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-: HAND WRITTEN NOTES:-

OF

CIVIL ENGINEERING

4

-: SUBJECT:-

SOIL MECHANICS

6

2

1st Choice

SOIL MECHANICS

GATE	IES
Obj : 6-7 Ques = 10-11 Marks	Obj - 20-25 Ques
	Conv - 50-60 Marks

3

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 Garg
6. Geotech Engg by Venkat Ramaiyah

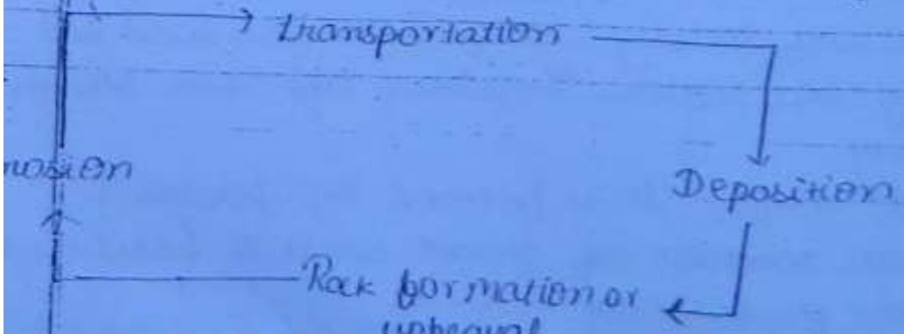
For advance reading

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

Introduction

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

The process of soil formation is called pedogenesis c is a cyclic process called geological cycle



Geological Cycle

The scientific study of soil mechanics was first started by Karl Terzaghi hence he is known as father of soil mechanics. He wrote first book on soil known as Earthbau Mechanic.

Date / /

Classification Of Soil

Almost all the soils are derived from weathering of rock. The weathered rock material is called sediment. 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.

(Fluvial soil)

1. Alluvial Soils (found in Indo-ganga- Bhramputra plains & River plains) These are deposited by river water. These may contain gravel, sands, silts & 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 sediments get deposit in lower part of mountain vallies.

(Lake)

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 salty (Saline). 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 sands & silts & clays. ∴ 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 particle eg - China clay.

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8. Caliche - It is a cemented soil c is rich in Calcium Carbonate consisting gravels sands & clays.

9. Loam - It is mixture of sand, silt & clay e 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.

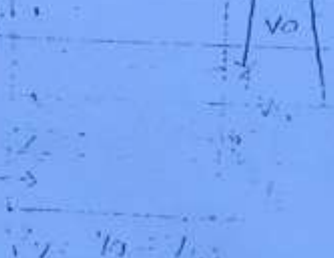
13. Tuff - These are small grain slightly cemented volcanic ash.

14. Bentonite - It is a volcanic ash having high compressibility & plasticity. It is often used in lubrication in drilling.

→ PROPERTIES OF SOILS

Soil
- 3 phase
  - Solids
  - Water
  - Air
- 2 phase
  - Oven dried sample
    - Solids
    - Air
  - Saturated sample
    - Solids
    - Water

Block diagram of 3 phase



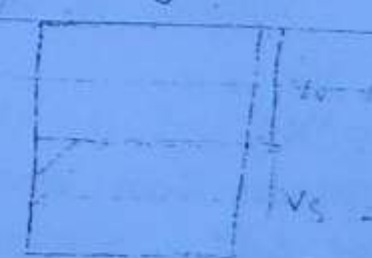
Voids = water + air

Volume = $V = V_v + V_s$

$= V_a + V_w + V_s$

(Moist Soil)

2 Phase
- Dry
- Saturated



(Dry Soil)

$(D_s = 0)$

$(D_s = 1)$

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

Soil moisture
- → Gravity water
- → Capillary water
- → Hygroscopic water

} Removed on oven drying at controlled temp (105°c to 110°c)

- → Chemical 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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Definition

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)

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

$$\% \, S = \frac{V_w}{V_v} \times 100 \qquad 0 \leq S \leq 100\%$$

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

$$\gamma_{sat} = \frac{\text{Saturated wt. of soil}}{\text{Total vol. of soil mass}} = \frac{W_{sat}}{V}$$

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

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→ $\boxed{\gamma_d < \gamma < \gamma_{sat}}$

(d) Submerged unit weight / Effective unit weight

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

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

γ_w = unit wt. of water = 9.81 kN/m³ = 9810 N/m³

ρ = 1 gm/cc

Specific Gravity

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

$$G = \frac{\text{Unit wt of solids}}{\text{Unit wt of water}} = \frac{\gamma_s}{\gamma_w}$$

G = 2.6 to 2.7 for inorganic solids

G = 1.22 - 1.4 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)

$G_m = \frac{\gamma_d}{\gamma_\omega}$. (Mass Specific gravity of dry soil.)

→ $G_m < G$

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

$$G_{27^\circ C} = G_{T^\circ C} \times \frac{\gamma_\omega \text{ at } T^\circ C}{\gamma \text{ at } 27^\circ C}$$

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

$G_{T_1^\circ} \cdot \gamma_{\omega T_1^\circ} = G_{T_2^\circ} \times \gamma_{\omega T_2^\circ}$

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 determine experimentally.

$$\boxed{\% \, I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100}$$

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

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

e = Actual ~~void~~ void ratio in the field condition

→ $0\% \leq I_d \leq 100\%$

→ 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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→ $0 \leq a_c \leq 1$

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-

$$a_c + S = 1$$

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

$$\therefore \boxed{a_c = 1 - S}$$

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

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

ulk density - ρ or ρ_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

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

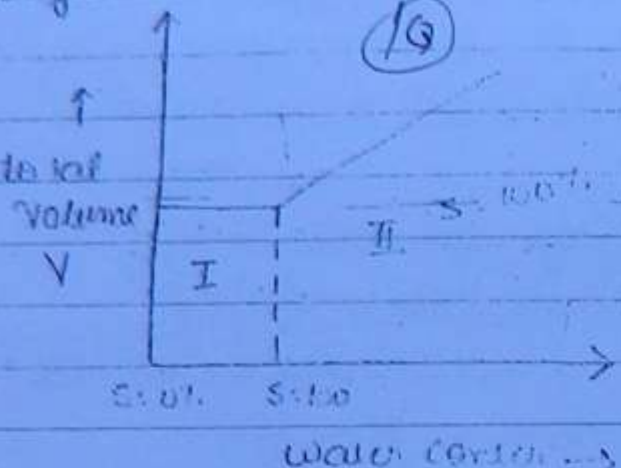
$$\rho_d = \frac{\text{Dry mass of soil}}{\text{Volume of soil}} = \frac{M_d}{V}$$

In partially saturated soil (3 phase) S is less 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

(Q)

Zone I - Partially saturated stage

Zone II - Super saturated stage



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)

$$n = \frac{\text{Volume of voids}}{\text{Total volume of soil mass}} = \frac{V_v}{V}$$

$$V = V_v + V_s$$

→ $0 < n < 1$

→ Significance of both porosity & void ratio is same

NOTE - Though significance of n & e is same but e is more often use than n becoz volume of solids (V_s) is used in e is more stable parameter than total volume V ($\underline{c}$) is used in porosity.

(1st Choice)

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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

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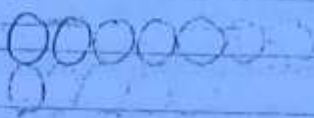

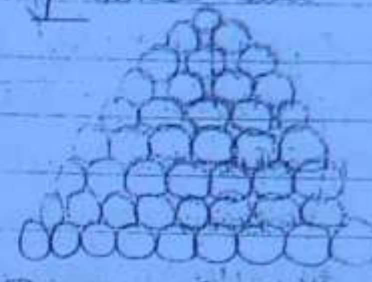

Loosest State
Cubical arrangement
e_{max} = 91%
n_{max} = [illegible].64%

Densest State
Prismoidal arrangement
e_{min} = 35%
n_{min} = 25.92%

Inter Relationship betn physical properties

① $S \cdot e = \omega \cdot G$

② γ = Bulk unit wt $= \dfrac{(G + S \cdot e)\gamma_w}{1+e}$ $\qquad G_m = G_s(1-n)$ $\qquad G_m = \dfrac{G_s}{1+e}$

③ $\gamma_{sat} = \dfrac{(G + e)\gamma_w}{1+e}$

④ $\gamma_d = \dfrac{G \cdot \gamma_w}{1+e} = \dfrac{\gamma}{1+\omega}$

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

⑥ $\gamma_d = \left(\dfrac{1 - n_a}{1 + \omega G}\right) G \gamma_\omega$

⑦ $n = \dfrac{e}{1+e}$

⑧ $e = \dfrac{n}{1-n}$

9 $1+e = \dfrac{1}{1-n}$

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

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Index properties of soil (Classification & ...

These 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, Sensivity, Activity

Determination of water Content :- [OPCSAT]

→ Oven drying Method
→ Pycnometer Method
→ Calcium Carbide / Rapid Moisture Tester Method
} laboratory Method
→ Sand bath Method
→ Alcohal Method
→ Torsion Balance Method / Infra-red Method

1. Oven drying Method - ① For inorganic soils temp is maintained betn 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 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)

% water content $= \frac{W_w}{W_s} \times 100$

$$\boxed{\% w = \frac{W_2 - W_3}{W_3 - W_1} \times 100}$$

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

10 ml volume) (500 gm soil + water)

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 bcoz 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 = Specific gravity of soil solids

$$\% \text{ water content } = \left[\frac{W_2 - W_1}{W_3 - W_4} \left(\frac{G-1}{G} \right) - 1 \right] \times 100$$

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 w.r.t kerosene instead of water

(14)

(3) Calcium Carbide Method - It is very Quick & takes Only 5-7 Mins. It is Crude Method c is used for Rough Computation. A Small wt. (5 gms) is placed in moisture testing equipment (Closed Chamber) & Calcium Carbide (CaC_2) powder is added. Sample is Shaken as a result acetylene gas may produced due to reaction $\bar{c}$ water

$$CaC_2 + 2H_2O \longrightarrow Ca(OH)_2 + C_2H_2 (\uparrow)$$

The pressure exerted by C_2H_2 is measured through a Caliberated Scale c is corelated $\bar{c}$ the water Content

Let w_r = Recorded water Content by Caliberated Scale

w_r is percentage of water / fraction present in Soil w.r.t 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

$$\boxed{w = \frac{w_r}{1 - w_r}}$$

(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 c is heated over a Stove. Since temp is uncontrolled Hence there is danger of Loss of Structural water. Water Content Computation is Similiar to oven drying Method.

(5) Alcohal Method - It is a quick field Method adopted for rough estimation. The methalated Spirit is mixed $\bar{c}$ the Soil sample In order to increase rate of evaporation. Since Alcohal is a oxidising 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 & wt of excavated soil is taken say W. The pit is filled by sand the volume of c is measured through a graduated glass cylinder hence ~~bulk unit wt~~ is

$$\text{Bulk unit wt} = \gamma = \frac{W}{V}$$

(15)

V = Vol of soil = Vol of sand needed to fill the pit.

This method is widely adopted for construction of roads & highway & is suitable gravely & 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 W. This sample is coated with a thin layer of paraffin wax & wax coated soil sample is weighted again say W_2.

W_2 = Wt of soil + wt of wax

$W_2 = W + W_{wax}$

$W_{wax} = W_2 - W$

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.

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

V = Vol of soil

$V = V_2 - V_{wax}$

$$V_{wax} = \frac{W_{wax}}{\gamma_{wax}}$$

Hence the bulk density of soil $\gamma = W/V$

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

(6) Torsion balance Mtd - It is a laboratory method in $\bar{c}$ drying & weighing is done Simultaneously. Infra red radiation is used for drying. This method is suitable for those soils $\bar{c}$ reabsorb moisture quickly from the atmosphere.

Determination of In-situ density (undisturb / field)

1. Core Cutter Method
2. Sand Replacement Method
3. Water Displacement Method.

(16)

(1) Core Cutter Method - Core cutter is a Cylindrical vessel open at top & bottom $\bar{c}$ sharp edges. Let D is the dia of Core-Cutter & h is height of Core Cutter / length. Then volume of Core Cutter is $V = \frac{\pi}{4} d^2 h$.

The empty wt of Core Cutter is known say W_1. The soil surface is prepared level & smooth & Corecutter is punched into the soil & it is taken out fully filled $\bar{c}$ Soil & top & bottom surfaces are levelled. Let W_2 is wt. of Core-Cutter + Soil hence

wt. of Soil = $W = W_2 - W_1$

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

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

(If dry unit wt is required then a small soil sample is required, w is water Content)

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

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 → Size > 300 mm
Cobbles → 80 to 300 mm
Gravels → 4.75 mm to 80 mm
Sand → 0.075 mm to 4.75 mm } Coarse Soil
Silts → 0.002 mm to 0.075 mm
Clays → < 0.002 mm } Fine Soil

(17)

For Coarse Soil - Sieve analysis
For Fine Soil - Sedimentation analysis

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

- (> 0.075 mm) Sieve analysis (Coarse Soil)
  - Coarse Sieving or Gravel Sieving
  - Fine Sieving / Sand Sieving
    - Dry (used when there is no clay content)
    - Wet (when there is clay content in Sand)
- (< 0.075 mm) Sedimentation analysis (Fine Soil) (Based on Principle of Stokes law)
  - Pippette Mtd.
  - Hydrometer Mtd

Coarse Sieving / Gravel Sieving

Sieves are represented either by there size or by no.- Size is dimension of square opening in mm or micron. Sieve no. represents the no. of square opening in 1 inch of length. The sieves have square opening c̄ 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 ith sieve
W = total wt. of soil

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

Find Cumulative % wt. retained

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

(18)

% finer (% N) than nth sieve size = (100 − % Cumulative wt retained in nth sieve)

A graph is plotted on a semilog paper $\bar{c}$ % finer in Y-axis & diameter in X-axis (x-axis is in log scale). From graph ~~the~~ D_{10}, D_{30}, D_{60} are computed directly & are used to ~~find~~ find Coefficient of uniformity (C_u) & Coefficient of Curvature (C_c) & are used in grading & classification of soil.

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

D_{30} is that size below $\bar{c}$ 30% particles are finer than this size

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

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

If C_u tends towards unity then 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 $C_u > 6$

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

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

It means floc / Settlement condition should be laminar / uniform

NOTE If particle size are is greater than 0.2 mm than turbulent motion may be setup & constant terminal velocity may be achieved. If particle size is smaller than 0.0002 mm than brownian motion may be setup & settling of particles may not occurs.

(19)

Procedure

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 pippette 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 - ~~so is~~ After preparing the suspension dispersing agents are added to break the flocs otherwise flocs will not give true settling. The common used dispersing agent is Sodium hexa metaphosphate (calgon)

(1) Pippette 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 olet 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 ($V =$ 500 ml)

Note (1) D_{10} 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}

(26)

* Fine sieving (Sand 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 sands. In fine sieving seven sieves are arranged in the decreasing size 2mm, 1mm, 600 μm, 425 μm, 212 μm, 150 μm, 75 μm (0.075 mm). 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 ~~settling~~ velocity"

$$v = \frac{(\gamma_s - \gamma_\ell) D^2}{18\mu}$$

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 flaky/plate ~~leads~~ lets c is not perfectly spherical)

(2) The medium of liquid suspension should be infinitely large it means there should be no resistance from boundary condition i.e side wall.

(3) The particle size should be less than 0.2 mm & should be greater than 0.0002 mm

Mass per unit volume of dispersing agent present in dried sample $= \frac{m'}{V} = \frac{m'}{500\,ml}$

(21)

Let M is total dry mass of soil sample used to make the suspension having total volume ($V = 500\,ml$)

Initial dry mass per unit vol. is $= \frac{M}{V}$

$$\%\ \text{finer at any instant } (\%N) = \frac{\frac{m_d}{v_p} - \frac{m'}{V}}{\frac{M}{V}} \times 100$$

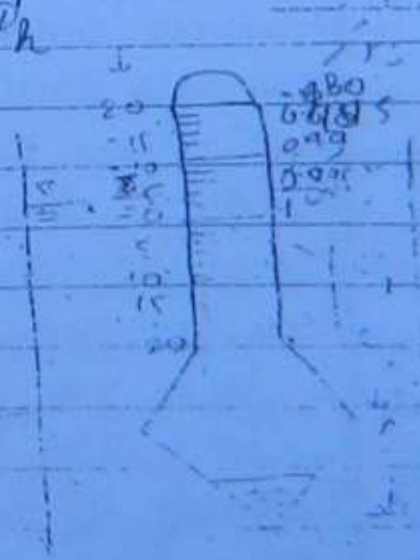
The above % finer are obtained at regular time interval (t)

let the specified depth from ċ observation is taken is He = 10cms then

$$v = He/t = \frac{(\gamma_s - \gamma_L) D^2}{18\mu}$$

From above realation D is found

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



Hydrometer Method

G_s = Specific gravity of suspension

$G_L = G_s = 1 + \frac{R_h}{1000}$ (for liquid)

$\gamma_L = G_L \cdot \gamma_w$

unit wt. of liquid suspension — specific gravity of liquid suspension

Let A_J = Area of Jar

V_h = vol. of hydrometer bulb

He = Effective height

$$He = H_1 + \frac{1}{2}\left[h - \frac{V_h}{A_J}\right]$$

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

$$v = He/t$$

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

$$\% N = \frac{G}{G-1} \times \frac{V}{M} \times \frac{\gamma_w}{d} \times \frac{R_h}{10}$$ (22)

Where G = Specific gravity of settling soils

V = total volume of soil suspension (500ml)

M = total wt. 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 ↑ 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°C

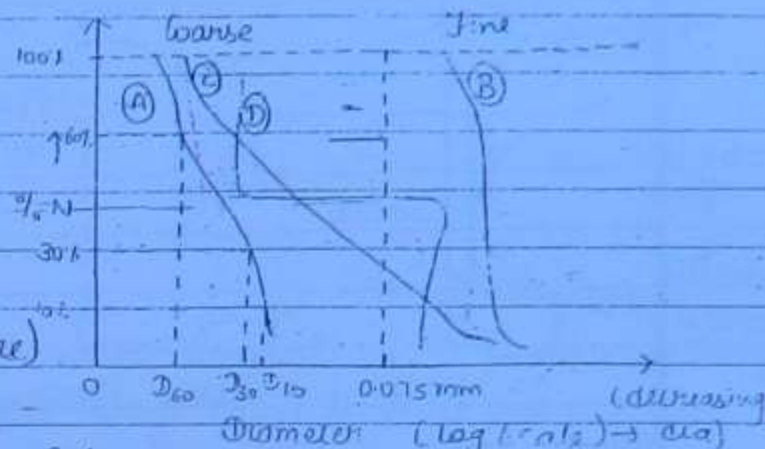
C_t = -ve - test temp < 27°C

(3) Dispersing agent correction factor - Due to addition of dispersing agent specific gravity of suspension is more. Hence observed R_h is greater than the actual C_d is always negative

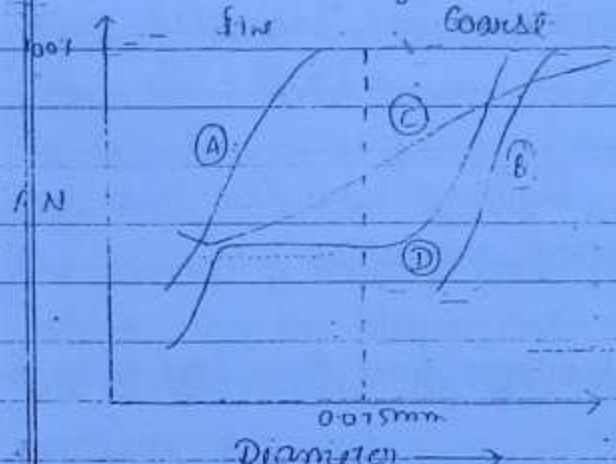
$C = C_m \pm C_t - C_d$

Graph b/w 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)



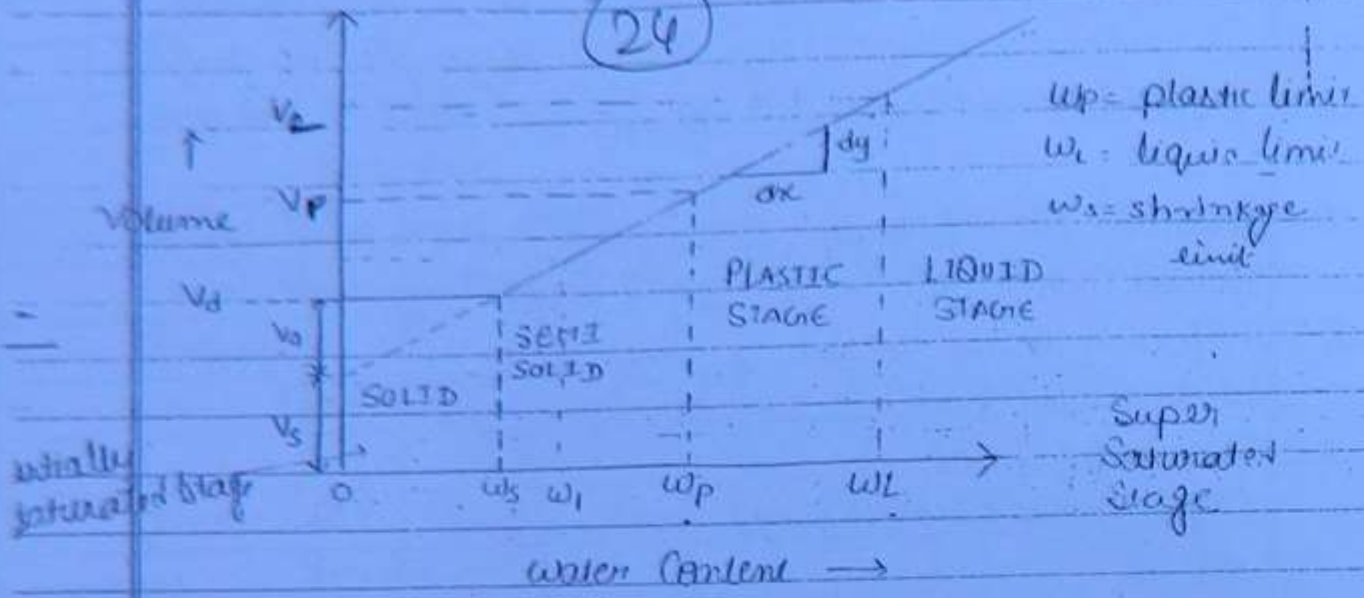
In graph D_{60}, D_{30} & D_{10} Can be Calculate
& Can be used to find C_c & C_u



→ Consistency limits

Consistency represents the realative 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 realated 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



When $w = 0$

then $V = V_d = V_a + V_s$ = Vol. of air + Vol. of Solids

w_s = Shrinkage limit, at So c Soil is just Saturated

At $w = w_s \Rightarrow S_s = 100\%$. $(S=1)$

Note that if water Content is below Shrinkage limit then there will not be Change in vol. with moisture variation.

If water Content is ≥ than Shrinkage limit then soil is in Super Saturated Stage.

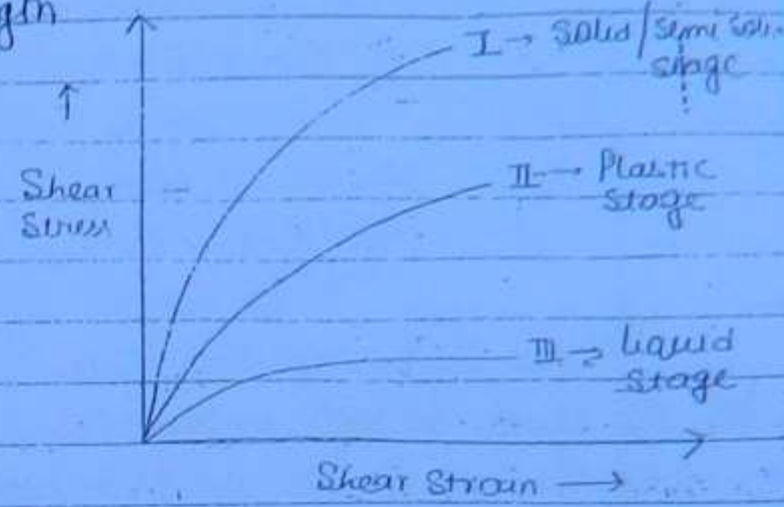
In this Stage vol. of Soil Change $\bar{c}$ Change in water Content. It means in partially Saturated Stage / Solid Stage void ratio is Constant ~~whereas~~ Whereas in Semisolid, plastic & liquid Stage void ratio will Change $\bar{c}$ Change in water Content.

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

The boundary b/w 2 Consistency Stages is defined as Consistency limits. At liquid limit most of the Soil Show negligible Shear Strength. & Shear Strength decreases with the ↑ in water Content

NOTE In Solid & Semisolid Stage Soils show diff soil Show 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 Cassagrande Apparatus. The height Of free fall Of Cup is 1 cm Only hard rubber base.

The Soil Sample is filled in the Cup & a groom is cut on the Soil Sample either using ASTM tool or Cassagrande 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 .... To find N_2, N_3 ...... A graph is plotted b/w 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 c is found from graph. The dimension Of tool used to Cut the groove are.

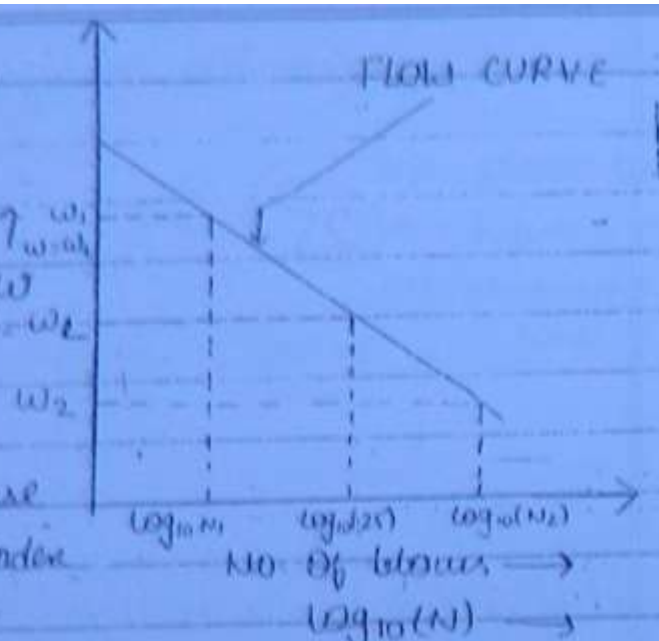
ASTM tool
Base - 2mm wide
top - 13.6 mm wide
Height 10 mm
It is preffered for less plastic soil such as Silts

Cassagrande tool
Base - 2mm wide
top - 11 mm wide
Height 8mm

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

26

$$I_f = \frac{\omega_1 - \omega_2}{\log_{10}\left(\frac{N_2}{N_1}\right)}$$



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 appears on the surface. Generally fine soils have greater liquid limit & plastic limit than coarse soil.

NOTE – ① 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%
Black	400-500%	200-250%

③ Hence High plastic soils have high liq. limit & greater liq. limit indicates more compressibility

④ If sand is mixed in Clay then L.L & P.L of Clay both are reduced but reduction in plastic limit is less than reduction in liquid limit hence plasticity of Clay reduced.

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

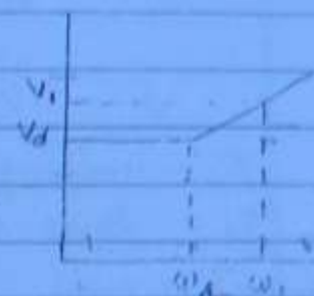
OR It is that max water content at which further decrease in water content do not result in decrease in volume. It means below shrinkage limit soil is partially saturated & at shrinkage limit soil is just saturated.

Determination of shrinkage limit

(1) At shrinkage limit $S = 1$, $\omega = \omega_s$

$$S \cdot e = \omega \cdot G$$

$$\text{i.e. } e = \omega_s \cdot G \Rightarrow \boxed{\omega_s = \frac{e}{G}}$$



(2) From graph $\omega_s = \omega_1 - \left[\frac{V_1 - V_d}{\omega_d}\right]\gamma_w$

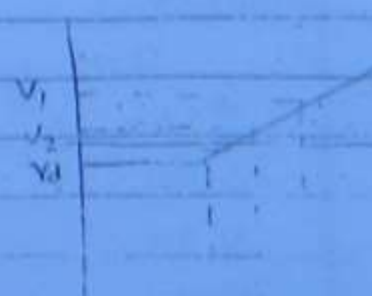
Where V_1 = vol of soil at w.c ω_1

V_d = vol of dry soil = vol of soil at shrinkage limit

ω_d = dry wt of soil mass

γ_w = unit wt of water

(3) $$G = \frac{1}{\frac{\gamma_w}{\gamma_d} - \frac{\omega_s}{100}} = \frac{1}{\frac{1}{R} - \frac{\omega_s}{100}}$$



R - Shrinkage ratio

G - Specific gravity of solid

ω_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

$$R = \frac{\frac{V_1 - V_2}{V_d} \times 100}{\omega_1 - \omega_2}$$

If $\omega_2 = \omega_s$ then $V_2 = V_d$

$$R = \frac{\frac{V_1 - V_d}{V_d} \times 100}{\omega_1 - \omega_s}$$

Shrinkage Ratio $= R = \frac{\gamma_d}{\gamma_\omega}$ 28

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

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

$$V_s = R \times (\omega_i - \omega_s)$$

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.OS	Suitability
$\nless 5\%$	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

$$I_P = \omega_L - \omega_P$$ (29)

For gravel & sand (coarse) $I_P \to 0$

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

$$I_S = \omega_P - \omega_S = \text{Plastic limit} - \text{Shrinkage limit}$$

Consistency Index / Relative Consistency (I_C)

$$I_C = \frac{\omega_L - \omega}{\omega_L - \omega_P} = \frac{\omega_L - \omega}{I_P}$$

If $I_C < 0$ then $\omega >$ liquid limit hence soil is in liquid stage.

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 –

$$I_L = \frac{\omega - \omega_P}{\omega_L - \omega_P} = \frac{\omega - \omega_P}{I_P}$$

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 $I_L \le 0$ Soil is in solid or semi solid stage

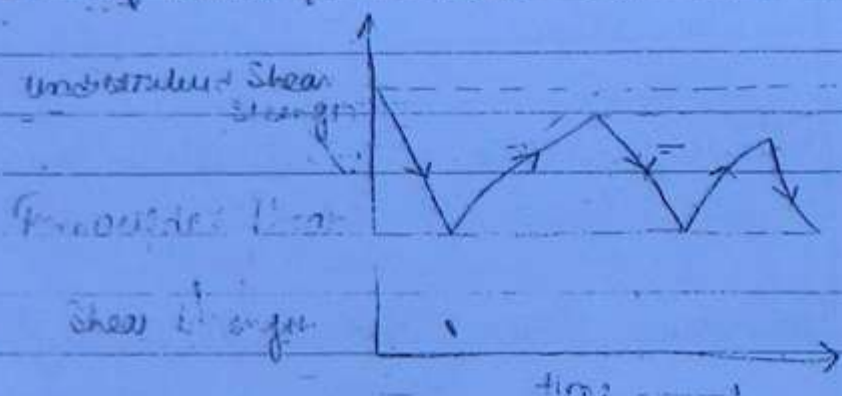
Toughness Index It is the ratio of plasticity Index to the flow Index. It gives the idea about Shear Strength of soil at plastic stage. If $I_t < 1$ then soil can be easily crushed & is friable

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

(30)

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

[illegible] Thixotropy – It is that property of soil due to which 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 which 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 & $\overset{\text{coarse}}{\text{Sand}}$)

If $S_t = 1$ then there is no loss in the Strength of Soil on remoulding

If S_t - 1 to 4 then soil is less sensitive or normal sensitive
If S_t = 4 to 8 then soil is sensitive & may have flocculant or honey comb structure.
If S_t = 8 to 16 then soil is extra sensitive such as fine clays.
If S_t > 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 with 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	Type Of Soil
> .75	Inactive / Non active
0.75 < A_c < 1.25	Normal active
A_c ≥ 1.25	Active
Activity	Type Of Mineral
0.38	Kaolinite
0.90	Illite
7.2	Montmorillonite

- Black Soil contain Montmorillonite mineral & are highly active

Collasibility - Those soils which undergo large decrease in volume due to rise in moisture content without rise in external load are called collasible soil. eg - loess

It is represented by C_p = Collasible 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
In above expression soil mass is considered semiinfinite i.e area is large & constant height is small & will change

% Cp	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

Miscellanous

(1) Diffn bem organic & Inorganic soils
If L.L of oven dried sample is less than 70% of liquid limit of air dried sample then soil is organic

(2) Field method to differentiate silts & clays
1. dialatancy 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-to.10 mins without much turbidity then soil is silt & if turbid suspension is formed then soil is clay
3. Toughness test The balls or threads of 3 mm size are rolled & dried in the air. After drying if the balls are pressable bern the fingers then it is silt otherwise clay.(hard)
4. Dry & wet test.

1st Choice

specific wt. or density (wt.) = $\rho \times g$

$\omega = \rho g$

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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.

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Sol:- wt. of soil M = 683 gm

wt. of soil & wax = 690.6 gm

wt. of wax = $690.6 - 683.0 = 7.6$ gm $= M_\omega$

Density of wax $\rho_{wax} = G_{wax} \times \rho_\omega$

$= 0.89 \times 1$ gm/cc

Vol. of wax $= \dfrac{\text{Mass of wax}}{\rho_{wax}} = \dfrac{7.6}{.89 \times 1} = 8.539$ cc

Vol. of water displaced = vol of soil + vol. of wax

$350 \text{ ml} = V + 8.539$

$V = 350 - 8.539 = 341.46$ cc

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

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

$$\rho_d = \frac{G \cdot \rho_\omega}{1+e} \qquad 1.71 = \frac{2.73 \times 1}{1+e}$$

$\boxed{e = 0.596}$

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

$$S = \frac{.17 \times 2.73}{0.596} = 0.778 = 77.8\%$$

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 core cutter filled with soil is 2970 gm. If $w = 6.2$ 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.c 40% is 1.88. On oven drying mass specific gravity drops to 1.74. Compute for SG of solids & S.L of soil.

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

$w = 0.40$ & $S = 1.0$

$S \cdot e = wG$

$1 \cdot e = 0.4G$

$e = 0.4G$

34

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

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

$1.4G = 1.88 + .752G$

$.648G = 1.88$

$G = 2.9$

~~Shrinkage condition~~

At dry condn $\quad G_m = \frac{\gamma_d}{\gamma_w} = 1.74$

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

$\Rightarrow \frac{100}{174 - w_s} = 2.9 \Rightarrow$ ~~$504.6 - 2.9w_s = 100$~~

~~$404.6 = 2.9w_s$~~

~~$w_s = 23\%$~~

$w_s = 23\%$

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

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

Height of core cutter $H = 12.6$ cm

diameter " " " $D = 10.2$ cm

(35)

Mass of empty core cutter $W_1 = 1071$ gms

Mass of core cutter + soil $W_2 = 2970$ gms

Mass of soil $= 2970 - 1071 = 1899$ gms

Vol. of soil = vol. of core cutter $= \frac{\pi}{4} D^2 H$ cc

$= 1029.58$ cc

Bulk density $\rho = M/V = 1899/1029.58 = 1.84$ gm/cc

$$\rho_d = \frac{\rho}{1+w} \Rightarrow \frac{1.84}{1+.06} = 1.735 \text{ gm/cc}$$

$$\rho_d = \frac{G\rho_w}{1+e} \Rightarrow 1+e = \frac{2.69 \times 1}{1.735}$$

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

At $S = 1$ $\quad S \cdot e = wG$

$1 \times 0.55 = w \times 2.69$

$w = 0.2044 = 20.44\%$

2011 ESE 6 Marks

An oven dried soil sample of vol. 225 cc weighs 390 gms. If $G = 2.72$ then determine void ratio & shrinkage limit. What will be the water content which will fully saturate the sample & cause a increase in vol. equal to 6% of original dry vol.

Sol. I Volume $V_d = 225\ cc$

Mass dry $= M_d = 390\ gm$

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

$$\rho_d = \frac{G\rho_w}{1+e} \Rightarrow \frac{2.72\times 1}{1.733} = 1+e$$

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void ratio $e = 0.569$

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

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

$$w_s = 20.938\ \%$$

II Initial vol. $V_d = 225\ cc$

Saturated vol. $= 1.08 \times 225 = 243\ cc$

~~$V = \frac{\text{vol. of void}}{\text{vol. of solid}} = 243$~~

$V =$ vol. of void + vol. of solids $=$ ~~243~~

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

$V_v = V - V_s$

$V_v = V_w = V - V_s$.

After saturation

$$\omega_2 = \frac{W_v}{W_s} = \frac{W_{w2}}{W_s}$$

Mass of dry Soil $=$ Mass of Solids $= 390\ gms$

~~ω_2~~ $=$ vol. of solid $= \frac{\text{Mass of Solids}}{G\cdot\rho_w}$

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

$V_v =$ ~~243-143.38~~ $V - V_s = 243 - 143.38$

$= 99.62\ cc$

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

(37)

$S \cdot e = wG$

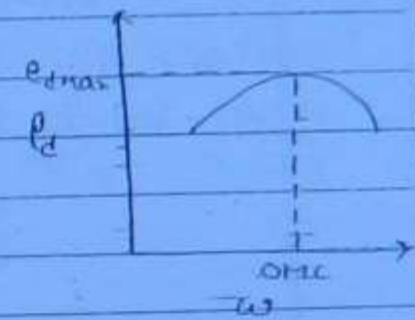
$1 \times 0.69 = w \times 2.72 \Rightarrow w = w_2 = 25.5\%$

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

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

$w = 16\% = 0.16$

$G = 2.65$



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

Void ratio $e = 0.47$

$S \cdot e = w \cdot G$

$S = \frac{.16 \times 2.65}{0.47} = 0.90 = 90\%$

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

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

$e = 0.424$

theoritical $\rho_{d\,max} = \frac{G\rho_w}{1+e} = \frac{2.65 \times 1}{1+0.424} \approx 1.86$ gm/cc

Ques A Soil is required to be excavated from a borrow pit for construction of an embankment of ht. 6m, top width 2m side slope 1:2. The unit wt. of soil in bulk (γ) = 18 kN/m³ & its natural water content is 8%. The dry density is required in the embankment is 20 kN/m³ with a $w = 10\%$. The $G = 2.7$ Estimate the quantity of soil required to be excavated from

from the borrow pit to construct 1 m length of embankment. If each truck has capacity to carry 80kN/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.

Sol-

Pit	Embankment
V_1 = total vol. of soil excavated	V_2 = total vol. of embankment soil
e_1 = void ratio of pit soil	e_2 = void ratio of embankment soil
$\rho_{d_1} = \frac{\rho}{1+\omega_1}$	$\rho_{d_2} =$
$\omega_1 = 8\%$	$\omega_2 = 10\%$
V_{s_1} = Vol. of Solids	$V_{s_2} = V_{s_1}$

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

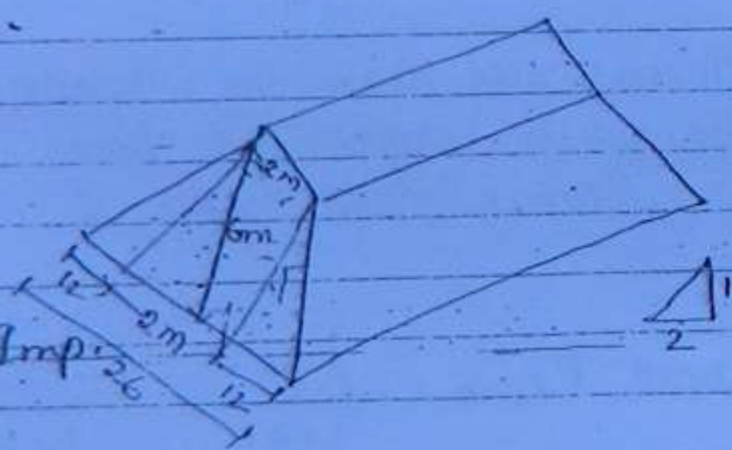
$$\Rightarrow \frac{V}{V_s} = 1+e$$

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

$$V_{s_1} = V_{s_2}$$

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

Imp.



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

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

$$\rho_{d_1} = \frac{\rho}{1+\omega_1} \Rightarrow \gamma_{d_1} = \frac{\gamma}{1+\omega_1} = \frac{18}{1+.08} = 16.67$$

$$\gamma_{d_1} = \frac{G\gamma_w}{1+e_1} \Rightarrow 1+e_1 = \frac{2.7 \times 9.81}{16.67} \Rightarrow e_1 = 0.589$$

$\gamma_{d_2} = \frac{G\gamma_w}{1+e_2} \Rightarrow e_2 = \frac{2.7\times 9.81}{20} - 1 = 0.324$

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

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$V_1 = 100.81\ m^3$

Wt. of excavation = vol × γ_{bulk}

$W = 100.81 \times 18 = 1814.58\ KN$

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

$= 1814.58/80$

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

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

$\boxed{n_2 = 0.24}$

$S.e_2 = w_2.G \Rightarrow S_2 = \frac{.10\times 2.7}{.324}$

$\boxed{S_2 = 83\%}$

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 $w = 15\%$ & % air void is 20% then Calculate the wt. of soil & water req. in the preparation of above Specimen If $G = 2.69$.

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

$w = 15\% = .15$

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

$\boxed{V = V_a + V_w + V_s}$ — (1)

$V_a = 196.35 \times .20 = 39.27$ cc

$w = \frac{W_w}{W_s} = \frac{V_w.\gamma_w}{V_s.\gamma_s} = \frac{V_w}{V_s}.G = .15$

$.15 = \frac{V_w}{V_s} \times 2.69$

$V_w = .15 \times 2.69\ V_s \Rightarrow V_w = 0.4035\ V_s$

From 1 $\quad 39.27 + 0.4035\, V_s + V_s = 196.35$

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

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

wt. of water $= V_w \times \gamma_w$

$$= 45.16 \times \cancel{9810}\ 10^{-6} \times \cancel{9.81}\ 9810 \text{ N}$$

$$= 0.443 \text{ N}$$

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wt. of solid $= \gamma_s \cdot V_s$

$$= G\gamma_w \times V_s$$

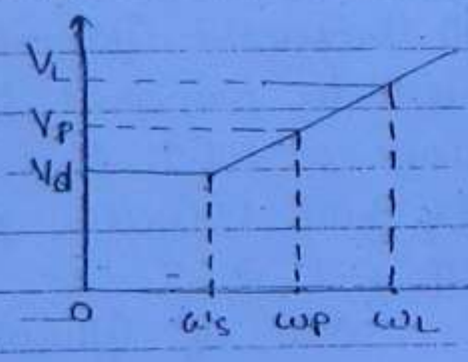
$$= 2.69 \times 9810 \times 111.92 \times 10^{-6} \text{ N}$$

$$= 2.95 \text{ N}$$

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.

