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-: HAND WRITTEN NOTES:-

OF

CIVIL ENGINEERING

4

-: SUBJECT:-

SOIL MECHANICS

6

SOIL MECHANICS

GATE

Obj: 6-7 Ques
= 10-11 Marks

(3)

IES

Obj - 20-25 Ques
Conw - 50-60 Marks

Ref. Books

1. Soil Mech. & foundation Engg By Gopal Ranjan & Rao
(Sometimes direct Ques in IES)
2. Soil Mech. by VNS Murthy
3. Soil Mech. by K.R. Arora
4. Soil Mech. by B.C. Punamia
5. Soil Mech. by S.K. Gang
6. Geotech Engg by Venkat Ramaiyah

For Advance reading

1. Soil Mech. by Muni Budhu
2. Soil Mech. by G. Goudon

Introduction

Soil is an unconsolidated material & comprises of solids, air, water or both or all & is derived from disintegration of rock.

The process of Soil formation is called pedogenesis & is a cyclic process called Geological cycle

Transportation

Geological Cycle

Deposition

WWW.GATENOTES.IN

Rock formation or
upheaval

The Scientific Study of Soil Mechanics was first started by Kari-terzaghi hence he is known as father of Soil Mechanics. He wrote first book on Soil known as Earthsoil Mechanics.

Classification Of Soil

Almost all the soils are derived from weathering of rocks. The weathered rock material is called bediment. Weathering may be due to physical or chemical agencies. If weathered material is transported & deposited at other place then it is called transported soil & If weathered material remains over parent rocks then soil is called residual soil.

(Alluvial soil)

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1. Alluvial Soils (found in Indo-ganga-Brahmaputra plains & River plains). These are deposited by river water. These may contain gravel, sand, silt & clays.
2. Colluvial Soil (Talus) - These are found in mountain valley & are formed by gravity forces. Due to temp. & moisture variation creeping of soil & land sliding may occur in sloping terrain & such bediments get deposit in lower part of mountain valleys.

me. talus

3. Lacustrine Soil - These are deposited by still water such as lakes. These are uniformly graded & consist mostly clays & may be silt.

4. Marine Soil - These are mostly ^(silt) silty. These are sand, silt or clays deposited by sea water. These have low shear strength & high compressibility.

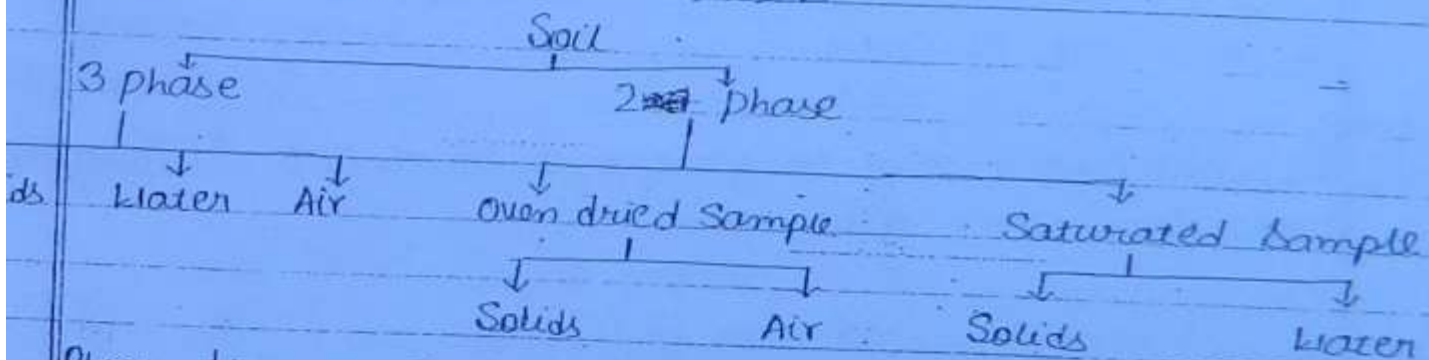
5. Glacial till / glacial drift - It is formed by glaciers & ice-berg. These contain mixture of gravel sand & silt & clays. $\therefore$ these soils are well graded. (non uniform)

Eolian soil - These are formed by wind. eg- sand dunes in desert region. These are loose & poorly graded (uniform).

Loess soil - It is also wind blown soil, uniformly graded containing fine particles. eg- China clay.

8. Caliche - It is a cemented soil & is rich in Calcium Carbonate consisting Gravels sands & clays.
9. Loam - It is mixture of sand, silt & clay & may consist organic content also.
10. Cumulose soils - These are peaty also called muck. These are formed due to accumulation of organic content such as vegetation matter.
11. Gumbo - These are highly sticky, plastic & dark coloured clay.
12. Marl - These are fine grained calcium carbonated soils of marine origin.
13. Tuff - These are small grain slightly cemented volcanic ash.
14. Bentonite - It is a volcanic ash having high compressibility & plasticity. It is often used in tuberculation in drilling.

→ PROPERTIES OF SOILS



Block diagram of 3 phase



$$d_s = \text{water} + \text{air}$$

$$V_{\text{voids}} = V = V_v + V_s$$

$$= V_a + V_w + V_s$$

(Moist Soil)

2 phase

Dry



(Dry Soil)

$$(D_s = 0)$$

Saturated



$$(D_s = 1)$$

NOTE - The dry soil is oven dried (at temp 105°C to 110°C). In air dried sample hygroscopic water may be present.

oil
moisture

→ Gravity water	} Removed on oven drying at controlled temp (105°C to 110°C)
→ Capillary water	
→ Hygroscopic water	
→ Chemically bounded water (Crystalline water)	

If temp is very high (more than 500°C) then Crystalline water also may be lost & hence Crystalline structure of soil may be destroyed.

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Definitions

① Water Content / Moisture Content (w)

$$w = \frac{\text{wt. of water}}{\text{wt. of solids}} = \frac{\text{Mass of water}}{\text{Mass of solids}}$$

$$\% w = \frac{\text{wt. of water}}{\text{wt. of solids}} \times 100$$

So water content may be greater or equal to 0. It may be more than 100% also.

② Degree of Saturation - (S)

$$S = \frac{\text{Volume of water}}{\text{Volume of voids}} = \frac{V_w}{V_v}$$

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

$$0 \leq S \leq 100\%$$

It cannot be greater than 100%.

For oven dried soil $S = 0\%$.

For fully saturated soil $S = 100\%$.

NOTE - If all the air is replaced by water the soil is just saturated. If water is further added then soil will be Super saturated.

(c) Saturated unit weight

$$\gamma_{sat} = \frac{\text{Saturated wt. of soil}}{\text{Total Vol. of soil Mass}} = \frac{M_{sat}}{V}$$

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

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$$\rightarrow \gamma_d < \gamma < \gamma_{sat}$$

(d) Submerged unit weight / Effective unit weight

$$\gamma_{sub} \text{ or } \gamma' = \gamma_{sat} - \gamma_w$$

$$\rho_{sub} \text{ or } \rho' = \rho_{sat} - \rho_w$$

$$\gamma_w = \text{unit wt. of water} = 9.81 \text{ kN/m}^3 = 9810 \text{ N/m}^3$$

$$\rho = 1 \text{ gm/cc}$$

Specific Gravity

(a) True specific gravity or absolute specific gravity or specific gravity

(G_s)

$$G_s = \frac{\text{Unit wt. of Solids}}{\text{Unit wt. of water}} = \frac{\gamma_s}{\gamma_w}$$

$$G_s = 2.6 \text{ to } 2.7 \text{ for inorganic Solids}$$

$$G_s = 1.22 - 1.4 \text{ for organic Solids}$$

(b) Apparent specific gravity / Mass specific gravity

$$G_m = \frac{\text{Bulk unit wt. of soil}}{\text{Unit wt. of water}} = \frac{\gamma}{\gamma_w}$$

If soil is saturated then $G_m = \frac{\gamma_{sat}}{\gamma_w}$ (Mass specific gravity of saturated soil)

$$G_{rm} = \frac{\gamma_d}{\gamma_w} \quad (\text{Mass Specific gravity of dry soil})$$

$$\rightarrow G_{rm} < G_r$$

(Q)

NOTE: Specific gravity depends on temperature. In India it is represented as 27°C . If test temp is different than standard temp then it can be modified.

$$G_{27^\circ\text{C}} = G_{T^\circ\text{C}} \times \frac{\gamma_w \text{ at } T^\circ\text{C}}{\gamma_w \text{ at } 27^\circ\text{C}}$$

$$\frac{G_{T_1^\circ}}{G_{T_2^\circ}} = \frac{\gamma_{w1^\circ}}{\gamma_{w2^\circ}}$$

$$G_{T_1^\circ} \cdot \gamma_{wT_1^\circ} = G_{T_2^\circ} \cdot \gamma_{wT_2^\circ}$$

Relative Density / Density Index / Degree of density (I_d)

It quantifies the degree of packing between the loosest and densest possible states of coarse grain soils. It is determined experimentally.

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

$e_{\max}$ = max. void ratio in loosest state of coarse grain soil

$e_{\min}$ = min. void ratio in densest state

e = Actual void ratio in the field condition

$$\rightarrow 0\% \leq I_d \leq 100\%$$

$\rightarrow$ There is inverse ratio of void ratio & unit weight.

$$\% I_d = \frac{\left(\frac{1}{\gamma_{\min}} - \frac{1}{\gamma} \right)}{\left(\frac{1}{\gamma_{\min}} - \frac{1}{\gamma_{\max}} \right)} \times 100$$

⑤ Air Content (a_c)

$$a_c = \frac{\text{Volume of air}}{\text{Volume of voids}} = \frac{V_a}{V_v}$$

$$\% a_c = \frac{V_a}{V_v} \times 100$$

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$$\rightarrow 0 \leq a_c \leq 1$$

⑥ Percentage air void (n_a)

$$n_a = \frac{\text{Volume of air}}{\text{Total Volume of Soil Mass}} = \frac{V_a}{V}$$

$$\% n_a = \frac{V_a}{V} \times 100$$

NOTE-

$$a_c + S = 1$$

$$\frac{V_a}{V_v} + \frac{V_w}{V_v} = 1 \Rightarrow \frac{V_a}{V_v} = 1 - \frac{V_w}{V_v}$$

$$\therefore a_c = 1 - S$$

$$\% a_c = 100 - \% S$$

⑦ Unit weight / density -

(a) Bulk unit weight / Bulk density (γ or γ_t)

$$\gamma = \frac{\text{Total wt. of Soil Mass}}{\text{Total Vol. of Soil Mass}} = \frac{W}{V}$$

It is KN/m^3

$$\text{Bulk density } \rho_o \text{ or } \rho_t = \frac{\text{Total Mass of Soil}}{\text{Total Vol. of Soil}} = \frac{M}{V}$$

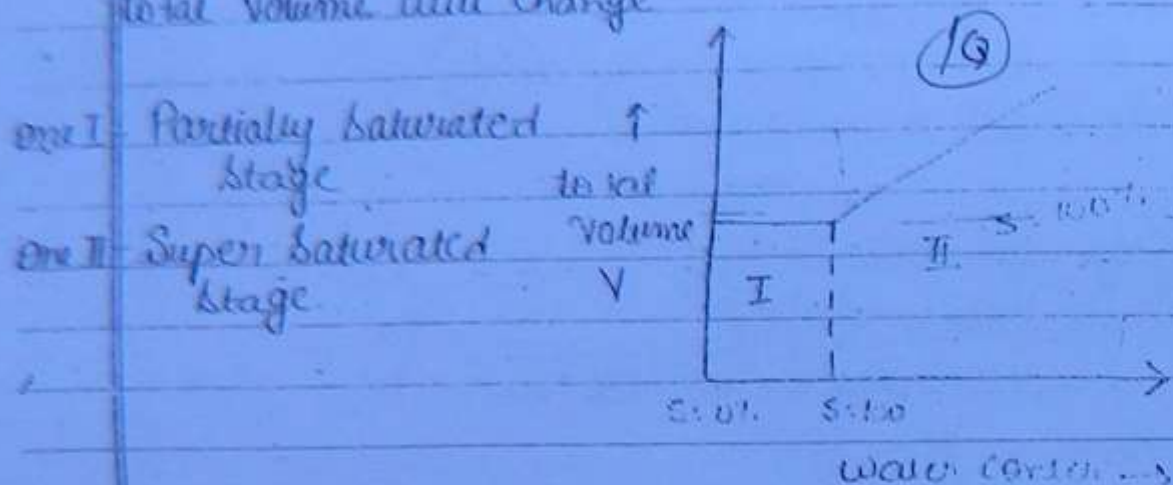
It is in gm/cc or kg/m^3 or t/m^3

(b) Dry unit weight

$$\gamma_d = \frac{\text{Dry wt. of Soil mass}}{\text{Volume of Soil}} = \frac{W_d}{V}$$

$$\rho_d = \frac{\text{Dry Mass of Soil}}{\text{Volume of Soil}} = \frac{M_d}{V}$$

In partially saturated soil (3 phase) S is betw. 0 to 100%. In partially saturated soil due to moisture variation there will be no change in total volume. But in Super saturated soils total volume will change.



③ Void ratio (e)

$$e = \frac{\text{Volume of voids}}{\text{Volume of Solids}} = \frac{V_v}{V_s}$$

Normally represented in fraction.

→ $e > 0$ & e may be > 1 also

→ e never will be equal to zero because soil is not in single phase

④ Porosity (n)

$$n = \frac{\text{Volume of voids}}{\text{Total Volume of Solids + Voids}} = \frac{V_v}{V}$$

$$V = V_v + V_s$$

→ $0 < n < 1$

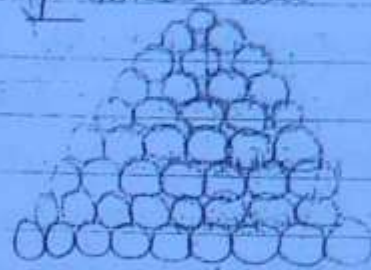
→ Significance of both porosity & void ratio is same

NOTE: Though significance of n & e is same but e is more often use than n bcoz volume of solids V_s is used in e is more stable parameter than total volume V e is used in porosity.

I_d	Description
0 to 15%	Very loose soil
15 to 30%	Loose soil
30 to 65%	Medium soil
65 to 85%	Dense soil
85 to 100%	Very dense soil

(//)

ITE-1



Loosest State

Densest State

Critical arrangement

Prismoidal arrangement

 $e_{max} = 91\%$ $e_{min} = 35\%$ $n_{max} = 71.6\%$ $n_{min} = 25.92\%$

Inter Relationship between physical properties

$$(1) S.e = W.G$$

$$(2) \gamma = \frac{\text{Bulk unit wt}}{1+e} = \frac{(G+Se)\gamma_w}{1+e} \quad G_m = G_s(1-n)$$

$$(3) \gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} \quad G_m = \frac{G_s}{1+e}$$

$$(4) \gamma_d = \frac{G\gamma_w}{1+e} = \frac{\gamma}{1+w}$$

$$(5) \gamma' = \gamma_{sat} - \gamma_w = \left(\frac{G-1}{1+e} \right) \gamma_w$$

$$(6) \gamma_d = \left(\frac{1-n_a}{1+wG} \right) G\gamma_w$$

$$(7) n = \frac{e}{1+e}$$

$$(8) e = \frac{n}{1-n}$$

$$9 \quad 1+e = \frac{1}{1-n}$$

$$10 \quad S = \frac{w}{\frac{\gamma_w}{\gamma}(1+w) - 1} \quad (12)$$

Index properties of Soil (Classification & Soil)

Those properties which are used for identification & Classification of Soil are called Index properties such as Water Content, Consistency limits, In-Situ density, particle size distribution, Specific gravity, Sensitivity, Activity

Determination of water Content :- [OPESAT]

- Oven drying Method
 - Pycnometer Method
 - Calcium Carbide / Rapid Moisture Tester Method
 - Sand bath Method
 - Alcohol Method
 - Torsion Balance Method / Infra-red Method
- (Laboratory Methods)

1. Oven drying Method - ① For inorganic soils temp is maintained bet^m 105°C to 110°C. ② For organic soils temp is maintained about 60°C but for longer time & if soil contains gypsum then temp maintained is 80°C. ③ 4 to 6 hours time is required for drying of sands & 16 to 20 hrs. for clays however 24 hours is usual provision. ④ If temp. is uncontrolled & exceeds 110°C then there is danger of loss of Molecular Structural water / Chemically bounded water. due to c. Soil Structure may be destroyed. ⑤ On oven drying at Controlled temp. gravity water, Capillary water & hygroscopic water is removed.

Let W_1 = wt. of empty container

W_2 = wt. of container + moist soil sample

W_3 = wt. of container + dry soil sample

$W_2 - W_3$ = wt. of water

$W_3 - W_1$ = wt. of dry soil = wt. of solids (13)

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

$$\% W = \frac{W_2 - W_3}{W_3 - W_1} \times 100 \quad \checkmark$$

NOTE - This method is quite accurate but time taken & widely adopted. It is not suitable for soil containing organic compound because of danger of oxidation of organic content.

sample volume) 500 gm soil sample

2. Pycnometer Method - This method is less accurate than previous method & it requires only 10-20 min. This method is suitable for coarse grained soils & if it is to be used for fine grained soils then kerosene should be used instead of water because kerosene is better wetting agent.

A small wt. of moist soil (200-400 gms) is taken in a clean pycnometer.

Let W_1 = wt. of empty pycnometer

W_2 = wt. of pycnometer + wt. of soil sample

W_3 = wt. of (pycnometer + soil sample + water) added to fill the pycnometer

W_4 = wt. of pycnometer filled with water fully

G_s = Specific gravity of soil solids

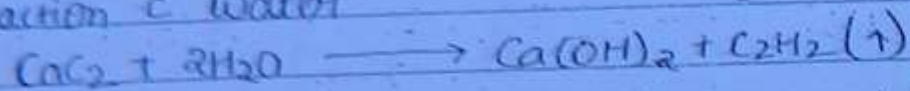
$$\% \text{ Water Content} = \left[\frac{W_2 - W_1}{W_3 - W_4} \left(\frac{G_s - 1}{G_s} \right) - 1 \right] \times 100$$

This method is applicable only when specific gravity of solids is known. If kerosene is used instead of water then specific gravity

of solid should cont. kerosene instead of water

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- ③ Calcium Carbide Method - It is very quick & takes only 5 mins. It is a field method & is used for rough computation. A small amt. (5gms) is placed in moisture testing equipment (Closed Chamber) & Calcium Carbide (CaC_2) powder is added. Sample is shaken as a result acetylene gas may be produced due to reaction w/ water.



The pressure exerted by C_2H_2 is measured through a calibrated scale & is correlated w/ the water content by the following formula -

Recorded water content by calibrated scale is percentage of water / fraction present in soil w/ its wet wt. of soil mass. The actual water content is represented as a fraction of dry wt. of soil / wt. of solids. Hence actual water content is given as

$$w = \frac{w_H}{1 - w_H}$$

- ④ Sand bath Method - It is a quick field method & it is used when oven is not available. The container with moist soil sample is placed on a sand bath & is heated over a stove. Since temp. is uncontrolled hence there is danger of loss of structural water. Water content computation is similar to oven drying method.

- ⑤ Alcohol Method - It is a quick field method adopted for rough estimation. The methalated spirit is mixed w/ the soil sample in order to increase rate of evaporation. Since alcohol is a oxidizing agent hence this method should not be used for soils containing organic compound & calcium compounds.

- ② Sand Replacement Method - In this method a small hole or pit is made on the ground & vol. of excavated soil is taken say k_1 . The pit is filled by sand the volume of c is measured through a graduated glass cylinder hence ~~total weight is~~
Bulk unit wt $= \gamma = \frac{W}{V}$ (15)

$V = \text{Vol of Soil} = \text{Vol of sand needed to fill the pit.}$

This method is widely adopted for construction of roads & highway & is suitable for gravelly & sandy soils.

- ③ Water displacement Method - This method is suitable for highly cohesive, sticky, plastic soils. A small sample of soil is taken having wt. k_1 . This sample is coated with a thin layer of paraffin wax & wax-coated soil sample is weighed again say k_2 .

$$k_2 = \text{wt. of soil} + \text{wt. of wax}$$

$$k_2 = k_1 + k_{\text{wax}}$$

$$k_{\text{wax}} = k_2 - k_1$$

If wax coated soil sample is immersed in a cylinder c is fully filled with water & the vol. of water displaced say V_2 is measured c is equal to vol. of soil + vol. of wax.

$$V_2 = V + V_{\text{wax}}$$

$$V = \text{Vol of Soil}$$

$$V = V_2 - V_{\text{wax}}$$

$$V_{\text{wax}} = \frac{k_{\text{wax}}}{\gamma_{\text{wax}}}$$

Hence the bulk density of soil $\gamma = \frac{W}{V}$

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

⑥ Torsion balance Method - It is a laboratory method in which drying & weighing is done simultaneously. Infra red radiation is used for drying. This method is suitable for those soils which loose moisture quickly from the atmosphere.

Determination of In-situ density (undisturbed / field)

- 1. Core Cutter Method
- 2. Sand Replacement method
- 3. Water displacement method.

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① Core Cutter Method - Core cutter is a cylindrical vessel open at top & bottom with sharp edges. Let D is the dia of Core Cutter & h is height of Core Cutter / length. Then Volume of Core Cutter is $V = \frac{\pi D^2 h}{4}$.

The empty cut of Core Cutter is known as L_1 . The soil surface is prepared level & smooth & Core cutter is pushed into the soil & it is taken out fully filled with soil & top & bottom surfaces are levelled. Let L_2 is cut of Core Cutter + Soil hence wt. of Soil = $W = W_2 - W_1$.

$\therefore$ Bulk unit wt. / In situ unit wt. $= \gamma = \frac{W}{V} = \frac{W_2 - W_1}{\frac{\pi D^2 h}{4}}$

② Dry unit wt / dry density $= \rho_d = \frac{\rho}{1+w}$

(If dry unit wt is required then a small soil sample is required to a water content)

Dry unit wt / dry density $= \rho_d = \frac{\gamma}{1+w}$

It is not suitable for dry soils & cohesionless soils hence it is applicable for moist cohesive soils only.

Particle Size analysis

IS SOIL CLASSIFICATION

Boulders $\rightarrow$ Size > 300 mm

Cobbles $\rightarrow$ 80 to 300 mm

Gravels $\rightarrow$ 4.75 mm to 80 mm

Sand $\rightarrow$ 0.075 mm to 4.75 mm

Silts $\rightarrow$ 0.002 mm to 0.075 mm

Clays $\rightarrow$ < 0.002 mm

> 300

80 10 300

4.75 80

0.075 4.75

Coarse Soil

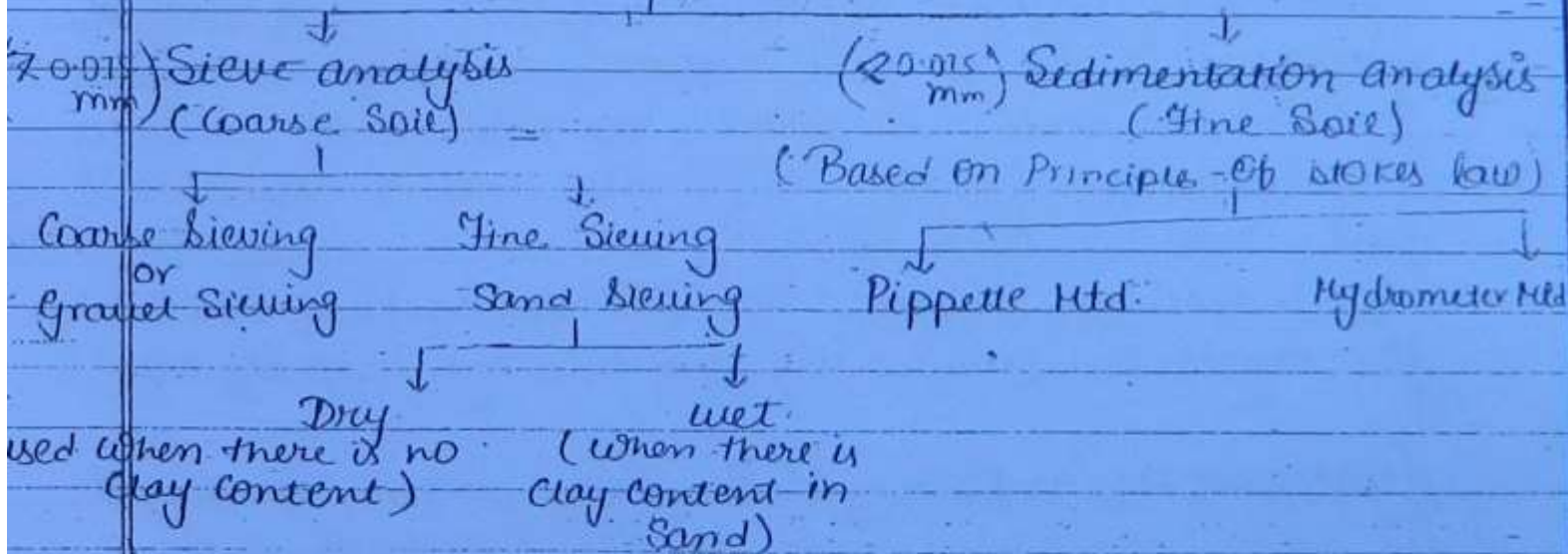
Fine Soil

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For Coarse Soil - Sieve analysis

For Fine Soil - Sedimentation analysis

Particle Size analysis is the process of grading the Soil & Size



Coarse Sieving / Gravel Sieving

Sieves are represented either by their size or by no. Size is dimension of square opening in mm or micron. Sieve no. ϕ represents the no. of square opening in 1 inch of length. The sieves have square opening & are arranged in decreasing order of size from top to bottom. In coarse sieving 4 sieves are used having size 80 mm, 20 mm, 10 mm, 4.75 mm. The sample is shaking for 10 min in the shaking machine & the % wt. retained in each sieve is computed.

Let W_i = wt. of soil retained in i th sieve

W = total wt. of soil

$$\% \text{ of wt. retained in } i\text{th sieve} = \frac{W_i}{W} \times 100$$

And Cumulative % wt. retained

$$= \sum_{i=1}^n \frac{W_i}{W} \times 100$$

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% finer (1-N) - then n th sieve size = (100 - % Cumulative wt. retained in n th sieve)

A graph is plotted on a semilog paper i.e. % finer in Y-axis & diameter in X-axis (X-axis is in log scale). From graph

D_{10} , D_{30} , D_{60} are computed directly & are used to find grad. Coefficient of Uniformity (C_u) & Coefficient of Curvature (C_c) & are used in grading & Classification of soil.

→ D_{10} represents that particle size below < 10% particles are finer than this size. It is also called effective size given by Allen Hazen.

D_{30} is that size below < 30% particles are finer than this size

D_{60} means 60% particles are finer than this size. (by weight)

1. C_u

$$\text{Coefficient of Uniformity } (C_u) = \frac{D_{60}}{D_{10}}$$

If C_u tends towards unity, Soil will be uniformly graded, poorly graded. C_u will be always greater than equal to 1.

If $C_u > 4$ then gravel is well graded & for well graded sands $C_u > 6$.

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

for well graded soil $1 \leq C_c \leq 3$. If C_c is not in the above range, Soil will be poorly graded or uniformly graded.

NOTE

It means flow / Settlement condition should be laminar / uniform
If particle size ~~are~~ is greater than 0.2 mm then turbulent motion may be setup & constant terminal velocity may be achieved.
If particle size is smaller than 0.0002 mm then brownian motion may be setup & settling of particles may not occur.

Procedure

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A soil specimen is taken & a suspension is prepared in a glass jar of 500 ml which is called. Volume of suspension.
The observation are taken at regular interval either using pipette method or using hydrometer method. Following 2 treatments are required before observation.

- ① Pretreatment - It is given to the soil before suspension is prepared. In order to remove organic compounds such as acids & Calcium compounds oxidising agents are added (H_2O_2)
- ② Post treatment - ~~It is~~ After preparing the suspension dispersing agents are added to break the floc otherwise floc will not give true settling. The common used dispersing agent is Sodium hexa Metaphosphate (Calgon)
- ③ Pipette Method - In 10 ml sample is collected from the soil suspension from a specified depth of 10 cm at different time interval. This sample is put in a container for oven drying & mass per unit volume is computed. Let m_d is mass of dried sample obtained from 10 ml volume of suspension (V_p) C is pipette volume.
Mass per unit vol. at any instant = $\frac{m_d}{V_p}$

Let M' is the mass of dispersing agent added in the total volume of soil suspension (~~500~~ 500 ml)

soln: ① D_m i.e. effective size is that size of spherical particles will have same effect as the effect of above soil mass

② Avg Size is D_{50}

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* Fine Sieving (hand sieving)

The particles passing from 4.75 mm are further sieve. Fine Sieving can be done in two way ① Dry sieving ② Wet sieving. Wet sieving is preferred when clay particles are present in sand. In fine sieving seven sieves are arranged in the decreasing size 2mm, 1mm, 600 μ m, 425 μ m, 212 μ m, 150 μ m, 75 μ m (0.075mm). The procedure is similar to previous case of coarse sieving.

Sedimentation analysis (Silt & Clay analysis)

It is based on the Stokes law. "If a spherical particle falls through an infinitely large medium then it achieves a constant terminal velocity."

$$V = \frac{(\gamma_s - \gamma_l) D^2}{18\mu}$$

γ_s = Unit wt. of settling solids

γ_l = Unit wt. of liquid suspension

D = Dia of settling particle

μ = Dynamic viscosity coefficient

Assumption involved in Stokes law

- ① The particle should be spherical (Note that shape of clay particle is platy / plate like ^{lets} ~~and~~ c is not perfectly spherical)
- ② The medium of liquid suspension should be infinitely large & means there should be no resistance from boundary condition i.e. free wall.
- ③ The particle size should be less than 0.2 mm & should be greater than 0.0002 mm.

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[illegible]

Abstract

✓

2

TEM

A graph is plotted betw % finer & diameter in semi-log paper from $\Sigma D_{10}, D_{30}, D_{60}$ are found to compute C_u & C_c .

Ph

Unit vol. of
Liquid Suspension

↑ Specific gravity of liquid Suspension

$$\frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} = \frac{\partial \mathcal{L}}{\partial \mathbf{z}_{t-1}} \mathbf{A}_t + \frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} \mathbf{B}_t$$
$$\frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} = \frac{\partial \mathcal{L}}{\partial \mathbf{z}_{t-1}} \mathbf{A}_t + \frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} \mathbf{B}_t$$
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$$\frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} = \frac{\partial \mathcal{L}}{\partial \mathbf{z}_{t-1}} \mathbf{A}_t + \frac{\partial \mathcal{L}}{\partial \mathbf{z}_t} \mathbf{B}_t$$

Hydrometer is a device used to measure specific gravity of liquids. Let H_e is the effective depth of particle settlement. At any time t after starting the settlement terminal velocity is $V = H_e/t$

Using Stokes law diameter will be known & using hydrometer reading corrected value of R_h can be known

$$\gamma_s = \frac{G_s}{G_s - 1} \times \frac{V}{K} \times \frac{\gamma_w}{d} \times \frac{R_h}{10} \quad (22)$$

Where G_s = Specific Gravity of settling soils

V = total volume of soil suspension (500ml)

K = total vol. of soil sample

γ_w = unit wt. of water at 27°C

R_h = corrected hydrometer reading

Corrected hydrometer reading is $R_h' \pm C$

C = final correction

R_h' = observed hydrometer reading

Hydrometer Correction

- ① Meniscus correction factor (C_m) - Due to turbidity upper meniscus level will be recorded but actual level is lower level hence C_m is always positive because hydrometer reading $\uparrow$ downward.
- ② Temp correction factor (C_t) - If test temp diff then 27°C then temp. correction is needed.
 $C_t = +ve$ - test temp $> 27^\circ\text{C}$
 $C_t = -ve$ - test temp $< 27^\circ\text{C}$
- ③ Dispersing agent correction factor - Due to addition of dispersing agent specific gravity of suspension is more. Hence assumed R_h is greater than the actual & is always negative

$$C = C_m + C_c - C_d$$

(23)

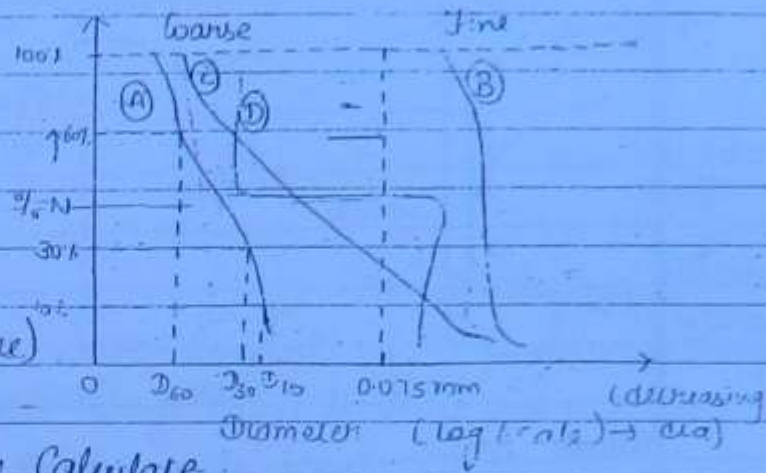
Graph between diameter & % finer

A - Uniformly graded / poorly Graded Coarse Soil

B - Uniformly graded fine Soil

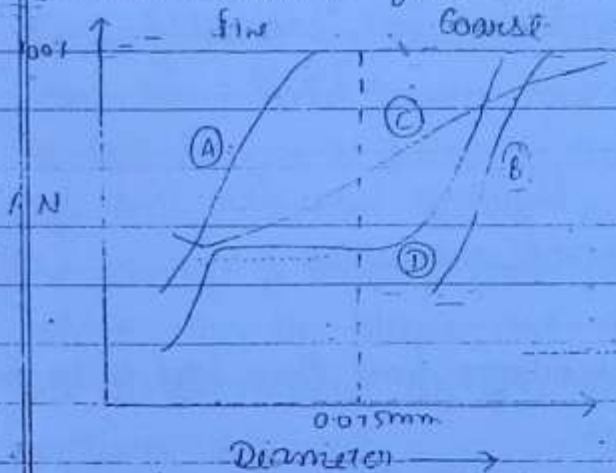
C - Well graded Soil

D - Gap graded Soil
(eg. - Gravel & Clay mixture)



Graph D_{60} , D_{30} & D_{10} Can be Calculate

C can be used to find C_c & C_u



→ Consistency limits

Consistency represents the relative ease with which soil can be deformed it represents degree of deformation due to presence of water. Consistency has its significance for fine grained soils & it was defined by Atterberg hence Consistency limit is also called Atterberg limit. It is related to the water content. Depending upon presence of water content consistency of fine soil is classified in 4 stage

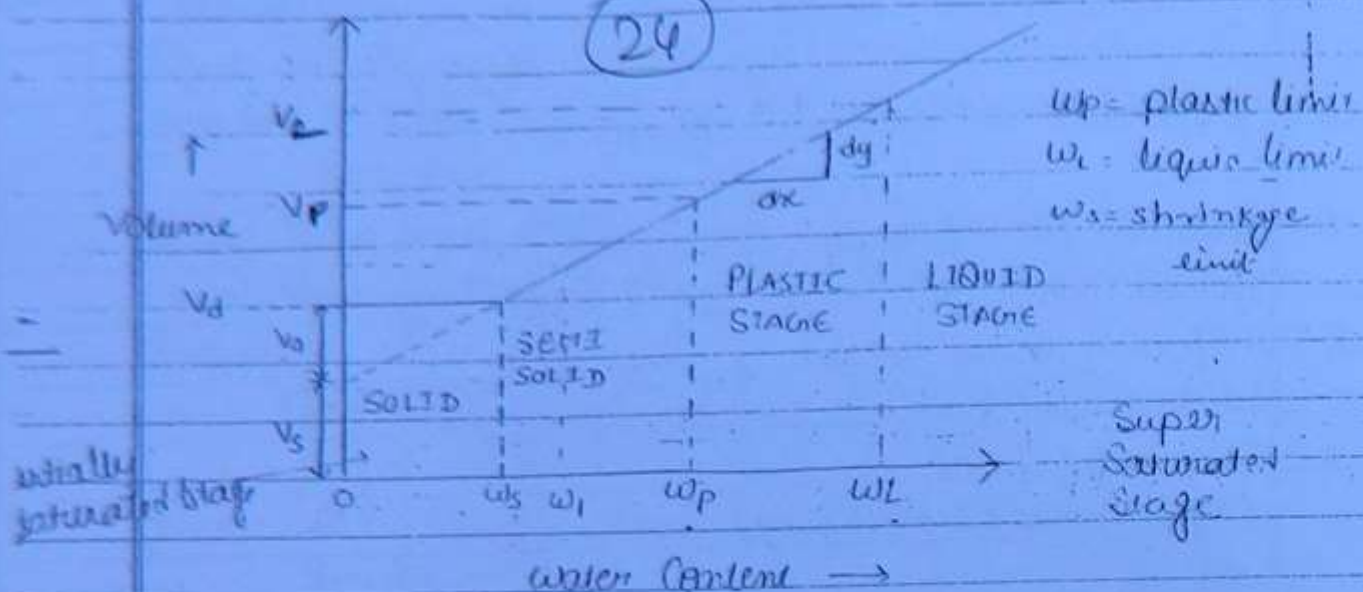
① Solid Stage

② Semi solid Stage

③ Plastic Stage

④ Liquid Stage

(24)



When $w=0$

then $V = V_d = V_a + V_s = \text{Vol. of air} + \text{Vol. of Solids}$

$w_s = \text{Shrinkage limit, at } S < \text{Soil is just saturated}$

At $w = w_s \Rightarrow D_s = 100\% \quad (S=1)$

Note that if water content is below shrinkage limit then there will not be change in vol. with moisture variation.

If water content $w >$ than shrinkage limit then soil is in Super Saturated stage.

In this stage vol. of soil change $\propto$ change in water content. It means in partially Saturated stage/Solid stage void ratio is constant ~~whereas~~ whereas in Semi-Solid, plastic & liquid stage void ratio will change $\propto$ change in water content.

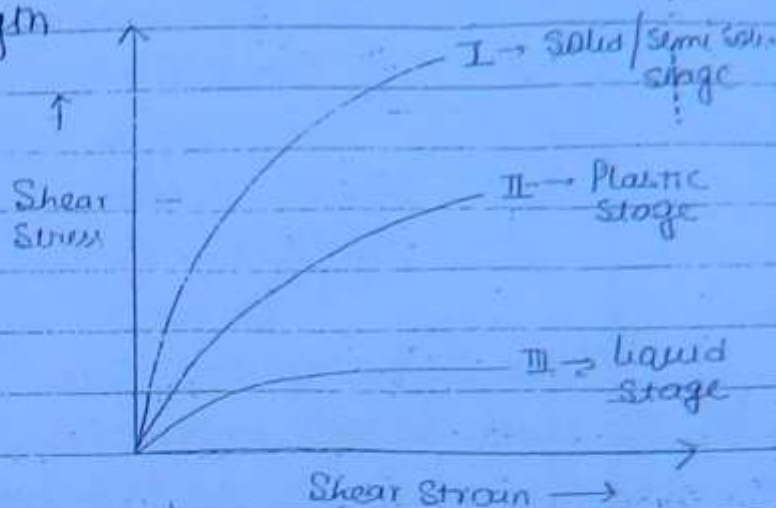
$$\frac{dy}{dx} = \frac{V_L - V_P}{w_L - w_P} = \frac{V_P - V_d}{w_P - w_s} = \frac{V_L - V_d}{w_L - w_s} = \text{constant}$$

The boundary between 2 consistency stages is defined as consistency limits. At liquid limit most of the soil show negligible shear strength. & Shear strength decreases with the $\uparrow$ in water content.

Note In solid & semi-solid stage soils show ^{diff. soil} "diff" shear strength & properties whereas in liquid stage most of the soil show

equal & negligible Shear Strength

(25)



1. Liquid limit - It is that water content at c consistency changes from plastic stage to liquid stage. or It is that min. water content at c soil has tendency to flow.

Determination of liquid limit - The test is performed by a Casagrande Apparatus. The height of free fall of cup is 1 cm. Only hard & uniform base.

The soil sample is filled in the cup & a groove is cut on the soil sample either using ASTM tool or Casagrande tool. The no. of blows / free fall are found to close the loop. Let N_1 no. of blows are required to close the loop when soil has water content w_1 . The test is repeated with different water contents $w_2, w_3, \dots$ to find $N_2, N_3, \dots$. A graph is plotted between water content in Y-scale (arithmetic scale) & no. of blows in X-scale (log scale).

Liquid limit is defined that water content c corresponds to 25 blows to close the ^{groove} loop c is found from graph. The dimensions of tool used to cut the groove are

ASTM tool

Base - 2mm wide

Top - 13.6 mm wide

Height 10 mm

It is preferred for less plastic soil such as silts

Casagrande tool

Base - 2mm wide

Top - 11 mm wide

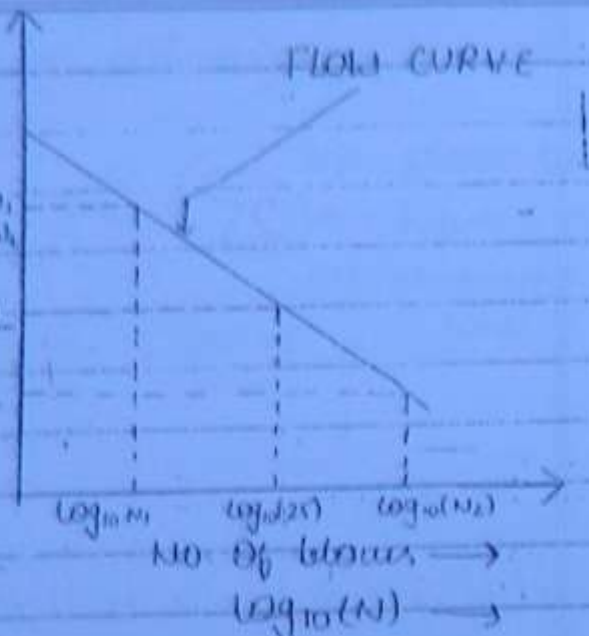
Height 8 mm

The Slope of flow Curve is
Constant & is called flow
Index.

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

(26)

$$\frac{\tau_{w=w_1}}{w_1 - w_L}$$



Flow Index represents rate
of loss of shear strength with increase
in water content. Greater the flow index
smaller is the shear strength.

2. Plastic limit - It is that water content at which consistency
of soil changes from semi solid stage to plastic stage.
OR It is that min. water content at which soil is in plastic
stage. If a soil is at plastic limit & threads of
3mm dia are rolled using the soil then cracks just
appear on the surface. Generally fine soils have greater
liquid limit & plastic limit than coarse soil.

NOTE - (i) For fine soils liquid limit is much greater than the plastic
limit whereas for coarse soil liquid limit & plastic
limit are nearly same.

(2) Type of Soil	Liquid limit	Plastic limit
Alluvial soil	40-60%	20-40%
Block	400-500%	200-250%

- (3) ~~These~~ High plastic soils have high liq. limit & greater
liq. limit indicates more compressibility.
- (4) If sand is mix. in clay then L.L & P.L of clay both
are reduced but reduction in plastic limit is less than
reduction in ^{liquid} plastic limit hence plasticity of clay reduced.

3. Shrinkage limit - It is that water content at c consistency changes from solid stage to semi-solid stage.

OR It is that max water content at c further decrease in water content do not result in decrease in volume. It means below shrinkage limit soil is partially saturated & at shrinkage limit soil is just saturated.

Determination of Shrinkage limit

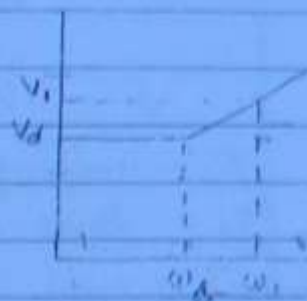
(27)

(1) At shrinkage limit $S = 1$, $w = w_s$

$$S \cdot e = w \cdot G_r$$

$$\text{i.e. } e = w_s \cdot G_r \Rightarrow \left[w_s = \frac{e}{G_r} \right]$$

(2) From graph $w_s = \left[\frac{V_1 - V_d}{w_d} \right] \gamma_w$



Where V_1 = Vol. of soil at w.c w_1

V_d = Vol. of dry soil = Vol. of soil at shrinkage limit

w_d = dry wt. of soil mass

γ_w = unit wt. of water

$$(3) \quad G_r = \frac{1}{\frac{\gamma_w - w_s}{\gamma_d \cdot 100}} = \frac{1}{R \cdot 100}$$

R - Shrinkage Ratio

G_r - Specific gravity of solids

w_s - Shrinkage limit in %

γ_d = dry unit wt.



Shrinkage Ratio (R) - It is defined as the ratio of given vol change in a soil expressed as dry vol. to the corresponding change in water content above the S.L.

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

$$= \frac{w_1 - w_2}{w_s - w_d} \times 100$$

If $w_1 = w_s$ then $V_1 = V_d$

$$R = \frac{V_i - V_d}{V_d} \times 100$$
$$w_1 - w_s$$

Shrinkage Ratio = $R = \frac{\gamma_d}{\gamma_w}$ (28)

Volumetric Shrinkage - It is the % reduction in vol. of soil on drying

$$V_s = \frac{V_i - V_d}{V_d} \times 100$$

$$V_s = R \times (w_i - w_s)$$

V_i = initial volume on drying

V_d = final volume on drying

Degree of shrinkage (D.O.S) - It is expressed as decrease in vol. on drying corresponding to its initial volume.

$$D.O.S = \frac{V_i - V_d}{V_i} \times 100$$

If D.O.S is greater then soil is less suitable for foundation material

D.O.S	Suitability
$\leq 5\%$	Good
5-10%	Medium good
10-15%	Poor
$> 15\%$	Very poor

Linear Shrinkage - It refers for decrease in one dimension of soil expressed as percentage of its initial dimension

$$L_s = \left[1 - \left(\frac{100}{100 + V_s} \right)^{1/3} \right] \times 100$$

V_s = Volumetric Shrinkage

Plasticity Index - It is defined as range of consistency within which soil is in plastic stage. Plasticity is due to presence of Clay Content

$$I_p = w_L - w_p$$

(29)

For gravel & ^{coarse} sand $I_p \rightarrow 0$

For fine soils I_p may be greater than 50%.

% I_p	Consistency
0	Non plastic
1-5	Slight plastic
5-10	Low plastic
10-20	Medium plastic
20-40	High plastic
Over 40	Very high plastic

Shrinkage Index - It represents semi solid stage of consistency

$$I_s = w_p - w_s = \text{Plastic limit} - \text{Shrinkage limit}$$

Consistency Index / Relative Consistency (I_c)

$$I_c = \frac{w_L - w}{w_L - w_p} = \frac{w_L - w}{I_p}$$

Stage

If $I_c < 0$ then $w > w_L$ liquid limit hence soil is in liquid

If $I_c > 0$ but $I_c < 1$ Soil is in plastic stage

If $I_c > 1$ Soil is in semi solid or solid stage

Liquidity Index $\rightarrow$

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

Note that $I_c + I_L = 1$

$$I_L = 1 - I_c$$

If $I_L > 1$ Soil is in liquid stage

If $0 < I_L < 1$ Soil is in plastic stage

If $I_L < 0$ Soil is in solid or semi solid stage

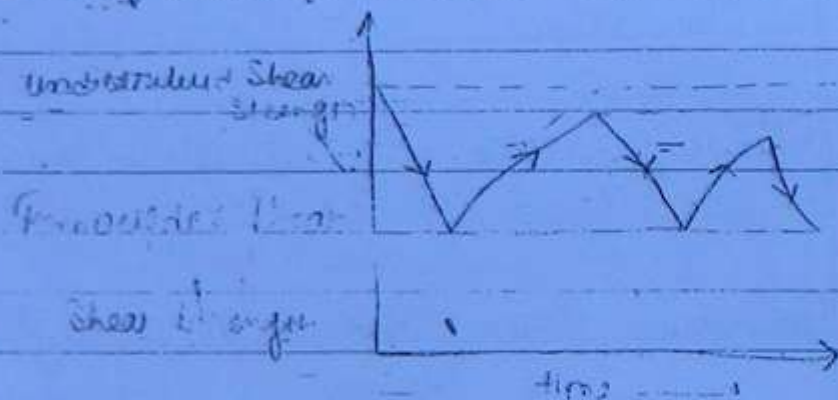
Toughness Index It is the ratio of plasticity Index to the flow Index. It gives the idea about Shear strength of soil at plastic stage. If $I_t < 1$ then Soil can be easily crushed & is brittle.

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

(30)

For most of the Clays I_t is between 0 to 3 ($0 < I_t < 3$)

Thixotropy - It is that property of soil due to a loss in shear strength may be regained. The loss in shear strength may be due to disturbance in orientation of molecule & permanent destruction of bond. permanent destruction is not recoverable.



NOTE The Coarse soil such as sand & gravels do not show much loss in shear strength on remoulding. Hence are less sensitive. This property of thixotropy is more prominent in fine soil specially in plastic clays.

Sensitivity - The degree of disturbance ϵ is achieved on remoulding is expressed by sensitivity.

$$S_t = \frac{\text{Unconfined Compressive strength of undisturbed soil}}{\text{Unconfined Compressive strength of Remoulded soil.}}$$

$$= \frac{q_{u \text{ undisturbed}}}{q_{u \text{ remoulded}}}$$

$S_t = 1$ for coarse (i.e. gravel & sand)

If $S_t = 1$ then there is no loss in the strength of soil on remoulding.

If $S_u = 1$ to 4 then soil is less sensitive or normal sensitive.
 If $S_u = 4$ to 8 then soil is sensitive & may have flocculent or honeycomb structure.
 If $S_u = 8$ to 16 then soil is extra sensitive such as fine clays.
 If $S_u > 16$ soil is quick not suitable for foundation & is unstable. eg - quick clays

(3)

Activity - (A_c) - The behaviour of fine soils such as clay & change in water content is influenced by presence of clay minerals & amount of clay minerals. Higher activity means soil is more compressible & will show more swelling & shrinkage on moisture variation. Acc to Skempton the activity is defined as

$$A_c = \frac{\% I_p}{\% C}$$

I_p = Plasticity Index in %

C = Percentage Clay Content (> 0.002 mm)

Activity	I_p	Type of Soil
> 0.75	1	Inactive / Non active
$0.75 \times A_c < 1.25$		Normal active
$A_c > 1.25$		Active
K Activity		Type of Mineral
Kaolin 0.38		Kaolinite
0.90		Illite
1.2		Montmorillonite

- Black Soil contain Montmorillonite mineral & are highly active

Collapsibility - Those soils & undergo large decrease in volume due to rain moisture content without rise in external load are called collapsible soil. eg - loess

It is represented by C_p = Collapsible potential

$$C_p = \frac{\Delta e}{1+e} = \frac{\Delta H}{H}$$

Δe = Change in void ratio due to change in moisture content

e = initial void ratio

ΔH = Change in height

H = Initial height

(32)

In above expression soil mass is considered semi-rigid i.e. area is large & constant height is small & will change

7. C_p	Description of soil
0-1	No trouble to structure
1-5	Moderate trouble
5-10	trouble
10-20	Severe trouble
more than 20	very severe trouble

Miscellaneous

① Diffⁿ betⁿ organic & Inorganic soils

If 7.1 of oven dried sample is less than 70% of liquid limit of air dried sample then soil is organic.

② Field Method to differentiate Silt & Clays

1. dilatancy test - A remoulded wet sample is placed in the palm & is shaken if water reflects in the surface of soil shiny surface then soil is silt otherwise clay.
2. dispersion test - A spoon of dry soil sample put in a glass of water. If soil is settled down in 5-10-10 mins without much turbidity then soil is silt & if turbid suspension is formed then soil is clay.
3. Toughness test The threads of 3 mm size are rolled & dried in the air. After drying if the balls are pressable betⁿ the fingers then it is silt otherwise clay (hard).
4. Dry & wet test.

Ques:- The mass of a soil is coated with thin layer of paraffin wax. The wt. of soil & wax is 690.6 gm. The soil alone has wt. of 683 gm. When soil sample is coated with wax is immersed in water it displaces 350 ml. of water. The specific gravity of solids is 2.73 & that of wax is 0.89. Find void ratio & degree of saturation of soil if $w = 17\%$. Given that mass density 1 gm/cc.

Sol:- wt. of soil $M_s = 683 \text{ gm}$

wt. of soil & wax = 690.6 gm

wt. of wax = $690.6 - 683.0 = 7.6 \text{ gm} = M_w$

Density of wax $\rho_{wax} = G_{wax} \times \rho_w$
 $= 0.89 \times 1 \text{ gm/cc}$

Vol. of wax = $\frac{\text{Mass of wax}}{\rho_{wax}} = \frac{7.6}{0.89 \times 1} = 8.539 \text{ cc}$

Vol. of water displaced = Vol. of soil + Vol. of wax
 $350 \text{ ml} = V + 8.539$

$V = 350 - 8.539 = 341.46 \text{ cc}$

Density of soil $\rho = \frac{683}{341.46} \approx 2 \text{ gm/cc}$

$\rho_d = \frac{\rho}{1+w} = \frac{2}{1+0.17} = 1.71 \text{ gm/cc}$

$\rho_d = \frac{G \cdot \rho_w}{1+e} = 1.71 = \frac{2.73 \times 1}{1+e}$

$e = 0.596$

W.K.T $S \cdot e = wG$

$S = \frac{w \times 2.73}{0.596} = 0.778 = 77.8\%$

Ques In Core Cutter of 12.6 cm ht & 10.2 cm dia has empty mass of 1011 gm it is used to determine insitu density of soil. If mass of Cone. Cutter filled with soil is 2970 gm. If $w = 62\%$. What is the insitu dry density & porosity of soil. If soil gets

Ques:- The mass specific gravity of a fully saturated soil of clay with w.e. 40% is 1.88. On oven drying mass specific gravity drops to 1.74. Compute for S_r of solids $S.L$ of soil.

Sol:- $G_{m_s} = \text{mass sp. gr. at sat stage} = \frac{\gamma_{sat}}{\gamma_w} = 1.88$

$$w = 0.40 \text{ \& } S = 1.0$$

$$S \cdot e = wG$$

$$1 \cdot e = 0.4G$$

$$e = 0.4G$$

(34)

$$\gamma_{sat} = \left(\frac{G + e}{1 + e} \right) \gamma_w$$

$$\frac{\gamma_{sat}}{\gamma_w} = \frac{G + e}{1 + e} = 1.88 \Rightarrow \frac{G + 0.4G}{1 + 0.4G} = 1.88$$

$$1.4G = 1.88 + 0.752G$$

$$0.648G = 1.88$$

$$G = 2.9$$

~~Given:-~~ ~~Find:-~~

At dry condⁿ $G_{m_s} = \frac{\gamma_d}{\gamma_w} = 1.74$

$$G = \frac{1}{\frac{\gamma_w - w_s}{\gamma_d} - 1} = \frac{1}{\frac{1.74 - w_s}{100} - 1} = 2.9 \quad \checkmark$$

$$\Rightarrow \frac{100}{1.74 - w_s} = 2.9 \Rightarrow \frac{504.6 - 2.9w_s}{404.6} = 100$$

$$404.6 = 2.9w_s$$

$$w_s = 23.7\%$$

$$w_s = 23.7\%$$

fully saturated due to heavy rain then what will be the water content if no change in vol. occurs. $G = 2.69$

Sol- Cone cutter method is also sometimes called as cylinder penetration method.

Height of Cone Cutter $H = 12.6 \text{ cm}$

diameter " " " $D = 10.2 \text{ cm}$

Mass of empty Cone Cutter $M_1 = 1071 \text{ gms}$

Mass of Cone Cutter + Soil $M_2 = 2970 \text{ gms}$

Mass of Soil $= 2970 - 1071 = 1899 \text{ gms}$

Vol. of Soil = Vol. of Cone Cutter $= \frac{\pi D^2 H}{4} \text{ cc}$

$$= 1029.58 \text{ cc}$$

Bulk density $P = M/V = 1899/1029.58 = 1.84 \text{ gm/cc}$

$$P_d = \frac{P}{1+w} \Rightarrow \frac{1.84}{1+0.06} = 1.735 \text{ gm/cc}$$

$$P_d = \frac{G \cdot P_w}{1+e} \Rightarrow 1+e = \frac{2.69 \times 1}{1.735}$$

$$\text{Void ratio } e = 0.55$$

$$\text{Porosity } n = \frac{e}{1+e} = \frac{0.55}{1.55} = 0.355$$

If Vol. is Constant then Volume of void / Vol. of Solid is Constant
i.e. void ratio is Constant

$$\text{At } S = 1 \quad S \cdot e = w \cdot G$$

$$1 \times 0.55 = w \times 2.69$$

$$w = 0.2044 = 20.44\%$$

Q11 marks

ESE An oven dried soil sample of vol. 225 cc weighs 340 gms.

If $G = 2.72$ then determine void ratio & shrinkage limit.

What will be the water content when soil fully saturate the sample & cause a increase in vol. equal to 6% of original dry vol.

Sol I Volume $V_d = 225 \text{ cc}$
 Mass dry = $M_d = 390 \text{ gm}$

$$\rho_d = \frac{M_d}{V} = \frac{390}{225} = 1.733 \text{ gm/cc}$$

$$\rho_s = \frac{G \cdot \rho_w}{1+e} \Rightarrow \frac{2.72 \times 1}{1.733} = 1.57$$

(30)

void ratio $e = 0.569$

Shrinkage limit $G_L = \frac{1}{\frac{\gamma_w - w_s}{\gamma_d \cdot 100}} = \frac{1}{\frac{g \cdot \rho_w - w_s}{g \cdot \rho_d \cdot 100}}$

$$\Rightarrow 2.72 = \frac{1}{\frac{1}{1.733} - \frac{w_s}{100}}$$

$w_s = 20.938 \%$

II Initial vol. $V_d = 225 \text{ cc}$

Saturated vol. $= 1.06 \times 225 = 243 \text{ cc}$

$$V = \frac{\text{Vol of void}}{\text{Vol of solid}} = \frac{243}{1.733}$$

$V = \text{Vol. of void} + \text{Vol. of solids} = 243$

$V_{\text{sat}} = V_v + V_s$

$V_v = V_{\text{sat}} - V_s$

$V_v = V_w = V_{\text{sat}} - V_s$

$\left[\frac{V_v}{V_s} = 0.569 \right]$

After saturation

$$w_2 = \frac{L_v}{L_s} = \frac{L_w}{L_s}$$

Mass of dry Soil = Mass of Solids = 390 gms

$$w_2 = \text{Vol. of solid} = \frac{\text{Mass of Solids}}{G \cdot \rho_w}$$

$$V_s = \frac{390}{2.72 \times 1} = 143.38 \text{ cc}$$

$$V_v = \cancel{243 - 143.38} \quad V - V_s = 243 - 143.38 = 99.62 \text{ cc}$$

$$\text{Void ratio after saturation} = \frac{V_v}{V_s} = \frac{99.62 - 1}{143.38} = 0.694$$

$$S \cdot e = w \cdot G_s$$

$$1 \times 0.69 = w \times 2.72 \Rightarrow w = w_2 = 25.5\%$$

(37)

Ques A Cohesive Soil yields a max. dry density of 1.8 gm/cc at a optimum moisture content of 16% if the sp. gravity of solids is 2.65 then find the degree of saturation & also find theoretical max. dry density e is possible to be achieved.

Sol. I dry density $\rho_d = 1.8 \text{ gm/cc}$

$$w = 16\% = 0.16$$

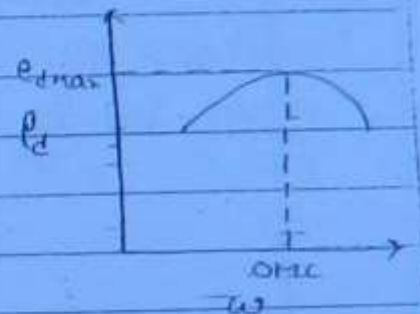
$$G_s = 2.65$$

$$\rho_d = \frac{G_s \rho_w}{1+e} \Rightarrow 1+e = \frac{2.65 \times 1}{1.8}$$

$$\text{Void ratio } e = 0.47$$

$$S \cdot e = w \cdot G_s$$

$$S = \frac{0.16 \times 2.65}{0.47} = 0.90 = 90\%$$



II Theoretically max dry density will be achieved when all the air is escaped out hence $S=1$ at

$$\rho_{d \text{ max}}, \text{ void ratio } e = \frac{w \cdot G_s}{S} = \frac{0.16 \times 2.65}{1}$$

$$e = 0.424$$

$$\text{theoretical } \rho_{d \text{ max}} = \frac{G_s \rho_w}{1+e} = \frac{2.65 \times 1}{1+0.424} = 1.86 \text{ gm/cc}$$

Ques A Soil is required to be excavated from a borrow pit for construction of an embankment of ht. 6 m , top width 2 m side slope $1:2$. The unit wt. of soil in bulk (γ) = 18 kN/m^3 & its natural water content is 8% . The dry density is required in the embankment is 20 kN/m^3 cum. a $w = 10\%$. The $G_s = 2.7$. Estimate the quantity of soil required to be excavated from

from the borrow pit to construct 1 m length of embankment
 If cam truck has capacity to carry 30kN/trip then find
 no. of trips needed to bring the soil from borrow pit to
 construct 1 m length of embankment. Also find porosity
 & degree of saturation for embankment soil.

Pit

V_1 = total vol. of soil excavated

e_1 = void ratio of pit soil

$$\rho_{d1} = \frac{P}{1+w_1}$$

w_1 = B.T.

V_{s1} = Vol. of Solids

Embankment

V_2 = total vol. of embankment soil

e_2 = void ratio of embankment soil

ρ_{d2} =

w_2 = 10%

V_{s2} = V_{s1}

38

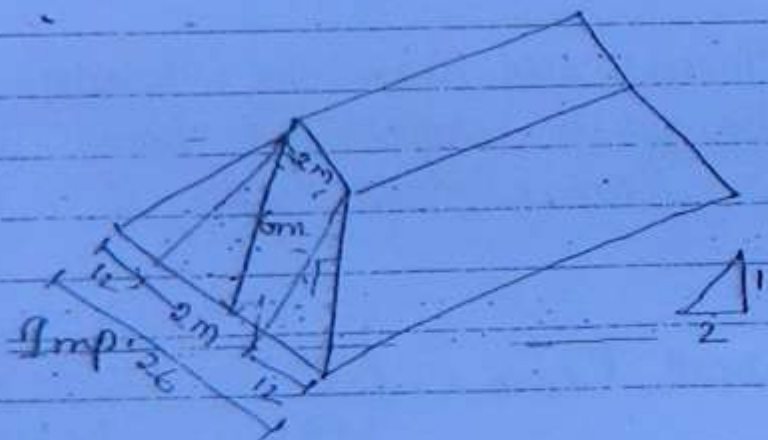
$$\frac{V_v}{V_s} = e \Rightarrow 1+e = \frac{V_v}{V_s} + 1 \Rightarrow \frac{V_v + V_s}{V_s} = 1+e$$

$$\Rightarrow \frac{V}{V_s} = 1+e$$

$$V_s = \frac{V}{1+e}$$

$$V_{s1} = V_{s2}$$

$$\frac{V_1}{1+e_1} = \frac{V_2}{1+e_2}$$



$$\text{Area} = \frac{a+b}{2} \times h = \frac{2+26}{2} \times 6 = 84 \text{ m}^2$$

$$V_2 = \text{Vol.} = A \times L = 84 \times 1 = 84 \text{ m}^3$$

$$\rho_{d1} = \frac{P}{1+w_1} \Rightarrow \gamma_{d1} = \frac{\gamma}{1+w_1} = \frac{18}{1+0.08} = 16.67$$

$$\gamma_{d1} = \frac{G \gamma_w}{1+e_1} \Rightarrow 1+e_1 = \frac{2.7 \times 9.81}{16.67} \Rightarrow e_1 = 0.589$$

$$\gamma_{d2} = \frac{G_1 \gamma_w}{1+e_2} \Rightarrow e_2 = \frac{2.7 \times 9.81}{20} = 0.324$$

$$V_1 = V_2 \times \frac{1+e_1}{1+e_2} = \frac{84 \times 1+0.589}{1+0.324}$$

(39)

$$V_1 = 100.81 \text{ m}^3$$

$$\text{Wt. of excavation} = \text{Vol} \times \gamma_{\text{bulk}}$$

$$W = 100.81 \times 18 = 1814.58 \text{ KN}$$

$$\text{No. of trips of truck required} = \frac{\text{total wt. of soil}}{\text{wt. taken per trip}}$$

$$= 1814.58 / 80$$

$$\boxed{\text{no.} = 22.66 \approx 23 \text{ trips}}$$

$$\text{Porosity for embankment soil} = n = \frac{e_2}{1+e_2} = \frac{0.324}{1.324}$$

$$\boxed{n_2 = 0.24}$$

$$S \cdot e_2 = w_2 \cdot G_1 \Rightarrow S_2 = \frac{10 \times 2.7}{0.324}$$

$$\boxed{S_2 = 83\%}$$

Ques A Compacted Cylindrical Specimen 50 mm in dia, 100 mm long is to be prepared from a dry soil. The Soil is required to have $w = 15\%$ & γ . Air void is 20%. then Calculate the wt. of soil & water req. in the preparation of above Specimen. If $G_r = 2.69$.

$$\text{Vol. of Cylindrical} = V_s = \frac{\pi \times 50^2 \times 100}{4} = 196.349 \text{ mm}^3 \times 10^3 = 196.35 \text{ cc}$$

$$w = 15\% = 0.15$$

$$n_a = 20\% = \frac{V_a}{V}$$

$$V = V_a + V_w + V_s \quad \text{--- (1)}$$

$$V_a = 196.35 \times 0.20 = 39.27 \text{ cc}$$

$$w = \frac{W_w}{W_s} = \frac{V_w \cdot \gamma_w}{V_s \cdot \gamma_s} = \frac{V_w}{V_s} \cdot G_r = 0.15$$

$$0.15 = \frac{V_w}{V_s} \times 2.69$$

$$V_w = 0.15 \times 2.69 V_s \Rightarrow V_w = 0.4035 V_s$$

$$\text{Item 1} \quad 39.27 \pm 0.4035 V_s \pm V_c = 196.35$$

$$V_s = 111.92 \text{ cc}$$

$$V_w = 0.4035 \times 111.92 = 45.16 \text{ cc}$$

$$\begin{aligned} \text{wt. of water} &= V_w \times \gamma_w \\ &= 45.16 \times 9810 \times 10^{-6} \times 9810 \text{ N} \\ &= 0.443 \text{ N} \end{aligned}$$

$$\begin{aligned} \text{wt. of solid} &= \gamma_s \cdot V_s \\ &= G \gamma_w \times V_s \\ &= 2.69 \times 9810 \times 111.92 \times 10^{-6} \text{ N} \\ &= 2.95 \text{ N} \end{aligned}$$

(40)

Ques The plastic limit of a soil is 25% & P.I = 8%. When the soil is dried from plastic limit the Vol. Change is 25% of vol. at plastic limit & when soil is dried from liquid limit to dry stage Vol. Change is 34% of the vol. at liquid limit. Determine Shrinkage limit & S.R.

