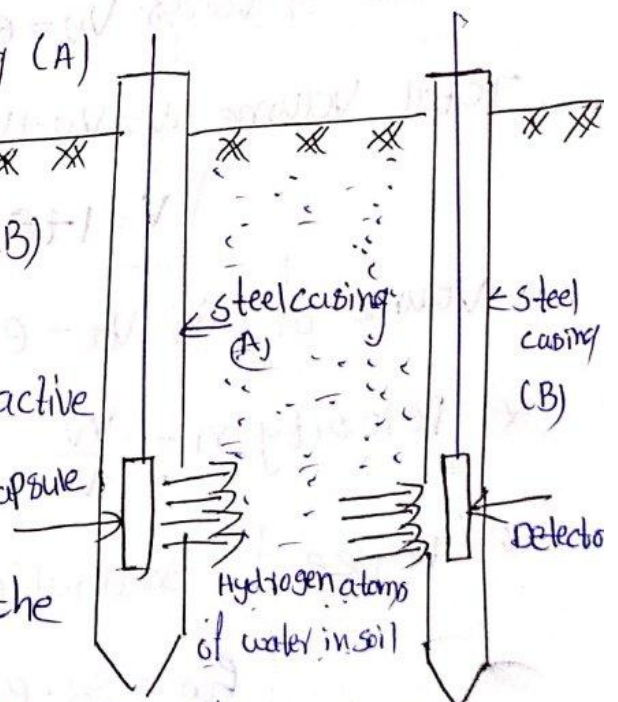
⑦ Radiation method:-

→ A device containing a radioactive isotope (cobalt-60) in a capsule is lowered into a bore hole in the soil.

→ The device includes a steel casing (A) with a small opening, and a detector is placed inside another steel casing (B) with a corresponding opening.

→ Neutrons emitted by the radioactive material are scattered by the hydrogen atoms present in the water within the soil.

→ The energy loss of these neutrons is proportional to the



quantity of water present, allowing the detector, which is calibrated to directly measure water content, to determine the soil's water content.

* The method is particularly useful for in-situ determination of soil water content.

* Careful handling and proper shielding precautions are necessary due to potential radiation hazards.

⇒ Relationship b/w void Ratio (e) and dry density: ^{(or) dry unit weight}

3 phase diagram in terms of void ratio (e)

The volume of solids is taken as unity in three phase diagram. If the area of cross section of the soil mass is also taken unity.

→ volume of solids is taken as unity, $V_s = 1$

* void Ratio $e = \frac{V_v}{V_s}$

$$e = \frac{V_v}{1}$$

Volume of voids $V_v = e$

Total Volume $V = V_v + V_s$

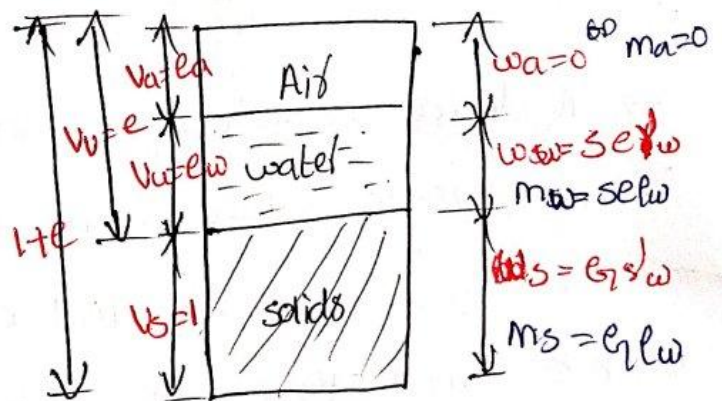
$$V = 1 + e$$

Volume of air $V_a = e_a$, volume of water $V_w = e_w$

* Porosity $n = \frac{V_v}{V}$, $n = \frac{e}{1+e}$

* degree of saturation $S_r = \frac{V_w}{V_v} = \frac{e_w}{e}$

$$e_w = S_r \cdot e$$



* Volume of air $V_a = V_v - V_w$

$$V_a = e - e_w = e - s_1 e$$

$$V_a = e(1 - s_1)$$

* Bulk unit weight ρ_1 or γ_{Bulk} (or) $\gamma = \frac{W}{V}$

$$\begin{aligned} \gamma &= \frac{W_s + W_w}{V} \\ &= \frac{e_1 \rho_w + \gamma_w \cdot V_w}{V} \\ &= \frac{e_1 \gamma_w + \gamma_w \cdot e_w}{1 + e} \end{aligned}$$

mass density

$$\rho = \frac{(e_1 + s e) \rho_w}{1 + e} \quad \text{or} \quad \boxed{\gamma = \frac{(e_1 + s e) \gamma_w}{1 + e}} \quad \text{--- (1)}$$

* Dry unit weight $\gamma_d = \frac{W_s}{V}$

$$\boxed{\gamma_d = \frac{e_1 \gamma_w}{1 + e} \quad \text{or} \quad \rho_d = \frac{e_1 \rho_w}{1 + e}} \quad \text{--- (2)}$$

* Saturated unit weight $\gamma_{\text{sat}} = \frac{W_{\text{sat}}}{V}$

$$\gamma_{\text{sat}} = \frac{(e_1 + (1)e) \gamma_w}{1 + e} = \frac{(e_1 + e) \gamma_w}{1 + e}$$

$$[s=1] \quad \text{or} \quad \rho_{\text{sat}} = \frac{(e_1 + e) \rho_w}{1 + e} \quad \text{--- (3)}$$

* Submerged unit weight (γ_{sub}) (or) $\gamma' = \gamma_{\text{sat}} - \gamma_w$

$$= \frac{(e_1 + e) \gamma_w}{1 + e} - \gamma_w$$

mass density of solids (or) $\rho_s = \frac{m_s}{V_s} = \frac{m_s}{\frac{1}{G_s}} = m_s G_s$

unit weight of solids (or) $\gamma_s = \frac{W_s}{V_s} = \frac{W_s}{\frac{1}{G_s}} = W_s G_s$

$$e_1 = \frac{\rho_s}{\gamma_w} = \frac{W_s}{\gamma_w}$$

$$e_1 = \frac{W_s}{\gamma_w}$$

$$W_s = e_1 \cdot \gamma_w$$

$$\rho_s \gamma_w = \frac{W_s}{V_w}$$

$$W_w = \gamma_w \cdot V_w$$

$$\gamma_{sub} \text{ or } \gamma' = \left(\frac{e + e}{1 + e} - 1 \right) \gamma_w$$

$$= \left[\frac{e_1 + e - (1 + e)}{1 + e} \right] \gamma_w$$

$$= \left[\frac{e_1 + e - 1 - e}{1 + e} \right] \gamma_w$$

In case of soil is not fully saturated the sub merged mass density $\gamma' = \gamma - \gamma_w \Rightarrow \gamma' = \frac{(e_1 + e)}{1 + e} \gamma_w - \gamma_w \Rightarrow (e_1 + e) \gamma_w - (1 + e) \gamma_w \Rightarrow \gamma' = \left[\frac{e_1 + e - (1 + e)}{1 + e} \right] \gamma_w$

$$\gamma_{sub} / \gamma' = \left[\frac{e_1 + e}{1 + e} \right] \gamma_w$$

$$\text{or } \rho_{sub} \text{ or } \rho' = \left[\frac{e_1 + e}{1 + e} \right] \rho_w$$

Relationship b/w Porosity γ' unit weight or dry density.

→ Total volume of soil $V = 1$

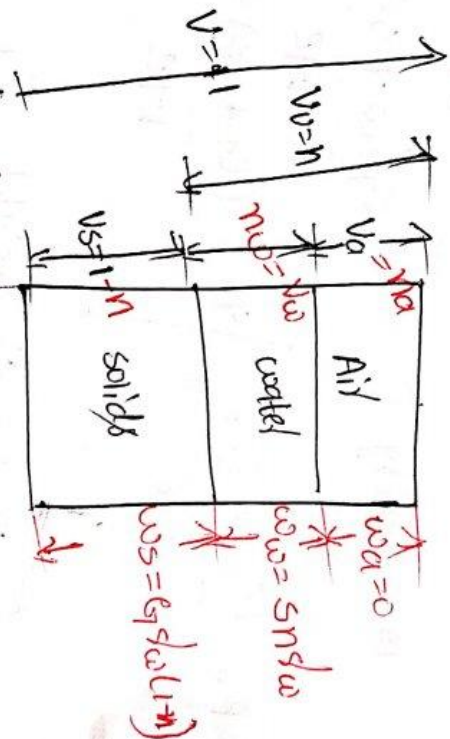
$$\text{Porosity } n = \frac{V_v}{V}$$

$$n = \frac{V_v}{1} = V_v$$

Volume of voids $(V_v) = n$

$$\text{Void ratio } e = \frac{V_v}{V_s} \Rightarrow e$$

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



$$\gamma_s = \frac{w_s}{V_s} = \frac{w_s}{1}$$

$$\gamma_s = w_s \Rightarrow e_1 = \frac{\gamma_s}{\gamma_w}$$

$$e_1 = \frac{w_s}{\gamma_w}$$

$$w_s = e_1 \cdot \gamma_w$$

$$\gamma_w = \frac{w_w}{V_w}$$

$$w_w = \gamma_w \cdot V_w$$

$$w_s = \gamma_s \cdot (1 - n)$$

∴ Bulk unit weight $(\gamma) = \frac{W}{V} = \frac{w_s + w_w}{V}$
unit weight of solids $(\gamma_s) \text{ or } \gamma_s = \frac{w_s}{V_s}$

$$w_s = \gamma_s \cdot V_s$$

* Specific gravity (G_s) = $\frac{\gamma_s}{\gamma_w}$

$$\gamma_s = G_s \gamma_w$$

$$w_s = G_s \cdot \gamma_w (1-n)$$

① Bulk unit weight $\gamma = \frac{w_s + w_w}{V}$

$$\gamma = \frac{G_s \cdot \gamma_w (1-n) + \gamma_w \cdot V_w}{V}$$

$$S_r = \frac{V_w}{V}$$

$$S = \frac{w_w}{n}$$

$$w_w = S \cdot n$$

$$\gamma = G_s \gamma_w (1-n) + \gamma_w \cdot S \cdot n$$

$$\boxed{\gamma = \gamma_w [G_s (1-n) + S \cdot n]} \quad \text{--- ①}$$

$$V_w = S \cdot n$$

* dry mass density / dry unit weight (γ_d / γ_d)

$$\text{dry } S = 0$$

$$\gamma_d = \gamma_w (G_s (1-n))$$

$$\boxed{\gamma_d = G_s \cdot \gamma_w (1-n)} \quad \text{--- ②}$$

* saturated unit weight (γ_{sat}) = γ_{sat} (saturated $S=1$)

$$\gamma = \gamma_{sat}$$

$$\gamma_{sat} = \gamma_w (G_s (1-n) + 1 \cdot n)$$

$$\boxed{\gamma_{sat} = \gamma_w [G_s (1-n) + n]} \quad \text{--- ③}$$

* submerged mass density (γ_{sub}) = $\gamma_{sat} - \gamma_w$

$$\gamma_{sat} = [G_s (1-n) + n] \gamma_w - \gamma_w$$

$$= [G_s - G_s n + n - 1] \gamma_w$$

$$\rho_{sub} = [\rho_g(1-n) - (1-n)] \rho_w$$

$$\rho'_{sub} = (\rho_g - 1)(1-n) \rho_w \quad \text{--- (4)}$$

$\Rightarrow$ In terms mass density

$$\rho = [\rho_g(1-n) + sn] \rho_w \quad \text{--- (1)}$$

$$\rho_d = \rho_g \rho_w (1-n) \quad \text{--- (2)}$$

$$\rho_{sat} = [\rho_g(1-n) + n] \rho_w \quad \text{--- (3)}$$

$$\rho'_{sat} \rho'_{sub} = (\rho_g - 1)(1-n) \rho_w \quad \text{--- (4)}$$

$\Rightarrow$ Relation b/w mass density in terms of water content

The expression for mass density can be written in terms of water content by writing the void ratio in terms of water content using equation

$$\phi \cdot \rho = \frac{(\rho_g + se) \rho_w}{1 + e}$$

$$\rightarrow se = w \rho_g \Rightarrow e = \frac{w \rho_g}{s}$$

$$\rho = \left(\frac{\rho_g + \frac{w \rho_g}{s}}{1 + \frac{w \rho_g}{s}} \right) \rho_w$$

$$\rho = \left(\frac{(1+w) \rho_g \cdot \rho_w}{1 + w \rho_g / s} \right) \quad \text{--- (1)}$$

* If the soil is fully saturated; $s = 1$

becomes eqn ①

$$\rho_{sat} = \frac{(1+w) \rho_w}{1+w e} \quad \text{--- ②}$$

* The submerged density is given by

$$\rho_{sub} = \rho_{sat} - \rho_w$$

$$= \frac{(1+w) \rho_w}{1+w e} - \rho_w$$

$$\rho_{sub} = \frac{\rho_w (1+w) - \rho_w (1+w e)}{1+w e}$$

$$= \frac{(\rho_w + \rho_w w - \rho_w - \rho_w e w)}{1+w e} \rho_w$$

$$\rho' \text{ (or)} \left[\rho_{sub} = \frac{(\rho_w (1+w) - \rho_w (1+w e))}{1+w e} \right] \quad \text{--- ③}$$

* Dry density

$$\rho_d = \frac{\rho_w}{1+e} \quad \text{--- ④}$$

From equation ①

and $\rho_w = \frac{w \rho_w}{1+w e}$

$$\rho_d = \frac{\rho_w}{1+w e} \quad \text{--- ⑤}$$

from equation (1) and (4)

$$P_d = \frac{G_f \ell \omega}{1 + \omega G_f / s} \quad \text{--- (4)}$$

$$P = \frac{(1 + \omega) G_f \ell \omega}{1 + (\omega G_f / s)} \quad \text{--- (1)}$$

for (1) equation

$$P_d = \frac{G_f \ell \omega}{1 + \omega G_f / s} \times \frac{1 + \omega}{1 + \omega}$$

$$P_d = \frac{P}{1 + \omega} \quad \text{--- (5)}$$

⇒ Specific Gravity Determination:-

The specific gravity of solid particles is determined in the laboratory using the following methods:

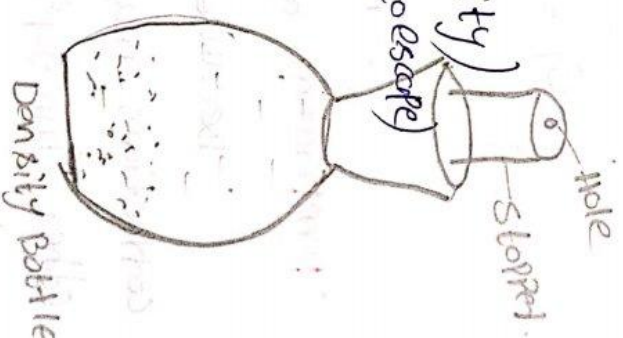
- ① Density bottle method - specific bottle method.
- ② Pycnometer method
- ③ measuring flask method
- ④ Gas jar method
- ⑤ shrinkage limit method — In this method shrinkage limit & specific gravity measured.

① Density Bottle method:-

Procedure To determine the specific gravity (G_s) of soil particles using a density bottle.

Apparatus:-

- * A density bottle (usually 50 ml capacity)
 - * stopper with a hole (to allow air/water to escape)
 - * Balance (to measure weight)
 - * Distilled water
 - * oven (for drying soil)
 - * Vacuum Pump (to desiccate (to remove air))
- Steps:-
- * To pass 2 mm sieve before the test.
 - ↳ clean and dry the bottle at 105-110°C and weigh it.



* Let this weight be M_1

② Add dry soil (5-10g): to the bottle, reweigh:

→ weight is now M_2 = mass of bottle + dry soil

③ Add distilled water: to cover the soil and allow it to soak for about 2 hours.

4) Fill the bottle completely with water (no air bubbles), and weigh;

* weight is now $m_3 = \text{bottle} + \text{soil} + \text{water}$.

⑤ Empty the bottle, clean it, and refill with water only to the ~~same~~ same level; weigh;

* weight is now $m_4 = \text{bottle} + \text{water}$ only.

Calculations:-

$m_1 = \text{mass of empty bottle}$

$m_2 = \text{mass of bottle} + \text{dry soil}$

$m_3 = \text{mass of bottle} + \text{soil} + \text{water}$

$m_4 = \text{mass of dry soil} = m_2 - m_1$

$\therefore \rho_w = \text{density of water (used in some formulas)}$

$$G_1 = \frac{m_2 - m_1}{(m_2 - m_1) - (m_3 - m_4)} \quad \text{or} \quad G_1 = \frac{m_5}{m_5 + m_4 - m_3}$$

$$\therefore m_5 = m_2 - m_1$$

Temperature corrections:-

Because specific gravity of water varies with temperature, corrections must be applied if the test is done at temperatures other than 27°C

$$\text{At } 27^\circ\text{C} \Rightarrow G_{27} = G_t \times \frac{\text{specific gravity of water at } t^\circ\text{C}}{\text{S.G. of water at } 27^\circ\text{C}}$$

$$\text{At } 4^\circ\text{C} \Rightarrow G_{14} = G_t \times \frac{\text{S.G. of water at } t^\circ\text{C}}{\text{S.G. of water at } 4^\circ\text{C}}$$

→ use table 2.4 (page 32) for values of water's specific gravity at various temperatures.

(3) Pycnometer method: Similar to density bottle but uses a larger container. more suitable for coarse soils.

about - 800 - 300 gm - 80 mm I_s sieve.

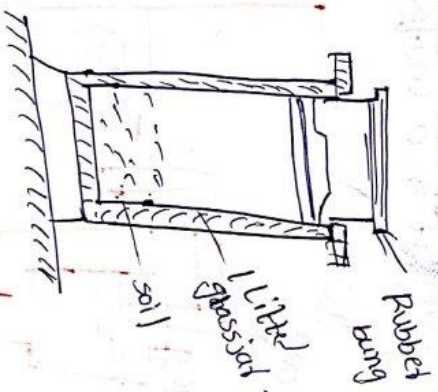
(3) measuring flask method: - uses a flask with a stopper.

and Vacuum line. good for medium and fine soils.

± measuring flask is of 850 ml (or 500 ml) capacity.

* 80 - 100 g of oven dry soil

(4) Gas jar method: - In this method, a gas jar of about 1 litre capacity is used. - similar to Pycnometer method. The jar is fitted with a rubber bung.



measurement of mass density: -

The bulk mass density of a soil sample, as per

equation, is the mass per unit volume. $\rho = \frac{M}{V}$

→ while the mass of a soil sample can be determined precisely, accurately determining its volume is difficult.

→ Different methods for measuring mass density primarily vary in how the volume of the soil sample is measured.

→ once the bulk mass density is determined, the dry mass density

can be calculated using equation

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

→ The volume of the specimen used in various tests can be computed from the measured dimensions, as they have regular shapes, such as a cylinder or a cube.

→ However, precise measurements are not possible. If the sample is made in a container of known dimensions much more accurate measurements are possible.

Water Displacement method:

This involves coating a soil specimen with paraffin wax to make it impervious, then immersing it in a water displacement container to measure the volume of displaced water, which corresponds to the volume of the waxed specimen. The volume of the wax is then subtracted to find the actual soil volume.

$$V = V_t - \frac{(M_t - M)}{\rho_p}$$

where V = volume of specimen

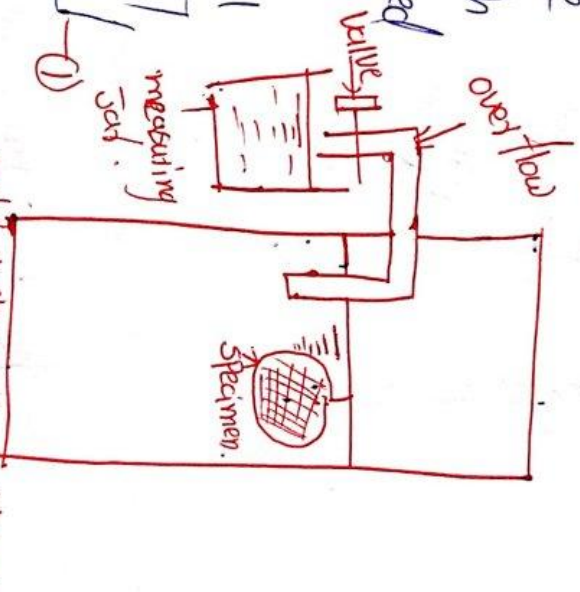
M_t = mass of waxed specimen, M = mass of specimen

ρ_p = mass density of paraffin [approximately 0.998 gm/ml]

→ calculate bulk density and dry density equation

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

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



② Submerged mass density method:-

This method utilizes Archimede's principle, where the reduction in mass of a core soil specimen when submerged in water is equal to the mass of the displaced water, allowing for the calculation of the specimen's volume.

$$V = \left(\frac{m_L - m_1}{\rho_w} - \frac{(m_L - m)}{\rho_p} \right) \quad \text{--- (2)}$$

m_L = mass of core specimen,

m = mass of specimen

m_1 = mass of core specimen in water,

ρ_p = mass density of core.

ρ_w = mass density of water.

→ can be derived, using Archimedes' principle

$$M_1 = m_L - V = m_L - V_L \rho_w$$

$$V_L = \frac{m_L - M_1}{\rho_w} \quad \text{--- [substituting this value } V_L \text{ in above equation (1)]}$$

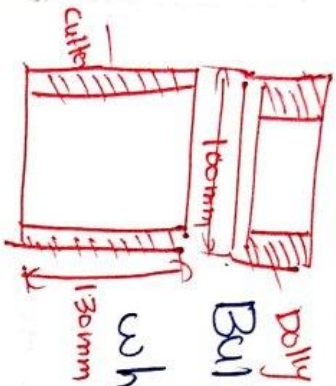
③ core cutter method:-

① This is a field method primarily used for determining the mass density of soft, fine-grained soils.

② It involves a core cutter, which is an open, cylindrical barrel with a sharp cutting edge, and a dolly, which is placed over the cutting during tamping to prevent damage to the cutter's edges.

③ The cutter is tamped into the soil is measured and a sample is taken for water content determination.

④ The volume of the soil is considered equal to the internal volume of the cutter, which can be determined by its dimensions and by filling it with water and measuring the mass of water.



Bulk mass density

$$\rho = \frac{m_2 - m_1}{V}$$

m_2 = mass of cutter, with soil

m_1 = mass of empty cutter

V = internal volume of cutter.

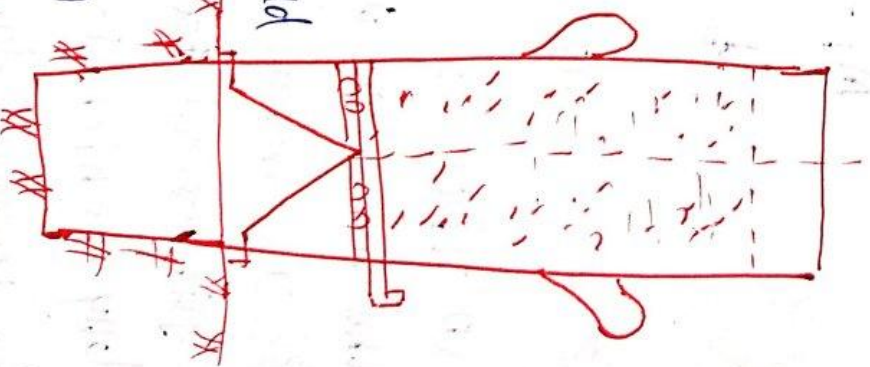
→ This method is quite suitable for soft, fine grained soils. It cannot be used for stoney, gravelly soils.

⑤ Sand Replacement method

→ This method uses a sand-pouring cylinder with a pouring cone at its base placed on the ground.

→ The cylinder is first calibrated to determine the mass density of sand.

→ For good results, the sand used should be uniform, dry and clean, passing a 600 micron sieve and retained on a 300 micron sieve.



Calibration of apparatus:-

- ① The cylinder is filled with sand and weighed.
- ② A calibrating container is then placed below the pouring cylinder and the shutter is opened. The sand falls the calibrating container and the cone. The shutter is closed, the mass of the cylinder is again taken.

③ The mass of the sand in the container and the cone is equal to the difference of the two observations.

④ The pouring cylinder is again filled to the initial mass. The sand is allowed to run out of the cylinder, equal to the volume of the calibrating container, and the shutter is closed.

⑤ The cylinder is removed and the sand filling the cone is collected and weighed (m_2).

* The mass density of the sand is determined as under:

$$\rho_s = \frac{m_1 - m_2 - m_3}{V_c}$$

where m_1 = initial mass of cylinder with sand.

m_2 = mass of sand in cone only

m_3 = mass of cylinder after pouring sand into the cone and the container.

V_c = Volume of the container.

∴ mass of sand in both the container and cone is $m_1 - m_3$

(B) measurement of volume of hole

① The sand replacement method involves excavating a hole in the ground and measuring the volume of sand required to fill it.

② The method uses a sand-pouring cylinder with a cone-shaped bottom to pour sand into the hole.

③ The Volume of the hole is determined by measuring the mass of sand required to fill it and the density of sand.

④ The in-situ density of the sand soil is calculated using the mass of soil excavated from the hole and the volume of the hole.

⑤ The method is widely used for fine-grained to coarse grained soils.

$$\text{Volume of hole} = \frac{m_1 - m_4 - m_2}{\rho_s}$$

m_1 = mass of cylinder and sand before pouring into the hole.

m_2 = mass of sand in cone only

m_4 = mass of cylinder after pouring sand into the hole.

ρ_s = mass density of sand, found from calibration.

→ The bulk mass density of the in-situ soil is determined from the mass of soil excavated and the volume of the hole.

⑤ Rubber Balloon method - (same as above diffn but sand to water)

① A density plate is placed on leveled ground and a cylinder containing a rubber balloon partially filled with water is placed over it.

② Pressure is applied to the balloon, causing it to inflate and conform to the shape of the hole displacing water in the cylinder.

③ The volume of water displaced in the cylinder is measured, which corresponds to the volume of the hole.

④ The cylinder is then removed, and the soil from the hole is collected for further analysis.

* The method is general and is suitable for all soils

However, it is not so accurate as it is difficult to fit the balloon exactly in an irregular hole.

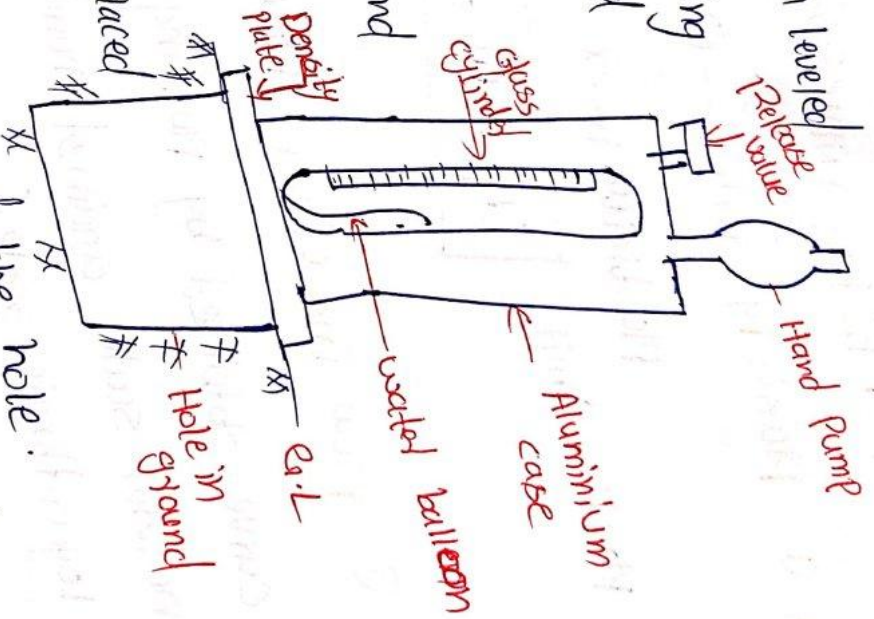
⑥ Radiation method -

① The meter consists of two probes.

② Radio isotope source ③ gamma ray detector.

④ The probes max depth 800mm to 300mm into the ground.

→ * Radiation methods for determination of the mass density of soils are quick and convenient and are gaining popularity. However, precautions must be taken against the radiation hazard.



⇒ ~~Veteran~~ Atterberg Indices

ATTERBERG INDICES

* ① Plasticity Index: — (I_P)

Plasticity index is defined as liquid limit minus Plastic limit.

$$I_P = w_L - w_P$$

② Flow Index: (I_F) Flow index is the slope of flow curve obtained by plotting water content as ordinate on natural scale, against, number of blows as abscissa on logarithmic scale.

$$I_F = \frac{w_1 - w_2}{\log_{10} \frac{N_2}{N_1}}$$

w_1 = water content corresponding to number of blows N_1

w_2 = water content corresponding to " " " " N_2

③ Toughness Index: — (I_T)

Toughness index is defined as the ratio of plasticity index to flow index.

$$I_T = \frac{I_P}{I_F}$$

* ④ Consistency Index (I_C)

as the ratio of natural water content minus Plastic limit to Plasticity index.

$$I_L =$$

* Consistency Index (I_c) consistency index is defined as the ratio of liquid limit minus natural water content to the plastic index.

$$I_c = \frac{w_L - w}{I_P}$$

- when soil is at L.L, $w = w_L$; $I_c = 0$
- when soil is at P.L, $w = w_P$; $I_c = 1$
- I_c varies from 0 to 1 (100%).
- when I_c is (-ve) negative the soil mass will be Liquid state
- when I_c is (+ve) but $< 100\%$ the soil mass will be in semisolid or solid state.

* Liquidity Index (I_L)

Liquidity index is defined as the ratio of natural water content minus plastic limit to plasticity index.

$$I_L = \frac{w - w_P}{I_P}$$

- when soil is at P.L $w = w_P$ and $I_L = 0$
- when the soil mass is at L.L $w = w_L$ and $I_L = 1$
- when I_L will be between 0 and 1
- when $I_L > 1$; The soil mass will be in liquid state.
- when I_L is (-ve) negative, it will be in semi-solid or solid state.

Consistency Limits:

→ consistency limits of fine-grained soils - specifically how water content affects the behavior of soil.)

Consistency refers to how firm (or) soft a fine-grained soil is depending on its water content. A Swedish engineer named Atterberg introduced the concept of Atterberg limits in 1911. These limits define the water contents at which soil changes from one state to another.

→ There are four main consistency states of soil:

- ① Liquid state
- ② Plastic state
- ③ Semi-solid state
- ④ Solid state.

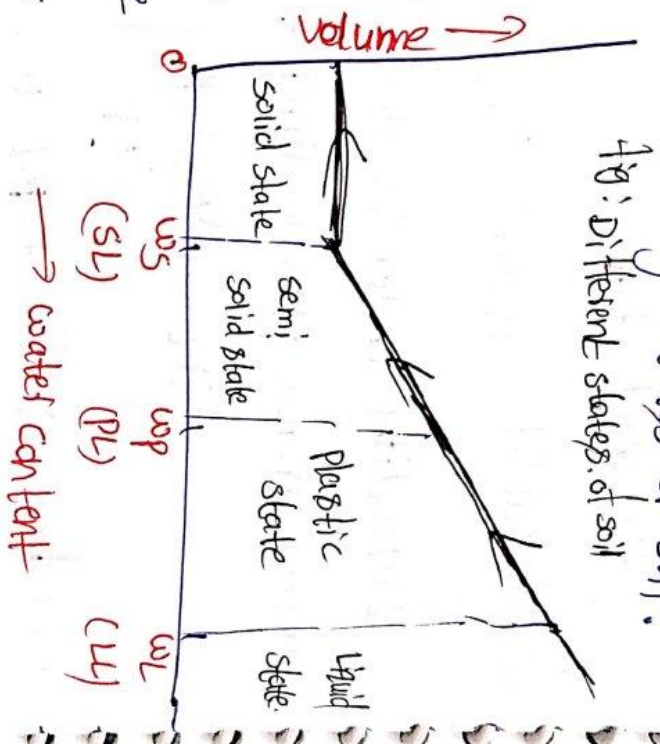
→ As water content decreases, soil changes from behaving like a liquid to behaving like a solid.

① Liquid state:

when a soil has high water content, it acts like a liquid. It has no resistance to shear. i.e. it can't resist shape changes (or loads), and its shear strength is zero.

② Plastic state:

As water content decreases, the soil starts gaining strength and offers resistance to shear deformation.



It behaves more like a flexible solid - moldable but still cohesive.

③ & ④ semi-solid & solid states:-

with even less water, the soil becomes stiff and hard, eventually behaving like a solid with little to no plasticity.

→ Atterberg limits:-

The diagram in the book shows these transitions as the water content changes:

① Liquid limit (LL):- The point where soil changes from liquid to plastic.

② Plastic limit (PL):- The point where soil changes from plastic to semi solid.

③ Shrinkage limit (SL):- The point where soil changes from semi-solid to solid.

Note:- These limits are important in soil engineering because they help determine how a soil will behave under different moisture conditions.

