TEEGALA KRISHNA REDDY ENGINEERING COLLEGE

(UGC-Autonomous)

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: GEOTECHNICAL ENGINEERING

SUBJECT CODE: 20CE5PC03

FACULTY NAME: B. SURESH

SYLLABUS:

UNIT – I

Introduction: Soil formation and structure – moisture content – Mass, volume relationships
Specific Gravity-Field density by core cutter and sand replacement methods-Relative density.

Index Properties of Soils: Grain size analysis – consistency limits and indices – I.S. Classification of soils.

Unit - I

Introduction of Soil mechanics

(*) **Introduction :- Soil :-** → Soil is defined as the solid particles formed by disintegration of Rock. These particles contains voids and the voids may be

(i) Air voids
(ii) water void
(iii) water and Air voids

→ **Types of Soil on the basis of nature of voids :-**

Solids + Air voids → Dry soil

Solids + water voids → fully Saturated soil

Solid + water voids + Air voids → Partially saturated soil.

(*) **Soil mechanics :-** → Dr. Karl Terzaghi is the father of Soil mechanics

→ Soil mechanics is the study of the engineering behaviour of soil when it is used either as a Construction material (or) as a foundation material.

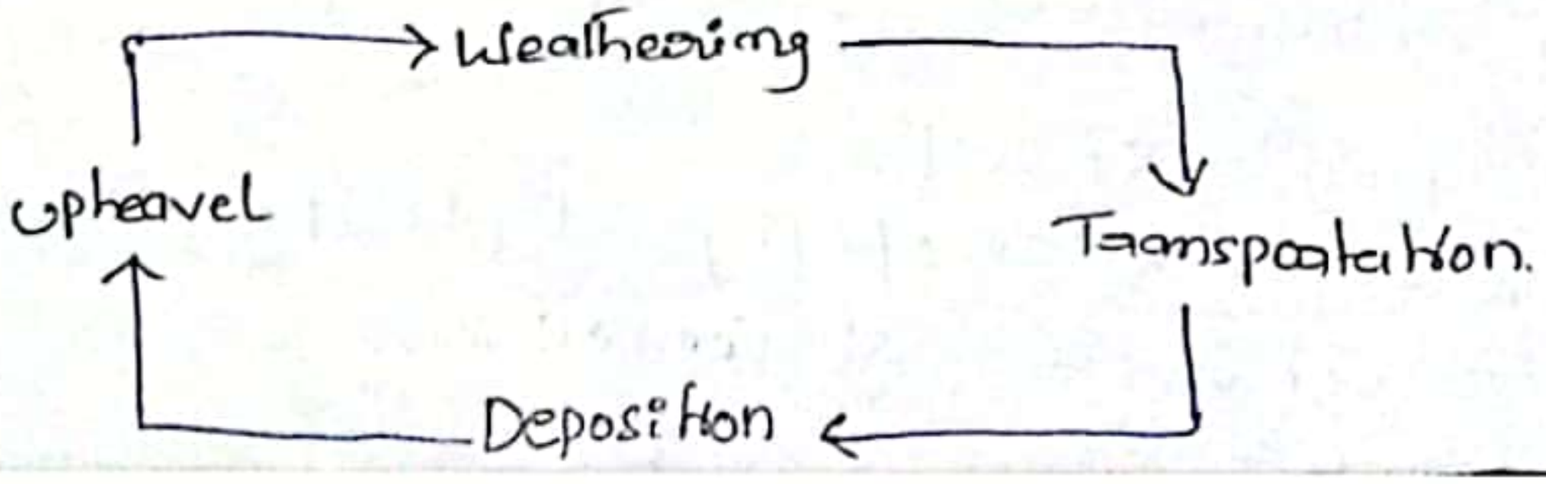
→ Soil is the application of Laws and hydraulics to the Solid particles in order to study their engineering.

(*) **Soil Formation :-**

→ As mentioned ~~above~~ Soils are formed by either

(A) physical disintegration

(B) chemical decomposition of Rocks.



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→ In this process, formation of geological cycle take place.

(A) Physical Disintegration :

→ Physical disintegration (or) mechanical weathering of rocks occurs due to the following physical process.

(1) Temperature changes :-

→ Different minerals of a rock have different coefficients of thermal expansion.

→ Unequal expansion and Contraction of these minerals occurs due to temperature changes.

→ When the stresses indeed 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) Abrasion :-

→ As water, wind and glaciers move over the surface of rocks, 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.

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→ 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.

(4) (2) 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, water combines with the rock minerals and results in the formation of a new chemical compound
→ The chemical reaction causes a change in volume and decomposition of rock into small particles.

② 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.

③ Oxidation :-

→ Oxidation occurs when oxygen ions combine with minerals in rocks
→ oxidation results in decomposition of rocks oxidation of rocks is some what similar to rusting of steel.

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④ 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.

⑤ 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.

(*) Soil structure. :
→ the geometrical arrangement of soil particles with respect to one another is known as soil structure.
→ the soils in nature have different structures depending upon the particle size and the mode of formation.
→ the following types of structures are usually found.
→ the first two ~~times~~ types are for coarse grained soils
(1) single-grained structure
(2) Honey-comb structure.
→ three and four types for clays.
(3) Flocculated structure.
(4) dispersed structure.
→ five and six types for are mixed soils.
(5) coarse grained skeleton.
(6) clay matrix structure.

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① Single-grained structure :-

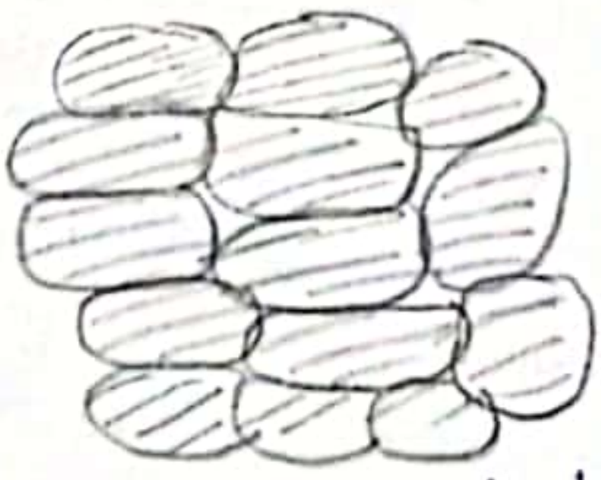
→ Cohesionless soils such as gravel and sand are composed of bulky grains in which the gravitational forces are more predominant the surface forces.

→ when deposition of these soils occurs the particely settle under gravitational forces and take an equilibrium position as shown in fig.

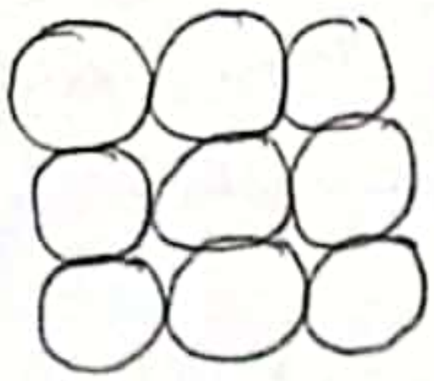
→ Each particle is in contact with those surrounding it. the soil structure so formed is known as single grained structure.

→ the arrangement is somewhat similar to the stacking of oranges on a grocery's counter or to that of marble pieces at a toy's shop.

→ Depending upon the relative position of the particles, the soil may have a loose structure (or) a dence structer. Shows spherical particles in the loosest and those in the densest conditions.



Single grained structure



Loose



Dense

→ the void ratio is 0.90 and that for the densest state is 0.35

→ In actual sand deposits, as the particles are not exactly spherical the void ratio b/w the loosest and densest condition varies b/w 0.90 & 0.35

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(1) Honey comb Structure :-

→ It is possible for fine Sands (or) Silts to get deposited such that the particles when settling develop a particle to particle contact that bridges over large voids in the soil mass

→ the particles wedge b/w one another into a stable condition and form a skeleton like on each to carry the weight of the overlying material.

→ the structure so formed is known as honey-comb structure.

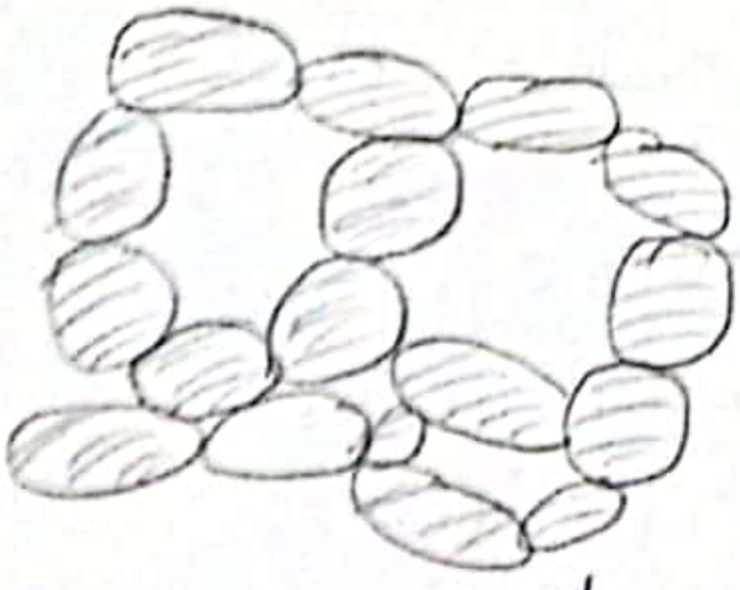
the honey comb structure usually develops when the particle size is b/w 0.002 mm and 0.02 mm

→ Honey comb structure occurs in soils having small granular particles which have cohesion because of their fineness.

→ the particles are held in position by mutual attraction due to cohesion. the particles however do not possess plasticity characteristics associated with clayey soils.

→ Soils in honey comb structure are loose. They can support loads only under static conditions. Under vibrations and shocks, the structure collapses and large deformations take place.

→ In nature, honey comb structure usually occurs in small pockets and can be easily detected.



honey-comb structure.

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(3) Flocculated structure :-

→ Flocculated structure occurs in clays the clay particles have large surface area and therefore the electrical forces are important in such soils.

→ the clay particles have a negative charge on the surface and a positive charge on the edges.

→ Interparticle contact develops b/w the positively charged edges and the negatively charged faces. This results in a flocculated structure.

→ Flocculent structure is formed when there is a net attractive force b/w particles.



flocculated structure.

→ when clay particles settle in water deposits formed have a flocculated structure.

→ the degree of flocculation of a clay deposit depends upon the type and concentration of clay particles and the presence of salts in water.

→ clays setting out in a salt water solution have a more flocculent structure than those ~~setti~~ settling out in a fresh water solution. Salt water acts as an electrolyte and reduces the repulsive forces b/w the particles.

→ Soils with a flocculent structure are light in weight and have a high void ratio and water content however these soils are quite strong and can resist external forces because of a strong bond due to attraction b/w particles.

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④ Dispersed Structure :-

→ Dispersed structure develops in clays that have been Reworked (or) remolded the particles develop more or less a parallel orientation.

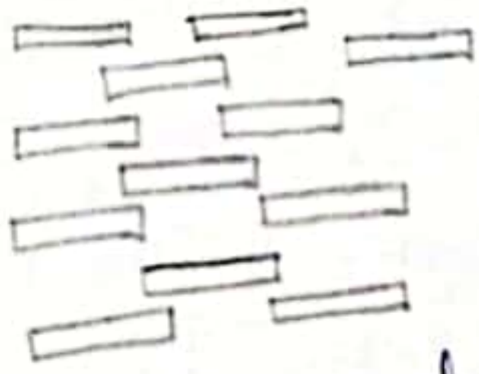
→ Clay deposits with a flocculent structure when transported to other places by nature or man get remoulded.

→ Remoulding converts the edge-to-face orientation to face-to-face orientation.

→ The dispersed structure is formed in nature when there is a net repulsive force b/w particles.

→ The soils in dispersed structure generally have a low shear strength, high Compressibility and low permeability.

→ Remoulding causes a loss of strength in a cohesive soil with the passage of time however, the soil may regain some of its lost strength.



Dispersed structure.

⑤ Coase grained Skeleton :-

→ A coarse grained skelton is a composite structure which is formed when the soil contains particles of different types.

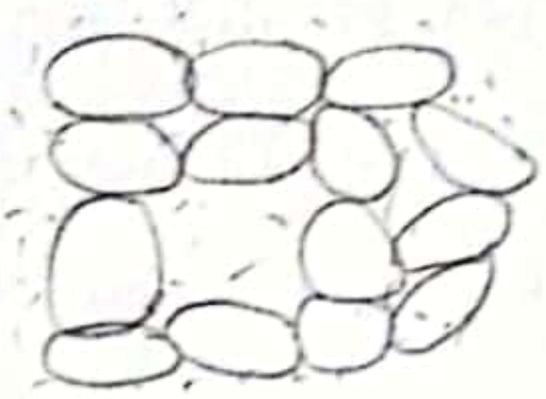
→ when the amount of bulky cohesionless particles is large compared with that of fine grained clayey particles. the bulky grains are in particle-to-particle contact.

→ these particles form a frame work or skelton the space b/w the bulky grains is occupied by clayey particles, known as binders. in nature. the bulky grains are deposited first during sedimentation

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-and the binder is subsequently deposited.



Coarse grained structh

⑥ clay matrix structure :-

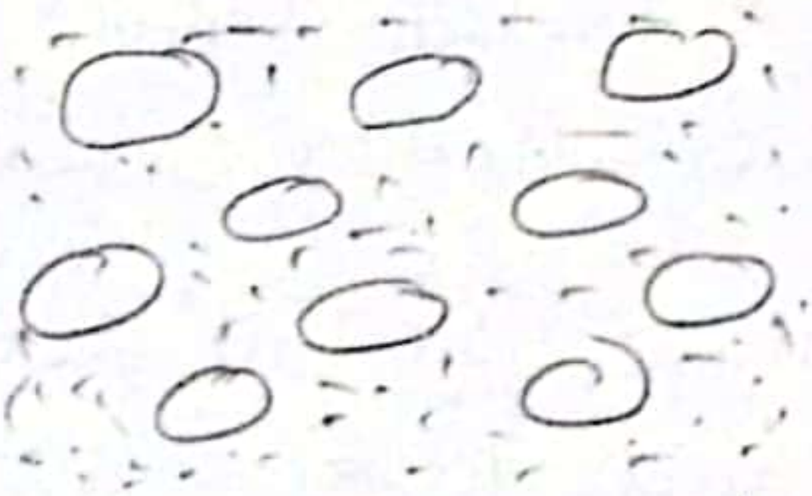
→ clay-matrix structure is also a composite structure formed by soils of different types

→ however in this case the amount of clay particles is very large as compared with bulky coarse grained particles

→ the clay forms a matrix in which bulky grains appear floating without touching one another.

→ the soils with a clay matrix structure have almost the same properties as clay.

→ their behaviour is similar to that of an ordinary clay deposit. However they more stable or disturbance have very little effect on the soil formation with a clay matrix structure.



Clay matrix structure.

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(*) Teaminology of Diffeaent types of Soils!-

→ A geolechnical engineeg should be well veaysed wiilh lhe nomenclaleae and teaminology of diffeaent types Soils

(1) Bentonite :→ 9l is a type of clay wiilh a veay hegh peaconlage of clay mineaal montmoaillonite. 9l is a highly plastic clay gesulting fuom lhe decompasition of volcanic ash. 9l is highly waleg abscabent and has hegh Shainkage and Swelling chaaacleristics.

(2) Block cotton soil!-

→ it is a gesidual soil containing a high peacentage of lhe clay mineaal montmoaillonite. 9t has veay Low beaaing Capacity and high Swelling and Shainkage paopeilies.

(3) Bouldeas!- → Bouldeas aae sock fagments of laage size, moae lham 300 mm in size.

(4) Calcaaeous Soils!→ These soils contain a laage quantity of Calcium Caabonate. Such soils effeavesee cohen tested wilh weak hydaochloaic acid.

(5) Caliche!- → it is a type of soil which Contains geavel. Sand & silt. lhe paalicles aae cemented by Calcium Caibonial.

(6) clay 9t!- it Consists of micaoscopic and Sub micaoscopic paalicles dealved fuom lhe chemical decomposition of socks.

→ It Containes a Laage quantity of clay mineaals.

→ It Can be made plastic by adjusting lhe waleg Content.

→ It exhibits Considerable Stiength when day. clay is a fine gained Soil. it is a ~~of~~ Choesive soil. ~~it is a~~.

lhe paalicle size is leh lhan 0.002 mm

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→ Organic clay contains finely divided organic matter and is usually dark grey or black in colour.

→ It has a conspicuous odour. organic clay is highly compressible and its strength is very high when dry.

(7) Cobbles :-

→ Cobbles are large size particles in the range of 80mm to 300mm.

(8) Diatomaceous earth :-

→ Diatoms are minute unicellular marine organism. Diatomaceous earth is a fine, light grey. Soft sedimentary deposit of the siliceous remains of skeltons of diatoms.

(9) Dispersive clays :-

→ These are special type of clays which deflocculate in still water. Such soils erode if exposed to low velocity water.

→ Susceptibility to dispersion depends upon the cations in the soil pore water.

(10) Dune Sands :-

→ These are wind transported soils. These are composed of relatively uniform particles of fine to medium sand.

(11) expansive clays :-

→ These are prone to large volume changes as the water content is changed. These soil contain the mineral montmorillonite.

(12) Fills :- → All man-made deposits of soil and waste minerals are called fills. These are the soil embankments raised above the ground surface. Engineering property of fills depend upon the type of soil its water content and the degree of compaction.

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(13) Gravel :- → Gravel is a type of coarse grained soil the particle size ranges from 4.75 mm to 80 mm it is a cohesionless material.

(14) Hardpans :-

→ Hardpans are types of soils that offer great resistance to the penetration of drilling tools during soil exploration the soils are designated hardpans regardless of their particle size

→ These are generally dense, well-graded, cohesive aggregates of mineral particles. Hardpans do not disintegrate when submerged in water.

(15) Humus :-

→ it is a dark brown, organic amorphous earth of the topsoil it consists of partly decomposed vegetal matter it is not suitable for engineering works.

(16) Kankar :- → it is an impure form of lime stone. it contains calcium carbonate mixed with some silicious material.

(17) Laterites :- → laterites are residual soils formed in tropical regions. Laterites are very soft when freshly cut but become hard after long exposure.

Hardness is due to cementing action of iron oxide and aluminum oxide. These soils are also called lateritic soils

(18) Loam :- → it is a mixture of sand, silt and clay. the loam is generally used in agronomy. the soil is well suited to tilling operations.

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(19) Loess :-

→ it is a wind blown deposit of silt. it is generally of Uniform gradation, with the particle b/w 0.01 to 0.05 mm. it consists of quartz & feldspare particles cemented with Calcium Carbonate or iron oxide.

(20) marl :-

→ it is a stiff, marine calcareous clay of greenish colour.

(21) moorum :-

→ the word moorum is derived from a Tamil word, meaning powdered rock. it consists of small pieces of disintegrated rock or shale, with or without boulders.

(22) muck :-

→ it denotes a mixture of fine soil particles and highly decomposed organic matter. it is black in colour and of extremely soft consi

(23) peat :- it is an organic soil having fibrous aggregates of microscopic and microscopic particles. formed from vegetal matter under conditions of excess moisture, such as in swamps. it is highly compressible and not suitable for foundations.

(24) Sand :- it is a coarse grained soil, having particle size b/w 0.075mm to 4.75mm. the particle are visible to naked eye. the soil is cohesionless and previous.

(25) silt :-

→ it is a fine grained soil, with particle size b/w 0.002mm & 0.075mm. the particles are not visible to naked eyes.

→ Inorganic silt consists of bulky equidimensional grains of quartz. it has little or no plasticity and cohesionless.

→ Organic silt contains an admixture of organic matter it is a plastic soil & is cohesive.

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(26) Till :- → it is an unstratified deposit formed by melting of a glacier. The deposit consists of particles of different sizes, ranging from boulders to clay. The soil is generally well graded. it can be easily densified by compaction till is also known as boulder-clay.

(27) Top soils :-

→ Top soils are surface soils that support plants. They contain a large quantity of organic matter and are not suitable for foundation.

(28) Tuff :-

→ it is a fine grained soil composed of very small particles ejected from volcanoes during its explosion and deposited by wind or water.

(29) Tundra :-

→ it is a mat of peat and shrubby vegetation that covers clayey subsoil in arctic regions. The deeper layers are permanently frozen and are called permafrost the surface deposit is the active layer which alternately freezes and thaws.

(30) Varved clays :-

→ These are sedimentary deposits consisting of alternate thin layers of silt and clay the thickness of each layer seldom exceeds 1cm. These clays are the results of deposition in lakes during periods of alternately high and low waters.

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(4) Cohesive Soils :-

→ Soils in which the adsorbed water and particle attraction are such that it deforms plastically at varying water contents are known as cohesive soils/clays

→ This cohesive property is due to presence of clay minerals in soils. Therefore the term cohesive soil is used synonymously for clayey soils.

(5) Cohesionless Soils :-

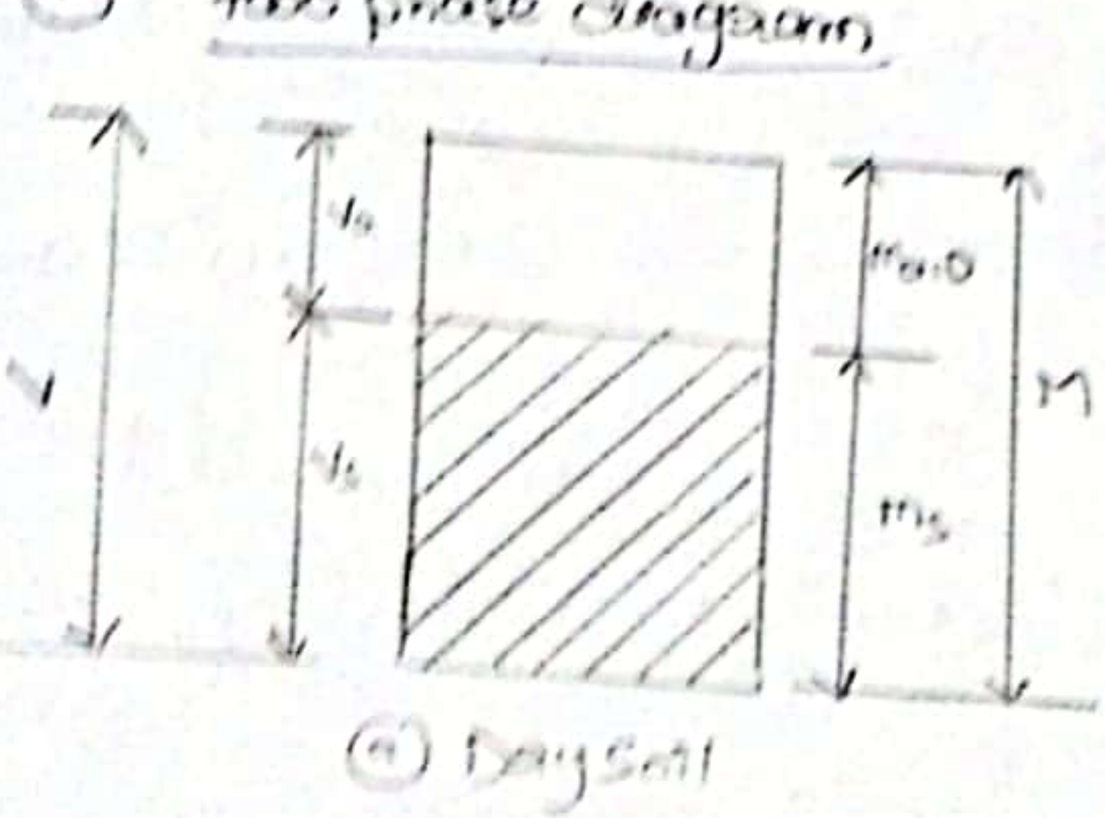
→ The soils composed of bulky grains are cohesionless regardless of the fineness of the particles. The rock flour is cohesionless even when it has the particle size smaller than 2 μ size. non plastic silts and coarse grained soils are cohesionless.

→ many soils are mixture of bulky grains and clay minerals and exhibit some degree of plasticity with varying water content. Such soils are termed cohesive if the plasticity effect is significant. otherwise cohesionless.

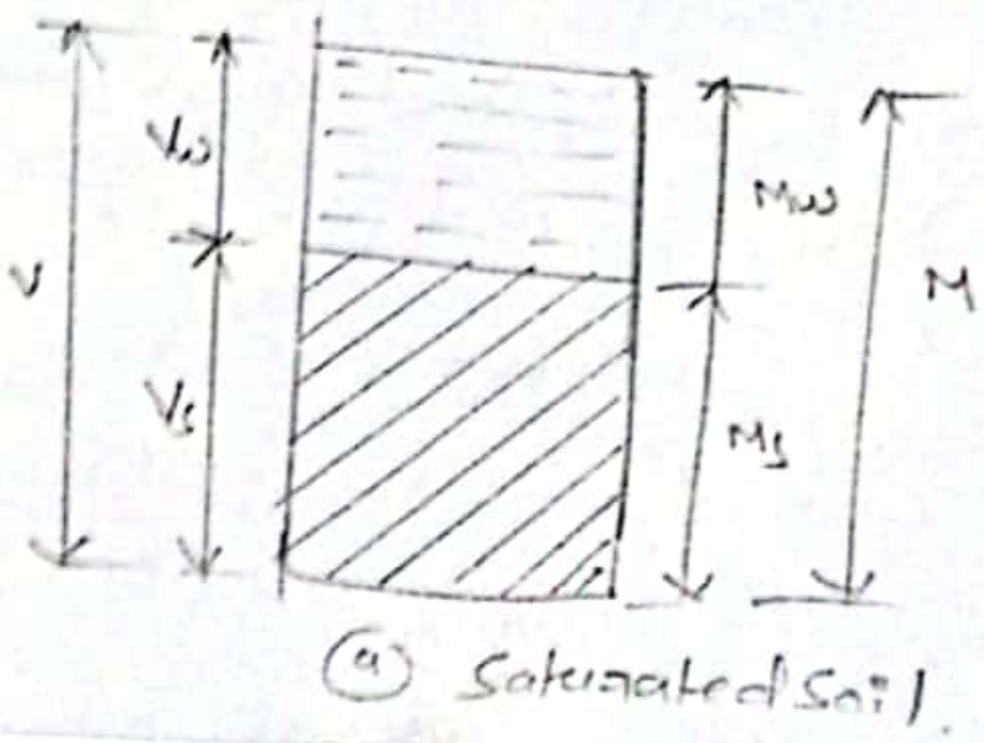
(6) properties of soils :-

→ ① 2 phase diagram
② three phase diagram.

① two phase diagram



(a) Dry Soil



(a) Saturated Soil

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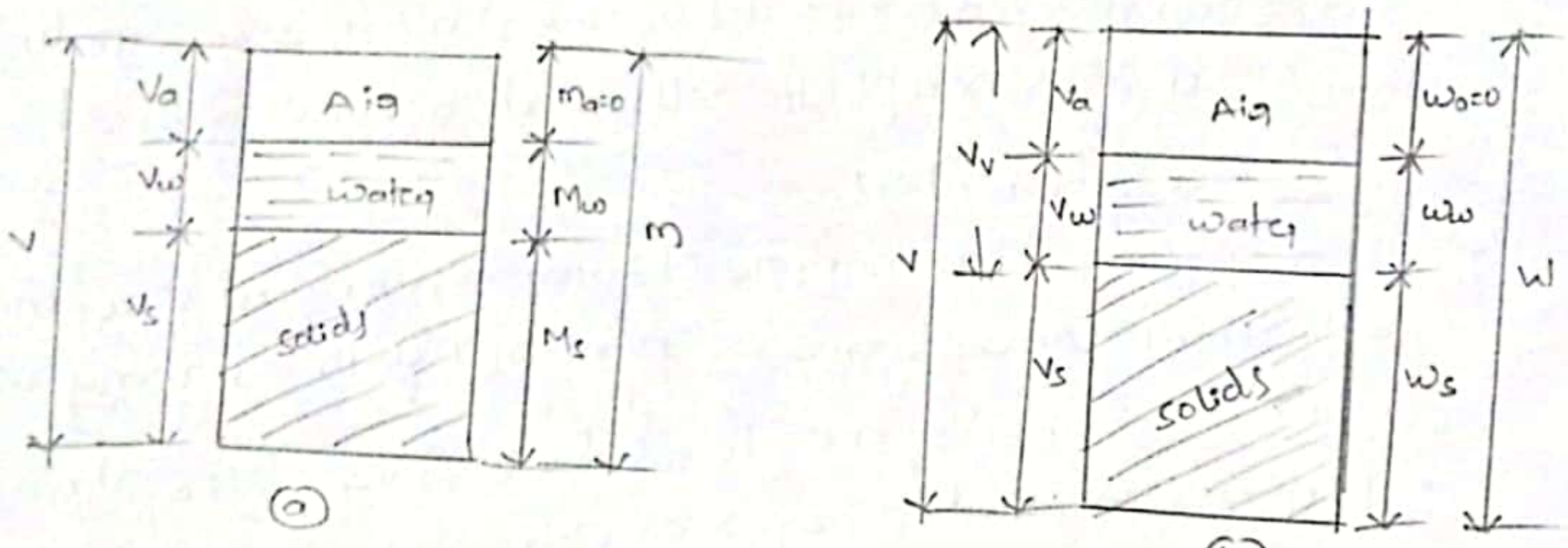
→ the soil is a it becomes a two-phase system in the following two case① when the soil is absolutely day the water phase disappears

case② :- when the soil is fully saturated there is no air phase it is the relative proportion of the three constituents and their interaction that governs the behaviour and properties of soils.

→ the phase diagram is a simple diagrammetic representation of a real soil.

→ it is extremely useful for studying the various terms used in soil engineering and their interrelationship.

(2) Three phase diagram :



→ In a 3-phase diagram it is conventional to write volumes on the left side and the mass on the right side the total volume of a given soil mass in designated as V.

→ It is equal to the sum of the volume of solids the volume of water (V_w) and the volume of air (V_a) the volume of voids (V_v) is equal to the sum of the volumes of water and air

→ the total mass of the soil mass is represented as M the mass of air (m_a) is very small and is neglected.

→ the total mass of the soil is equal to the mass of solids (M_s) and the mass of water (M_w).

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(A) Volumetric Relationships :-

(1) Void Ratio (e) :-

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

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

→ The void ratio is expressed as a decimal such as 0.4, 0.5 etc for coarse grained soils, the void ratio is generally smaller than that for fine grained soils for some soils it may have a value even greater than unity.

(2) porosity (n) :-

→ It is defined as the ratio of the volume of voids to the total volume.

thus $n = \frac{V_v}{V}$ — (1)

→ porosity is generally expressed as percentage. However in equations. it is used as a ratio

→ for example a porosity of 50%. will be used as 0.5 in equations.

→ the porosity of a soil cannot exceed 100%. as it should mean V_v is greater than V which is absurd. In fact it will have a much smaller value porosity is also known as percentage voids.

→ An inter relationship can be found b/w the void ratio & the porosity as under.

from equation (1) $\frac{1}{n} = \frac{V}{V_v} = \frac{V_v + V_s}{V_v}$

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

$= \frac{1}{n} = 1 + \frac{1}{e} = \frac{e+1}{e} = \boxed{\frac{1+e}{e}}$ — (11)

$\boxed{n = \frac{e}{1+e}}$ — (11)

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Also from eqn (2) $= \frac{1+e}{e}$ $= \frac{1}{e} + \frac{e}{e}$ $= \frac{1}{e} + 1$

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

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

$$\boxed{e = \frac{n}{1-n}} \quad \text{--- (3)}$$

→ equation (2) & (3) the porosity should be expressed as a ratio.

(3) Degree of Saturation (S) :-

→ the degree of saturation (S) is the ratio of the volume of water to the volume of voids.

$$\boxed{S = \frac{V_w}{V_v}}$$

→ the degree of saturation is generally expressed as a percentage it is equal to zero when the soil is absolutely dry and 100% when the soil is fully saturated.

In expressions, the degree of saturation is used as a decimal.

(4) percentage Air voids (n_a) :-

→ it is the ratio of the volume of air to the total volume

thus $\boxed{n_a = \frac{V_a}{V}}$

(5) Air Content (a_c) :-

→ Air content is defined as the ratio of the volume of air to the volume of voids.

thus $\boxed{a_c = \frac{V_a}{V_v}}$

→ Air content is usually expressed as a percentage.

→ Both air content and the percentage air voids are zero when the soil is saturated ($V_a = 0$)

from equation $n_a = \frac{V_a}{V} = \frac{V_a}{V_v} \times \frac{V_v}{V} = \boxed{n_a = n a_c}$

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(A) **Water Content :-**

→ The water content (w) is defined as the ratio of the mass of water to the mass of solids.

$$w = \frac{M_w}{M_s}$$

→ The water content is also known as the moisture content (m). it is expressed as a percentage but used as a decimal in computation.

(B) **Volume - mass Relationships :-**

→ The volume-mass relationship are in terms of mass density the mass of soil per unit volume is known as mass density. In soil engineering the following 5 different mass densities are used.

① **Bulk mass Density :-**

→ The bulk mass density (ρ) is defined as the total mass (M) per unit total volume (V).

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

→ The bulk mass density is also known as the wet mass density or simply bulk density or density. it is expressed as

kg/m^3

② **Dry mass Density :-**

→ The dry mass density (ρ_d) is defined as the mass of solids per unit total volume.

$$\rho_d = \frac{M_s}{V}$$

→ As the soil may shrink during drying the mass density may not be equal to the bulk mass density of the soil in the dried condition

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(3) Saturated mass Density :

→ The Saturated mass density (ρ_{sat}) is the bulk mass density of the soil when it is fully Saturated thus

$$\rho_{sat} = \frac{m_{sat}}{V}$$

(4) Submerged mass Density :

→ When the soil exists below water it is in a Submerged Condition when a volume V of soil is Submerged in water it displaces an equal volume of water thus the net mass of soil when Submerged is reduced.

→ The Submerged mass density (ρ') of the soil is defined as the Submerged mass per unit of total volume.

$$\rho' = \frac{m_{sub}}{V}$$

(5) mass Density of Solids :

→ The mass density of Solids (ρ_s) is equal to the ratio of the mass of Solids to the Volume of Solids.

$$\rho_s = \frac{M_s}{V_s}$$

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(*) Index properties :-

→ These are the properties useful for classification and Identification of soils, also give rough estimation of Engineering properties of soils.

Ex :- particle size distribution and relative density for coarse grained soils and consistency limits for fine grained soils.

→ Important Index property of fine grained soils.

1. Consistency limits.
2. Liquid limit
3. Plastic limit
4. Shrinkage limit
5. plasticity, liquidity and consistency indexes.
6. Flow index
7. Toughness index

(1) Consistency limits :-

→ The consistency of a fine grained soil is the physical state in which it exists.

→ It is used to denote the degree of firmness of a soil. Consistency of a soil is indicated by such terms as soft firm (or) hard.

→ Swedish agriculture engineer Atterberg mentioned that a fine grained soil can exist in four states namely, liquid, plastic, semi-solid (or) solid state. The water contents at which the soil changes from one state to the other are known as consistency limits (or) Atterberg's limits.

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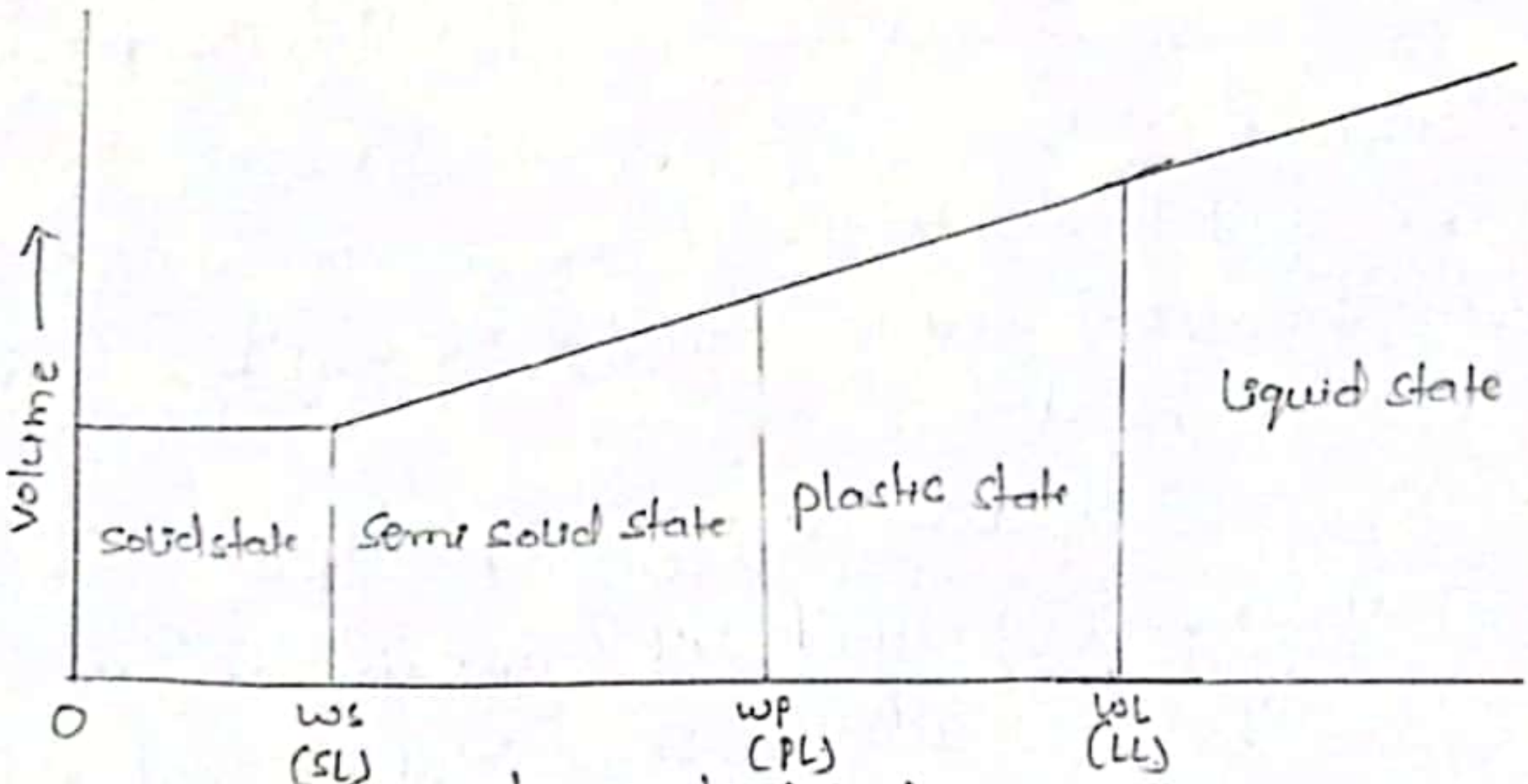
→ The water content alone is not an adequate index property of a soil. At the same water content, one soil may be relatively soft whereas another soil may be hard.

→ However the soils with the same consistency limits behave somewhat in a similar manner. These consistency limits are very important index properties of fine grained soils.

→ A soil containing high water content is in a liquid state. It offers no shearing resistance and can flow like liquids. it has no resistance to shear deformation and Therefore the shear strength is equal to zero.

→ As the water content is reduced the soil becomes stiffer and starts developing resistance to shear deformation.

→ At some particular water content, the soil becomes plastic. The water content at which the soil changes from the liquid state to the plastic state is known as liquid limit (LL, w_L). in other words, the liquid limit is the water content at which the soil ceases to be liquid..



Different states of soil.

→ The soil in the plastic state can be moulded into various shapes. As the water content is reduced the plasticity

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- of the soil decreases.

→ Ultimately the soil passes from the plastic state to the semisolid state when it stops behaving as a plastic.

→ It cracks when moulded the water content at which the soil becomes semi solid is known as the plastic limit (PL, wp) in other words the plastic limit is the water content at which the soil just fails to behave plastically.

→ the numerical difference b/w the liquid limit & the plastic limit is known as plasticity Index (P.I, Ip)

they $\boxed{PI = LL - PL}$

→ The soil remains plastic when the water content is b/w the liquid limit & the plastic limit the plasticity index is an important index property of fine grained soils.

→ when the water content is reduced below the plastic limit the soil attains a semi solid state. The soil cracks when moulded.

→ In the semi solid state the volume of the soil decrease with a decrease in water content till a stage is reached when further reduction of the water content does not cause any reduction in the volume of the soil

→ the soil is solid to have reached a solid state (Insolid, no appreciable change in volume is observed with a change in water content).

→ the water content at which the soil changes from the semi solid state to the solid state is know as the Shrinkage Limit (SL, ws)

→ Below the Shrinkage limit the soil does not remain saturated. Air enters the voids of the soil however because of capillary tension developed the volume of the soil does not change.

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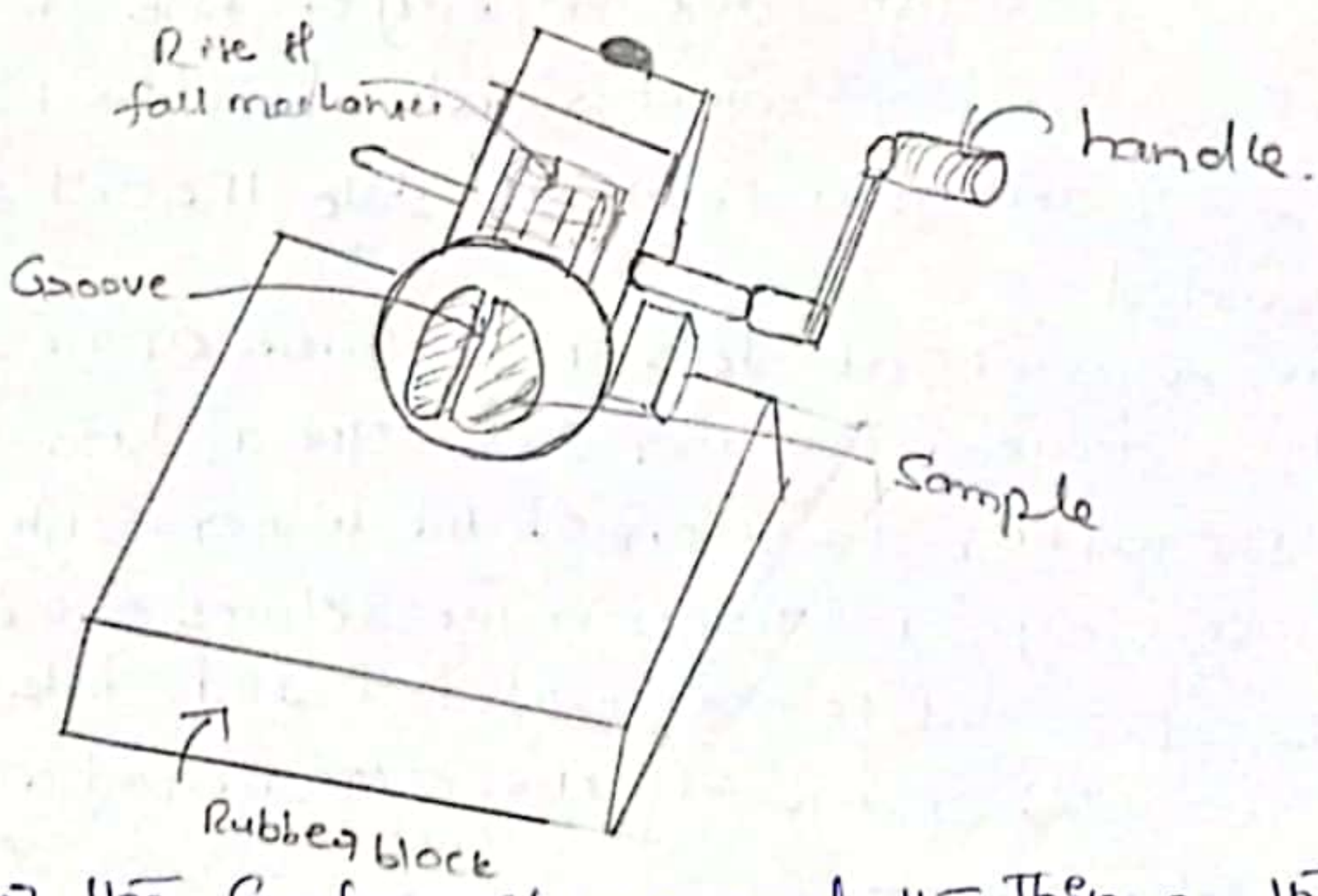
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→ Thus the shrinkage limit is the water content at which the soil stops shrinking further and attaines a constant volume. The shrinkage limit may also be defined as the lowest water content at which the soil is fully saturated.

② Liquid limit :-

→ The liquid limit is the water content at which the soil changes from the liquid state to the plastic state. At the liquid limit, the clay is practically like a liquid but possesses a small shearing strength.

→ ~~the~~ the shearing strength at that stage is the smallest value that can be measured in the laboratory the liquid limit of soil depends upon the clay mineral present.



→ The stronger the surface charge and the thinner the particle the greater will be the amount of adsorbed water and therefore the higher will be the liquid limit.

→ The liquid limit is determined in the laboratory either by casagrande's apparatus by by cone penetration method

→ The device used in casagrande's method consists of a brass cup which drops through a height of 1cm on a hard base when operated by the handle

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→ The device is operated by turning the handle which raises the cup and lets it drop on the rubber base. The height of drop is adjusted with the help of adjusting screws.

→ About 120 gm of an air-dried sample passing through 425 μ IS sieve is taken in a dish and mixed with distilled water to form a uniform paste

→ A portion of this paste is placed in the cup of the liquid limit device, and the surface is smoothened and a levelled with a spatula to a maximum depth of 1cm.

→ A groove is cut through the sample along the symmetrical axis of the cup, preferably in one stroke using a standard grooving tool. IS 2720 part V recommends two types of grooving tools. ① Casagrande tool ② ASTM tool. The Casagrande tool cuts a groove of width 2 mm at the bottom, 11mm at the top & 8 mm deep.

→ The ASTM tool cuts a groove of width 2 mm at the bottom, 13.6 mm at the top and 10mm deep.

→ The Casagrande tool is recommended for normal fine grained soils, whereas the ASTM tool is recommended for sandy, fine grained soils in which the Casagrande tool tends to tear the soil in the groove.

→ After the soil pat has been cut by a proper grooving tool the handle is turned at a rate of 2 revolutions per second until the two parts of the soil sample come in to contact at the bottom of the groove along a distance of 12mm.

→ The groove should close by a flow of the soil, and not by slippage b/w the soil and the cup. When the groove closes by a flow it indicates the failure of slopes formed on the two sides of the groove.

→ The soil in the cup is again mixed the test is repeated until two consecutive tests give the same no. of blows.

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→ About 15 gm of soil near the closed groove is taken for water content determination.
→ The soil in the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding more water.
→ The soil sample is again taken in the cup of the Liquid limit device and the test is repeated the Liquid limit is the water content at which the soil is sufficiently fluid to flow when the device is given 25 blows.
→ As it is difficult to get exactly 25 blows for the sample to flow, the test is conducted at different water contents so as to get blows in the range of 10 to 40

(*) Plastic limit :-

→ plastic limit is the water content below which the soil stops behaving as a plastic material it begins to crumble when rolled into a thread of soil of 3mm diameter. At this water content the soil loses its plasticity and passes to a semi solid state.
→ for determination of the plastic limit of a soil it is air dried and sieved through a 425 μ IS sieve. About 30 gm of soil is taken in an evaporating dish it is mixed thoroughly with distilled water till it becomes plastic and can be easily moulded with fingers.
→ About 10gm of the plastic soil mass is taken in one hand and a ball is formed. The ball is rolled with fingers on a glass plate to form a soil thread of uniform diameter.
→ The rate of rolling is kept about 80 to 90 strokes per minute.
→ if the diameter of thread becomes smaller than 3 mm without cracle formation it shows that the water content is more than the plastic limit.

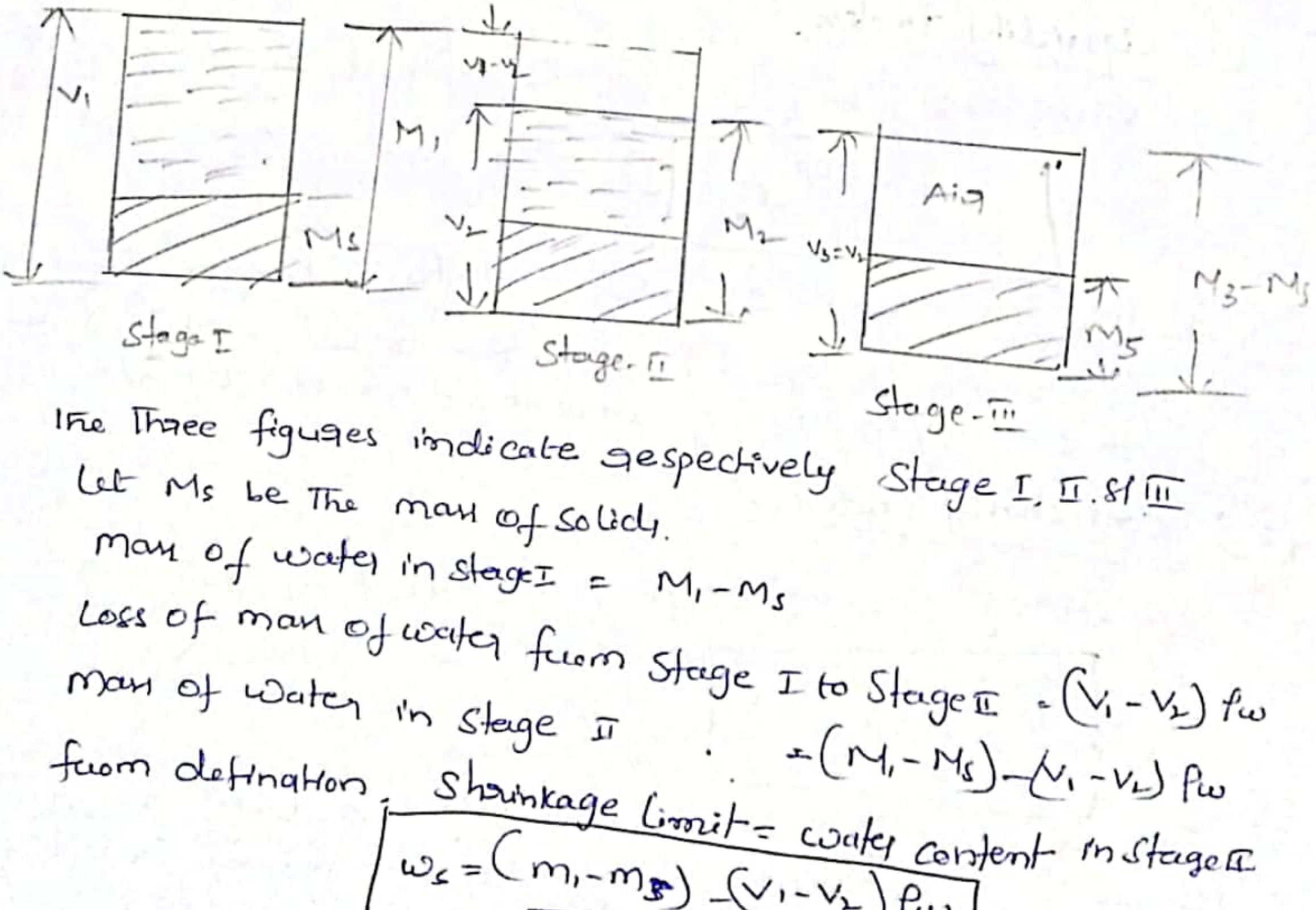
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→ The soil is kneaded further this results in the reduction of the water content, as some water is evaporated due to the heat of the hand. the soil is re-rolled and the ~~pro~~ procedure repeated till the thread crumbles.

(x) **Shrinkage limit:**

→ Shrinkage limit is the smallest water content at which the soil is ~~saturated~~ saturated. it is also defined as the maximum water content at which a reduction of water content will not cause a decrease in the volume of the soil mass.

→ ~~fig~~ (a) the block diagram of a soil sample when it is fully saturated and has the water content greater than the expected shrinkage limit. fig (b) shows the sample at shrinkage limit fig (c) depicts the condition when the soil sample has been ovendried. the total volume V_3 in fig (c) is the same as the total volume V_2 in fig (b).