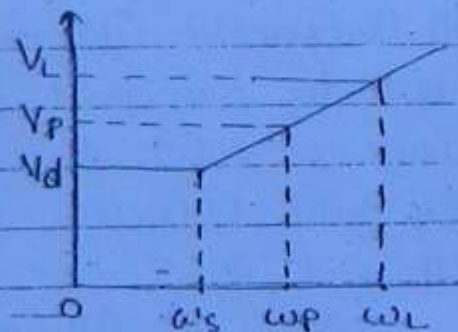
$$\text{Soln:-} \quad I_p = w_L - w_p$$

$$w_L \pm I_p \pm w_p = 8\% + 25\% = 33\%$$

$$\textcircled{1} \quad V_p - V_d = 0.25 V_p$$

$$0.75 V_p = V_d$$

$$V_p = V_d / 0.75$$



$$\textcircled{2} \quad V_L - V_d = 0.34 V_L$$

$$0.66 V_L = V_d$$

$$V_L = \frac{V_d}{0.66}$$

Vol. at Shrinkage limit = dry volume = V_d
 $\frac{dy}{dx} = \text{constant}$

$$\frac{V_L - V_d}{w_L - w_s} = \frac{V_p - V_d}{w_p - w_s} = \frac{\frac{V_d}{0.66} - V_d}{0.33 - w_s} = \frac{V_d / 0.75 - V_d}{25 - w_s}$$

$$\frac{1-0.66}{0.66(33-w_s)} = \frac{1-0.75}{0.75(25-w_s)}$$

$$\Rightarrow w_s = 10.36\% \text{ ans}$$

$$S.R. = \frac{V_1 - V_d}{V_d}$$

~~210~~ 219

(41)

$$w_1 - w_s$$

$$S.R. = \frac{V_d - V_d}{0.66}$$

$$\times 100 = 2.27 \text{ ans}$$

$$V_d$$

$$33 - 10.36$$

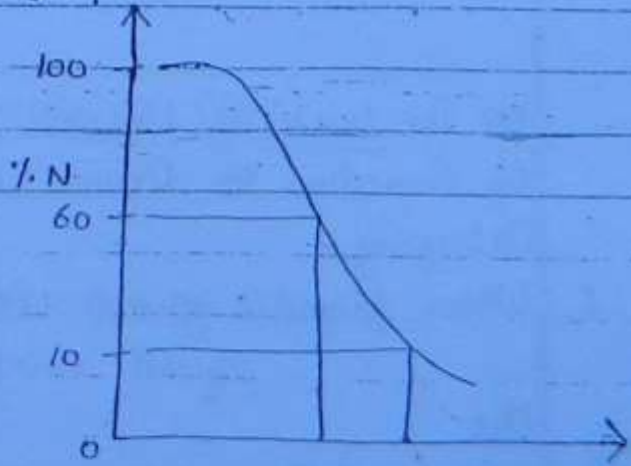
Find D_{10} & C_u

S.NO	Mesh opening (mm)	wt. Retained gms	% Retained	Cumulative % Ret.	% N
1	2.4	0.0	0	0	100
2	1.2	0.0	0	0	100
3	0.6	30.0	6	6	94
4	0.3	215.0	43	49	51
5	0.15	225.0	45	94	6
6	0.075	25.0	5	99	1
7	Pan at bottom	5.0	1	100	0

$$ZK = 500 \text{ gms}$$

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

$$Eff \text{ size} = D_{10}$$


log scale D_{10} in mm

1st Choice

SOIL CLASSIFICATION SYSTEM

Page No.
 Date / /

Unified Soil Classification

→ Given by A. Casagrande

→ Adopted by I.S.

→ Based on particle size ^{distribution} for coarse soil & plasticity characteristics of fine soils

Unified Soil Classification

Coarse grain classification

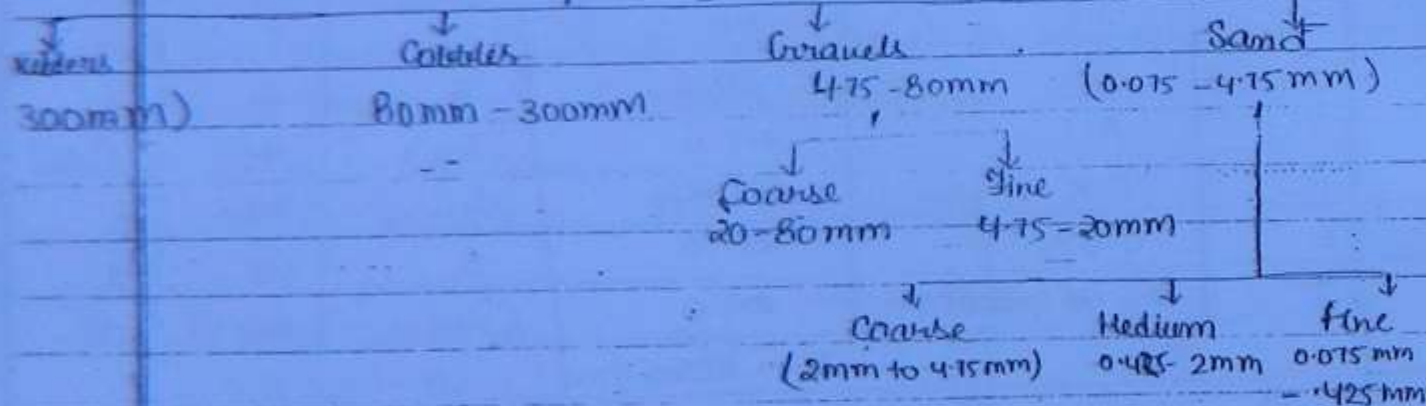
> 50% by weight is greater than 0.075 mm

(I.S. Sieve no 200)

Fine grain classification

> 50% by wt. is smaller than 0.075 mm

A Coarse grain Classification



On the basis of fineness, - Coefficient of uniformity, Coefficient of Curvature & Coarse soils are classified in following category.

1. When fineness $\leq 5\%$. (fineness means % of fine soil till & clay present in coarse soil)

① G₁ - (well graded gravel)

$C_u > 4$

$1 \leq C_c \leq 3$

fineness $< 5\%$

② GP → Poorly graded gravel (uniformly graded)
 C_u & C_c are not in above range

③ SW - Well graded Sand

$$C_u > 6$$

$$1 \leq C_c \leq 3$$

$$\text{fineness} < 5\%$$

(43)

④ SP - Poorly graded sand

C_u & C_c are not in above range

2. When fineness is between 5% to 12%.

under this condition dual symbol is used

C → Clay

A when Clay Content is greater than Silt Content --

① GW-GC (well graded gravel containing clay)

$$C_u > 4$$

$$1 \leq C_c \leq 3$$

$$5\% \leq \text{fineness} \leq 12\%$$

② GP-GC (Poorly graded gravel containing clay)

C_u & C_c are not in above range

③ SW-SC (well graded sand containing clay)

$$C_u > 6$$

$$1 \leq C_c \leq 3$$

④ SP-SC (Poorly graded sand containing clay)

C_u & C_c are not in above range

B when Silt Content is greater than Clay Content

M - Silt

① GW-GM (well graded gravel containing silt)

$$C_u > 4 ; 1 \leq C_c \leq 3$$

$$5\% \leq \text{fineness} \leq 12\%$$

② GP-GM (Poorly graded gravel containing silt)
 C_u & C_c are not in above range

③ SU-SM

$C_u > 6$; $1 < C_c < 3$

(44)

④ SP-SM

C_u & C_c are not in above range

3. When fineness is greater than 12%.

A. When Clay Content is greater than Silt Content

① GC (Gravel > Sand, Clay > Silt) Clayey gravel

$I_p > 7\%$ C_u & C_c are not there.
 fineness > 12%.

② SC (Clayey sand) (Sand > Gravel, Clay > Silt)

$I_p > 7\%$
 fineness > 12%.

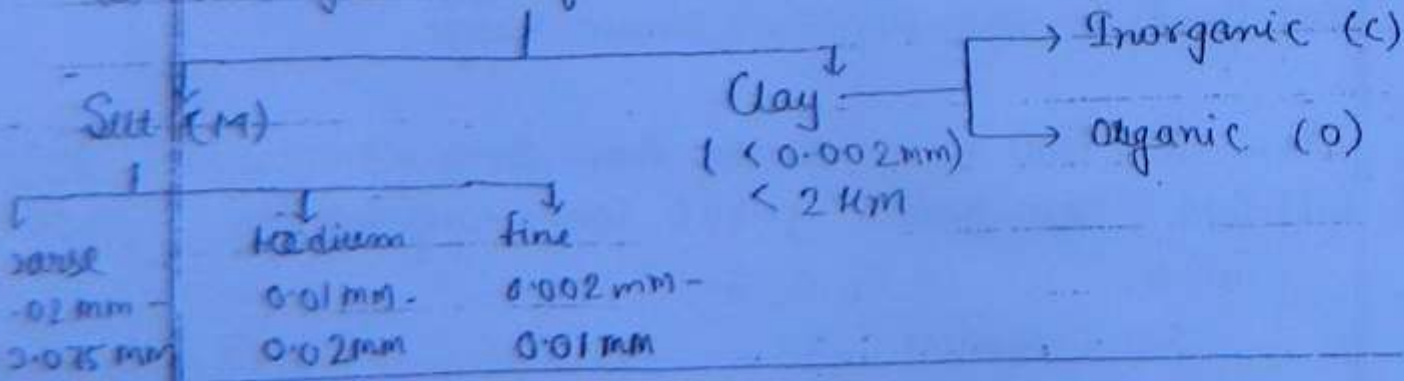
③ GM (Silty gravel) (Silt > Clay Gravel > Sand)

$I_p < 7\%$ fineness > 12% Silt is less plastic than clay)

④ SM (Silty sand) (Silt > Clay Sand > Gravel)

$I_p < 7\%$ fineness > 12%.

(B) Fine Grain Classification (1. cut. of Soil < 0.075 mm is more than 50%)



If soil contains organic compound (mo) $\rightarrow$ organic soil

CO - organic Clay

C - Inorganic Clay

(45)

The fine soils are further classified on the basis of plasticity / liquid limit / compressibility

A. Soils having low plasticity ($LL < 35\%$)

(i) CL - low plastic inorganic clay

(ii) ML - low plastic inorganic silt

(iii) OL - low plastic organic clay

① CL Soils lie above A-line

B. Medium plastic soils ($LL = 35-50\%$)

(i) CI - Medium plastic inorganic clay

(ii) MI " " " silt

(iii) OI Medium " - organic clay

C. High Plastic Soils ($LL \geq 50\%$)

(i) CH - High Plastic inorganic clay

(ii) MH " " " silt

(iii) OH

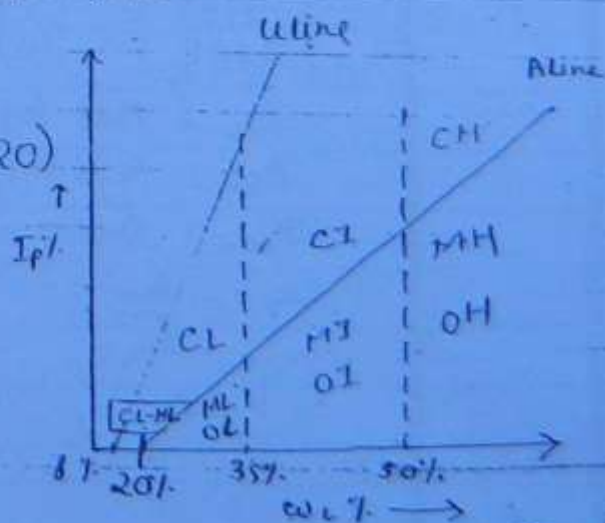
MI, OI, MH, OH located below A-line

A-line -

Eq. of A-line $I_p = 0.73 (w_L - 20)$

$w_L = \%$ liquid limit

$I_p = \%$ PI



NOTE: Eq. of U-line $I_p = 0.9 (w_L - 6)$

(46)

NOTE (1) A line is a boundary & represents difference between clay soils & silty soils. If I_p of soil ($w_L - w_p$) is greater than I_p of A-line then soil will be located above A-line hence will be clay.

(2) If I_p of soil ($w_L - w_p$) is less than I_p of A-line then soil will be located below A-line hence will be silt or organic clay. If it is organic soil then w_L of oven dried soil will be less than 70% liquid limit of air dried soil.

(3) U-line represents upper boundary beyond which no result should lie. If any result is found above U-line then experiments should be repeated.

eg- A soil contains 40% gravel, 50% sand, 10% silt. This soil can be classified as

(a) Silty sandy gravel (b) Silty-gravelly sand $C_u = 10$
 $C_u < 60$

(c) Gravelly silty sand $C_u > 60$ (d) Gravelly silty sand for C_u cannot be determined

SOL	S.NO	Sieve Size	% Retained	% Cum	% Finer
1	1	4.75	40%	40%	60
2	2	0.075	50%	90%	10
3	3	0.002	10%	100%	0

D_{60} = that size below $\leq 60\%$ particles are finer

$D_{60} = 4.75 \text{ mm}$

$D_{10} = 0.075 \text{ mm}$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{4.75}{0.075} = 63.33$$

$C_u > 60$

Ans- (c)

2006

Q. Laboratory sieve analysis was carried out on a soil sample using complete set of the I.S Sieves. Out of 500 gmst soil used in the test 200 gm was retained on 600 μ m sieve, 250 gm was retained on 500 μ m sieve remaining 50 gm was retained on 425 μ m sieve.

(1) The C_u of soil is

(47)

(a) 0.9 (b) 1.0 (c) 1.1 (d) 1.2

(2) The Classification of soil is

(a) SP (b) SL (c) GP (d) GH

No	Sieve	gm wt retained	% wt Rt	% Cum	% N
1	600 μ m	200	40	40	60
2	500 μ m	250	50	90	10
3	425 μ m	50	10	100	0

$$D_{60} = 600 \mu\text{m}$$

$$D_{10} = 500 \mu\text{m}$$

$$C_u = \frac{D_{60}}{D_{10}} = 1.2$$

$$D_{10}$$

$C_u > 4$ gravel
 $C_u > 6$ sand

C_u not greater than 6 so it is poorly graded & size are of sand; for gravel $4.75 \times 1000 = 4750 \mu\text{m}$.

2002

In a soil specimen 70% of particles are passing through 4.75 mm I.S Sieve 40% are passing through 75 μ m I.S Sieve. If its C_u is 6 & $C_c = 2$ then the soil classified as

(a) SP (b) GP (c) SL (d) GH

No	Sieve	% wt retained	% Cum
1	4.75	30%	30
2	75 μ m	60	90

(c)

Clay Minerals & Structures of Soils

Most of the soil are made of primarily 3 Material

↓
Kaolinite

48

Made up of H₂O bond
b/w molecules

H₂O bond is strongest
hence these soil shows
min Change in volume.
On water Adds

↓
Montmorillonite

Water bond b/w
molecules

H₂O bond is weakest

Show max. vol. change
highly compressible
eg. black soil

↓
Illite / polygonite

Tonic bond b/w
molecules

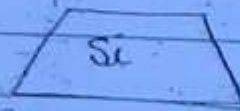
Tonic bond is
medium strong.

Almost all the minerals are made of 2 fundamental units

1. Silica Sheet

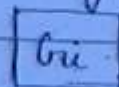
2. Alumina Sheet / Gibbsite Sheet

Silica Sheet is a tetrahedral unit in $\subset$ 4 Oxygen or hydroxyl atom enclose a Silicon atom & form a tetrahedron unit. Its symbolic representation is



Silica Sheet

Gibbsite Sheet is an Octahedral unit in $\subset$ Aluminium / Iron / magnesium atom is enclosed by 6 hydroxyl ions & its symbolic representation is

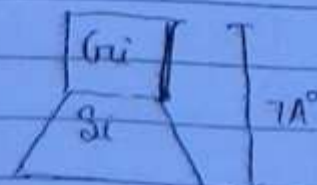


Gibbsite

1. Kaolinite Mineral

One molecule of kaolinite mineral is made of one Silica Sheet & one Gibbsite Sheet

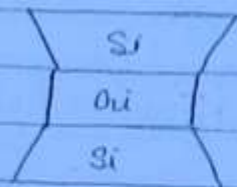
Various such molecules are joined by hydrogen bond. These soil shows least change in vol. due to change in moisture content. eg. China Clay.



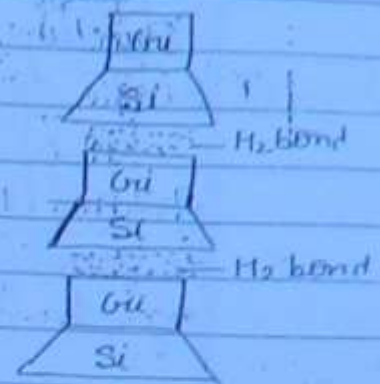
$$1 \text{ Å} = 10^{-10} \text{ m}$$

Montmorillonite Minerals

One molecule of Montmorillonite minerals is made of 2 Si sheets & 1 gibbsite sheet



(49)



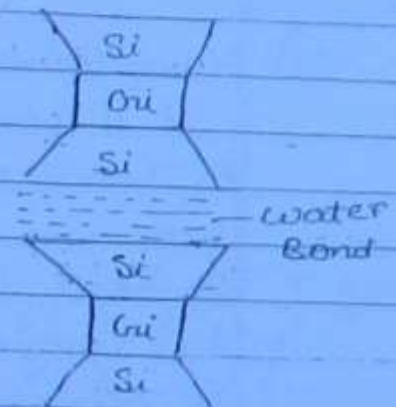
Kaolinite

Sandwiched

Gibbsite sheet is sandwiched between silica sheet. Various such molecules are loosely bonded through water bond. Thickness of water bond may be as high as $200 \text{ \AA}$

eg. black soil & leontomite soils.

These soil shows high vol. change on moisture variation (i.e. large shrinkage & swelling)

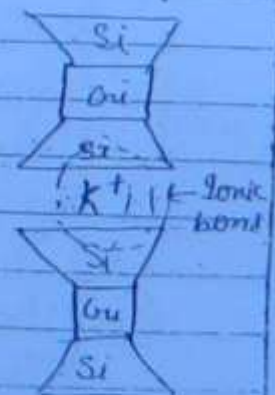


Montmorillonite

Illite Minerals

One molecule of Illite Soil is also made of 2 Silica sheet & one gibbsite sheet but in silica sheet Silicon atom is replaced by Aluminium atom. Various such molecules are joined together by Ionic bond (potassium ion). Ionic bond is neither very weak nor very strong. Hence these shows medium swelling & shrinkage properties

eg. Alluvial soils



Permeability in increasing order = Montmorillonite, Illite, Kaolinite
 Activity in increasing order = Kaolinite, Illite, Montmorillonite

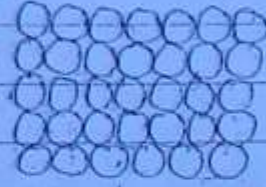
Structure of Soils

(50)

① Single grained Structure

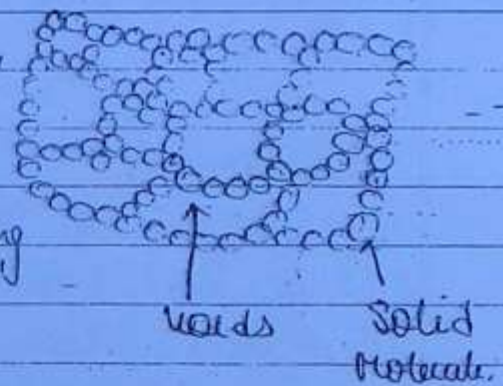
It is found in Coarse grained soils having size greater than 0.02 mm . The major cause of formation is gravity force, Surface electric charges are negligible. eg - Sands & Gravels.

Single grain



② Honeycombed Structure

It is found in soils having particle size between 0.02 mm to 0.0002 mm . Gravity force & surface electric force both are responsible. These soils enclose large volume of voids & form a structure like honey net. When structure is unbroken it has ability to ~~carry~~ bear large load but if structure is destroyed on loading the load carrying capacity is lost.



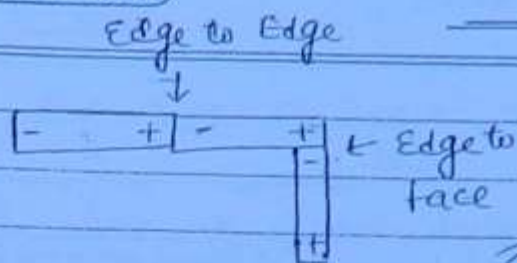
eg - Silt & Coarse Clays

③ Flocculated & Dispersed Structure

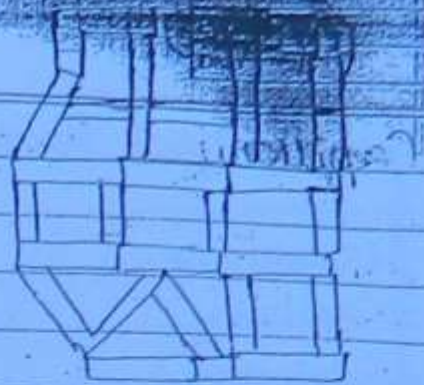
These are found in the soils having size less than 0.0002 mm . The fine particles are flaky / platelets are not spherical. These particles are charged & surface electric forces. The effect of gravity forces is negligible hence surface electric forces play dominant role.

In flocculated structure clay particles are join edge to edge or edge to face through attractive forces. These soils enclose high volume of voids & on remoulding

(1st Choice)



(5)



these may get converted into dispersed structures.

Dispersed structure are formed when clay particles are jointed through face to face through repulsive electric forces.

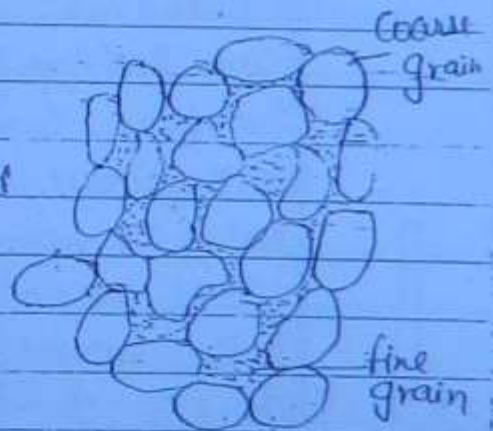


These enclose less void of voids & have structure similar to brick structure.

④ Structure of Composite soil.

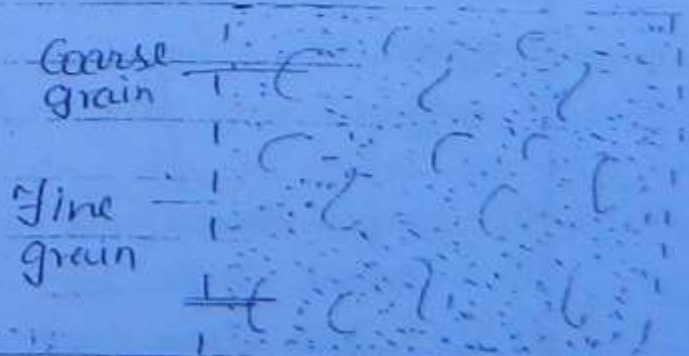
① Coarse grain skeleton

The Coarse grain particle are in contact with each other & voids are filled by fine grained particles. Such soils are less compressible.



② Cohesive matrix

Fine grained particle are in contact with each other & some of the coarse grain particle are also present in the soil mass without contact to each other.



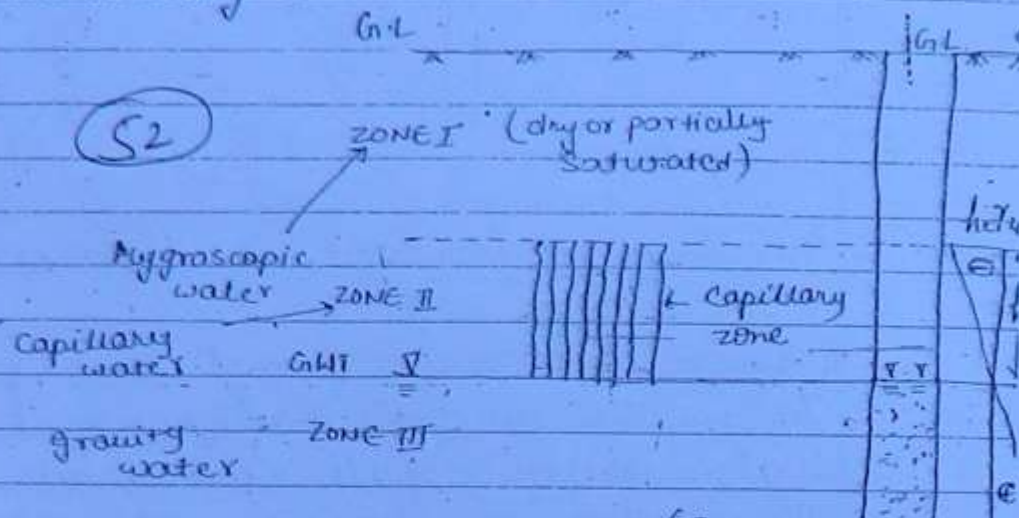
Capillary & Permeability

Zone I - Moist/dry

Zone II - Capillary

Zone III - Submerged

$S = 100\%$



Zone III below Ground water table soil is submerged & fully saturated. Below G.W.T. in zone III pore pressure is hydrostatic (+ve)

Zone II (Capillary zone) In fine soils size of voids is small therefore Capillary ht. is large. In coarse soil Capillary effect is negligible. In zone II soil may be fully or partially saturated depending upon pore size. If degree of saturation is not mention in zone II then this zone should be assume fully saturated.

Zone I - It is under hygroscopic condition & soil is in partially saturated stage or dry stage. If D.O.S is not mentioned then soil may be treated dry.

In Capillary Zone pore pressure is (-ve) or tension due to suction effect & depends upon Capillary height. The Capillary ht is given as h_c

$$h_c = \frac{4\sigma \cos\theta}{\gamma_w d}$$

σ = Surface tension force = force per unit length

θ = Contact angle betⁿ water and Capillary

$\theta \leq 90^\circ$ for clean water & soil force.

Effective pressure. It is grain to grain contact pressure between soil particles. Under dry condition moist (normal) eff. pressure is equal to total pressure but in submerged condition or capillary condition eff. pressure is reduced / increased due to pore pressure. Hence, total pressure / effective pressure / effective stress = total pressure - pore pressure

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

(54)

Total stress - Total stress at any level due to self weight of soil is equal to cut of soil under bulk condition divided by area. If external load is also acting then total stress is self weight effect + applied load effect.

Total pressure at 1-1 $\sigma_1 = \gamma_1 z_1$

Pore " " " " $u_1 = 0$

$$\bar{\sigma}_1 = \sigma_1 - u_1 = \gamma_1 z_1$$

Total pressure at 2-2 $\sigma_2 = \gamma_1 z_1 + \gamma_2 z_2$
 $\gamma_1 z_1 + \gamma_{sat} z_2$

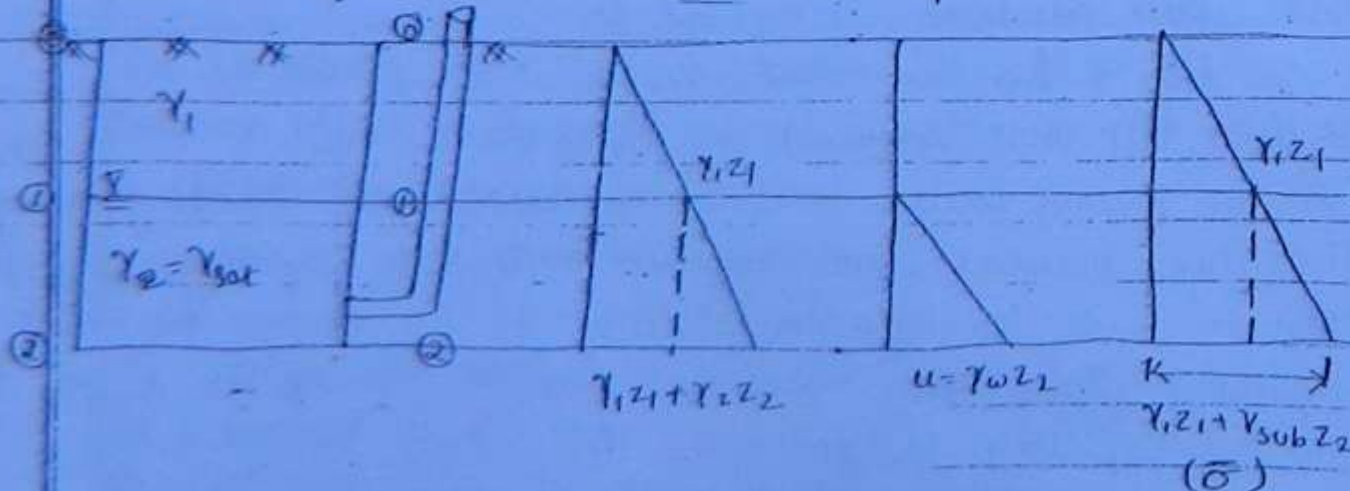
$$u_2 = \gamma_w \cdot h_w \Rightarrow \gamma_w \cdot z_2$$

$$\bar{\sigma}_2 = \sigma_2 - u_2$$

$$= \gamma_1 z_1 + \gamma_{sat} z_2 - \gamma_w z_2$$

$$\bar{\sigma}_2 = \gamma_1 z_1 + z_2 \gamma_{sub}$$

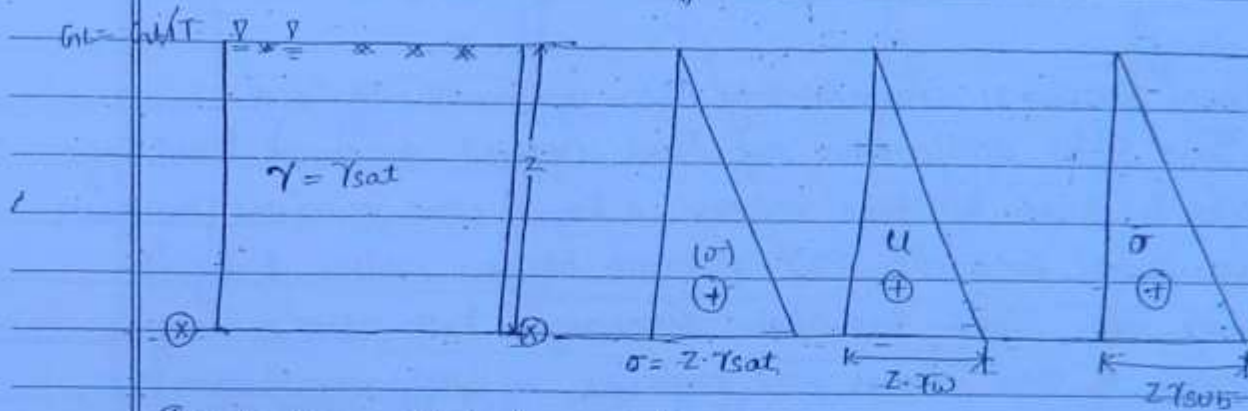
Variation of σ , u , $\bar{\sigma}$ with the depth is linear



NOTE - Shear strength, Consolidation, Settlement, bearing capacity are function of effective stress & not the function of total stress

(SS)

ASE I Plot total stress, eff. stress & pore water pressure diagram when water table is at ground level.



Total stress at X-X = $z \cdot \gamma_{sat}$

Pore pressure = $z \cdot \gamma_w$

$$\sigmā = z \cdot \gamma_{sat} - z \cdot \gamma_w$$

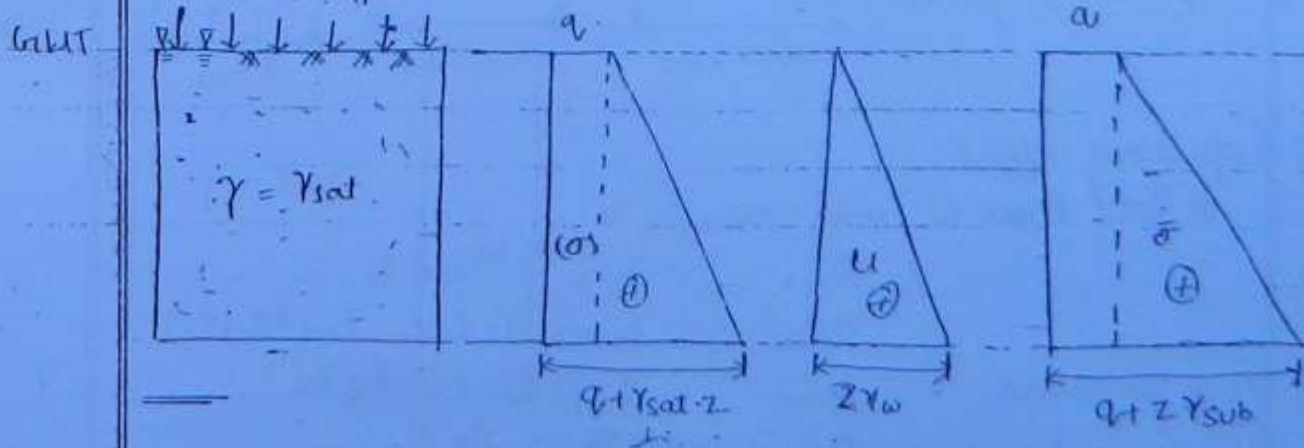
$$z (\gamma_{sat} - \gamma_w)$$

$$\sigmā = z \gamma_{sub}$$

ASE II Effect of Surcharge when water level is at ground level.

If water table is above the ground level then due to further rise in water table $\uparrow$ in total stress & pressure will be equal hence eff. stress will not change.

$q \text{ kN/m}^2$



① When discharge starts from long period of time, then total stress & effective pressure will ↑ & pore pressure remains unchanged.

$$\text{at stress } \sigma_x = q + \gamma_{\text{sat}} \cdot z$$

$$u = 0 \cdot 0 \cdot z \cdot \gamma_w$$

$$\bar{\sigma} = q + \gamma_{\text{sat}} \cdot z - z \cdot \gamma_w$$

$$q + z \cdot \gamma_{\text{sub}}$$

(56)

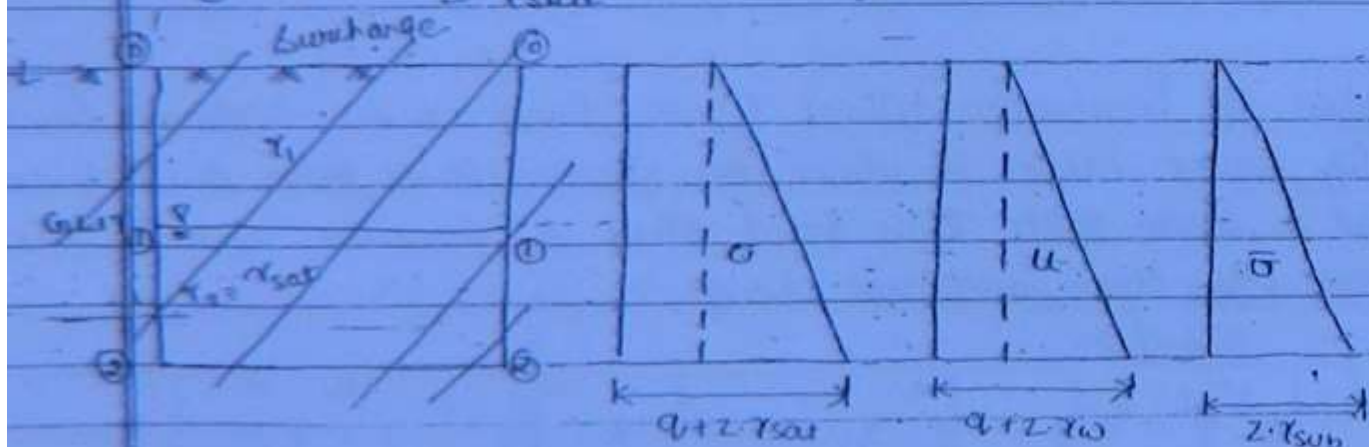
② Just after application of load applied external discharge load is taken by pore water & hence pore pressure increases by equal amount (q) & total stress also ↑ by (q) hence eff. stress remains unchanged but after a long period of time, excess pore pressure is dissipated hence discharge is transferred to grain hence eff. stress increases

$$\sigma = q + z \cdot \gamma_{\text{sat}}$$

$$u = q + z \cdot \gamma_w$$

$$\bar{\sigma} = q + z \cdot \gamma_{\text{sat}} - q - z \cdot \gamma_w$$

$$\bar{\sigma} = z \cdot \gamma_{\text{sub}}$$



III Capillary effect

• Total stress at 0-0 $\sigma_{0-0} = 0$

$$\sigma_{1-1} = \gamma_1 z_1 = \gamma_{\text{sat}} z_1$$

$$\sigma_{2-2} = \gamma_1 z_1 + \gamma_2 z_2 = \gamma_{\text{sat}} (z_1 + z_2)$$

Pore pressure

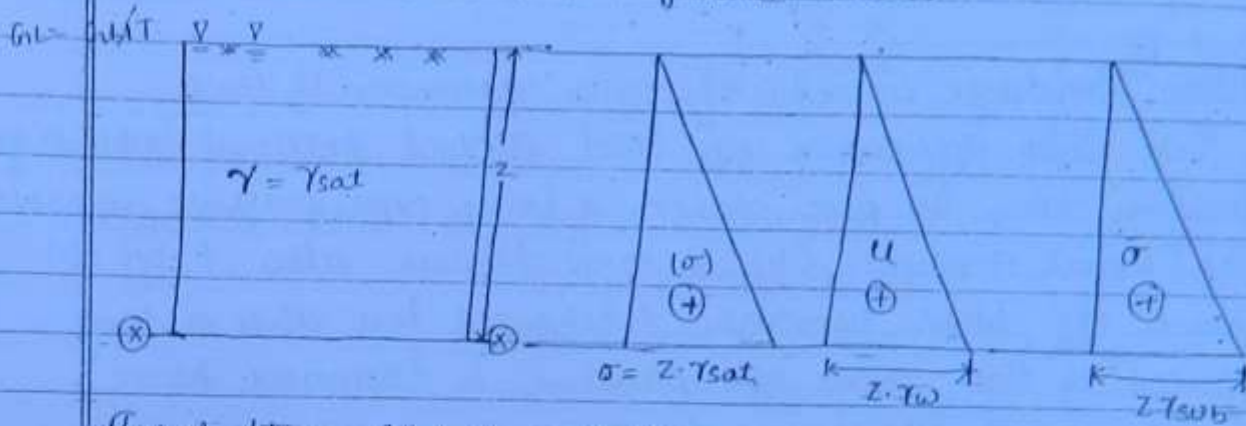
$$u_{0-0} = 0 - z_1 \gamma_w = -h_c \gamma_w$$

$$u_{1-1} = 0 \cdot 0 = 0$$

NOTE - Shear strength, Consolidation, settlement, bearing capacity are function of effective stress & not the function of total stress

(57)

CASE I Plot total stress, eff. stress & pore water pressure diagram when water table is at ground level.



$$\text{Total Stress at X-X} = z \cdot \gamma_{sat}$$

$$\text{Pore pressure} = z \cdot \gamma_w$$

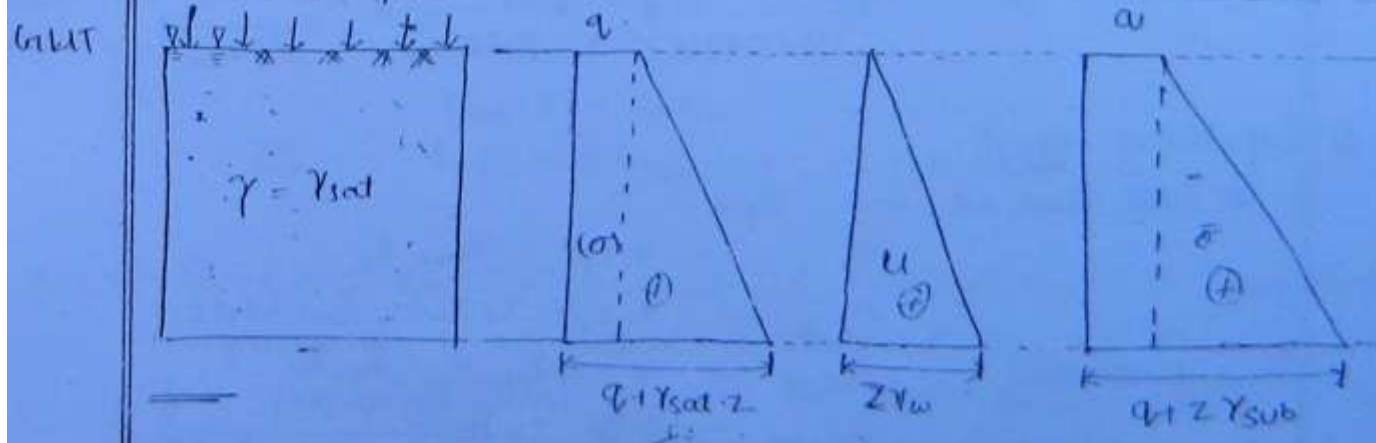
$$\bar{\sigma} = z \cdot \gamma_{sat} - z \cdot \gamma_w$$

$$z (\gamma_{sat} - \gamma_w)$$

$$\bar{\sigma} = z \gamma_{sub}$$

CASE II Effect of Surcharge when water level is at ground level
If water table is above the ground level then due to further rise in water table $\uparrow$ in total stress & pressure will be equal hence eff stress will not change.

$$q \text{ kN/m}^2$$



- ① When surcharge acts from long period of time, then total stress & effective pressure will ↑ & pore pressure remain unchanged.

$$\text{Total stress } \sigma_x = q + \gamma_{\text{sat}} \cdot z$$

$$u = 0 \cdot 0 \cdot z \cdot \gamma_w$$

$$\bar{\sigma} = q + \gamma_{\text{sat}} \cdot z - z \cdot \gamma_w$$

$$q + z \cdot \gamma_{\text{sub}}$$

(58)

- ② Effect of surcharge immediately after application of load

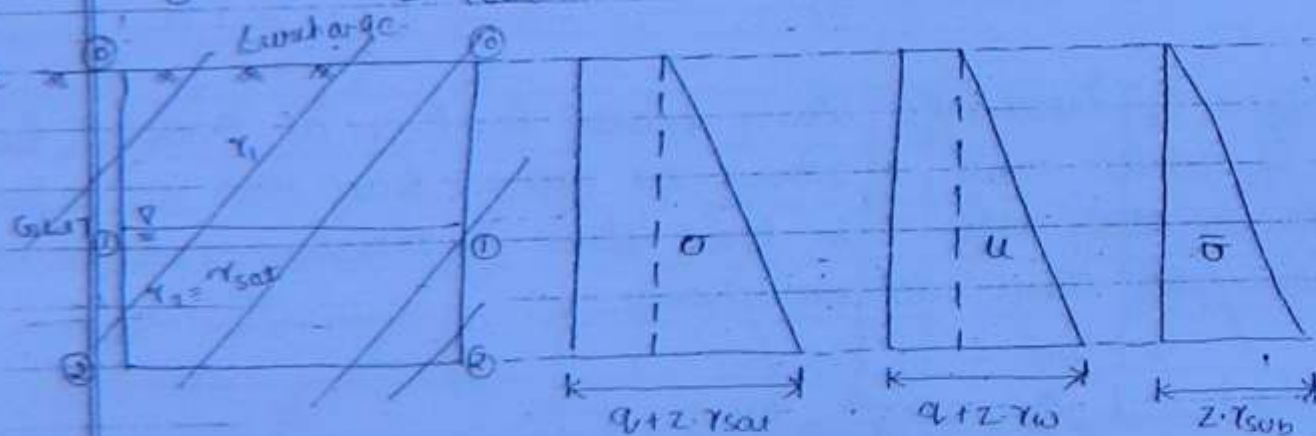
Just after application of load applied external surcharge load is taken by pore water & hence pore pressure increases by equal amount (q) & total stress also ↑ by (q) hence eff. stress remains unchanged but after a long period of time excess pore pressure is dissipated hence surcharge is transferred to grain hence eff. stress increases

$$\sigma = q + z \cdot \gamma_{\text{sat}}$$

$$u = q + z \cdot \gamma_w$$

$$\bar{\sigma} = q + z \cdot \gamma_{\text{sat}} - q - z \cdot \gamma_w$$

$$\bar{\sigma} = z \cdot \gamma_{\text{sub}}$$



Capillary effect

① Total stress at o-o $\sigma_{o-o} = 0$

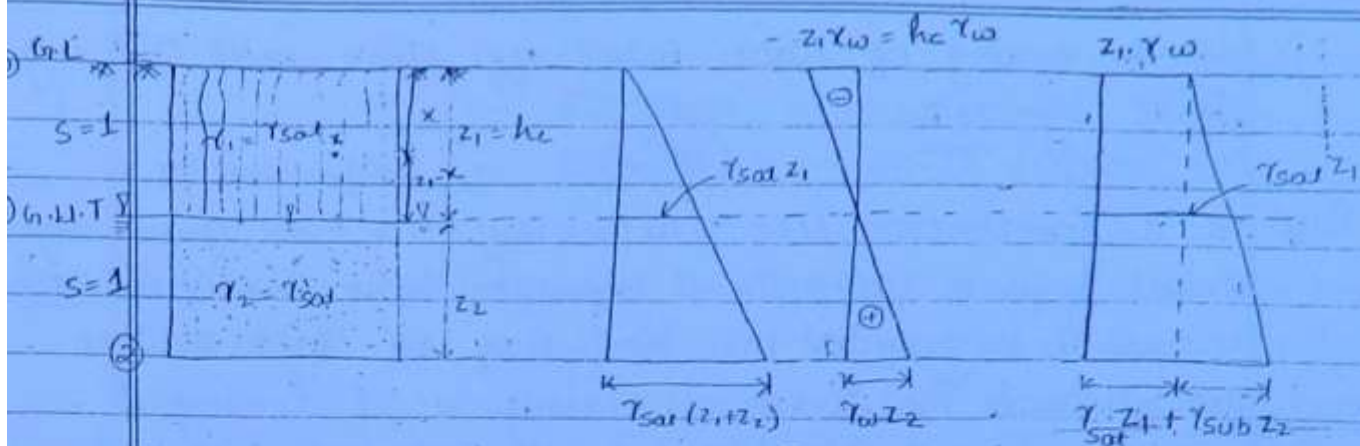
$$\sigma_{1-1} = \gamma_1 z_1 = \gamma_{\text{sat}} z_1$$

$$\sigma_{2-2} = \gamma_1 z_1 + \gamma_2 z_2 = \gamma_{\text{sat}} (z_1 + z_2)$$

Pore pressure

$$u_{o-o} = 0 - z_1 \gamma_w = -h_c \gamma_w$$

$$u_{1-1} = 0 \cdot 0 = 0$$



Pore pressure at 2-2 $u_{2-2} = \gamma_w \cdot z_2$

Effective stress at $\bar{\sigma}_{0-0} = 0 + z_1 \gamma_w$

$$\bar{\sigma}_{1-1} = \gamma_{sat} z_1 - 0$$

$$\bar{\sigma}_{2-2} = \gamma_{sat} \cdot (z_1 + z_2) - \gamma_w \cdot z_2$$

$$\gamma_{sat} z_1 + \gamma_{sub} z_2$$

$$\text{from } \bar{\sigma}_{2-2} = \gamma_{sat} \cdot z_1 + \gamma_{sub} \cdot z_2 - \gamma_w z_1 + \gamma_w z_1$$

$$= \gamma_{sub} (z_1 + z_2) + \gamma_w z_1$$

$$= \gamma_{sub} (z_1 + z_2) + \gamma_w h_c$$

NOTE: The effect of Capillary action i.e. h_c is similar to a surcharge q is equal to $\gamma_w h_c$ acting at the top surface of soil & ground water table present at ground level.

In above fig. at a depth x

The total & effective stress at a depth x in a capillary zone is

$$\sigma_x = z_1 \cdot \gamma_{sat} = x \cdot \gamma_{sat}$$

$$\text{@ } u_x = - \gamma_w (z_1 - x)$$

$$\bar{\sigma}_x = x \cdot \gamma_{sat} + \gamma_w (z_1 - x)$$

$$= \gamma_{sub} \cdot x + z_1 \gamma_w$$

$$\bar{\sigma}_x = h_c \gamma_w + x \gamma_{sub} = q + x \cdot \gamma_{sub}$$

NOTE: If water table is present at ground level & then though the pumping water table is lowered by z_1 then immediately after pumping zone z_1 will be still fully saturated & effect will be similar to that of capillary effect.

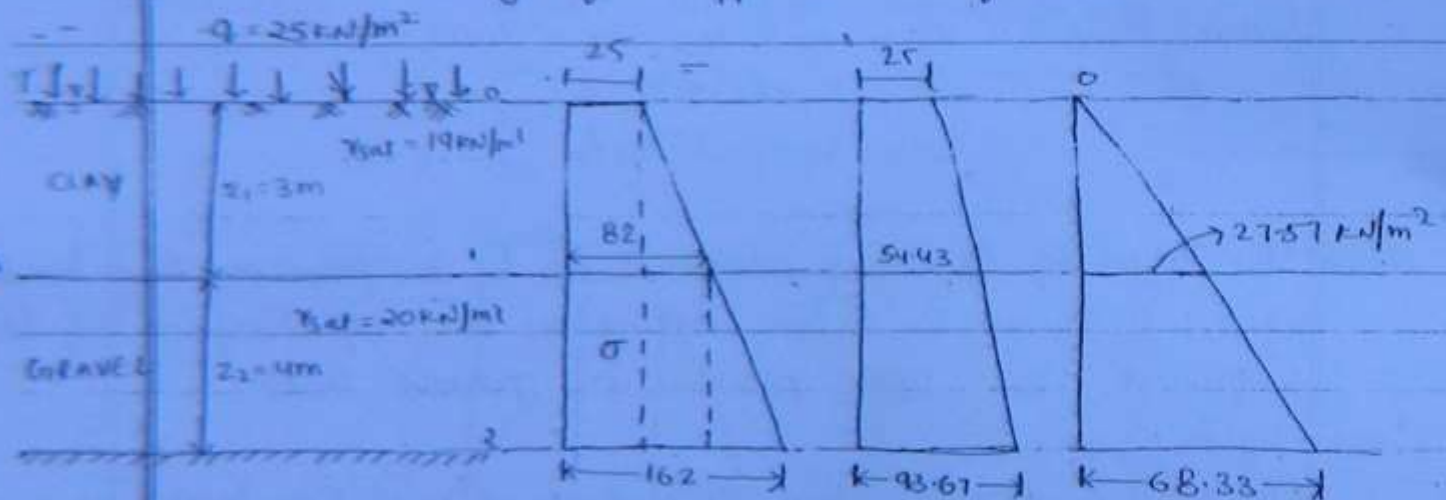
∴ due to loading of water table eff. stress will ↑ at any level by $\gamma_w z$

(68)

layer is bounded by

Ques At a construction site 3m thick clay, 4m thick gravel layer is resting on an impervious rock. A load of 25 kN/m^2 is applied suddenly at the surface of soil. The w.t. is at ground level. The saturated unit wt. for clay & gravel layers are 19 kN/m^3 & 20 kN/m^3 respectively. Draw total, effective, pore stress diagrams immediately after application of load.

Ans In suddenly applied load on submerged soil mass pore pressure will increase by an equal amt. to the applied load immediately after application of load.



① Total stress

$$\sigma_{0-0} = q \text{ kN/m}^2 = 25$$

$$\sigma_{1-1} = q + \gamma_1 z_1$$

$$= 25 + 19 \times 3 = 82 \text{ kN/m}^2$$

$$\sigma_{2-2} = q + \gamma_1 z_1 + \gamma_2 z_2$$

$$= 25 + 19 \times 3 + 20 \times 4 = 162 \text{ kN/m}^2$$

Pore pressure

$$u_{0-0} = q = 25$$

$$u_{1-1} = q + \gamma_w z_1 = 25 + 9.81 \times 3 = 54.43$$

$$u_{2-2} = q + \gamma_w z_1 + \gamma_w z_2$$

$$= 25 + 9.81 (3 + 4) = 93.67$$

$$\sigma_{0-0} = 25 - 25 = 0$$

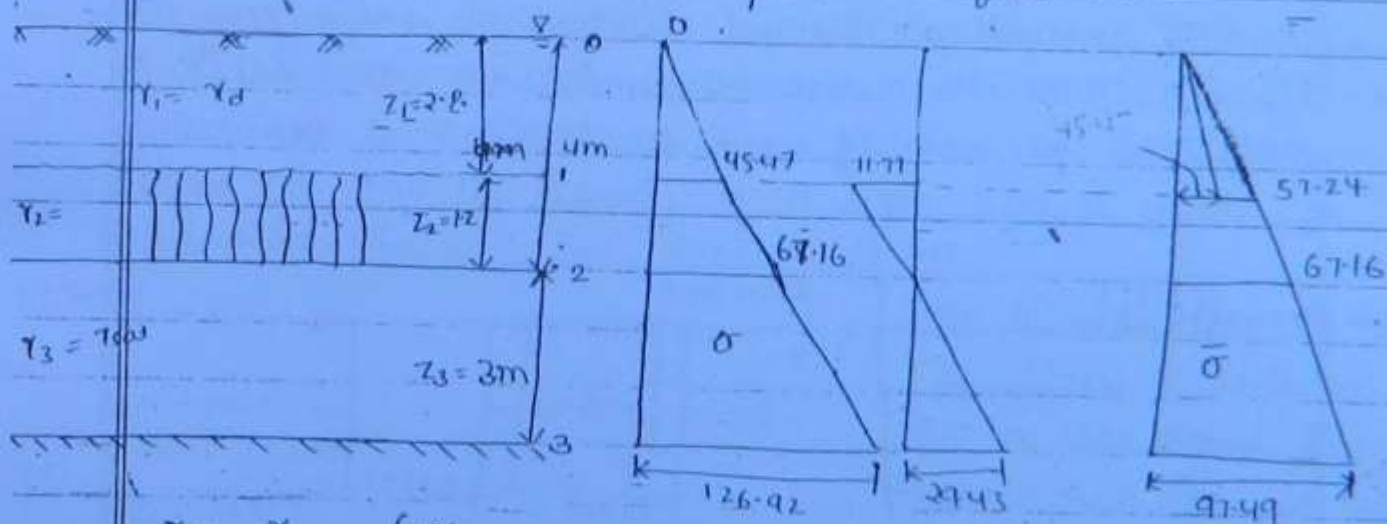
$$\sigma_{1-1} = 82 - 54.43 = 27.57 \text{ kN/m}^2$$

$$\sigma_{2-2} = 162 - 93.67 = 68.33 \text{ kN/m}^2$$

(61)

Ques A granular soil deposit is 7m deep resting over an impervious layer has GWL at 4m below ground level. The deposit has zone of capillary with D.O.S 50%. The capillary ht is 1.2 m above. The GWT If $G = 2.65$ & $e = 0.6$ then plot variation of total, pore & effective pressure with the depths.

sol The soil above the zone of capillary may be considered dry. In granular soils hygroscopic effect is negligible. However in granular capillary ht is less as compared to fine soils & D.O.S is also less as compared to fine soil.



$$\gamma_1 = \gamma_d = \frac{G \gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+0.6} = 16.24 \text{ kN/m}^3$$

$$\gamma_2 = \frac{(G + Se) \gamma_w}{1+e} = \frac{(2.65 + 0.5 \times 0.6) \times 9.81}{1+0.6} = 18.08 \text{ kN/m}^3$$

$$\gamma_3 = \frac{(G + e) \gamma_w}{1+e} = \frac{(2.65 + 0.6) \times 9.81}{1+0.6} = 19.92 \text{ kN/m}^3$$

$$\text{Total stress } \sigma_{0-0} = 0 = 0$$

$$\sigma_{1-1} = Z_1 \cdot \gamma_1 = 2.8 \times 18.08 = 50.624 \text{ kN/m}^2$$

$$\sigma_{2-2} = Z_2 \cdot \gamma_2 + \gamma_1 Z_1 = 45.47 + 12 \times 18.08 = 67.16 \text{ kN/m}^2$$

$$\sigma_{2-3} = \gamma_{s2} + \gamma_{w2} + \gamma_{w2} = 126.92 \text{ kN/m}^2$$

(62)

Pore pressure

$$u_{0-0} = 0$$

$$u_{1-1} = -h_w \gamma_w = -1.2 \times 9.81 = -11.77$$

$$u_{2-2} = 0$$

$$u_{3-3} = 3.8 \gamma_w = 3 \times 9.81 = 29.43$$

Eff. pressure

$$\sigma_{0-0} = 0$$

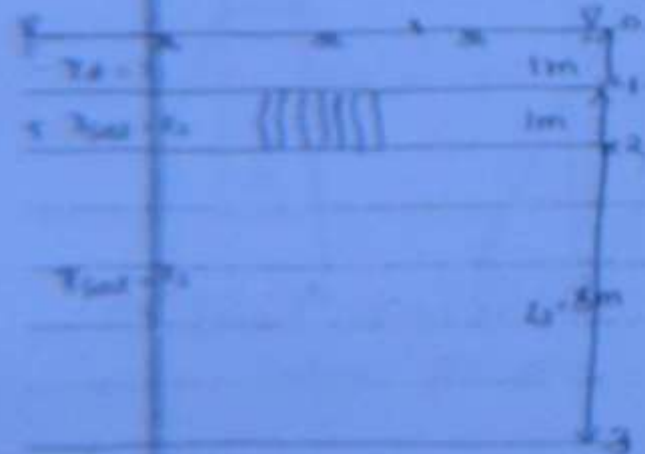
$$\sigma_{1-1} = 45.41 + 11.77 = 57.18$$

$$\sigma_{2-2} = 67.16 - 0 = 67.16$$

$$\sigma_{3-3} = 126.92 - 29.43 = 97.49$$

Ques

Draw total, pore, effective stress diagrams for a sand stratum having a thickness of 10m. The water table is at depth of 2m below the G.L. & there is a capillary rise of 1m above the water table. Assume $\gamma_d = 17 \text{ kN/m}^3$ & 20 kN/m^3 for sand and water respectively.



$$\text{Total stress } \sigma_0 = 0$$

$$\sigma_{0-1} = \gamma_d \cdot 1 = 17 \times 1 = 17$$

$$\sigma_{1-2} = \gamma_d \cdot 1 + \gamma_{s1} = 17 + 21 = 38$$

$$\sigma_{2-3} = 38 + 8 \times 21 = 38 + 168 = 206$$

Pore pressure

$$u_{0-0} = 0$$

$$u_{1-1} = -1 \times \gamma_w = -9.81$$

$$u_{2-2} = 20 \cdot 0.8 = 0$$

$$u_{3-3} = 7.5 \cdot \gamma_w = 8 \times 9.81 = 78.48$$

$$\bar{\sigma}_0 = 0 - 0 = 0$$

$$\bar{\sigma}_{1-1} = 17 + 9.81 = 26.81$$

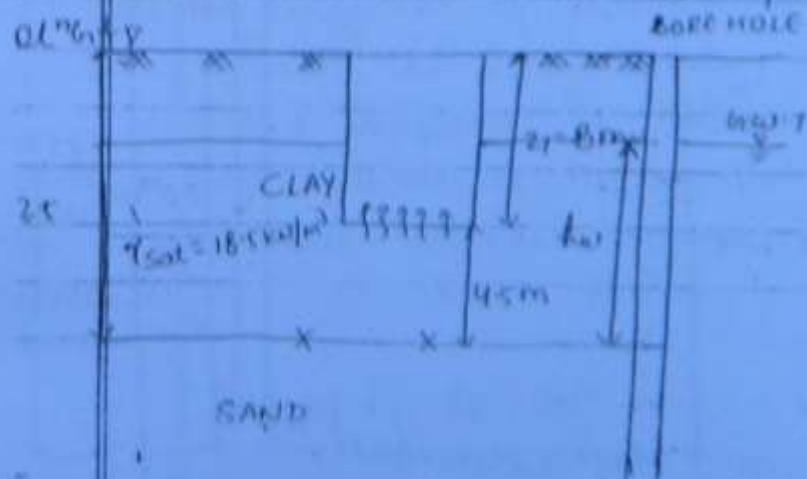
$$\bar{\sigma}_{2-2} = 38 - 0 = 38$$

$$\bar{\sigma}_{3-3} = 206 - 78.48 = 127.52$$

(58)

ATE

Ques In layer of clay of thickness 12.5m is underlying by sand. The saturated unit wt. of clay is 18.5 kN/m^3 . When depth of an open excavation in the clay reached to a depth of 8m the bottom of trench cracked & water started coming from the base. Determine the position of W.T. present above the top of sand if a bore hole is made through the soil strata. Take $\gamma_w = 10 \text{ kN/m}^3$



Due to excavation of trench Eff. stress is reduced over top of the sand layer & when net eff. stress at the level of sand surface is reduced 0 then sand particles will be in floating condition & water will start flowing into the trench from the centre of base of

trench & may carry floating sand particles also

Net eff. stress over sand layer is 0

$$\bar{\sigma} = \sigma - u = 0$$

$$\text{total } \sigma = \gamma_{\text{sat}} \cdot 4.5 = 18.5 \times 4.5$$

$$u = h_w \cdot \gamma_w = h_w \times 10$$

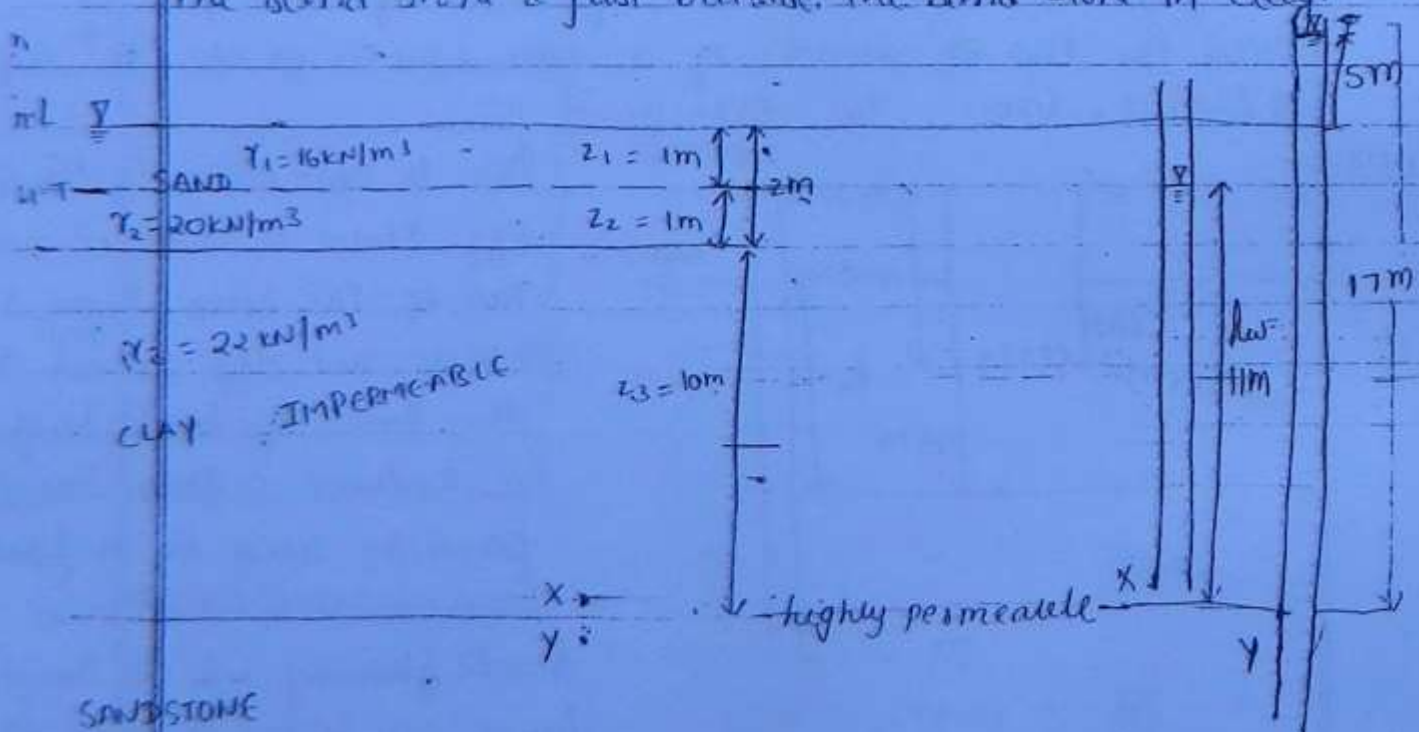
$$\therefore \sigma - u = 0$$

$$18.5 \times 4.5 - 10 h_w = 0$$

$$h_w = 8.325 \text{ m}$$

(64)

Ques The soil profile at the bottom of the valley comprises 2m of sand overlying 10m of clay. The clay layer itself is resting on highly permeable weathered sand stones. Unit wt of sand & above the W.T is 16 kN/m^3 . W.T is 1m below ground level & unit wt of sand below the water table is 20 kN/m^3 . The saturated unit wt of clay is 22 kN/m^3 . If the water table in the sand stone layer is under artesian pressure condition & is equal to a level of 5m above the ground level then plot find total stress, pore pressure, & eff. stress just inside the sand stone & just outside the sand stone in clay.