Soln:- $I_p = w_L - w_p$

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



① $V_P - V_d = 0.25 V_P$

$0.75 V_P = V_d$

$V_P = V_d / 0.75$

② $V_L - V_d = .34 V_L$

$0.66 V_L = V_d$

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

Vol. at Shrinkage limit = dry volume $= V_d$

$\dfrac{dy}{dx}$ = constant

$$\frac{V_L - V_d}{w_L - w_s} = \frac{V_P - V_d}{w_P - w_s} = \frac{\frac{V_d}{0.66} - V_d}{33 - w_s} = \frac{V_d/0.75 - V_d}{25 - w_s}$$

Date / /

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

$\Rightarrow w_s = 10.36\,\%$ ans.

$$S.R = \frac{\frac{V_1 - V_d}{V_d}}{w_1 - w_s}$$

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$V_1 = V_d$, $w_1 = w_l$

$$S.R = \frac{\frac{\frac{V_d}{0.66} - V_d}{V_d}}{33 - 10.36} \times 100 = 2.27 \text{ ans}$$

MB 16 Ex NO-57

Find D_{10} & C_u

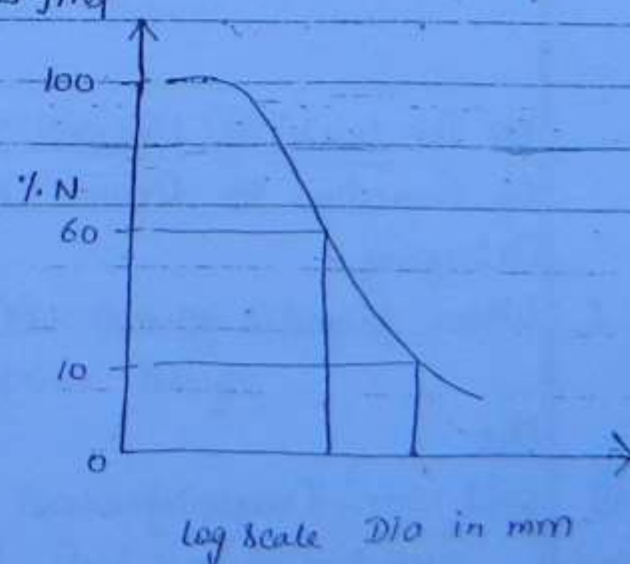
Sol:-

S.NO	Mesh opening (mm)	wt. Retained gms	% wt Retained	Cum % Ret	100-%Cum % 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

$\Sigma W = 500$ gms

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

Eff. size $= D_{10}$



1st Choice

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SOIL CLASSIFICATION SYSTEM

Unifide Soil Classification
→ Given by A. Cassagrande
→ Adopted by I.S
→ Based on particle size (distribution) for coarse soil & plasticity Characteristics of fine soils

Unifide Soil Classification

Coarse grain classification
> 50% by weight is greater than 0.075 mm
(IS sieve no 200)

Fine grain Classification
≥ 50% by wt. is smaller than 0.075 mm

A Coarse grain Classification

Boulders (> 300mm)

Cobbles 80mm - 300mm

Gravels 4.75 - 80mm
- Coarse 20-80mm
- Fine 4.75-20mm

Sand (0.075 - 4.75mm)
- Coarse (2mm to 4.75mm)
- Medium 0.425 - 2mm
- Fine 0.075 mm - .425 mm

On the basis of fineness, Coefficient of uniformity, Coefficient of Curvature Coarse Soils are Classified in following Category.

1 When fineness < 5%. (fineness means % of fine soil silt & clay present in Coarse Soil)

① GW - (well graded gravel)
Gravel, well graded

$C_u > 4$

$1 \le C_c \le 3$

fineness < 5%

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

③ SW – Well graded Sand
$C_u > 6$ (43)
$1 \le C_c \le 3$
fineness < 5%.

④ SP – Poorly graded sand
C_u & C_c are not in above range

2. When fineness is betn 5% to 12%.
under this condition dual symbol is used $\boxed{C \rightarrow \text{Clay}}$
A When Clay Content is greater than Silt content
① GW – GC (well graded gravel containing clay)
$C_u > 4$
$1 \le C_c \le 3$
$5\% \le \text{fineness} \le 12\%$

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)
$C_u > 6$
$1 \le C_c \le 3$
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 $\boxed{M - \text{Silt}}$
① GW–GM (well graded gravel containing silt)
$C_u > 4$; $1 \le C_c \le 3$
$5\% \le \text{fineness} \le 12\%$.

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

③ SW-SM
$C_u > 6$; $1 < C_c < 3$

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 (% wt of Soil < 0.075 mm is more than 50%.)

Silt (M)
- oarse: -02 mm - 0.075 mm
- Medium: 0.01 mm - 0.02 mm
- fine: 0.002 mm - 0.01 mm

Clay (< 0.002 mm) < 2 μm
- Inorganic (C)
- Organic (O)

If silt contains organic compound (MO) → organic Silt
CO – organic Clay
C – Inorganic Clay

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

A. Soils having low plasticity (L.L < 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 ≥ 50%)

(i) CH – High Plastic Inorganic Clay
(ii) MH " " " Silt.
(iii) OH

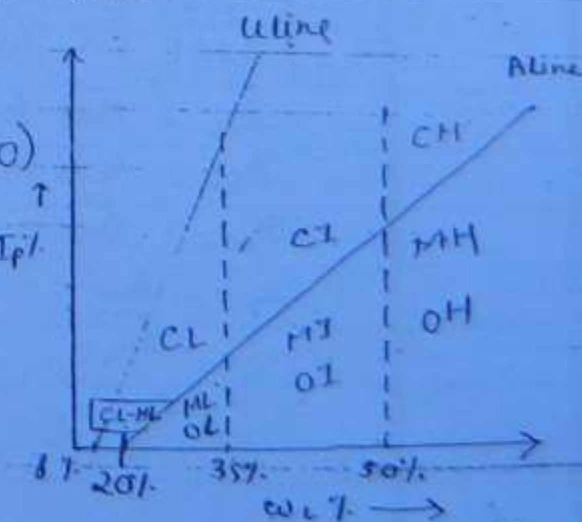
MI, OI, MH, OH located below A-line

A-line –

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

ω_L = % liquid limit

I_p = % PI



NOTE? Eq. of U-line $I_p = 0.9 (w_L - 8)$

NOTE (1) A line is a boundary c represents difference b/w 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 clays. 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 beyound which no result should lie. If any result is found above U-line then experiments should be repeated.

2005

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

(a) Silty sandy gravel (b) Silty gravely sand $\bar{c}$ $C_u = 10$ $\bar{c}$ $C_u < 60$

(c) Gravely silty sand $\bar{c}$ $C_u > 60$ (d) Gravely silty sand for $\bar{c}$ C_u cannot be determined

Sol-

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

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

$D_{60} = 4.75$ mm

$D_{10} = 0.075$ mm

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

$C_u > 60$

Ans- (c)

(1st Choice)

2006

Q. Laboratory sieve analysis was carried out on a soil sample using complete set of the I.S sieves. Out of 500 gm of soil used in the test 200 gm was retained on 600 μm sieve & 250 gm was retained on 500 μm sieve remaining 50 gm was retained on 425 μm sieve. (47)

(1) The C_u of soil is

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

(2) The classification of soil is

(a) SP (b) SM (c) GP (d) GM

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

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

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

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

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

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

2002

In a soil specimen 70% of particles are passing through 4.75 mm IS sieve 40% are passing through 75 μm IS-sieve. If its C_u is 8 & $C_c = 2$ then the soil classified as

(a) SP (b) GP (c) SM (d) GM

sand

S.No	Sieve	% wt retained
1	4.75	30%
2	75 μm	60

Ans (c)

1st Choice

Page No. / Date

Clay Minerals & Structure of Soils

Most of the Soil are made of primarily 3 Material

Kaolinite | Montmorillonite | Illite / polygorskite
Made up of H_2 bond b/w molecules | Water bond b/w molecules | Ionic bond b/w molecules.
H_2 bond is strongest hence these soil shows min change in volume on water adds | H_2O bond is weaker. Show max. vol. change. Highly compressible. eg- black soil. | Ionic bond is medium strong.

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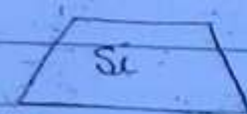
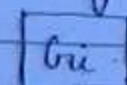
Almost all the minerals are made of 2 fundamental units
1. Sillica Sheet
2. Alumina Sheet / Gibbsite Sheet

Sillica Sheet is a tetrahydral unit in c 4 oxygen or hydroxyl atom enclose a silicon atom & form a tetra hydron unit. Its symbolic representation is



Sillica Sheet

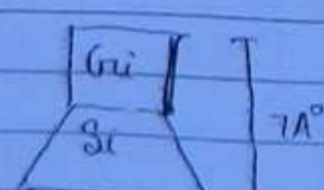
Gibbsite Sheet is an Octahydron unit in c aluminium / iron / magnesium atom is enclosed by 6 hydroxyl ions & its symbolic representation is



Gibbsite

1. Kaolinite Mineral

One molecule of kaoline 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.

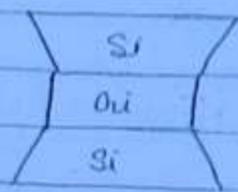


$1 A^{\circ} = 10^{-10} m$

(1st Choice)

Montmorillonite Minearals

One molecule of montmorillonite minearals is made of 2 Si sheets & 1 gibbsite sheet



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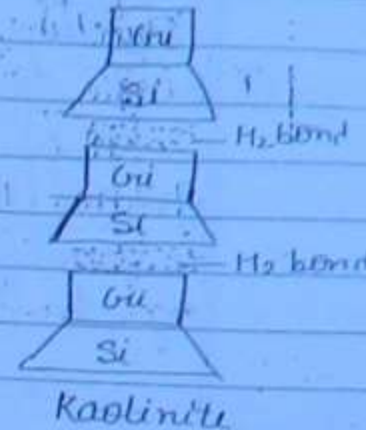
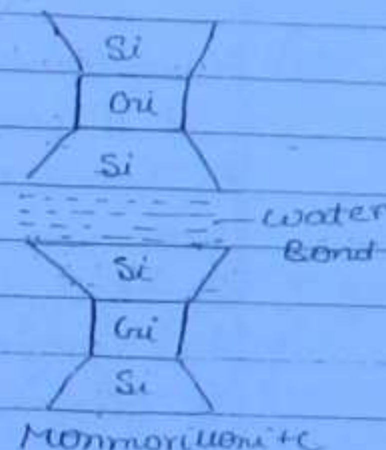


Gibbsite sheet is Sandwiched b/w Silica Sheet. Various such mol. are loosely bonded through water bond.

Thickness of water bond may be as high as 200 A°

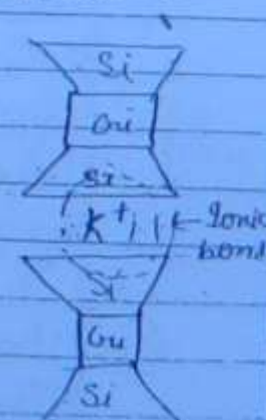
eg. black soil & bentonite soils.

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



Illite Minearals

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 jointed together by Ionic bond (pottasium ion). Ionic bond is neither very weak & nor very strong. Hence these shows medium swelling & shrinkage properties



eg. Alluvial Soils

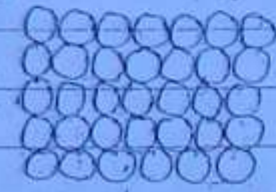
Preameability in increasing order = Montmorillonite, Illite, Kaolinite

Activity in increasing order = Kalinite, Illite, Montmorillonite

Structure of Soils

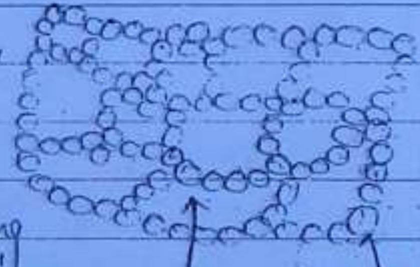
1. Single grained Structure

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.



2. HoneyCombed Structure

It is found in soils having particle size betn 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

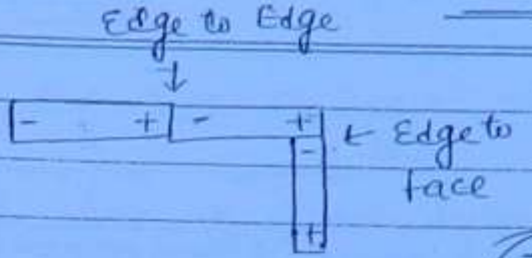


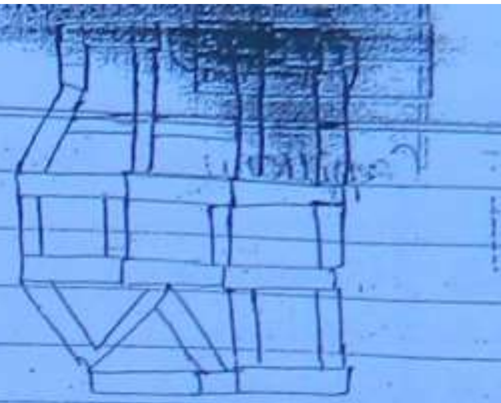
eg- Silt & Coarse Clays

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 c̄ surface electric forces. The effect of gravity forces is negligible hence surface electric forces play dominant role

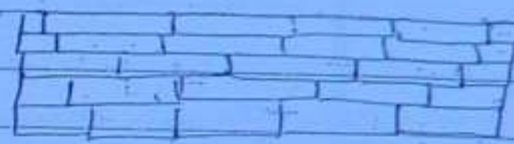
In flocculated structure Clay particles are join edge to edge or edge to face through attractive forces These soils enclose high volume of voids & On remoulding





these may get converted into dispersed Structures.

Dispersed Structure are formed when Clay particles are joint through face to face through repulsive electric forces

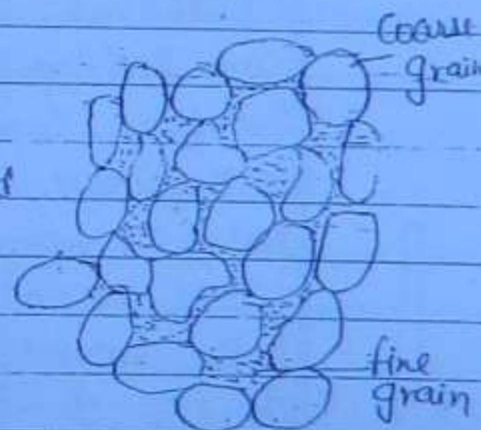


These enclose less vol of voids & have structure similar to brick structure

④ Structure of Composite soil.

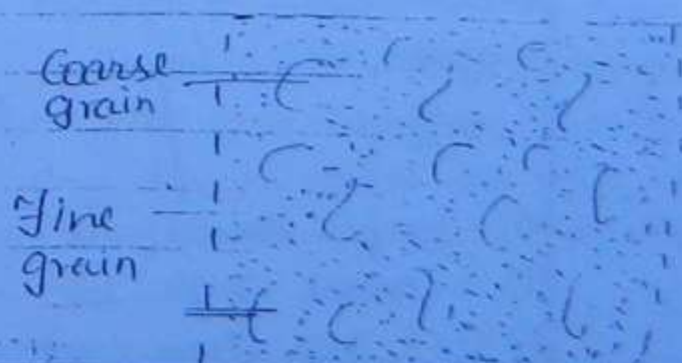
① Coarse grain Skeleton

The Coarse grain particle are in Contact c̄ each other & voids are filled by fine grained particles Such soils are less Compressible



② Cohesive matrix

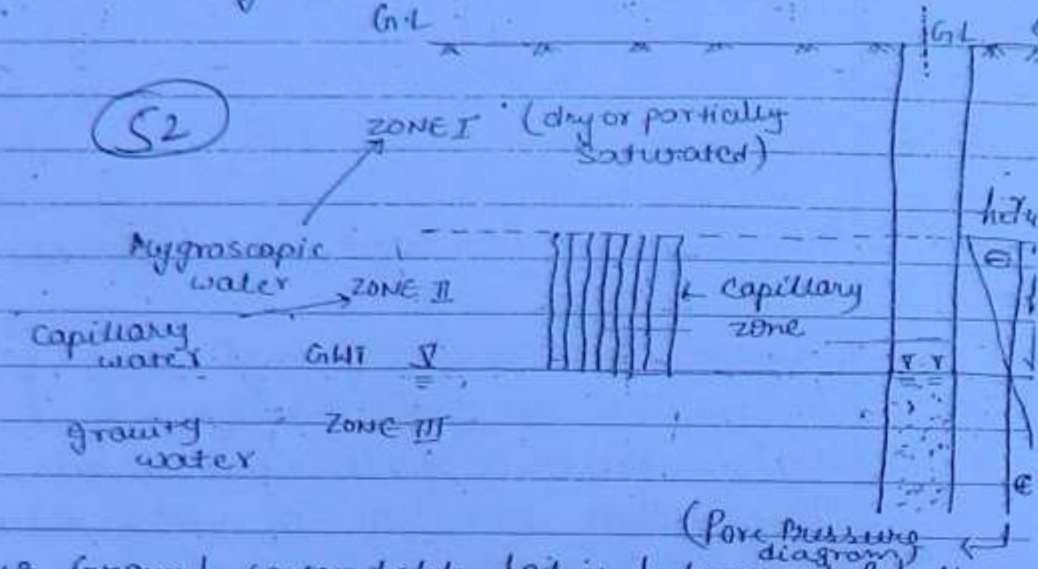
Fine grained particle are in Contact c̄ each Other & Some of the Coarse grain particle are also present in the soil mass without contact to each other.



Capillary & Permeability

Zone I – Moist/Dry.
Zone II – Capillary.
Zone III – Submerged
$S = 100\%$.

(S2)



Zone III below Ground water table soil is submerged & fully saturated. Below GWT 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

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

σ = surface tension force = force per unit length
θ = Contact angle betn water and Capillary
$\theta \approx 0°$ for clean water & soil force.

Effective pressure. It is grain to grain contact pressure b/w soil particles. Under dry condition/moist condn 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 ~~self~~ wt of soil is equal to wt. of soil under bulk condition divided by area. If external load is also acting then total stress is self wt. effect + applied load effect.

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

Pore " " " $u_1 = 0$

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

& total pressure at 2-2 $\sigma_2 = \gamma_1 z_1 + \gamma_2 z_2$

$\gamma_1 z_1 + \gamma_{sat} z_2$

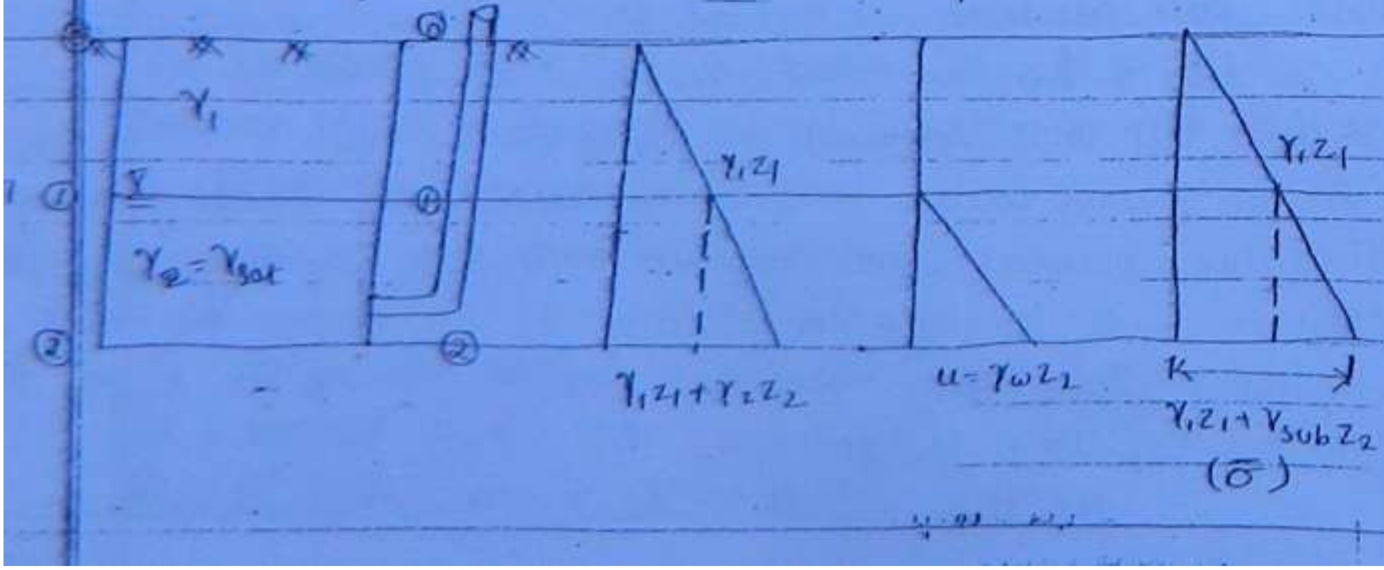
$u_2 = \gamma_w h_w \Rightarrow \gamma_w \cdot z_2$

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

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

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

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

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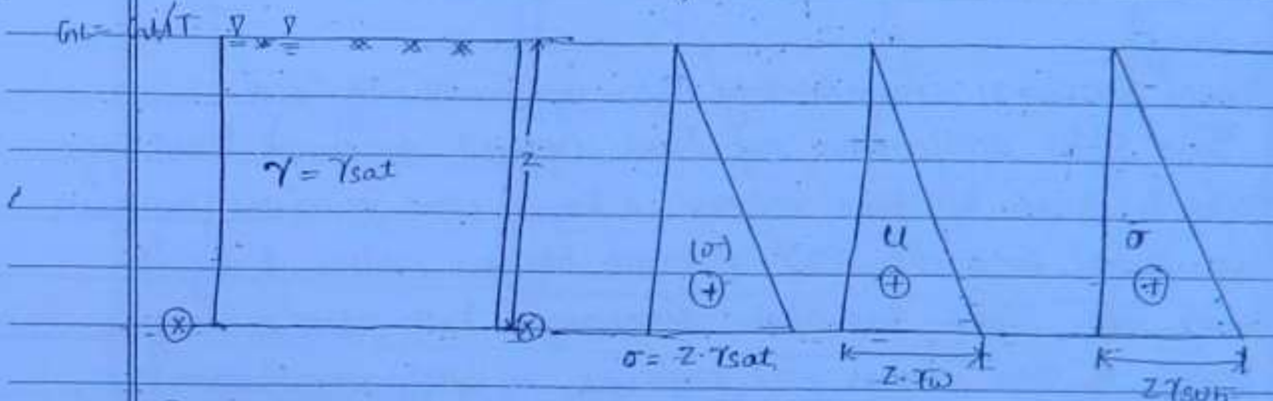
CASE I Plot total stress, eff. stress & pore water pressure diagram when water table is at ground level.



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

Pore pressure $= z \cdot \gamma_w$

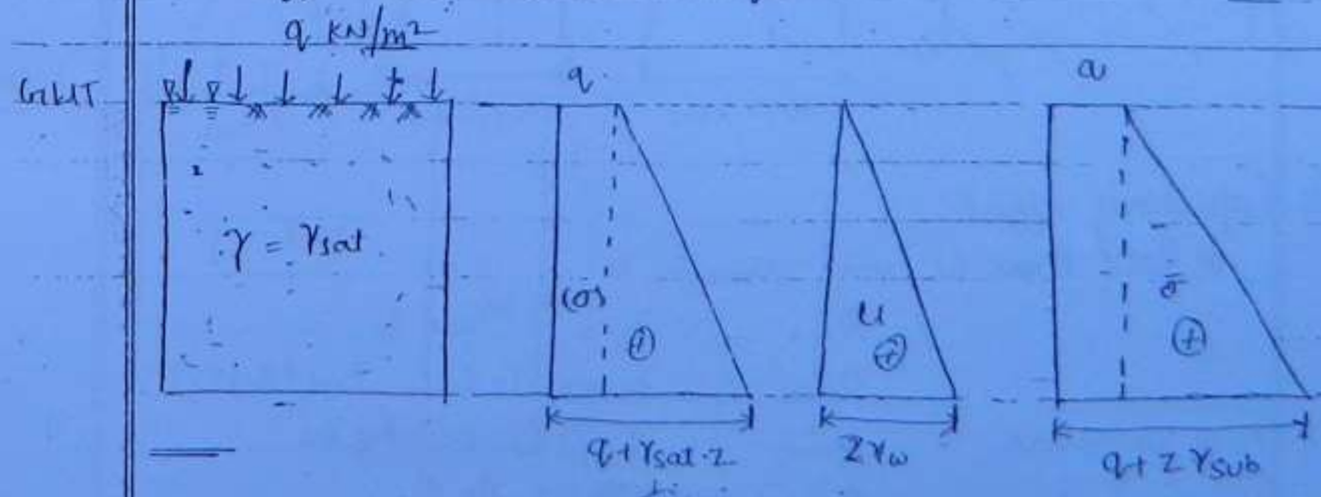
$\bar{\sigma} = z \cdot \gamma_{sat} - z \cdot \gamma_w$

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

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

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 ↑ in total stress & pressure will be equal hence eff. stress will not change.



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

Total Stress $\sigma_x = q + \gamma_{sat} \cdot Z$

$u = Z \cdot \gamma_w$

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

$q + Z\gamma_{sub}$

56

② Effect of surcharge immediately after application of load

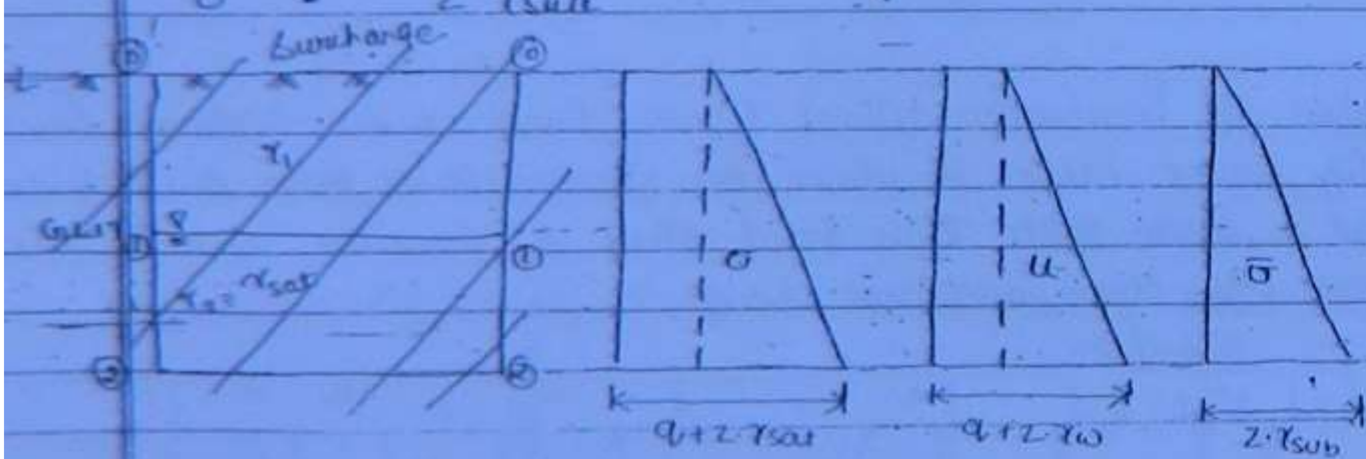
Just after application of load applied external surcharge load is taken by pore water & hence pore pressure increases by equal amount (q) & total stress also ↑ by (q) hence eff. stress remains unchanged but after a long period of time excess pore pressure is dissipated hence surcharged is transferred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

$\sigma = q + Z \cdot \gamma_{sat}$

$u = q + Z \cdot \gamma_w$

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

$\bar{\sigma} = Z\,\gamma_{sub}$



Case III Capillary effect

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

$\sigma_{1\text{-}1} = \gamma_1 Z_1 = \gamma_{sat} Z_1$

$\sigma_{2\text{-}2} = \gamma_1 Z_1 + \gamma_2 Z_2 = \gamma_{sat}(Z_1 + Z_2)$

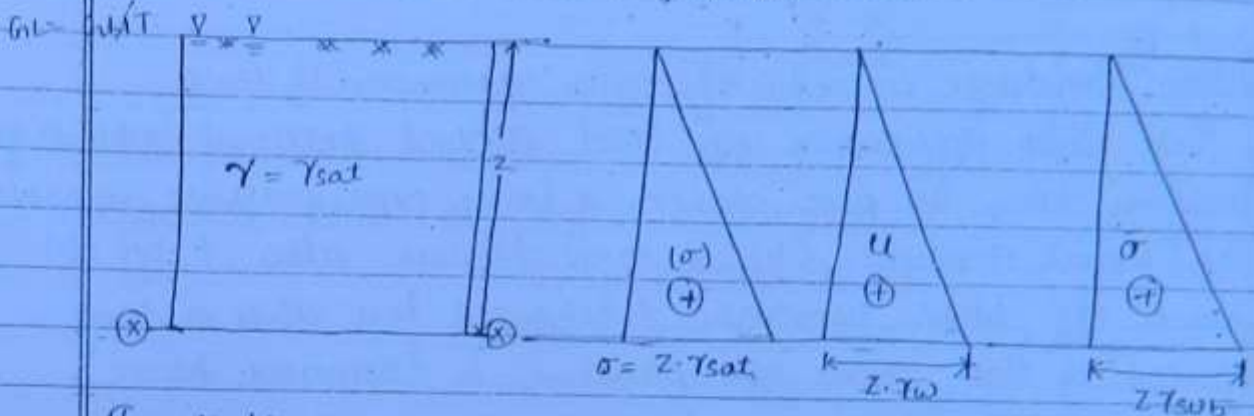
Pore pressure $u_{0\text{-}0} = 0 - Z_1 \gamma_w = -h_c \gamma_w$

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

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.



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

Pore pressure $= z \cdot \gamma_w$

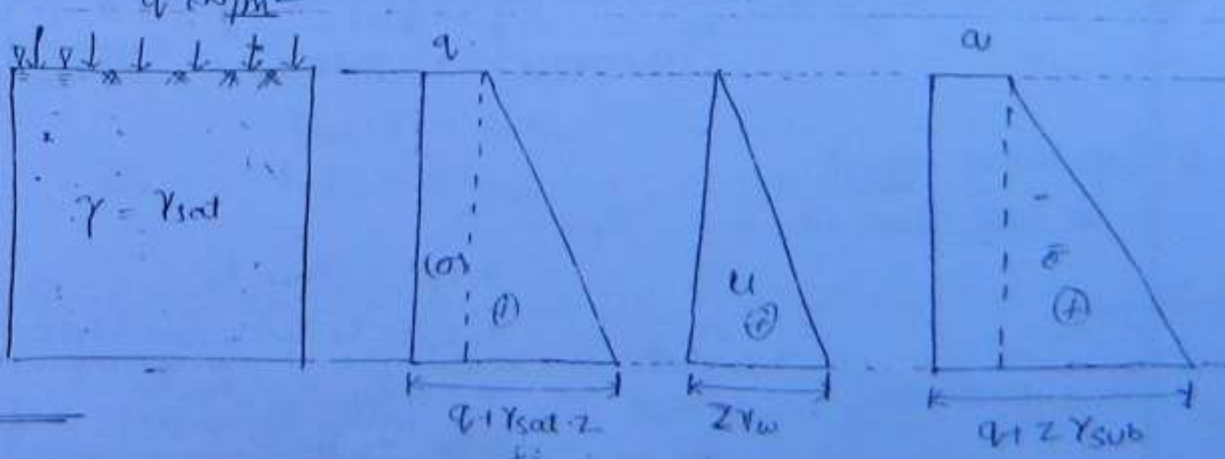
$\bar{\sigma} = z \cdot \gamma_{sat} - z \cdot \gamma_w$

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

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

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 ↑ in total stress & pressure will be equal hence eff stress will not change.



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

Total Stress $\sigma_x = q + \gamma_{sat} \cdot Z$

$u = Z \cdot \gamma_w$

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

$q + Z \gamma_{sub}$

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② Effect of surcharge immediately after application of load

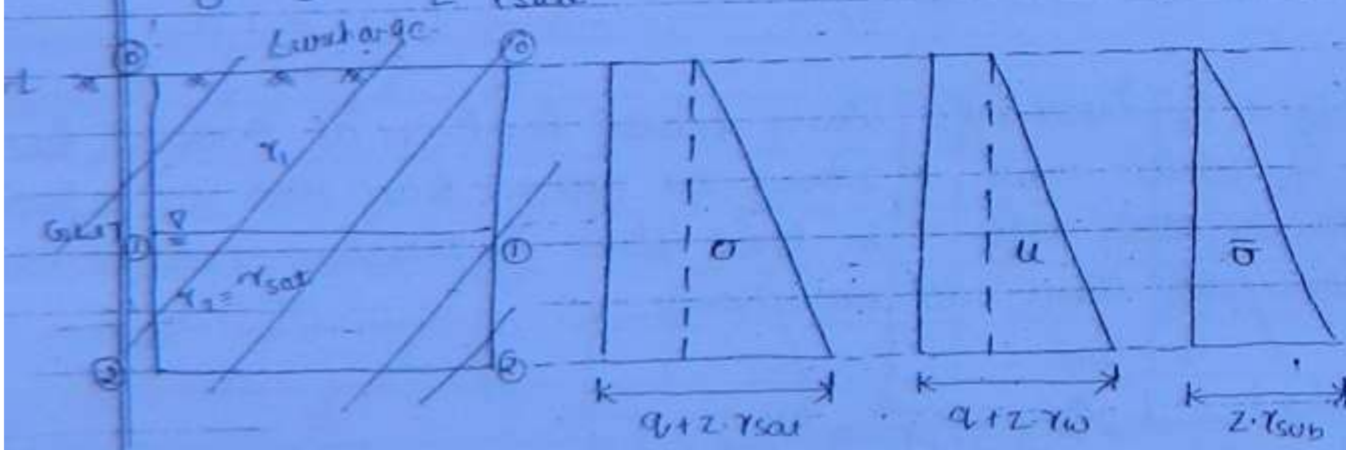
Just after application of load applied external surcharge load is taken by pore water & hence pore pressure increases by equal amount (q) & total stress also ↑ by (q) hence eff. stress remains unchanged but after a long period of time excess pore pressure is dissipated hence surcharged is transferred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

$\sigma = q + Z \cdot \gamma_{sat}$

$u = q + Z \cdot \gamma_w$

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

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



iii Capillary effect

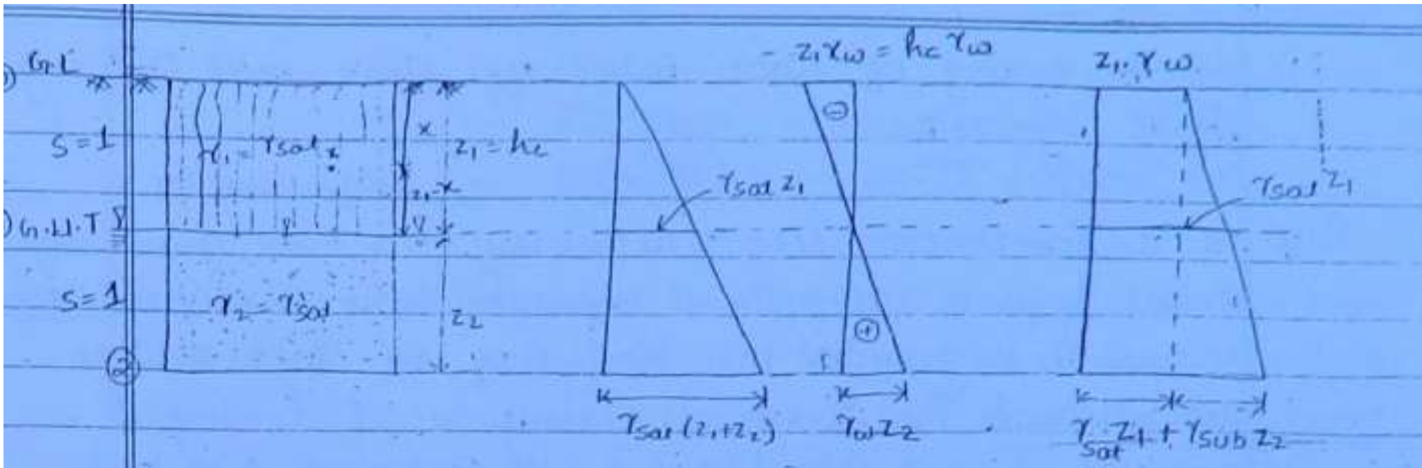
Total stress at 0-0 $\sigma_{0\text{-}0} = 0$

$\sigma_{1\text{-}1} = \gamma_1 Z_1 = \gamma_{sat} Z_1$

$\sigma_{2\text{-}2} = \gamma_1 Z_1 + \gamma_2 Z_2 = \gamma_{sat}(Z_1 + Z_2)$

Pore pressure $u_{0\text{-}0} = 0 - Z_1 \gamma_w = -h_c \gamma_w$

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



Pore pressure at 2-2 $\quad u_{22} = \gamma_w \cdot z_2$

Effective stress at $\bar{\sigma}_{0-0} = 0 + z_1\gamma_w$ (54)

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

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

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

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

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

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

NOTE① The effect of capillary action with ht. 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 distn. x

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

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

$$\& \; u_x = -\gamma_w(z_1 - x)$$

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

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

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

NOTE② If water table is present at ground level & then through 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.

$\therefore$ due to lowering of water table eff. stress will ↑ at any level by $z_1 \cdot \gamma_w$

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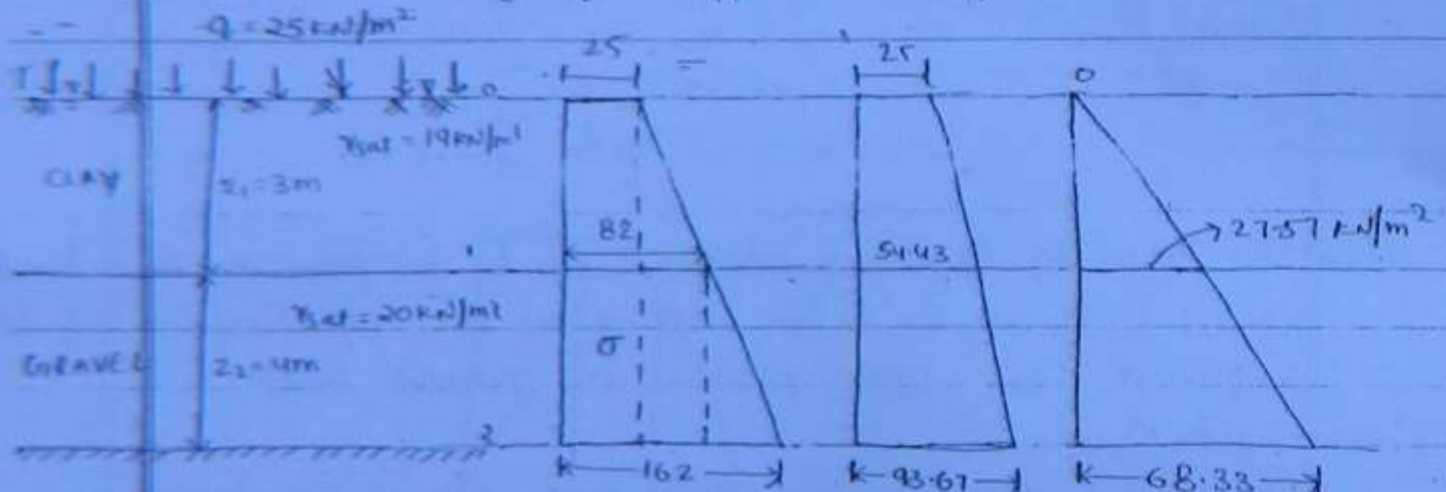
Ques At a construction site 3 m thick clay layer is followed by 4m thick gravel layer; gravel layer is resting on a impervious rock. A load of $25/m^2$ is applied suddenly at the surface of soil. The W.T is at ground level. The saturated unit wt for clay & gravel layers are $19\ kN/m^3$ & $20\ kN/m^3$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

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



① Total Stress

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

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

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

$$\sigma_{2-2} = q + \gamma_1 Z_1 + \gamma_2 Z_2 =$$

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

Pore pressure

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

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

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

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

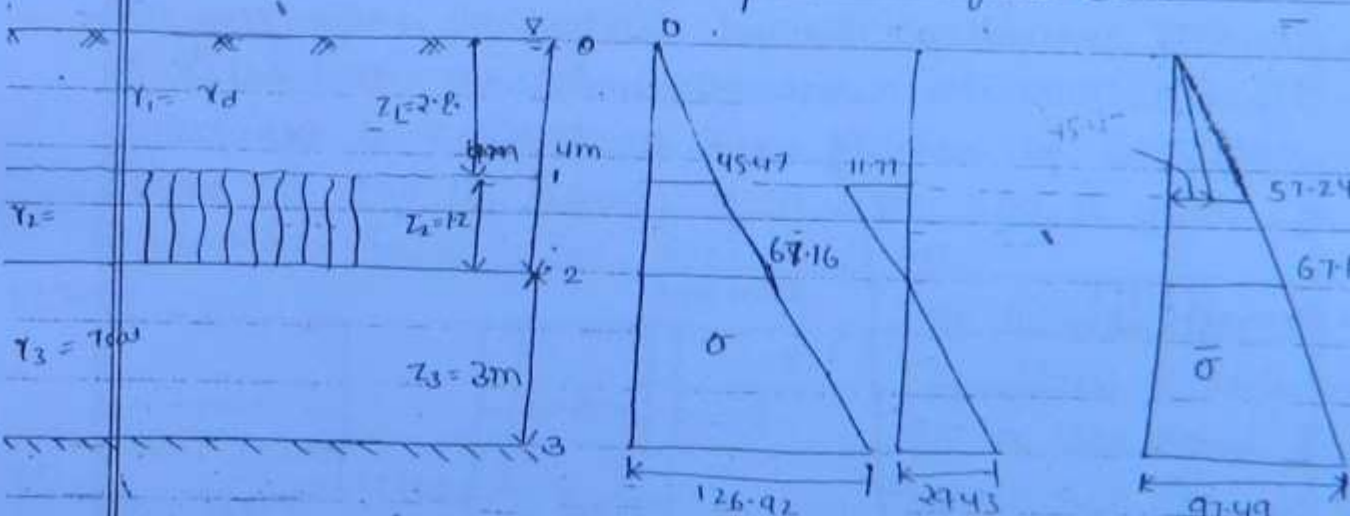
Eff. Stress $\bar{\sigma}_{0-0} = 25 - 25 = 0$

$\bar{\sigma}_{1-1} = 82 - 54.43 = 27.57\ kN/m^2$

$\bar{\sigma}_{2-2} = 162 - 93.67 = 68.33\ kN/m^2$

Ques A granular soil deposit is 7m deep resting over an Impervious layer has GWT 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 depth.