The three figures indicate respectively Stage I, II. & III

Let Ms be the mass of solids.

mass of water in stage I = $M_1 - M_s$

Loss of mass of water from Stage I to Stage II = $(V_1 - V_2)\rho_w$

mass of water in stage II = $(M_1 - M_s) - (V_1 - V_2)\rho_w$

from definition, Shrinkage limit = water content in stage II

$$\omega_s = \frac{(m_1 - m_s) - (V_1 - V_2)\rho_w}{M_s}$$

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$$\boxed{w_s = w_1 - \frac{(V_1 - V_2)}{M_s} P_w}$$

where w_1 represents the water content in stage I.

→ for Determination of the shrinkage limit in the Laboratory about 50gm of soil passing ~~0.0~~ a 425 M sieve is taken and mixed with distilled water to make a creamy paste the water content (w_1) of the soil is kept greater than the liquid limit.

(*) Plasticity, Liquidity And Consistency indexes :-

(1) Plasticity Index :-

→ Plasticity Index (Ip or PI) is the range of water content over which the soil remains in the plastic state it is equal to the difference b/w the liquid (w_l) and the plastic limit (w_p)

thus $\boxed{I_p = w_l - w_p}$

(2) Liquidity Index :

→ Liquidity Index (I_l or L_I) is defined as

$$\boxed{I_l = \frac{w - w_p}{I_p} \times 100}$$

→ The Liquidity Index of a soil indicates the nearness of its water content to its liquid limit the liquidity index is 100% and it behaves as a liquid.

(3) Consistency Index :-

→ Consistency Index (I_c, CI) is defined as

$$\boxed{I_c = \frac{w_l - w}{I_p} \times 100}$$

→ The consistency index indicates the consistency of a soil it shows the nearness of the water content of the soil to its plastic limit. A soil with a consistency index of zero:

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(*) Flow Index :-

→ Flow index (I_f) is the slope of the flow curve obtained by the no of blows and the water content in Casagrande's method of determination of the liquid limit

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

N_1 = no of blows required at water content of w_1

N_2 = no of blow required at water content of w_2

(*) Toughness Index :

→ Toughness index (I_t) of a soil is defined as the ratio of the plasticity Index (I_p) to the flow index (I_f)

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

(*) Grain Size Analysis :-

→ The ~~re~~ Grain Size Analysis also known as mechanical ~~Size~~ Analysis is a method of separation of soils into different fractions based on the particle size.

→ it expresses quantitively the proportions by mass of various sizes of particles present in a soil it is shown graphically on a particle size distribution curve.

→ The Grain size analysis is done in two stage.

(1) Sieve Analysis.

(2) Sedimentation Analysis.

→ The first analysis is meant for coarse grained soil (particle size greater than 75μ) which can easily pass through

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a set of sieves

→ The second analysis is used for fine grained soils (size smaller than 75 micron) sedimentation analysis is also known as wet analysis.

→ As a soil mass may contains the particles of both types of soils a combined analysis comprising both sieve analysis & sedimentation analysis may be required for such soils.

→ particle size smaller than 0.2 microns cannot be determined by the sedimentation method.

→ These can be determined by an electron microscopic (or) by x ray diffraction techniques. However such analyses is of little particle practical Importance in Soil engineering.

① Sieve Analysis :-

→ The soil is sieved through a set of sieves Sieves are generally made of Spun brass and phosphor bronz (stainless steel) sieve cloth

→ According to IS : 1498 - 1970. The sieves are designated by the size of square opening, in mm (or) microns (1 micron = 10^{-6} m = 10^{-3} mm)

→ Sieves of various sizes ranging from 80mm to 75 microns are available. The diameter of the sieve is generally b/w 15 to 20 cm.

→ As mentioned before, the sieve analysis is done for coarse-grained soils.

→ The coarse-grained soils can be further subdivided into gravel fraction (size > 4.75mm) and sand fraction (75M < size < 4.75mm) where Greek letter μ is used to represent microns.

→ A set of coarse sieves, consisting of the sieves of size 80mm, 40mm, 20mm, 10mm & 4.75mm is required for the gravel fraction.

→ The second set of sieves, consisting of the sieves of size 2mm, 1mm, 600μ, 425μ, 212μ, 150μ, & 75μ is used for sand fraction.

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→ All the Sieves may not be required for a particular soil the selection of the required no. of sieves is done to obtain a good particle size distribution curve.

→ the Sieves are stacked one over the other with decreasy size from the top to the bottom.

→ they the Sieve of the Largest opening is kept at the top A lid(cv) Cover is placed at the top of the Largest Sieve A receiver known as pan, which has no opening is placed at the bottom of the Smallest Sieve.

(a) Dry sieve Analysis :-

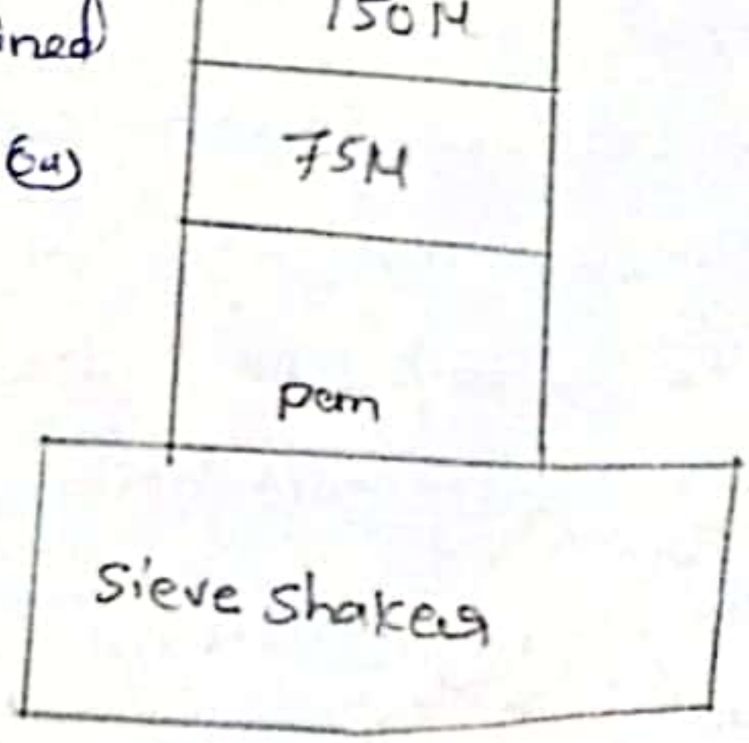
→ The Soil Sample is taken in Suitable quantity as given table the Larger the particle size, the greater is the quantity of soil required.

→ The soil should be oven dry it should not Contain any Lump if necessary it should be pulverized if the Soil Contains Organic matter it can be taken air dry instead of oven dry.

→ the Sample is Sieved through a 4.75mm IS Sieve. the portion retained on the Sieve is the gravel fraction (or) plus 4.75mm material.

→ the gravel fraction is sieved through the set of Coarse Sieve manually (or) Using a mechanical Shaker.

→ Hand sieving is normally done the weight of soil retained on each sieve is obtained.



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→ The minus 4.75 mm fraction is sieved through the set of fine sieves. The sample is placed on the top sieve and the set of sieves is kept on a mechanical shakes and the machine is started.

→ Normally, 10 minutes of shaking is sufficient for most soils. The mass of soil retained on each sieve and on pan is obtained to the nearest 0.1 gm the mass of the retained soil is checked against the original mass.

→ Dry sieve analysis is suitable for cohesionless soils, with little (or) no fines. if the sand is sieved in wet condition. the surface tension may cause a slight increase in the size of the particles and the particles smaller than the aperature size may be retained on the sieve and the result would be erroneous.

(b) wet sieve Analysis:

→ if the soil contains a substantial quantity (say. more than 5%) of fine particles, a wet sieve analysis is required.

→ All lumps are broken into individual particles. A representative soil sample in the required quantity is taken using a riffler. and dried in an oven.

→ The dried sample is taken in a tray and soaked with water.

→ If deflocculation is required, Sodium hexameta-phosphate at the rate of 2g per litre of water is added.

→ The sample is stirred and left for a soaking period of at least one hour.

→ The slurry is then sieved through a 4.75 mm IS sieve and washed with a jet of water. The material retained on the sieve is the gravel fraction. it is dried in an oven & sieved through set of coarse sieves.

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→ The material passing through 4.75mm sieve is sieved through a 0.75 M sieve. The material is washed until the wash water becomes clear.

→ The material retained on the 75M sieve is collected and dried in an oven.

→ It is then sieved through the set of sieves of the size 2mm, 1mm, 600M, 425M, 212 M, 150M, & 75M. The material retained on each sieve is collected and weighed. The material that should have been retained on pan is equal to the total mass of soil minus the sum of the masses of material retained on all sieves.

④ Computation of percentage finer:

→ For determination of the particle size distribution curve, percentage of particles finer than a particular size is required. This can be found as under.

→ Let us consider the case when the sieving has been done through seven sieves, no-1 to no-7

→ Let the mass of soil retained on these sieves be respectively, $m_1, m_2, \ldots m_7$, and the mass of soil retained on the pan be m_8 the sum of all these masses is obviously, equal to the total mass of sample M.

→ Expressed as percentage, the materials retained on the sieves and pan are

$$P_1 = \frac{m_1}{M} \times 100 \quad , \quad P_2 = \frac{m_2}{M} \times 100 \quad \text{etc.}$$

$$P_7 = \frac{m_7}{M} \times 100 \quad , \quad P_8 = \frac{m_8}{M} \times 100$$

→ The cumulative percentage (c) of material retained on any sieve is equal to the sum of the % of soil retained on the sieve & that retained on all sieves coarser than that sieve

$$C_1 = P_1 \quad , \quad C_2 = P_1 + P_2 \quad , \quad C_7 = P_1 + P_2 + \ldots + P_7$$

→ The percentage finer (N) than any sieve size is obtained

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by subtracting the cumulative percentage retained on the sieve from 100%.

$N_1 = 100 - C_1$ $N_2 = 100 - C_2$ etc

$N_F = 100 - C_F$

(*) **Stokes law :-**

→ Soil particles finer than 75μ size cannot be sieved. The particle size distribution of such soils is determined by Sedimentation or analysis.

→ The analysis is based on Stokes' law which gives the terminal velocity of a small sphere settling in a fluid of infinite extent.

→ When a small sphere settles in a fluid its velocity first increases under the action of gravity but the drag force comes into action and retards the velocity.

→ After an initial adjustment period, steady conditions are attained and the velocity becomes constant.

→ The velocity attained is known as terminal velocity. The expression for terminal velocity can be obtained from the equilibrium of the particle.

→ The drag force F_D, experienced by a sphere of radius 'r' when it falls through a fluid of viscosity 'η' is given by $\boxed{F_D = 6\pi\eta r v}$

where v is the velocity.

→ The other two forces acting on the sphere are the weight (W) of the sphere and the buoyant force (U).

$W = 4/3\ \pi r^3 \gamma_s$

$W = 4/3\ \pi r^3 (\rho_s g)$

where γ_s is the unit weight of the material of sphere.

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$$U = 4/3\,\pi a^3 \gamma_w$$

$$= 4/3\,\pi a^3 (\rho_w g)$$

→ from equilibarum of foaces im veaticad diaection

$$W = U + F_D$$

$$4/3\,\pi a^3 \gamma_s = 4/3\,\pi a^3 \gamma_w + 6\pi\eta a v$$

$$4/3\,\pi a^3 \rho_s g = 4/3\,\pi a^3 \rho_w g + 6\pi\eta a v$$

$$\rho_s g = \rho_w g + 6\pi\eta a v$$

$$6\pi\eta a v = (\rho_s g - \rho_w g)$$

$$6\pi\eta a v = g(\rho_s - \rho_w)$$

$$v = \frac{2}{9} \cdot \frac{a^2}{\eta} (\rho_s - \rho_w) g$$

$$\boxed{v = \frac{1}{18} \cdot \frac{g D^2 (G-1) \rho_w}{\eta}} \quad \text{—— (1)}$$

→ wheaae D is the Diameteay of the Spheaae 'G' is the Sp. g of the matavial of Spheaae and g is the acceleaation dueae to gaavity.

→ If a Spheaical paaticle falls thaough a height He Centimeteas im t minute.

$$\boxed{v = \frac{He}{6t} \text{ cm/sec}} \quad \text{—— (2)}$$

fuom equation (1) & (2)

$$\frac{He}{6t} = \frac{1}{18} \frac{g D^2 (G-1) \rho_w}{\eta}$$

$$D = \sqrt{\frac{0.3\eta \times He}{g(G-1)\rho_w \times t}}$$

$$\boxed{D = M \sqrt{\frac{He}{t}}} \quad \text{—— (3)}$$

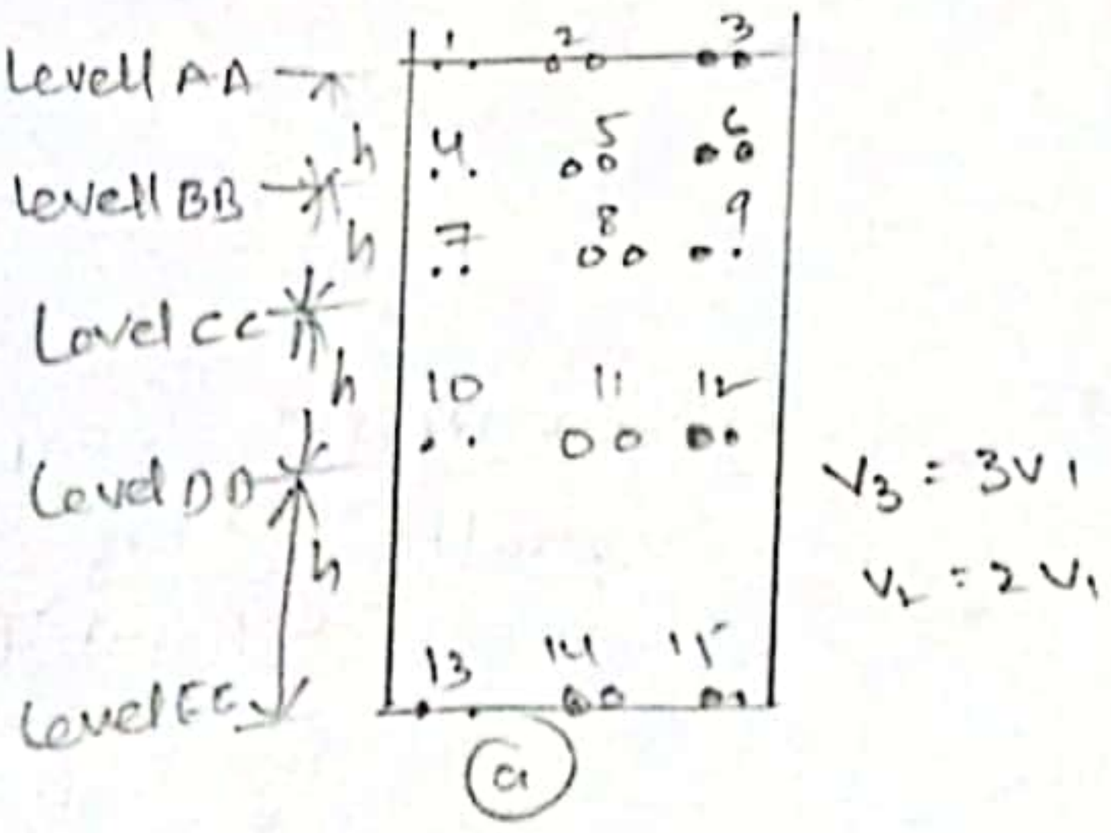
where m is a faday equal to $= \left[\frac{0.3\eta}{g(G-1)\rho_w}\right]^{\frac{1}{2}}$

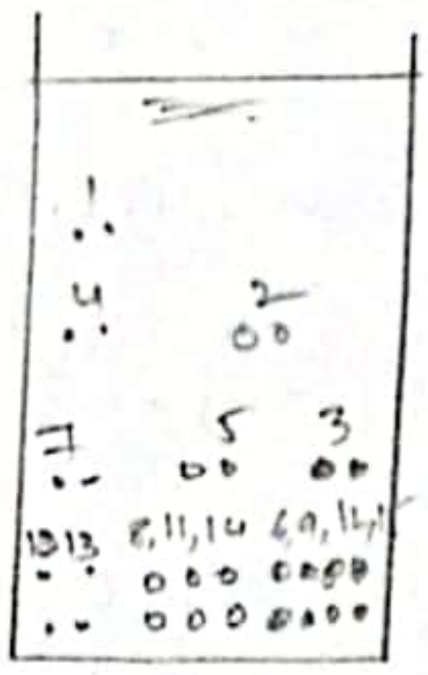
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→ In which n is the viscosity in poise (dyne-sec/cm²)
g = 981 cm/sec² & ρ is in gm/ml
D is in centimeter.

(*) Theory of Sedimentation :

→ At the commencement of the Sedimentation, the soil particles are uniformly dispersed throughout the suspension and the concentration of particles of different sizes is the same at all depths.
→ After a time period at a particular depth, only those particules remain which have not settled.
→ Because all particles of the same size have the same velocity the particles of a given size if they exist at any level are in the same Concentration as at the beginning of Sedimentation.
→ In other words all particles smaller than a particular size D will be present at a depth He in the same degree of concentration as at the beginning. All particles larger than the size 'D' would have settled below that depth





$$M = \frac{MD}{MS} \times 100$$

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(*) Combined Sieve and Sedimentation Analysis :

→ If the soil mass consists of particles of both coarse grained and fine grained soils a combined analysis is done.

→ The slurry of the soil is made as mentioned in the wet sieve analysis. The slurry is sieved through a 0.75mm IS sieve. The material retained on the sieve is oven dried and a coarse sieve analysis is done.

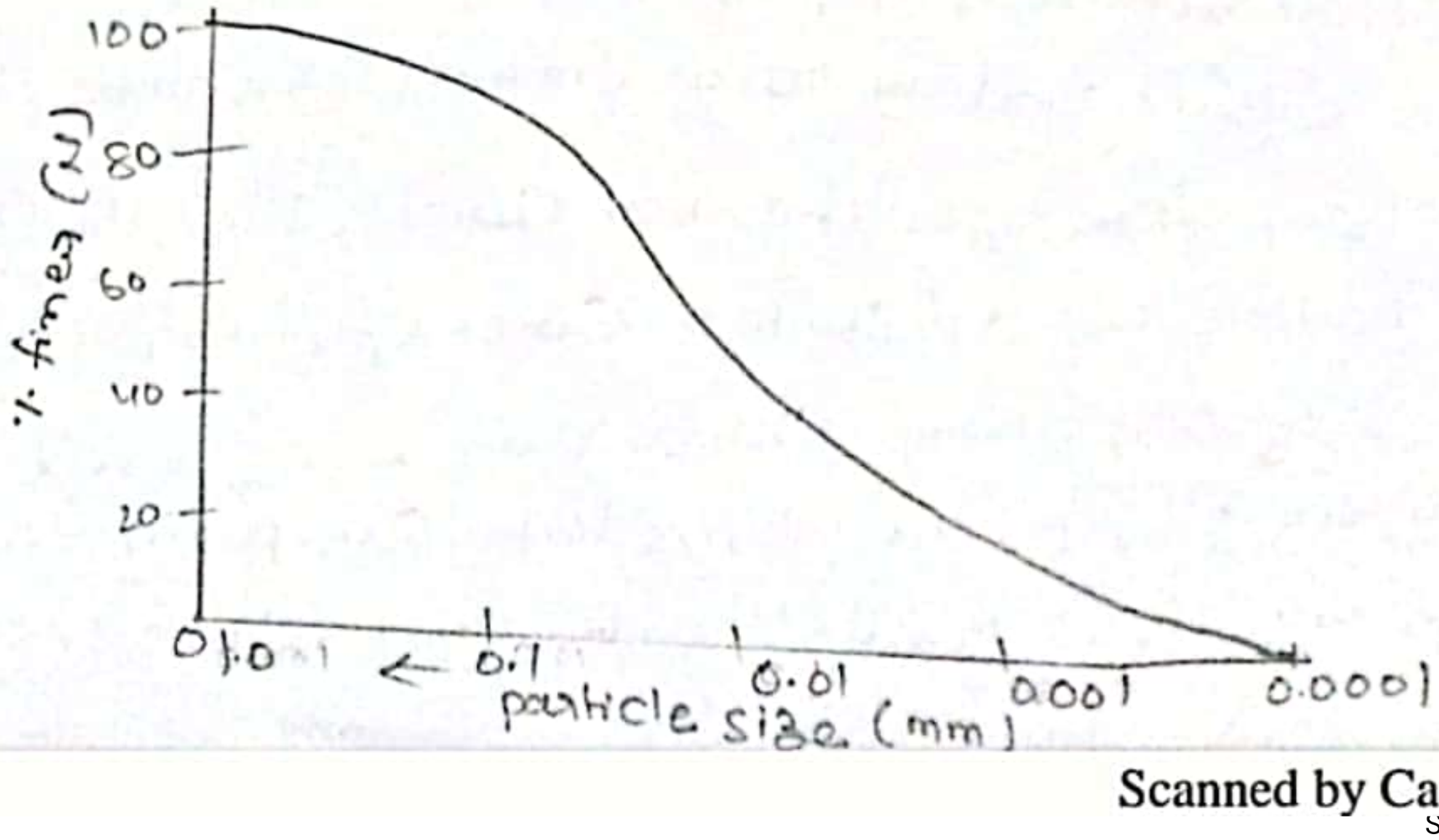
→ The material retained on a #75μ IS sieve is also oven dried and the sieve is analysis is done using the set of fine sieves.

→ The suspension passing the #75μ sieve is mixed with a deflocculating agent. if not already done the percentage finer than any size can be calculated on the basis of the original mass of soil taken for the combined analysis.

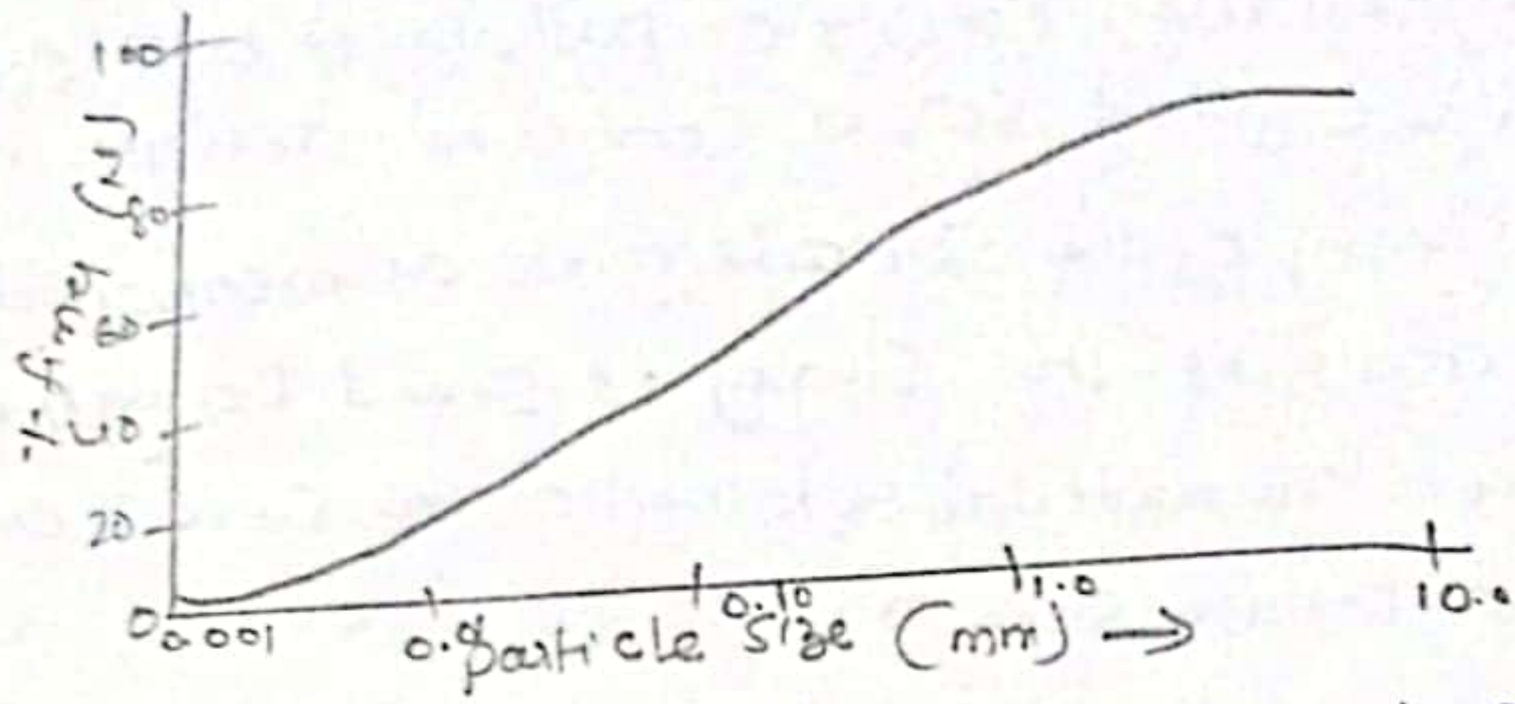
(*) particle size Distribution Curve :

→ The particle size distribution curve also known as a gradation curve, represents the distribution of particles of different sizes in the soil mass

→ The percentage finer N than a given size is plotted as cadinate (on natural scale) and the particle size as abscissa (on log scale). in fig (a) the particle size decrease from



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- left to right, whereas in fig (b) the particle size increases from left to right.

→ Both the methods are relevant the reader should carefully observe the horizontal scale of the particle size distribution curve.

→ In this text the particle size distribution is shown in fig (b) the particle size increases from left to right which is also the usual convention.

→ The semi log plot for the particle size distribution as shown in fig has the following advantages over natural plots.

(1) the soils of equal uniformity exhibit the same shape irrespective of the actual particle size

(2) As the range of the particle sizes is very large for better representation a log scale is required.

(4) Grading of soils:-

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

→ A curve with a hump such as curve A. represents the soil in which some of the intermediate size particles are missing. Such a soil is is called gap graded (or) skip graded.

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→ A flat S-curve such as curve B represents a soil which contains the particles of different sizes in good proportion. Such a soil is called a well-graded (uniformly graded) soil.

→ A steep curve like 'C' indicates a soil containing the particles of almost the same size. Such soils are known as uniform soils.

→ The particle size distribution curve also reveals whether a soil is coarse grained or fine grained. In general a curve situated higher up and to the left (curve D) indicates a relatively fine grained soil, whereas a curve situated to the right (curve E) indicates a coarse grained soil.

→ The uniformity of a soil is expressed qualitatively by a term known as uniformity coefficient

where

Cu given by

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

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

D_{10} = particle size such that 10% of the soil is finer than this size.

→ D_{10} size is also known as the effective size. In fig D_{60} & D_{10} for the soil B are respectively, 0.08 mm & 0.004 mm

Therefore $C_u = \frac{0.08}{0.004} = 20$.

→ The larger the numerical value of C_u the more is the range of particles. Soils with a value of C_u less than 2 are uniform soils.

→ Sands with a value of C_u of 6 (or) more are well-graded Gravels with a value of C_u of 4 (or) more are well graded.

→ The general shape of the particle size distribution curve is described by another coefficient known as the

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Coefficient of curvature (C_c) by the coefficient of gradation

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

where D_{30} is the particle size corresponding to 30% finer.

→ for a well graded soil the value of the coefficient of curvature lies b/w 1 and 3

→ It may be noted that the gap grading of the soil cannot b/w detected by C_u only the value of C_c is also required to detect it.

→ For the soil shown by curve B in fig the particle size D_{30} is 0.025 mm. Therefore.

$$C_c = \frac{(0.025)^2}{0.06 \times 0.004} = 1.95$$

(*) Relative Density :—

→ The most important index: aggregate property of a cohesionless soil is its relative density. The engineering properties of a mass of cohesionless soil depend to a large extent on its relative density (D_R) also known as density index (I_D)

The relative density is defined as.

$$D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

e_{max} = maximum void ratio of the soil in the loosest condition

e_{min} = minimum void ratio of the soil in the densest condition

e = void ratio in the natural state.

→ The relative density of a soil gives a more clear idea of the denseness than does the void ratio.

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→ two types of Sands having the same void ratio may have entirely different state of denseness and engineering properties. However if the two sands have the same relative density, they usually behave in identical manner.

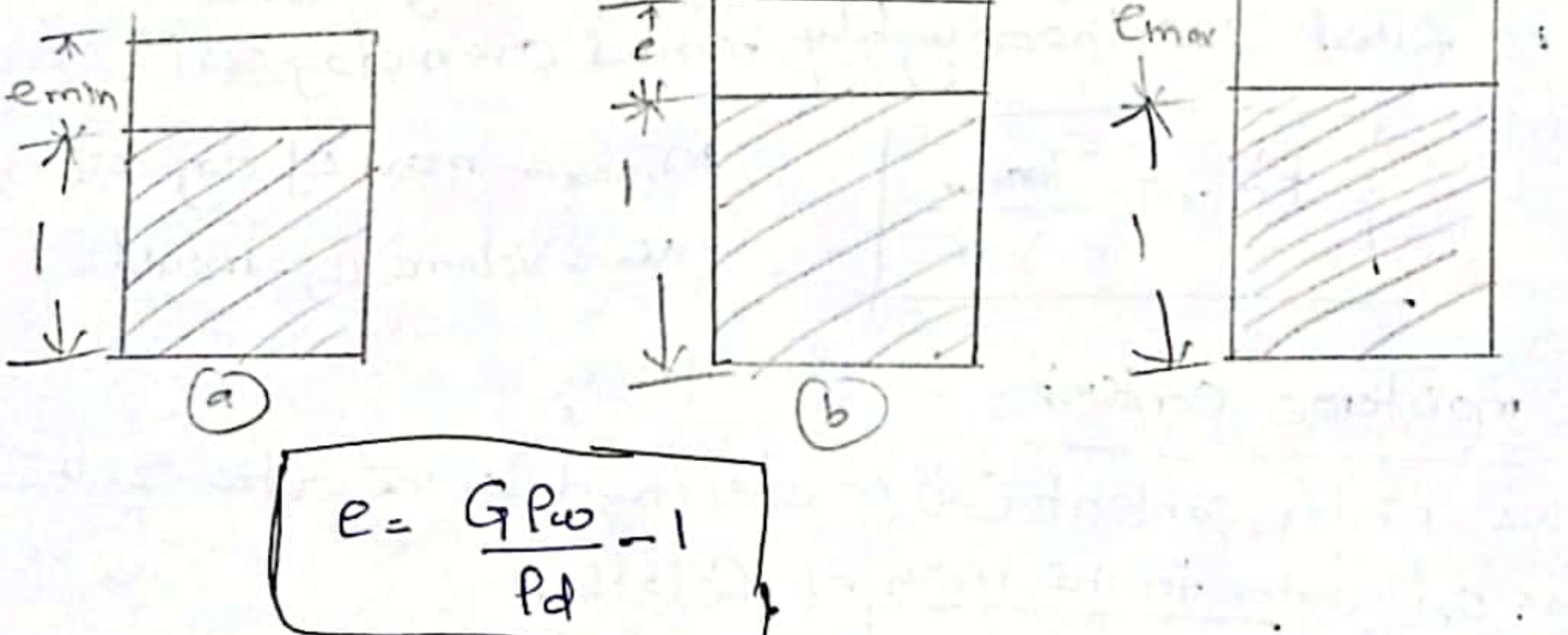
→ The relative density of a soil indicates how it would behave under loads. if the deposits is dense, it can take heavy loads with very little settlements.

→ Depending upon the relative density the soils are generally divided into 5 categories.

(*) Determination of Relative Density :

→ Shown a fig (a), (b) & c show the soil in the densest natural & loosest states.

→ As it is difficult to measure the void ratio directly equation cannot be used. However it is convenient to express the void ratio in terms of dry density (ρ_d)



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

→ Representing the dry density in the loosest, densest and natural conditions as ρ_{min}, ρ_{max} & ρ_d becomes

$$D_R = \frac{\left(\frac{G\rho_w}{\rho_{min}} - 1\right) - \left(\frac{G\rho_w}{\rho_d} - 1\right)}{\left(\frac{G\rho_w}{\rho_{min}} - 1\right) - \left(\frac{G\rho_w}{\rho_{max}} - 1\right)}$$

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$$D_r = \frac{\rho_{max}}{\rho_d}\left(\frac{\rho_d - \rho_{min}}{\rho_{max} - \rho_{min}}\right) \quad -(1)$$

equation (1) is used to determine the relative density of an in situ deposit. The methods for the determination of the day density (ρ_d) have been the methods for the determining of ρ_{max} and ρ_{min} are described below.

→ for determination of the minimum day density a represen tative. Oven day sample of soil is taken.

→ the sample is then pulverised and sieved through the required sieve.

$$\rho_{min} = \frac{M_{min}}{V_m}$$

M_{min} = mass of day soil

V_m = Volume of soil deposited in the mould.

→ the maximum day density is determined either by the day method (or) wet method In the day method the mould is filled with thoroughly mixed oven day soil.

$$\rho_{max} = \frac{M_{max}}{V_m}$$

M_{max} = mass of day soil

V_m = Volume of mould.

(*) moisture content :-

→ the water content (w) is defined as the ratio of the mass of water to the mass of solids.

$$w = \frac{M_w}{M_s}$$

→ the water content is also known as the moisture content (m) it is expressed as a percentage but used as a decimal in computation.

→ the water content of the fine grained soils such as silts & days the generally more than that of the

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coarse grained soils such as gravels and sands.

→ The water content of some of the fine grained soils may be even more than 100%. which indicates that more than 50% of the total mass is that of water.

→ The water content of a soil is an important property the characteristics of a soil especially a fine grained soil. change to a marked degree with a variation of its water content.

→ In geology and some other disciplines the water content is defined as the ratio of the mass of water to the total mass.

→ Some of the instruments such as moisture testers also give the water content as a ratio of the total mass In this text water content (w) will be taken as given equation unless mentioned otherwise.

→ The symbol 'm' shall be used in this text for the water content based on the total wet mass.

thus $\boxed{m' = \frac{M_w}{M} \times 100}$

example problem

(1) The mass of a chunk of moist soil is 20kg, and its volume is $0.011\ m^3$. After drying in an oven the mass reduces to 16.5kg. Determine the water content the density of moist soil, the dry density, void ratio, porosity and the degree of saturation. Take $G = 2.70$

sol:- mass of water, $M_w = 20 - 16.5 = 3.50\ kg$.

water content: $w = \frac{M_w}{M_s} = \frac{3.50}{16.5} \times 100 = 0.2121 \times 100$

$= 21.21\%$.

wet mass density: $\rho = \frac{M}{V} = \frac{20}{0.011} = 1818.18\ kg/m^3$

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The day density : $\rho_d = \frac{M_s}{V} = \frac{16.5}{0.011} = 1500.0 \text{ kg/m}^3$

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

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

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

voidratio: $e = \frac{G\rho_w}{\rho_d} - 1 = \frac{2.70 \times 1000}{1500} - 1 = 0.80$

porosity $n = \frac{e}{1+e} = \frac{0.80}{1+0.80} = \frac{0.80}{1.80} = 0.444 = 44.44\%$.

∴ void ratio (e)

$$w = \frac{Se}{G}$$

$$wG = Se$$

degree Saturated soil: $S = \frac{wG}{e} = \frac{0.2121 \times 2.70}{0.80} = 0.7158$ (71.58

(*) A soil specimen has a water content of 10% and a well unit weight of 20 kN/m³ if the specific gravity of solids is 2.70 determine the dry unit weight, void ratio, & the degree of Saturation Take $\gamma_w = 10 kN/m^3$

Sol:- day unit weight = $\gamma_d = \frac{\gamma}{1+w} = \frac{20}{1+0.1} = 18.18 \text{ kN/m}^3$

day unit weight = $\gamma_d = \frac{G.\gamma_w}{1+e}$

$$1+e = \frac{G.\gamma_w}{\gamma_d} = \frac{2.7 \times 10}{18.18} = 1.49$$

$$e = 1.49 - 1 = 0.49$$

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degree of saturation : $S = \frac{Wq}{e} = \frac{0.1 \times 2.70}{0.49}$

$= 0.551 \ (55.1\%)$

(*) The results of a sieve analysis of a soil are given below

Total mass of sample = 900gm

IS sieve	20 mm	10 mm	4.75 mm	2 mm	1.0 mm	0.6 mm	4.25 H	212 H	150 H	75 H	Pan
mass of soil retained (gm)	35	40	80	150	150	140	115	55	35	25	75

Draw the particle size distribution curve and hence determine the uniformity coefficient and the coefficient of curvature.

Sol: The calculations for percentage finer in different sizes are shown.

IS Sieve (1)	mass retained (2)	percentage retained $\frac{2}{900} \times 100$ (3)	cumulative percentage retained (4)	percentage of finer (5) = 100 - (4)
20 mm	35 g	3.89	3.89	96.11
10	40	4.44	8.33	91.67
4.75	80	8.89	17.22	82.78
2	150	16.67	33.89	66.11
1	150	16.67	50.56	49.44
0.6	140	15.56	66.12	33.88
4.25 H	115	12.78	78.90	21.10
212 H	55	6.11	85.01	14.99
150 H	35	3.89	88.90	11.10
75 H	25	2.78	91.68	8.32
Pan	75	8.32	100	0

$\Sigma = 900.0g$.

→ The particle size distribution curve shown.

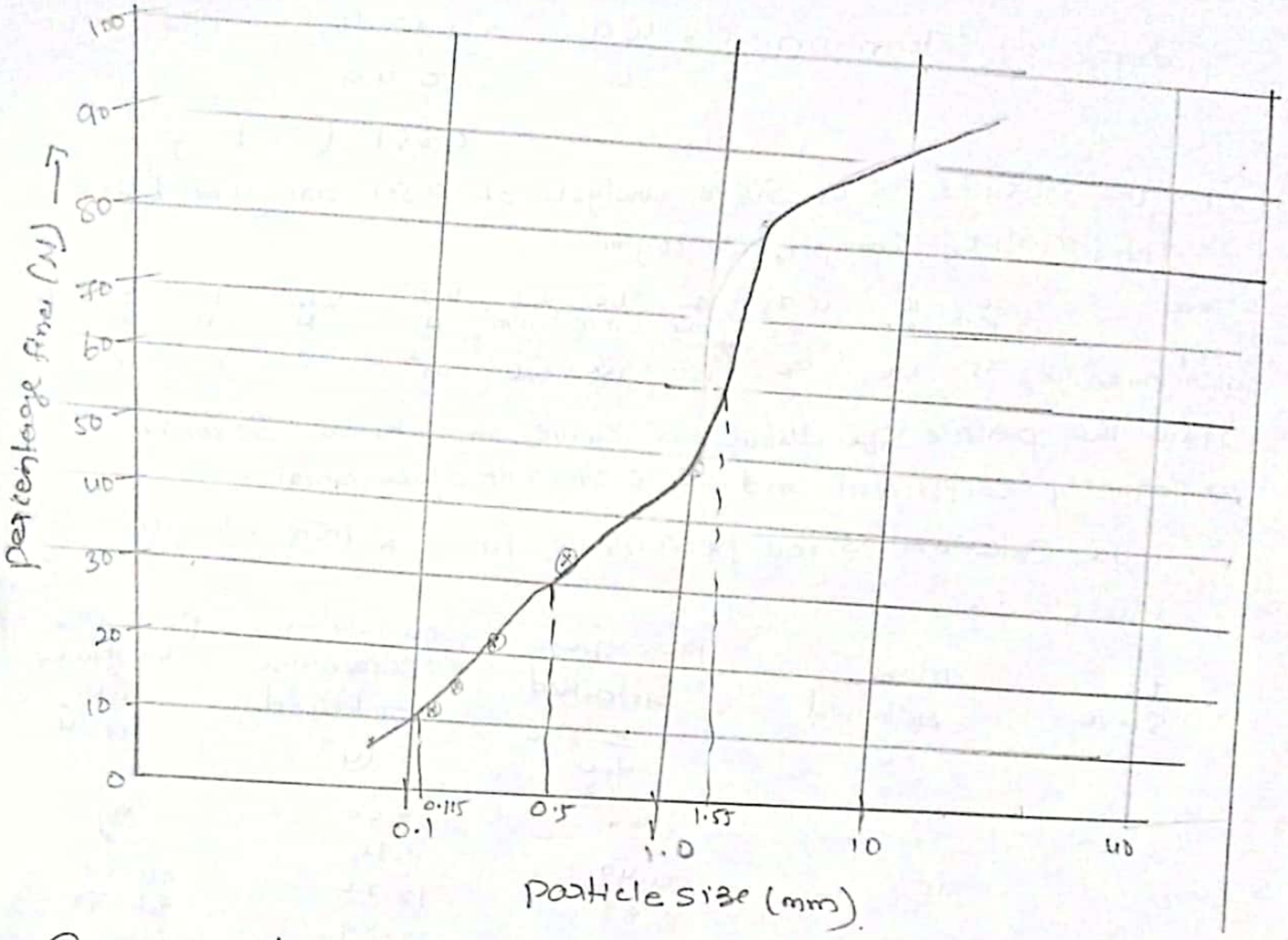
From plot. $D_{60} = 1.55$, $D_{30} = 0.53$, $D_{10} = 0.115$

from equation $C_u = \frac{D_{60}}{D_{10}} = \frac{1.55}{0.115} = 13.48$

$C_c = \frac{(D_{30})^2}{} = \frac{(0.53)^2}{1.55 \times 0.115} = 1.58$

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(7) In order to find the relative density of a sand, a mould of volume 1000 ml was used when the sand was dynamically compacted in the mould, its mass was 2.10 kg, whereas when the sand was poured in loosely its mass was 1.635 kg if the in-situ density of the soil was 1.50 mg/m³. Calculate the relative density. G = 2.70. Assume that the sand is saturated.

Sol:

$$(\rho_{max})_{sat} = \frac{M_{max}}{V_m} \quad \frac{2.1 \times 10^3}{1000} = 2.1\ g/ml.$$

$$(\rho_{min})_{sat} = \frac{M_{min}}{V_m} = \frac{1.635 \times 10^3}{1000} = 1.635\ g/ml.$$

As $\rho_d = 1.5\ mg/m^3 = = 1.50\ g/ml$

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

$$e = \frac{G\cdot\rho_w}{\rho_d} - 1 = \frac{2.70\times1}{1.5} - 1 = 0.80$$

$$\text{Now } (\rho_{max})_{sat} = \left(\frac{G+e_{min}}{1+e_{min}}\right)\rho_w$$

$$2.1 = \left(\frac{2.70+e_{min}}{1+e_{min}}\right)\times 1.0$$

$$e_{min} = 0.545$$

$$(\rho_{min})_{sat} = \left(\frac{G+e_{max}}{1+e_{max}}\right)\rho_w$$

$$1.635 = \left(\frac{2.70+e_{max}}{1+e_{max}}\right)\times 1.0$$

$$e_{max} = 1.677$$

$$D_r = \frac{e_{max}-e}{e_{max}-e_{min}} \times 100$$

$$= \frac{1.677-0.80}{1.677-0.545} \times 100$$

$$= 77.47\%.$$

(*) A soil has a liquid limit of 25%. and a flow index of 12.5%. if the plastic limit is 15%. determine the plasticity index and the toughness index.

if the water content of the soil in its natural condition in the field is 20%. find the liquidity index & the relative consistency.

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Sol: from equation

$$I_p = \omega_1 - \omega_p$$

$$= 25 - 15 = 10\%$$

equation $I_t = \frac{I_p}{I_f} = \frac{10}{12.5} = 0.8 = 80\%.$

equation $I_1 = \frac{\omega - \omega_p}{I_p} \times 100 = \frac{0.20 - 0.15}{0.10} \times 100 = 50\%.$

equation $I_c = \frac{\omega_1 - \omega}{I_p} \times 100 = \frac{0.25 - 0.20}{0.1} \times 100 = 50\%.$

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(1) A soil specimen has a water content of 10% and a wet unit weight of 20 kN/m³. If the specific gravity of solids is 2.70 Determine the dry unit weight, void ratio, & degree of Saturation

Take $\gamma_w = 10 kN/m^3$

Sol: Given data

Water Content = $w = 10\% = 0.1$

wet unit weight = $\gamma_{wet} = 20 kN/m^3$

$G = 2.70$

→ dry unit weight = $\gamma_d = \frac{\gamma}{1+w} = \frac{20}{1+0.1} = 18.18 kN/m^3$

→ void ratio (e): $\gamma_d = \frac{G.\gamma_w}{1+e}$

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

$e = \frac{G.\gamma_w}{\gamma_d} - 1 = \frac{2.70 \times 10}{18.18} - 1$

$= 0.49$

→ degree of Saturation (S)

$e = \frac{wG}{S} = S = \frac{wG}{e} = \frac{0.1 \times 2.70}{0.49}$

$= 0.55$

55.1%.

(2) A moist soil sample weighs 3.52 N. After drying in an oven its weight is reduced to 2.9 N the specific gravity of solids and the mass specific gravity are respectively 2.65 & 1.85 Determine the water content, voids ratio, porosity and the degree of saturation. Take $\gamma_w = 10 kN/m^3$

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Sol!: Given data

moist soil Sample weight = 3.52N

day soil weight = 2.9N.

$G_s = 2.65$

$G_m = 1.85$

$\gamma_w = 10 kN/m^3$

water Content =

weight of water = 3.52 − 2.90 = 0.62N.

water content = $\frac{\text{Weight of water}}{\text{day soil}} = \frac{M_w}{M_s}$

$= \frac{0.62}{2.9} = 0.2138 = (21.38\%)$

$\left(G = \frac{\gamma}{\gamma_w}\right)$

$\gamma = G_m \times \gamma_w$

$= 1.85 \times 10 = 18.5 kN/m^3$

day density / unit weight: $\gamma_d = \frac{\gamma}{1+w} = \frac{18.5}{1+0.2138} = 15.24 kN/m^3$

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

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

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

$= \frac{2.65 \times 10}{15.24} - 1 = 0.74.$

- porosity (n) = $\frac{e}{1+e} = \frac{0.74}{1+0.74} = 0.4253\ 42\%$

degree of Saturation = S: $e = \frac{wG}{S} \Rightarrow S = \frac{wG}{e}$

$S = \frac{0.2138 \times 2.65}{0.74}$

$= 0.76$

$= 76\%.$

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(3) A sample of clay soil has mass 68 gm. Find the volume of voids if the total volume of the sample 40 ml. If the Specific gravity of solids is 2.65. Also determine the void ratio.

Sol: Given data

$M_s = 68$ gm.

$V = 40 ml$

$G = 2.65$

Clay density $\rho_d = \frac{M_s}{V} = \frac{68}{40} = 1.7 gm/ml$

Volume of solids (V_s) : $\rho_s = \frac{M_s}{V_s}$

$V_s = \frac{M_s}{\rho_s}$

$= \frac{M_s}{G \cdot \rho_w}$

$\because G = \frac{\rho_s}{\rho_w}$

$\rho_s = G \cdot \rho_w$

$V_s = \frac{68}{2.65 \times 1} = 25.66 ml$

Volume of voids :

$V_v = V - V_s = 40 - 25.66$

$= 14.34 ml.$

Voids ratio = $e = \frac{V_v}{V_s} = \frac{14.34}{25.66} = 0.56$

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(1) A soil has a day density of 1.816 gm/ml in the natural conditions, when 410 gm of the soil was poured in a vessel in a veay loose state. its volume was 290 ml the same soil when vibrated & compacted was found to have a volume of 215 ml Determine the relative density.

Sol:- Given data

$\rho_d = 1.8169 \text{ gm/ml}$

$M_{min} = M_{max} = 410 \text{ gm}$.

$V_{max} = 290 \text{ ml}$

$V_{min} = 215 \text{ ml}$

$$\rho_{min} = \frac{M_{min}}{V_{max}} = \frac{410}{290} = 1.414 \text{ gm/ml}$$

$$\rho_{max} = \frac{M_{max}}{V_{min}} = \frac{410}{215} = 1.907 \text{ gm/ml}$$

$$D_a = \frac{\rho_{max}}{\rho_d}\left(\frac{\rho_d - \rho_{min}}{\rho_{max} - \rho_{min}}\right) \times 100$$

$$= \frac{1.907}{1.816}\left(\frac{1.816 - 1.414}{1.907 - 1.414}\right) \times 100$$

$D_a = 85.63\%$.

(2) A test for the relative density of soil in place way performed by digging a small hole in the soil the volume of the hole was noomd & the moist weight of the excavated soil was 9 N After oven drying

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: GEOTECHNICAL ENGINEERING

SUBJECT CODE: 20CE5PC03

FACULTY NAME: B. SURESH

SYLLABUS:

UNIT - II Permeability:

Soil water – capillary rise – flow of water through soils – Darcy's law- permeability – Factors affecting permeability – laboratory determination of coefficient of permeability – Field measurement, pumping out in unconfined and confined aquifer - Permeability of layered soils. Effective Stress & Seepage Through Soils: Total, neutral and effective stress – principle of effective stress – quick sand condition – Seepage through soils – Flow nets: Characteristics and Uses.

UNIT - II
PERMEABILITY OF SOIL

Def: → The property of soil which allows the movement of water from its interconnecting voids is known as permeability of soil.

→ The property of a soil which permits flow of water through it is called the permeability. In other words the permeability is the ease with which water can flow through it.

→ The material which allows the movement of water from it is known as permeable material.

→ The material which doesnot allow movement of water from it is known as Impermeable material.

→ permeability of soil increase with void size.

permeability order:

gravel > Sand > silt > clay

→ The material which can store water in its voids is known as porous material.

→ The material which can not store water in its voids is known as non-porous material.

Porosity order:

clay > silt > Sand > Gravel

→ Gravels are highly permeable but less porous

→ clay is less permeable but highly porous.

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(*) soil water :-

→ The soil water is broadly classified in two categories
  (1) Free water
  (2) Held water.

→ Free water moves in the pores of the soil under the influence of gravity the held water is retained in the pores of the soil and it cannot move under the influence of gravitational force.

(1) Free water flows :-

→ Free water flows from one point to the other wherever there is a difference of total head. the rate at which the head is lost along the flow passage is equal to the hydraulic gradient.

→ the flow of free water in soils is just like laminar flow in pipes. Because of very small flow passages in the soil the velocity head is generally neglected. and the total head is taken equal to the sum of the elevations head and the pressure head

(2) Held water :.

→ Held water is further divided in to three types.
  (a) structural water
  (b) Adsorbed water
  (c) Capillary water.

→ the structural water is chemically combined water in the crystal structure of the mineral of the soil this water cannot be removed without breaking the structure of the mineral..

→ A temperature of more than 300°c is required for removed without breaking the structure of the

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- removing the structural water. In soil engineering the structural water is considered as an integral part of the soil solid. (2)

(b) Adsorbed water :-

→ the water held by electrochemical forces existing on the soil surface is known as adsorbed water as the quantity of adsorbed water depends upon the colloidal fraction in the soil, the chemical composition of the clay mineral and the environment surrounding the particle.

→ the adsorbed water is important only for clayey soils for coarse grained soils its amount is negligible or zero.

→ the adsorbed water is also sometimes called hygroscopic water. the amount of water in an air dried soil is defined as hygroscopic water.

(c) Capillary :-

→ the water held in the interstices of soils due to capillary forces is called capillary water.

(*) Capillary rise :-

→ the water which falls on the ground as rain flows under gravity and passes through the soil and reaches a surface known as ground water table

→ the soil is saturated below the ground water table. the level to which underground water rises in an observation well is called ground water table (or) simply water table.

→ the ground water table is also called as the ground phreatic surface a term derived from the Greek word ~~phreas~~ phreas meaning a well.

→ Ground water which is a form of free water is not static it is a moving stream which flows under gravitational forces.

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→ The water table is not horizontal it takes the shape according to the topography the water is drawn above the water table due to Capillary action.

→ A soil mass consists of a no. of interconnected interstices which act as Capillary tubes of varying diameters.