Consistency Index:-

= Shrinkage Parameters:-

① shrinkage limit:- (w_s) The shrinkage limit can be determined using dry soil but if the value of G is known, we have

Shrinkage limit, $w_s = \frac{\text{mass of water at shrinkage limit}}{\text{mass of soil solids}}$

$$w_s = \frac{(V_d - V_s) \rho_w}{M_d} = \frac{V_d \rho_w - \frac{V_s}{G} \rho_w}{M_d}$$

$$= \frac{V_d \rho_w}{M_d} - \frac{M_d}{\rho_s M_d} \rho_w = \frac{V_d \rho_w}{M_d} - \frac{\gamma_w}{G \rho_w}$$

$$w_s = \frac{V_d \rho_w - \frac{1}{G}}{\frac{M_d}{\rho_w}}$$

$$w_s = w_c - \left[\frac{V_{wet} - V_{dry}}{w_{dry}} \right] \times 100$$

① Shrinkage Index (I_s):- The difference b/w liquid limit to shrinkage limit is known as shrinkage index (I_s)

$$I_s = w_L - w_s$$

② Shrinkage Ratio (SR):- It is defined as the ratio of given volume change expressed as a dry volume to the change in water content.

$$SR = \frac{V_1 - V_2 / V_d}{w_1 - w_2} \times 100$$

V_d = volume of dry soil
 V_1 = volume of soil mass at water content w_1
 V_2 = volume of soil mass at water content w_2

③ Volumetric shrinkage :-

It is defined as the ~~ex~~ shrinkage ratio another expressed has a dry volume when the water content is reduced from a given value to the shrinkage limit

$$V_s = \frac{V_1 - V_2}{V_d \times 100}$$

$$V_s = SR (w_1 - w_2)$$

shrinkage ratio another expression $SR = \frac{w_1 - w_2}{w_s} = \frac{(V_1 - V_2) \rho_w}{m_s}$

$$SR = \frac{m_s}{V_d \cdot \rho_w}$$

$$SR = \frac{\rho_d}{\rho_w} = e_{sm}$$

④ Linear shrinkage (L_s) :-

It is defined as the change in length divided by the initial length at the shrinkage limit.

$$L_s = \frac{\text{Final length} - \text{Initial length}}{\text{Initial length}} \times 100$$

① A soil has a liquid limit of 25% and a flow For a soil sample having natural moisture content 45% plastic limit 40% and liquid limit 55% the liquidity index of the soil sample.

$$I_L = \frac{w - w_p}{I_p}$$

$$I_p = w_L - w_p$$

$$= 0.55 - 0.40 = 0.15$$

Given data $w = 45\%$ $= 0.45$ $I_L = \frac{0.45 - 0.40}{0.15}$

$$L.L (w_L) = 55\% = 0.55$$

$$P.L (w_p) = 40\% = 0.40$$

$$= \frac{0.05}{0.15} = \frac{1}{3} = 0.33$$

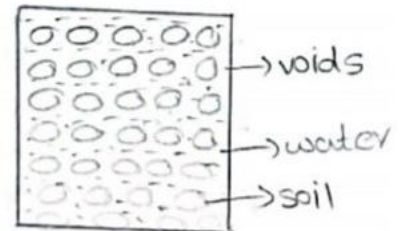
UNIT - II

PERMEABILITY

soil water:-

The water which is present in the voids of the soil mass is called as soil water. It can be divided into different types.

1. free (or) gravitational water.
2. Held water.
3. structural water
4. Absorbed water
5. capillary water.



1. free (or) gravitational water:-

The water which is free to move under the gravitational force is known as free water.



2. Held water:-

Held water is held in pores of voids of soil mass due to some forces in the porous and soil mass.

3. structural water:-

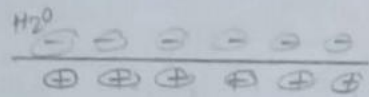
It is the water which is chemically combined in crystal structure of the minerals is called as structural water. This can be not easily removable only by heating of sample above 110°C temperature. It can be breaking the structure of soil mass.

4. absorbed water:-

This type of water is present especially in fine grain soils over the surface of solids this water is having high viscosity.

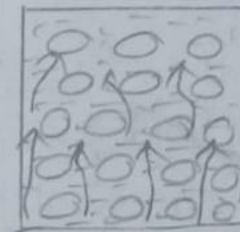
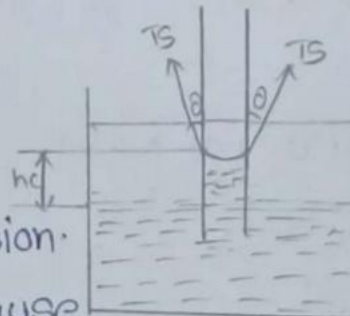
* usually the solid particles carries negative charge

by which it attracts the moisture from the atmosphere is called as absorbed water and is held over the surface of the particles by the force of adhesion.



5. capillary water :-

water rise in a small capillary tube because of adhesion and cohesion. Adhesion occurs because



water is a adhesion property are sticks to the solids walls of the tube. cohesion is due to attraction between the particles. If the effect is less significant than effect of adhesion liquid rises at the point of contact. The liquid level is depends at the point of contract as in case of mercury also.

* If A glass tube both sides open is lower end in water and upper end in atmosphere the water level rises in a tube. let θ angle of contact between the water and tube walls.

F_u = upword pull due to surface tension

$$F_u = (T_s \cos \theta) \pi d$$

T_s = surface tension

d = diameter of the tube.

F_d = down word force

ρ_w = density of water

$$F_d = \rho_w (A) h_c$$

$$= \sqrt{\rho_w \frac{\pi}{4} d^2 h_c}$$

2

equate the f_u and f_d

$$TS \cos \theta \pi d = \sqrt{\rho_w \frac{\pi}{4} d^2 h_c}$$

$$h_c = \frac{4TS \cos \theta}{\rho_w \times d}$$

case 2:- for a clean glass tube and pure water the meniscus is horizontal $\theta = 0$ and surface tension becomes 0.073 N/m and density of water $\rho_w 9810 \text{ N/m}^3$ what is the rise of water.

$$h_c = \frac{4 \times 0.073 \times \cos 0}{9810 \times d}$$

$$= \frac{2.9765 \times 10^{-5}}{d}$$

d in meters converted in to cm and h_c also cm we get $\frac{0.297}{d} \text{ cm}$.

Rising of water in different soils :-

sandy soils (sand)

5 to 10 cm

silt

10 mts

clayey soils

30 mts.

Imp

Permeability :-

It is the property of soil by which it's permits the passage of water or seepage of water through their internal connecting voids is called as permeability.

usually gravel soil are highly permeable compared to Darcy's soils and claye soils can be considered as a impervious soils. It is important to determine the settlement of compressible soil layers.

To identify the seepage through the earth and dams ground water flow to wards wells.

stability of slopes:-

In 1856 Darcy's law has studied flow of water according to Darcy's law the velocity of flow of water through the soil is directly proportional to hydraulic gradient in laminar flow condition in saturated soil.

$$V \propto i \quad V = ci$$

$$\Rightarrow V = ki$$

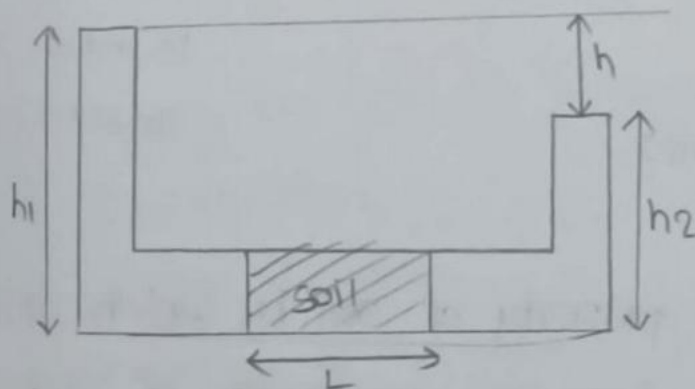
where k = coefficient of permeability

$$Q = V \times A$$

Q = Rate of discharge

A = Area of soil sample

V = velocity of flow.



Loss of head for unit length of flow through the soil is equal to hydraulic gradient.

$$i = \frac{h}{L}$$

$$i = \frac{(h_1 - h_2)}{L}$$

$$Q = VA$$

$$= K i A$$

$$Q = K \left(\frac{h_1 - h_2}{L} \right) A$$

Type of soils	coefficient of permability cm/s
1. gravel	≥ 1
2. sand	10^{-2} to 1
3 silt	10^{-3} to 10^{-5}
4. clay	$< 10^{-6}$

According to USBR, soil has been classified into three groups.

1. Pervious $> 10^{-4}$ cm/s

2. semi pervious 10^{-4} to 10^{-6}

3. Impervious $< 10^{-6}$

validity of Darcy's law:-

It is valid only in laminar flow condition. It is valid in soils Reynolds no $Re \leq 1$

$$Re = \frac{\rho v d}{\mu} \leq 1$$

v = velocity of flow

d = Diameter of particles

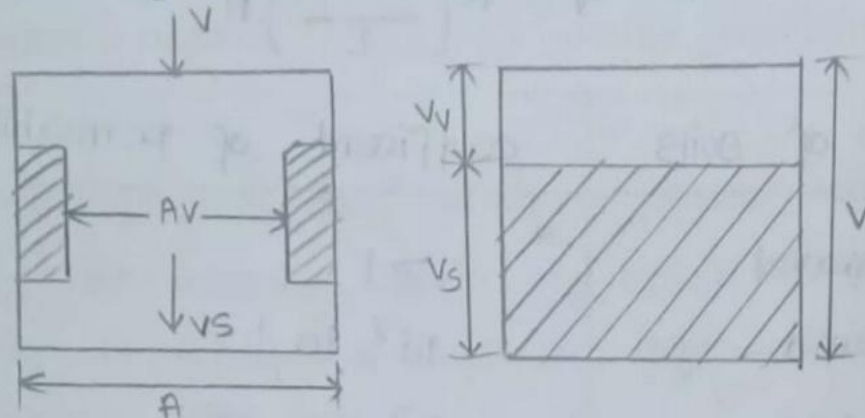
μ = viscosity of the particles

ρ = density of water.

Align hygen theory :-

According to align hygen theory laminar flow condition occurs in soil when ever the maximum size of the particles is less than 3mm.

Discharge velocity and seepage velocity :-



The discharge velocity V is not the actual velocity through the intercept's of the soil it is a velocity obtained by dividing the total discharge cube.

Total cross section Area 'A'

Total area consider solids area and voids area.

The flow of water can be takes place to the voids only. The actual velocity through the voids is much greather thane the discharge velocity. The actual velocity on a microscopic scale we can find is known as seepage velocity V_s

$$\text{seepage velocity } V_s = \frac{q_v}{AV}$$

$$\text{discharge velocity } V = \frac{q_v}{A}$$

$$= \frac{q_v}{AS + AV}$$

True or actual velocity $q = V_A = V_S \cdot A_V$

$$V_S = \frac{V_A}{A}$$

Now multiplied the numerator and denominator by the length (L) we get $V_S = V \times \frac{A \times L}{A_V \times L}$

$$V_S = V \frac{A \times L}{A_V \times L}$$

$$V_S = V \times \left(\frac{V}{V_V} \right)$$

$$\frac{V_V}{V} = \eta$$

$$\frac{V}{V_V} = \frac{1}{\eta}$$

$$V_S = V \left(\frac{1}{\eta} \right)$$

from darcy's law $V = K i \rightarrow (1)$

$$V_S = K_P i \rightarrow (2)$$

dividing this two equations

$$\frac{(1)}{(2)} = \frac{V}{V_S} = \frac{K i}{K_P i}$$

$$\frac{V}{V_S} = \frac{K}{K_P}$$

from that equation $K_P = \frac{V_S K}{V}$

$$K_P = \frac{V_S}{V} \times K$$

$$V_S = \frac{V}{\eta} \quad \left[\because \eta = \frac{V}{V_S} \right] = \frac{V_S}{V}$$

rate of actual velocity $q = V A = V_s \cdot A_v$

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

Now multiplied the numerator and denominator by the length (L) we get $V_s = V \times \frac{A \times L}{A_v \times L}$

$$V_s = V \frac{A \times L}{A_v \times L}$$

$$V_s = V \times \left(\frac{V}{V_v} \right)$$

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

$$\frac{V}{V_v} = \frac{1}{\eta}$$

$$V_s = V \left(\frac{1}{\eta} \right)$$

from darcy's law $V = K i \rightarrow (1)$

$$V_s = K_p i \rightarrow (2)$$

dividing this two equations

$$\frac{(1)}{(2)} = \frac{V}{V_s} = \frac{K i}{K_p i}$$

$$\frac{V}{V_s} = \frac{K}{K_p}$$

from that equation $K_p = \frac{V_s K}{V}$

$$K_p = \frac{V_s}{V} \times K$$

$$V_s = \frac{V}{\eta} \quad \left[\because \eta = \frac{V}{V_s} = \frac{1}{\eta} = \frac{V_s}{V} \right]$$

$$= K_p = \frac{K}{n}$$

where K_p = coefficient of percolation

K = coefficient of permeability.

Factors affecting coefficient of permeability:-

Hagen Poiseuille equation

$$K = D_s^2 (\text{or}) D_d \times \frac{\sqrt{w}}{\eta} \times \frac{e^3}{17e} \times c$$

$K = D_s^2$	$K = \sqrt{w}$	$K = \frac{1}{\eta}$	$K = e$
$K \propto D_s^2$	$K \propto \sqrt{w}$	$K \propto \frac{1}{\eta}$	$K \propto e$

1. size of the particles
2. structure of the soil mass
3. properties of liquid.
4. shape of the particles.
5. void ratio
6. degree of saturation
7. adsorbed water
8. Impurities in water.

Determination of coefficient of permeability:-

The coefficient of permeability of a soil can be determined using the following methods.

a) laboratory methods:-

The coefficient of permeability of a soil sample can be determined by the following methods.

- (i) constant head permeability method.
- (ii) variable head permeability test method.

b) field method:-

The coefficient of permeability of a soil deposit in situ conditions can be determined by the following field methods.

- (i) Pumping out tests
- (ii) Pumping in - tests.

c) Indirect method:-

The coefficient of permeability of the soil can also be determined indirectly from the soil parameters.

- (i) computation from the particle size or its specific surface.
- (ii) computation from the consolidation test data.

d) capillarity permeability test:-

The coefficient of permeability of an unsaturated soil can be determined by the capillarity permeability test.

constant head permeability test :-

The constant head permeability test is recommended for the coarse-grained soil. The measurement is based on the amount of water flows under a given hydraulic gradient with known dimensions. length and cross sectional area of sample in a given time.

The test is conducted in an instrument known as constant head permeameter.

Procedure

1. Apply the vacuum to the soil sample and supply the deaired water to the reservoir to avoid the air bubbles following through the pipe.
2. close the outlet value 'c' and open the inlet values A and B. Value A is used to control the discharge
3. fill the perspex with soil sample.
4. Allow the water to flow through the sample.
5. measure the head of water (h) from manometer construction points provided at the side of the cylinder
6. measure the discharge and repeat the procedure for several time.
7. calculate the value of K as given below

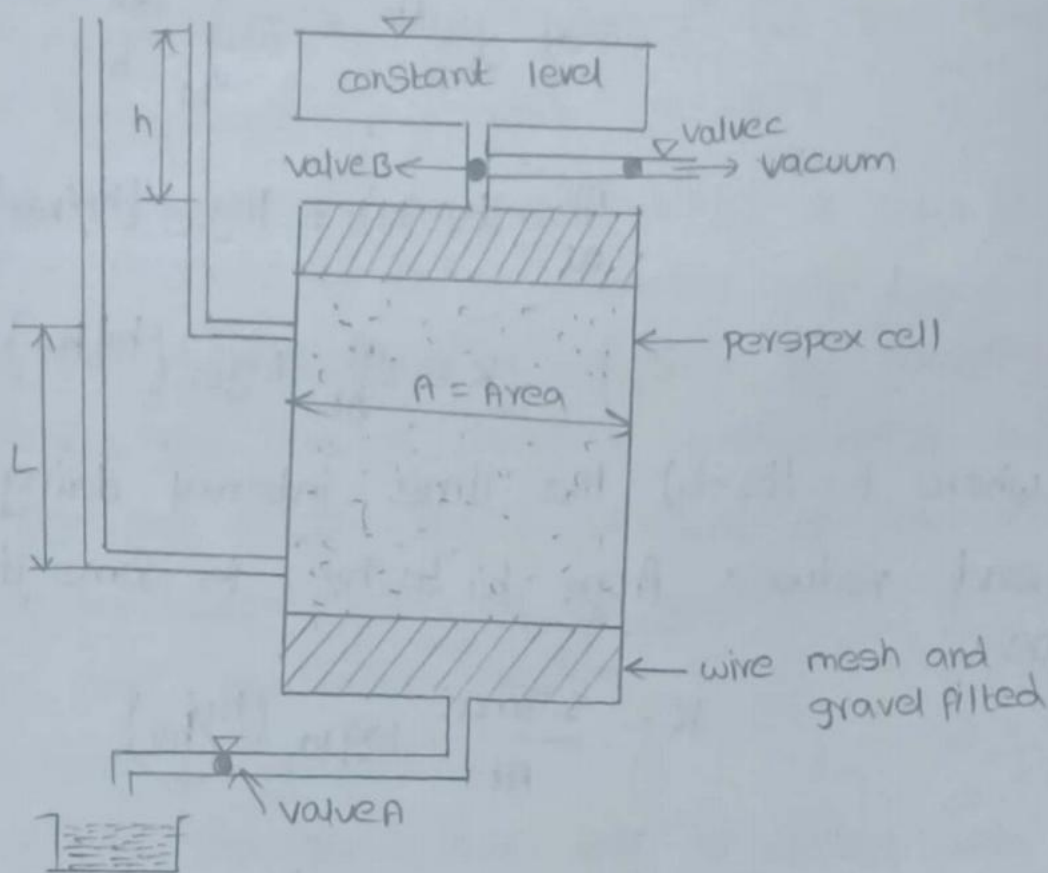
$$K = \frac{Q}{IA} = \frac{QL}{Ath} \text{ cm/sec}$$

where K = coefficient of permeability (cm/s)
 Q = Discharge (cm³) with is collected in time (t)
 A = cross sectional area of the sample (cm²)



h = difference in manometer levels (cm)

L = distance between manometer tapping points (cm)



constant head permeameter.

VARIABLE-head permeability test:-

Let us consider the instand when the head is h .
For the infinitesimal small time dt , the head falls by dh
let the discharge through the sample be q . from
continuity of flow.

$$a dh = -q dt$$

where a is crosssection area of the standpipe.

$$a dh = -(A \times k \times i) dt$$

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

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

Integrating

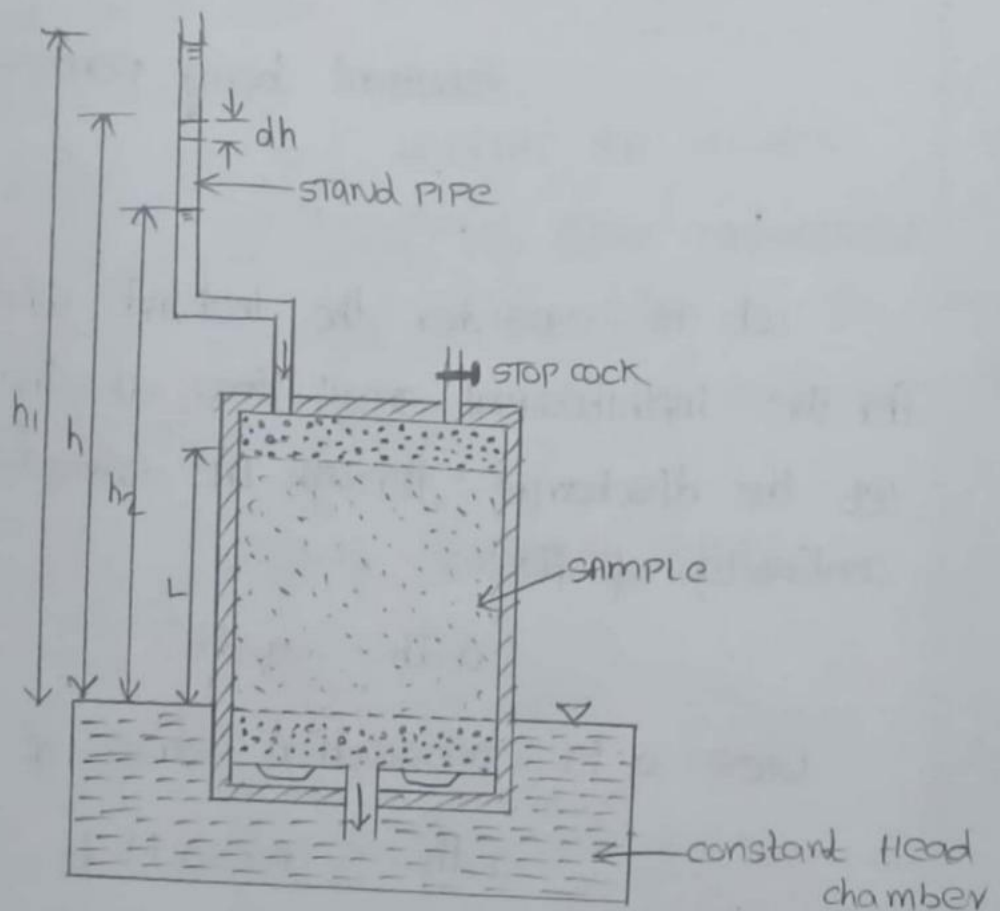
$$\frac{AK}{aL} \int_{t_1}^{t_2} dt = - \int_{h_1}^{h_2} \frac{dh}{h}$$

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

$$K = \frac{aL}{At} \log_e (h_1/h_2)$$

where $t = (t_2 - t_1)$ the time interval during which the head reduces from h_1 to h_2 is some times written as

$$K = \frac{2.30 aL}{At} \log_{10} (h_1/h_2)$$



variable Head permeameter.

Pumping-out Test:-

The field tests are conducted to obtain accurate results of coefficient of permeability. The field tests can be similar to that of pumping out test where the water is pumped out from the wells.

The measurement of permeability of soils is common in major projects by using pumping-out tests. The soil deposit over a huge area is effected by pumping out test and hence the results show a permeability coefficient of a huge soil mass. The coefficient of permeability of soil can be determined by the equations of unconfined aquifer and confined aquifer.

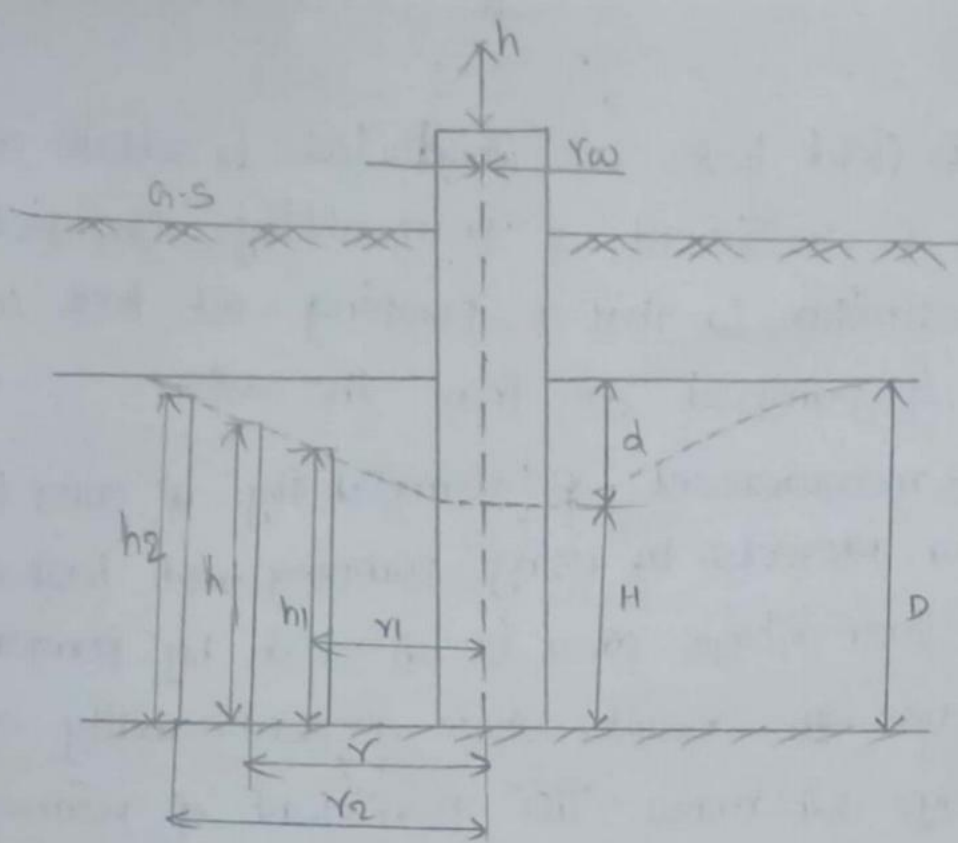
unconfined Aquifer:-

In this process a tube well is drilled upto the underlying impermeable stratum. The tube is perforated and surrounded by a screen known as strainer to monitor the passing of soil particles.

After pumping the water upto the steady state the level of water does not alter and the discharge becomes uniform.

The water table develops an inverted cone close to the well called as cone of depression.

consider the flow through a cylindrical surface of height ' h ' at the radial distance of ' r ' from the centre of well. Therefore from Darcy's law.



unconfined Aquifer

$$q = k i A$$

from figure area $A = 2\pi r h$ and hydraulic gradient is given by $i = \frac{dh}{dr}$

$$q = k \left(\frac{dh}{dr} \right) (2\pi r h)$$

(or)

$$\frac{dr}{r} = \frac{2\pi k h dh}{q}$$

By integrating $\int_1^2 \frac{dr}{r} = \frac{2\pi k}{q} \int_1^2 h dh$

$$\log_e \left(\frac{r_2}{r_1} \right) = \frac{2\pi k}{q} \frac{(h_2^2 - h_1^2)}{2}$$

$$k = \frac{q}{\pi (h_2^2 - h_1^2)} \log_e \left(\frac{r_2}{r_1} \right)$$

(or) $k = \frac{2.30 q}{\pi (h_2^2 - h_1^2)} \log_{10} \left(\frac{r_2}{r_1} \right)$

Therefore, the approximate value of coefficient of permeability is given by

$$K = \frac{qv}{\pi(D^2 - H^2)} \log_e \left(\frac{R}{r_w} \right)$$

Where R = Radius of influence.

$R = 3000 d\sqrt{K}$ according to sichardt.

$R = \left[(12t/n) (qK/\pi)^{1/2} \right]^{1/2}$ according to kozeny

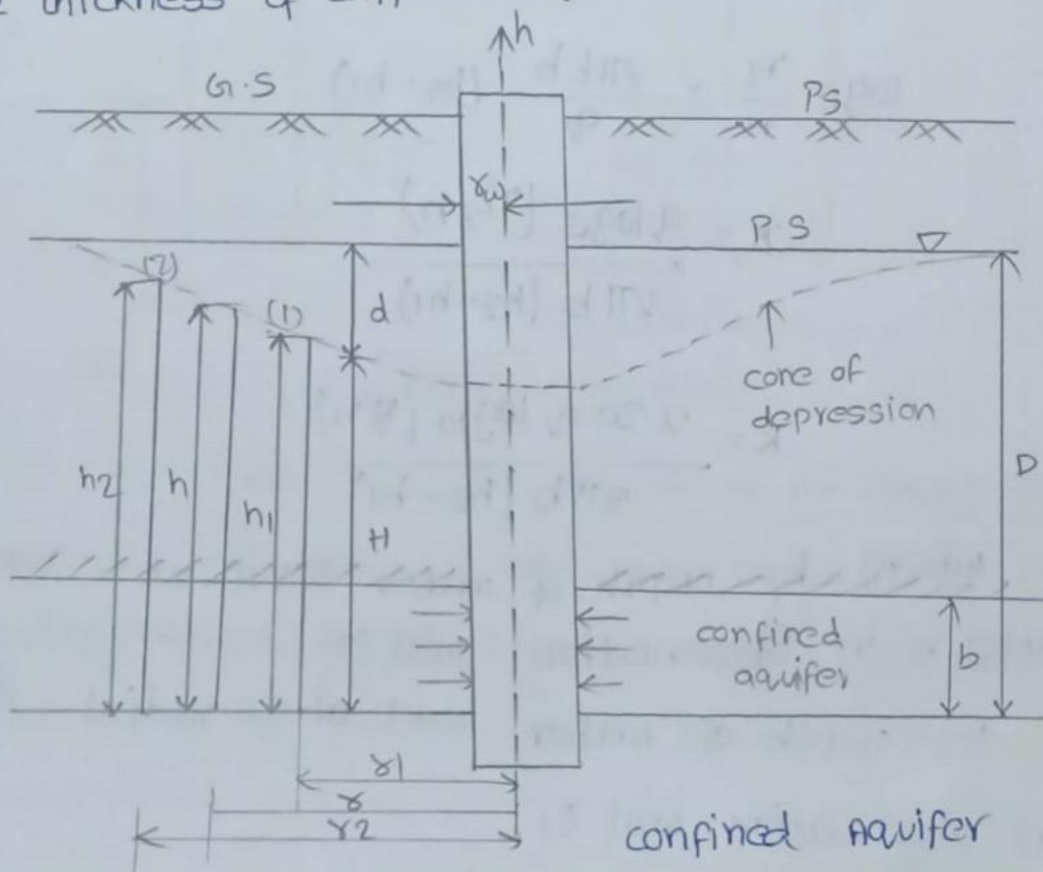
H = depth of water in the well

D = Height of aquifer measured below the water table.

r_w = Radius of test well.

confined Aquifer :-

In confined aquifer the piezometric surface is above the aquifer and the water pressure is denoted by (P_s) . The thickness of confined aquifer is taken as (b) .



The piezometric surface is horizontal in the initial stage but it is depressed during the pumping from the well and the cone of depression is formed.

consider the flow through a cylindrical surface of height 'h' at a radial distance of r from the centre of the well

Therefore from Darcy's law.

$$q = k i a$$

from fig area = $A = 2\pi r b$ and hydraulic gradient is given by $i = \frac{dh}{dr}$

$$\Rightarrow q = k \left(\frac{dh}{dr} \right) (2\pi r b)$$

Integrating

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi k b \cdot dh}{q}$$

$$(\log_e r)_{r_1}^{r_2} = \frac{2\pi k b}{q} [h]_{h_1}^{h_2}$$

$$\log_e \frac{r_2}{r_1} = \frac{2\pi k b}{q} (h_2 - h_1)$$

$$k = \frac{q \log_e (r_2/r_1)}{2\pi b (h_2 - h_1)}$$

$$k = \frac{2.30 q \log_{10} (r_2/r_1)}{2\pi b (h_2 - h_1)}$$

where h_1 = depth of water level at a radial distance r_1 in observation well (1)

h_2 = depth of water level at a radial distance r_2 in observation well (2)

$$K = \frac{q}{55YH}$$

where

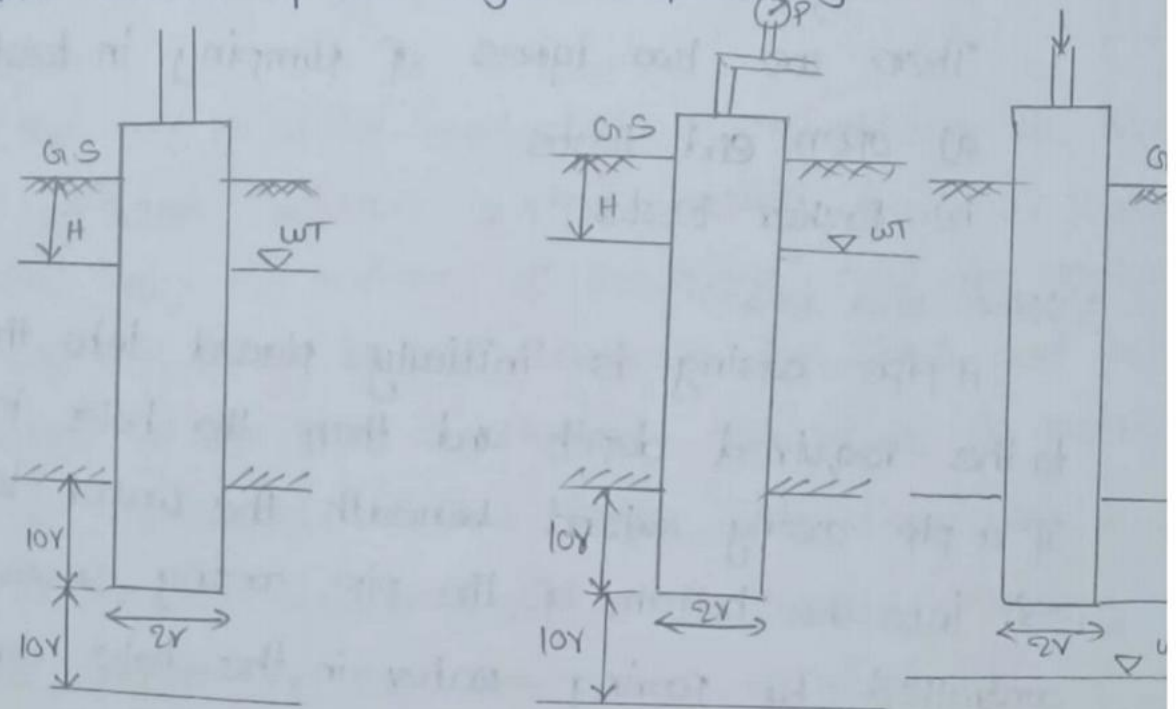
r = Inner radius of pipe casing.