⑥ Properties of water / fluid - $K \propto \frac{\mu}{\gamma_w}$

fracture
velocity
efficient

μ is inversely proportionally to temp. in liquid.

(65)

⑦ Impurities present in water - Impurity block the voids hence K is reduced

⑧ Adsorbed water - The fine grained soils have a layer of adsorbed water which is strongly attached to the surface of particles which reduces seepage flow.

NOTE - Generally K is represented at 20°C but if test temp is depend then 20°C then temp corrections can be made.

$$K_{20^\circ} = K_T \times \frac{\mu_T}{\mu_{20^\circ}}$$

$K_T =$ Coefficient of permeability at $T^\circ\text{C}$ then

$$\left[\frac{\mu_T}{\mu_{20^\circ}} = R_T = 2.42 - 0.475 \log_e T^\circ\text{C} \right]$$

$$K_{20^\circ} = K_T \times R_T$$

② The Coefficient of permeability (K) depends upon properties of water (μ & γ_w). Another coefficient called Coefficient called Coefficient of absolute permeability can be defined which is independent of fluid properties. Darcy Absolute Coefficient of permeability (k_a)

specific
permeability
or

intrinsic
permeability

$$k_a = K \frac{\mu}{\gamma_w}$$

Unit of k_a is m^2 or cm^2 or mm^2 or darcy

$$1 \text{ darcy} = 0.987 \times 10^{-8} \text{ m}^2$$

if test

cells

Permeability of stratified soils

What is intrinsic permeability of a water saturated medium which has hydraulic conductivity of 15.24 m/day Assume the groundwater is at atmospheric pressure at 20°C

$$\left. \begin{aligned} v_s &= k_p \cdot i \\ v &= k \cdot i \end{aligned} \right\} \quad \frac{k_p}{k} = \frac{v_s}{v} = \frac{1}{n}$$

$$k_p = \frac{k}{n}$$

(66)

Coefficient of percolation is Coefficient of permeability divided by porosity.

Types of Soil	K
Clean gravel	$> 1 \text{ cm/sec}$
Clean sand	$1 \times 10^{-3} \text{ cm/sec}$ to 1 cm/sec
Silt	$1 \times 10^{-7} \text{ cm/sec}$ to $1 \times 10^{-3} \text{ cm/sec}$
Clays	$< 1 \times 10^{-7} \text{ cm/sec}$

Factors affecting permeability

- ① Particle size - $k \propto D_{10}^2$
- ② Void ratio - $k \propto e^2$ [Keeping size of particle is constant]
- ③ Shape of particles - The effect of shape of particles is expressed by specific surface area. $k \propto \frac{1}{S^2}$
 S = Specific surface area
 S = Surface area per unit volume.

For spherical particle $\Rightarrow$ Sp. surface area = $\frac{4\pi R^2}{\frac{4}{3}\pi R^3} = \frac{3}{R} = \frac{6}{D}$

Note In natural soil deposits soils having same void ratio but diff shape of particles these soil will have greater permeability in which particles are angular than those soils which have rounded particle due to better connectivity of pores.

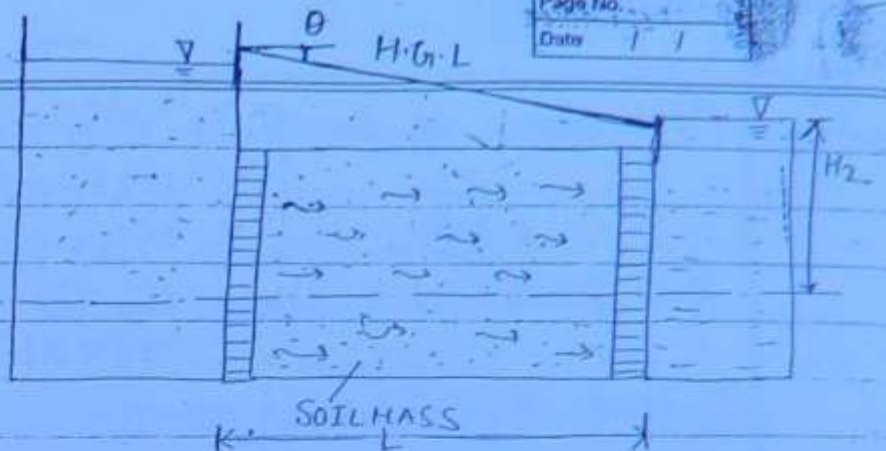
- ④ Degree of saturation - (s) $k \propto S$

In partially saturated soils air voids provide the air logs which hinder in the seepage flow.

- ⑤ Structure of soil - In a stratified soil deposit k is always greater parallel to bedding planes than normal to the bedding planes.

$$H_1 = H_2 = H$$

$$i = \frac{H}{L} \text{ DATUM}$$

 H_1


Q = discharge through the soil mass

$$Q = v \times A \quad (\text{m}^3/\text{sec})$$

v = discharge velocity

A = Area of Soil Mass

$$Q = K \cdot i \cdot A \quad \leftarrow \text{Darcy's equation}$$

flow per
unit time

NOTE

In above case area of flow is considered entire cross-section area of soil hence which is not true area of flow of water. Because water flows only through voids hence actual plane area is equal to area of voids.

The above discharge velocity $v = Ki$ is not true velocity of flow. True velocity of flow will be greater than discharge velocity for equal discharge as above. The true velocity of flow is called seepage velocity v_s .

$$v_s = k_p \cdot i$$

k_p Coefficient of percolation

$$v_s = \frac{Q}{A_v} = \frac{K i A}{A_v} = \frac{v \cdot A}{A_v}$$

$$\Rightarrow \frac{v_s}{v} = \frac{A}{A_v} = \frac{A \cdot L}{A_v L} = \frac{V}{V_v}$$

$$\frac{v_s}{v} = \frac{1}{n} \Rightarrow \boxed{v_s = \frac{v}{n}} \quad \begin{matrix} \text{Imp } n < 1 \\ \text{Hence } v_s > v \end{matrix}$$

Total stress at X $\sigma_x = \gamma_1 z_1 + \gamma_2 z_2 + \gamma_3 z_3$

$$= 16 \times 1 + 20 \times 1 + 22 \times 10$$

$$= 256 \text{ kN/m}^2$$

Pore pressure at X $u_x = \gamma_w \cdot z_w$

$$11 \times 9.81 = 107.91 \text{ kN/m}^2$$

Eff. stress $\bar{\sigma}_x = 256 - 107.91 = 148.09 \text{ kN/m}^2$

Total stress at Y $\sigma_y = \sigma_x = 256 \text{ kN/m}^2$

Pore pressure at Y $u_y = 17 \times 9.81 = 166.77 \text{ kN/m}^2$

$\bar{\sigma} = 256 - 166.77 \text{ kN/m}^2 = 89.23$

Permeability (Hydraulic Conductivity) ^{rate at which water flows through} that soil.

It refers to the water seepage capacity of a soil if pores/voids are interconnected & their size is bigger then soils are more permeable.

Darcy law's

Statement - For laminar condition in a saturated soil mass the discharge velocity is directly proportional to hydraulic gradient (hydraulic gradient is slope of hydraulic grade line / piezometric line)

$$\Rightarrow v \propto i \Rightarrow v = Ki$$

$i =$ hydraulic gradient

$$v \propto \frac{H_L}{L}$$

$i = H_L / L$ (loss of head through L length of soil mass)

$$v = K \frac{H_L}{L}$$

(Homogeneous soil mass)

$K =$ Constant of prop. = Coefficient of permeability

$$K = \frac{v}{i} = \text{m/s or cm/sec or mm/sec}$$

SOL. c density of 998.2 kg/m^3 & viscosity $= 1.002 \times 10^{-3} \text{ kg/m sec}$

$$K_0 = \frac{k \mu}{\gamma_w} = \frac{k \mu}{\rho_w \cdot g}$$

$$\mu = \frac{N \cdot \text{sec}}{m^2}$$

$$\mu = \frac{kg}{m \cdot \text{sec}}$$

$$\frac{\mu}{g} = \frac{kg \cdot m \cdot \text{sec}}{m \cdot \text{sec}^2}$$

$$\mu = \frac{N \cdot \text{sec}}{m^2}$$

$$N = \frac{kg \cdot m}{\text{sec}^2}$$

(69)

$$K_0 = \frac{k \mu}{\gamma_w} = \frac{k \mu}{\rho_w \cdot g}$$

$$\mu = kg/m \cdot \text{sec}$$

$$\frac{\mu}{g} = \frac{kg/m \cdot \text{sec}}{m \cdot \text{sec}^{-2}} = \frac{kg \cdot \text{sec}}{m}$$

$$K_0 = \frac{k (\mu/g)}{\rho_w}$$

$$K = 15.24 \text{ m/day}$$

$$K = \frac{15.24 \times 24 \times 3600}{1000} = \frac{1312.07}{1000} \text{ m/sec}$$

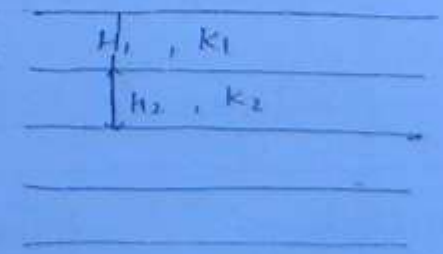
$$K_0 = 1.77 \times 10^{-10} \text{ m}^2$$

Permeability of Stratified Soil

Let there are diff. layers of soils having thickness $H_1, H_2, H_3, \dots, H_n$ & Coefficient of permeability is $k_1, k_2, \dots, k_n$ (hydraulic conductivity) of each layer. let bedding planes are parallel to X-dir or in Y-dir

X & Y $\rightarrow$ horizontal dirn
Z $\rightarrow$ vertical dirn

The equivalent Coefficient of permeability parallel to bedding planes in X or Y dirn is given as



$$K_y = \frac{k_1 H_1 + k_2 H_2 + \dots + k_n H_n}{H_1 + H_2 + \dots + H_n}$$

The equivalent or effective coefficient of permeability normal to the bedding planes is k_z

$$K_x = \frac{H_1 + H_2 + H_3}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \frac{H_3}{k_3}} \quad \text{---} \quad \frac{H_m}{K_m}$$

(70)

NOTE $K_x > K_z$ (i.e. permeability parallel to bedding plane is always greater than normal)

If flow is 2-D in X-Z plane then equivalent coefficient of permeability is geometric average

$$K_e = \sqrt{K_x \cdot K_z}$$

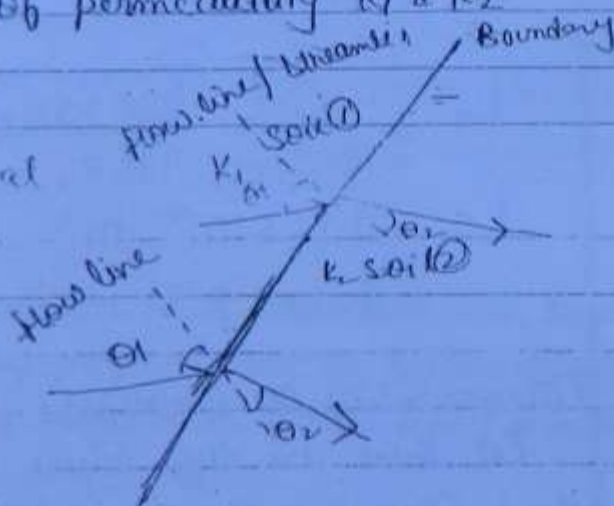
If flow is 3-D $K_e = \sqrt[3]{K_x \cdot K_y \cdot K_z}$

(2) If flow passes through a stratified strata having two soils with coefficient of permeability k_1 & k_2

The flow line makes an angle θ_1 with the normal of interface of two soils within Soil (1) then flow line deflects in Soil (2) & makes angle θ_2 with the normal of interface such that

$$\frac{\tan \theta_1}{\tan \theta_2} = \frac{k_1}{k_2}$$

depends on permeability



Methods to determine coefficient of permeability.

Methods	Field Methods	Indirect Methods
Constant head permeability test (used for coarse soils)	- Pumping out test (area of influence is large) - Pumping in test (area of influence is small)	- Kozney Karman eqⁿ - Allen Hazen eqⁿ - Chaudens eqⁿ

Let at any time $t=0$ the head diff = head loss = h_1
 After time t head diff = h_2 ; let at any stage head diffⁿ
 h which falls by dh in time dt

Let Area of upstream pipe is a & area of bore hole is A
 At any stage when head loss is H

$$Q = \frac{K \cdot A}{L} \cdot h \quad \text{--- (7)}$$

Vol. of water $\leftarrow$ will flow in time $dt =$

$$dV = Q \cdot dt = a(-dh)$$

$$Q \cdot dt = a(-dh)$$

$$\int_0^t \frac{K \cdot A}{L} \cdot dt = \int_{h_1}^{h_2} a(-dh)$$

$$\frac{K \cdot A}{L} \cdot dt = \frac{a \cdot L}{A} \left(\frac{-dh}{h} \right)$$

$$K \int dt = -\frac{aL}{A} \int_{h_1}^{h_2} \frac{dh}{h}$$

$$K \cdot t = -\frac{aL}{A} \left[\log_e h \right]_{h_1}^{h_2}$$

$$K \cdot t = \frac{aL}{A} \log_e \left(\frac{h_1}{h_2} \right)$$

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

NOTE: If in time t head difference falls from h_1 to h_2 & again same time t head difference falls from h_2 to h_3

then $h_1/h_2 = h_2/h_3$

$$h_2 = \sqrt{h_1 h_3}$$

- Falling head permeability test (suitable for fine soils)
- Capillary permeability test (Partially saturated soils)

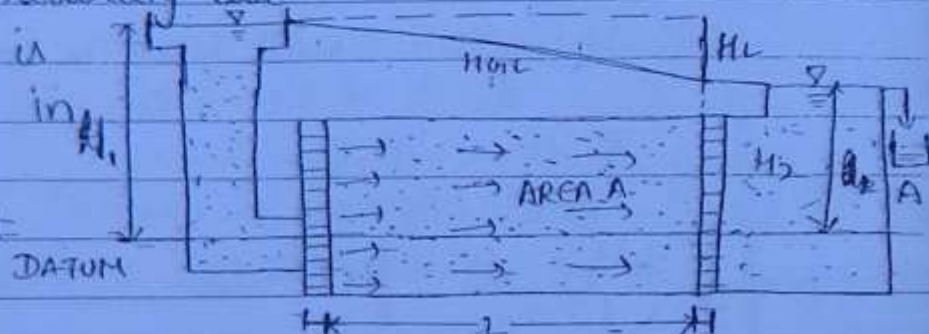
(72)

Permeability
Consolidation eqn
(Suitable when k is very small $< 10^{-6}$ cm/sec)

Constant head permeability test

Volume of the water is collected in time t in the tank A

H_2 & H_1 are constant



Let Q = Quantity of water collected in time t (i.e. vol. of water collected in time t)

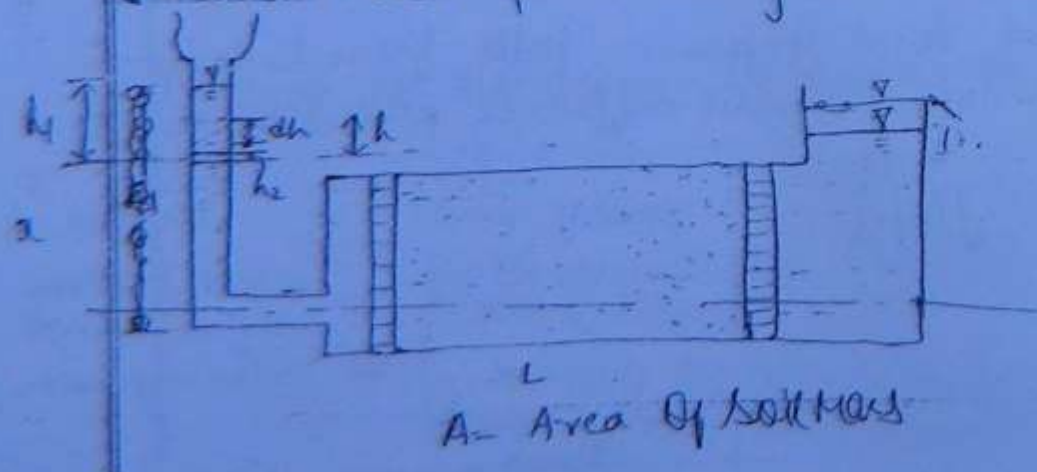
$$q = \frac{Q}{t}$$

$$Q = k i A$$

$$k = \frac{Q}{i \cdot A} = \frac{Q}{t i A} = \frac{Q}{t (H_1/L) A} = \frac{Q \cdot L}{A \cdot t \cdot H_1}$$

$$k = \frac{Q \cdot L}{A \cdot t \cdot H_1} \quad \text{m/sec or cm/sec}$$

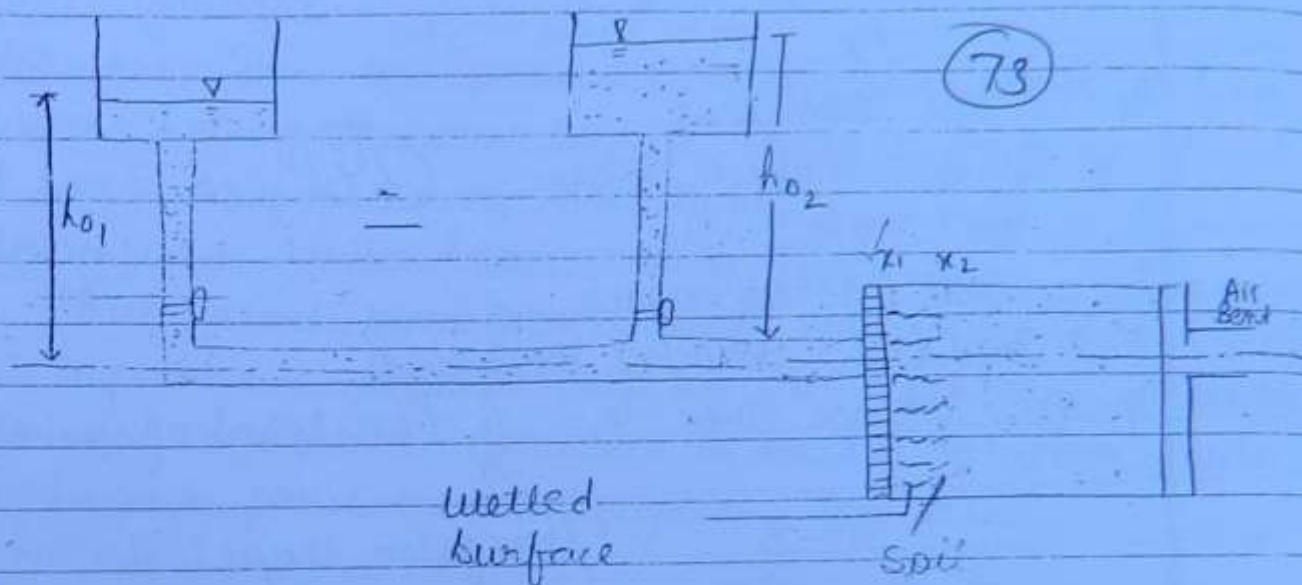
Variable head permeability test or falling head permeability



A = Area of soil mass

Capillary Permeability test

$$i = \frac{h_{o1} + h_c}{x_2 - x_1}$$



Dry or partially saturated soil is placed in a transparent tube of 4 cm dia & 35 cm length. Water is allowed to flow from one end under a constant head h_{o1} & other end is kept open to the atmosphere through air pipe. The time is recorded when water surface in the tube moves from

x_1 to x_2 in time t_1 to t_2

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K}{S \cdot n} (h_{o1} + h_c) \quad \text{--- (1)}$$

h_c = Capillary head

S = degree of saturation of soil sample

n = Porosity

K = Coefficient of permeability

The test is repeated at diffⁿ head say h_{o2} & let water surface moves from x'_1 to x'_2 in time t'_1 to t'_2 hence

$$\frac{x'^2_2 - x'^2_1}{t'_2 - t'_1} = \frac{2K}{S \cdot n} (h_{o2} + h_c) \quad \text{--- (2)}$$

Solving eqⁿ (1) & (2) K & h_c can be found

The biggest advantage of this method is that it can be used

Indirect Methods

① Kozeny Carman's equations

$$K = \frac{1}{K_k} \times \frac{\gamma_w}{\mu} \times \frac{e^3}{1+e} \times \frac{1}{S^2}$$

K_k = Shape Coefficient

S = Specific Surface Area

e = void ratio

γ_w = unit wt. of water

μ = dynamic viscosity coefficient

Specific Surface Area $S = \frac{6}{D}$ (for spherical particles)

$$S = \frac{6}{\sqrt{a \times b}} \quad (\text{irregular shape}) \quad a = \text{Size of Sieve through which Particles Pass}$$

b = Size of sieve through at $\frac{1}{2}$ Particles retained.

② Allen Hazen's equation

$$K = C \cdot D_{10}^2 \quad \text{cm/sec}$$

D_{10} = eff. size in cm

C = Constant $\leq$ is near about 1 (Range 1-150)

This eqⁿ is valid only when particle size is in the range of 0.1 to 3mm (i.e. sands)

③ Kozeny's equation

$$\log_{10}(K \cdot S^2) = a + b \cdot n$$

a & b are constant depending upon type of soil & temp.

n = Porosity

S = Specific surface area

④ Consolidation eqⁿ (K is less than applicable)

$$K = C_v \cdot \gamma_w \cdot m_v$$

C_v = Coefficient of consolidation

m_v = Coefficient of volume compressibility

for partially saturated soil.

(75)

• Well Hydraulics

1. Aquifers - These are porous & permeable formations & have water holding & water seepage capacity both eg- gravels & sands. Aquifers are of three types

① Unconfined aquifer / Non artesian type

- These extend from ground level upto certain depth below & impermeable strata may lie. The G.W.T will be under ordinary hydrostatic condition. At the top of water table hydrostatic pressure is atmospheric (i.e 0).

② Confined aquifer / artesian type

The porous & permeable formation are sandwiched between two impermeable strata at top & bottom. The aquifer is under artesian pressure / ~~or~~ confined pressure.

③ Perched aquifer

If water cavity or porous strata is surrounded by impermeable strata & has water under high confining pressure then it is called perched aquifer. If impermeable part is punctured then water may come out under pressure from perched aquifer. Normally it is found inside the hills & mountains.

② Aquiclude - These are porous but non permeable & can hold water but do not permit flow of water eg- Clay

③ Aquicard - These are less porous & less permeable. Their seepage capacity is low & discharge is small eg- Silt

Aquifuge - These are neither porous nor permeable.
eg- Rocks.

(76)

Specific Yield (S_y)

The specific yield of an unconfined aquifer is the ratio of vol. of water to $\leq$ will flow under gravity conditions in a saturated soil to the total volume of soil mass.

(unconfined aquifers)
$$S_y = \frac{V_{wy}}{V} = \frac{\text{Volume of water yielded under gravity}}{\text{Total vol. of soil mass (aquifer)}}$$

Specific Retention (S_R)

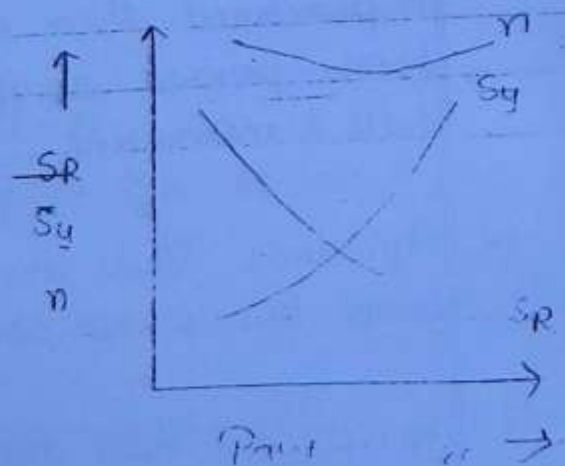
It is the ratio of vol. of water retained in the voids against the gravity flow to the total vol. of aquifers.

$$S_R = \frac{V_{WR}}{V}$$

Note that
$$S_R + S_y = \frac{V_{wy}}{V} + \frac{V_{WR}}{V} = \frac{V_v}{V} = n$$

$$S_R + S_y = n$$

NOTE Coarse soils have greater S_y at lower S_R than fine soils



The phreatic line follows parabolic path and focus of Parabola F is located at toe. The Phreatic line joins d/s surface of dam at P and follows PF. PF is wetted and is called discharge face. This portion of d/s slope must be pitched by stone or concrete in order to prevent toe failure.

Let α is angle of d/s face $\bar{c}$ horizontal

Case I When α is less than 30° ($\alpha < 30^\circ$)

$$q = K \cdot A \sin \alpha \tan \alpha$$

$$A = \frac{D}{\cos \alpha} = \frac{\sqrt{D^2 + H^2}}{\cos^2 \alpha \sin \alpha}$$

(77)

Case II When $\alpha \geq 30^\circ$

$$q = A \cdot K \cdot \sin^2 \alpha$$

$$A = \sqrt{D^2 + H^2} \cdot \sin^2 \alpha$$

Ques A homogenous isotropic earthen dam ^{18m} is 20m high is constructed on an impermeable foundation. K_x & K_y are respectively 4.8×10^{-8} m/s & 1.6×10^{-8} m/s. water level on the reservoir side is 60m from the base & on the d/s side is at base. It is found that there are 4 flow channels & 18 equipotential drops on flownet drawn on the dam secⁿ. estimate the quantity of seepage discharge per unit length of dam.

$$q = K \cdot h \frac{N_f}{N_d}$$

$$K_e = \sqrt{K_x \times K_y} = 0.277 \times 10^{-8}$$

$$q = 0.277 \times 60 \times \frac{4}{18}$$

$$q = 1.108 \times 10^{-8} \text{ m}^3/\text{s}/\text{m}$$

Ques Calculate Seepage through an earthen dam per unit length $\bar{c}$ is resting on an impervious foundation following data -

(i) height of dam = 60m

(ii) u/s Slope = 2.75 to 1
horizontal vertical

d/s Slope - 2.5 horizontal to 1 vertical

tree board = 25m

Grass width = 8m

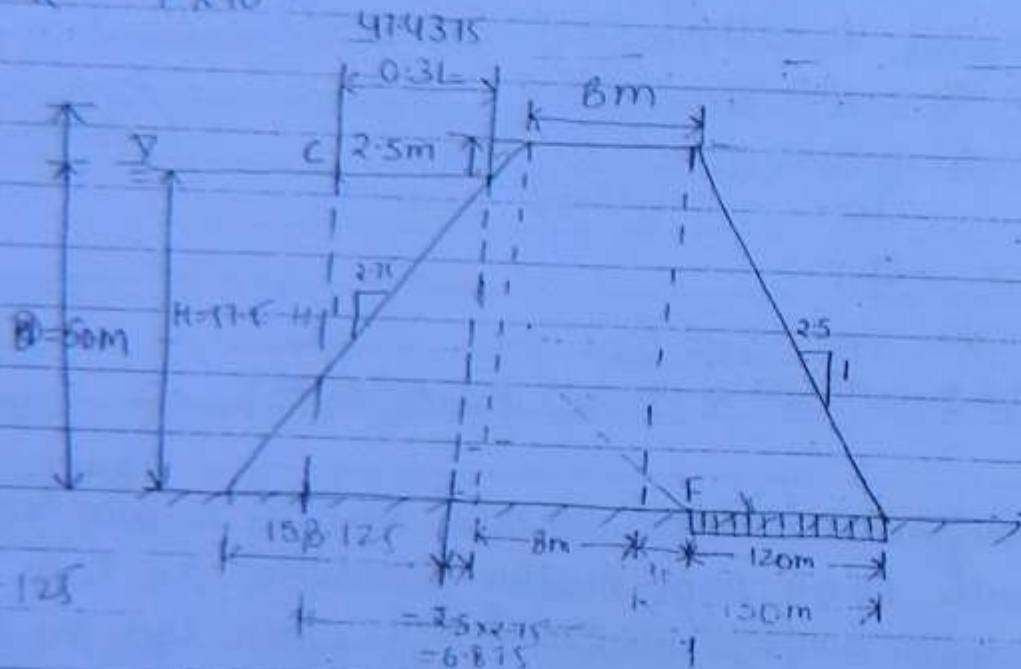
length of drainage filter = 120 m from toe

$$K_{re} = 8 \times 10^{-7} \text{ m/s}$$

$$K_f = 2 \times 10^{-7} \text{ m/s}$$

Sair

$$k = 4 \times 10^{-7}$$



$$S = \sqrt{D^2 + H^2} \quad \text{--- ①}$$

D = horizontal dist of C from F
 $= 90.275 \text{ m}$

$$H = 57.5 \text{ m}$$

$$S = \sqrt{92.275^2 + 57.8^2} - 92.275 = 16.45 \text{ m}$$

$$q_v = k_v S$$

$$= 4 \times 10^{-7} \times 16.4 \times$$

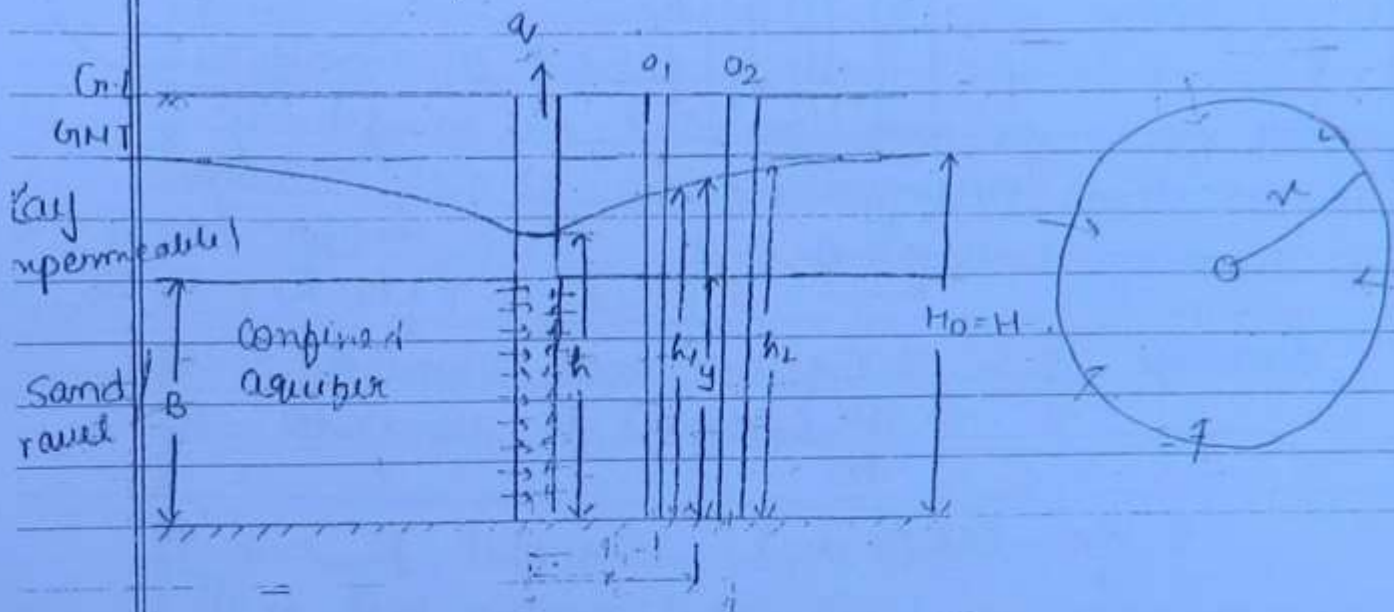
$$= 65.8 \times 10^7 \text{ m}^3/\text{sec}/\text{m}$$

$$k = \frac{2.303 q \log_{10} (R/r)}{\pi [H^2 - h^2]}$$

(79)

② Confined aquifer

In Confined aquifer microscopic flow is ignored (flow through impermeable strata is microscopic)



$$q = k i A$$

$$q = K \frac{dy}{dx} (2\pi x \cdot B)$$

$$q \frac{dx}{x} = 2\pi K B \, dy$$

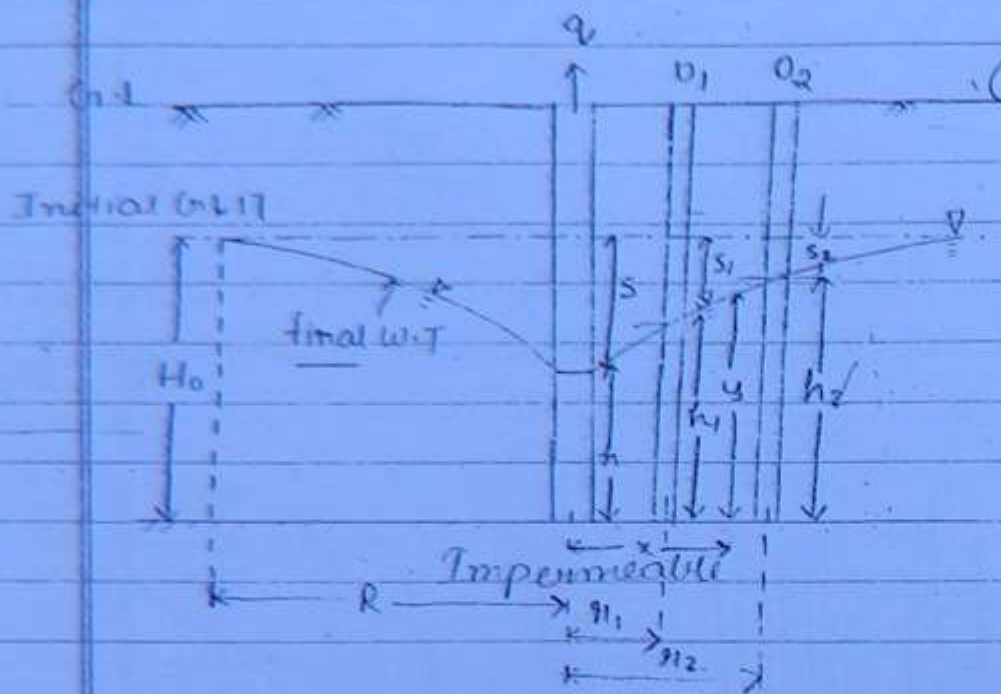
$$2\pi K B \int_{h_1}^{h_2} dy = q \int_{r_1}^{r_2} \frac{dx}{x}$$

$$K = \frac{q \ln(r_2/r_1)}{2\pi B (h_2 - h_1)}$$

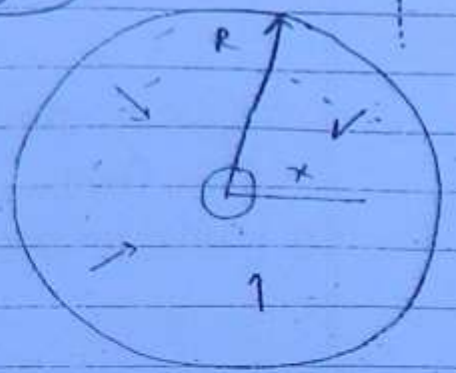
$$K = \frac{2.303 q \log(r_2/r_1)}{2\pi B (h_2 - h_1)}$$

dupits $r_2 = R$, $r_1 = r$
 $h_2 = H$, $h_1 = h$

$$K = \frac{2.303 q \log(R/r)}{2\pi B (H - h)}$$



80



from fig.

$$h_1 + s_1 = h_2 + s_2 = H_0$$

$$h_2 - h_1 = s_2 - s_1$$

$$h + s = H_0$$

$$s = H_0 - h$$

discharge $q = k \cdot i \cdot A$

$$q = k \frac{dy}{dx} (2\pi x \cdot y)$$

$$q \cdot \frac{dx}{x} = 2\pi k (y \cdot dy)$$

$$q \cdot \int_{r_1}^{r_2} \frac{dx}{x} = 2\pi k \int_{h_1}^{h_2} y \cdot dy$$

$$k = \frac{q \ln(r_2/r_1)}{2\pi (h_2^2 - h_1^2)}$$

$$k = \frac{2.303 \cdot q \log(r_2/r_1)}{\pi (h_2^2 - h_1^2)}$$

In Dupuit's theory observation wells are not considered the observation are taken at $r_1 = r_w$ & $r_2 = R$ where r_w is radius of test well & R is radius of influence. Since radius of influence is a guess work hence results are not accurate.

If $r_1 = r_w$ & $r_2 = R$
 $h_1 = h_w$ & $h_2 = H = H_0$

then we get

Storage Coefficient (A):- The water yielding capacity of a confined aquifer can be expressed by Storage Coefficient. It is the vol. of water $\leq$ can be released by a confined aquifer ~~per~~^{per} unit surface area of a aquifer ~~per~~^{per} unit change in head. It is dimensionless. It lies in the range of 5×10^{-5} to 5×10^{-3}

(81)

Coefficient of transmissibility - (T)

It is defined as rate of flow of water through a vertical strip of aquifer of unit width extending to the full saturation depth under unit hydraulic gradient.

$$T = K \cdot H$$

K - hydraulic Conductivity

H - thickness of aquifer

Radius of Influence

It is that distance from the centre of well upto which drawdown is observed in the groundwater table at constant pumping rate. This circular area $\leq$ contribute in the flow to the well is called area of influence. For most of the soils radius of influence is 150 to 300 metres. However its value can be obtained by

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

Where S = drawdown in the well in metre

K = Coefficient of permeability in m/sec

R = Radius of Influence in metre

Pumping out test

Analysis of Aquifers

(82)

(i) unconfined aquifers

There are two theories

1. Dupuit's theory

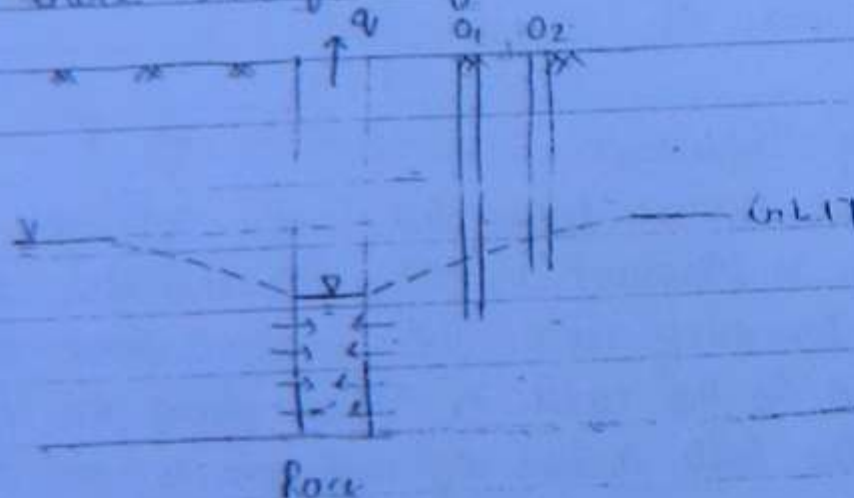
2. Thiem's theory

Thiem's theory is an improvement over Dupuit's theory & is more accurate hence Thiem's theory is widely adopted

Thiem's theory

→ Assumption

- ① Test well penetrates through the entire thickness of aquifer & is resting over an impermeable strata. It means there is no flow from the base of well



- ② Darcy law is valid & flow is laminar
- ③ Flow is radial
- ④ Flow is steady & drawdown is stabilised. Soil is homogeneous & isotropic (means K & T are constant with depth)
- ⑤ Observation wells are made to read the position of water or draw down & are located within the radius of influence

$$H - h = S \text{ (draw down)}$$

$$K = \frac{2303 q \log(R/r)}{2\pi B S}$$

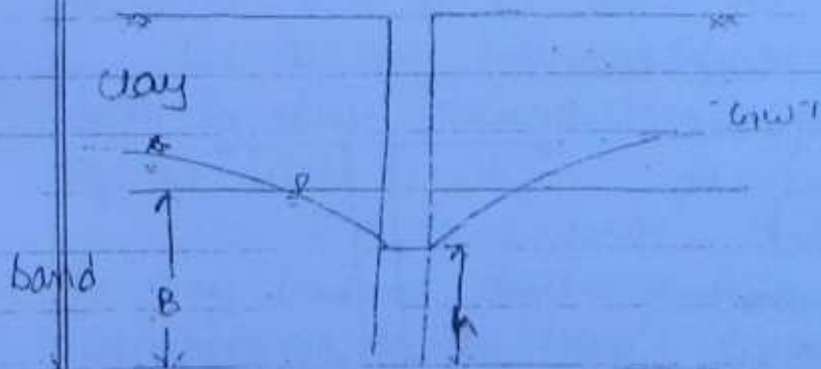
(83)

NOTE- (1) Flow Contribution by Clay into the well is neglected

(2) The condition remain artesian throughout stage of pumping it means steady drawdown level should be always above the layer of Confined aquifer it means $h > B$

(3) Mixed aquifer

In above case if drawdown falls such that steady state drawdown level is within Confined aquifer then near the test well flow will be unconfined but outside the test well flow will be under Confined stage.



$$K = \frac{2303 q \log_{10} \left(\frac{r_2}{r_1} \right)}{2\pi (h_2^2 + B^2 - 2h_1 B)}$$

$$2\pi (h_2^2 + B^2 - 2h_1 B)$$

NOTE- In Confined Aquifer if test well do not penetrate fully

If well is resting just over Confined aquifer then flow is essentially from the base of well & is spherical. The spherical discharge is nearly $1/30$ times of radial discharge in fully penetrating well.



$$Q = K \cdot 2\pi r (H-h)$$

$$Q = K \cdot 2\pi r h S$$

$\left\{ \begin{array}{l} r = \text{radius of well} \\ S = \text{draw down} \end{array} \right.$

(84)

Pumping on test / Slug test

① Open end test

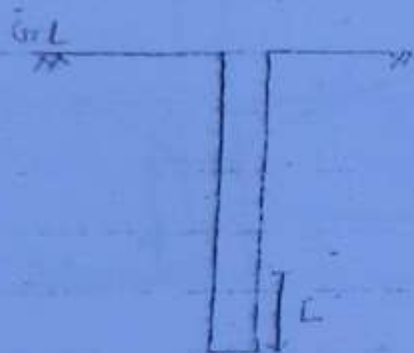
Open end pipe is sunk into the ground & soil in the pipe is removed. The clean water having temp little greater than the ground water temp is added & discharge added is measured through measuring system. The head is kept constant. Let r is the radius of pipe, h is the head under which water is added above the base of pipe.

$h = \text{gravity head} + \text{Pressure head (if any)}$

$$K = \frac{Q}{5.5 r h}$$

② Packers test / Open side test

In this test bottom of the pipe is plugged & side of pipe is perforated. Let L = length of pipe from the base & is perforated.

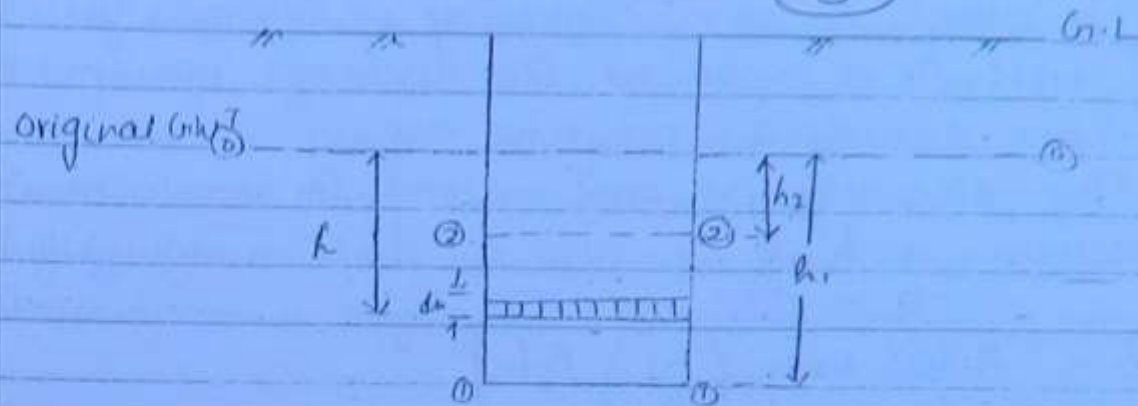


$$K = \frac{Q}{2\pi h L} \log_{10} \left(\frac{L}{10r} \right) \quad (\text{When } L > 10r)$$

$$K = \frac{Q}{2\pi L h} \sin^{-1} \left(\frac{h}{2r} \right) \quad (\text{When } L \leq 10r)$$

Analysis of open well (Recuperation test)

(8)



The flow in open well is essentially from base because sides are lined through brick or stonework. Let original W.T before pumping is at 0-0 & after pumping W.T falls through 1-1 at depth h_1 . When pumping is stop. After pumping is stoped well recuperated & time of recuperation is noted let water table rises from 1-1 to 2-2. 2-2 is at depth h_2 in time T .

Let at any instant of time water level is at depth h which recuperates by dh in time dt . Let area of the well is A

$$q \propto h \Rightarrow q = Ch$$

At any instant discharge into the well is directly proportional to h (head difference). The vol. of water recuperated in time dt

$$-dV = q \cdot dt = A(-dh)$$

$$Ch \, dt = -A \, dh$$

$$\frac{C \cdot dt}{A} = -\frac{dh}{h}$$

$$\frac{C}{A} \int_0^T dt = -\int_{h_1}^{h_2} \frac{dh}{h}$$

$$\frac{CT}{A} = \ln\left(\frac{h_1}{h_2}\right)$$

$$\frac{C}{A} = \frac{2.303}{T} \log_{10} \left(\frac{h_1}{h_2} \right)$$

(86)

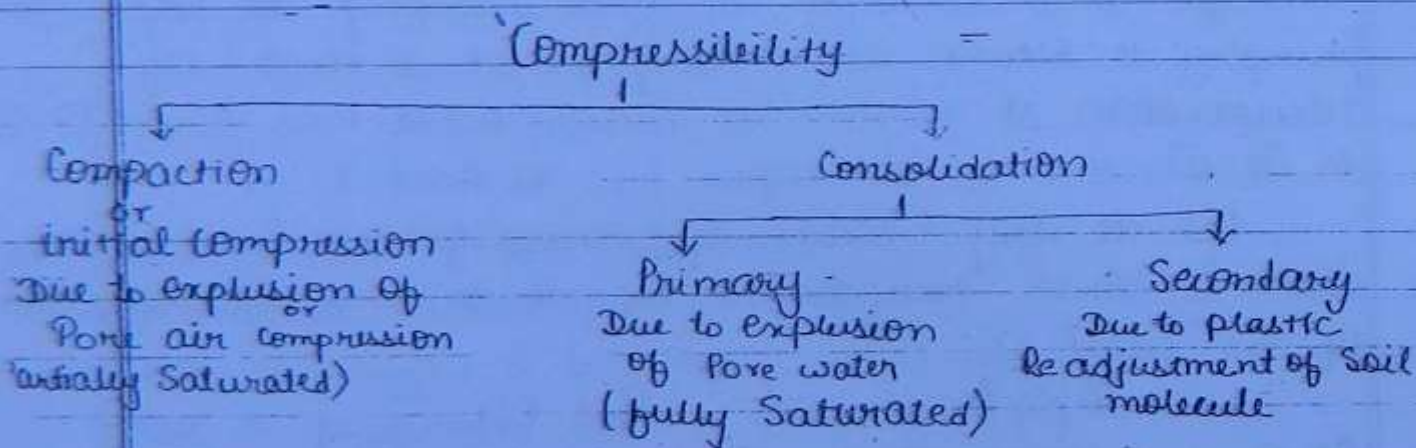
QA is called specific capacity or specific yield of open well. It is expressed as discharge per unit time per metre drawdown per unit area.

The discharge at any instant of time when drawdown is h is

$$Q = Ch$$

$$Q = \frac{2.303}{T} \log_{10} \left(\frac{h_1}{h_2} \right) Ah$$

COMPRESSIBILITY AND CONSOLIDATION



The process of gradual decrease of vol. due to applied load is called Compressibility. The vol. change may occur due to

- ① Compression of pore air
- ② Expulsion of pore air
- ③ Compression of pore water
- ④ Expulsion of pore water
- ⑤ Compression of solids

The decrease in vol. due to compression & expulsion of pore air is called initial compression or compaction where as ↓ in vol. due to expulsion of pore water is called primary consolidation. Note that water & solid molecule are considered

incompressible.

Secondary Consolidation occurs after the completion of primary consolidation when expulsion of pore water is stopped. With the passage of time some of the soils show further $\downarrow$ in vol. due to plastic readjustment of molecules. The sec. Consolidation may be 10 to 20% of total consolidation for highly plastic soils whereas for granular soils secondary consolidation is negligible.

Primary Consolidation

When soil is reached into fully saturated stage then due to applied effective stress over the soil mass excess pore pressure develops & dissipates over the time resulting into consolidation. It is complete when expulsion of pore water is stop. During the process of consolidation soil remains saturated it means decrease in vol of soil mass is equal to expulsion of pore water vol.

Soil mass is considered semi-infinite (area infinitely large & height to be finite) hence the vol change of soil mass is due to height change

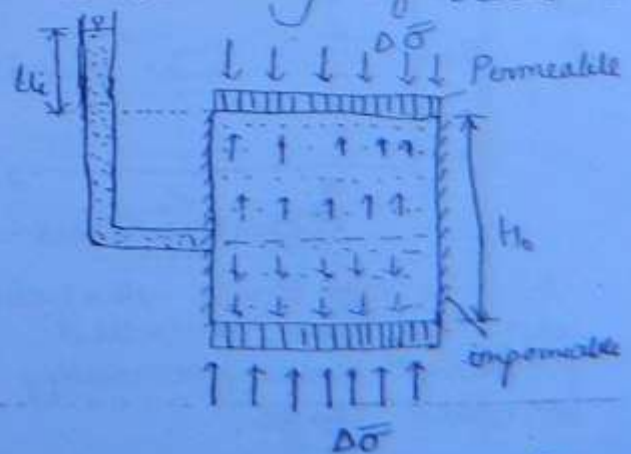
$$\frac{\Delta V}{V} = \frac{\Delta e}{1+e} = \frac{\Delta H}{H}$$

The rate of Consolidation depends on

- ① Rate of loading / applied effective stress
- ② Permeability of soil / size & connectivity of voids
- ③ length of drainage path

$$u_i = \frac{\Delta \bar{\sigma}}{2}$$

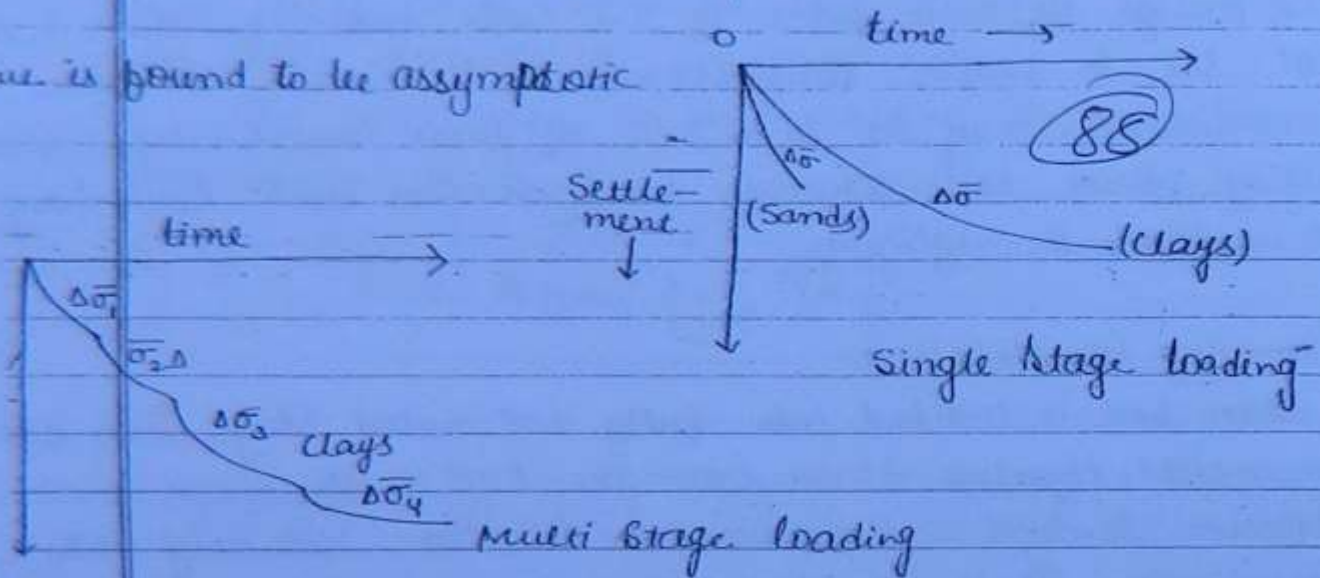
at time $t=0$



Time v/s Settlement Curve in Consolidation

at pore press - $u_i = \Delta \bar{\sigma}$ (at the beginning of consolidation)
 $u_o = 0$ (after completion of consolidation)

u.e. is found to be asymptotic

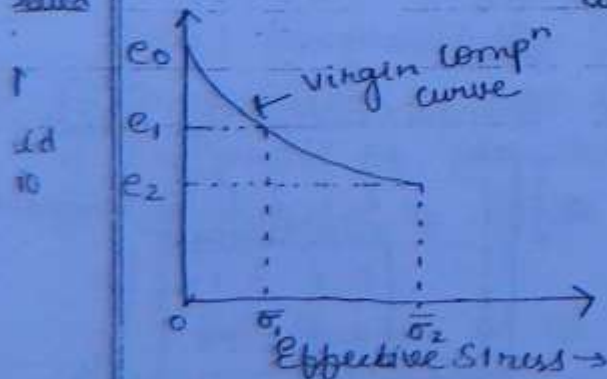


re Due to high permeability, early completion of Consolidation occurs in granular soil but settlement of granular soil is small whereas due to low permeability in clays settlement/Consolidation takes more time but due to large void ratio total settlement is large. Generally Consolidation Settlement of granular soils is neglected.

of void
and
water
ratio

Pressure/Effective stress v/s void ratio Curve

Curve is Asymptotic



to $\uparrow$ in eff. stress void ratio
 decreases. This curve is called
 'virgin compn curve' / normally
 consolidated stage

The Slope of eff. stress v/s void
 ratio curve is called Coefficient
 of Compressibility if taken in
 arithmetic scale.

$$a_v = - \frac{\Delta e}{\Delta \bar{\sigma}}$$

$$a_v = - \left[\frac{e_2 - e_1}{\bar{\sigma}_2 - \bar{\sigma}_1} \right]$$

Δe is -ve [decrease in vol.]

$$\eta_b = \frac{W + e^2 P}{W + P} \quad \text{when } W > eP$$

$$\eta_b = \frac{W + e^2 P}{W + P} - \left(\frac{W - eP}{W + P} \right)^2 \quad \text{when } W \leq eP \quad (89)$$

e = coeff. of restitution b/w hammer & Pile

$e = 0.25$ for wooden pile & cast iron hammer

$= 0.4$ for concrete pile & cast iron hammer

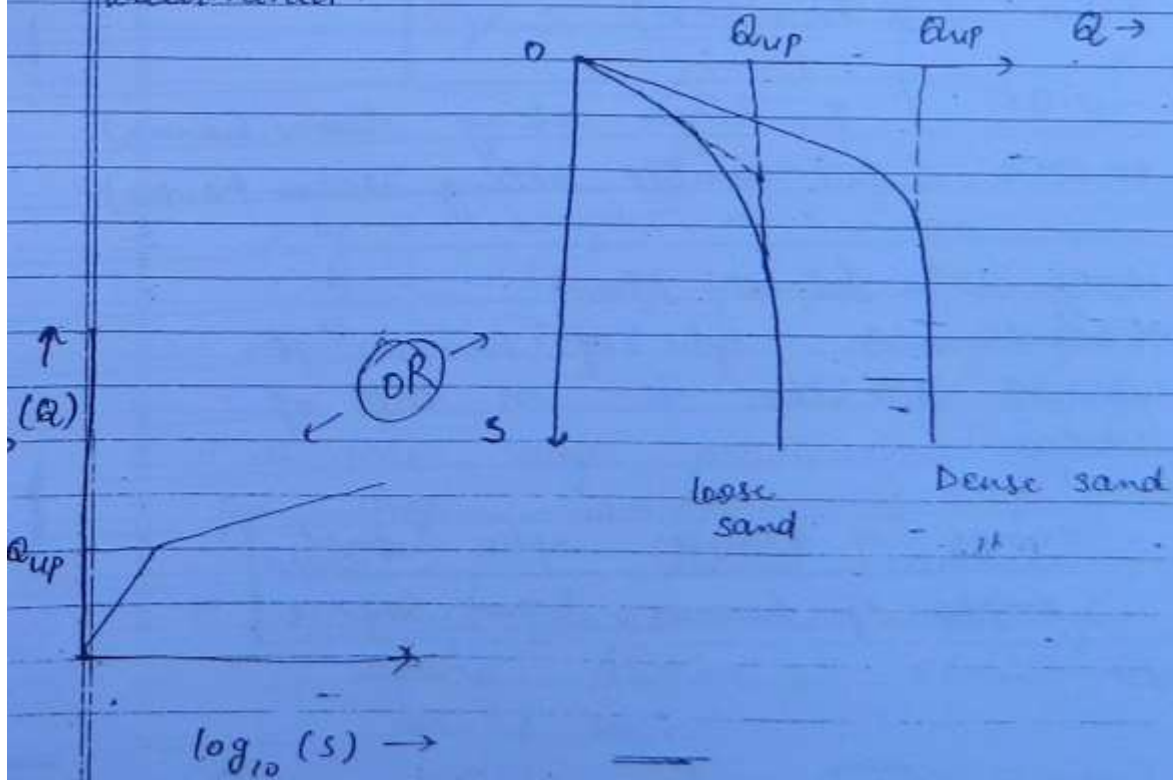
$= 0.5$ for Steel pile & cast iron hammer

S = final set per set of hammer

C = total elastic compression of pile, pile cap & soil

Pile load Test :- The test pile is loaded and load settlement curve is plotted. At failure pile settles at faster rate without much increase in load.

From load settlement curve Q_{up} can be determined.



$c = 0.75 \text{ m}$ (single to double acting steam hammer).

(90)

b) Hikey Formula :- This formula is adopted by IS code.

ultimate load on pile

$$Q_{up} = \frac{\eta_h \cdot \eta_p \cdot W \cdot H}{S + \frac{c}{2}}$$

allowable load on pile

$$Q_{ap} = \frac{Q_{up}}{F.O.S.}$$

$$F.O.S. = 3 \text{ to } 6$$

η_H = efficiency of hammer
= 1 (for drop hammer)

= 0.75 - 0.85 (for single acting steam hammer).

= 0.7 to 0.8 (for double acting steam hammer).

η_b = efficiency of hammer blow
it depends upon type of hammer & type of pile that is on coeff of restitution.

$$\eta_b = \frac{\text{energy of hammer after impact}}{\text{energy of hammer before impact}}$$

This method is suitable for clays not suitable for sandy soil. For sandy soil SPT is best.

(91)

2) Dynamic Method :-

a) Engineering News Record Formula [ENR]

$$\checkmark \left[Q_{up} = \frac{WH}{S+C} \right] \left\{ \begin{array}{l} \text{For drop hammer \&} \\ \text{single acting steam hammer} \end{array} \right.$$

$$Q_{ap} = \frac{Q_{up}}{FOS} = \frac{WH}{FOS(S+C)} = \frac{WH}{6(S+C)}$$

$$\left[Q_{ap} = \frac{WH}{6(S+C)} \right] \quad [FOS = 6]$$

$$\left[Q_{up} = \frac{(W+ap)H}{(S+C)} \right] \left\{ \begin{array}{l} \text{for double acting} \\ \text{steam hammer} \end{array} \right.$$

a = area of piston, p = pressure (steam) acting on piston/hammer

$\checkmark W$ = weight of hammer

$\checkmark H$ = ht. of free fall of hammer (cm)

$\checkmark S$ = final set (avg. penetration of pile per blow of hammer in cm)

S is taken avg. penetration of last 5 blows for drop hammer & avg. of last 20 blows for steam hammer.

C = const. which accounts for energy loss against friction, elastic compression of pile & soil

C = 2.5 cm (for drop hammer).

top to bottom so we will take avg. value of q_{rs} .

$$\begin{aligned} A_b &= \text{bearing area of pile} \\ &= \frac{\pi}{4} D^2 \quad (\text{if pile is circular}) \\ &= B^2 \quad (\text{if pile is square}) \end{aligned}$$

$$\begin{aligned} A_s &= \text{Surface area of pile in contact of soil} \\ &= \pi DL \quad (\text{for circular}) \\ &= 4BL \quad (\text{for square pile}) \\ &= (\text{perimeter} \times \text{length}). \end{aligned} \quad (92)$$

avg. cohesion at the base

$$q_{rb} = q_c \quad \text{for clayey soils.}$$

$$q_{rs} = \alpha \bar{c} + \bar{\sigma}_h \tan \delta \quad [\text{for } c-\phi \text{ soil}]$$

δ = wall friction angle of soil

$\bar{\sigma}_h$ = avg. horizontal thrust = $K \bar{\sigma}_v$

If $\delta = 0$ for clay then

$$q_{rs} = \alpha \bar{c}$$

For clays -

ultimate load on pile

avg. cohesion at the depth of pile

$$Q_{up} = q_c A_b + \alpha \bar{c} A_s \quad \text{imp.}$$

α = Adhesion factor b/w pile & soil

= 0.6 to 0.9 for loose clay

= 0.5 to 0.6 for med^m clay

= 0.3 to 0.5 for dense clay

Methods to determine Pile load Capacity

- ① Static load Method :- It is analytical approach which is suitable for cohesive soils.
- ② Dynamic Method :- It is based on hammer test & it is suitable for dense sands only.
- ③ Field method :- It includes - Q3
 - Pile load test
 - Cyclic Pile load test
 - SPT
 - CPT

Pile load test is most accurate and best method but test pile becomes waste hence it is destructive test.

Cyclic Pile load test has advantage that it gives end bearing resistance or skin friction resistance separately.

① Static Method :-

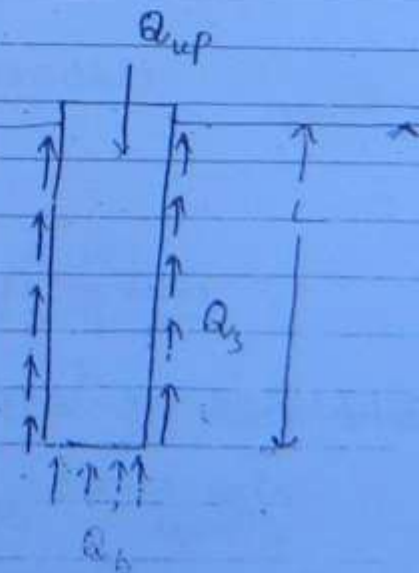
ultimate load on pile

$$Q_{up} = Q_b + Q_s$$

$$Q_{up} = q_b \cdot A_b + q_s \cdot A_s$$

where - q_b = unit bearing resistance

q_s = unit skin friction resistance



Anchor piles :- Below hydraulic structures. - to ~~minimize~~ ^{increase} seepage force to prevent piping failure / quick sand failure. (94)

Fender pile :- These are used to protect water front structures against the impact from waves produced by ships and other floating objects.

Type of pile on the basis of method of installation

1) Driven pile :- precast piles are driven through dynamic action using hammer blow.

2) Bored pile :- precast or cast in situ. Generally driven pile have greater friction resistance than bored pile hence ultimate load capacity of driven pile is ^{always} greater than bored pile.

Allowable Load on the pile

$$Q_{ap} = \frac{\text{Ultimate load on pile}}{F.O.S.}$$

$$= \frac{Q_{up}}{F}$$

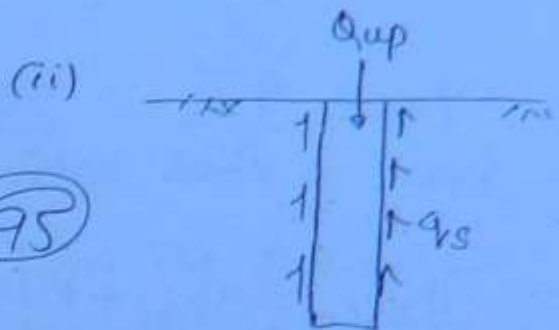
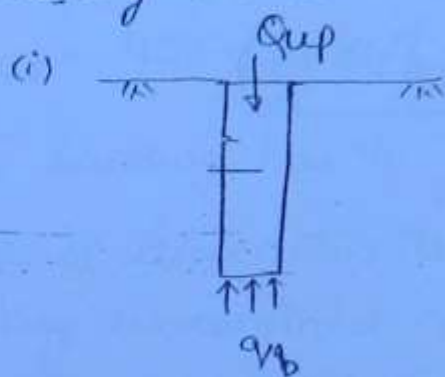
Overall FOS of ^{2.5} 2 to 3 may be used.

$$Q_{ap} = \frac{Q_b + Q_s}{F}$$

$$Q_{ap} = \frac{Q_b}{C_1} + \frac{Q_s}{C_2}$$

Types of pile on basis of their action

- 1) End bearing piles / Bearing piles :- These piles rest over stiff strata and load carrying capacity is due to end bearing action.