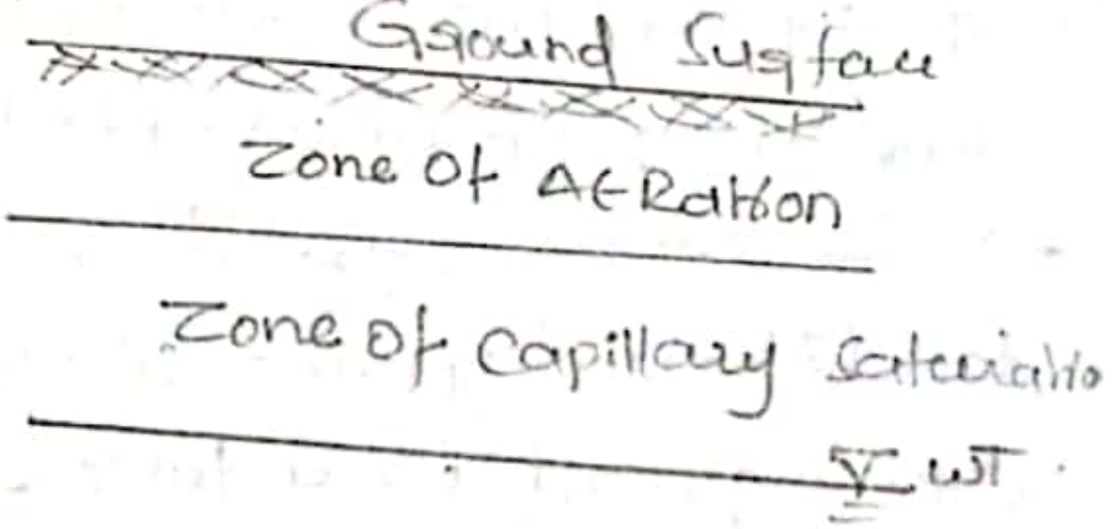
→ Although the channels formed by interconnected interstices are not circular in Cross-Section the results of capillary rise in circular tubes are useful for understanding the phenomenon of Capillary rise in soils.

→ The channels formed in the soil are a sort of capillary tubes of varying diameter but not necessarily vertical. These capillary tubes may be inclined in any direction.

→ Capillary rise in soils depends upon the size and grading of the particles. The diameter (d) of the channels in pore passage depends upon the diameter of the particle. it is generally taken as one fifth of the effective diameter (D_{10}) in case of Coarse-grained soils.

$$d = 0.2 D_0$$

→ As the capillary rise is inversely proportional to the diameter of the tube the capillary rise is small in coarse grained soils. In some very fine grained soils it may be even more than 30 m.



(*) Flow of water through soil (permeability).

→ It is necessary for a civil engineer to study the principal of fluid flow and the flow of water through soil in order to solving problems involving.

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(a) the rate at which water flows through soil

(b) Compression

(c) Strength

→ water can flow through the pore spaces in the soil and the soil is considered to be permeable the property of a porous medium such as soil by virtue of which water can flow through it is called its permeability.

→ while all soils are permeable to a greater (or) smaller degree, certain clays are more (or) less impermeable for all practical purposes.

→ permeability is one of the most important of soil properties the path of flow from one point to another is considered to be a straight one, on a macroscopic scale & the velocity of flow is considered uniform at an effective value.

(*) Darcy's Law:-

→ the flow of free water through soil is governed by Darcy's law. Darcy demonstrated experimentally that for laminar flow in a homogeneous soil, the velocity of flow (v) is given by

$$v = ki \quad \text{--- (1)}$$

where k = coefficient of permeability
i = hydraulic gradient

→ the velocity of flow is also known as the discharge velocity (or) the superficial velocity.

→ equation (1) is known as Darcy's law which is one of the corner stones of the soil engineering. the discharge 'q' is obtained by multiplying the velocity of flow (v) by the total cross-sectional area of soil (A) normal to the direction of flow.

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Thus $\boxed{q = vA = kIA}$ — (1)

The area A includes both the solids & the voids.

→ The coefficient of permeability can be defined as can be defined using eq (2) If the hydraulic gradient is unity the coefficient of permeability is equal to the velocity of flow.

→ In other words the coefficient of permeability is defined as the velocity of flow which would occur under unit hydraulic gradient.

→ The coefficient of permeability has the dimensions of velocity (L/T)

→ it is measured in mm/se, cm/se, m/se, m/day & other velocity units.

→ The coefficient of permeability depends upon the particle size & upon many other factors. eg.

~~Soil~~ Soil type · ~~coefficient of permeability~~

(*) Validity of Darcy's Law

→ Darcy's law is valid if the flow through soils is Laminar the flow of water through soils depends upon the dimension of interstices, which, in turn depends upon the particle size.

→ In fine grained soils the dimensions of the interstices are very small and the flow is necessarily laminar.

→ In coarse grained soils the flow is also generally laminar However in very coarse grained soils such as coarse gravel the flow may be turbulent.

→ For flow of water through pipes the flow is laminar when the Reynolds number is less than 2000 for flow through

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it has been found that the flow is laminar if the Reynolds (4)
number is less than unity.

→ For flow through only the characteristic length in the Reynolds number is taken as the average particle diameter (D)

thus = $\boxed{\frac{\rho V D}{\eta} \leq 1}$

where ρ = is the mass density and η is the coefficient of viscosity.

→ Using Allen Hazen's equation for velocity it can be shown that the maximum diameter of the particle for the flow to be laminar is about 0.50mm

→ the value of the critical Reynolds number of unity is however conservative. it has been demonstrated that the flow remains laminar even upto the Reynold's number of 75.

→ For flow through coarse sands, gravels and boulders, the actual relationship b/w the velocity and the hydraulic gradient is non linear. Hough gave the following equation for the velocity when the flow is turbulent.

$\boxed{V = k(i)^n}$

n: with a value of 0.65

(*) Factors Affecting permeability of soils :-

(1) Particle Size:- As it is the coefficient of permeability of a soil is proportional to the square of the particle size(D) the permeability of coarse grained soils is very large as compared to that of fine grained soils the permeability of coarse sand may be more than one million times as much that of clay.

(2) Structure of soil mass :

→ the coefficient c in equation takes into account the shape of the flow passage

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→ . The size of the flow passage depends upon the structural arrangement for the same void ratio, the permeability is more in the case of flocculated structure as compared to that in the dispersed structure.

→ Stratified soil deposits have greater permeability parallel to the plane of stratification than that perpendicular to this plane

→ permeability of a soil deposit also depends upon shrinkage cracks, joints fissures and shear zones. Loess deposits have greater permeability in the vertical direction than in the horizontal direction.

→ the permeability of a natural soil deposit should be determined is undisturbed condition.

(3) Shape of particles!.

→ the permeability of a soil depends upon the shap of particles. Angular particles have greater specific surface area as compared with the rounded particles.

→ for the same void ratio, the soils with angular particles are less permeable than those with rounded particles. as the permeability is inversely proportional to the specific surface

(4) Void Ratio :-

→ the coefficient of permeability varies as $\frac{e^3}{(1+e)}$ for a given soil the greater the void ratio, the higher is the value of the coefficient of permeability.

→ Based on other concepts it has been established that the permeability of a soil varies as e^2 (or) $\frac{e^2}{(1+e)}$ whatever may be the exact relationship all soils have e versus log k plot as a straight line.

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(5) Properties of water :-

→ The coefficient of permeability is directly proportional to the unit weight of water (γ_w) and is inversely proportional to its viscosity (η).

→ The unit weight of water does not vary much over the range of temperature ordinarily encountered in soil engineering problems.

→ However there is a large variation in the value of the coefficient of viscosity (η).

→ The coefficient of permeability increases with an increase in temperature due to reduction in the viscosity.

→ it is usual practice to report the coefficient of permeability at 27°c the following equation can be used for conversion of the permeability to 27°c

$$K_{27} = k_t \frac{\eta_t}{\eta_{27}}$$

(6) Degree of Saturation :-

→ if the soil is not fully saturated it contains air pockets formed due to entrapped air or due to air liberated from percolating water.

→ whatever may be the cause of the presence of air in soils the permeability is reduced due to presence of air which cause blockage of passage.

→ Consequently the permeability of a partially saturated soils is considerably the permeability of a partially saturated soils is ~~cond~~ considerably smaller than that of a fully saturated soil. Infact Darcy's law is not strictly applicable to such soil.

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(7) Adsorbed water :

→ the fine grained Soils have a layer of adsorbed water strongly attached to their surface. This adsorbed water layer is not free to move under gravity.

→ it causes an obstruction to flow of water in the pores and hence reduces the permeability of soils.

→ it is difficult to estimate the voids occupied by the Adsorbed water. According to one estimate the void ratio occupied by adsorbed water is about 0.10.

(8) Impurities in water :-

→ Any foreign matter in water has a tendancy to plug the flow passage and reduce the effective voids and hence the permeability of soils.

(*) Laboratory 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 test

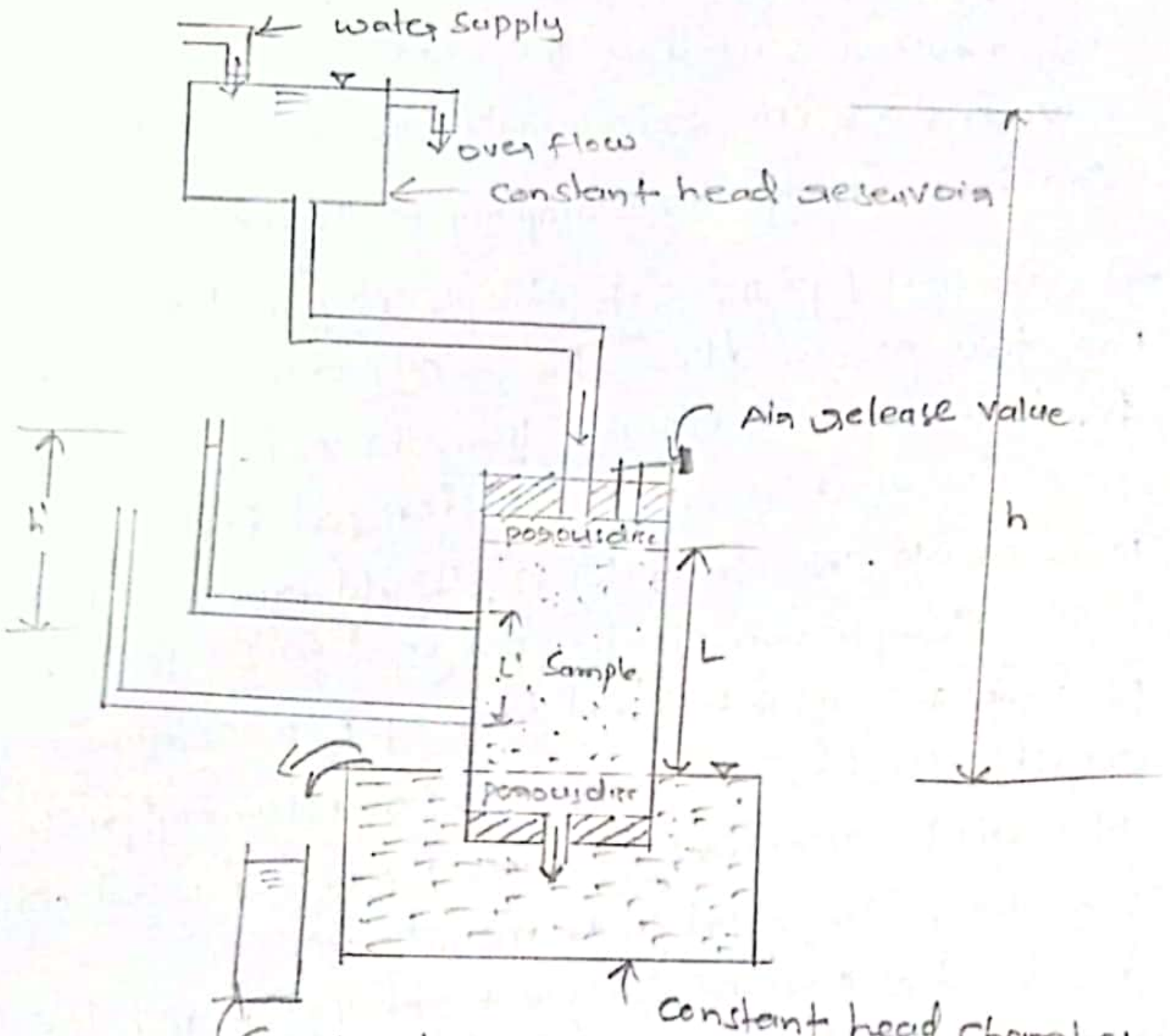
(ii) Variable-head permeability test

→ the Instruments used are known as permeameters. the former test is suitable for relatively more pervious soils and the latter for less pervious soils.

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(2) Constant Head permeability Test :-

→



→ The coefficient of permeability of a relatively more permeable soil can be determined in a Laboratory by the constant head permeability test.

→ The Test is conducted in an instrument known as Constant head permeameter.

→ It Consists of a metalic mould, 100 mm internal diameter 127.3mm effective height and 1000 ml Capacity according to IS: 2720 (part XVII)

→ The mould is provided with a detachable extension collar 100mm diameter and 60mm high required during Compaction of Soil.

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→ The mould is provided with a drainage base plate with a recess for porous Stone.

→ The mould is fitted with a drainage cap having an inlet Valve and an air release Valve. The drainage ~~base~~ base and cap have fittings for clamping to the mould.

→ Shown a fig the Soil Sample is placed inside the mould blw two porous discs. The porous discs should be at (east ten times more permeable than the soil.

→ The porous discs should be deaired before these are placed in the mould. The water tubes should also be deaired.

→ The Sample can also be prepared in the permeameter by pouring the Soil into it and tamping it to obtain required density. The base is provided with a dummy plate 12 mm thick and 108 mm ⌀ in diameter which is used when the Sample is Compacted in the mould.

→ it is essential that the sample is fully Saturated This is done by one of the following three methods.

(i) By pouring the Soil in the permeameter filled with water and thus depositing the Soil under water.

(ii) By allowing water to flow upward from the base to the top after the Soil has been placed in the mould. This is done by attaching the constant-head reservoir to the drainage base the upward flow is maintained for sufficient time till all the air has been expelled out.

(iii) By applying a vacuum pressure of about 700 mm of mercury through the drainage cap for about 15 minutes after closing the drainage Valve. Then the Soil is Saturated by allowing deaired water to enter from the drainage base. The air-release valve is kept open

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- during Saturation process.

→ After the soil Sample has been Saturated the Constant head reservoir is Connected to the drainage cap. water is allowed to flow out from the drainage base for some time till a Steady State is established.

→ the water level in the Constant-head chamber in which the mould is placed is kept Constant.

→ the chamber is filled to the brim at the start of the experiment.

→ the water which enters the chamber after flowing through the Sample spills over the chamber and is collected in a graduated jar for a convenient period.

→ the head causing flow (h) is equal to the difference in water levels b/w the Constant-head reservoir and the constant head chamber

if the cross-sectional area of the Specimen is A the discharge is given by

$$q = kiA$$

$$q = k \cdot \frac{h}{L} A$$

$$k = \frac{qL}{Ah}$$

where L = length of specimen

h = head causing flow.

→ the discharge q is equal to the volume of water collected divided by time.

(2) Variable Head Permeability Test :-

→ for relatively less permeable soils the quantity of water collected in the graduated jar of the Constant head permeability test is very small and cannot be measured accurately.

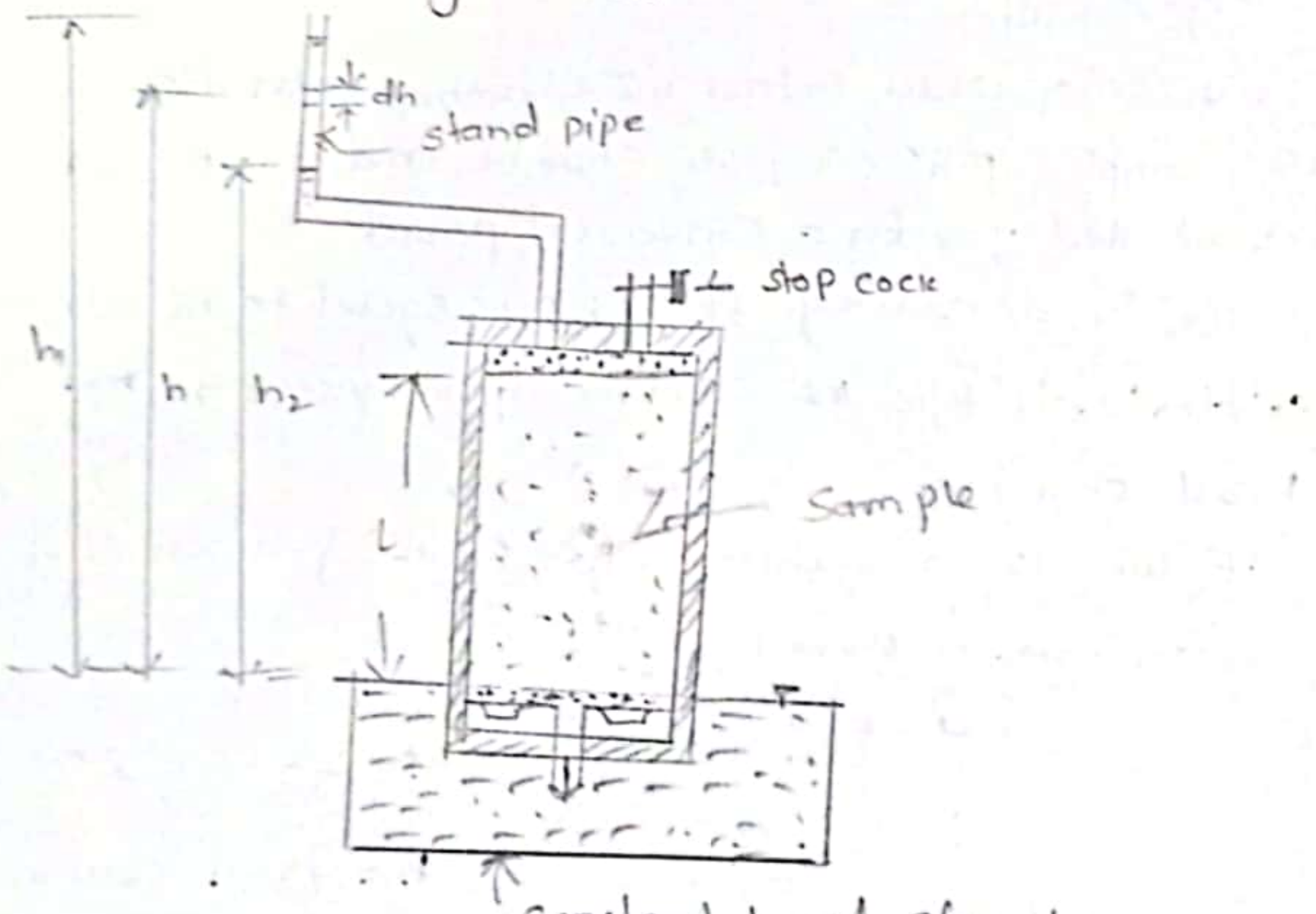
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→ For such soils the variable head permeability test is used. The permeameter mould is the same as that used in the constant head permeability test.

→ A vertical, graduated stand pipe of known diameter is fitted to the top of permeameter.

→ The sample is placed b/w two porous discs. The whole assemly is placed in a constant head chamber filled with water to the barometric stage of the test.



→ As shown in fig the porous discs and water tubes should be de aired before the sample is placed. If in situ undisturbed sample is available the same can be used otherwise the soil is taken in the mould and compacted to the required density.

→ The valve at the drainage base is closed and a vacuum pressure is applied slowly through the drainage cap to remove air from the soil.

→ The vacuum pressure is increased to 400mm of mercury and maintained for about 15 minutes.

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→ The Sample is Saturated by allowing deaired water to flow upward from the drainage base when under vacuum. when the soil is Saturated both the top and bottom outlets are closed the Standpipe is filled with water to the required height

→ the test is started by allowing the water in the Stand pipe to flow Through the Sample to the Constant head chamber from which it overflows and spills out.

→ As the water flows through the soil the water level in the Stand pipe falls. The time required for the water level to fall from a known initial head (h_1) to a known final head (h_2) is determined.

→ The head is measured with reference to the level of water in the Constant-head chamber.

→ Let us Consider the Instant 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.

$$\boxed{a\,dh = -q\,dt}$$

→ where a is Cross-Sectional area of the Stand pipe

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

$$a\,dh = -Ak \times \frac{h}{L} \times dt.$$

$$\frac{a\,dt}{AL} = -\frac{dh}{h}$$

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

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$$\frac{Ak}{aL}(t_2-t_1) = \log_e\left(\frac{h_1}{h_2}\right)$$

$$\boxed{k = \frac{aL}{At}\log_e\left(\frac{h_1}{h_2}\right)} \quad \text{——— (1)}$$

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

$$\boxed{k = \frac{2.30aL}{At}\log\left(\frac{h_1}{h_2}\right)}, \quad \text{——— (2)}$$

→ The rate of fall of water level in the Stand pipe and the rate of flow can be adjusted by changing the area of the Cross-section of the Standpipe.

→ The Smaller diameter pipes are required for less previous soils.

→ The Coefficient of permeability is expressed at 27°c as per IS ~~[illegible]~~ 2720(part xvii). The void ratio of the Soil is also generally determined.

→ The Variable head permeameter is Suitable for Very fine Sand and Silt with $k = 10^{-2}$ to 10^{-5} mm/sec

→ Sometimes, the permeability test is conducted using the Consolidometer Instead of the permeameter mould.
The fixed Ring Consolidometer is used as a Variable head permeameter by attaching a Stand pipe to its base.

(*) In situ permeability test

(1) pumping in test

(2) pumping out test.

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(-x) pumping in tests:

→ pumping in test are conducted to be determine the Coefficient of permeability of an individual stratum through which a hole is drilled

→ These tests are more economical than the pumping out test However the pumping out test give more reliable Values. than that given by pumping in test.

→ The pumping in tests give the value of the coefficient of peameability of stratum just close to the hole whereas the pumping out tests give the value for a large area around the hole.

→ There are basically two types of pumping in tests.

(1) open-end tests.

(2) packer tests.

→ In an open end tests the water flows out of the test hole through its bottom end whereas in packere tests the water flows out through the sides of the section of a hole enclosed by packers.

→ The Value of the Coefficient of permeability is obtained from the quantity of water accepted by the hole.

→ the Water pumped in Should be Clean of the impurities Such as Silt clay or any other foreign matter may Cause plugging of the flow passages.

→ If the water available is turbid it Should be classified in a Settling tank or by using a filter.

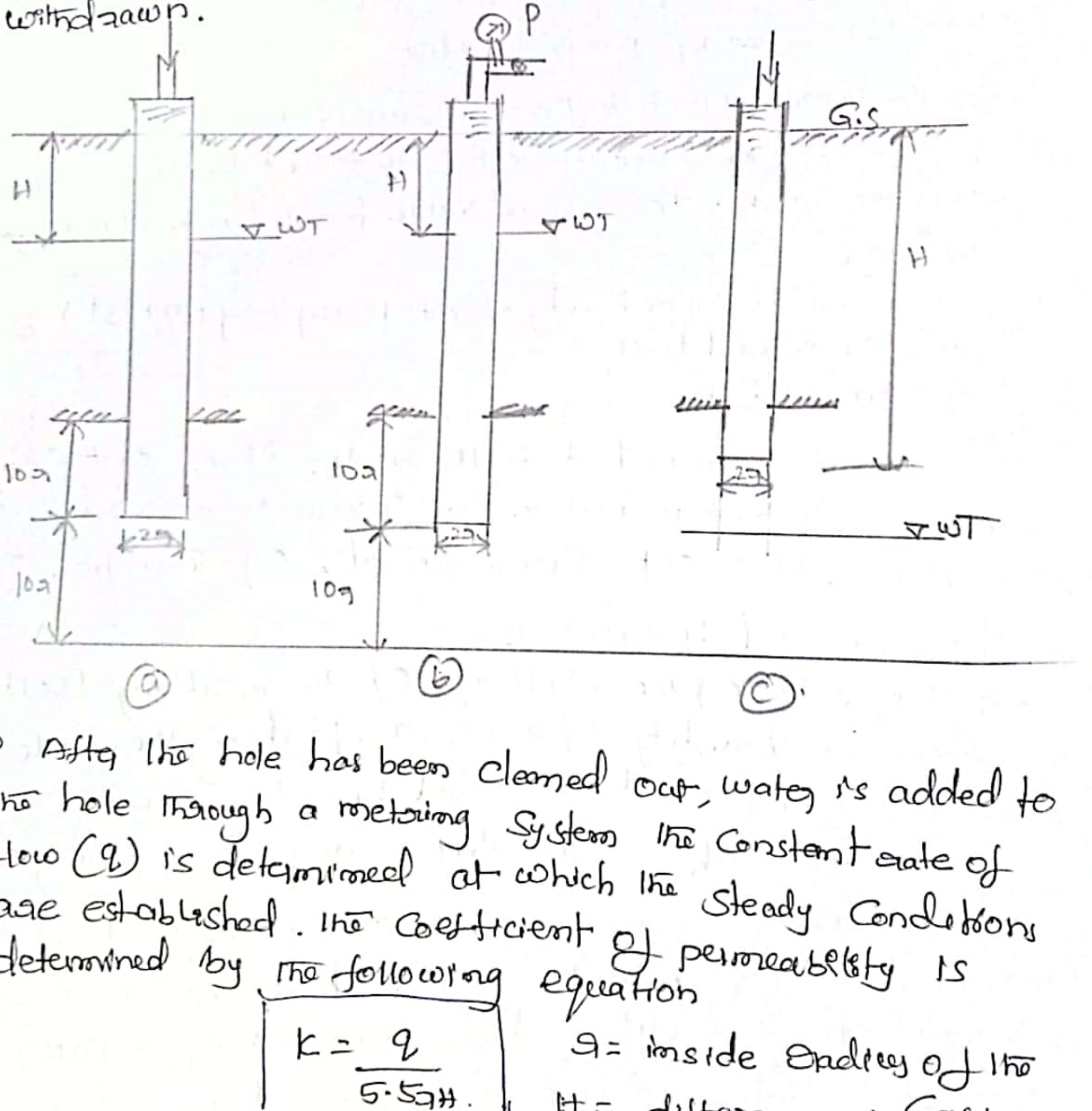
→ the temperature of the water pumped in Should be slightly higher than the temperature of the ground water to preclude the formation of air bubbles in strata.

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(i) Open-end tests :-

→ A pipe casing is inserted into the bore hole to the desired depth and it is cleaned out. The hole is kept filled with water during cleaning if it extends below the water table.

→ This is necessary to avoid squeezing of the soil into the bottom of the pipe casing when the drawing tool is withdrawn.



→ After the hole has been cleaned out, water is added to the hole through a metering system the constant rate of flow (Q) is determined at which the steady conditions are established. The coefficient of permeability is determined by the following equation

$$K = \frac{Q}{5.5aH}$$

a = inside radius of the casing.

H = difference of levels b/w the inlet to the casing and the water table

Q = discharge.

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→ if required the discharge can be increased by pumping in water under a pressure P, For this case the value of H becomes equal to $(H + P/\gamma_w)$

→ For accurate results the lower end of the pipe should be at a distance of not less than 10q from the top as well as from the bottom of the stratem.

→ the open end test can be also conducted above the water table in this case however. It is difficult to maitain a constant water level in the casing and some sagging of this level has to be tolerated. Can also be used in this case However in this case H is equal to the differe of inlet level and the bottom end of the pipe if required. the rate the rate of flow (q) can be increased by pumping in water under a pressure p. with a total head of $\frac{H+L}{\gamma_w}$

(2) packer tests :-

→ the packer tests are performed in an uncated portion of the pipe casing the packer test are more commonly used for testing of rocks.

→ this tests are occasionaly used for testing of the soils if the bore hole can stay open without any casing.

(a) single packer tests :-

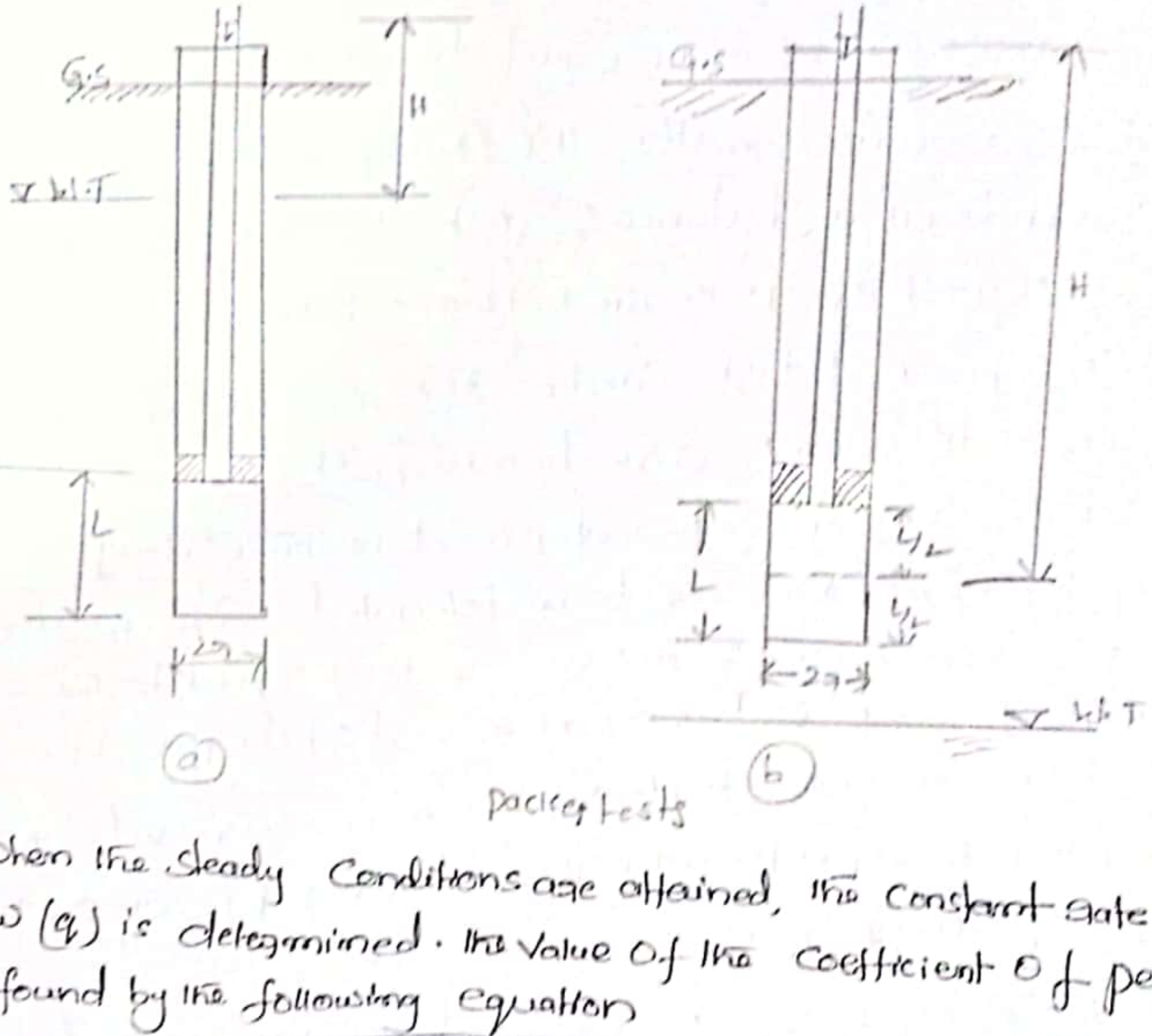
→ If the hole cannot stand without a casing single packer test is used the packer is placed as shown in fig

→ water is pumped into the hole. It comes out of the sides of uncased portion of the hole below the packer. If the casing is used for the full depth. it should have perforations in the portion of the stratem being test

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the lower end of the casing is plugged.



packer tests

→ when the steady conditions are attained, the constant rate of flow (q) is determined. the value of the coefficient of permeability is found by the following equation

$$K = \frac{q}{2\pi L H} \log_e\left(\frac{L}{r}\right) \quad \text{if } L \geq 10r$$

$$K = \frac{q}{2\pi L H} \sinh^{-1}\left(\frac{L}{2r}\right) \quad \text{if } 10r > L \geq r$$

where r = inside radius of hole, L = length of the hole tested
H = difference of water levels at the entry and the ground water table for the hole tested below the water table.

$\sinh^{-1}$ = arc hyperbolic sine.

→ for the hole tested above the water table, H is equal to the difference of levels of water at the entry and the middle of the test section.

→ if the water is applied under pressure (p) the value of H

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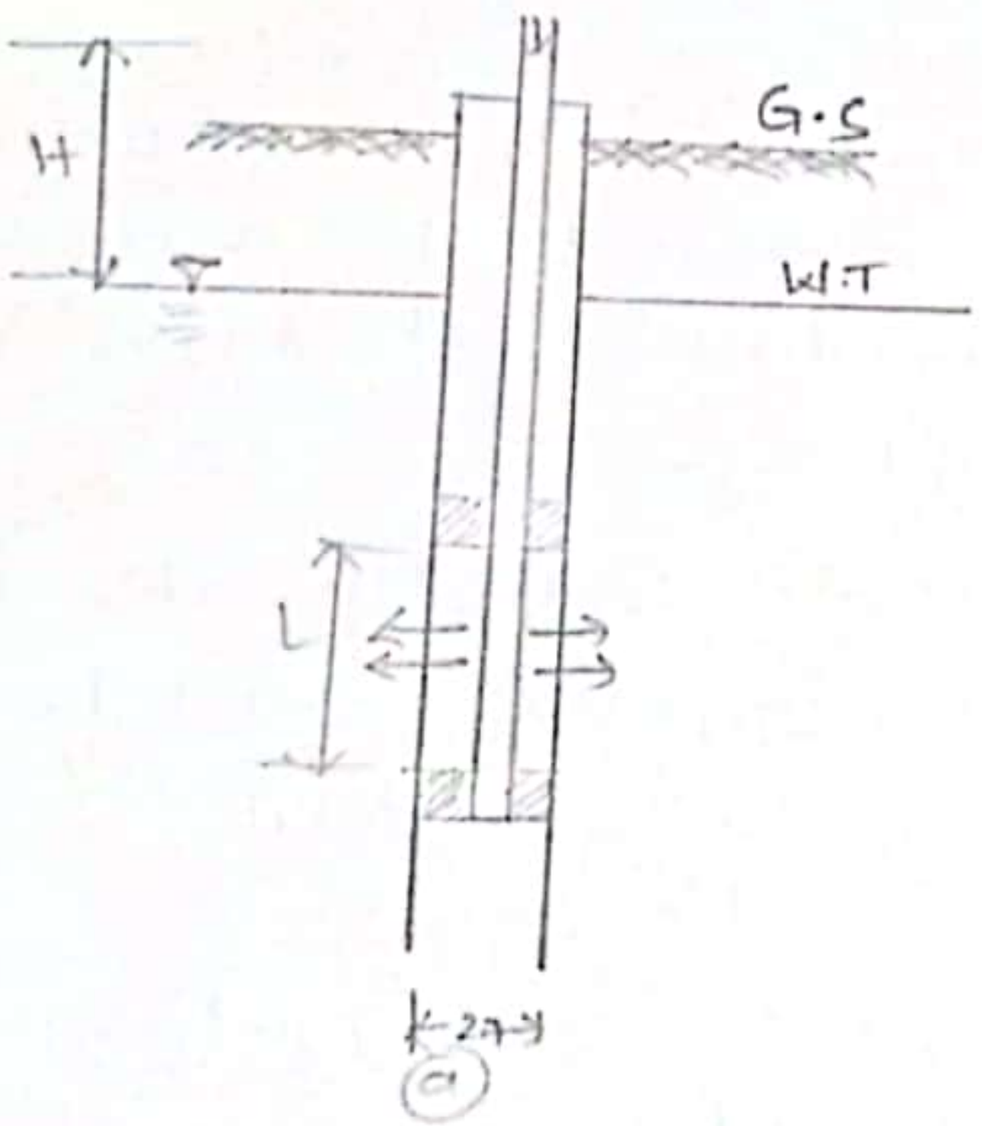
becomes $\left(H + \frac{P}{\gamma_w}\right)$, as in the case of an open-end tests.

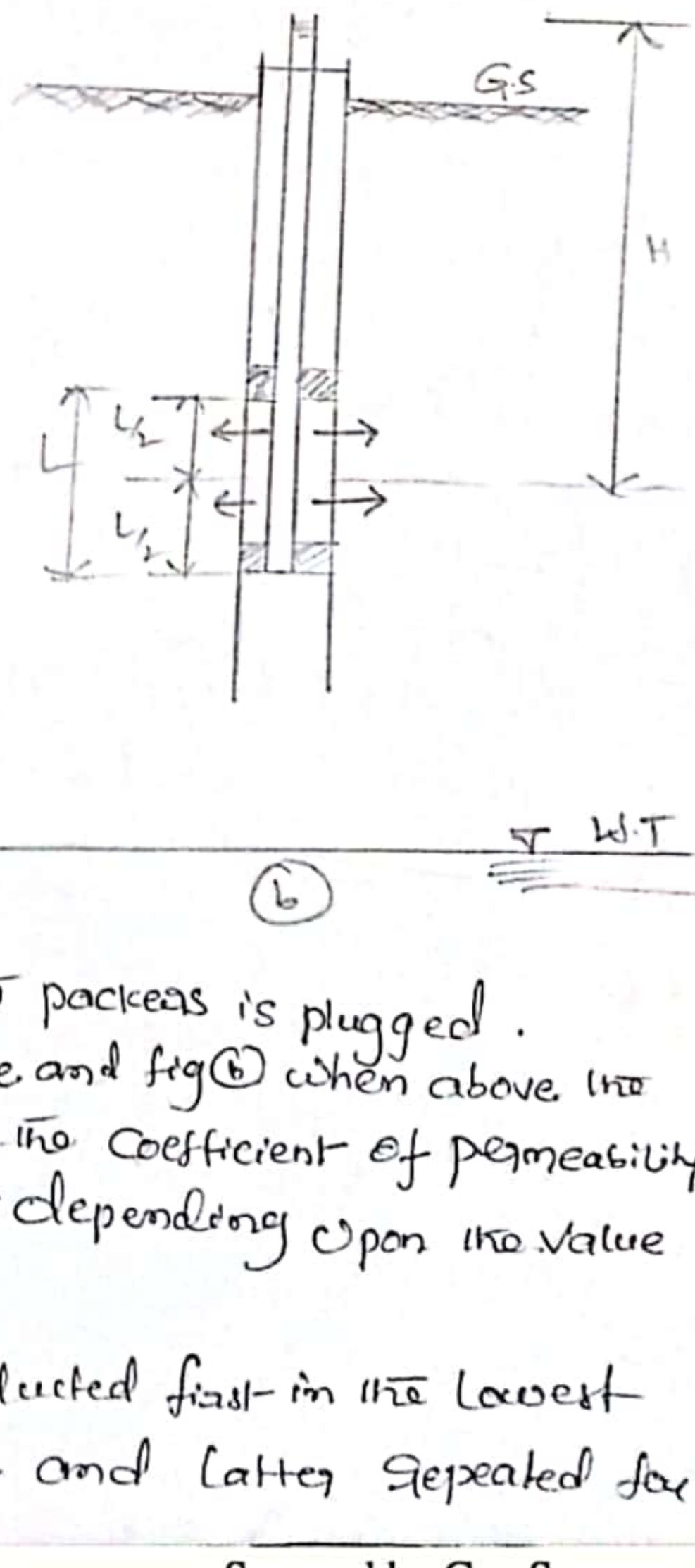
→ After the test is complete, the packer is removed if required the hole is made deeper and again a packer is placed and the test repeated for that position.

(b) Double-packer test :-

→ If the hole can stand without a casing, double-packer test can be used. The hole is drilled to the final depth.

→ it is filled with water surged and boiled out Two packers are fitted to a small diameter pipe. as shown in fig.





→ if the hole can the bottom of the pipe fitted with packers is plugged. (a) shows the ground water table and fig (b) when above the ground water table. the value of the coefficient of permeability is determined using equations depending upon the value of L and a as specified.

→ the double-packer test is conducted first in the lowest position near the bottom of the hole and latter repeated for

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for the upper layers.

→ the packer tests give better results when conducted below the water table than when above the water table for reliable results, the Thickness of the Stratum should be at least five times the length (L) of the hole tested.

(*) pumping-out Tests :-

→ the laboratory methods for the determination of the Coefficient of permeability, as discussed before, do not give correct results -

→ the Samples used are generally disturbed and do not represent the true in Situ structure.

→ For more accurate, representative Values, the field tests are Conducted. the field tests may be in the from of pumping out test wherein the water is pumped out from the wells drilled for this purpose. the other type of the field tests are pumping in tests, where in the water is pumped into the drilled holes, as discussed in the following article.

→ For large engineering projects, it is the usual practice to measure the permeability of Soils by pumping out tests wherein the water is pumped out from the wells drilled for this purpose.

→ the other type of the field tests are pumping in tests, wherein the water is pumped in to the drilled holes as discussed in the following article.

→ for large Engineering projects it is the usual practice to measure the permeability of soils by pumping out tests. the method is extremely useful for a homogeneous Coarse grained deposits for which is difficult to obtain undisturbed Samples.

→ In a pumping out test the Soil deposits over a large area is influenced.

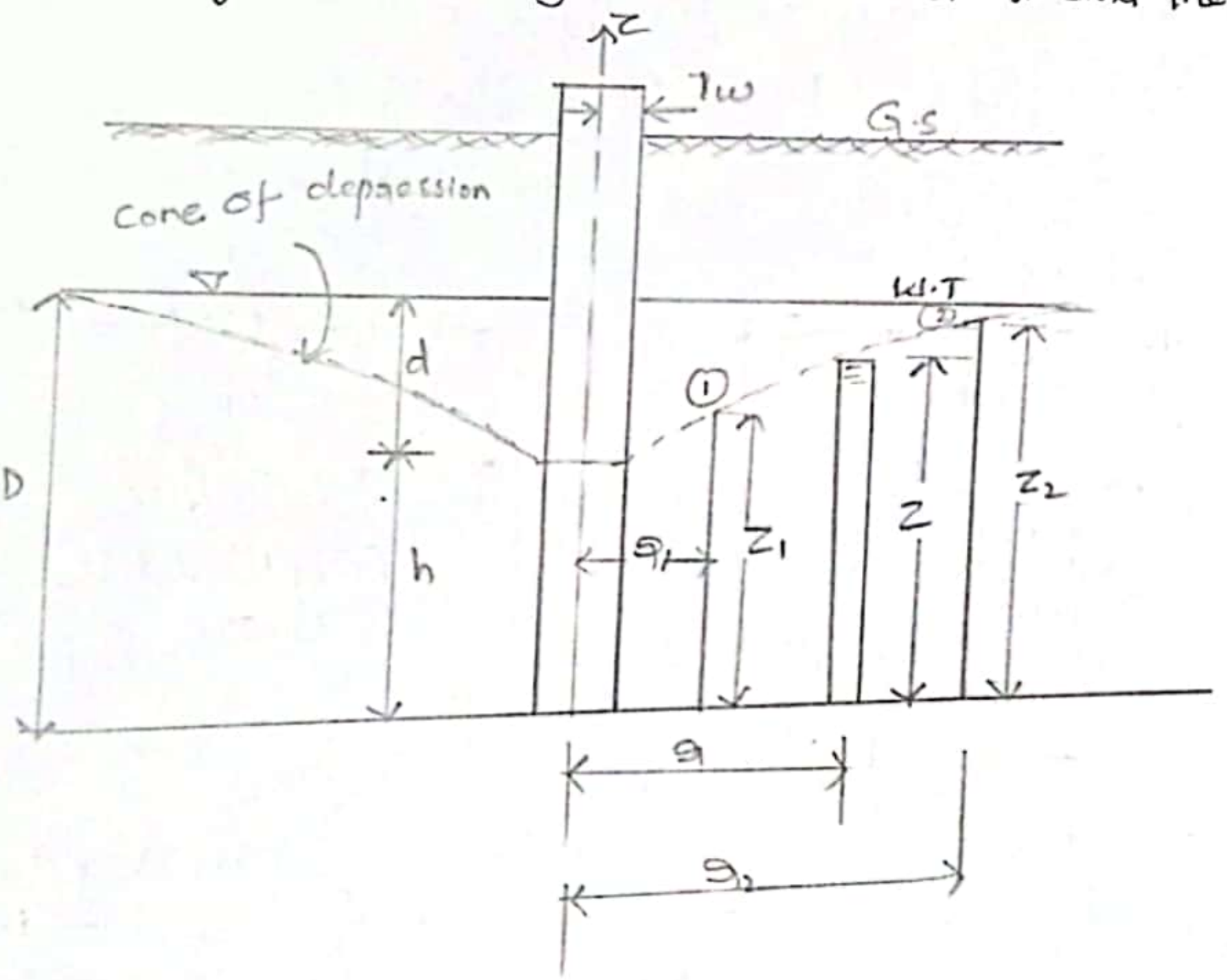
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(a) Unconfined Aquifer :

→ In an unconfined Aquifer a tube well is drilled as shown in fig the well reaches the underlying impervious stratum. the tube used for the well is perforated so that water can enter the well.

→ the tube is surrounded by a screen called strainer to check the flow of soil particles into the well.

→ water is pumped out of the tube well till a steady state is reached At that stage the discharge becomes constant and the water



level in the well does not change.

→ the water table which was originally horizontal before the pumping was started is depressed near the well.

→ the water table near the well forms and inverted cone known as the cone of depression. the maximum depression of the water table is known as the draw down (d).

→ the expression for the coefficient of permeability can be derived making the following assumptions

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→ The following assumptions known as Dupuit's assumption.

(i) The flow is laminar and Darcy's law is Valid.

(ii) The soil mass is Isotropic and homogeneous

(iii) The well penetrates the entire thickness of aquifer

(iv) The flow is Steady

(v) The Coefficient of permeability remains Constant Throughout.

(vi) The flow towards the well is radial and horizontal.

(vii) Natural ground water regime remains Constant.

(viii) The slope of the hydraulic gradient line is Small and can be taken as the tangent of the angle in place of the sine of angle. i.e

$$\boxed{i = \frac{dz}{dr}} \quad \text{---(i)}$$

→ Let us Consider the flow through a cylindrical Surface of height z at a radial distance of 'r' from the centre of the well. from Darcy's law

$$\boxed{q = k i A}$$

→ Substituting the Value of i from equation and taking A equal to $2\pi rz$.

$$q = k\left(\frac{dz}{dr}\right)(2\pi rz)$$

$$\frac{dr}{r} = \frac{2\pi k\, z\, dz}{q}$$

→ integrating

$$\int_1^2 \frac{dr}{r} = \frac{2\pi k}{q}\int_1^2 z\,dz$$

$$\log_e\left(\frac{r_2}{r_1}\right) = \frac{2\pi k}{q}\left(\frac{z_2^2 - z_1^2}{2}\right) \quad \text{---(ii)}$$

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$$K = \frac{q}{\pi(z_2^2 - z_1^2)} \log_e \left(\frac{r_2}{r_1}\right) \quad \text{— (2)}$$

$$K = \frac{2.30q}{\pi(z_2^2 - z_1^2)} \log_{10} \left(\frac{r_2}{r_1}\right) \quad \text{— (3)}$$

→ near the test well there is a rapid drop in head and the slope of the hydraulic gradient is steep, and assumption (5) is not satisfied.

→ The observation wells ① & ② should be drilled at considerable distance from the well for accurate measurement.

→ The radial distance r_1 of the well should be at least equal to the thickness of aquifer (D).

→ The observation wells are usually arranged in two orthogonal lines, one along the general direction of flow of the ground water and the other at right angle to this direction.

→ An approximate value of the coefficient of permeability can be determined if the radius of influence (R). is known as 'R' is estimated.

→ The circle of influence over which the effect of pumping is observed. extends to a very large area. In fact it gradually merges asymptotically to the water table. The radius of influence varies b/w 150 to 300 m.

→ According to Sichardt it can be found using the relation

$$R = 3000 d \sqrt{K}$$

where R = radius of influence (m), d = drawdown (m)

K = coefficient of permeability (m/s)

→ According to kozeny the radius of influence

$$R = \left[\left(\frac{12t}{n}\right)\left(\frac{2k}{\pi}\right)^{1/2}\right]^{1/2}$$

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where t is the time required to establish steady condition and n is the porosity.

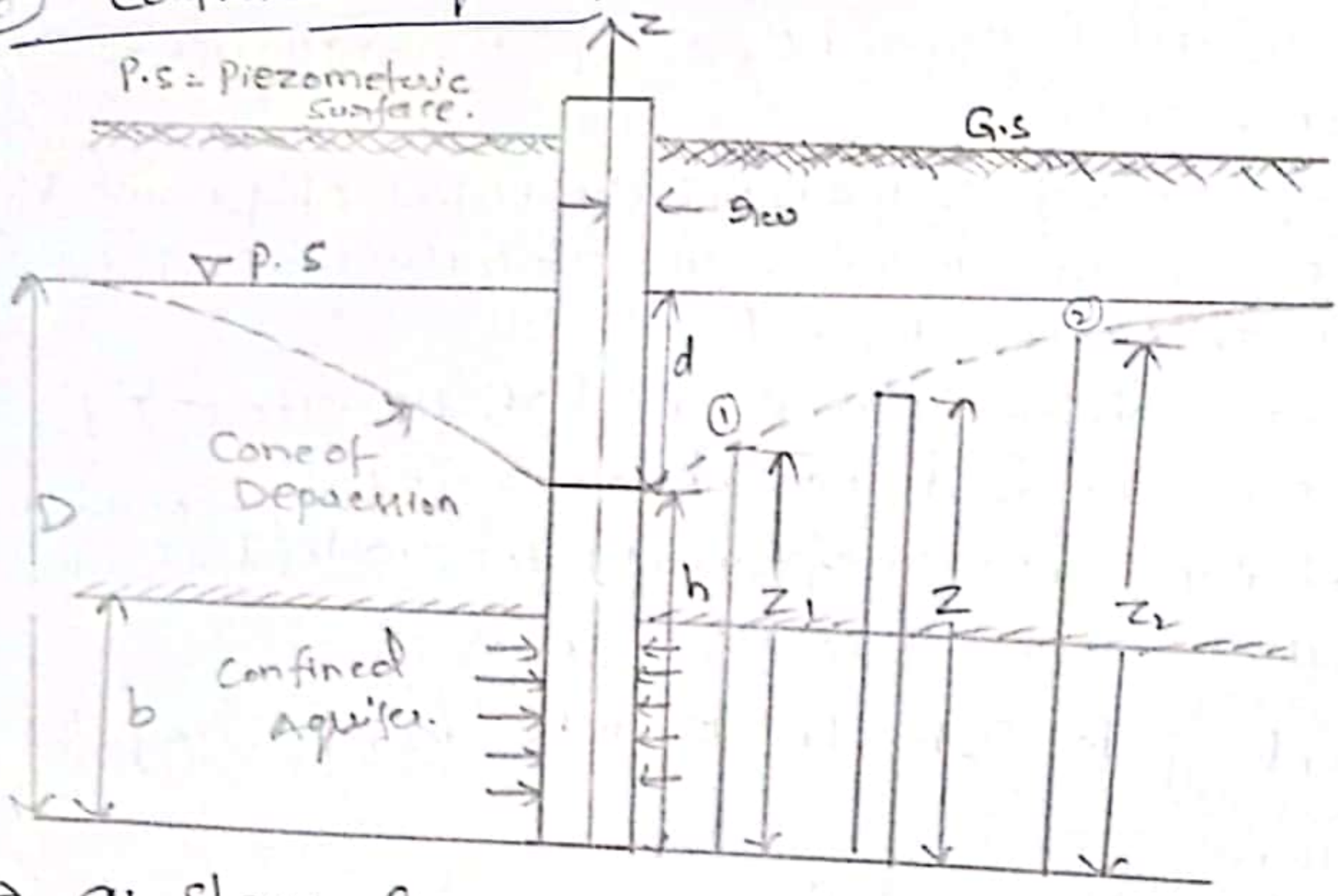
can be written as

$$K = \frac{q}{\pi (D^2 - h^2)} \log_e \left(\frac{R}{r_w}\right) \quad \text{--- (1)}$$

r_w = radius of test well
R = radius of influence.
D = depth of aquifer measured below the water table.
h = depth of water in the well.

→ equation an approximate value of the coefficient of permeability because the slope of the water surface near the well is steep and Dupuit's assumption is not justified further the value of the radius of influence (R) is also approximate.