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



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

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

$\gamma_3 \quad \dfrac{(G+e)\gamma_w}{1+e} = \left(\dfrac{2.65+0.6}{1+0.6}\right) 9.81 = 19.92\ kN/m^3$

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

$\sigma_{1-1} = Z_1 \gamma_1 = \cancel{1.2 \times 18.08}\ 2.8 \times 16.24 = 45.47\ kN/m^2$

$\sigma_{2-2} = Z_2 \gamma_2 + \gamma_1 Z_1 = 45.47 + 1.2 \times 18.08 = 67.16\ kN/m^2$

$\sigma_{3-3} = \gamma_1 z_1 + \gamma_2 z_2 + \gamma_3 z_3 = 126.92 \ kN/m^2$

(62)

Pore pressure $\quad u_{0-0} = 0$

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

$u_{2-2} = 0$

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

Eff. pressure $\quad \bar{\sigma}_{0-0} = 0$

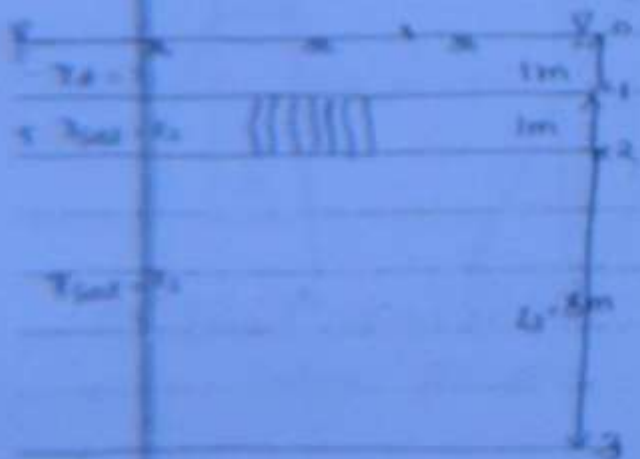
$\bar{\sigma}_{1-1} = 45.47 + 11.77 = 57.24$

$\bar{\sigma}_{2-2} = 67.16 - 0 = 67.16$

$\bar{\sigma}_{3-3} = 126.92 - 29.43 = 97.49$

-2014

Exam: 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 the dry & saturated unit wt. of sand as 17 kN/m³ & 21 kN/m³



Total stress - $\sigma_0 = 0$

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

$\sigma_{2-2} = \gamma_d \cdot 1 + \gamma_2 \cdot 1 = 17 + 21 = 38$

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

Pore pressure $\quad u_{0-0} = 0$

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

$u_{2-2} = 0 \cdot D_{CB} = 0$

$u_{3-3} = z_3 \cdot \gamma_w = 8 \times 9.81 = 78.48$

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

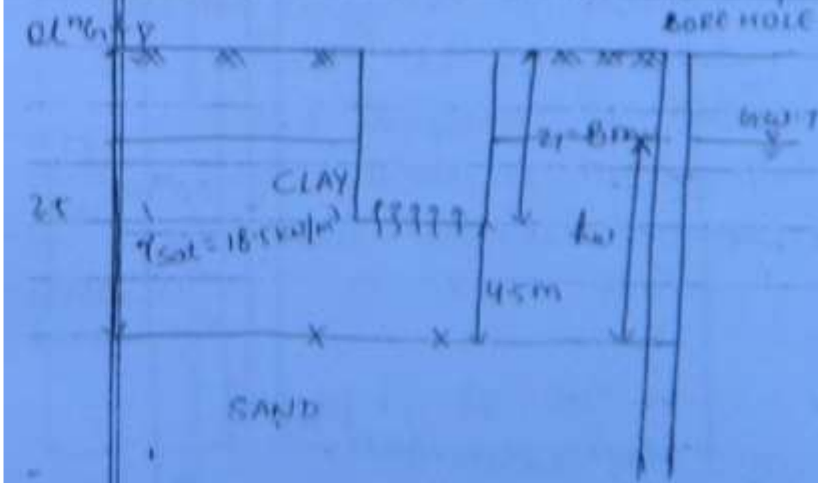
$\bar{\sigma}_{1-1} = 17 + 9.81 = 26.81$

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

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

(63)

Ques In layer of clay of thickness 12.5m is underlying by sand. The saturated unit wt. of clay is 18.5 kN/m³. 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 $\gamma_w = 10 kN/m^3$



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

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

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

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

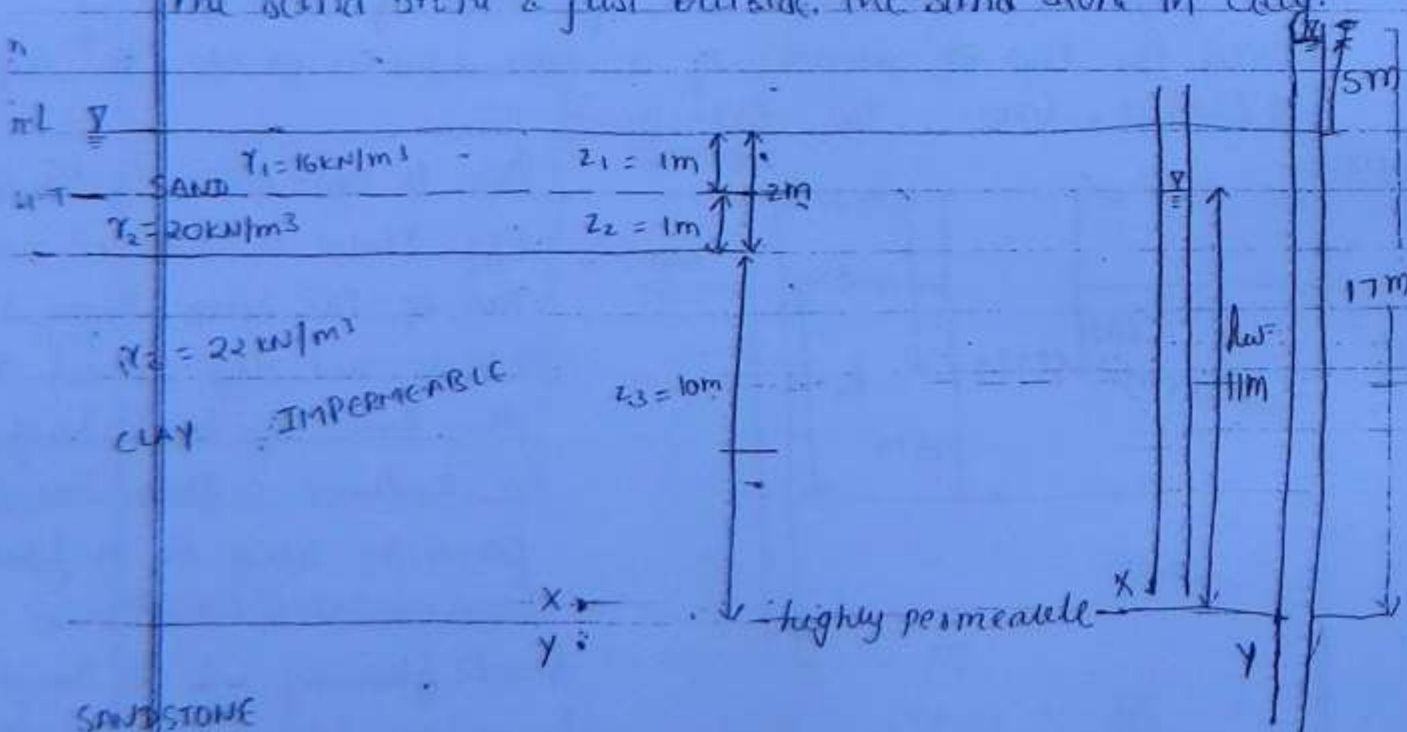
$\therefore \sigma - u = 0$

$18.5 \times 4.5 - 10 h_w = 0$

$h_w = 8.325\ m$

(64)

<u>Ques</u> The soil profile at the bottom of the valley comprises 2m of sand overlying 10 m of clay. The clay layer itself is resting on highly permeable weathered sand stones. Unit wt of sand γ above the W.T is 16 kN/m³. W.T is 1m below ground level & unit wt. of sand below the water table is 20 kN/m³. The saturated unit wt of clay is 22 kN/m³. If the water table in the sand stone layer is under artesian pressure condition & is equal to a level of 5 m 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 – $K \propto \frac{\gamma_w}{\mu}$

Dynamic viscosity coefficient — μ is inversly proportionly to temp. in liquid. (65)

(7) Impurities present in water – ~~Impurity~~ 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 correction can be made

$$K_{20°} = K_{T°} \times \frac{\mu_{T°}}{\mu_{20°}}$$

$K_{T°}$ = Coefficient of permeability at T°C then

$$\boxed{\frac{\mu_{T°}}{\mu_{20°}} = R_T = 2.42 - 0.475 \log_e T°C}$$

$$K_{20°} = K_{T°} \times R_T$$

(2) The Coefficient of permeability (k) depends upon properties of water (μ & γ_w). Another Coefficient ~~called~~ ~~Coefficient~~ called Coefficient of absolute premeability can be defined which is independent of fluid properties. Darcy

Specific permeability or intrinsic permeability (k_0) — Absolute Coefficient of permeability (k_0)

$$k_0 = k \frac{\mu}{\gamma_w}$$

Unit of k_0 is m^2 or cm^2 or ~~mm~~^{2}, or darcy

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

Permeability of Stratified soils

11 ESE Ques — What is instrinc permeability of a water saturated medium which has hydraulic conductivity of 15.24 m/day. Assume the groundwater is at atm. pressure at 20°C

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

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

66

Coefficient of percolation is Coefficient of permeability divided by porosity.

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

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 ⇒ Sp. surface area $= \frac{4\pi R^2}{\frac{4}{3}\pi R^3} = \frac{3}{R} = \frac{6}{D}$

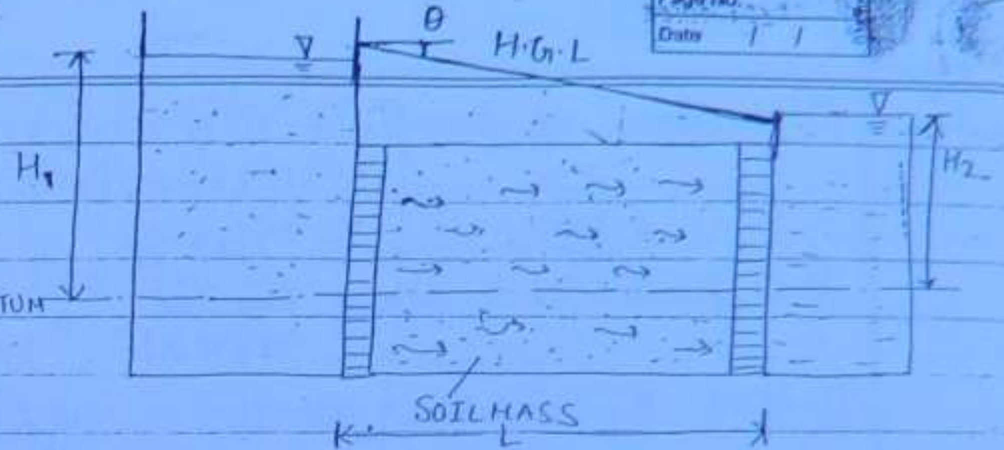
Note In natural soil deposits soils having same void ratio but diff shape of particles these soil will have greater permeability in which particles are angular than those soils which have rounded particle due to of better Connectivity of pores

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.

$H_L = H_1 - H_2$



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

q = discharge through the soil mass (67)

$q = v \times A$ (m^3/sec)

v = discharge velocity

A = Area of Soil Mass

flow per unit time $\boxed{q = K \cdot i \cdot A}$ ← Darcy's equation

<u>NOTE</u> 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 flow 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

$$\overline{v_s} = k_p \cdot i$$

k_p Coefficient of percolation

$$v_s = \frac{q}{A_v} = \frac{KiA}{A_v} = \frac{v \cdot A}{A_v}$$

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

$$\frac{v_s}{v} = \frac{1}{n} \Rightarrow \boxed{v_s = \frac{v}{n}}$$

<u>Imp</u> $n < 1$

Hence $v_s > v$

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

68 $= 16 \times 1 + 20 \times 1 + 22 \times 10$

$= 256 \ kN/m^2$

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

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

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

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

Pore pressure at $u_y = 17 \times 9.81 = 166.7 \ kN/m^2$

$\bar{\sigma} = 256 - 166.7 \ kN/m^2 = 89.23$

Permeability (Hydraulic Conductivity) case ē ̄c water flows through that soil

It refers to the water seepage capacity of a soil if pores/voids are interconnected & there 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)

$\Rightarrow v \propto i \Rightarrow v = Ki$ $\quad i$ = hydraulic gradient

$v \propto \frac{H_L}{L}$ $\quad i = H_L/L$ (H_L Loss of head through L length of soil mass)

$v = K\frac{H_L}{L}$ $\quad$ (Homogenous soil mass)

K = Constant of prop = Coefficient of permeability

$K = \frac{v}{i}$ = m/s or cm/sec or mm/sec c density of 998.2 kg/m³ & viscosity = 1.002 × 10⁻³ kg/m sec

Sol. $K_0 = \frac{K\,\mu}{\gamma_w} = \frac{K\,\mu}{\rho_w \cdot g}$

$\mu = \frac{N\text{-sec}}{m^2}$ $\quad N = \frac{kg\text{-}m}{sec^2}$

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

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

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

$\mu = kg/m\,sec$

$\frac{\mu}{g} = kg/m\,sec = \frac{kg}{m \cdot sec} \times \frac{sec^2}{m} = kg\,sec/m^2$

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

$K = 15.24$ m/day

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

$K_0 = \frac{15.24}{24 \times 3600} \cdot \frac{1.002 \times 10^{-3}}{998.2}$

$K_0 = 1.77 \times 10^{-10}$ m²

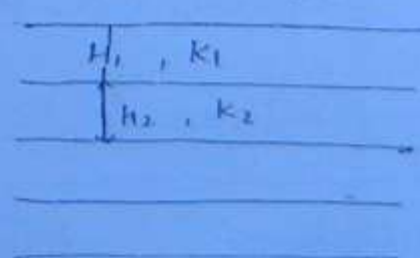
Permeability of Stratified Soil

Let there are diff. layers of soils having thickness H_1, H_2, H_3 -- H_n & Coefficient of permeability is k_1, k_2 -- k_n (hydraulic conductivity) of each layer. Let bedding planes are parallel in x-dirn or in Y-dirn

X & Y → horizontal dirn

Z → vertical dirn

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



K_x or $K_y = \frac{k_1 H_1 + k_2 H_2 + H_3 k_3 \cdots k_n H_n}{H_1 + H_2 + \cdots H_n}$

The Equivalent or effective coefficient of permeability normal to the bedding planes is K_z

$$K_z = \frac{H_1+H_2+H_3 \text{ —— } H_n}{\frac{H_1}{K_1}+\frac{H_2}{K_2}+\text{ —— }\frac{H_n}{K_n}}$$

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average

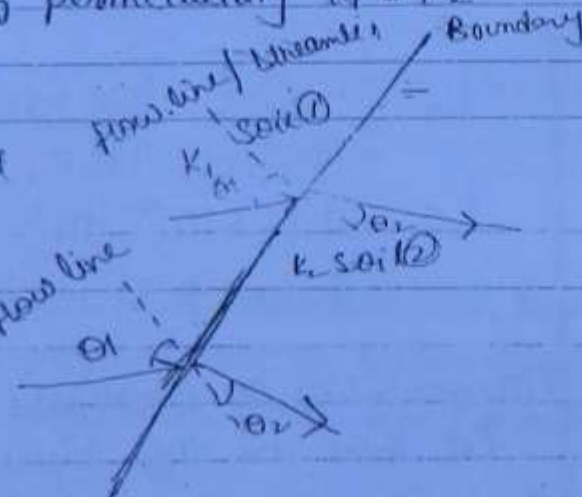
NOTE $K_x > K_z$ (i.e. permeability parallel to bedding plane is ~~more~~ greater than normal)

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 $\quad K_e = \sqrt[1/3]{K_x \cdot K_y \cdot K_z}$

(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

- Lab Methods
  - Constant Head Permeability test (used for coarse soils)
- Field Methods
  - → Pumping out test (area of influence is large)
  - → Pumping in test (area of influence is small)
- Indirect Methods
  - → Kozeny Karman eq^n
  - → Allen Hazen eq^n
  - → Loudens eq^n

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 soil mass A
At any stage when head loss is H

$$q = KiA$$

$$q = K \cdot \frac{h}{L} \cdot A \quad \text{— (71)}$$

Vol. of water c will flow in time dt =

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

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

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

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

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

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

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

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

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)

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

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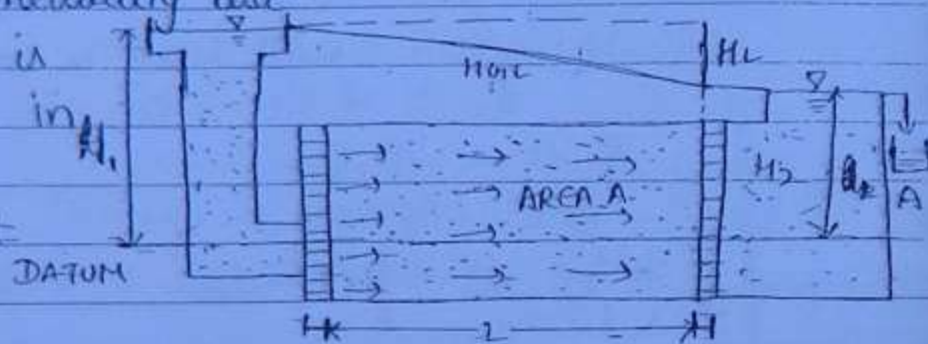
Constant head permeability test
volume of the water is collected in time t in the tank A
H_2 & H_1 are Constant



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

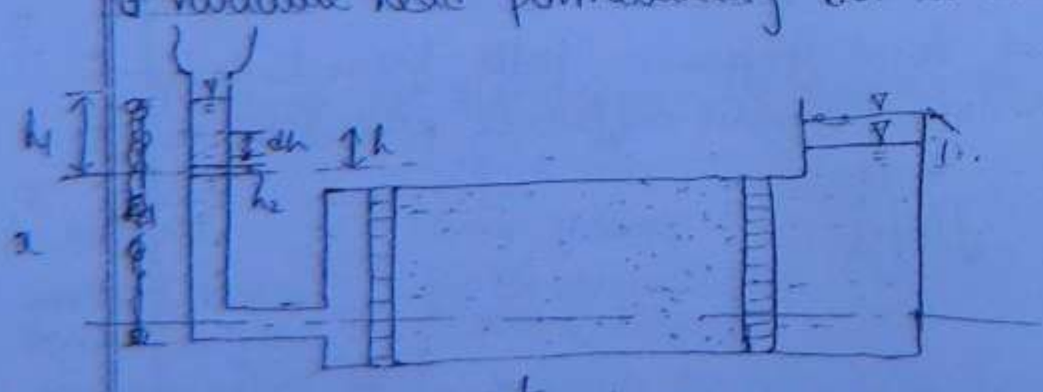
$$q = \frac{Q}{t}$$

$$q = kiA$$

$$k = \frac{q}{i \cdot A} = \frac{Q}{tiA} = \frac{Q}{t\left(H_L/L\right)A} = \frac{Q \cdot L}{A \cdot t \cdot H_L}$$

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

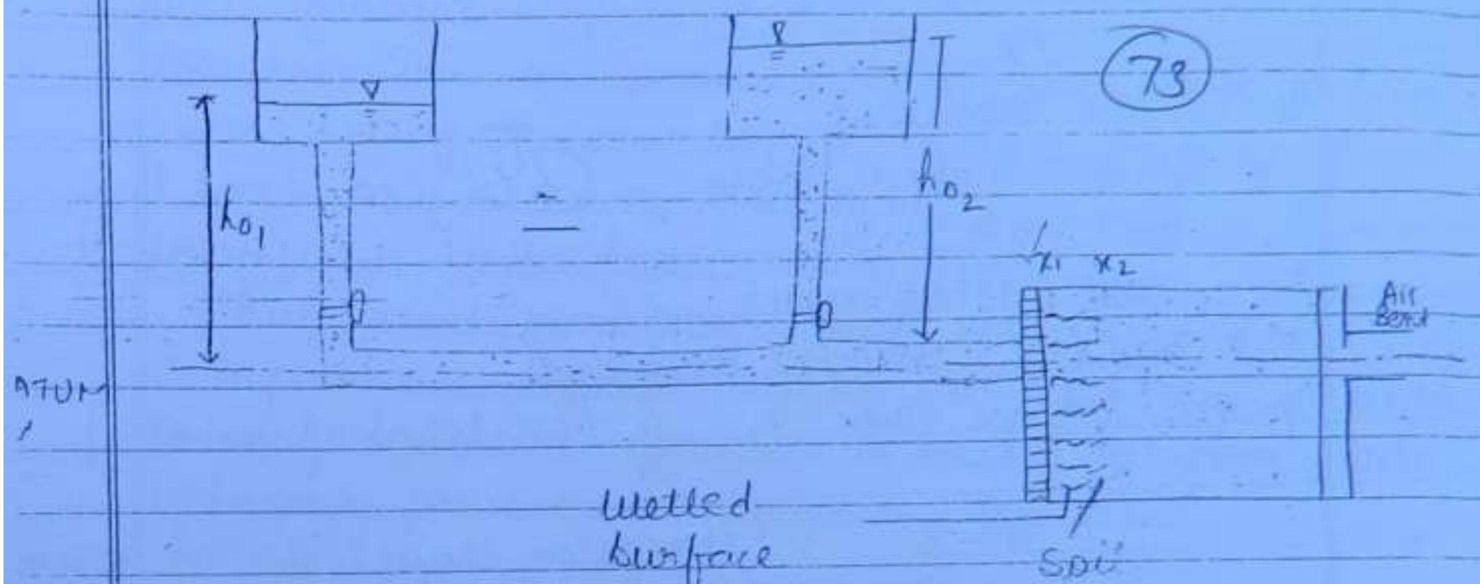
Variable head permeability test or falling head permeability



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

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

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

$$\frac{x_2'^2 - x_1'^2}{t_2' - t_1'} = \frac{2K}{S \cdot n}\left(h_{o2} + h_c\right) \quad \text{---- (2)}$$

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 Kozney Karman's equations

$$K = \frac{1}{K_K} \times \frac{\gamma_\omega}{\mu} \times \frac{e^3}{1+e} \times \frac{1}{S^2}$$

K_K = Shape Coefficient

S = Specific Surface area

e = void ratio

γ_ω = Unit wt of water

μ = dynamic viscocity Coefficient

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

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

b = Size of Sieve ~~through at~~ Particles Retained.

② Allen Hazen's equation

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

D_{10} = eff. size in cm

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

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

③ Louden'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 eq^n (K is less than applicable)

$$K = C_v \cdot \gamma_\omega \cdot m_v$$

C_v = Coefficient of consolidation

m_v = Coefficient of volume Compressibility for partially saturated soil.

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- Well Hydraulics

1 Aquifers– These are porus & permeable formations c 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 c 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 porus & permeable formation are sandwiched betn two impermeable strata at top & bottom. The aquifer is under artesian pressure / or confined pressure.

(3) Perched aquifer

If water cavity or porus strata is surrounded by impermeable strata c 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 porus but non permeable c can hold water but do not permit flow of water eg- Clay

(3 Aquitard - These are less porus & less permeable. There seepage capacity is low & discharge is small eg- Silt

Aquifuse – These are neither porus nor permeable.
eg- Rocks

(76)

Specific yield (S_y)

The specific yield of an unconfined aquifer is the ratio of vol. of water to c 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)

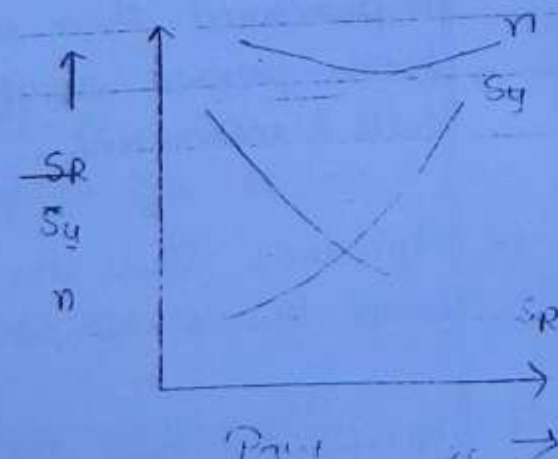
It is the ratio of vol of water retained in the voids against the gravity flow to the total vol. of aquifers.

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

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

$$\boxed{S_R + S_y = n}$$

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 the failure

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

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

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

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$$a = \frac{D}{\cos\alpha} - \sqrt{\frac{D^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}$$

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

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

$$a = \sqrt{D^2 + H^2} - \sqrt{D^2 - H^2 \cot^2\alpha}$$

Ques A homogenous isotropic earthen dam $\bar{c}$ is ~~20m~~ 18m heigh 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 80m 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 sec^n. estimate the quantity of seepage discharge per unit length of dam.

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

$$K_e = \sqrt{K_x \times K_y} = 0.277 \times 10^{-7}$$

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

$$q = 1108 \times 10^{-8} \text{ m}^3/\text{s/m}$$

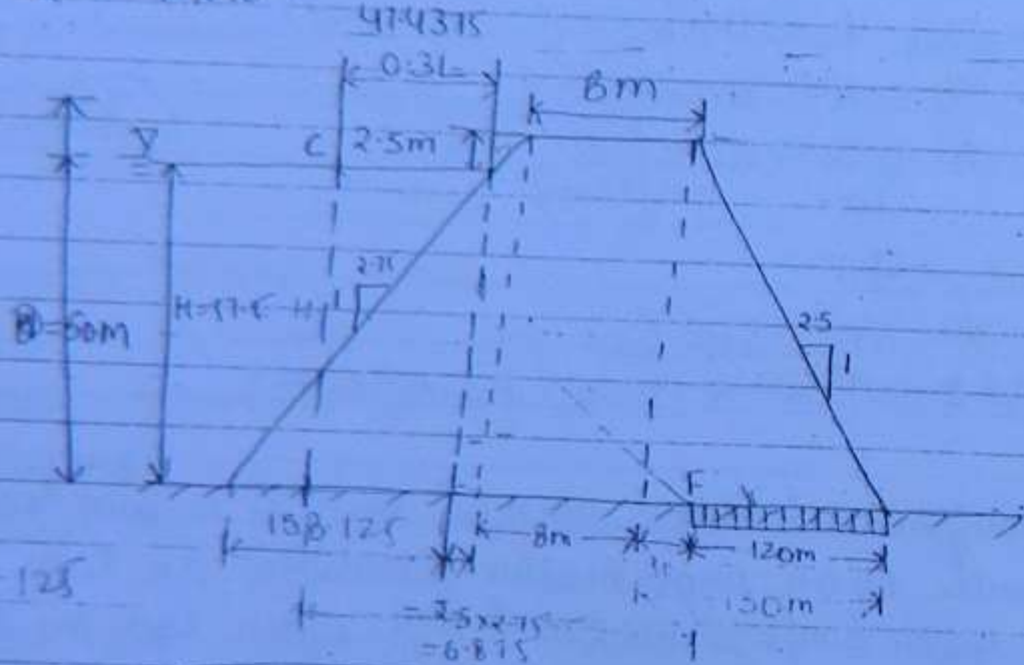
Ques Calculate Seepage through an earthen dam per unit length $\bar{c}$ is resting on an impervious foundation following data —

(i) height of dam = 60 m

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

d/s Slope - 2.5 horizontal to 1 vertical
Free board = 2.5m
Crest width = 8m (78)
length of drainage filter = 120 m from toe
$K_x = 8 \times 10^{-7}$ m/s $K_y = 2 \times 10^{-7}$ m/s

Soln $K = 4 \times 10^{-7}$



L = 158.125

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

D = horizontal distn of C from F
= 92.275 m

H = 57.5 m

$S = \sqrt{92.275^2 + 57.5^2} - 92.275 = 16.45\,m$

$q = K.S$
$= 4 \times 10^{-7} \times 16.45$
$= 65.8 \times 10^{-7}$ m³/sec/m

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

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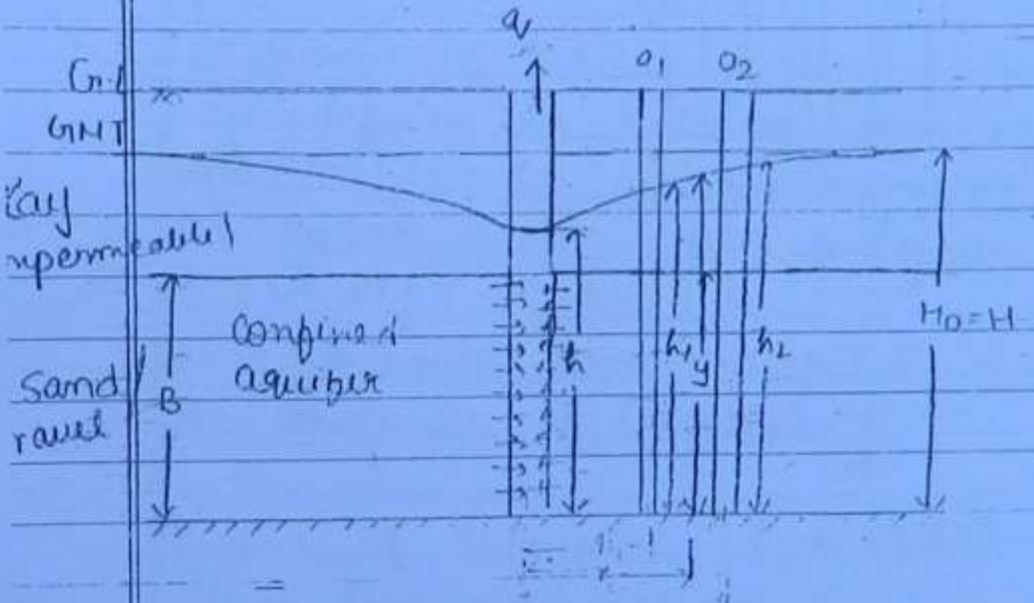
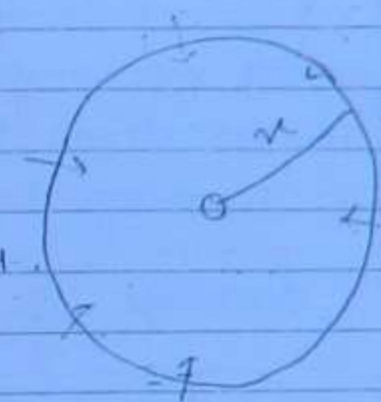
② Confined aquifer

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





$$q = KiA$$

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

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

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

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

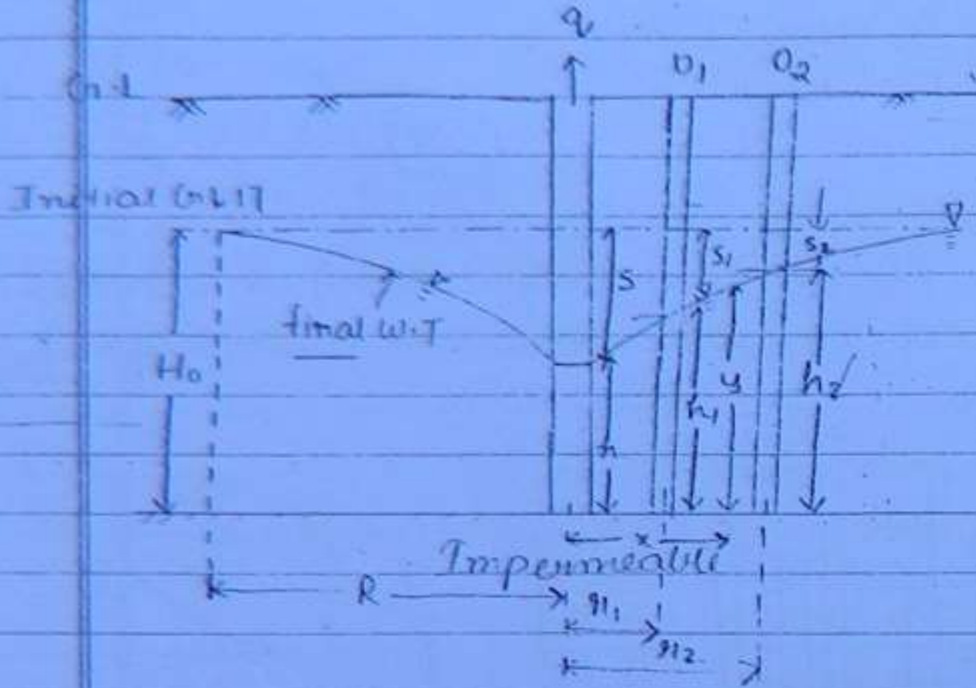
$$K = \frac{2.303\, q \log(r_2/r_1)}{2\pi B\,(h_2 - h_1)}$$

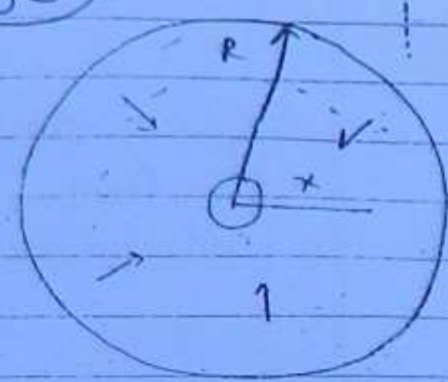
dupuits $r_2 = R$; $r_1 = r$

$h_2 = H$; $h_1 = h$

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

(80)





from fig.

$$h_1+S_1 = h_2+S_2 = H_0$$

$$h_2-h_1 = S_1-S_2$$

$$h+S = H_0$$

$$S = H_0-h$$

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

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

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

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

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

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

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

If $r_1 = r$ & $r_2 = R$
$h_1 = h$ 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 c̄ can be realised by a Confined aquifer ~~for~~ per unit surface area of a aquifer ~~fo~~ per unit Change in head. It is dimensionless. It lies in the range of 5×10^{-5} to 5×10^{-3}

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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.

$$\boxed{T = K \cdot H}$$

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 c̄ Contributes 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

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

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

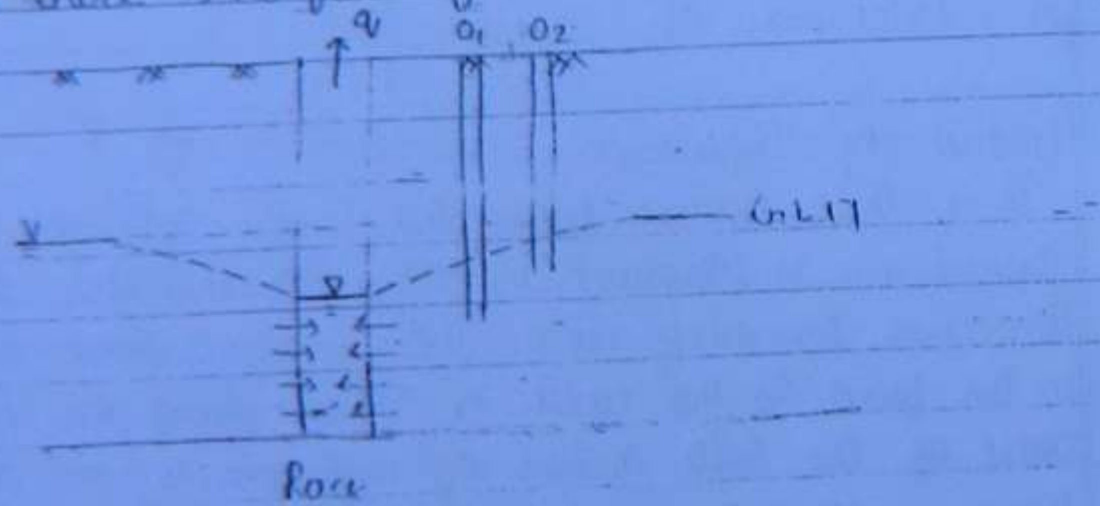
1 Dupuit's theory   2. Them's theory.

Them's theory is an improvement over dupuit's theory & is more accurate hence them's theory is widely adopted

Them's theory

→ assumption

(1) Test well penetrates through the entire thickness of aquifers & 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 it means K & T are constant with depth

(5) Two observation wells are made to read the position of W.T or draw down & are located within the radius of influence

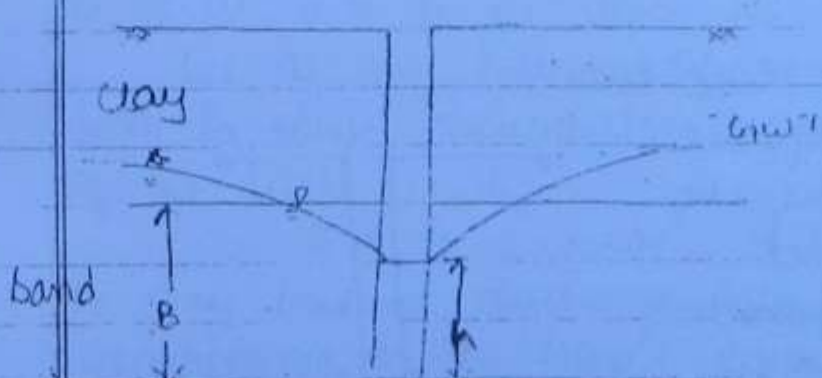
$$H - h = S \text{ (drawdown)}$$

$$K = \frac{2.303\, q \log(R/r)}{2\pi B S}$$

(83)

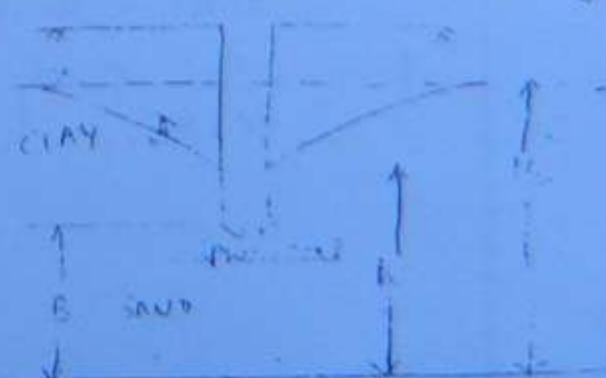
NOTE- (1) Flow contribution by Clay into the well is neglected
(2) The Condition remain artesian throughout stage of pumping it means steady drawdown level should be always above the layer of Confined aquifer it means $h > B$

(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.



$$K = \frac{2.303\, q \log_{10}\left(\frac{r_2}{r_1}\right)}{\pi\,(h_2^2 + B^2 - 2h_1 B)}$$

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 1/3 times of radial discharge in fully penetrating well.



$q = K \cdot 2\pi r (H-h)$ $\left\{ r = \text{radius of well} \right.$
$q = K 2\pi r S$ $\left. S = \text{draw down} \right.$

Pumping in test / Slug test (84)

① Open end test

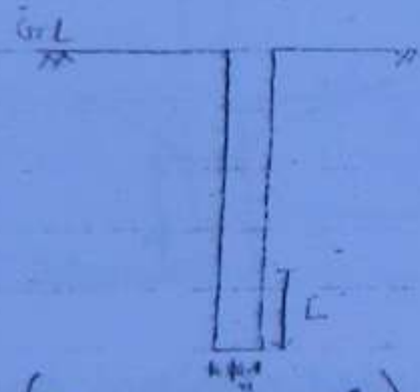
Open end pipe is sunk into the ground & soil in the pipe is remove. The clean water having temp little greater than the ground water temp. is added & discharge added is measured through meeting system. The head is kept constant. Let r is the radius of pipe, h is the head under which water is added above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

h = gravity head + Pressure head (If any)

$$\boxed{K = \frac{q}{5.5\, r h}}$$

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)

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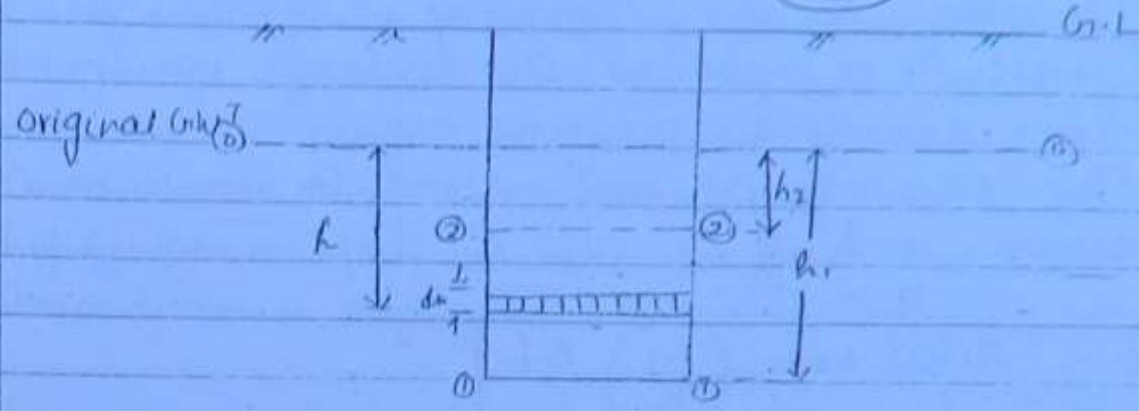


The flow in open well is essentially from base because sides are lined through brick or stone work. Let original W.T before pumping is at 0-0 & after pumping W.T falls through (1)-(1) at depth h_1 when pumping is stop. After pumping is stoped well recuperated & time of recuperation is noted. Let water 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 recuprated in time dt

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

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

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

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

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

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

C/A 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

$$q = Ch$$

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

COMPRESSIBILITY AND CONSOLIDATION

Compressibility

- Compaction or initial compression — Due to explusion of or compression Pore air (partially Saturated)
- Consolidation
  - Primary — Due to explusion of Pore water (fully Saturated)
  - Secondary — Due to plastic Readjustment of Soil molecule

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. Explusion of pore air
3. Compression of pore water
4. Explusion of pore water
5. Compression of solids

The decrease in vol. due to Compression & explusion of pore air is called initial Compression or Compaction where as ↓ in vol. due to explusion of pore water is called Primary consolidation. Note that water & Solid molecule are Considered incompressible.