(95)

- 2) Friction piles or hanging piles :- These are driven through soft strata which extends to the great depth. The load supporting power is due to skin friction action which acts on the surface area of pile.
- 3) Bearing and friction piles :- Ultimate load carrying capacity is due to end bearing action and skin friction action both.

$$Q_{up} = Q_b + Q_s$$

$$Q_{up} = q_b A_b + q_s A_s$$

Types of pile on basis of their function

- (i) Compaction pile :- To compact loose sands, to increase bearing capacity. ex. sand piles.
2. Uplift pile :- Tension pile :- To anchor structures.

$$Q_{nu} = 50 \times 5.67 \left(1 + \frac{Df}{70} \right)$$

$$= 283.5 \left(1 + \frac{Df}{70} \right)$$

(96)

$$Q_{ns} = \frac{Q_{nu}}{FOS} = \frac{283.5 \left(1 + \frac{Df}{70} \right)}{3}$$

$$Q_{safe} = Q_{ns} + r \cdot Df$$

$$= 94.5 \left(1 + \frac{Df}{70} \right) + 19 \times Df$$

$$94.5 \left(1 + \frac{Df}{70} \right) + 19 \times Df = 140$$

$$\underline{Df} = 2.2385 \text{ m} \quad \underline{\text{ans.}}$$

$$C_w = 0.5 \left[1 + \frac{3}{3+1.5} \right] = 0.83$$

$$q_{\text{anet}} = 0.41 \times 33.4725 \times 40 \times 0.83$$

$$q_{\text{anet}} = 455.62 \text{ kN/m}^2$$

(97)

Question A building has to supported over a raft foundation of dimension $14 \text{ m} \times 21 \text{ m}$. The sub soil is clay having unconfined comp. strength of 100 kN/m^2 . The pressure on the soil due to weight of building and applied load is 140 kN/m^2 at the base of raft. What should be depth of raft which should be provide with FOS 3 against shear failure. Use γ for clay 19 kN/m^3 and adopt skempton's method.

Ans $B = 14 \text{ m}$, $L = 21 \text{ m}$, $D_f = ?$

$$C = \frac{q_u}{2} = \frac{100}{2} = 50 \text{ kN/m}^2$$

applied pressure at base of footing $\leq$ safe bearing capacity.

$$140 \leq q_{\text{safe}}$$

Skempton's method

$$q_{nu} = C N_c$$

$$N_c = 5 \left[1 + \frac{0.2B}{L} \right] \left[1 + \frac{0.2D_f}{B} \right]$$

$$= 5 \left[1 + \frac{0.2 \times 14}{21} \right] \left[1 + \frac{0.2D_f}{14} \right]$$

Corrected Values for Overburden pressure

N

$$N'_0 = N_0 \times \frac{350}{\bar{\sigma}_1 + 70}$$

$$\begin{aligned} \text{YDF} \\ \bar{\sigma}_1 &= 18 \times 1.5 \\ &= 27 \end{aligned}$$

$$N'_0 = 10 \times \frac{350}{27 + 70} = 36.08$$

(98)

$$N'_0 = 15 \times \frac{350}{3 \times 18 + 70} = 42.33$$

$$N'_0 = 20 \times \frac{350}{(3 \times 18) + (1.5 \times 10) + 70} = \frac{4000}{50.35}$$

$$N'_0 = 25 \times \frac{350}{(3 \times 18) + (1.5 \times 10) + 1.5 \times 8 + 70} = 57.94$$

Corrected value for dilatancy

$$N''_0 = 36.08 \quad [\text{No corr}^n \text{ needed because WT is below test level}]$$

$$N''_0 = 15 + \frac{[42.33 - 15]}{2} = 28.66$$

$$N''_0 = 15 + \frac{[50.35 - 15]}{2} = 32.68$$

$$N''_0 = 15 + \frac{[57.94 - 15]}{2} = 36.47$$

Corrected average SPT no.

$$N = \frac{36.08 + 28.66 + 32.68 + 36.47}{4} = 33.4725$$

$$e_0 = \frac{0.40 \times 2.7}{1} = e_0 = 1.08$$

$$\gamma_{sat} = \frac{G + e}{1 + e} \gamma_w = \frac{2.7 + 1.08}{1 + 1.08} \times 9.81 = 17.82 \text{ kN/m}^3$$

$$\gamma' = (17.82 - 9.81) = 8.01 \text{ kN/m}^3$$

(99)

Initial effective overburden pressure

$$\bar{\sigma}_0 \text{ at } \alpha = \sigma_0 - u_0 = 2.5 \gamma_{sat} - 2.5 \gamma_w$$

$$\bar{\sigma}_0 = 2.5 \times 8.01 = 20.025 \text{ kN/m}^2$$

$$q_u = 1.3 C N_c + \gamma' D_f N_q + 0.4 \gamma' B N_\gamma$$

$$q_u = 1.3 \times 50 \times 6.9 + 8.01 \times 1.5$$

$$q_u = 460.515 \text{ kN/m}^2$$

$$q_{nu} = 460.515 - 8.01 \times 1.5$$

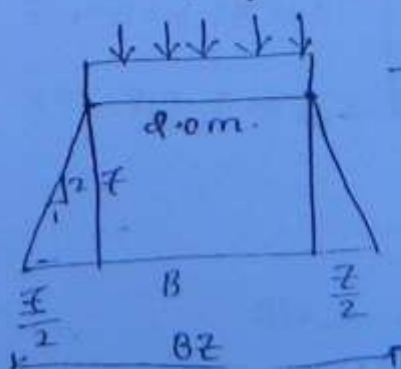
$$q_{nu} = 448.5$$

$$q_{ns} = \frac{448.5}{3} = 149.5$$

$$q_{safe} = q_{ns} + \gamma' D_f = 149.5 + 8.01 \times 1.5$$

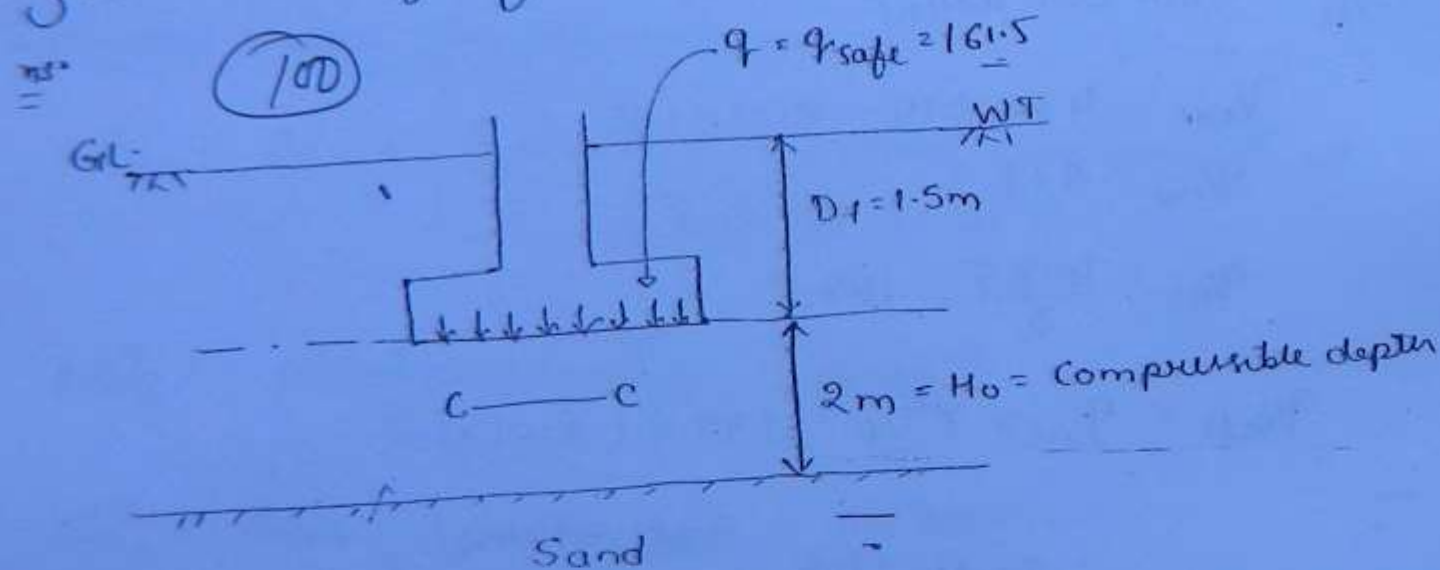
$$= 161.5 \text{ kN/m}^2$$

$$q_{safe} = 161.5$$



$$\alpha + \beta = 3$$

a soft clay with its base at a depth of 1.5m from the ground surface. If clay ^{thickness} 3.5m thick and is underlay by a firm sand. Clay has L.L. of 30%, S.L. 2.7 and Degree of Saturation = 100%, $w_c = 40\%$, unconfined compressive strength 100 kN/m^2 and $\phi = 0$. It is known that clay is normally consolidated then compute the settlement which will be resulted if load intensity = safe bearing capacity is allowed at the base of footing. Natural w_c is close to G.L. and for given condition bearing capacity factor are $N_c = 6.9$, $N_q = 1$, $N_r = 0$. $\phi = 0$, $\delta = 30^\circ$. Assume load distribution vertical to horizontal below footing.



$$\Delta H = \frac{C_c H_0}{1 + e_0} \log \left[\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right]$$

$$C_c = 0.009 (w_c - 10)$$

$$\therefore 0.009 (30 - 10) = 0.18$$

$$q_{nu} = 439.065$$

$$q_{ns} = \frac{q_{nu}}{F.O.S} = \frac{439.065}{2.5} = 175.626$$

$$q_{safe} = q_{ns} + \bar{\sigma} = 175.626 + 12.285$$

$$= 187.911 \text{ kN/m}^2.$$

(10)

Case-II when WT at 3m from G.L.

$$R_y^x = 0.5 \left[1 + \frac{z_y}{B} \right] = 0.5 \left[1 + \frac{1.5}{2.5} \right] = 0.8$$

$$R_q^x = 1$$

$$q_u = 0 + 18 \times 1.5 \times 20.3 \times 1 + 0.4 \times 18 \times 2.5 \times 19.7 \times 0.8$$

$$q_u = 831.78$$

$$q_{nu} = q_u - \gamma D_f = 831.78 - 18 \times 1.5 = 804.78$$

$$q_{ns} = \frac{q_{nu}}{F.O.S} = \frac{804.78}{2.5} = 321.912$$

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

$$= 321.912 + 18 \times 1.5$$

$$= 348.912 \text{ kN/m}^2$$

provided.

1. If soil is loose, then to improve bearing capacity and to compact loose sand sand piles may / compaction pile should be used. (102)

If piles are driven in clays, then structure must not be loaded immediately after driving the piles because while driving the piles sensitivity of clay may occur and shear strength may be lost.

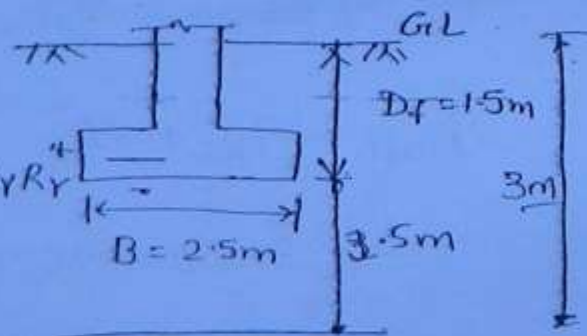
The pile driving process should start from centre and gradually outwards so as to minimise resisting of driving.

2. The subsoil at a building site consist of medium sand with $\gamma_{sat} = 18 \text{ kN/m}^3$, $c = 0$, $\phi = 32^\circ$ and WT at G.L.

2.5 m square footing is to be placed at 1.5 m below G.L. compute safe bearing capacity of footing.

What will be safe SBC if WT goes 3 m below G.L. when that $\phi = 32^\circ$, $N_q = 20.3$, $N_r = 19.7$, F.O.S = 2.5

Case-I When WT is at G.L.



$$q_u = 1.3 c N_c + \gamma D_f N_q \cdot R_q^* + 0.4 \gamma B N_r R_r$$

$$\gamma^* = 0.5 ; R_q^* = 0.5 ; c = 0$$

$$q_{u1} = 0 + 18 \times 1.5 \times 20.3 \times \frac{1}{2} + 0.4 \times 18 \times 2.5 \times 19.7 \times \frac{1}{2}$$

$$q_u = 451.35 \text{ kN/m}^2$$

$$q_{ns} = \frac{148.5 + 26.4B}{3}$$

$$q_{safe} = 49.5 + 8.8B + 11 \times 1.5 = 8.8B + 66$$

$$q_{safe} \cdot B^2 = 150$$

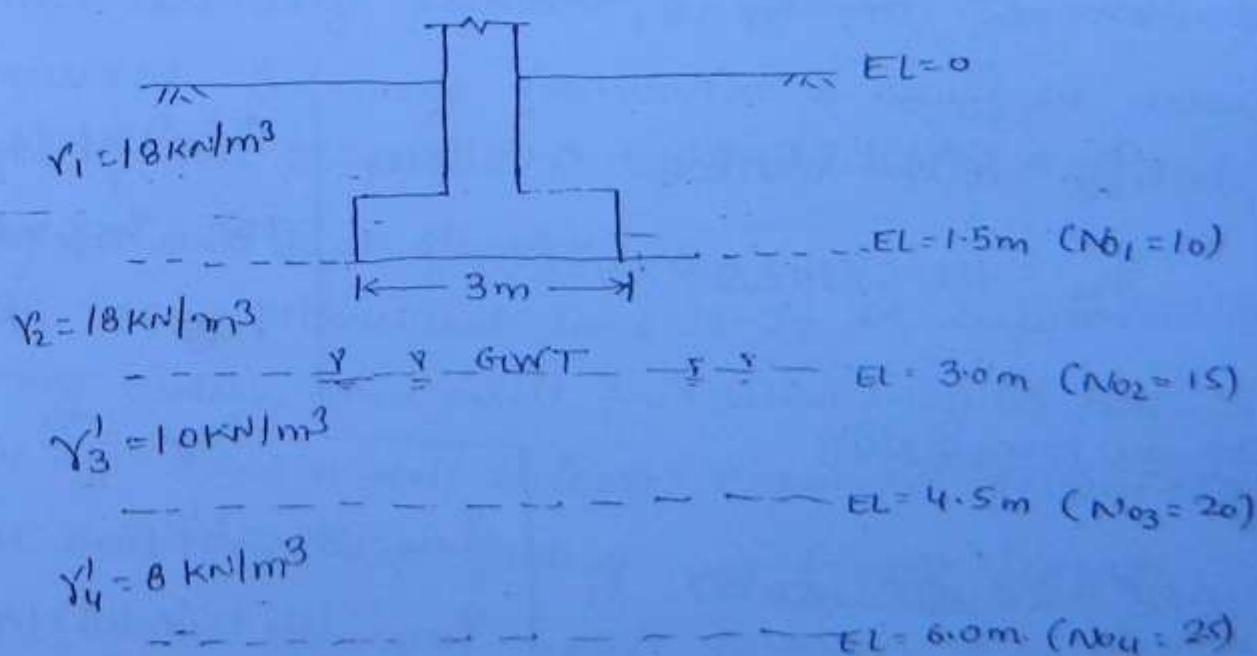
(103)

$$(8.8B + 66)B^2 = 150$$

$$B = 1.385m$$

Ques. Using Rack Hensen's method find Net safe bearing pressure for the footing shown in figure having permissible settlement of 40mm. SPT is conducted at 1.5m interval as shown in figure. The observed value of SPT no. are 10, 15, 20, 25 at a depth of 1.5m, 3.0m, 4.5m and 6.0m respectively. Unit weight of foundation details are shown in figure.

Ans.



$$\Delta \sigma = \frac{q_{safe} \cdot A_{col}}{(B+z)^2}$$

$$= \frac{161.5 \times 2 \times 2}{3^2}$$

(104)

$$\Delta \sigma = 71.78 \text{ kN/m}^2$$

$$H = \frac{0.18 \times 2.0}{1 + 1.68} \log_{10} \left(\frac{20.625 + 71.78}{20.625} \right)$$

$$= 0.114 \text{ m}$$

$$= 11.4 \text{ cm}$$

Square footing located at a depth of 1.5 m. from carries column load of 150 kN. The soil is submerged having effective unit weight of 11 kN/m³ and angle of wing resistance $(\phi) = 30^\circ$. Find the size of footing using Terzaghi's Theory. Given FOS = 3, $\phi = 30^\circ$ and $\phi = 30^\circ$ $N_q = 10$, $N_\gamma = 6$, $c = 0$.

$$q_u = \cancel{1.3}^{1.3} c N_c + \gamma' D_f N_q + 0.4 \gamma' B N_\gamma$$

$$q_u = 11 \times 1.5 \times 10 + 0.4 \times 11 \times B \times 6.0$$

$Q = Q_{\text{safe load}}$

$$150 = q_{\text{safe}} \times A$$

$$q_{\text{safe}} \times B^2 = 150$$

$$q_{\text{safe}} = \frac{150}{B^2}$$

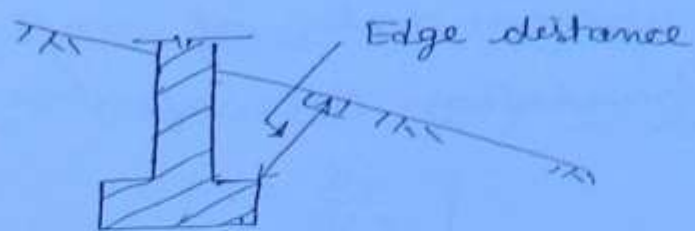
$$q_{\text{net}} = q_u - \gamma' D_f$$

$$q_{\text{net}} = 165 + 6.4B - 11 \times 1.5$$

$$\frac{150}{B^2} = 165 + 6.4B - 16.5$$

$$150 = B^2 (165 + 6.4B - 16.5)$$

ground a minimum edge distance of 60 cm in rock and 90 cm in soil should be provided.



(105)

Selection of foundation

- 1) If structural load is small and soil is medium to dense then shallow footing may be provided.
- 2) If column spacing is less and footing area is more than 50% of total foundation area, then combined footing or mat foundation may be provided.
- 3) If structural load is heavy and soil is loose extending to the great depth, then either raft or pile foundation may be provided.
- 4) If the soil is expansive such as black cotton soil then balancing foundation / floating foundation may be provided. Balancing foundation is raft in which weight of soil excavated is equal to weight of substructure and superstructure.
- 5) If heavy structural load is to be supported on running water, then well foundation may be provided.
- 6) If top 3 to 2 m soil is loose / expansive followed by dense soil, then deep footing may be preferred.
- 7) If there is chance of uplift pressure and soil is

$$q_{\text{net safe}} = 1.38 (N-3) \left(\frac{B+0.3}{2B} \right)^2 S \cdot C_w \quad \text{--- KN/m}^2$$

(106)

S code recommends minimum depth of raft foundation 3m and for raft foundation Net safe bearing pressure is given as

$$q_{\text{net safe}} = 0.88 NS C_w \quad \text{--- KN/m}^2$$

$$\approx 21 N \quad (\text{for sand})$$

Mayerhoff equation

Net safe bearing pressure in KN/m^2 is given as

$$q_{\text{net safe}} = 0.49 NS C_w C_D \quad \text{--- KN/m}^2 \quad \text{when } B < 1.2 \text{ m}$$

$$q_{\text{net safe}} = 0.32 N \left(\frac{B+0.3}{B} \right)^2 S C_w C_D \quad \text{--- } B > 1.2 \text{ m}$$

$$\text{here } C_w = 0.5 \left(1 + \frac{D_w}{B} \right) \quad D_w \Rightarrow \text{Depth of WT below found}^n \text{ level.}$$

$$C_D = \left(1 + \frac{D_f}{B} \right) (\leq 2)$$

S. code Recommendation :-

Minimum depth of foundation should be always more than depth of frost action irrespective of bearing capacity of soil.

Minimum depth of foundation should be always more than depth of organic fill.

Depth of foundation $> 30 \text{ cm}$ for single and double story building and $> 60 \text{ cm}$ for multistory building

$$Q_{\text{net}} = 0.41 N S C_w \quad \text{KN/m}^2$$

where N = corrected average SPT number

S = permissible settlement of footing in mm.

C_w = Water table (WT) correction factor

$$= 0.5 \left[1 + \frac{D_w}{B + D_f} \right]$$

(107)

where D_w = Depth of WT below G.L.

D_f = Depth of Footing

B = width of footing

② Teng's equation :- The net safe bearing pressure in KN/m^2 is given as

$$Q_{\text{net safe}} = 1.4 (N-3) \left(\frac{B+0.3}{2B} \right)^2 S C_w C_d \dots \dots \frac{\text{KN}}{\text{m}^2}$$

where S = permissible total settlement in mm.

N = Corrected SPT no.

B = width of footing

C_w = WT correction factor = $0.5 \left[1 + \frac{D_w}{B} \right]$

D_w = Depth of WT below base of foundation

C_d = Depth correction factor

$$C_d = 1 + \frac{D_f}{B} \quad (C \leq 2)$$

P.S. code equation :- IS code recommends use of Teng's equation without depth correction factor and with

N_0 = Observed value of SPT no. then correct value for overburden pressure is given as

$$N_1 = N_0 \times \frac{350}{\bar{\sigma} + 70} \quad (108)$$

where $\bar{\sigma}$ = effective overburden pressure at the test level if test is ~~perfor~~ performed at depth D_f

$$\bar{\sigma} = \gamma' D_f \quad \bar{\sigma} \nless 280 \text{ kN/m}^2$$

• $\bar{\sigma} > 280 \text{ kN/m}^2$ then overburden pressure correction not required.

latency correction :- If WT is present at test level or above the test level, then this correction is not required. Corrected value for overburden pressure (N_1) further corrected for latency.

$$N_2 = 15 + \frac{1}{2} (N_1 - 15)$$

The above corrections are applied in sequence as mentioned above.

If multilevel test is performed then final SPT no. = average of all the corrected SPT values.

procedure to determine Net allowable bearing pressure / Safe bearing pressure using SPT test data on the basis of settlement criteria

Peck-Hansen Eqⁿ

The net allowable bearing pressure in kN/m^2 is

(i) Angle of shearing resistance. —

(ii) Relative density —

(iii) Allowable bearing pressure on the basis of settlement criteria

(109)

(iv) Point Resistance of pipe. —

(v) Unconfined compressive strength of cohesive soils.

This test is specially designed for dense granular soil because clay may get remoulded and will be affected by pore pressure. A split spoon sampler is used in the bore hole and hammer blows are given to drive the sampler under dynamic action. The standard weight of hammer is 65 kg and height of freefall is 75 cm. The test is conducted at every 2 m interval or at change of strata.

SPT Number (N) is defined as no. of blows of hammer required for 300 mm penetration of sampler. The penetration per blow of hammer is called 'set'. Usually test is performed in 3 stages 150 mm penetration each.

SPT no. is taken as no. of blows required for last 300 mm penetration. The observed value of SPT no is corrected for —

(i) Overburden pressure correction

(ii) WT correction (Dilatency Correction).

Hence allowable pressure on footing can be found.

$$q_{af} = q_{ap} \quad \text{--- Clays}$$

$$q_{af} = q_{ap} \times \frac{B_f}{B_p} \quad \text{--- Sands}$$

(1/0)

NOTE If S_1 is the settlement of foundⁿ of width B_f is placed at depth D_1 & S_2 is the settlement of same foundⁿ is placed at depth D_2 then

$$\frac{S_2}{S_1} = \left[\frac{1 + \frac{2D_1}{B_f}}{1 + \frac{2D_2}{B_f}} \right]^{1/2}$$

Standard penetration test

Plate load Test

This method was designed to determine the modulus of Subgrade reaction c is used to designed rigid pavement but it can also be used to find bearing capacity based on shear criteria & allowable bearing pressure based on settlement criteria.

(112)

Description of test

1. A rigid plate of size 30cm/45cm/60cm/75cm/90cm is used c may be circular or square. Usually smaller size plate is used for dense or stiff soils.
2. A pit of dimension not less than 5 times dimension of plate is excavated at same depth equal to depth of footing & plate is seated at the centre of pit. If H.T is present above the foundation level then it must be lowered by pumping below the foundation level.
3. The load on the plate is applied through jacking mechanism & settlement of plate is recorded by 3 dial gauges attached at 120° to each other & average settlement is taken. The settlement vs load curve is plotted by taking settlement in y-axis & pressure / load in x-axis. The shear failure stage is represented when plate suddenly starts settling.

From curve q_{up} (ultimate B.C for plate) on shear criteria is found. And $q_{uf} = q_{up}$

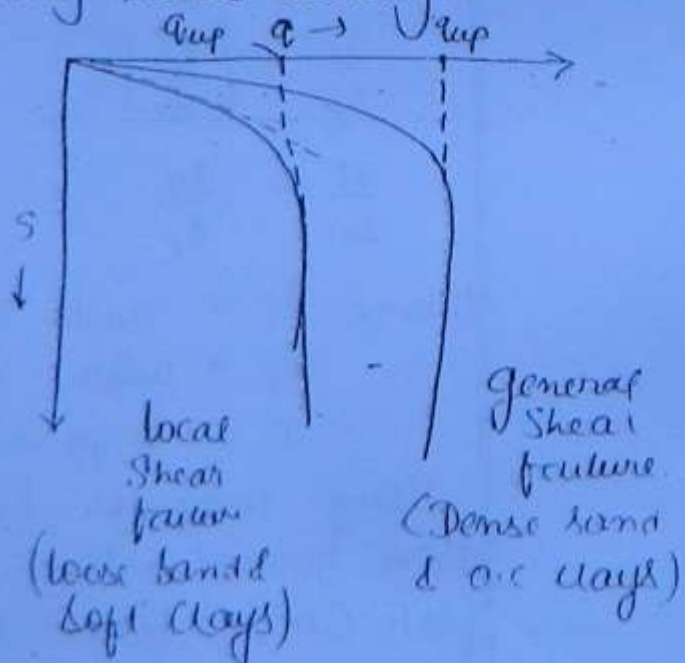
q_{uf} = ultimate B.C for footing)

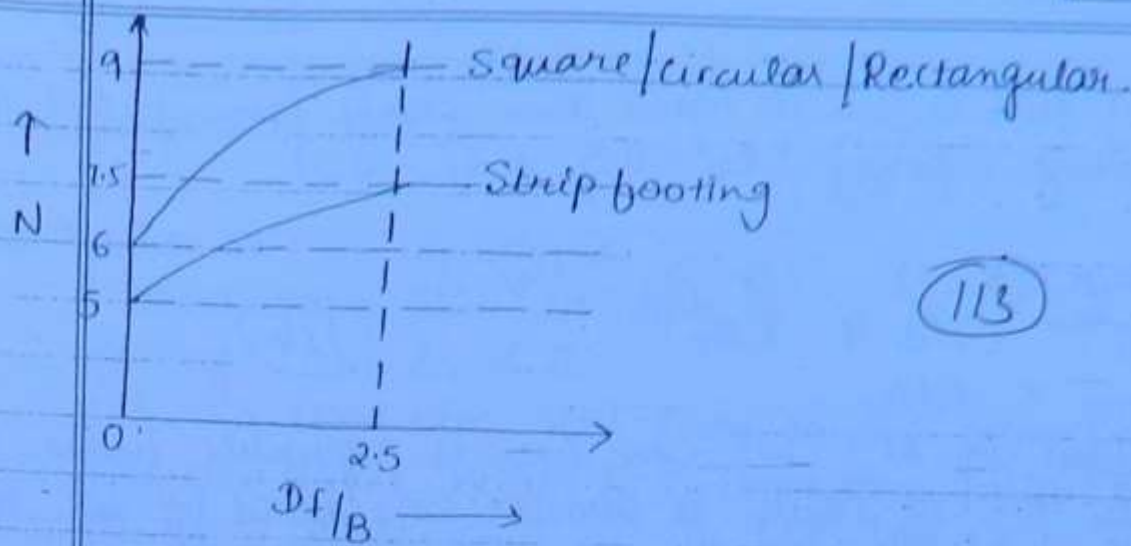
Can be found as follows.

$q_{uf} = q_{up}$ for clays

$q_{uf} = \frac{B_f}{B_p} \times q_{up}$ for sand

(For clays B.C is independent of size)





Meyerhoff equation

This theory can be applied for shallow & deep footings both. He considered the failure surface & stress zone to extend upto ground level & he accounted for side shear resistance also. He introduced additional correction factors for shape, depth & inclination.

$$q_u = (C \cdot N_c \cdot S_c \cdot d_c \cdot i_c) + (q + N_q \cdot S_q \cdot d_q \cdot i_q) + \frac{1}{2} \gamma B \cdot N_\gamma \cdot S_\gamma \cdot d_\gamma \cdot i_\gamma$$

$S_c, S_q, S_\gamma \rightarrow$ Shape Correction factor
 $d_c, d_q, d_\gamma \rightarrow$ depth "
 $i_c, i_q, i_\gamma \rightarrow$ inclination "

NOTE-1 IS Code recommends use of Meyerhoff eqⁿ & general shear failure & Additional K₁ Correction factor.

2. Terzaghi has considered general shear failure but if soil fails in local shear then modified ~~the~~ factor soil parameter should be used

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

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

If u -T rises to Orl in sands then nearly there is 50% decrease in bearing capacity ($R_q^* = 0.5$ & $R_r^* = 0.5$)

Q2 For Clays ($\phi = 0$) $N_r = 0$ $N_q = 1$

$$q_{ult} = C N_c + \gamma D_f$$

(1/4)

$$q_{net} = C N_c$$

The effect of u -T on $C N_c$ & N_c is negligible hence net ultimate B.C. of Clays is almost unaffected by the rise of u -T at Orl .

Skempton's theory [c- ϕ Soils]

This theory is applicable only for cohesive soil in c bearing capacity is independent of size. This method does not neglect side shear resistance hence it can be applied for deep footing also.

Net ultimate B.C. is given by

$$q_{net} = C N_c$$

Where N_c is B.C. factor & depends upon $\left(\frac{D_f}{B}\right)$ ratio.

Q2 When footing is on the surface ($D_f/B = 0$) then

$$N_c = 5.0$$

(Strip footing)

$$N_c = 6.0$$

[Square / circular / Rectangular footing]

Q2 When D_f/B is $0 < D_f/B < 2.5$

$$N_c = 5 \left(1 + 0.2 \frac{D_f}{B}\right)$$

Strip footing

$$N_c = 6.0 \left(1 + 0.2 \frac{D_f}{B}\right)$$

Square / circular footing

[B=D]

$$N_c = 5.0 \left(1 + 0.2 \frac{B}{L}\right) \left(1 + 0.2 \frac{D_f}{B}\right)$$

Rectangular footing & Raft foundation

Q3 When $D_f/B \geq 2.5$

$$N_c = 7.5$$

for Strip footing

$$N_c = 9.0$$

for Square, Circular, Rectangular footing

(Max. value)

of soil because stress zone below the foundation is located in zone 2

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- Case 2 When L.T is in zone 2

When water table at depth of z_r from the base of footing $0 \leq z_r \leq B$.

In this condition 3rd term of B.C Capax eqⁿ is affected hence either L.T Correction factor (R_r^*) should be used or eff. unit wt. of soil should be used in 3rd term.

$$R_r^* = 0.5 \left[1 + \frac{z_r}{B} \right]$$

When L.T remains below the foundation level then there will be no effect of W.T in 1st term & 2nd term. Hence for 2nd term γ of soil zone I should be used.

Case 3 - When L.T is in zone 1

In 3rd term R_r^* will be 0.5 or $\gamma_{effective} (\gamma')$ will be ~~sho~~ used & in 2nd term R_q^* ~~its~~ should be used or eff. unit wt. of zone 1 should be used.

$$R_q^* = 0.5 \left[1 + \frac{z_q}{B_f} \right]$$

In 1st term also $c-\phi$ will be affected at the base of footing hence either eff. unit wt should be used
eff. friction friction angle & cohesion should be used

NOTE - For sands ($c=0$) then

$$Q_u = \gamma D_f N_q + \frac{1}{2} \gamma B N_\gamma$$

$$\text{L.T is present} \rightarrow Q_u = \gamma D_f N_q R_q^* + \frac{1}{2} \gamma B N_\gamma R_r^*$$

Of failure it remains in plastic equilibrium. For pure cohesive soils failure surface of zone 2 is circular whereas for $c-\phi$ soils it is spiral.

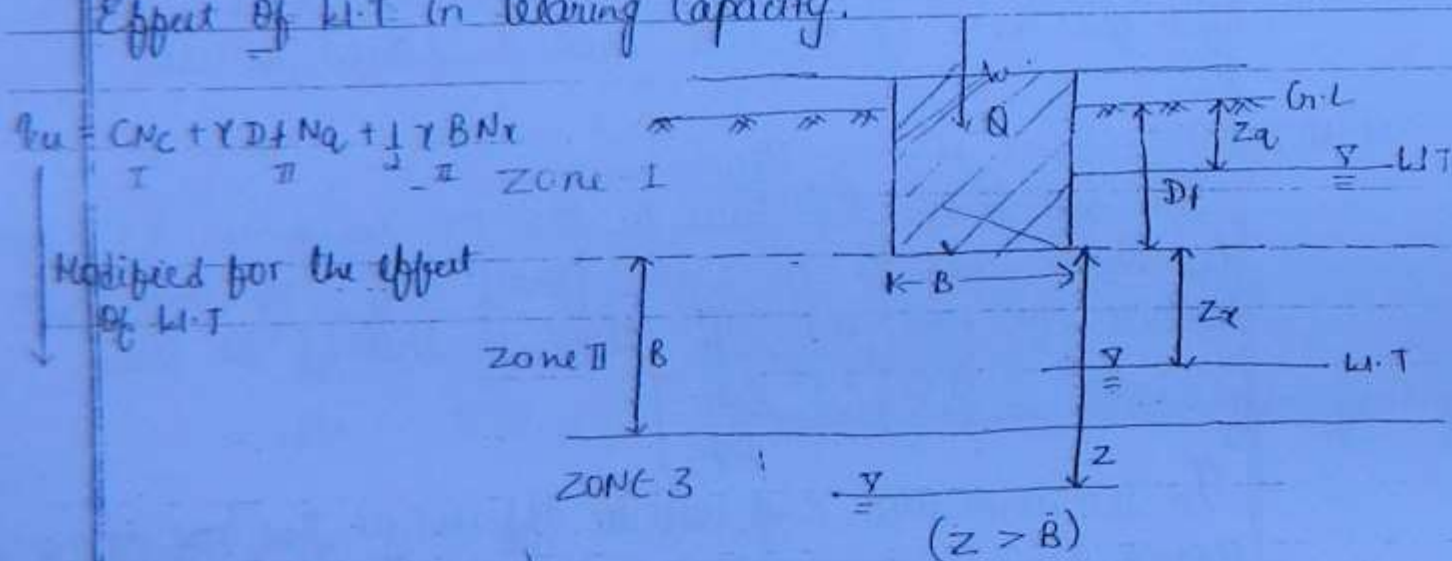
3 Zone 3 - Rankine's passive zone - It makes an angle $(45 - \frac{\phi}{2})$ to the horizontal.

(116)

NOTE - For pure clays - $q_u = C N_c + \gamma D_f$ (from table values are)
Hence B.C is independent of size. Whereas in sandy soils B.C is directly proportion to size (B).

In order to support heavy load on clayey soils either raft / pipe foundation should be preferred.

Effect of L.T in bearing capacity.



$$q_u = c' N_c + \gamma D_f N_q R_q^* + \frac{1}{2} \gamma B N_\gamma R_\gamma^*$$

R_q^* & R_γ^* are L.T Correction factors

c' - effective cohesion with respect to L.T

X) When L.T is in zone 3 it means depth of L.T below the foundation is greater than width of foundation ($z > B$) under this condition there will be no effect in bearing capacity

(117)

$$q = \gamma D_f$$

5. Terzaghi ignored the stress in adjacent soil above the foundation level becoz stress zone is extended upto foundation level only.
7. He ignored the side resistance & considered only base resistance.

Terzaghi's theory is an improvement over prandtl theory. Prandtl considered the base of footing to be smooth whereas Terzaghi's considered base of footing to be rough. This theory is valid for c & ϕ soil both. The bearing capacity eqn is same in prandtl & terzaghi theory but bearing capacity factors are different. The ultimate bearing capacity of strip footing is

$$q_u = C N_c + \gamma D_f N_q + \frac{1}{2} \gamma B N_\gamma$$

Where N_c , N_q , N_γ are bearing capacity factors $\propto$ depend upon friction angle of soil.

For pure clays ($\phi = 0$) $\downarrow$

Prandtl B.C factor

$$N_c = 5.14$$

$$N_q = 1$$

$$N_\gamma = 0$$

Terzaghi B.C factor

$$N_c = 5.7$$

$$N_q = 1$$

$$N_\gamma = 0$$

$$N_\phi = 1 + \tan^2 \left(45 + \frac{\phi}{2} \right) \quad \text{Influence value}$$

$$N_c = \tan \phi e^{\lambda \tan \phi}$$

$N_q =$ $N_H =$

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(iv) Square footing

$$q_m = 1.3 C N_c + \gamma D_f N_q + 0.4 \gamma B N_{\gamma}$$

B = size of footing

iii) Circular footing

$$q_{u1} = 1.3 C_{Nc} + \gamma D_f N_q + 0.3 \gamma B N_{\gamma}$$

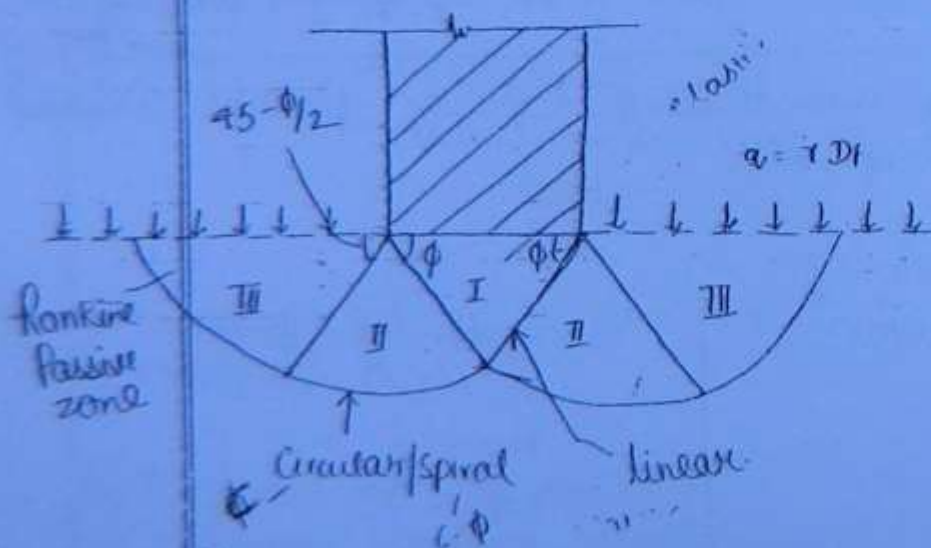
D → Dia of footing

(iii) Rectangular footing & Raft foundation

$$q_{u1} = \left(1 + 0.3 \frac{B}{L}\right) \left[C_{NG} + \gamma D_f N_q + \frac{1}{2} \left[1 - 0.2 \frac{B}{L} \right] \gamma B N_r \right]$$

Where $B \rightarrow$ width of footing ($B < L$)

$L \rightarrow$ length of footing



Veryagi divided soil below the foundation in 3 zone

1. Central zone (Zone I) - It is called linear shear zone. Zone I remains in elastic state.
Zone II - It is called Radial shear zone & at the time

Vold ratio

$$\leq 0.55$$

$$\geq 0.75$$

unconfined comp.

$$> 100 \text{ kN/m}^2$$

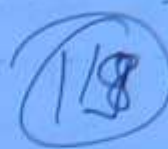
$$< 80 \text{ kN/m}^2$$

Strength of Clays

load settlement

Curve

Curve I



Curve II

1 Rankine's theory. [ϕ Soils]

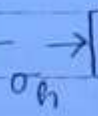
(ultimate bearing capacity)

Gr L

$$\sigma_v = q_u$$

$$\sigma_v = \gamma \cdot D_f \text{ (wt of soil)}$$

$$\sigma_v > \sigma_h$$

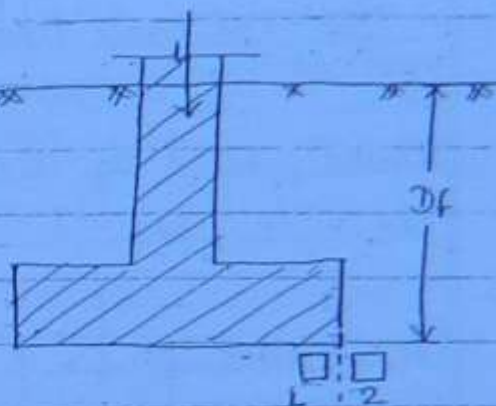


$$\sigma_h$$

$$\sigma_h$$

$$\sigma_h$$

$$\sigma_h$$



$$\sigma_h \text{ for element 2 is in passive state } \sigma_h = K_p \cdot \gamma D_f = \frac{1 + \sin \phi}{1 - \sin \phi} \gamma D_f$$

$$= \tan^2(45 + \frac{\phi}{2}) \gamma D_f$$

$$K_p \cdot K_p \gamma D_f$$

$$\text{For element 1 } \sigma_v \text{ is in passive state } = K_p \cdot \sigma_h = q_u = \sigma_v$$

$$q_u = \tan^2(45 + \frac{\phi}{2}) \tan^2(45 + \frac{\phi}{2}) \gamma D_f$$

$$q_u = \tan^4 \tan(45 + \frac{\phi}{2}) \gamma D_f$$

Rankine Consider the equilibrium of two elements adjacent to each other 1 below the footing at the corner (element 1) & other adjacent but outside the footing (element 2) & he applied condition of passive pressure.

$$q_u = \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^2 \gamma D_f$$

$$q_u = \tan^4 \left(45 + \frac{\phi}{2} \right) \gamma D_f$$

Limitations :-

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1. Not applicable for clays
2. At $D_f = 0$, $q_u = 0$ is not practicable.
3. The width of foundation is accounted. (No effect)
4. There is no effect of shape & size of footing

Bell's Methods [C- ϕ soils]

The ultimate bearing capacity is given as

$$q_u = C N_c + \gamma D_f N_q$$

Where, N_c & N_q are bearing capacity factors.

$$N_c = 4, N_q = 1 \quad (\text{for clays}) \quad (\phi = 0)$$

Limitation :-

1. The base of footing is considered smooth.
2. The size of footing is not accounted

Getlinius Method

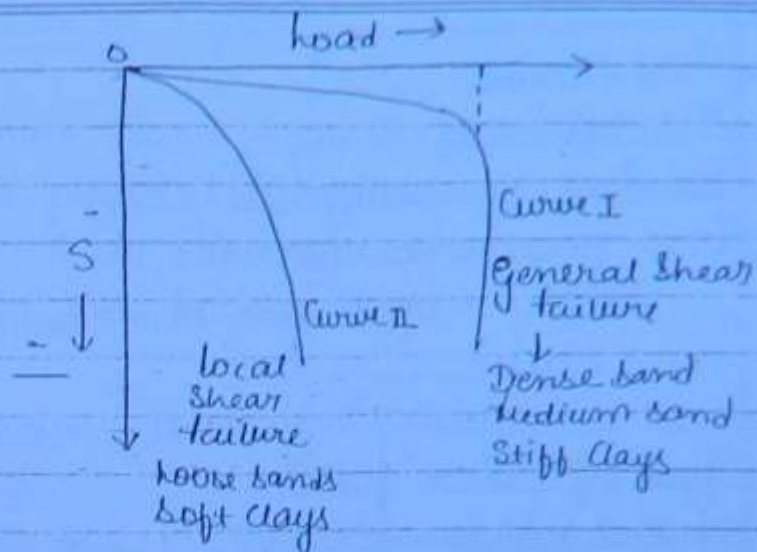
The foundation is resting on cohesive soil (C-soils only) & failure is assumed due to ~~strip~~ general shear condition. & the failure surface is an arc of a circle.

Imp Terzaghi's theory

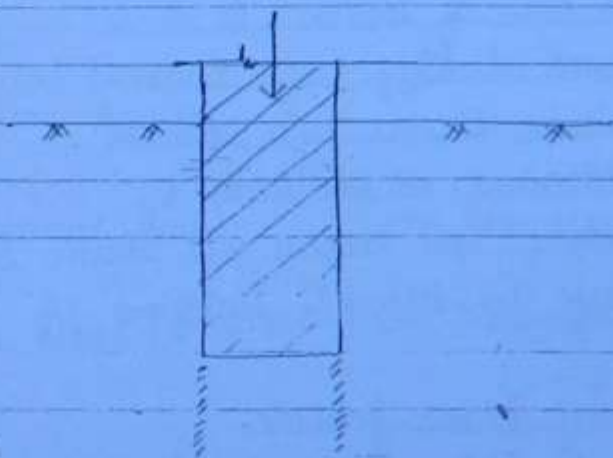
Imp Assumptions -

1. The foundation is considered as shallow ($D_f \leq B$)
2. The base of foundation is rough
3. Footing is continuous, such as strip footing therefore analysis is 2-dimensional
4. Terzaghi replace the soil above the base of foundation by an equivalent surcharge $q = \gamma D_f$
5. He considered general shear failure & at the time of failure soil reaches into plastic equilibrium.

(12)



(iii) Punching Shear failure:



In case of very loose soils using deep footing or piles, the soil adjacent to the foundation gets sheared hence adjacent soil is not fully stressed & excessive settlements

are recorded w/o change in characteristics of adjacent soil but this failure is not common in shallow foundⁿ. There will be no side bulging of soil.

Guidelines to diffⁿ betⁿ General & local shear failure

Parameter	G.S.F	L.S.F
1. Friction angle	More than 36°	$\phi < 28^\circ$
2. Shear strain at failure	$< 5\%$	$> 15\%$
3. SPT Number	> 30 blows	< 5 blows
4. Relative density / density index	$> 70\%$	$< 20\%$

3. Field Methods.

1. Plate load test
2. Standard penetration test (SPT)
3. Cone penetration test (C.P.T)

(122)

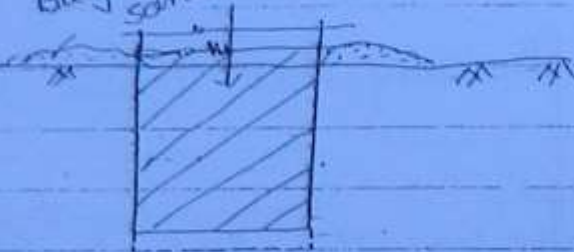
Field methods are more suitable for granular soils (dense sands).

Types of Shear failure.

(i) General Shear failure.

In Case of Medium to dense sands & stiff clays shear failure occurs w/o excessive settlement. At the time of

Bulging of sands



failure soil reaches into plastic stage & due to shear foundation may get tilted & bulging of soil may occur from the sides.

(ii) Local Shear failure.

In Case of loose sands & soft clays large settlement may occur before the foundation before soil reaches into plastic equilibrium. There may be little or no bulging from the sides. If soil fails in local shear then modified values of cohesion & friction should be used.

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

$$\tan \phi' = \frac{2}{3} \tan \phi \Rightarrow \phi' = \frac{2}{3} \tan^{-1} \left(\frac{2}{3} \tan \phi \right)$$

During plate load test general & local shear failure will give following types of load settlement curve.

Factors affecting bearing Capacity.

- (i) Position of GWT w.r.t size & depth of foundation
- (ii) Type of Soil & its physical & engineering properties
- (iii) Type of foundation & its dimension
- (iv) Initial stresses on the soil if any

(123)

Methods to determine bearing capacity

1. Analytical Methods

The properties of soil (c, ϕ) & characteristics of foundⁿ are used. Analytical Methods are based on following 3 theories

Elastic theory	Classical earth Pressure theory	Plastic theory
→ Schlier's Mtd. (cohesive soils)	✓ Rankine Mtd. [ϕ soils] (frictional soil)	→ Jellineq Mtd. [c -soils]
	→ Pauker's Mtd. [ϕ soils]	→ Prandtl Mtd. [c, ϕ both]
	✓ Bell's Mtd. [c, ϕ soils]	✓ Terzaghi Mtd. [c, ϕ both]
		→ Skempton Mtd. [c, ϕ]
		→ Meyerhoff Mtd. [c, ϕ]
		→ Brinch Hansen [c, ϕ]
		→ I.S. Code Mtd. [c, ϕ]
		→ Vesic's Mtd. [c, ϕ]

2. Bearing Capacity from Codes.

Various building codes are published by codal agencies such as IRC, BIS, CPWD etc. c Carry bearing capacity of Zonal soils. For rough computation & approximate analysis such values can be used

Net Safe bearing Capacity (q_{ns})

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

$$F.O.S = 2 \text{ to } 3$$

(124)

Safe bearing Capacity (q_{safe})

$$q_{safe} = q_{ns} + \bar{\sigma}$$

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

$$q_{safe} = \frac{q_{nu}}{F.O.S} + \gamma D_f$$

$$q_{safe} = \frac{q_u - \bar{\sigma}}{F.O.S} + \gamma D_f$$

NOTE - The total pressure at the base of foundation due to applied load & self wt. should be less than or equal to safe bearing capacity of soil.

Classification of foundation

Foundation

Shallow foundation

$$(D_f \leq B)$$

D_f = depth below G.L

B = least lateral dimension

↓
footing

Isolated

→ Rectangular

→ Square

→ Circular

Continuous

→ Strip

→ Combined

Strap

trapezoidal

Rectangular

Deep foundation

$$(D_f > B)$$

→ Deep footing

→ Pile foundation

→ Well's foundation
or caissons

→ Pier foundation

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FOUNDATION ENGINEERING

Normally foundation failure may be due to

- (i) Shear failure or Shear Criteria
- (ii) Settlement failure or Settlement Criteria

(126)

The allowable load on the basis of Shear Criteria is called bearing capacity & allowable load on Settlement Criteria is called bearing pressure. The design load should be taken min. of the above two criteria.

NOTE Generally it is found that sands do not fail in shear because the strength imparted by friction & interlocking is much greater but clayey soils shear failure or settlement failure anyone may occur depending upon properties of soil & foundation.

Determination of bearing capacity on the basis of Shear Criteria.

(1) Gross pressure - It is the total pressure at the base of footing due to self wt. of soil, wt. of footing & applied loading on the footing (q_g).

(2) Net pressure - It is the pressure at the base of foundation in axis to the initial effective overburden pressure.

$$q_n = q_g - \bar{\sigma}$$

$\bar{\sigma}$ - initial eff. overburden pressure

(3) Ultimate bearing capacity - It is the max. pressure at the base of foundation which can be applied without shear failure. It is min. gross pressure at $\leq$ soil is likely to fail in shear.

Net ultimate bearing capacity (q_{nu}) -

$$q_{nu} = q_u - \bar{\sigma}$$

$$\bar{\sigma} = \gamma \cdot D_f$$

γ = Eff. unit wt.



It is based on total stress analysis. It assumes circular slip surface of radius 'r'. The resultant acts at a distance from soil mass is dependent on curvature smaller circle of radius (A sin β) which is called friction circle. The inclination of resultant with the normal of failure surface is at an angle (β).

Taylor's Method

Taylor defined a dimensionless parameter called stability No. which is given as

$$S_m = \frac{C}{\gamma H} = \frac{C}{\gamma Z F_e} \quad \text{--- (1)}$$

Z = ht. of slope

$$S_m = \tan \beta \cos \beta \quad \text{--- [for clay]}$$

$$= [\tan \beta - 1.5m \beta] \cos \beta \quad \text{--- [for c- ϕ soil]}$$

Using Taylor's chart stability no. is obtained for given value of β & ϕ .

Use: In Eq (1) γ & c . γ should be used when soil is dry or partially saturated also use assumed friction angle (ϕ_u) to determine β_u .

Eq (2) - If soil is fully submerged then use γ' submerged unit wt & ϕ' (eff. friction angle)

If soil is sat. & submerged but not submerged water due to capillary effect or sudden drawdown of water Table use γ_{sat} & ϕ_u should be used & assumed friction angle (ϕ_u)

$$\gamma_w = \frac{\gamma'}{\gamma_{sat}} \quad \text{--- (2)}$$

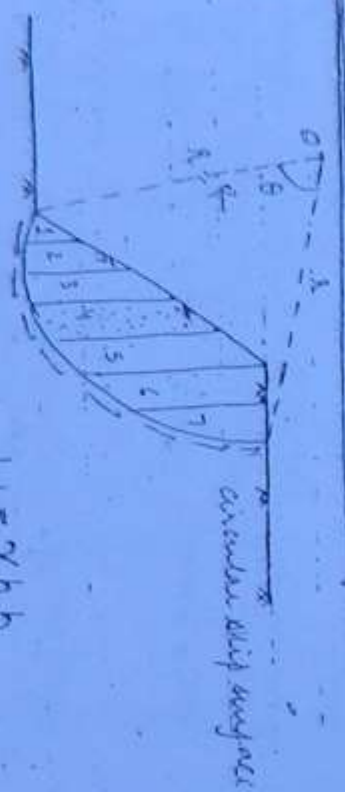
(127)

C_u

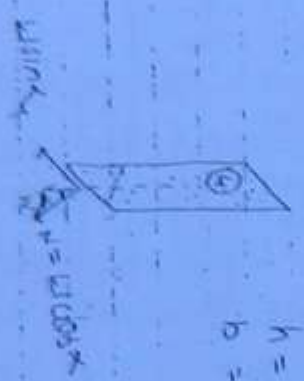
mobilised unit cohesion for β slope of ht Z

$$\frac{C}{\gamma H} = \frac{f}{\gamma H f_c}$$

used by Swedish Circle Method on Mtd of slices
 (26) analysis



$L = \sum h b$
 $h = \text{avg. height of slice}$
 $b = \text{width of slice}$



In this method the soil mass above the assumed slip surface is divided into 6 to 12 no. of slices of equal width. The forces b/w adjacent slices are neglected. (no shear stress b/w the slices is considered). Each slice is considered to be an independent column of unit thickness but restoring moment is given by friction & cohesion about point O (vertical equilibrium moment is given by tangential component of wt of each slice).

The Swedish Circle can be used for homogeneous or stratified soils, can also be used where surface is taking place & pore pressure are present in

(128)

Pos - Restoring moment
overturning moment

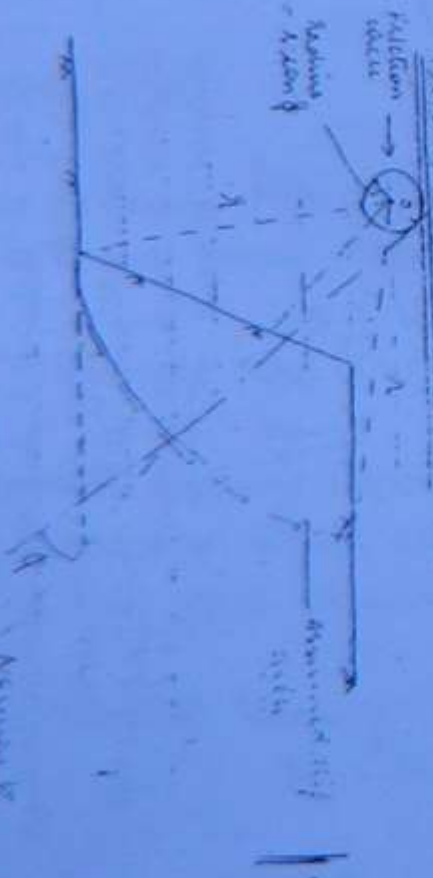
$M_0 = \sum T \cdot R$

$\sum T$ is total tangential component of wt of soil mass of all slices

$M_R = (C \cdot R \cdot \theta) \cdot \sum L + (\sum N \tan \phi) \cdot R$
 $\text{longm. epax. (slip surface)}$
 $FOC = \frac{C \cdot R \cdot \theta + \sum N \tan \phi}{\sum T} = \frac{C L + \sum H \cos \alpha \cdot R}{\sum W \sin \alpha}$

$c = \text{cohesion (in kN/m}^2)$
 $R = \text{radius of slip surface}$

Friction Circle Method



Restoring friction on slip surface

$$\sin \alpha = \sin \beta \cos \phi = \frac{\sin 2\phi}{2}$$

If water table is present in the influence zone then use effective soil parameters (c', ϕ')

Stability analysis of finite slope

Assumptions:

- 1. Failure is due to rotation
- 2. The slip surface is circular

Methods of Analysis

Generally stability analysis is made in two stages: I

1. Immediately after construction along time after construction

The significance is don't pore pressure.

① In case of sands & gravels, effective stress analysis should be used for short term & long term conditions both.

② For normally consolidated clays (O.C.C) $\rightarrow$ since with σ_{total} stress analysis for short & term and effective stress analysis for long term.

③ For O.C.C. which behave similar to dense sand, effective stress analysis should be used for short & long term both.

Methods Based on Total Stress Analysis

1. Swedish Circle Method or Method of slices

↳ It is an improvement over Fellenius method

2. Friction circle method

3. Total stress analysis of freely seepage soils called $[\phi_u = 0]$ analysis

Methods based on Effective Stress Analysis

① Taylor's stability method

② Bishop's method

③ Effective stress analysis of saturated dense soil to sudden draw down and seepage

Note: - For total stress analysis (TSA) results of V-U Test are used for effective stress analysis (ESA) results of U tests are used

γ = avg total unit weight of soil above the dip surface.

For γ_1 & γ_2 by dipping above water table

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The dipping below WT & above dip surface have total unit wt γ_2 is γ_{sat} .

$$\gamma = \frac{\gamma_1 h_1 + \gamma_2 h_2}{h_1 + h_2}$$

Special case : if WT rises to ground level then $h_1 = z$

$$FOS = \left(\frac{1 - \frac{\gamma_{sat}}{\gamma_{sub}} \right) \frac{\tan \phi'}{\left(1 - \frac{1}{2} \frac{\gamma_{sat}}{\gamma_{sub}} \right) \frac{\tan \phi'}{\gamma_{sub}}}$$

$$= \frac{\gamma'}{\gamma_{sat}} \frac{\tan \phi'}{\tan \phi} \left[\frac{\gamma' + \frac{1}{2} \gamma_{sat}}{\gamma_{sub}} \right]$$

$$= \frac{1}{2} \frac{\tan \phi'}{\tan \phi} \left[\frac{\gamma' + \frac{1}{2} \gamma_{sat}}{\gamma_{sub}} \right]$$

to correct under steady state seepage at on dikes to ground level on under submerged

condition up to 5%
FOS reduced by 50%

$$SN = \frac{C}{\gamma H_c} = \frac{C}{\gamma H_c}$$

It means under dry condition & if FOS is less than 2 then for some sand or submerged FOS will reduce by 50% hence it may become unsafe. In under dry condition overall FOS is 2.

Case 3: Infinite slope in cohesionless soil

$$FOS = \frac{\text{shear strength}}{\text{shear stress}} = \frac{c}{c + \gamma \tan \phi}$$

$$\gamma z \cos \phi \sin \phi$$

$$FOS = \frac{c}{\gamma z \sin \phi \cos \phi}$$

$$FOS = \frac{Mc}{z}$$

$$= \frac{c}{\gamma z \sin \phi \cos \phi} = \frac{Mc}{z}$$

$$\frac{c}{\gamma H_c} = \sin \phi \cos \phi = SN$$

Taylor's stability No. (for clay)

max theoretical value of $SN = 0.5$
however, max practical value of $SN = 0.15$

Ques 2: FOS w.r.t. friction

(i) $FOS = \frac{\tan \phi}{\tan \beta}$

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where, ϕ is friction angle
 β is slope angle

(ii) $FOS = \frac{\tan \phi}{\tan \phi_m}$

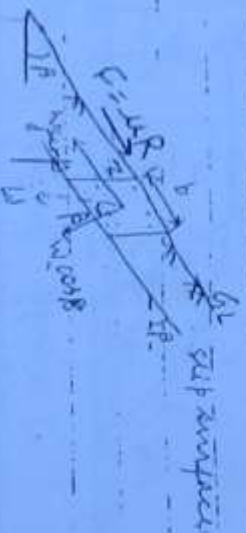
where,
 ϕ_m is mobilised friction angle or actual
friction angle developed

Stability analysis of infinite slope

Assumptions:

- ① soil is assumed homogeneous
- ② Failure surface is plane surface which is due to translation i.e. sliding

Case 1: Frictionless soil under dry or moist conditions ($0 \leq \beta < 100$)



The failure / sliding is due to tangential component of weight along the slip surface & resistance is equal to frictional force

$FOS = \frac{\text{Resisting force}}{\text{Sliding force}}$

$$= \frac{\frac{1}{2} \gamma b^2}{\frac{1}{2} \gamma b^2 \cos \beta}$$

$FOS = \frac{\tan \phi}{\tan \beta}$
 ϕ = friction angle
 β = slope angle

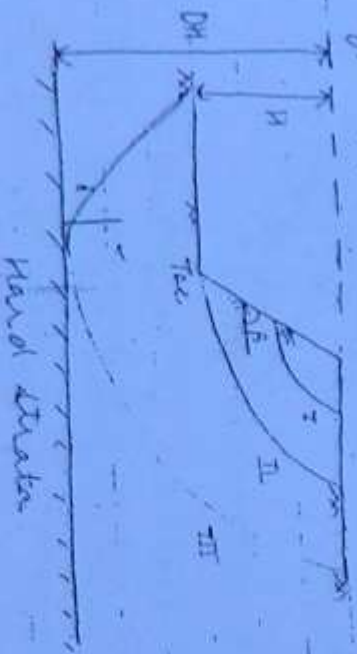
Case 2: If water table is present above the slip surface at a distance parallel to ground and soil is cohesionless



$FOS = \left(1 - \frac{1}{2} \frac{\gamma_w}{\gamma} \right) \frac{\tan \phi}{\tan \beta}$

where
 ϕ' = effective friction angle
 b = water table

Types of failure planes:-



(132)

- I - face failure / slope failure $\beta = \alpha$
- II - toe failure $\beta > 55^\circ$
- III - base failure

FACE FAILURE:

It occurs when angle of slope (β) is very high and soil is loose part close to the surface and in upper part soil is weak.

TOE FAILURE:

It occurs when slope is steep ($\beta > 55^\circ$) and soil is homogeneous in upper & lower parts.

BASE FAILURE:

Failure surface forms below the toe when soil is hard & strong. It occurs when soil below toe is relatively weak and

above the soil is strong & steep.

If H is $1/2$ of slope and DH is $1/2$ of H point of failure surface then it is called depth factor.

Factor of Safety (FOS):

FOS = Shear strength of soil / shear stress developed on slip surface

CASE 1: FOS w.r.t cohesion

(i) $FOS = \frac{M_c}{Z}$

where M_c = critical height = $\frac{4c}{\gamma} \cdot \tan(15^\circ + \frac{\phi}{2})$

For fine clays $\phi = 0^\circ$

Z = actual depth of cut

(ii) $FOS = \frac{c}{c_m}$

where c = unit cohesion
 c_m = mobilised or developed cohesion

$$\begin{aligned} \text{Net stress at top of bottom layer} &= 70.56 - 42 = 28.56 \\ \text{" " bottom of " " } &= 70.56 + 19.74 \\ &= 90.30 \end{aligned}$$

$$Pa_1 = -23.1 \times \frac{1}{2} \times 2.89 \times 1 = -33.379 \quad \text{acts at } 6 + 3.11 + \frac{3 \times 1}{3} = 11.04 \text{ m}$$

$$Pa_2 = 24.9 \times \frac{1}{2} \times 3.11 \times 1 = 38.72 \quad \text{acts at } 6 + \frac{3.11}{3} = 7.04 \text{ m}$$

$$Pa_3 = 28.56 \times 6 = 171.36 \quad \text{acts at } 3 \text{ m}$$

$$Pa_4 = \frac{1}{2} \times 61.74 \times 6 = 185.22 \quad \text{acts at } 2 \text{ m from}$$

$$\text{Total} = 361.921 \text{ kN/m}$$

(133)

$$\bar{H} = \frac{-33.379 \times 11.04 + 38.72 \times 7.04 + 171.36 \times 3 + 185.22 \times 2}{361.921}$$

$$= 2.18 \text{ m}$$

Stability of Earth Slopes

Forces responsible for failure :-

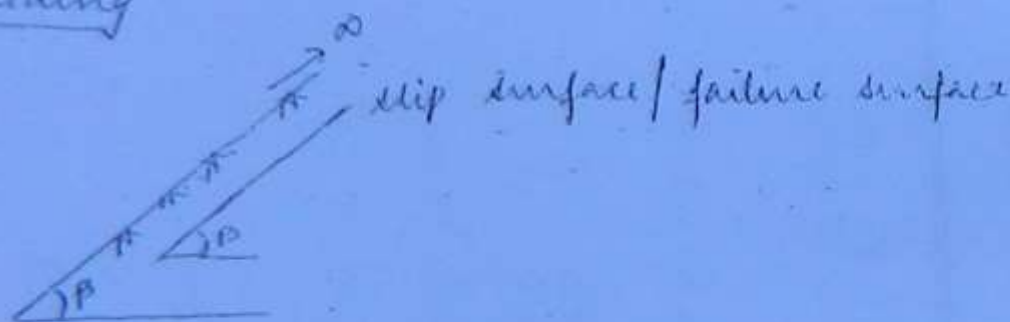
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1. gravitational force
2. seepage force
3. sudden drawdown of water table
4. Earthquake force
5. erosion due to water
6. Excavation near the soil mass

Types of Earth slopes :-

1. Infinite Slope

It represents surface of semi-infinite inclined soil mass such as mountain slope and desert sand dune. The failure surface is parallel to slope of landfill and failure is due to slipping or sliding.