(b) Confined Aquifer :



→ as shown fig a confined aquifer of thickness b and lying blw the two aquicludes.
→ the piezometric surface is above the top of the aquifer. in confined aquifer the water pressure is indicated by the piezometric surface (PS)
→ the piezometric surface is the water table equivalent

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for a confined aquifer. Initially the piezometric surface is horizontal when the pumping is started from the well, it is depressed and a cone of depression is formed.

→ The expression for the coefficient of permeability can be derived making the same assumptions as in the case of unconfined aquifer.

→ Let us consider the discharge through a cylindrical surface at a radial distance 'r' from the centre and of height z.

from Darcy's law.

$$q = KiA$$

$$q = k\left(\frac{dz}{dr}\right)(2\pi rb)$$

integrating $\int \log_e(r)_1^2 = \frac{2\pi kb}{q}(z)_1^2$

$$\boxed{k = \frac{q \log_e\left(\frac{r_2}{r_1}\right)}{2\pi b(z_2 - z_1)}} \quad \text{--- (1)}$$

$$\boxed{k = \frac{2.30q \log_{10}\left(\frac{r_2}{r_1}\right)}{2\pi b(z_2 - z_1)}} \quad \text{--- (2)}$$

where z_1 = height of water level in observation well (1) at a radial distance of r_1

z_2 = height of water level in observation well (2) at a radial distance of r_2.

→ As in the case of an unconfined aquifer an approximate value of k can be determined if the radius of influence R is known (or) estimated in this case

$$\boxed{k = \frac{q \log_e\left(\frac{R}{r_w}\right)}{2\pi b(D - h)}}.$$

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Ex:- ① In a constant head permeameter test the following observations were taken

Distance b/w piezometer tappings = 100mm

Difference of water levels in piezometer = 60mm.

Diameter of the test sample = 100mm

Quantity of water collected = 350 ml

Duration of the test = 270 sec

Determine the Coefficient of permeability of the soil.

Sol:-

$$K = \frac{qL}{Ah} \qquad q = \frac{350}{270} = 1.296 \text{ ml/sec}.$$

$$K = \frac{1.296 \times 10.0}{\frac{\pi}{4}(10)^2 \times 6} = 0.0275 \text{ cm/sec}.$$

② The coefficient of permeability of a soil at a void ratio of 0.7 is 4×10^{-4} cm/sec. Estimate its value at a void ratio of 0.50

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

→ As all the parameters remain constant except.

$$\frac{K_{0.7}}{K_{0.5}} = \frac{(0.70)^3}{(1+0.70)} \times \frac{(1+0.50)}{(0.50)^3}$$

$$= \frac{4 \times 10^{-4}}{K_{0.5}} = 2.421$$

$$K_{0.5} = 1.65 \times 10^{-4} \text{ cm/sec}.$$

Alternative method

$$K = 1.41 K_{0.85} e^2$$

$e = 0.50$) $\quad 4 \times 10^{-4} = 1.4 K_{0.85} \times (0.7)^2$

$$K_{0.85} = 5.83 \times 10^{-4}$$

$$K = 1.40 \times 5.83 \times 10^{-4} \times (0.5)^2 = 2.04 \times 10^{-4} \text{ cm/sec}.$$

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(X) Effective Stress :-

→ The effective stress principle enunciated by Karl Terzaghi in 1936 forms an extremely useful basis of the most important theories in Soil engineering.

(*) total stress :- → due to self weight of the soil (σ_s) due to external applied forces (σ_w) due to both, at any point inside a soil mass is resisted by the soil grains as also by water present in the pores (σ_w) void spaces in the case of a saturated soil.

(*) Neutral Stress :-

→ is defined as the stress carried by the pore water and it is the same in all directions when, there is static equilibrium since water cannot take static shear stress. This is also called pore water pressure and is designated by 'u' this will be equal to $\gamma_w \cdot z$ at a depth z below the water table

$$u = \gamma_w \cdot z$$

(*) Effective Stress :-

→ is defined as the difference b/w the total stress and the neutral stress; this is also referred to as the intergranular pressure and is denoted by

$$\bar{\sigma} = \sigma - u$$ (1) equation Skiert equation.

→ The effective stress has influence in decreasing the void ratio of the soil and in mobilising the shear strength while the neutral stress does not have any influence on the void ratio and is ineffective in mobilising the shearing strength.

(*) Seepage Through Soils :-

→ Seepage is the flow of water under gravitational forces in a permeable medium. Flow of water takes place from a point of high head to a point of low head. The flow is generally laminar

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→ The path taken by a water particle is represented by a flow line. Although an infinite number of flow lines can be drawn for convenience only a few are drawn.

→ At certain points on different flow lines, the total head will be the same.

→ The lines connecting points of equal total head can be drawn these lines are known as equipotential lines.

→ As flow always takes place along the steepest hydraulic gradient, the equipotential lines cross the flow lines at right angles.

→ The flow lines and equipotential lines together from a flow net. The flow net gives a pictorial representation of the path taken by water particles and the head variation along that path.

① principle of effective stress:

→ the effective stress principle consists of two parts.

(1) Definition of the effective stress.

(2) Importance of the effective stress in engineering behaviour of soil.

(1) Definition of effective stress

→ as shown in fig a soil mass which is fully saturated, Let us consider a prism of soil with a cross-sectional area. A the weight P of the soil in the prism is given by

$$P = \gamma_{sat} h A \quad \text{---(1)}$$

where γ_{sat} is the saturated weight of the soil and h is the height of the prism.

(2) Total stress (or) on the base of the prism is equal to the force per unit area.

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$$\boxed{\sigma = \frac{P}{A} = \gamma_{sat} h} \quad \text{—} (2)$$

→ while dealing with stresses it is more convenient to use in terms of unit weights rather than density.

$$\boxed{\gamma = \rho . g}$$

where γ is in N/m^3 and ρ is in kg/m^3, $g = 9.81\ m/sec^2$

Thus $\gamma_{sat} = \rho_{sat} \times g$

$= 9.81\ \rho_{sat}$

→ Generally, the unit weights are expressed in kN/m^3 and the mass density in kg/m^3 in that case.

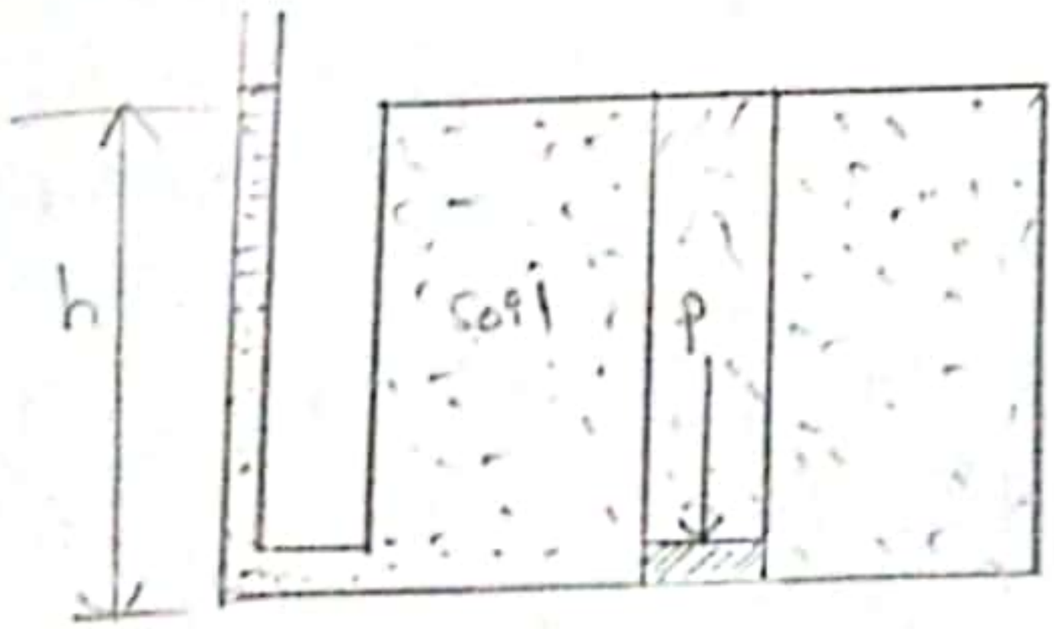
$$\boxed{\gamma_{sat} = \frac{\rho_{sat} \times g}{1000}} = 9.81 \times 10^3\ \rho_{sat} \text{ —}$$

for example if $\rho_{sat} = 2000\ kg/m^3$

$\gamma_{sat} = 9.81 \times 10^3 \times 2000 = 19.62\ kN/m^3$

$\gamma_{sat} \approx 0.01\ \rho_{sat}$ — (3)

$\gamma_{sat} = 0.01 \times 2000 = 20.00\ kN/m^3$



→ for convenience eqn (2) is sometimes used.

pore water pressure (u) is the pressure due to pore water filling the voids of the soil thus

$$\boxed{u = \gamma_w h}$$

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→ pore water pressure is also known as neutral pressure (u) neutral stress, because it cannot resist shear stresses. pore water pressure is taken as zero when it is equal to atmospheric pressure because in soil engineering the pressures used are generally gauge pressure and not absolute pressures.

→ the effective stress ($\bar{\sigma}$) at a point in the soil mass is equal to the total stress minus the pore water pressure.

thus $\boxed{\bar{\sigma} = \sigma - u}$ ——— (1)

for saturated soils, it is obtained as

$$\boxed{\bar{\sigma} = \gamma_{sat} h - \gamma_w h}$$

$$\bar{\sigma} = (\gamma_{sat} - \gamma_w) h$$

$$\boxed{\bar{\sigma} = \gamma' h}$$

where γ' is the submerged unit weight.

→ the effective stress is also represented by σ' in some texts.

it may be noted that the effective stress is an abstract quantity as it cannot be measured directly in the laboratory.

② Importance of effective stress :-

→ the effective stress controls the engineering properties of soils. Compression and shear strength of a soil are dependent on the effective stress. thus

Compression = $f(\bar{\sigma})$ where f & ϕ represent some functions.

Shear strength = $\phi(\bar{\sigma})$

→ As the effective stress in a soil increases, the compression of the soil occurs. This causes settlement of structures built on soils.

→ the shear strength of a soil depends on its effective stress. As the effective stress is changed the shear strength changes. the stability of slopes the earth pressures against retaining structure and the bearing capacity of soil depend upon the shear strength of the soil and hence the effective stress.

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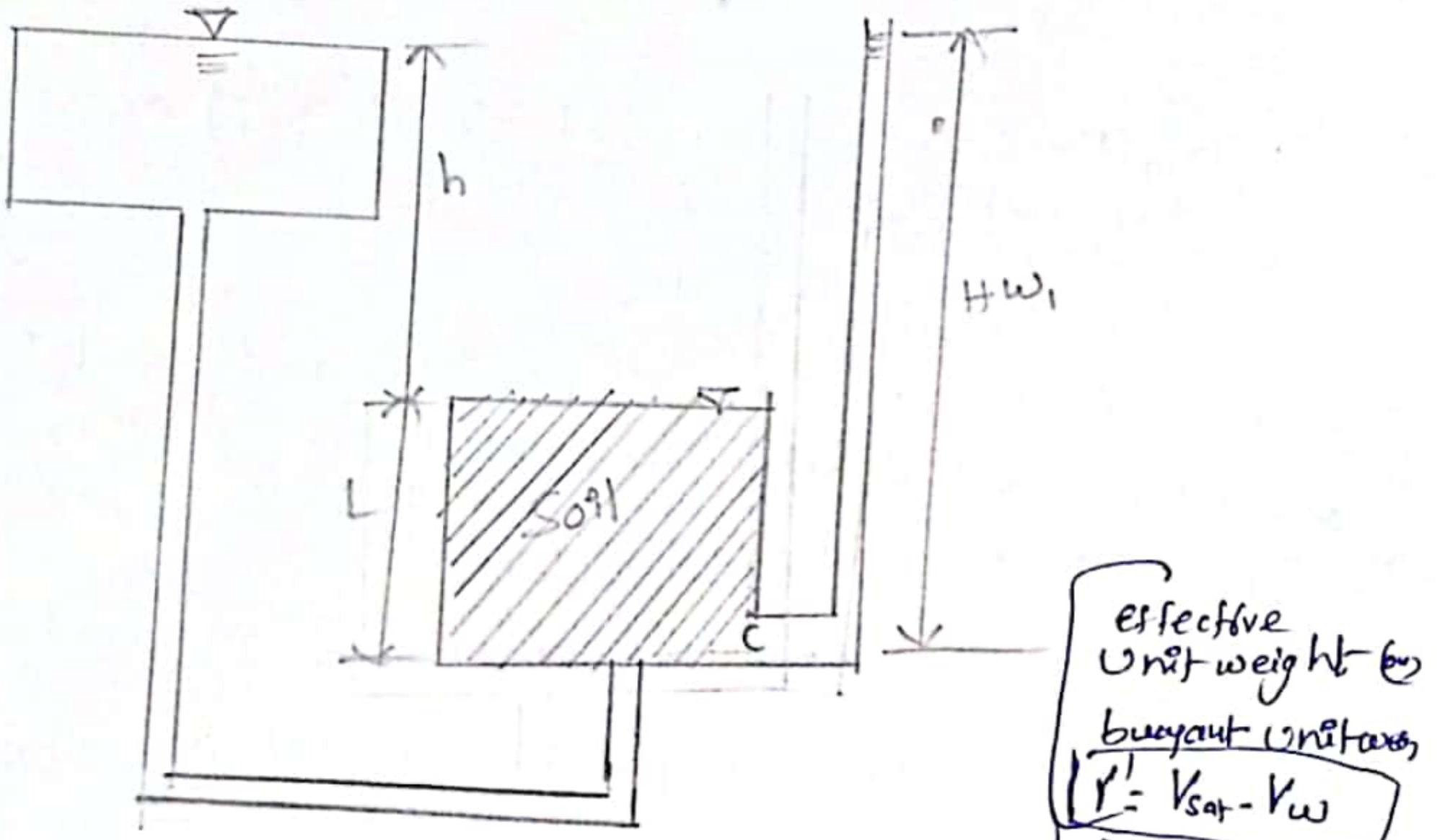
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(*) Quick Sand Conditions :-

→ The effective stress is reduced due to upward flow of water. When the head causing upward flow is increased a stage is eventually reached when the effective stress is reduced to zero.

→ The condition so developed is known as quick sand condition

a soil specimen of length 'L' subjected to an upward pressure. Let us consider the stresses developed at section C-C.



Quick Sand condition

$\sigma = \gamma_{sat} L = (\gamma' + \gamma_w) L$ .. $u = \gamma_w H_{w1} = \gamma_w (L+h)$

$\bar{\sigma} = (\gamma' + \gamma_w) L - \gamma_w (L+h)$

$\bar{\sigma} = \gamma' L - \gamma_w h$.

The second term can be written in terms of the gradient as under

$\gamma_w h = \gamma_w (h/L) . L = \gamma_w \cdot i \cdot L$

$\bar{\sigma} = \gamma' L - \gamma_w i L$

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

$= \gamma' L - \gamma_w \cdot h$

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→ The effective stress becomes zero if

$$\gamma' i_c = \gamma_w i_c$$

$$\boxed{i = \gamma'/\gamma_w}$$

→ The hydraulic gradient at which the effective stress becomes zero is known as the critical gradient (ic)

$$\boxed{i_c = \frac{\gamma'}{\gamma_w}} \quad \left\{ \gamma' = \frac{(G-1)\gamma_w}{1+e} \right\} \therefore \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e}$$

→ Substituting the value of the submerged unit weight in terms of void ratio

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

Taking the specific gravity of solids (G) as 2.67 and the void ratio (e) as 0.67.

$$i_c = \frac{2.67-1}{1+1.67} = 1.0$$

→ The effective stress becomes zero for the soil units above values of G and e when the hydraulic gradient is unity the head causing flow is equal to the length of the specimen

(a) The quick sand conditions may be summarised as under;

(1) Quick sand is not a special type of soil. it is a hydraulic condition.

(2) A cohesionless soil becomes quick when the effective stress is equal to zero.

(3) The critical gradient at which a cohesionless soil becomes quick is about unity.

(4) The discharge required to maintain a quick condition in a soil increases as the permeability of the soil increases.

(5) The cohesive soil does not becomes quick when the effective stress is equal to zero. as it still possesses some strength equal to the cohesion intercept.

(6) A quick condition is most likely to occurs in silt and fine

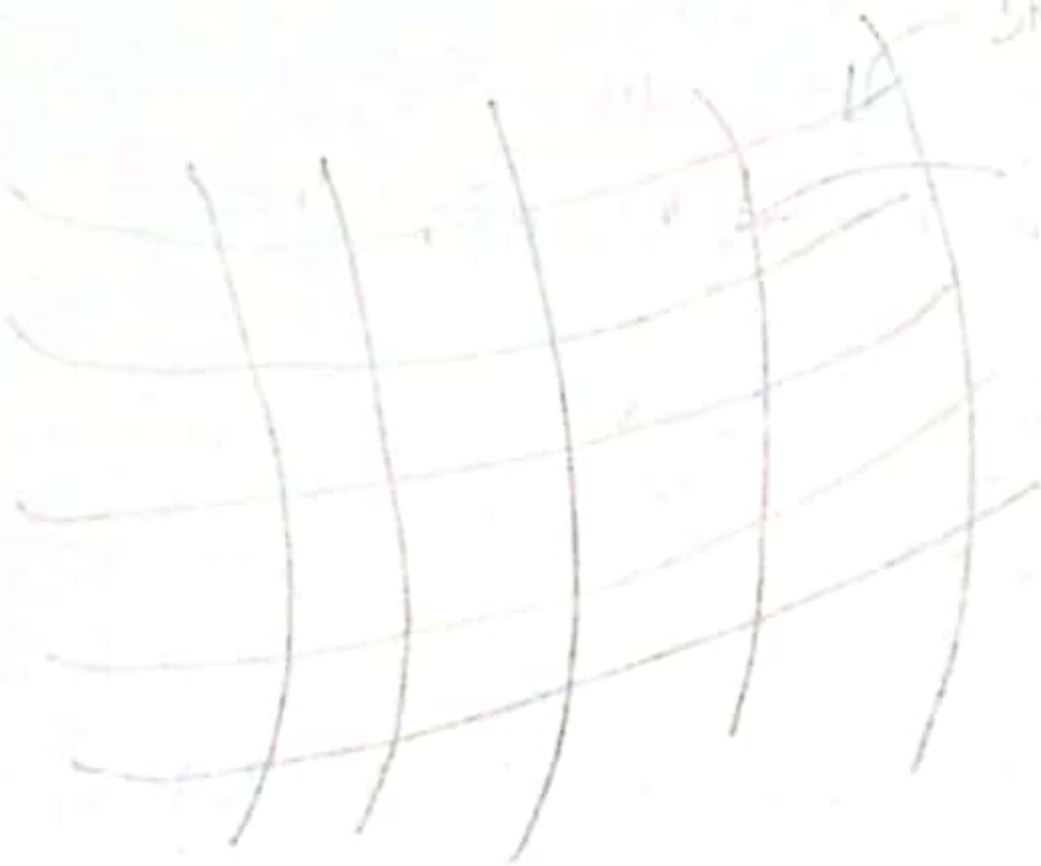
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(*) Flownets :-

→ The path taken by a water particle is represented by a flow line. Although an infinite number of flow lines can be drawn for convenience only a few are drawn.

→ At certain points on different flow lines the total head will be the same.

→ The lines connecting points of equal total head can be drawn these lines are known as equipotential lines. As flow always takes place along the steepest hydraulic gradient the equipotential lines cross flow lines at right angles the flow lines and equipotential lines together form a flow net. The flownet gives a pictorial representation of the path taken by water particles and the head variation along that path.



(*) Characteristics of flownets :-

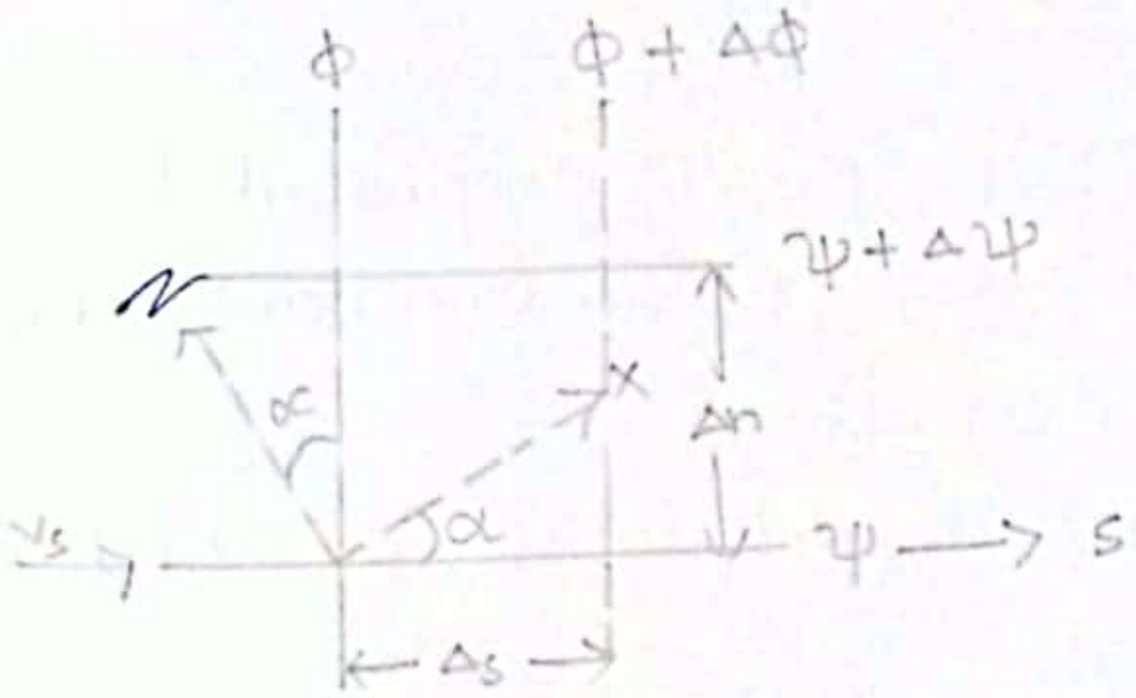
→ a flow field formed b/w two adjacent flow lines and equipotential lines. if v_s is the velocity along the stream lines represented by ψ.

$$v_x = v_s \cos\alpha \quad (1)$$

$$v_z = -v_s \sin\alpha \quad (2)$$

→ the potential function ϕ can be written as

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$$\frac{\partial \phi}{\partial s} = \frac{\partial \phi}{\partial x} \cdot \frac{\partial x}{\partial s} + \frac{\partial \phi}{\partial z} \cdot \frac{dz}{ds}$$

Using equation $\frac{\partial \phi}{\partial s} = V_x \cos\alpha + V_z(-\sin\alpha)$

Substituting the values of V_x and V_z from equation

$$\frac{\partial \phi}{\partial s} = V_s \cos^2\alpha + V_s \sin^2\alpha = V_s \quad \text{—— (3)}$$

$$\frac{\partial \psi}{\partial n} = \frac{\partial \psi}{\partial x} \cdot \frac{\partial x}{\partial n} + \frac{\partial \psi}{\partial z} \cdot \frac{dz}{dn}$$

$$\frac{\partial \psi}{\partial n} = -V_z \sin\alpha + V_x \cos\alpha$$

$$\frac{\partial \psi}{\partial n} = V_s \sin^2\alpha + V_s \cos^2\alpha = V_s$$

$$\frac{\partial \phi}{\partial s} = \frac{\partial \psi}{\partial n}$$

$$\boxed{\frac{\Delta \phi}{\Delta s} = \frac{\Delta \psi}{\Delta n}} \quad \text{—— (4)}$$ the flow net must satisfy eq (4)

→ it is convenient to construct the flow net such that the change in stream function ($\Delta\psi$) b/w two adjacent flow lin and the change in potential function ($\Delta\phi$) b/w two adjacent equipotential lines are constant:

$$\therefore \frac{\Delta \phi}{\Delta \psi} = \text{constant}$$

$$\frac{\Delta s}{\Delta n} = \text{constant}.$$

→ Although any fixed ratio $\frac{\Delta s}{\Delta n}$ can be used for convenience

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$\frac{\Delta s}{\Delta n}$ is kept Unity.

∴ in actual practice the flow net consists of approximate Square.

$\Delta s = \Delta n$.

→ the distance b/w two adjacent flow lines is equal to the distance b/w two adjacent equipotential lines.

(*) the characteristics of flow net can be Summarised as Under

① the fundamental Condition that is to be Satisfied is that every intersection b/w a flow lines & an equipotential lines Should be at right angles.

② the Second Condition to be Satisfied is that the discharge (Δq) b/w any two adjacent flow lines is Constant and the drop of head (Δh) b/w the two adjacent equipotential lines is Constant.

③ the ratio of the length and width of each field ($\frac{\Delta s}{\Delta h}$) is Constant. The ratio is generally taken as Unity for Convenience In other words the flow net consists of approximate Squares.

(*) Uses of flownets :-

→ the flow net can be used for a number of purposes as explained below.

① Discharge :-

→ the Space b/w two adjacent flow lines is called a flow channel Let Nf be the number of flow channels.

→ the difference b/w two adjacent equipotential line is Called equipotential drop.

→ Let Nd be the no. of equipotential drops. There are 5 flow channels and 10 equipotential drops.

→ Let us consider the flow through the flow field shown in

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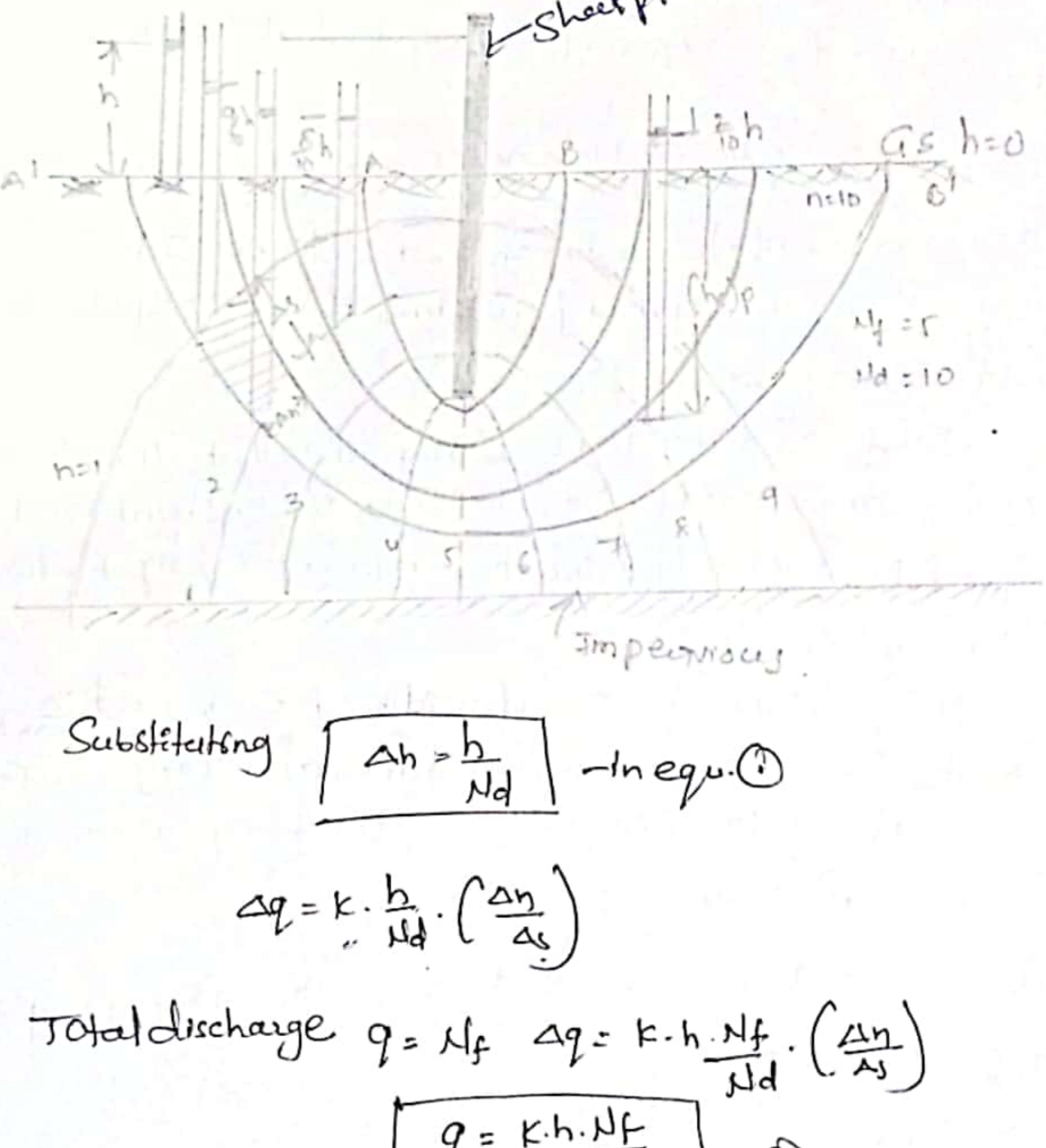
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→ From Darcy's Law discharge through the flow field per unit length $\Delta q = k i A$

$$\boxed{\Delta q = k \cdot \left(\frac{\Delta h}{\Delta s}\right)(\Delta n \times 1)} \quad \text{--- (1)}$$

Δh is equipotential drop in the flow field

Δs & Δh are dimensions of the flow field.



Substituting $\boxed{\Delta h = \frac{h}{N_d}}$ — in equ. (1)

$$\Delta q = k \cdot \frac{h}{N_d} \cdot \left(\frac{\Delta n}{\Delta s}\right)$$

Total discharge $q = N_f \ \Delta q = k \cdot h \frac{N_f}{N_d} \cdot \left(\frac{\Delta n}{\Delta s}\right)$

$$\boxed{q = k \cdot h \cdot \frac{N_f}{N_d}} \quad \text{--- (2)}$$

$$q = k \times h \times \frac{5}{10} = 0.5 k h.$$

→ The ratio $\left(\frac{N_f}{N_d}\right)$ is a characteristic of the flow net it is known as Shape factor (β) it is independent of the permeability (k) of the soil.

→ it depends only on the Configuration or the shape of the soil mass it's not necessary that N_f & N_d be always full integers

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(2) Total head :-

→ The loss of head (Δh) from one equipotential line to the next is h/Nd.. The total head at any point (P) can be determined as under.

$$h_p = h - n \times \left(\frac{h}{Nd}\right) \quad \text{— (1)}$$

where n is the no. of the equipotential drop upto point P.

n = 8 for point P.

∴ total head at P is

$$h_p = h - 8 \times \left(\frac{h}{10}\right) = 0.2h$$

→ It may be noted that if piezometers were placed at different points on the same equipotential line water would arise in these piezometers to the same elevation.

(3) pressure head :-

→ The pressure at any point is equal to the total head minus the elevation head As mention above the downstream water level is generally taken as datum.

for point P. the pressure head is given by

$$(h_p)_p = h_p - (-(h_e)_p)$$

$$= h_p + (h_e)_p$$

where $(h_p)_p$ = pressure head at P and $(h_e)_p$ = elevation head at P and h_p is the total head.

(4) Hydraulic gradient :-

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

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

Δs is the length of the flow field and Δh is the loss of head. the hydraulic gradient is generally maximum

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Example

(1) In a constant head permeameter test. the following observations were taken

Distance b/w piezometer tappings = 100 mm

Difference of water levels in piezometer = 60 mm

Diameter of the test sample = 100 mm

Quantity of water collected = 350 ml

Duration of the test = 270 seconds.

Determine the coefficient of permeability of the soil.

Sol:

$$\boxed{K = \frac{qL}{Ah}}$$

$$q = \frac{350}{270} = 1.296 \text{ ml/sec}.$$

$$K = \frac{1.296 \times 10.0}{\frac{\pi}{4} \times (10)^2 \times 6.0} = 0.0275 \text{ cm/sec}.$$

(*) The coefficient of permeability of a soil at a void ratio of 0.7 is 4×10^{-4} cm/s. Estimate its value at a void ratio of 0.50

Sol:

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

As all the parameters remains constant

$$\frac{K_{0.7}}{K_{0.5}} = \frac{(0.70)^3}{(1+0.70)} \times \frac{(1+0.5)}{(0.5)^3}$$

$$\frac{4 \times 10^{-4}}{K_{0.5}} = 2.421$$

$$K_{0.5} = 1.65 \times 10^{-4} \text{ cm/sec}$$

from $K = 1.4 K_{0.85} e^2$

$$4 \times 10^{-4} = 1.4 K_{0.85} \times (0.7)^2$$

$$K_{0.85} = 5.83 \times 10^{-4}$$

$e = 0.50$ $K = 1.40 \times 5.83 \times 10^{-4} (0.5)^2$

$= 2.04 \times 10^{-4}$ cm/sec.

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(8) A Sand deposit is 10m Thick and overlies a bed of soft clay. The ground water table is 3m below the ground surface if the Sand above the ground water table has a degree of Saturation of 45%. plot the diagram showing the Variation of the total stress, pore water pressure and the effective stress. The void ratio of the Sand is 0.70 Take G = 2.65

Sol:-

$$\text{Bulk density. } \rho = \left(\frac{G+Se}{1+e}\right)\rho_w$$

$$\rho = \left(\frac{2.65+0.45\times0.70}{1+0.70}\right)\times 1000$$

$$= 1744.12 \text{ kg/m}^3 \qquad \gamma = \rho g$$

$$\gamma = 1744.12 \times 9.81 \times 10^{-3}$$

$$= 17.11 \text{ kN/m}^3$$

For Saturated Soils. $S = 1.0$ &

$$\rho = \left(\frac{2.65+0.70}{1+0.70}\right)\times 1000$$

$$= 1970.59 \text{ kg/m}^2$$

$$\gamma = 1970.59 \times 9.81 \times 10^{-3}$$

$$= 19.33 \text{ kN/m}^3$$

The Soil profile. The Stresses at Section B-B & C.

$$\sigma = 17.11 \times 3 = 51.33 \text{ kN/m}^2 \qquad u = 0$$

$$\bar{\sigma} = 51.33 \text{ kN/m}^2 \qquad \bar{\sigma} = \sigma - u$$

$$\sigma = 17.11\times3 + 19.33\times7 \qquad \sigma = \gamma_{sat} \times h - \gamma_w$$

$$= 186.64 \text{ kN/m}^2 \qquad \bar{\sigma} = \gamma' h$$

$$u = 7\times9.81 = 68.67 \text{ kN/m}^2 \qquad u = \gamma_w . z$$

$$\bar{\sigma} = 186.64 - 68.67$$

$$= 117.97 \text{ kN/m}^2$$

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: GEOTECHNICAL ENGINEERING

SUBJECT CODE: 20CE5PC03

FACULTY NAME: B. SURESH

SYLLABUS:

UNIT - III

Stress Distribution in Soils: Bossiness's and Westergaard's theories for point load, uniformly loaded circular and rectangular areas, pressure bulb, variation of vertical stress under point load along the vertical and horizontal plane, and Newmark's influence chart for irregular areas. Compaction: Mechanism of compaction – factors affecting compaction – effects of compaction on soil properties – Field compaction Equipment – compaction quality control.

Unit - III

Stress Distribution In soil

(*) Boussinesq Theories for point load:

→ The Boussinesq gave the theoretical solution; for, the Stress distribution in an elastic medium Subjected to a concentrated load on its Surface

→ The Solutions are Commonly used to obtain the Stresses in a Soil mass due to externally applied load.

(*) The following assumptions are made:

(1) The Soil mass in an elastic Continuum having a constant Value of modulus of elasticity (E) i.e the ratio b/w the Stress and Strain is Constant

(2) The Soil is homogeneous i.e it has identical properties at different points.

(3) The Soil is Isotropic i.e it 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 & extends to infinity in all other direction.

(5) The Soil is weightless and is free from residual Stress before the application of the load.

Derivation:

→ Shown afig a horizontal Surface of the elastic Continuum Subjected to a point Load "Q" at a point O

→ The origin of the Coordinates is taken at 'O' using Laplace's Stress function for the Solution of elasticity problem Boussinesq proved that the polar Stress σ_R at point P(x, y, z) is given by

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$$\sigma_R = \frac{3}{2\pi} \frac{Q \cos\beta}{R^2} \quad \text{---(1)}$$

where R = polar distance b/w the origin O and point P
β = angle which the line OP makes with the vertical

obviously

$$R = \sqrt{x^2+y^2+z^2}$$

$$R = \sqrt{r^2+z^2} \quad \text{where } r^2 = x^2+y^2$$

$\sin\beta = r/R$ & $\cos\beta = z/R$

→ the vertical stress (σ_z) at point P is given by

$$\sigma_z = \sigma_R \cos^2\beta$$

$$= \frac{3}{2\pi}\left(\frac{Q\cos\beta}{R^2}\right)\cos^2\beta$$

$$\sigma_z = \frac{3Q}{2\pi} \frac{\cos^3\beta}{R^2}$$

$$\sigma_z = \frac{3Q}{2\pi} \cdot \frac{\left(\frac{z}{R}\right)^3}{R^2}$$

$$= \frac{3Q}{2\pi} \cdot \frac{z^3}{R^5}$$

$$\sigma_z = \frac{3Q}{2\pi} \frac{1}{z^2} \cdot \frac{z^5}{R^5}$$

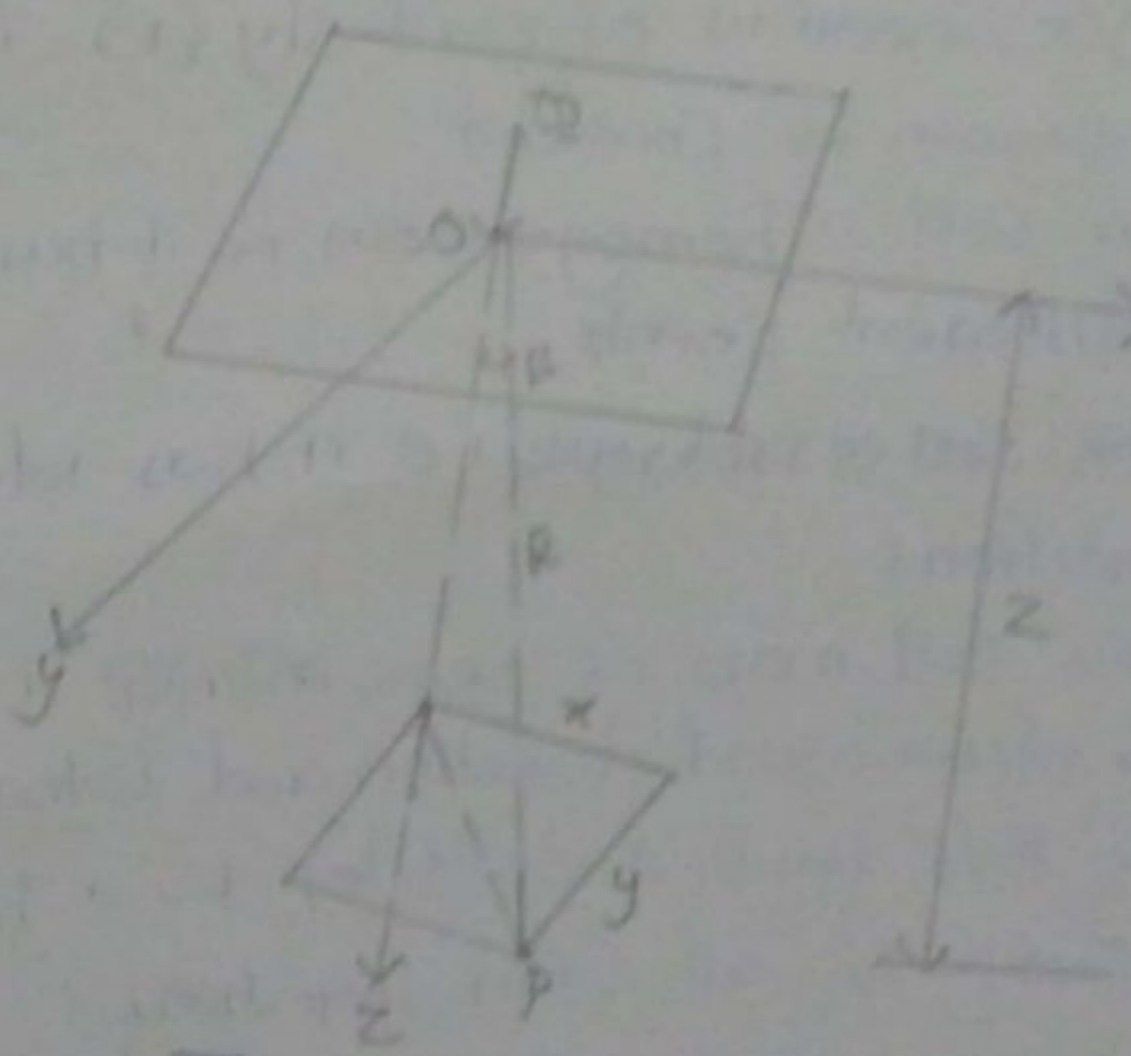
$$\sigma_z = \frac{3Q}{2\pi} \cdot \frac{1}{z^2} \left[\frac{z^5}{(r^2+z^2)^{5/2}}\right]$$

$$\sigma_z = \frac{3Q}{2\pi} \cdot \frac{1}{z^2} \cdot \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} \quad \text{---(2)}$$

$$\sigma_z = I_B \frac{Q}{z^2} \quad \text{---(3)}$$

$$I_B = \frac{3}{2\pi\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} \quad \text{---(4)}$$

→ The coefficient I_B is known as the Boussinesq influence coefficient for the vertical stress



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(*) Limitations of Boussinesq's Solution:

→ The solution was initially obtained for determination of stresses in elastic solids. Its application to soils may be questioned as the soils are far from purely elastic solids.

→ The application of Boussinesq's solution can be justified when the stress changes are such that only a stress increase occurs in the soil.

→ The real requirement for use of the solution is not that the soil be elastic but it should have a constant ratio b/w stress and strain.

→ when the stress decrease occurs the relation b/w stress and strain is not linear and therefore the solution is not strictly applicable.

→ If the stresses induced in the soil are small in comparision with the shear strength of the soil the soil behaves somewhat elastically and the Boussinesq solution can be used.

(*) Westergaard's Solution:-

⇒ Boussinesq' solution assumes that the soil deposit is isotropic Actual sedimentary deposits are generally anisotropic.

→ These are generally thin layers of sand embedded in homogeneous clay strata.

→ westergaard's solution assumes that these are thin sheets of rigid materials sandwiched in a homogeneous soil mass.

→ These thin sheets are closely spaced and are of infinite rigidity and are therefore incompressible.

→ These permit only downward displacement of the soil mas as a whole without any lateral displacement, therefore westergaard's solution represents more closely the actual sedimentary deposits.

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→ According to Westergaard, the vertical stress at a point P at a depth z below the Concentrated load 'Q' is given by

$$\sigma_z = \frac{c/2\pi}{\left[c^2+\left(\frac{r}{cz}\right)^2\right]^{3/2}} \cdot \frac{Q}{z^2} \quad \text{---(1)}$$

where c depends upon the poisson Ratio(ν) and is given by

$$c = \sqrt{(1-2\nu)/(2-2\nu)}$$

→ for an elastic material the value of ν varies b/w 0.0 to 0.50 for the case when ν is zero. it is simplified Considerably, taking $c = \frac{1}{\sqrt{2}}$

$$\sigma_z = \frac{1}{\pi\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}} \cdot \frac{Q}{z^2} \quad \text{---(2)}$$

$$\sigma_z = I_w \frac{Q}{z^2} \quad \text{---(3)}$$

where is known as Westergaard influence coefficient

$$I_w = \frac{1}{\pi\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}} \quad \text{---(4)}$$

→ the values of I_w are Considerably Smaller than the Boussinesq influence factor (I_B)

(*) Uniform Load on Circular Area :

→ this problem may arise in Connection with Settlement Studies of Structures on circular foundations, such as gasoline tanks, grain elevators, and storage bins.

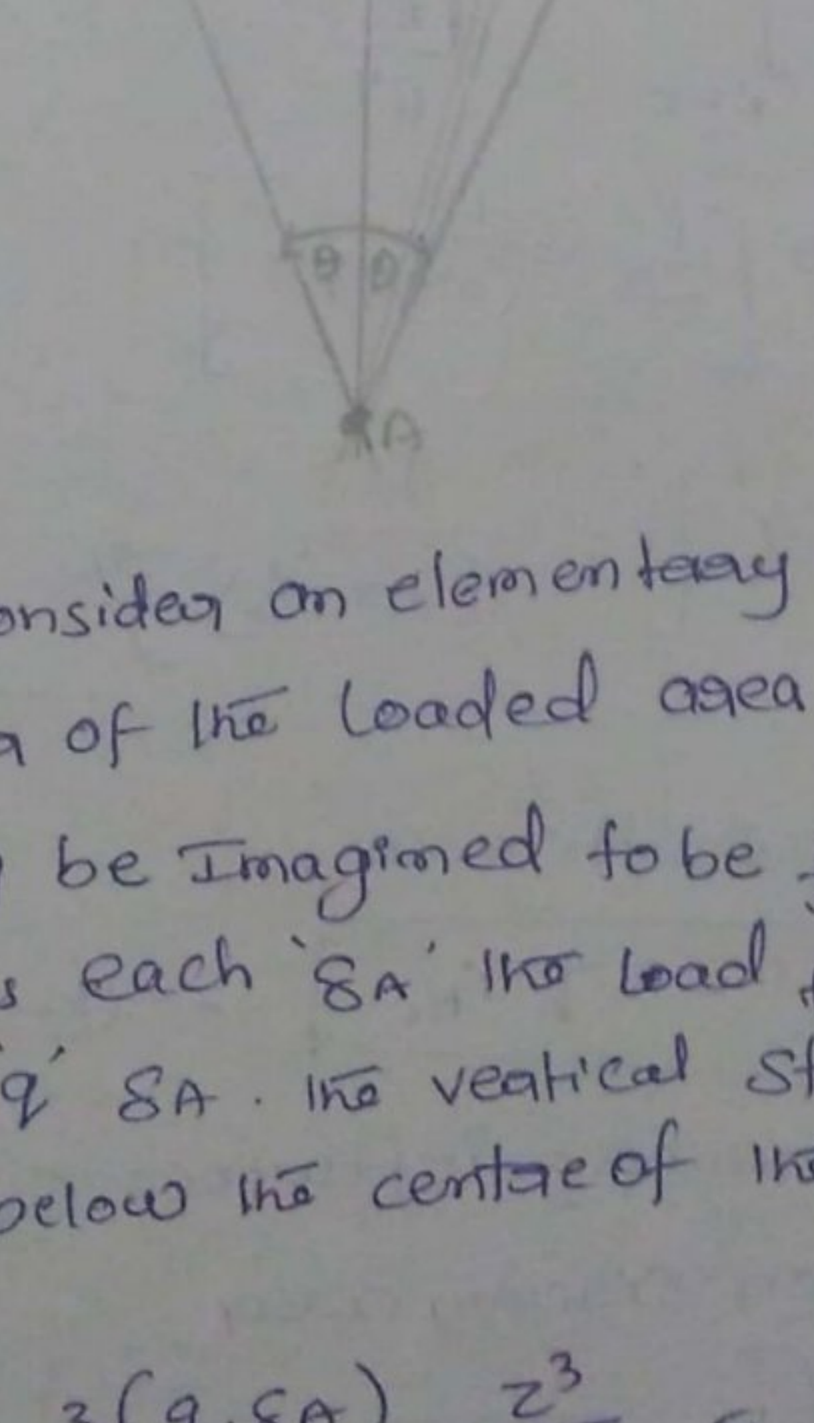
→ the Boussinesq equation for the vertical stress due to

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- a point load can be extended to find the vertical stress at any point beneath the centre of a uniformly loaded circular area.

→ Let the circular area of radius a be loaded uniformly with q per unit area as shown in fig



→ Let us consider an elementary ring of radius 'r' and thickness dr of the loaded area.

→ This ring may be Imagined to be further divided into to elemental areas each 'δA' the load from such an elemental is 'q' δA. the vertical stress $\delta\sigma_z$ at point 'A' at a depth z below the centre of the loaded area is given by.

$$\delta\sigma_z = \frac{3(q \cdot \delta A)}{2\pi} \cdot \frac{z^3}{(r^2+z^2)^{5/2}}$$

→ the stress $d\sigma_z$ due to the entire ring is given by

$$d\sigma_z = \frac{3q}{2\pi}(\Sigma \delta A) \cdot \frac{z^3}{(r^2+z^2)^{5/2}}$$

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$$= \frac{3q}{2\pi} \times (2\pi r\, dr) \cdot \frac{z^3}{(r^2+z^2)^{5/2}}$$

$$d\sigma_z = \frac{3qz^3 \cdot r\, dr}{(r^2+z^2)^{5/2}}$$

→ The total vertical stress σ_z at A due to entire loaded area is obtained by integrating $d\sigma_z$ with the limits $r=0$ to $r=a$

$$\sigma_z = 3qz^3 \int_{r=0}^{r=a} \frac{r\, dr}{(r^2+z^2)^{5/2}}$$

Setting $r^2+z^2=R^2$, $r\,dr = R\,dR$ the limits for R will be z and $(a^2+z^2)^{1/2}$

$$\sigma_z = 3qz^3 \int_{R=z}^{R=(a^2+z^2)^{1/2}} \frac{dR}{R^4}$$

$$= qz^3 \left[\frac{1}{z^3} - \frac{1}{(a^2+z^2)^{3/2}}\right]$$

$$\boxed{\sigma_z = q\left[1 - \frac{1}{\left[1+\left(\frac{a}{z}\right)^2\right]^{3/2}}\right]} \quad \text{— (1)}$$

This may be written as

$$\boxed{\sigma_z = q \cdot K_{Bc}} \quad \text{— (2)}$$

where K_{Bc} = Boussinesq influence coefficient for uniform load on circular area.

$$\boxed{K_{Bc} = \left[1 - \frac{1}{\left\{1+\left(\frac{a}{z}\right)^2\right\}^{3/2}}\right]} \quad \text{— (3)}$$

→ if θ is the angle made by OA with the tangent of the periphery of the loaded area from A

$$\boxed{\sigma_z = q(1-\cos^3\theta)} \quad \text{— (4)}$$

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→ The vertical stress at a point not lying on the vertical axis through the centre of the loaded area may also be found; but it requires the evaluation of a more difficult integral.

→ Spangler gives the influence coefficients for both cases the values for the case where the point lies directly beneath the centre of the loaded area being those in the column for $\frac{a}{a} = 0$

→ if Westergaard's theory is to be used, equation may be integrated to obtain the vertical stresses at a point beneath the centre of a uniformly loaded circular area which is given by

$$\sigma_z = q\left[1 - \frac{1}{\sqrt{1+\left(\frac{a}{nz}\right)^2}}\right] \quad \text{—(5)}$$

where $n = \sqrt{\frac{1-2\nu}{2-2\nu}}$, ν being passon's ratio

for $\nu = 0$ $\quad n = \frac{1}{\sqrt{2}} = 0.707$

(*) Uniform Load on Rectangular Area:

→ The more common shape of a loaded area in foundation engineering practice is a Rectangle especially in the case of ~~built~~ buildings.

→ Applying the principle of integration, one can obtain the vertical stress at a point at a certain depth below the centre or a corner of a uniformly loaded rectangular area, based either on Boussinesq's or on Westergaard's solution for a point load.



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① Uniform load on Rectangular Area based on Boussinesq's theory:

→ Newmark (1935) has derived an expression for the Vertical Stress at a point below the corner of a rectangular area ~~based either on Boussinesq's (or) on Westergaard's solution~~ for loaded uniformly as shown in

→ The following are the two popular forms of Newmark's equation for σ_z

$$\sigma_z = \frac{q}{4\pi}\left[\left(\frac{2mn\sqrt{m^2+n^2+1}}{(m^2+n^2+1+m^2n^2)}\right)\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \sin^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\right)\right] \quad \text{--- (1)}$$

$$\sigma_z = \frac{q}{4\pi}\left[\left(\frac{2mn\sqrt{m^2+n^2+1}}{(m^2+n^2+1+m^2n^2)}\right)\cdot\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \tan^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{(m^2+n^2+1-m^2n^2)}\right] \quad \text{--- (2)}$$

where $m = \frac{B}{z}$ & $n = \frac{L}{z}$

→ The second term within the brackets is an angle in radians

It is of interest to note that the above expressions do not contain the dimension z.

may be written in the form

$$\boxed{\sigma_z = q \cdot I_\sigma}$$

I_σ = influence value

$$= \left(\frac{1}{4\pi}\right)\left[\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\right)\cdot\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)\right] \quad \text{--- (3)}$$

→ Based on this equation Fadum has prepared a chart for influence values for sets.