Secondary Consolidation occur after the completion of primary consolidation when explusion of pore water is stopped. With the passage of time some of the soils show further ↓ 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 explusion of pore water is stop. During the process of consolidation soil remains saturated it means decrease in vol of soil mass is equal to explusion 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

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

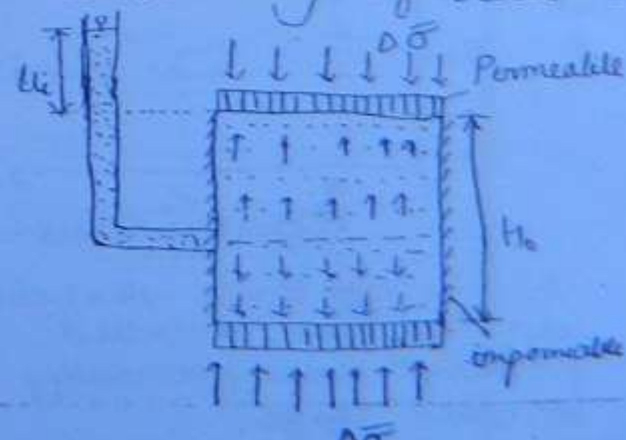
The rate of consolidation depends on

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

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

at time $t = 0$

④

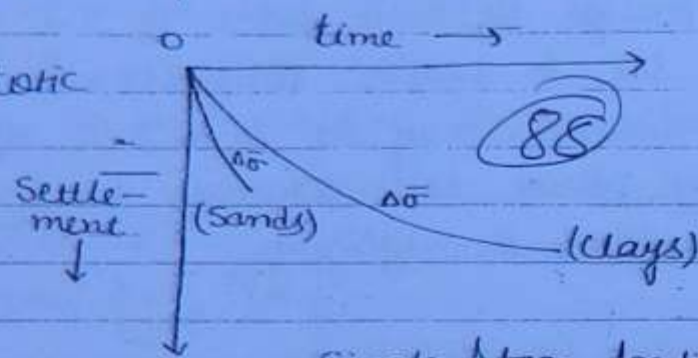


Time v/s Settlement Curve in Consolidation

xss pore press - $u_i = \Delta\bar{\sigma}$ (at the beginning of Consolidation)

$u_o = 0$ (after Completion of Consolidation)

u is found to be assymptotic



Single Stage loading

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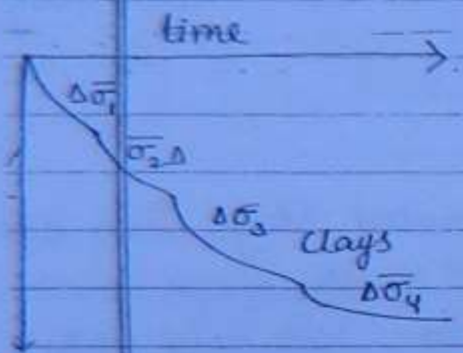


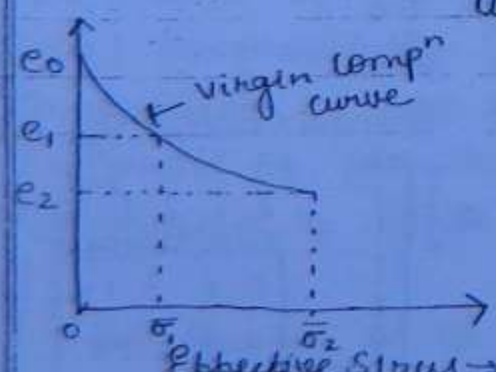
Multi Stage loading

IE 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 reduced

Pressure / effective stress V/s void ratio Curve.

Curve is assymptotic



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

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

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

to ↑ in eff. stress void ratio decreases. This Curve is called virgin comp^n Curve / Normally consolidated stage

Δe is -ve [decrease in vol.]

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

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

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

e = 0.25 for wooden pile & cast iron hammer

= 0.4 for concrete pile & cast iron hammer

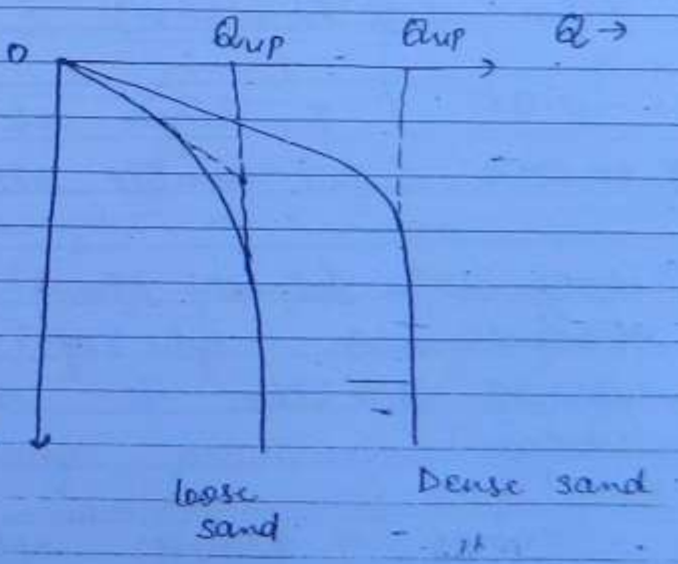
= 0.5 for Steel pile & cast Iron hammer

s = final set per set of hammer

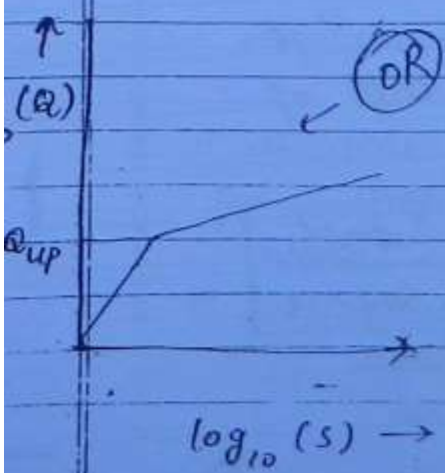
c = total elastic compression of pile, pile cap & soil

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.



OR



c = ~~0.25~~ cm (single & double acting steam hammer).

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

ultimate load on pile

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

allowable load on pile

$$Q_{ap} = \frac{Q_{up}}{F.OS}$$

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

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 coff of restitution.

$$\boxed{\eta_b = \frac{\text{energy of hammer after impact}}{\text{energy of hammer before impact}}}$$

This method is suitable for clays not suitable for sandy soil. For sandy soil SPT is best.

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2. Dynamic Method :-

a) Engineering News Record Formula [ENR]

$$\boxed{Q_{up} = \frac{WH}{S+C}}$$ For drop hammer & single acting steam hammer

$$Q_{ap} = \frac{Q_{up}}{FOS} = \frac{WH}{FOS\,(S+C)} = \frac{WH}{6(S+C)}$$

$$\boxed{Q_{ap} = \frac{WH}{6\,(S+C)}} \qquad [FOS = 6]$$

$$\boxed{Q_{up} = \frac{(W+ap)\,H}{(S+C)}}$$ for double acting steam hammer

a = area of piston, p = pressure (steam) acting on piston/hammer

W = weight of hammer

H = ht. of free fall of hammer (cm)

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_s.

A_b = bearing area of pile
$= \frac{\pi}{4} D^2$ (if pile is circular)
$= B^2$ (if pile is square)

A_s = surface area of pile in contact of soil
$= \pi DL$ ... (for circular)
$4BL$ (for square pile)
= (perimeter x length)

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unit cohesion at the base

$\boxed{q_b = 9c}$ for clayey soils.

$\boxed{q_s = \alpha \bar{c} + \bar{\sigma}_h \tan\delta}$ [for c-φ soil]

δ = wall friction angle of soil
$\bar{\sigma}_h$ = avg. horizontal thrust $= K\bar{\sigma}_v$

if $\delta = 0$ for clay then
$\boxed{q_s = \alpha \bar{c}}$

For clays –
ultimate load on pile.

unit avg. cohesion in the depth of pile

$\boxed{Q_{up} = 9c A_b + \alpha \bar{c} A_s}$ 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-
Pile load test
Cyclic Pile load test
SPT
CPT

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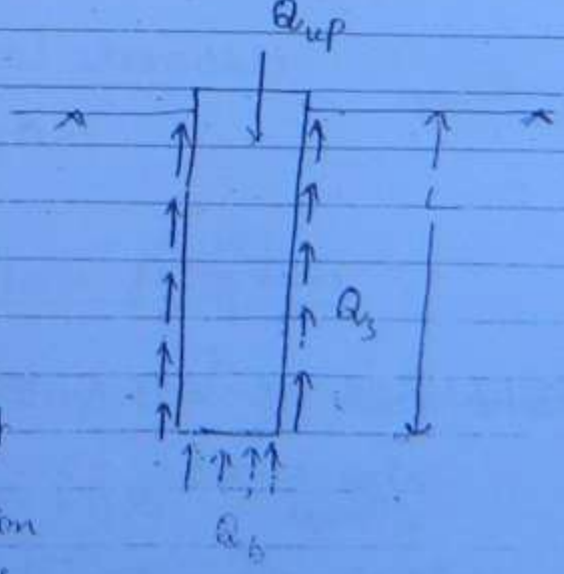
Pile load test is most accurate and best method but test pile becomes waste hence it is destructive test.

Cyclic Pile load test has advantage that it gives end bearing resistance or skin friction resistance seperately

(1) Static Method :-



ultimate load on pile

$$\boxed{Q_{up} = Q_b + Q_s}$$

$$Q_{up} = q_b \cdot A_b + q_s \cdot A_s$$

where - q_b = unit bearing/base resistance

q_s = unit skinfriction resistance

...piles :- Below hydraulic structures to minimise seepage force to prevent piping failure / quick sand failure.

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Fender pile :→ These are used to protect water front structures against the impact from wave produced by ships and other floating objects.

Type of pile on the basis of method of installation

i) Driven pile :→ precast piles are driven through dynamic action using hammer blow.

ii) Bored pile :→ precast or cast in situ. Generally driven pile have greater friction resistance then bored pile hence ultimate load capacity of driven pile is always greater than bored pile.

Allowable Load on the pile

$$Q_{ap} = \frac{\text{Ultimate Load on pile}}{\text{F.O.S.}}$$

$$= \frac{Q_{up}}{F}$$

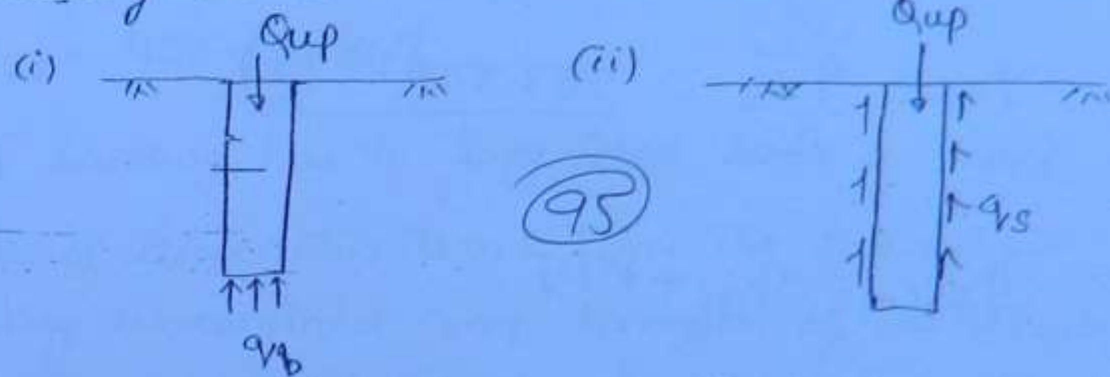
Overall FOS of 2.5 to 3 may be used.

$$Q_{ap} = \frac{Q_b + Q_s}{F}$$

$$Q_{ap} = \frac{Q_b}{C_1} + \frac{Q_s}{C_2}$$

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.



2) Friction piles or hanging piles :- These are driven through soft strata which extents 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 endbearing 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 structure

$N_c = 5.67\left(\cdots \frac{}{70}\right)$

$$q_{nu} = 50 \times 5.67\left(1 + \frac{D_f}{70}\right)$$

$$= 283.5\left(1 + \frac{D_f}{70}\right)$$

$$q_{ns} = \frac{q_{nu}}{F.O.S} = \frac{283.5\left(1 + \frac{D_f}{70}\right)}{3}$$

$$q_{safe} = q_{ns} + \gamma \cdot D_f$$

$$= 94.5\left(1 + \frac{D_f}{70}\right) + 19 \times D_f$$

$$94.5\left(1 + \frac{D_f}{70}\right) + 19 \times D_f = 140$$

$$D_f = 2.2385\,m \quad \underline{\underline{\text{ans.}}}$$

$$\left[\quad \overline{B+D_f} \right]$$

$$C_\omega = 0.5\left[1+\frac{3}{3+1.5}\right] = 0.83$$

$$q_{anet} = 0.41 \times 33.4725 \times 40 \times 0.83$$

$$q_{anet} = 455.62\ KN/m^2$$

97

<u>Question</u> A building has to supported over a raft foundation of dimension 14 m × 21m. The sub soil is clay having unconfined comp. strength of 100 KN/m². The pressure on the soil due to weight of building and applied load is 140 KN/m² at the base of raft. What should be depth of raft which should be provi with FOS 3 against shear failure. Use γ for clay 19 KN/m³ and adopt skempton's method.

<u>Ans</u> $B = 14m$, $L = 21m$, $D_f = ?$

$$C = \frac{q_u}{2} = \frac{100}{2} = 50\ KN/m^2$$

applied pressure at base of footing ≤ safe bearing capacity.

$$140 \leq q_{safe}.$$

<u>Skempton's method</u>

$$q_{nu} = CN_c$$

$$N_c = 5\left[1+\frac{0.2B}{L}\right]\left[1+\frac{0.2D_f}{B}\right]$$

$$= 5\left[1+\frac{0.2\times14}{21}\right]\left[1+\frac{0.2D_f}{14}\right]$$

orrected values for Overburden pressure

N=

$$N'_{o_1} = N_{o_1} \times \frac{350}{\bar{\sigma}_1 + 70} \qquad \bar{\sigma}_1 = \overset{\gamma_1 D_f}{18 \times 1.5} = 27$$

$$N'_{o_1} = 10 \times \frac{350}{27+70} = 36.08$$

98

$$N'_{o_2} = 15 \times \frac{350}{3 \times 18 + 70} = 42.33$$

$$N'_{o_3} = 20 \times \frac{350}{(3 \times 18) + (1.5 \times 10) + 70} = \cancel{50.3} \; 50.35$$

$$N'_{o_4} = 25 \times \frac{350}{(3 \times 18) + (1.5 \times 10) + 1.5 \times 8 + 70} = 57.94$$

orrected value for dilatancy

$N''_{o_1} = 36.08$ [No corr^n needed because WT is below test level]

$$N''_{o_2} = 15 + [42.33 - 15]\frac{1}{2} = 28.66$$

$$N''_{o_3} = \cancel{20} \, 15 + \frac{1}{2}[50.35 - \cancel{15}] = 32.68$$

$$N''_{o_4} = 15 + \frac{1}{2}[57.94 - 15] = 36.47$$

rrected average SPT no.

$$N = \frac{36.08 + 28.66 + 32.68 + 36.47}{4} = 33.4725$$

$e_0 = \frac{0.40\times 2.7}{1} = e_0 = 1.08$

$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w = \frac{2.7+1.08}{1+1.08}\times 9.81 = 17.82\ kN/m^3$

$\gamma' = (17.82-9.81) = 8.01\ kN/m^3$ 99

Intial effective overburden pressure

$\bar{\sigma}_0$ at $x = \sigma_0 - u_0 = 2.5\gamma_{sat} - 2.5\gamma_w$

$\bar{\sigma}_0 = 2.5\times 8.01 = 20.025\ kN/m^2$

$q_u = 1.3CN_c + \gamma' D_f N_q + 0.4\gamma' B N_\gamma$

$q_u = 1.3\times 50\times 6.9 + 8.01\times 1.5$

$q_u = 460.515\ kN/m^2$

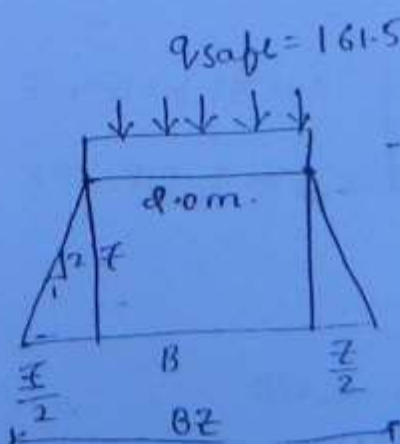
$q_{nu} = 460.515 - 8.01\times 1.5$

$q_{nu} = 448.5$

$q_{ns} = \frac{448.5}{3} = 149.5$

$q_{safe} = q_{ns} + \gamma' D_f = 149.5 + 8.01\times 1.5$

$= 161.5\ kN/m^2$

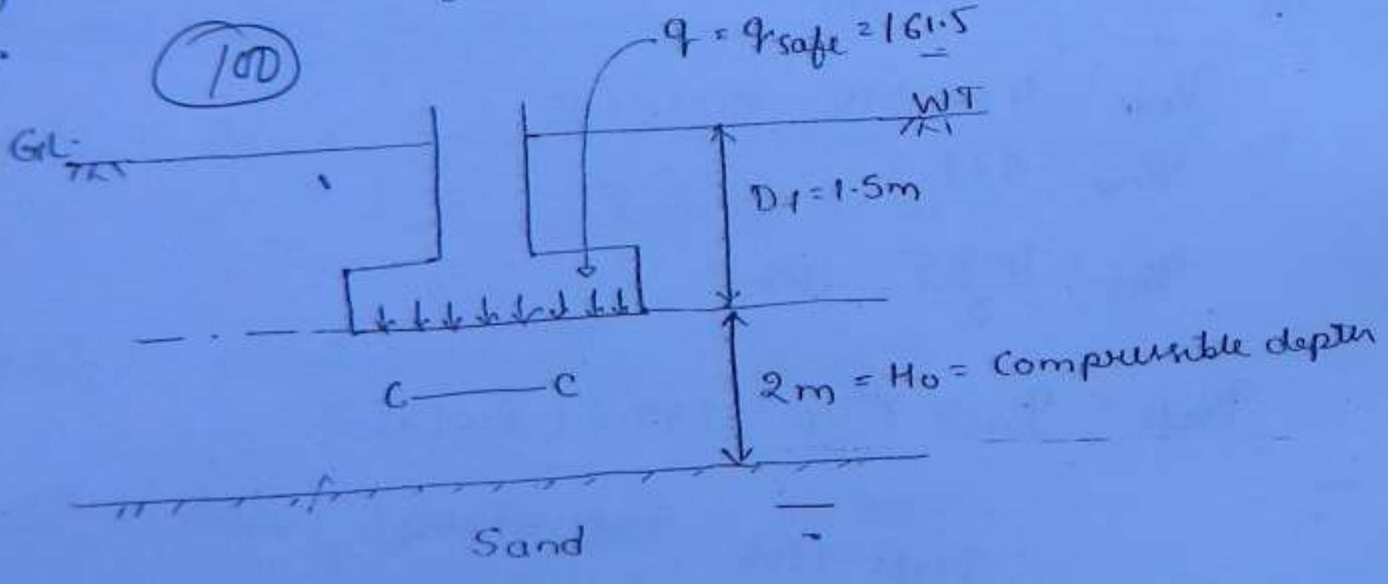


2+B-3

a soft clay with its base at a depth of 1.5m
rom the ground surface. If clay stratum 3.5 m thick and is
nderlay by a firm sand. Clay has L.L. of 30%, S.G.
2.7 and Degree of saturation = 100%, w_c = 40%,
nconfined compressive strength 100 kN/m^2 and $\phi = 0$. It is
nown that clay is normally consolidated then compute the
ettlement which will be resulted if load intensity =
afe bearing capacity is allowed at the base of footing.
atural w_c is close to G.L and for given condition
earing capacity factor are $N_c = 6.9$, $N_q = 1$, $N_\gamma = 0$.
F.O.S. = 3. Assume load distribution 2 vertical to 1
orizontal below footing.

ns.

100



$$\Delta H = \frac{C_c H_0}{1+e_0} \log\left[\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right]$$

$$C_c = 0.009(w_L - 10)$$

$$= 0.009(30 - 10) = 0.18$$

$q_{nu} = 439.065$

$q_{ns} = \frac{q_{nu}}{F.O.S} = \frac{439.065}{2.5} = 175.626$

$q_{safe} = q_{ns} + \bar{\sigma} = 175.626 + 12.285$

$= 187.911 \ kN/m^2.$

101

<u>Case-II</u> <u>when WT at 3m from GL.</u>

$R_\gamma^* = 0.5\left[1 + \frac{Z_\gamma}{B}\right] = 0.5\left[1 + \frac{1.5}{2.5}\right] = 0.8$

$R_q^* = 1$

$q_u = 0 + 18 \times 1.5 \times 20.3 \times 1 + 0.4 \times 18 \times 2.5 \times 19.7 \times 0.8$

$q_u = 831.78$

$q_{nu} = q_u - \gamma D_f = 831.78 - 18 \times 1.5 = 804.78$

$q_{ns} = \frac{q_{nu}}{F.OS} = \frac{804.73}{2.5} = 321.912$

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

$= 321.912 + 18 \times 1.5$

$= 348.912 \ KN/m^2$

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 should not be loaded immediately after driving the piles because while driving the piles remoulding of clay may occur and shear strength may be lost.

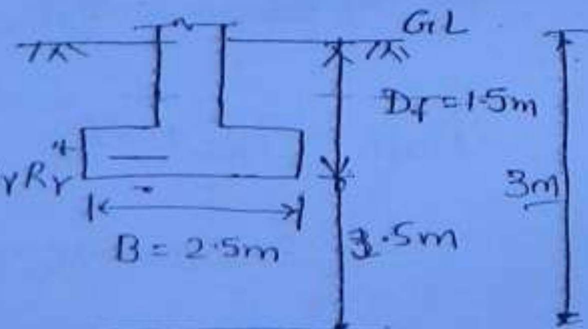
, The pile driving process should start from centre moving radially outward so as to minimise resisting of driving.

Qb. The subsoil at a building site consist of medium sand with $\gamma_{sat} = 18\ KN/m^3$. $C = 0$ $\phi = 32°$ and WT at GL.

2.5 m square footing is to be placed at 1.5 m below GL. Compute safe bearing capacity of footing.

What will be safe SBC if WT goes 3 m below GL. Given that for $\phi = 32°$ $N_q = 20.3$, $N_\gamma = 19.7$. F.O.S 2.5

Case-I When WT is at GL



$$q_u = 1.3 CN_c + \gamma D_f N_q \cdot R_q^* + 0.4 \gamma B N_\gamma R_\gamma^*$$

$R_q^* = 0.5$; $R_\gamma^* = 0.5$; $C = 0$

$$q_u = 0 + 18 \times 1.5 \times 20.3 \times \frac{1}{2} + 0.4 \times 18 \times 2.5 \times 19.7 \times \frac{1}{2}$$

$$q = 451.35\ KN/m^3$$

$$q_{ns} = \frac{148.5 + 26.4B}{3}$$

$$q_{safe} = 49.5 + 8.8B + 11 \times 1.5 = 8.8B + 66$$

$$q_{safe} \cdot B^2 = 150$$

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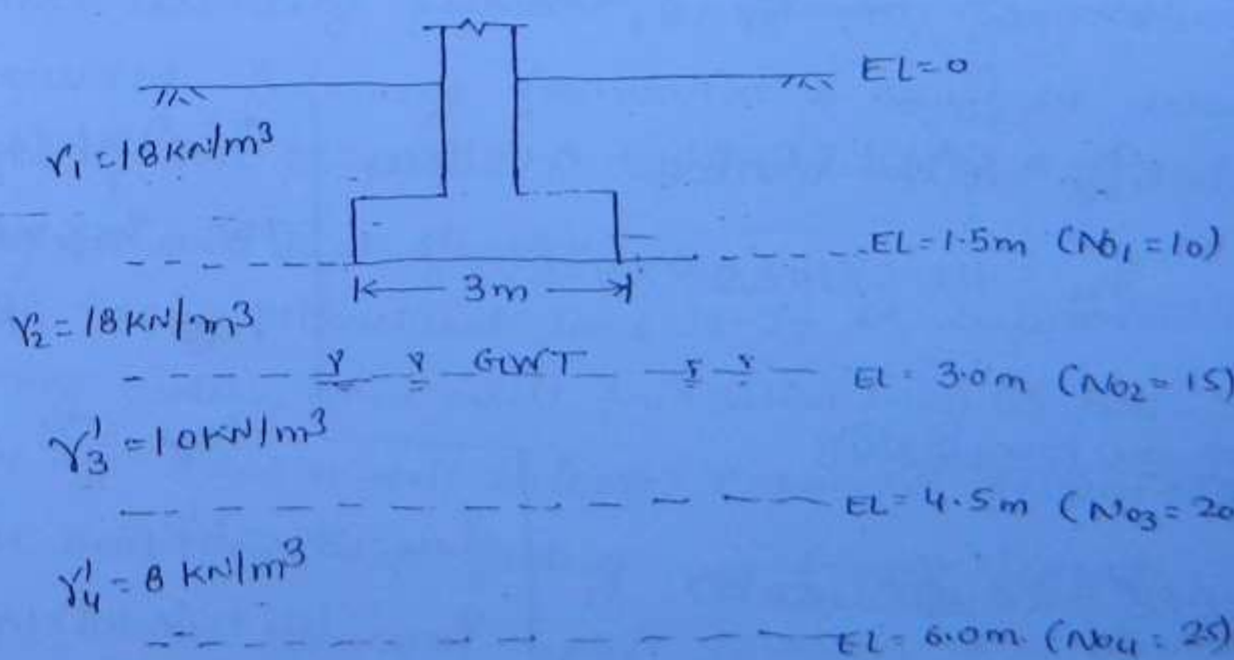
$$(8.8B + 66)B^2 = 150$$

$$\boxed{B = 1.385m}$$

Que. Using Pack 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



$$\Delta\sigma = \frac{q_{safe} \times B \times B}{(B+Z)^2}$$

$$= \frac{161.5 \times 2 \times 2}{3^2}$$

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$$\Delta\sigma = 71.78\ kN/m^2$$

$$H = \frac{0.18 \times 2.0}{1+1.08} \log_{10}\left(\frac{20.025 + 71.78}{20.025}\right)$$

$= 0.114 m$

$= 11.4 cm$

Square footing located at a depth of 1.5 m. from carries column Load of 150 kN. The soil is submerged ving effective unit weight of 11 kN/m³ and angle of ring resistance (ϕ) = 30°. Find the size of footing ng Terzaghi's Theory. Given FOS = 3, ϕ = 30° and ϕ = 30° N_q = 10, N_γ = 6, c = 0.

$$q_u = 1.3\cancel{cN_c} + \gamma' D_f N_q + 0.4\gamma' B N_\gamma$$

$$q_u = 11 \times 1.5 \times 10 + 0.4 \times 11 \times B \times 6.0$$

~~$\frac{q_u}{B^2}$ = 165 + 26.4B~~

~~165B² + 26.4B³ - 150 = 0~~

Q = Q safe Load

$150 = q_{safe} \times A$

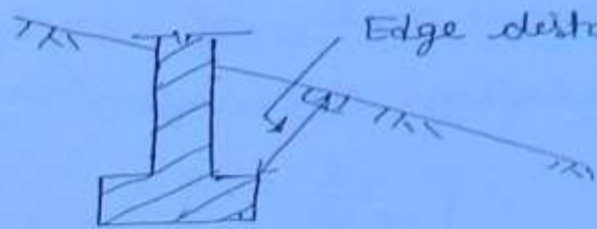
$q_{safe} \times B^2 = 150$

$$q_{safe} = \frac{150}{B^2}$$

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

$q_{nu} = 165 + 26.4B - 11 \times 1.5$

ground a minimum edge distance of 60cm in rocks and 90cm in soil should be provided.



(103)

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 are, 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 sub-structure and super structure.
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

$$q_{net\,safe} = 1.38(N-3)\left(\frac{B+0.3}{2B}\right)^2 S \cdot C_w \quad \text{— kN/m}^2$$

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S code recommends minimum depth of raft foundation 5m and for raft foundation Net safe bearing pressure is given as

$$q_{net\,safe} = 0.88\,N S C_w \quad \text{— kN/m}^2$$
$$\simeq 21\,N \quad (\text{for sand})$$

Mayerhoff equation.

:- safe bearing pressure in kN/m² is given as

$$q_{net\,safe} = 0.49\,N S C_w C_D \cdots \text{kN/m}^2 \quad \text{when } B < 1.2\text{m}$$

$$q_{net\,safe} = 0.32\,N\left(\frac{B+0.3}{B}\right)^2 S C_w \cdot C_D \cdots B \geq 1.2\text{m.}$$

here $C_w = 0.5\left(1 + \frac{D_w}{B}\right)$ $D_w \Rightarrow$ Depth of WT below found^n level.

$C_D = \left(1 + \frac{D_f}{B}\right)(\leq 2)$

S. code Recommendation :-

Minimum depth of foundation should be always eater than depth of frost action irrespective of aring capacity of soil.

Minimum depth of foundation should be always eater than depth of organic fill.

Depth of foundation > 30 cm. for single and double tory building and > 60 ~~mm~~ cm for multistory building

$$q_{a\,net} = 0.41\, N\, S\, C_\omega \quad kN/m^2$$

where N = Corrected average SPT number

S = permissible settlement of footing in mm.

C_ω = Water table (WT) correction factor

$$= 0.5\left[1 + \frac{D_\omega}{B + D_f}\right]$$

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where D_ω = Depth of WT below GL

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

$$q_{net\,safe} = 1.4\,(N-3)\left(\frac{B+0.3}{2B}\right)^2 S\, C_\omega C_D \quad \cdots \cdots \quad \frac{kN}{m^2}$$

where S = permissible total settlement in mm.

N = Corrected SPT no.

B = Width of footing

C_ω = WT correction factor $= 0.5\left[1 + \frac{D_\omega}{B}\right]$

D_ω = Depth of WT below base of foundation

C_d = Depth correction factor

$$C_d = 1 + \frac{D_f}{B} \quad (\le 2)$$

I.S. code equation :- IS code recommends use of Teng's equation without depth correction factor and with

N_o = Observed value of SPT no. then correct [value] for overburden pressure is given as

$$N_1 = N_o \times \frac{350}{\bar{\sigma} + 70}$$ 108

[where] $\bar{\sigma}$ = effective overburden pressure at the test level if test is ~~perfo~~ performed at depth D_f

$\bar{\sigma} = \gamma' D_f$ $\quad\quad \bar{\sigma} \not> 280\ kN/m^2$

[If] $\bar{\sigma} > 280\ kN/m^2$ then overburden pressure correction [is n]ot required.

[Di]latency correction :- If WT is present at test [lev]el or above the test level, then this correction is [req]uired. Corrected value for overburden pressure (N_1) [is] further corrected for dialatency.

$$\boxed{N_2 = 15 + \frac{1}{2}(N_1 - 15)}$$

The above corrections are applied in sequence as [men]tioned above.

If multilevel test is performed then final SPT no. [is] average of all the corrected SPT values.

[Pro]cedure to determine Net allowable bearing [pre]ssure / 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

(ii) Angle of shearing resistance
(i) Relative density
(iii) Allowable bearing pressure on the basis of settlement criteria
(v) Point Resistance of pipe.
) 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 75cm. The test is conducted at every 2m interval or at change of strata.

SPT Number (N) is defined as no of blows of hammer required for 300mm penetration of sampler. The penetration per blow of hammer is called 'set'. Usually test is performed in 3 stages 150mm 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.

(Ist Choice) Date / /

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 found^n of width B_f & is placed at depth D_1 & S_2 is the settlement of same found^n & is placed at depth D_2 then

$$\frac{S_2}{S_1} = \left[\frac{1 + \frac{2D_1}{B_f}}{1 + 2\frac{D_2}{B_f}}\right]^{1/2}$$

Standard penetration test

Plate load test

This Method was designed to determine the modulus of Subgrade reaction & 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

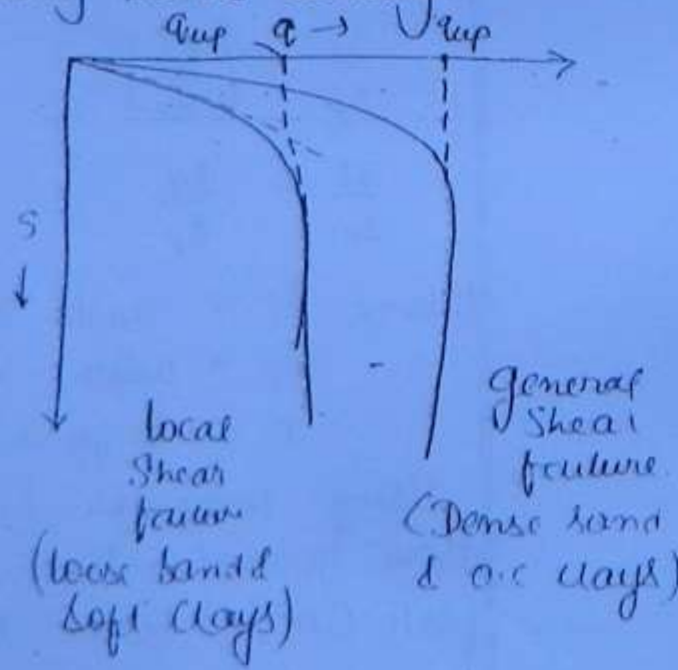
1. A rigid plate of size 30cm/45cm/60cm/75cm/90cm is used & may be circular or square. Usually smaller size plate is used for dense or stiff soils
2. A pit of dimension not less than 5 times dimesion of plate is excavated at same depth equal to depth of footing & plate is seated at the centre of pit. If W.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 ~~100~~ 120° to each other & average settlement is taken. The settlement v/s 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

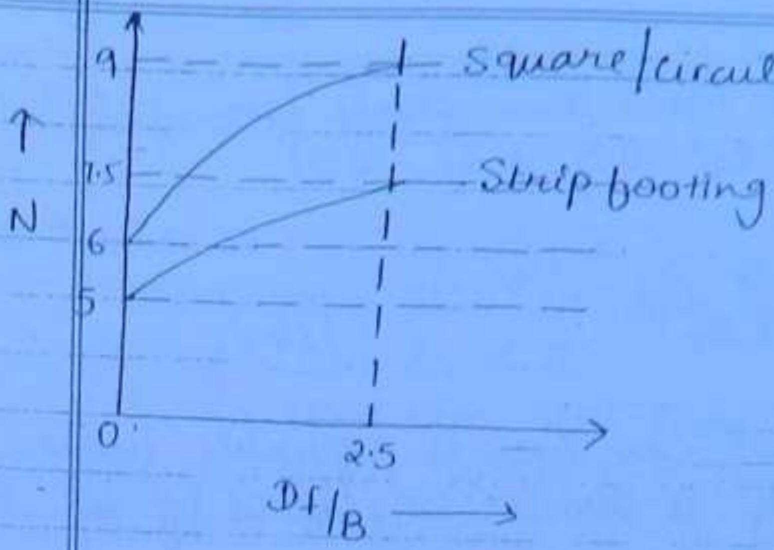
From curve q_{up} (ultimate B.C for plate) on shear criteria is found. And $q_{uf} = q_u$ (q_{uf} = ultimate B.C for footing) can be found as follows.

$q_{uf} = q_{up}$ for clays

$q_{uf} = \frac{B_f}{B_p} \times q_{up}$ for sands

(For clays B.C is independent of size)





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Meyorhoff 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

$$q_u = cN_c \cdot S_c \, d_c \, i_c + \gamma D_f N_q \cdot S_q \, d_q \, i_q + \frac{1}{2}\gamma B \cdot N_\gamma \, S_\gamma \cdot d_\gamma \, i_\gamma$$

$S_c, S_q, S_\gamma \rightarrow$ Shape Correction factor
$d_c, d_q, d_\gamma \rightarrow$ depth " "
$i_c, i_q, i_\gamma \rightarrow$ inclination " "

NOTE-1 I.S Code recommends use of Meyorhoff eq^n & general shear failure $\bar{c}$ additional W.T Correction factor.

2 Terzaghi has Considered general shear failure but if soil fails in local shear then modified ~~be~~ ~~factor~~ soil parameter should be used

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

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

If W.T Rises to G.L in Sands then nearly there is 50% decrease in bearing capacity ($R_q' = 0.5$ & $R_r' = 0.5$)

(2) For Clays ($\phi = 0$) $N_\gamma = 0$ $N_q = 1$

$q_u = CN_c + \gamma D_f$ (114)

$q_{nu} = CN_c$

The effect of W.T on CN_c & N_c is negligible hence net ultimate B.C of Clays is almost unaffected by the rise of W.T at G.L.

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

$q_{nu} = CN_c$

Where N_c is B.C factor & depends upon $\left(\frac{D_f}{B}\right)$ ratio.

(1) 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]

(2) When D_f/B is $0 < D_f/B < 2.5$

$N_c = 5 \times (1 + 0.2 D_f/B)$ 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

(3) 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 W.T is in zone 2

When water table at depth of Z_γ from the base of footing $0 \le Z_\gamma \le B$.

In this condition 3rd term of B.C capac eqn is affected. Hence either W.T correction factor (R_γ^*) should be used or eff. unit wt. of soil should be used in 3rd term.

$$R_\gamma^* = 0.5\left[1 + \frac{Z_\gamma}{B}\right]$$

When W.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 W.T is in zone 1

In 3rd term R_γ^* will be 0.5 or $\gamma_{effective}$ (γ') will be ~~shou~~ used & in 2nd term R_q^* ~~its~~ should be used or eff. unit wt. of zone 1 should be used.

$$R_q^* = 0.5\left[1 + \frac{Z_q}{D_f}\right]$$

In 1st term also c-ϕ 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

$$q_u = \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

W.T is present $\rightarrow$ $q_u = \gamma D_f N_q R_q^* + \frac{1}{2}\gamma B N_\gamma R_\gamma^*$

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 $\left(45-\frac{\phi}{2}\right)$ $\bar{c}$ the horizontal.

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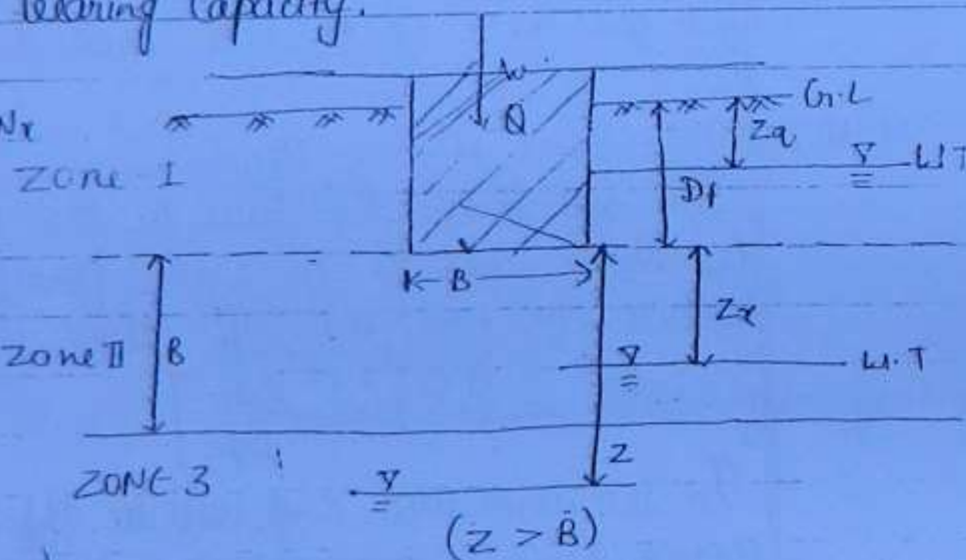
NOTE - For pure clays - $q_u = CN_c + \gamma D_f$ (from table values are)
Hence B.C is independent of size. Whereas in sandy soils B.C is directly proportion to size (B)

In order to support heavy load on clayey soils either raft / pile foundation should be preferred.

Effect of W.T in bearing capacity.