2. Finite Slope

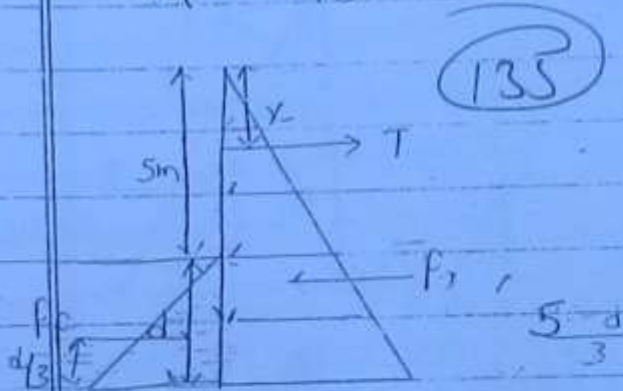
It is bounded between top & bottom surfaces of soil. The failure is due to rotation of slip surface and plane of failure is either circular or spiral. There are three

For an Anchored sheet pile wall shown in fig determine penetration depth of (d) & tension force in the tie bar.
 The properties of soil are $\gamma = 20 \text{ kN/m}^3$
 $\phi = 30^\circ$ height (H) = 5 m & tie bar is anchored at 1 m below the G.L.

$$K_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} \quad \& \quad K_p = \frac{1 + \sin 30^\circ}{1 - \sin 30^\circ}$$

$$K_a = \frac{1}{3}$$

$$K_p = 3$$



$$\sum F_x = 0$$

$$T + P_p - P_a = 0$$

$$T + \frac{1}{2} \times 3 \times 20 \times d^2 - \frac{1}{2} \times \frac{1}{3} \times 20 (5+d)^2 = 0$$

$$T + 31.5d^2 - 3.5(5+d)^2 = 0$$

$$T + 31.5d^2 - 3.5(25 + d^2 + 10d) = 0$$

$$T + 31.5d^2 - 87.5 - 3.5d^2 - 35d = 0$$

$$28d^2 - 35d + T - 87.5 = 0$$

$$\sum M_B = 0$$

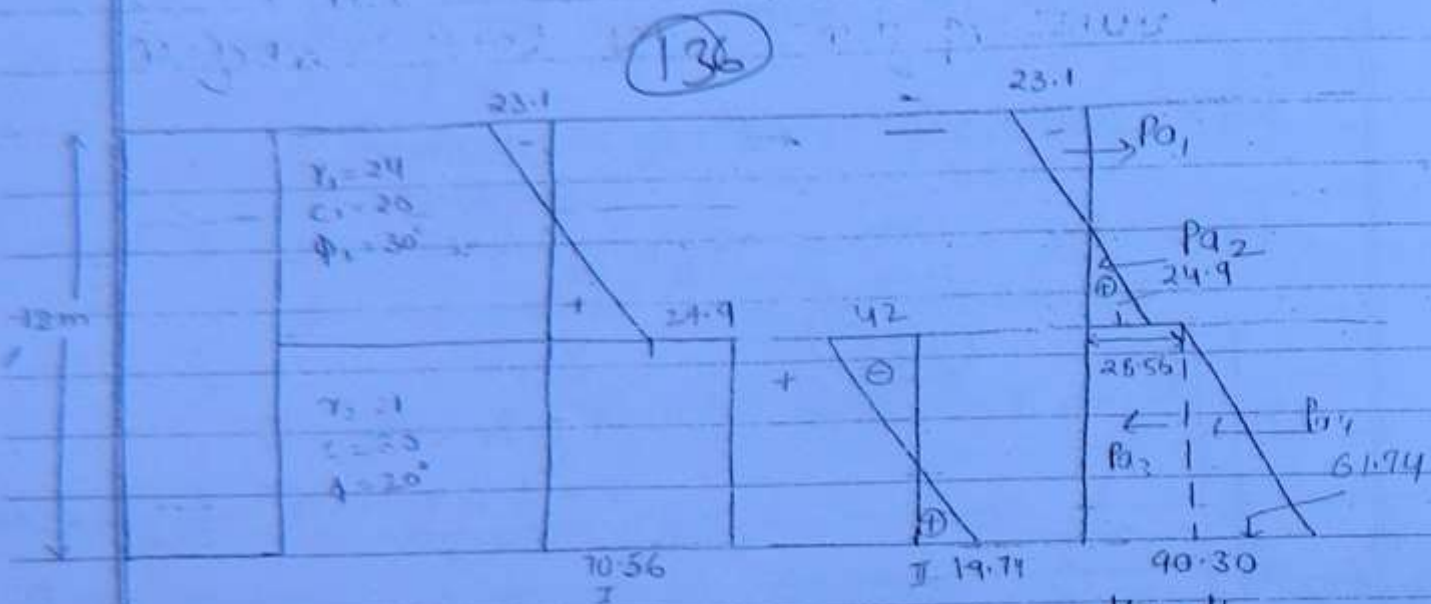
$$T \times (5+d-1) + P_p \times \frac{d}{3} - P_a \times \frac{5+d}{3} = 0$$

$$T \times (4+d) + \frac{1}{2} K_p \gamma d^2 \times \frac{d}{3} - \frac{1}{2} K_a \gamma (5+d)^2 \left(\frac{5+d}{3} \right) = 0$$

$$4T + Td + 10.5d^3 - 1.167(5+d)^3 = 0$$

$$4T + Td + 9.33d^3 - 145.85 = 0$$

A retaining wall 12m ht retains backfill & vertical smooth surface. The top 6m soil has $\gamma = 24 \text{ kN/m}^3$, $c = 20 \text{ kN/m}^2$, $\phi = 30^\circ$ whereas bottom 6m soil has $\gamma = 21 \text{ kN/m}^3$, $c = 30 \text{ kN/m}^2$, $\phi = 20^\circ$. Plot active earth pressure & surcharge.



$$Z_{c1} = \frac{2c_1 \tan(45^\circ + \frac{\phi_1}{2})}{\gamma_1} = 2.89 \text{ m}$$

$$K_{a1} = \frac{1}{3}$$

$$Z_{c2} = \frac{2c_2 \tan(45^\circ + \frac{\phi_2}{2})}{\gamma_2} = 4.08 \text{ m}$$

$$K_{a2} = 0.49$$

At top of top layer $P_a = 2c_1 \sqrt{K_{a1}} = 2 \times 20 \sqrt{\frac{1}{3}} = 23.1$

At bottom of top layer $P_a = K_{a1} \gamma_1 H_1 - 2c_1 \sqrt{K_{a1}}$
 $= \frac{1}{3} \times 24 \times 6 - 2 \times 20 \sqrt{\frac{1}{3}} = 24.9$

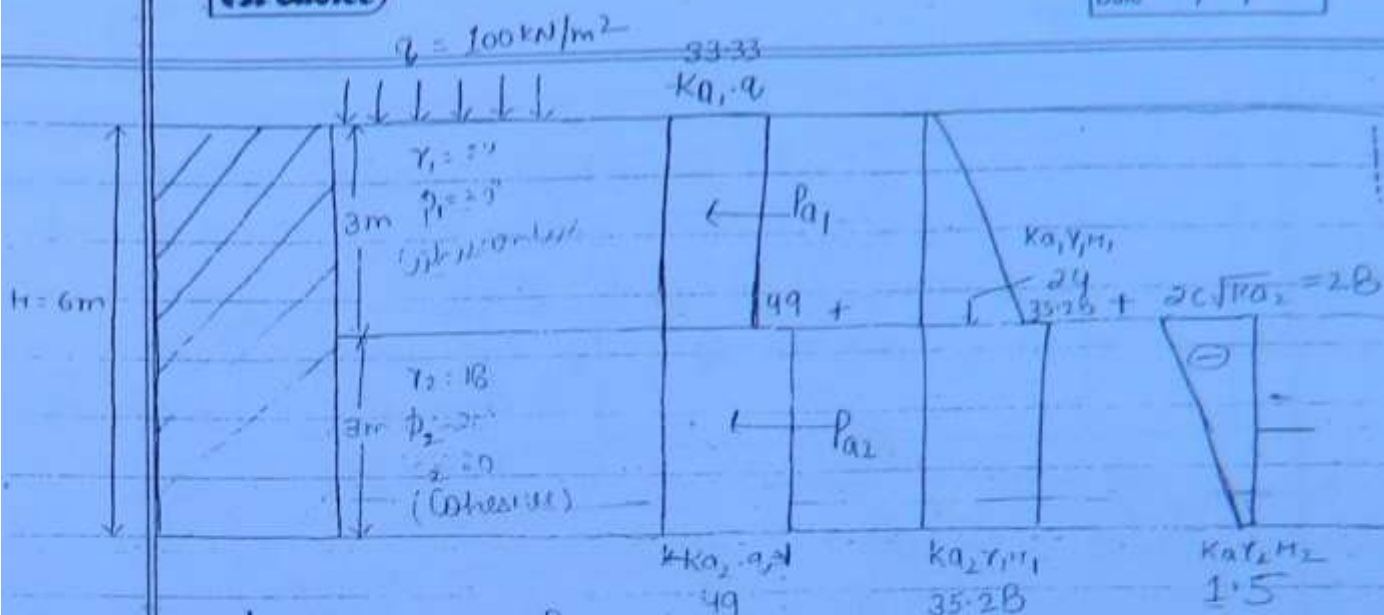
Top layer will be treated as surcharge over bottom layer
 Surcharge $q = \gamma_1 H_1 = 24 \times 6 = 144 \text{ kN/m}^2$

$$P_a = K_{a2} q = 0.49 \times 144 = 70.56 \text{ kN/m}^2$$

In diagram II

Top layer $= 2c_2 \sqrt{K_{a2}} = 2 \times 30 \sqrt{0.49} = 42$

bottom $= K_{a2} \gamma_2 H_2 - 2c_2 \sqrt{K_{a2}}$
 $= 0.49 \times 21 \times 6 - 42 = 19.71$



$$K_{a1} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

$$K_{a2} = \frac{1 - \sin 20^\circ}{1 + \sin 20^\circ} = 0.49$$

$$Z_c = \frac{2c_2}{\gamma_2} \tan\left(45^\circ + \frac{\phi_2}{2}\right) = 3.17\text{m}$$

at Stress on top of Clay

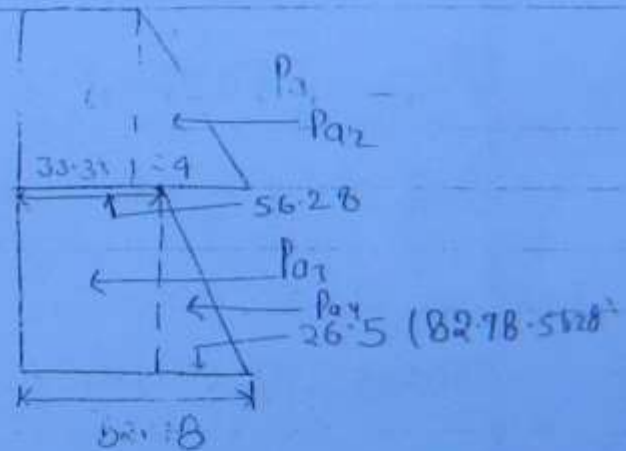
$$P_{a1} = 30.33 \times 3 + \frac{1}{2} \times 24 \times 3$$

$$= 49 + 35.28 - 28 = 56.28$$

at Stress on bottom of Clay

$$49 - 35.28 - 15$$

$$= 8.78$$



$$P_{a1} = 30.33 \times 3 = 90.99 = 10\text{ kN/m acts at } 4.5\text{ from base}$$

$$P_{a2} = \frac{1}{2} \times 24 \times 3 = 36\text{ acts at } 4\text{ m from base}$$

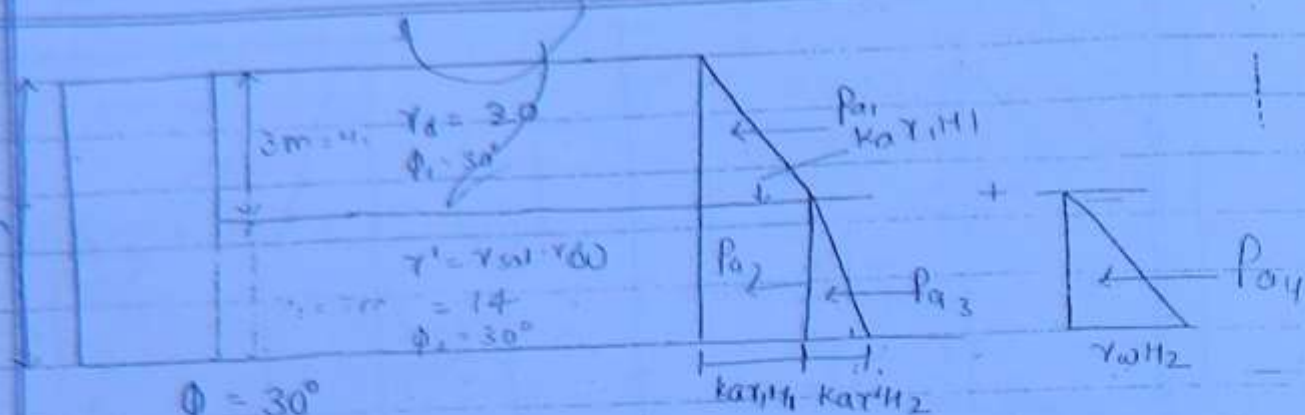
$$P_{a3} = 56.28 \times 3 = 168.84\text{ acts at } 1.5\text{m}$$

$$P_{a4} = \frac{1}{2} \times 26.5 \times 3 = 39.75\text{ kN/m acts at } 1\text{m}$$

$$\text{Total } P_a = 335.58$$

$$H = 2.57\text{m}$$

H = 5m



$$\phi = 30^\circ$$

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1}{3}$$

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$$P_{a1} = \frac{1}{2} K_a \gamma_1 H_1^2 = \frac{1}{2} \times \frac{1}{3} \times 20 \times 3^2$$

= 30 kN/m acts at 4m from base

$$P_{a2} = K_a \gamma_1 H_1 H_2 = \frac{1}{3} \times 20 \times 3 \times 2 = 40 \text{ kN/m acts at } 1.5 \text{ m from base}$$

$$P_{a3} = \frac{1}{2} K_a \gamma'_2 H_2^2 = \frac{1}{2} \times \frac{1}{3} \times 18 \times 2^2 = 12 \text{ kN/m acts at } 1 \text{ m from base}$$

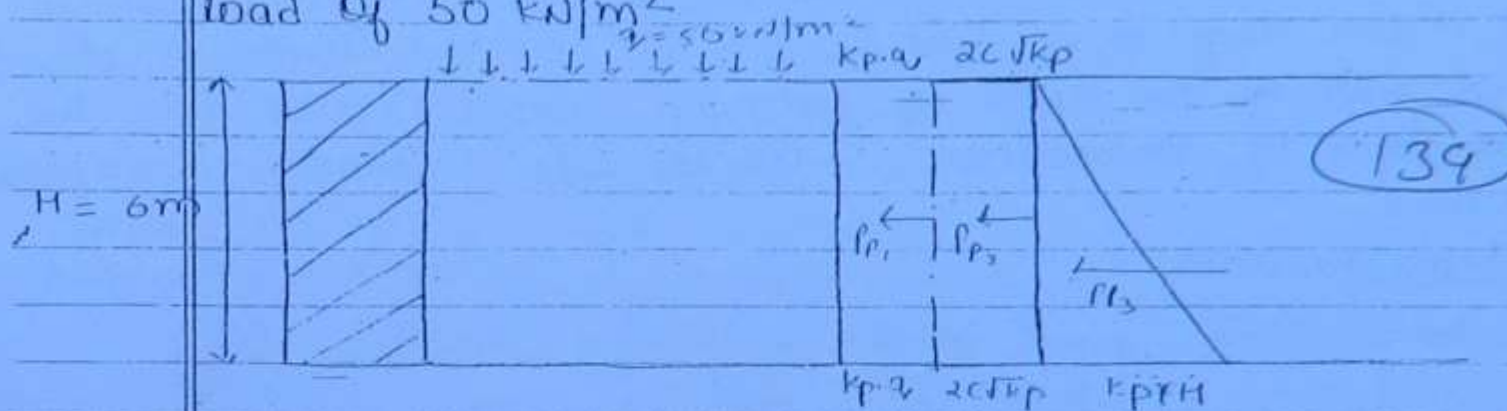
$$P_{a4} = \frac{1}{2} \gamma_w H_2^2 = \frac{1}{2} \times 10 \times 2^2 = 20 \text{ kN/m acts at } 1 \text{ m from base}$$

$$\text{Total } P_p = 156 \text{ kN/m}$$

$$\bar{H} = \frac{30 \times 4 + 40 \times 1.5 + 12 \times 1 + 20 \times 1}{156} = 1.77 \text{ m from base}$$

Ques A retaining wall 6m high retains sand $\phi = 30^\circ$ & $\gamma = 24 \text{ kN/m}^3$ upto depth of 3m from top. From 3m to 6m material is cohesionless $c = 0$ is 20 kN/m^2 $\phi = 25^\circ$ & $\gamma = 16 \text{ kN/m}^3$. A uniform surcharge of 100 kN/m^2 (also) act on the top surface. Determine the total lateral pressure acting on the wall & its pt. of application.

Ques - A retaining wall 6m height is a smooth vertical (passive) pushed against a soil mass having $c' = 40 \text{ kPa}$, $\phi' = 19^\circ$ & $\gamma = 19 \text{ kN/m}^3$. Using Rankine's theory compute the pressure & pt. of application of resultant thrust. The horizontal soil surface carries a uniform load of 50 kN/m^2 .



$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} = 1.7$$

$$q = 50 \text{ kN/m}^2$$

$$P_{p1} = K_p \cdot q \cdot H = 510 \text{ kN/m} \quad \text{acts at } 3 \text{ m from base}$$

$$P_{p2} = 2c\sqrt{K_p} \cdot H = 625.84 \text{ kN/m} \quad \text{acts at } 3 \text{ m from base}$$

$$P_{p3} = \frac{1}{2} K_p \gamma H^2 = 1.7 \times 19 \times 6^2 = 581.4 \text{ kN/m} \quad \text{acts at } 2 \text{ m from base}$$

$$\text{Total } P_p = P_{p1} + P_{p2} + P_{p3} = 1717.24 \text{ kN/m}$$

$$\bar{H} = \frac{510 \times 3 + 625.84 \times 3 + 581.4 \times 2}{1717.24} = 2.66 \text{ m}$$

Ques - A retaining wall 6m high retains sand with friction 30° & $\gamma = 24 \text{ kN/m}^3$ (γ_{sat}). The top 3m depth is unit wt 20 kN/m^3 & bottom 3m is submerged. Find total active thrust on the wall per m length. Line of action. Assume there is no water on or side of wall.

$$\text{Net pressure at B} = P_p - P_a$$

$$q + \gamma d + 2c - \gamma d + 2c$$

$$q + 4c$$

140

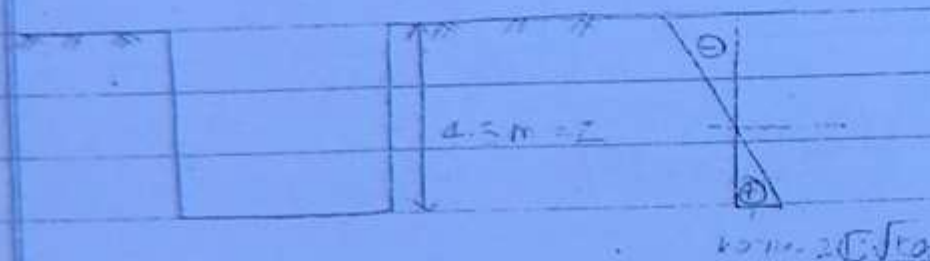
Ques Determine the stresses at the top & bottom of a vertical cut at 4.5m deep in a soil $c = 16^\circ$, $c = 19.1 \text{ kN/m}^2$, $\gamma = 18.5 \text{ kN/m}^3$. What will be the depth of potential cracks & also find the max depth of excavation $\leq$ can be left unsupported without failure.

$$c = 19.1 \text{ kN/m}^2$$

$$\gamma = 18.5 \text{ kN/m}^3$$

$$\phi = 16^\circ$$

Sol Active Condition



$$\text{Depth of } Z_c = \frac{2c}{\gamma} \tan(45 + \frac{\phi}{2})$$

$$= \frac{2 \times 19.1}{18.5} \tan(45 + \frac{16}{2})$$

$$Z_c = 2.740$$

$$\text{Max. Ht. } \leq \text{ can be left unsupported} = H_c = 2Z_c = 5.4 \text{ m}$$

$$\text{F.O.S} = 1.2 = \frac{H_c}{H} = \frac{5.4}{H}$$

$$P_a = -2c\sqrt{K_a}$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.57$$

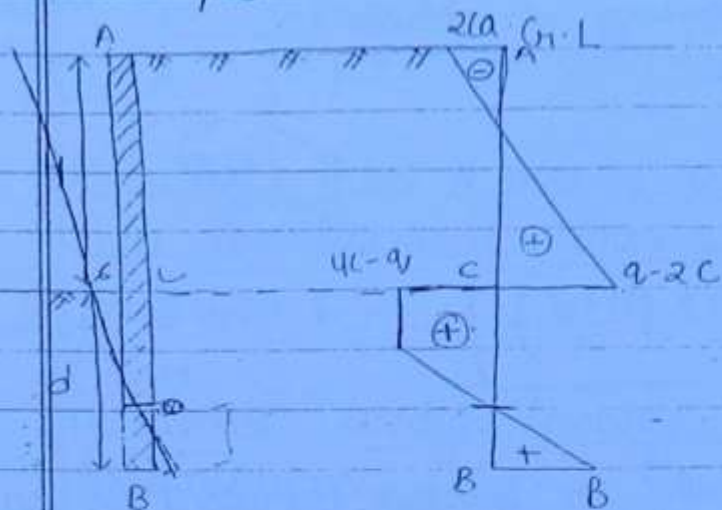
$$\text{top } P_a = -2 \times 19.1 \times \sqrt{0.57} = -28.54$$

$$\text{(max)} P_a = K_a \times H - 2c\sqrt{K_a}$$

$$0.57 \times 18.5 \times 4.5 - 2 \times 19.1 \sqrt{0.57}$$

$$= 118.46 \text{ kN/m}^2$$

Sheet pile wall in Cohesive Soil



(142)

Right face AO - In Active State & OB in passive State
 * Left face CO - passive state & left face OB in Active State

For pure clays $\phi = 0$
 $K_a = 1$

$$P_a = K_a \gamma \cdot z + 2c\sqrt{K_a}$$

$$P_{aA} = -2c$$

$$P_{aC} \text{ (on right face.)} = 1 \cdot \gamma H - 2c\sqrt{1} \quad [\gamma H = q]$$

$$= \gamma H - 2c$$

$$= q - 2c$$

$$P_{pC} \text{ (on left face)} = K_p \cdot \gamma \cdot z + 2c\sqrt{K_p}$$

$$K_p = 1 \quad \Delta \quad z = 0$$

$$P_{pC} \text{ (left)} = 2c$$

$$\text{Net pressure at C} = p_p - \text{Acti } P_a$$

$$2c - q - 2c = 4c - q$$

$$P_B \text{ (right face)} \quad P_p = K_p \cdot \gamma \cdot (H+d) + 2c\sqrt{K_p}$$

$$\gamma H + \gamma d + 2c$$

$$q + \gamma d + 2c$$

$$P_B \text{ (left face)} \quad P_a = K_a \gamma d - 2c\sqrt{K_a}$$

$$\gamma d - 2c$$

If F.O.S against overturning F then

$$F.O.S(F) = \frac{M_R}{M_o}$$

(142)

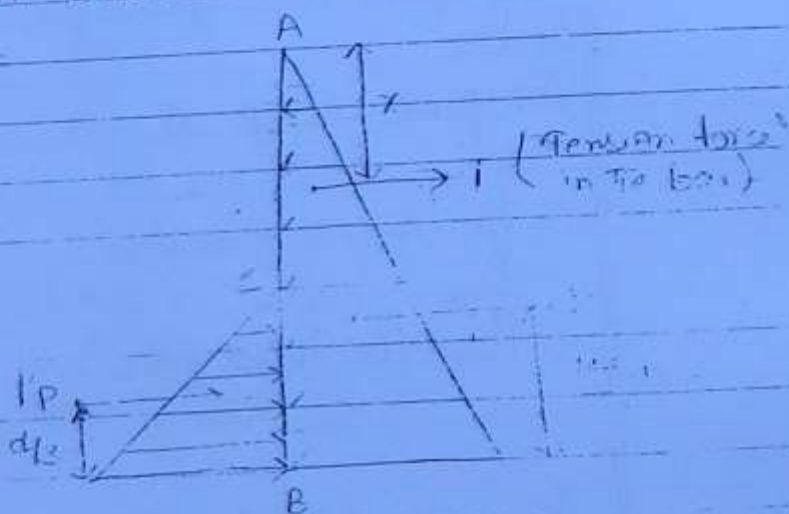
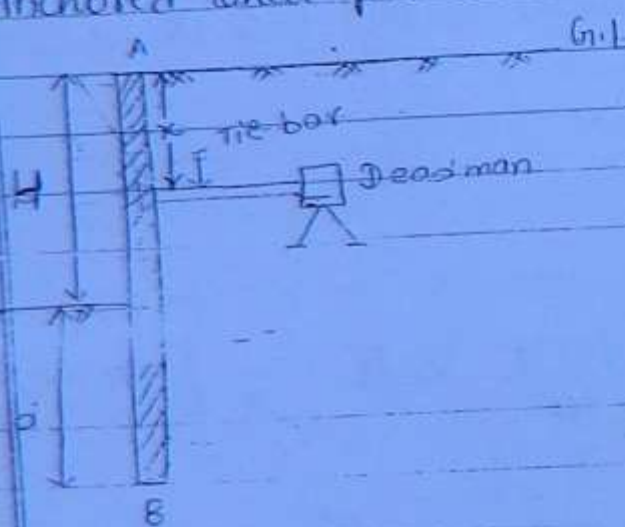
then $\frac{M_R}{F} = M_o$

$$\frac{P_p \times d/3}{F} = \frac{P_a \times (H+d)/3}{F}$$

(d can be found from this)

The depth Computed Should be further increased by 20 to 25%.

Anchored Sheet pile wall in Cohesionless Soil



$$\sum F_x = 0$$

$$T + P_p - P_a = 0$$

$$T + \frac{1}{2} k_p \gamma d^2 - \frac{1}{2} k_a \gamma (H+d)^2 = 0 \quad \text{--- (1)}$$

T - Tension force ~~per unit~~ per unit length of tie bar
 P_a - Total active thrust per unit length
 P_p - Passive thrust per unit length

$$\sum M_B = 0$$

$$P_p \times \frac{d}{3} + T \times (H+d-x) - P_a \times \left(\frac{H+d}{3} \right) = 0 \quad \text{--- (2)}$$

From 1 & 2 we can find T & d can be found.

Total passive Pressure $P_p = P_{p1} + P_{p2}$
 $P_p = \frac{1}{2} k p \gamma H^2 + 2cH\sqrt{k_p}$

(143)

Note that - active & passive pressure diagrams in Cohesive soils are different whereas in Cohesionless soils are similar.

Coulomb's Wedge theory

Assumptions

- (i) Coulomb Consider equilibrium of entire wedge whereas
- (ii) Rankine Consider element equilibrium
- (iii) The backfill is dry, homogeneous, cohesionless & isotropic
- (iv) The back of wall is assumed rough. c may be vertical or inclined.
- (v) Failure is assumed on a plane c is called rupture surface & failure is 2-dimensional.
- (vi) Wedge acts as a rigid body & friction is distributed uniformly on the rupture surface
- (vii) The position & dirⁿ of the resultant thrust is known c acts at $\frac{H}{3}$ from base at an angle δ to the normal of the wall.

NOTE - The failure surface (rupture plane) is considered linear at angle ϕ however actual surface in passive case is spiral $\therefore$ for passive condition this theory is less accurate & errors get further $\uparrow$ $\propto \uparrow \phi$ in friction angle

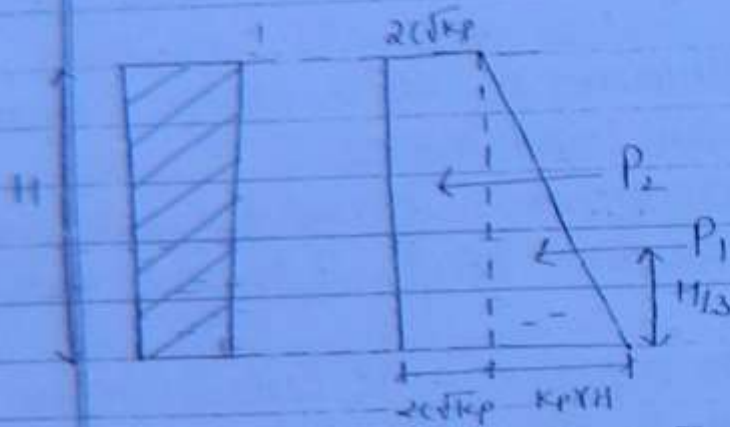
Special case - For pure clays $\phi = 0$

$$z_c = \frac{2c}{\gamma}$$

$$H_c = \frac{4c}{\gamma}$$

(744)

Passive pressure on Cohesive Soils



In passive state $\sigma_h > \sigma_v$

$$\begin{array}{c} \downarrow \quad \downarrow \\ \sigma_1 \quad \sigma_3 \\ \downarrow \quad \downarrow \\ (P_p) \end{array}$$

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$P_p = \gamma z \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

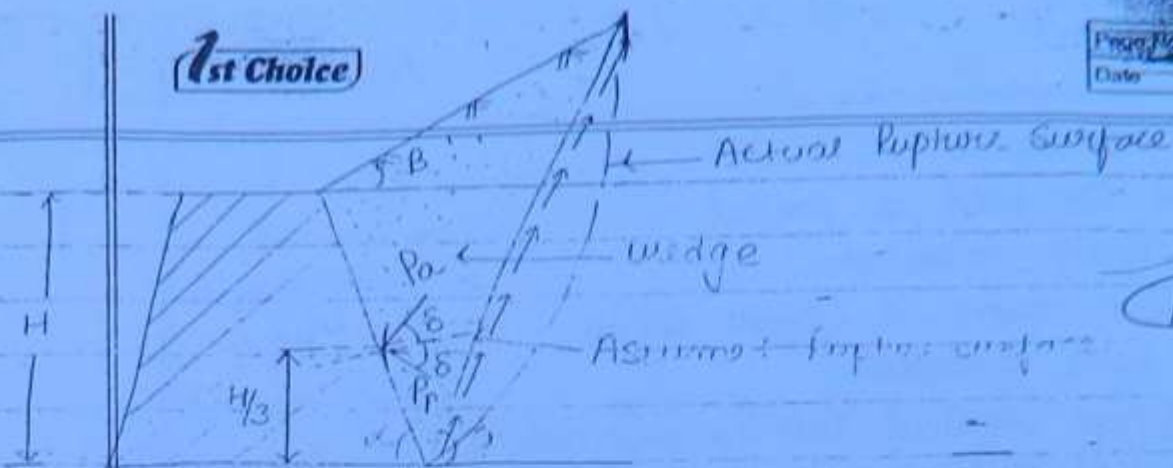
$$P_p = K_p \gamma z + 2c \sqrt{K_p}$$

$$\text{At } z = 0 \quad P_p = 2c \sqrt{K_p}$$

$$\text{At } z = H \quad P_p = K_p \gamma H + 2c \sqrt{K_p}$$

Total passive force per unit length of wall
 $P_p = \frac{1}{2} K_p \gamma H^2$ acts at $\frac{H}{3}$ from base

$P_p = 2cH\sqrt{K_p}$ acts at $\frac{H}{3}$ from base



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Total $P_a = \frac{1}{2} K_a \gamma H^2$ -- Acts at $\frac{H}{3}$ from the base

$P_p = \frac{1}{2} K_p \gamma H^2$ -- Acts at $\frac{H}{3}$ from the base

$$K_a = \left[\frac{\sin(\alpha + \phi) / \sin \alpha}{\sqrt{\sin(\alpha - \delta)} + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\sin(\alpha + \beta)}}} \right]^2$$

$$K_p = \left[\frac{\sin(\alpha - \phi) / \sin \alpha}{\sqrt{\sin(\alpha + \delta)} + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi + \beta)}{\sin(\alpha + \beta)}}} \right]^2$$

α = Angle of back of wall $\bar{c}$ the horizontal

β = Angle of back fill $\bar{c}$ the horizontal

ϕ = friction angle of soil

δ = friction angle betⁿ wall & soil

$$\delta < \alpha = \frac{\phi}{3} < \delta < \frac{3\phi}{2}$$

$$\delta \approx \phi$$

imp
> Case If wall is vertical ($\alpha = 90^\circ$), back fill is horizontal ($\beta = 0^\circ$)
friction angle of wall is equal to friction angle of soil ($\delta = \phi$). Then

$$K_a = \frac{\cos \phi}{[1 + \sqrt{2} \sin \phi]^2} ; K_p = \frac{\cos \phi}{[1 - \sqrt{2} \sin \phi]^2}$$

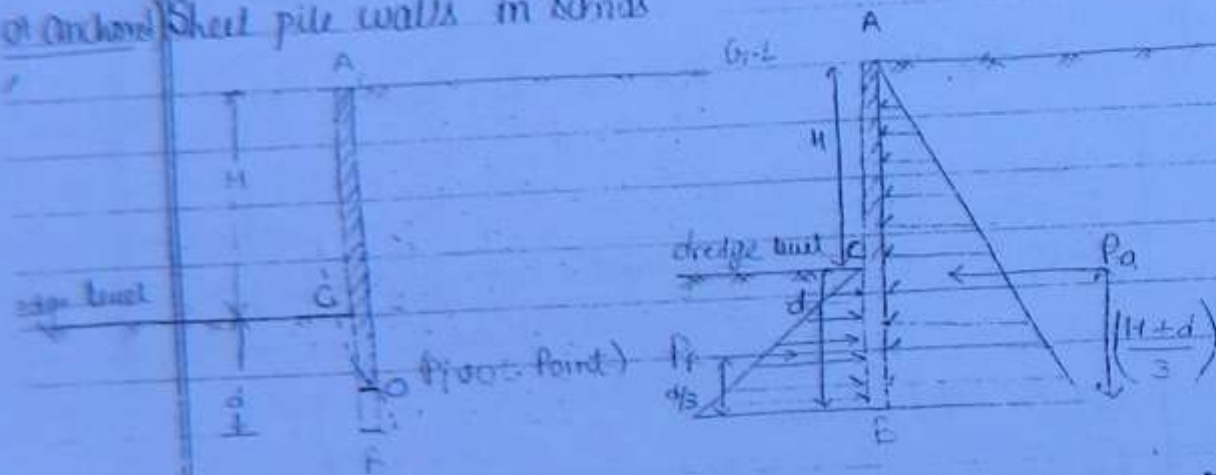
Sheet pile walls

These are used to protect :-

- Water front structure.
- Division dams & Cofferdams.
- River bank protection.
- To support vertical cuts in excavation.
- To reduce uplift pressure below hydraulic structures.

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Sheet pile walls in bents



The actual rotation of wall is abt pivot point D. But for the purpose of Analysis & Computation the wall is assumed to be rotated abt ~~point~~ base B. Hence right face of the Cantilever wall will be in active state and left face BC will be in passive state. The active pressure on right face of AB is producing overturning moment & passive pressure on left face of BC produces restoring moment.

The appropriate depth below dredge level may be found by summation of moment abt B is zero

$$\sum M_B = 0$$

Restoring moment $P_p \times \frac{d}{3}$ Overturning moment M_o
 $P_a \left(\frac{H+d}{3} \right)$

$$\frac{1}{2} K_p \gamma d^3 \times \frac{d}{3} = \frac{1}{2} K_a \gamma (H+d)^2 \left(\frac{H+d}{3} \right)$$

$$a = K_a \gamma H - 2c\sqrt{K_a}$$

$$b = 2c\sqrt{K_a}$$

$$\text{Height} = H - 2Z_c$$

(147)

Total Active thrust on unit length of wall

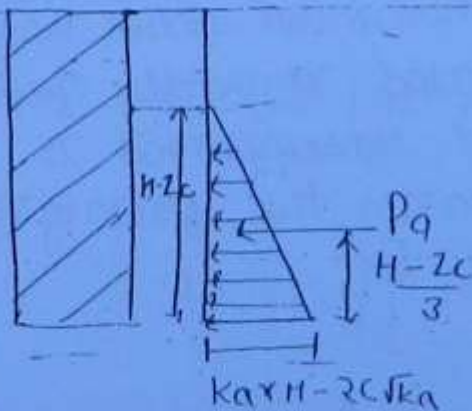
$$P_a = \frac{1}{2} (a+b)(H-2Z_c) \quad \text{act at } \bar{H}$$

$$\bar{H} = \frac{a+2b}{a+b} \cdot \frac{(H-2Z_c)}{3}$$

$$= \frac{1}{2} (K_a \gamma H - 2c\sqrt{K_a} + 2c\sqrt{K_a}) (H-2Z_c)$$

$$P_a = \frac{1}{2} K_a \gamma H^2 - 2cH\sqrt{K_a}$$

se - 2 When Cracks are developed - The negative pressure will not be developed hence net active thrust will be over the depth $H - Z_c$ from base.



$$P_a = \frac{1}{2} (K_a \gamma H - 2c\sqrt{K_a}) (H - Z_c) \quad \text{act at } \frac{H - Z_c}{3} \text{ from base}$$

$$P_a = \frac{1}{2} K_a \gamma H^2 - 2cH\sqrt{K_a} + \frac{2c^2}{\gamma}$$

At $z = 0$, at top

$$p_a = -2c\sqrt{K_a}$$

At $z = H$, $K_a \gamma H - 2c\sqrt{K_a}$ at baseAt $z = z_c$, $p_a = 0$

$$K_a \gamma z_c - 2c\sqrt{K_a} = 0$$

$$K_a \gamma z_c = 2c\sqrt{K_a}$$

$$z_c = \frac{2c\sqrt{K_a}}{K_a \gamma}$$

$$z_c = \frac{2c}{\sqrt{K_a} \gamma}$$

$$z_c = \frac{2c}{\sqrt{K_a} \gamma}$$

$$z_c = \frac{2c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right)$$

 z_c = Depth of potential cracks. H_c = Critical depth or Critical height. So $H_c = 2z_c$

$$H_c = \frac{4c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right) \quad \text{Imp.}$$

It is that depth at which net horizontal active thrust is zero.

If a Cohesive Soil is excavated vertically then the max.

Vertical depth z_c can be left unsupported is H_c . If H is Actual depth of excavation the factor of safety

against Slope failure will be

$$F.O.S = \frac{H_c}{H}$$

Over the depth z_c from ground surface, Active pressure is negative (tension) & pulls the wall toward the soil.

If tension cracks get developed then the pulling effect will not be available. Hence, total Active earth pressure

will increase

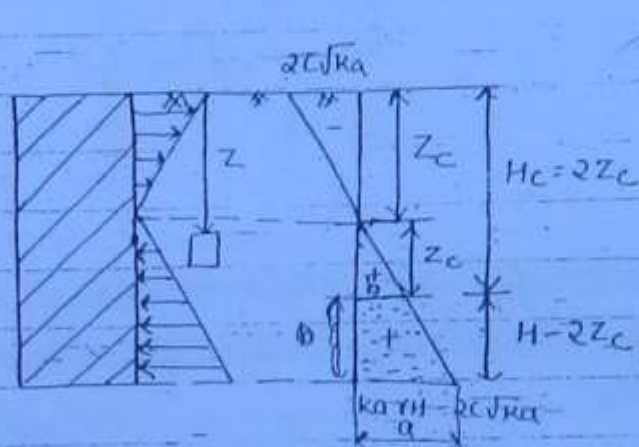
Use 1. Total Active earth pressure when tension cracks are not developed

Imp

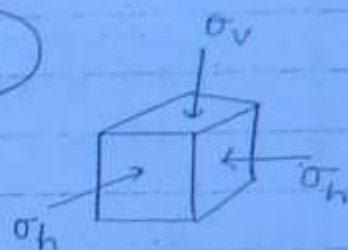
Active earth pressure on Cohesive Soils

Though Rankine theory is given for Cohesionless soil but it is extended for Cohesive Soils by Bell.

Cohesive soils are self-supporting due to interaction between molecules (cohesion) hence active earth pressure is reduced whereas passive earth pressure is ↑



(149)



$$P_a = \frac{\sigma_v}{1 - \frac{ka\sigma_v}{\sigma_v}}$$

In Active State $\sigma_v > \sigma_h$

$$\sigma_v = \gamma \cdot z$$

P

$$\sigma_1 = \sigma_3 \tan^2(45^\circ + \frac{\phi}{2}) + 2c \tan(45^\circ + \frac{\phi}{2})$$

$$\sigma_1 = \sigma_3 \tan^2(45^\circ + \frac{\phi}{2}) + 2c \tan(45^\circ + \frac{\phi}{2})$$

$$\gamma \cdot z = P_a \tan^2(45^\circ + \frac{\phi}{2}) + 2c \tan(45^\circ + \frac{\phi}{2})$$

$$P_a = \frac{\gamma \cdot z}{\tan^2(45^\circ + \frac{\phi}{2})} - \frac{2c}{\tan(45^\circ + \frac{\phi}{2})}$$

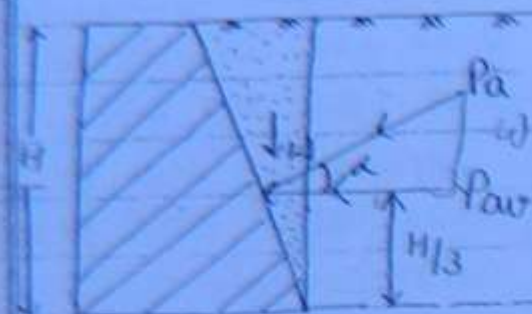
$$\tan^2(45^\circ + \frac{\phi}{2}) = N\phi \text{ (influence value)}$$

vi) P

$$\frac{1}{ka}$$

$$P_a = \frac{\gamma \cdot z}{N\phi} - \frac{2c}{\sqrt{N\phi}} = ka\gamma \cdot z - 2c\sqrt{ka}$$

(180)



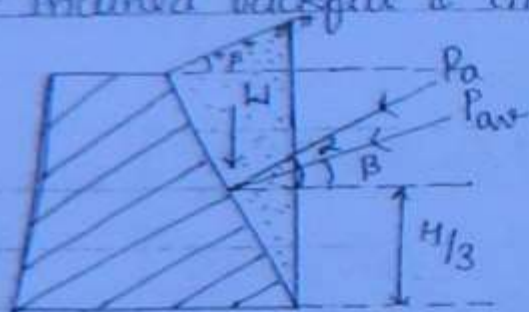
$$P_a = \sqrt{P_{av}^2 + L^2}$$

$$\tan \alpha = \frac{L}{P_{av}}$$

$$P_{av} = \frac{1}{2} K_a \gamma H^2$$

Let P_{av} is lateral earth pressure on imaginary vertical wall & let L is weight of soil acting vertically down on inclined back of wall.

Case 5 - Inclined backfill & inclined back of wall.



Let P_{av} is lateral earth pressure on imaginary vertical wall due to inclined backfill & acts at an angle β to horizontal. Let L is weight of soil acting vertically down on vertical inclined on back of wall.

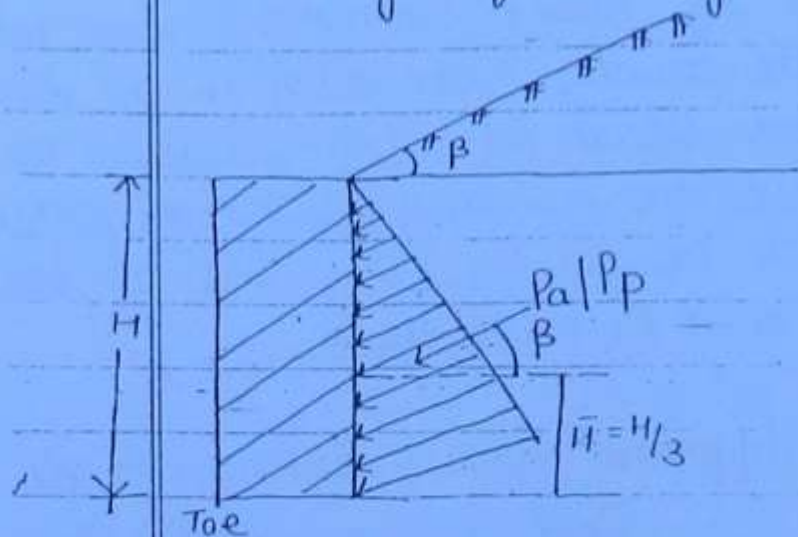
$$\begin{aligned} \sum F_H &= P_{av} \cos \beta & (\text{Horizontal Component of Thrust}) \\ \sum F_V &= P_{av} \sin \beta + L & (\text{Vertical " " "}) \end{aligned}$$

$$P_a = \sqrt{(\sum F_H)^2 + (\sum F_V)^2}$$

$$P_a = \sqrt{(P_{av} \cos \beta)^2 + (P_{av} \sin \beta + L)^2}$$

$$\tan \alpha = \frac{\sum F_V}{\sum F_H} = \frac{P_{av} \sin \beta + L}{P_{av} \cos \beta}$$

Case 4 - Effect of inclined backfill (Inclined Surcharge)
 let angle of surcharge $\bar{\alpha}$ horizontal is β



(157)

The resultant active or passive thrust acts $\parallel$ to the Surcharge
 At an angle β to the horizontal at $\frac{H}{3}$ from the base.

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

$$= P_b = \frac{1}{2} K_p \gamma H^2$$

$$K_a = \cos \beta \left[\frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \right]$$

If $\beta = 0$ then previous result is obtained

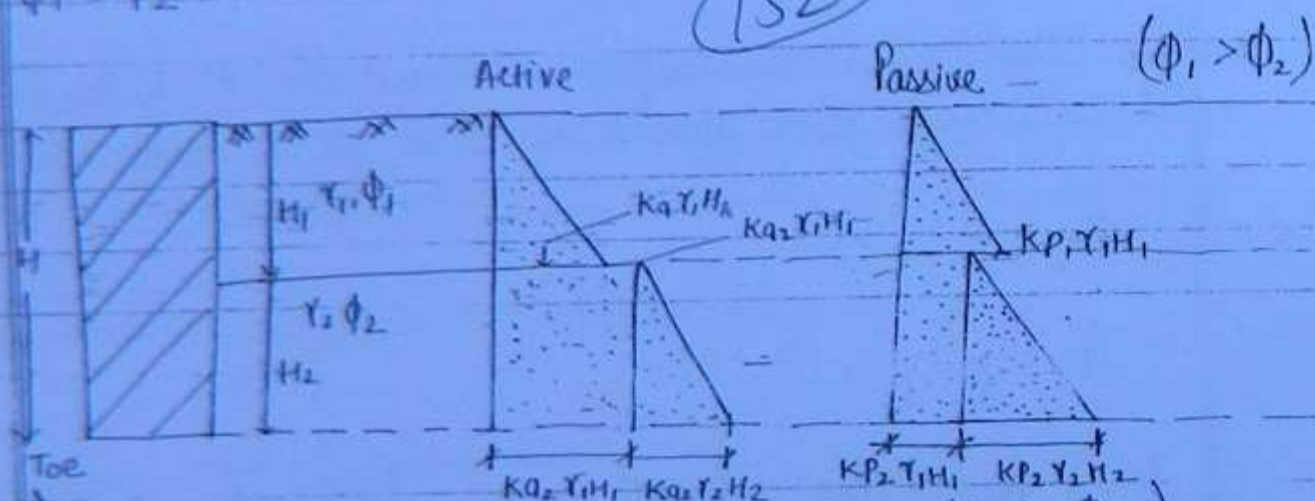
$$K_p = \cos \beta \left[\frac{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}} \right]$$

$$K_a \times K_p = \cos^2 \beta$$

Case 5 Horizontal backfill & inclined back of wall.

NOTE If L.T is present then grain to grain contact pressure below the LT should be considered by taking submerged unit wt. and pore pressure will be equally transmitted in all dirn under pascal's law hence below L.T pore pressure on the wall will be hydrostatic & same in active, passive & rest condn. If L.T is present on both side of the wall then pore pressure will be neutralized.

Ex. 1.3 Backfill contains two different soils ϕ_1 & ϕ_2



If $\phi_1 > \phi_2 \Rightarrow K_{a1} < K_{a2}$ $K_a = \tan^2 (45^\circ - \phi/2)$
 $K_{p1} > K_{p2}$ $K_p = \tan^2 (45^\circ + \phi/2)$

NOTE The top layer should be treated as surcharge over bottom layer

$$P_{a1} = \frac{1}{2} K_{a1} \gamma_1 H_1^2 \quad \bar{H}_1 = H_2 + \frac{H_1}{3}$$

$$P_{a2} = K_{a2} \gamma_1 H_1 H_2 \quad \bar{H}_2 = \frac{H_2}{2}$$

$$P_{a3} = \frac{1}{2} K_{a2} \gamma_2 H_2^2 \quad \bar{H}_3 = \frac{H_2}{3}$$

Total active thrust per unit length of wall

$$P_{a1} = (K_a \gamma H) \times 1 \quad \text{acts at } \frac{H}{2} \text{ from base } (\bar{H}_1)$$

$$P_{a2} = \frac{1}{2} K_a \gamma H^2 \quad \text{acts at } \frac{H}{3} \text{ from base } (\bar{H}_2)$$

$$\text{Total } P_a = P_{a1} + P_{a2}$$

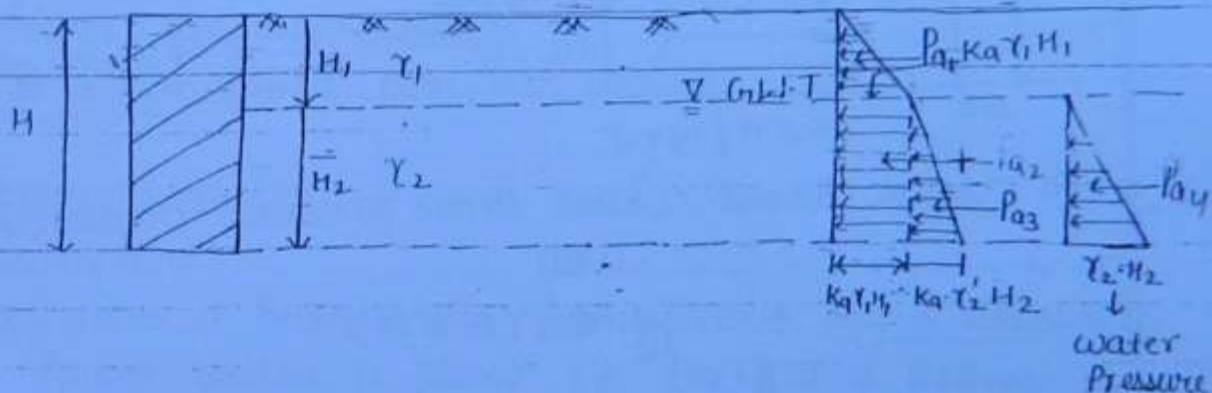
line of action of resultant from base

$$\bar{H} = \frac{P_{a1} \cdot \bar{H}_1 + P_{a2} \cdot \bar{H}_2}{P_{a1} + P_{a2}}$$

(153)

Case 2 Effect of water table

let w.t is present at a depth of H_1 from ground level



$$P_{a1} = \frac{1}{2} K_a \gamma_1 H_1^2 \quad \bar{H} = H_2 + \frac{H_1}{3} \text{ from base}$$

$$P_{a2} = K_a \gamma_1 H_1 H_2 \quad \bar{H} = \frac{H_2}{2} \quad "$$

$$P_{a3} = \frac{1}{2} K_a \gamma_2' H_2^2 \quad \bar{H} = \frac{H_2}{3} \quad "$$

$$P_{a4} = \frac{1}{2} \gamma_2 H_2^2 \quad \bar{H} = \frac{H_2}{3} \quad "$$

$$P_a = \sum P_a$$

$$\bar{H} = \frac{\sum P_a \bar{H}}{\sum P_a}$$

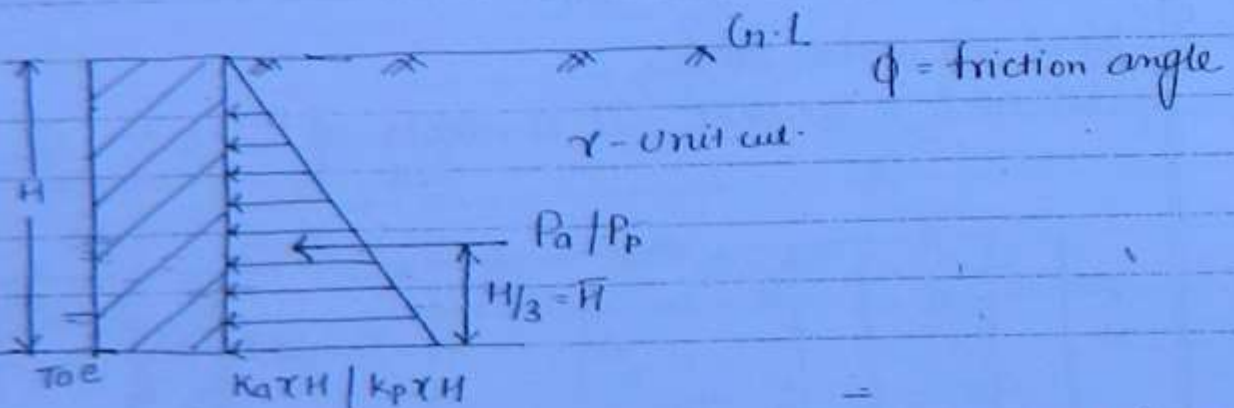
rupture plane is plastic failure.

(vi) The stress relationship for any two element adjacent to the wall is similar but diff^r than the stress distribution in adjacent element which is outside the wall.

(vii) Rankine Consider Element failure whereas Coulomb Consider wedge failure.

(154)

Total active & passive earth pressure on unit length of wall.



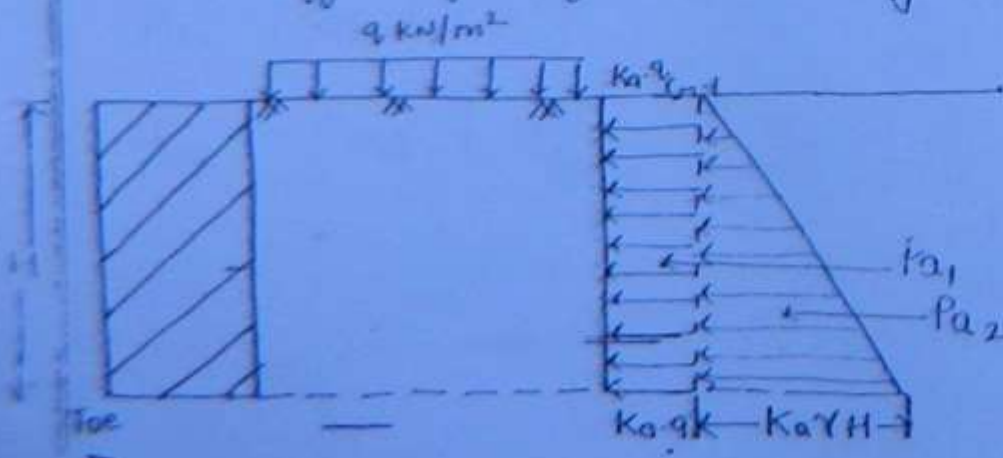
P_a = Active thrust = Total active pressure on unit length of wall

$$= \frac{1}{2} \times K_a \gamma H \times H \times 1$$

$$= \frac{1}{2} K_a \gamma H^2 \quad \text{--- Acts at } \frac{H}{3} \text{ from base}$$

P_p = Passive thrust = $\frac{1}{2} K_p \gamma H^2$... Acts at $\frac{H}{3}$ from base

Special Case 1 - Effect of uniform surcharge as shown in fig



Earth pressure theories

1. Rankine's theory
More versatile & widely adopted due to its simplicity.

(153)

2. Coulomb's theory
More realistic for concrete retaining wall especially in active state but results in passive states are erroneous.

Culmann's

3. ~~Culmann's~~ Graphical Method

It is based on Coulomb's theory & Coulomb's theory is a modified version of Rankine theory.

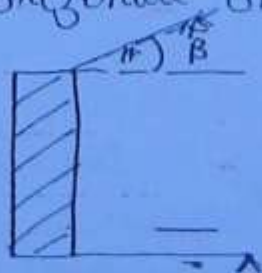
4. Rebhan Graphical Method

It is also called Poncetele Method

RANKINE'S THEORY (for $c=0$)

Assumptions

- (i) Soil mass is semi-infinite, homogenous, dry & cohesionless.
- (ii) The ground surface is plane i.e. backfill is plane c may be horizontal or inclined.

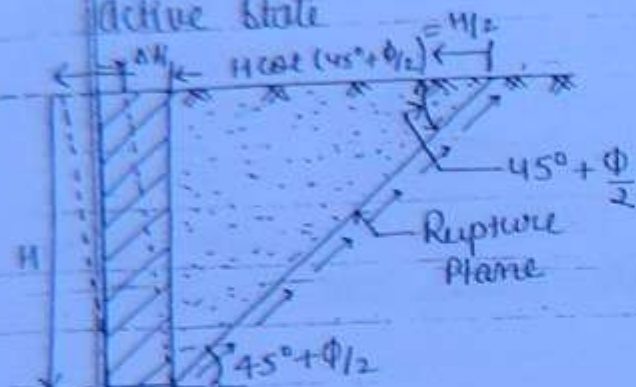


- (iii) The face of the wall in contact with backfill is vertical & smooth.
- (iv) The wall yields at the base sufficiently to mobilise active & passive earth pressure fully.
- (v) Strains in soil are considered when soil reaches into plastic equilibrium. It means failure due to shear in

Active & passive States

If wall has tendency to move away from fill then it is

Active State



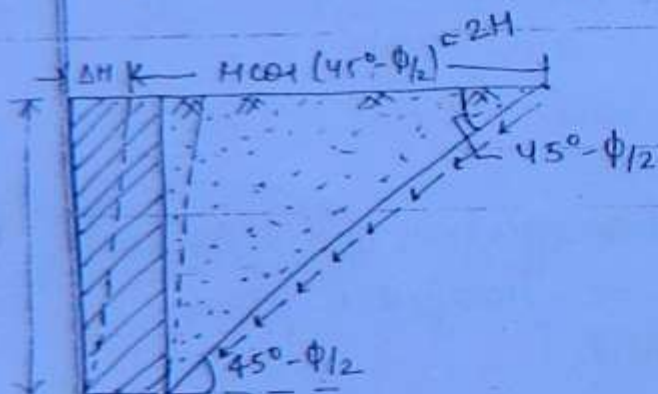
For full active condition to develop/mobilize

$$\text{Strain} = \frac{\Delta H}{H} \times 100 \geq 0.1 \text{ to } 0.2\% \text{ for dense sand}$$

$$\frac{\Delta H}{H} \times 100 \geq (0.2 \text{ to } 0.5)\% \text{ for loose sand}$$

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In passive state the wall is pushed against the backfill i.e. towards the backfill. The angle of rupture surface to the horizontal is $45^\circ - \frac{\Phi}{2}$. Relatively more horizontal strain is req. to mobilize full passive state.



$$\frac{\Delta H}{H} \times 100 \geq 2\% \text{ for dense sand}$$

$$\frac{\Delta H}{H} \times 100 \geq 10\% \text{ for loose sand}$$

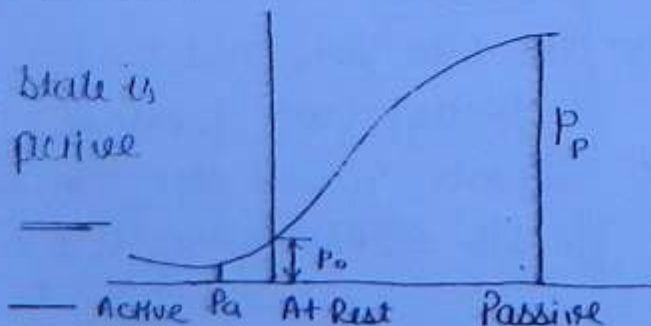
Acc. to Rankine $K_A = \frac{1 - \sin \Phi}{1 + \sin \Phi} = \tan^2 \left(45^\circ - \frac{\Phi}{2} \right)$

$$K_P = \frac{1 + \sin \Phi}{1 - \sin \Phi} = \tan^2 \left(45^\circ + \frac{\Phi}{2} \right)$$

$$K_A = \frac{1}{K_P}$$

$$\Rightarrow K_A \cdot K_P = 1$$

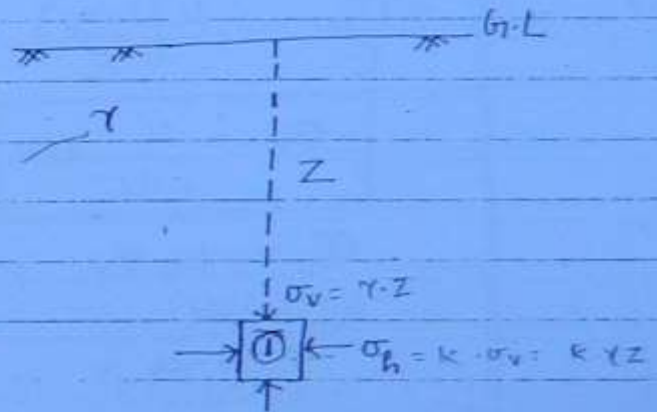
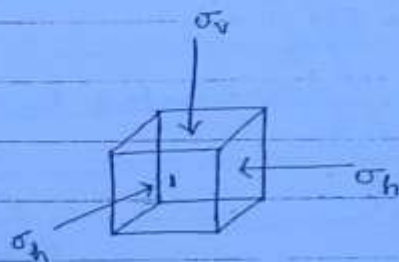
ΔH req. to mobilize passive state is much greater than that for active state.



Stress Isobars are the lines joining the points of equal vertical stress. Generally effect of vertical stress is neglected beyond the zone of $\rho \approx 0.29$ isobar. The approximate depth of 20% isobar is $1.5B$ where B is the width of footing.

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LATERAL EARTH PRESSURE THEORY



$$\sigma_v = \gamma \cdot Z$$

$$\sigma_h = K \cdot \sigma_v \quad (\text{lateral earth pressure})$$

$$K = K_0 \quad \text{At rest Condition}$$

$$K = K_a \quad \text{At Active Condition}$$

$$K = K_p \quad \text{At Passive Condition}$$

$$K = \text{lateral earth pressure Coefficient in evaluation.}$$

$$\therefore K_a < K_0 < K_p$$

the earth pressures are statically indeterminate and hence pose problems in evaluation.

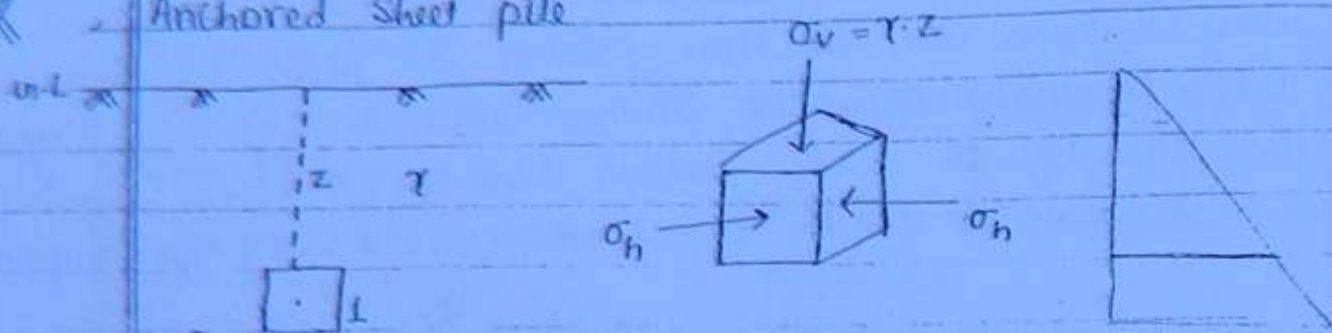
In order to support lateral earth pressure retaining walls & sheet pile walls are used. Retaining walls may be classified as —

- (i) Gravity Retaining wall
- (ii) Cantilever Retaining wall
- (iii) Counterfort Retaining wall
- (iv) Buttress Retaining wall

Whereas Sheet pile walls may be classified as

- (i) Cantilever Sheet pile.

Anchored Sheet pile



At Rest Condition

Lateral Strain = 0 (at rest) (158)

$$\epsilon_h = 0$$

$$\frac{-\sigma_h}{E} - \mu \left(\frac{-\sigma_h}{E} \right) - \mu \left(\frac{-\sigma_v}{E} \right) = 0$$

$$\sigma_h = \frac{\mu}{1-\mu} \sigma_v$$

Saturated $k_0 = \frac{\sigma_h'}{\sigma_v'}$

effective

$$\sigma_h = k_0 \sigma_v \leftarrow \text{lateral pressure}$$

$$k_0 = \frac{\mu}{1-\mu}$$

If friction angle of soil is given (ϕ) then k_0 is defined as

$$k_0 = 1 - \sin \phi$$

k_0 is more or greater for loose sand (In loose sand ϕ is less) than that for Dense sand.

Type of soil

Dense sand

loose sand

N.C. clays

O.C clays

k_0

0.4 to 0.5

0.45 to 0.55

0.5 to 0.6

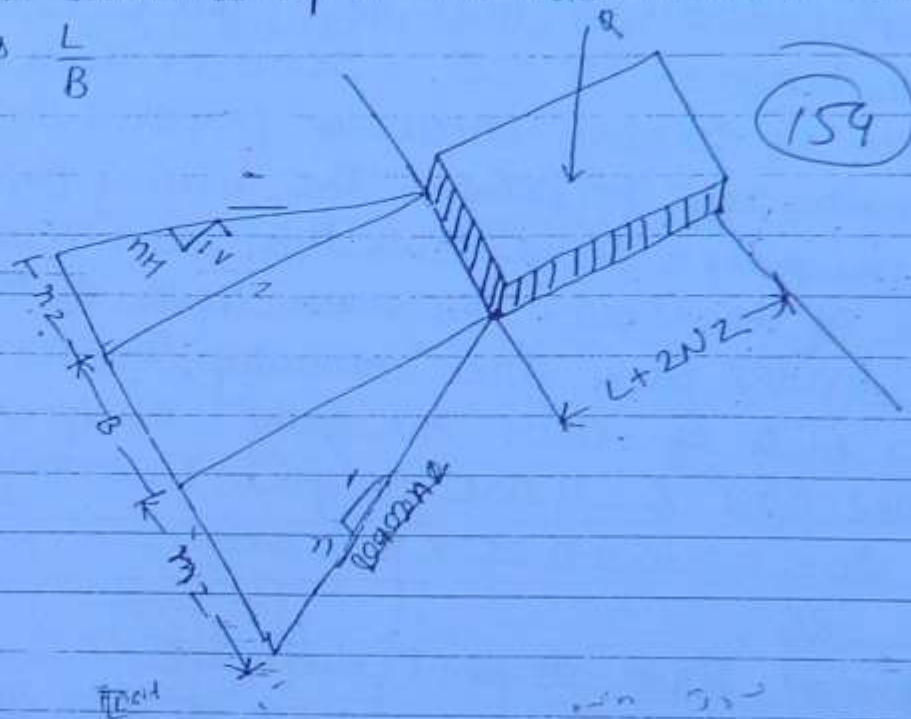
0.1 to 0.4

$$(k_0)_{\text{O.C. clays}} = (k_0)_{\text{N.C. clays}} \times \sqrt{O.C.R.}$$

$O.C.R. > 1$ for O.C. Soil.