H = difference between the ground surface and water tables.

q = discharge.

water can be pumped in under a pressure P for increasing the discharge, if needed. In this situation the value of H will be equal to $(H + P/\gamma_w)$

To get accurate results the bottom end of the pipe should be at a distance of not less than $10r$ from the top and bottom of the layer respectively.



open-end Tests.

This test can also be carried out above the water table in this case, a constant water level in the pipe casing cannot be maintained easily but allows some increase of the water level. The coefficient of permeability can be determined by using the equation

Therefore, the approximate value of permeability is given by $K = \frac{q \log_e (R/r_w)}{2\pi b (D-H)}$

Pumping in tests:-

In these tests a hole is drilled through a layer and the clean water is filled in it. If the water is turbid then it should be cleaned with the help of filter. The temperature of the water pumped in should be slightly greater than the ground water temperature so that no air bubbles should be developed in layer. Pumping in tests are inexpensive as compared to pumping out test. In these tests, the values of the coefficient of permeability and the hole may be approximately equal.

There are two types of pumping in tests.

- a) open end tests
- b) packer tests

a) open end tests:-

A pipe casing is initially placed into the bore hole to the required depth and then the hole is cleaned out. If a pipe casing extends beneath the water table then the soil into the bottom of the pipe casing squeezes. This is controlled by pouring water in the hole during cleaning.

After cleaning water is filled in the hole by means of metering system. The steady conditions are established when the constant rate of flow is calculated.

The coefficient of permeability is computed by using an equation.

$$K = \frac{q}{5.5 \gamma H}$$

10

where H = Difference of inlet level and the bottom end of the pipe.

The rate of flow can be increased if needed by pumping in water under a pressure P with a total head of $(H + P/\gamma_w)$.

b) Packer test:-

Packer test are conducted in a perforated portion of the pipe casing. If the test is conducted during drilling a top packer is kept just inside or beneath the pipe casing. Due to which water is pumped to the bottom of the hole.

If the test is to be conducted after drilling the hole which can stand without a pipe casing then two Packers are fitted at the bottom of the pipe. Now the perforated portion of the pipe is kept between the plugs and then the bottom of the pipe is plugged. The length of packer should be increased to five times the diameter of the hole. The double packer test is performed initially from the bottom of the hole and continued for the upper layers.

The coefficient of permeability is calculated by using equations.

$$(i) \text{ If } L \geq 10r, K = \frac{q}{2\pi L H} \log_{10} \frac{L}{r}$$

$$\text{iii) If } l_0 r > L \geq r, K = \frac{q}{2\pi L H} \sin H^{-1} \frac{L}{2r}$$

where L = length of part of the hole tested.

Problems:-

1. If the soil sample height 6 cm cross section area 100 cm^2 subjected to a constant head of 18 cm calculate the quantity of water will be collected in 5 minutes time find the coefficient of permeability of the sample.

$$K = \frac{QL}{A \Delta t H}$$

$$K = 4.86 \times 10^{-4} \text{ cm/sec}$$

$$4.86 \times 10^{-4} = \frac{Q \times 6}{100 \times 300 \times 18}$$

$$= \frac{100 \times 300 \times 18}{6} \times 4.86 \times 10^{-4}$$

$$Q = 43.74$$

2. Falling head parameter accommodate a soil sample 6 cm height 250 cm^2 area the permeability of the sample expected to $1 \times 10^{-4} \text{ cm/sec}$. If it is design that the head in standard pip should fall from 30 cm to 10 cm in 40 minutes determine the size of the standard pip which should be used.

$$K = 2.3 \frac{aL}{At} \log_{10} \left(\frac{h_1}{h_2} \right)$$

$$1 \times 10^{-4} = 2.3 \frac{a \times 6}{250 \times 40 \times 60} \log_{10} \left(\frac{30}{10} \right)$$



$$\text{iii) If } 10r > L \geq r, K = \frac{q}{2\pi L H} \sin H^{-1} \frac{L}{2r}$$

where L = length of part of the hole tested.

Problems:-

1. If the soil sample height 6 cm cross section area 100 cm^2 subjected to a constant head of 18 cm calculate the quantity of water will be collected in 5 minutes time find the coefficient of permeability of the sample.

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$$K = 2.3 \frac{aL}{At} \log_{10} \left[\frac{h_1}{h_2} \right]$$

$$1 \times 10^{-4} = 2.3 \frac{a \times 6}{250 \times 40 \times 60} \log_{10} \left[\frac{30}{10} \right]$$



$$= 1 \times 10^{-4} = 2.3 \cdot \frac{9 \times 6}{600000} \times 0.477$$

$$60 = 6.58 \times a$$

$$a = \frac{60}{6.58} = 9.118$$

$$d^2 = 9.118$$

$$d = \sqrt{9.118} \times \pi/4$$

$$d = 3.40$$

- 3 In during a permeability test on a soil sample with a falling head method equal to the time intervals are noted for drops of head from h_1 to h_2 and again from h_2 to h_3 find the relation between h_1, h_2, h_3 .

$$K = 2.3 \frac{aL}{At} \log_{10} \left[\frac{h_1}{h_2} \right]$$

(i) 1 time duration

$$K_1 = 2.3 \frac{aL}{At} \log_{10} \left[\frac{h_1}{h_2} \right]$$

(ii) 2 time duration

$$K_2 = 2.3 \frac{aL}{At} \log_{10} \left[\frac{h_2}{h_3} \right]$$

now equating the K_1 and K_2

$$\Rightarrow 2.3 \times \frac{aL}{At} \log_{10} \left[\frac{h_1}{h_2} \right] =$$

$$2.3 \times \frac{aL}{At} \log_{10} \left[\frac{h_2}{h_3} \right]$$

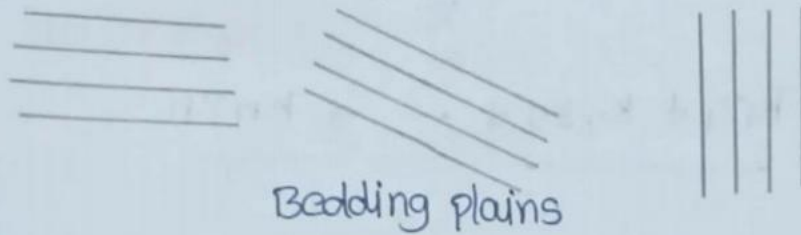
$$\Rightarrow \log_{10} \left[\frac{h_1}{h_2} \right] = \log_{10} \left[\frac{h_2}{h_3} \right]$$

$$\Rightarrow \frac{h_1}{h_2} = \frac{h_2}{h_3}$$

$$\Rightarrow \underline{h_2^2 = h_1 h_3}$$

Permiability of stratified soil deposits:- (2)

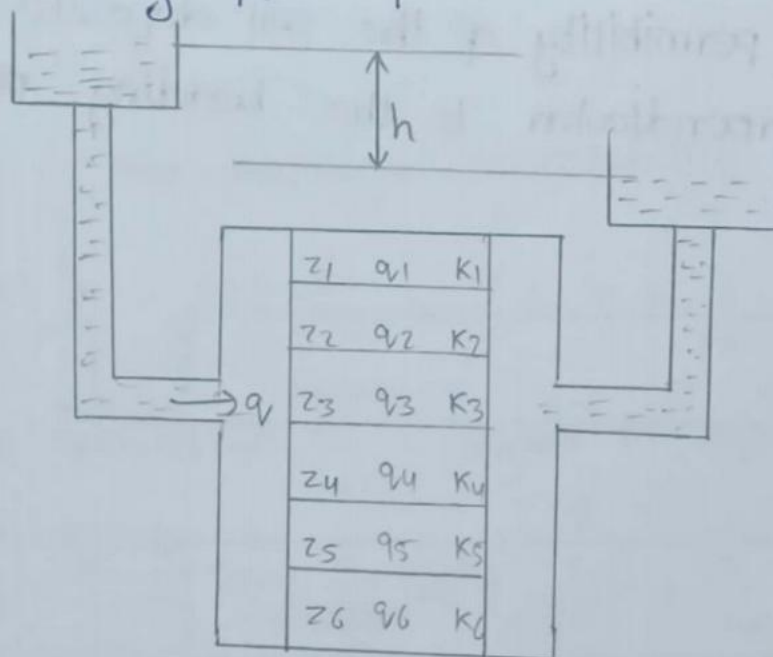
A soil are deposited one over the another this are deposited in different plains. planes.



Bedding plains

case(i) :-

Average permiability of the soil deposite when the flow is taking place parallel to the bedding plain planes.



let us consider $z_1, z_2, z_3 \dots z_n$ Thick ness of soil layers . $K_1, K_2, K_3 \dots K_n$ coefficent of permiability of individual layers. h = head difference between upstream side and down stream side discharge

$$q = q_1 + q_2 + q_3 + \dots + q_n$$

$$q = KiA \quad \text{from darcy's law}$$

$$Kiz(l) = K_1 iz_1(l) + K_2 iz_2(l) + \dots + K_n iz_n(l)$$

$$KZ = K_1Z_1 + K_2Z_2 + \dots + K_nZ_n$$

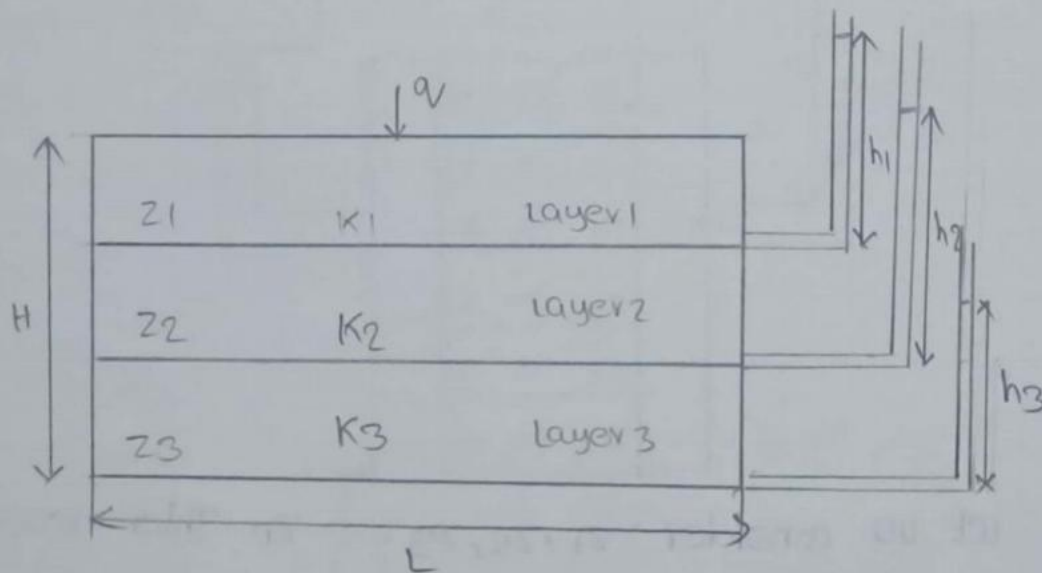
$$K = \frac{K_1Z_1 + K_2Z_2 + \dots + K_nZ_n}{Z}$$

$$K_n = \frac{K_1Z_1 + K_2Z_2 + \dots + K_nZ_n}{Z}$$

$$Z = Z_1 + Z_2 + Z_3 + \dots + Z_n$$

case (iii)

Average permeability of the soil deposit when the flow is perpendicular to the bedding plain planes



$$h = h_1 + h_2 + h_3 + \dots + h_n$$

The average permeability of the soil K_v from Darcy's law

$$v = Ki$$

$$= K_v \frac{h}{L}$$

$$[\because i = h/L]$$

$$= K_v \frac{h}{Z}$$

$$L = Z$$

Now $Z = Z_1 + Z_2 + Z_3 + \dots + Z_n$

$$i_1 = \frac{h_1}{Z_1}$$

$$h_1 = i_1 Z_1$$

similarly $h_2 = i_2 Z_2$

$$h_3 = i_3 Z_3$$

$$h_n = i_n Z_n$$

$$iZ = i_1 Z_1 + i_2 Z_2 + i_3 Z_3 + \dots + i_n Z_n \rightarrow (1)$$

But $v = Ki$

$$i = \frac{v}{K}$$

K_v is the average permeability of soil

$$i_1 = \frac{v}{K_1}, i_2 = \frac{v}{K_2} \dots i_n = \frac{v}{K_n}$$

now substitute in i value in eqn(1) we get

$$\frac{v}{K_v} Z = \frac{v}{K_1} Z_1 + \frac{v}{K_2} Z_2 + \dots + \frac{v}{K_n} Z_n$$

$$\frac{Z}{K_v} = \frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \dots + \frac{Z_n}{K_n}$$

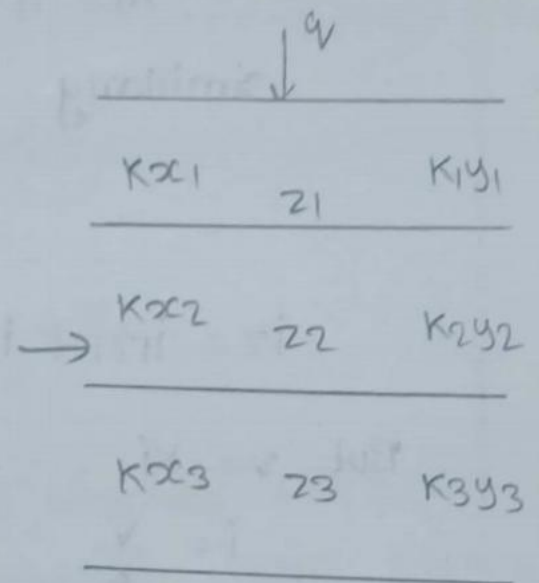
$$K_v = \frac{Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \dots + \frac{Z_n}{K_n}}$$

If the same permeability is having when the flow is perpendicular or parallel that soil is said to be isotropic soil. $K_x = K_v$

when $K_x \neq K_v$ that type of soil is called as non isotropic soils.

$$K_h = \frac{Kx_1z_1 + Kx_2z_2 + \dots + Kx_nz_n}{z}$$

$$K_v = \frac{z}{\frac{z_1}{Kx_1} + \frac{z_2}{Kx_2} + \frac{z_3}{Kx_3}}$$



14
A horizontal stratified soil deposit consist of 3 layers the Permiability of each layer is given 8×10^{-4} cm/sec, 52×10^{-4} cm/sec, and 6×10^{-4} cm/sec and their thickness is 5mts, 3mts, 7mts Respectively find the coefficient of permiability in horizontal and vertical direction.

$$z_1 = 5 \text{ mts}$$

$$z_2 = 3 \text{ mts}$$

$$z_3 = 7 \text{ mts}$$

$$k_1 = 8 \times 10^{-4} \text{ cm/sec}$$

$$k_2 = 52 \times 10^{-4} \text{ cm/sec}$$

$$k_3 = 6 \times 10^{-4} \text{ cm/sec}$$

z_1	5mts
z_2	3mts
z_3	7mts

$$k_h = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3}{z}$$

$$= \frac{8 \times 10^{-4} \times 5 + 52 \times 10^{-4} \times 3 + 6 \times 10^{-4} \times 7}{5+3+7}$$

$$= 1.586 \times 10^{-3}$$

$$k_v = \frac{z}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}}$$

$$= \frac{5+3+7}{\frac{5}{8 \times 10^{-4}} + \frac{3}{52 \times 10^{-4}} + \frac{7}{6 \times 10^{-4}}}$$

$$= 8.1109 \times 10^{-4}$$

- 2 A soil deposite has 4 layers the thickness of each layer is 2mts, 3mts, 4mts, 2mts. The Permiability of each horizontal layer 6×10^{-4} , 2×10^{-4} , 3×10^{-3} , 5×10^{-3} and The Permiability in vertical direction 1×10^{-4} , 4×10^{-3} , 7×10^{-4} .

6×10^{-3} find the average permeability of horizontal and vertical direction.

Given that

$$z_1 = 2 \text{ mts}$$

$$z_2 = 3 \text{ mts}$$

$$z_3 = 4 \text{ mts}$$

$$z_4 = 2 \text{ mts}$$

$$K_1 = 6 \times 10^{-4}$$

$$K_2 = 2 \times 10^{-2}$$

$$K_3 = 3 \times 10^{-3}$$

$$K_4 = 5 \times 10^{-3}$$

2m	$K_1 = 6 \times 10^{-4}$
3m	$K_2 = 2 \times 10^{-2}$
4m	$K_3 = 3 \times 10^{-3}$
2m	$K_4 = 5 \times 10^{-3}$

$$K_h = \frac{K_1 z_1 + K_2 z_2 + K_3 z_3 + K_4 z_4}{z}$$

$$= \frac{60 \times 10^{-4} \times 2 + 2 \times 10^{-2} \times 3 + 3 \times 10^{-3} \times 4 + 5 \times 10^{-3} \times 2}{2+3+4+2}$$

$$= \frac{1.2 \times 10^{-3} + 0.06 + 0.012 + 0.01}{2+3+4+2}$$

$$= 7.563 \times 10^{-3}$$

$$K_v = \frac{z}{\frac{K_1}{z_1} + \frac{K_2}{z_2} + \frac{K_3}{z_3} + \frac{K_4}{z_4}}$$

$$= \frac{2+3+4+2}{\frac{1 \times 10^{-4}}{2} + \frac{4 \times 10^{-3}}{3} + \frac{7 \times 10^{-4}}{4} + \frac{6 \times 10^{-3}}{2}}$$

$$= \frac{2+3+4+2}{5 \times 10^{-5} + 1.33 \times 10^{-3} + 1.75 \times 10^{-4} + 3 \times 10^{-3}}$$

$$= 4.104 \times 10^{-4}$$

2m	$K_1 = 1 \times 10^{-4}$
3m	$K_2 = 4 \times 10^{-3}$
4m	$K_3 = 7 \times 10^{-4}$
2m	$K_4 = 6 \times 10^{-3}$

3. The sub soil at a site consists of fine sand layer lying in between a clay layer at top and a silt layer at the bottom the coefficient of permeability of sand layer is $\frac{1}{10}$ that of clay and $\frac{1}{4}$ that of silt find out the equivalent coefficient permeability deposit in direction parallel and perpendicular to the bedding plane in terms of coefficient of permeability of clay layer.

$$K_{\text{sand}} = \frac{1}{10} K_c = 0.1 K_c$$

$$K_{\text{sand}} = \frac{1}{4} K_{\text{silt}}$$

$$K_{\text{silt}} = 0.4 K_c$$

$$Z_1 = Z_2 = Z_3 = Z$$

$$K_h = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z}$$

$$= \frac{K_c \cdot Z + 0.1 K_c \cdot Z + 0.4 K_c \cdot Z}{Z}$$

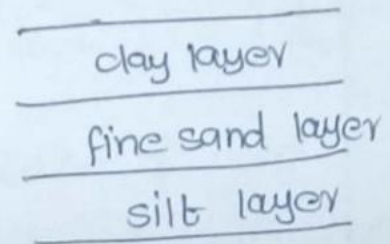
$$= \frac{Z(K_c + 0.1 K_c + 0.4 K_c)}{Z}$$

$$K_h = 1.5 K_c$$

$$K_v = \frac{Z}{\left(\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3} \right)}$$

$$= \frac{Z}{\left(\frac{Z}{K_c} + \frac{Z}{0.1 K_c} + \frac{Z}{0.4 K_c} \right)}$$

$$= \frac{Z}{K_c} \left(1 + \frac{1}{0.1} + \frac{1}{0.4} \right)$$



$$= \frac{Z}{\frac{Z}{KC} (13.5)}$$

$$= \frac{Z}{\frac{13.5Z}{KC}}$$

$$= \cancel{Z} \times \frac{KC}{13.5\cancel{Z}} = \frac{KC}{13.5}$$

$$KV = \underline{\underline{0.074 KC}}$$

Total effective and natural pressure:-

16

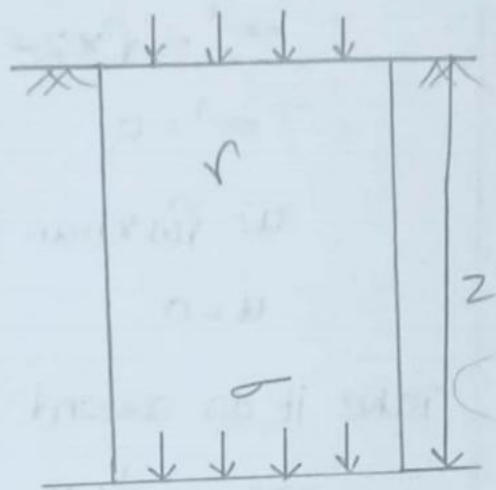
$$\gamma = \frac{P}{V}$$

$$P = \gamma V = \gamma \cdot h \times A$$

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

$$\sigma = \frac{\gamma \cdot h \times A}{A}$$

$$\sigma = \gamma \cdot h$$



Total stress σ — It is the combination of two distribution of Total stress $\sigma =$ effective pressure + pore water pressure.

Effective pressure (or) Inter granular pressure (σ'):-

It is the pressure transmitted from particle to particles in their point of contact with the particles in soil.

Pore water pressure:-

The pressure which is transmitted from pore to pore in the soil is called as pore water pressure denoted by u .

$$\sigma = \sigma' + u$$

$$\sigma' = \sigma - u$$

$$u = \sigma - \sigma'$$

case (i) :- To find the total stress of submerged soil shown in fig.

Take it as first section A-A

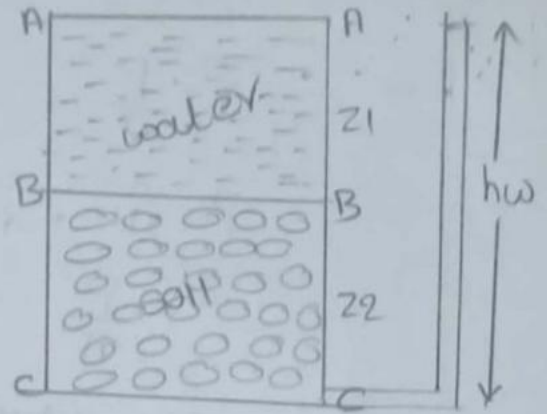
$$\sigma = \sigma' + u$$

$$\sigma' = \gamma \times z$$

$$\sigma' = 0$$

$$u = \gamma_w \times h_w$$

$$u = 0$$



Take it as second section B-B

$$\sigma = \sigma' + u$$

$$\sigma' = 0$$

$$u = \gamma_w \cdot z_1$$

Take it as section Third c-c

$$\sigma = \sigma' + u$$

$$\sigma' = \gamma_{sat} \times z_2$$

$$u = \gamma_w \times h_w$$

$$u = \gamma_w \times z_1$$

$$u = \gamma_{sat} \times z_2 + \gamma_w \times z_1$$

Pore water pressure $u = \gamma_w h_w$

$$= \gamma_w [z_1 + z_2]$$

now find the effective stress

$$\sigma' = \sigma - u$$

$$\sigma' = \gamma_{sat} \times z_2 - \gamma_w [z_1 + z_2] + \gamma_w z_1$$

$$= \gamma_{sat} z_2 - \gamma_w z_1 - \gamma_w z_2 + \gamma_w z_1$$

$$= \gamma_{sat} z_2 - \gamma_w z_2$$

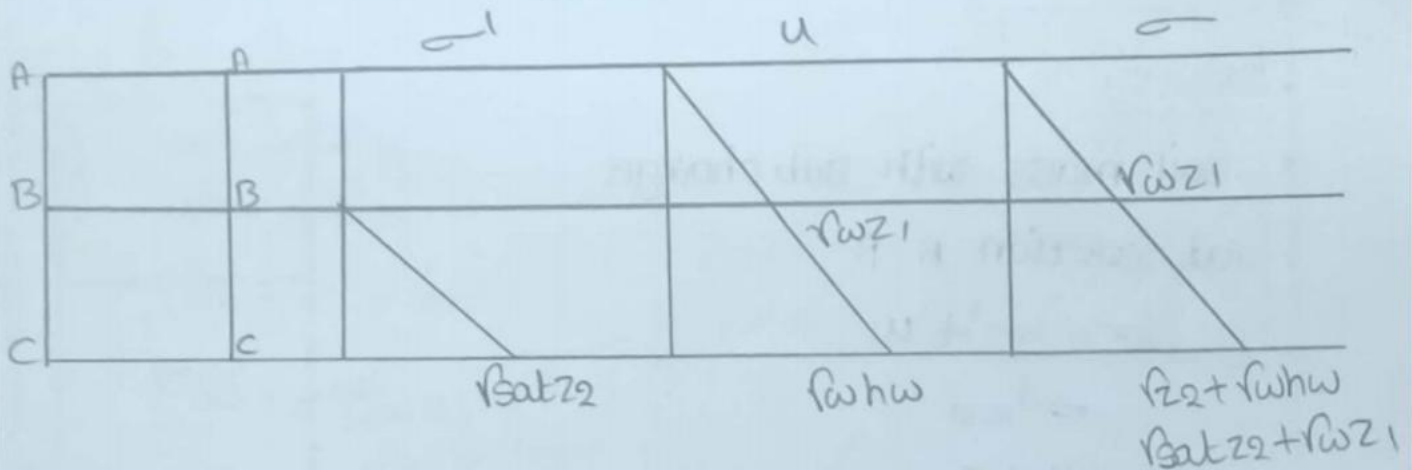
17

$$= [\gamma_{sat} - \gamma_w] z_2$$

$$= [\gamma' + \gamma_w - \gamma_w] z_2$$

$$= \gamma' z_2$$

stress distribution diagram:-



2) At section A-A

$$\sigma = \sigma' + u$$

$$\sigma = 0$$

at section B-B

$$\sigma = \sigma' + u$$

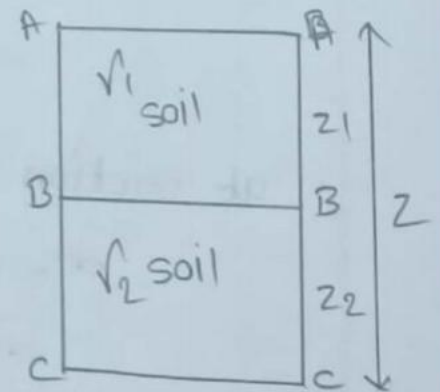
$$\sigma' = \gamma h = \gamma_1 z_1 \quad u = 0$$

$$\sigma = \gamma_1 z_1$$

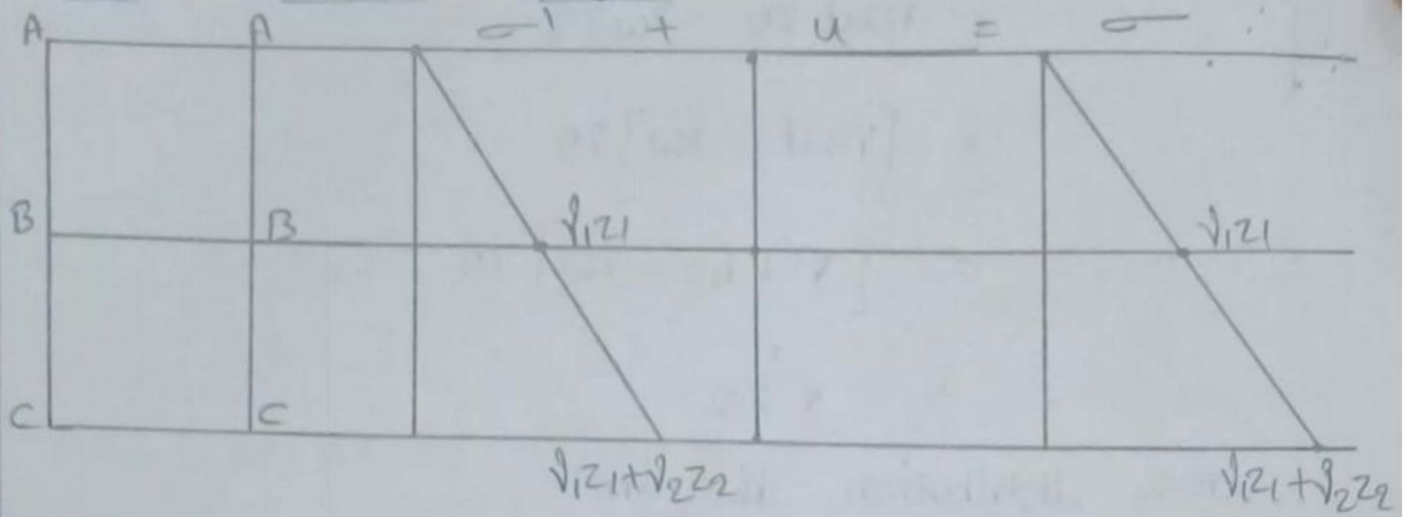
at section C-C

$$\sigma = \sigma' + u$$

$$\sigma' = \gamma_1 z_1 + \gamma_2 z_2 \quad u = 0$$



stress distribution diagram :-



case (ii)

soil mass with sub charge
at section A-A

$$\sigma = \sigma' + u$$

$$\sigma' = 0$$

$$u = 0$$

$$\sigma = 0 + 0 + q$$

$$\sigma = q$$

at section B-B

$$\sigma = \sigma' + u + q$$

$$\sigma' = \gamma_1 z_1$$

$$u = 0$$

$$\sigma = \gamma_1 z_1 + q$$

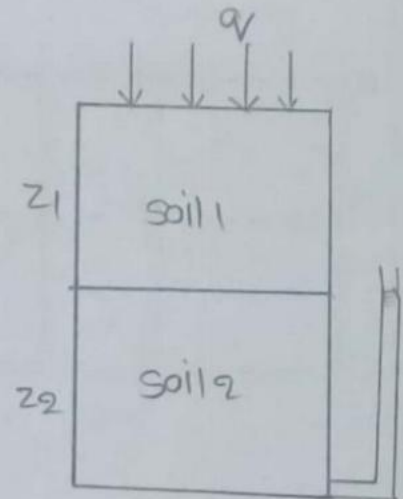
at section c-c

$$\sigma = \sigma' + u + q$$

$$\sigma' = \gamma_1 z_1 + \gamma_{sat} z_2$$

$$\therefore u = \gamma_w z_2$$

$$\sigma = \gamma_1 z_1 + \gamma_{sat} z_2 + q$$



$$\sigma' = \sigma - u$$

$$= \gamma z_1 + \gamma_{sat} z_2 + q - \gamma_w z_2$$

$$= \gamma z_1 + \gamma_w z_2 + \gamma' z_2 - \gamma_w z_2 + q$$

$$\sigma' = \gamma z_1 + \gamma' z_2 + q$$

case (iii) saturated soil with capillary rising.

at section A-A

$$u = -\gamma_w z_1$$

$$\sigma = \sigma' + u$$

$$0 = \sigma' + u$$

$$0 = \sigma' - \gamma_w z_1$$

$$\sigma' = \gamma_w z_1$$

$$\sigma = \sigma' + \gamma_w z_1$$

$$\gamma_w z_1 - \gamma_w z_1 = 0$$

at section B-B

$$\sigma = \gamma_{sat} z_1$$

$$u = 0$$

$$\sigma' = \gamma_{sat} z_1$$

at section C-C

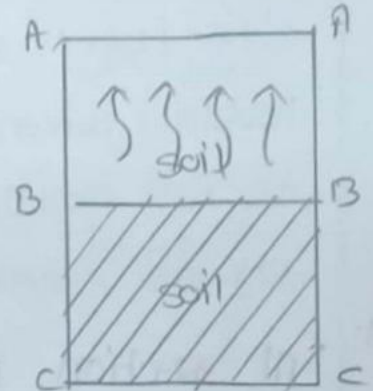
$$\sigma = \gamma_{sat} (z_1 + z_2)$$

$$u = \gamma_w z_2$$

$$\sigma' = \sigma - u$$

$$= \gamma_{sat} z_1 + \gamma_{sat} z_2 - \gamma_w z_2$$

$$= \gamma' z_1 + \gamma_{sat} z_2$$



Problems:-

1. A soil profile consist of a surface layer are sand 3mts thick (saturated density) $\gamma_{sat} = 16 \text{ kN/m}^3$ and intermediate clay layer 2mts thick saturated unit weight $\gamma_{sat} = 19.25 \text{ kN/m}^3$ and a bottom layer of gravel 4 mts thick saturated unit weight $\gamma_{sat} = 19 \text{ kN/m}^3$. The water table is at the top of clay layer determine the effective stress at varies levels. when a subcharge of 50 kN/m^2 is placed at the ground surface draw the total effective stress pore water pressure diagram.

Sol.