$q_u = \underset{I}{CN_c} + \underset{II}{\gamma D_f N_q} + \underset{III}{\frac{1}{2}\gamma B N_\gamma}$

Modified for the effect of W.T

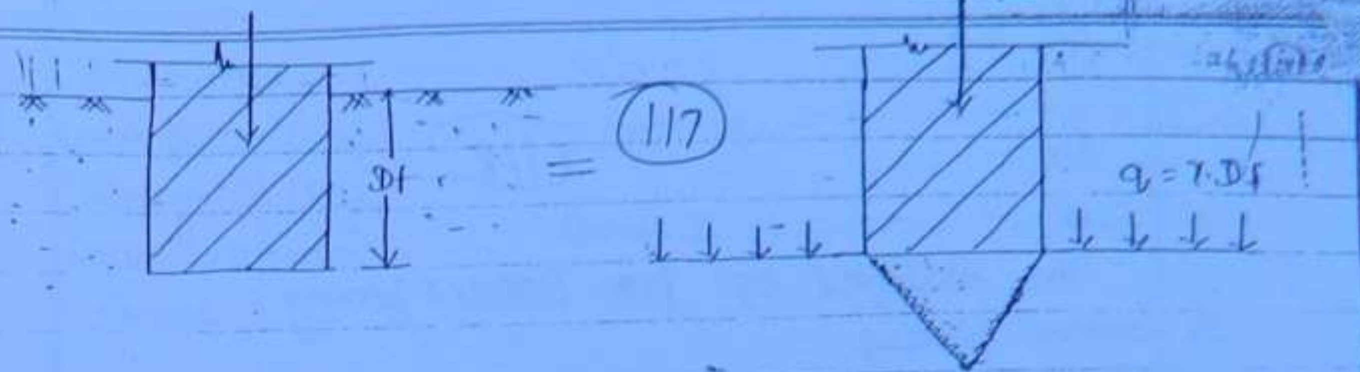


$q_u = c'N_c + \gamma D_f N_q R_q^* + \frac{1}{2}\gamma B N_\gamma R_\gamma^*$

R_q^* & R_γ^* are W.T correction factors
c' - effective cohesion with respect to W.T

SC I - When W.T is in zone 3 it means depth of W.T below the foundation is greater than width of foundation $(Z > B)$ under this condition there will be no effect in bearing capacity

(Ist Choice)



6. Terzaghi ~~igor~~ 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 prandtle theory. Prandtle 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 prandtle & terzaghi theory but bearing capacity factors are different. The ultimate bearing capacity of strip footing is $\boxed{q_u = cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma}$

Where N_c, N_q, N_γ are bearing capacity factors & depend upon friction angle of soil.

For pure clays ($\phi = 0$)

Prandtle B.C factor	Terzaghi B.C factor
$N_c = 5.14$	$N_c = 5.7$
$N_q = 1$	$N_q = 1$
$N_\gamma = 0$	$N_\gamma = 0$

$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$ — Influence value

$N_c = \tan\phi \; e^{\pi \tan\phi}$

$N_q =$

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$N_\gamma =$

The modified eqn for isolated footing :-

(i) Square footing

$$q_u = 1.3CN_c + \gamma D_f N_q + 0.4\,\gamma B N_\gamma$$

B = size of footing.

(ii) Circular footing

$$q_u = 1.3CN_c + \gamma D_f N_q + 0.3\,\gamma B N_\gamma$$

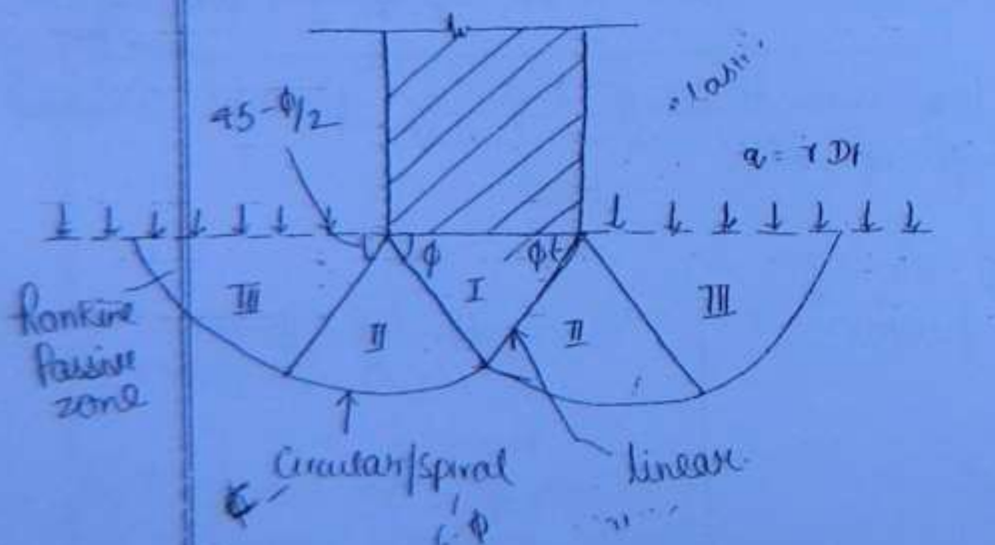
D → Dia of footing

(iii) Rectangular footing & Raft foundation

$$q_u = \left(1 + 0.3\frac{B}{L}\right) C N_c + \gamma D_f N_q + \frac{1}{2}\left[1 - 0.2\frac{B}{L}\right]\gamma B N_\gamma$$

Where B → width of footing (B < L)

L → length of footing



Terzaghi divided soil below the foundation in 3 zone

1. Central zone (Zone I) - It is called linear shear zone

Zone 1 remains in elastic state

Zone II - It is called Radial shear zone & at the time

Void ratio	$\not< 0.55$	$\not> 0.75$
unconfined Comp. Strength Of Clays	$> 100\ kN/m^2$	$< 80\ kN/m^2$
Load Settlement Curve	Curve I	Curve II

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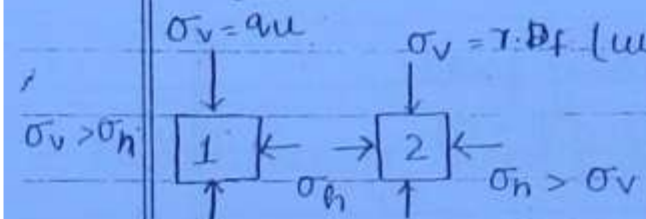
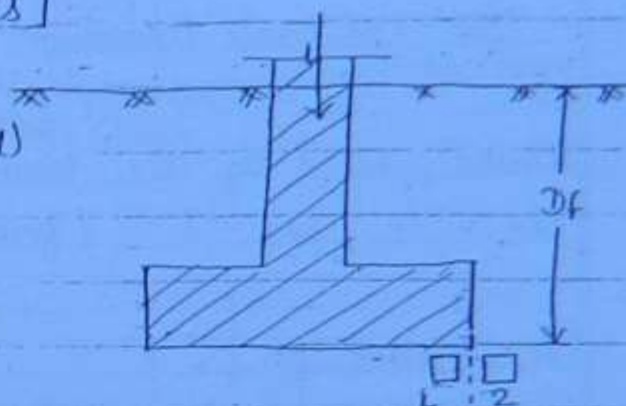
1 Rankine's theory. [ϕ Soils]

(ultimate bearing capacity)





σ_h for element 2 is in passive state $\sigma_h = K_p \cdot \gamma D_f = \frac{1+\sin\phi}{1-\sin\phi} \gamma D_f$

$$= \tan^2\left(45 + \frac{\phi}{2}\right) \gamma D_f$$

For element 1 σ_v is in passive state $= K_p \cdot \sigma_h = q_u = \sigma_v$ ($K_p \cdot K_p \gamma D_f$)

$$q_u = \tan^2\left(45+\frac{\phi}{2}\right)\tan^2\left(45+\frac{\phi}{2}\right) \gamma D_f$$

$$q_u = \tan^4 \cancel{\tan}\left(45+\frac{\phi}{2}\right) \gamma D_f$$

Rankine Consider the equillibrium 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.

$$q_u = \left[\frac{1+\sin\phi}{1-\sin\phi}\right]^2 \gamma D_f$$

$$q_u = \tan^4\left(45 + \frac{\phi}{2}\right) \gamma D_f$$

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Limitations :-

1. Not applicable for clays
2. At $D_f = 0$, $q_u = 0$ & is not practiable.
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

$$q_u = CN_c + \gamma D_f N_q$$

Where, N_c & N_q are bearing capacity factors.

$N_c = 4$, $N_q = 1$ (for clays) ($\phi = 0$)

Limitation :-

1. The base of footing is considered smooth.
2. The size of footing is not accounted

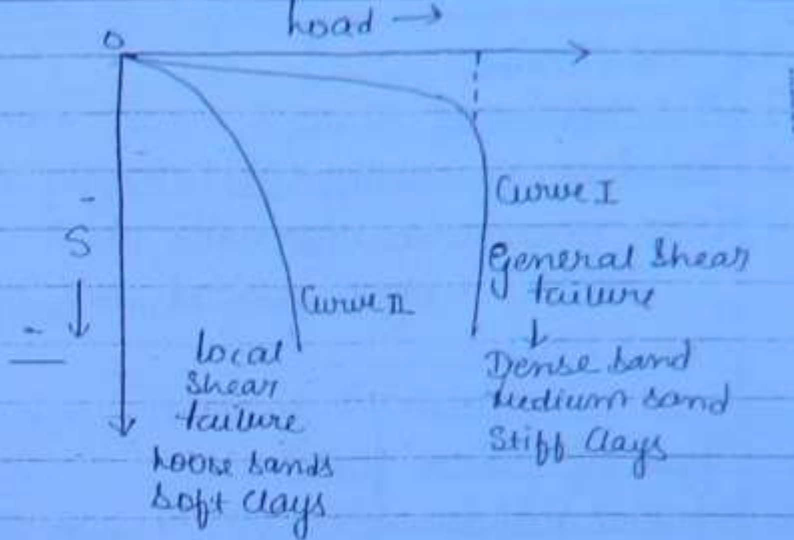
Jetlinius Method

- The foundation is resting on cohesive soil (c-soils only) & failure is assumed due to ~~slip~~ ~~under~~ 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.



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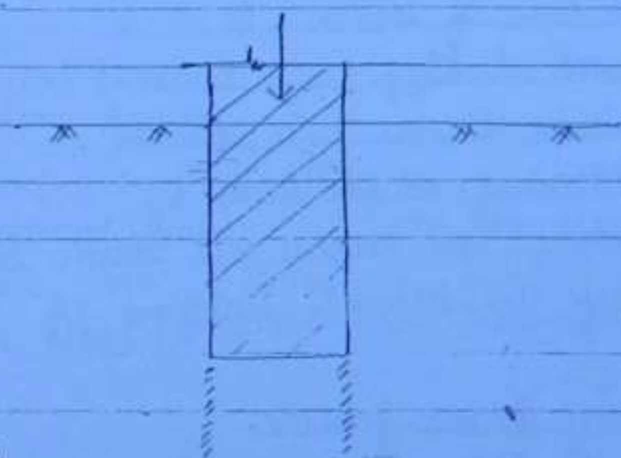
(iii) Punching Shear failure.



n Case of very loose soils
using deep footing or piles
e soil adjacent to the
oundation gets sheared hence
djaent soil is not fully
tressed & excessive settlements
re recorded w/o Change in characteristics of adjacent soil but
his failure is not Common in Shallow foundⁿ. There will
e no side bulding of soil.

Guidelines to diffⁿ beeⁿ general & local shear failure

	Parameter	G.S.F	L.S.F
1.	Friction angle	More than 36°	$\phi < 28°$
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)

Fields methods are more suitable for granular soils (dense sands).

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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 failure soil reaches into plastic stage & due to shear foundation may get tilted & bulging of soil may occurs from the sides.

(ii) Local shear failure

In Case of loose sands & soft Clays large settlement may occur below 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

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Methods to determine bearing Capacity

1. Analytical Methods

The properties of soil (c, ϕ) & characteristics of found^n are used. Analytical Methods are based on following 3 theories

Elastic theory
- Schleir's Mtd. (cohesive soils)

Classical earth Pressure theory
- Rankine Mtd [ϕ soils] (frictional soil)
- Pauker's Mtd [ϕ soils]
- Bell's Mtd. [c-ϕ soils]

Plastic theory
- Fellinius Mtd [c-soils]
- Prandtle Mtd. [c-ϕ both]
- Terzaghi Mtd [c-ϕ both]
- Skempton Mtd [c-ϕ]
- Meyorhoff Mtd [c-ϕ]
- Brinch Hansen [c-ϕ]
- I.S Code Mtd [c-ϕ]
- Veasics Mtd [c-ϕ]

2. Bearing Capacity from Codes.

Various building codes are published by codal agencies such as IRC, BIS, CPWD etc. which carry bearing capacity of Zonal soils. For rough computation & approximate analysis such values can be used

Net Safe bearing Capacity (q_{ns})

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

$F.O.S = 2 \text{ to } 3$

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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
  - ft / Mat
- Deep foundation ($D_f > B$)
  - Deep footing
  - Pile foundation
  - Well's foundation or Cassions
  - Pier foundation

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(Substructure)

FOUNDATION ENGINEERING

Normally foundation failure may be due to
(i) Shear failure or Shear Criteria
(ii) Settlement failure or Settlement Criteria

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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 in 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.

(q_g) 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)

(q_n) 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

(q_u) 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 which 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 react^n b/w the portion of soil mass is tangential to concentric smaller circle of radius ($r \sin \phi$) which is called Friction circle.
The inclination of resultant with the normal of failure surface is at an angle (ϕ).

Taylor's Method

Taylor defined a dimensionless parameter called stability No. which is given as

$$\boxed{S_n = \frac{C}{\gamma H_c} = \frac{C}{\gamma z F_c}} \quad —— (A)$$

z = ht. of slope

$S_n = \sin\beta \cos\beta$ [For clays]

F_c

$= [\tan\beta - \tan\phi] \cos^2\beta$ [for c-ϕ soils]

Using Taylor's chart stability No. is obtained for given value of ϕ & β.

Note: In Eq (A) γ_d or γ_{bulk} should be used when soil is dry or partially saturated also use undrained friction angle (ϕ_u) to determine S_n

If soil is fully submerged then use γ' (submerged unit wt) & ϕ' (eff friction angle)

If soil is saturated but not submerged such as due to capillary effect or sudden drawdown of water Table the γ_{sat} should be used & weighted friction angle (ϕ_w) should be used

$$\boxed{\phi_w = \frac{\gamma'}{\gamma_{sat}} \phi}$$

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mobilised unit cohesion for a slope of ht z

$$\frac{C}{\gamma H_c} = \frac{C}{\gamma H F_c}$$

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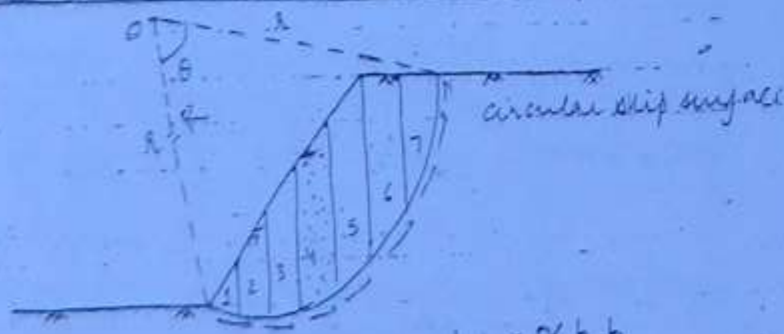
...mius
...126)
...φ analysis

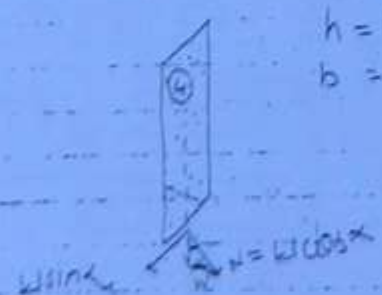
Swedish Circle Method or Mtd of Slices



$W = \gamma h b$

h = avg. height of slice

b = 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. The restoring moment is given by friction & cohesion about point O, whereas overturning moment is given by tangential component of wt of each slice.

The method of slice can be used for homogeneous or stratified soils & can also be used where seepage is taking place & pore pressure are present in

$$FOS = \frac{\text{Restoring moment}}{\text{overturning moment}} = \frac{M_R}{M_O}$$

$M_O = \Sigma T \cdot R$

ΣT is total tangential component of wt of soil mass of all slices

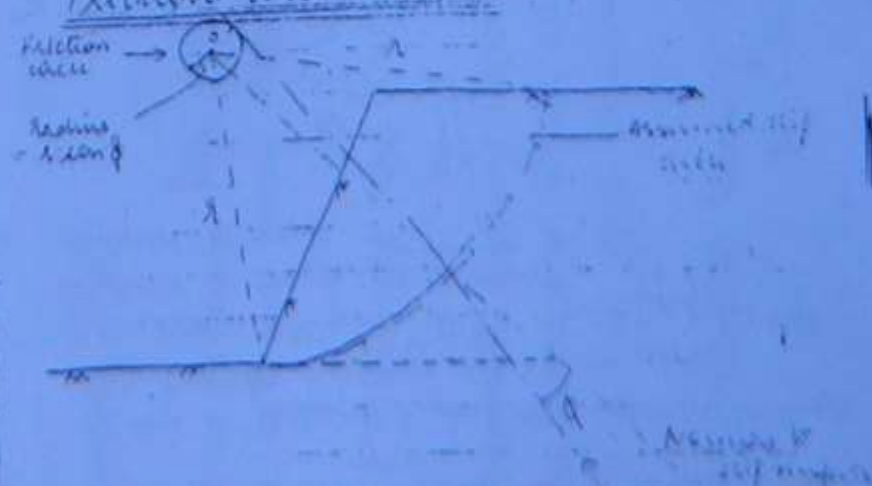
$$M_R = \underbrace{(c\,r\,\theta)}_{\text{length of arc (slip surface)}} \times r + (\Sigma N \tan\phi)\, r$$

$$FOC = \frac{c(r\theta) + \Sigma N \tan\phi}{\Sigma T} = \frac{cL + \Sigma W \cos\alpha \tan\phi}{\Sigma W \sin\alpha}$$

c = cohesion (in kN/m^2)

r = radius of slip surface

Friction Circle Method



R → Resultant reaction on slip surface

$S_n = \sigma' \sin\beta \cos\beta = \frac{\sin 2\beta}{2}$

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:-

1) Immediately after construction
2) Long time after construction

The significance is w.r.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 (NCC) & fine silts total stress analysis for short term and effective stress analysis for long term.

(3) For OCC which behave similar to dense sand, effective stress analysis should be used for short or 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 purely cohesive 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 earthen dams due to sudden draw down and seepage

Note:- For total stress analysis (TSA) results of U-U Test are used & for effective stress analysis (ESA) results of CD tests are used

γ = avg total unit weight of soil above the slip surface.

For eg:- if h_1 depth above water table

h_2 depth below WT & above slip 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 $h = Z$

$$FOS = \left(1 - \frac{\gamma_w}{\gamma_{sat}}\right)\frac{\tan \phi'}{\tan \beta} \qquad \left(1 - \frac{h}{z}\frac{\gamma_w}{\gamma_1}\right)\frac{\tan \phi'}{\tan \beta}$$

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

$$= \frac{\gamma'}{\gamma_{sat}} \cdot \frac{\tan \phi'}{\tan \beta}$$

$$= \frac{1}{2}\frac{\tan \phi'}{\tan \beta} \qquad \left[\gamma' = \tfrac{1}{2}\gamma_{sat}\right]$$

It means under steady state seepage at or parallel to ground level or under submerged condition upto GL

* FOS reduced by 50%.

$$Sn = \frac{c}{\ldots} = \frac{c}{\ldots}$$

It means under dry condition • If FOS is less than 2 then for same sand on submergence FOS will reduce by 50% hence it may become unsafe. i.e. Under dry condition desired FOS is 2.

Case 3: Infinite slope in cohesive soil

$$FOS = \frac{\text{shear strength}}{\text{shear stress}} = \frac{s}{\tau}$$

$$= \frac{c + \sigma \tan \phi}{\gamma z \cos\beta \sin\beta} \qquad [\phi = 0 \text{ for clay}]$$

$$FOS = \frac{c}{\gamma z \sin\beta\cos\beta}$$

$$FOS = \frac{H_c}{z}$$

$$= \frac{c}{\gamma z \sin\beta \cos\beta}$$

$$\boxed{\frac{c}{\gamma H_c} = \sin\beta \cdot \cos\beta = S_n}$$ — Taylor's stability No. [for clay]

max^m theoretical value of $S_n = 0.5$

however max^m practical value of $S_n = 0.261$

Case 2: FOS w.r.t. friction

(i) $FOS = \frac{\tan\delta}{\tan\beta}$

where, δ is friction angle
β is slope angle

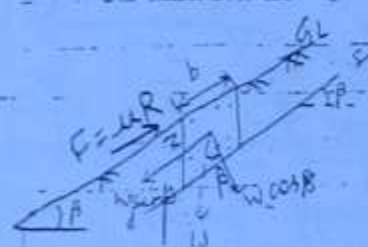
(ii) $FOS = \frac{\tan\delta}{\tan\delta_m}$

where,
δ_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 condition $(0 \le S < 100)$



The failure / sliding is due to tangential component of width along the slip surface & resistance is equal to frictional force.

$$FOS = \frac{\text{Restoring force}}{\text{Sliding force}}$$

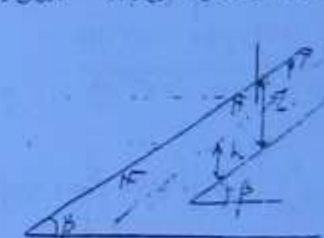
$$= \frac{\mu R}{W\sin\beta}$$

$$= \frac{\mu.(W\cos\beta)}{W\sin\beta}$$

$$FOS = \frac{\tan\delta}{\tan\beta}$$

ϕ = friction angle
β = slope angle

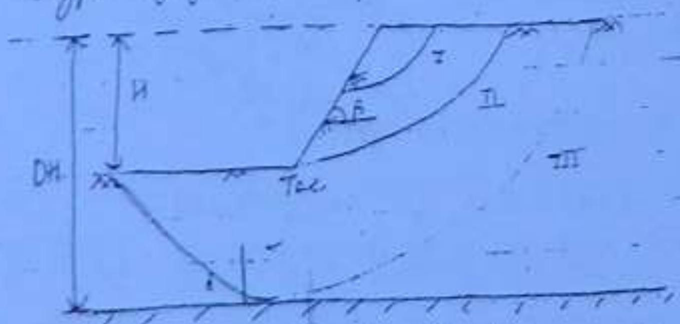
Case 2: If water table is present above the slip surface at a ht H parallel to ground level and soil is cohesionless.



$$FOS = \left(1 - \frac{h}{z}\frac{\gamma_w}{\gamma}\right)\frac{\tan\delta'}{\tan\beta}$$

where
δ' : effective friction angle
β = slope angle

Types of failure planes :-



Hard strata

I - face failure / slope failure $\beta = high$

II - Toe failure $\beta > 53°$

III - Base failure

FACE FAILURE :

It occurs when angle of slope (β) is very high and soil in lower part close to toe is strong and in upper part soil is weak.

TOE FAILURE :

It occurs when slope is steep ($\beta > 53°$) and soil is homogeneous in upper & lower parts.

BASE FAILURE :

Failure surface passes below the toe over some hard strata. It occurs when soil below the toe is relatively weak and

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above the toe soil is strong & stiff.

If H is ht of slope and DH is depth of lowest point of failure surface then D is called depth factor.

Factor of Safety (FOS) :

$$FOS = \frac{\text{Shear strength of soil}}{\text{shear stress developed on slip surface}}$$

Case 1 : FOS w.r.t cohesion

(i) $FOS = \frac{H_c}{z}$

where,

H_c = critical height

$= \frac{4c}{\gamma} \tan\left(45° + \frac{\phi}{2}\right)$

for pure clays $\phi = 0°$

z = actual depth of cut

(ii) $FOS = \frac{c}{c_m}$

where,

c = unit cohesion

c_m = mobilised or developed cohesion

Net stress at top of bottom layer = $70.56 - 42 = 28.56$

" " bottom of " " $= 70.56 + 19.74$

$= 90.30$

$Pa_1 = -23.1 \times \frac{1}{2} \times 2.89 \times 1 = -33.379$ acts at $6 + 3.11 + \frac{2}{3} \times 2.89$

$= 11.031\,m$

$Pa_2 = 24.9 \times \frac{1}{2} \times 3.11 \times 1 = 38.72$ acts at $6 + \frac{3.11}{3} = 7.04$

$Pa_3 = 28.56 \times 6 = 171.36$ acts at $3\,m$

$Pa_4 = \frac{1}{2} \times 61.74 \times 6 = 185.22$ acts at $2\,m$ from

Total = 361.921 kN/m

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$$\bar{H} = \frac{-33.379 \times 11.04 + 38.72 \times 7.04 + 171.36 \times 3 + 185.22 \times 2}{361.921}$$

$= 2.18\,m$

Imp for Gate

Stability Of Earth Slopes

Forces responsible for failure :-

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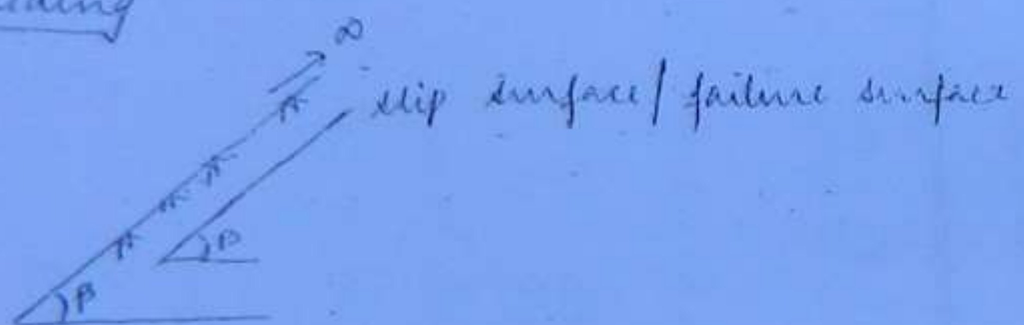
1. gravitational force
2. seepage force "
3. Sudden drawdown of water table
4. Earthquake force
5. Erosion due to water
6. Excavation near the soil mass

Types of Earth slopes :-

1. Infinite Slope

It represents surface of semi-infinite inclined soilmass 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 [illegible] γ = 21 [illegible] $\phi = 30^\circ$ height (H) = 5 m & the bar is connected at [illegible] below [illegible]

$$K_a = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} \quad \& \quad K_p = \frac{1+\sin 30^\circ}{1-\sin 30^\circ} =$$

$$K_a = 1/3 \qquad K_p = 3$$

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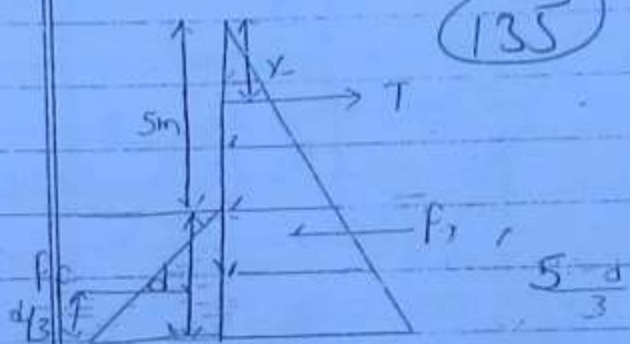

$$\Sigma F_x = 0$$

$$T + P_p - P_a = 0$$

$$T + \frac{1}{2}\, 3 \times 21 \times d^2 - \frac{1}{2} \times \frac{1}{3} \times 21\,[5+d]^2 = 0$$

$$T + 31.5\,d^2 - 3.5\,(5+d)^2 = 0$$

$$T + 31.5d^2 - 3.5(25 + d^2 + 10d) = 0$$

$$T + 31.5d^2 - 87.5 - 3.5d^2 - 35d = 0$$

$$28d^2 - 35d + T - 87.5 = 0$$

$$\Sigma M_B = 0$$

$$T \times (5+d-1) + P_p \times \frac{d}{3} - P_a \times \frac{5+d}{3} = 0$$

$$T \times (4+d) + \frac{1}{2} K_p \gamma d^2 \times \frac{d}{3} - \frac{1}{2} K_a \gamma (5+d)^2 \left(\frac{5+d}{3}\right) = 0$$

$$4T + Td + 10.5d^3 - 1.167\,(5+d)^3 = 0$$

~~4T + Td + 9.33d^3 - 145.85 = 0~~

A retaining wall 12m ht retains backfill $\bar{c}$ vertical smooth surface. The top 6m soil has $\gamma = 24 kN/m^3$, $c = 20 kN/m^2$, $\phi = 30°$ whereas bottom ... $\gamma = 21 kN/m^3$ ... plot active earth pressure ...

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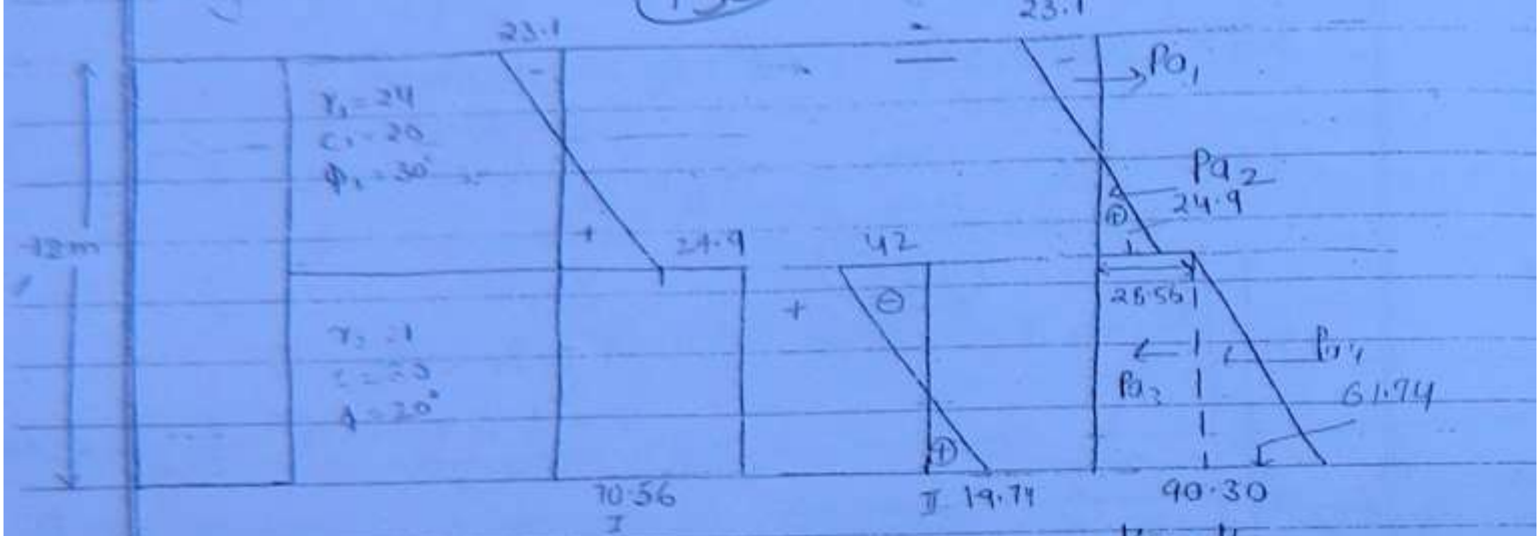


$Ka_1 = 1/3$

$$Z_{c_1} = \frac{2C_1}{\gamma_1} \tan\left(45° + \frac{\phi_1}{2}\right) = 2.89m$$

$Ka_2 = 0.49$

$$Z_{c_2} = \frac{2C_2}{\gamma_2} \tan\left(45° + \frac{\phi_2}{2}\right) = 4.08m$$

At top of top layer $Pa = 2C_1\sqrt{Ka_1} = 2 \times 20\sqrt{1/3} = -23.1$

At bottom of top layer $Pa = ka_1\gamma_1 H_1 - 2C_1\sqrt{ka_1}$

$$= \frac{1}{3} \times 24 \times 6 - 2 \times 20\sqrt{1/3} = 24.9$$

Top layer will be treated as surcharge over bottom layer

Surcharge $q = \gamma_1 H_1 = 24 \times 6 = 144 \; kN/m^2$

$Pa = ka_2 q = 0.49 \times 144 = 70.56 \; kN/m^2$

In diagram II

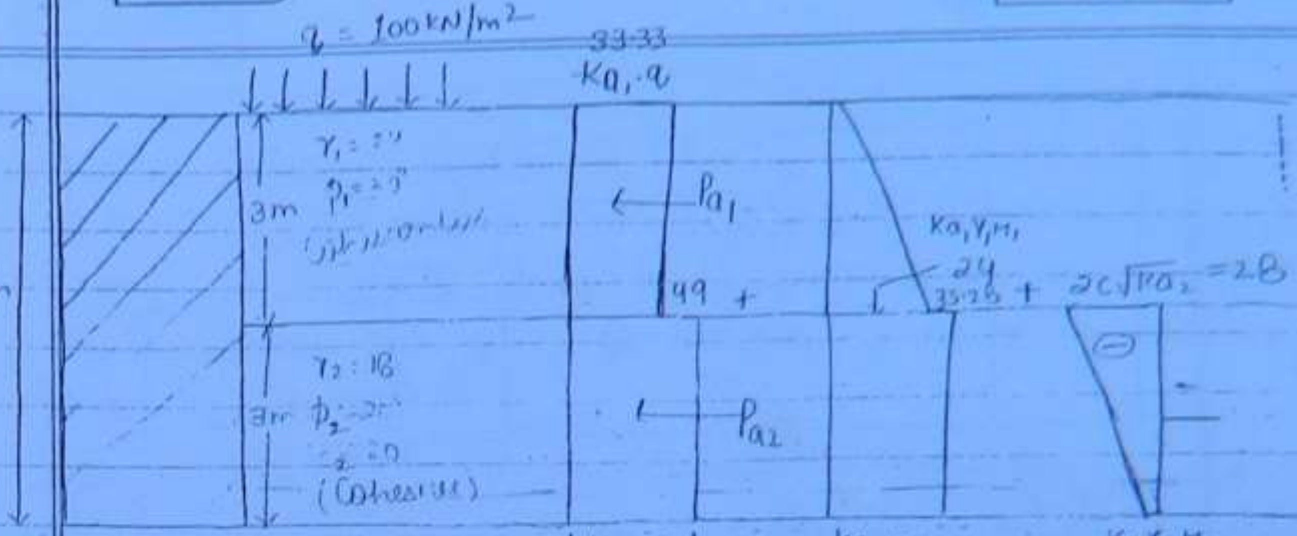
Top layer $= 2C_2\sqrt{ka_2} = 2 \times 30\sqrt{0.49}$

$= 42$

bottom $\quad k a_2 \gamma_2 H_2 - 2C_2\sqrt{ka_2}$

$= 0.49 \times 21 \times 6 - 42$

$= 19.74$



$$K_{a_1} = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1}{3}$$

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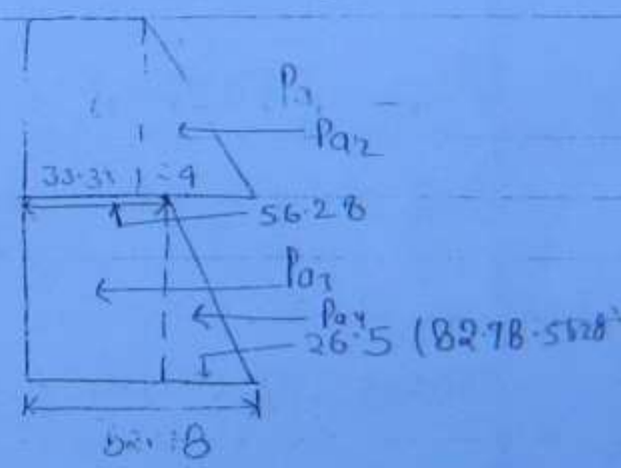
$$K_{a_2} = \frac{1-\sin 20°}{1+\sin 20°} = 0.49$$

$$Z_c = \frac{2c_2}{\gamma_2} \tan\left(45 + \frac{\phi_2}{2}\right) = 3.17\text{m}$$

Let Stress on top of Clay

$\cancel{P_{a_1}} = \cancel{33.33 \times 3} + \cancel{\frac{1}{2} \times 24 \times 3}$

$= 49 + 35.28 - 28 = 56.28$



Let Stress on bottom of Clay

$49 + 35.28 - 15$

$= 82.78$

$P_{a_1} = 30.33 \times 3 = 90.99 = 10$ kN/m acts at 4.5 from base

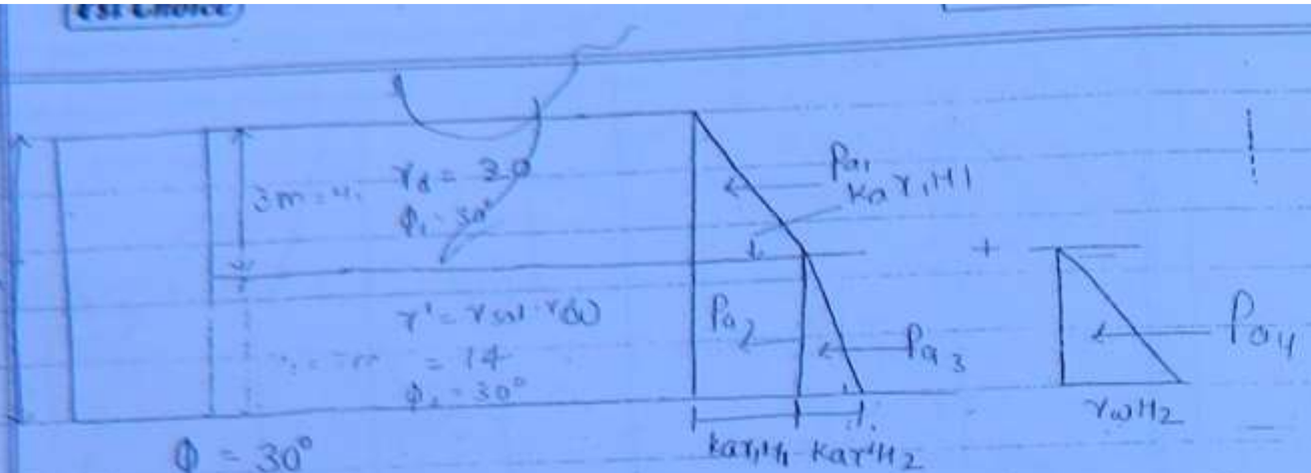
$P_{a_2} = \frac{1}{2} \times 24 \times 3 = 36$ acts at 4 m from base

$P_{a_3} = 56.28 \times 3 = 168.84$ acts at 1.5m

$P_{a_4} = \frac{1}{2} \times 26.5 \times 3 = 39.75$ kN/m act at 1m

Total $P_a = 335.58$

$\bar{H} = 2.57$m



$\phi = 30°$

$K_a = \dfrac{1-\sin\phi}{1+\sin\phi} = \dfrac{1}{3}$

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$Pa_1 = \frac{1}{2} K_a \gamma_1 H_1^2 = \frac{1}{2} \times \frac{1}{3} \times 20 \times 3^2$

$= 30\ kN/m$ acts at 4m from base

$Pa_2 = K_a \gamma_1 H_1 H_2 = \frac{1}{3} \times 20 \times 3 \times 3 = 60\ kN/m$ act at 1.5 m from base

$Pa_3 = \frac{1}{2} K_a \gamma' H_2^2 = \frac{1}{2} \times \frac{1}{3} \times 14 \times 3^2 = 21\ kN/m$ act at 1m from base

$Pa_4 = \frac{1}{2} \gamma_w H_2^2 = \frac{1}{2} \times 10 \times 3^2 = 45\ kN/m$ act at 1m from base

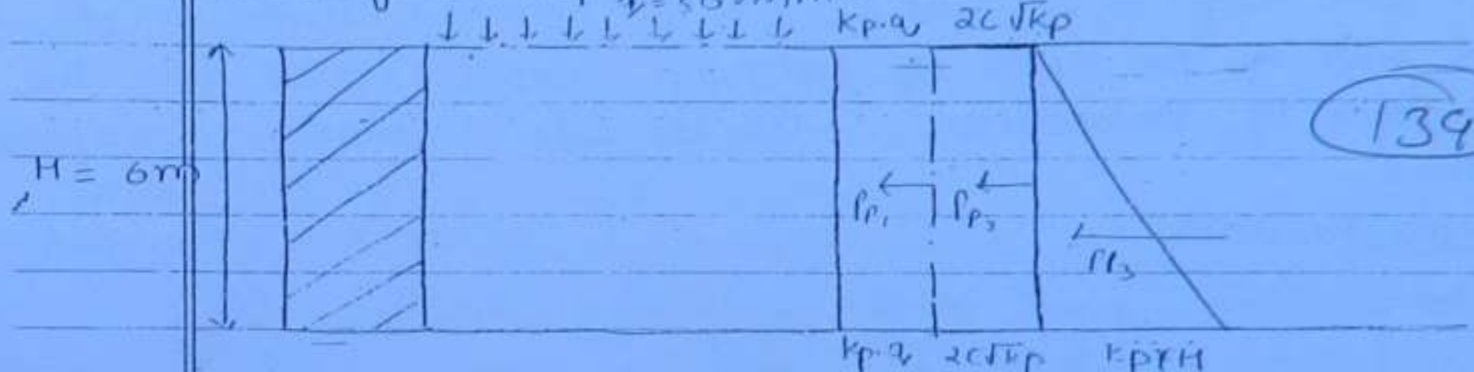
Total $P_P = 156\ kN/m$

$\bar{H} = \dfrac{30 \times 4 + 60 \times 1.5 + 21 \times 1 + 45 \times 1}{156}$

$= 1.77$ m from base.

Ques A retaining wall 6m high retains sand c $\phi = 30°$ & $\gamma = 24 kN/m^3$ upto depth of 3m from top. From 3m to 6m material is cohesive c is 20 kN/m² $\phi = 20°$ & $\gamma = 18 kN/m^3$. A uniform surcharge of 100 kN/m² also act on the top surface. Determine the total lateral pressure acting on the wall & its pt of application.