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(2) Uniform Load on Rectangular Area based on Westergaard Theory

→ If the soil conditions correspond to those assumed in Westergaard's theory that is the soil consists of very thin horizontal sheets of infinite rigidity which prevent the occurrence of any lateral strain and the vertical stress at a point below a corner of a uniformly loaded rectangular area then it may be obtained by integration of the stress due to a point load under similar conditions. and shown to be

$$\sigma_z = \left(\frac{q}{2\pi}\right)\left[\cot^{-1}\sqrt{\left(\frac{1-2v}{2-2v}\right)\left(\frac{1}{m^2}+\frac{1}{n^2}\right)+\left(\frac{1-2v}{2-2v}\right)^2\cdot\frac{1}{m^2n^2}}\right] \quad \text{--- (1)}$$

→ if the poisson's ratio. v. is taken as zero this reduced to

$$\sigma_z = \left(\frac{q}{2\pi}\right)\left[\cot^{-1}\sqrt{\frac{1}{2m^2}+\frac{1}{2n^2}+\frac{1}{4m^2n^2}}\right] \quad \text{--- (2)}$$

→ the notation is the same as that for equations

$$\boxed{\sigma_z = q \cdot I_{\sigma w}}$$

where $I_{\sigma w}$: influence coefficient for vertical stress at the corner of a uniformly loaded Rectangular area from Westergaard's Theory.

(3) pressure bulb '- (or) Isobar Diagram'

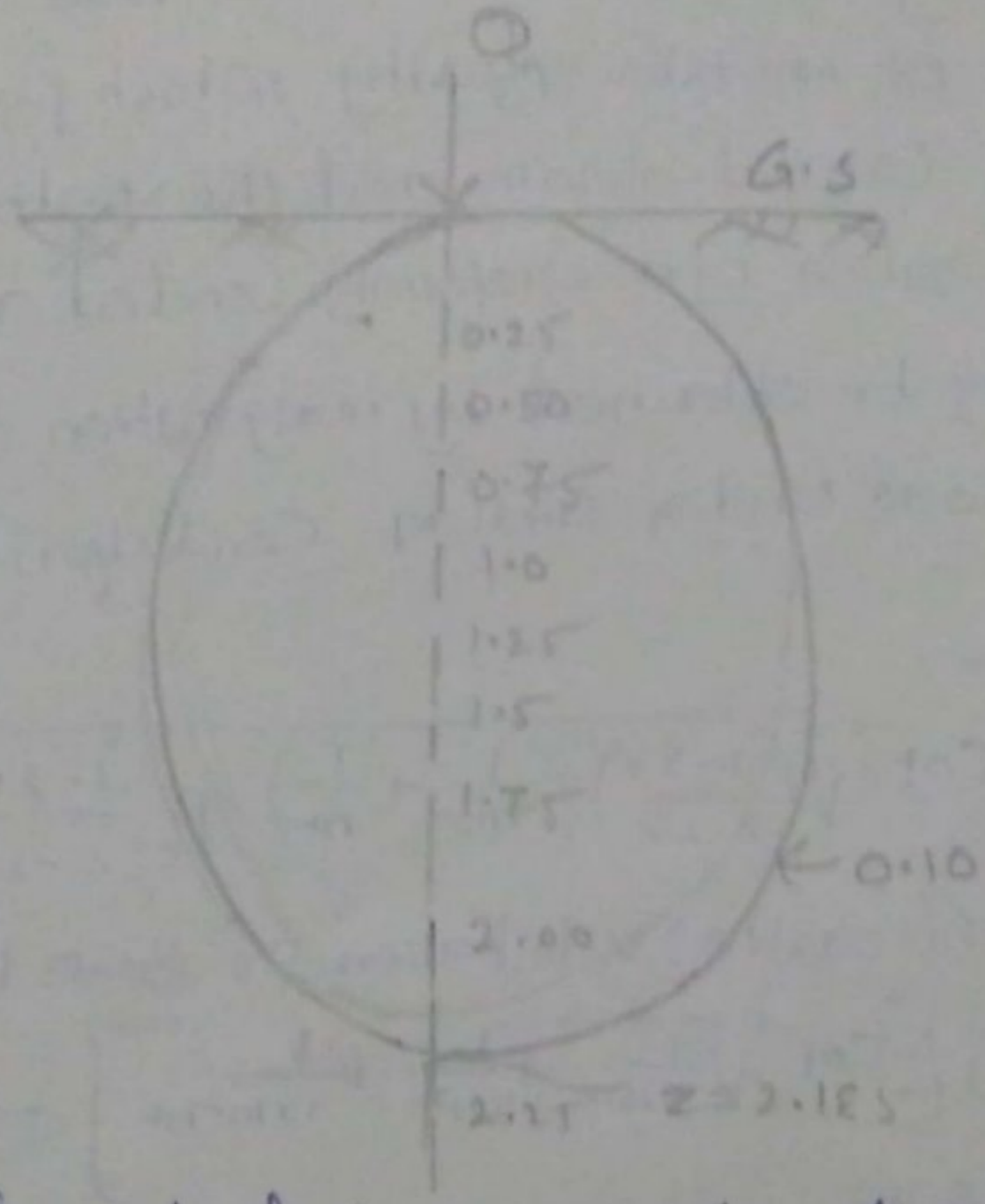
→ An Isobar is a curve joining the points of equal stress intensity. In other words, an Isobar is a Contour of equal stress

→ An Isobar is a Spatial Curved Surface of the shape of an electrical bulb (or) an Onion. the curved Surface is symmetrical about the vertical axis passing through the load point

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→ The Isobar of a particular intensity can be obtained using equation. The calculations are shown in a tabular form gives calculations for an Isobar of intensity 0.1 Q per unit area.



$$\sigma_z = I_b \frac{Q}{z^2} \quad (1)$$

$$0.1Q = I_B \frac{Q}{z^2}$$

$$I_B = 0.1 z^2 \quad (2)$$

→ for different depths z. The value of I_B is computed from eq (2) as shown in the second row of Table.

→ The values of $\frac{r}{z}$ for computed values of I_B are obtained from equation (1)

→ Once the values of $\frac{r}{z}$ have been determined the radial distance r can be obtained as shown in table

→ it may be observed that "r" is zero at the load point and it attains a maximum value at $\frac{r}{z} = 0.75$ and again decreases

→ As the Isobar is symmetrical about the load axis the other half can be drawn from symmetry. The shape of an Isobar approaches a laminar crack curve shows the pressure bulb of intensity 0.1 Q.

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→ In the same manner, the Isobars of other intensity 0.2Q, 0.3Q. etc. can be drawn. Obviously the Isobars of higher intensity shall lie within the Isobar of 0.1Q

→ Isobars are useful for determining the effect of the load on the vertical stresses at various points. The zone within which the stresses have a significant effect on the settlement of structure is known as the pressure bulb.

→ it is generally assumed that an Isobar of 0.1Q forms a pressure bulb. The area outside the pressure bulb is assumed to have negligible stresses.

(*) Vertical Stress Distribution on a horizontal plane:-

→ The vertical stresses at various points on a horizontal plane at a particular depth z can be obtained using equation

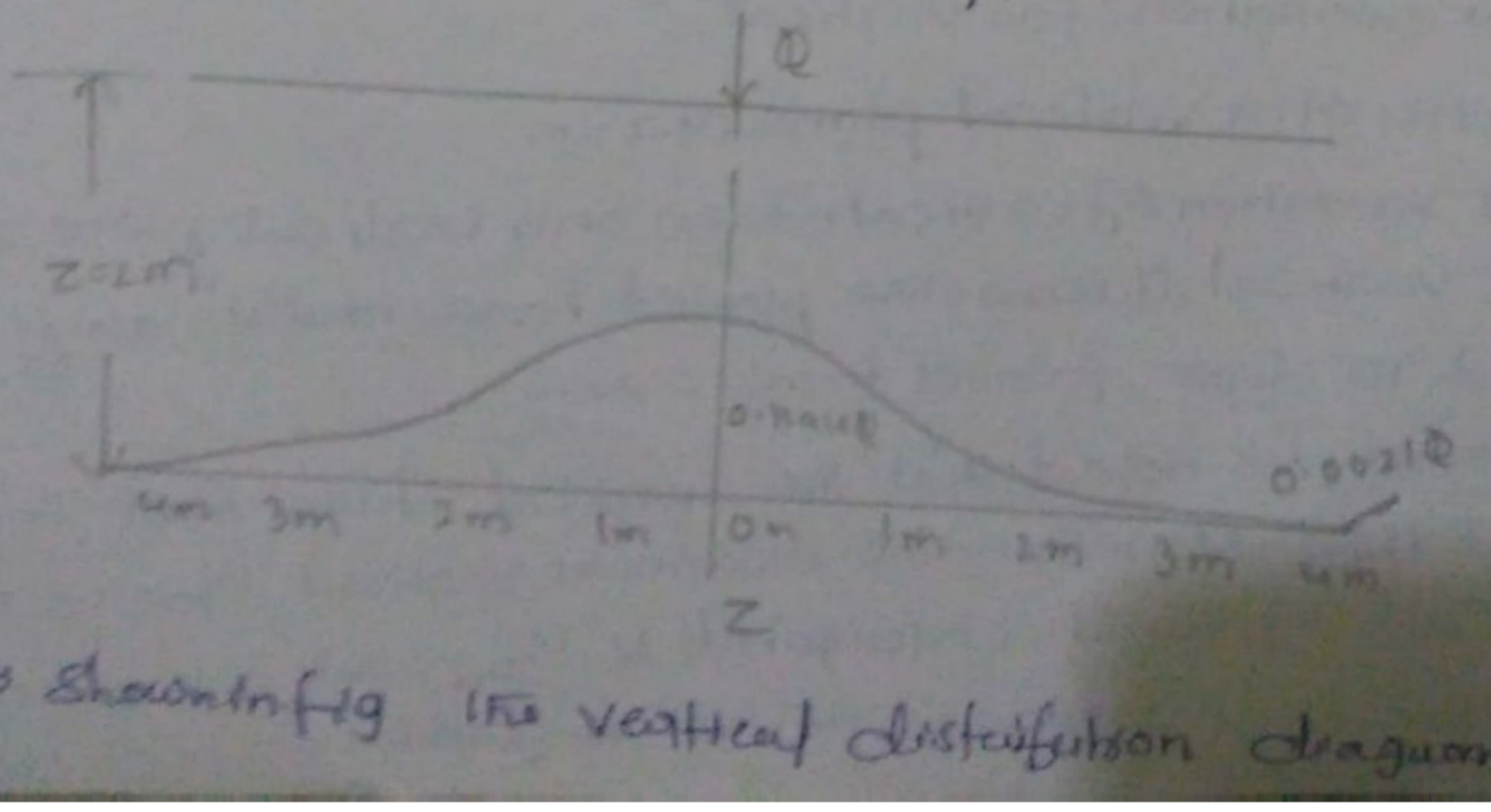
Let us determine the stresses at a depth of 2m

$$\therefore \sigma_z = I_B\left(\frac{Q}{z^2}\right)$$

$$\sigma_z = I_B\left(\frac{Q}{4}\right)$$

$$= I_B 0.25Q$$

→ The value of σ_z are computed for different values of I_B after obtaining I_B from eq.



→ as shown in fig the vertical distribution diagram

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The diagram is symmetrical about the vertical axis the maximum stress occurs just below the load (z=0) and it decreases rapidly as the distance r increases.

→ The vertical stress distribution diagram on a horizontal plane can be obtained graphically if the Isobars of different intensity are available. The horizontal plane is drawn on the Isobar diagram the points of intersection of the horizontal plane with the Isobar of a particular intensity give that vertical stresses.

(*) Vertical stress Distribution on a vertical plane:

→ The vertical stress distribution on a vertical plane at a radial distance of 'r' can be obtained using eq In this case the radial distance 'r' is constant and the depth changes.

→ The Values of $\frac{r}{z}$ are obtained for different depths z. The Values of I_B are obtained from eq * Table the stresses computed by

$$\sigma_z = \left(\frac{I_B}{z^2}\right) Q$$

The calculations for vertical stresses on a vertical plane at r=1m.

→ The Variation of vertical stress on a vertical plane r=1 m the vertical stresses are plotted horizontally along r-axis and the depth, parallel to the z axis.

→ It may be noted that the vertical stress first increases and then decreases. The maximum vertical stresses occurs at $\frac{r}{z}$ = 0.817 this corresponds to the point of intersection of the vertical plane with the line drawn at 39° to the vertical axis of the load.

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Newmark's Influence Charts :

→ The methods for the determination of vertical stresses under a strip, a circular and a rectangular area have been discussed in the preceding sections.

→ In practice, sometimes one has to find the vertical stress under a uniformly loaded areas of other shapes. For such cases Newmark's influence charts are extremely useful

→ Newmark's chart is based on the concept of the vertical stress below the centre of the circular area. Let us consider a uniformly loaded circular area of radius R_1, divided into 20 equal ~~section~~ sectors

→ the vertical stress at point p at depth z just below the centre of the loaded area due to load on one sector (hatched area ①) will be ($1/20$) of that due to load on full circle from eqn

$$\sigma_z = \frac{1}{20} q \left[1 - \left\{ \frac{1}{1+\left(\frac{R_1}{z}\right)^2} \right\}^{3/2} \right] \quad \text{— (i)}$$

→ If the vertical stress (σ_z) is given on arbitrary fixed value, say 0.005q eq ① becomes

$$0.005q = \frac{q}{20} \left[1 - \left\{ \frac{1}{1+\left(\frac{R_1}{z}\right)^2} \right\}^{3/2} \right] \quad \text{— (2)}$$

Solving eq (b) $\frac{R_1}{z} = 0.270$.

→ Thus every one-twentieth sector of the circle with a radius R_1 equal to 0.270 z. would give a vertical stress of 0.005q at the centre

→ Let us now consider another concentric circle of radius R_2 and divide it again into 20 equal sectors.

→ Each larger sector is divided into two sub areas if the small area exerts a stress of 0.005 q at P, the vertical

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Stress due to bolt area ① & ② would be equal to 2×0.005q

Thus

$$2 \times 0.005q = \frac{q}{20}\left[1 - \left\{\frac{1}{1+\left(\frac{R_2}{z}\right)^2}\right\}^{3/2}\right] \quad \text{— (c)}$$

Solving. $R_{2/z} = 0.40$.

→ In other words, the radius of the second circle should be equal to 0.40z

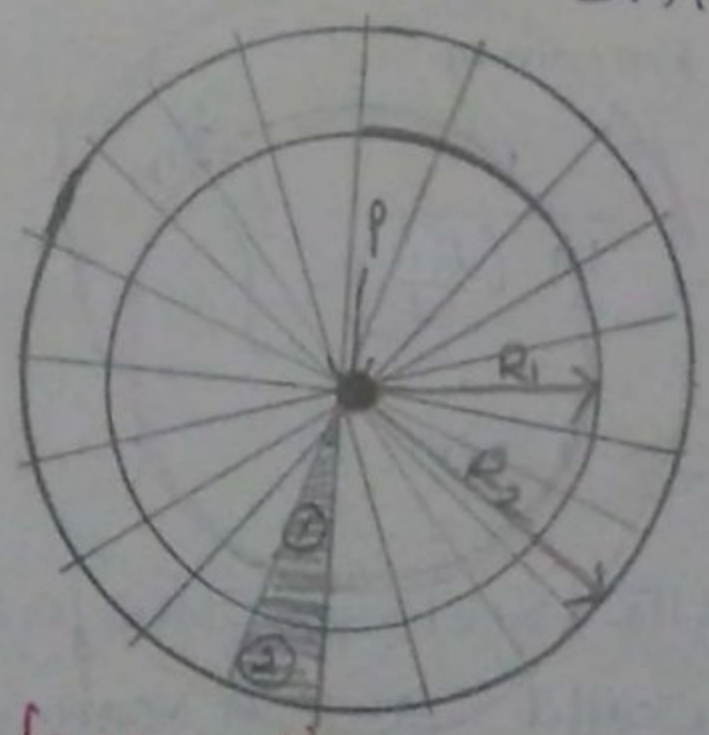
→ Likewise, the radii of the third to the ninth circles can be determined. The values obtained are 0.52z, 0.64z, 0.77z, 0.92z, 1.11z, 1.39z and 1.91z the radius of 9½ circle is 2.54z. The radius for the tenth circle R_{10} is given by

$$10 \times 0.005q = \frac{q}{20}\left[1 - \left\{\frac{1}{1+\left(\frac{R_{10}}{z}\right)^2}\right\}^{3/2}\right]$$

$$R_{10} = \infty$$

→ Therefore, the tenth circle cannot be drawn

→ as shown in fig the complete Newmarks influence chart in which only 9½ circles have been drawn for z equal to the distance AB marked on the sheet.



(*) Use of Newmarks Chart :-

→ The chart can be used to determine the vertical stress at point p below the loaded area.

→ A plan of the loaded area is drawn on a tracing paper to a scale such that the length AB (=z cm in the

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— is equal to the depth (z) of the point P below the surface.

→ For example, if the pressure is required at a depth of 1m the plane should be drawn to a scale of 2cm = 1m (or) R.F = 1/50.

→ The traced plan of the loaded area is placed over the Newmark chart such that the point 'p' at which the pressure is required coincides with the centre of the chart.

The vertical stress at point P is given by.

$$\sigma_z = I \times n \times q$$

where I = influence coefficient (=0.005 in this case).

n = no. of small area units covered by the plan

Each area b/w two successive radial lines and two successive concentric circle is taken as one unit.

q = Intensity of load.

* The following points are worth noting:

(1) The fractions of the unit areas should also be counted and properly accounted for

(2) If the plan of the loaded area extends beyonds the 9 1/2 15 circle it may be assumed to approach the 10 15 circle for the purpose of counting the unit areas.

(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 AB on the chart.

example problem

(1) A concentrated load of 2000kN is applied at the ground surface. Determine the vertical stress at a point P which is 6m directly below the load. Also calculate the vertical stress at a point R which is at a depth of 6m but at a

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horizontal distance of 5m from the axis of the load.

Sol: Given data

$Q = 2000\,kN$

$z = 6m$

$r = 5m$

$\sigma_z = \frac{3Q}{2\pi z^2} \cdot \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$

point P at $r=0$: $\frac{r}{z} = 0$ $\quad \sigma_z = \frac{3 \times 2000}{2\pi(6)^2} \cdot \frac{1}{[1+0]^{5/2}}$

$= 26.53\,kN/m^2$

point R. at $r = 5m = \frac{r}{z} = \frac{5}{6}$ $\quad \sigma_z = \frac{3 \times 2000}{2\pi(6)^2} \cdot \frac{1}{\left[1+\left(\frac{5}{6}\right)^2\right]^{5/2}}$

$= 7.1\,kN/m^2$

(2) Determine the vertical stress at a point P which is 3m below and at a radial distance of 3m from the vertical load of 100 kN Use Westergaard's Solution. ($\nu = 0.0$)

Sol: Given data.

$Q = 100\,kN$

$r = 3m$

$z = 3m$

$\therefore \sigma_z = \frac{1}{\pi\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}} \cdot \frac{Q}{z^2}$

$\sigma_z = \frac{1}{\pi\left[1+2\left[\frac{3}{3}\right)^2\right]^{3/2}} \cdot \frac{100}{(3)^2} = 0.681\,kN/m^2$

Alternatively

using equation $\sigma_z = I_w \cdot \frac{Q}{z^2}$

Taking I_w from table as 0.0613

σ_z $0.0613 \times \frac{100}{(3)^2} = 0.681\,kN/m^2$

(3) Calculate the vertical stress at a point P at a depth of 2.5m directly under the centre of the circular area of radius 2m and subjected to a load 100 kN/m². Also Calculate the vertical stress at a point Q which is at the same depth of 2.5m but 2.5m away from the centre of the loaded area.

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Sol: Given data:

$z = 2.5m$
$r = 2m$
$q = 100 kN/m^2$ } about P point.

about Q point: $z = 2.5$
$r = 2.5$

$$\sigma_z = q\left[1 - \left\{\frac{1}{1+\left(\frac{R}{z}\right)^2}\right\}^{3/2}\right]$$

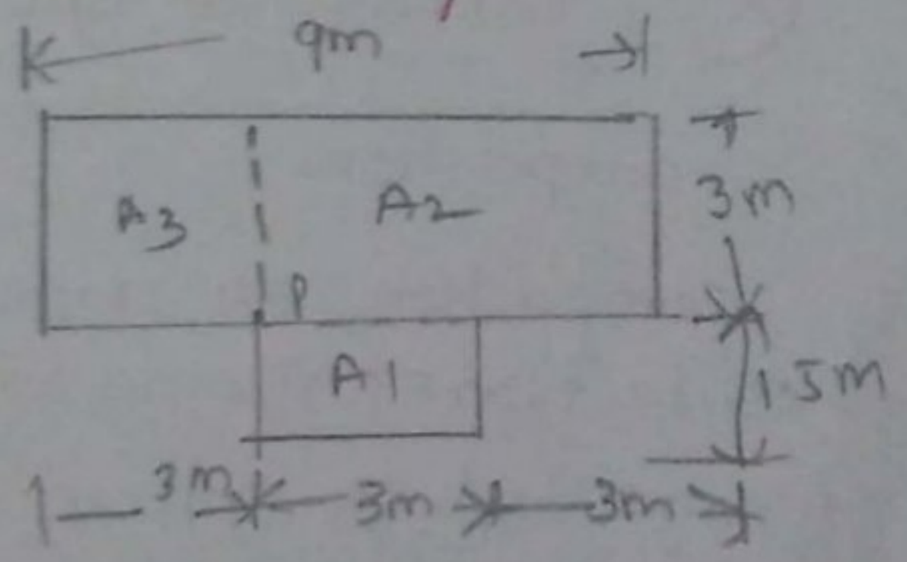
σ_z at P point $= 100\left[1 - \left\{\frac{1}{1+\left(\frac{2}{2.5}\right)^2}\right\}^{3/2}\right] = 52.39 \text{ kN/m}^2$

→ The vertical stress at $z = 1.25R$ & $r = 1.25R$ is about 0.2q.

$\therefore \sigma_z$ at Q $= 0.2 \times 100 = 20 \text{ kN/m}^2$

(4) A T-Shaped foundation is loaded with a Uniform load of 120 kN/m² Determine the vertical stress at point p at a depth of 5m. Use Newmarks influence chart. Compare the answer by exact method.

Sol: The foundation plan is drawn on a tracing paper with a scale such that the distance AB in fig represents 5m



→ The plan is placed on the Newmark chart such that point P is at the centre of the chart.

No. of area units occupied by plan = 63.

From equ. $\sigma_z = I \times n \times q$

$\sigma_z = 0.005 \times 63 \times 120 = 37.8 \text{ kN/m}^2$

$\left[\sigma_z = 120\left[0.0629 + 0.1431 + 0.1069\right] = 37.55 \text{ kN/m}^2\right]$

Exact method: The loaded area is divided 3 area such that they have one corner at P.

Area A_1: $m = 3/5 = 0.60$; $n = \frac{1.5}{5} = 0.30$ $(I_N)_1 = 0.0629$
Area A_2: $m = 3/5 = 0.60$; $n = 6/5 = 1.2$ $(I_N)_2 = 0.1431$
Area A_3: $m = 3/5 = 0.60$; $n = 3/5 = 0.60$ $(I_N)_3 = 0.1069$

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(*) Compaction :-

→ Compaction means passing the soil particles close to each other by mechanical methods. Air during compaction is expelled from the void space in the soil mass and there fore the mass density is increased.

→ Compaction of a soil mass is done to improve its engineering properties. Compaction generally increases the shear strength of the soil and hence the stability and bearing capacity. It is also useful in reducing the compressibility and permeability of the soil.

→ To increase the density of the soil and decrease the air voids.

→ The densification of soil by the application of mechanical energy is called Compaction.

→ it is measured in terms of dry unit weight

(*) Difference b/w Compaction and Consolidation

compaction	Consolidation
(1) compaction is a rapid process of reduction of volume by mechanical means such as rolling, tamping and vibration.	(1) Consolidation is a gradual process of reduction of volume under sustained static loading.
(2) Compaction the volume of a partially saturated soil decrease because of expulsion of air from the voids at the unaltered water content.	(2) Consolidation cause a reduction in volume of saturated soil due to squeezing out of water from the soil.
(3) Compaction is an artificial process which is done to increase the density of the soil to improve its properties before it is put to any use	(3) Consolidation is a process which occurs in nature when the saturated soil deposits are subjected to static loads caused by the weight of the buildings & other structures

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(x) Factors Affecting Compaction :-

→ The dry density of the soil is increased by compaction the increase in the dry density depends upon the following factors.

① Water content :-

→ At low water content the soil is stiff and offers more resistance to compaction. As the water content is increased the soil particles get lubricated.

→ the soil mass becomes more workable and the particles have closer packing.

→ the dry density of the soil increases with an increase in the water content till the optimum water content is reached

→ At that stage, the air voids attain approximately a constant volume

→ with further increase in water content the air voids do not decrease, but the total voids increase and the dry density decrease

→ the highest dry density is achieved upto the optimum water content due to forcing air out from the soil voids

→ After the optimum water content is reached, it becomes more difficult to force air out and to further reduce the air voids.

② Amount of Compaction :-

→ the effect of increasing the amount of compactive effort is to increase the maximum dry density and to decrease the optimum water content

→ At a water content less than the optimum the effect of increased compaction is more predominant

→ At a water content more than the optimum the volume of air voids be becomes almost constant and the effect of increased compaction is not significant

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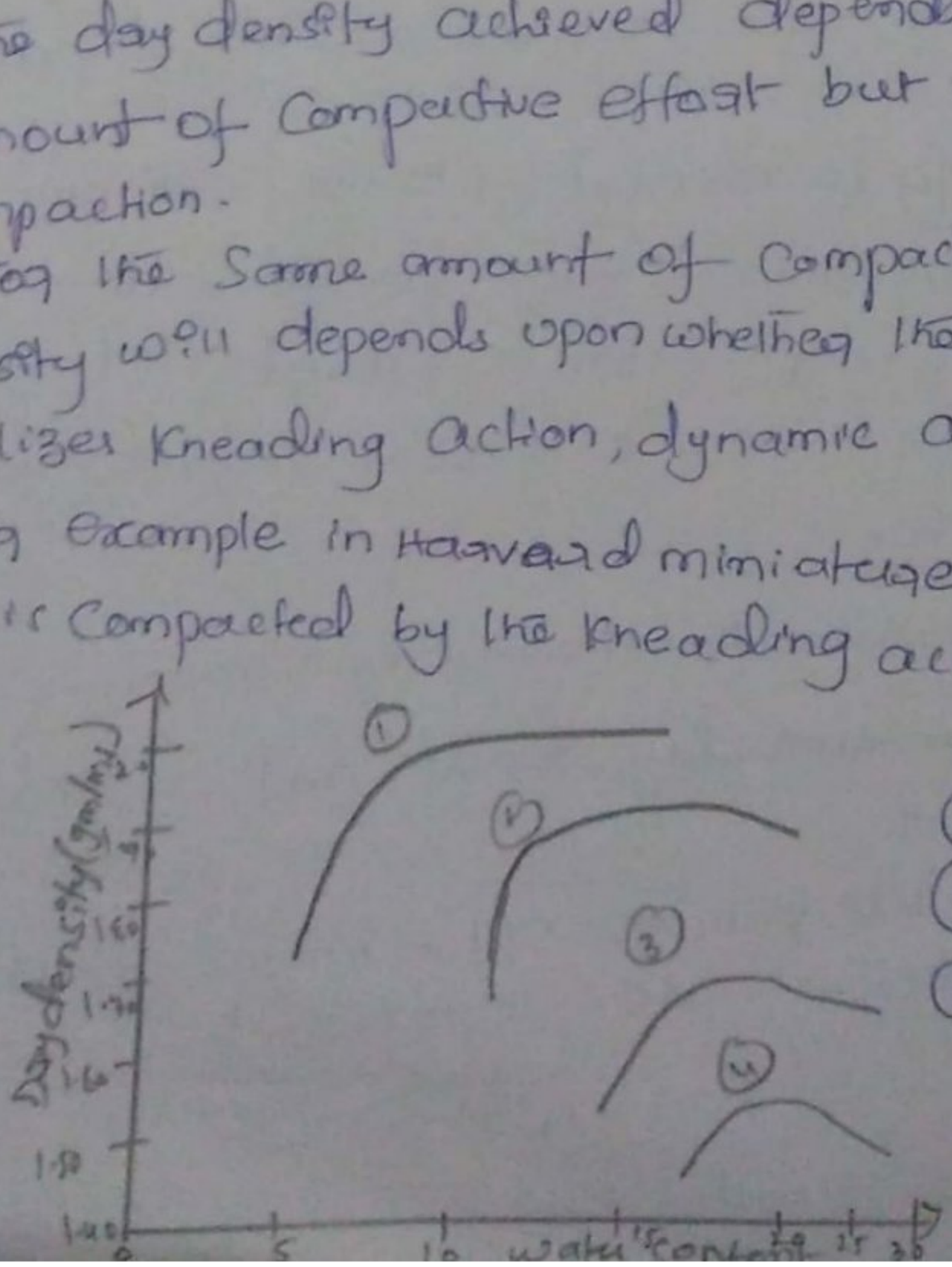
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(3) Type of Soil :-

→ The day density achieved depends upon the type of soil.
→ The maximum day density and the optimum water content for different soils are shown in fig.
→ In general coarse grained soils can be compacted to higher day density than fine grained soils.
→ Wilh the addition of even a small quantity of fines to a coarse grained soils, the soil attains a much higher day density for the same compactive effort.
→ However if the quantity of fines is increased to a value more than that required to fill the voids of the coarse grained soil s, the maximum day density decreases.
→ A well graded sand attains a much higher day density than a poorly graded soil.

(4) Method of Compaction :-

→ The day density achieved depends not only upon the amount of compactive effort but also on the method of compaction.
→ For the same amount of compactive effort, the day density will depends upon whether the method of compaction utilizes kneading action, dynamic action (or) static action.
→ For example in Haward miniature compaction test the soil is compacted by the kneading action &



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→ Compaction curve obtained is different from that obtained from the other Conventional tests in which an equal compactive effort is applied

(5) Admixture :-

→ The compaction characteristics of the soils are improved by adding other materials known as admixtures.

→ The most commonly used admixtures are Lime, Cement and bitumen as discussed in

(*) Effect of Compaction on properties of Soils :-

→ The engineering properties of soils are improved by Compaction. The desirable properties are achieved by proper ~~Sel~~ Selection of the soil type. the mode of placement and the method of Compaction.

(1) Soil structure :-

→ the water content at which the soil is Compacted plays an important role in the engineering properties of the soil.

→ Soils Compacted at a water content less than the optimum water content generally have a flocculated structure. Regardless of the method of Compaction.

→ Soils Compacted at a water content more than the Optimum water content usually have a dispersed structure if the Compaction induces large shear strains and a flocculated structure if the shear strains are relatively small.

(2) permeability :-

→ The permeability of a soil depends upon the size of voids as the permeability of a soil decrease with an increase in water content on the dry side of the optimum water content.

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→ There is an improved orientation of the particles and a corresponding reduction in the size of voids which cause a decrease in permeability.

→ The minimum permeability occurs at (or) slightly above the optimum water content.

(3) Swelling :-

→ A soil compacted dry of the optimum water content has high water deficiency and more random orientation of particles.

→ Consequently it imbibes more water than the sample compacted wet of the optimum & has therefore more swelling

(4) Pore water pressure :-

→ A sample compacted dry of the optimum has low water content the pore water pressure developed for the soil compacted dry of the optimum is therefore less than that for the same soil compacted wet of the optimum.

(5) Shrinkage :-

→ Soils compacted dry of the optimum shrink less on drying compared soils those compacted wet of the optimum.

→ The soils compacted wet of the optimum shrink more because the soil particles in the dispersed structure have nearly parallel orientation of particles & can pack more efficiently.

(6) Compressibility :-

→ The flocculated structure developed on the dry side of The optimum offers greater resistance to compression than the dispersed structure on the wet side

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(7) Stress-Strain relationship :

→ The soils Compacted dry of the optimum have a steeper stress-strain curve than those on the wet side

→ The modulus of elasticity for the soils Compacted dry of the optimum is Therefore high

(8) Shear Strength :

→ In general, at a given water Content the Shear Strength of the soil increases with an increase in the Compactive effort till a critical degree of ~~Saturation~~ Saturation is reached

(a) Shear Strength at moulded water Content

→ two samples are Compacted to the same dry density one dry of the optimum & the other wet of the optimum and tested for Shear Strength

(b) Shear Strength after Saturation

→ two samples are Compacted to the same dry density one dry of the optimum & the other wet of the optimum and then Soaked in water without any Volume change to have full Saturation.

(*) Field Compaction methods :

(a) Rolling (Rollers) (b) Ramming (Rammers) (c) Vibration (Vibrators)

Rolling equipments →
- Smooth wheel Rollers
- Pneumatic tyred rollers
- Sheep foot rollers
- Vibratory Pneumatic
- tyred Construction plant
- Track laying vehicles

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→ The mould had a detachable base plate and a Removable collar of 2 inches (5cm) height at the top.

→ The Soil was Compacted in the mould in 3 equal layers each layer was given 25 blocks of 5.5 pounds rammer falling through a height of 12 inches. A curve was obtained btw the dry density and the water content.

→ The same specifications as in Standard proctor test with some minor modifications and metrification.

→ The mould Recommended is of 100mm diameter, 127.3 mm height of 1000ml Capacity.

→ The Rammer Recommended is of 2.6 kg mass with a free drop of 310mm of a face diameter of 50mm.

→ The Soil is Compacted in three layers. The mould is fixed to a detachable base plate the collar is of Commhagy

→ If the percentage of soil detained on 4.75mm Sieve is more than 20%, a larger mould of Internal diameter 150mm effective height of 127.3mm of Capacity 2250ml is recommended

(iv) Compaction Control :

→ The Laboratory Compaction tests give the optimum water Content and the maximum dry density

→ In the field during the Compaction of the soils it is essential to check the dry density and the water Content in order to effect proper quality control

→ The geotechnical engineering has to ensure that the specified amount of Compaction of the desired dry densities are achieved.

→ Compaction Control is done by measuring the dry density and the water Content of the Compacted soil in the field.

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(1) Dry Density :-

→ The dry density is measured using the methods the core-cutter method and the sand replacement method are commonly used

→ The nuclear methods are occasionally used as these are non-destructive and require no physical (or) chemical processing of the soil & are very convenient.

(2) Water Content :-

→ The oven drying method of the determination of the water content takes 24 hours

→ This method though very accurate, cannot be used for controlling construction as the soil layer from which the sample was taken would be buried by the time the water content is known

→ The basic requirement is that the method used be such that it gives quick results in the field.

→ The water contents generally determined using the sand bath method, alcohol method (or) calcium carbide method

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: GEOTECHNICAL ENGINEERING

SUBJECT CODE: 20CE5PC03

FACULTY NAME: B. SURESH

SYLLABUS:

UNIT – IV

Consolidation: Types of compressibility – Immediate Settlement, 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 - reconsolidation pressure and its determination - Terzaghi's 1-D consolidation theory – coefficient of consolidation: square root time and logarithm of time fitting methods - computation of total settlement and time rate of settlement.

UNIT - IV
Consolidation

(*) Consolidation :-

→ It is the compression of the soil mass due to expulsion of water from the voids under a steady, static, long term pressure is called consolidation.

→ When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting thereby in an excess pore water pressure.

→ If drainage is permitted, the resulting hydraulic gradient initiate a flow of water out of the clay mass and the mass begins to compress.

→ A portion of the applied stress is transfered to the soil skelton which in turn causes a reduction in the excess pore pressure.

→ This process involving a gradual compression occuring simultaneously with a flow of water out of the mass & with a gradual transfer of the applied pressure from the pore water to the mineral skeleton is called consolidation.

→ This the peocess opposite to consolidation is called swelling.

(*) Compressibility :-

→ It is the property due to which reduction in volume occurs under compression in soil. Compression takes place due to
- Compression and expulsion of air in the voids
- Expulsion of water in the voids.

→ The compression of solid grains and compression of water in the voids are negligibly small and are neglected.

(*) Types of Consolidation :-

→ The consolidation of a soil deposit can be divided in to 3 stages.

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① 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 solid particles.

→ The reduction in volume of the soil just after the application of the load is known as initial consolidation (or) initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles.

② primary consolidation :-

→ After initial consolidation, further reduction in volume occurs due to expulsion of water from voids.

→ When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as an excess pore water pressure, as water is almost incompressible as compared with solid particles.

→ A hydraulic gradient develops and the water starts flowing out and a 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, the primary consolidation occurs over a long time. on the other hand in coarse grained soils the primary consolidation occurs rather quickly due to high permeability.

→ As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles. The effective stress is increased.

③ Secondary Consolidation :-

→ The reduction in volume continues at a very slow rate even after the excess hydrostatic pressure developed by the

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- applied pressure is fully dissipated and the primary consolidation is complete.
→ This additional reduction in the volume is called secondary consolidation.
→ The causes for a secondary consolidation are not fully established it is attributed to the plastic re adjustment of the solid particles and the adsorbed water to the new stress system.
→ In most Inorganic soils, It is generally small.

(*) Normally consolidated soil and over-consolidation soils

→ A Normally consolidated soil is one which had not been subjected to a pressure greater than the present existing pressure.
→ A soil is said to be over-consolidated if it had been subjected in the past to a pressure in excess of the present pressure.
→ The portion AB of the curve the represents the soil in normally consolidated condition.
→ The curve in this range is also called the virgin Compression curve.
→ The soil in the range CD when it is recompressed represents overconsolidated condition. as the soil had been previously subjected to a pressure $\bar{\sigma}_2$ which is greater than the pressure in the range CD.
→ The maximum pressure to which an over-consolidated soil had been subjected in the past divided by the present pressure is known as the overconsolidation ratio (O.C.R)
→ For example the soil indicated by the condition at point c has an over consolidation ratio $\bar{\sigma}_2/\bar{\sigma}_1$
→ It may be emphasized that normally consolidated soils and overconsolidated soils are not different types of soils but these are conditions in which a soil exists.

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→ The same type of soil can behave as normally consolidated in a certain pressure range and an over-consolidated in some other pressure range.

→ The liquidity index of a normally consolidated clay is generally b/w 0.6 & 1.0 whereas that for an over consolidated clay b/w 0.0 & 0.60.

→ As the recompression index (C_r) is very small as compared with the compression index (C_c). The soils in the overconsolidated state have smaller compressibility. The settlements of the structure built on over consolidated clays are small.

(*) **Under consolidated clays (or) Soil:**

→ If the clay deposit has not reached equilibrium under the applied overburden loads. It is said to be under consolidated. This normally occurs in areas of recent land fill.

(*) **Preconsolidation pressure:-**

→ The maximum pressure to which an overconsolidated soil had been subjected in the past is known as the preconsolidation pressure (or) over-consolidation pressure (σ_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.

→ The specimen is generally preconsolidated on overconsolidated when the specimen is loaded in the consolidation test initial portion AB of the compression curve ABCD is actual a recompression curve

→ Consequently the initial portion AB is relatively flat

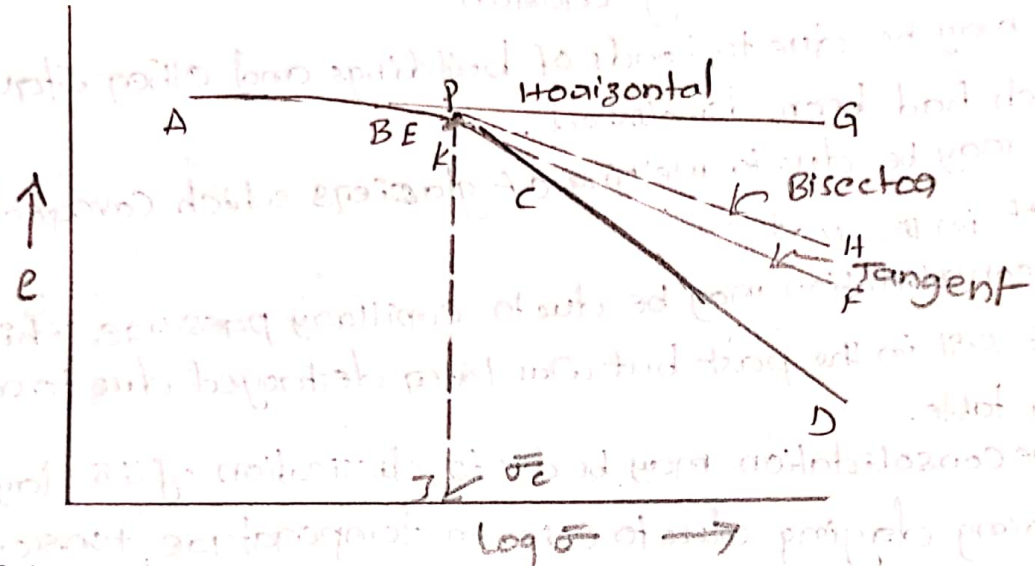
→ It is followed by a straight line CD with a steep slope which indicates the compression of a virgin soil.

→ In the transition range BC. The slope gradually changes.

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→ The preconsolidation pressure ($\bar{\sigma}_c$) falls in this range it can be obtained using the method given by Casagrande.

→ The procedure consists of the following steps.



(1) Determine the point E on the curve where the curvature is maximum. i.e the radius of the curvature is minimum

(2) Draw the tangent EF to the curve at E.

(3) Draw a horizontal line EG at E

(4) Bisect the angle between the tangent EF and the horizontal EG. and draw the bisector EH.

(5) produce back the straight line portion CD of the curve and determine ~~EG, and draw the bisector EH.~~ 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 J. 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.

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(*) Causes of preconsolidation in soils :

→ Preconsolidation in a soil deposits may be due to one (or) more of the causes mentioned below.

(1) preconsolidation may be due to the 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 capillary pressure which acted on the soil in the past but was later destroyed due to a rise in water table.

(5) preconsolidation may be due to dissication of the clay deposit. During drying due to rise in temperature, tension develops in pore water and the effective stress increases. After dessicate the soil remained preconsolidated.

(6) Sustained downward seepage forces cause an increase in the effective stress. When the seepage later stops. the soil becomes Pre consolidated due to reduction in the effective stress.

(7) pre consolidation may be due to tectonic forces caused by the movement of earth's crust which later become less severe.

(*) Consolidation test Results :-

(1) Dial Gauge reading - time plot :-

→ as shown in fig the plot b/w the dial gauge reading and time for a typical load increment for clay and sand samples.

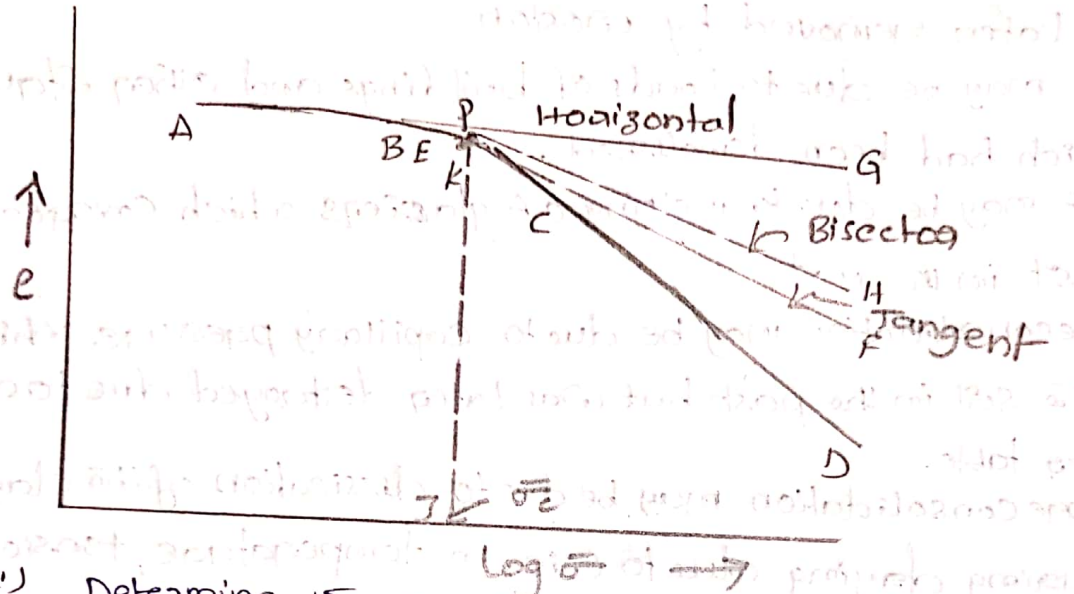
→ the thickness just after the application of the load increment ($t=t_0$), is a maximum which decreases as the time increases.

→ the decrease is rapid initially but it slows down as the time passes.

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→ The preconsolidation pressure ($\bar{\sigma_c}$) falls in this range it can be obtained using the method given by Casagrande.

→ The procedure consists of the following steps.



(1) Determine the point E on the curve where the curvature is maximum. i.e the radius of the curvature is minimum.

(2) Draw the tangent EF to the curve at E.

(3) Draw a horizontal line EG at E

(4) Bisect the angle between the tangent EF and the horizontal EG. and draw the bisector EH.

(5) Produce back the straight line portion CD of the curve and determine ~~EG, and draw the bisector EH.~~ the point of intersection P of the bisector EH and the backward extension of CD.

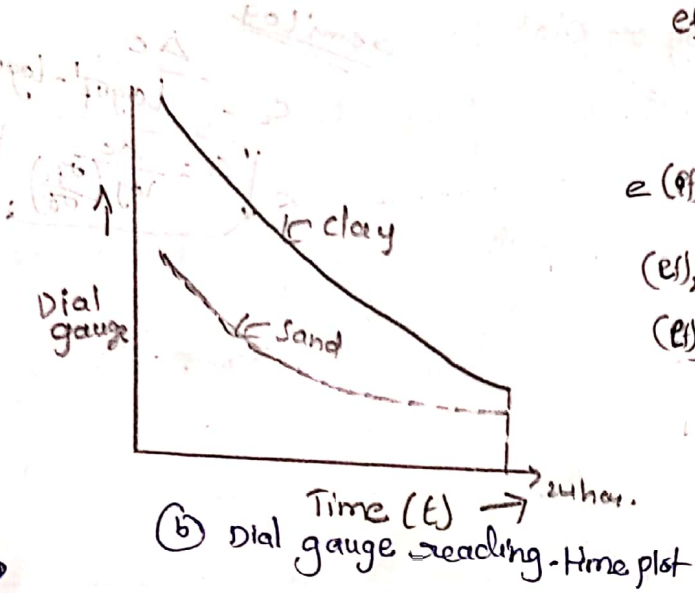
(6) Draw the vertical PJ through P which cuts the log $\bar{\sigma}$-axis at J. The point J indicates the preconsolidation pressure $\bar{\sigma_c}$.

(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.

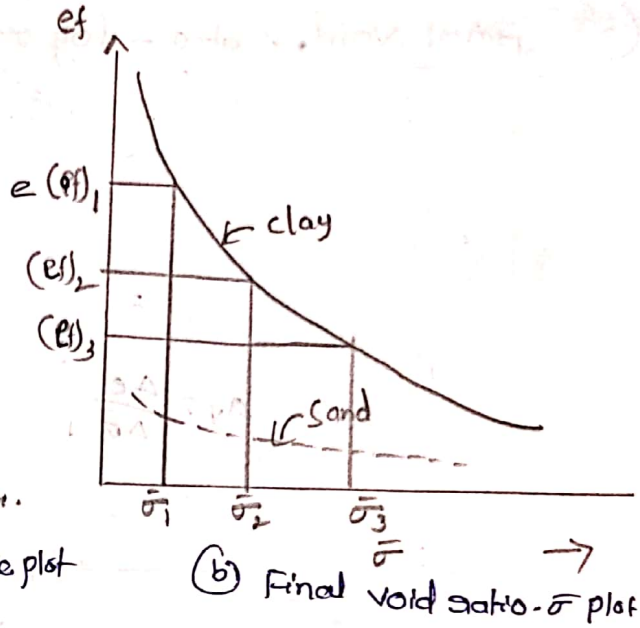
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→ There is practically no change in Thickness after 24 hours.

→ The Consolidation at that Load increament is considered to the Complete at 24 hours.



(b) Dial gauge reading - Time plot



(b) Final void ratio - $\bar{\sigma}$ plot

→ For Sand, the change in thickness occurs very quickly and stops after a few minutes. This is due to high permeability of the Sand which permits easy flow of water.

→ The plot between the dial gauge reading and time is required for determinationning the coefficient of Consolidation. which is useful for obtaining the rate of consolidation in the field.

(2) Final void ratio - effective Stress plot :-

→ The Thickness of the Specimen after 24 hours of application of the Load increment is taken as the final thickness for that increment.

→ The final void ratio (ef) corresponding to the final thickness for each increament is determined using the methods discussed in the preceding section.

→ as shown a fig the plot between the final void ratios $(e_f)_1$, $(e_f)_2$, $(e_f)_3$ --- etc. and the corresponding effective Stresses $\bar{\sigma_1}$, $\bar{\sigma_2}$, $\bar{\sigma_3}$ -- for Load increament 1, 2, 3 etc.

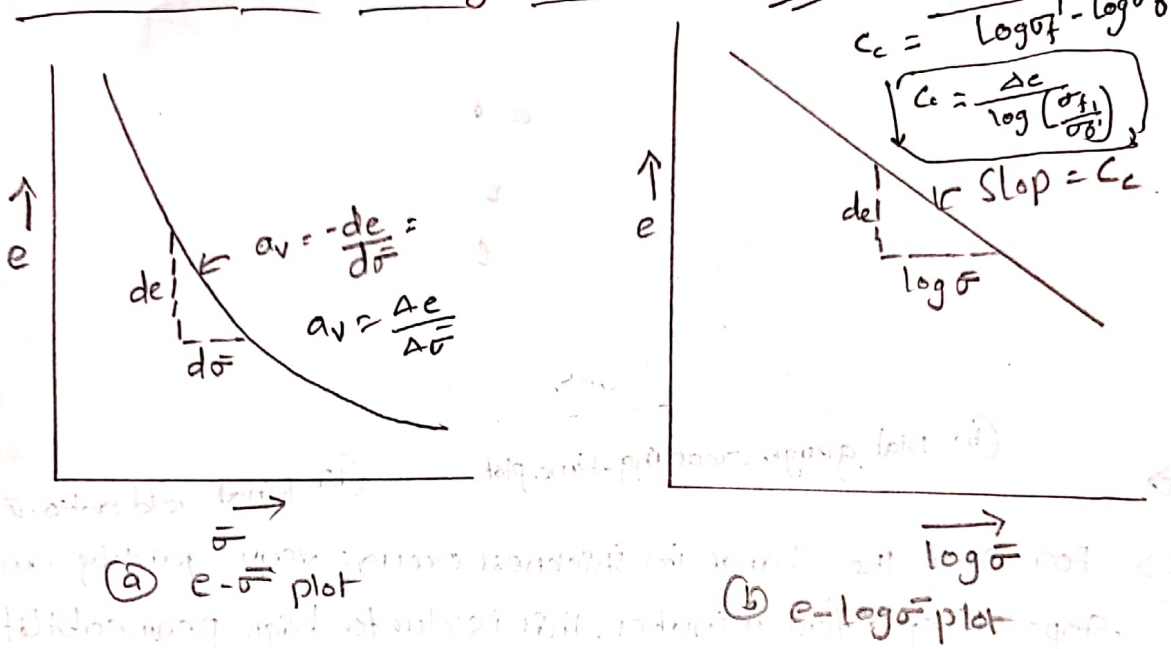
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→ As the sand is relatively less compressible the change in void ratio is small. The plot b/w the final void ratio and the effective stress is required for determination of the magnitude of the consolidation settlement in the field.