At section A-A

$$\sigma = 50$$

$$\sigma = \sigma' + u$$

$$u = 0$$

$$\sigma' = 50$$

At section B-B

$$\sigma = \sigma' + u$$

$$\sigma = 50 + 16 \times 3$$

$$u = 0$$

$$\sigma = \sigma' + u$$

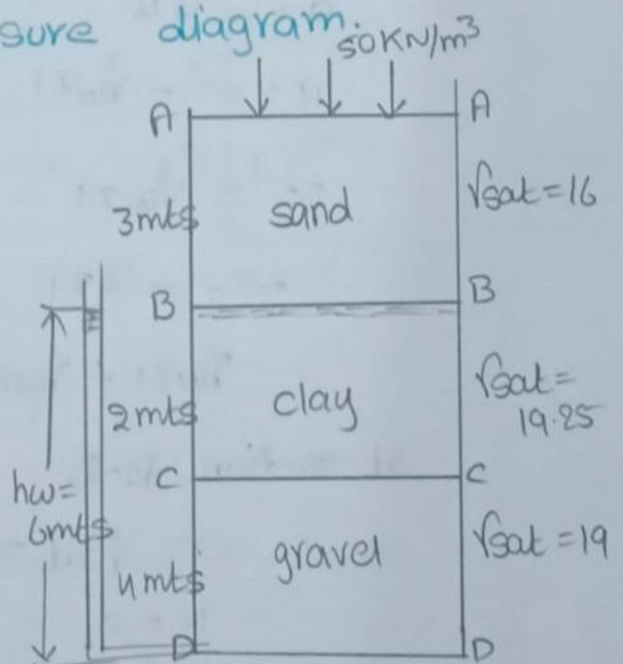
$$\sigma' = 98$$

At section C-C

$$\sigma = 50 + 16 \times 3 + 19.25 \times 2$$

$$= 136.5$$

$$u = \gamma_w \times h_w$$



$$= 9.81 \times 2 = 19.62 \text{ kN/m}^2$$

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$$\sigma = \sigma' + u$$

$$\sigma' = 136.5 - 19.62$$

$$= 116.88 \text{ kN/m}^2$$

At section D-D

$$\sigma = 50 + 16 \times 3 + 19.25 \times 2 + 19 \times 4$$

$$= 212.5 \text{ kN/m}^2$$

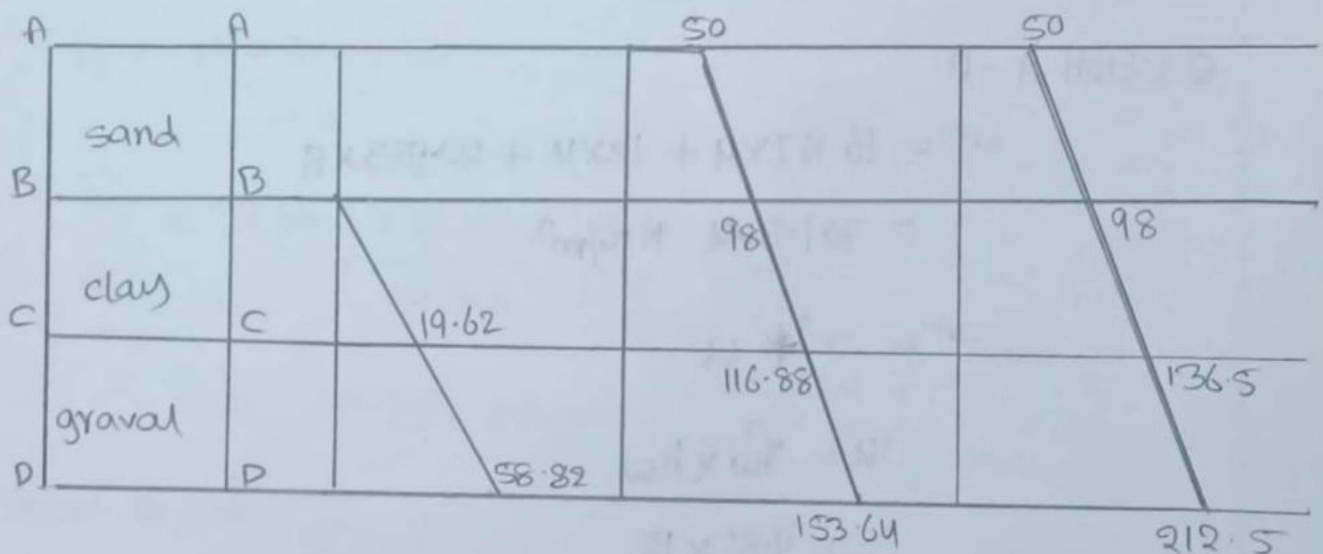
$$\sigma = \sigma' + u$$

$$u = \gamma_w \times h_w$$

$$= 9.81 \times 6 = 58.86 \text{ kN/m}^2$$

$$\sigma' = 212.5 - 58.86$$

$$= 153.64$$



$$u + \sigma' = \sigma$$

2. A sand deposit 8 mts thick over lies a clay stratum of 12 mts thick the void ratio and specific gravity of clay is 0.6 and 2.7 the water table is at a depth of 4 mts from the submerged sand has a state unit weight of 18 kN/m^3 and the moisture

sand which is above the water table the density is 16.873 kN/m^3 calculate the total effective pore water pressure at a depth of 8 mts from the surface of clay layer.

sol

given that

$$\text{dry sand} = 16.873 \text{ kN/m}^3$$

$$\text{submerged sand} = 18 \text{ kN/m}^3$$

step 1:- find the saturated density

$$\gamma_{\text{sat}} = \frac{(G+e)\gamma_w}{1+e}$$

$$= \frac{0.6 + 2.7}{1 + 0.6} \times 9.81$$

$$= 20.233$$

section A - A

$$\sigma = 16.87 \times 4 + 18 \times 4 + 20.233 \times 8$$

$$= 301.344 \text{ kN/m}^2$$

$$\sigma = \sigma' + u$$

$$u = \gamma_w \times h_w$$

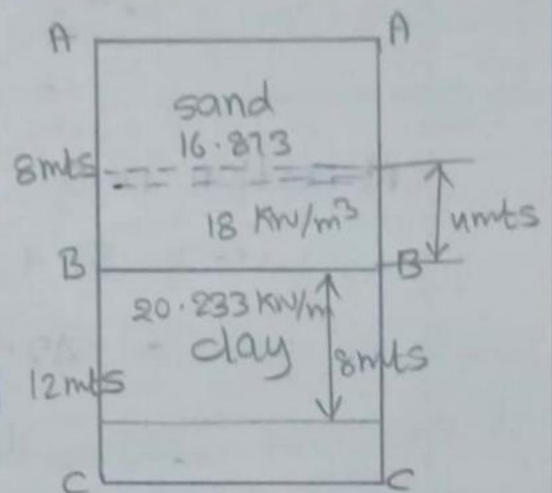
$$= 9.81 \times 12$$

$$= 117.72$$

$$\sigma' = \sigma - u$$

$$\sigma' = 301.344 - 117.72$$

$$\sigma' = 183.634$$



3 For a given soil profile compute the effective 20 Pressure and pore water pressure at a depth of 12mts from the surface.

First layer

$$\gamma_1 = \frac{(n + sre) \gamma_w}{1 + e}$$

$$e = \frac{n}{1 - n} = \frac{0.4}{1 - 0.4} = 0.666$$

$$\gamma_1 = \frac{(2.65 + 0.3 \times 0.666) 9.81}{1 + 0.666}$$

$$= 16.77 \text{ (or)} 16.84 \text{ kN/m}^3$$

second layer:-

$$\gamma_2 = \frac{(n + sre) \gamma_w}{1 + e}$$

$$= \frac{(2.68 + 0.8 \times 0.6) 9.81}{1 + 0.6}$$

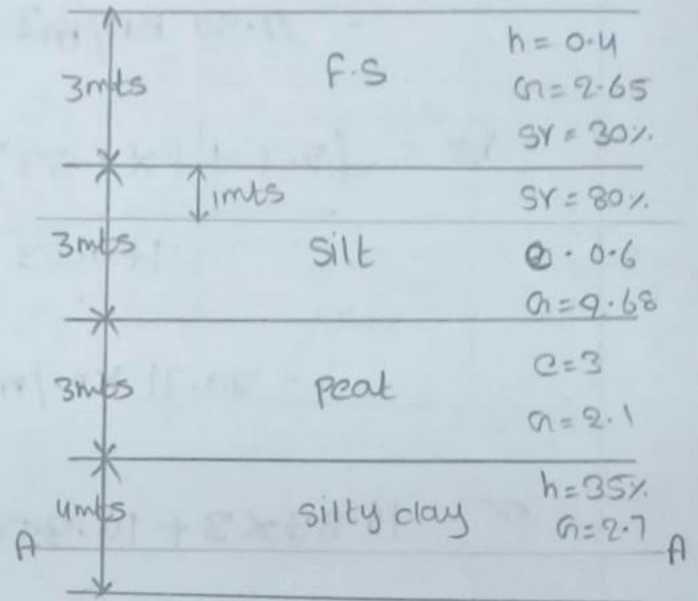
$$= 19.35 \text{ kN/m}^3$$

Third layer:-

$$\gamma_3 = \frac{(n + sre) \gamma_w}{1 + e}$$

$$= \frac{(2.68 + 1 \times 0.6) 9.81}{1 + 0.6}$$

$$= 9.85 \text{ kN/m}^3$$



$$\gamma_u = \frac{(2.1 + 1 \times 3) \cdot 9.81}{1 + 3}$$

$$= 12.50 \text{ kN/m}^3$$

$$\gamma_s = \frac{(2.7 + 1 \times 0.53) \cdot 9.81}{1 + 0.53}$$

$$= 20.71 \text{ kN/m}^3$$

$$e = \frac{n}{1-n} = \frac{0.35}{1-0.35} = 0.53$$

$$\sigma = 16.83 \times 3 + 19.35 \times 1 + 9.85 \times 2 + 12.50 \times 3 + 20.71 \times 4$$

$$= 209.88$$

$$u = \gamma_w \times h_w$$

$$= 9.81 \times 8 = 78.48$$

$$\sigma' = 209.83 - 78.48$$

$$= 131.4 \text{ kN/m}^2$$

=

- u A soil profile consists of a surface layer of sand 3 mts thick ($\gamma_{sat} = 16 \text{ kN/m}^3$) an intermediate clay layer 1.5 mts thick ($\gamma_{sat} = 19 \text{ kN/m}^3$) and a bottom layer of gravel 5 mts thick ($\gamma_{sat} = 20 \text{ kN/m}^3$). The water table is at the top of the clay layer. determine the effective stress at various levels when a surcharge of 50 kN/m^2 is placed at the ground surface draw the total effective and pore water pressure diagram.

sol) at section A-A

$$\sigma = 50$$

$$\sigma = \sigma' + u$$

$$u = 0$$

$$\sigma = \sigma'$$

$$\sigma' = 50$$

at section B-B

$$\sigma = \sigma' + u$$

$$\sigma = 50 + 16 \times 3$$

$$u = 0$$

$$\sigma = \sigma' + u$$

$$\sigma' = 98$$

at section C-C

$$\sigma = 50 + 16 \times 3 + 19 \times 1.5$$

$$= 126.5$$

$$\sigma = \sigma' + u$$

$$u = \gamma_w \times h_w$$

$$= 9.81 \times 1.5$$

$$= 14.715$$

$$\sigma' = 126.5 - 14.715$$

$$= 111.785$$

at section D-D

$$\sigma = 50 + 16 \times 3 + 19 \times 1.5 + 20 \times 5$$

$$= 226.5$$



$$\sigma_2 = \sigma_1 + u$$

$$u = \gamma_w \times h_{1/2}$$

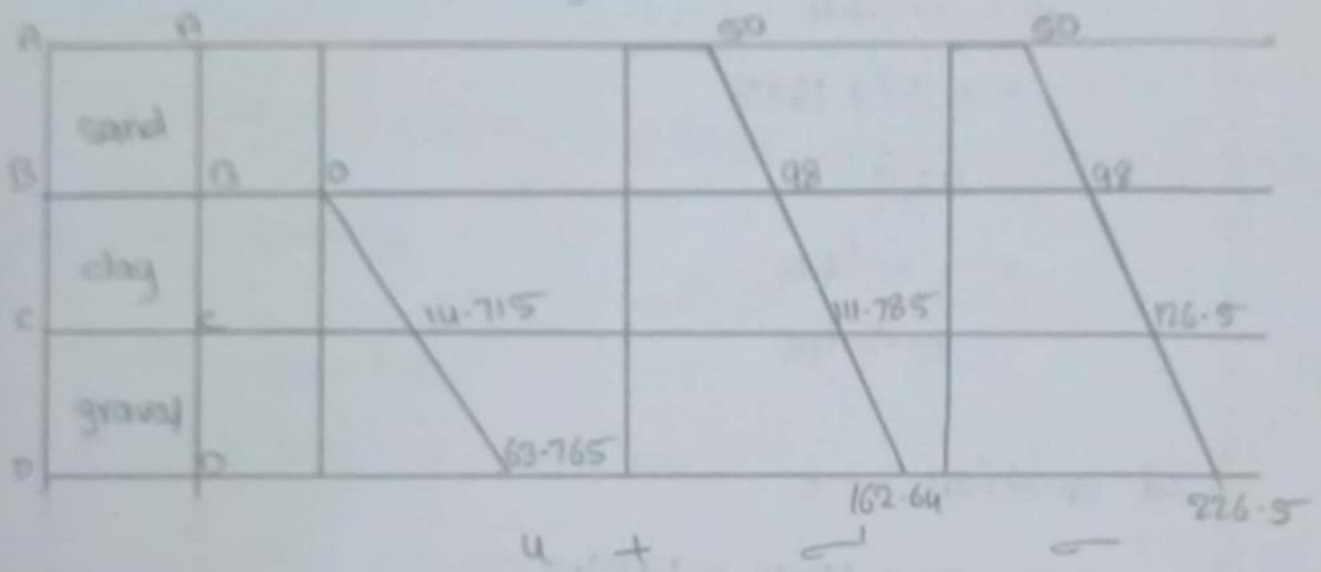
$$= 9.81 \times 6.5$$

$$= 63.765$$

$$\sigma_1 = 226.5 - 63.765$$

$$= 162.735$$

stress distribution diagram:-

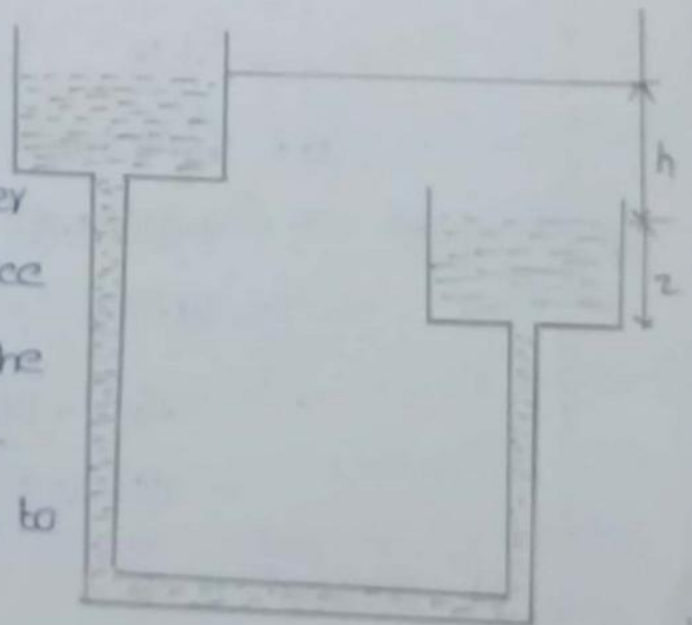


Seepage pressure:-

seepage:-

Seepage is the flow of water under gravity or gravitational force in a permeable medium. The flow of water takes place from a point of high head to a point of lower head.

This flow is laminar flow.



seepage pressure :-

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seepage pressure is the pressure exerted by the water on the soil mass through which it's percolates. The force acts in a direction of flow.

This force is known as drag force (or) seepage force.

seepage force represented by P_s .

$$P_s = \gamma_w h$$

h = head loss (or) head difference due to the flow of water through the soil

$$P_s = \gamma_w \frac{h}{z} \times z$$

$$P_s = \gamma_w i \cdot z \quad \therefore i = \frac{h}{z}$$

The above case water moves upward direction the net effective pressure decreased due to seepage pressure and the effective pressure is increased. In the water flow in downward direction because downward direction the soil particles become closer.

$$\sigma = \sigma' + u$$

$$\sigma = \sigma' \pm P_s$$

$$\sigma = \sigma' - P_s \quad \text{= up word flow}$$

$$\sigma = \sigma' + P_s \quad \text{= down word flow}$$

now the hydrolic pressure is due to head (h)

The seepage force (J) act on soil skeleton due to flow of water through frictional drag given by

$$J = \gamma_w \times h \times A$$

The seepage pressure $P_s = \frac{J}{A} = \gamma_w \times h$

$$\text{now } P_s = i \cdot \gamma_w \cdot h$$

now the seepage force J can be expressed as the force per unit volume $J = \frac{J}{A \cdot L}$

$$= \frac{\gamma_w \cdot h \cdot A}{A \cdot L}$$

$$= \gamma_w \frac{h}{L} \quad (\because L = z)$$

$$J = \gamma_w i$$

Quick sand condition (or) Boiling condition:-

Quick sand is a type of flow condition when the flow taking place in upward direction especially in sandy soils. The seepage is always acts a upward direction by which some times the net effective pressure becomes 0 at that situation a cohesion less soils loose all the ends shear strength and solid particles have a tendency to move up in the direction of water flow the phenomena of lifting of soil particles in the direction of flow is known as quick sand condition.

$$\text{net effective pressure } \sigma = \sigma' - P_s$$

$$0 = \sigma' - P_s$$

$$\Rightarrow r'_z = r_w i \cdot z = 0$$

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$$r'_z = r_w i z$$

$$i = \frac{r'_z}{r_w}$$

The hydraulic gradient i at quick sand condition is known as critical hydraulic gradient.

$$i_c = \frac{r'_z}{r_w}$$

$$i_c = \frac{(G-1)r_w}{\frac{1+e}{\gamma_w}} = \frac{G-1}{1+e}$$

Now the head corresponding to critical hydraulic gradient is known as critical hydraulic.

$$i_c = \frac{h_c}{z}$$

1 A stream consisting of fine sand is 2mts. thick and ~~on~~ at the what head of water following in an upward direction will a quick sand condition develop take $G = 2.68$, $e = 0.6$

$$i_c = \frac{G-1}{1+e} = \frac{2.68-1}{1+0.6} = 1.05$$

$$1.05 = \frac{h_c}{2} = 1.05 \times 2 = 2.1 = h_c = 2.1$$

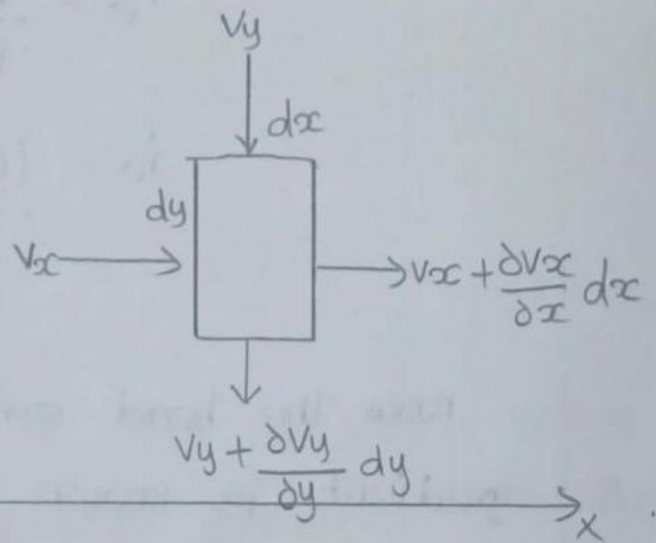
Two dimensional flow equation (Laplace's equation):-

Assumptions:-

- * flow in water is only in two direction The materials Soil and water or in compressible.
- * The soil is fully saturated.
- * material is homogeneous and isotropic
- * The flow is studied and valied dacey's law

let us consider a elementary
strip of size $dx \times dy$

let the v_x and v_y
velocity components with
 x and y directions



* at entrance of the
soil element

* at exit of the soil)

* at exit point at velocity component

$$v_x + \frac{\partial v_x}{\partial x} dx \quad \& \quad v_y + \frac{\partial v_y}{\partial y} dy$$

* The quantity of water entering in x and y direction

$\Rightarrow$ in let = out let

$$\Rightarrow v_x dy + v_y dx = \left(v_x + \frac{\partial v_x}{\partial x} dx \right) dy + \left(v_y + \frac{\partial v_y}{\partial y} dy \right) dx$$

$$v_x dy + v_y dx = v_x dy + \frac{\partial v_x}{\partial x} dx dy + v_y dx + \frac{\partial v_y}{\partial y} dy dx$$

$$= \frac{\partial v_x}{\partial x} dx dy + \frac{\partial v_y}{\partial y} dx dy = 0$$

$$= \left(\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} \right) dy \cdot dx = 0$$

$$= \frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} = 0 \rightarrow (1)$$

This equation is continuity equation for two dimensional flow.

let use ∂h be the total head at any point in soil mass then the horizontal and vertical components of hydraulic gradient is

$$i_x = \frac{\partial h}{\partial x}, \quad i_y = \frac{\partial h}{\partial y}$$

$\therefore$ from Darcy's law $v_x = k_x i_x$

$$v_y = k_y i_y$$

now substituting i_x and i_y values we get

$$v_x = k_x \frac{\partial h}{\partial x}, \quad v_y = k_y \frac{\partial h}{\partial y}$$

now substitute v_x and v_y values in equation (1)

$$\frac{\partial \left[k_x \frac{\partial h}{\partial x} \right]}{\partial x} + \frac{\partial \left[k_y \frac{\partial h}{\partial y} \right]}{\partial y} = 0$$

$$\frac{\partial^2 h}{\partial x^2} K_x + \frac{\partial^2 h}{\partial y^2} K_y = 0 \rightarrow (2)$$

When eqn (2) denoted different values of K_x and K_y this equation is used for non isotropic soils. for isotropic soils

$K_x = K_y$ we get

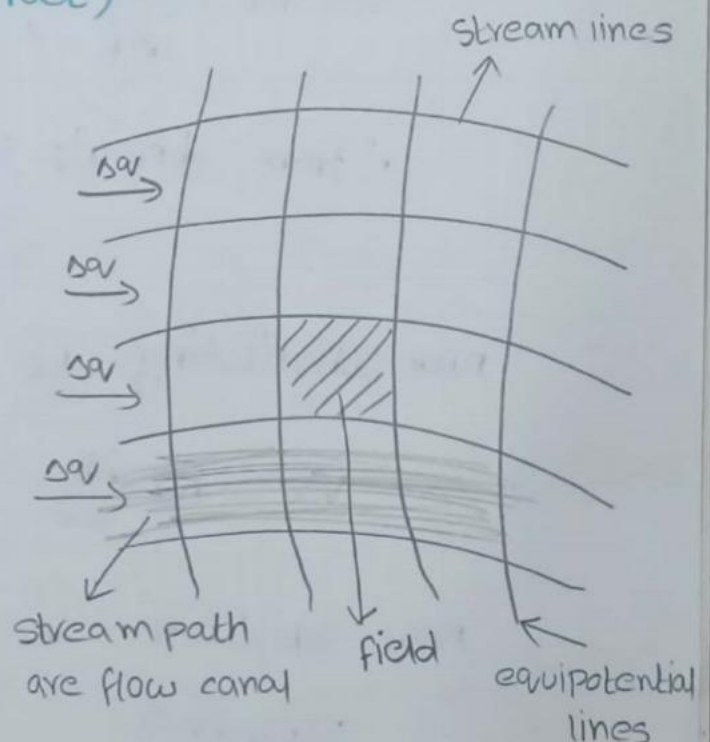
$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

The above equation is known as Laplace's equation.

This solution can be obtained by several methods but most commonly used method is graphical method.

Graphical method :- (flow net)

The solution for this in the form of flow net diagram. It consists of two sets of curves. equipotential lines and stream lines.



characteristics of flow net diagram:-

- * The flow lines and equipotential lines intersect at right angle to each other 90°
- * The head drop will be the same between successive equipotential lines and also the flow in each canal will be same.

Flow canal:-

The portion enclosed between two successive flow lines is known as stream lines.

Field:-

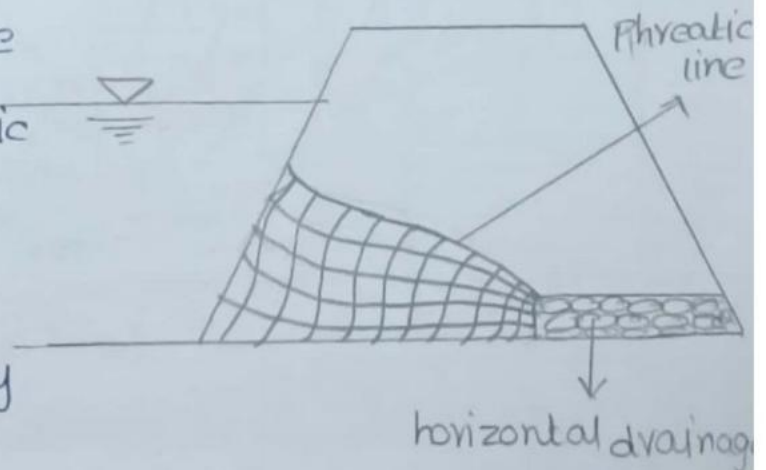
The portion enclosed between two flow lines and two stream lines is called as field.

The field is approximately square shape so that a circle can be drawn touching all four sides.

The size of the square in flow canal should change gradually from up stream side to down stream side.

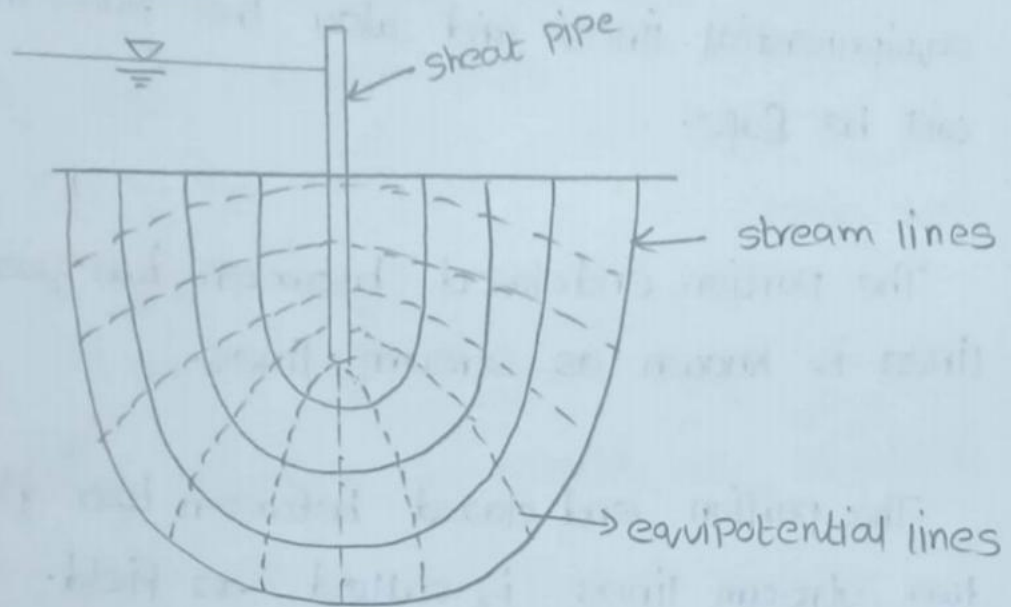
Phreatic line:-

The top flow line of flow field diagram is known as phreatic line its pressure is equal to the atmospheric pressure. Smaller is the dimension of the field, greater will be the velocity and hydraulic gradient.

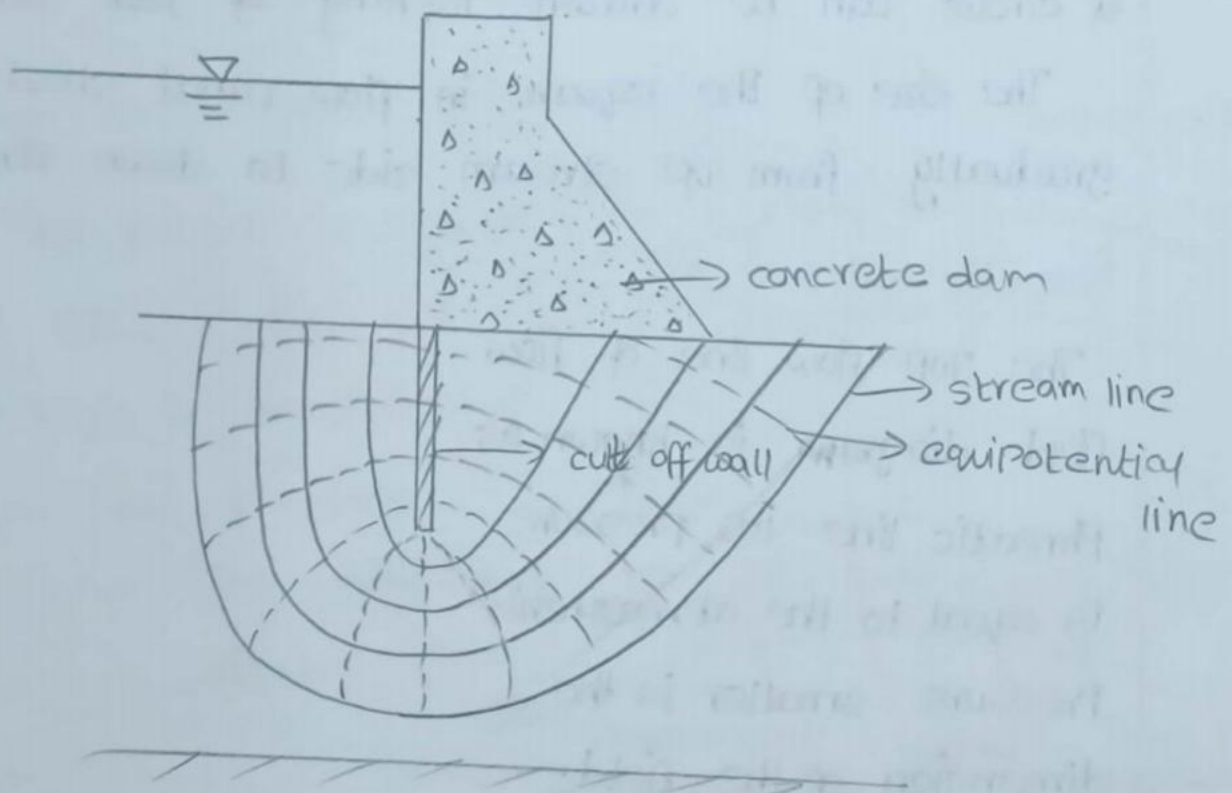


The transition are smooth bigan eleptical or parabolic shape.

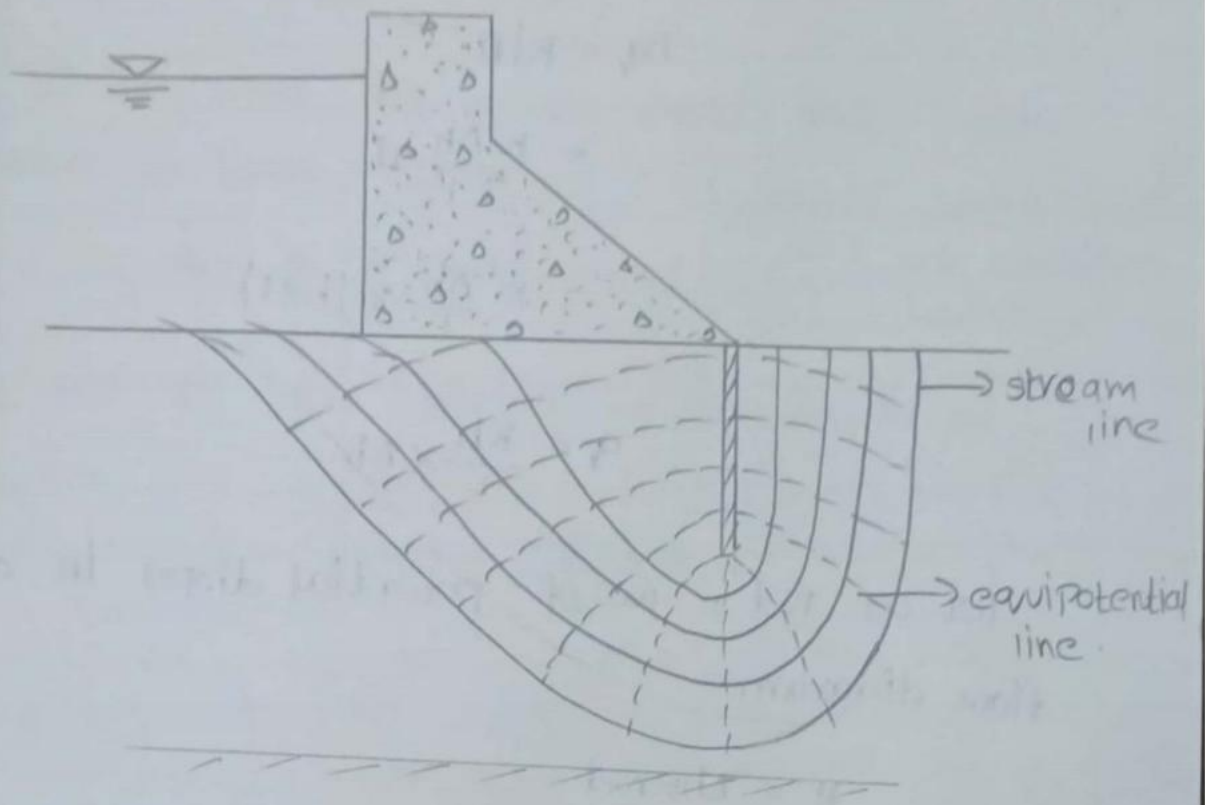
Flow net diagram from different hydraulic structures
(ii) sheat pipe wall:-



(iii) concrete dam with cut off wall at hill:-



(iii) concrete dam with cut of wall at toe :- 26



uses of flow net :-

The flow net diagram is use full to determine the rate of seepage and hydro statical pressure and seepage pressure and exit gradient.