Ques (Passive) A retaining wall 6m height $\bar{c}$ a smooth vertical b pushed against a soil mass having $c' = 40 kPa$, ϕ' = & $\gamma = 19 \, kN/m^3$. Using Rankine's theory compute th pressure & pt. of application of resultant thrust the horizontal soil surface carries a uniform su load of $50 \, kN/m^2$



$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = 1.7$$

$$q = 50 \, kN/m^2$$

$P_{P_1} = K_p \cdot q \, H = 510 \, kN/m$ acts at 3m from

$P_{P_2} = 2c\sqrt{K_p} \cdot H = 625.84 \, kN/m$ acts at 3m fro

$P_{P_3} = \frac{1}{2} K_p \gamma H^2 = 1.7 \times 19 \times 6^2 = 581.4$ ac

2m from

$$\text{Total } P_P = P_{P_1} + P_{P_2} + P_{P_3}$$

$$= 1717.24 \, kN/m$$

$$\bar{H} = \frac{510 \times 3 + 625.84 \times 3 + 581.4 \times 2}{1717.24} = 2.66$$

Ques- A retaining wall 6m high retains sand with frict 30° & $\gamma = 24 \, kN/m^3$ (γ_{sat}). The top 3m depth is unit wt $20 \, kN/m^3$ & bottom 3m is submerged. D total active thrust on the wall per m length line of action. Assume there is no water on o side of wall

Net pressure at B $= P_p - P_a$

$$q + \gamma d + 2c - \gamma d + 2c$$

$$q + 4c$$

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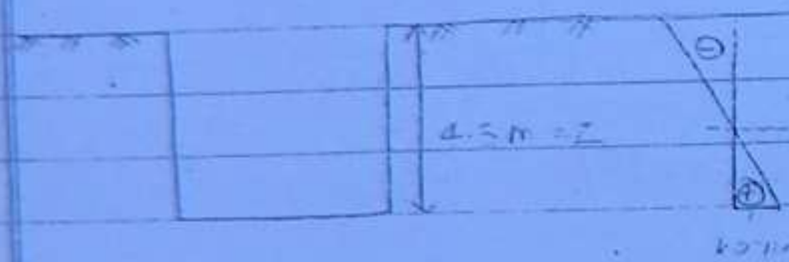
Ques Determine the stresses at the top & bottom of a vertical cut at 4.5 m deep in a soil $\bar{c}$ $\phi = 16°$, $c = 19.1\ kN/m^2$, $\gamma = 18.5\ kN/m^3$. What will be the depth of potential cracks & also find the max. depth of excavation $\underline{c}$ can be left unsupported without failure.

$c = 19.1\ kN/m^2$

$\gamma = 18.5\ kN/m^3$

$\phi = 16°$

Sol Active Condition



Depth of $Z_c = \frac{2c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right)$

$$= \frac{2 \times 19.1}{18.5} \tan\left(45 + \frac{16}{2}\right)$$

$$Z_c = 2.740$$

Max. ht. $\underline{c}$ can be left unsupported $= H_c = 2Z_c$

$$= 5.4 m$$

$$F.O.S = 1.2 = \frac{H_c}{H} = \frac{5.4}{4.5}$$

$$P_a = -2c\sqrt{K_a}$$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.57$$

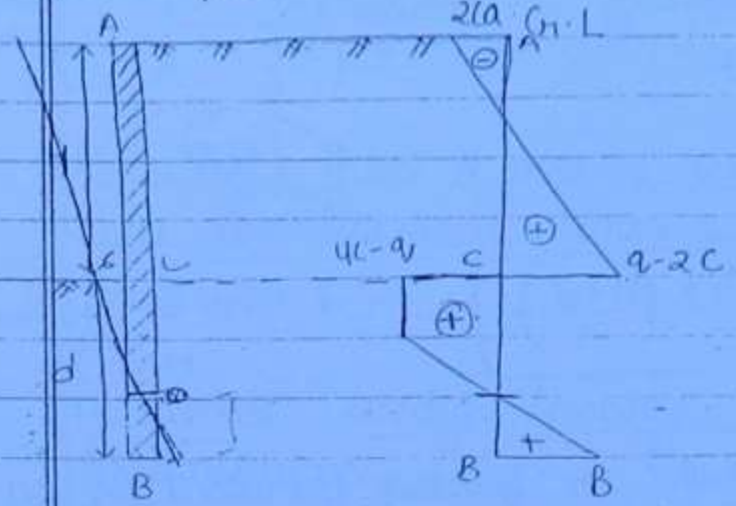
(top) $P_a = -2 \times 19.1 \times \sqrt{0.57} = -28.54$

(base) $P_a = K_a \gamma H - 2c\sqrt{K_a}$

$$0.57 \times 18.5 \times 4.5 - 2 \times 19.1 \sqrt{0.57}$$

$$= 18.46\ kN/m^3$$

Sheet pile wall in Cohesive soil



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Right face AO - In active state & OB in passive state

* Left face CO - passive state & left face OB in active state

For pure Clays $\phi = 0$

$K_a = 1$

$P_a = K_a \gamma . z - 2c\sqrt{K_a}$

$P_{a_A} = -2c$

P_{a_C} (on right face) $= 1 \cdot \gamma H - 2c\sqrt{1}$ $\quad [\gamma H = q]$

$= \gamma H - 2c$

$= q - 2c$

P_{p_C} (on left face) $= K_p \cdot \gamma \cdot z + 2c\sqrt{K_p}$

$K_p = 1 \quad \& \quad z = 0$

P_{p_C} (left) $= 2c$

Net pressure at C = P_p - Acti P_a

$2c - (q - 2c) = 4c - q$

P_B (right face) $P_p = K_p \cdot \gamma \cdot (H+d) + 2c\sqrt{K_p}$

$\gamma H + \gamma d + 2c$

$q + \gamma d + 2c$

P_B (left face P_a - $K_a \gamma d - 2c\sqrt{K_a}$

$\gamma d - 2c$

If F.O.S against overturning F then

$$F.O.S(F) = \frac{M_R}{M_o}$$

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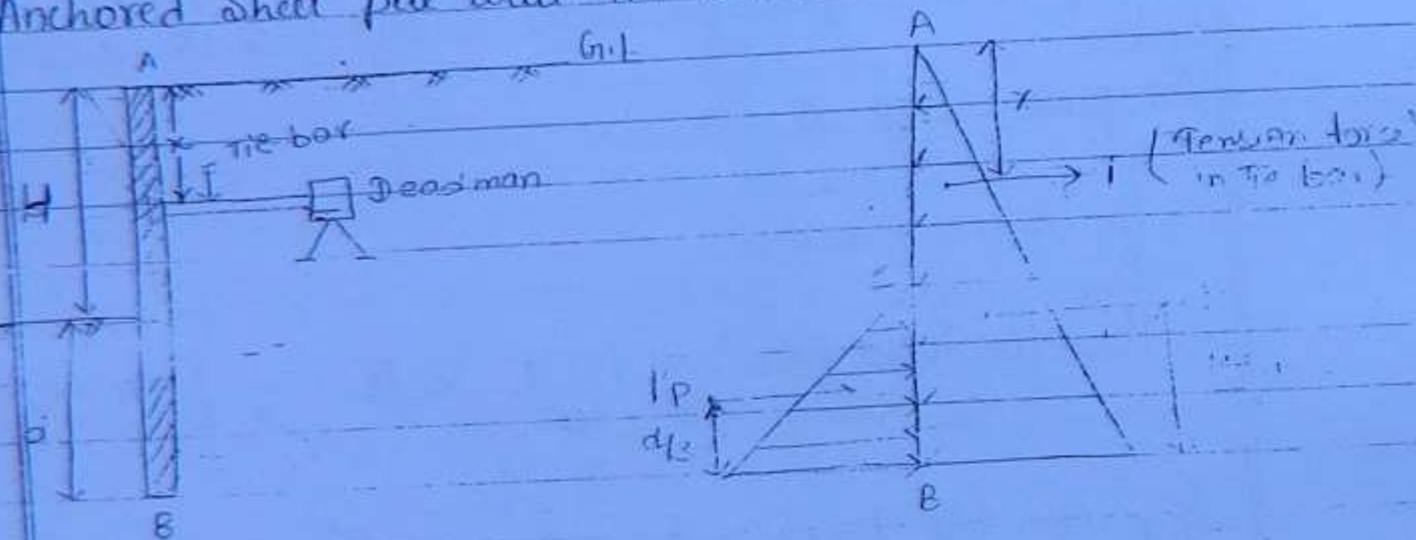
then $\frac{M_R}{F} = M_o$

$$\boxed{\frac{P_P \times d/3}{F} = P_a \times \left(\frac{H+d}{3}\right)}$$

(d can be found from this)

The depth computed should be further increased by 20 to 25%.

Anchored Sheet pile wall in Cohesionless Soil



$$\Sigma f_x = 0$$

$$T + P_P - P_a = 0$$

$$T + \frac{1}{2} K_P \gamma d^2 - \frac{1}{2} K_a \gamma (H+d)^2 = 0 \quad \text{——} ①$$

T - Tension force ~~present~~ per unit length of tie bar

P_a - Total active thrust per unit length

P_P - Passive thrust per unit length

$$\Sigma M_B = 0$$

$$P_P \times \frac{d}{3} + T \times (H+d-x) - P_a \times \left(\frac{H+d}{3}\right) = 0 \quad \text{——} ②$$

From 1 & 2 we T & d can be found.

1st Choice

Date / /

Total passive Pressure $P_P = P_{P_1} + P_{P_2}$

$P_P = \frac{1}{2} K_P \gamma H^2 + 2CH\sqrt{K_P}$

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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
(ii) The backfill is dry, homogenous, Cohesionless & isotropic
(iii) The back of wall is assumed rough & may be vertical or inclined.
(iv) Failure is assumed on a plane & is called rupture surface & failure is 2-dimensional.
(v) Wedge acts as a rigid body & friction is distributed uniformly on the rupture surface
(vi) The position & dirn of the resultant thrust is known & acts at $\frac{H}{3}$ from base at an angle δ with 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 ∴ for passive condition this theory is less accurate & errors get further ↑ with ↑ in friction angle

Special Case - For pure clays $\phi = 0$

$$Z_c = \frac{2c}{\gamma}$$

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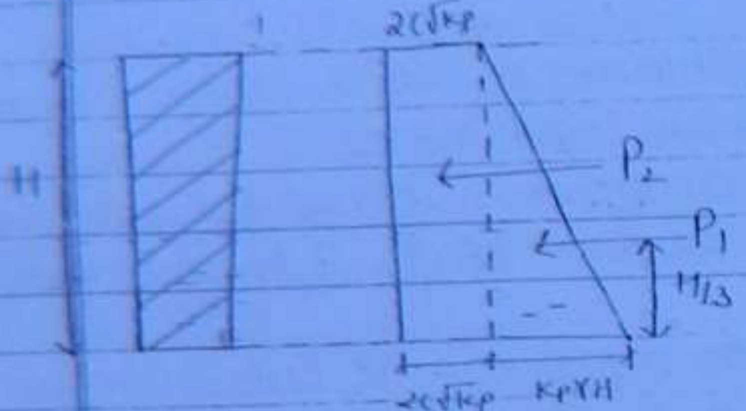
$$H_c = \frac{4c}{\gamma}$$

Passive pressure on Cohesive Soils



- In passive state $\sigma_h > \sigma_v$

$$\sigma_h \to \sigma_1 \to (P_p), \quad \sigma_v \to \sigma_3$$

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$P_p = \gamma \cdot z \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$P_p = K_p \gamma \cdot z + 2c\sqrt{K_p}$$

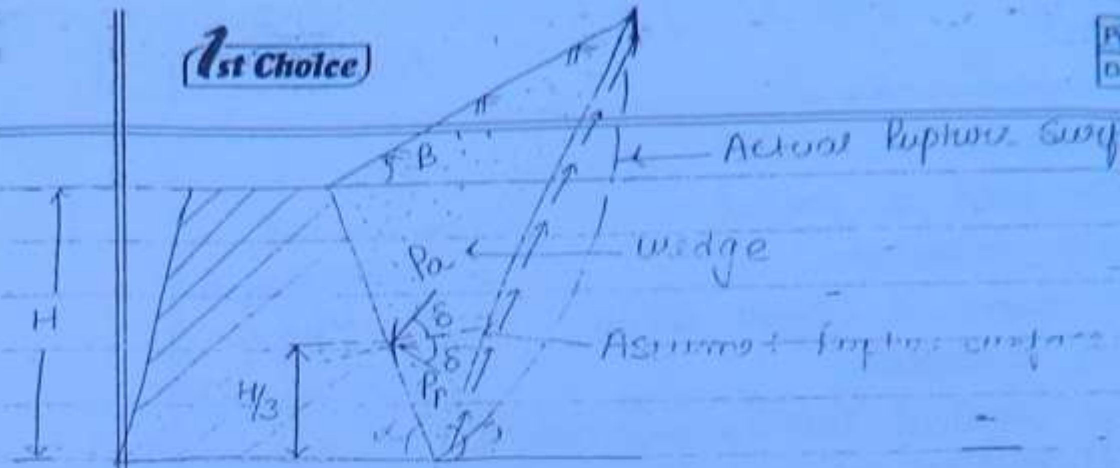
If $z = 0$ $\quad P_p \quad 2c\sqrt{K_p}$

$z = H$ $\quad P_p \quad K_p \gamma \cdot H + 2c\sqrt{K_p}$

Total passive force per unit length of wall

$P_{P_1} = \frac{1}{2} K_p \gamma H^2$ acts at $\frac{H}{3}$ from base

$P_{P_2} = 2cH\sqrt{K_p}$ acts at $\frac{H}{2}$ from base



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 b/w wall & soil
& α = $\frac{\phi}{3} < \delta < \frac{3\phi}{2}$

$$\delta \approx \phi$$

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> Case If wall is vertical ($\alpha = 90°$), back fill is horizontal ($\beta = 0°$) friction angle of wall is equal to friction angle of soil then ($\delta = \phi$) then

$K_a = \frac{\cos\phi}{[1+\sqrt{2}\sin\phi]^2}$; $K_p = \frac{\cos\phi}{[1-\sqrt{2}\sin\phi]^2}$

1st Choice

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Sheet pile walls

These are used to protect :-

(i) Water front structure

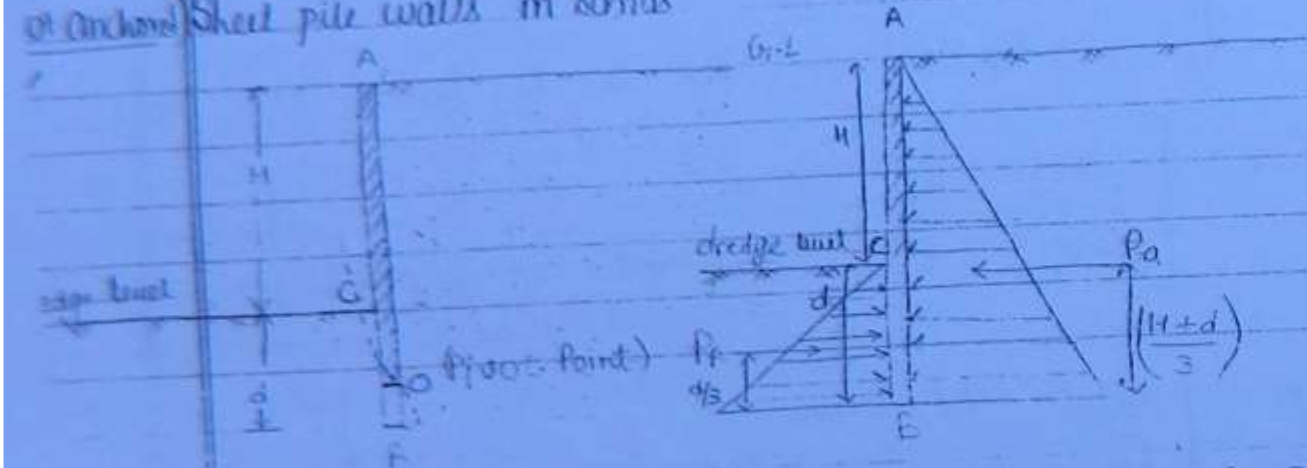
(ii) Diversion dams & Coffer dams.

(iii) River bank protection

(iv) To support vertical cuts in excavation.

(v) To reduce uplift pressure below hydraulic structures

(a) Anchored Sheet pile walls in bonds



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 base B. Hence right face of the cantilever wall will be in active state and left of the cantilever wall will be in passive state. The active pressure on 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

$$\Sigma M_B = 0$$

Restoring Moment M_R = Overturning Moment M_O

$$P_p \times \frac{d}{3} \qquad P_a\left(\frac{H+d}{3}\right)$$

$$\frac{1}{2} K_p \gamma d^2 \times \frac{d}{3} = \frac{1}{2} K_a \gamma (H+d)^2 \left(\frac{H+d}{3}\right)$$

$a = K_a \gamma H - 2c\sqrt{K_a}$

$b = 2c\sqrt{K_a}$

Height $= H - 2Z_c$

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Total active thrust on unit length of wall

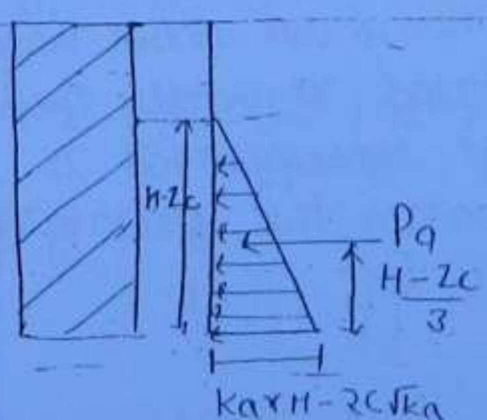
$$P_a = \frac{1}{2}(a+b)(H-2Z_c) \quad \text{act at } \bar{H}$$

$$\bar{H} = \frac{a+2b}{a+b}\left(\frac{H-2Z_c}{3}\right)$$

$$= \frac{1}{2}\left(K_a\gamma H - 2c\sqrt{K_a} + 2c\sqrt{K_a}\right)(H-2Z_c)$$

$$\boxed{P_a = \frac{1}{2}K_a\gamma H^2 - 2CH\sqrt{K_a}}$$

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.



$P_a = \frac{1}{2}(K_a\gamma H - 2c\sqrt{K_a})(H-Z_c)$ acts at $\frac{H-Z_c}{3}$ from base

$$\boxed{P_a = \frac{1}{2}K_a\gamma H^2 - 2CH\sqrt{K_a} + \frac{2C^2}{\gamma}}$$

1st Choice

$K_a = 1$ (Pure Clay)

At $z = 0$, at G.L

$p_a = -2c\sqrt{K_a}$

At $z = H$ = $K_a \gamma H - 2c\sqrt{K_a}$ at base

At $z = z_c$ $p_a = 0$

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$$K_a \gamma z_c - 2c\sqrt{K_a} = 0$$

$$K_a \gamma z_c = 2c\sqrt{K_a}$$

$$z_c = \frac{2c\sqrt{K_a}}{K_a \gamma}$$

$$z_c = \frac{2c}{\sqrt{K_a} \cdot \gamma}$$

$$z_c = \frac{2c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right)$$

z_c = Depth of potential Cracks.

H_c = Critical depth or Critical height So $H_c = 2z_c$

$$= \boxed{H_c = \frac{4c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right)}$$ 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 $\bar{c}$ 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) $\bar{c}$ 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

$\bar{c}$ Total active earth pressure when tension Cracks are not developed

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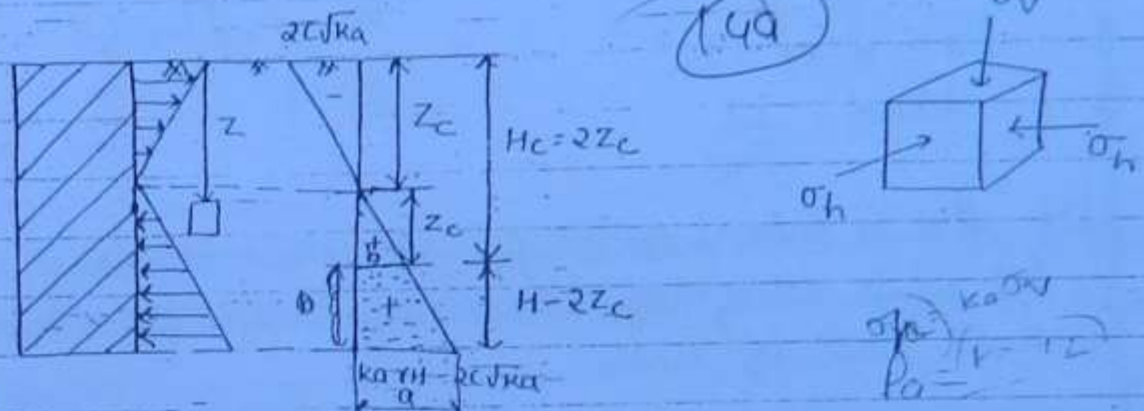
Active earth pressure on Cohesive Soils

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 betn molecule (Cohesion) hence active earth pressure is reduced whereas passive earth pressure is ↑



In active state $\sigma_v > \sigma_h$

$\sigma_v = \gamma \cdot z$ $\qquad \sigma_1 \qquad (\sigma_3) = \sigma_2 = Pa$

$$\sigma_1 = \sigma_3' \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2C \tan\left(45 + \frac{\phi}{2}\right)$$

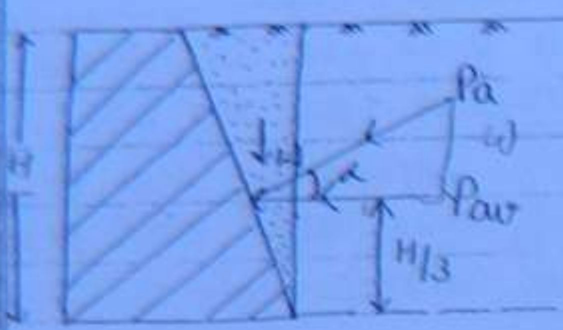
$$\gamma \cdot z = Pa \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2C \tan\left(45 + \frac{\phi}{2}\right)$$

$$Pa = \frac{\gamma \cdot z}{\tan^2\left(45 + \frac{\phi}{2}\right)} - \frac{2C}{\tan\left(45 + \frac{\phi}{2}\right)}$$

$$\tan^2\left(45 + \frac{\phi}{2}\right) = N\phi \text{ (influence value)}$$

$$Kp = \frac{1}{Ka}$$

$$Pa = \frac{\gamma \cdot z}{N\phi} - \frac{2C}{\sqrt{N\phi}} = Ka\,\gamma \cdot z - 2C\sqrt{Ka}$$



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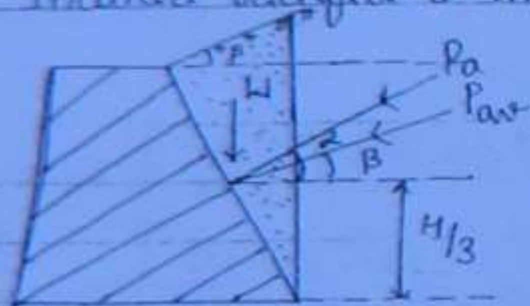
$$P_{av} = \frac{1}{2} K_a \gamma H^2$$

$$P_a = \sqrt{P_{av}^2 + W^2}$$

$$\tan\alpha = \frac{W}{P_{av}}$$

Let Pav is lateral earth pressure on imaginary vertical wall & let W is weight of soil acting vertically down on inclined back of wall.

Case 5 - Inclined backfill & inclined back of wall.



Let Pav is lateral earth pressure on imaginary vertical wall due to inclined backfill c acts at an angle β c̄ horizontal. Let W is weight of soil ~~retained~~ Retained on vertica inclined on back of wall -

$\Sigma F_H = P_{av}\cos\beta$ (Horizontal Component of thrust)

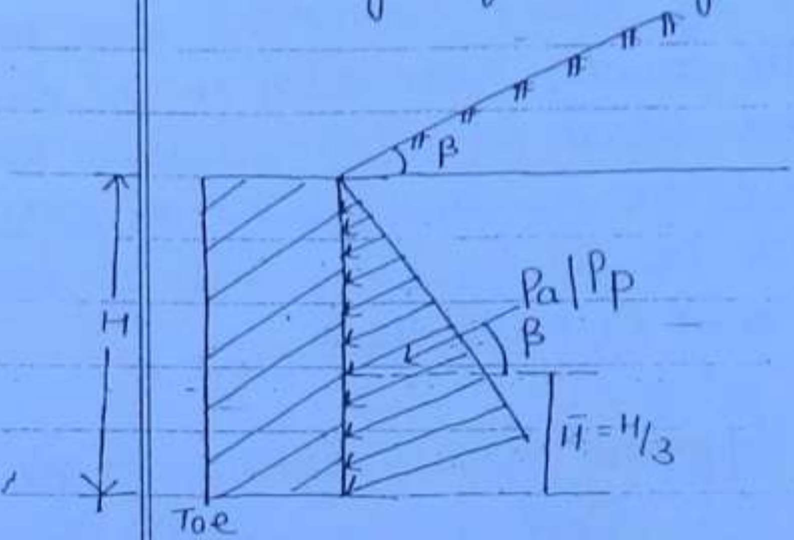
$\Sigma F_V = P_{av}\sin\beta + W$ (vertical " " ")

$$P_a = \sqrt{(F_H)^2 + (F_V)^2}$$

$$P_a = \sqrt{(P_{av}\cos\beta)^2 + (P_{av}\sin\beta + W)^2}$$

$$\tan\alpha = \frac{\Sigma F_V}{\Sigma F_H} = \frac{P_{av}\sin\beta + W}{P_{av}\cos\beta}$$

). Case 4 - Effect of inclined backfill (Inclined surcharge)
Let angle of surcharge $\bar{c}$ horizontal is β



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The resultant active or passive thrust acts $\parallel^{el}$ to the surcharge at an angle β $\bar{c}$ the horizontal at $\frac{H}{3}$ from the base.

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

$$P_p = \frac{1}{2} K_p \gamma H^2$$

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]$$

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 $\bar{c}$ inclined back of wall.

NOTE If W.T is present then grain to grain contact pressure below the W.T should be consider by taking submerged unit wt and pore pressure will be equally transmitted in all dirn under pascal's law hence below W.T pore pressure on the wall will be hydrostatic & same in active, passive & rest condin. If W.T is present on both side of the wall then pore pressure will be neutralized.

Sp. Case: Backfill contains two different soils $\bar{c}$ friction angle ϕ_1 & ϕ_2.

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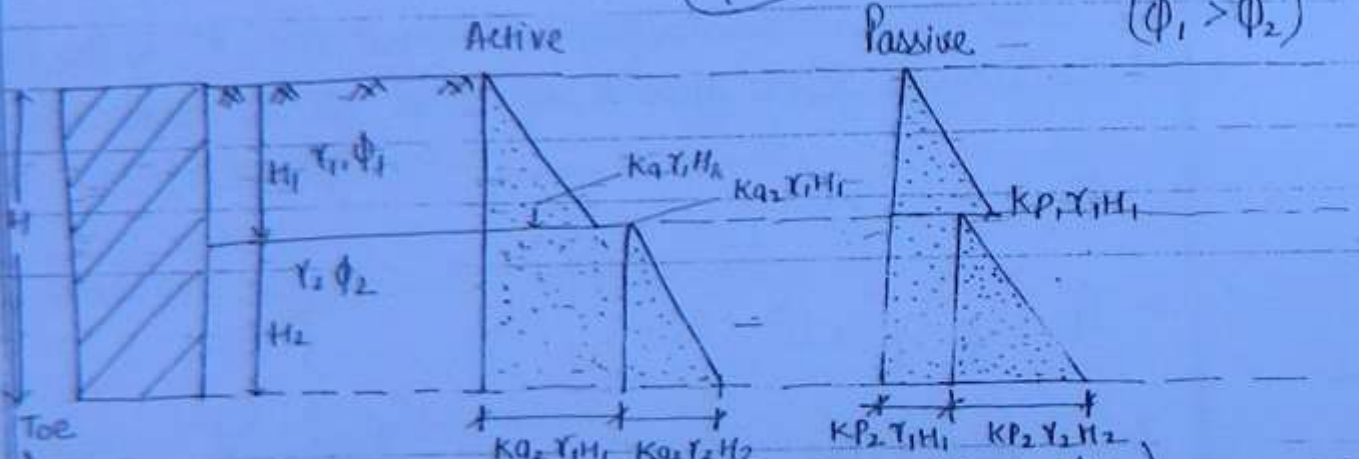

If $\phi_1 > \phi_2 \Rightarrow K_{a_1} < K_{a_2}$ $\quad K_a = \tan^2(45^\circ - \phi/2)$

$K_{p_1} > K_{p_2}$ $\quad K_p = \tan^2(45^\circ + \phi/2)$

NOTE The top layer should be treated as surcharge over bottom layer

$P_{a_1} = \frac{1}{2} K_{a_1}\gamma_1 H_1^2$ $\quad \bar{h}_1 = H_2 + \frac{H_1}{3}$

$P_{a_2} = K_{a_2}\gamma_1 H_1 H_2$ $\quad \bar{h}_2 = \frac{H_2}{2}$

$P_{a_3} = \frac{1}{2} K_{a_2}\gamma_2 H_2^2$ $\quad \bar{h}_3 = \frac{H_2}{3}$

Total active thrust per unit length of wall

$P_{a_1} = (K_a q \times H) \times 1$ — acts at $\frac{H}{2}$ from base ($\bar{H}_1$)

$P_{a_2} = \frac{1}{2} K_a \gamma H^2$ — acts at $\frac{H}{3}$ from base ($\bar{H}_2$)

Total $P_a = P_{a_1} + P_{a_2}$

Line of action of resultant from base

$$\bar{H} = \frac{P_{a_1} \cdot \bar{H}_1 + P_{a_2} \cdot \bar{H}_2}{P_{a_1} + P_{a_2}}$$

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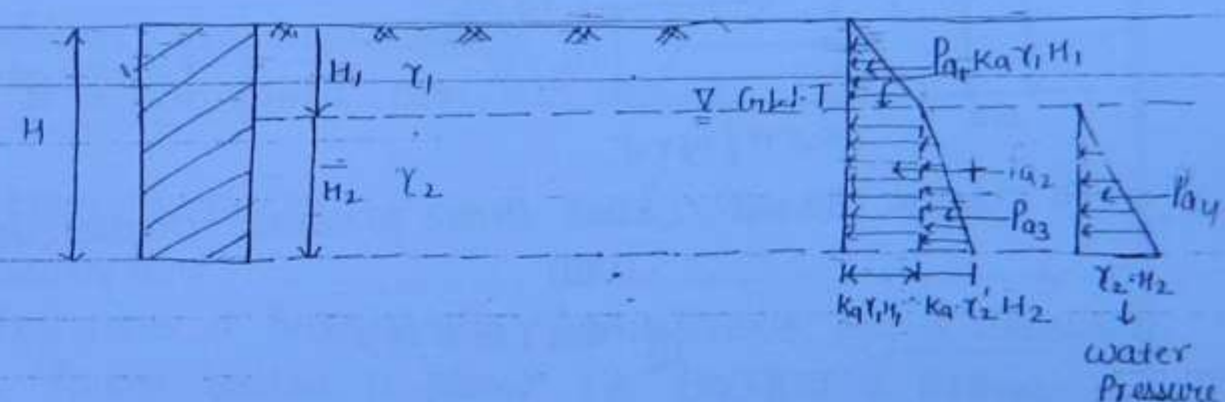
Case 2 Effect of water table

Let W.T is present at a depth of H_1 from ground level



$P_{a_1} = \frac{1}{2} K_a \gamma_1 H_1^2$ — $\bar{H} = H_2 + \frac{H_1}{3}$ from base

$P_{a_2} = K_a \gamma_1 H_1 H_2$ — $\bar{H} = \frac{H_2}{2}$ "

$P_{a_3} = \frac{1}{2} K_a \gamma_2' H_2^2$ — $\bar{H} = \frac{H_2}{3}$ "

$P_{a_4} = \frac{1}{2} \gamma_w H_2^2$ — $\bar{H} = \frac{H_2}{3}$ "

$P_a = \Sigma P_a$

$\bar{H} = \frac{\Sigma P_a \bar{H}}{\Sigma P_a}$

rupture plane is plastic failure.

(vi) The stress relationship for any two element adjacent to the wall is similar but diffⁿ then the stress distribution in adjacent element which is outside the wall.

(vii) Rankine Consider element failure whereas Coulomb Consider wedge failure.

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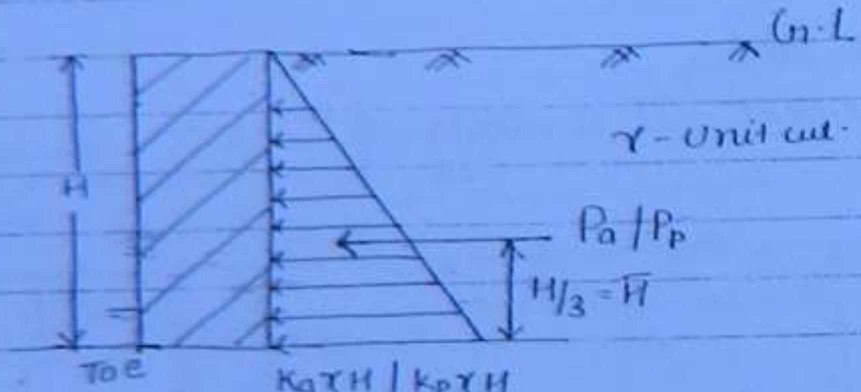
Total active & passive earth pressure on unit length of wall.



ϕ = friction angle

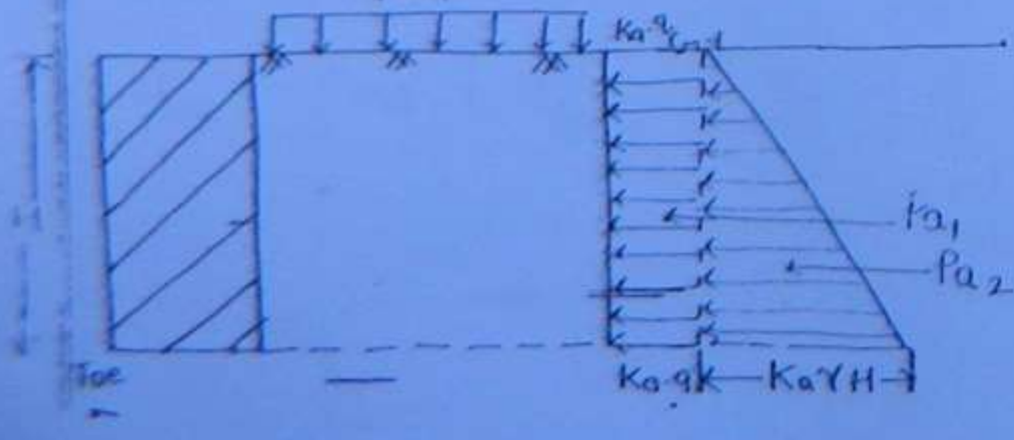
P_a = Active thrust = Total active pressure on unit length of wall

$= \frac{1}{2} \times K_a \gamma H \times H \times 1$

$= \frac{1}{2} K_a \gamma H^2$ — acts at $\frac{H}{3}$ 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.

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2. Coulomb's theory
More realistic for Concrete retaining wall especially in active state but results in passive states are eronious.

3. Culmann's graphical Method
It is base on Coulomb's theory & Coulomb's theory is modified version of Rankine theory.

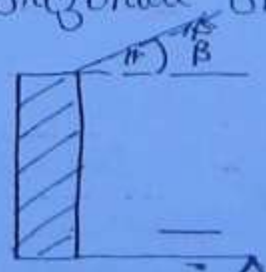
4. Rebhan graphical Method
It is also called poncelete Method

RANKINE'S THEORY

Assumptions

(i) Soil mass is semi-infinite, homogenous, dry & cohesionless. $(c=0)$

(ii) The ground surface is plane i.e backfill is plane $\bar{c}$ may be horizontal or inclined.



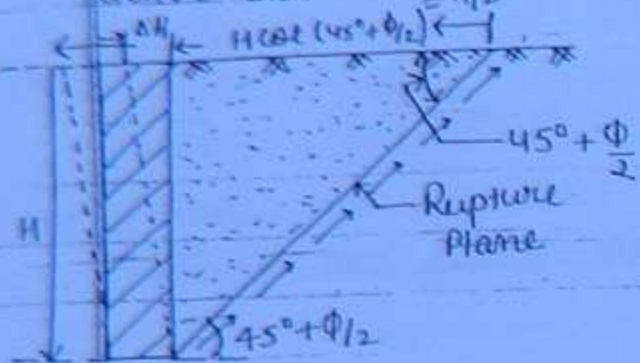
(iii) The face of the wall in contact with backfill is vertical & smooth

(iv) The wall yields abt the base sufficiently to mobilise active & passive earth pressure fully.

(v) Stresses 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 / Mobilise

Strain $= \frac{\Delta H}{H} \times 100 \geq 0.1$ to 0.2% for dense sand

$\frac{\Delta H}{H} \times 100 \geq (0.2$ to $0.5)\%$ for loose sand

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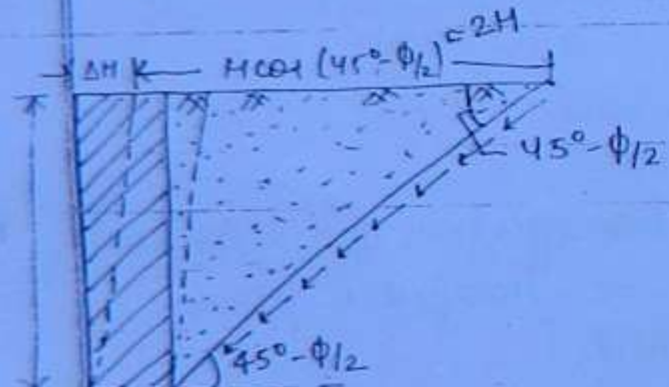
In passive state the wall is pushed against the backfill i.e towards the backfill. The angle of rupture surface with the horizontal is $45° - \frac{\phi}{2}$. Relatively more horizontal strain is req. to mobilize full passive state.



$\frac{\Delta H}{H} \times 100 \geq 2\%$ for dense sand

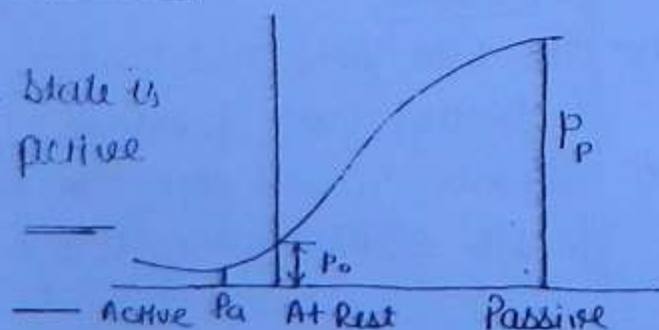
$\frac{\Delta H}{H} \times 100 > 10\%$ for loose sand

Acc to Rankine $K_A = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45° - \frac{\phi}{2}\right)$

$K_P = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45° + \frac{\phi}{2}\right)$

$K_A = \frac{1}{K_P} \Rightarrow \boxed{K_A \cdot K_P = 1}$

ΔH req. to mobilise 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 ~~ø~~ 0.2q isobar. The approximate depth of 20% isobar is 1.5 B where B is the width of footing.

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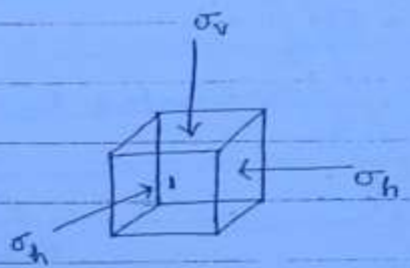
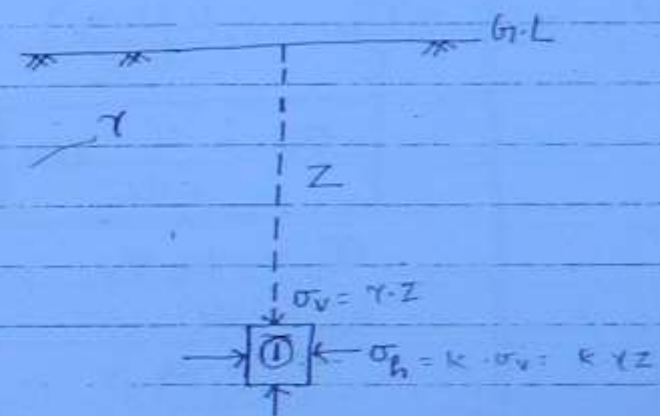
LATERAL EARTH PRESSURE THEORY





$\sigma_v = \gamma \cdot Z$

$\sigma_h = K \cdot \sigma_v$ (lateral earth pressure)

$K = K_0$ At rest Condition

$K = K_a$ At active Condition

$K = K_p$ At Passive Condition

K = lateral earth pressure Coefficient

$\therefore \quad K_a < K_0 < K_p$

The earth pressures are statically indeterminate and hence pose problems in evaluation.

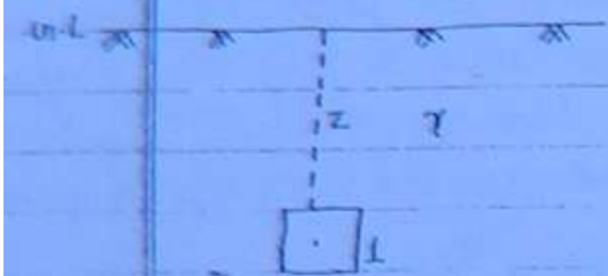
In order to Support lateral earth pressure retaining walls & sheet pile walls are used. Retaining walls may be classified as —

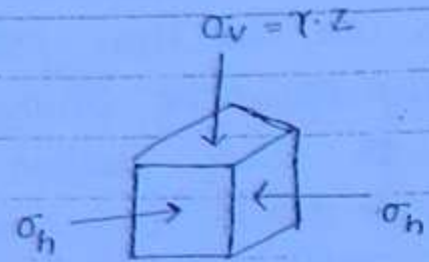
(i) Gravity Retaining wall
(ii) Cantilever Retaining wall
(iii) Counterfort Retaining wall
(iv) Buttress Retaining wall

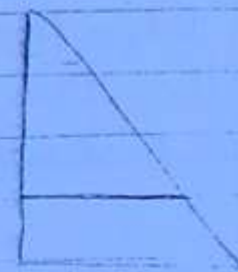
Whereas Sheet pile walls may be classified as

(i) Cantilever Sheet pile.