(ii) Trapezoidal Method. On-load Strip Method.

Load is assumed to be distributed trapezoidal below the foundation with the depth. The loaded area at the level of footing is $\frac{L}{B}$



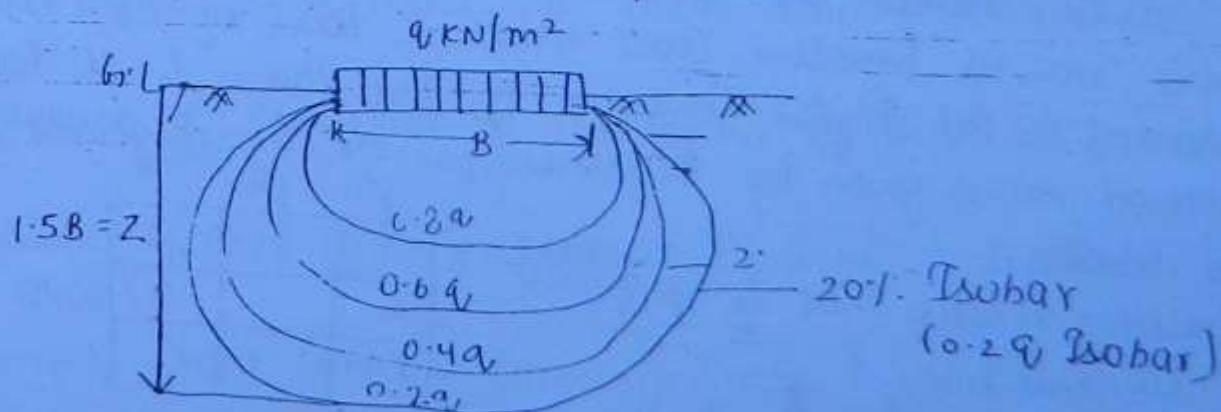
$$\sigma_z = \frac{Q}{(B + 2nz)(L + 2nz)} = \frac{q \times (L \times B)}{(B + 2nz)(L + 2nz)}$$

q = stress / pressure at footing level

Q = Total applied load

σ_z = vertical stress at depth z below footing.

Stress Isobar / Stress bulb / Pressure bulb.



Influence

$$\frac{1}{m \times n} = \frac{1}{10 \times 20} = 0.005$$

(160)

Let vertical stress is req. at P for c. o. is projected pt on the surface. $OP = Z = 5\text{cms}$ or 30ms . On this scale plan of loaded area is prepared on a tracing paper with ref. to point o. The tracing paper is placed over the Newmark's Chart such that point o coincides with the centre of the Chart. The area units of the Chart occupied by loaded plan are manually counted say N_A . If 20 units of area are fully occupied, 10 are occupied 50% & 4 are occupied Quantity.

$$20 + 0.5 \times 10 + 0.25 \times 4 = 26$$

$$N_A = 26$$

So vertical stress at pt. P will be

$$\sigma_z = q \times \frac{\text{Influence factor of each area} \times N_A}{\frac{q}{r} \frac{1}{m \times n}} \times N_A$$

$$\text{If } m=10 \text{ \& } n=20 \text{ then } \sigma_z = 0.005q \cdot N_A$$

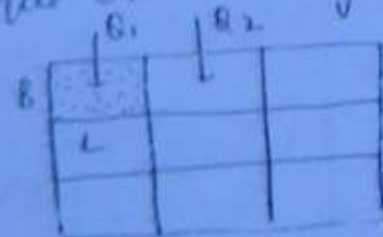
Approximate Method

(i) Equivalent point load Method -

In this Method the loaded area is divided into a convenient no. of smaller area units & load of each area is assumed to act at the Centroid of that area & pt. load Method either given by Boussinesque or Westergaard may be used.

Division is such that $\frac{L}{Z} \neq 3$

$$1 \frac{L}{Z} \neq 3$$



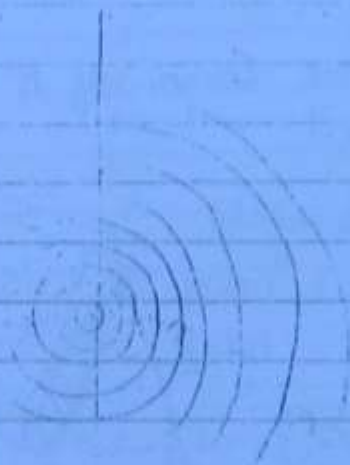
$$\sigma_{z1} = K_{B1} \frac{B_1}{Z^2} \quad , \quad \sigma_{z2} = K_{B2} \frac{B_2}{Z^2}$$

$$\sigma_z = \sigma_{z1} + \sigma_{z2} + \dots$$

mp Newmarks Chart Method

It is also called Influence Chart Method. It is applicable for homogeneous, isotropic, semi infinite elastic soil mass. It is loaded with U.D.L ($q \text{ kN/m}^2$). The loaded area may have any shape.

(16)



1/15 - 3

OP = 5 cm or 3 cm

It contains m concentric circles
 n radial lines

$m = 10$

$n = 20$

Chart divides entire area into $m \times n$ No. of area units (200)

Influence of each area at the Centre of Chart is equal

Influence of Area (1) = Influence of Area (2) = Influence of Area (3)

Newmark's Chart contains concentric circles & radial lines at an equal distance in the chart OP = 5 cm or 3 cm

Generally there are 10 circles & 20 radial lines hence, influence of each area at the Centre of Chart is

Case Vertical Stress due to Circular loaded Area π U.D.L at a depth z below the Centre line of load.

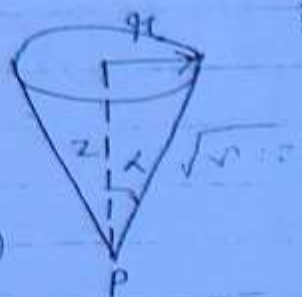
Let q is intensity of load (KN/m^2)

& r = radius of loaded area.

$$\cos \alpha = \frac{z}{\sqrt{r^2 + z^2}}$$

$$\sigma_z = q(1 - \cos^3 \alpha)$$

(162)



Case Vertical Stress below the corner of a Rectangular loaded Area π U.D.L

$$m = \frac{L}{z}$$

$$n = \frac{B}{z}$$

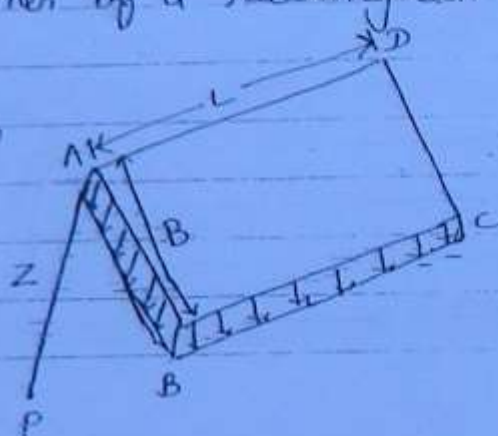
Interchangable

$$\sigma_z = q \cdot I_o$$

I_o = Influence factor

$$I_o = \frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}$$

$$I_o = \frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \left(\frac{m^2+n^2+z}{m^2+n^2+1} \right) + \tan^{-1} \left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2} \right)$$



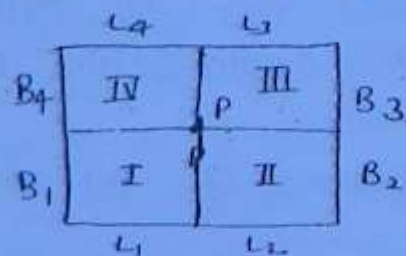
$q = \text{KN/m}^2$

Special Case 1 - If pt 'P' is located inside the loaded area

$$m_1 = \frac{L_1}{z}$$

$$\& n_1 = \frac{B_1}{z}$$

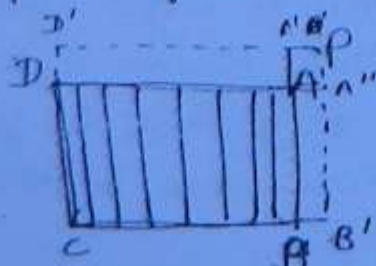
total influence factor $I_o = I_{o1} + I_{o2} + I_{o3} + I_{o4}$



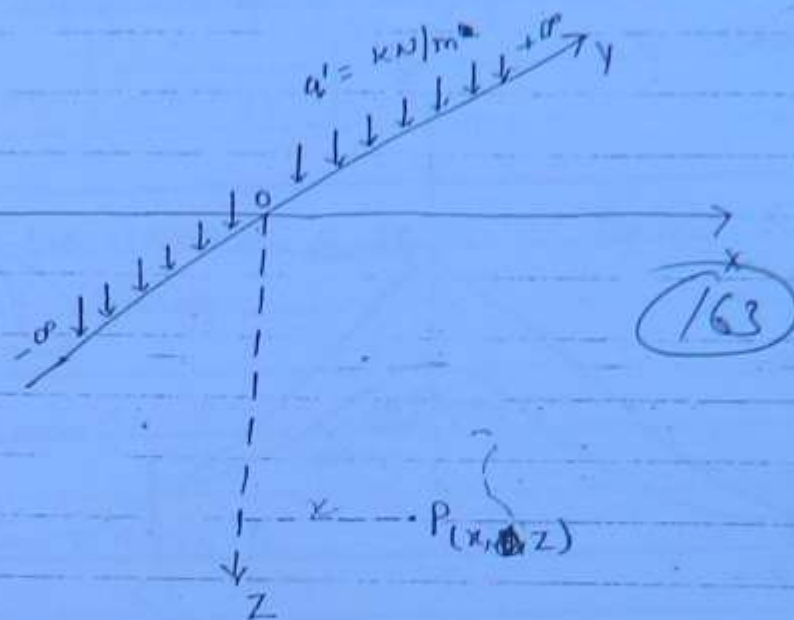
Special Case 2 - If Pt 'P' ~~located~~ lies outside the loaded area
Let P is projection of stress pt. on the plan of the loaded area.

I_o = Influence factor at P due to loaded area ABCD

I_o for $PB'CD'P$ - I_o for $PA''DD'P$ - I_o for $PA'B'B''P$ + I_o for $PA'A'A''$



Case Vertical stress due to line load



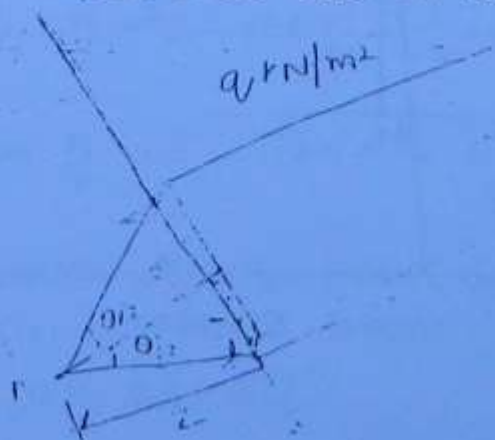
The vertical stress at point P is at depth z & horizontal distance x from line load

$$\sigma_z = \frac{2q'}{\pi z} \left[\frac{1}{1 + \frac{x^2}{z^2}} \right]^2$$

Special case. If pt 'P' is located vertically below the line load then

$$x=0 \text{ then } \sigma_z = \frac{2q'}{\pi z}$$

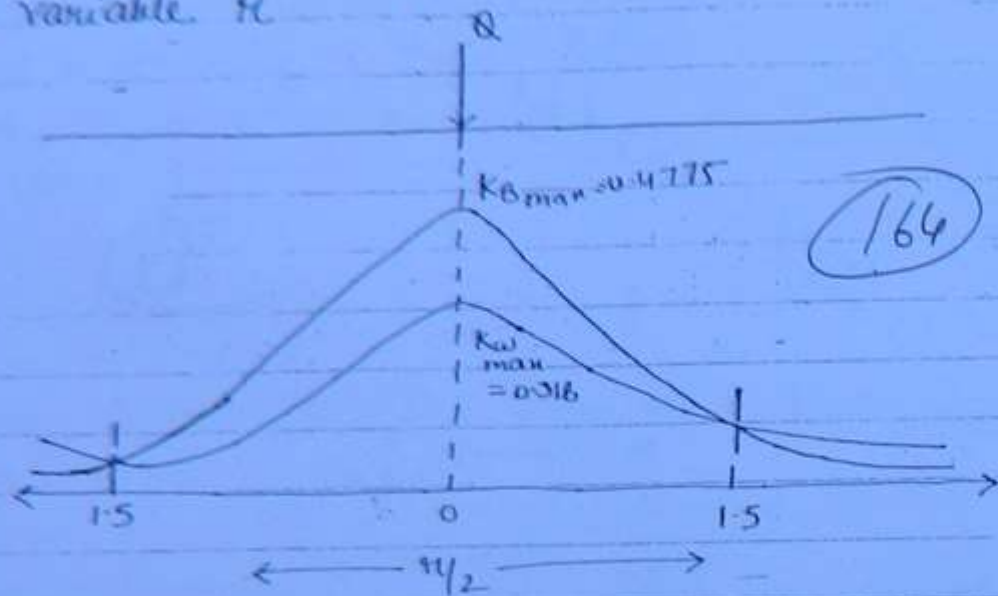
Case Vertical stress due to Strip loading



Vertical stress at pt 'P' is located at a depth z below the centre line of load. At point P the strip load subtends angle θ . Then

$$\sigma_z = q' (\theta + \sin \theta)$$

Comparison between Westergaard's & Boussinesque theory at a depth z with variable H



If $\frac{H}{z} < 1.5$ then $K_B > K_W$

If $\frac{H}{z} = 1.5$ then $K_B = K_W$

If $\frac{H}{z} > 1.5$ then $K_B < K_W$

Application

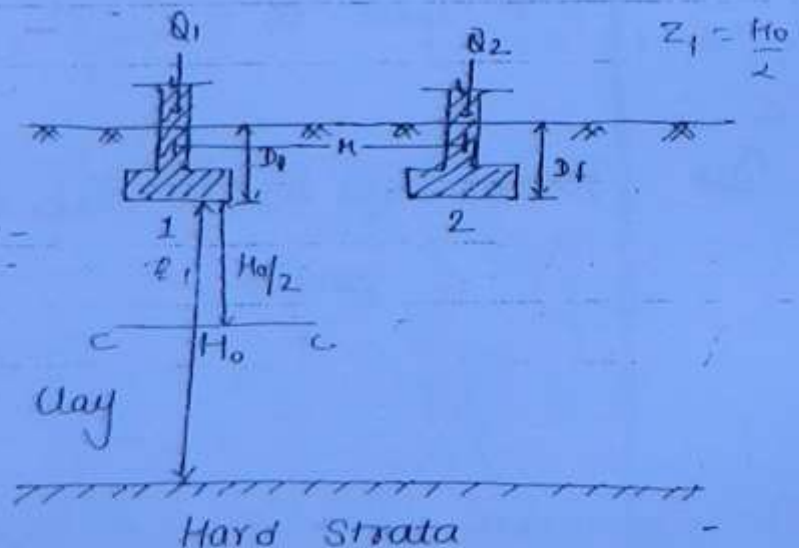
$$\Delta H = \frac{C_c H_0}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)$$

e_0 = Initial void ratio of Comp. Clay layer

H_0 = thickness of Comp. layer

$\bar{\sigma}_0$ = Initial eff. overburden pressure at Centre of Comp. layer at c-c

$\Delta \bar{\sigma}$ = Increase in eff. stress at c-c
This is due to Q_1 & Q_2



$$\Delta \sigma_1 = \sigma_{z1} = K_{B1} \frac{Q_1}{z_1^2}$$

$$\Delta \sigma_2 = \sigma_{z2} = K_{B2} \frac{Q_2}{z_2^2}$$

$$\Delta \bar{\sigma} = \sigma_{z1} + \sigma_{z2}$$

K_B = Boussinesque influence factor

$$K_B = \frac{3}{2\pi} \left[\frac{1}{1 + \eta^2/2} \right]^{5/2}$$

Special case -

If point P is located at a depth 'z' below the point Z in the same line of loading

Hence $\eta = 0$

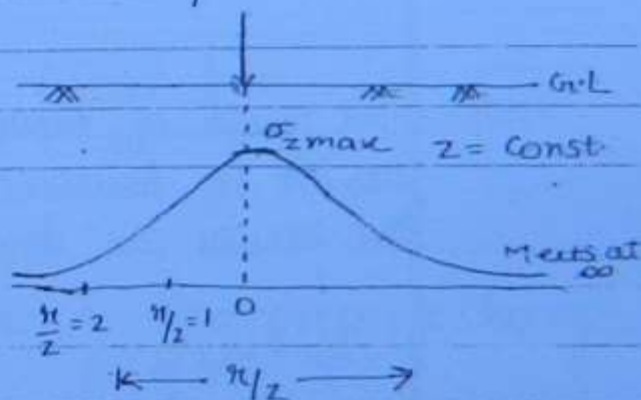
(165)

$$K_B = K_{B \max} = \frac{3}{2\pi} = 0.4775$$

$$\therefore \sigma_z = 0.4775 \cdot \frac{Q}{z^2}$$

I Variation of σ_z w.r.t η at constant depth

$$\sigma_{z \max} = 0.4775 \frac{Q}{z^2}$$



NOTE: $\sigma_{z \max}$ occurs when $\frac{\eta}{z} = 0$ & $\sigma_{z \max} = 0.4775 \frac{Q}{z^2}$

When $\frac{\eta}{z} = 2$ then σ_z is 18% of $\sigma_{z \max}$

When $\frac{\eta}{z} = 3$ then σ_z is only 0.3% of $\sigma_{z \max}$

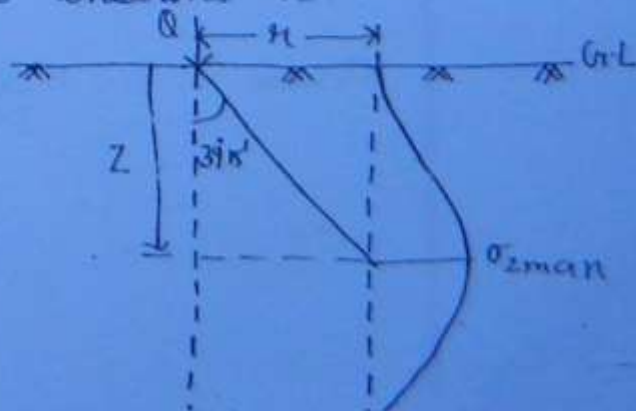
II Variation of σ_z with depth at constant η .

$$\tan(39^\circ 15') = 0.817$$

$$\frac{\eta}{z} = 0.817$$

$$\sigma_{z \max} = 0.08880 \frac{Q}{z^2}$$

$$\sigma_{z \max} = 0.1332 \frac{Q}{z^2}$$



Westergaard's theory

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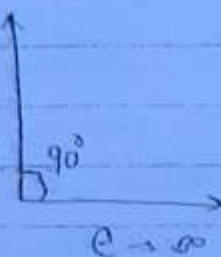
→ Assumption

(i) The material is elastic, homogeneous & non-isotropic.

NOTE - The clay strata is made of thin lenses which makes the soil nonisotropic.

(ii) The material is semiinfinite & laterally reinforced with numerous closely spaced beds of negligible thickness. It means soil mass is considered stratified in which soil in horizontal dir is considered rigid & in vertical dir the soil is considered elastic. (So $\mu = 0$)

$$\frac{E_h}{E_v} \rightarrow \infty$$



NOTE - The results of Westergaard's theory are more realistic but results of Boussinesque theory are widely adopted because the results are conservative.

$$\sigma_z = k_w \frac{Q}{z^2}$$

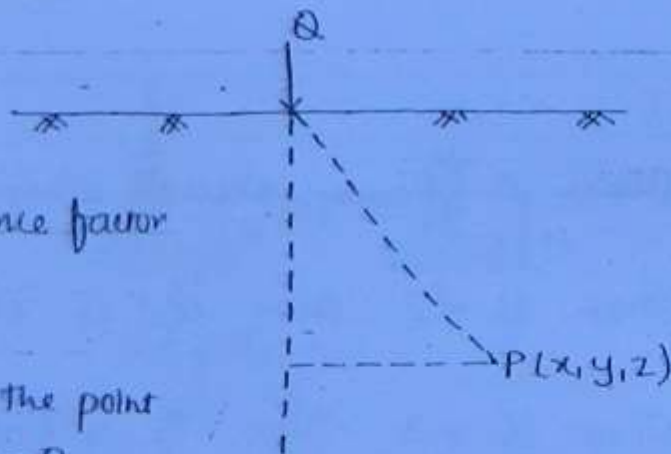
k_w = Westergaard's influence factor

$$k_w = \frac{1}{\pi} \left[\frac{1}{1 + 2 \cdot \frac{y^2}{z^2}} \right]^{3/2}$$

axial case - If Point P' lies below the point load at depth z then $y = 0$

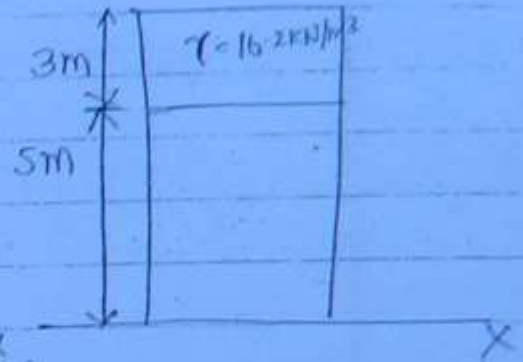
$$k_{w \max} = \frac{1}{\pi} = 0.318$$

$$\sigma_{z \max} = 0.318 \frac{Q}{z^2} \quad (\text{at depth } z, \text{ below the pt load})$$



no pore pressure before the $\uparrow$ of height & during the construction of height there is no dissipation of pore pressure. Also assume lateral pressure at any pt is half of vertical pressure.

Sol. $S = c' + \bar{\sigma}_n \tan \phi'$
 $\bar{\sigma}_n = \sigma_x - U_x = \text{---} \quad (167)$



~~Q. 2.100~~

$\Delta \sigma_1 = 3 \times 16.2 = 48.6 \text{ kN/m}^2$
 $\Delta \sigma_3 = \frac{1}{2} \sigma_1 = \frac{48.6}{2} = 24.3 \text{ kN/m}^2$

Change in pore pressure

$\Delta U = B [\Delta U_c + \Delta U_d]$
 $B [\Delta \sigma_3 + A (\Delta \sigma_1 - \Delta \sigma_3)]$
 $0.92 [24.3 + 0.4 (48.6 - 24.3)]$
 $\Delta U = 31.2984$

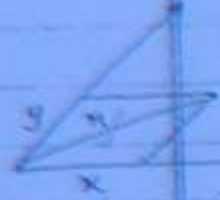
Initial Pore Pressure = 0

final " " = 31.2984 = Pore pressure change

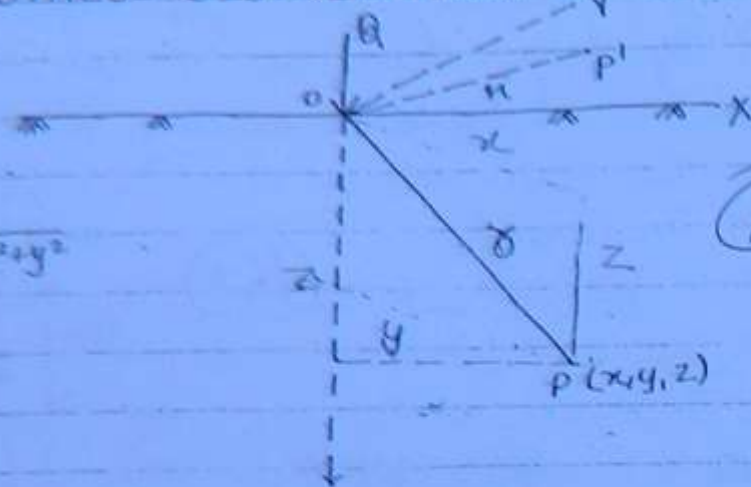
Eff stress = $\bar{\sigma}_x - U_x$
 $B \times \gamma - 31.29$
 $B \times 16.2 - 31.29 = 98.31$

$S = 50 + 98.31 \tan 16^\circ$
 $= 76.18 \text{ kPa}$

STRESS DISTRIBUTION IN SOIL



$$r = \sqrt{x^2 + y^2}$$



The vertical stress at any depth z in the soil mass may be caused by (i) Self wt. of soil (ii) applied structural load

The effect applied load can be found using

- (i) Boussinesque's theory
- (ii) Westergaard's theory

Boussinesque's theory

→ Assumption

- (i) Soil is Consider elastic, homogenous, isotropic & semi infinite
- (ii) The medium obeys hooke's law
- (iii) The self wt. of soil is ignored
- (iv) The Soil is initially unstressed (Normally consolidated)
- (v) The Change in vol. due. to load is insignificant
- (vi) The top surface of soil is free from shear stresses & loading is point loading
- (vii) The stresses are distributed Symmetrically w.r.t the vertical axis of loading

Let horizontal distⁿ of point P is $r = \sqrt{x^2 + y^2}$ & vertical distⁿ is z hence vertical stress at P is given as

$$\sigma_z = k_B \frac{Q}{z^2}$$

∴ at a given depth z is dependent upon $1/z^2$ ratio.

$$\begin{aligned} \text{II} \quad \sigma_3 &= 350 & \sigma_1 &= 350 + 242 = 592 \\ 592 &= 350 \tan^2 \left(45 + \frac{\phi}{2} \right) + 2c \tan \left(45 + \frac{\phi}{2} \right) \end{aligned}$$

$$592 - 429 = 350 \tan^2 \left(45 + \frac{\phi}{2} \right) - 250 \tan^2 \left(45 + \frac{\phi}{2} \right)$$

$$\begin{aligned} \phi &= 13.86^\circ \\ c &= 8.42 \text{ kPa} \end{aligned}$$

(169)

Effective parameters

$$\begin{aligned} \text{I} \quad \bar{\sigma}_3 &= 250 - 101 = 149 \\ \bar{\sigma}_1 &= 149 + 179 = 328 \end{aligned}$$

$$\begin{aligned} 2 \quad \bar{\sigma}_3 &= 350 - 145 = 205 \\ \bar{\sigma}_1 &= 205 + 242 = 447 \end{aligned}$$

from Sample 1

$$328 = 149 \tan^2 \left(45 + \frac{\phi'}{2} \right) + 2c' \tan \left(45 + \frac{\phi'}{2} \right)$$

$$447 = 205 \tan^2 \left(45 + \frac{\phi'}{2} \right) + 2c' \tan \left(45 + \frac{\phi'}{2} \right)$$

$$\begin{aligned} \phi' &= 3.98 \text{ kN/m}^2 \\ c' &= 211^\circ \end{aligned}$$

Ques Laboratory results on a Soil Mass show that unconfined comp. strength is 1.2 kg/cm^2 . Another specimen of same soil is tested in triaxial test under drained condition which resultant deviator stress at failure is 1.6 kg/cm^2 at a cell pressure of 0.4 kg/cm^2 . Estimate the shear strength of the same soil along a horizontal plane at a depth of 7 m from the ground level. γ_{sat} in the soil is 2.5 m below the ground level. The soil above the H.T is dry have $\rho_d = 1.7 \text{ gm/cc}$, $G_s = 2.7$.

(1st Choice)

uncoupled compressive strength

$$1.7 = \frac{2 \times 1 \times 1}{1 + e}$$

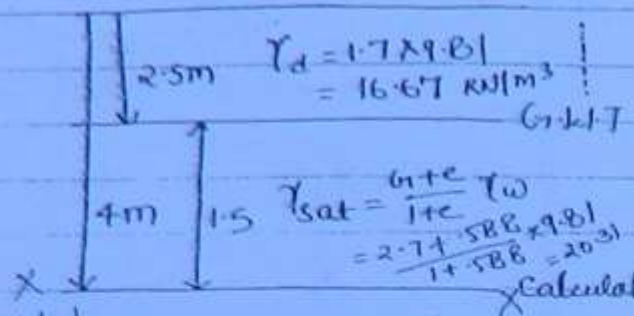
$$e = 0.588$$

G.L.

$$\tau_u = 2c' \tan(45 + \frac{\phi'}{2})$$

$$\bar{\sigma}_3 = 0.4 \text{ kg/cm}^2$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \Delta\sigma = 0.4 + 1.6 = 2.0$$



$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \frac{\phi'}{2})$$

$$2 = 0.4 \tan^2(45 + \frac{\phi'}{2}) + 1.2$$

$$\phi' = 19.47^\circ$$

$$c' = 0.636 \text{ kg/cm}^2 \text{ kg/cm}^2$$

170

$$\frac{0.636 \times 9.81 \times 10^{-3}}{10^{-4}} \frac{\text{KN}}{\text{m}^2}$$

$$c' = 62.39 \text{ kN/m}^2$$

Eff Normal stress at XX

$$\bar{\sigma}_n = \bar{\sigma}_x = \sigma_x - u_x$$

$$2.5\gamma_d + 1.5\gamma_{sat} - 1.5\gamma_w$$

$$2.5 \times 16.67 + 1.5 \times 20.31 - 1.5 \times 9.81$$

$$57.425 \text{ kN/m}^2$$

$$\text{Shear strength } S = c' + \bar{\sigma}_n \tan \phi'$$

$$= 62.39 + 57.425 \tan 19.47^\circ$$

$$= 82.67 \text{ kN/m}^2$$

Ques:- An embankment 5m height is made of eff. stress parameter $c' = 50 \text{ kPa}$ & $\phi' = 16^\circ$ having bulk unit wt. 16.2 kN/m^3 . The pore pressure parameter A & B are 0.4 & 0.93 respectively. Find Shear Strength of the soil using eff. parameter at the base of embankment just after the height of embankment is raised from 5m to 8m. Assume there is

Ques A direct shear test is carried out on a sandy soil & gave following results

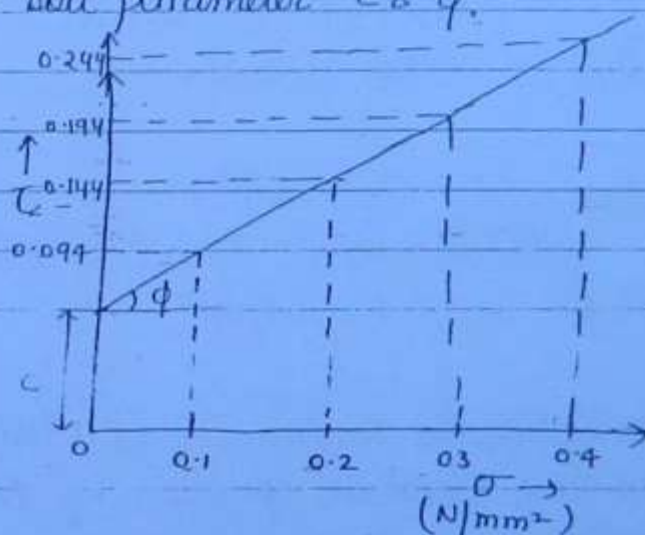
$n = \frac{P}{A}$ N/mm^2	Vertical load (kg) (N) P	Division of dial gauge (1 Div. = 1 μm)	S.F.W) Kx Reading	$\tau = \frac{S.F}{A}$ N/mm^2
0.1	36.8 36.8×9.81	17	20x17 = 340	0.094
0.2	73.5 73.5×9.81	26	520	0.144
0.3	110.2 110.2×9.81	35	700	0.194
0.4	146.9 146.9×9.81	44	880	0.244

The Shear box Size is 60mm x 60mm & proving ring constant is 20 N/ μm . Determine soil parameter c & ϕ .

$$K = \frac{20 \text{ N}}{1 \mu\text{m}}$$

Sol $c = 44.6 \text{ kN/m}^2$

$$\phi = 26.5^\circ$$



Ques A CD test conducted on a clay sample resulted the following data. Cell pressure - 250 kN/m^2 , deviator stress at failure - 275 kN/m^2 . Determine drained friction angle (ϕ_d) if drained cohesion is zero, Angle of Critical plane to the Major principal plane, Normal stress on Critical plane & Shear strength of the soil.

Sol In CD test - $\sigma_3 = \sigma_1 = 250 \text{ kN/m}^2$

$$\Delta \sigma = \sigma_d = 275$$

$$\sigma_1 = 250 + 275 = 525 \text{ kN/m}^2$$

$$\begin{array}{ccc} \tau_d & c' & 0 \\ \phi_d & \phi' & ? \end{array}$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2 \left(45 + \frac{\phi'}{2} \right) + 2c' \tan \left(45 + \frac{\phi'}{2} \right)$$

$$525 = 250 \tan^2 \left(45 + \frac{\phi'}{2} \right)$$

$$\phi' = 20.78^\circ$$

$$\theta_c = 45 + \frac{\phi'}{2}$$

$$\theta_c = 45 + \frac{20.78}{2} = 55.39^\circ$$

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$$\bar{\sigma}_n = \bar{\sigma}_1 (1 - \sin \phi')$$

$$525 (1 - \sin 20.78^\circ)$$

$$\bar{\sigma}_n = 338.74 \text{ kN/m}^2$$

$$s = c' + \bar{\sigma}_n \tan \phi'$$

$$= 0 + 338.74 \tan (20.78^\circ)$$

$$s = 128.54 \text{ kN/m}^2$$

5-2011
Que
marks

Consolidated undrained (CU test) triaxial test when performed on two identical specimen of same saturated clay & pore pressure measurement as given in table. Determine total & eff shear strength parameter using analytical approach.

Pressure (kPa)	Deviator Stress at failure (kPa)	Pore Pressure at failure (kPa)	
50	179	101	- Sample - 1
50	242	145	" 2

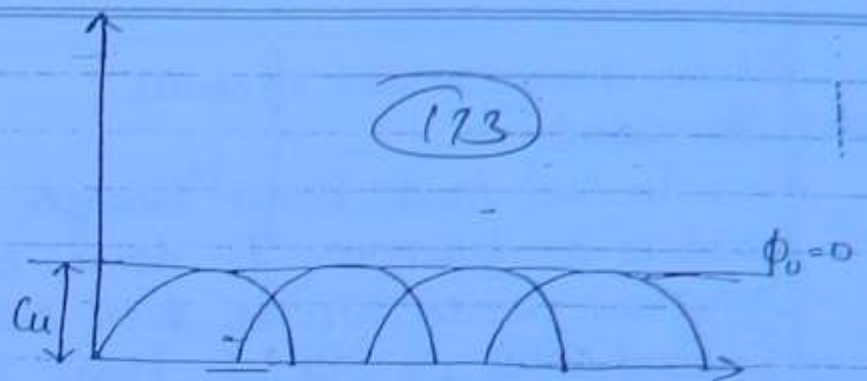
Total parameter

$$\bar{\sigma}_3 = 250 \quad \bar{\sigma}_1 = 250 + 179 = 429$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2 \left(45 + \frac{\phi}{2} \right) + 2c \tan \left(45 + \frac{\phi}{2} \right)$$

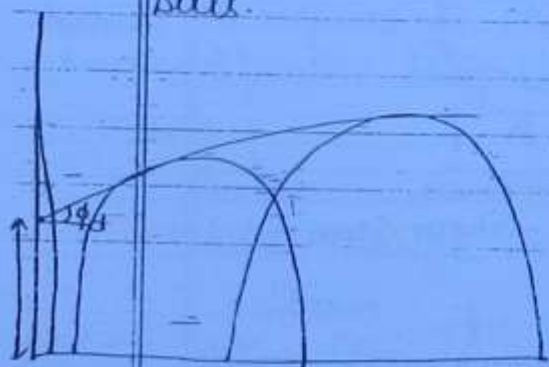
$$\text{Sample 1} \quad 429 = 250 \tan^2 \left(45 + \frac{\phi}{2} \right) + 2c \tan \left(45 + \frac{\phi}{2} \right)$$

$\phi_u = 0$
 $c_u = 25 \text{ to } 30 \text{ kN/m}^2$
 for N.C. Clay
 $= 100 \text{ to } 300 \text{ kN/m}^2$
 for O.C. Clay.



(ii) Saturated Clay under drained Condition (CD test)

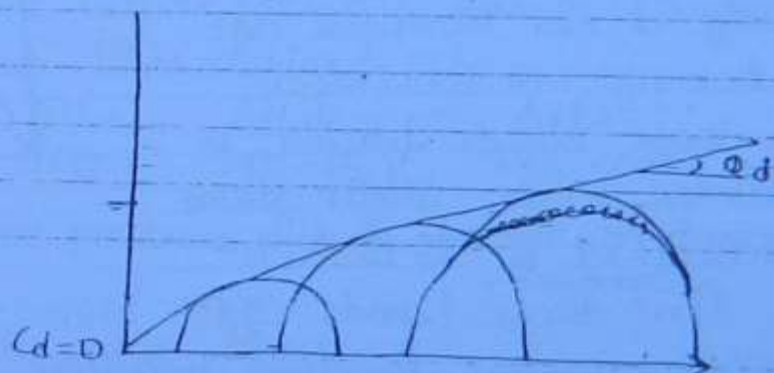
N.C. Clays in CD test represents give results similar to that of sand & O.C. clay give results similar to that of Silt.



for O.C.C.

$c_d = 60 \text{ to } 100 \text{ kN/m}^2$

$\phi_d = 10 \text{ to } 15^\circ$



for N.C.C. (Sand)

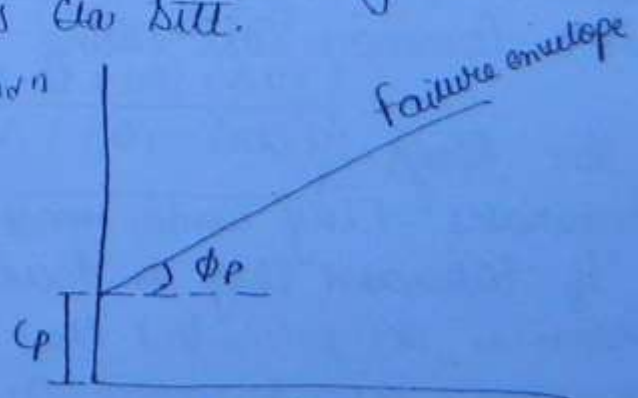
$c_d = 0$

$\phi_d = 20^\circ \text{ to } 30^\circ$

(iii) Partially Saturated Clays.

Under drained & undrained condition both partially saturated clays behaves similar to Clays & Silt.

$c_p = 60 \text{ to } 80 \text{ kN/m}^2$
 $\phi_d = 20 \text{ to } 30^\circ$ } drained condition
 N.C.C.



$c_p = 80 \text{ to } 150 \text{ kN/m}^2$

$\phi_p = 0^\circ \text{ to } 10^\circ$

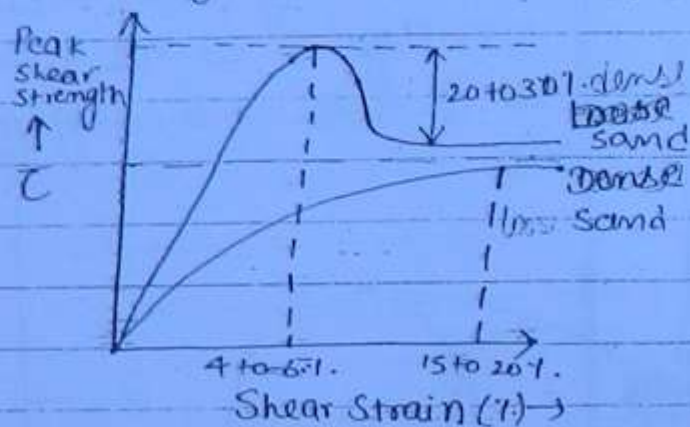
(Underdrained condition)

Shear Characteristics of Soils

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Case 1: For Sands -

In the sands Shear Strength is due to interlocking between particle & friction. In dense sands interlocking resistance may be 20 to 30% of total resistance. On shear failure $\bar{c}$ may occur at 4 to 6% shear strain interlocking resistance will be destroyed hence peak strength will be suddenly reduced by 20 to 30%.



In loose sands sudden failure is not recorded due to lack of interlocking.

In loose sands failure is assumed when shear strain has reached to 15 to 20%.

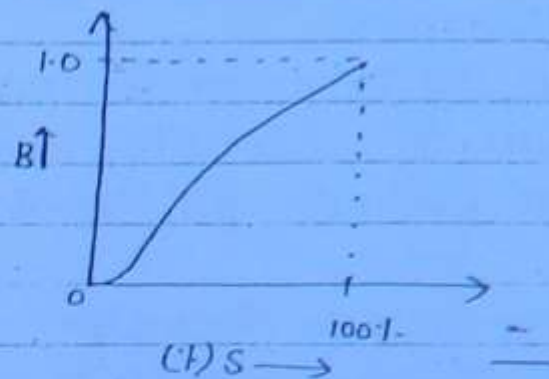
In ^{Saturated} dense sands after shear disturbance $\bar{c}$ increases & pore pressure becomes negative due to $\bar{c}$ eff stress $\uparrow$ & hence shear strength increases whereas in loose saturated sands on shear disturbance $\bar{c}$ decreases, pore pressure increases hence eff. stress reduces $\bar{c}$ causes sudden loss in shear strength. Such a phenomena of loss in shear strength in loose saturated sands on earth quake, dynamic vibration, shear disturbance is called liquefaction. It is common case along the sea shore & river valleys.

Case 2: For Clays -

(i) Saturated Clay under undrained condition (UU test)

If saturated clay is loaded rapidly & pore pressure dissipation is not permitted then results of undrained triaxial test represent zero friction angle. Same Mohr's circle is found & constant radius.

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- (ii) Shear Stage - The parameter A is defined in terms of another parameter $\bar{A}$ such that

$$\bar{A} = A \cdot B$$

$\bar{A}$ represents change in pore pressure due to change in deviator stress. It is defined as

$$\bar{A} = \frac{\Delta u_d}{\Delta \sigma_d} = \frac{\Delta u_d}{\Delta(\sigma_1 - \sigma_3)} = \frac{\Delta u_d}{\Delta \sigma_1 - \Delta \sigma_3}$$

Δu_d - Change in pore pressure due to change in deviator stress

$\Delta \sigma_d$ - Change in deviator stress

The value of A depends on $\bar{A}$ may be as low as -0.5 for O.C soils to as high as 2 to 3 for loose sands. The total pore pressure change in both stage is

$$\Delta u = \Delta u_c + \Delta u_d$$

$$= B \cdot \Delta \sigma_3 + \bar{A} \cdot \Delta \sigma_d$$

$$= B \cdot \Delta \sigma_3 + \bar{A} (\Delta \sigma_1 - \Delta \sigma_3)$$

$$= B \Delta \sigma_3 + A \cdot B (\Delta \sigma_1 - \Delta \sigma_3)$$

$$\Delta u = B [\Delta \sigma_3 + A (\Delta \sigma_1 - \Delta \sigma_3)] \quad \underline{\underline{T_{mp}}}$$

Limitations

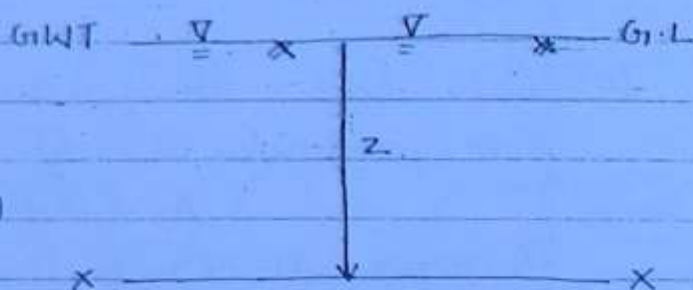
- (i) No mechanism to measure pore pressure hence it is gives us essentially undrained shear strength
- (ii) Not suitable for sands & dry soils.

(176)

15/11/2011

Pore Pressure Parameter.

If it is not possible to determine pore pressure practically then theoretical approach given by Skempton may be adopted. The pore pressure parameter represent



$$u_{x-x} = z \cdot \gamma_w$$

the response of pore pressure due to change in stress under undrained condition.

The pore pressure change may be classified in two stages (i) Cell pressure stage (ii) Shear stage (deviator).

- (i) Cell pressure stage - The parameter B represents the ratio of change in ^{pore} pressure due to change in cell pressure.

$$B = \frac{\Delta u_c}{\Delta \sigma_c} = \frac{\Delta u_c}{\Delta \sigma_3 - \text{Confining pressure or Cell pressure}}$$

$\Delta u_c \rightarrow$ Change in pore pressure due to change in cell pressure.

$B = 0$ when soil is dry

$B = 1$ when soil is fully saturated.

Range of B is $0 \leq B \leq 1$

$\rightarrow$ B increase with increase in S% (degree of saturation)
Relationship with S% is not linear.

$$B = \frac{1}{1 + \frac{n C_c}{C_c}}$$

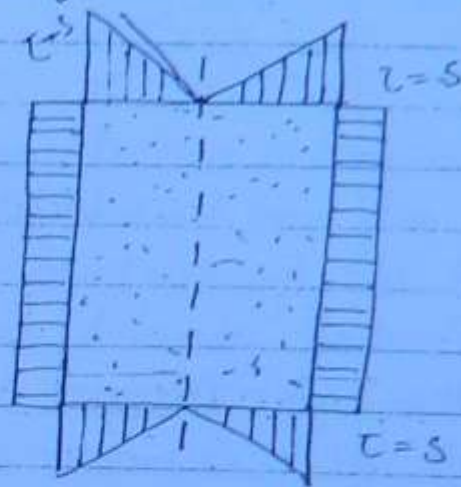
C_c : Coefficient of Consolidation

C_c : Coefficient of Compression also called Compression index

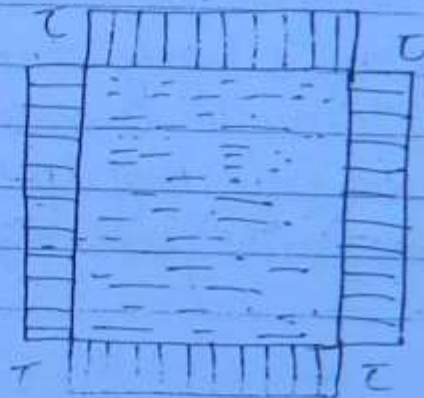
Cylindrical surface. will be as follows

$$S = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{6} \right)}$$

If Vane is quickly rotated then Soil get remoulded & shear stress distribution on the Soil surface becomes uniform.

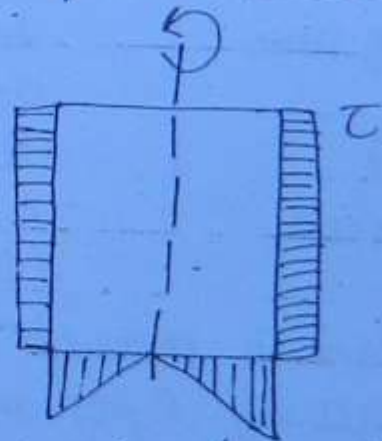


(177)



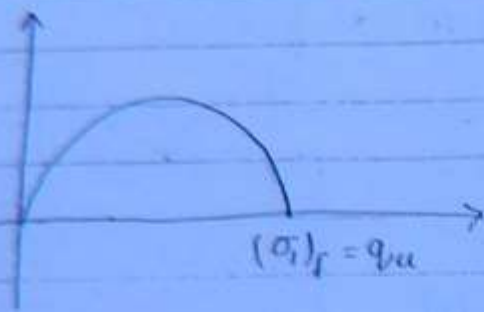
Case (ii) When Only bottom & sides participate in shearing

$$S = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{12} \right)}$$



NOTE - This test can also be used to determine sensitivity of plastic clays.

$$\text{(Sensitivity)} \quad S_t = \frac{(q_u)_{\text{undisturbed}}}{(q_u)_{\text{remoulded}}} = \frac{(2c)_u}{(2c)_r} = \frac{c_u}{c_r} = \frac{(S)_{\text{u}}}{(S)_{\text{r}}}$$

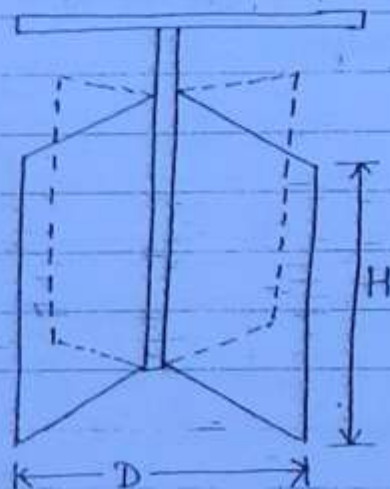


Unique Mohr's circle c passes from origin

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(iv) Vane Shear test

This test is essentially undrained & it is suitable for saturated loose clays & highly plastic clays. It can be conducted in lab or field only diffⁿ is inside in the size of vane.



Description	Lab Size	Field Size
1. Height of vane (H)	20 mm	10-20 cm
2. Dia (D)	8 mm	5-10 mm/cm
Vane thickness	0.5-1 mm	2-3 mm

The vane is pushed into the soil & torque is applied manually at rate of 1° per minute. The torque is measured at failure to a calibrated torsional spring & torsional constant k . Let soil fails at θ angle of rotation, then torque at failure is $T = k \cdot \theta$

There may be two cases.

- (a) When top, bottom and side participate in shearing. Under such condition the distribution of shear stress on sheared

in all possible drainage conditions & results are quite accurate

(179)

(iii) Unconfined Compression Test

It is a special case of triaxial test in which confining pressure is zero. No rubber membrane is provided and there is only 2nd stage of loading (i.e. axial loading). This test cannot be conducted in dry soils & sandy soils. It is suitable for saturated clay & may also be conducted for silts.

σ_1 at failure is called unconfined comp. strength (q_u)

$$\sigma_3 = \sigma_c = 0$$

$$(\sigma_3)_f = (\sigma_1)_f = q_u$$

NOTE that $\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi_{max}}{2}) + 2c \tan(45 + \frac{\phi_{max}}{2})$

$$\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi}{2}) + 2c \tan(45 + \frac{\phi}{2})$$

$$\sigma_1 = \sigma_3 \tan^2 \theta_c + 2c \tan \theta_c \quad \left[\theta_c = 45 + \frac{\phi}{2} \right]$$

For unconfined condition $\sigma_3 = 0$

$$\sigma_1 = 2c \tan(45 + \frac{\phi}{2})$$

$$q_u = (\sigma_1)_f = 2c \tan(45 + \frac{\phi}{2})$$

special case - For pure clays $\phi = 0$

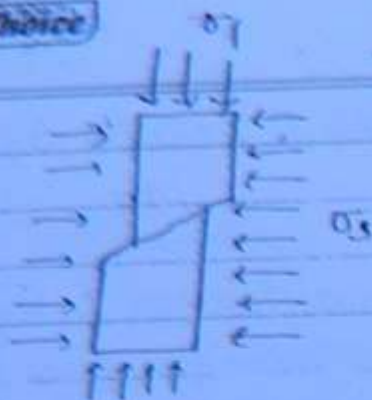
$$q_u = 2c \tan 45^\circ$$

$$q_u = 2c$$

$$c = \frac{q_u}{2}$$

For pure clays $s = c + \tan \phi$

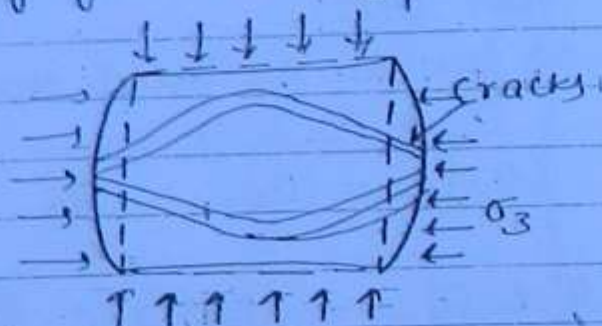
$$s = c$$



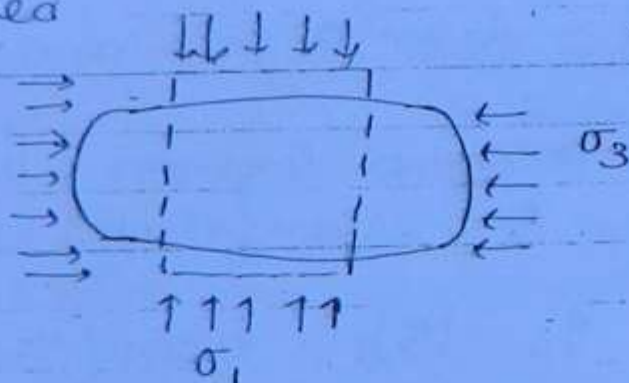
Brittle failure

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Semi brittle - (Semi plastic failure) e.g. Silts ($c-\phi$ soils)
in such failure lateral bulging is recorded & cracks
developed as shown in fig



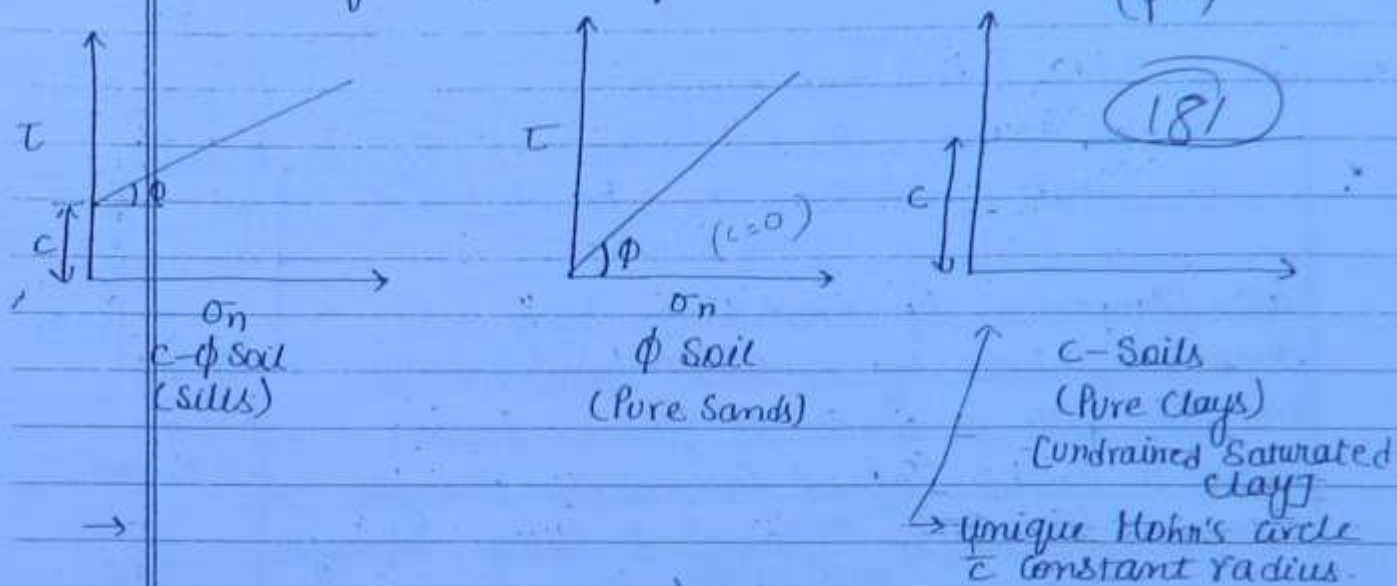
Plastic failure - It occurs in loose sand & normally consolidated clay. large axial deformation are recorded & high lateral bulging is obtained



Merits of triaxial test

- i) Failure occurs along weakest plane ϕ is not predetermined
- ii) Pore pressure measurement is possible & vol change can be determined
- iii) There is complete control over drainage condition
- iv) The stress distribution in failure plane is much more uniform than in direct shear test.
- v) It is applicable for all types of soils & can be conducted

- (loose)
- NOTE (i) The failure envelope for N.C. Soil is found straight line for O.C. Soil is found Curve.
- (ii) The type of failure envelope depends upon type of Soil there may be following 3 Cases -



- (iii) In drained test due to expulsion of pore water there is significant vol change. The area at failure is given as

$$A_f = \frac{A_0 (1 + E_v)}{1 - E_L}$$

A_0 — Original Area

E_v — Volumetric Strain

E_L — Axial Strain / Longitudinal Strain

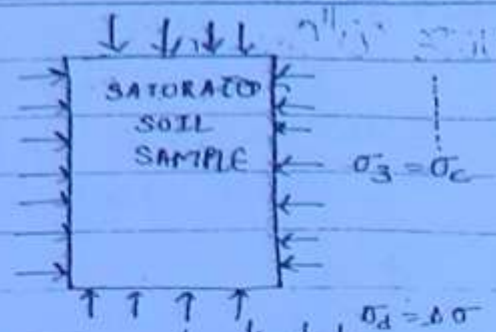
Special Case — If test is undrained then change in vol. is insignificant then ($E_v \approx 0$)

$$A_f = \frac{A_0}{1 - E_L}$$

- (iv) At the time of failure in triaxial test following 3 types of failure may occur depending upon type of Soil

- (i) Brittle failure — It occurs in dense sands & over consolidated clay c — high O.C.R.

will occur & will be completed when expulsion of pore water is stopped. The net change in soil will be equal to vol. of water expelled.

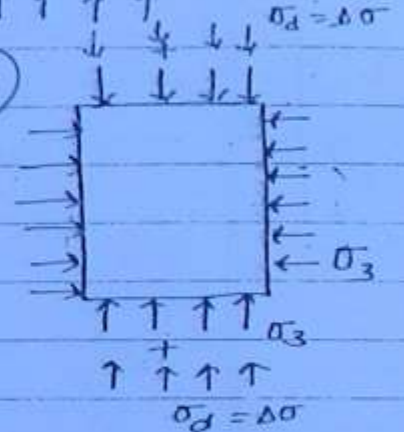


(iii) Shear Stage / Deviator Stage.

$$\text{Total Axial} = \sigma_1 = \sigma_3 + \sigma_d$$

$$\text{Stress or } \sigma_1 = \sigma_3 + \Delta\sigma$$

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After completion of Consolidation Stage axial stress is increased by σ_d called deviator stress till soil fails in shear. At failure deviator stress is called Confined Compressive Strength.

$$(\sigma_d)_f = (\sigma_1 - \sigma_3)_f$$

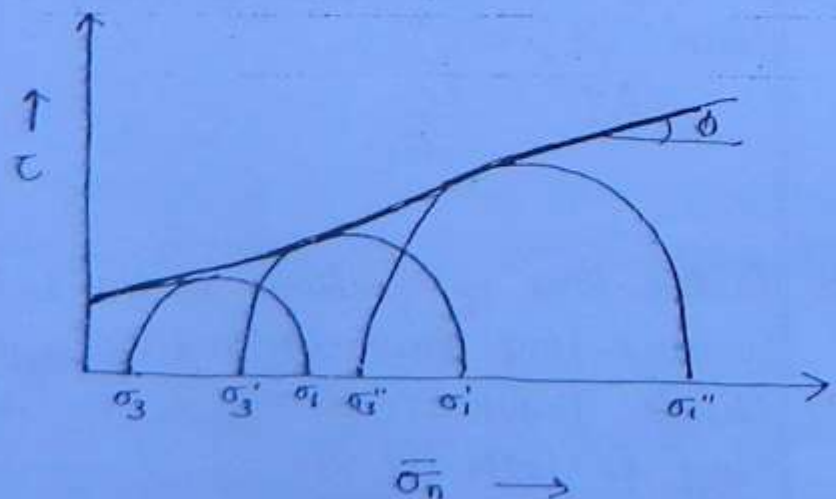
$$= \frac{P}{A_f}$$

Confined Comp. Strength

P = Axial load applied

A_f = Area of soil at failure

To find soil parameter (c & ϕ) test is repeated at different cell pressure ^{or} axial stress ^{or} at failure is found & Mohr's circle are plotted. A common tangent to Mohr's circle at failure is called Mohr's failure envelope from c & ϕ can be obtained.

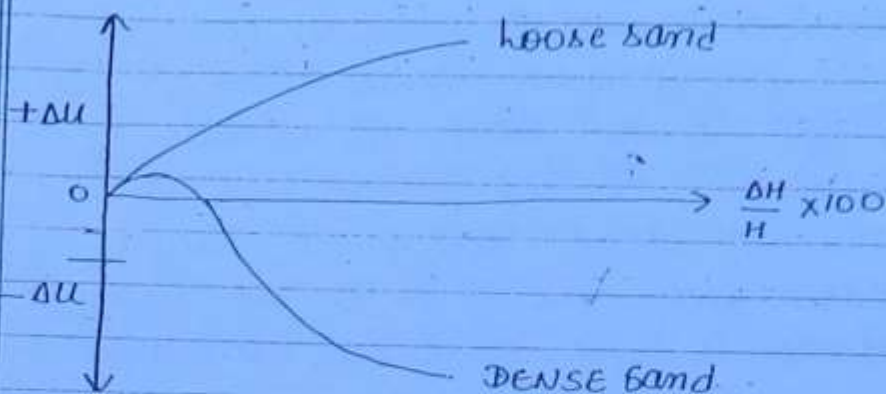


$$\frac{L}{D} = 2$$

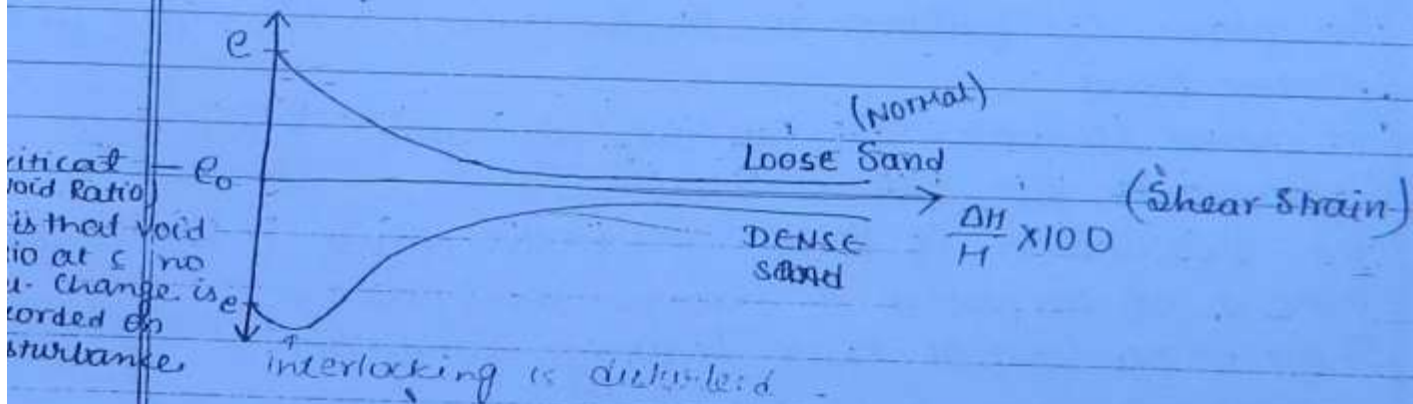
Size of apparatus - 76×38 mm and 100×50 mm

* direct **(1st choice)** test is generally conducted on cohesionless soil as ed test.

Change in ~~void ratio~~ ^{Pore Pressure} v/s Shear Strain Curve



Ob Void ratio v/s Shear strain curve



If $e > e_0$ - Soil is loose
If $e < e_0$ - Soil is dense.

(ii) Triaxial Shear test

1st Stage Triaxial test may be performed under drained or undrained conditions both. It can be used for all type of soils. A cylindrical soil specimen of length $L =$ twice of its dia is prepared & is encased inside a rubber membrane. The test is performed in two stage

(i) Consolidation stage or ~~shell~~ pressure stage or confining pressure stage -

In 1st stage a constant stress $\sigma_3 = \sigma_c$ is applied from all dirn. If drainage is permitted then consolidation

Shear box is either a square or circular having size 60-90 mm. There is no mechanism to measure pore pressure hence this test is to be performed under drained condition & it is suitable for sandy soil. At certain normal stress ($\bar{\sigma}_{n1}$) shear loading is ↑ till failure ($\bar{\tau}_1$) & test is repeated at diff normal stress ($\bar{\sigma}_{n2}$) to obtain $\bar{\tau}_2$ & so on. A graph is plotted between $\bar{\sigma}_n$ & $\bar{\tau}$ at failure from c soil parameter c & ϕ can be obtained (effective)

$$s = c' + \bar{\sigma}_n \tan \phi'$$

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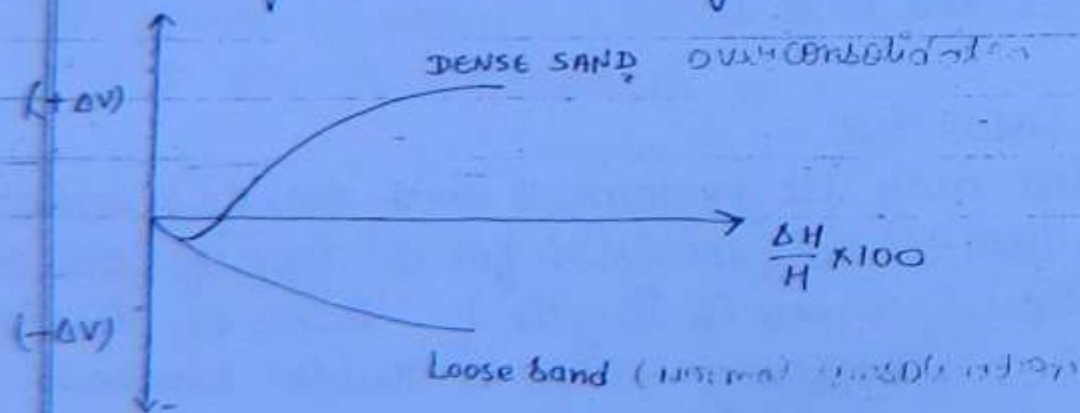
Limitations -

- (i) The plane of failure is predetermined & may not be the weakest plane.
- (ii) The stress condition are not only at failure hence it is difficult to plot Mohr's Circle.
- (iii) The distribution of stress on failure plane is not uniform.
- (iv) There is no mechanism to measure pore pressure.
- (v) There is no control over drainage condition.

RESULTS PRESENTATION:-

Strain

Vol Change v/s Shear ~~Strength~~ Curve.



values of shear stress at failure are plotted against the normal stress for each test.