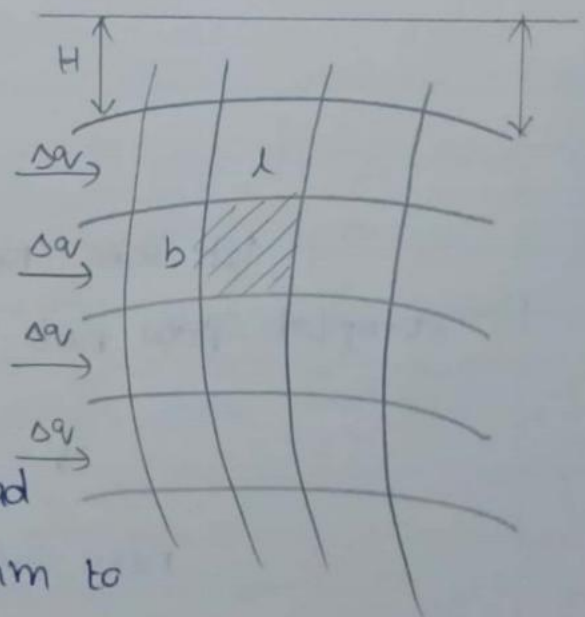
Determination of seepage by using flow net method (or) diagram :-

H = Hight of water

consider a field of size L, b, l , and let Δh is the Potential drope between two equipotential lines and

' h ' is the Total hydraulic head causing flow from up stream to

down stream. let Δq = Rate of flow in each flow canal



$$Q = K i A$$

$$\Delta Q = K i A$$

$$= K \frac{\Delta h}{L} A$$

$$= K \frac{\Delta h}{L} \times (b \times 1)$$

$$Q = \frac{Kb}{L} \times \Delta h$$

let us n_d = no of potential drops in complete flow diagram.

$$H = \Delta h \times n_d$$

$$\Delta h = \frac{H}{n_d}$$

now substitute in above equation

$$Q = \frac{Kb}{L} \times \frac{H}{n_d}$$

$$\Delta Q = K \frac{\frac{H}{n_d}}{L} \times b$$

$$= K \frac{H}{n_d} \times \frac{b}{L}$$

let use N_f = Total no of flow canals in the complete flow net diagram.

$$Q = \Delta Q \times N_f$$

now substitute ΔQ value

$$Q = K \frac{H}{n_d} \times \frac{b}{L} \times N_f$$

$$q_v = K \cdot \frac{NF}{nd} \times H \times \frac{b}{L} \rightarrow (1)$$

Note:- If the field are in rectangle shape above equation is used. usually the field are in square shape

$$q_v = K \frac{NF}{nd} \times H \rightarrow (2) \quad b=L \text{ the equation becomes.}$$

If the soil non isotropic or the equation becomes.

$$q_v = \sqrt{K_x K_y} \frac{NF}{nd} \times H$$

$$q_v = \frac{NF}{nd}$$

$$q_v \propto NF$$

$$q_v \propto \frac{1}{nd}$$

The ratio of $\frac{NF}{nd}$ = shape factor for flow net diagram.

Problems:-

1. A homogenies dam is 21.5 mts high and as a free board 1.5 mts if the flow net was constructed and the following results observed no of potential drops = 12, no flow canals = 3, The dam as a horizontal filter of 15 mts length calculate the discharge per meter length of dam if the coefficient of permeability of the dam is 2.75×10^{-6} m/sec.

given data

height = 21.5 mts

free board = 1.5 mts

no of potential drops $n_d = 12$

no of flow canals $n_f = 3$

Permiability of the dam = 2.75×10^{-6} m/sec

horizontal filter of = 15 mts

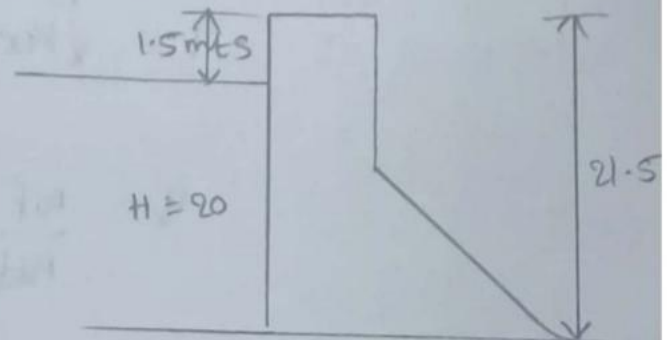
$q = ?$

$$q = K \cdot \frac{n_f}{n_d} \times H$$

$$H = 21.5 - 1.5 = 20$$

$$= 2.75 \times 10^{-6} \times \frac{3}{12} \times 20$$

$$= 1.375 \times 10^{-5}$$



- 2 An earth dam is built on impervious for with a horizontal filters at the base near that to the K of soil is different in horizontal at Vertical direction Respectively 4×10^{-2} mm/sec and 2×10^{-2} mm/sec The full Reservoir level 30 mts above the filters the flow net constructed from the transformed section of dam consists of 6 flow canals and 18 potential drops estimate the seepage loss per meter length of dam.

sol given that

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Horizontal $K_x = 4 \times 10^{-2} \text{ mm/sec}$

vertical $K_y = 2 \times 10^{-2} \text{ mm/sec}$

Reservoir level $H = 30 \text{ mts}$

Potential drops $N_d = 18$

flow canal $N_f = 6$

$$q = \sqrt{K_x K_y \frac{N_f}{N_d}} \times H$$

$$= \sqrt{4 \times 10^{-2} \times 2 \times 10^{-2}} \times \frac{6000}{18000} \times 30$$

$$= 282.84 \text{ mm/sec}$$

- 3 A flow net for seepage under the sheet pile wall shown in fig estimate the approximate quantity per/m² meter length.

$$K = 3.47 \times 10^{-9} \text{ m/day}$$

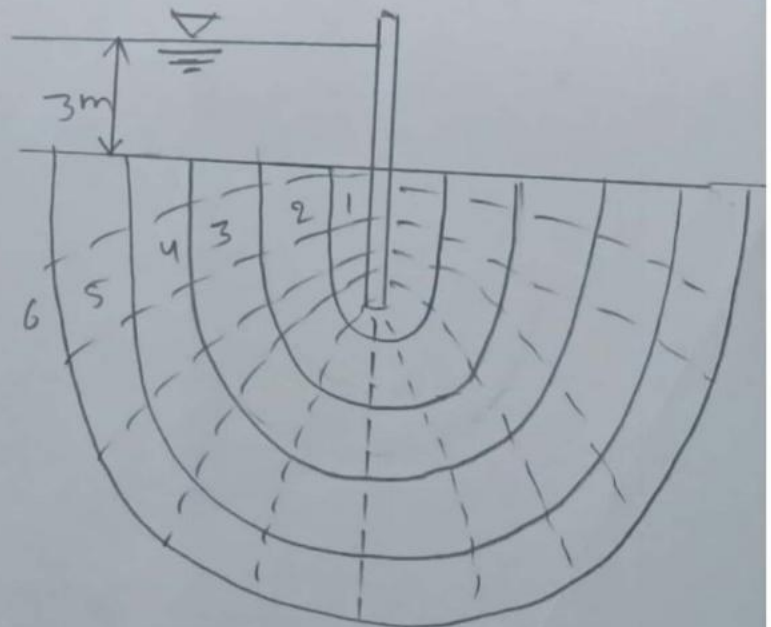
$$q = K \cdot H \frac{N_f}{N_d}$$

$$N_f = 6$$

$$N_d = 10$$

$$q = 3.47 \times 10^{-9} \times \frac{6}{10} \times 3$$

$$= 6.246 \times 10^{-9} \text{ m}^2/\text{day}$$



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Stress distribution in soils

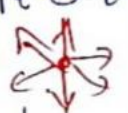
→ stress distribution in soil — stresses are induced in a soil mass due to weight of overlying soil and due to the applied loads.

→ These stress are required for the stability analysis of the soil mass, the settlement analysis of foundations and the determination of the earth pressure.

Boussinesq's Equation:-

Boussinesq's gave the theoretical solutions for the stress distribution in a elastic medium subjected to a concentrated load on surface. The solutions are used to obtain the stresses in a soil mass due to externally applied loads.

Assumptions:-

- ① The soil mass is an elastic continuum; having a constant value of modulus of elasticity (E), i.e; the ratio b/w the stress and strain is constant. *homogeneous* → 
- ② The soil is homogeneous, i.e; it has identical properties at different points.
- ③ The soil is isotropic, i.e; it ~~extends to infinity~~ has identical properties in all directions. → 
- ④ The soil mass is semi-infinite; i.e; it extends to infinity in the downward direction and lateral directions. In other words, it is limited on its top by a horizontal plane and extends to infinity in all other directions. 

(1) The soil is weightless and is free from residual stresses before the application of the load.

Vertical stress due to concentrated load (or) Point load

→ consider to horizontal surface of the elastic continuous soil mass at the ground level subjected to a Point load Q acting at any point on the horizontal surface that point is 'o'. The origin of the co-ordinates y is taken at 'o'.

→ The distribution of stress in the soil distributed in different directions. Assume located 'P' point at a depth of z below the horizontal surface.

→ Boussinesq proved that the Polar stress (σ_r) at Point $P(x, y, z)$ is given by.

$$\sigma_r = \frac{3}{2\pi} \cdot \frac{Q \cos \beta}{R^2} \quad [\text{proved}]$$

Q = Point load (N)

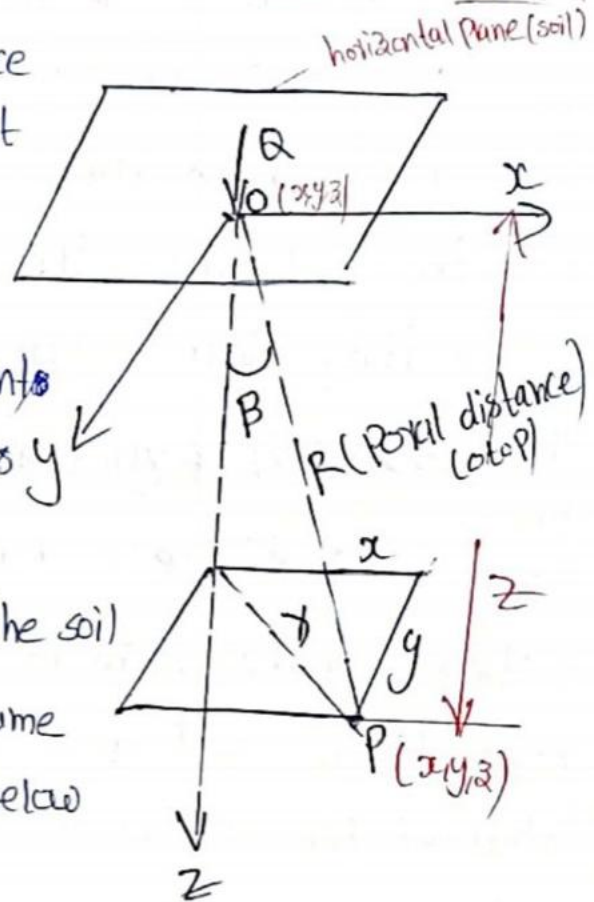
z = depth below the surface (m)

r = horizontal distance from the axis of loading (m)

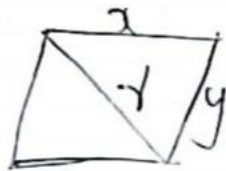
R = Polar distance b/w the origin 'o' and point 'P'.

$$R = \sqrt{x^2 + y^2 + z^2}$$

$$R = \sqrt{r^2 + z^2}$$



Cartesian coordinate,
(x, y, z)
Polar - (r, theta)



$$\text{Hyp}^2 = \text{opp}^2 + \text{Adj}^2$$

$$r^2 = x^2 + y^2$$

$$r = \sqrt{x^2 + y^2}$$



β = angle b/w R and z axis (3)

$$\sin \beta = r/z \quad ; \quad \cos \beta = z/R$$

→ The vertical stress (σ_z) at point P is given by

$$\sigma_z = \sigma_R \times \cos^2 \beta$$

$$= \frac{3}{2\pi} \times \frac{Q \cos \beta}{R^2} \times \cos^2 \beta$$

$$= \frac{3}{2\pi} \times \frac{Q \cos^3 \beta}{R^2} = \frac{3}{2\pi R^2} \times Q \times \left(\frac{z}{R}\right)^3 \times \frac{z^2}{z^2}$$

$$= \frac{3Q}{2\pi} \frac{z^3}{R^5} \times \frac{z^2}{z^2}$$

$$= \frac{3Q}{2\pi} \times \frac{1}{z^2} \times \frac{z^5}{R^5}$$

$$R = \sqrt{r^2 + z^2} \\ = (r^2 + z^2)^{1/2}$$

$$\sigma_z = \frac{3Q}{2\pi} \times \frac{1}{z^2} \times \frac{z^5}{((r^2 + z^2)^{1/2})^5}$$

$$= \frac{3Q}{2\pi} \times \frac{1}{z^2} \left[\frac{z^5}{(r^2 + z^2)^{5/2}} \right] = \frac{3Q}{2\pi} \frac{1}{z^2} \left[\frac{z^5}{\left(\frac{r^2}{z^2} + 1\right)^{5/2}} \right]$$

$$\sigma_z = \frac{3Q}{2\pi} \times \frac{1}{z^2} \times \frac{z^5}{\left(z^2 \left(\frac{r^2}{z^2} + 1\right)\right)^{5/2}}$$

$$\sigma_z = \frac{3Q}{2\pi} \frac{1}{z^2} \left[\frac{z^5}{(z^2)^{5/2} \left(\frac{r^2}{z^2} + 1\right)^{5/2}} \right]$$

$$\sigma_z = \frac{3Q}{2\pi} \times \frac{1}{z^2} \left[\frac{1}{\left(\frac{r^2}{z^2} + 1\right)^{5/2}} \right]$$

* $\sigma_z = \frac{3Q}{2\pi} \frac{1}{z^2} \left[\frac{1}{\left(1 + \left(\frac{r}{z}\right)^2\right)^{5/2}} \right] \rightarrow (1)$

2

$$\sigma_z = I_B \times \frac{Q}{z^2}$$

where $I_B = \frac{3}{2\pi} \left[\frac{1}{(1 + (\gamma/z)^2)^{5/2}} \right]$

$\therefore I_B$ = is the Boussinesq influence co-efficient for Vertical stress. (or) Boussinesq's influence factor.

Radial shear stress (τ_{rz}) = $\sigma_z (\gamma/z)$

→ Vertically below the load $\tau_{rz} = 0$ ($\gamma = 0$)

eg ① → $\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi}$

$\sigma_z \propto \frac{1}{z^2}$

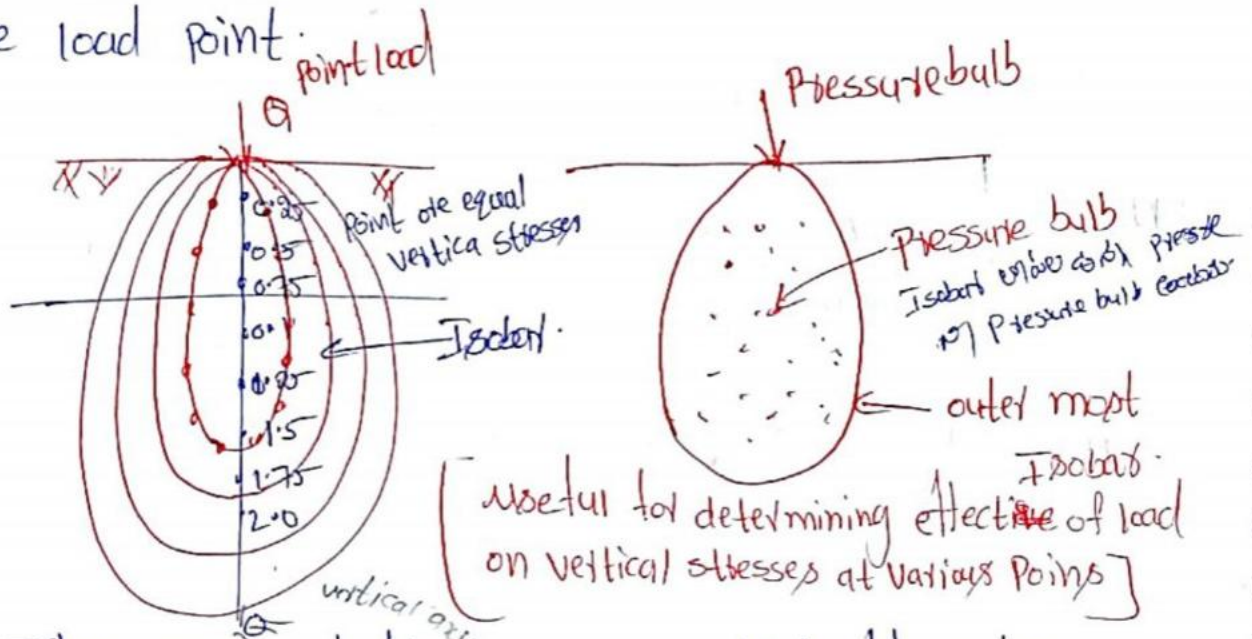
↑ ↓

For Every change in the depth of soil, the vertical stress also changes inversely.

Isobar Diagram:⁽⁵⁾ An Isobar is a curve joining the points of equal stress intensity (or) equal pressure.

→ An Isobar is a spatial curved surface of the shape of an electrical bulb. The curved surface is symmetrical about the vertical axis passing through the load point.

Ex: contour lines on same elevation plots.



* The zone of the soil in which there is significance stress is called Pressure bulb.

* out side the pressure bulb the stress is negligible but not zero.

$$\Rightarrow \sigma_z = I_B \cdot Q / z^2$$

$$\Rightarrow \sigma_z = 0.1 Q \quad \left(\begin{array}{l} 0.1 \text{ depth of } Q \text{ on stress} \\ \text{depend on variation of depth } (z) \\ \text{vertical stress} \end{array} \right)$$

$$\Rightarrow 0.1 \times Q = I_B \cdot Q / z^2$$

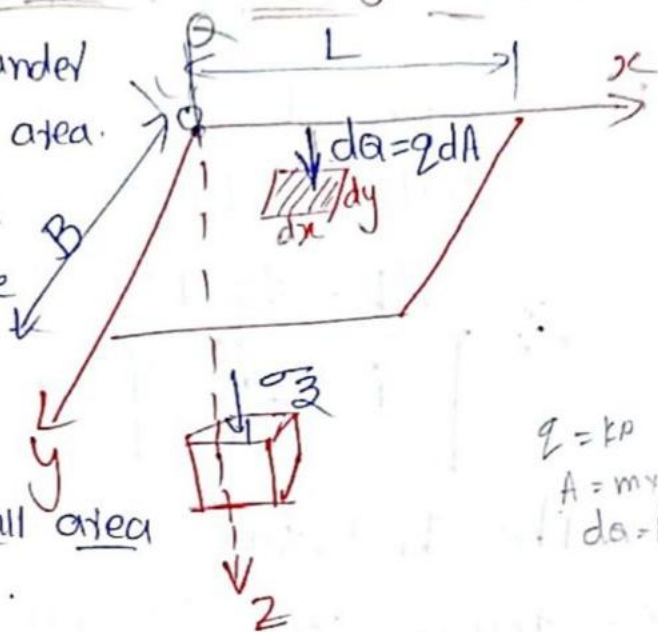
$$\Rightarrow \boxed{I_B = 0.1 \times z^2}$$

6

Vertical stress under a corner of rectangular area:-

The vertical stress under a corner of a rectangular area with a uniformly distributed load of intensity q can be obtained from Boussinesq's equation.

→ The intensity on small area $da = q dA = q dx dy$.



$$q = kP$$

$$A = m \times m = m^2$$

$$da = k \times m^2$$

$$\sigma_z = \frac{3qz^3}{2\pi(r^2+z^2)^{5/2}}$$

$$dA = da \times dy$$

where q = point load $r^2 = x^2 + y^2$

$$da = q dA$$

$$da = q \cdot dx dy$$

→ So stress at depth z is obtained by integrating over the area

$$\sigma_z = \iint \frac{3qz^3}{2\pi(x^2+y^2+z^2)^{5/2}} dx dy$$

$$\sigma_z = \int_0^B \int_0^L \frac{3qz^3}{2\pi(x^2+y^2+z^2)^{5/2}} dx dy$$

→ This double integral is mathematically heavy.

→ To simplify Newmark derived a closed-form solution for the stress at depth z under a rectangular area

* He defined two ratios: $m = B/2$, $n = L/2$

② → So the vertical stress becomes

$$\sigma_z = I_N q$$

where I_N is the influence coefficient, depending only on m and n .

→ Expression for influence coefficient I_N

∴ There are two equivalent forms:

$$I_N = \frac{q}{2\pi} \left[\frac{mn}{\sqrt{m^2+n^2+1}} \cdot \frac{m^2+n^2+2}{m^2+n^2+m^2n^2+1} + \sin^{-1} \left[\frac{mn}{\sqrt{m^2+n^2+1}} \right] \right]$$

$$I_N = \frac{L}{2\pi} \left[\frac{mn}{\sqrt{m^2+n^2+1}} \cdot \frac{m^2+n^2+2}{m^2+n^2+m^2n^2+1} + \sin^{-1} \left[\frac{mn}{\sqrt{m^2+n^2+1}} \right] \right]$$

$$I_N = \frac{1}{4\pi} \left[\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1} \frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1-m^2n^2} \right] \quad (or)$$

→ I_N depends only on the ratios $B/2$ and $L/2$ not on actual dimensions or load.

→ The formula is symmetrical: swapping m and n gives the same result.

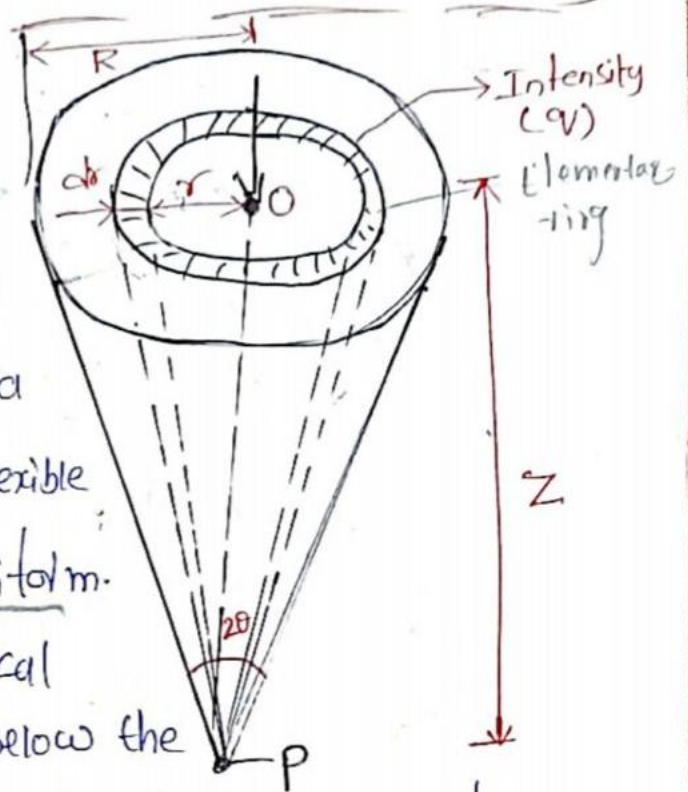
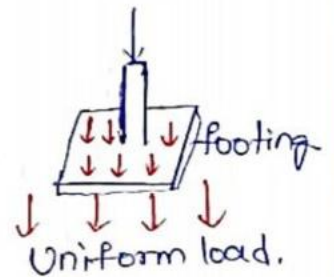
→ At very large m and n [i.e., very large loaded area], $I_N \rightarrow 0.25$ (meaning maximum 25% of load transmits to infinite depth).

⑧ ⇒ Vertical stress under a circular Area (uniform load)

→ The loads applied to soil surface by footings are not concentrated loads. These are usually spread over a finite area of the footing. The footing is flexible and the contact pressure is uniform.

→ Let us determine the vertical stress at point 'P' at depth 'z' below the center of a uniformly loaded circular area.

→ Let the intensity of load q per unit area, and R are be the radius of the loaded area.



→ Boussinesq's solution can be used to determine the σ_z the load on the elementary or small ring of radius ' r ' and depth dr .

∴ The load on Elementary ring $dP = q, dA = 2\pi r dr$

∴ The vertical stress for elementary ring

Area of the ring
 $dA = 2\pi r dr$

$$\text{eq-①} \rightarrow \sigma_z = \frac{3q}{2\pi} \frac{1}{z^2} \left[\frac{1}{(1+(r/z)^2)^{5/2}} \right]$$

$$d\sigma_z = \frac{3q(2\pi r dr)}{2\pi} \times \frac{1}{z^2} \left[\frac{1}{(1+(r/z)^2)^{5/2}} \right]$$

$$d\sigma_z = 3q r dr \times \frac{1}{z^2} \times \left[\frac{1}{(1+(r/z)^2)^{5/2}} \right]$$

① ∴ Total vertical stress on circular area.

$$\delta \sigma_z = \int_0^R \frac{3q r dr}{z^2} \cdot \frac{1}{(1+(r/z)^2)^{5/2}}$$

$$\sigma_z = \frac{3q}{z^2} \int_0^R \frac{r dr}{\left(\frac{z^2+r^2}{z^2}\right)^{5/2}} = \frac{3q}{z^2} \int_0^R \frac{r dr}{\frac{(z^2+r^2)^{5/2}}{z^5}}$$

$$= \frac{3q}{z^2} \int_0^R \frac{r dr}{(z^2+r^2)^{5/2}} \cdot z^5$$

$$\sigma_z = 3q z^3 \int_0^R \frac{r dr}{(r^2+z^2)^{5/2}} \quad \text{--- (1)}$$

Upper limit
 $r^2+z^2 = u$ any constant
 $r^2+z^2 = u$
 Lower limit
 $0+z^2 = u$

Let $r^2+z^2 = u$

$2r dr = du$

$\frac{du}{2} = r dr$

eq-① $\sigma_z = 3q z^3 \int_{z^2}^{R^2+z^2} \frac{du}{2 \cdot u^{5/2}}$

$$= \frac{3q z^3}{2} \left[\frac{u^{-5/2+1}}{-5/2+1} \right]_{z^2}^{R^2+z^2}$$

$$= \frac{3q z^3}{2} \left[\frac{u^{-3/2}}{-3/2} \right]_{z^2}^{R^2+z^2}$$

$$\sigma_z = -\frac{3q z^3}{2} \cdot \frac{2}{3} \left[(R^2+z^2)^{-3/2} - (z^2)^{-3/2} \right]$$

$$\sigma_z = -q z^3 \left[\frac{1}{(R^2+z^2)^{3/2}} - \frac{1}{(z^2)^{3/2}} \right]$$

$$\int \frac{1}{x^n} = \frac{x^{1-n}}{1-n}$$

$$1^{-5/2}$$

$$\frac{2^{-5}}{2} = -3/2$$

(10)

$$\sigma_z = + q z^3 \left[\frac{1}{z^3} - \frac{1}{(R^2 + z^2)^{3/2}} \right]$$

$$\sigma_z = q \frac{z^3}{z^3} \left[1 - \frac{1}{\left(1 + \left(\frac{R^2}{z^2} \right) \right)^{3/2}} \right]$$

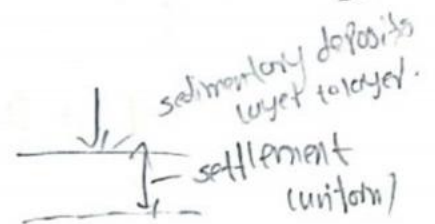
$$\sigma_z = q \left[1 - \frac{1}{\left(1 + \left(\frac{R^2}{z^2} \right) \right)^{3/2}} \right]$$

$$\sigma_z = I_c \cdot q$$

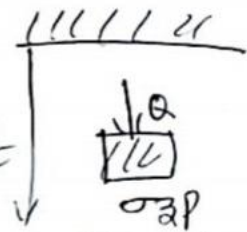
$$I_c = 1 - \left[\frac{1}{\left(1 + \left(\frac{R^2}{z^2} \right) \right)^{3/2}} \right]$$

use:
Below footing point 'P'
to find the vertical
stress = ?

⇒ westergaard's solution:



→ westergaard's assumes there are thin sheets of rigid materials in a homogeneous soil mass. These thin sheets are closely spaced and are of infinite rigidity and incompressible these permit only downward displacement of the soil mass as a whole without any lateral displacement.



→ westergaard's solution / equation represents are more closely the actual sedimentary deposits. According to westergaard's the vertical stress at a point P at depth z below the concentrated load Q is given by.

→ point load elastic, semi-infinite

$$\sigma_z = \frac{\frac{C}{2\pi}}{(C^2 + (r/z)^2)^{3/2}} \times \frac{Q}{z^2} \quad \text{--- (1)}$$

where; C = constant depends on the Poisson's ratio (μ)

$$C = \sqrt{\frac{1-2\mu}{2-2\mu}}$$

$$\left(\begin{array}{l} \frac{1}{2} \sqrt{2} = 2^{1/2} \\ 2^{1/2} \cdot 2^{1/2} \\ 2^{3/2} \end{array} \right)$$

→ For elastic materials, the value of μ where $\mu = 0$ to 0.5

→ we take $\mu = 0 \Rightarrow C = \sqrt{\frac{1-0}{2-0}} \Rightarrow C = \frac{1}{\sqrt{2}} \quad \text{--- (2)}$

Substituting eqn (1) & (2)

$$\sigma_z = \frac{1}{\frac{1}{\sqrt{2}} 2\pi} \times \frac{Q}{z^2} \times \frac{1}{\left[\left(\frac{1}{\sqrt{2}} \right)^2 + \left(\frac{r}{z} \right)^2 \right]^{3/2}}$$

[note: It is best suited to stratified soil & (or) sedimentary soils]

$$\sigma_z = \frac{1}{2^{3/2} \pi \left[\frac{1}{2} + \left(\frac{r}{z} \right)^2 \right]^{3/2}} \times \frac{Q}{z^2}$$

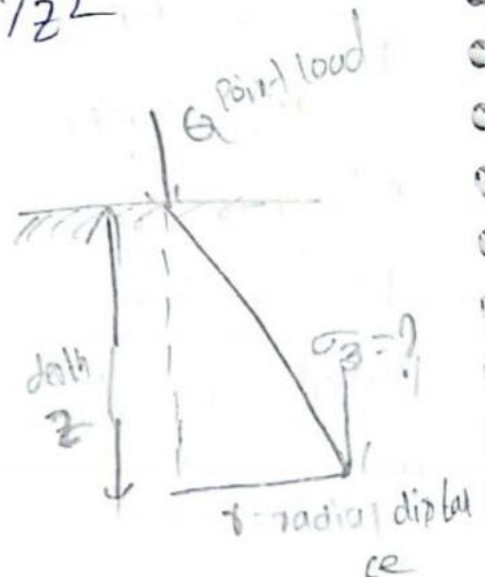
$$\sigma_z = \frac{1}{\pi \left[\frac{2}{2} + 2 \left(\frac{r}{z} \right)^2 \right]^{3/2}} \times \frac{Q}{z^2}$$

$$\sigma_z = \frac{1}{\pi \left[1 + 2 \left(\frac{r}{z} \right)^2 \right]^{3/2}} \times \frac{Q}{z^2}$$

$$\sigma_z = I_w \times \frac{Q}{z^2}$$

where $I_w = \frac{1}{\pi \left(1 + 2 \left(\frac{r}{z} \right)^2 \right)^{3/2}}$

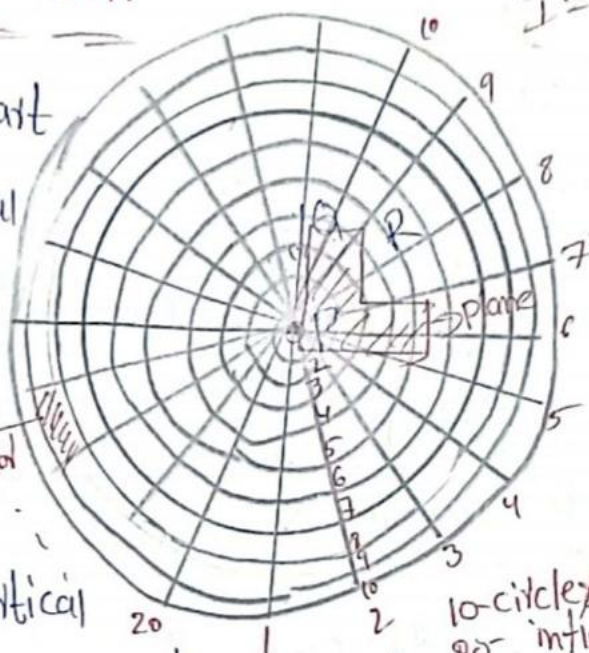
I_w = westergaard's influence co-efficient



Newmark's Influence chart

(12)
 $I = 0.005$
 for
 0.01

(1) Newmark's influence chart can be used for the vertical stresses at any point center a uniformly loaded areas of other shapes.