Anchored Sheet pile







At rest Condition

Lateral Strain = 0 (at rest) 158

$$\epsilon_h = 0$$

$$-\frac{\sigma_h}{E} - \mu\left(\frac{-\sigma_h}{E}\right) - \mu\left(-\frac{\sigma_v}{E}\right) = 0$$

$$\boxed{\sigma_h = \frac{\mu}{1-\mu}\,\sigma_v}$$

Saturated $K_0 = \frac{\sigma_h'}{\sigma_v'}$

effective

$\sigma_h = K_0 \sigma_v$ ← 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.

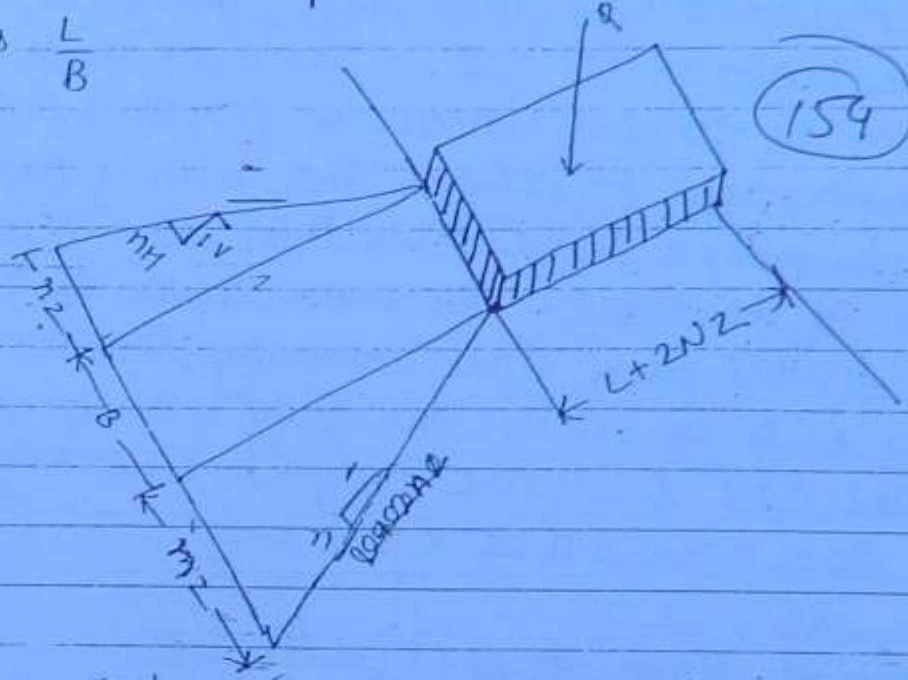
Type of Soil	K_0
Dense sand	0.4 to 0.5
loose sand	0.45 to 0.55
N.C. Clays	0.5 to 0.6
O.C Clays	1 to 4.0

$$(K_0)_{O.C\ \text{soil}} = (K_0)_{N.C.\ \text{soil}} \times \sqrt{O.C.R}$$

O.C.R > 1 for O.C. Soil.

(ii) Trapezoidal Method or 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 $\frac{L}{B}$



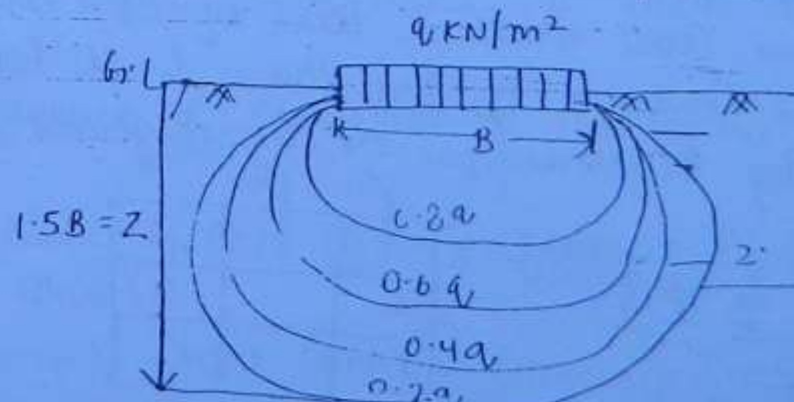
$$\sigma_z = \frac{Q}{(B+2nz)(L+2nz)} = \frac{q \times (L \times B)}{(B+2nz)(L+2nz)}$$

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.



1st Choice

$$\frac{1}{m \times n} = \frac{1}{10 \times 20} = 0.005$$

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The vertical stress is req. at P for C. O is projected on the surface. $OP = Z = 5$ cms or 3 cms. On this scale plan of loaded area is prepared on a tracing paper with ref to point O. The tracing paper is placed on 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.

eg – 20 units of area are fully occupied, 10 are occupied 50% & 4 are occupied quarterly

$$20 + 0.5 \times 10 + 0.25 \times 4 = 26$$

$$N_A = 26$$

The vertical stress at pt. P will be

The vertical stress = q × Influence factor of each area × N_A

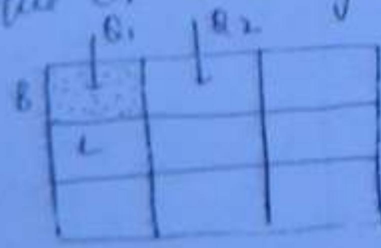
$$\sigma_z = q \times \text{Influence} = q \times \frac{1}{m \times n} \times N_A$$

If $m = 10$ & $n = 20$ then $\sigma_z = 0.005 q \cdot N_A$

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 $\frac{L}{Z} \not> 3$



$$\& \ \frac{B}{Z} \not> 3$$

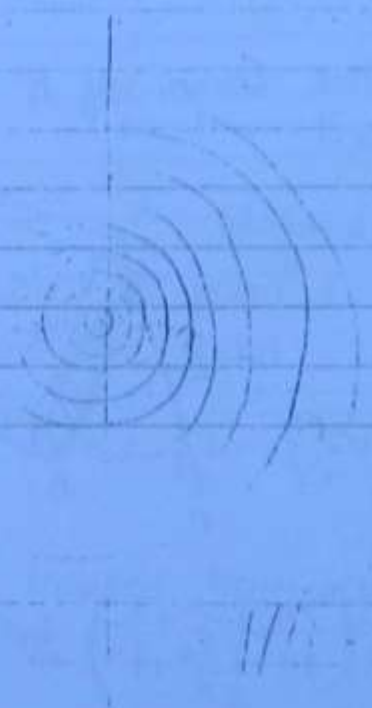
$$\sigma_{z_1} = K_{B_1} \frac{Q_1}{Z^2}, \quad \sigma_{z_2} = K_{B_2} \frac{Q_2}{Z^2}$$

$$\sigma_z = \sigma_{z_1} + \sigma_{z_2} + \dots$$

mp Newmarks Chart Method

It is also called Influence Chart Method. It is applicable for homogenous, isotropic, semi infinite & elastic soil mass & is loaded & U.D.L ($q\,kN/m^2$). The loaded area may have any shape

(161)



O —— P

OP = 5cms or 3cms

It contains m concentric circles $m = 10$

n radial lines $n = 20$

Charts 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)

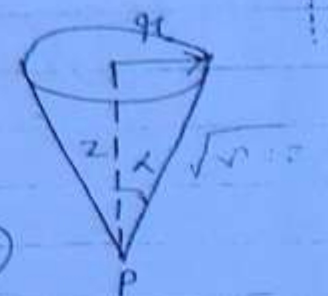
Newmarks chart contain concentric circles & radial lines at a scale mention in the chart OP 5cm or 3cm

Generally there 10 circles & 20 radial lines hence influence of each area at the centre of chart is

Case Vertical Stress due to Circular loaded area $\bar{c}$ U·D·L at a depth z below the Centre line of load.

Let q is intensity of load (kN/m²)

& r = radius of loaded area.



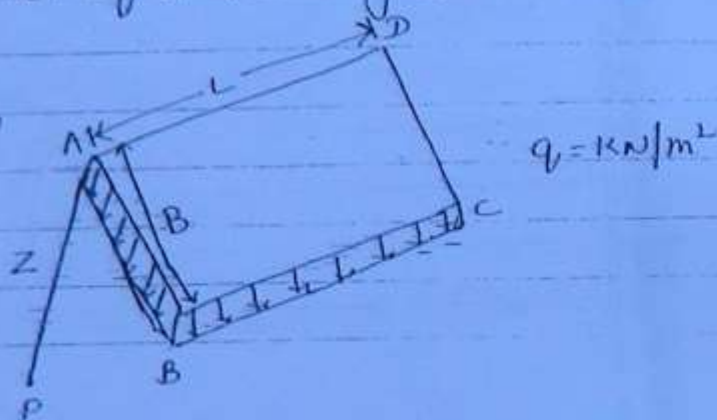
$$\cos\alpha = \frac{z}{\sqrt{r^2+z^2}}$$

$$\sigma_z = q(1-\cos^3\alpha)$$

(162)

Case Vertical Stress below the corner of a rectangular loaded area $\bar{c}$ U·D·L

$$\left.\begin{aligned} m &= \frac{L}{z} \\ n &= B/z \end{aligned}\right\}\text{ Interchangable}$$



$$\sigma_z = q \cdot I_\sigma$$

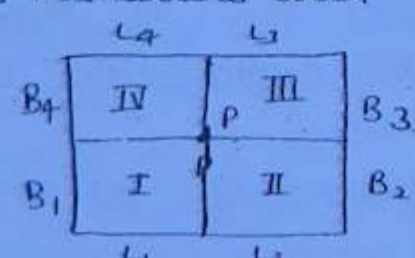
I_σ = Influence factor

$$\cancel{I_\sigma = \frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}}$$

$$I_\sigma = \frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)$$

Special Case 1 – If pt 'P' is located inside the loaded area

$$m_1 = \frac{L_1}{z} \quad \& \quad n_1 = \frac{B_1}{z}$$

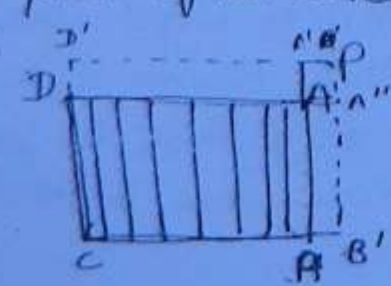


Total influence factor $I_\sigma = I_{\sigma_1} + I_{\sigma_2} + I_{\sigma_3} + I_{\sigma_4}$

Special Case 2 If Pt 'P' lies outside the loaded area

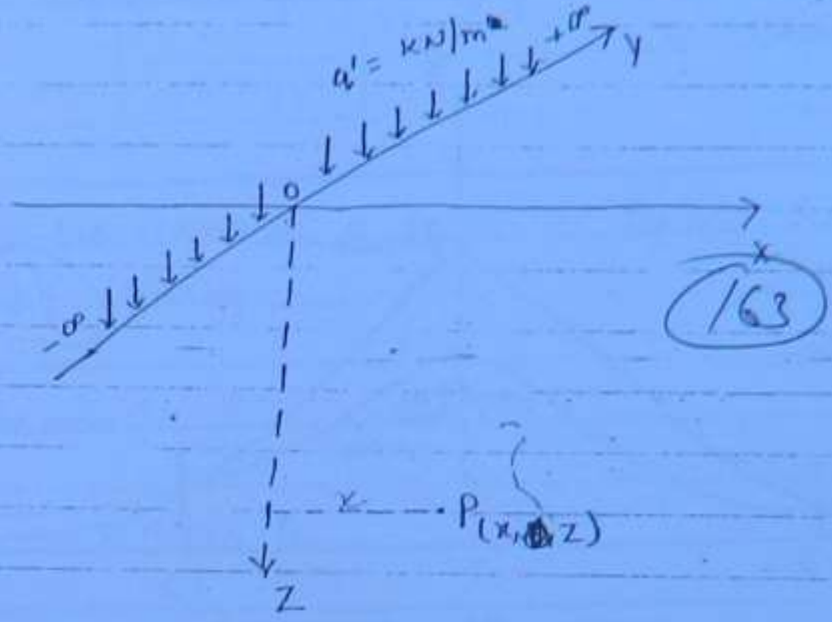
Let P is projection of stress pt. on the plan of the loaded area.

I_σ = Influence factor at P due to loaded area ABCD



= I_σ for PB'CD'P – I_σ for PA''DD'P – I_σ for PA'BB''P + I_σ for PA'AA''

Case Vertical Stress due to line load

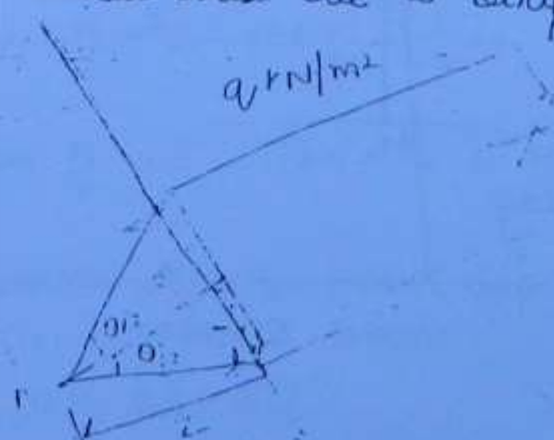


The vertical stress at point P which is at depth 'z' & horizontal distance x from line load

$$\sigma_z = \frac{2q'}{\pi z}\left[\frac{1}{1+\frac{x^2}{z^2}}\right]^2$$

Special case. If pt 'P' is located vertically below the line load then $x = 0$ then $\sigma_z = \frac{2q'}{\pi z}$

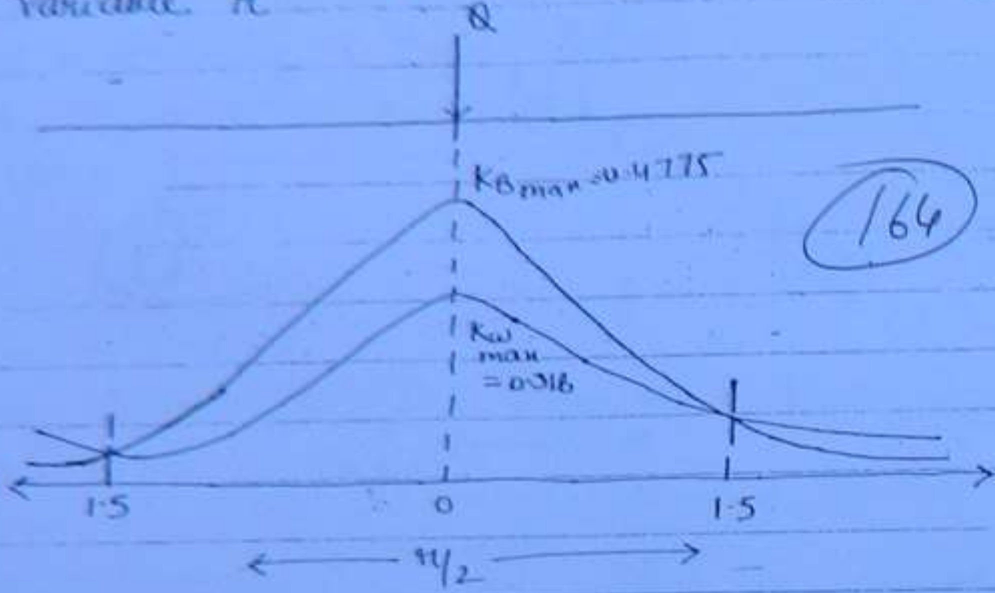
Case Vertical stress due to Strip loading



Vertical stress at pt 'P' which 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 betn Westergaard's & Boussinesque theory at a depth z with variable r.



If $\frac{r}{z} < 1.5$ then $K_B > K_W$

If $\frac{r}{z} = 1.5$ then $K_B = K_W$

If $\frac{r}{z} > 1.5$ then $K_B < K_W$

Application

$$\Delta H = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

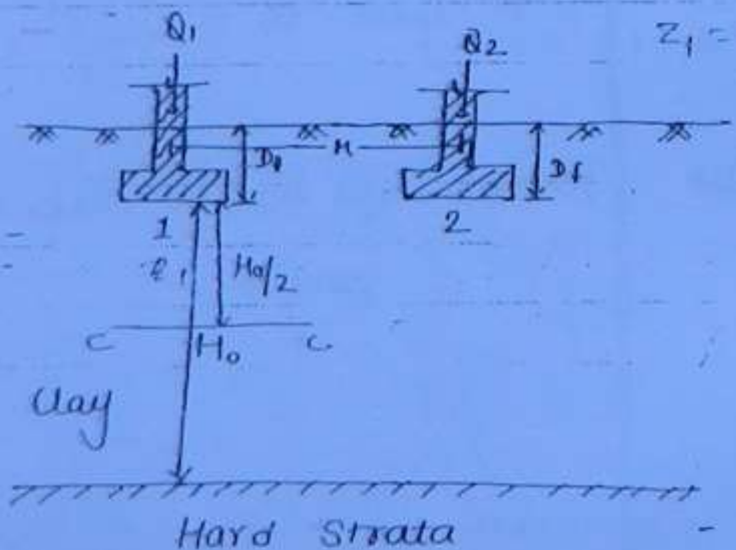
e_0 = Initial void ratio of Compr- Clay layer

H_0 = thickness of Compr. 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

$z_1 = \frac{H_0}{2}$



$\Delta\sigma_1 = \sigma_{z_1} = K_{B_1} \frac{Q_1}{z_1^2}$

$\Delta\sigma_2 = \sigma_{z_2} = K_{B_2} \frac{Q_2}{z_2^2}$

$\Delta\bar{\sigma} = \sigma_{z_1} + \sigma_{z_2}$

K_B = Boussinesque influence factor

$$K_B = \frac{3}{2\pi}\left[\frac{1}{1+r^2/z^2}\right]^{5/2}$$

Special Case -

If point P is located at a depth 'z' below the point Z in the same line of loading

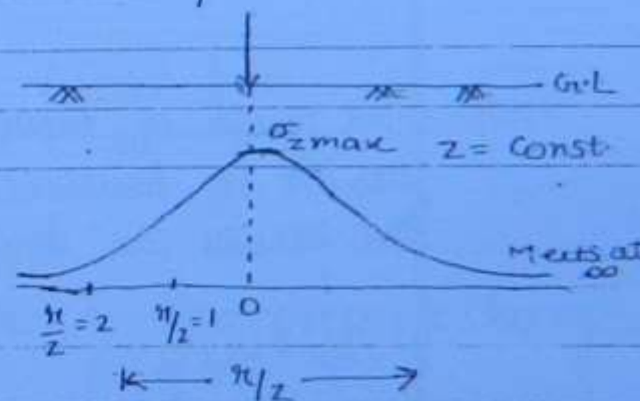
Hence $r = 0$

$$K_B = K_{B\,max} = \frac{3}{2\pi} = 0.4775$$

$$\therefore \quad \sigma_z = 0.4775 \cdot \frac{Q}{z^2}$$

I Variation of σ_z w.r.t r at constant depth

$$\sigma_{z\,max} = 0.4775\,\frac{Q}{z^2}$$



NOTE - $\sigma_{z\,max}$ occurs when $\frac{r}{z} = 0$ & $\sigma_{z\,max} = 0.4775\frac{Q}{z^2}$,

When $\frac{r}{z} = 2$ then σ_z is 1.8 % of $\sigma_{z\,max}$

When $\frac{r}{z} = 3$ then σ_z is only 0.3 % of $\sigma_{z\,max}$

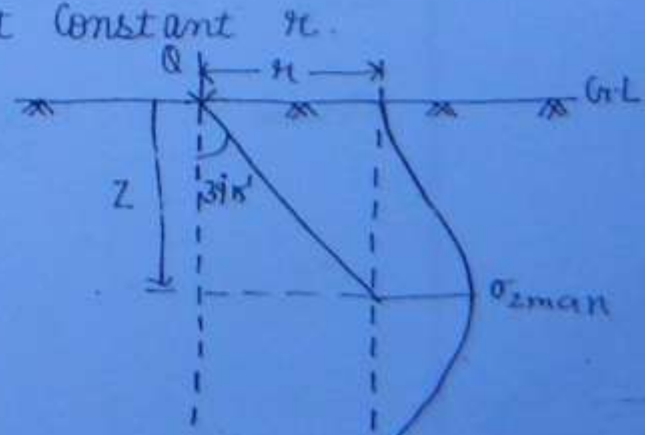
II Variation of σ_z with depth at constant r.

$\tan(39°15')$ 0.817

$$\frac{r}{z} = 0.817$$

$$\sigma_{z\,max} = \frac{0.0888\,Q}{r^2}$$

$$\sigma_{z\,max} = \frac{0.1332\,Q}{z^2}$$



Westerguard's theory (166)

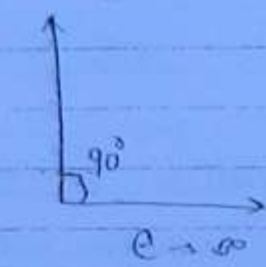
→ Assumption

(i) The material is elastic, homogenous & non-isotropic

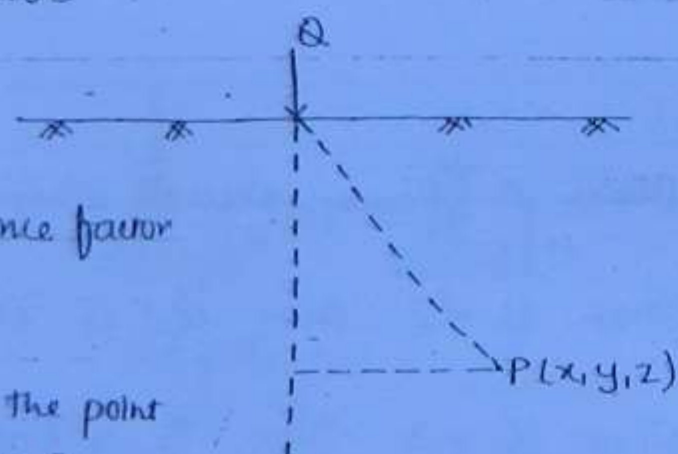
NOTE - The Clay Strata is made of thin lenses which makes the soil nonisotropic.

(ii) The material is semiinfinite & laterally reinforced with numerous closely spaced sheets of negligible thickness i.e. means soil mass is consider stratified in which soil in horizontal dirn is consider rigid & in vertical dirn the soil is consider elastic. (So $\mu = 0$)

$$\frac{E_h}{E_v} \rightarrow \infty$$



NOTE - The results of westerguard'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 = westerguard's influence factor

$$K_w = \frac{1}{\pi}\left[\frac{1}{1+2\cdot\frac{r^2}{z^2}}\right]^{3/2}$$

Special Case - If Point 'P' lies below the point load at depth z then $r = 0$

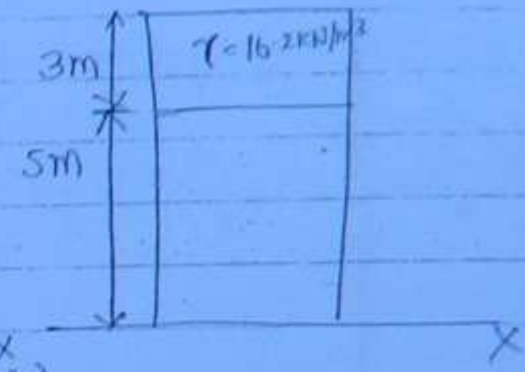
$$K_{w\,max} = \frac{1}{\pi} = 0.318$$

$$\sigma_z = 0.318\frac{Q}{z^2} \quad \text{(at depth z, below the pt load)}$$

no pore pressure before the ↑ 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. $S = c' + \bar{\sigma}_n \tan\phi'$

$\bar{\sigma}_n = \sigma_x - U_x$ — 167



$\Delta\sigma_1 = 3 \times 16.2 = 48.6 \ KN/m^2$

$\Delta\sigma_3 = \frac{1}{2}\sigma_1 = \frac{48.6}{2} = 24.3 \ KN/m^2$

Change in pore pressure

$\Delta U = \Delta U_c + \Delta U_d$

$B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$

$0.92\ [24.3 + 0.4(48.6 - 24.3)]$

$\Delta U = 31.2984$

Initial Pore pressure $= 0$

final " " $= 31.2984 =$ Pore pressure change

Eff Stress $= \sigma_x - U_x$

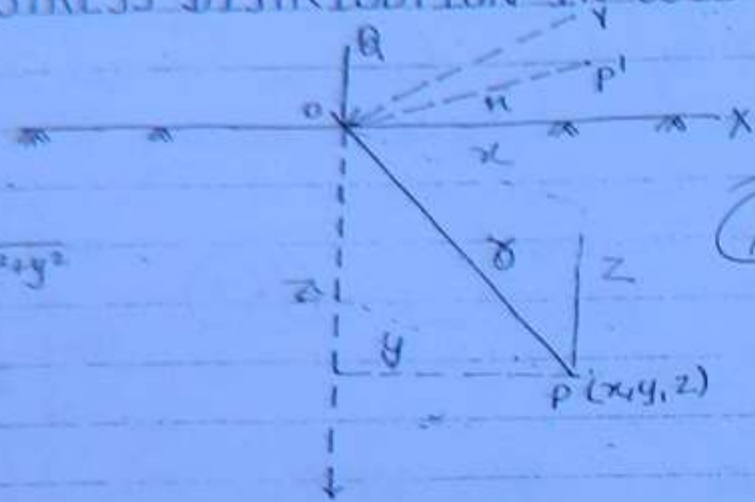
$8 \times \gamma - 31.29$

$8 \times 16.2 - 31.29 = 98.31$

$S = 50 + 98.31 \tan 16°$

$= 78.18 \ kPa$

STRESS DISTRIBUTION IN SOIL



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$r = \sqrt{x^2+y^2}$

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) Westerguard'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 disⁿ of point P is $r = \sqrt{x^2+y^2}$ & vertical disⁿ is Z hence vertical Stress at P is given as

$$\sigma_z = K_B \frac{Q}{z^2}$$

σ_z at a constant pt Z is dependent upon r/z ratio.

II $\sigma_3 = 350 \qquad \sigma_1 = 350 + 242 = 592$

$592 = 350\tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$

$592 - 429 = 350\tan^2\left(45 + \frac{\phi}{2}\right) - 250\tan^2\left(45 + \frac{\phi}{2}\right)$

$\phi = 13.86°$ 169

$c = 8.42$ kPa

Effective parameters

① $\bar{\sigma}_3 = 250 - 101 = 149$

$\bar{\sigma}_1 = 149 + 179 = 328$

2 $\bar{\sigma}_3 = 350 - \underset{105}{145} = 205$

$\bar{\sigma}_1 = 205 + 242 = 447$

from Sample 1

$328 = 149\tan^2\left(45' + \frac{\phi'}{2}\right) + 2c'\tan\left(45 + \frac{\phi'}{2}\right)$

$447 = 205\tan^2\left(45° + \frac{\phi'}{2}\right) + 2c'\tan\left(45 + \frac{\phi'}{2}\right)$

$\phi' = 3.98\ \text{kN/m}^2$

$c' = 21°$

ies Laboratory results on a soil mass show that unconfined comp. strength is $1.2\ \text{kg/cm}^2$. Another specimen of same soil is tested in triaxial test under drained condition which resultant deviator stress at failure is $1.6\ \text{kg/cm}^2$ at a cell pressure of $0.4\ \text{kg/cm}^2$. Estimate the shear strength of the same soil along a horizontal plane at a depth of 4 m from the ground level. GWT in the field is 2.5 m below the ground level. The soil above the W.T is dry have $\gamma_d = 1.7$ gm/cc, $G = 2.7$.

1st Choice

$$1.7 = \frac{2.7 \times 1}{1+e}$$

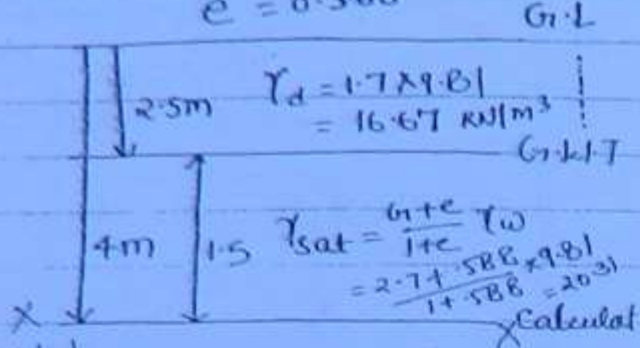
$$e = 0.588$$

unconfined compressive strength

$$q_u = 2c' \tan\left(45+\frac{\phi'}{2}\right)$$

$$\bar{\sigma}_3 = 0.4\ \text{kg/cm}^2$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \Delta\bar{\sigma} = 0.4 + 1.6 = 2.0$$



$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2\left(45+\frac{\phi'}{2}\right) + 2c \tan\left(45+\frac{\phi'}{2}\right)$$

$$\tan^2\left(45+\frac{\phi'}{2}\right) = 2$$

$$2 = 0.4 \tan^2\left(45+\frac{\phi'}{2}\right) + 1.2$$

$$\phi' = 19.47^\circ$$

$$c' = 0.636\ \text{kg/cm}^2$$

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$$\frac{0.636 \times 9.81 \times 10^{-3}}{10^{-4}}\ \frac{\text{kN}}{\text{m}^2}$$

$$c' = 62.39\ \frac{\text{kN}}{\text{m}^2}$$

- Eff Normal stress at XX

$$\bar{\sigma}_n = \bar{\sigma}_x = \sigma_x - u_x$$

$$2.5\gamma_d + 1.5\gamma_{sat} - 1.5\gamma_w$$

$$2.5 \times 16.67 + 1.5 \times 20.31 - 1.5 \times 9.81$$

$$57.425\ \text{kN/m}^2$$

Shear strength $S = c' + \bar{\sigma}_n \tan\phi'$

$$= 62.39 + 57.425 \tan 19.47^\circ$$

$$= 82.69\ \text{kN/m}^2$$

Ques :- An embankment 5m height is made of eff. stress parameter $c' = 50$ kpa & $\phi' = 16^\circ$ having bulk unit wt. $16.2\ \text{kN/m}^3$. The pore pressure parameter A & B are 0.4 & 0.92 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/mm²	Vertical load (kg)	(N) P	Division of dial gauge (1 Div. = 1 μm)	S.F (N) K× Reading	$\tau=\frac{S.F}{A}$ N/mm²
0.1	36.8	36.8×9.81	17	20×17 = 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

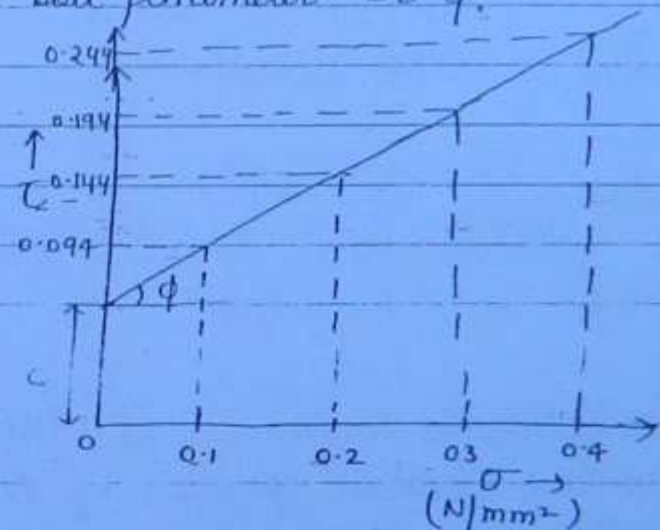
(17)

The shear box size is 60mm × 60mm & proving ring constant is 20 N/μm. Determine soil parameter C & φ.

$K = 20 \frac{N}{\mu m}$

Sol $C = 44.6$ kN/m²

$\phi = 26.5°$



Ques- A CD test conducted on a clay sample resulted the following data. Cell pressure- 250 kN/m², deviator stress at failure = 275 kN/m². Determine drained friction angle (ϕ_d) if drained cohesion is zero, angle of critical plane $\bar{c}$ the major principal plane, normal stress on critical plane & shear strength of the soil.

Sol In CD test - $\sigma_3 = \sigma_1 = 250$ kN/m²

$\Delta\sigma = \bar{\sigma}_d = 275$

$\sigma_1 = 250 + 275 = 525$ kN/m²

$C_d = C' = 0$

$\phi_d = \phi' = ?$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2\left(45 + \frac{\phi'}{2}\right) + \cancel{2c'}\tan\left(45 + \frac{\phi'}{2}\right)$$

$$525 = 250 \tan^2\left(45 + \frac{\phi'}{2}\right)$$

$$\phi' = 20.78^\circ$$

$$\theta_c = 45 + \frac{\phi'}{2}$$

$$\theta_c = 45 + \frac{20.78}{2} = 55.39^\circ$$

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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 Ques 4 marks

Consolidated undrained (CU test) triaxial test when performed on two identical specimen of same saturated clay $\bar{c}$ pore pressure measurement as given in table. Determine total & eff shear strength parameter using analytical approach.

ll pressure (kPa)	Deviator stress at failure (kPa)	Pore Pressure at failure (kPa)	
50	179	101	– Sample-1
50	242	145	" 2

Total parameter

$\sigma_3 = 250 \qquad \sigma_1 = 250 + 179 = 429$

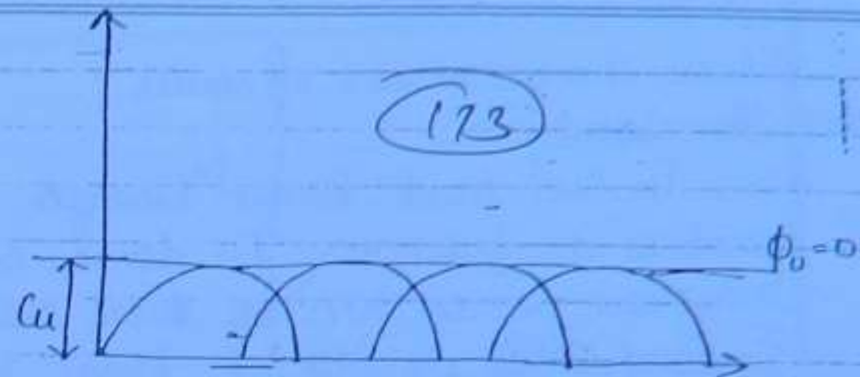
$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

Sample 1 $\quad 429 = 250 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$

$\phi_u = 0$

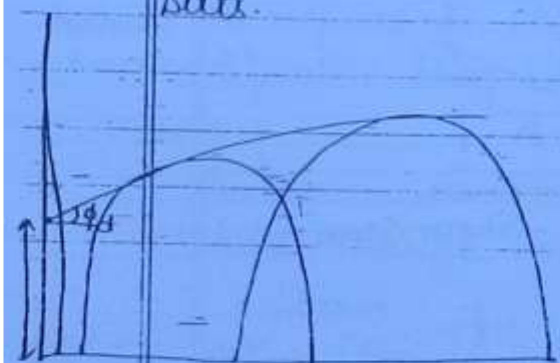
$C_u = 25$ to $30\ kN/m^2$ for N.C. Clay

$= 100$ to $300\ kN/m^2$ for O.C Clay.



(ii) Saturated Clay under drained Condition (CD test)

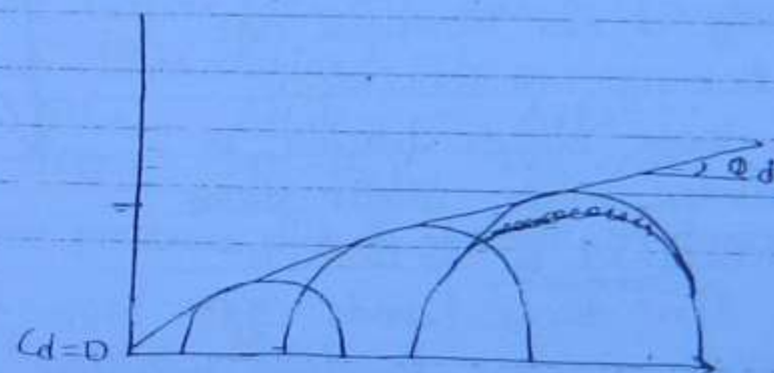
N.C Clays in CD test represents give results similar to that of sands & O.C clay give results similar to that of silts.



for O.C.C

$C_d = 60$ to $100\ kN/m^2$

$\phi_d = 10$ to 15^0



for N.C.C , (Sand)

$C_d = 0$

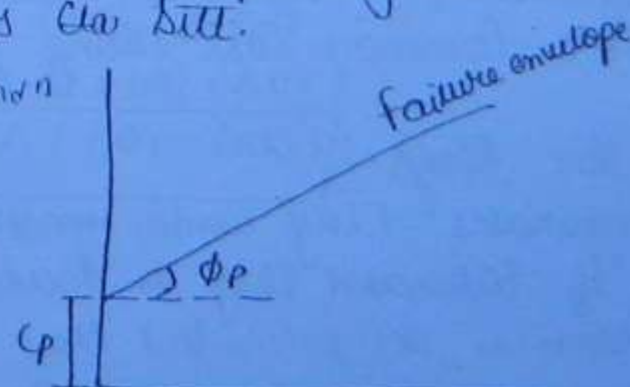
$\phi_d = 20^0$ to 30^0

(iii) Partially saturated Clays.

Under drained & undrained Condition both partially saturated clays behaves similar to ~~Clays~~ Cla Silt.

$C_p = 60$ to $80\ kN/m^2$ } drained Condn N.C.C

$\phi_d = 20$ to 30^0



$C_p = 80$ to $150\ kN/m^2$

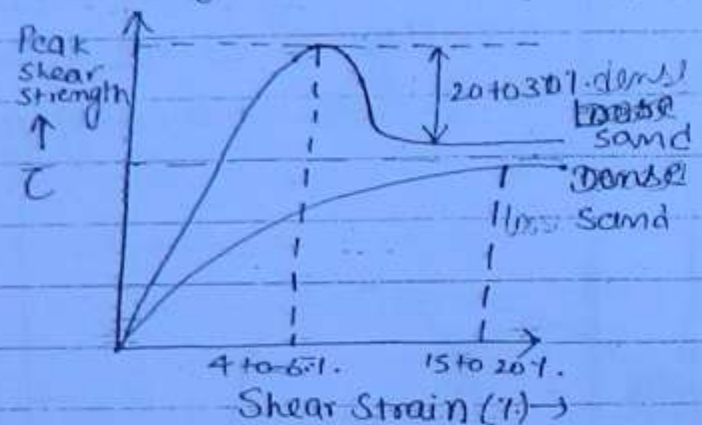
$\phi_p = 0^\circ$ to 5^0

(undrained condn)

Shear Characteristics of soils

Case 1 For sands —

In the sands Shear Strength is due to interlocking been particle & friction. In dense sands interlocking resistance may be 20 to 30% of total resistance. On shear failure ē 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 vol increases & pore pressure becomes negative due to ē eff stress ↑ & hence shear strength increases whereas in loose saturated sands on shear disturbance vol decreases, pore pressure increases hence eff. stress reduces ē 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 ē constant radius

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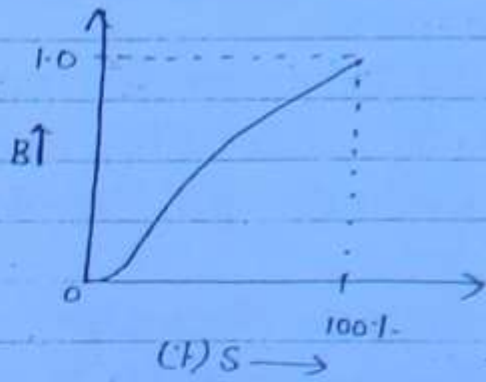

(ii) Shear Stage – The parameter A is defined in terms of another parameter $\bar{A}$ such that

$$\bar{A} = A \cdot B$$

$\bar{A}$ represents change in pore pressure due to change in deviator stag 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

$$\Delta u = \Delta u_c + \Delta u_d$$
$$= B \cdot \Delta\sigma_3 + \bar{A} \cdot \Delta\sigma_d$$
$$= B \cdot \Delta\sigma_3 + \bar{A}\,(\Delta\sigma_1 - \Delta\sigma_3)$$
$$= B\,\Delta\sigma_3 + A \cdot B\,(\Delta\sigma_1 - \Delta\sigma_3)$$

$$\boxed{\Delta U = B\,[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]}$$ Imp

Limitations

(i) No mechanism to measure pore pressure hence it gives essentially undrained shear strength

(ii) Not suitable for sands & dry soils.

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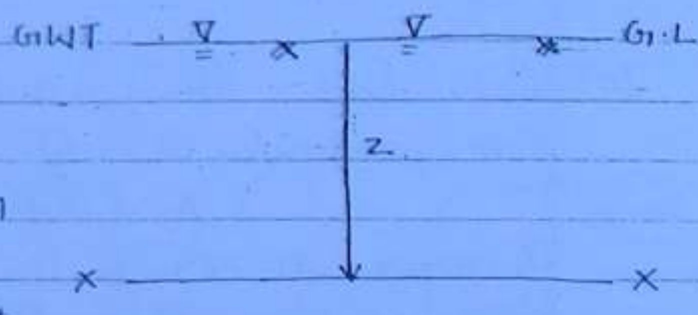
15/11/2011

Pore Pressure Parameter.

If it is not possible to determine pore-pressure praticaly then theoritical approach given by Skempton may be adopted. The pore pressure parameter represent the response of pore pressure due to change in stress under undrained condition.



$u_{x-x} = z \cdot \gamma_w$

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.

$$B = \frac{\Delta u_c}{\Delta \sigma_c} = \frac{\Delta u_c}{\Delta \sigma_3}$$ – Confining pressure or cell pressure (↑)

Δu_c → 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 – $0 \leq B \leq 1$

→ B increase c̄ increase in S% (degree of saturation)

Relationship betn S & B is not linear.

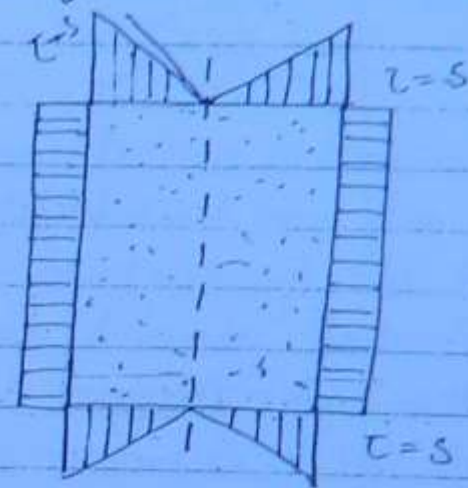
$$B = \frac{1}{1 + \frac{n C_v}{C_c}}$$

C_v 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.