③ Final void ratio – log $\bar{\sigma}$ plot :- Semi log

$$C_c = \frac{\Delta e}{\log\sigma_f' - \log\sigma_i'}$$

$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma_{f'}}{\sigma_i'}\right)}$$



(a) e – $\bar{\sigma}$ plot (b) e – log $\bar{\sigma}$ plot

→ as shown in fig a plot b/w the final void ratio and the effective stress which is similar to one in fig (b) for convenience the suffix f has been dropped.

→ The curves has concavity upward. the slope of the curve at different points is different the slope decrease with an increase in effective stress.

→ it is more common to plot the results on a semi log graph in which the final voids ratio is ploted on the natural scale and the effective stress as abscissa on the log scale

④ Unloading and Reloading plot :-

→ As shown in fig the curve AB indicates the decrease in void ratio with an increase in the effective stress.

→ the curve is similar to one shown in fig 5 it is the loading curve

→ After the sample has reached equilibrium at the effective stress of $\bar{\sigma_2}$ as shown by point B. the pressure

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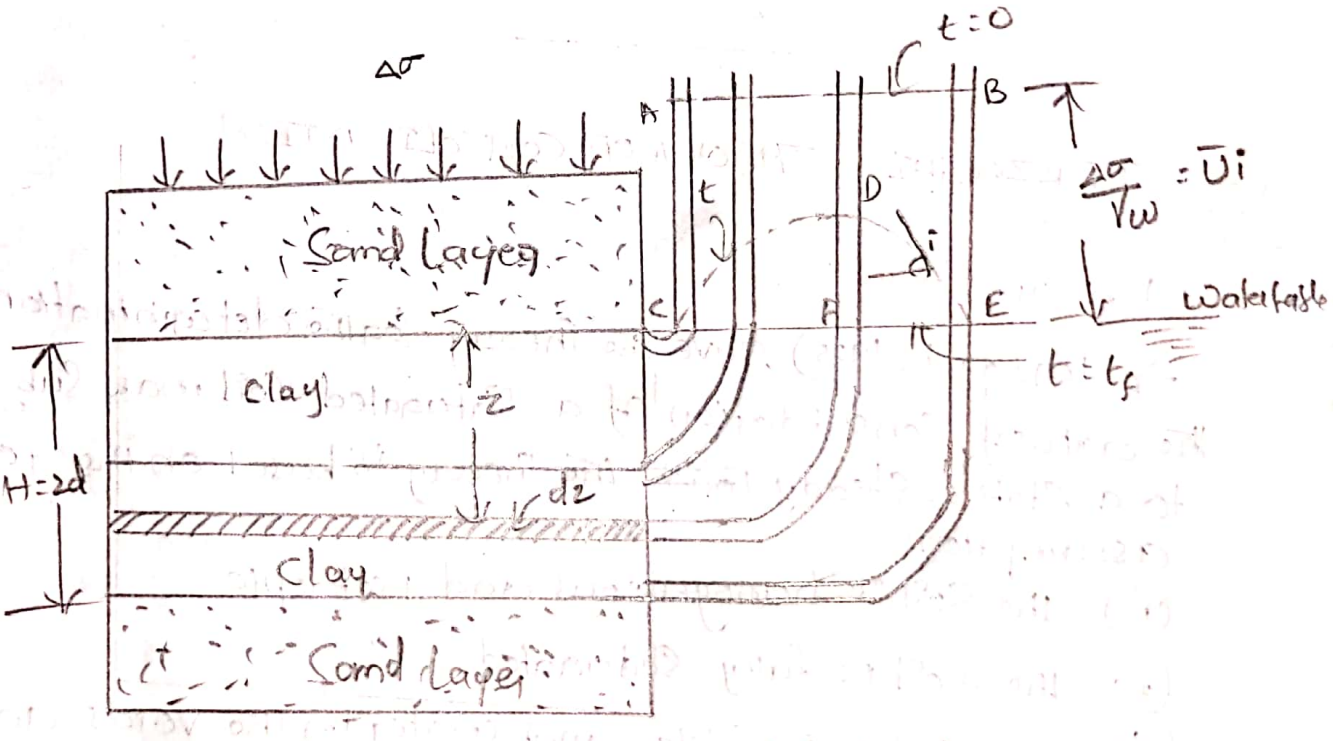
(5) Darcy's law is valid throughout the consolidation process.

(6) Soil is laterally confined, and the consolidation takes place only in axial direction. Drainage of water also occurs only in the vertical direction.

(7) The time lag in consolidation is due entirely to the low permeability of the soil.

(8) There is a unique relationship b/w the void ratio & the effective stress, and this relationship remains constant during the load increment.

Derivation of Differential equation :



→ The basic differential equation of one dimensional Consolidation can be derived as under.

→ Let us consider a saturated clay layer of thickness (2d=H) sandwiched b/w two layers of sand

→ when a uniform pressure of Δσ is applied on the surface of the top sand layer the total stress

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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 excess pressure of $\frac{\Delta\sigma}{\gamma_w}$ develops.

→ The excess hydrostatic pressure diagram on the right side it is assumed that various points along the thickness of the clay layer are connected by flexible tubes to the piezometers.

→ At time $t=0$, just after the application of the load the excess hydrostatic pressure u_i is equal to $\frac{\Delta\sigma}{\gamma_w}$ throughout the clay layer.

→ This is represented by the horizontal line AB the excess hydrostic pressure is independent of the position of the water table For convenience, the water table is assumed at the level of the surface of the clay layer.

→ Water starts escaping towards the upper and the lower sand layers due to excess hydrostatic pressure developed.

→ The hydrostatic excess pressure at the top and the bottom of the clay layer, indicated by points C and E in the pressure diagram. drops to zero. However, the excess hydrostatic pressure in the middle portion of the clay layer at D remains high.

→ The curves indicating the distribution of excess hydro static pressure are known as Isochrones.

→ The Isochrone CDE indicates the distribution of excess hydrostatic pressure at time t.

→ As the consolidation progresses, the excess hydrostatic pressure in the middle of the clay layer decreases. Finally at time $t=t_f$ the whole of the excess hydrostatic pressure has been dissipated and the pressure distribution is indicated by the horizontal Isochrone CFE

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→ Let us consider the equilibrium of an element of the clay at a depth of z from its top at time 't'. The consolidating pressure $\Delta\sigma$ is partly carried by water and partly by solid particles as

$$\Delta\sigma = \Delta\bar{\sigma} + \bar{u} \quad \text{—(1)}$$

where $\Delta\bar{\sigma}$ is the pressure carried by solid particles.

$\bar{u}$ is the excess hydro-static pressure

→ The hydraulic gradient (i) at that depth is equal to the slope of the Isochrone CDE at a horizontal distance 'z' from the point c in the pressure diagram.

$$i = \frac{dh}{dz} = \frac{\partial(\bar{u}/\gamma_w)}{\partial z} = \frac{1}{\gamma_w}\left(\frac{\partial \bar{u}}{\partial z}\right) \quad \text{—(2)}$$

where $\bar{u}$ is the excess hydrostatic pressure at depth z

→ The expression for the hydraulic gradient 'i' can also be obtained as under.

→ Let us consider a thin slice of clay layer Δz at depth 'z' the pressure difference ($\Delta\bar{u}$) across this thickness is given by.

$$\Delta\bar{u} = \left(\not{u} + \frac{\partial \bar{u}}{\partial z}\cdot dz\right) - \not{u} = \frac{\partial \bar{u}}{\partial z}\cdot dz.$$

$$\boxed{\Delta\bar{u} = \frac{\partial \bar{u}}{\partial z}\cdot dz.}$$

→ The unbalanced head across the thickness is given by.

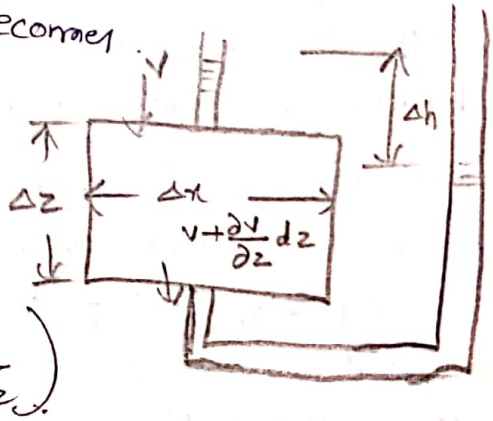
$$\Delta h = \frac{\Delta\bar{u}}{\gamma_w} = \frac{1}{\gamma_w}\left(\frac{\partial \bar{u}}{\partial z}\cdot dz\right)$$

The hydraulic gradient becomes

$$i = \frac{\Delta h}{dz} = \frac{1}{\gamma_w}\left(\frac{\partial \bar{u}}{\partial z}\right)$$

→ From Darcy's Law the velocity of flow at depth z is given by

$$v = ki = k\cdot\frac{1}{\gamma_w}\left(\frac{\partial \bar{u}}{\partial z}\right)$$



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→ The velocity of flow at the bottom of the element of thickness Δz can be written as.

$$v + \frac{\partial v}{\partial z} \cdot dz = v + \frac{\partial}{\partial z}\left[\frac{k}{\gamma_w}\left(\frac{\partial \bar{u}}{\partial z}\right)\right] dz$$

$$v + \frac{\partial v}{\partial z} dz = v + \frac{k}{\gamma_w}\left(\frac{\partial^2 \bar{u}}{\partial z^2}\right) dz$$

$$\therefore \boxed{\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \cdot \frac{\partial^2 \bar{u}}{\partial z^2}} \quad \text{——— (3)}$$

→ The discharge entering the element Q_{in} is

$$Q_{in} = v(\Delta x \times \Delta y)$$

where Δx and Δy are the dimensions of the element in plan the discharge leaving the element Q_{out} is

$$Q_{out} = \left(v + \frac{\partial v}{\partial z} \cdot dz\right)(\Delta x \times \Delta y)$$

∴ The net discharge squeezed out of the element is given by

$$\Delta Q = Q_{out} - Q_{in}$$

$$\Delta Q = \left[\left(v + \frac{\partial v}{\partial z} \cdot dz\right) - v\right] \Delta x \times \Delta y$$

$$\boxed{\Delta Q = \frac{\partial v}{\partial z}(\Delta x \times \Delta y \times \Delta z)} \quad \text{——— (4)}$$

→ As the water is squeezed out the effective stress increases and the volume of the soil mass decreases.

$$\Delta V = -m_v V_0 \Delta \bar{\sigma}$$

where V_0 = initial volume of soil mass $(\Delta x \Delta y \Delta z)$.

$\Delta \bar{\sigma}$ = increase in effective stress.

→ The decrease in volume of soil per unit time

$$\boxed{\frac{\partial(\Delta V)}{\partial t} = -m_v(\Delta x \Delta y \Delta z)\frac{\partial(\Delta \bar{\sigma})}{\partial t}} \quad \text{——— (5)}$$

→ As the decrease in volume of soil mass per unit time is equal to the volume of water squeezed out per

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Unit time eqn (4) & (5) give.

$$\frac{\partial v}{\partial z}(\Delta x \Delta y . \Delta z) = -m_v (\Delta x \Delta y \Delta z) \frac{\partial (\Delta \bar{\sigma})}{\partial t}$$

$$\boxed{\frac{\partial v}{\partial z} = -m_v \frac{\partial (\Delta \bar{\sigma})}{\partial t}} \quad \text{—— (6)}$$

$$\Delta \bar{\sigma} = \Delta \sigma - \bar{u}$$

$$\frac{\partial \Delta \bar{\sigma}}{\partial t} = \frac{\partial (\Delta \bar{\sigma})}{\partial t} - \frac{\partial \bar{u}}{\partial t}$$

For a given pressure increment. $\partial \Delta \sigma = 0$.

$$\therefore \frac{\partial (\Delta \bar{\sigma})}{\partial t} = -\frac{\partial \bar{u}}{\partial t}$$

equation (6) becomes $\frac{\partial v}{\partial z} = -m_v \left(-\frac{\partial \bar{u}}{\partial t}\right)$

$$\boxed{= m_v \left(\frac{\partial \bar{u}}{\partial t}\right)} \quad \text{—— (7)}$$

equating two values of $\frac{\partial v}{\partial z}$ from eqs (3)

$$\frac{k}{\gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2} = m_v \left(\frac{\partial \bar{u}}{\partial t}\right)$$

$$c_v \frac{\partial^2 \bar{u}}{\partial z^2} = \frac{\partial \bar{u}}{\partial t} \quad \text{—— (8)}$$

Where c_v is the coefficient of consolidation and is given by

$$\boxed{c_v = \frac{k}{\gamma_w m_v} = \frac{k}{g \rho_w m_v}} \quad \text{—— (9)}$$

The equation of one dimensional consolidation. it gives the distribution of hydrostatic excess pressure $\bar{u}$ with dep15 z and time t.

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(*) Basic Definations

(1) Coefficient of Compressibility :-

→ The coefficient of compressibility (a_v) is defined as decrease in void ratio per unit Increase in effective Stress.

→ it is equal to the Slope of the $e - \bar{\sigma}$ curve at the point under consideration.

$$\therefore \boxed{a_v = \frac{-de}{d\bar{\sigma}} = \frac{-\Delta e}{\Delta \bar{\sigma}}}$$

(2) Coefficient of Volume change :

→ The coefficient of volume change is defined as the Volumetric Strain per unit increase in effective Stress.

$$\therefore \boxed{m_v = \frac{\frac{-\Delta V}{V_0}}{\Delta \bar{\sigma}}}$$

$m_v = 1 \times 10^{-3}$ to 1×10^{-4} m^2/kN

m_v = Coefficient of volume change.

V_0 = initial volume.

ΔV = change in volume

$\Delta \bar{\sigma}$ = change in effective Stress.

(a) Let e_0 be the initial void ratio. Let the volume of solids be Unity. Therefore the initial volume V_0 is equal to $(1+e_0)$ if is the change in void ratio e_0+1 due to change in volume ΔV. we have $\Delta V = \Delta e$.

$$\therefore \boxed{\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0}} \qquad \boxed{m_v = \frac{\frac{-\Delta e}{1+e_0}}{\Delta \bar{\sigma}}}$$

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(b) As the area of cross-section of the sample in the consolidometer remains constant the change in volume is also proportional to the change in height.

Thus $\Delta V = \Delta H$

$$\frac{\Delta V}{V_0} = \frac{\Delta H}{H_0}$$

$$m_v = \frac{-\frac{\Delta H}{H_0}}{\Delta \sigma}$$

$\therefore$ ~~$\Delta V \times H_0 = -\Delta H \times \Delta \sigma$~~

~~$\Delta H = \frac{m_v \times H_0}{\Delta \sigma}$~~

→ The relationship b/w a_v & m_v can be obtained

$$m_v = \frac{a_v}{1+e_0}$$

(3) Compression Index : (C_c)

→ The Compression Index (C_c) is equal to the slope of the linear portion of the void ratio vs log $\bar{\sigma}$ plot

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

where $\bar{\sigma}_0$ = initial effective stress, $\bar{\sigma}$ = final effective

Δe = change in void ratio.

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

(4) Expansion Index : (C_e)

The expansion index (or) swelling index (C_e) is the

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→ Slope of the e-log σ̄ plot obtained during unloading

$$C_e = \frac{\Delta e}{\log_{10}\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

(5) Recompression index.

→ Recompression is the compression of a soil which had already been loaded and unloaded the load during recompression is less than the load to which the soil has been subjected previously. the slope of the recompression curve obtained during reloading when plotted as e - log σ̄ is equal to the recompression index.

$$C_r = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

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(x) Coefficient of consolidation :-

→ The curve b/w dial gauge reading and time 't' obtained in the laboratory by testing the soil sample is similar in shape to the theoretical curve b/w U and T_v obtained from the consolidation theory.

→ This similarity b/w the laboratory curve and the theoretical curve is used for the determination of the coefficient of consolidation (c_v) of the soil. The methods are known as the fitting methods.

→ The following two methods are commonly used.

① Square-root of time method :-

→ The method, devised by Taylor utilizes the theoretical relationship b/w U and $\sqrt{T_v}$ the relationship is linear upto the value of U equal to about 60%.

→ it has been further established that at 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 sample of the soil whose coefficient of consolidation is required is tested as explained for a given load increment. The dial gauge readings are taken for different time intervals.

→ A curve is plotted b/w the dial gauge reading (R) as ordinate, and the $\sqrt{t}$ as abscissa

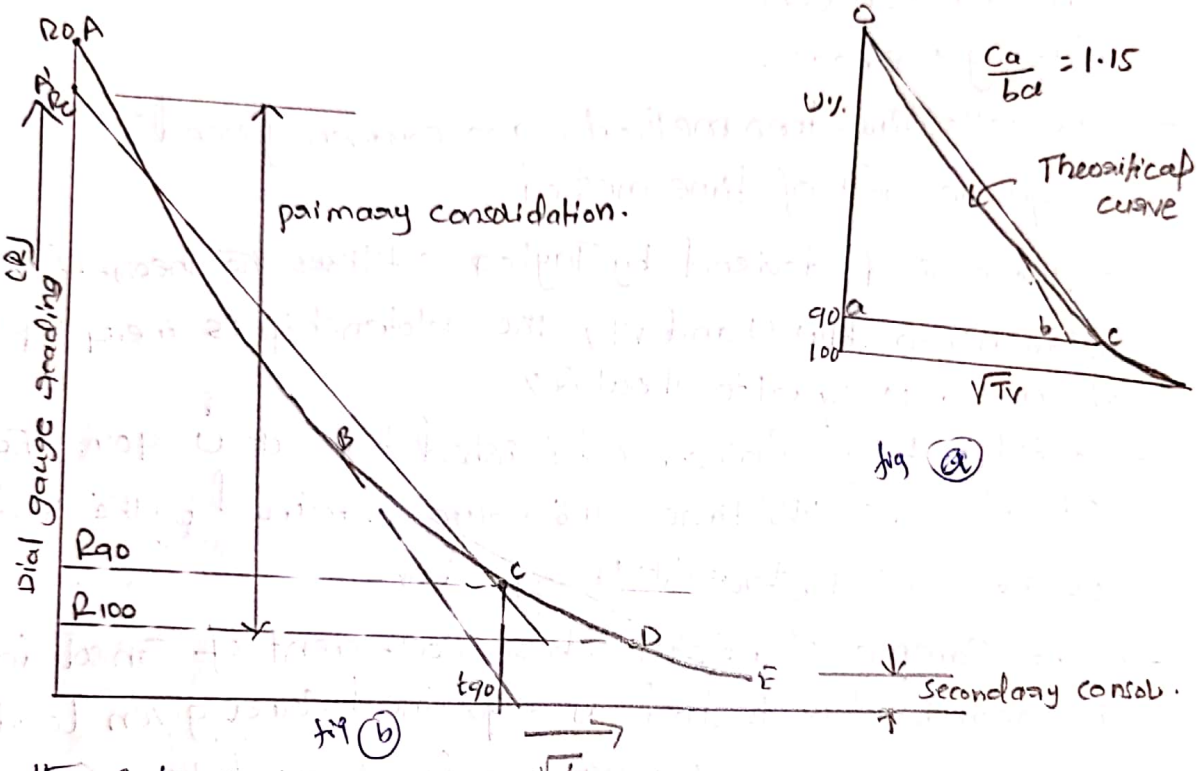
→ The curve ABCDE shows the plot. The curve begins at the dial gauge reading R_0 at time t_0 indicated by point A.

→ As the load increment is applied there is an initial compression. it is obtained by producing back the initial linear part of the curve to intersect the dial-gauge reading axis at point 'A' this corresponding to corrected zero reading (R_c).

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→ The consolidation b/w the dial gauge reading R_0 and R_c is the initial ~~corrected zero reading~~ (R_c compression the Terzaghi theory of consolidation is not applicable in this range.

→ From the corrected zero reading point 'A', a line $A'C$ is drawn such that its abscissa is 1.15 times the of the initial linear portion $A'B$ of the curve.



→ The intersection of this line with the curve at a point c indicates 90% of U. The dial gauge reading corresponding to 'c' is shown as R_{90} and the corresponding abscissa as $\sqrt{t_{90}}$.

→ The point D for 100% primary consolidation can be obtained from R_{90} as

$$R_c - R_{100} = \frac{10}{9}(R_c - R_{90})$$

→ The consolidation after 100% of primary consolidation in the range DE is the secondary consolidation the value of the coefficient of consolidation of the soil for that load increment is obtained from the value of $\sqrt{t_{90}}$.

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- obtained from that plot. from table for U = 90%, the value of $T_v = 0.848$

∴ using equation

$$\boxed{C_v = \frac{T_v d^2}{t}} = \frac{0.848 d^2}{(\sqrt{t_{90}})^2} = \frac{0.848 d^2}{t_{90}}$$

→ the distance of the drainage path d is half the total thickness the total thickness may be taken as the average of the initial thickness (H_i) and final thickness (H_f) of the sample.

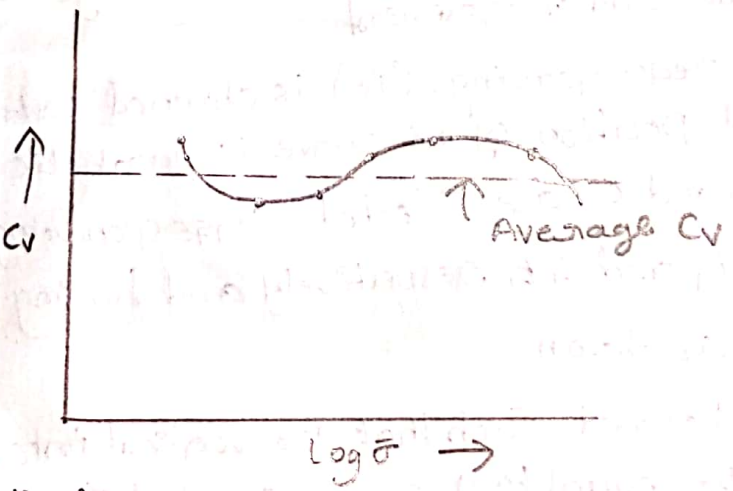
$$d = \frac{H}{2} = \frac{1}{2}\left[\frac{H_i + H_f}{2}\right]$$

for single drainage $d = H = \left[\frac{H_i + H_f}{2}\right]$

→ the test is repeated for different load increament & an average value of C_v obtained. as shown in fig.

(2) Logarithm of time method :-

→ the method given by Casagrande uses the theoretical curve b/w U and log T_v as shown in fig.



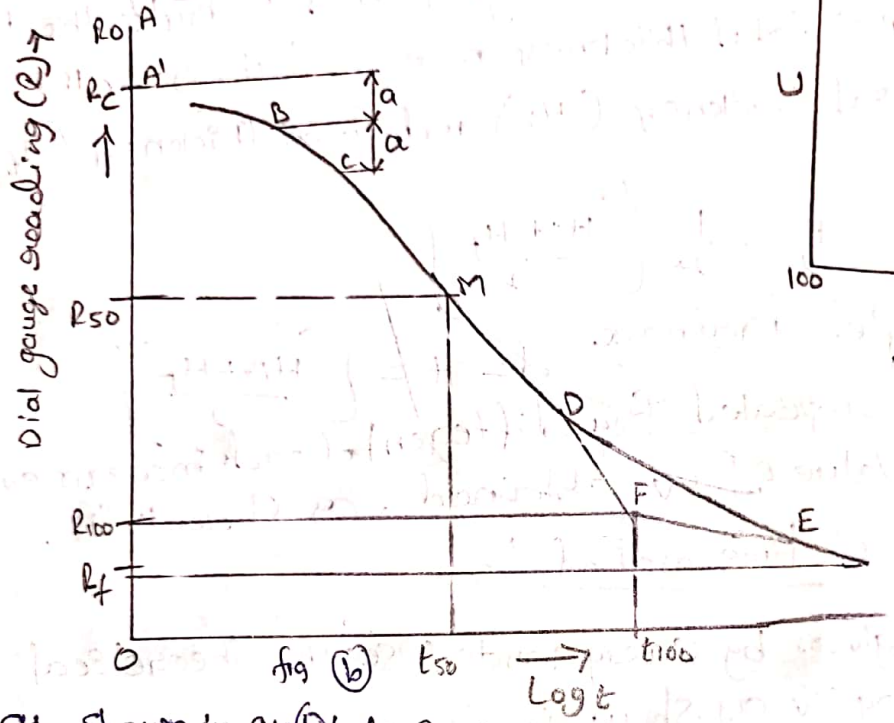
→ ~~the method given by Casa~~

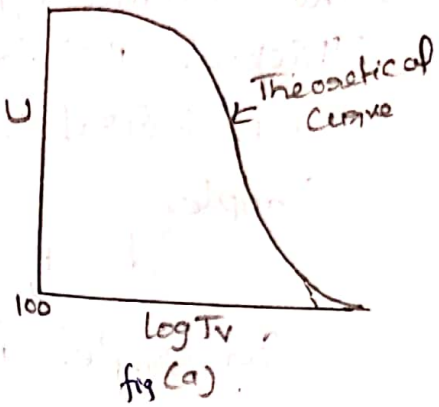
→ the curve consists of three parts (i) an initial portion which is parabolic in shape. (ii) A middle portion which is almost linear and (iii) the last portion to which the horizontal axis is an asymptote.

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→ It is observed that the point of intersection of the tangent drawn at the point of inflexion on the curve and the asymptote of the lower position gives the value of 100% Consolidation.

→ The sample of the soil is tested as explained in section for a given load increment a curve is plotted between the dial gauge reading R and log t





→ as shown in fig (b) Let R0 be the initial dial gauge before the application of the load increment.

→ The corrected zero readings (R0) is obtained using the fact that the initial position of the curve is parabolic.

→ Two points B and C are selected corresponding to some arbitrary time t_1 and $4t_1$ respectively and having the vertical Intercept 'a' as shown

→ point A' is located such that the vertical intercept b/w B and A' is also equal to a. It represents the corrected dial gauge reading Rc corresponding to zero primary Consolidation.

→ As a check the procedure can be repeated by selecting two other points with the time ratio 1:4 it should also

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– give approximately the same location of point A' obviously the consolidation b/w the dial gauge reading R_0 to R_c represented by A and A', is initial compression.

→ the final position of the experimental curve is linear the point F corresponding to 100%. Consolidation is obtained from the intersection of the two linear parts as shown.

→ The values of R_{100} and t_{100} are obtained corresponding to point F. The compression b/w the dial gauge reading R_c & R_{100} is the primary consolidation and that b/w R_{100} and R_f is the secondary consolidation.

→ The point M corresponding to 50% primary consolidation (R_{50}) is located midway b/w R_c & R_{100} and the value of time t_{50} is obtained.

i.e $R_c - R_{50} = \frac{1}{2}(R_c - R_{100})$.

From Table for U = 50%. the value of T_v is equal to 0.196.

$$C_v = \frac{0.196 d^2}{t_{50}}$$

(*) Compression Ratios

(1) initial compression ratio (a_i) :-

→ it is the ratio of the initial compression to the total compression in terms of dial gauge reading. it is expressed as.

$$a_i = \frac{R_0 - R_c}{R_0 - R_f}$$

where R_0 = zero dial gauge
R_c = corrected zero
R_f = final dial gauge.

(2) primary compression ratio (a_p) :

→ it is ratio of the primary compression to the total compression in terms of the dial gauge reading. it is given by.

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$$\alpha_p = \frac{R_c - R_{100}}{R_0 - R_f}$$

where
R_{100} = dial gauge reading corresponding to 100% primary consolidation.

(iii) Secondary Compression ratio (α_s):

→ It is the ratio of the secondary compression to the total compression in terms of dial gauge readings

$$\alpha_s = \frac{R_{100} - R_f}{R_0 - R_f}$$

It can also be written $\alpha_s = 1 - (\alpha_i + \alpha_p)$

because $\alpha_s + \alpha_i + \alpha_p = 1$

(*) Final Settlement of a Soil Deposit in the field

→ Computation of ~~magnitude of~~ settlement of a soil deposit in the field consists of two parts.

(i) Computation of magnitude of final settlement
(ii) Determination of the time-rate of settlement.

→ For the computation of final settlement, the coefficient of volume change or the compression index is required which is based on the plot b/w void ratio and the effective stress

→ For the time rate of computation the Terzaghi theory is used. It requires the coefficient of consolidation and is

(i) Final settlement using Coefficient of Volume change:

→ Let us consider a small element of thickness Δz at a depth z in the clay deposit of total thickness H_0

→ Let the effective pressure increment causing the settlement be $\Delta\bar{\sigma}$ from equation.

$$\Delta H = m_v H_0 (\Delta\bar{\sigma})$$

→ Representing the final settlement as Δs_f and taking

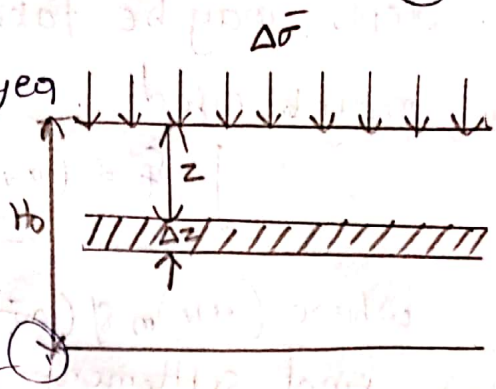
$H_0 = \Delta z$

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$\Delta S_f = m_v \Delta z (\Delta \bar{\sigma})$

→ total Settlement of the complete layer

$$S_f = \int_0^{H_0} \Delta S_f = \int_0^{H} m_v \Delta \bar{\sigma} \, dz$$



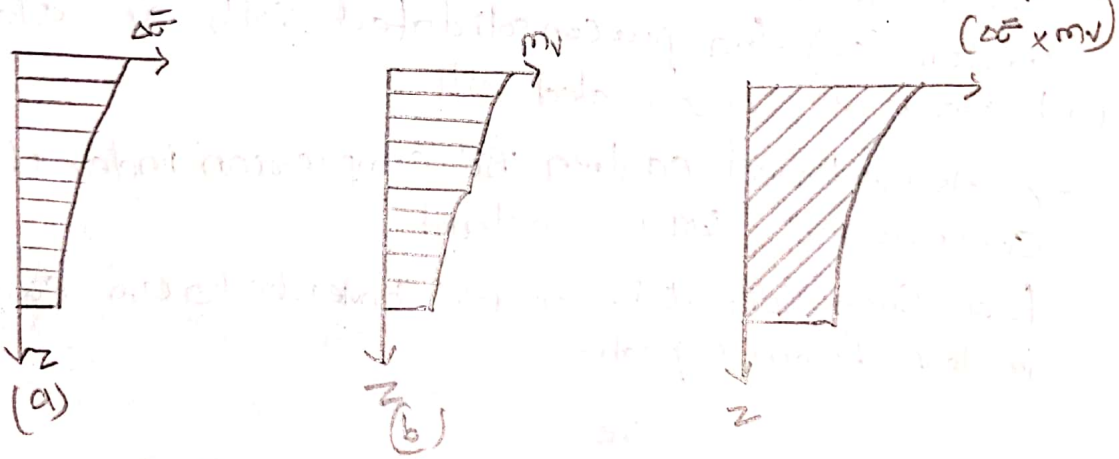
if both m_v and $\Delta \bar{\sigma}$ are constant

$$S_f = m_v \Delta \bar{\sigma} H_0 \quad \text{— (1)}$$

→ In an actual case as the pressure intensity $\Delta \bar{\sigma}$ and the coefficient of volume change m_v decrease with depth z. it is convenient to divide the stratum into a number of small layers and to assume $\Delta \bar{\sigma}$ and m_v as constant in each layer. The final Settlement is given by

$$S_f = \sum_{i=1}^{i=n} (m_v)_i \, (\Delta \bar{\sigma})_i \, (\Delta z)_i \quad \text{— (2)}$$

→ The settlement S_f may also be obtained by the method of graphical integration. The variation of $\Delta \bar{\sigma}$ and m_v with the depth z is shown in figs. (a) & (b) respectively.



→ Shows a fig (c) the variation of the product $(\Delta \bar{\sigma} \times m_v)$ with the depth z. The final settlement is equal to the area of the diagram.

→ However if the thickness of the clay layer is relatively small the variation of $\Delta \bar{\sigma}$ and m_v with depth z may be assumed as linear the value at the mid

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depth may be taken as representative values. equation may be used

$$\boxed{S_f = (m_v)_m (\Delta\bar{\sigma})_m H_0} \quad -(3)$$

where $(m_v)_m$ & $(\Delta\bar{\sigma})_m$ are the values at mid-depth.

(4) Final settlement using void Ratio:

→ if e-$\bar{\sigma}$ plot for the soil is available it can be used to determine the final settlement. The values of Δe corresponding to the given load increament is read off from the plot and substituted

$$\Delta H = H_0 \left(\frac{\Delta e}{1+e_0}\right)$$

$$\boxed{S_f = H_0 \cdot \left(\frac{\Delta e}{1+e_0}\right)} \quad -(1)$$

where e_0 is the initial void ratio.

→ The usual practice is not to use Δe but to use the Coefficient of compression index (C_c) for normally Consolidated soils and the Coefficient of decompression Index (C_s) for preconsolidated soils as explained below

(a) Normally consolidated soils:

→ As mentioned earlier the compression index of a normally consolidated soil is constant.

for such soils it is more convenient to use compression index from equation.

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

$$-\Delta e = C_c \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

where Δe is change in void ratio when the effective pressure is increased from $\bar{\sigma}_0$ to $(\bar{\sigma}_0 + \Delta\bar{\sigma})$. Substituting this value of Δe in equation.

$$\boxed{S_f = \frac{C_c}{1+e_0} \cdot H_0 \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)} \quad -(1)$$

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(b) Preconsolidated Soils :-

→ The final settlement are small in the case of preconsolidated soils as the recompression index C_r is considerably smaller than the compression index from equation.

$$-\Delta e = C_r \log\left(\frac{\bar{\sigma_o} + \Delta\bar{\sigma}}{\bar{\sigma_o}}\right)$$

$$S_f = \frac{C_r}{1+e_o} \cdot H_o \log\left(\frac{\bar{\sigma_o} + \Delta\bar{\sigma}}{\bar{\sigma_o}}\right)$$

→ The above equation is applicable when $(\bar{\sigma_o} + \Delta\bar{\sigma})$ is smaller than to ~~pre~~ preconsolidation pressure $\bar{\sigma_c}$.

→ if the preconsolidation pressure $\bar{\sigma_c}$ is greater than $\bar{\sigma_o}$ but less than $(\bar{\sigma_o} + \Delta\bar{\sigma})$ the settlement is computed in two parts.

(i) Settlement for pressure $\bar{\sigma_o}$ to $\bar{\sigma_c}$

(ii) Settlement for pressure $\bar{\sigma_c}$ to $(\bar{\sigma_o} + \Delta\bar{\sigma})$

→ for the first part the recompression index is applicable whereas for the second part, the compression index is used.

i.e $$S_f = \frac{C_r}{1+e_o} \cdot H_o \log\left(\frac{\bar{\sigma_c}}{\bar{\sigma_o}}\right) + \frac{C_c H_o}{1+e_o} \log\left(\frac{\bar{\sigma_o} + \Delta\bar{\sigma}}{\bar{\sigma_c}}\right)$$

(*) Time Settlement Curve :-

→ The load is not applied to a soil deposit in the field instantaneously as assumed in the Consolidation theory

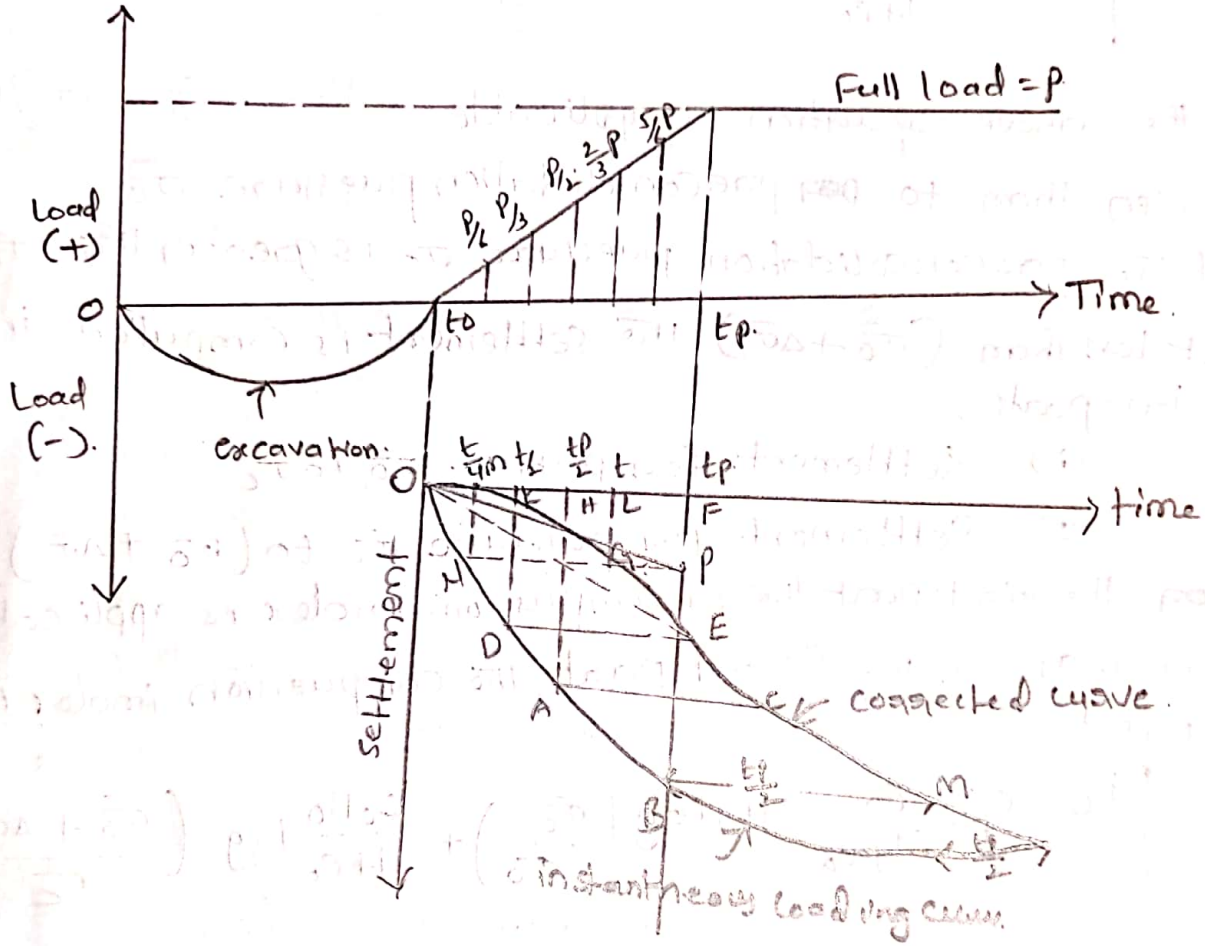
→ First excavation is done for the footing it causes a decrease in the stresses. After the excavation the footing is constructed and the load of the superstructure is applied gradually as shown in fig.

→ The net load on the soil becomes zero say at time $t = t_0$ it then gradually increases to full load p at $t = t_p$ they

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There is expan of soil due to removal of load in the beginning and it is followed by recompression it is generally assumed that the net compression of the soil during the period $t=0$ to $t=t_0$ is negligible.

→ The actual loading period is from time $t=t_0$ to $t=t_p$. during this period the loading is assumed to vary linearly from zero to full load P.



→ The time settlement curve in the field is observed based on the assumption that the settlement at the end of construction period (t_p) is the same as that would have occurred in half as much time had the entire load been applied instantaneously

→ In other words the actual settlement at time t_p is equal to that at $t_p/2$ due to instantaneous loading. as shown in fig.

→ The curve OB is the load settlement curve obtained using the Terzaghi theory assuming that the full load P is applied instantaneously at time t_0.

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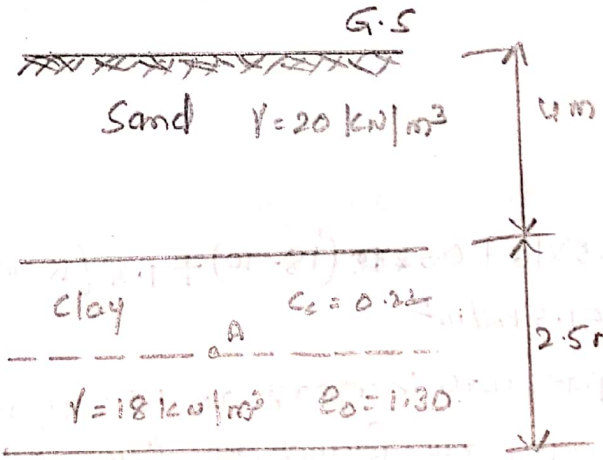
→ The corrected curve is obtained from the instantaneous loading curve. The point c on the corrected curve has the settlement Fc at time t_f This settlement is equal to the settlement AH at time $\frac{t_p}{2}$ of the instantaneous curve.

→ A horizontal line AC is drawn from point A to cut the vertical Fc at time t_p at C.

→ The settlement on the corrected curve at any other time is also obtained from the Instantaneous curve the settlement 'LG' at time t is obtained from the settlement KD at time $\frac{t}{2}$ of the instantaneous curve but correction is applied.

example problem

① Calculate the final settlement of the clay layer shown in fig due to an increase of pressure of 30 kN/m² at mid-height of the layer. Take $\gamma_w = 10$ kN/m³. Also calculate the settlement when the water table arises to the ground surface.



Sol: initial pressure at the centre of the clay layer.

$\bar{\sigma_0} = h \times \gamma + 1.25 \times \gamma$

$= 4 \times 20 + 1.25 \times 18$

$= 102.5$ kN/m²

$S_f = \frac{C_c}{1+e_0} \cdot H_0 \log_{10}\left(\frac{\bar{\sigma_0} + \Delta\bar{\sigma}}{\bar{\sigma_0}}\right)$

$= \frac{0.22}{1+1.30} 2.5 \log_{10}\left(\frac{102.5+30}{102.5}\right)$

$= 0.0263$ m $= 2.63$ cm

→ When the water table arises to the ground surface

$\bar{\sigma_0} = 4(20-10) + 1.25 \times (18-10) = 50$ kN/m²

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$$S_f = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{\bar{\sigma_0}+\Delta\bar{\sigma}}{\bar{\sigma_0}}\right)$$

$$= \frac{0.22}{1+1.30} \times 2.5 \log_{10}\left(\frac{50+30}{50}\right)$$

$$= 0.0488\,m = 4.88\,cm$$

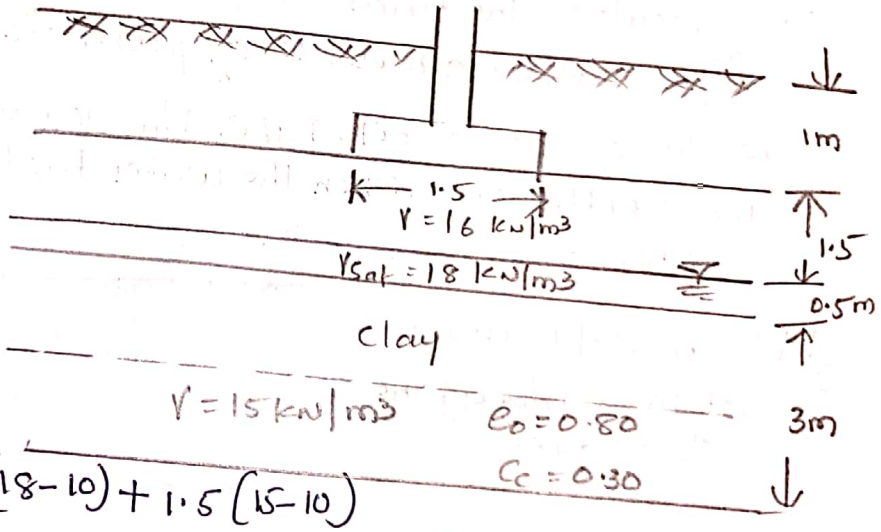
→ As expected the settlement increase due to the rise of the water table.

(2) A footing has a size of 3 m by 1.5 m and it cause a pressure increment of 200 kN/m² at its base as shown in fig Determine the consolidation settlement at the middle of the clay layer. Assume 2:1 pressure distribution and Consider the variation of pressure across the depth of the clay layer

$\gamma_w = 10\,kN/m^3$.

Sol:

initial pressure at the centre of the clay layer,



$$\bar{\sigma_0} = 2.5\times16 + 0.5(18-10) + 1.5(15-10)$$

$$= 51.5\,kN/m^2$$

→ The pressure increase at the top, middle & the bottom of the clay layer are found as follows from equation.

for Rectangular Area (B×L)

$$\sigma_z = \frac{q(B\times L)}{(B+z)(L+z)}$$

$$(\Delta\sigma)_{top} = \frac{200\times(3\times1.5)}{(3+2)(1.5+2)} = 51.4\,kN/m^2$$

$$(\Delta\sigma)_{middle} = \frac{200(3\times1.5)}{(3+3.5)(1.5+3.5)} = 27.7\,kN/m^2$$

$$(\Delta\sigma)_{bottom} = \frac{200(3\times1.5)}{(3+5)(1.5+5)} = 17.3\,kN/m^2.$$

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→ the average pressure can be found from the following equation.

$$\Delta\sigma = \frac{1}{6}\left[(\Delta\sigma)_t + 4(\Delta\sigma)_m + (\Delta\sigma)_b\right]$$

$$\Delta\sigma = \frac{1}{6}\left[51.4 + 4\times 27.7 + 17.3\right] = 29.9 \text{ kN/m}^2$$

$$S_f = \frac{C_c}{1+e} H_0 \log_{10}\left(\frac{\Delta\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= \frac{0.30}{1+0.80} \times 3 \log_{10}\left(\frac{51.5+29.9}{51.5}\right)$$

$$S_f = 0.09941 \text{ m} = 99.41 \text{ mm}$$

$S_2 = \frac{\Delta x}{3}[f(x_0) + 4f(x_1) + f(x_2)]$

$\Delta x = \frac{b-a}{n}$

$\Delta x = \frac{2-1}{2} = \frac{1}{2}$

$\Delta x = \frac{b-a}{n} = \frac{3-1}{3} = \frac{1}{3}$

$\frac{2}{3}\times 3 = \frac{y}{94.5}$

(3) A 3m thick clay layer beneath a building is overlain by a permeable stratum and is underlain by an impervious rock. The coefficient of consolidation of the clay was found to be 0.025 cm²/minute. The final expected settlement for the layer is 8cm (a) How much time will it take for 80% of the total settlement to take place? (b) Determine the time required for a settlement of 2.5cm to occur (c) Compute the settlement that would occur in one year.

Given data
$d = 300$ cm.
$C_v = 0.025$
$U = 80\%$

Sol: (a) Length of drainage path $d = 3\text{m} = 300\text{cm}$

from equation $C_v = \frac{T_v d^2}{t}$

Substituting the values

$$0.025 = \frac{T_v (300)^2}{t}$$

$$t \times 0.025 = T_v (300)^2$$

$$t = 3.6 \times 10^6 T_v$$

→ from Table $U = 80\%$. $T_v = 0.567$

$$t = 3.6\times 10^6 \times 0.567$$

$$= 2.041\times 10^6 \text{ minutes}$$

$$= 3.883 \text{ years}.$$

1 min $= 1.90259^{-6}$

1 minute $= 1.90259\times 10^{-6}$ y.

(b) when the settlement is 2.5cm

$$U = \frac{2.5}{8} \times 100 = 31.25\%$$

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from Table $U = 31.25\%$.

$T_V = 0.078$

$t = 3.6 \times 10^6 \times 0.078$

$= 0.2808 \times 10^6$ minute

$\approx$ 195 days.

(c) from equation $1 \times 365 \times 24 \times 60 = 3.6 \times 10^6 T_V$

$= 3.6 \times 10^6 T_V$

$T_V = \dfrac{1 \times 365 \times 24 \times 60}{3.6 \times 10^6}$

$= 0.146$

from table $T_V = 0.146$

$U = 0.429$.

$\therefore$ Settlement $= U \times 8 = 0.429 \times 8 = 3.432$ cm //

1x365x24x60

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: GEOTECHNICAL ENGINEERING

SUBJECT CODE: 20CE5PC03

FACULTY NAME: B. SURESH

SYLLABUS:

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 strength envelops – Shear strength of sands - dilatancy – critical void ratio., Introduction to stress path method.

Unit- V

Shear Strength of Soils

(x) Introduction & Importance of Shear Strength :

→ The shear strength of a soil is its maximum resistance to shear stress just before the failure.

→ Soils are seldom subjected to direct shear. However the shear stresses develop when the soil is subjected to direct compression

→ Although shear stresses may also develops when the soil is subjected to direct tension, but these shear stress are not relevant as the soil in this case fails in tension & does not fail in shear.

→ In field soils are seldom subjected to tension as it cause opening of the cracks and fissures. These cracks are not only undesirable but are also detrimental to the stability of the soil masses

(2) Importance of the Shear Strength :-

→ Shear strength is the principal engineering property which controls the stability of a soil mass under loads.

→ it governs the bearing capacity of soils the stability of slopes in soils the earth pressure against retaining structures and many other problems

→ All the problems of soil engineering are related in one one way or the other with the shear strength of the soil

→ Unfortunately the shear strength is on of the most complex engineering properties of the soil

→ The current research is giving new concepts & theories

→ For the design of embankment for dams, roads, pavement

- excavations, levees etc. the analysis of the stability the slope is done using shear strength.

→ the shear strength of a soil mass is essentially made up of
(i) Due to the interlocking of the grains the structural resistance of the movement of the soil is very essential.
(ii) An other important component is the frictional resistance b/w the individual soils grains at their contact point on sliding

(*) MOHR'S CIRCLE:

→ 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 and is extremely useful.

→ In this method an origin O is selected and the normal stresses are plotted along horizontal axis and the shear stresses on the vertical axis.

→ As the compressive stresses are taken positive in soil engineering these are stresses on the vertical axis. As the compressive stresses are taken positive in soil engineering these are plotted towards the right of the origin i.e along positive x-axis.

→ the shear stress is generally taken as positive if it causes a counter clockwise couple at a point inside the wedge ABC in fig.

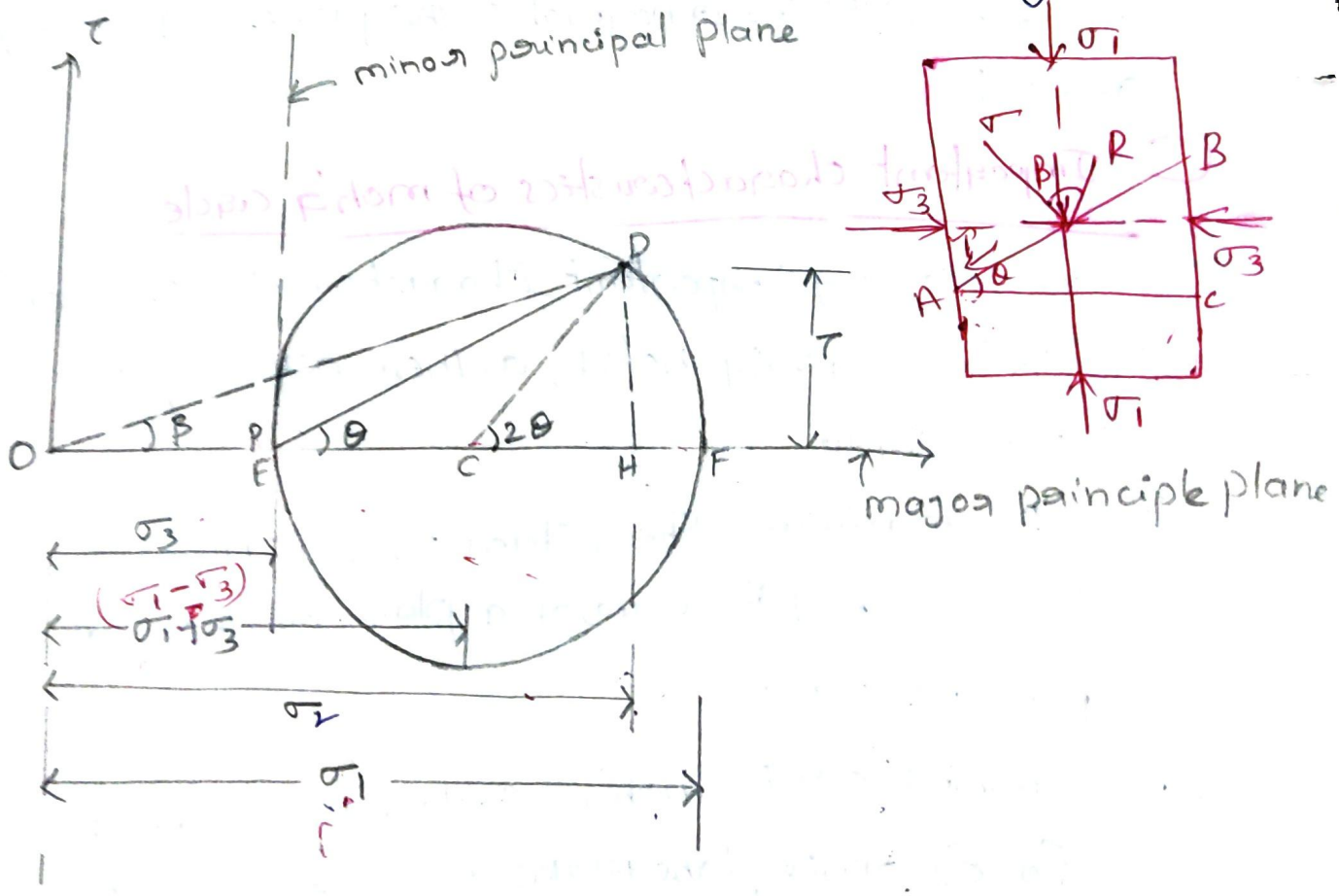
→ Thus the shear stress marked on the plane AB is positive the positive shear stresses are plotted upward from the origin i.e along positive y-axis Let us draw the mohr circle for the stresses shown in fig.