The failure plane is always horizontal in direct shear test.

this test is not much used for fine grained soils.

of earthen dam during sudden drawdown case.

Consolidated drained test

This is most time taken test. During the both stages drainage is permitted hence consolidation takes place. In this case significant volume change may be expected due to expulsion of pore water. This test is suitable for sand & high permeability & slow rate of loading. eg - stability analysis of retaining walls.

Shear Strength test

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(i) Lab Test

(a) Direct Shear test ✓

(b) Triaxial test ✓

(c) Unconfined Compression test

(d) Vane Shear test ✓

(e) Torsion Shear test

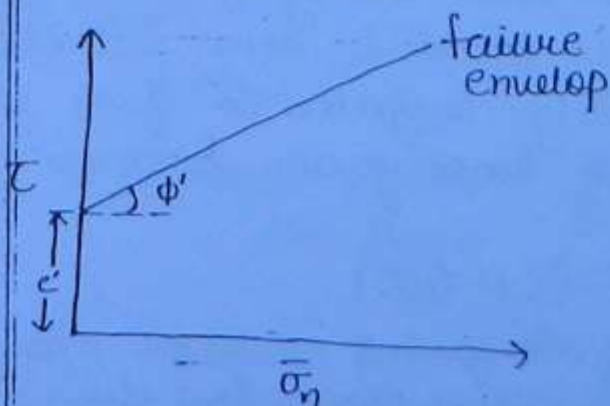
(f) Ring Shear test

(ii) Field Test

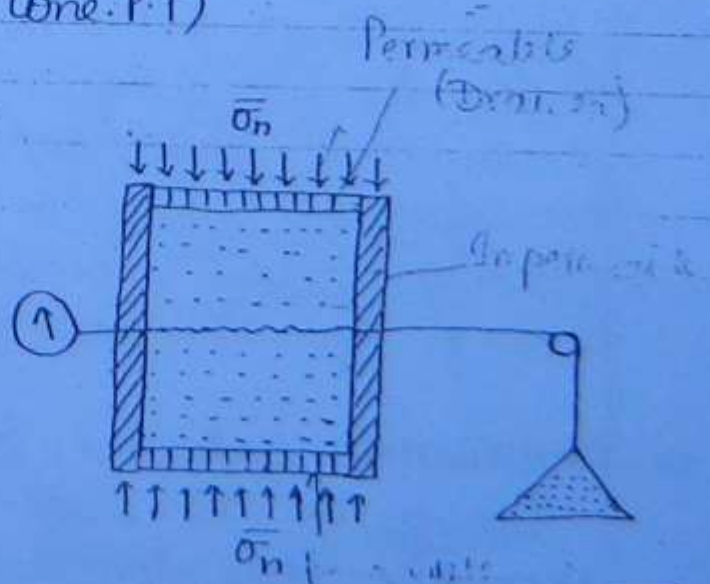
(a) Vane Shear test

(b) Penetration test (S.P.T & Cone.P.T)

(c) Direct Shear test (eq. 18.11)



effective stress will come directly



Modified Mohr's Coulomb's Coulomb's theory

It is found that S.S is a function of effective parameter not of total parameter

$$S = c' + \bar{\sigma}_n \tan \phi'$$

Where $\bar{\sigma}_n$ is effective normal stress & c' & ϕ' are effective soil parameter.

NOTE: The effect of pore pressure in c & ϕ if not mention then original c & ϕ may be used with normal stress is significantly affected.

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Limitation

1. The theory assumes failure envelope into a straight line c may be a little curve for over consolidated soils.
2. It neglects the effect of intermediate principal stress (σ_2)
3. In case of pure clays shear strength is constant c the depth however in actual practice small $\uparrow$ is observed.

TYPE OF SHEAR TEST ON THE BASIS OF DRAINAGE CONDITION

The choice of test depends upon type of soil, purpose of test, drainage condition available at the field & loading rate

(i) Unconsolidated - Undrained test (U-U)

It is a quick test & may be completed in 5 to 10 minutes. In this test neither water is allowed to leave the soil in consolidation stage nor in shear stage. There will be no significant change in vol. of soil. Such test are suitable for clays & low permeability subjected to fast rate of loading. eg - Construction of building over saturated clay.

(ii) Consolidation - Undrained test (C-U test)

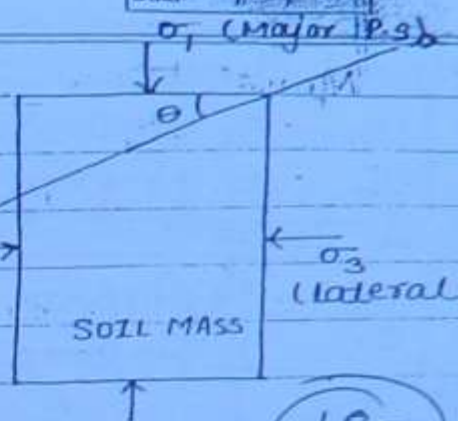
During the 1st stage of confining pressure drainage is permitted hence consolidation takes place but during vertical shear loading drainage is not permitted. eg - Stability Analysis

SHEAR STRENGTH OF SOIL

Definition

Shear Strength of Soil is resistance σ_n or σ_r to the Shear deformation & may be due to

- 1) Interlocking between particles } sand & gravel
- 2) Frictional resistance
- 3) Cohesion & adhesion (Clay pure)



Constant τ depth (Clays)

Shear $\uparrow$ τ depth (Granular Soil)

$$\sigma_r = \sqrt{\sigma_n^2 + \tau^2}$$

In Clays Shear Strength comes from Cohesion & Adhesion whereas in Gravel & sand Shear Strength comes from Interlocking of particles & frictional resistance.

Shear Strength is equal to Shear Stress on the Critical plane a-a. Critical plane is that plane on which resultant stress is most inclined to the normal of that plane ($\beta = \beta_{max}$ & $\theta = \theta_c$) then $\tau =$ Shear Strength. Note that Critical plane is not necessary to be plane of τ_{max} . However, in pure clays plane of τ_{max} may coincide with Shear failure plane.

$$\theta_c = 45^\circ + \frac{\beta_{max}}{2}$$

$$= 45^\circ + \frac{\phi}{2}$$

[$\beta_{max} = \phi$ (friction angle of soil)]

For pure pure Clays Shear Strength $\Rightarrow S = \tau_{max}$

For granular Soil Shear Strength $\Rightarrow S < \tau_{max}$

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$$

which acts at 45° from

the σ_1 (major principal stress) & σ_3 (Minor Principal stress)

$$\tan \beta_{max} = \frac{\tau}{\sigma_n}$$

σ_n = Normal stress on Critical plane

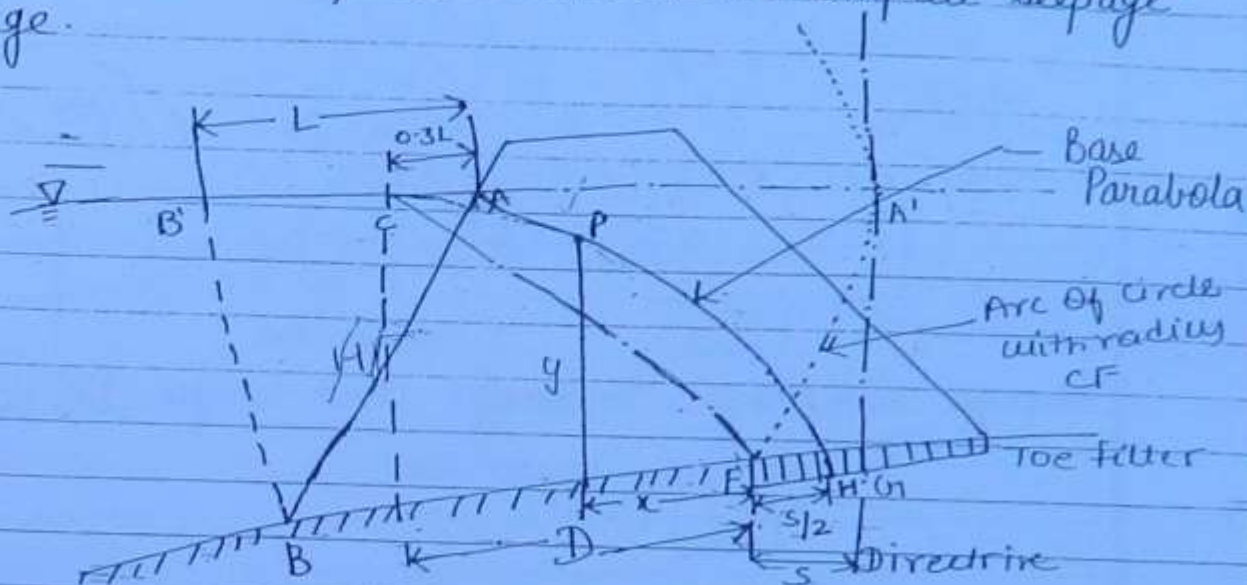
$\tau = S$ = Shear strength on failure / Critical plane

β_{max} = Angle of Obliquity

as 801, 701, 601. . . so on.

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Procedure to draw phreatic line & to compute seepage discharge.



- Let AB is U/s wetted Surface. Horizontal projection of AB is L. The entry point of base parabola is located at $0.3 L$ from A (ie at c)
 → Acc. to the property of parabola any pt on the parabola is equidistant from focus and directrix. Let Horizontal disⁿ of c from focus is D and vertical height of c from base is H.
 Hence $CF = \sqrt{D^2 + H^2}$
 $CA' = CF = D + S$
 Acc. to property of parabola $CF = CA' = \sqrt{D^2 + H^2} = D + S$
 Focal distance $S = \sqrt{D^2 + H^2} - D$
 Extend horizontal line from c and draw an arc of a circle taking C as centre and CF as radius. The arc cuts horizontal CA at A'. Hence A' must lie on the directrix. Draw a vertical line from A' to meet the filter at cn. A'En is directrix. $FC_n = S$ is focal disⁿ. Hence vertex of parabola is located at $S/2$ from F (ie H). Draw a parabola to originates from c and meets to the filter at H.
 Phreatic line follows path of base parabola with entry correction at A. Phreatic line originates at A normal

to Surface AB.

Consider any pt. P in Horizontal distⁿ x & Vertical distⁿ y from focus

$$PF = \sqrt{x^2 + y^2}$$

$$= x + S$$

$$\sqrt{x^2 + y^2} = x + S$$

$$x^2 + y^2 = x^2 + S^2 + 2XS$$

$$y = \sqrt{S^2 + 2XS}$$

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Seepage discharge through unit length of dam

$$q = kiA$$

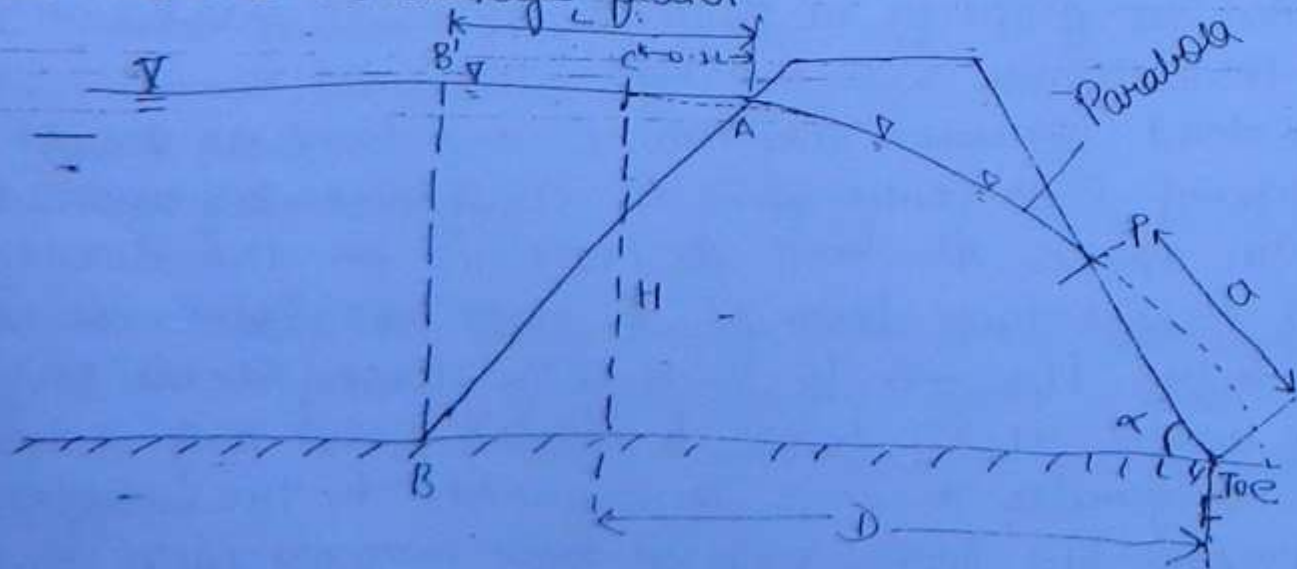
$$q = k \left(\frac{y}{H} \right) (y \times 1)$$

$$\frac{dy}{dx} = \frac{1}{2} \frac{2S + 2x}{\sqrt{S^2 + 2XS}} = \frac{S}{\sqrt{S^2 + 2XS}}$$

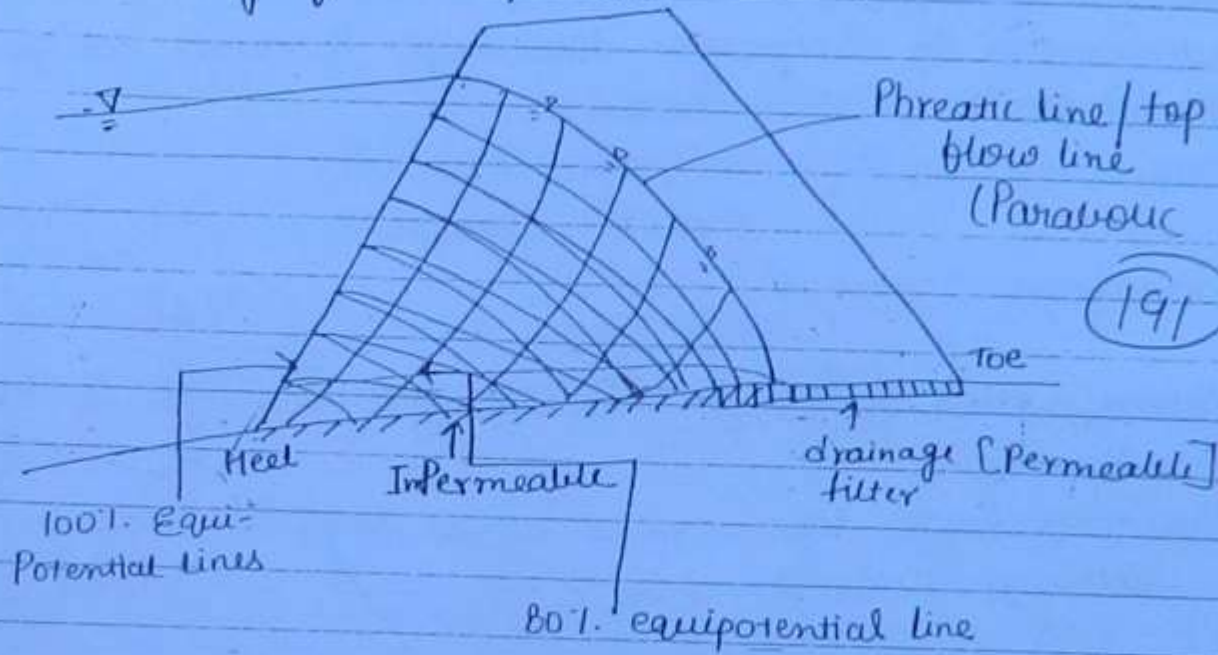
$$q = k \frac{S}{\sqrt{S^2 + 2XS}}$$

$$q = k \cdot S$$

2. When there is no drainage filter



1. When drainage filter is provided



Phreatic line is top flow line at $\leq$ pressure is atmospheric. Below phreatic line soil is submerged & pressure is hydrostatic & whereas - above the phreatic line circulation of air is permitted hence pressure remains atmospheric. Phreatic line follows path of base parabola. The upstream wetted surface of dam is 100% equipotential line it means there is no head loss at the surface but in the dirⁿ of seepage flow head loss occurs & therefore available equipotential decreases.

Application for flow net

1. Determination of seepage discharge through earthen dams
2. Determination of exit gradient
3. Determination of seepage pressure
4. " " uplift pressure or pore pressure or hydrostatic pressure

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Determination of Seepage Discharge through a dam.

$$h = H_1 - H_2$$

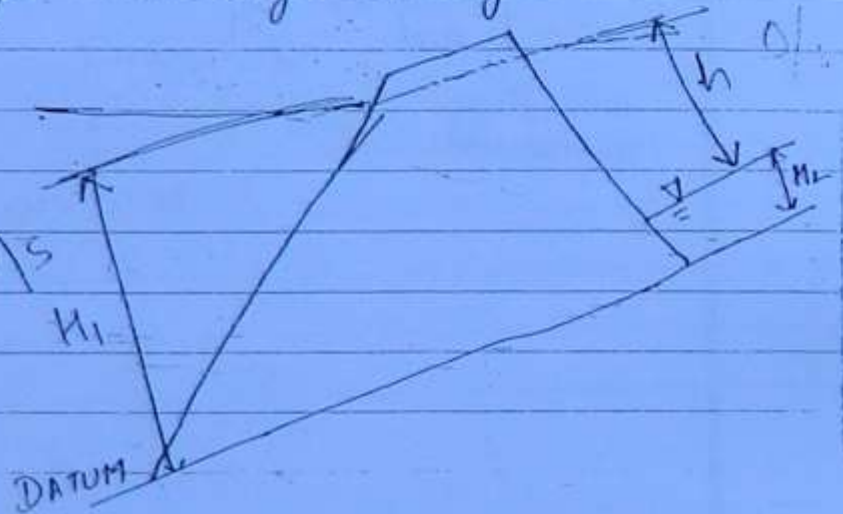
Consider unit length of dam, discharge through

$$\Delta q = K \Delta h$$

$$\Delta q = K \left(\frac{\Delta h}{b} \right) (b \times L) \quad \checkmark$$

$$= K \Delta h$$

$$= K \cdot \frac{h}{N_d}$$



Let there are N_f No. of flow Channels

$$q = \Delta q \times N_f = \frac{K \cdot h}{N_d} N_f$$

$$q = K \cdot h \frac{N_f}{N_d}$$

$\frac{N_f}{N_d}$ = Shape factor.

K = equivalent coeffⁿ of permeability. $= K_e = (K_x \cdot K_y)^{1/2}$ - 2-D

$K = K_e = (K_x \cdot K_y \cdot K_z)^{1/3}$ - 3-D

Phreatic line

hydraulic gradient at exit i.e. $= \frac{\Delta h}{b_n}$

4. The discharge through each flow channel is equal. (Δq) & remains constant hence continuity eqⁿ may be applied b/w two points in a flow channel.

Hence $q_1 v_1 = q_n v_n = \Delta q$

Let depth of flow is unit then

$$(b_1 \times 1) v_1 = (b_n \times 1) v_n = \Delta q$$

$$b_1 v_1 = b_2 v_2 = b_n v_n \quad \text{Imp}$$

b = Size of flow field.

5. Let there are N_f no. of flow channels then

$$N_f = \text{No. of flow lines} - 1$$

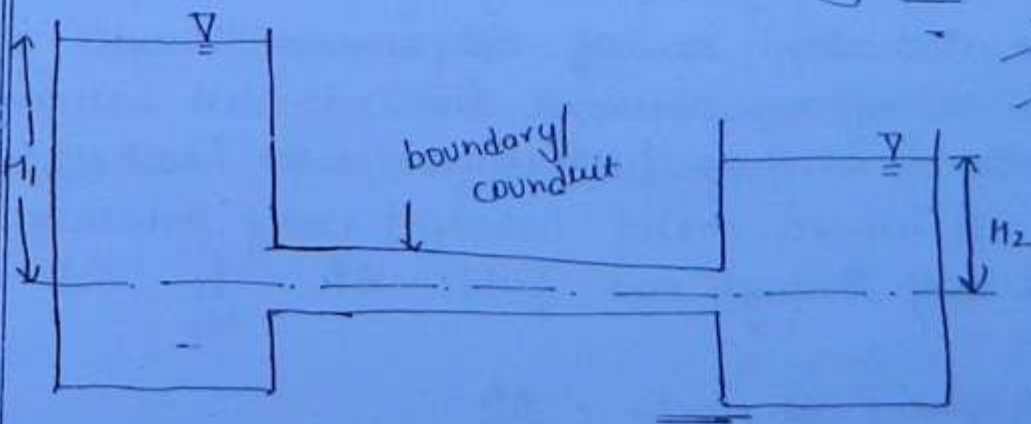
Hence total discharge per unit depth $= q = \Delta q \times N_f$

Δq = discharge through flow channel is also equal to diffⁿ betⁿ each flow stream lines.

$$\Delta q = \psi_1 - \psi_2$$

6. Two equipotential lines or two stream lines ^{never} can join.

7. Flow net will be same if u/s & d/s water level are interchange provided there is no change in boundary condition where as flow net will change if boundary condition are changed. It means flow net will be unique for given set of boundary condition.



Assumption in Laplace eqⁿ

- (i) Darcy law is valid $Q = k i A$
- (ii) Soil is homogeneous & saturated & isotropic
- (iii) Flow is 2-dimensional
- (iv) Water & Soil molecules are incompressible.
- (v) Due to seepage there no change in volume.
- (vi) Steady state condition exists. i.e. flow condition does not change wth time.

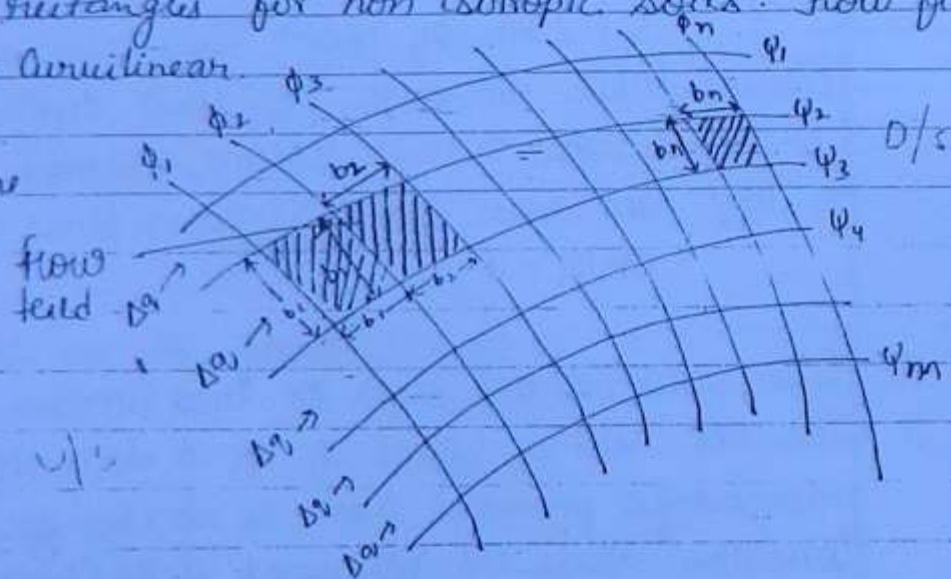
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Imp. Def: Properties of flownet

1. Potential lines or equipotential line (ϕ lines) & Stream lines (flow lines) intersect each other orthogonally.
2. The flow fields are approximate square for isotropic soil & approximate rectangles for non isotropic soils. Flow fields may be linear or curvilinear.

ϕ - Equipotential line

ψ - Stream lines



3. The head loss through each flow field is equal & is called equipotential drop.

Let there are N_d = no. of equipotential drops

N_d = No. of equipotential lines - 1

Let 'h' is total head drop betⁿ u/s & d/s = head difference betⁿ u/s & d/s.

Equipotential drop through each field = $\Delta h = \frac{h}{N_d}$

Hydraulic gradient at inlet = $\frac{\Delta h}{b_1}$

floating gradient or bursting gradient.

To prevent Quick Sand Condition hydraulic gradient (i) should be kept smaller than Critical hydraulic gradient (i_c). Factor of Safety against piping failure / Quick Sand failure is given as

$$F.O.S = \frac{i_c}{i}$$

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Friday
11/11/2011

eg- A 1.2 m layer of Soil is subjected to upward seepage flow under a head of 1.8 m. A layer of Coarse sand is placed over the Soil layer. to attain F.O.S 2.0 against piping failure Soil & sand both have $G_r = 2.67$ & $e = 0.67$. the depth of sand req. is

(a) 0.9 m (b) 1.2 m (c) 2.4 (d) 3.6

$$i_c = \frac{G_r - 1}{1 + e} = \frac{2.67 - 1}{1 + 0.67} = 1$$

$$i = \frac{i_c}{F.S} = \frac{1}{2.0} = 0.5$$

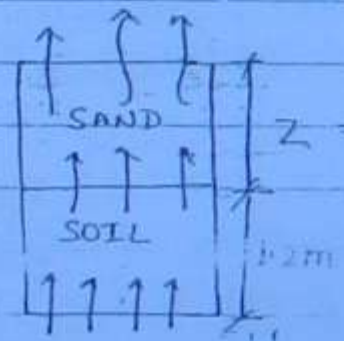
$$i = \frac{h}{L} \Rightarrow 0.5 = \frac{1.8}{L}$$

$$L = \frac{1.8}{0.5} = 3.6 \text{ m}$$

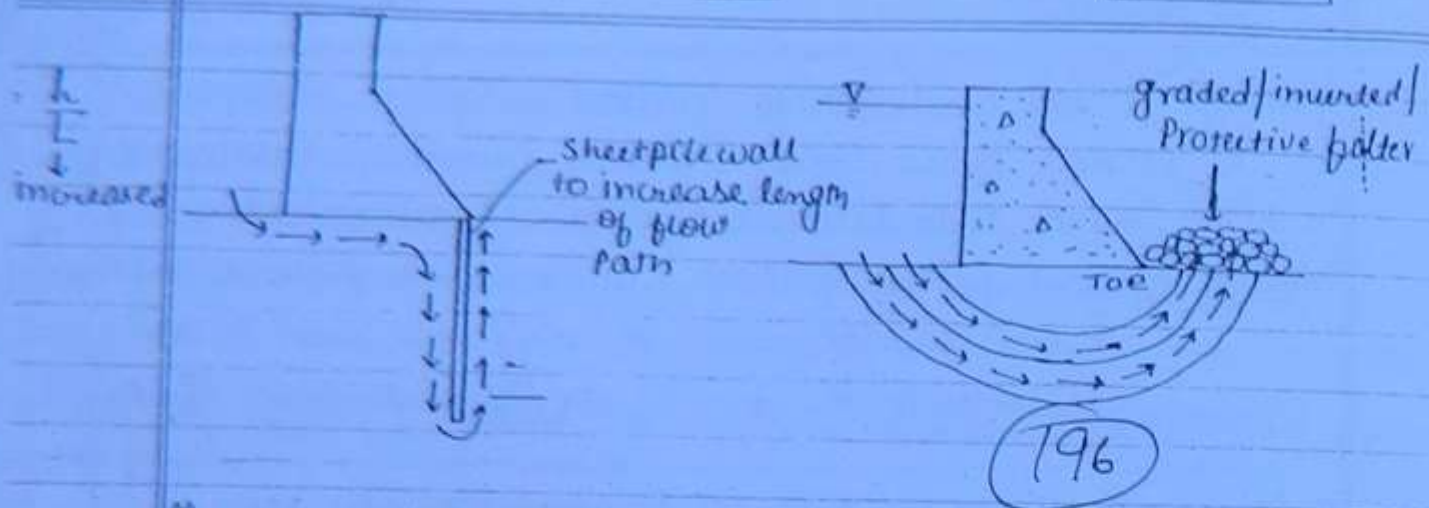
$$L = Z + 1.2 \text{ m}$$

$$3.6 - 1.2 = Z \Rightarrow Z = 2.4 \text{ m}$$

Ans (c)



→ Due to upward seepage flow piping failure in hydraulic structure may occur. To prevent such failure hydraulic gradient is to be controlled below critical hydraulic gradient. To control hydraulic gradient either length of flow path should be increased or protective filter / graded filter should be provided near the toe.



The protective filter is designed in such a way that it will permit flow of water molecules but not of soil molecules.

Design Specification filter

- 1 $\frac{(D_{15})_{\text{filter}}}{(D_{85})_{\text{filter}}} \leq 5$
- 2 $4 < \frac{(D_{15})_{\text{filter}}}{(D_{15})_{\text{soil}}} \leq 20$
- 3 $\frac{(D_{50})_{\text{filter}}}{(D_{50})_{\text{soil}}} \leq 25$

are eqⁿ

$H=0$

Flow net

Flow net is graphical representation of solⁿ of Laplace eqⁿ
 $\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$ — Laplace eqⁿ for 2D flow in isotropic soil

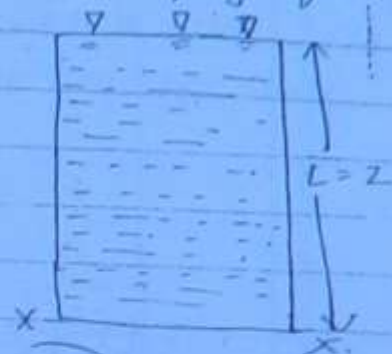
$k_x \frac{\partial^2 H}{\partial x^2} + k_y \frac{\partial^2 H}{\partial y^2} = 0$ — Laplace eqⁿ for 2D flow in non-isotropic soil

k_x & k_y are permeability coeffⁿ in x & y dirⁿ

If soil mass has z vertical depth in $\downarrow$ Seepage flow is vertically upward. then net eff. pressure at $x-x$ will be

eff. stress

$$\begin{aligned} &= \sigma - p_s \\ &= (\sigma - u) - p_s \\ &= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - h \cdot \gamma_w \\ &= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - \frac{h}{z} \cdot \gamma_w \cdot z \\ &= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - i \cdot z \cdot \gamma_w \\ &= \gamma_{sub} \cdot z - i \cdot z \cdot \gamma_w \\ &= 0 \end{aligned}$$



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If net eff. stress is reduced to zero then soil particles will be in floating condition. Such a condition in cohesionless soil is Quick Sand Condition.

Quick Sand Condition - is a stage in $\downarrow$ due to vertically upward flow net eff. pressure at any level becomes zero. Under such condition soil particles are floating $\downarrow$ may flow $\downarrow$ the water. Such as in case of sand & fine gravels. At Quick Sand Condition

Net eff. stress = 0

$$z \cdot \gamma' - i z \gamma_w = 0$$

$$\left| \gamma' = \frac{i \gamma_w}{i} \right|$$

$$i = \frac{\gamma'}{\gamma_w}$$

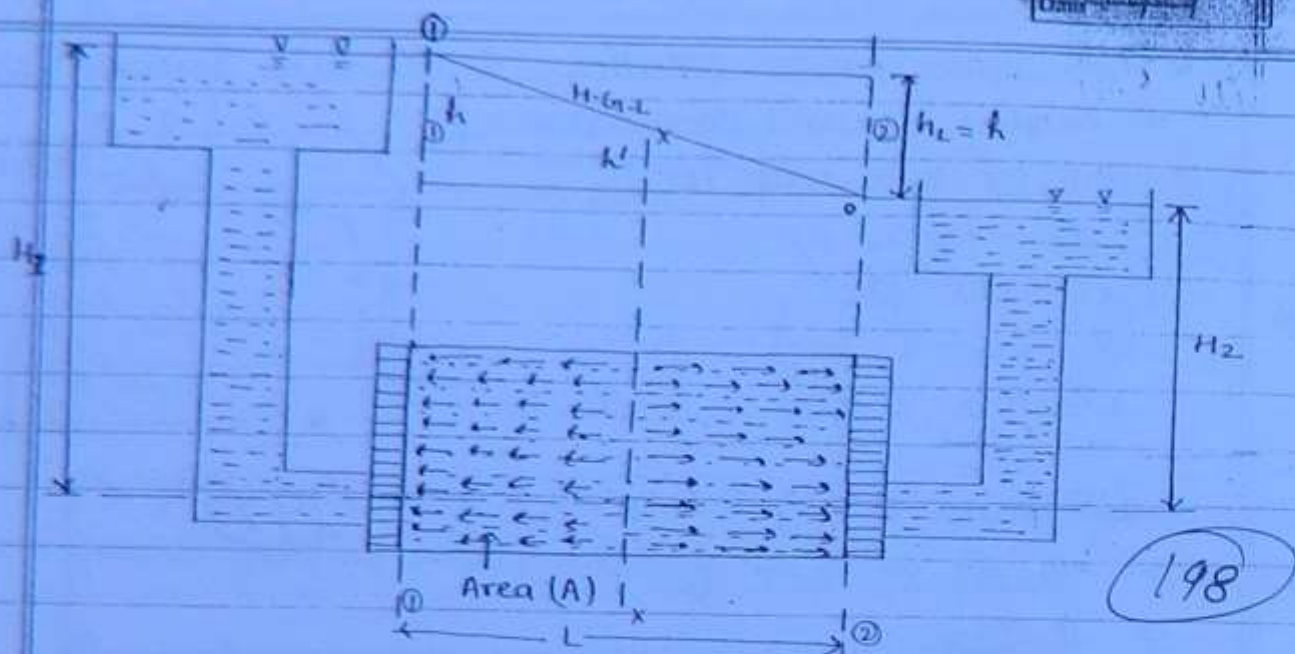
$$i = i_c = \frac{(n-1)}{1+e}$$

$$i_c \approx 1$$

Critical hydraulic gradient

Cohesionless less $e = 0.6-0.7$

At Quick Sand Condition hydraulic gradient is called critical hydraulic gradient (i_c) nearly equal to 1. It is also called



Let $h_L = h = H_1 - H_2 =$ hydraulic head (i.e. head diffn b/w upstream & downstream level). It is also equal to head loss ^{through} to the soil of length L .

The seepage force is the drag force exerted by the water molecules in the soil molecule $\vec{e}$ always acts in the dirn of flow & is the effect of friction. The seepage pressure at 1-1 inlet of soil mass is $p_s = h \cdot \gamma_w$

at $x-x = p_s = h' \cdot \gamma_w$

at 2-2 $= p_s = 0 \cdot \gamma_w = 0$

Seepage force at 1-1 $= p_s \times A = h \cdot \gamma_w \cdot A$

$\frac{F_s}{L} = \frac{h}{L} \cdot \gamma_w \cdot (A \cdot L) = i \cdot \gamma_w \cdot (\text{Volume})$

Seepage force per unit volume $= \boxed{\frac{F_s}{V} = i \cdot \gamma_w = f_s}$

Acts Always in the dirn of flow.

Special case—

If flow is in vertical dirn then net eff. stress will be $= \sigma \pm p_s$

+ when flow is downward

— when flow is upward.

Under a certain loading the layer of clay is expected to undergo full settlement of 18 cm. Also it is expected to settle 5 cm in 2 months at same loading. Find the time req. for clay layer to settle by 10 cm

$\Delta H = 18 \text{ cm}$ (ultimate settlement)

$$U_1 = \frac{5}{18} = 0.278$$

$$U_2 = \frac{10}{18} = 0.55$$

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$$T_{v1} = \frac{\pi}{4} U_1^2 = 0.0606 \frac{C_v \cdot t_1}{d_1^2}$$

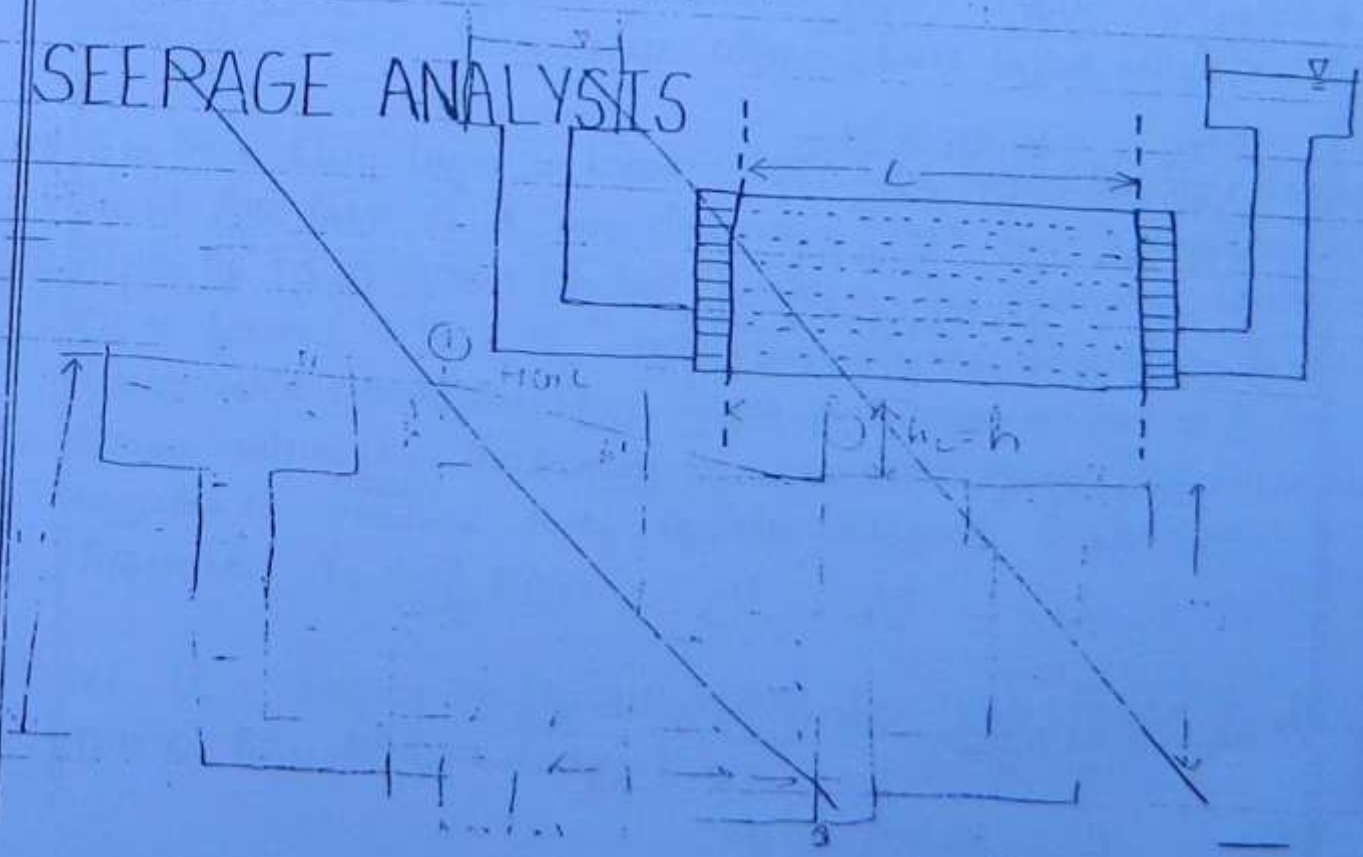
$$T_{v2} = \frac{\pi}{4} U_2^2 = \frac{C_v \cdot t_2}{d_2^2}$$

$$\frac{T_{v1}}{T_{v2}} = \frac{U_1^2}{U_2^2} = \frac{t_1 \cdot d_1^2}{t_2 \cdot d_2^2}$$

$$\frac{t_1}{t_2} = \frac{U_1^2}{U_2^2} =$$

$$t_2 = t_1 \times \frac{U_2^2}{U_1^2} = \frac{0.55^2}{0.278^2} \times 2 = 8 \text{ months}$$

SEEPAGE ANALYSIS



A hand-drawn diagram of a rectangular field divided into two horizontal sections. The top section is labeled (1) and the bottom section is labeled (2). The field is bounded by a dashed line on the left and a solid line on the right. The top section is shaded with diagonal lines.

$$T_{V_1} = \frac{\lambda}{4} U_1^2 = \frac{C_v \cdot t}{d_1^2}$$

$$T_{V2} = \frac{\bar{\Lambda}}{4} U_2^2 = \frac{C_v \cdot t}{d_2^2}$$

$$\frac{T_{V1}}{T_{V2}} = \frac{\rho_1 U_1^2}{\rho_2 U_2^2} = \frac{d_2^2}{d_1^2}$$

$$\frac{U_1}{U_2} = \frac{dz}{dI} = \frac{H_{O2}/2}{H_{O1}/2} = \frac{2.25}{0.75} = 3$$

$$U_1 = 3U_2 \quad \text{--- (1)}$$

Let after time t Settlement of top layer is Δh_1
 " " " " " " bottom " " Δh_2
 Overall Settlement is = Δh

Let ΔH_1 is ultimate settlement of layer 1

$$\Delta H_1 = H_{O_1} \cdot m_0 \cdot \Delta \bar{\sigma}_1$$

$$= H_{O_1} \cdot X_K$$

$$U_1 = \frac{\Delta H_1}{\Delta H_1} = \frac{\Delta H_1}{H_{01} \cdot X_K} \Rightarrow \Delta h_1 = K \cdot U_1 \cdot X_{H_{01}}$$

$$\Delta h_2 = k \cdot U_2 \times H_{O_2}$$

$$\Delta h = -k \cdot U \times H_0$$

$$\Delta h = \Delta h_1 + \Delta h_2$$

$$U \cdot H_0 = U_1 \cdot H_{01} + U_2 \cdot H_{02}$$

$$0.5 \times 6 = 0.1 \times 1.5 + 4.5 \times 0.2$$

$$3 = 1.5U_1 + 4.5U_2$$

$$U_1 + 3U_2 = 2 \quad \text{--- (2)}$$

$$6U_2 = 2$$

$$U_2 = \frac{2}{6} = \frac{1}{3} = 0.33$$

$$-U_1 = 1$$

$$T_{v2} = \frac{\lambda}{4} \times \frac{1^2}{3} = \frac{4.92 \times 10^{-2} \times 10^{-6} \cdot 1}{(2.25)^2}$$

$$t = 103.9 \text{ days}$$

- (iii) Ballon
(iv) The Sand Cone

(291)

Methods of field Compaction

- (a) Rollers - (i) Smooth wheel rollers - These are suitable for gravels & boulders & may also be used for fine soils.
(ii) Pneumatic tyred roller - These have kneading action & may convert flocculant structure into dispersed structure. These are suitable for cohesive soils but also may be used for other type of soils.
(iii) Sheep foot rollers - These are suitable only for cohesive soils & not for granular soils.
(iv) Grid rollers - Suitable for silt & clays. Often used for moist soils.
- (b) Rammers - These are dropping weights such as brog hammer.
- (c) Vibrators - These are suitable for cohesionless soil & concrete work especially when soil is in loose stage.

Ques 6m thick clay layer is located betn two layers of free drainage sand. Also there is a thin drainage layer within the clay at a depth of 1.5 m from its top surface. The average value of C_v is found $4.92 \times 10^{-2} \text{ mm}^2/\text{sec}$. If the structure is to be constructed above the clay layer & cause uniform $\uparrow$ in clay stress within the thickness of clay. How much time will be require to ^{attain} produce half of its ultimate settlement.
Assume $T_v = \frac{\pi}{4} u^2$

solⁿ Let U_1 = Degree of consolidation for top layer 1 & U_2 D.O.C for bottom layer 2.

The diagram shows a horizontal beam of length \$L\$ with a curved profile. Two point loads, \$q_1\$ and \$q_2\$, are applied vertically downwards at distances \$H_1\$ and \$H_2\$ from the left end, respectively. The beam is supported by a vertical line on the left and a vertical dashed line on the right.

$$T_{v_1} = \frac{\lambda}{4} v_1^2 = \frac{C_v \cdot t}{d_1^2}$$

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$$\frac{T_{V_1}}{T_{V_2}} = \frac{d_1^2 U_1^2}{d_2^2 U_2^2} = \frac{d_1^2}{d_2^2}$$

$$\frac{U_1}{U_2} = \frac{d_2}{d_1} = \frac{H_{O_2/2}}{H_{O_1/2}} = \frac{2.25}{0.75} = 3$$

$$U_1 = 3U_2 \quad \text{--- (1)}$$

" " " " " " bottom " Δh_2

" " " " Overhaul Settlement is $= \Delta h$

Let ΔH_1 is ultimate settlement of layer 1

$$\Delta H_1 = H_{0,1} \cdot m_v \cdot \Delta \bar{\sigma}_1$$

$$= H_{0,1} \times K$$

$$U_1 = \frac{\Delta H_1}{\Delta H_1} = \frac{\Delta h_1}{H_{01} \times K} \Rightarrow \Delta h_1 = K \cdot U_1 \times H_{01}$$

$$\Delta h_2 = k \cdot U_2 \times H_{O_2}$$

$$\Delta h = k \cdot U \times H_0$$

$$\Delta h = \Delta h_1 + \Delta h_2$$

$$U \cdot H_0 = U_1 \cdot H_{01} + U_2 \cdot H_{02}$$

$$0.5 \times 6 = 0.1 \times 5 + 4.9 \times 0.2$$

$$3 = 1.5U_1 + 4.5U_2$$

$$U_1 + 3U_2 = 2 \quad \text{--- (2)}$$

$$6U_2 = 2$$

$$U_2 = \frac{2}{6} = \frac{1}{3} = 0.33$$

$$-U_1 = 1$$

$$T_{V2} = \frac{\lambda}{4} \times \frac{12}{3} = \frac{4.92 \times 10^{-2} \times 10^{-6}}{(2.25)^2} \text{ s}$$

$$t = 103.9 \text{ days}$$

- (iii) Ballon
(iv) The Sand Cone

(291)

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Assume $T_v = \frac{\pi}{4} u^2$

Solⁿ Let U_1 = Degree of Consolidation for top layer 1 & U_2 D.O.C for bottom layer 2.

- No. of layers - 5 & blows - 25 to each layer.
- Height of free fall - 457.2 mm (18 inches)
- Weight of hammer = 4.54 kg (10 lb or pounds)

(iii) I.S light Compaction test

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- Volume of mould - 1000 c.c
- Height of free fall - 310 mm
- Weight of hammer = 2.6 kg
- No. of layer = 3 & No. of blow = 25 to each layer.

(iv) I.S heavy Compaction test

- Volume of Mould - 1000 c.c
- Height of free fall - 450 mm
- Weight of hammer - 4.9 kg
- No. of layers = 5 & No. of blow to each layer = 25

Ques The ratio of total energy imparted to the soil in heavy compaction test to the light compaction test is equal to

- (a) 2.0 (b) 3.5 (c) 4.5 (d) 5.0

$$\frac{(E)_{\text{heavy}}}{(E)_{\text{light}}} = \frac{(MgH)_{\text{heavy}} \times \text{No. of layer} \times \text{No. of blows}}{(MgH)_{\text{light}} \times \text{No. of layer} \times \text{No. of blow}}$$

$$\frac{4.9 \times 9 \times 450 \times 5 \times 25}{2.6 \times 9 \times 310 \times 3 \times 25} = 4.56$$

Factors affecting Compaction

1. Compactive effort (total energy given in compaction)
2. Water Content
3. Method of Compaction
4. Type of Soil

The quality of compaction can be checked by following test - (i) nuclear density meter

(ii) Proctor needle test

Compaction of Soils

Date: / / 2011

Compaction is an instantaneous process in partially saturated soil by which density is increased by giving external pressure. Compaction curve between dry density & moisture content.

- (i) For clayey soil - When soil is compacted at certain water content the bulk density is obtained at that water content. The dry density is computed as $\rho_d = \frac{\rho}{1+w}$. The test is repeated

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at diffⁿ Water Content & following graph is found

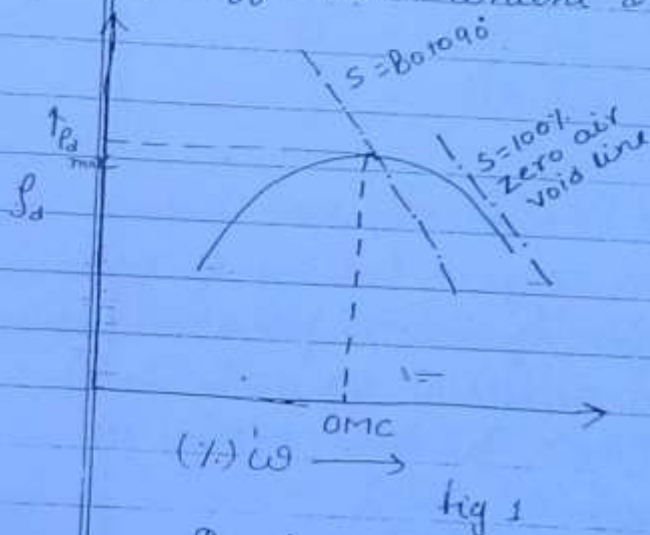


fig 1

When density is max. degree of saturation is 80 to 90%. ~~because~~ practically it is not feasible to expell all the air. However theoretically max. dry density will be achieved when all the air is expelled out. (i.e. $s = 100\%$.) Degree of compactive effort is constant (i.e. 25 blows are given)

If compactive effort is $\uparrow$ then greater max dry density will be achieved at smaller optimum moisture content.

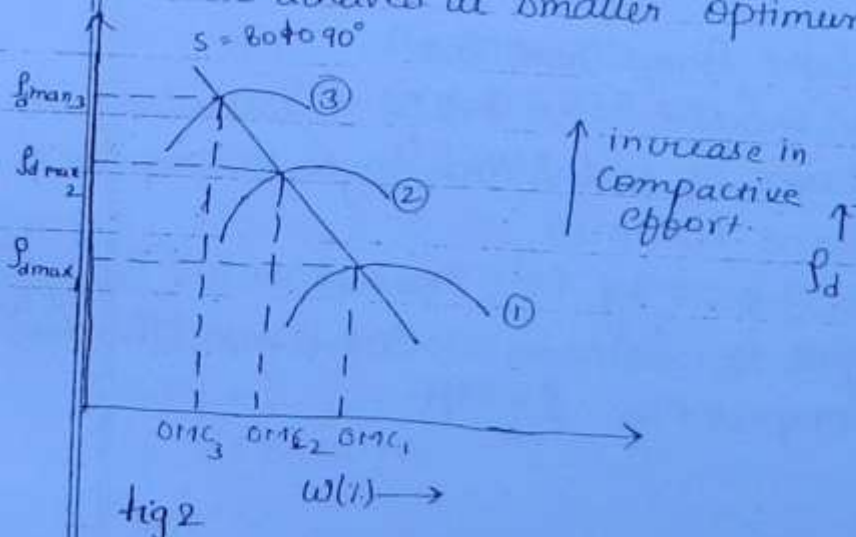
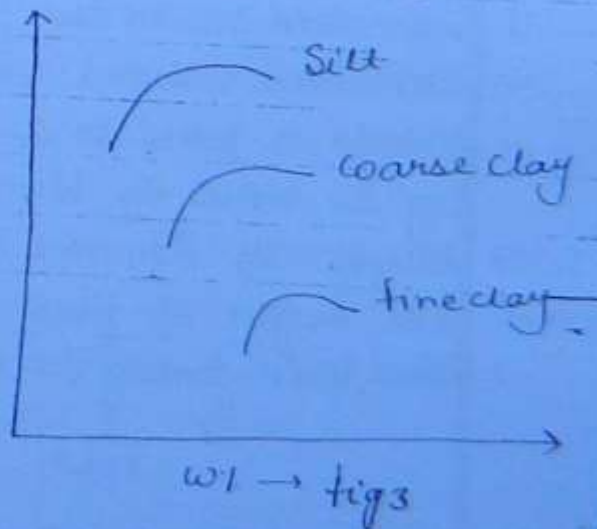


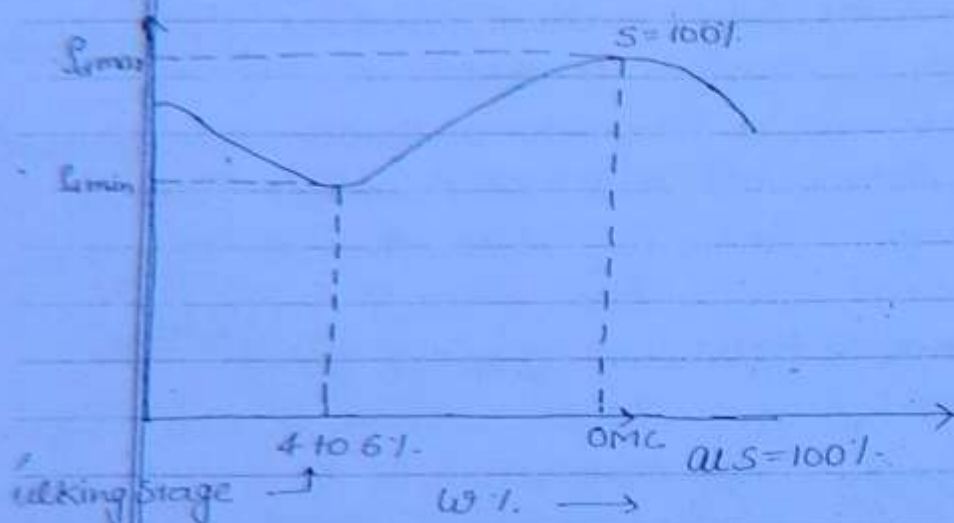
fig 2



w -> fig 3

NOTE If compactive efforts are kept constant but curves are plotted for three diffⁿ soil then following results are obtained from as above fig 3.

Compaction Curve of sands.



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Sands show min density at 4 to 6% moisture content & results in max. volume such as stage is bulking of sand. At this water content a thin layer of water film is available around the solid particle & have repulsive effect. In sand max. density is achieved at $S = 100\%$. The best Method of Compaction of sand is vibration.

Compaction test

(i) Standard proctor test / light Compaction test

- Standard volume of mould = 942 CC (1/30 Cubic feet)
 - Mould is filled in 3 layer of soil & each layer is compacted by 25 blows of standard hammer
 - Weight of hammer is 2.454 kg (5.5 Pound)
 - The height of free fall of hammer is 304.8 mm (12 inches)
 - The bulk density is computed as $\rho = M/V$
- $$\rho_d = \frac{\rho}{1+w}$$

(ii) Modified proctor test / heavy Compaction test

- It was designed during IInd world war to construct pavement to support heavy air traffic
- Volume of mould = 942 CC (1/30 Cubic feet)

Soln Clay layer is compressible

Initial eff Overburden pressure at Centre of Clay layer

$$\begin{aligned}\text{At C-C } \bar{\sigma}_0 &= \sigma - u \\ &\text{effective } 10 \times \gamma_{\text{sat}} + 3 \cdot \gamma_{\text{sat}} - 13 \times \gamma_w \\ &10 \times 19.62 + 3 \times 19.62 - 13 \times 9.81 \\ &= 127.53\end{aligned}$$

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CASE II If water table is lowered by six metre then

$$\bar{\sigma}_0 = \sigma - u$$

$$\sigma = 6 \times \gamma_{\text{sat}} + 4 \times \gamma_{\text{sat}} + 3 \times \gamma_{\text{sat}} - 7 \gamma_w$$

$$= 186.39 \text{ kN/m}^2$$

Increase in eff stress At C-C

$$\Delta \bar{\sigma} = 186.39 - 127.53 = 58.89 \text{ kN/m}^2$$

$$m_v = 0.02 \text{ cm}^2/\text{kg}$$

$$= \frac{0.02 \times 10^{-4}}{9.81 \times 10^{-3}} = \frac{\text{m}^2}{\text{kN}} = 2.038 \times 10^{-4}$$

Settlement of Clay

$$\Delta H = H_0 m_v \cdot \Delta \bar{\sigma}$$

$$= 6 \text{ m} \times 2.038 \times 10^{-4} \times 58.89 = 0.0720 \text{ m}$$

$$= 72 \text{ mm}$$

A saturated soil has compression ratio of 0.25 if unit ratio of the stress of compression is 205 kPa and the initial effective stress is 100 kPa. Compute the settlement of the soil layer having thickness of 1.5 m and unit weight of 19.6 kN/m³ & water content of 40% and liquid limit of 60%.

= 0.25

In the laboratory test on a clay sample of thickness 25mm drained at top only 50% Consolidation occurred in 11 minutes. Find the time required for the same clay layer in the field having 2m thickness & drained at top & bottom both. Go through 70% Consolidation given that time factor $T_{50} = 0.197$ & $T_{70} = 0.405$

Lab test $d = 25\text{mm} = H_0$ (Single drainage)
 $t = 11$ minutes for 50% Consolidation
 $T_{50} = 0.197$ $U = 0.5$

$$T_{50} = \frac{C_v \cdot t_{50}}{d^2} \Rightarrow C_v = \frac{0.197 \times 25^2}{11} \text{ mm}^2/\text{min}$$

Field Condition $H_0 = 2\text{m}$

$$d = \frac{2}{2} = 1\text{m} \text{ (double drainage)}$$

$$T_{70} = 0.405$$

$$T_{70} = \frac{C_v \cdot t_{70}}{d^2}$$

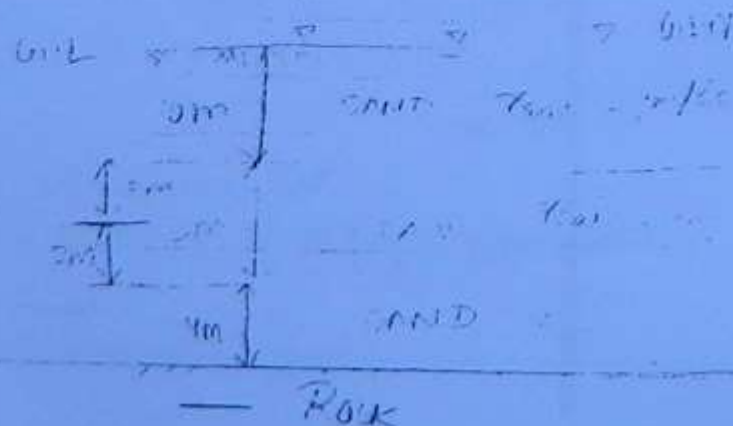
$$t_{70} = \frac{T_{70} \times d^2}{C_v} \Rightarrow \frac{0.405 \times 1^2 \times (1000)^2 \times 11}{0.197 \times 25^2} \text{ min}$$

$$\frac{24 \times 60}{1} \rightarrow \text{days} = 25.13 \text{ days}$$

At a given lat the soil profile is shown as below. The water table is well above the clay layer. The clay layer is 2m thick. The soil is saturated. The unit weight of soil is $\gamma_{sat} = 19.62 \text{ kN/m}^3$ and the unit weight of water is $\gamma_w = 9.81 \text{ kN/m}^3$.

$$\gamma_{sat} = 19.62 \text{ kN/m}^3$$

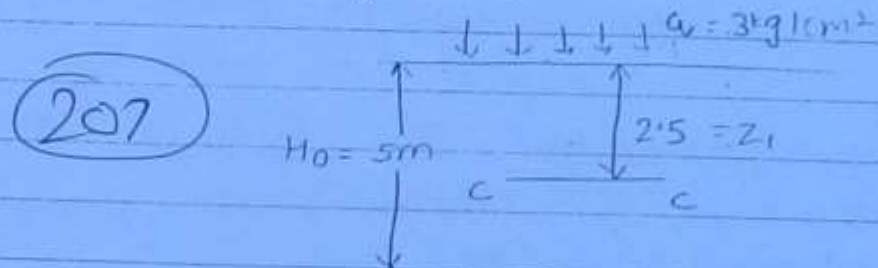
$$\gamma_w = 9.81 \text{ kN/m}^3$$



$$\gamma_{sat} = 2 \times 9.81 = 19.62 \text{ kN/m}^3$$

$$\gamma_w = 9.81 \text{ kN/m}^3$$

A layer of soft clay 5m thick lies under newly constructed building. The eff pressure due to overlying strata on the center of clay layer is 3 kg/cm^2 & new construction increases the over burden pressure by 1.2 kg/cm^2 . If compression index of the clay is 4.5 then compute the settlement of the clay assuming natural water content of clay 43% & $G_r = 2.7$, degree of saturation = 100%.



Initial eff over burden pressure at centre of clay layer

$$\bar{\sigma}_0 = q_0 + \gamma_{sub} \cdot Z_1$$

$$S \cdot e = w \cdot G_r$$

$$e = 1.161$$

$$\gamma_{sub} = \frac{G_r - 1}{1 + e} \gamma_w \Rightarrow \frac{2.7 - 1}{1 + 1.161} 9.81$$

$$= 7.72 \text{ kN/m}^3$$

$$q = 3 \text{ kg/cm}^2 = \frac{3 \times 9.81 \times 10^{-3}}{10^{-4}} = 294.3 \text{ kN/m}^2$$

$$\bar{\sigma}_0 = q_0 + \gamma_{sub} \cdot Z_1$$

$$= 294.3 + 7.72 \times 2.5$$

$$= 313.6 \text{ kN/m}^2$$

$$\Delta \bar{\sigma}_0 = 1.2 \text{ kg/cm}^2 = 1.2 \times 9.81 \times 10^{-3} / 10^{-4}$$

$$= 117.72 \text{ kN/m}^2$$

$$C_c = 4.5$$

$$e_0 = 1.161$$

$$S_c = \Delta H = \frac{C_c H_0}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}_0}{\bar{\sigma}_0} \right)$$

$$= \frac{4.5 \times 5}{1 + 1.161} \log_{10} \left(\frac{313.6 + 117.7}{313.6} \right)$$

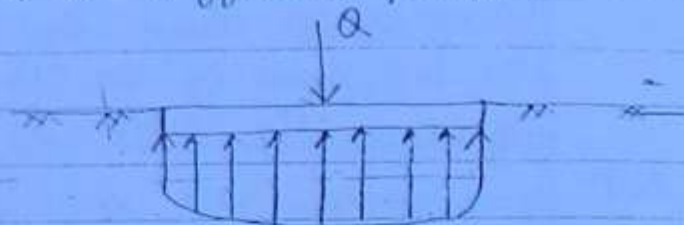
$$= 1.44 \text{ m}$$

Contact pressure at the base of foundation.

1. Flexible foundation - In flexible foundation Contact pressure for all type of soils is nearly same & uniform however settlement may be different at different points.

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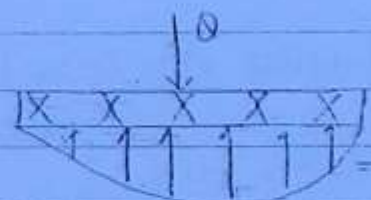
Flexible foundation
(for all soils)



2. Rigid foundation Such as Raft - In Rigid foundation the settlement is uniform but pressure distribution depends upon type of soil.

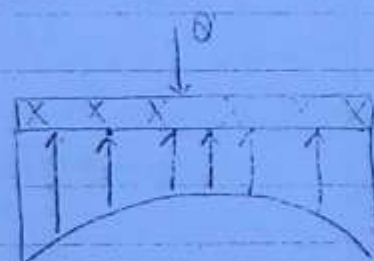
① Cohesionless soil (sand)

Zero at edge



Rigid foundation in sand

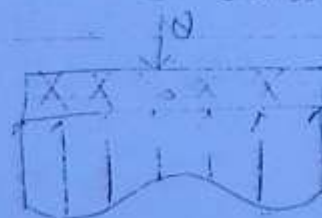
Max at edge



Rigid foundation in Clay
= soils

Max at center

Min at edge



Max at center
Min at edge

Date / /

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_0 - \Delta \bar{\sigma}}{\bar{\sigma}_0} \right) + \frac{C_c H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_1 - \Delta \bar{\sigma}'}{\bar{\sigma}_1} \right)$$

Where $\bar{\sigma}_1 = \bar{\sigma}_0 - \Delta \bar{\sigma}$

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$(\bar{\sigma}_0 \text{ to } \bar{\sigma}_1 (\bar{\sigma}_0 + \Delta \bar{\sigma})) \rightarrow$ Over Consolidated
 $\bar{\sigma}_1 \text{ to } \bar{\sigma}_1 + \Delta \bar{\sigma}' \rightarrow$ Normally Consolidated

- ③ If below the foundation soil mass consist of sand Clay & gravel layer then total settlement is due to settlement of all the layers but since settlement of gravel & sand is insignificant as compared to that of Clay layer hence for all practical purposes settlement of gravel & sand is neglected.

Determination of Secondary Consolidation settlement

The Secondary Consolidation is due to plastic readjustment of soil molecules & for coarse soils it is negligible but for highly plastic clays its value may be 10 to 20% of total settlement. It is time dependent & occurs at very slow rate.

$$S_s = \frac{C_s H_{100}}{1+e_{100}} \log_{10} \left(\frac{t}{t_{100}} \right)$$

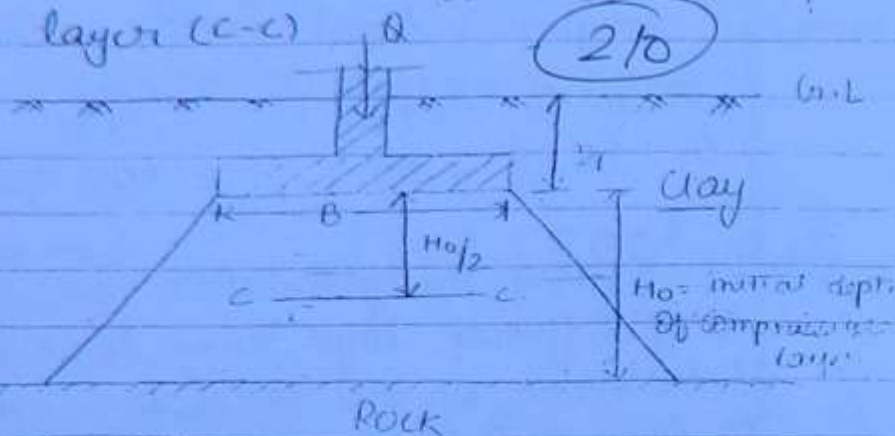
here H_{100} - thickness of soil layer after primary consolidation (100% consolidation)
 e_{100} - void ratio of after end of primary consolidation
 C_s - Secondary compression index
 t_{100} - time req. for primary consolidation (90% cons)
 t - any time after primary consolidation at $\underline{\underline{c}}$ Secondary Consolidation is required.
 $t \gg t_{100}$

2. If m_v is coefficient of volume compressibility at the centre of compressible layer & $\Delta \bar{\sigma}$ = increase in effective stress at the centre of compressible layer (c-c)

Find $\bar{\sigma}_0$ at c-c

" $\Delta \bar{\sigma}$ at c-c

" m_v at c-c



$$\Delta H = S_c = H_0 \cdot m_v \cdot \Delta \bar{\sigma}$$

III. Since $\Delta \bar{\sigma}$ & m_v vary w the depth hence $\Delta \bar{\sigma}$ & average value is taken at the centre but for precise computation total depth H_0 should be divide in more no. of layers & m_v & $\Delta \bar{\sigma}$ should be computed at the centre of each layer.

$$\Delta H = \sum H m_v \cdot \Delta \bar{\sigma}$$

H = height of any layer

3. If Coefficient of Compression then

$$\Delta H = S_c = \frac{C_c H_0}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)$$

H_0 - initial height at the beginning of consolidation

e_0 - initial void ratio "

C_c - Coefficient of Compression

$\bar{\sigma}_0$ - initial eff. overburden pressure at the centre of compressible layer

$\Delta \bar{\sigma}$ - increase in eff. stress at the centre of compressible layer.