(2) It is a simple graphical procedure for computing the vertical stresses at any point, loaded by uniformly distributed vertical load at surface. → To find vertical stress at any point, under any shape of loading.
 → Developed based on Boussinesq eqn

(3) This chart can be applicable to a semiinfinite, homogeneous, isotropic, and elastic soil mass.

(4) Newmark's chart is based on the concept of the vertical stress below the center of the circular area of radius R , are divided into 20 equal sectors.

(5) The vertical stress at a point 'P' at depth 'z' just below center of the loaded area due to load on one sector will be $1/20$ of due to load on full circle.

→ The vertical stress under the center of a uniformly loaded circular area has been → each circle gaps are different distance
 → each sector causes equal stress at centre of chart.

construction steps

$$\sigma_z = q \left[1 - \frac{1}{(1 + (R/z)^2)^{3/2}} \right] \quad (2)$$

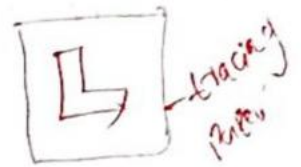
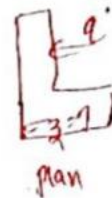
$$\rightarrow \sigma_z = \frac{1}{20} q \left[1 - \frac{1}{(1 + (R/z)^2)^{3/2}} \right]$$

If the vertical stress is fixed value $\sigma_z = 0.005 q$

$$\sigma_z = 0.005 q$$

$$I = \frac{1}{10} \times \frac{1}{20} = 0.005$$

$$0.005q = \frac{1}{20} q \left[1 - \left(\frac{1}{1 + (R_1/z)^2} \right)^{3/2} \right]$$



where $\frac{R_1}{z} = 0.27 \Rightarrow R_1 = 0.27z$

→ every $1/20$ of a circle with a radius $R_1 = 0.27z$ would give a vertical stress of $0.005q$ at its center.

→ now, consider another circle of radius R_2 can divided into 20 equal sectors. each large sector is divided into sub areas.

The vertical stress at a circle ②

$$2 \times 0.005q = \frac{q}{20} \left[1 - \left(\frac{1}{1 + (R_2/z)^2} \right)^{3/2} \right]$$

$$\frac{R_2}{z} = 0.40 \Rightarrow R_2 = 0.40z$$

→ The radius of the 3 to 9 circles can be determine the values $0.52z, 0.64z, 0.77z, 0.92z, 1.12z, 1.39z$ and $1.91z$

→ The radius of $q/2$ is $2.54z$

→ The radius for 10 circle are;

$$10 \times 0.005q = \frac{q}{20} \left[1 - \left(\frac{1}{1 + (R_{10}/z)^2} \right)^{3/2} \right]$$

$$\frac{R_{10}}{z} = 2 \Rightarrow R_{10} = 2z$$

→ The 10th circle can be drawn.

Practical use:- * Place the chart (transparent sheet) over the loaded area (to scale)

* count the number of units n covered by the load shape.

* stress at depth z is → $\sigma_z = I_p \times n \times q$

I_p = influence value of the chart. (or) Influence coefficient of the chart

$n = \text{no. of units covered by the plan.}$

$q = \text{intensity of load} \cdot (\text{N/m}^2 \text{ or } \text{kg/m}^2)$

Disadvantages:

- ① For many loaded arcs Present (drawn with depth) many influence charts is required.
- ② For each different depth counting of units is must be done. (depth change in plot is not circle drawn)

Advantages :-

- used for loaded area of any shape and any point is calculated.

⇒ Important points:-

- (1) The fractions of the unit areas should also be counted.
- (2) If the plan of the loaded area extends beyonds may be assumed to the 10th circle.
- (3) The point 'P' at which the vertical stress is required may be anywhere within (or) outside the loaded area.
- (4) If the depth at which the stress is required is changed, a fresh plan is required such that the new depth is equal to the distance on the chart.

For $\forall e$
each by $\rightarrow$ discharge
layers.
 $q = q_1$
 $h = h_1$
discharge
 $q = q_1$
 $u \notin A$ same
 $q = q_1$
total 11707
Kv

cube!!!

Difference b/w Boussinesq's and Westergaard's

① If $(\frac{r}{z}) < 1.5$, Boussinesq's equation gives relatively higher stresses compared to Westergaard's equation.

② If $(\frac{r}{z}) = 1.5$, Both equations give same stress value.

③ If $(\frac{r}{z}) > 1.5$, Westergaard's equation gives slightly lower value compared to Boussinesq's eqn.

⇒ Fenske's chart: is used to find the vertical stress based on Westergaard's equation.

③ $w > B$

(b) Compaction

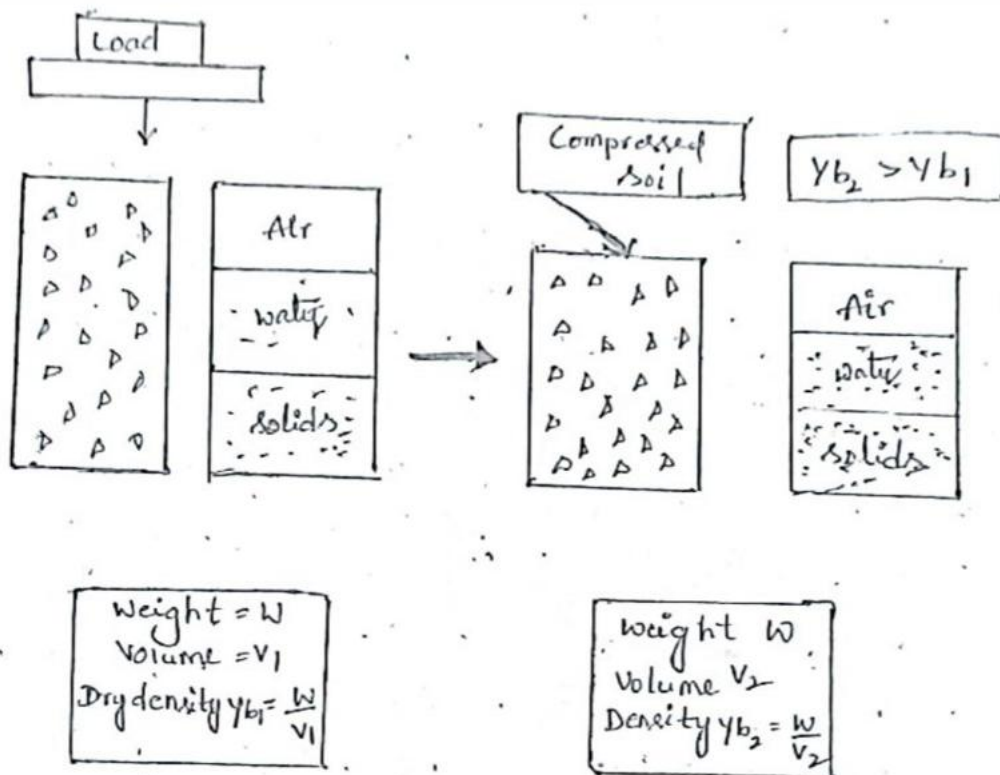
Definition of compaction:

compaction is the process of increasing the bulk density of a soil or aggregate by driving out air. For any soil, at a given compactive effort, the density obtained depends on the moisture content. An "optimum moisture content" exists at which it will achieve a maximum density. compaction is the method of mechanically increasing the density of soil. The densification of soil is achieved by reducing air void space. During compaction, air content reduces, but not water content. It is not possible to compact saturated soil. It should be noted that higher the density of soil mass, stronger, stiffer, more durable will be the soil mass.

Hence, compaction!

1. Increases density
2. Increases strength characteristics
3. Increases load-bearing capacity
4. Decreases undesirable settlement
5. Increases stability of slopes and embankments
6. Decreases permeability
7. Reduces water seepage
8. Reduces swelling & shrinkage
9. Reduces frost damage
10. Reduces corrosion damage

11. Develops high negative pore pressures (suctions) increasing effective stress



Applications of compactions:

The following are the situations in which compaction will improve the existing field condition.

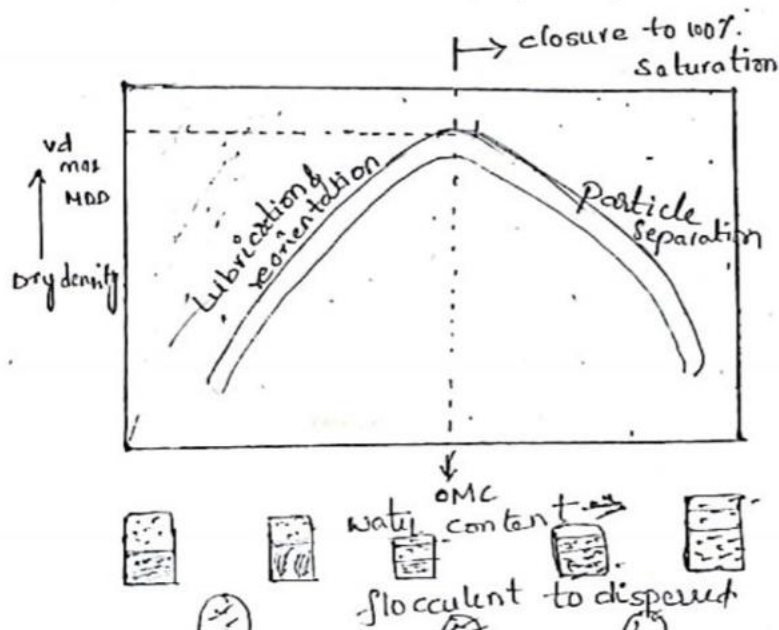
1. compaction of foundation soil for house construction.
2. compaction of soil/gravel/crushed rock/asphalt in road & pavement construction.
3. compaction of soil behind retaining walls.
4. compaction of soil in earth embankments.
5. compaction of soil backfill in trenches.
6. Dam construction
7. construction of clay liners for waste storage areas
8. Ground improvement

distinction between compaction & consolidation: 2

The following are the important distinctions between compaction and consolidation.

compaction	consolidation
1. Man made	1. Natural
2. Expulsion of air	2. Expulsion of pore water
3. Fast process	3. Gradual process
4. possible in dry or partially saturated soil	4. possible in saturated
5. Results in increased density	5. Results in dense packing

Mechanism of compaction:



optimum moisture content (OMC) is the moisture content at which the maximum possible dry density is achieved for a particular compaction energy or compaction method. The corresponding dry density is called maximum dry density (MDD). water is added to lubricate the contact surfaces of soil particles and improve the compressibility of the soil mixture. It should

be noted that increase in water content increases the dry density in most soil up to one stage (dry side). Water acts as lubrication. Beyond this level, any further increase in water (wet side) will only add more void space, thereby reducing the dry density. Hence OMC indicates the boundary between the dry side and wet side. Hence the compaction curve as shown in figure indicates the initial upward trend up to OMC and the downward trend.

Factors affecting compaction

1. water content
2. Amount of compaction
3. method of compaction
4. Type of soil
5. Addition of Admixtures

Effect of water content

1. With increase in water content, compacted density increases up to a stage, beyond which compacted density decreases.
2. The maximum density achieved is called MDD and the corresponding water content is called OMC.
3. At lower water contents than OMC, soil particles are held by electrical forces that prevent the development of diffused double layer leading to low inter-particle repulsion.
4. Increase in water results in expansion of double layer and reduction in net attractive force between particles. Water replaces air in void space.
5. particles slide over each other easily increasing lubrication, helping in dense packing.
6. After OMC is reached, air voids remain

constant — further increase in water, increase⁶ the void space, thereby decreasing dry density.

Effect of amount of compaction:-

1. As discussed earlier, effect of increasing compactive effort is to increase MDD and reduce OMC (Evident from standard proctor's Test).
2. However, There is no linear relationship between compactive effort and MDD

Effect of method of compaction

The dry density achieved by the soil depends on the following characteristics of compacting method.

1. Weight of compacting equipment
2. Type of compaction
3. Area of contact of compacting equipment with soil
4. Time of exposure
5. Each of these approaches will yield different compactive effort. Further, suitability of a particular method depends on type of soil.

Effect of Type of soil:

1. Maximum density achieved depends on type of soil.
2. coarse grained soil achieves higher density at lower water content and fine grained soil achieves lesser density, but at higher water content.

Effect of Addition of Admixtures:

1. stabilizing agents are ^{or} the admixtures added to soil.
2. The effect of adding These admixtures is to stabilize the soil.
3. In many cases they accelerate the process of densification.

Field compaction control:

It is extremely important to understand the factors affecting compaction in the field and to estimate the correlation between laboratory and field compaction. Field compaction control depends on i) placement water content ii) Type of equipment for compaction and iii) Lift thickness and iv) Number of passes based on soil type & degree of compaction desired.

placement water content is the water content at which the ground is compacted in the field. It is desirable to compact at or close to optimum moisture content achieved in laboratory so as to increase the efficiency of compaction. However, in certain jobs the compaction is done at lower than or higher than omc (by about 1-2%) depending on the desired function as detailed in the table.

compaction control in field:

There are many variables which control the vibratory compaction or densification of soils.

characteristics of the compactor:

1. Mass size
2. operating frequency and frequency range

characteristics of the soil:

1. Initial density
2. Grain size and shape
3. Water content

construction procedures:

1. Number of passes of the roller
2. Lift thickness
3. Frequency of operation vibrator
4. Towing speed

Degree of compaction

Relative compaction or degree of compaction

$$R.C = \frac{\gamma_d - \text{field}}{\gamma_d \text{ max-laboratory}} \times 100\%$$

correlation between relative compaction & relative density

$$R.C = 80 + 0.2 D_r$$

It is a statistical result based on 47 soil samples.

Typical required $R.C. > 95\%$

... .. equipment:



Compressibility:- The property of the soil by which the decrease in soil mass under compressive force is known as Compressibility.

Reasons for Compressibility of soil:-

- i. Compression of solid particles and compression of water in the pores of soil mass.
2. Expulsion of air from the voids of soil mass and compression of air.
3. Expulsion of water from the voids of soil mass
 - i> Not possible
 - ii> known as Compaction
 - iii> known as Consolidation.

* Consolidation is the term used for cohesive soil i.e. clay soils.

Consolidation:- The expulsion of pore water from the voids of the soil mass under normal long term static loads without replacement of water by air is known as Consolidation. It is simply known as Primary Consolidation or primary Compression or primary time effect.

* Difference b/w Compaction & Consolidation.

S.No Compaction

Consolidation.

- | | |
|--|--|
| 1. Expulsion of pore air | 1. expulsion of pore water |
| 2. Soil involved is partially saturated | 2. Soil involved is fully saturated. |
| 3. Applies to cohesive as well as cohesionless soils | 3. Applies to cohesive soils only.
(clayey soils) |
| 4. Artificial process | 4. Natural process |
| 5. Dynamic loads is commonly applied | 5. static loads is commonly applied |
| 6. Relatively quick process | 6. Relatively slow process |
| 7. useful primarily in embankment and earthdams | 7. useful as a means of improving the properties of foundation soil. |

* The Consolidation of soil deposit can be divided into 3 stages:

→ Initial Consolidation:- When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion and compression of air in the voids.

→ A small decrease in volume also occurs due to compression of

→ The reduction in volume of the soil just after the application of load is known as initial consolidation or initial compression. The initial consolidation is mainly due to compression of solid particles.

* primary Consolidation: - After initial consolidation, further reduction in volume occurs due to expulsion of water from voids.

→ When a saturated soil is subjected to pressure, initially all the applied pressure is taken by water as an excess pore water pressure, as water is almost incompressible compared with solid particles.

→ A hydraulic gradient develops and the water starts flowing out and decrease in volume occurs.

→ The decrease depends upon the permeability of the soil and is therefore time dependent this reduction in volume is called primary Consolidation.

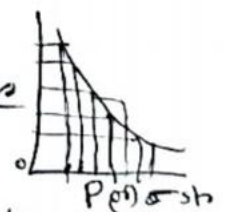
→ In fine grained soils primary Consolidation occurs over a long time. In coarse grained soils primary Consolidation rather quickly due to high permeability.

* Secondary Consolidation: The reduction in volume continues at a very slow rate even after the excess hydrostatic pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in volume is called Secondary Consolidation. The causes for Secondary Consolidation are not fully established.

→ The primary Consolidation is most important component of the total Consolidation and forms the main subject matter.

③ → Compression Index (C_c): - it is defined as the slope of Compression Curve in the e vs $\log_{10} \sigma'$ of virgin Compression Curve.

$$C_c = \frac{-\Delta e}{\log_{10} \left(\frac{\sigma'_1}{\sigma'_0} \right)} \Rightarrow \sigma'_1 = \sigma'_0 + \Delta \sigma'$$



It is also defined as the ratio of change in void ratio by $\log_{10}$ effective stress to the initial effective stress.

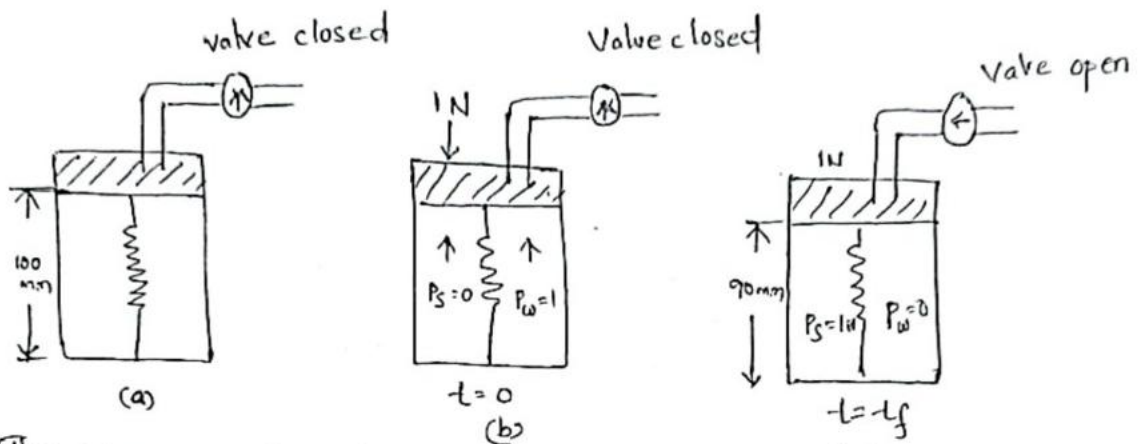
$$C_c = \frac{-\Delta e}{\log_{10} \left(\frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \right)}$$

(or) Expansion Index / Recompression Index.

σ'_0 = initial eff. stress
 σ'_1 = final effective stress
 $\Delta \sigma'$ = change in eff. stress.

* Spring Analogy for primary Consolidation:-

(2)



1. The process of primary Consolidation Can be explained with the help of the Spring analogy given by Terzaghi.
2. fig (a) shows a Cylinder fitted with a tight-fitting piston having a valve. the Cylinder is filled with water, and contains a Spring of Specified stiffness. let the initial length of the Spring Can be 100mm and stiffness of Spring be 10mm/N. let us assume that the piston weightless and Spring and water are initially free of Stress.
3. when a load (P) Say (1N) is applied to the piston, with its valve closed the entire load is taken by water fig (b). the stiffness of the Spring is negligible Compared with that of water, and Consequently no load is taken by Spring. from equilibrium.

$$P_w + P_s = P$$

when P_w = Load taken by water, P_s = Load taken by Spring

$$P = \text{total load for } P = 1 \text{ N.}$$

$$P_w + P_s = 1.0$$

Initially ($t=0$) when valve is closed $P_s = 0.00$ Therefore $P_w = 1.0$

4. The valve is now gradually opened, water starts escaping from the Cylinder the spring starts sharing some load and decrease in its length occurs. when a portion (ΔP) of the load is transferred from the water to the Spring becomes $\Delta P + (1.0 - \Delta P) = 1.0$

5. AS more and more water escapes, the Load carried by the Spring increases. fig shows the transfer of the load from the water to the Spring. eventually, when the steady conditions are established. the water stops escaping finally at the time $t = t_f$. the entire Load is taken by Spring thus $P_w = 0$ $P_s = 1.00$

6. The load causes a decrease in length of the Spring by 10% the final length is 90mm fig (c). As the load carried by water zero, it is again free of excess pressure. Now if the valve is closed and the load is P is increased to 2N, the process of transferring load repeats and finally the Spring takes and the complete load its length becomes 80mm. Like wise, the process is repeated till the final increment of the load has been applied.

* Empirical equations for C_c :-

Skempton equation:

for remoulded clayey soils $C_c = 0.007 [W_L - 10]$
(Normally Consolidated)

for undisturbed clay soils $C_c = 0.009 [W_L - 10]$

Note:- C_c for undisturbed clay soil = 1.3 times of C_c of remoulded (or) disturbed clay soil.

** Classification of Soil deposits with respect to Consolidation history :-

1. preconsolidated or precompressed or overconsolidated soil:-
If the soil had been subjected to a pressure greater than the present existing pressure. that the soil is said to be preconsolidated soil.

2. Normally Consolidated soil:- if the soil has not been subjected to a pressure greater than the present existing pressure. but due to the present pressure itself the consolidation process is over.

3. under Consolidated soil:- if the Consolidation process is still under progress then that soil is said to be under Consolidated soil.

4. over Consolidation ratio:- (OCR) :- it is the ratio of preconsolidation pressure to the present existing pressure

$$OCR = \frac{\text{preconsolidation pressure}}{\text{Present existing pressure}}$$

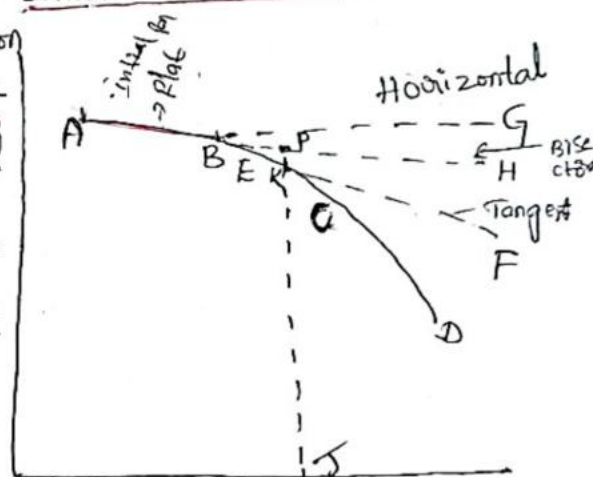
$OCR > 1$ known as over Consolidated soil.

* preconsolidation pressure :-

- The maximum pressure to which an overconsolidated soil had been subjected in the past is known as the preconsolidation pressure or overconsolidation pressure ($\bar{\sigma}_c$). When a soil specimen is taken from a natural deposit, the weight of the overlying material is removed.
- This causes an expansion of the soil due to a reduction in pressure.
- Thus the specimen is generally preconsolidated or overconsolidated. When the specimen is loaded in the Consolidation test,

the initial portion AB of the Compression Curve ABCD (fig) is actually a recompression curve. Consequently, the initial portion AB is relatively flat.

- it is followed by a straight line CD with a steep slope which indicates the compression of a virgin soil (normally consolidated soil)



In the transition range BC, the slope gradually changes. the determination of $\bar{\sigma}_c$ preconsolidation pressure ($\bar{\sigma}_c$) falls in the range. it can be obtained using the method given by Casagrande.

* The procedure consist of following steps.

1. determine the point E on the Curve where the Curvature is maximum. i.e. the radius of curvature is maximum
2. draw the tangent EF to the Curve at E
3. draw a horizontal line EG at E.
4. Bisect the angle b/w tangent EF and the horizontal EG, and draw the bisector EH.
5. produce back the straight line portion CD of the Curve and determine the point of intersection P of the bisector EH and the backward extension of CD.
6. Draw the vertical PJ through P which cuts the log $\bar{\sigma}$ -axis at the point J indicates the preconsolidation pressure $\bar{\sigma}_c$
7. The vertical PJ cuts the Curve at point K. The portion ABK of the Curve represents the recompression Curve and the portion KCD as the virgin Compression Curve.

* Causes of preconsolidation in Soils:

preconsolidation in a soil deposit may be due to one or more of the causes mentioned below.

1. preconsolidation may be due to overburden which had been later removed by erosion.
2. it may be due to loads of buildings and other structures which had been demolished.
3. it may be due to melting of glaciers which covered the soil deposit in the past.
4. preconsolidation may be due to dessication of the clay deposit. During drying due to rise in temperature, tension develops in pore water and the effective stress increases. After dessication, the soil remained preconsolidated.
5. preconsolidation may be due to Capillary pressure which acted on the soil in the past but was later destroyed due to rise in water table.

* final settlement of a Soil deposit in the field:-

Computation of Settlement of a soil deposit in the field consist of two parts:

1. Computation of magnitude of final Settlement:-

for the computation of final Settlement, the Coefficient of volume change C_v on the Compression C_c index is required, which is based on the plot between e and $\log \sigma_v$.

2. Determination of the time-rate of Settlement:-

for the time-rate of computation, the terzaghi theory is used. it requires the Coefficient of Consolidation (C_v) and is discussed.

for the computation of final settlement, the Coefficient of volume change C_v on the Compression C_c index is required, which is based on the plot between e and $\log \sigma_v$.

* Coefficient of compressibility (a_v) : (log -1)

(4)

it is the ratio of decrease in void ratio to the increase in effective stress (or) pressure.

①

$$a_v = \frac{-\Delta e}{\Delta \sigma'}$$

$$a_v = \frac{e_0 - e}{\sigma' - \sigma'_0}$$

$$a_v = - \frac{(e - e_0)}{\sigma' - \sigma'_0}$$

units for a_v m^2/kN .

Δe = decrease in void ratio
 $\Delta \sigma'$ = increase in effective stress

In the above expression -ve sign because the void ratio decreases with increase in stress.

e = Final void ratio final stress σ'

e_0 = Initial void ratio under initial

stress of σ'_0

* Coefficient of volume change (m_v) :- it is the ratio b/w volumetric strain to the effective stress

②

$$m_v = \frac{e_v}{\Delta \sigma'}$$

$$m_v = \frac{-\frac{\Delta V}{V_0}}{\Delta \sigma'}$$

$$m_v = \frac{e_v}{\Delta \sigma'} \quad e_v = \text{volumetric strain} = \frac{\Delta V}{V_0}$$

is volumetric strain in terms of void ratio

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

$$m_v = \frac{\text{volumetric strain}}{\text{increase in stress}}$$

$$e = \frac{V - V_s}{V_s} = \frac{V}{V_s} - 1$$

$$\frac{V}{V_s} = e + 1$$

$$V = V_s (e + 1)$$

let us assume $V_0 = V_s (1 + e_0)$

$$\Delta V = V_0 - V = V_s (1 + e_0) - V_s (1 + e) = V_s (e_0 - e)$$

$$\frac{\Delta V}{V_0} = \frac{V_s (e_0 - e)}{V_s (1 + e_0)}$$

$$= \frac{\Delta e}{1 + e_0}$$

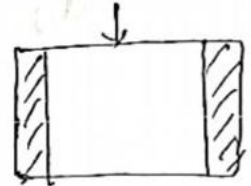
$$m_v = \frac{-\left[\frac{\Delta e}{1+e_0}\right]}{\Delta \sigma_1}$$

$$= \left(\frac{-\Delta e}{1+e_0} \times \frac{1}{\Delta \sigma_1} \right)$$

$$m_v = \frac{\frac{\Delta V}{V_0}}{1+e_0}$$

ii) Volumetric strain in terms of height of soil sample..

$$\frac{\Delta V}{V_0} = \frac{V_0 - V}{V_0} = \frac{\Delta H_0 - \Delta H}{\Delta H_0}$$



ΔH = change in height (or) thickness

H = Height (or) thickness

$\Delta \sigma_1$ = increase in stress.

$$= \frac{\Delta H}{H_0}$$

$$= \frac{\Delta H}{H_0}$$

$$\Delta H = H_0 \cdot \frac{\Delta e}{1+e_0} \quad (m_v)$$

$$m_v = \frac{-\Delta H}{H_0} \times \frac{1}{\Delta \sigma_1}$$

$$m_v = \left(\frac{-\Delta H}{H_0} \right) \frac{1}{\Delta \sigma_1}$$

$$C_c = \frac{-\Delta e}{\log_{10} \left(\frac{\sigma'_1}{\sigma'_0} \right)} \Rightarrow \Delta e = C_c \cdot \log \left(\frac{\sigma'_1}{\sigma'_0} \right)$$

$$\Delta H = -m_v \Delta \sigma_1 \cdot H_0 \quad \Delta H = \frac{H_0 \cdot C_c \log_{10} \left(\frac{\sigma'_1}{\sigma'_0} \right)}{1+e_0}$$

Settlement (s) = $m_v \Delta \sigma_1 H_0$

Consolidation Settlement $S =$

$$S_c = \frac{H_0 \cdot C_c}{1+e_0} \log \left(\frac{\sigma'_1}{\sigma'_0} \right)$$

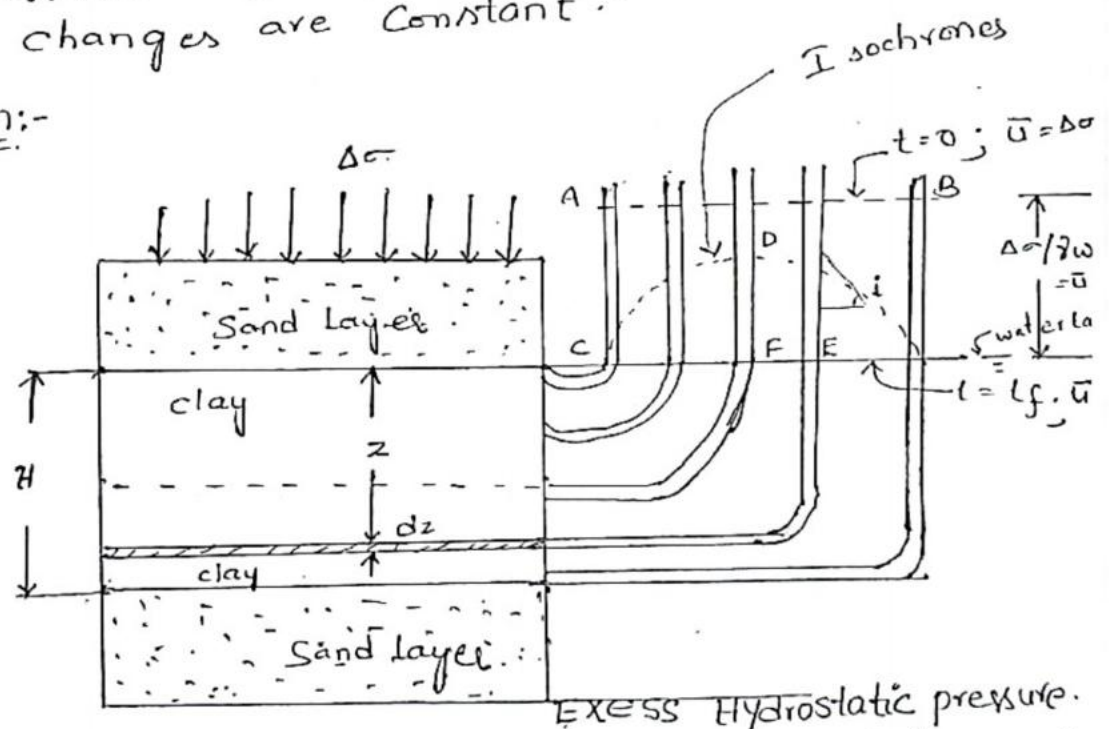
* determination of preconsolidation pressure: (Casagrande method)
 * it is conducted on a undisturbed clay soil
 (refer to preconsolidation fig)

Terzaghi's 1-D Consolidation equation:-

* Assumptions:-

1. The soil is homogeneous and Isotropic
2. The soil is fully saturated.
3. The solid particles and water in voids on the soil mass are incompressible
4. The coefficient of (fully) permeability is constant during consolidation process.
5. Darcy's law is valid throughout the Consolidation process
6. Soil is laterally confined and the Consolidation process takes place in axial direction only.
7. The time lag in consolidation is entirely due to the low permeability of soil.
8. The Coefficient of Compressibility and coefficient of volume changes are constant.

Derivation:-



→ Let us consider a Saturated clay layer of thickness $2d$ sandwiched between two layers of sand.

→ When a uniform pressure $\Delta\sigma$ is applied on the surface of top sand layer, the total stress developed at all points in the clay layer is increased by $\Delta\sigma$.

→ As explained in the Spring analogy model initially the whole of the pressure is taken up by water and the hydrostatic pressure of $\Delta\sigma/2w$ develops.

→ the excess hydrostatic pressure is right side on the diagram
 → it is assumed that various points along the thickness of the clay layer are connected flexible tubes to the piezometers.