→ as shown in fig the point E represents the minor principal stress σ_3 and the point F, the major principal stress σ_1, the point C is the middle point with the normal stress coordinate equal to $\left(\frac{\sigma_1+\sigma_3}{2}\right)$

→ The circle is drawn with C as the centre and EF as diameter. The circle is known as mohr's circle. each point on the circle gives the stresses σ and τ on a particular plane.

→ It can be shown that the point D on the circle gives the stress on the plane AB inclined at an angle θ to the major principle plane

→ The line DE makes an angle θ with σ-axis the angle DCF subtended at the centre is obviously twice the angle DEC.



→ From the fig $OH = OC + CH$.

$$OH = \frac{\sigma_1+\sigma_3}{2} + \left(\frac{\sigma_1-\sigma_3}{2}\right)\cos 2\theta = \sigma$$

$$DH = CD\sin 2\theta = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta = \tau$$

→ The point E is a unique point, which is known as the pole (P) (or) the origin of planes (OP) if a line is

drawn from any point (D) on the mohr circle parallel to the plane (say AB) whose stresses are represented by that point it will intersect the circle at the pole P.

→ When the major principal plane is horizontal (parallel to x axis) the minor principal plane is vertical and the pole lies at the point E which indicates the minor principal stress.

→ Once the pole has been located the stresses any other plane making an angle α with the major principal plane can be determined graphically by drawing a line through the pole and making an angle α with σ axis.

→ the coordinates of the point obtained by the intersection of this line with the circle give the stresses on that plane

→ the line OD represent the magnitude of the resultant stress on the inclined plane AB. the angle of the obliquity of the resultant with the normal of the plane AB is equal to the angle B.

(iv) Important characteristics of mohr circle:

→ the following important characteristics of mohr's circle should be carefully noted, as these are required for further study.

(1) the maximum shear stress τ_{max} is numerically equal to $\left(\frac{\sigma_1 - \sigma_3}{2}\right)$ and it occurs on a plane inclined at 45° to the principal planes.

(2) point 'D' on the mohr's circle represents the stresses (σ, τ) on the plane make an angle θ with the major principal plane. the resultant stress on that plane is equal to $\sqrt{\sigma^2 + \tau^2}$ and its angle of obliquity with the normal of the plane is equal to angle B. given by

$$B = \tan^{-1}\left(\frac{\tau}{\sigma}\right)$$

(3) The maximum angle of obliquity β_{max} is obtained by drawing a tangent to the circle from the

$$\beta_{max} = \sin^{-1}\frac{\left(\frac{\sigma_1-\sigma_2}{2}\right)}{\left(\frac{\sigma_1+\sigma_2}{2}\right)} = \sin^{-1}\left(\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}\right)$$

(4) The shear stress τ_f on the plane of the maximum obliquity is less than the maximum shear stress τ_{max}.

(*) Mohr - Coulomb Theory :-

⇒ The soil is a particulate material the shear failure occurs in soils by slipage of particles due to shear stresses.

→ The failure is essentially by shear, but shear stresses at failure depend upon the normal stresses on the potential failure plane.

→ According to mohr, the failure is caused by a critical combination of the normal and shear stresses.

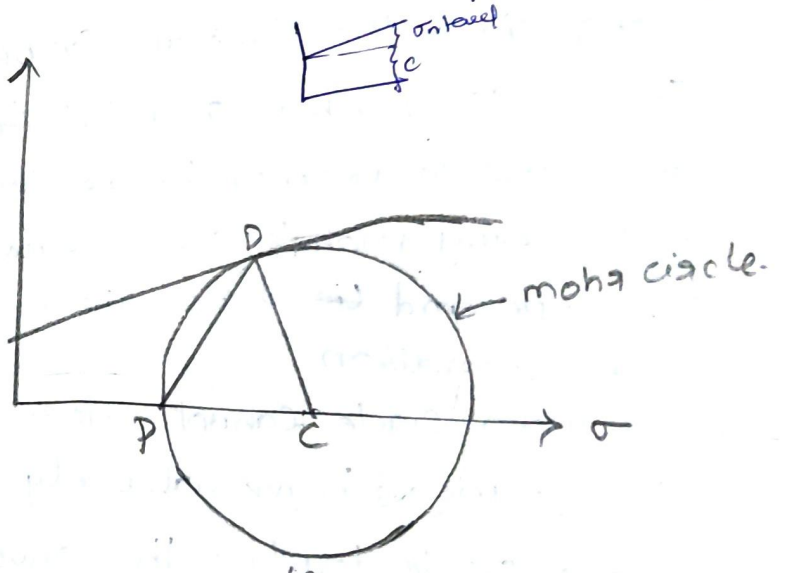
→ The soil fails when the shear stress (τ_f) on the failure plane at failure is a unique function of the normal stress (σ) acting on that plane

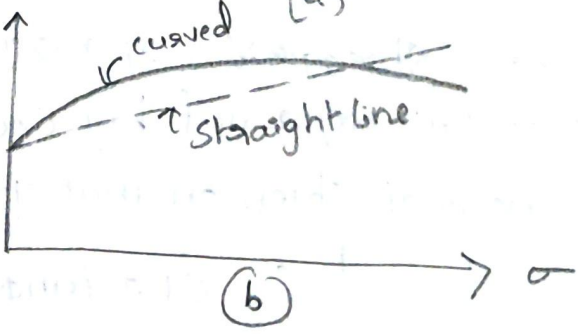
$$\tau_f = f(\sigma)$$

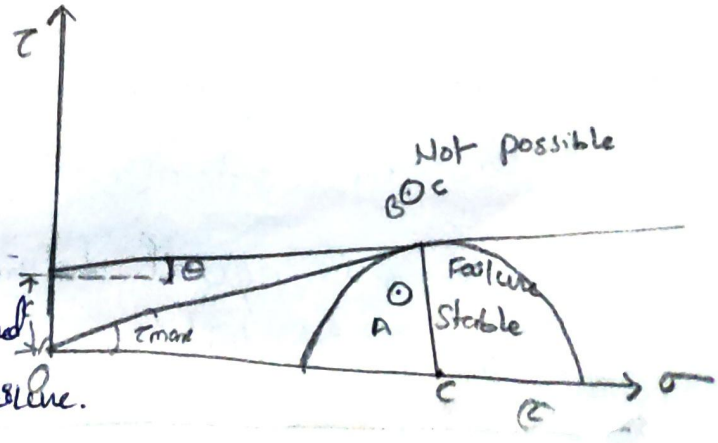
→ Since the shear stress on the failure plane at failure is defined as the shear strength (s), the above equation can be written as

$$s = f(\sigma)$$

The mohr theory is concerned with the shear stress at failure.







→ A plot can be made b/w the shear stress τ and the normal stress σ at failure.

→ The curve defined by equation is known as the mohr envelope shown in fig (a) There is a unique failure envelope for each material.

→ Failure of the material occurs when the mohr circle of the stresses touches the mohr envelope.

→ As discussed in the preceding sections the mohr circle represent all possible combinations of shear and normal stresses at the stressed point.

→ At the point of contact (D) of the failure envelope and the mohr circle the critical combination of shear and normal stresses is reached and the failure occurs.

→ The plane indicated by the line PD is therefore the failure plane. Any mohr's circle which does not cross the failure envelope and lies below the envelope represents a (non-failure) stable condition.

→ The mohr circle cannot cross the mohr envelope, as the failure would have already occurred as soon as the mohr circle touched the envelop.

→ The shear strength (s) of a soil at a point on a particular plane was expressed by Coulomb as a linear function of the normal stress on that plane as.

$$\boxed{S = c + \sigma \tan\phi} \quad \text{--- (1)}$$

→ In other words, the mohr envelope is replaced by a straight line by Coulomb as shown in fig.

→ in equation (1) c is equal to the intercept on τ-axis and ϕ is the angle which the envelope makes with σ-axis

→ The component c of the shear strength is know as cohesion. cohesion holds the particles of the soil together in a soil mass and is independent of the normal stress.

→ The angle ϕ is called the angle of internal friction

(*) Revised Moha - Coulomb equation :-

→ Later research showed that the parameters c and ϕ in equation (1) are not necessarily fundamental properties of the soil as was originally assumed by Coulomb

→ These parameters depend upon a number of factors such as the water content, drainage conditions, conditions of testing.

→ The current practice is to consider c' and ϕ' as mathematical parameters which represent the failure conditions for a particular soil under given conditions.

→ That is the reason why c and ϕ are now called cohesion intercept and the angle of shearing resistance. These indicate the intercept and the slope of the failure envelope, respectively.

→ Terzaghi established that the normal stresses which control the shear strength of a soil are the effective stresses and not the total stress.

→ In terms of effective stress it is written as. ϕ' c' = effective

$$\boxed{S = c' + \bar{\sigma} \tan\phi'} \quad (1)$$

where c' and ϕ are cohesion intercept and the angle of shearing resistance in terms of the effective stresses.

→ in equation (1) is known as the Revised moha-Coulomb equation for the shear strength of the soil.

(*) Mohr's - Coulomb failure theories

→ Coulomb observed that one component of the shearing strength called intrinsic cohesion (or apparent cohesion) is constant for a given soil and is independent of applied stress.

→ The other component, namely the frictional resistance varies directly as the magnitude of the normal stress on the plane of rupture.

→ Coulomb equation is written as

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

where τ_f = Shear strength of soil
c = Apparent cohesion
σ = Normal stress on plane of rupture.

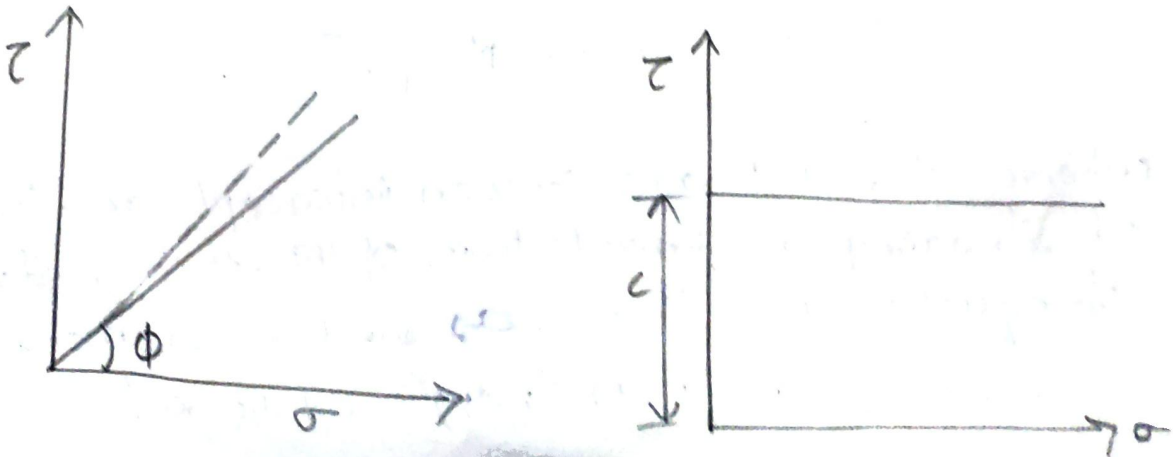
→ Mohr-Coulomb failure criteria can be expressed in the form of

$$\tau_{ff} = c + \sigma_{ff} \tan\phi$$

Also angle of failure can be expressed in the form of angle of shearing resistance ϕ

$$\theta_{FF} = 45^\circ + \phi/2$$

→ The shearing strength envelope will be tangential to any mohr circle at failure as follows

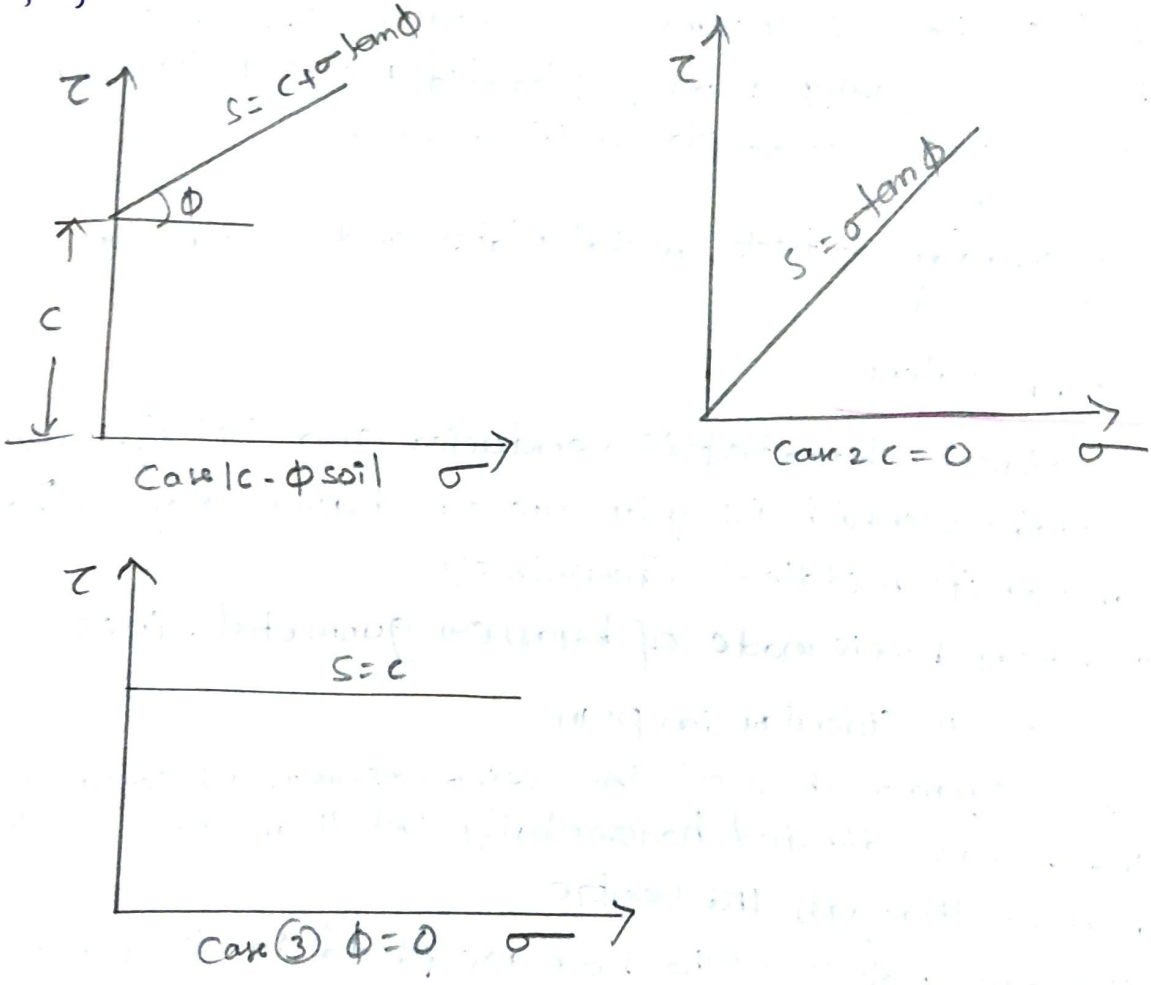


Torsion test

→ Based on values of shear strength parameters, soil can be classified.

(1) cohesive soil
(2) cohesionless soil
(3) purely cohesive soil.

→ The strength envelops for the three cases are shown in fig.



(*) Types of laboratory tests for strength parameters:

→ The following tests are available for this purpose.

(*) Laboratory tests.

① Direct shear test
② Triaxial Compression Test
③ Unconfined compression test
④ Laboratory vane shear test
⑤ Torsion test

(6) Ring shear tests.

(*) Field Tests

(1) Vane shear test
(2) penetration Test.

(1) Direct shear test:

→ The direct shear device, also called the 'shear box apparatus' essentially consists of a brass box, split horizontally at mid height of the soil specimen, as shown schematically in fig.
→ The soil is gripped in perforated metal grilles, behind which porous discs can be placed if required to allow the specimen to drain.
→ For undrained tests, metal plates and solid metal grilles may be used.

(a) Apparatus:

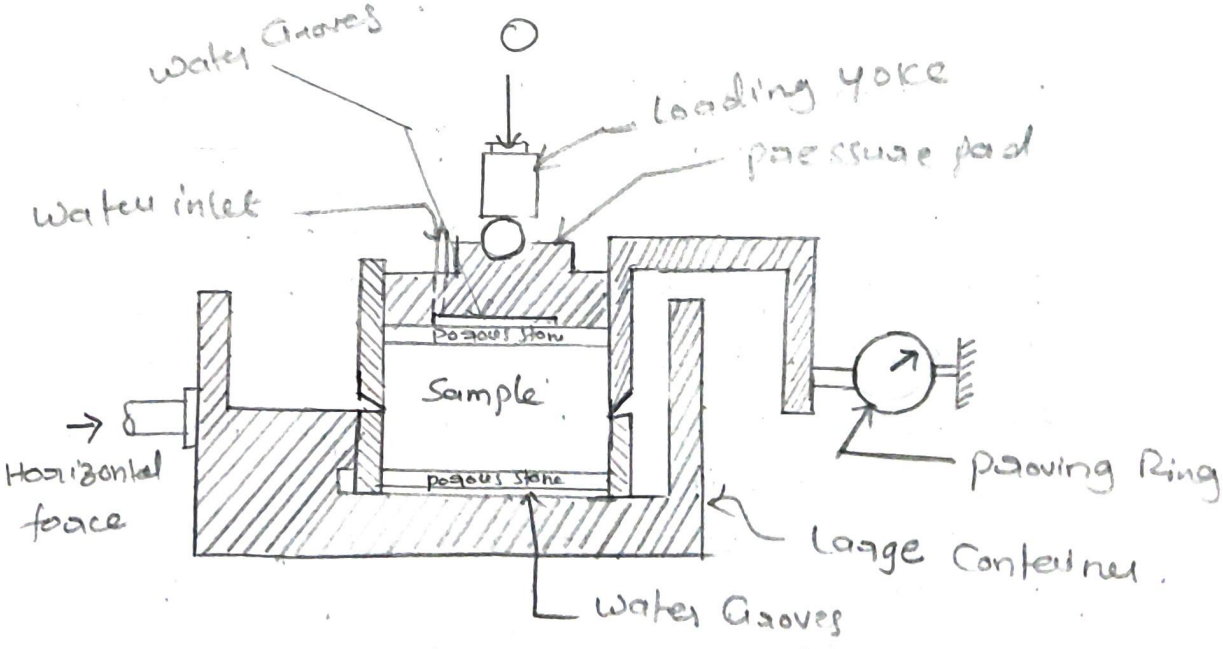
→ A direct shear test is conducted on a soil specimen in a shear box which is split into two halves along a horizontal plane at its middle as shown in fig.
→ The shear box is made of brass (or) gunmetal. it is either square (or) circular in plan.
→ A square box of size 60x60x50mm is commonly used the box is divided horizontally such that the dividing plane passes through the centre.
→ The two halves of the box are held together by locking pins.
→ Suitable spacing screws to separate the two halves are provided. The spacing screws are fixed to the upper half and they butt against the top of the lower half.
→ The box is provided with the gripper on the grid plates which are toothed and fitted inside it the gripper plates are plains for undrained tests and perforated for drained tests.

→ A pressure pad of brass or gun metal is fitted into the box at its top to transmit the normal load to the sample

→ The normal load from the loading yoke is applied on the top of the specimen through a steel ball bearing upon the pressure pad.

→ 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. The container can be pushed forward at a constant rate by a geared jack which works as a strain-controlled device the jack may be operated manually (or) by an electric motor.



→ A loading frame is used to support the large container it has the arrangement of a loading yoke and 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.

→ The proving ring bears against a fixed support As the box moves the proving ring records the shear for

→ The shear displacement is measured with dial gauge fitted to the container. Another dial gauge is fitted to the top of the pressure pad to measure the change in the thickness of the specimen.

(b) Test :-

→ A soil specimen of size 60 x 60 x 25mm is taken it may be either an undisturbed sample or made from compacted and remoulded soil.

→ The specimen may be prepared directly in the box and compacted.

→ The base plate is attached to the lower half of the box. A porous stone is placed in the box.

→ ~~A porous stone is placed in the~~

→ For undrained tests, a plain grid is kept on the porous stone, keeping its segregations at right angles to the direction of shear for drained tests.

→ perforated grids are used instead of plain grids. The mass of the base plate, porous stone and grid is taken. The specimen if made separately is transferred to the box and its mass taken

→ The upper grid, porous stone and the pressure pad are placed on the specimen. The box is placed inside the large container and mounted on the loading frame.

→ The upper half of the box is brought in contact with the proving ring.

→ The loading yoke is mounted on the steel ball placed on the pressure pad.

→ The dial gauge is fitted to the container to give the shear displacement. The other dial gauge is mounted on the loading yoke to record the vertical movement.

The shear test must be conducted under appropriate drainage conditions that simulate the actual problem. In shear tests, there are two stages :

(1) Consolidation stage in which the normal stress (or confining pressure) is applied to the specimen it is allowed to consolidate.

(2) Shear stage in which the shear stress (or deviator stress) is applied to the specimen to shear it.

Depending upon the drainage conditions, there are three types of tests as explained below :

(1) Unconsolidated—Undrained Condition. In this type of test, no drainage is permitted during consolidation stage. The drainage is also not permitted in the shear stage.

As no time is allowed for consolidation or dissipation of excess pore water pressure, the test can conducted quickly in a few minutes. The test is known as unconsolidated—undrained test (*UU* test) or qu test (*Q*-test).

(2) Consolidated—Undrained Condition. In a consolidated—undrained test, the specimen is allowed onsolidate in the first stage. The drainage is permitted until the consolidation is complete.

n the second stage when the specimen is sheared, no drainage is permitted. The test is known consolidated—undrained test (*CU* test) It is also called a *R*-test, as the alphabet *R* falls between the alpha *Q* used for quick test, and the alphabet *S* used for slow test.

The pore water pressure can be measured in the second stage if the facilities for its measurement available. In that case, the test is known as $\overline{CU}$ test.

(3) Consolidated—Drained Condition. In a consolidated—drained test, the drainage of the specimen permitted in both the stages. The sample is allowed to consolidate in the first stage. When the consolidati is complete, it is sheared at a very slow rate to ensure that fully drained conditions exist and the excess po water is zero.

The test is known as a consolidated—drained test (*CD* test) or drained test. It is also known as the slo test (*S*—test).

13.10. MODE OF APPLICATION OF SHEAR FORCE

The shear force in a shear test is applied either by increasing the shear displacement at a given rate by increasing the shearing force at a given rate. Accordingly, the shear tests are either strain—controlled stress-controlled.

(1) Strain controlled tests. In a strain-controlled test, the test is conducted in such a way that th shearing strain increases at a given rate. Generally, the rate of increase of the shearing strain is kept constan and the specimen is sheared at a uniform strain rate.

The shear force acting on the specimen is measured indirectly using a proving ring. The rate of shearin strain is controlled manually or by a gear system attached to an electric motor.

Most of the shear tests are conducted as strain—controlled. The stress—strain characteristic are easi obtained in these tests, as the shape of the stress—strain curve beyond the peak point can be observed on n a strain—controlled test. A strain—controlled test is easier to perform than a stress- controlled test.

(2) Stress—Controlled tests. In a stress—controlled test, the shear force is increased at a given rate Usually, the rate of increase of the shear force is maintained constant. The shear load is increased such tha the shear stresses increase at a uniform rate. The resulting shear displacements are obtained by means of dial gauge.

Stress—controlled tests are preferred for conducting shear tests at a very low rate, because an applie load can easily be kept constant for any given period of time. Further, the loads can be conveniently applie and removed. The stress-controlled test represents the field conditions more closely.

13.11 DIRECT SHEAR TEST

(a) **Apparatus.** A direct shear test is conducted on a soil specimen in a shear box which is split into two halves along a horizontal plane at its middle (Fig. 13.7). The shear box is made of brass or gunmetal. It

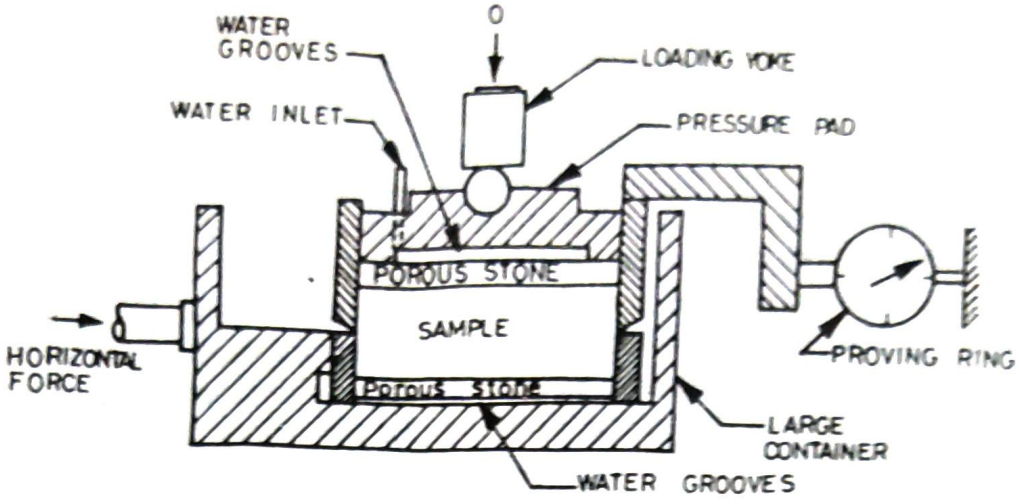


Fig. 13.7. Direct Shear Test.

either square or circular in plan. A square box of size 60 × 60 × 50mm is commonly used. The box is divided horizontally such that the dividing plane passes through the centre. The two halves of the box are held together by locking pins. Suitable spacing screws to separate the two halves are also provided. The spacing screws are fixed to the upper half and they butt against the top of the lower half.

The box is provided with the gripper or the grid plates which are toothed and fitted inside it. The gripper plates are plain (without perforations) for undrained tests and perforated for drained tests. Porous stones are placed at the top and the bottom of the specimen in drained tests. A pressure pad of brass or gun metal is fitted into the box at its top to transmit the normal load to the sample. The normal load from the loading yoke is applied on the top of the specimen through a steel ball bearing upon the pressure pad.

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 (rollers not shown). The container can be pushed forward at a constant rate by a geared jack which works as a strain-controlled device. The jack may be operated manually or by an electric motor.

A loading frame is used to support the large container. It has the arrangement of a loading yoke and 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.The proving ring butts against a fixed support. As the box moves, the proving ring records the shear force. The shear displacement is measured with a dial gauge fitted to the container. Another dial gauge is fitted to the top of the pressure pad to measure the change in the thickness of the specimen.

(b) **Test.** A soil specimen of size 60 × 60 × 25 mm is taken. It may be either an undisturbed sample or made from compacted and remoulded soil. The specimen may be prepared directly in the box and compacted. The base plate is attached to the lower half of the box. A porous stone is placed in the box. For undrained tests, a plain grid is kept on the porous stone, keeping its segregations at right angles to the direction of shear. For drained tests, perforated grids are used instead of plain grids. The mass of the base plate, porous stone and grid is taken. The specimen if made separately is transferred to the box and its mass taken.

The upper grid, porous stone and the pressure pad are placed on the specimen. The box is placed inside the large container and mounted on the loading frame. The upper half of the box is brought in contact with the proving ring. The loading yoke is mounted on the steel ball placed on the pressure pad. The dial gauge is fitted to the container to give the shear displacement. The other dial gauge is mounted on the loading yoke to record the vertical movement.

The locking pins are removed and the upper half box is slightly raised with the help of spacing screws. The space between the two halves is adjusted, depending upon the maximum particle size. The space should be such that the top half of the box does not ride on soil grains which come between the edges.

The normal load is applied to give a normal stress of 25 kN/m^2. Shear load is then applied at a constant rate of strain. For undrained tests, the rate is generally between 1.0 mm to 2.00 mm per minute. For

tests, the strain rate depends upon the type of soil. For sandy soils, it may be taken as 0.2 mm/minute whereas for clayey soils, it is generally between 0.005 to 0.02 mm/min. The sample shears along the horizontal plane between the two halves. The readings of the proving-ring and the dial gauges are taken every 30 seconds. The test is continued till the specimen fails. The failure is indicated when the proving ring dial gauge begins to recede after having reached the maximum. For the soils which do not give a peak point, the failure is assumed to have occurred when a shearing strain of 20% is reached. At the end of the test, the specimen is removed from the box and its water content found.

The test is repeated under the normal stress of 50, 100, 200 and 400 kN/m^2. The range of the normal stress should cover the range of loading in the field problem for which the shear parameters are required. The shear stress at any stage during shear is equal to the shear force indicated by the proving ring divided by the area of the specimen. A plot can be made between the shear stress and the shear strain. The shear strain is equal to the shear displacement (ΔH) divided by the length of the specimen (L). The shear stress is obtained from the shear load indicated by the proving ring and the cross-sectional area.

Direct shear tests can be conducted for any one of the three drainage conditions. For *U-U* test, plain grids are used and the sample is sheared rapidly. For *CU* test, perforated grids are used. The sample is consolidated under the normal load and after the completion of consolidation, it is sheared rapidly in about 5—10 minutes. In a *CD* test, the sample is consolidated under the normal load and then sheared slowly so that excess pore water pressure is dissipated. A *CD* test may take a few hours for cohesionless soils. For cohesive soils, it may take 2 to 5 days.

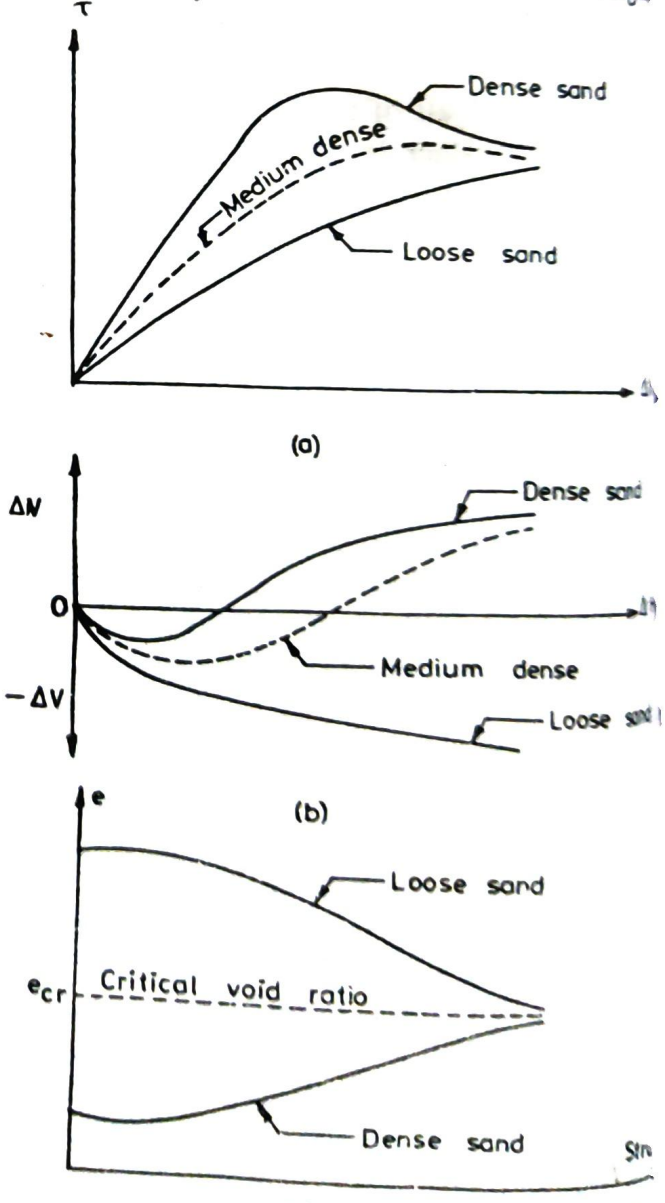
The direct shear test is generally conducted on cohesionless soils as *CD* test. It is convenient to perform and it gives good results for the strength parameters. It is occasionally used to determine the strength parameters of silt and clay under unconsolidated—undrained and consolidated drained conditions, but it does not offer the flexibility of a triaxial compression test, as explained later.

13.12. PRESENTATION OF RESULTS OF DIRECT SHEAR TEST

(*a*) Stress-Strain Curve. A stress-strain curve is a plot between the shear stress τ and the shear displacement ($\Delta H/L$) [Fig. 13.8 (*a*)]. In case of dense sand (and also over-consolidated clays), the shear stress attains a ***peak value*** at a small strain. With further increase in strain, the shear stress decreases slightly and becomes more or less constant, known as ultimate stress. In case of loose sands (and normally consolidated clays), the shear stress increases gradually and finally attains a constant value, known as the *ultimate stress* or residual strength. It has been observed that the ultimate shear stress attained by both dense and loose sands tested under similar conditions is approximately the same. The figure also shows the stress-strain curve of a medium dense sand.

Generally, the failure strain is 2 to 4% for dense sand and 12 to 16% for loose sand.

Fig. 13.8 (*b*) shows the volume changes with an increase in shear strain for *CD* tests. Since the cross-sectional area of the specimen remains unchanged, the volume change is proportional to the change in thickness measured by the dial gauge. In case of dense sands (and over-consolidated clays), the volume first decreases slightly,



(c) Fig. 13.8. Stress-Strain Cu

but it increases with further increase in strain. In the case of loose sands (and normally consolidated clays), the volume decreases with an increase in shear strain. The figure also shows the curve for medium dense sand.

It may be observed that the void ratio of an initial loose sand decreases with an increase in shear strain, whereas that for the initially dense sand increases with an increase in strain [Fig. 13.8 (*c*)]. The void ratio at which there is no change in it with an increase in strain is known as the *critical void ratio*. If the sand initially is at the critical void ratio, there would be practically no change in volume with an increase in shear strain.

(*b*) **Failure Envelope.** For obtaining a failure envelope, a number of identical specimens are tested under different normal stresses. The shear stress required to cause failure is determined for each normal stress. The failure envelope is obtained by plotting the points corresponding to shear strength at different normal stresses and joining them by a straight line [Fig. 13.9 (*a*)]. The inclination of the failure envelope to

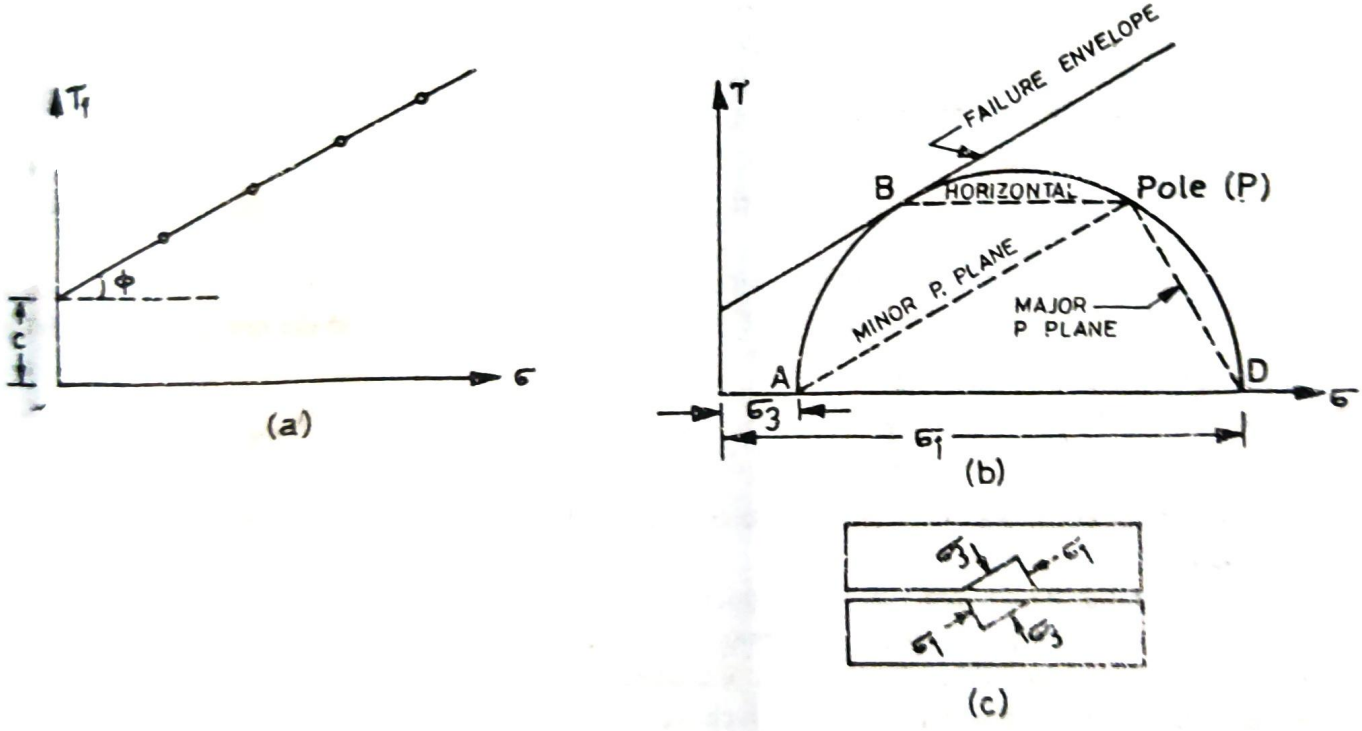


Fig. 13.9. Failure Envelope.

the horizontal gives the angle of shearing resistance ϕ and its intercept on the vertical axis is equal to the cohesion intercept c.

For dense sands, the failure envelope can be drawn either for peak stress or for ultimate stress. The values of the parameters ϕ and c for the two envelopes will be different. For loose sands, the failure envelope is drawn for ultimate stress, which is usually taken as the shear stress at 20% shear strain.

(*c*) **Mohr-Circle.** In a direct shear test, the stresses on planes other than the horizontal plane are not known. It is, therefore, not possible to draw Mohr stress circle at different shear loads. However, the Mohr circle can be drawn at the failure condition assuming that the failure plane is horizontal.

In Fig. 13.9 (*b*), the point *B* represents the failure condition for a particular normal stress. The Mohr circle at failure is drawn such that it is tangential to the failure envelope at *B*. The horizontal line *BP* gives the direction of the failure plane. The point *P* is the pole. The lines *PD* and *PA* give the directions of the major and minor principal planes, respectively. The principal planes are also shown in Fig. 13.9 (*c*).

Merits and Demerits

Merits.

(1) The sample preparation is easy. The test is simple and convenient.

(2) As the thickness of the sample is relatively small, the drainage is quick and the pore pres dissipates very rapidly. Consequently, the consolidated-drained and the consolidated- undrained take relatively small period.

(3) It is ideally suited for conducting drained tests on cohesionless soils.

(4) The apparatus is relatively cheap.

Demerits.

(1) The stress conditions are known only at failure. The conditions prior to failure are indeterminate therefore, the Mohr circle cannot be drawn.

(2) The stress distribution on the failure plane (horizontal plane) is not uniform. The stresses are at the edges and lead to the progressive failure, like tearing of a paper. Consequently, the strength of the soil is not mobilised simultaneously on the entire failure plane.

(3) The area under shear gradually decreases as the test progresses. But the corrected area cannot determined and, therefore, the original area is taken for the computation of stresses.

(4) The orientation of the failure plane is fixed. This plane may not be the weakest plane.

(5) Control on the drainage conditions is very difficult. Consequently, only drained tests can conducted on highly permeable soils.

(6) The measurement of pore water pressure is not possible.

(7) The side walls of the shear box cause lateral restraint on the specimen and do not allow it to defo laterally.

13.13. DIFFERENT TYPES OF SOILS

On the basis of shear strength, soils can be divided into three types.

(1) Cohesionless soils. (2) Purely cohesive soils and

(3) Cohesive-frictional soils.

1. Cohesionless soils. These are the soils which do not have cohesion *i.e.*, $c' = 0$. These soils derive the shear strength from the intergranular friction. These soils are also called *frictional soils*. For example, sands and gravels.

2. Purely cohesive soils. These are the soils which exhibit cohesion but the angle of shearing resistance $\phi = 0$. For example, saturated clays and silts under undrained conditions. These soils are also called $\phi_u = 0$ soils.

3. Cohesive-frictional soils. These are composite soils having both c' and ϕ'. These are also called c—ϕ soils. For example, clayey sand, silty sand, sandy clay, etc.

[**Note.** Sometimes, cohesive-frictional soils are also called cohesive soils. Thus any soil having a value of c' is called a cohesive soil.]

13.14. TRIAXIAL COMPRESSION TEST APPARATUS

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. In this test, a cylindrical specimen is stressed under conditions of axial symmetry, as shown in Fg. 13.10. In the first stage of the test, the specimen is subjected to an all round confining pressure (σ_c) on the sides and at the top and the bottom. This stage is known as the consolidation stage.

In the second stage of the test, called the shearing stage, an additional axial stress, known as the deviator stress (σ_d), is applied on the top of the specimen through a ram. Thus, the total stress in the axial direction at the

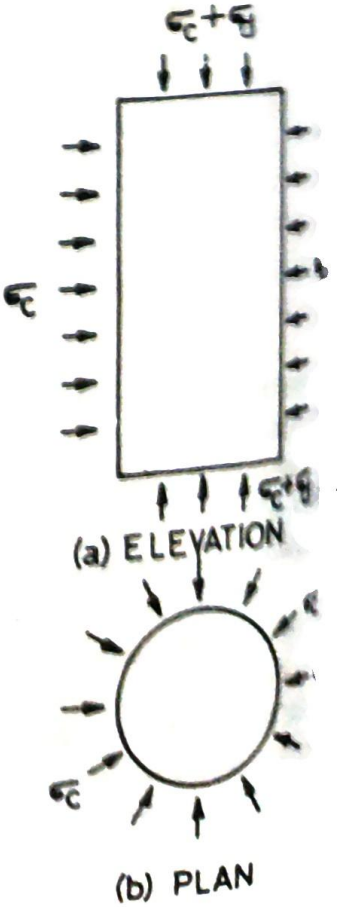


Fig. 13.10.

time of shearing is equal to $(\sigma_c + \sigma_d)$. It may be noted that when the axial stress is increased, the shear stresses develop on inclined planes due to compressive stresses on the top.

The vertical sides of the specimen are principal planes, as there are no shear stresses on the sides. The confining pressure σ_c is equal to the minor principal stress (σ_3). The top and bottom planes are the major principal planes. The total axial stress which is equal to the sum of the confining pressure and the deviator stress, is the major principal stress (σ_1). Because of axial symmetry, the intermediate principal stress (σ_2) is also equal to the confining pressure (σ_c).

[**Note.** The above interpretation of the stress conditions in the triaxial test is not strictly correct according to the theory of elasticity. In the case of cylindrical specimens, the three principal stresses are the axial, radial and the circumferential stresses. The state of stress is statically indeterminate throughout the specimen. For convenience, in the triaxial test, the circumferential stress is taken equal to the radial stress and the principal stresses σ_2 and σ_3 are assumed to be equal].

The main features of a triaxial test apparatus are shown in Fig. 13.11. It consists of a circular base that has a central pedestal. The pedestal has one or two holes which are used for the drainage of the specimen in a drained test or for the pore pressure measurement in an undrained test. A triaxial cell is fitted to the top of the base plate with the help of 3 wing nuts (not shown in the figure) after the specimen has been placed on the pedestal. The triaxial cell is a perspex cylinder which is permanently fixed to the top cap and the bottom brass collar. There are three tie rods which support the cell. The top cap is a bronze casting with its central boss forming a bush through which a stainless steel ram can slide. The ram is so designed that it has minimum of friction and at the same time does not permit any leakage. There is an air-release valve in the top cap which is kept open when the cell is filled with water (or glycerine) for applying the confining pressure. An oil valve is also provided in the top cap to fill light machine oil in the cell to reduce the leakage of water past the ram in long duration tests. The apparatus is mounted on a loading frame. The deviator stress is applied to the specimen from a strain-controlled loading machine. The loading system consists of either a screw jack operated by an electric motor and gear box or a hydraulic ram operated by a pump.

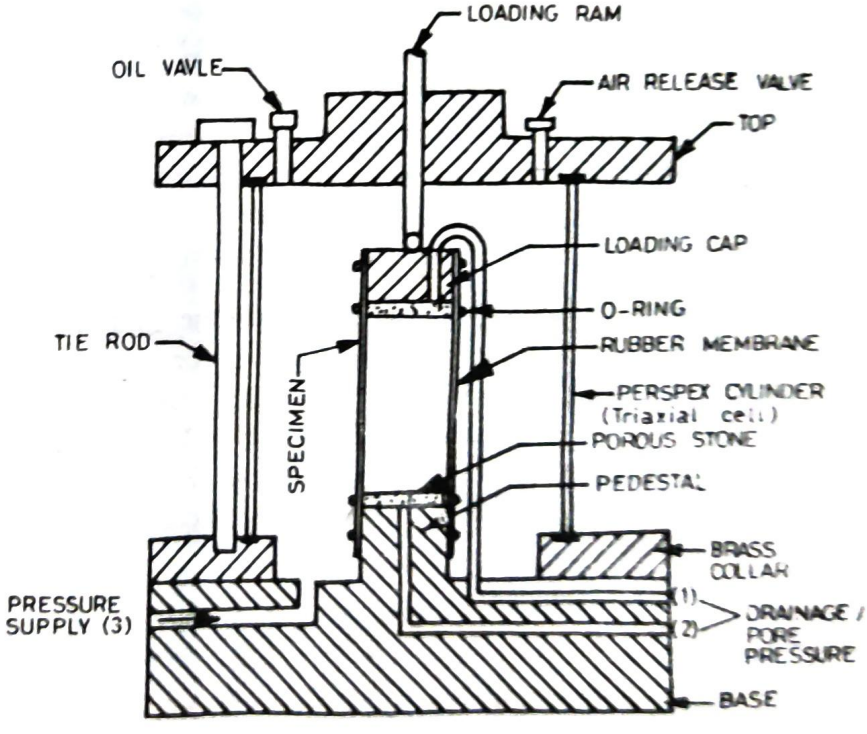


Fig. 13.11. Triaxial Tes Apparatus.

The triaxial test apparatus has the following special attachments.

1. Mercury Control System. The cell pressure is a triaxial test in maintained constant with a self-compensating mercury control system, developed by Bishop and Henkel. It consists of two limbs of a water-mercury manometer (Fig. 13.12). The pressure in the water of the triaxial cell develops due to the difference in levels of the mercury in the two pots. The water pressure at the centre of the specimen in the triaxial cell, at a height of h_3 above the datum, can be calculated using the theory of manometers. As the mercury surface in the upper pot is open to atmosphere, the (gauge) pressure there is zero. From the manometer equation,

$$h_1 \gamma_m - h_2 \gamma_w - (h_3 - h_2) \gamma_w = \sigma_c$$

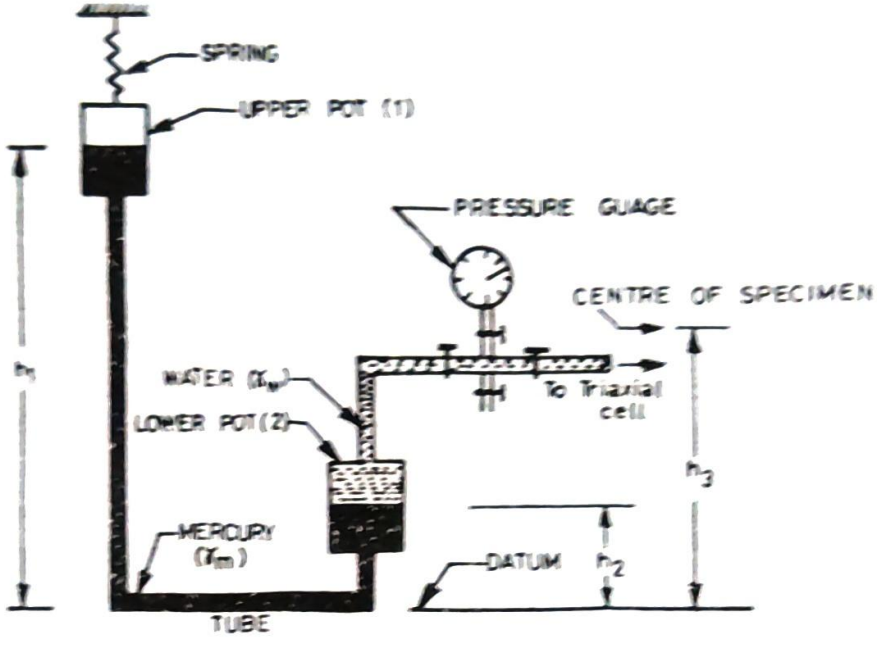


Fig. 13.12. Mercury Control System.

where σ_c is the cell pressure at the centre of the specimen,

γ_w is the unit weight of water, and

γ_m is the unit weight of mercury.

The above equation can be simplified as

$$\sigma_c = \gamma_m (h_1 - h_2) + (h_2 - h_3)\gamma_w \qquad \ldots(13.$$

The upper pot is supported by a spring. When the volume of the specimen decreases due to consolidation or when the water leaks past the ram, water flows from the lower pot to the cell and the mercury level in the lower pot rises by a small amount Δh. The mercury level in the upper pot would also fall by the same amount if the two pots are of the same cross-sectional area. However, the difference of mercury levels in the two pots is maintained constant by the spring. The stiffness (k) of the spring is selected such that it reduces in length and causes a rise of the upper pot as soon as its weight decreases due to flow of mercury. The stiffness of the spring is given by

$$k = A\,\gamma_m \left[\frac{1}{2 - (\gamma_w/\gamma_m)}\right] - W \qquad \ldots(13.$$

where A = cross-sectional area of the mercury pot,

and W = weight of unit length of the tube filled with mercury which is also lifted above the floor.

2. Pore water Pressure Measurement Device. The pore water pressure in the triaxial specimen is measured by attaching it to the device shown in Fig. 13.13. It consists of a null indicator in which no-flow condition is maintained. For accurate measurement, *no- flow condition is essential because* the flow of water from the sample to the gauge would modify the actual magnitude of the pore water pressure. Further, the flow of water leads to a time lag in the attainment of a steady state in samples of cohesive soils because of low permeability.

The null indicator is essentially a U-tube partly filled with mercury. One limb of the null indicator is connected to the specimen in the triaxial cell and the other limb is connected to a pressure gauge. A control cylinder, which is filled with water, is attached to the system. The water can be displaced by a screw controlled piston in the control cylinder. The whole system is filled with deaired water. The tubes connecting the specimen and the null-indicator should be such that these undergo negligible volume changes under pressure and are free from leakage.