11.10. If Soil is over consolidated then use C_u instead of C_c

12. If for initial part soil is over consolidated or for later part soil is normally consolidated then

$$S_c = S_{c1} + S_{c2}$$

Where $\bar{\sigma}_0$ - initial eff. overburden pressure at the centre of soil layer.
 $\Delta \bar{\sigma}$ - increase in eff. pressure at the centre of soil layer
 $C_s = 1.5 \left(\frac{C_u}{\bar{\sigma}_0} \right) = C_n \rightarrow$ Static pore ^{cone} resistance in KN/m^2
 H_0 = initial depth of soil layer

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For cohesive soils (clays) (distortion settlement)

In case of saturated clays immediate settlement is insignificant because pore pressure cannot dissipate immediately due to low permeability however small settlement may occur due to deformation of clay & squeezing of water.

Immediate settlement below a rectangular flexible foundation - is given by

$$S_i = \frac{q \cdot B}{E_s} (1 - \mu^2) I_t \quad \text{Schleicher's eqn}$$

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q = uniform pressure at the base of foundation

B = width of foundation elastic (Plastic condition)

μ = Poisson ratio of soil ($0.2 - 0.5$)

For soft saturated clays under undrained condition $\rightarrow 0.5$

E_s = Young's modulus of soil

I_t = Influence factor or shape factor depends upon shape & rigidity of foundation.

Determination of Consolidation Settlement (S_c)

1. Let $\Delta H = S_c$ is settlement of soil layer due to applied eff. stress $\hookrightarrow$ Causes change in volume $\hookrightarrow$ results into change in void ratio.

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0}$$

$$\Delta H = S_c = \left(\frac{\Delta e}{1 + e_0} \right) H_0$$

Δe = change in void ratio
 H_0 - initial depth
 e_0 - initial void ratio

in organic soil - S_s assumes great significance
 independent of total settlement.
 (1st choice)

② Primary Compression Ratio (η_p)

$$\eta_p = \frac{R_o - R_{100}}{R_i - R_f}$$

③ Secondary Compression Ratio (η_s)

$$\eta_s = \frac{R_{100} - R_f}{R_i - R_f}$$

2/2

NOTE THAT $\eta_i + \eta_p + \eta_s = 1$

SETTLEMENT ANALYSIS

Total Settlement $S = S_i + S_c + S_s$

S_i = Immediate Settlement

S_c = Primary Consolidation Settlement or Simply Consolidation Settlement

S_s = Secondary Consolidation Settlement

Permissible Settlement as Per BIS

Type of Foundation	Permissible total Settlement
Isolated foundation on Clay	65 mm
Isolated foundation on Sand	40 mm
Raft foundation on Clay	65-100 mm
Raft foundation on Sand	40-65 mm
Permissible differential Settlement	
Raft foundation on Clay	40 mm
Raft foundation on Sand	25 mm

Permissible Angular Distortion

→ In case of large framed structure angular distortion must not exceed $\frac{1}{500}$ in general & if all types of minor damages are to be prevented then angular distortion must not exceed $\frac{1}{1000}$

(not independent)

Determination of Immediate Settlement (take place immediately or less than 1 day after the load is placed)

1. For Cohesionless Soil (Sand) - To determine immediate Settlement Standard penetration test data may be used

$$S_i = \frac{H_o}{C_{s\%}} \log_{10} \left(\frac{\bar{\sigma}_o + \Delta \bar{\sigma}}{\bar{\sigma}_o} \right)$$

Determination of C_v (Coefficient of Consolidation)

C_v depends on type of soil & $\downarrow \bar{e} \uparrow$ in liquid limit.

For most of the soils its value lies betⁿ $5 \times 10^{-4} \text{ mm}^2/\text{sec}$ to $2 \times 10^{-2} \text{ mm}^2/\text{sec}$.

Thou Through value of C_v changes $\bar{e}$ change in effective stress but for the purpose of calculation its ^{avg} value is taken

There are two methods to determine C_v

1. Square root of time fitting method

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It was given by Taylor & Taylor eqⁿ is

$$C_v = \frac{T_{90} \cdot d^2}{t_{90}}$$

T_{90} = time factor for 90% Consolidation - 0.848

t_{90} = time req. for 90% Consolidation

d = length of drainage path

This method is better for those soils $\leq$ have high Secondary Consolidation such as highly plastic clays

2. Logarithm of time fitting Method

It was given A Casagrande

$$C_v = \frac{T_{50} \cdot d^2}{t_{50}}$$

$T_{50} = 0.196 \text{ or } 0.197$

Compression Ratios

Let R_i is initial reading of dial gauge, R_o is reading of dial gauge at the beginning of primary consolidation, R_{100} is reading of dial gauge after 100% primary consolidation, R_f final reading of dial gauge after secondary consolidation.

① Initial Compression Ratio

$$r_c = \frac{R_i - R_o}{R_i - R_f}$$

$$\begin{aligned} \text{At } t=0, \quad e &= e_0 \\ \text{At } t \rightarrow \infty, \quad e &= e_{100} \end{aligned}$$

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Time factor (T_v)

It is the parameter $\leq$ relates the degree of consolidation & time req. for consolidation

$$T_v = \frac{C_v \cdot t}{d^2}$$

$$\begin{cases} d = \frac{H_0}{2} & \text{double drainage} \\ d = H_0 & \text{Single drainage} \end{cases}$$

C_v = Coefficient of Consolidation

t = time req. for any stage of consolidation at c time factor is T_v

d = length of drainage path

In precise computation length of drainage path should be taken average value (ie $d = \frac{d_i + d_f}{2}$)

d_i = initial length of drainage path H_0 (double drainage)

d_f = final length of drainage path at the time of completion of consolidation $[d_f = \frac{H_0 - \Delta H}{2}]$ (double drainage)

T_v is related to degree of consolidation $\Rightarrow$

Valid — When $u \leq 0.6$

$$T_v = -0.9332 \log_{10} (1-u) - 0.0851 \quad \text{When } u > 0.6$$

① degree of

If consolidation is 50% $u = 0.5$ then $T_{50} = \frac{\pi}{4} (0.5)^2 = 0.196$

If $u = 0.9$ then $T_v = T_{90} = -0.9332 \log_{10} (1-0.9) - 0.0851$
 $T_v = T_{90} = 0.848$

② When degree of consolidation reaches greater or equal to 96% consolidation is assumed to be completed.

$$m_v = - \left(\frac{\Delta H}{H_0} \right) \div - \left(\frac{\Delta e}{1+e_0} \right) = \frac{a_v}{1+e_0}$$

a_v = Coefficient of compressibility (215)

Degree of Consolidation

It is the fraction of Consolidation stage completed at any time to the total primary Consolidation

$$0 \leq U \leq 1$$

$$0\% \leq U \leq 100\%$$

At beginning $t=0$ $U=0\%$

At completion $U=100\%$

Theoretically it is infinite

Determination of degree of Consolidation

1. If H_0 is initial depth of Clay layer & ΔH is final settlement of Clay layer after primary Consolidation is completed ($U=100\%$)
If Δh is settlement at any stage then degree of Consolidation at this stage will be $\frac{\Delta h}{\Delta H} \times 100 = \% U$

2. Let $u_i (= \Delta \bar{\sigma})$ is initial excess pore pressure developed due to τ in eff. stress ($\Delta \bar{\sigma}$). Let u is excess pore pressure at any stage of time then degree of Consolidation is given as $\frac{u_i - u}{u_i} \times 100 = \% U$

At Beginning ($t=0$) $u = u_i \Rightarrow \% U = 0$

At Completion ($t=\infty$) $u = 0 \Rightarrow \% U = 100\%$

3. Let e_0 is initial void Ratio, e_{100} is void ratio after completion of primary Consolidation then at any stage when void Ratio is e then

$$\% U = \frac{e_0 - e}{e_0 - e_{100}} \times 100$$

$$\frac{\partial U}{\partial t} = C_v \cdot \frac{\partial^2 U}{\partial z^2}$$

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$\frac{\partial U}{\partial t}$ = Rate of Change of pore pressure. τ the time i.e. rate of Consolidation

C_v = Coefficient of Consolidation C depends upon type of Soil & is defined as

$$C_v = \frac{K}{\gamma_w \cdot m_v}$$

K = Coefficient of permeability / Hydraulic Conductivity

γ_w = Unit wt. of water

m_v = Coefficient of Vol. Change / Coefficient of Volume Compressibility / modulus of Vol. Change.

$\frac{\partial U}{\partial z}$ = Rate of Change of pore pressure. τ the depth

Inverse of m_v is called Compression modulus

$$E_c = \frac{1}{m_v}$$

Coefficient of Volume Compressibility is defined as Ratio of Unit Vol. Change to the Corresponding Change in effective Stress.

$$m_v = - \left(\frac{\frac{\Delta V}{V}}{\Delta \bar{\sigma}} \right) \quad \frac{m^2}{KN}$$

$m_v \uparrow \tau \downarrow$ in $\Delta \bar{\sigma}$

TE Since Soil mass is Considered infinite (area is large & constant & depth is finite C may change) Hence $\frac{\Delta V}{V} = \frac{\Delta H}{H_0}$

$H = H_0$ = initial height

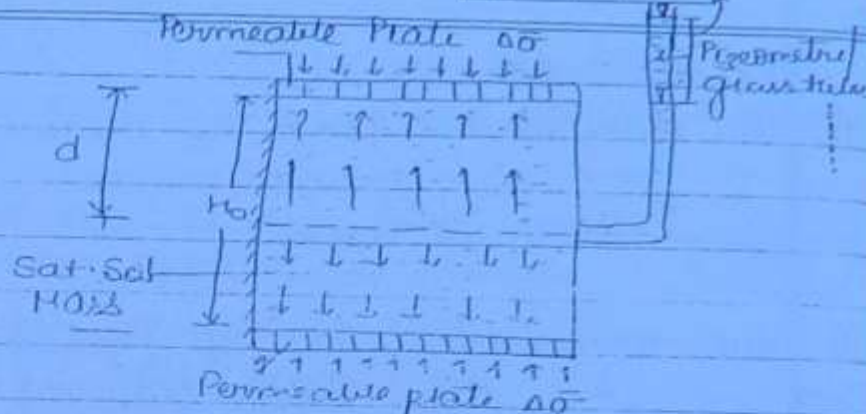
$$\frac{\Delta V}{V} = \frac{\Delta H}{A H_0} \Rightarrow \frac{\Delta V}{V} = \frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0}$$

e_0 = initial void Ratio

Δe = Change in void Ratio due to Change in effective Stress

$$\frac{\Delta V}{V} = \frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0}$$

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Let top & bottom both plates are permeable hence soil is in double drainage system. If only one plate is permeable top/bottom then soil is in single drainage condition.
 length of drainage path = $d = \frac{H_0}{2}$ in double drainage
 $d = H_0$ in single drainage

Let u is excess pore pressure developed in the soil mass due to increase in effective stress. At the beginning of consolidation

$$u = u_i = \Delta \sigma \quad \text{--- (At time } T = 0 \text{)}$$

After completion of primary consolidation excess pore pressure reduces to 0. So At time $t = \infty$

Theoretically Consolidation will be completed in time $t = \infty$ but practically Consolidation will complete in finite time & generally when degree of consolidated Consolidation is $\geq 90\%$. it is assumed to be completed.

The laboratory conditions are designed analogous to field condition i.e. if field soil is in double drainage then consolidometer has top & bottom^{both} face permeable & if field soil is in single drainage then either top or bottom plate is permeable in consolidometer.

The rate of Consolidation $\propto \frac{1}{D^2} \quad \frac{\Delta V}{\Delta t} \propto \frac{1}{D^2}$

D = length of drainage path.

The one dimensional eqⁿ of consolidation given by Terzaghi is

Determination of C_c

① For undisturbed clays of medium sensitivity

$$C_c = 0.009 (w_L - 10)$$

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w_L = % liquid limit

② For remoulded soil of medium to low sensitivity

$$C_c = 0.007 (w_L - 10)$$

③ If natural moisture content is given then

$$C_c = 0.115 w_n$$

w_n = % natural moisture content

④ If void ratio is given

$$C_c = 0.40 (e_0 - 0.25) \text{ or } 1.15 (e_0 - 0.35)$$

e_0 = initial void ratio at the beginning of consolidation

Tergaghi theory of one dimensional consolidation

Assumption

1. The soil mass is homogenous & isotropic hence $K_x = K_y = K_z$
 2. Soil is fully saturated ($S = 100\%$) & remains saturated throughout the process of consolidation
 3. Consolidation is one dimensional & movement of water is unidirectional (vertical) & there is no change in area
 4. Soil mass is semi-infinite (area large & constant & height small & may change)
 5. Darcy law is valid & flow is laminar
 6. The strains are small (due to $\Delta \sigma$)
- The hydrodynamic lag is considered but plastic lag is ignored where as plastic lag is known to exist. It means theory is valid for primary consolidation not secondary consolidation.

Ratio of

Over Consolidation Ratio - It is the max. eff. Stress upto c Soil is stressed in the past to the present applied Effective Stress

$$O.C.R = \frac{\text{Max applied eff. Stress in past}}{\text{Present applied eff. Stress.}}$$

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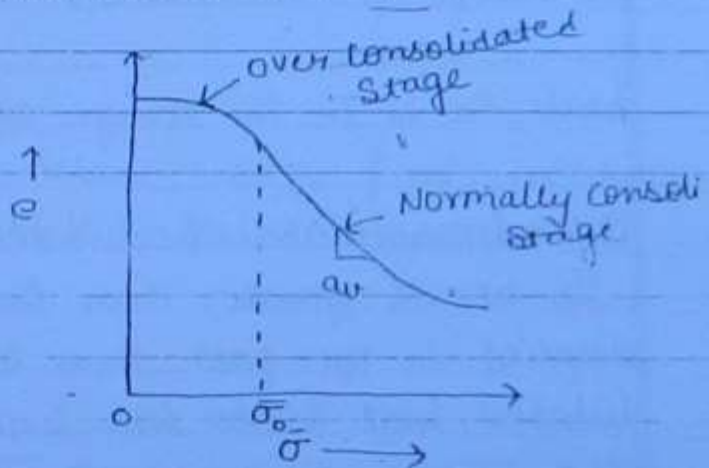
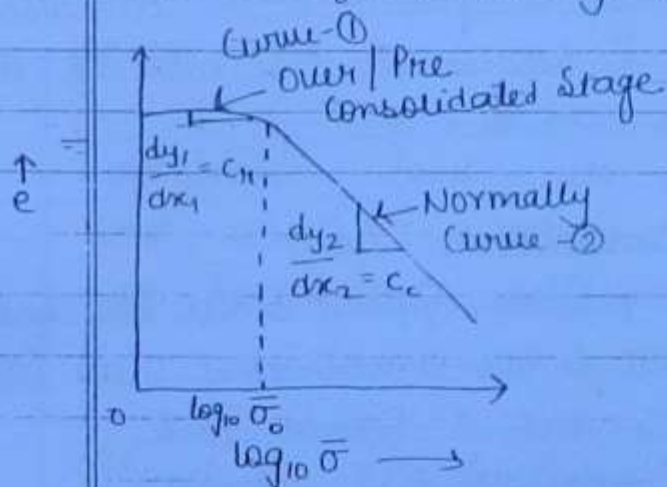
For O.C Soil O.C.R will be greater than 1.

Heavily pre Consolidated Soils have $O.C.R \gg 1$

O.T.E If heavily pre Consolidated Clay with high O.C.R behave like dense sand

For Normally Consolidated Soil $O.C.R < 1$

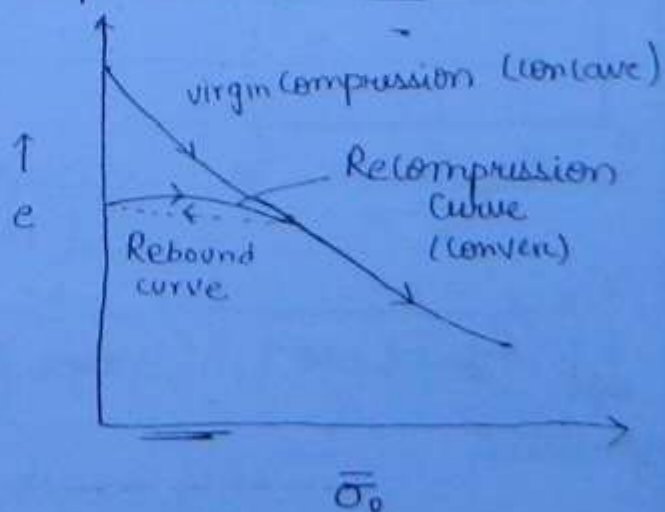
Max. OCR for normally Consolidated Soil is 1



Slope of Curve 1 is Called C_n

Slope of Curve 2 is Called C_c

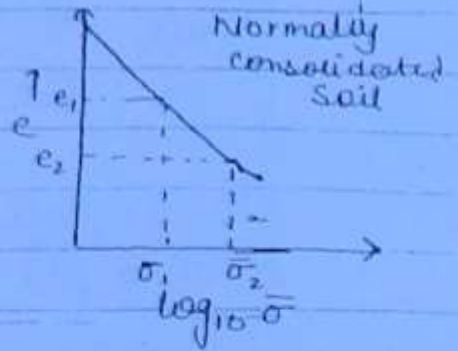
Cyclic loading (Eff Stress v/s void ratio curve)



α_v - Coefficient of Compressibility depends upon type of soil & magnitude of load.

If τ in eff. stress is taken in log scale then curve is found to be straight line & slope is called Coef. of Compression.

(220)



Coef. of Compression $C_c = - \frac{\Delta e}{\log_{10}(\bar{\sigma}_2 / \bar{\sigma}_1)}$

$$C_c = \frac{-\Delta e}{\log_{10} \frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0}}$$

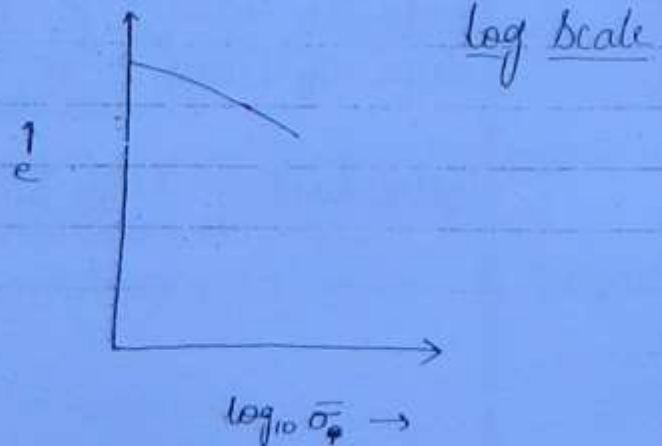
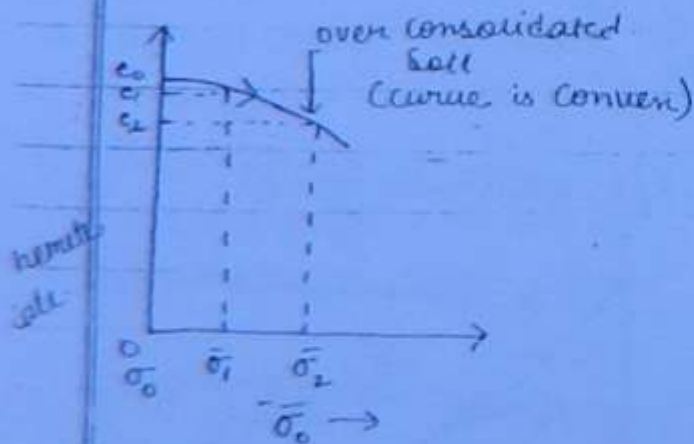
C_c does not change $\bar{e}$ change in eff. stress but it depends on type of soil. Greater the mag. of C_c greater is the compressibility & settlement. For most of the

soils C_c is in the range of 0.1 to 0.8.

Over Consolidated Soils / Reconsolidated Soils

If stress greater than the present applied stress has been applied in the past then soil is over consolidated. Over Consolidated soil shows less settlement & slower rate.

Eff. Stress v/s void ratio curve



$$C_{\alpha} = \frac{-\Delta e}{\log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)} = \frac{-\Delta e}{\log_{10} \left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1} \right)}$$

Coef. of Compression

$$C_{\alpha} = \left(\frac{1}{5} \text{ to } \frac{1}{10} \right) \text{ of } C_c$$

From load settlement curve Q_{up} is known

$$\text{hence } Q_{ap} = \frac{Q_{up}}{P.O.S.} \quad \text{P.O.S. } 2.5 \text{ to } 3$$

(221)

IS code recommends procedure to find allowable load on the pile using settlement criteria as follows -

1. The allowable load is taken as 50% of ultimate load at which total settlement of pile is $\frac{1}{10}$ of dia. or $\frac{1}{10}$ of dia.
2. Allowable load is taken as $\frac{2}{3}$ of ultimate load at which total settlement of pile is 12 mm or
3. Allowable load is taken as $\frac{2}{3}$ of ultimate load which causes net plastic settlement or permanent settlement or 6 mm.

Standard Penetration Test :-

Let N is corrected SPT no. at the base of pile.

Let $\bar{N}$ is avg. corrected SPT no. over the depth of pile.

1. ultimate load capacity of driven piles (Displacement Piles) -

$$Q_{up} = (400N)A_b + (2\bar{N})A_s$$

$P.O.S. = 4.0$ for driven piles

2. For bored piles :-

(222)

$$Q_{up} = (133N) A_b + \left(\frac{2}{3} \bar{N}\right) A_s \quad [FOS = 2.5]$$

3. For H-Piles (Non displacement piles) -

Generally Q_{up} for Non displacement piles is 50% of Q_{up} for displacement piles.

Cone Penetration Test :-

Pile load capacity can be determined using static cone resistance q_c (kg/cm^2)

Let q_c = cone resistance at base of pile

$\bar{q}_c$ = Avg. cone resistance in kg/cm^2 over the depth of pile

then ultimate load on pile in KN is given as -

$$Q_{up} = q_c A_b + \frac{\bar{q}_c}{2} A_s$$

where A_s & A_b are in m^2
↳ Q_{up} in KN

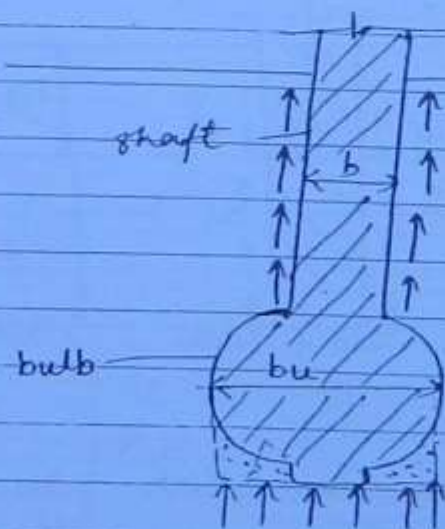
⇒ For non displacement pile / H pile Q_{up} is taken as 50% of Q_{up} of displacement pile.

UNDER REAMMED PILES :-

Underreamed piles contains one or more than one bulbs to increase the load carrying capacity. These are used in expansive soil to take care of uplift pressure due to swelling eg - BLACK COTTON SOIL.

The depth at which bulb should be placed should be in stable condition i.e. no variation of water content. Due to increase in bulb area, bearing area increases. The ratio of bulb dia to pile dia (shaft dia) should be 2 to 3 and min^m spacing b/w adjacent bulbs should be 1.5 times of bulb dia.

Underreamed piles cannot be driven hence these are essentially bored piles.



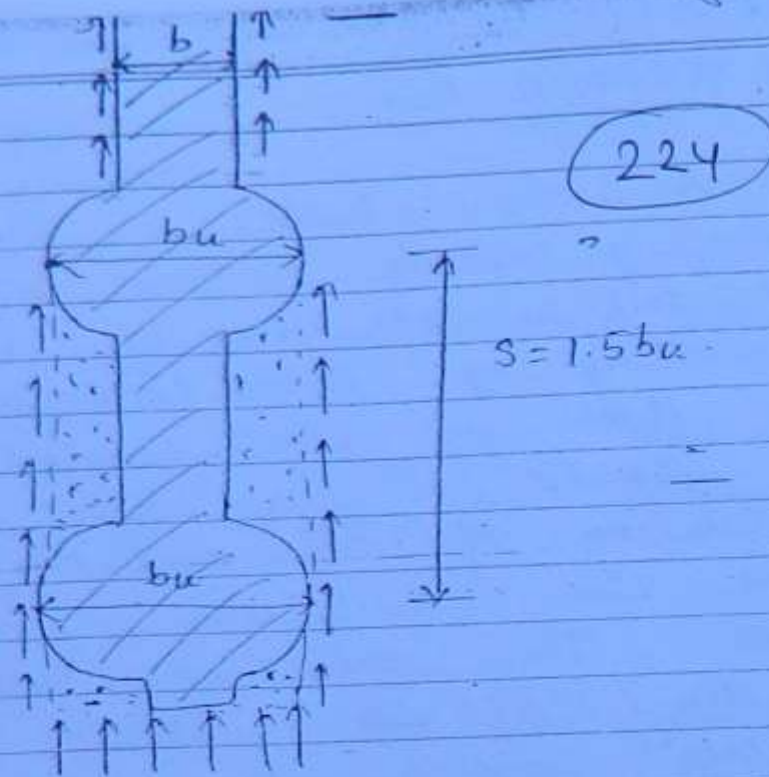
(223)

$$\frac{b_u}{b} = 2 \text{ to } 3$$

$$A_b = \frac{\pi}{4} (b_u)^2$$

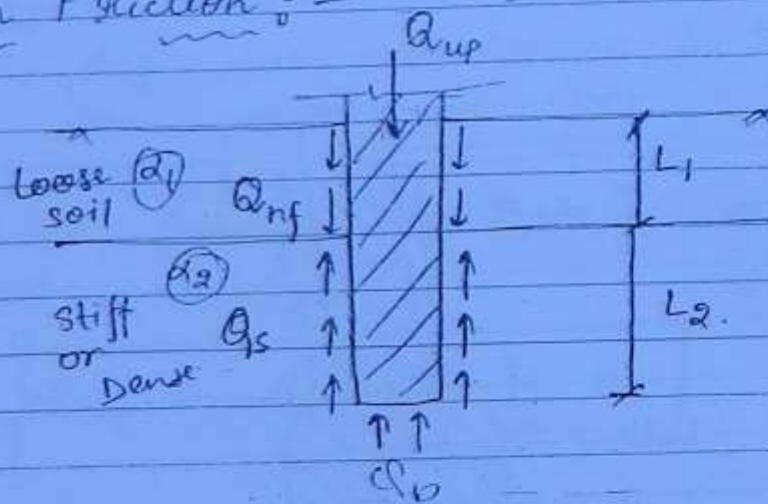
(Single bulb U/R. Pile)
Area of bulb

$$Q_u = \underbrace{C N_c}_{\downarrow q} A_{b1} \propto \underbrace{C N_c}_{\downarrow q}$$



Multibulb U/R Pile.

Negative Skin Friction :-



It is the phenomenon in which surrounding soil settles more than the settlement of pile. This condition occurs when surrounding soil is very loose or soft & pile rest on stiff strata. Following condition may cause negative skin friction.

- 1) increase in surcharge over surrounding soil.
- 2) lowering of GWL
- 3) disturbance due to earthquake & other dynamic effects.

The negative skin friction reduces the load carrying capacity of pile

$$\text{loss} \left\{ Q_{nf} = \bar{q}_s (11D) L_1 \right\}$$

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$\bar{q}_s$ = avg. unit skin resistance over depth L_1 .

for cohesive soils $\left\{ \bar{q}_{s1} = \alpha_1 \bar{c}_1 \right\}$

surf $\left\{ \bar{q}_s = \bar{q}_{s2} (11D) L_2 \right\}$

$$\bar{q}_{s2} = \alpha_2 \bar{c}_2$$

$$Q_b = q_b \cdot A_b$$

$$Q_b = (1.9c) \frac{\pi D^2}{4}$$

The P.O.S will be reduced and it is given by

$$\text{P.O.S.} = \frac{Q_{up}}{\text{working load} + Q_{nf}}$$

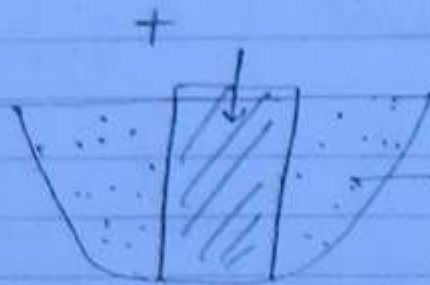
Stress zone in loaded Piles :-

1. End bearing Resistance :-

(226)

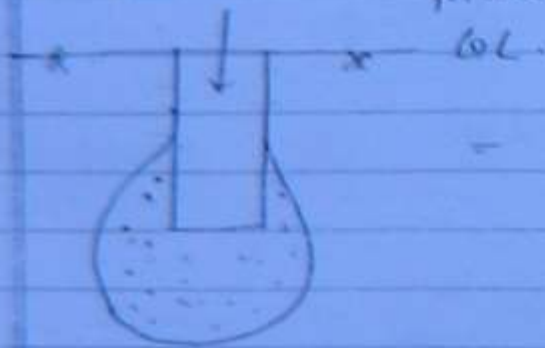


(end bearing resistance).



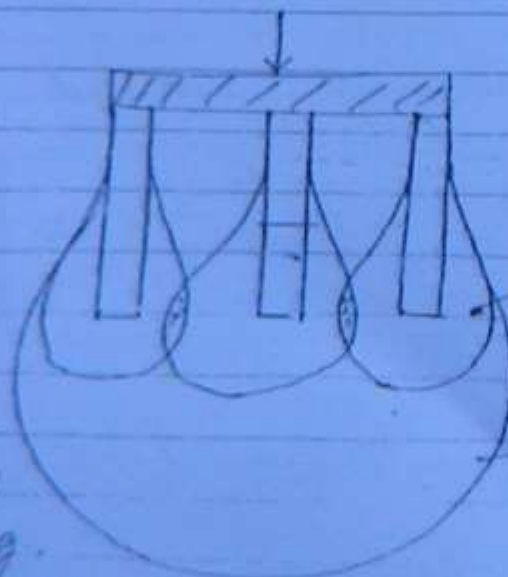
Friction action

equivalent/eff. stress zone in bearing & friction.



Group action of pile :-

If load applied
to larger & more
number of piles
are used then
piles may act
individually
or in group
action depending
upon the spacing.



individual action.

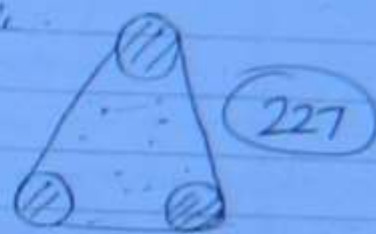
Group Action

If spacing is less than $4d$ ($d = \text{dia}$) the soil may get

compacted b/w piles & these may be group action.
 On group action all piles & compacted soil b/w piles act as a single wedge / single pile. Under such condition more soil area beneath the soil is stressed and hence more settlement will occur and load carrying capacity may increase.

The min^m no. of piles for group action should be 3 which are placed in the form corner of equilateral triangle.

Pile group may be triangular, rectangular, square, circular & polygonal but square is preferred.



For grp. action spacing b/w pile should be kept b/w $2.5D$ to $3.5D$. Generally spacing is less than in friction pile than the end bearing piles.

Determination of ultimate load carrying capacity of pile load :-

$$Q_{ug} = Q_b + Q_s$$

$$= q_b A_b + q_s A_c \rightarrow 1$$

$$Q_{ug} = (q_c) b^2 + (\alpha c) (4B \cdot L)$$

= ultimate load carrying capacity of pile group.

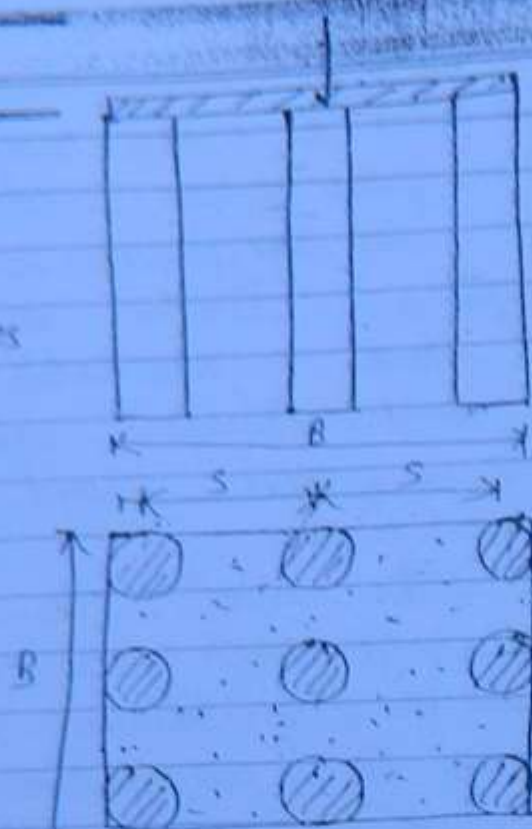
If there is individual action then Q_{up} is ultimate load of single pile than ultimate load of all pile will be n times Q_{up} .

$$B = 2S + D$$

D = Dia. of piles

S = of spacing

B = size of pile group



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The allowable load in pile group should be taken min^m of $\frac{Q_{ug}}{F.O.S}$ & $\frac{n Q_{up}}{F.O.S}$

F.O.S - 2.5 to 3

Following Parameter may be assumed for the pile group if required -

1) ADHESION FACTOR -

$\alpha = 0.6$ to 0.9 (loose clay)

$\alpha = 0.3$ to 0.6 (dense clay)

2) LENGTH OF PILE -

For friction piles $L = 10$ to 20 m

for bearing piles $L =$ depends upon depth of stiff strata.

3) Spacing :- the spacing b/w piles should be kept 2.5 to 3.5 D. More commonly S is taken 3D.

4) Dia. :- 0.3 to 0.9 m is commonly adopted.

5) Group efficiency :- should be ≥ 1 .

It is defined as $\eta_g = \frac{Q_{ug}}{n Q_{up}}$

6) No. of Piles :- Generally a square pile group is preferred having $3 \times 3 = 9$ piles or $4 \times 4 = 16$ piles or $5 \times 5 = 25$ piles or $6 \times 6 = 36$ piles.

Group Settlement Ratio :-

$$S_r = \frac{\text{settlement of pile group (} S_g \text{)}}{\text{settlement of individual pile (} S_i \text{)}}$$

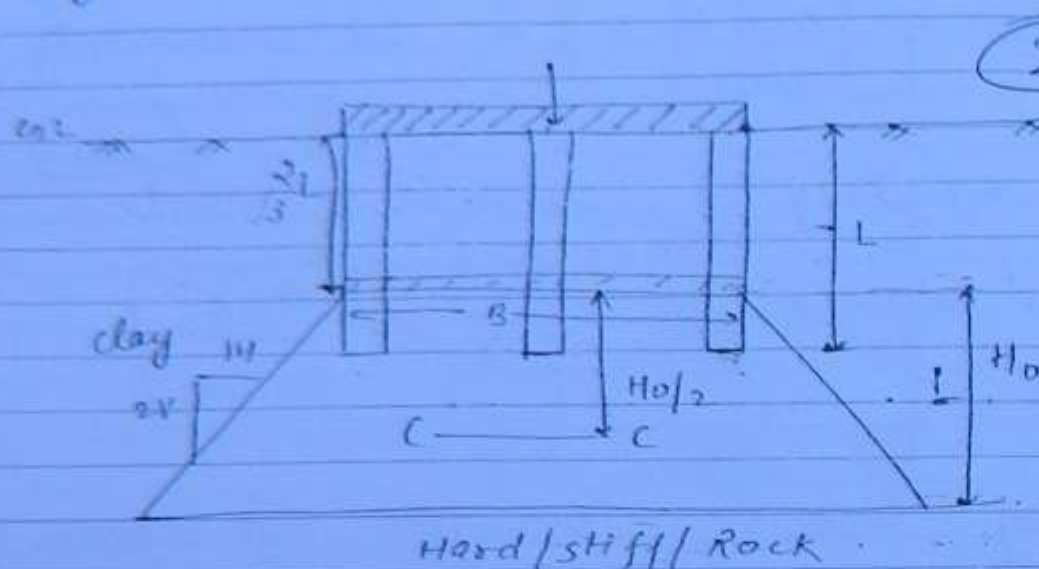
$$S_r = \frac{S_g}{S_i}$$

> 1 may be as high as 16

$S_r > 1$ b/c in group action the stress zone is much larger than the stress zone in individual action.

Determination of settlement of Pile group:-

Q.1 When piles are embedded in a uniform clay deposit and piles act as friction piles.



It is assumed that applied load acts on equivalent shaft of size $(B \times B)$ at a depth $\frac{2}{3}L$ from the CL. The load below the shaft is assumed to be distributed with $2V:1H$ distribution. The compressible soil layer has initial thickness H_0 located below the shaft. Let C-C is the centre of the compressible soil layer at the depth of $H_0/2$ below the shaft.

Let $\bar{\sigma}_0$ = initial eff. overburden pr. at C-C

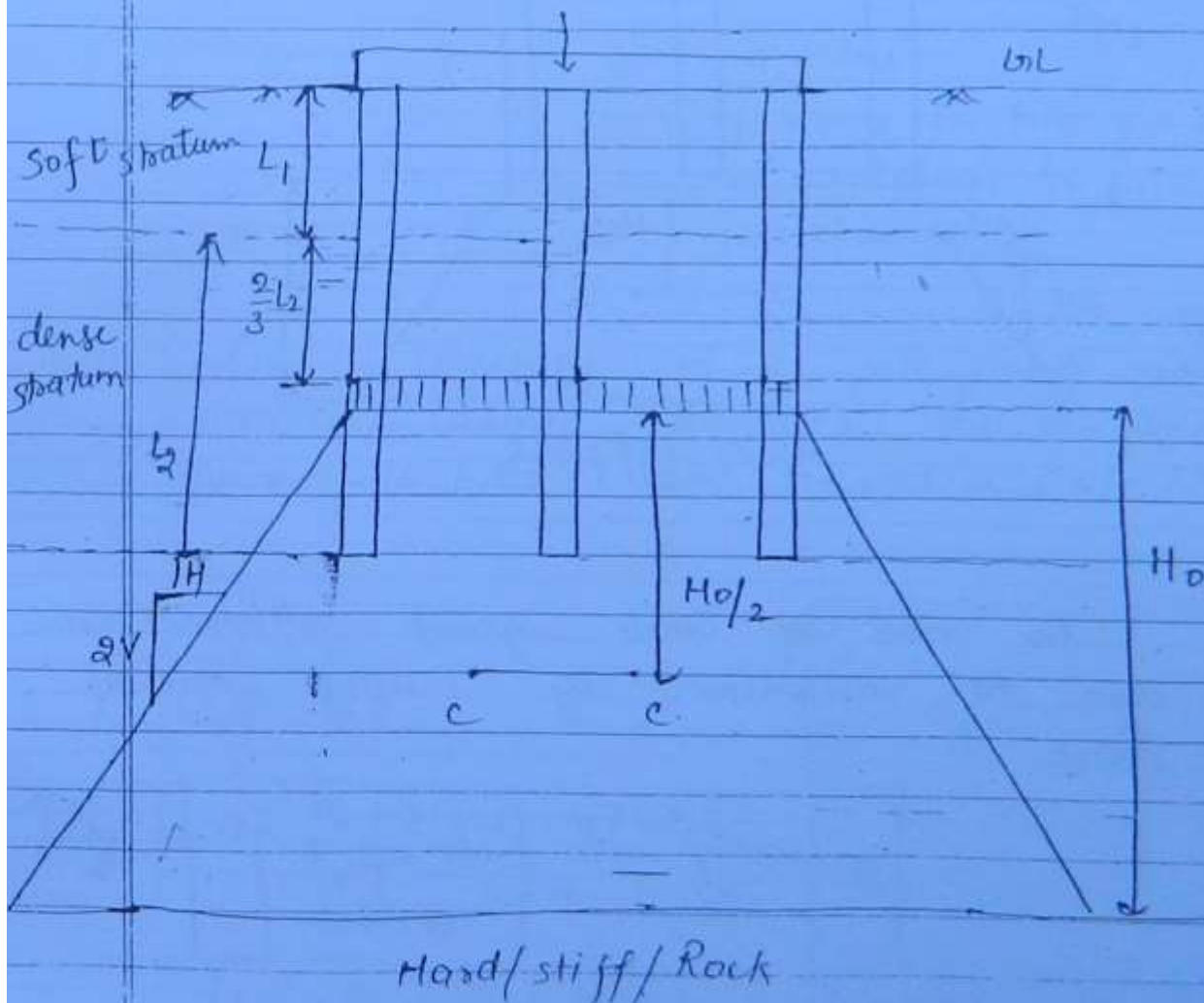
$\Delta \sigma$ = increased in eff. stress at C-C due to load on pile group which is assumed to act at the equivalent shaft.

Hence settlement of pile group = consolidation settlement of clay layer below the shaft.

$$S_g = \frac{c_c H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)$$

Case 2: If piles are driven into a strong/dense stratum of depth L_2 over which a soft stratum lies of depth L_1 , as shown in fig. —

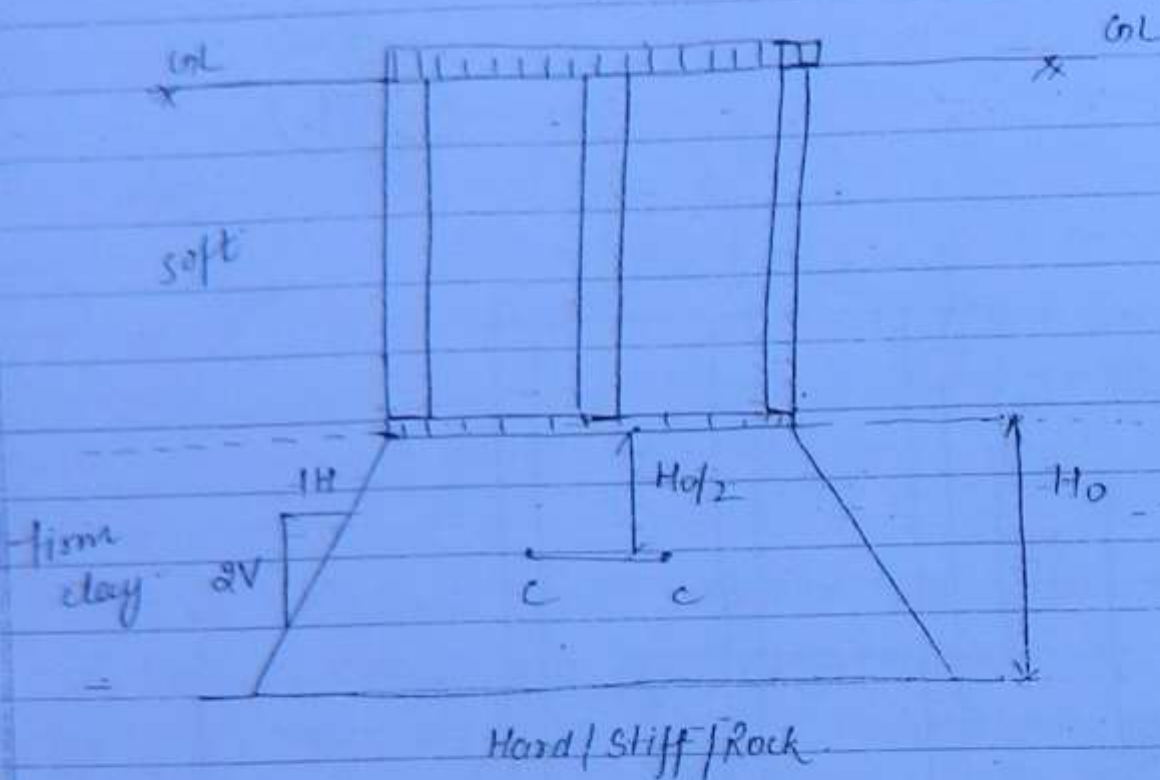
(231)



The equivalent raft is assumed at $\frac{2}{3}L_2$ below the soft stratum and rest of the procedure is similar to previous case.

Q3 If pile group is end bearing resting on a firm clay the equivalent shaft is assumed at the base of pile i.e. on the top of firm clay and the suit procedure is same.

(232)



Q4 In case of Sands group settlement ratio can be determine by using size of pile group.

$$S_n = S_g - [4B + 2.7]^2$$

Converse

Converse - Labbare Formula to determine group efficiency :-

(233)

$$\% \eta_g = \frac{Q_{ug}}{n \cdot Q_{up}} \times 100$$

$$\% \eta_g = \left\{ 1 - \frac{\phi}{90} \left[\frac{m(n-1) + n(m-1)}{m \cdot n} \right] \right\} \times 100$$

where ϕ = arc tan value

m = no. of rows in pile group

n = no. of column in pile group

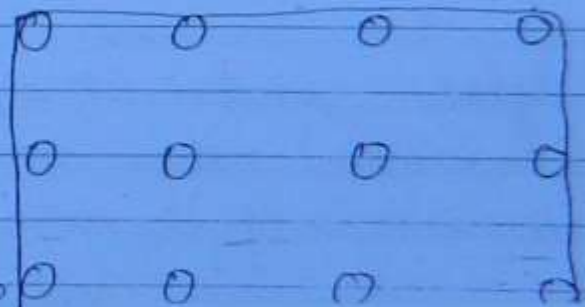
m & n are interchangeable.

Ques. Find efficiency of a pile group using converse Labbare formula for a square group of 16 piles. arc tan value (ϕ) = 18

$$m = 4$$

$$n = 4$$

$$\phi = 18$$



$$\% \eta_g = 1 - \frac{18}{90} \left[\frac{4(4-1) + 4(4-1)}{4 \cdot 4} \right] \times 100$$

Methods of soil stabilization :-

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Soil stabilization is the process of improvement of shearing strength and bearing capacity.

Mechanical stabilization / Mechanical methods :-

The sands & clay behaves opposite to each other. Hence it is possible to obtain a good composition in mixing both.

Compaction can also be used for mech. stabilization and no chemicals are added.

Chemical stabilization - The chemical compounds are added to

improve the str. such as - calcium chloride (CaCl_2) :- CaCl_2 acts as water repulsive agent and it is used to stabilize sandy soils such as base & subbase courses of roads.

Sodium Silicate (NaSiO_3) :- NaSiO_3 with H_2O & CaCl_2 is known as water glass. This mixture is injected for stabilization of deep soil deposits. It forms soluble precipitate of silica gel within the pores of soil and makes the soil impervious & also increases the shear strength.

Soil exploration :-

(235)

Methods of soil exploration :-

1. Auger Boring :- Hand operated augers are used to make the bore hole upto the depth of 6m in soft soils whereas power driven augers may be used for hard strata and for greater depth.
2. Wash Boring :- It consist of driving a pipe casing through chop hammer & water is forced under pressure inside the pipe casing to remove the soil. It can be conveniently used below W.T & for all type of soils except hard rocks & boulders.
3. Percussion Boring :- Percussion drilling is carried out by repeated blows of a heavy weight inside cast iron pipe. The bore hole is usually kept dry & for reducing friction pulverised slurry is added. This method is suitable for gravelly & boulderly strata.
4. Rotary Drilling :- It is useful for those soils which are highly resistance to auger & wash boring. It can be used for sands & clay both.

It is commonly used to stabilized sands.

3. Lime Stabilization :- The hydrated lime (Ca(OH)_2) is used to stabilize highly plastic clays such as black soils. 4-6% volume of soil is the lime requirement.

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Bitumin Stabilization :- Bitumin, tar & emulsion are used to stabilization of granular soils. Such materials act as binder as well as water proofing agent often used of surface course of roads.

Cement Stabilization :- To increase the strength & durability of all type of soils except highly organic clays. Cement grouting can be done. Cement requirement is nearly 5% of volume of soils in sands & 15% of volume of soils in clays.

Georeinforcement :- Geomembrane and geotextiles are polypropylene material which are available in synthetic form in open market.

They are transparent, permeable & colourless material. Their thickness is very small often measured in 'mil'.

Types of Soil Samples :- (237)

1. Disturbed Samples :- In such samples natural soil str. is modified or destroyed. If mineral composition & w.c. remain same then it is called representative sample but if mineral composition & w.c. is also modified then it is called non representative sample.

2. Undisturbed Samples :- If original soil str., mineral content & w.c. do not change while sampling then it is called undisturbed sampling. Such samples are practically difficult to obtain. However small disturbance may be neglected.

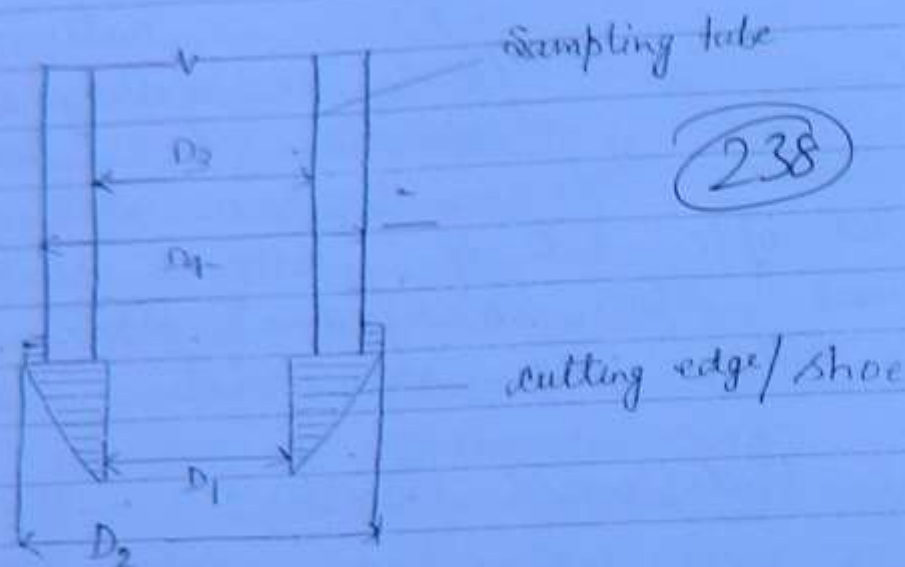
For the purpose of consistency limit, sp. gr., grain size distribution, representative sample of undisturbed samples may be used.

But for coeff. of permeability, consolidation parameters, shear str. parameters undisturbed samples will be used.

The degree of disturbance of the sample depends upon design of

1. cutting edge.
2. inside wall friction
3. Non return wall

Following are the design parameter of a good Sampler -



D_1 = inside dia of cutting edge

D_2 = outside " " " "

D_3 = inside dia of sampling tube

D_4 = outside " " " "

Inside clearance (C_i) :-

$$\% C_i = \frac{D_3 - D_1}{D_1} \times 100$$

It is governed to reduce friction b/w soil sample & sampler when soil enters/ in the tube. It allows for elastic expansion of soil. If C_i is large then there will be more lateral expansion. It's value should be b/w 1% to 3%.

Outside clearance (C_o) :-

$$\left[\% C_o = \frac{D_2 - D_1}{D_1} \times 100 \right]$$

(239)

It will help to reduce friction while sampler is driven & withdrawn.
It's value b/w 1% to 2%.

Area Ratio :-

$$\% A_{sr} = \frac{\frac{1}{4} \pi (D_2^2 - D_1^2)}{\frac{1}{4} \pi D_1^2} \times 100$$

$$\left[\% A_{sr} = \frac{D_2^2 - D_1^2}{D_1^2} \times 100 \right]$$

for sensitive case area ratio should be less than or equal to 10% & no case it should be greater than 20%. As far as possible it should be as low as possible.

Recovery Ratio (L_r) :- It is defined as the ratio of recovered length of sample to the penetration length of sampler.

$$L_r = \frac{\text{recovered length of sample}}{\text{penetration length of sampler}}$$

if $L_n = 1$ (Good Recovery)

if $L_n < 1$ (Sample is shrank)

if $L_n > 1$ (Sample is swelled)

Field methods used for subsurface p
investigation

Penetration Test - SPT
CPT

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Plate load Test

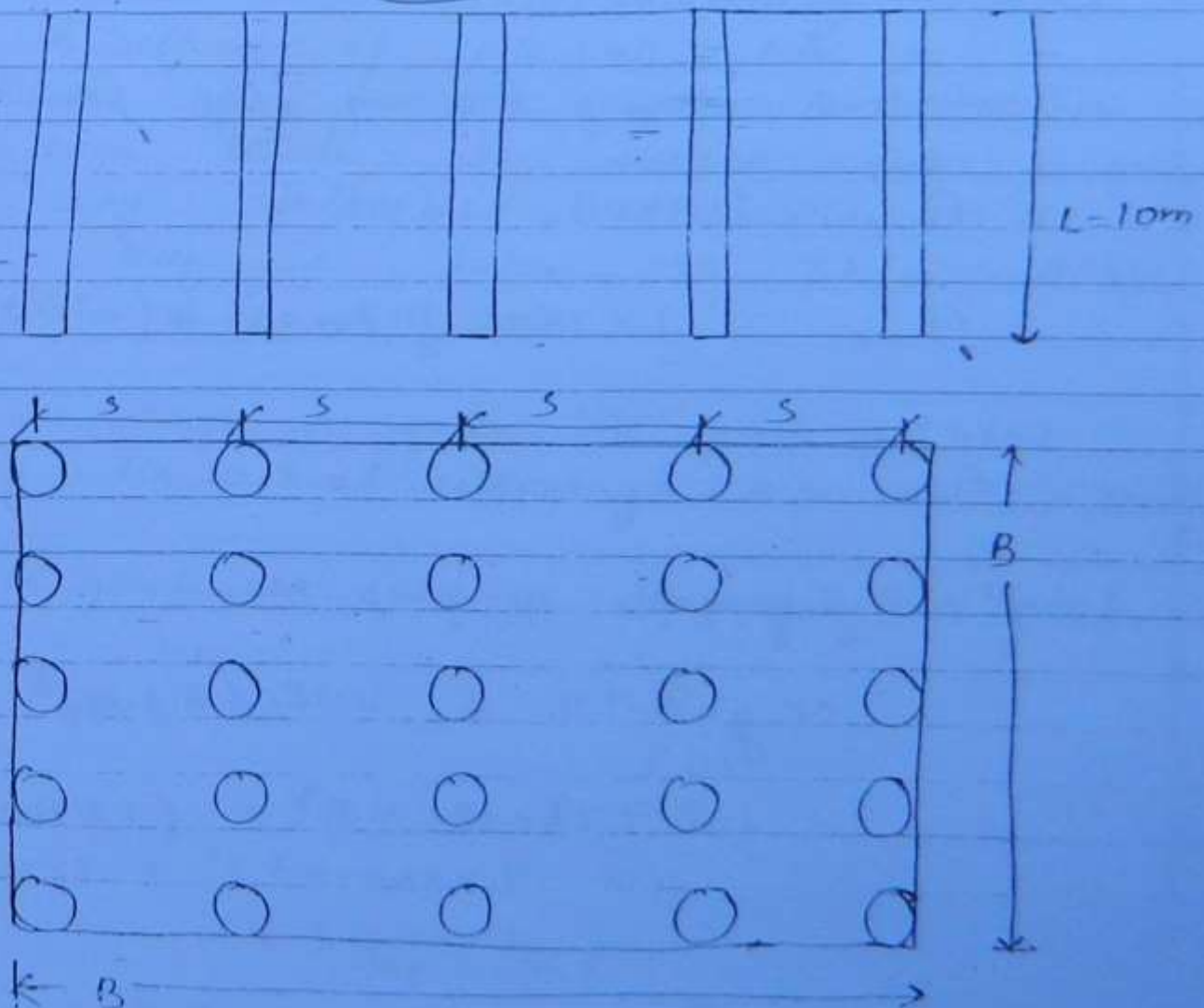
Vane Shear Test

Per meter Test

Geo physical methods - Seismic refraction mtd.
electrical resistivity mtd.

Find group efficiency and safe load which can be applied on the pile group as shown in fig. The dia of pile is 0.3 m and c/c spacing b/w piles is 0.9 m . length of pile is 10 m . The shear strength of clayey soil at the base of pile is 180 kN/m^2 & avg. shear strength over the depth of 10 m is 110 kN/m^2 & adhesion factor is 0.75 . Adopt factor of safety 2.5 there are 25 piles in the group (5×5).

(241)



Size of pile group

$$\begin{aligned} B &= 4.5 + 0 \\ &= 4 \times 0.9 + 0.3 \\ &= 3.9 \text{ m} \end{aligned}$$

shear strength at base = 180 kN/m^2

$$S = c + \bar{\sigma}_n \tan \phi^{10}$$

$$S = c$$

$$c = 180 \text{ kN/m}^2$$

$\bar{c}$ = avg cohesion over depth of pile

$$\bar{c} = \bar{s} = 110 \text{ kN/m}^2$$

ultimate load carrying cap of single pile

$$Q_{up} = q_c A_b + (\alpha \bar{c}) A_s$$

$$Q_{up} = 9 \times 180 \times \frac{\pi}{4} (0.3)^2 + \frac{(0.45 \times 110) \times \pi \times 0.3 \times 10}{0.3 \times 10}$$

$$Q_{up} = 581.03 \text{ kN}$$

Load carrying cap of group pile

$$Q_{ug} = q_c A_b + (\alpha \bar{c}) A_s$$

$$= 9 \times 180 \times B^2 + (1 \times 110) (4B \cdot L)$$

$$= 9 \times 180 \times 3.9^2 + 1 \times 110 \times 4 \times 3.9 \times 10$$

$$= 41800.2 \text{ kN}$$

$$\eta_g = \frac{Q_{ug}}{n Q_{up}} = \frac{41800.2}{25 \times 581.03} = 2.81$$

$$\frac{Q_{ug}}{FOS} \leq \frac{n Q_{up}}{FOS}$$

- Q_{ag} = allowable load on pile group

$$= \frac{n Q_{up}}{FOS} = \frac{25 \times 581.03}{2.5}$$

$$Q_{ag} = 5810.3 \text{ kN}$$

(242)

Design a friction pile group to carry a load of 300t including applied load & self wt of pile cap over a uniform clay deposit upto a depth of 20m which is resting over a rock. The avg. unconfined compressive str. of clay may be taken as 7 t/m^2 . Adopt $FOS = 3$, $\alpha = 0.6$

Let us design a square pile group of (5×5) having dia 0.4m & spacing $3d = 3 \times 0.4 = 1.2 \text{ m}$

The pile group is always designed for $\eta_g \geq 1$

$$Q_{ag} = Q_{safe} = 300 \text{ t} = \frac{n Q_{up}}{FOS}$$

$$300 \text{ t} = \frac{25 \times Q_{up}}{3}$$

$$Q_{up} = 36 \text{ t}$$

For friction pile

$$Q_{up} = Q_b^0 + Q_s$$
$$Q_{up} = q_s \cdot A_s$$
$$= \alpha \bar{c} \cdot \pi D L$$

$$36 t = 0.6 \times 3.5 \times \pi \times 0.4 \times L$$

$$L = 13.64 \text{ m.}$$

(for friction pile
 $L = 10 \text{ to } 20$)

$$\bar{c} = \frac{2 \cdot 4}{2} = \frac{7}{2}$$
$$= 3.5$$

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check for group efficiency.

$$Q_{ug} = Q_b^0 + Q_s$$
$$= q_s \cdot A_s$$
$$= (\alpha \bar{c}) (4 B L)$$
$$= 1 \times 3.5 \times 4 \times 5.2 \times 13.64$$
$$= 992.99 \text{ t}$$

$$B = 4s + D$$
$$= 4 \times 1.2 + 0.4$$
$$= 5.2$$

$$\eta_g = \frac{Q_{ug}}{n Q_{up}} = \frac{992.99}{25 \times 36} = 1.1 > 1$$

$$\eta_g > 1 \quad \text{OK.}$$

Design parameters are

$$\text{no. of piles} = 5 \times 5 = 25$$

$$\text{dia} = 0.4 \text{ m}$$

$$\text{spacing} = 3d = 3 \times 0.4 = 1.2 \text{ m}$$

$$\text{length} = 13.64 \text{ m}$$

$$\eta_g = 1.1$$

In this end bearing effect is negligible b/c
design of friction pile

$$Q_{ag} = \frac{n \times Q_{up}}{P.O.S}$$

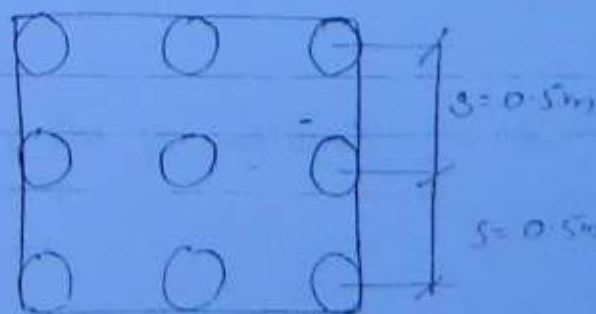
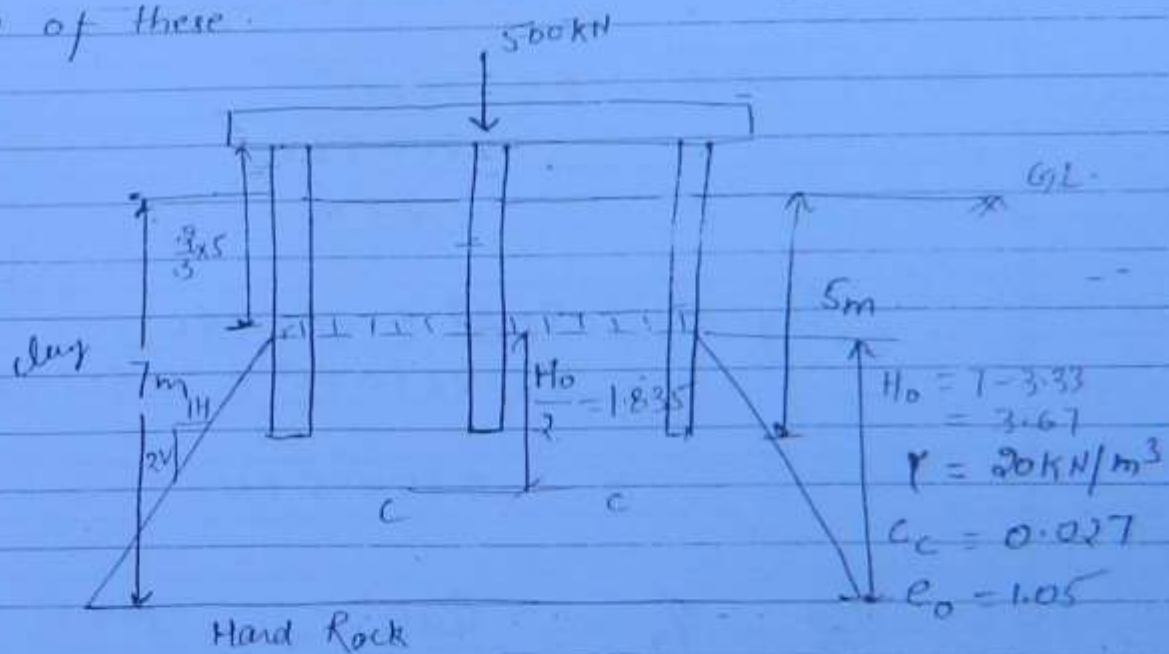
$$= \frac{9 \times 371.16}{2.5}$$

(245)

$$Q_{ag} = 1335.96 \text{ kN}$$

For 3x3 pile group shown in fig find the settlement of pile group in a normally consolidated clay having property shown in fig.

- a) 13.2 mm b) 12.775 mm c) 7.345 mm
d) none of these



Dia of piles $D = 0.2 \text{ m}$

Q. 9 piles of 0.3 m dia & 8 m length are arranged in a square pattern for the foundation of a column of uniform deposits of medium stiff clay ($\bar{q}_u = 100 \text{ kN/m}^2$) c/c spacing = 0.9 m. & $\alpha = 0.9$.
 Calculate the pile load capacity of pile group. FOS = 2.5.

(246)

$$\begin{aligned} Q_{up} &= q_c A_b + \alpha \bar{c} A_s \\ &= 9 \times \left(\frac{100}{2} \right) \times \frac{\pi (0.3)^2}{4} + 0.9 \times \frac{100}{2} \times \frac{\pi \times 0.3 \times 8}{2} \\ &= 371.100 \text{ kN} \end{aligned}$$

$$\begin{aligned} \text{size of group} = B &= 2s + D \\ &= 2 \times 0.9 + 0.3 \\ &= 2.1 \text{ m} \end{aligned}$$

$$\begin{aligned} Q_{ug} &= q_c A^2 + \alpha \bar{c} (4BL) \\ &= 9 \times 50 \times (2.1)^2 + 1 \times 50 (4 \times 2.1) \times 8 \\ &= 5344.5 \text{ kN} \end{aligned}$$

$$\begin{aligned} \frac{Q_{up}}{FOS} \cdot \eta_g &= \frac{Q_{ug}}{n Q_{up}} = \frac{5344.5}{9 \times 371.100} \\ &= 1.6 \end{aligned}$$

$$\text{safe load} = \text{smaller of } \frac{n Q_{up}}{FOS} \text{ \& } \frac{Q_{ug}}{FOS}$$