$\bar{u}$ = pore water pressure

t_f = final time

$$\bar{u} = \gamma_w \cdot h$$

$$h = \bar{u} / \gamma_w = \frac{\partial (\bar{u})}{\partial z} \cdot \frac{1}{\gamma_w}$$

$$\frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \cdot \frac{\partial \bar{u}}{\partial z}$$

from darcy's law $v = k i$
 $= k \cdot \frac{\partial h}{\partial z}$

$$\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \cdot \frac{\partial}{\partial z} \left(\frac{\partial \bar{u}}{\partial z} \right)$$

$$= \frac{k}{\gamma_w} \cdot \frac{\partial^2 \bar{u}}{\partial z^2} \rightarrow \textcircled{1}$$

The quantity of water entering into the soil element at entry point per unit time = $v \cdot dx \cdot dy$

The quantity of water leaving into the soil element at exit point per unit time = $\left[v + \frac{\partial v}{\partial z} \cdot dz \right] dx \cdot dy$

the quantity of water squeezed out of soil element per unit time = $\frac{\partial v}{\partial z} \cdot dx \cdot dy \cdot dz \rightarrow \textcircled{2}$

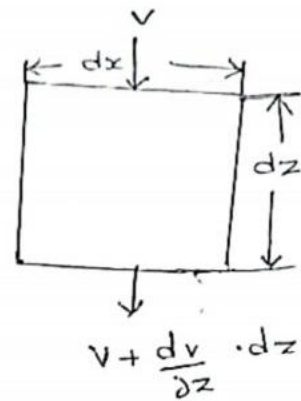
from the definition of coefficient of volume change.

$$m_v = \left(\frac{-\Delta v}{v_0} \right) \frac{1}{\Delta \sigma'}$$

$$\Delta v = -m_v \cdot v_0 \cdot \Delta \sigma'$$

$$\text{But } v_0 = dx \cdot dy \cdot dz$$

$$\Delta v = -m_v \cdot dx \cdot dy \cdot dz \cdot \Delta \sigma'$$



Rate of change of Volume per unit time

$$\frac{\partial (\Delta v)}{\partial t} = -m_v \cdot dx \cdot dy \cdot dz \cdot \frac{\partial (\Delta \sigma')}{\partial t}$$

with respect to time

$$\Delta \sigma' = \Delta \sigma - u' \quad \Delta \sigma = \Delta \sigma' + u' \quad \Delta u = \Delta \sigma - u' \quad \sigma = \sigma' + u$$

$$\frac{\partial \Delta u}{\partial t} = \frac{\partial \Delta \sigma}{\partial t} - \frac{\partial u'}{\partial t} \quad 0 = \frac{\partial \Delta \sigma'}{\partial t} + \frac{\partial u}{\partial t}$$

for a given pressure increment $\Delta \sigma = 0$

$$\frac{\partial (\Delta \sigma')}{\partial t} = - \frac{\partial u}{\partial t}$$

substitute above eqn

$$\therefore \frac{\partial (\Delta \sigma')}{\partial t} = - \frac{\partial u}{\partial t} \quad \frac{\partial (\Delta v)}{\partial t} = m_v \cdot dx \cdot dy \cdot dz \cdot \frac{\partial u}{\partial t} \rightarrow \textcircled{3}$$

from Consolidation theory $\textcircled{2}$ & $\textcircled{3}$ must be equal

$$\frac{\partial v}{\partial z} \cdot dx \cdot dy \cdot dz = m_v \cdot dx \cdot dy \cdot dz \cdot \frac{\partial u}{\partial t}$$

$$\frac{\partial v}{\partial z} = m_v \cdot \frac{\partial u}{\partial t}$$

from equation $\textcircled{1}$

$$\frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} = m_v \cdot \frac{\partial u}{\partial t}$$

$$\boxed{\frac{\partial u}{\partial t} = \frac{k}{\gamma_w \cdot m_v} \cdot \frac{\partial^2 u}{\partial z^2}}$$

$$\frac{\partial u}{\partial t} = C_v \cdot \frac{\partial^2 u}{\partial z^2}$$

where $C_v = \frac{k}{m_v \cdot \gamma_w}$

= coefficient of Consolidation

units are "cm²/s" (Stokes)

Solution for above equation is obtained by means

of Fourier Series $T_v = \frac{C_v \cdot t}{d^2}$

T_v = Time factor (Dimensionless quantity)

t = time

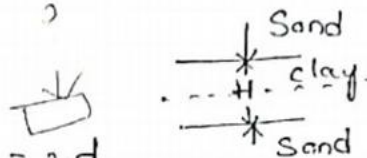
C_v = Coefficient of consolidation

d = drainage path

* Drainage path (d): The maximum distance that the water particle has to travel to reach the free drainage surface.

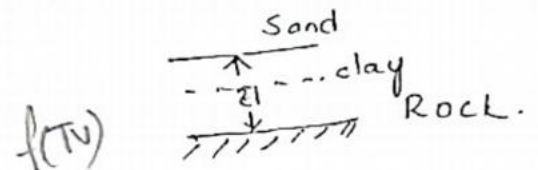
i) double drainage:

$$d = H/2 \Rightarrow H = 2d$$



ii) Single drainage

$$d = H$$



* Degree of Consolidation (U): $\frac{s}{s_f} \times 100$

where s = settlement at any time

s_f = final settlement

it is the function of time factor $u = f(T_v)$

if u is unknown, the approximate expression for T_v are

xx
Imp for
Quiz

$$T_v = \frac{\pi}{4} \left[\frac{u}{100} \right]^2 \quad \text{if } u < 60\%$$

$$T_v = -0.9332 \log_{10} \left[1 - \frac{u}{100} \right] - 0.0851 \quad \text{if } u > 60\%$$

problems:

1. The time to reach 60% consolidation is 32.5 sec for sample 10mm thick tested in the laboratory under the conditions of double drainage. how long corresponding to layer in nature required to reach same degree of consolidation. If it is 10m thick and designed at one side only

Sol:

Laboratory

$$u = 60\%$$

$$t = 32.5 \text{ Sec}$$

$$H = 10 \text{ mm}$$

Double drainage

$$d_1 = H/2$$

$$= \frac{10}{2} = 5 \text{ mm}$$

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

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

$$\boxed{\frac{t_1}{t_2} = \frac{d_1^2}{d_2^2}}$$

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

$$\frac{T_v}{C_v} = \frac{t_n}{d_n^2}$$

$$\frac{t_1}{t_n} = \frac{d_1^2}{d_n^2}$$

$$\frac{32.5}{t_n} = \frac{(5)^2}{(10,000)^2}$$

$$t_n = 1504.62 \text{ days.}$$

In Natural field

$$u = 60\%$$

$$t = ?$$

$$H = 10 \text{ m}$$

Single drainage

$$d_2 = H$$

$$d_2 = 10 \text{ m}$$

$$10 \times 1000 = 10000$$

$$\Rightarrow \frac{t_1}{t_2} = \frac{d_1^2}{d_2^2}$$

$$\Rightarrow \frac{32.5}{t_2} = \frac{(5)^2}{(10,000)^2}$$

$$t_2 = 1504.62 \text{ days}$$

2.

The Settlement analysis of a proposed structure proposes that 6 cm of Settlement occur in 3 years and that ultimate total Settlement will be about 25 cm analysis based on the assumption that the Compaction. clay layer both top and bottom Surface. However it is Suspended that there may not be drainage bottom. for the Single drainage case is ultimate total Settlement

ii) the time required for 6 cm of Settlement

Sol:

Settlement of structure in 3 years = 6 cm

ultimate total settlement = 25 cm

is ultimate total settlement = 25 cm

for double drainage

for single drainage

T_v

T_v

C_v

C_v

$d = \frac{H}{2}$

$d = H$

$t_2 = 9$

$t_1 = 3 \text{ years}$

$$\frac{t_1}{t_2} = \frac{d_1^2}{d_2^2} = \frac{3}{t_2} = \frac{(12.5)^2}{(25)^2}$$

3.

In laboratory consolidation test on a 20 mm thick saturated clay layer from a site 50% consolidation point was reached in 10 min. estimate the time required for clay layer of 5 m thickness at the site for 50% consolidation. if there is drainage only towards the top.

Sol:

Laboratory

site

$H = 20 \text{ mm}$

$U = 50\%$

$U = 50\%$

$H = 5 \text{ m}$

$t = 10 \text{ min}$

$t = ?$

$d = H/2 = 10 \text{ mm}$

$d = H = 5 \text{ m}$

(60 x 60 x 24 x 36)

$$\frac{t_1}{t_2} = \frac{d_1^2}{d_2^2} \Rightarrow t_2 = \frac{10}{\frac{100}{(5000)^2}}$$

4.

In a consolidation test on a soil the void ratio of the sample decreases from 1.24 to 1.12 where the pressure is increasing from 200 to 400 kN/m². calculate the coefficient of consolidation in m²/year given that the coefficient of permeability of the soil during this pressure increment is 2.5 x 10⁻⁸ m/s.

Sol:

$$e_0 = 1.124$$

$$e = 1.120$$

$$\sigma'_0 = 200 \text{ kN/m}^2$$

$$\sigma'_1 = 400 \text{ kN/m}^2$$

$$C_v = ?$$

$$K = 8.5 \times 10^{-8} \text{ cm/s}$$

$$m_v = \frac{\Delta e}{1+e_0} \cdot \frac{1}{\Delta \sigma'_1}$$

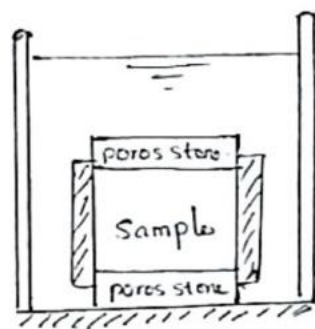
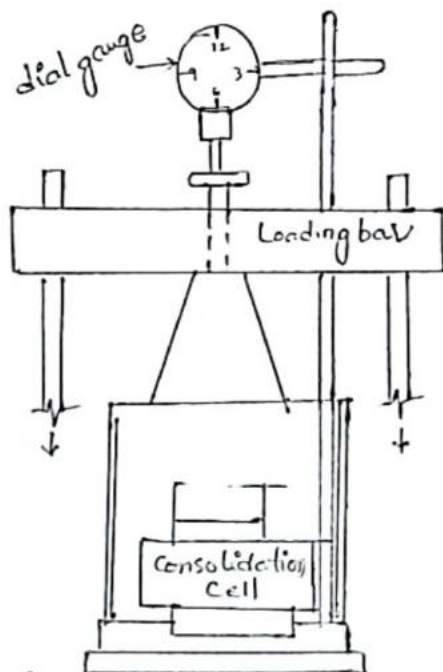
$$= \frac{(1.124 - 1.12)}{1 + 1.124} \cdot \frac{1}{(400 - 200)}$$

$$= 9.416 \times 10^{-6} \text{ m}^2/\text{kN}$$

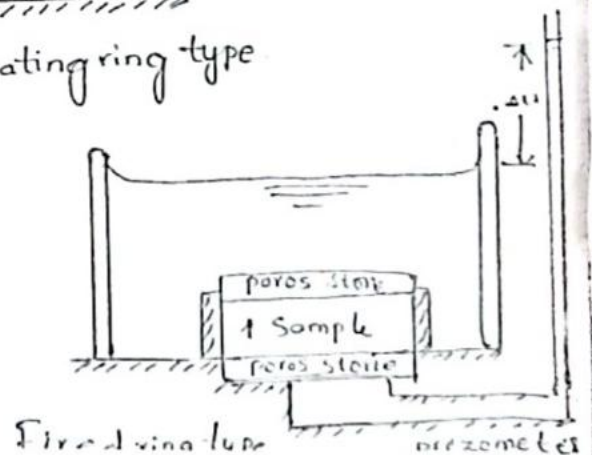
$$C_v = \frac{K}{m_v \cdot \gamma_w} = \frac{8.5 \times 10^{-10}}{9.416 \times 10^{-6} \times 9.81}$$

$$= 2.91 \times 10^{-13} \text{ m}^2/\text{year}$$

* Laboratory Consolidation Test: (oedometer or consolidometer)



floating ring type



Fixed ring type

piezometer

→ The consolidation test is conducted in a laboratory to study the compressibility of a soil. The test is performed in the consolidation test apparatus, known as the consolidometer or an oedometer.

→ It consists of a loading device and a cylindrical container called consolidation cell. The soil specimen is placed in the cell between top and bottom porous stones. The consolidation cell are of two types. (1) floating & freeing cell: In which both top and bottom porous stones are free to move. the porous stone can be move upward as the sample consolidates.

(2) fixed ring cell: in which the bottom porous stone cannot move only the top porous stone can move downward as the specimen consolidates. The fixed ring cell can also be used as the variable head permeability test apparatus for this purpose, a piezometer is attached to the base of the cell.

→ The inside surface of the ring should be smooth and polished to reduce friction. The ring imposes a condition of zero lateral strain on the soil sample. The internal diameter of the cell is usually 60 mm, but the cells with a diameter up to 100 mm also available. The thickness of the sample is fixed from the following considerations.

1) The thickness of the sample should be as small as possible to reduce side friction, but a minimum thickness of 20 mm is usually required to get uniform distribution of pressure on the sample.

2) The diameter to the thickness ratio should be a minimum of 3.

3) The thickness of the specimen should not be less than 10 times the maximum size of the particle.

→ The thickness of the sample for 60 mm dia. cell is usually as 20 mm. The specimen of diameter 50, 70 and 100 mm may be used in special cases.

→ It is useful to know the compressibility characteristics of the saturated soil.

→ It consists of a loading frame (upto a load of 1000 kN/m^2) ($5, 50, 10, 100, 150, 400, 800, 1000 \text{ kN/m}^2$)

* In any stage of loading, it acts as a variable head method and it is useful to measure directly coefficient of permeability by the fixed ring cell.

* For each loading the reading should be taken at different intervals. Generally they are 0.25', 1.00', 2.25', 4', 6.25', 9.00', 12.25', 16', 20.25', 25', 36.49', 60', 2 hrs, 4 hrs, 8 hrs, 24 hrs.

→ The data obtaining from consolidation is useful to know the m_v, C_v, k .

$$C_v = \frac{k}{m_v \gamma_w}$$

* Determination of Coefficient of Volume change (m_v):-

To determine this there are two methods

1. Height of solids method.
2. equivalent void ratio method:

→ Height of Solids method:-

$$G = \frac{\gamma_s}{\gamma_w} = \frac{\gamma_s}{1} = \frac{W_d}{V_s}$$

$$V_s = \frac{W_d}{G}$$

$$A \times H_s = \frac{W_d}{G}$$

$$H_s = \frac{W_d}{GA}$$



H_s = Thickness of sample or height of solid

A = Area

H = Thickness of sample

$$\text{Void ratio } (e) = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{AH - AH_s}{AH_s} = \frac{(H - H_s)A}{AH_s}$$

$$e = \frac{H - H_s}{H_s} \quad \text{where}$$

H = Specimen height (cm) at equilibrium under various pressures

$$H = H_0 \pm \sum \Delta H$$

ΔH = change in Specimen thickness under any pressure increment

H_0 = Initial height of Specimen.

* Data Sheet for determining m_v : Initial dial gauge reading.

Applied pressure (σ') kN/m ²	final dial gauge	change in dial gauge reading ΔH	Specimen height $H = H_0 \pm \Delta H$	$H - H_s$	$e = \frac{H - H_s}{H_s}$
5					
10					
15					
20					

$$m_v = \frac{\Delta e}{1+e_0} \cdot \frac{1}{\Delta \sigma'}$$

$$H_s = \frac{W_d}{G_d}$$

* equivalent void ratio method (or) change in void ratio method

$$e_f = w_f G$$

$$S_r e = w G$$

$$\therefore \frac{\Delta e}{1+e_0} = \frac{\Delta H}{H}$$

$$\Delta e = \frac{\Delta H}{H} \times (1+e_0)$$

$$\therefore \Delta e = \frac{\Delta H}{H_f} \times (1+e_f)$$

or $e = \frac{V-V_s}{V_s} = \frac{V}{V_s} - \frac{V_s}{V_s} = \frac{V}{V_s} - 1$
 $1+e = V/V_s \Rightarrow (1+e)V_s = V$
 $(1+e)V_s = A \times H - W \quad V_s + eV_s = A \times H$
 By partial differentiation
 $A \times dH = V_s de - (2) \quad \text{divide eq (1)}$
 $\frac{A dH}{H} = \frac{V_s de}{(1+e)V_s} = \frac{de}{1+e}$

Final dial gauge reading	ΔH	$H = H_0 \pm \Delta H$	$\Delta e = \frac{\Delta H}{H_f} (1+e_f)$	e
5				
10				
...				
800				
			Δe	e_f

* determination of co-efficient of consolidation (C_v):
 Theoretical relationship b/w u and T_v vs dial gauge reading and time.

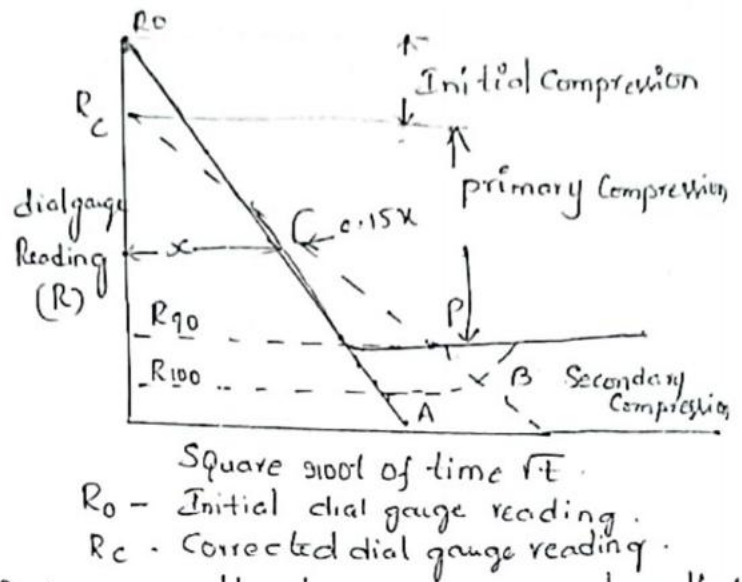
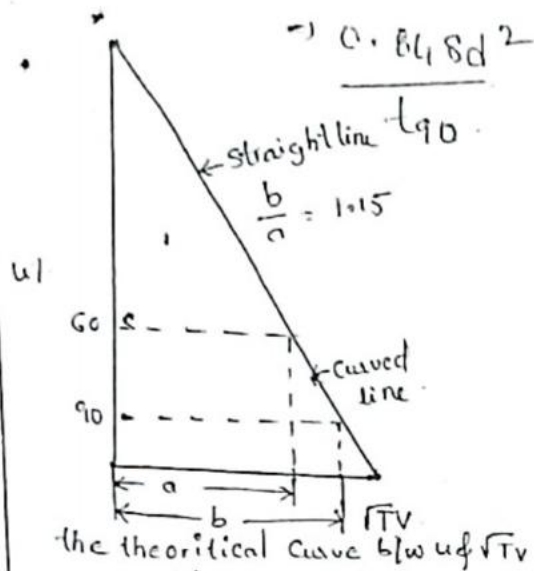
- 1) Square root of time fitting method (Taylor's method).
- 2) Logarithm of time fitting method (Casagrande method)

* Square root of time method: The method devised by Taylor, utilizes the theoretical relationship between u and $\sqrt{T_v}$. The relationship is linear up to the value of u equal to about 60%. It has further established that $u = 90\%$. The value of $\sqrt{T_v}$ is 1.15 times the value obtained by the extension of the initial straight line portion.

→ The settlement b/w R_0 & R_c Initial compression

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

$$C_v = \frac{(T_v)_{90} d^2}{1.15^2}$$



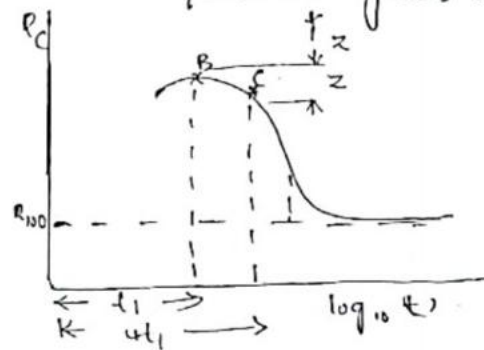
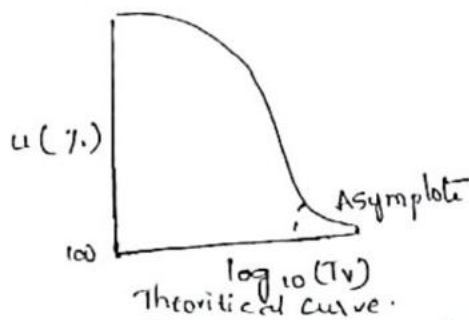
* Logarithm of time fitting method: (or) Casagrande method

The method given by Casagrande uses the theoretical curve between u and $\log T_v$. The curve consists of three parts.

1) an initial portion which is parabolic in shape

2) an middle portion which is almost linear

3) the last portion to which the horizontal axis is an asymptote. It is observed that the point of inflexion of the curve and tangent drawn at the point of inflexion on the curve and the asymptote of the lower portion gives the value of 100% Consolidation.



$$C_v = \frac{T_v 50d^2}{t_{50}}$$

$$(T_v)_{50} = \frac{C_v t_{50}}{d^2}$$

$$K = C_v m_v \gamma$$

* Comparison of two methods: - The two methods for determination of the coefficient of consolidation give comparable results for most of the soils. However, the following points must be carefully noted.

1) For some soils, the square-root of time plot does not give a straight line for the initial portion and therefore, to locate the corrected zero R_c becomes difficult. For such soils the log-of-time method is better.

2) The square-root of time method is more suitable for soil exhibiting high secondary consolidation. In such soils the log-t plot does not show the characteristic shape required to locate the point corresponding to 100% consolidation.

3. The Square-root of time method is more convenient for general case, as it requires dial gauge readings covering a much shorter period of time compared with the log-time method. The latter method requires accurate plotting of the secondary Consolidation Curve in order to locate the asymptote.

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HAND WRITTEN NOTES ON.

GEO-TECHNICAL ENGINEERING - I.

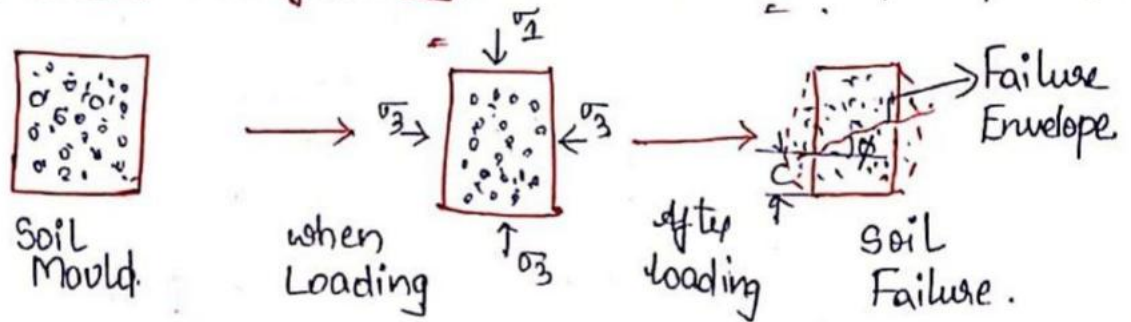
Chapter: - V.

Topic: Shear Strength of Soils.

Civil Engineering Department

- ! Importance of Shear Strength - Mohr's-Coulomb Failure Theories
- Type of laboratory tests for strength parameters - Strength tests based on drainage conditions - Strength envelopes - Shear strength of sands - Dilatancy - Critical Void Ratio - Liquefaction
- Shear strength of clays.

The shear strength of a soil is its maximum to shear stresses just before the failure. [The σ_1 are developed when the soil is subjected to direct compressions. At that stage, the soil starts the failure], $\rightarrow$ (Frictional Movement B/N Soil particles).



Here, " $C + \phi$ " are the shear strength parameters, mostly used to find the shear strength of soil due to failure condition.

where:- $C \rightarrow$ Cohesion.
 $\phi \rightarrow$ Angle of Friction

$C \rightarrow$ for cohesionless $C=0$.
 $\rightarrow$ for cohesive $C=C$.

Generally, In field the soils are subjected to tension, It Causes the two stresses. $\rightarrow$ (1) Major Principal Stress (σ_1).

Here, σ_2 gets unity

$\rightarrow$ (2) Minor Principal Stress (σ_3)

$\rightarrow$ (2) Intermediate Principal stress (σ_2)

Due to these stresses causes an opening of cracks & fissures to the soil mass. Thus, the shear failure of a soil mass occurs i.e., "the strength of the soil mass decreases".

Important Note :-

(τ) of a soil is the major principal engineering property which controls the stability of a soil mass under loads. It governs the bearing capacity of soils, Stability of slopes, Earth pressure against retaining structures.

Different Types of Soils :-

The testing of a ~~soil~~ shear strength, there are 3 different types of soils were divided. They are —

1.) Cohesionless Soil $\rightarrow C=0$!

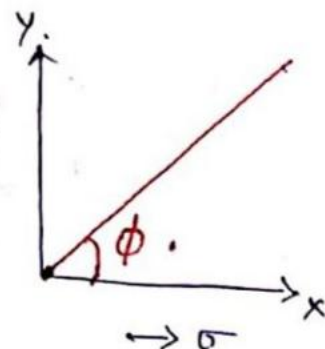
2.) Purely Cohesive Soil $\rightarrow \phi=0$.

3.) Cohesive - frictional Soil $\rightarrow C; \phi$ are present.

① Cohesionless Soils :-

Generally, Cohesionless Soils are C.G.S. $\approx$
Where the soils do not have Cohesion $\uparrow$
($C=0$). These soils derive the shear strength from the intergranular friction. Is also called frictional soils.

Eg:- Sand ; Gravel.

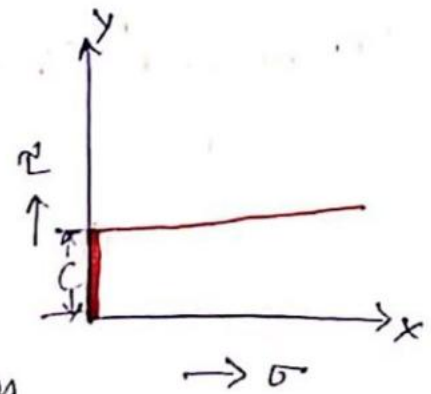


② Purely Cohesive Soil := $\phi = 0$.

Generally, Purely cohesive soil is a fine grained soil. So, cohesion is present. When the load is applied on the soil, it ~~resists~~ carries the load upto its strength after it failure.

These soils exhibit cohesion but angle of shear resistance $\phi = 0$.

For Example := Saturated clay; Silts. } $\rightarrow$ Under Undrained condition.

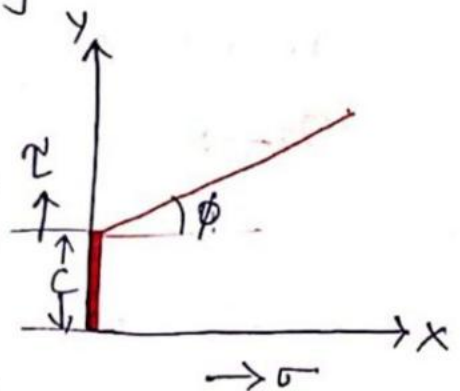


③ Cohesive - frictional Soil := $c; \phi$ having.

These are composite soils, having both cohesion (c) & angle of shear resistance (ϕ).

These are called $c-\phi$ soils.

For Example := Clayey sand; Sandy clay; Silty sand; etc.,.



Theories were developed - the Shear Strength :=

To find the Shear strength parameters, There are 3 theories have developed ($\tau' \rightarrow c, \phi$).

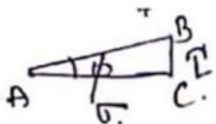
- 1) Coulomb's Theory
- 2) Mohr's Theory
- 3) Terzaghi's Theory.

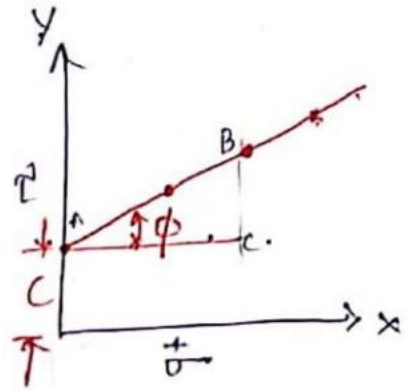
① Coulomb's Theory :-

The Coulombs have depended on the normal (or) total stresses to find the shear strength of a soil.

$C \rightarrow$ The cohesion will gets the strength of the soil particle reads.

$\phi \rightarrow$ The angle of shear resistance. readings shows the failure points.

$\Delta^k ABC$, 



① $\rightarrow \tan \phi = \frac{\tau}{\sigma} \Rightarrow \tau = \sigma \cdot \tan \phi \rightarrow$ For Angle Friction

② $\rightarrow \tau = C.$

$$\tau = C + \sigma \cdot \tan \phi$$

where, τ = Shear Strength

C = Cohesion

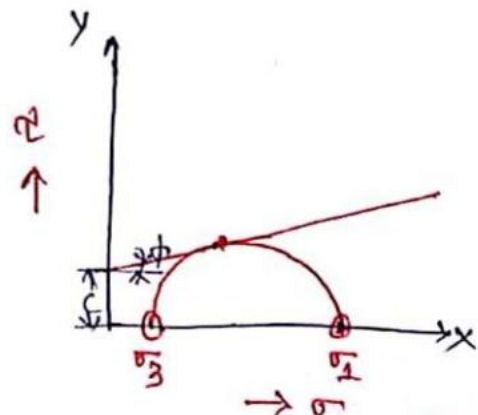
σ = Normal Stress / Total Stress

ϕ = Angle of Shear Resistance

In this method, Coulombs developed the shear strength equation to find " C & $\phi \rightarrow$ parameters".

② Mohr's Theory :-

Here, Mohr's developed the shear strength equation by depending on the Major (σ_1) & Minor (σ_3) stresses.



The Shear strength of a soil for Mohr's Theory is — .

$$\tau = c + \sigma \cdot \tan \phi$$

③ Terzaghi's Theory :-

The Terzaghi's theory was depended on the effective stress. & pore water pressure. So, the shear strength of a soil is — .

$$\tau = c' + \sigma' \cdot \tan \phi' \Rightarrow c' + (\sigma - u) \tan \phi'$$

where :-

c' = Effective Cohesion

σ' = Effective Stress

ϕ' = Effective angle of shearing resistance.

Mohr's Circle Failure Method :-

Otto Mohr, a German scientist, devised a graphical method for the determination of stresses on a plane inclined to the principal planes. The graphical construction is known as Mohr's Circle & is extremely useful.

Construction of Mohr's Circle :-

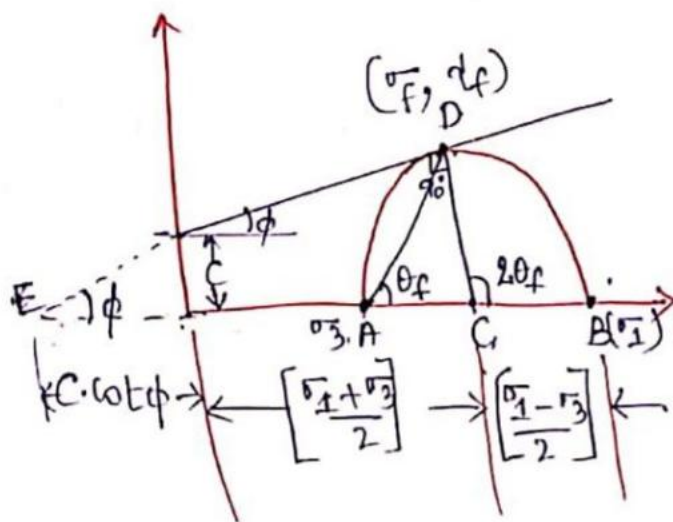
⇒ Select the origin point, the normal stresses ($\sigma_1; \sigma_3$) are plotted along horizontal axis & the shear stress (τ) are plotted along vertical axis.

⇒ The point A represents the minor stress (σ_3) & the point B represents the major stress (σ_1).

⇒ The point C is the middle point with the normal stress co-ordinate equal to $\left[\frac{\sigma_1 + \sigma_3}{2} \right]$.

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⇒ The Circle is drawn, with "C" as the Centre & AB as diameter. This Circle is known as "Mohr's Circle".



Note:- $\triangle EDC$,
 $2\theta_f = 90^\circ + \phi'$
 $\theta_f = 45^\circ + \frac{\phi'}{2}$

Figure : Mohr's Circle Failure.