Any change in the pore-water pressure in the specimen tends to cause a movement of the mercury level in the null-indicator. However, the no-flow condition is maintained by making a corresponding change in the

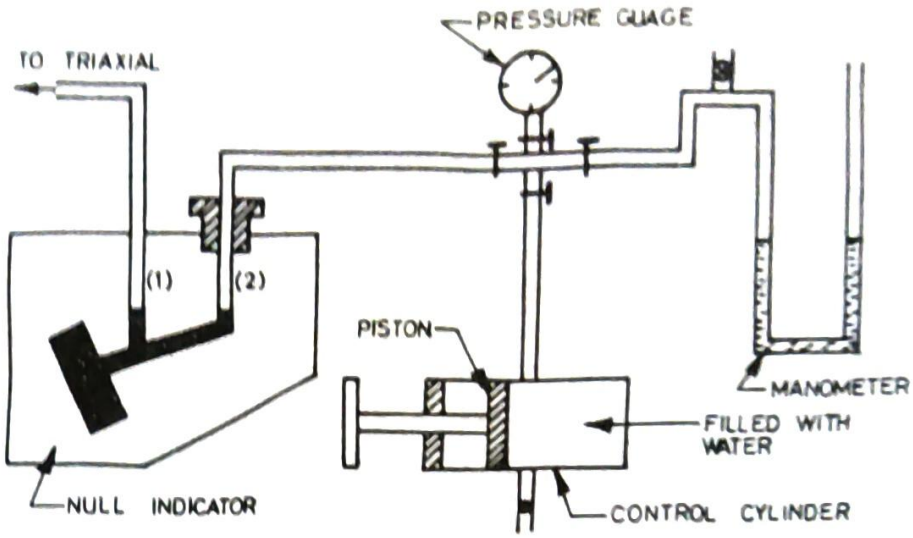


Fig. 13.13. Pore Water Pressure Measurement Device.

other limbs by means of the control cylinder. Thus the mercury levels in the two limbs remain constant. The pressure applied by the control cylinder is recorded by pressure gauge or the manometer.

If the specimen is partially saturated, a special fine, porous ceramic disc is placed below the sample in the triaxial cell. The ceramic disc permits only pore water to flow, provided the difference between the pore air pressure and pore water pressure is below a certain value, known as *the air-entry value* of the ceramic disc. Under undrained conditions, the ceramic disc will remain fully saturated, provided the air-entry value is high. It may be mentioned that if the required ceramic disc is not used and instead the usual coarse, porous disc is used, the device would measure air pressure and not water pressure in a partially saturated soil.

In modern equipment, sometimes the pore water pressure is measured by means of a transducer and not by conventional null indicator.

3. Volume Changes Measurement. Volume changes in a drained test and during consolidation stage of a consolidated undrained test are measured by means of a burette connected to the specimen in the triaxial cell. For accurate measurements, the water level in the burette should be approximately at the level of the centre of the specimen (Fig. 13.14).

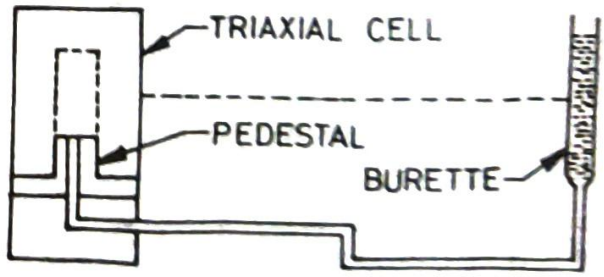


Fig. 13.14. Volume Change Measurement.

During consolidation stage, the volume of the specimen decreases and the water level in the burette rises. The change in the volume of the specimen is equal to the volume of the water increased in the burette. During shearing of specimens of dense sand when the volume of the sample increases, the water flows from the burette to the specimen. The increase in volume of the specimen is equal to the volume of water decreased in the burette.

13.15. TRIAXIAL TESTS ON COHESIVE SOILS

The following procedure is used for conducting the triaxial tests an cohesive soils.

***(a)* Consolidated-undrained test.** A deaired, coarse porous disc or stone is placed on the top of the pedestal in the triaxial test apparatus. A filter paper disc is kept over the porous stone. The specimen of the cohesive soil is then placed over the filter paper disc. The usual size of the specimen is about 37.5 mm diameter and 75.0 mm height. A porous stone is also placed on the top of the specimen. Deaired vertical filter strips are placed at regular spacing around the entire periphery such that these touch both the porous stones. The sample in then enclosed in a rubber membrane, which is slid over the specimen with the help of a membrane stretcher. The membrane is sealed to the specimen with O-rings.

The triaxial cell is placed over the base and fixed to it by tightening the nuts. The cell is then filled with water by connecting it to the pressure supply. Some space in the top portion of the cell is filled by injecting oil through the oil valve. When excess oil begins to spill out through the air-vent valve, both the valves (oil valve and air-vent valve) are closed. Pressure is applied to the water filled in the cell by connecting it to the mercury-pot system. As soon as the pressure acts on the specimen, it starts consolidating. The specimen is connected to the burette through pressure connections for measurement of volume changes. The consolidation is complete when there is no more volume change.

When the consolidation is complete, the specimen is ready for being sheared. The drainage valve is closed. The pore water pressure measurement device is attached to the specimen through the pressure connections. The proving ring dial gauge is set to zero. Using the manual control provided in the loading frame, the ram is pushed into the cell but not allowed to touch the loading cap. The loading machine is then run at the selected speed. The proving ring records the force due to friction and the upward thrust acting on the ram. The machine is stopped, and with the manual control, the ram is pushed further into the cell bringing it in contact with the loading cap. The dial gauge for the measuring axial deformation of the specimen is set to zero.

The sample is sheared by applying the deviator stress by the loading machine. The proving ring readings are generally taken corresponding to axial strains of 1/3%, 2/3%, 1%, 2%, 3%, 4%, 5%, ...until failure or 20% axial strain.

Upon completion of the test, the loading is shut off. Using the manual control, all additional axial stress is removed. The cell pressure is then reduced to zero, and the cell is emptied. The triaxial cell is unscrewed and removed from the base. O-rings are taken out, and the membrane is removed. The specimen is then recovered after removing the loading cap and the top porous stone. The filter paper strips are peeled off. The post- shear mass and length are determined. The water content of the specimen is also found.

(*b*) Unconsolidated Undrained test. The procedure is similar to that for a consolidated-undrained test with one basic difference that the specimen is not allowed to consolidate in the first stage. The drainage valve during the test is kept closed. However, the specimen can be connected to the pore-water pressure measurement device if required.

Shearing of the specimen is started just after the application of the cell pressure. The second stage is exactly the same as in the consolidated-undrained test described above.

(*c*) Consolidated Drained test. The procedure is similar to that for a consolidated-undrained test, with one basic difference that the specimen is sheared slowly in the second stage. After the consolidation of the specimen in the first stage, the drainage valve is not closed. It remains connected to the burette throughout the test. The volume changes during the shearing stage are measured with the help of the burette. As the permeability of cohesive soils is very low, it takes 4-5 days for the consolidated drained test.

13.16. TRIAXIAL TESTS ON COHESIONLESS SOILS

Triaxial tests on specimens of cohesionless soils can be conducted using the procedure as described for cohesive soils. As the samples of cohesionless soils cannot stand of their own, a special procedure is used for preparation of the sample as described below.

A metal former, which is a split mould of about 38.5 mm internal diameter, is used for the preparation of the sample (Fig. 13.15). A coarse porous stone is placed on the top of the pedestal of the triaxial base, and the pressure connection is attached to a burette (not shown). One end of a membrane is sealed to the pedestal by O-rings. The metal former is clamped to the base. The upper metal ring of the former is kept inside the top end of the rubber membrane and is held with the help of a clamp before placing the funnel and the rubber bung in position as shown in figure.

The membrane and the funnel are filled with deaired water. The cohesionless soil which is to be tested is saturated by mixing it with enough water in a beaker. The mixture is boiled to remove the entrapped air. The saturated soil is deposited in the funnel, with a stopper in position, in the required quantity. The glass rod is then removed and the sample builds up by a continuous rapid flow of saturated soil in the former. The

funnel is then removed. The sample may be compacted if required. The surface of the sample is leveled and a porous stone is placed on its top. The loading cap is placed gently on the top porous stone. O-rings are fixed over the top of the rubber membrane.

A small negative pressure is applied to the sample by lowering the burette. The negative pressure gives rigidity to the sample and it can stand without any lateral support. For sample of 37.5 mm diameter, a negative pressure of 20 cm of water (or 2 kN/m^2) is sufficient. As soon as the negative pressure is applied, the consolidation of the sample occurs and it slightly shortens. The diameter of the upper porous stone should be slightly smaller than that of the specimen so that it can go inside when the sample shortens; otherwise, a neck is formed.

The split mould is then removed, and the diameter and the height of the sample are measured. The thickness of the membrane is deducted from the total diameter to get the net diameter of the sample. The cell is then placed over the base and clamped to the base. It is then filled with water.

The rest of the procedure is the same as for cohesive soils.

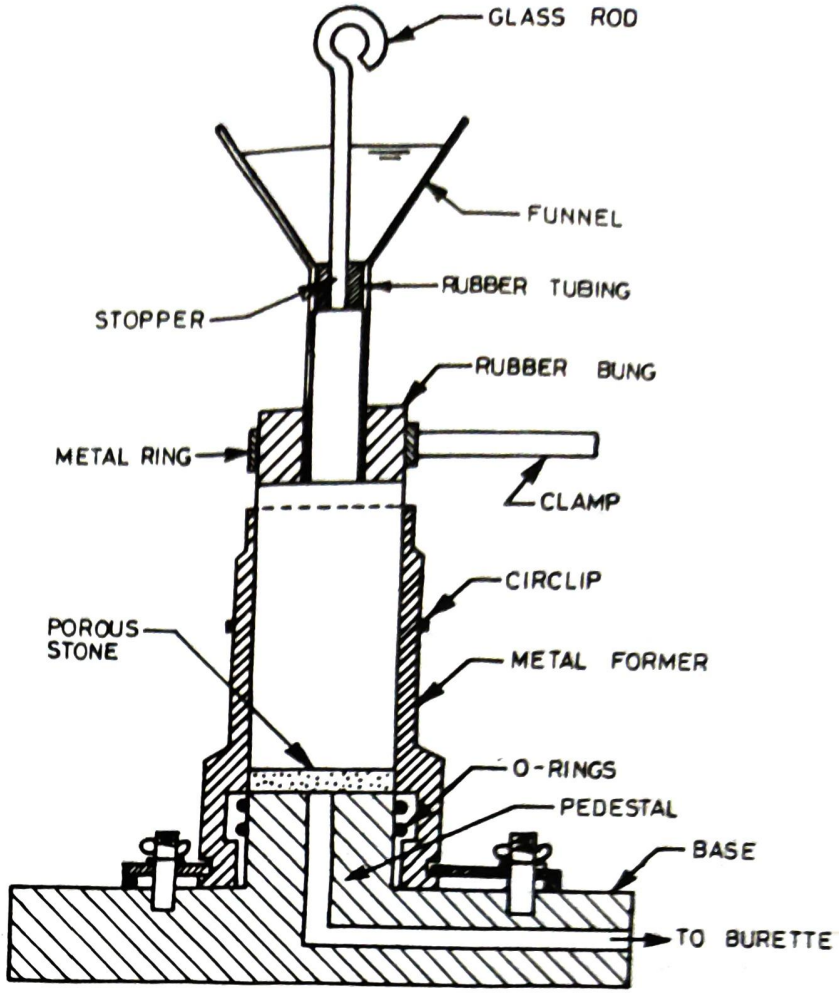


Fig. 13.15. Preparation of Sample of Cohesionless Soil.

13.17. MERITS AND DEMERITS OF TRIAXIAL TEST

The triaxial test has the following merits and demerits.

Merits.

(1) There is complete control over the drainage conditions. Tests can be easily conducted for all three types of drainage conditions.
(2) Pore pressure changes and the volumetric changes can be measured directly.
(3) The stress distribution on the failure plane is uniform.
(4) The specimen is free to fail on the weakest plane.
(5) The state of stress at all intermediate stages upto failure is known. The Mohr circle can be drawn at any stage of shear.
(6) The test is suitable for accurate research work. The apparatus is adaptable to special requirements such as extension test and tests for different stress paths.

Demerits.

(1) The apparatus is elaborate, costly and bulky.
(2) The drained test takes a longer period as compared with that in a direct shear test.
(3) The strain condition in the specimen are not uniform due to frictional restraint produced by the loading cap and the pedestal disc. This leads to the formation of the dead zones at each end of the specimen.

The non-uniform distribution of stresses can be largely eliminated by lubrication of end surfaces. However, non-uniform distribution of stresses has practically no effect on the measured strength if length/diameter ratio is equal to or more than 2.0.

(4) It is not possible to determine the cross-sectional area of the specimen accurately at large strains, the assumption that the specimen remains cylindrical does not hold good.

(5) The test simulates only axis-symmetrical problems. In the field, the problem is generally 3-dimensional. A general test in which all the three stresses are varied would be more useful.

(6) The consolidation of the specimen in the test is isotropic; whereas in the field, the consolidation is generally anisotropic.

Despite the above-mentioned demerits, the triaxial test is extremely useful. It is the only reliable test for accurate determination of the shear characteristics of all types of soils and under all the drainage conditions.

13.18. COMPUTATION OF VARIOUS PARAMETERS

(a) Post-Consolidation Dimensions. In consolidated-drained and consolidated-undrained tests, consolidation of the specimen takes place during the first stage. As the volume of the specimen decreases, post-consolidation dimensions are different from the initial dimensions. The post consolidation dimensions can be determined approximately assuming that the sample remains cylindrical and it behaves isotropically. Let L_i, D_i, and V_i be the length, diameter and the volume of the specimen before consolidation. Let L_0, D_0 and V_0 be the corresponding quantities after consolidation.

Therefore, volumetric change, $\Delta V_i = V_i - V_0$

The volumetric change (ΔV_i) is measured with the help of burette.

Volumetric strain, $$\varepsilon_v = \frac{\Delta V_i}{V_i}$$

For isotropic consolidation, the volumetric strain is three times the linear strain (ε_l). Thus

$$\varepsilon_l = \varepsilon_v / 3$$

Thus $$L_0 = L_i - \Delta L_i = L_i - L_i \times \varepsilon_l$$

or $$L_0 = L_i (1 - \varepsilon_l) = L_i (1 - \varepsilon_v/3) \quad \ldots(13.$$

Likewise, $$D_0 = D_i (1 - \varepsilon_v/3)$$

The post consolidation diameter D_0 can also be computed after L_0 has been determined from the relation

$$(\pi/4 \cdot D_0^2) \times L_0 = V_0$$

or $$D_0 = \sqrt{\frac{V_0}{(\pi/4) \times L_0}} \quad \ldots(13.$$

(b) Cross-sectional Area During Shear Stage. As the sample is sheared, its length decreases and its diameter increases. The cross-sectional area A at any stage during shear can be determined assuming that the sample remains cylindrical in shape. Let ΔL_0 be the change in length and ΔV_0 be the change in volume. The volume of the specimen at any stage is given by $V_0 \pm \Delta V_0$.

Therefore, $$A (L_0 - \Delta L_0) = V_0 \pm \Delta V_0$$

or $$A = \frac{V_0 \pm \Delta V_0}{L_0 - \Delta L_0} = \frac{V_0 \left(1 \pm \dfrac{\Delta V_0}{V_0}\right)}{L_0 \left(1 - \dfrac{\Delta L_0}{L_0}\right)} \quad \ldots(13.$$

Eq. 13.18 is the general equation which gives the cross-sectional area of the specimen.

The above equation can be written as

$$A = \frac{A_0 L_0 \left(1 \pm \frac{\Delta V_0}{V_0}\right)}{L_0 (1 - \varepsilon_1)} = \frac{A_0 \left(1 \pm \frac{\Delta V_0}{V_0}\right)}{(1 - \varepsilon_1)} \quad \ldots(13.19)$$

where ε_1 is the axial strain in the sample.

For an undrained test, as the volumetric change (ΔV_0) is zero, Eq. 13.19 becomes

$$A = \frac{A_0}{1 - \varepsilon_1} \quad \ldots(13.20)$$

The stresses in the specimen at various stages of shear should be calculated using the cross-sectional area A as found above.

The correction so applied is known as *area correction.*

(c) Stresses (*i*) *Deviator stress.* The deviator stress (σ_d) acting on the specimen when the axial load applied by the loading machine is P can be obtained as

$$\sigma_d = \frac{P}{A} \quad \ldots(13.21)$$

The deviator stress (σ_d) is equal ($\sigma_1 - \sigma_3$)

It may be noted that the load indicated by the proving ring is slightly more than P because of friction on the ram and the upward thrust on the ram due to pressure of the water in the cell. The correction can be determined separately.

A more convenient procedure is to lift the ram above the specimen when the cell pressure has been applied. The machine is started keeping the strain rate the same as to be used in the actual test. The proving ring records the load. To account for correction, the dial gauge on the proving ring is set to zero to indicate zero load. This automatically compensates the ram friction and the upward thrust on the ram due to cell pressure. Thus the load indicated by the proving ring during shear would be equal to the load P applied to the specimen.

(*ii*) *Principal stresses.* The minor principal stress (σ_3) is equal to the cell pressure (σ_c). The major principal stress (σ_1) is equal to the sum of the cell pressure and the deviator stress.

Thus $$\sigma_1 = \sigma_3 + (\sigma_1 - \sigma_3)$$

or $$\sigma_1 = \sigma_3 + \sigma_d \quad \ldots(13.22)$$

(*iii*) *Compressive strength.* The deviator stress at failure, $(\sigma_1 - \sigma_3)_f$, is known as the compressive strength of the soil.

13.19. PRESENTATION OF RESULTS OF TRIAXIAL TESTS

(*a*) **Stress—Strain Curves.** *Drained Test.* Fig. 13.16 (*a*) shows the stress—strain curve for a drained test. The y-axis shows the deviator stress ($\sigma_1 - \sigma_3$) and the x-axis, the axial strain (ε_1). For dense sand (and over-consolidated clay), the deviator stress reaches a peak value and then it decreases and becomes almost constant, equal to the ultimate stress, at large strains. For loose sand (and normally consolidated clay), the deviator stress increases gradually till the ultimate stress is reached.

The volumetric strain is shown in Fig. 13.16 (*b*). In dense sand (and over-consolidated clay), there is a decrease in the volume at low strains, but at large strains, there is an increase in the volume. In loose sand (and normally consolidated clay),

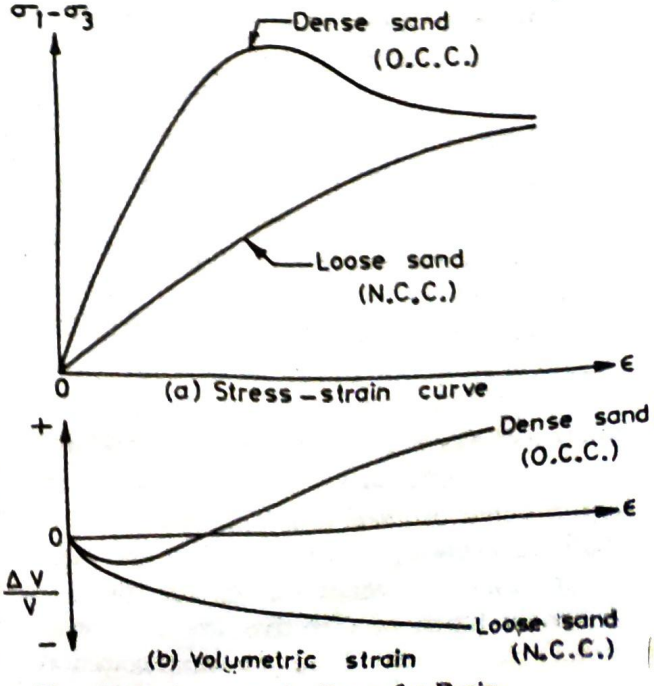


Fig. 13.16. Stress-strain Curve for Drained

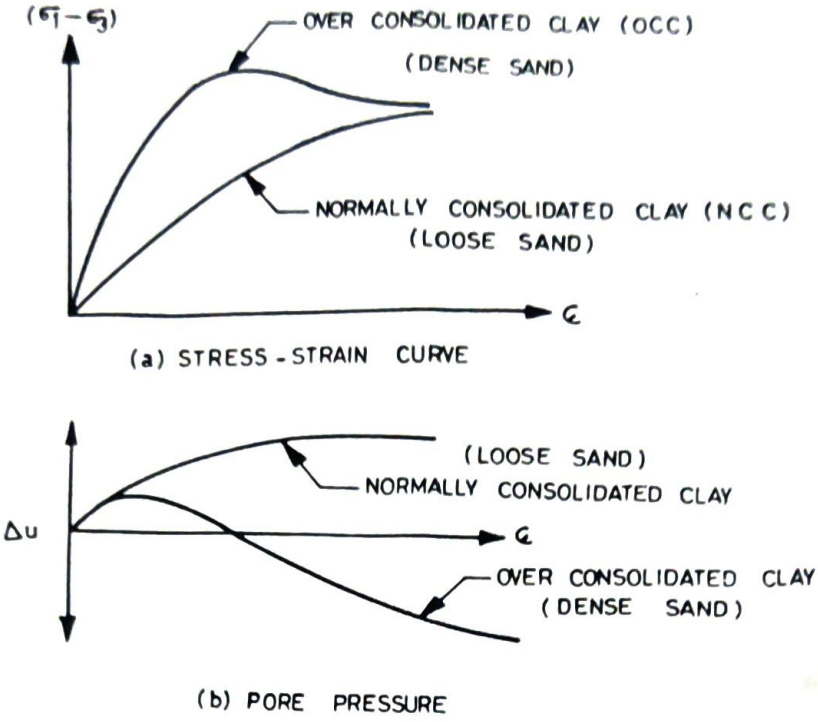


Fig. 13.17. Sterss-Strain Curve, *CU* Test.

volume decreases at all strains. (For some loose sands, there is a slight tendency to increase in the volume large strains).

Consolidated—undrained test. Fig. 13.17 (*a*) shows the stress-strain curve for a consolidated—undrained test. The shape of the curves is similar to that obtained in a consolidated-drained test. In a consolidated-undrained test, there is an increase in the pore water pressure throughout for loose sand (and normally consolidated clay), as shown in Fig. 13.17 (*b*). However, in the case of dense sands (and over- consolidated clay), the pore water pressure increases at low strains but at large strains it becomes negative (below atmospheric pressure).

(*b*) Mohr Envelopes. For drawing the failure envelopes, it is necessary to test at least three samples at three different cell pressures in the stress range of interest. For dense sands and over-consolidated clays, the failure envelope can be drawn either for the peak stress or for the ultimate stress. For loose sands and normally consolidated clays, the failure envelope is drawn for the ultimate stress, which is usually taken at 20% strain. Further, the failure envelope can be drawn either in terms of effective stresses or in terms of total stress. Of course, the two envelopes will give different values of strength parameters (c and φ).

Britlleness I_B is expressed as

$$I_B = \frac{\text{Peak strength} - \text{Ultimate strength}}{\text{Ultimate strength}}$$

(*i*) *Effective Stresses.* Fig. 13.18 (*a*) shows the failure envelope for a normally consolidated clay in terms of effective stresses obtained from a *consolidated drained test.* The failure envelope has an angle of shearing resistance of ϕ and passes through origin. First the Mohr circles for the three tests are drawn in terms of effective stresses corresponding to failure conditions. Then the best common tangent is drawn to the three circles. The common tangent is the

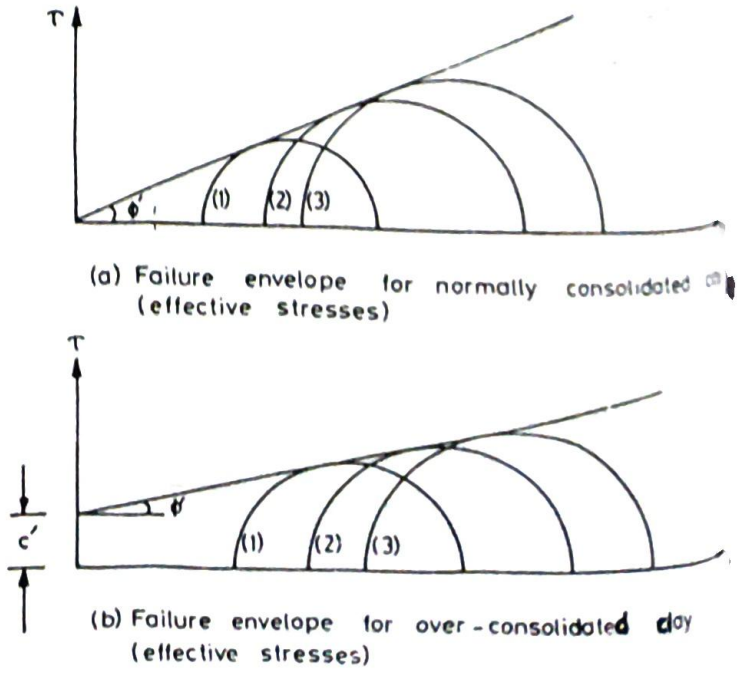


Fig. 13.18. Failure Envelopes (effective stress).

failure envelope. As each circle represents a failure, there must be at least one point on it which gives the stresses satisfying the failure criterion. Obviously, the common tangent joins all such points of the three circles.

Thus for normally consolidated clays, shear strength is

$$s = \bar{\sigma} \tan \varphi'.$$

Fig. 13.18 (*b*) shows the failure envelope for overconsolidated clay in terms of effective stresses. The failure envelope is slightly curved in the initial portion, but, for convenience, it is approximated as a straight line. The failure envelope has an intercept c' on the τ- axis. The angle of shearing resistance is ϕ'. In the case of over-consolidated clays, shear strength is

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

The failure envelopes in terms of effective stresses can also be drawn from the results of a consolidated-undrained test (*CU* test) when the pore water pressure measurements are also taken. The shear strength parameters c' and φ' obtained from the consolidated-undrained tests and that from consolidated drained tests are approximately equal. Drained tests on dense sands and over-consolidated clays give slightly higher values of ϕ' due to extra work required during dilation (increase in volume), but the difference is small, and, therefore, usually neglected.

(*ii*) *Total stresses*. The failure envelope in terms of total stresses can be drawn from the test results of a *consolidated-undrained test.*

[**Note.** In consolidated-drained tests, the total stresses are also equal to the effective stresses, as the pore water pressure throughout is zero].

The failure envelopes are similar in shape to that in terms of effective stresses but the values of the strength parameters are quite different. Fig. 13.19 shows the failure envelopes for effective stresses and also for total stresses for a normally consolidated clay. The angle of shearing resistance in terms of total stresses (ϕ_{cu}) is much smaller than that for the effective stresses (ϕ').

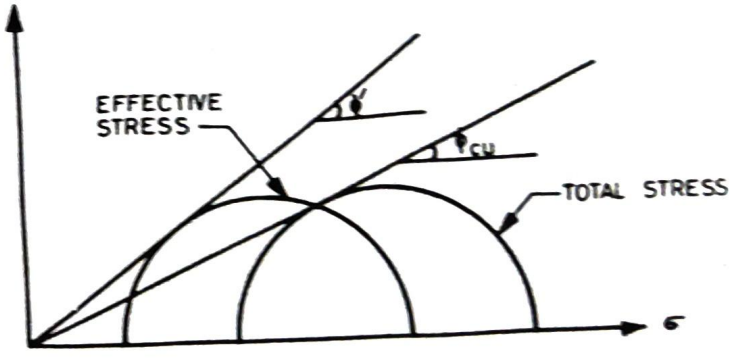


Fig. 13.19.

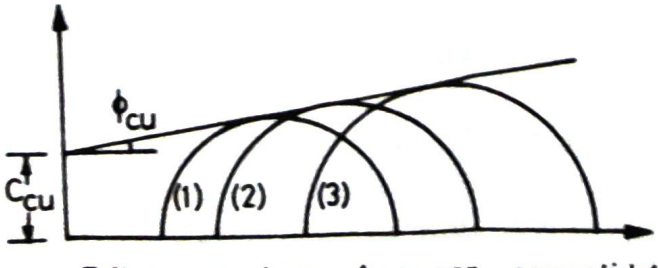
In the case of normally consolidated clays, shear strength is

$$s = \sigma \tan \phi_{cu}$$

Fig. 13.20 shows the failure envelope for an overconsolidated clay in terms of total stresses. The angle of shearing resistance (ϕ_{cu}) is much smaller than the angle ϕ' obtained in terms of effective stresses. In the case of overconsolidated clays, shear strength is

$$s = c_{cu} + \sigma \tan \phi_{cu}$$

The angle of shearing resistance ϕ_{cu} obtained from the total stress envelopes is also known as apparent angle of shearing resistance.



Failure envelope for over-consolidated clay
(Total stresses)

Fig. 13.20. Strength Envelope.

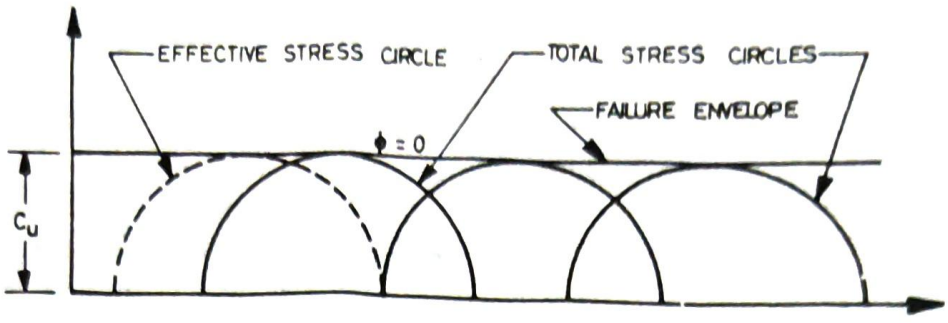


Fig. 13.21.

Fig. 13.21 shows the failure envelope in terms of total stress obtained from an *unconsolidated-undrain test* on a normally consolidated clay. The failure envelope is horizontal ($\phi = 0$), and has a cohesion intercept of c_u. In this case, shear strength is $s = c_u$. The failure envelope for an over-consolidated clay is horizontal, but the value of c_u will be more, depending upon the degree of overconsolidation.

For an unconsolidated-undrained test, the failure envelope cannot be drawn in terms of effective stress. In all the tests conducted at different confining pressures, the effective stress remains the same. This is due to the fact that an increase in confining pressure results in an equal increase in pore water pressure for saturated soil under undrained conditions. Thus only one Mohr circle (shown dotted) in terms of effective stresses is obtained from all the three tests. It may be noted that the deviator stress at failure is the same for all specimens.

13.20 EFFECT OF CONSOLIDATION PRESSURE ON UNDRAINED STRENGTH

As discussed in the preceding section, the shear strength of a cohesive soil under undrained condition depends upon the consolidation pressure. If a remoulded specimen of saturated clay is first consolidated under a confining pressure, say $(\sigma_c)_1$, and then sheared under undrained conditions, with different confining pressures, the deviator stress is independent of the cell pressure. The failure envelope is horizontal and the cohesion intercept is c_{u1} [Fig. 13.22 (*a*)].

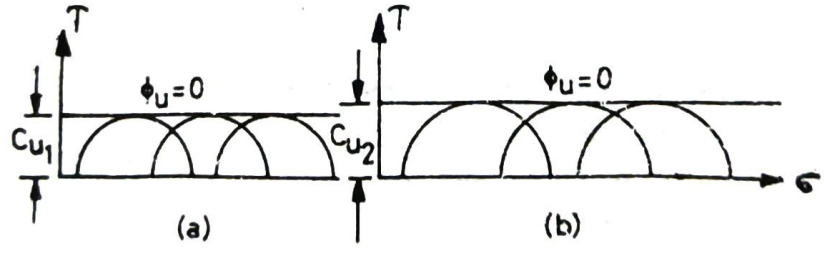


Fig. 13.22. Effect of Consolidation Pressure on C_u

The reader should carefully note the difference between the conventional consolidated-undrained test as discussed earlier and the test described in this section. In the former, each specimen is consolidated under a certain pressure, equal to confining pressure, and then sheared under undrained conditions with the same cell pressure. Thus, only one Mohr circle at failure is obtained for one confining pressure. In the latter, a set of specimens are consolidated under the same confining pressure and then sheared with different cell pressures. Thus, a number of Mohr circles at failure are obtained in which the deviator stress at failure is the same for all specimens.

If another set of identical specimens is consolidated under another pressure, say $(\sigma_c)_2$, and then sheared under different cell pressure, another horizontal failure envelope is obtained with the cohesion intercept as [Fig. 13.22 (*b*)]. Likewise, the test can be repeated after consolidating the set of specimens under another pressure, say $(\sigma_c)_3$. It is observed that the greater the consolidation pressure, the greater the undrained shear strength represented by the cohesion intercept. Fig. 13.23 shows a plot between the consolidation pressure and the undrained strength. The plot is a straight line for normally consolidated clays.

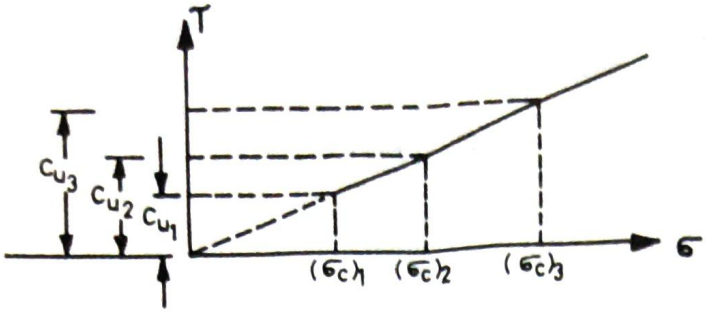


Fig. 13.23. C_u – σ Plot for Normally consolidated Clay.

or over-consolidated clays, there is a discontinuity in the plot between c_u and σ at a pressure equal to reconsolidation pressure ($\bar{\sigma}_c$), as shown in Fig. 13.24.

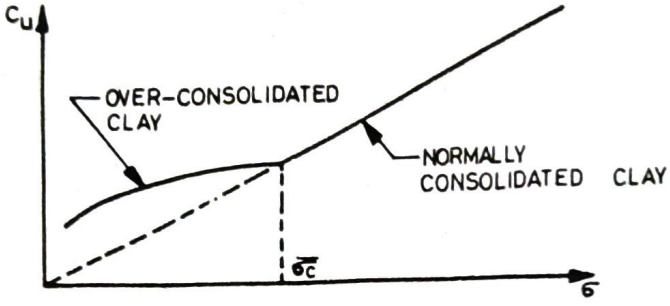


Fig. 13.24. C_u – σ Plot for Over-consolidated Clay

13.21. RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND EFFECTIVE OVERBURDEN PRESSURE

A relationship can be obtained between the undrained strength (c_u) and the effective overburden pressure $\bar{\sigma}_3$ for a normally consolidated natural deposit. Fig. 13.25 shows the Mohr circle in terms of total stresses obtained from a conventional *CU* test conducted at a confining pressure of $\bar{\sigma}_3$ on a normally consolidated clay.

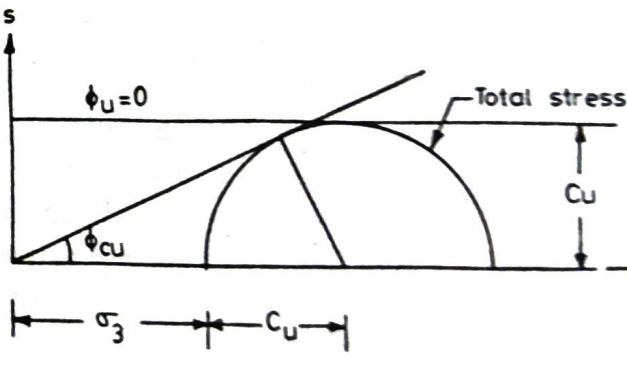


Fig. 13.25. Relationship between C_u, σ_3 and ϕ_{cu}.

By geometry, $$\frac{c_u}{\sigma_3 + c_u} = \sin \phi_{cu}$$

or $$\frac{\sigma_3 + c_u}{c_u} = \frac{1}{\sin \phi_{cu}}$$

or $$\frac{\sigma_3}{c_u} = \frac{1}{\phi_{cu}} - 1 = \frac{1 - \sin \phi_{cu}}{\sin \phi_{cu}}$$

or $$\frac{c_u}{\sigma_3} = \frac{\sin \phi_{cu}}{1 - \sin \phi_{cu}}$$

The ratio c_u / σ_3 is a constant for a given clay. Skempton suggested that a similar constant ratio exists between the undrained shear strength of normally consolidated natural deposits and the effective overburden pressure. It has been established that the ratio ($c_u / \bar{\sigma}$) is constant provided the plasticity index

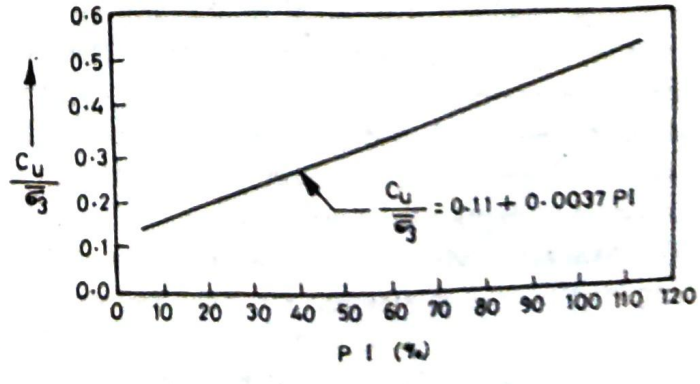


Fig. 13.26. Plot Between $C_u/\bar{\sigma}_3$ and

(*PI*) of the soil remains constant (Fig. 13.26). An approximate value of the undrained shear strength of normally consolidated deposit can be obtained from Fig. 13.26, if the plasticity index has been determined. The relationship is expressed as (Skempton, 1957).

$$\frac{c_u}{\bar{\sigma}} = 0.11 + 0.0037\ PI$$

where c_u = undrained cohesion intercept,
$\bar{\sigma}$ = effective over-burden pressure
PI = plasticity index (%)

The value of the ratio $(c_u / \bar{\sigma})$ determined in a consolidated-undrained test on undisturbed samples is generally greater than actual value because of anisotropic consolidation in the field. The actual value is better determined by in-situ shear vane test, as explained later.

13.22. UNCONFINED COMPRESSION TEST

The unconfined compression test is a special form of a triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soils which can stand without confinement. The test is generally performed on intact (non- fissured), saturated clay specimens. Although the test can be conducted in a triaxial test apparatus as a *U—U* test, it is more convenient to perform it in an unconfined compression testing machine. There are two types of machines, as described below.

(1) Machine with a spring. Fig. 13.27 shows the unconfined compression testing machine in which a loaded spring is used. It consists of two metal cones which are fixed on horizontal loading plates *B* and *C* supported on the vertical posts *D*. The upper loading plate *B* is fixed in position, whereas the lower plate *C* can slide on the vertical posts The soil specimen is placed between the two metal cones.

When the handle is turned, the plate *A* is lifted upward. As the plate *A* is attached to the plate *C*, the latter plate is also lifted. When the handle is turned slowly, at a speed of about half a turn per second, a compressive force acts on the specimen. Eventually, the specimen fails in shear. The compressive load is proportional to the extension of the spring.

The strain in the specimen is indicated on a chart fixed to the machine. As the lower plate *C* moves upward, the pen attached to this plate swings sideways. The lateral movement of the pen (in arc) is proportional to the strain in the specimen.

The chart plate is attached to the yoke *Y*. As the yoke moves upward when the handle is rotated, the chart plate moves upward. The pivot of the arm of the pen also moves upward with the lower plate. The vertical movement of the pen relative to the chart is equal to the extension of the spring and hence the compressive force. Thus the chart gives a plot between the deformation and the compressive force. Springs of different stiffnesses can be used depending upon the expected compressive strength of the specimen.

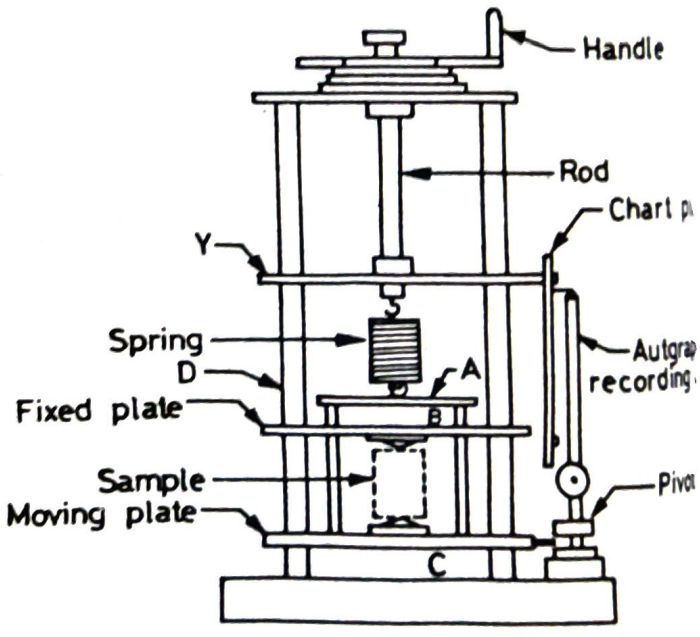


Fig. 13.27. Unconfined Compression Testing Machine (Spring Type).

(2) Machine with a Proving Ring. In this type of the unconfined compression testing machine, a proving ring is used to measure the compressive force (Fig. 13.28). There are two plates, having cone seatings for the specimen. The specimen is placed on the bottom plate so that it makes contact with the upper plate. The dial gauge and proving ring are set to zero.

The compressive load is applied to the specimen by turning the handle. As the handle is turned, the upper

plate moves downward and causes compression. (In some machines, the upper plate is fixed and the compressive load is applied by raising the lower plate). The handle is turned gradually so as to produce an axial strain of 1/2% to 2% per minute. The shearing is continued till the specimen fails or till 20% of the axial strain occurs, whichever is earlier.

The compressive force is determined from the proving ring reading, and the axial strain is found from the dial gauge reading.

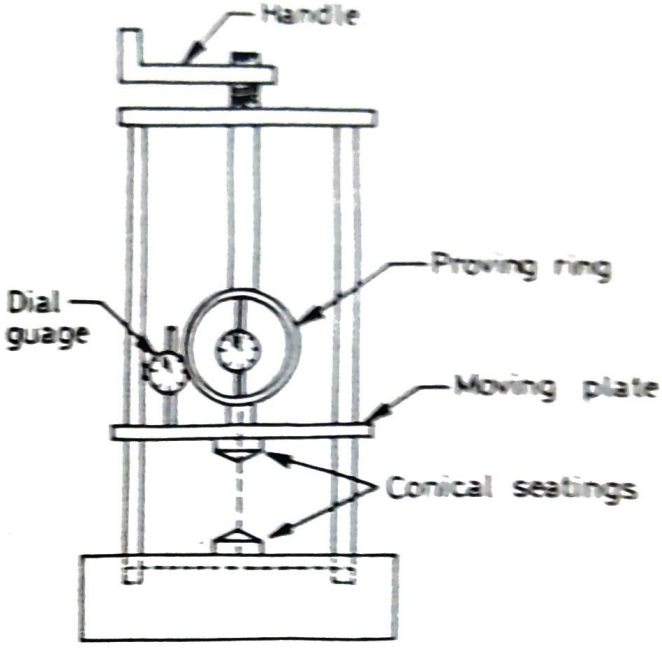


Fig. 13.28. Unconfined Compression Testing Machine (Proving Ring Type).

Presentation of Results. In an unconfined compression test, the minor principal stress (σ_3) is zero. The major principal stress (σ_1) is equal to the deviator stress, and is found from Eq. 13.21.

$$\sigma_1 = P/A$$

where P = axial load,
and A = area of cross-section.

The axial stress at which the specimen fails is known as the unconfined compressive strength (q_u). The stress- strain curve can be plotted between the axial stress and the axial strain at different stages before failure.

While calculating the axial stress, the area of cross-section of the specimen at that axial strain should be used. The corrected area can be obtained from Eq. 13.20 as

$$A = A_0/(1 - \varepsilon)$$

The Mohr circle can be drawn for stress conditions at failure. As the minor principal stress is zero, the Mohr circle passes through the origin (Fig. 13.29). The failure envelope is horizontal ($\phi_u = 0$). The cohesion intercept is equal to the radius of the circle, *i.e.*

$$s = c_u = \frac{\sigma_1}{2} = \frac{q_u}{2} \qquad \ldots(13.25)$$

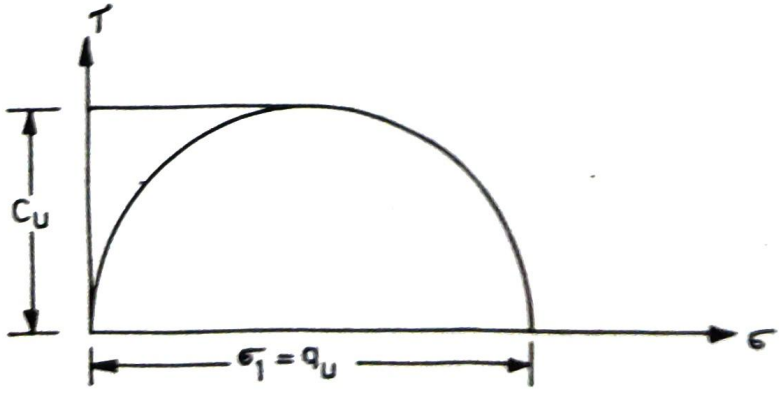


Fig. 13.29. Mohr Circle for Unconfined Compression Test.

Merits and Demerits of the test

Merits

(1) The test is convenient, simple and quick.
(2) It is ideally suited for measuring the unconsolidated-undrained shear strength of intact, saturated clays.
(3) The sensitivity of the soil may be easily determined by conducting the test on an undisturbed sample and then on the remoulded sample.

Demerits

(1) The test cannot be conducted on fissured clays.

(2) The test may be misleading for soils for which the angle of shearing resistance is not zero. For such soils, the shear strength is not equal to half the compressive strength.

(See Chapter 30, Sect. 30.17 for the laboratory experiment).

13.23. VANE SHEAR TEST

The undrained shear strength of soft clays 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.

The apparatus consists of a vertical steel rod having four thin stainless steel blades (vanes) fixed at its bottom end. IS : 2720—XXX—1980 recommends that the height H of the vane should be equal to twice the overall diameter D. The diameter and the length of the rod are recommended as 2.5 mm and 60 mm respectively. Fig. 13.30 (*a*) shows a vane shear test apparatus.

For conducting the test in the laboratory, a specimen of the size 38 mm diameter and 75 mm height is taken in a container which is fixed securely to the base. The vane is gradually lowered into the specimen till the top of the vane is at a depth of 10 to 20 mm below the top of the specimen. The readings of the strain indicator and torque indicator are taken.

Torque is applied gradually to the upper end of the rod at the rate of about 6° per minute (*i.e.* 0.1° per second). The torque acting on the specimen is indicated by a pointer fixed to the spring. The torque is continued till the soil fails in shear. The shear strength of the soil is determined using the formula derived below.

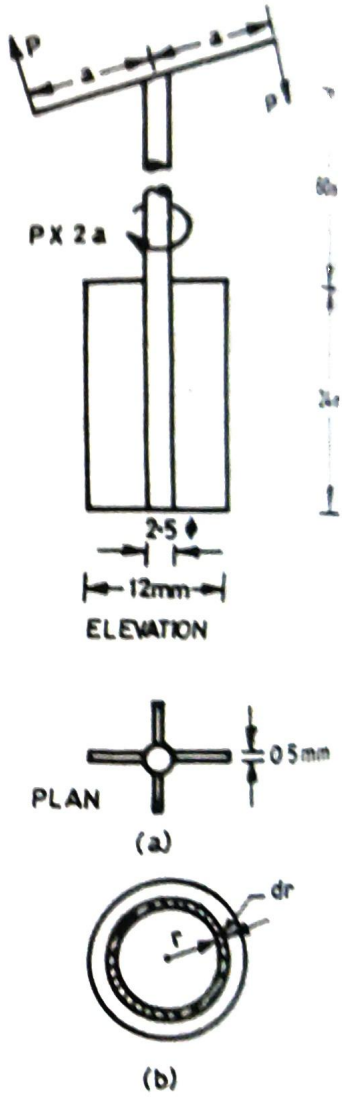


Fig. 13.30. Vane Shear Test.

Derivation of Formula. In the deviation of the formula, it is assumed that the shear strength (s) of the soil is constant on the cylindrical sheared surface and at the top and bottom faces of the sheared cylinder. The torque applied (T) must be equal to the sum of the resisting torque at the sides (T_1) and that at the top and bottom (T_2). Thus,

$$T = T_1 + T_2 \qquad \ldots(a)$$

The resisting torque on the sides is equal to the resisting force developed on the cylindrical surface multiplied by the radial distance. Thus,

$$T_1 = (s\pi DH) \times D/2 \qquad \ldots(b)$$

The resisting torque T_2 due to the resisting forces at the top and bottom of the sheared cylinder can be determined by the integration of the torque developed on a circular ring of radius r and width dr [Fig. 13.30 (*b*)]. Thus.

$$T_2 = 2\int_0^{D/2} [s\,(2\pi r)\,dr]\,r = 4\pi s \left[\frac{r^3}{3}\right]_0^{D/2}$$

or

$$T_2 = \pi s \frac{D^3}{6}$$

From Eqs. (*a*), (*b*) and (*c*), $T = \pi s\,[D^2 H/2 + D^3/6]$

$$s = \frac{T}{\pi (D^2 H/2 + D^3/6)} \quad \text{...(13.27)}$$

or

For example, if $D = 1.2$ cm, and $H = 2.4$ cm, $s = 0.158\ T$

where T is in N-cm and s in N/cm^2.

Eq. 13.27 is modified if the top of the vane is above the soil surface and the depth of the vane inside the sample is H_1. In such a case.

$$s = \frac{T}{\pi (D^2 H_1/2 + D^3/12)} \quad \text{...(13.28)}$$

The shear strength of the soil under undrained conditions is equal to the apparent cohesion c_u.

The vane shear test can be used to determine the sensitivity of the 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 and the shear strength in remoulded state is determined. Thus,

$$\text{Sensitivity } (S_t) = \frac{(s) \text{ undisturbed}}{(s) \text{ remoulded}}$$

Merits and Demerits of Shear Vane Test

Merits.

(1) The test is simple and quick.

(2) It is ideally suited for the determination of the in-situ undrained shear strength of non-fissured, fully saturated clay.

(3) The test can be conveniently used to determine the sensitivity of the soil.

Demerits.

(1) The test cannot be conducted on the fissured clay or the clay containing sand or silt laminations.

(2) The test does not give accurate results when the failure envelope is not horizontal.

13.24. PORE PRESSURE PARAMETERS

A knowledge of the pore water pressure is essential for the determination of effective stresses from the total stresses. The pore water pressure is usually measured in the field by installing piezometers. However, in some cases, it becomes difficult and impractical to install the piezometers and measure the pore water pressure directly in the field. For such cases, a theoretical method for the determination of the pore water pressure is useful. Skempton gave the pore pressure parameters which express the response of pore pressure due to changes in the total stresses under undrained conditions. These parameters are used to predict pore water pressure in the field under similar conditions. The expressions for pore pressure parameters are derived separately for isotropic consolidation, for deviatoric stress and for the combined effect.

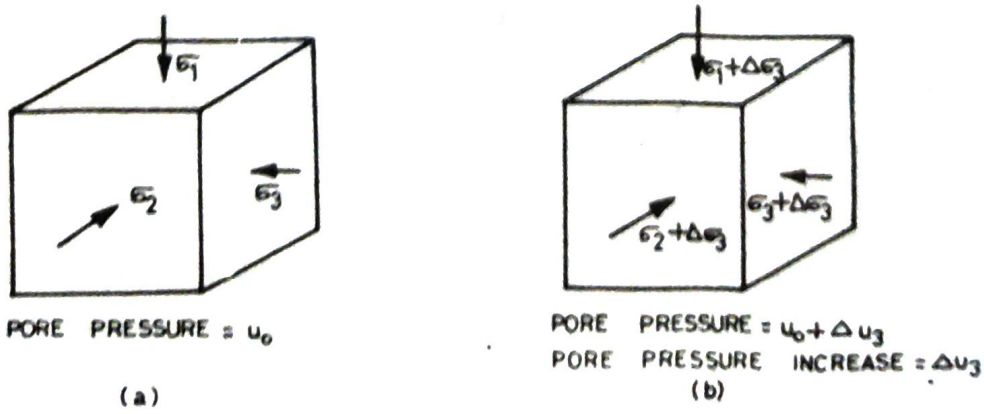


Fig. 13.31. Pore Pressure Under Isotropic Consolidation.