The shown figure is the failure envelope of a soil. It is tangent to the Mohr circle @ failure. The stress on the failure plane @ failure is indicated by point D on the Mohr circle. The failure plane is indicated by the line BF. The failure plane makes an angle (θ_f) with the major principal plane. The failure envelope has an intercept (C) on τ -axis & it cuts the σ -axis @ point E.

$$\therefore \triangle EDC, \quad \sin \phi = \frac{DC}{EC} \Rightarrow \frac{(\sigma_1 - \sigma_3)/2}{2C \cdot \cot \phi + \frac{(\sigma_1 + \sigma_3)}{2}}$$

$$(\sigma_1 - \sigma_3) = 2C \cdot \frac{\cos \phi}{\sin \phi} \times \sin \phi + \sigma_1 \cdot \sin \phi + \sigma_3 \sin \phi$$

$$\sigma_1 - \sigma_1 \sin \phi = 2C \cdot \cos \phi + \sigma_3 + \sigma_3 \sin \phi,$$

$$\therefore \sigma_1 = \frac{2C \cdot \cos \phi}{1 - \sin \phi} + \sigma_3 \cdot \frac{(1 + \sin \phi)}{(1 - \sin \phi)}$$

$$\text{Similarly } \sigma_3 = \frac{2C \cdot \cos \phi}{1 + \sin \phi} - \sigma_1 \cdot \frac{(1 - \sin \phi)}{(1 + \sin \phi)}$$

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For Modified Mohr's Circle of Shear Strength equation.

$$\bar{\sigma}_1 = \bar{\sigma}_3 \cdot \frac{(1 + \sin \phi')}{(1 - \sin \phi')} + 2c' \cdot \frac{\cos \phi'}{(1 - \sin \phi')}$$

$$\bar{\sigma}_3 = \bar{\sigma}_1 \cdot \frac{(1 - \sin \phi')}{(1 + \sin \phi')} - 2c' \cdot \frac{\cos \phi'}{(1 + \sin \phi')}$$

Here, $\tan^2(45^\circ + \frac{\phi}{2}) \rightarrow \text{Flow Ratio } N_\phi$.

$$\bar{\sigma}_1 = \bar{\sigma}_3 \cdot \tan^2(45^\circ + \frac{\phi}{2}) + 2c' \cdot \sqrt{\tan^2(45^\circ + \frac{\phi}{2})}$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \cdot N_\phi + 2c' \cdot \sqrt{N_\phi}$$

~~Soil~~ If Cohesionless, $c' = 0$.

$$\therefore \bar{\sigma}_1 = \bar{\sigma}_3 \cdot \tan^2(45^\circ + \frac{\phi}{2}) \cdot (\text{or})$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \cdot \frac{(1 + \sin \phi')}{(1 - \sin \phi')}$$

The $\bar{\sigma}_1$ Equation is called as a plastic Equilibrium, where the soil mass is said to have reached a state of plastic equilibrium.

Types of Laboratory tests for shear strength Parameters.

To measure the shear strength parameters $(c; \phi)$ by using 4-types.

- 1.) ~~Tri~~ Tri-Axial Shear Test. $\rightarrow$ Most important to measure $c; \phi$.
- 2.) Direct Shear Test.
- 3.) Unconfined Compression Test.
- 4.) Vane Shear Test.

① Tri-Axial Shear Test :-

Here, the Triaxial shear test must be conducted under drainage conditions which occurs in ~~problem~~ field problems. There are 2-stages taken to the soil.

$\rightarrow$ ① = Consolidation stage (σ), where normal stresses applied to the specimen & is allowed to consolidate.

$\rightarrow$ ② = Shearing stage, where the shear stress (τ) applied to the specimen to shear the soil specimen.

Note:-

Before going to triaxial shear test, let we know the drainage conditions in this test.

The triaxial test depending upon 3-drainage conditions, They are — .

Unconsolidated
Undrained
Test

UU-Test
(or)
"Q"-Test

- * There is no drainage is permitted during the consolidation stage.
- * The drainage is also not permitted in the shearing stage.
- * There is no time allowed for consolidation.
- * Due to heavy pressure of 1st stage & 2nd stage, the soils become failure in quickly.
- * It is called Quick-Test (or) Q-Test.

Consolidated
Undrained
Test

CU-Test
(or)
"R"-Test

- * The drainage is permitted during the consolidation stage until its complete.
- * The drainage is not permitted in the shearing stage.
- * The time allowed to failure of soil greater than UU-Test.
- * By the Value of σ_d @ 1st stage & Value of σ_d @ 2nd stage the soil becomes failure depend on Pressure.
- * It is called CU-Test (or) R-Test.

Consolidated
Drained
Test

CD-Test
(or)
"S"-Test

- * The drainage is permitted during the consolidation stage & shearing stage.
- * When consolidation completed, the shear stage started @ a very slow rate to ensure fully drained condition.
- * By the Value of σ_d @ both 1st & 2nd stages, the soil failures very slowly.
- * It is called Slow Test (or) CD-Test.

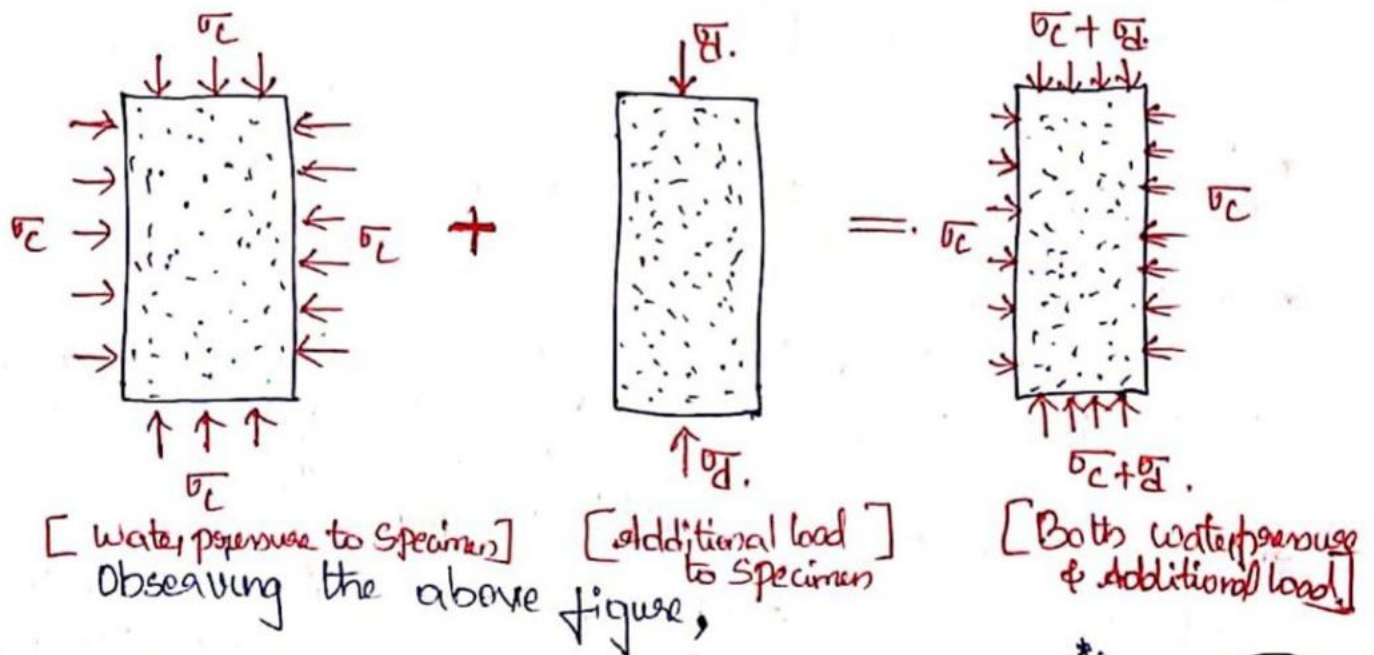
Imp-Note :-

The above 3-drainage conditions are depends on the outlet Value, Shown in figure in Triaxial Test.

The triaxial Compression Test (or) Simply triaxial test, is used for the determination of Shear characteristics of all types of soils under different drainage conditions.

~~Here~~, In First stage of test, the specimen is subjected to an all round Confining pressure (σ_c) on both top & bottom sides (or) around the sides of the soil specimen. This stage is called Consolidation stage.

In Second stage of test, an additional load applied to the soil specimen. This stage is called Shearing stage & the additional load is called deviator stress (σ_d). σ_d is applied on the top of the specimen through a ram.



Major principal stress, $\sigma_1 = \sigma_c + \sigma_d$
 Minor principal stress, $\sigma_3 = \sigma_c$
 Intermediate " " , $\sigma_2 = \sigma_c$

σ_c = Confining pressure / water pressure / pore water pressure / Consolidation pressure

Procedure of a Triaxial Test:-

The triaxial test apparatus consists of a circular base that has a central pedestal. The pedestal has 1 (or) 2-hobs which are used for drainage of the specimen in a drained test (or) for the pore pressure measurement in an undrained test.

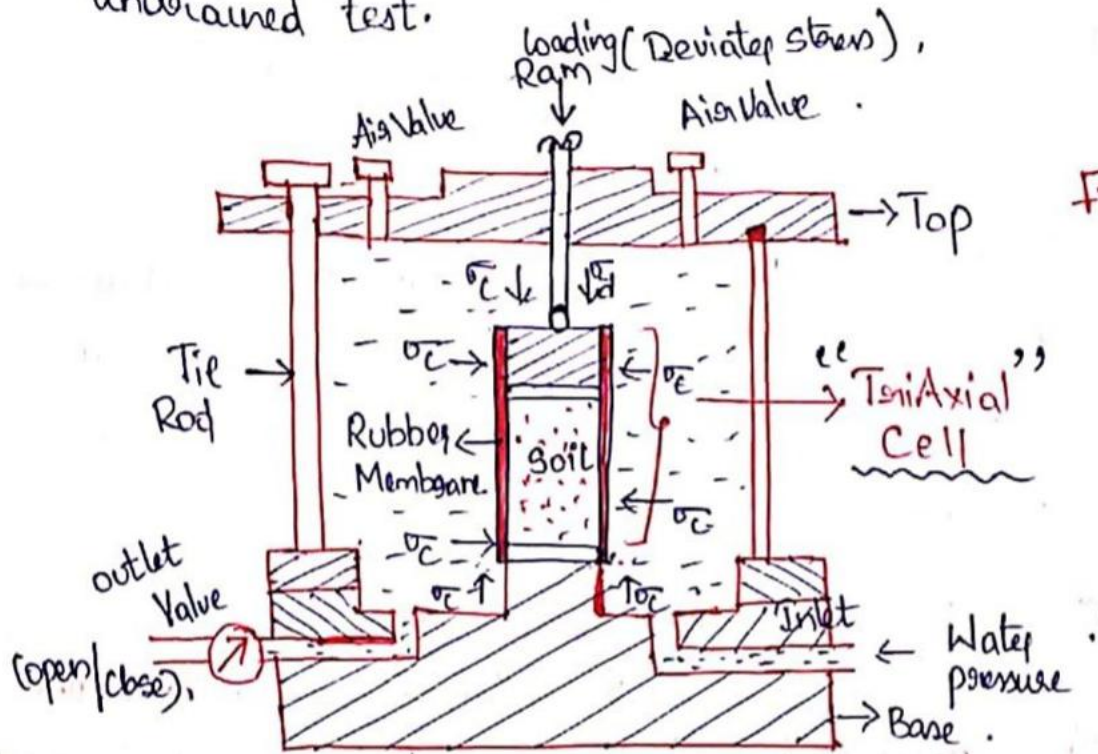


Figure: Triaxial Apparatus

A Triaxial Cell is fitted to top of base plate with the help of 3-wing nuts.

- * The normal size of the sample will be 76mm x 38mm & 100mm x 50mm.
- * The Soil Specimen was placed in rubber membrane, because not to enter the water in soil otherwise, the soil will ~~get~~ start ~~starts~~ liquidity.
- * The soil specimen of rubber membrane is fixed to the triaxial cell by O-Ring.
- * The triaxial cell was fixed to the triaxial apparatus with placing of loading ram, drainage pipe @ inlet & outlet.
- * The outlet pipe is measure the pore pressure. (The valve will be may closed (or) opened). The inlet pipe is to enter the water pressure.

* After all fixing up the soil specimen $\rightarrow$ Triaxial Cell,
Triaxial Cell $\rightarrow$ Triaxial Apparatus,
Connecting of drain pipes etc.,

* Now, start the process of this test by different drainage conditions which explained as previous concept. (C.U-Test; U.U-Test; C.D-Test).

* The soil specimen gets failure @ different drainage conditions by (σ_1 & σ_3) pressures. Take the readings of failure points & plot the graph to find shear strength parameters ($c; \phi$).

$C \rightarrow$ Cohesion. [for Granular Soil, $C=0$,
" Silt & clay Soil, $C = \text{constant value}$].

$\phi \rightarrow$ Angle of Shearing Resistance [failure points to measure ϕ].

$$\tau = c + \sigma \cdot \tan \phi \rightarrow \text{For Triaxial Test,}$$

② Direct Shear Test:

A direct shear test is conducted on a soil specimen in a shear box, which is split into 2-halves along a horizontal plane @ its middle.

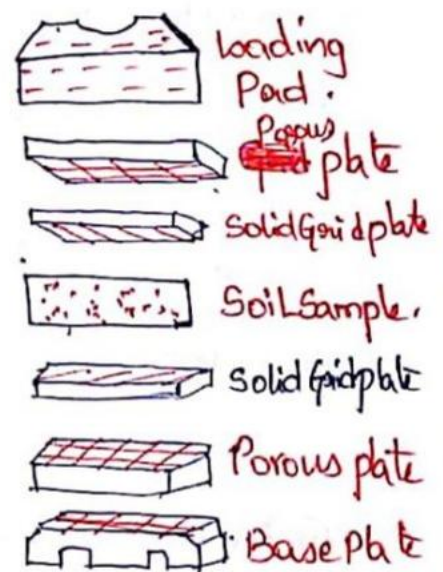
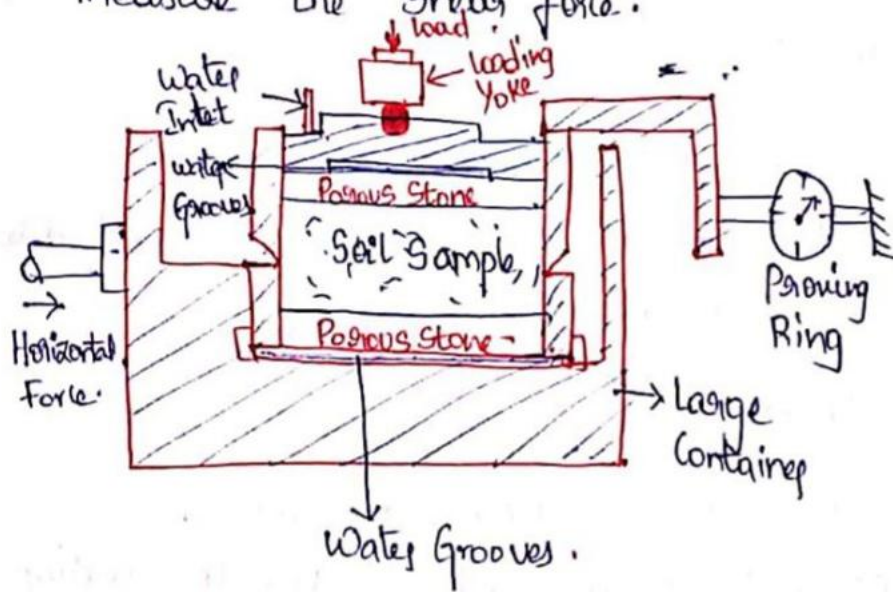
$\Rightarrow$ A Square box of size 60mm x 60mm x 50mm is commonly used.

$\Rightarrow$ The box is divided horizontally such that the dividing plane passes through the centre.

⇒ The lower half of the box is fixed to the base plate which is rigidly held in position in a large container. The large container is supported on rollers.

⇒ A loading frame is used to support the large container. It has the arrangement of a loading yoke () & a lever system for applying the normal load.

⇒ A proving ring is fitted to the upper half of the box to measure the shear force.



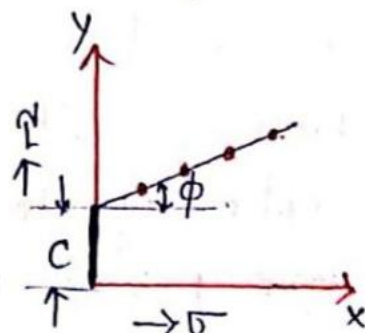
Step by Step procedure of Direct Shear Test :-

- 1.) Collect the soil specimen which is either undisturbed (or) remoulded. The should be taken using sampler. ~~It is not required~~ If cohesionless soil is taken, then sampler & rammer are not required.
- 2.) Take the box in dimensions $60\text{ mm} \times 60\text{ mm} \times 50\text{ mm}$. The thickness of soil in box contains 25 mm . Now attach the two halves of the shear box with locking pins & place the base plate @ bottom.

- 3.) Above the bottom plate, place the porous stone & also place the grid plate above it. Similarly, Below the loading plate, place the porous stone & grid plate. The soil should be placed in between the porous plates.
- 4.) Place the Shear box with soil specimen, in a container & mounted on the loading frame. The proving ring is arranged in such a way & loading yoke is placed on steel ball of loading pad of shear box.
- 5.) Two Dial gauges are fitted, one - to measure Shear displacement (connected to container), another (connected to loading yoke) to measure vertical displacement.
- 6.) Now, remove the locking pins of shear box & start the ~~process~~ loading 25 kN/m^2 & apply shear load.
- 7.) Now the box ^{starts} reacting to load applied & note the readings for every 30 seconds from proving ring & dial gauges.
- 8.) Repeat the process with different normal stresses 50 kN/m^2 ; 100 kN/m^2 ; 150 kN/m^2 ; 200 kN/m^2 ; ~~250~~ 250 kN/m^2 ; 300 kN/m^2 ; 400 kN/m^2 . & note down the reading upto failure stage.
- 9.) Then finally, find C & ϕ - parameters of a soil failure.

Formula, $\tau = C' + \sigma' \tan \phi'$

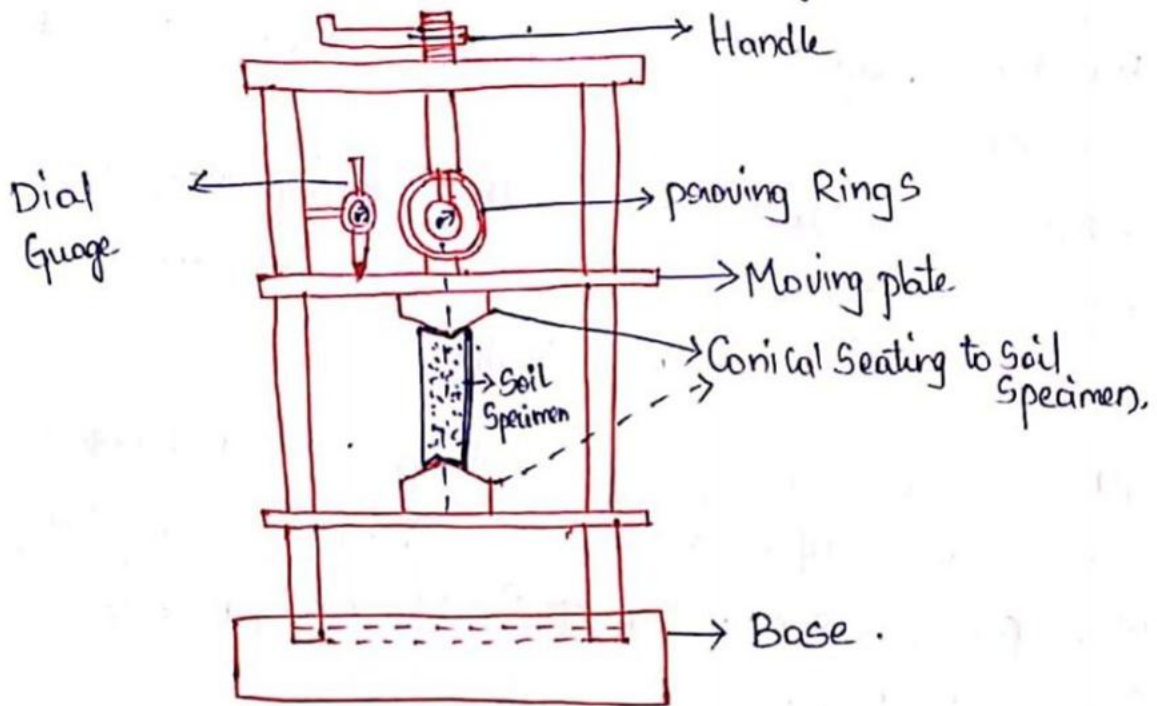
(or) $\tau = C + \sigma \tan \phi$



③ Unconfined Compression Test :-

The unconfined compression test is a special form of a tri-axial test in which the confining pressure is zero.

This test can be conducted only on clayey soils which can stand without confinement.



The unconfined compressive strength (q_u) is the load per unit area @ which the cylindrical specimen of a cohesion soil fails in compression.

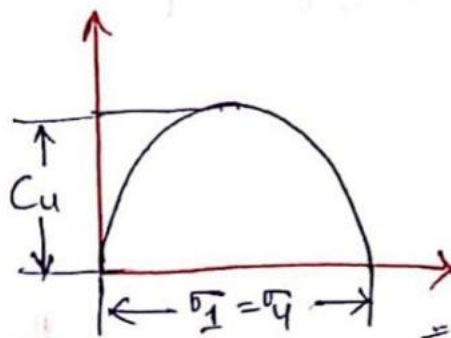
$$q_u = \frac{P}{A}$$

$\Rightarrow$ Load.
 $\Rightarrow$ Area of Soil Specimen (Circular)

The undrained shear strength (s) of the soil is equal to one half of the unconfined compressive strength.

Procedure of Unconfined Compression Test :-

- 1) Prepare the Soil Sample in split mould. ($\phi = 38 \text{ mm}$; $h = 76 \text{ mm}$) same as a proctor Compaction (3-layers $\rightarrow$ 25 blows). After remove the mould (or) Soil Specimen.
- 2) place the Soil Specimen on the bottom plate of Compression machine. Adjust the upper plate to make contact with the Specimen.
- 3) Adjust the dial gauge & the proving ring gauge to zero, & apply the Compression load to cause an axial strain at the rate of $1\frac{1}{2}\%$ to 2% per minute.
- 4) Record the dial gauge reading, & the proving reading every 30 sec upto a strain of 6% . The reading may be taken after every 60 sec for a strain b/w 6% , 12% & every 2 min (or) so beyond 12% .
- 5) Continue the test until the failure surfaces have clearly developed (or) until an axial strain of 20% is reached.
- 6) Measure the angle between the failure ~~specimen~~ surface & the horizontal, if possible.
- 7) Take the Sample from the failure zone of the Specimen for the water content determination.



$$S = C_u = \frac{\sigma_1}{2} = \frac{\sigma_3}{2}$$

④ Vane Shear Test:-

The undrained shear strength of a soft clay can be determined in a laboratory by a "Vane Shear Test". The test can also be conducted in the field on the soil at the bottom of a bore hole. The field test can be performed even without drilling a bore hole by direct penetration of the vane from the ground surface if it is provided with a strong shoe to protect it.

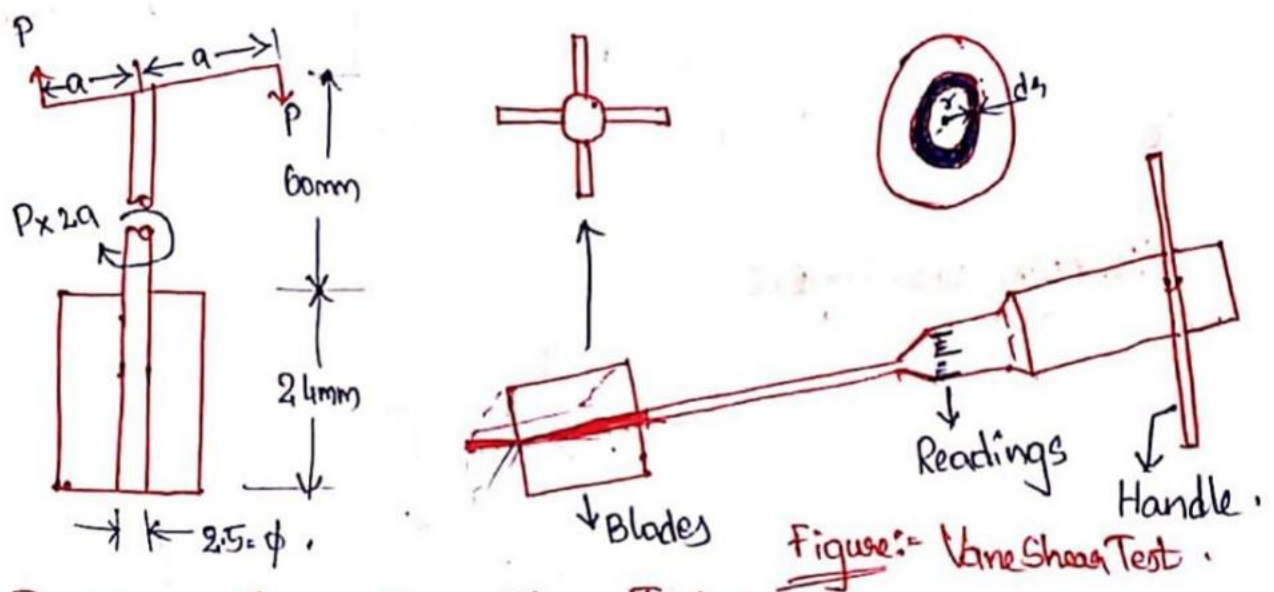


Figure:- Vane Shear Test.

Procedure of a Vane Shear Test:-

1.) Clean the Vane Shear Apparatus & apply grease to the lead screw for better movement of handles.

Take the soil specimen in container which is generally $H = 75\text{mm}$; $\phi = 37.5\text{mm}$.

2.) level the soil surface on the top & mount the container on the base of Vane Shear test apparatus using screws provided.

3.) lower the vane gradually into the soil specimen until the top of vane is @ depth = $10\text{mm} - 20\text{mm}$ below the top of soil specimen.

4.) Note down the reading of pointer on circular graduated scale which is initial reading.

5.) Rotate the vane inside the soil specimen using torque applying handle @ a rate of 0.1° per second.

6.) When the specimen fails, the strain indicator pointer will move backwards on the circular graduated scale & at this point stop the test & note down the final reading of pointer.

7.) The difference between the initial & final readings is nothing but the angle of torque. Repeat the procedure on 209 more soil specimens & calculate the τ_{avg} value.

Strength Envelope:

$$S = \frac{T}{\pi \left[\frac{D^2 H}{2} + \frac{D^3}{6} \right]}$$

If the depth of vane inside the height (H_1).

$$S = \frac{T}{\pi \left[\frac{D^2 H_1}{2} + \frac{D^3}{12} \right]}$$

Sensitivity:- The vane shear test can be used to determine the sensitivity of soil. After the initial test, the vane is rotated rapidly through several revolutions such that the soil becomes remoulded. The test is repeated on the remoulded soils & the shear strength in remoulded state is determined.

$$S_t = \frac{\text{Undisturbed}}{\text{Remoulded}}$$

(Sensitivity)

Liquefaction of Sand:

The Shear strength of sandy soils is given by Mohr-Coulomb equation, taking cohesion intercept as ($c' = 0$).

$$S = c' + \bar{\sigma} \cdot \tan \phi' \Rightarrow S = \bar{\sigma} \cdot \tan \phi' \rightarrow (1)$$

If the sand deposit is @ a depth of z - below the ground & the water table is @ the ground surface, the $\Delta \bar{\sigma}$ —

$$\bar{\sigma} = (\gamma_{sat} - \gamma_w) z \Rightarrow \gamma' \cdot z \Rightarrow (2)$$

Sub eq (2) in eq (1). $S = \gamma' z \cdot \tan \phi'$

If the sand deposit is shaken due to an earthquake (or) any other oscillatory load, extra pore pressure (u') develops. The shear strength equation becomes —

$$S = (\gamma' z - u') \cdot \tan \phi'$$

$$S = (\gamma' z - \gamma_w \cdot h) \cdot \tan \phi'$$

τ of sand decreases as the pore water increases, ultimately a stage is reached when the soil loses all its strength.

If $S = 0$,

$$\gamma' z - \gamma_w \cdot h = 0$$

$$\gamma' \cdot z = \gamma_w \cdot h$$

Critical Hydraulic Gradient $\leftarrow \left(\frac{h}{z} \right) = \frac{\gamma'}{\gamma_w}$

$$i_{crit} = \frac{(G-1) \cdot \gamma_w}{1+e} \times \frac{1}{\gamma_w}$$

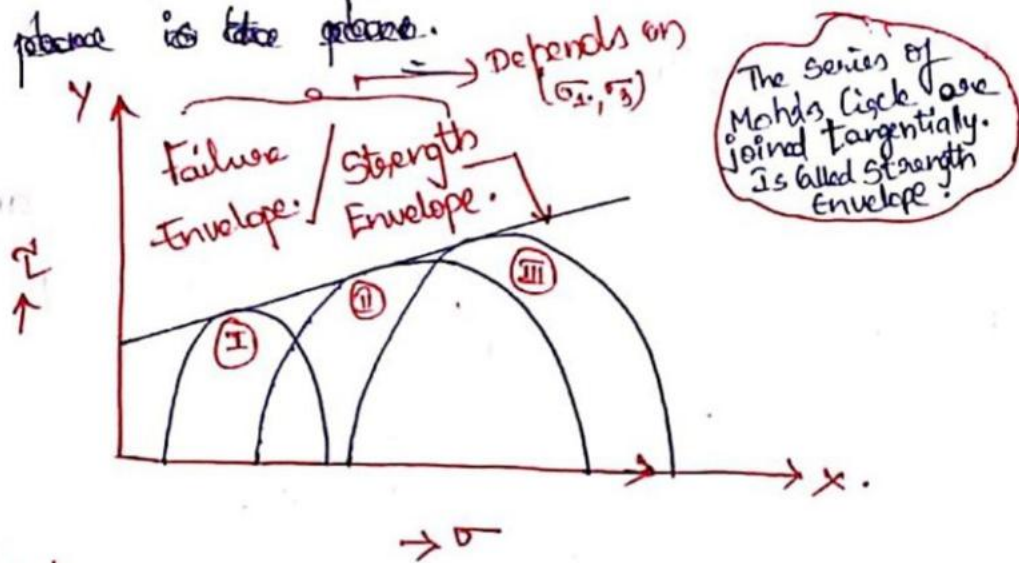
$$\therefore i_{cr} = \frac{G-1}{1+e}$$

When the sand loses its shear strength due to oscillatory motion, it is known as liquefaction of sand.

Strength Envelope:-

The soil strength is evaluated from the Shear Strength. It is done with the soil sample for estimating the shear strength.

When the Vane shear test is done using the soil sample, the soil sample will fail. The failed sample will determine its shear strength on the failure plane. The failure plane is the plane.



Failure Envelope:-

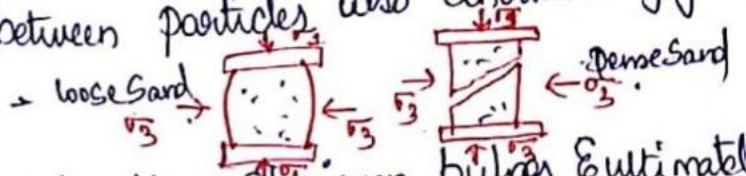
The failure plane is the plane on which the maximum shear stress exists.

The failure plane is measured in the lab through the application of principal stress by means of Triaxial Test laboratory.

With the principal stress parameters (σ_1, σ_3) ; the Mohr circle for the stress @ failure is drawn.

Shear Strength of Sand :-

It is in a cohesionless soils, such as sands & non-plastic silts, is mainly due to friction between particles. Dense sands, interlocking between particles also contribute significantly to the strength.



Here, In case of Loose Sand, the specimen bulges & ultimately fails by sliding simultaneously on numerous planes. The failure is known as "Plastic Failure".

In case of "Dense Sand", the specimen shows a clear failure plane & the failure is known as "Brittle Failure".

Factors affecting on τ of Sand :-

- 1.) Shape of particles ; 2.) Gradation ; 3.) Denseness ;
- 4.) Confining Pressure ; 5.) Deviator Stress ; 6.) Intermediate Principal Stress ;
- 7.) Loading ; 8.) Vibrations & Repeated loading ; 9.) Type of Minerals ;
- 10.) Capillary Moisture .

Shear Strength of Clay :-

These are a cohesive soils, depend upon the whether a soil is "normally Consolidated (or) over-Consolidated".

However, the ~~strain~~ σ - ϵ Curve of an over-consolidated clay is similar to that of a dense sand & that of a normally-consolidated clay is identical to that of a loose sand. The strain required to reach peak stress are generally greater in clay than in sand. The high strength @ the peak point in an over-consolidated clay is due to structural strength. Whereas in the dense-sand, it is mainly due to interlocking.

Loose Sands tend to increase in volume at large strains whereas normally Consolidated clays show no tendency to expand after a decrease in volume.

For ~~not~~ normally Consolidated clay, the failure envelope passes through the origin & hence $c' = 0$.

$$\therefore S = \bar{\sigma} \cdot \tan \phi'$$

Factors affecting on Clay:-

- 1.) Structure of Clay ; 2.) Clay Content ; 3.) Drainage Conditions ;
- 4.) Rate of Strain ; 5.) Intermediate principal stress ;
- 6.) Repeated loading ; 7.) Confining pressure ; 8.) Plasticity Index ;
- 9.) Stress History ; 10.) Disturbance.

Critical Void Ratio $:= (e_{cr})$. $\Rightarrow$ 

The void ratio at which there is no change in the soil with an increase ~~with~~ in strain, is known as Critical Void Ratio.

If the sand initially is @ ~~at~~ e_{cr} , there would be practically no change in volume with an increase in shear strain.

Dilatancy $:=$

It is the volume change observed in granular materials when they are subjected to shear deformations.

A sample of a material is called Dilative if its volume increases with increasing shear & Contractive if the volume decreases with increasing shear.

The Dilatancy is a common features of the "Soils & Sands".