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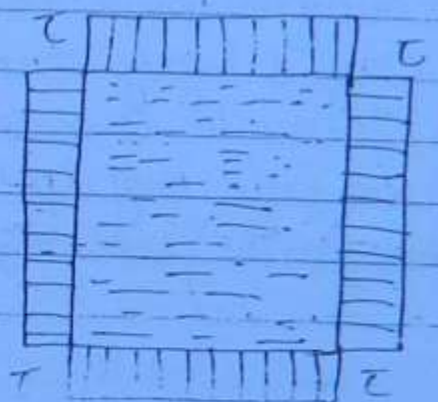
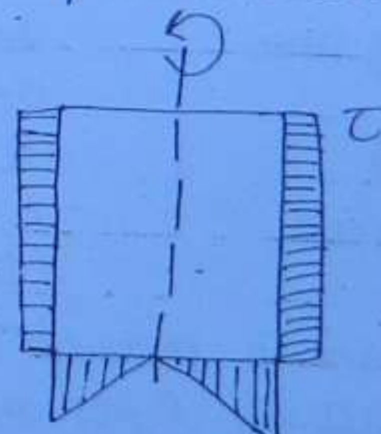


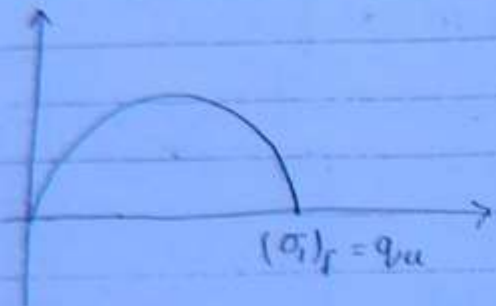
Case (ii) When only bottom & sides participate in shearing

$$S = \frac{T}{\pi D^2\left(\frac{H}{2}+\frac{D}{12}\right)}$$



NOTE - This test can also be used to determine sensitivity of plastic clays.

$$S_t \text{ (Sensitivity)} = \frac{(q_u)_{undisturbed}}{(q_u)_{remoulded}} = \frac{(2C)_u}{(2C)_r} = \frac{C_u}{C_r} = \frac{(S)_u}{(S)_r}$$



Unique Mohr's circle $\bar{c}$ passes from origin

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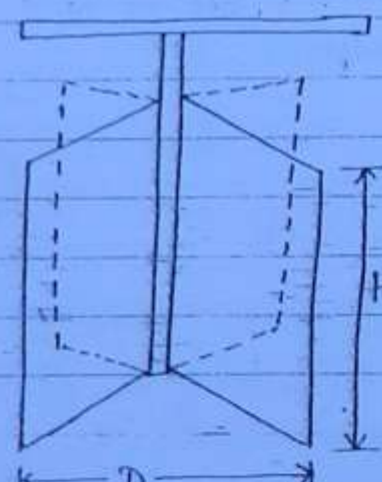
(iv) Vane shear test

This test is essentially undrained & it is suitable for saturated loose clays & highly plastic clays. It can be conducted in lab or field only diff^n 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)	12 mm	5-10 ~~mm~~ cm
3. Vane thickness	0.5-1 mm	2-3 mm

The vane is punched into the soil & torque is applied manually at rate of 1° per minute. The torque is measured at failure to a calibrated torsional spring $\bar{c}$ 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.

(i) 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

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(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)

$$\sigma_3 = \sigma_c = 0$$

$$(\sigma_d)_f = (\sigma_1)_f = q_u$$

NOTE that $\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\beta_{max}}{2}\right) + 2c \tan\left(45^\circ + \frac{\beta_{max}}{2}\right)$

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$\sigma_1 = \sigma_3 \tan^2 \theta_c + 2c \tan\theta_c \qquad \left[\theta_c = 45 + \frac{\phi}{2}\right]$$

For unconfined condition $\sigma_3 = 0$

$$\sigma_1 = 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$$q_u = (\sigma_1)_f = 2c \tan\left(45 + \frac{\phi}{2}\right)$$

Special case - For pure clays $\phi = 0$

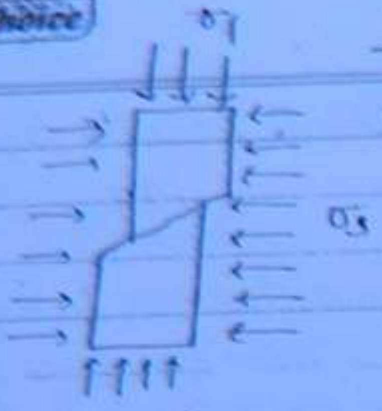
$$q_u = 2c \tan 45^\circ$$

$$\boxed{q_u = 2c}$$

$$\boxed{c = \frac{q_u}{2}}$$

For pure clay $s = c + \sigma \tan\phi$ ($\phi \to 0$)

$$s = c$$



Brittle failure

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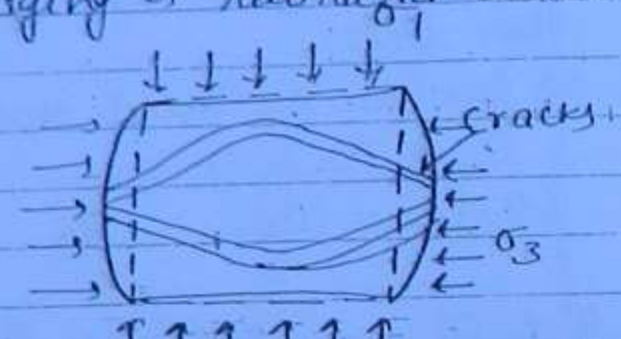
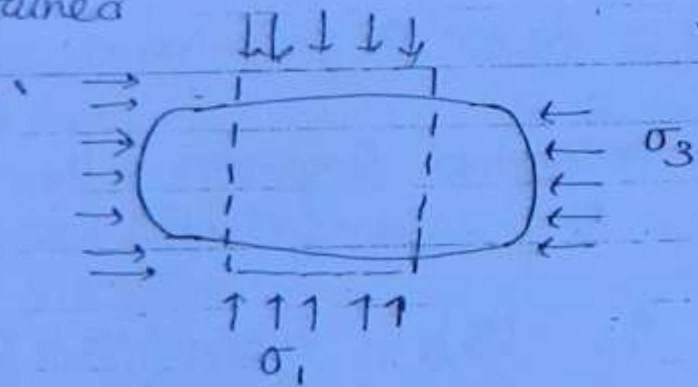
ii) Semi brittle – (Semi plastic failure) – eg Silts (c-ϕ soils) in such failure lateral buldging is recorded & cracks developed as shown in fig



Plastic failure – It occurs in loose sand & normally consolidated clay. Large axial deformation are recorded & high lateral buldging is obtained



Merits of triaxial test

i) Failure occurs along weakest plane & is not predetermined

ii) Pore pressure measurement is possible & vol change can be determine

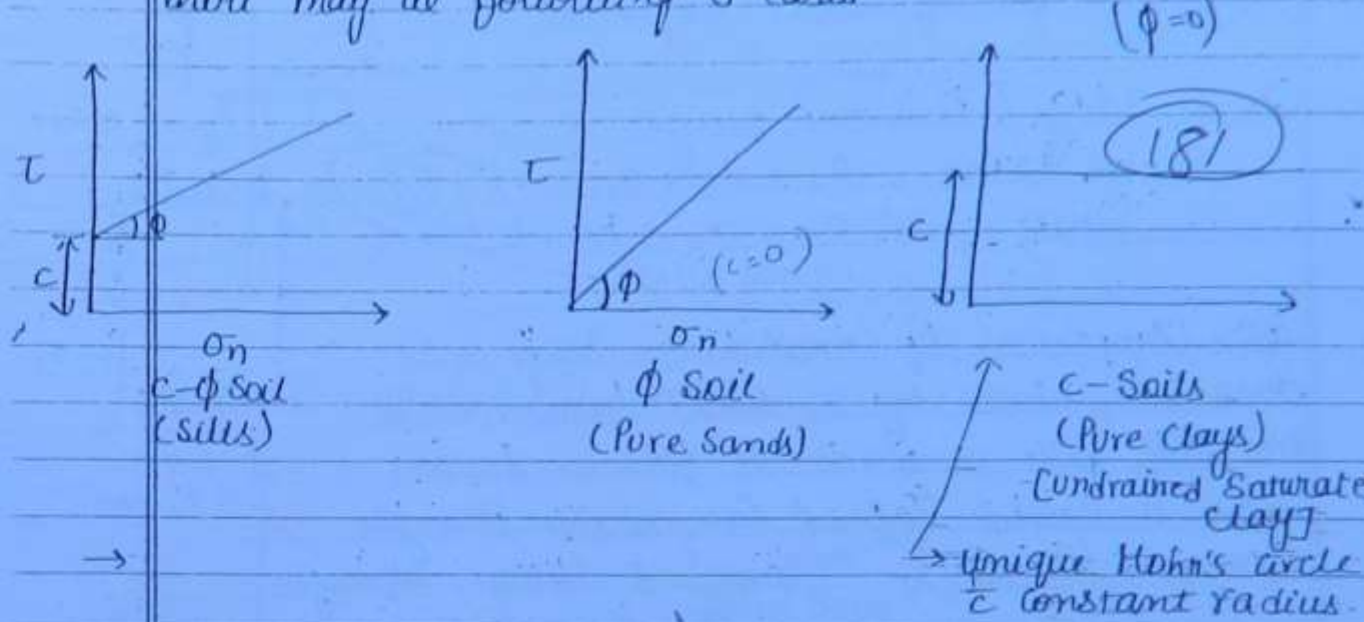
iii) There is complete control over drainage condition

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

NOTE (i) The failure envelope for (loose) 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

$$A_f = \frac{A_0 (1 \pm \epsilon_v)}{1 - \epsilon_L}$$

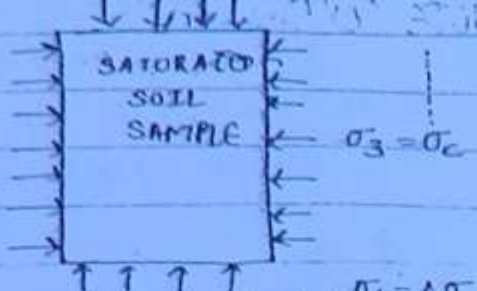
A_0 — Original area
ϵ_v - Volumetric Strain
ϵ_L → axial Strain / Longitudinal Strain

Special Case - If test is undrained then change in vol. is insignificant then ($\epsilon_v \simeq 0$)

$$A_f = \frac{A_0}{1 - \epsilon_L}$$

(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 $\bar{c}$ high O.C.R will occur & will be Completed when Expulsion of pore water is Stopped. The vol Change in Soil will be equal to vol of water expelled



(iii) Shear Stage / Deviator Stage :-

Total Axial Stress $= \sigma_1 = \sigma_3 + \sigma_d$

or $\sigma_1 = \sigma_3 + \Delta\sigma$

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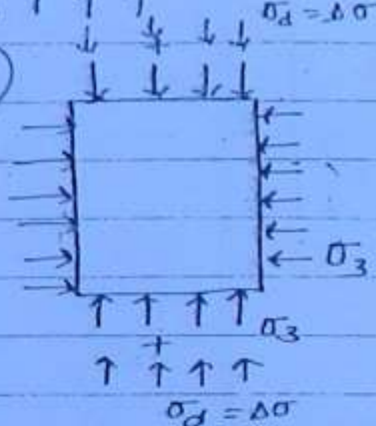


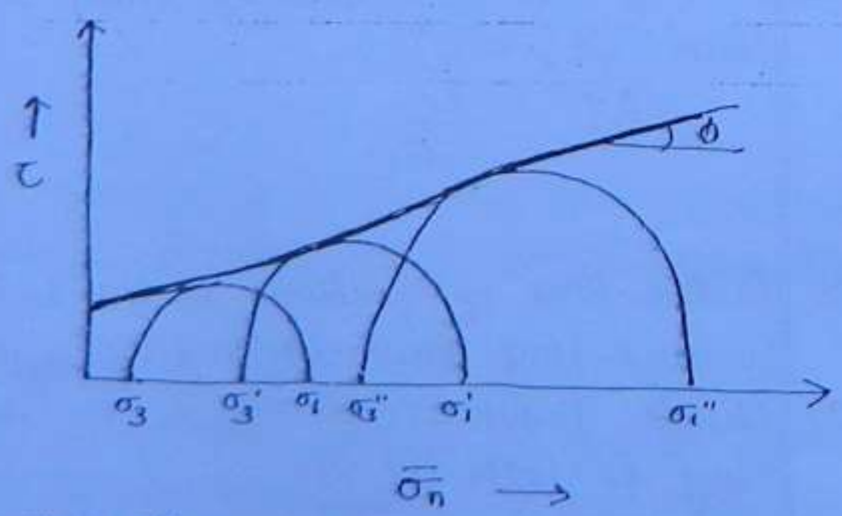
After Completion of Consolidation Stage axial Stress is increased by σ_d called deviator Stress till Soil fails in Shear. at failure deviator Stress is called Confined Compressive Strength

$(\sigma_d)_f = (\sigma_1 - \sigma_3)_f$ Confined Comp. Strength

$= \frac{P}{A_f} \rightarrow$ P = Axial load applied

A_f = Area of Soil at failure

To find Soil parameter (c & ϕ) test is repeated at diffⁿ Shell pressure corresponding axial stress (σ_1) at failure is found & mohr 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 cohesion-less soil as CD test.

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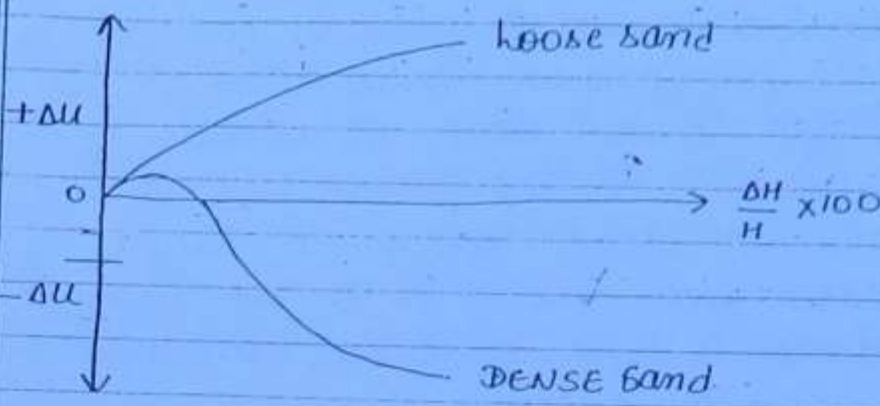
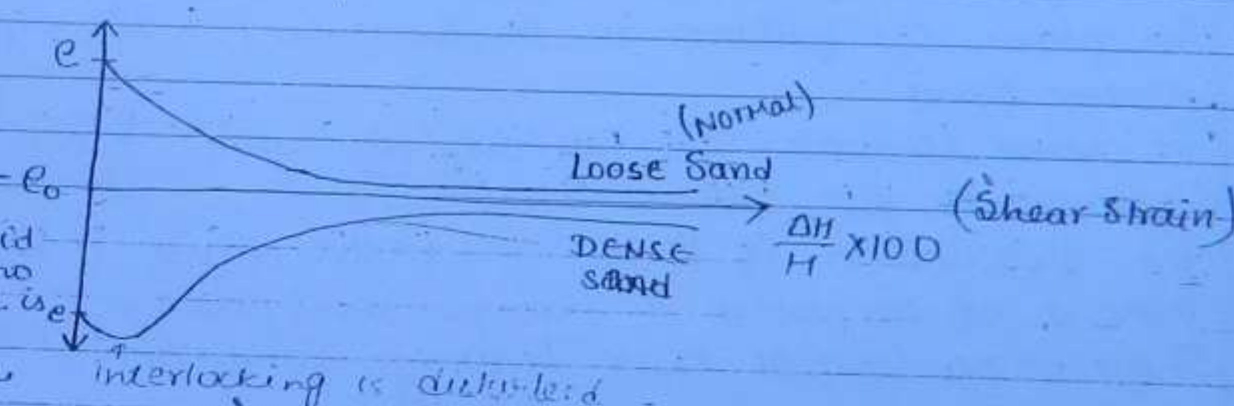
Change in ~~void ratio~~ Pore Pressure v/s Shear Strain Curve



(183)

Ch Void ratio v/s Shear strain curve

critical void ratio is that void ratio at which no u- change is recorded on disturbance



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 en-cased 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 ($\bar{\sigma}_{n_1}$) shear loading is ↑ till failure (τ_1) & test is repeated at diff normal stress ($\bar{\sigma}_{n_2}$) to obtain τ_2 & so on. A graph is plotted b/w $\bar{\sigma}_n$ & τ 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

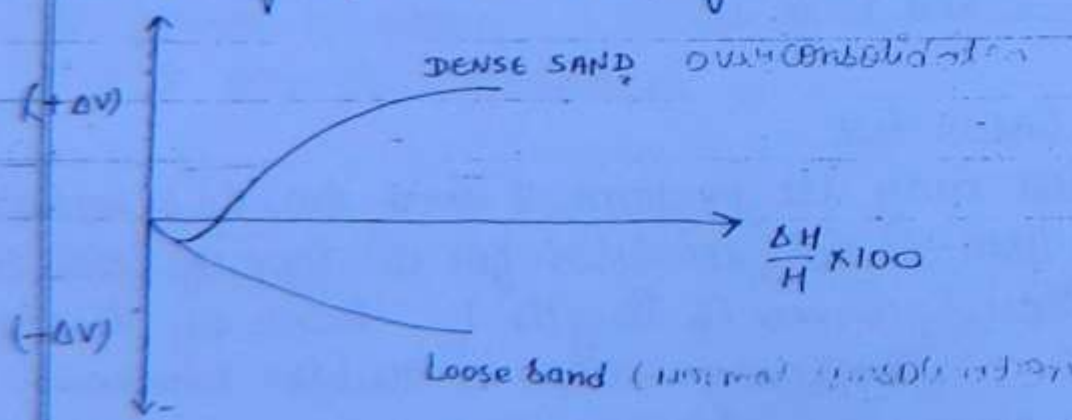
(iii) The distribution of stress on failure plane is not uniform

(iv) There is no-mechanism to measure pore pressure.

(v) There is no control over drainage condition

RESULTS PRESENTATION:—

Vol. change v/s Shear ~~Strength~~ Strain 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 earther 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 vol. change may be expected due to expulsion of pore water. This test is suitable for sand c high permeability & slow rate of loading. eg - stability analysis of retaining walls.

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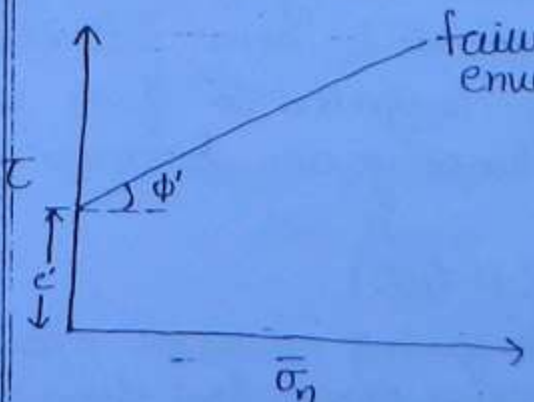
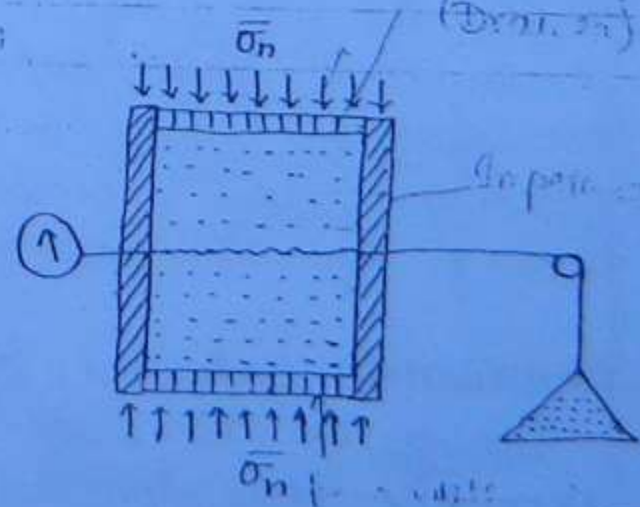
Shear Strength test

(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) Feild Test
(a) Vane Shear test
(b) Penetration test (S.P.T & Cone.P.T)

(i) Direct Shear test





effective stress will come, diretly

Modified Mohr's ~~Colomb'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 ~~shear~~ ϕ may be used but normal stress is significantly affected

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Limitation

1. The theory assumes failure envelope into a straight line c̄ 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 c̄ the depth however in actual practice small ↑ 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)

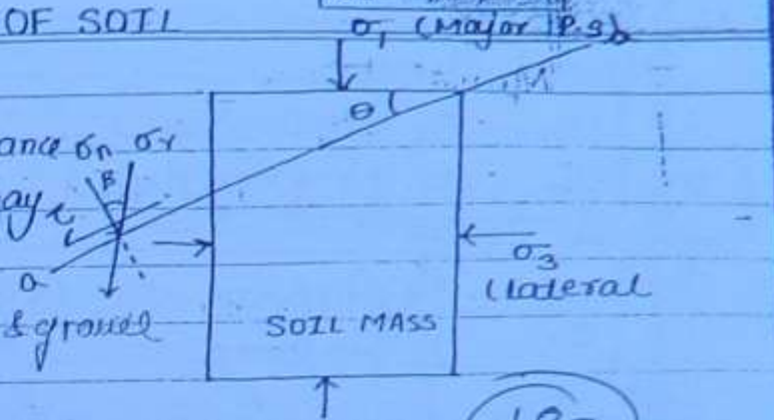
It is a quick test & may be completed in 5 to 10 minutes. In this test neither water is allowed to leave the soil in consolidation stage nor in shear stage. There will be no significant change in vol. of soil. Such test are suitable for clays c̄ low permeability subjected to fast rate of loading. eg - Construction of building over saturated clay.

(ii) Consolidation - Undrained test (C.U test)

During the Ist stage of confining pressure drainage is permitted hence consolidation takes place but during vertical shear loading drainage is not permitted. eg - Stability analysis

(1st Choice)

SHEAR STRENGTH OF SOIL



Definition

Shear strength of soil is resistance to the shear deformation c may be due to

Interlocking b/w particles] sand & gravel
i) Frictional resistance
ii) Cohesion & adhesion (Clay pure).

Constant c depth cohesive
Shear ↑ c 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 particle & frictional resistance.

Shear strength is equal to shear stress on the critical plane a-a. Critical plane is that plane on c resultant stress is most inclined c the normal of that plane ($\beta = \beta_{max}$ & $\theta = \theta_c$) then τ = 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 c shear failure plane.

$$\theta_c = 45° + \frac{\beta_{max}}{2}$$

$$= 45° + \frac{\phi}{2}$$

[$\beta_{max} = \phi$ (friction angle of soil)

For pure ~~pure~~ clays shear strength ⇒ $S = \tau_{max}$
For granular soil shear strength ⇒ $S < \tau_{max}$

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$$

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

Date / /

Normal Stress

$$\boxed{\begin{aligned} \sigma_n &= \sigma_1 (1-\sin\beta_{max}) \\ \text{or} \quad \sigma_n &= \sigma_3 (1+\sin\beta_{max}) \end{aligned}}$$ Imp

$$\boxed{\frac{\sigma_1}{\sigma_3} = \frac{1+\sin\beta_{max}}{1-\sin\beta_{max}} = \frac{1+\sin\phi}{1-\sin\phi}}$$

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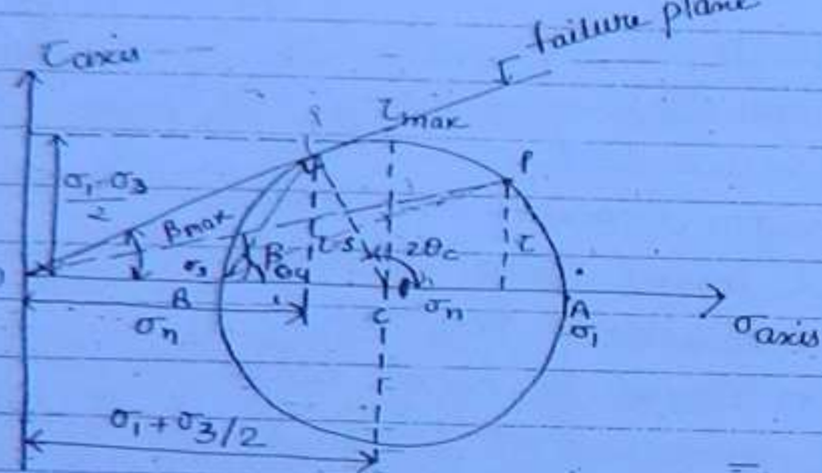
Mohr circle



$OA = \sigma_1$

$OB = \sigma_3$

$OP = \sqrt{\sigma_n^2 + \tau^2}$

$$2\theta_c = \beta_{max} + 90^\circ$$

$$\theta_c = 45^\circ + \frac{\beta_{max}}{2}$$

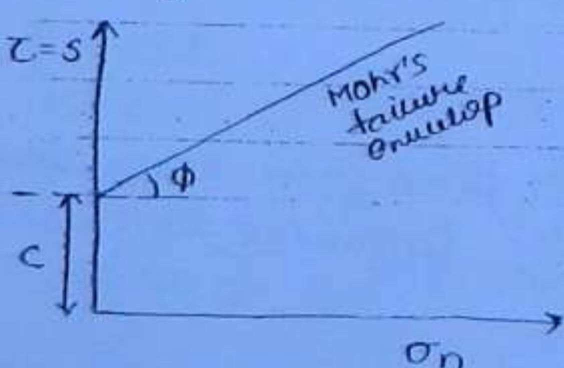
Mohr's Columbus theory of Shear Strength.

$$S = C + \sigma_n \tan\phi$$

C - ~~unit~~ Cohesion (total)

σ_n - normal stress (total) on failure plane

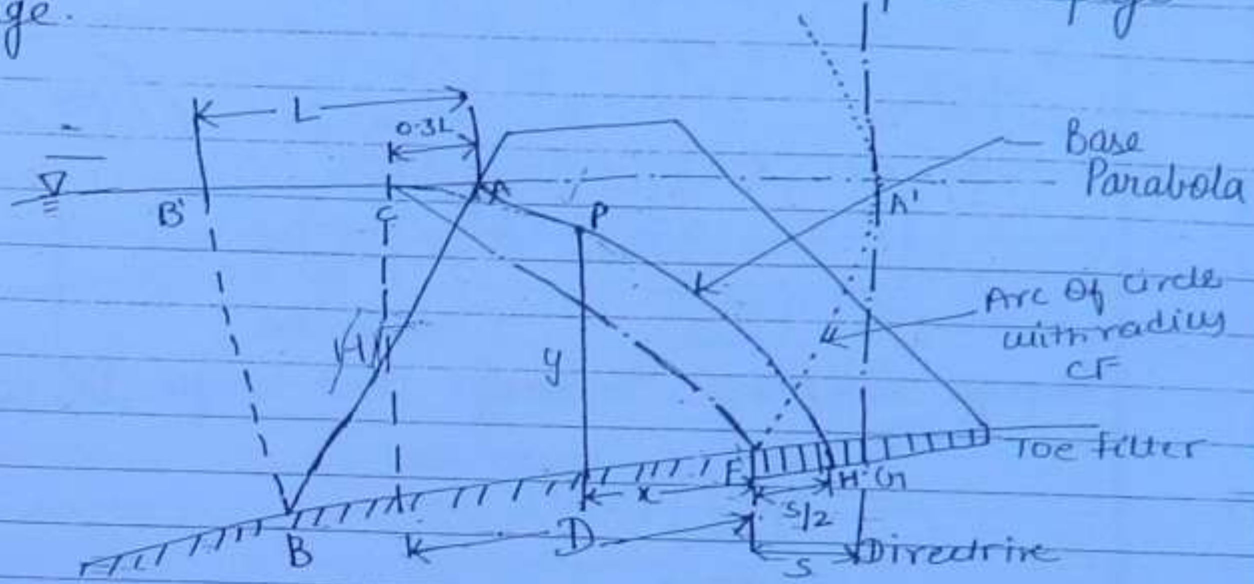
ϕ = total friction angle.



NOTE - The above results are found erronius when soil is submerged because due to presence of pore pressure cohesion is reduced & grain to grain contact pressure is reduced & also friction angle is affected.

as 80%, 70%, 60% . . . so on.

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

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 directrir. Draw a vertical line from A' & meet the filter at G. A'G is directrix. FG = S is focal disn. Hence vertex of parabola is located at S/2 from F (ie H). Draw a parabola & 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 $\bar{c}$ Horizontal dis^n x & vertical dis^n y from focus

$$PF = \sqrt{x^2+y^2}$$
$$= x+S.$$
$$\sqrt{x^2+y^2} = x+S.$$
$$x^2+y^2 = x^2+S^2+2xS$$
$$\boxed{y = \sqrt{S^2+2xS}}$$

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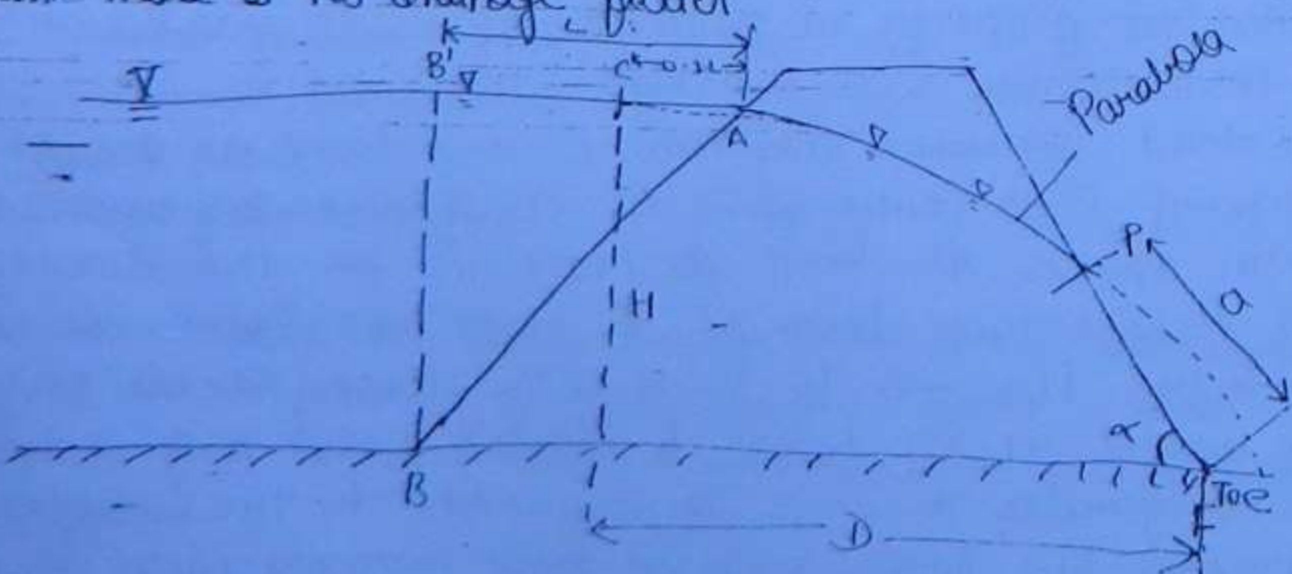
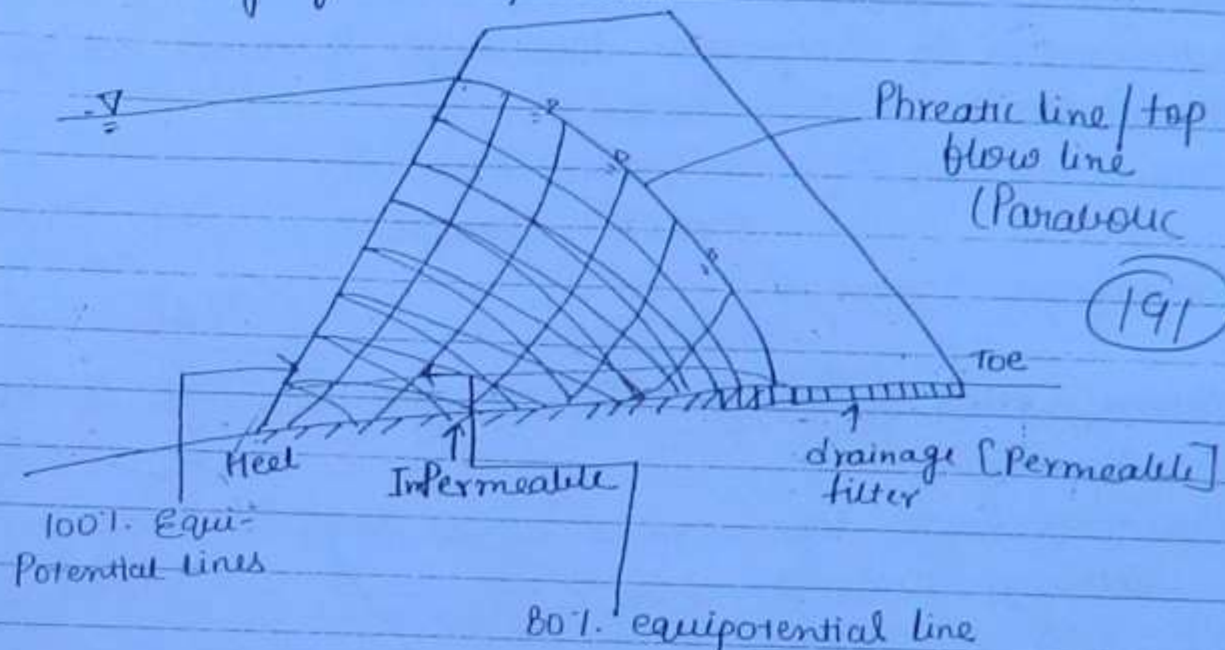
Seepage discharge through unit length of dam

$$q = kiA.$$
$$q = k\left(\frac{dy}{dx}\right)(y \times 1)$$
$$\frac{dy}{dx} = \frac{1}{2}\frac{2S \cdot 2}{\sqrt{S^2+2xS}} = \frac{S}{\sqrt{S^2+2xS}}$$
$$q = k\frac{S}{\sqrt{S^2+2xS}}\sqrt{S^2+2xS}$$
$$\boxed{q = k \cdot S}$$

2. When there is no drainage filter



1. When drainage filter is provided



Phreatic line is top flow line at c 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 U/s 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 at therefore available equipotential decreases

Date / /

Application for flownet
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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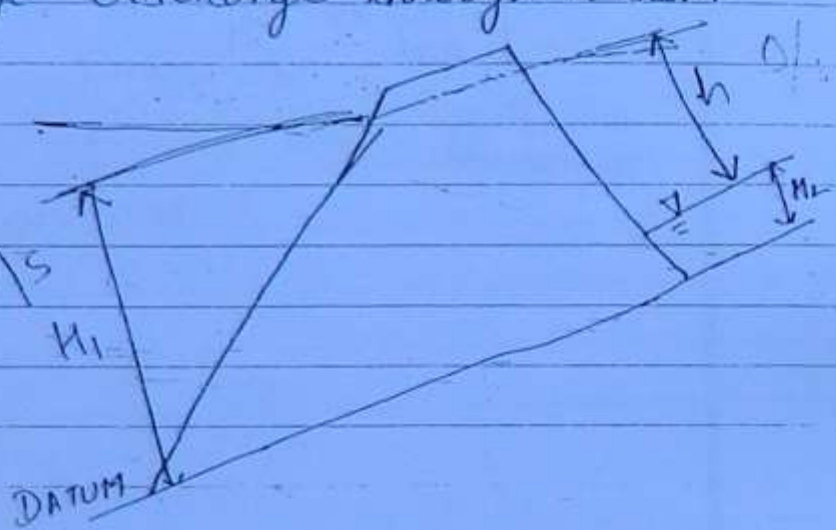
Determination of Seepage Discharge through a dam.

$h = H_1 - H_2$

Consider unit length of dam, discharge through flownet



$\Delta q = KiA$

$\Delta q = K\left(\frac{\Delta h}{b}\right)(b \times 1)$

$= K \Delta h$

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

Let there are N_f No. of flow channels

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

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

$\frac{N_f}{N_d}$ = 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 flowchannel is equal. (Δq) remains Constant hence Continuity eq^n may be applied. b/w two points in a flow channel.

Hence $a_1 v_1 = a_n v_n = \Delta q$

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Let depth of flow is unit then

$(b_1 \times 1) v_1 = (b_n \times 1) v_n = \Delta q$

$\boxed{b_1 v_1 = b_2 v_2 = b_n v_n}$ Imp

b = Size of flow feild.

5. Let there are N_f no. of flow Channels then

N_f = No. of flow lines -1

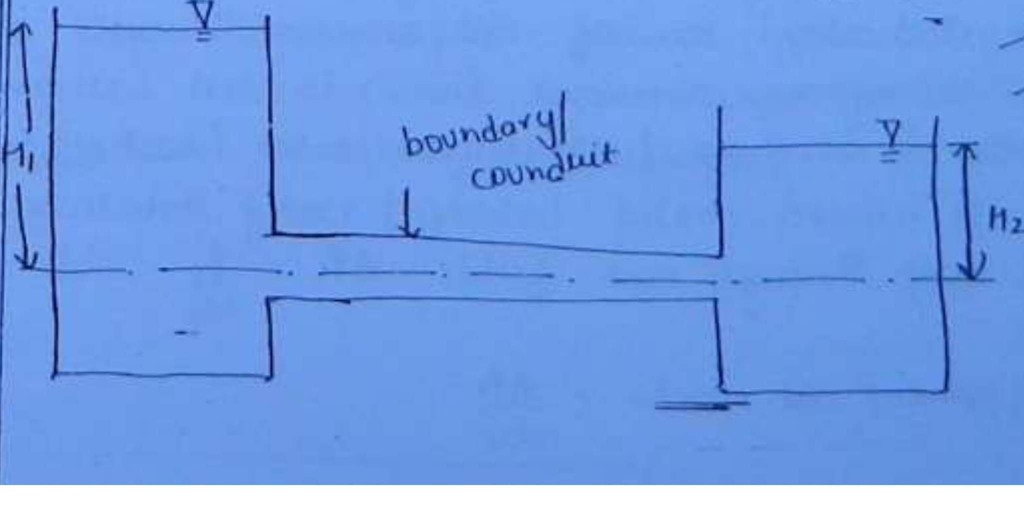
Hence total discharge per unit depth $= q = \Delta q \times N_f$

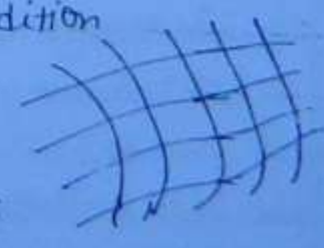
Δq = discharge through flow channel is also equal to diff^n b/w each flow Stream lines.

$\Delta q = \psi_1 - \psi_2$

6. Two equipotential lines or two Stream lines Can never 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 eq,n

(i) Darcy law is valid $Q = kiA$

(ii) Soil is homogenous & 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 do not changes with time.

imp obj

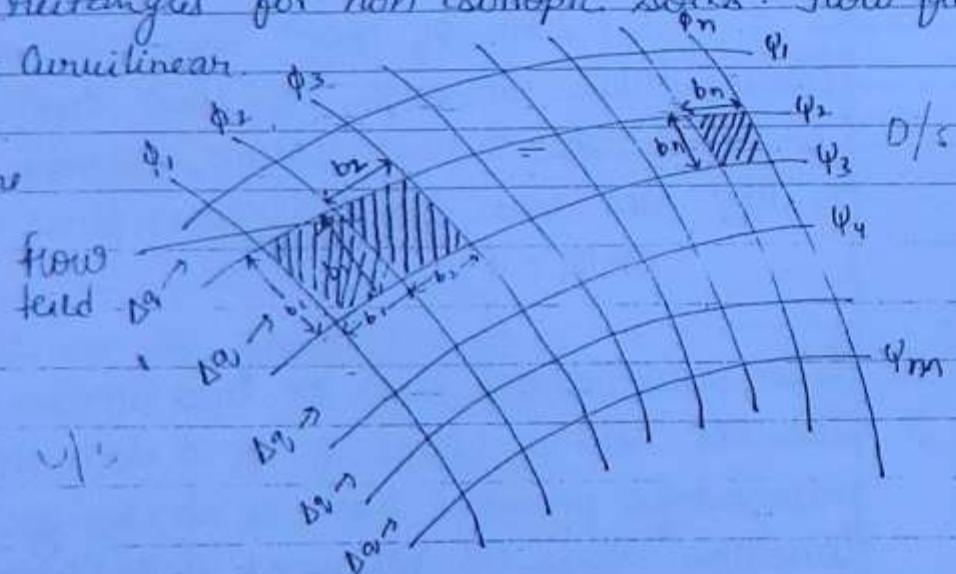
Properties of flownet

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 $= \Delta h = \dfrac{h}{N_d}$

Hydraulic gradient at inlet $= \dfrac{\Delta h}{b_1}$

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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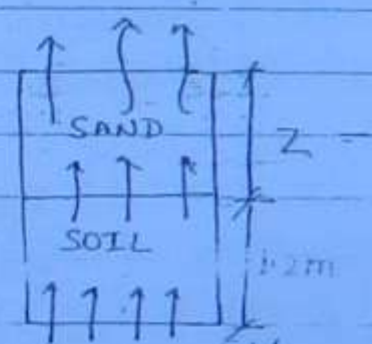
Friday
11/11/2011

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 = 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

$$i_c = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.67} = 1$$



$$i = \frac{i_c}{F.S} = \frac{1}{2.0} = 0.5$$

$$i = \frac{h}{L} \Rightarrow 0.5 = \frac{1.8}{L}$$

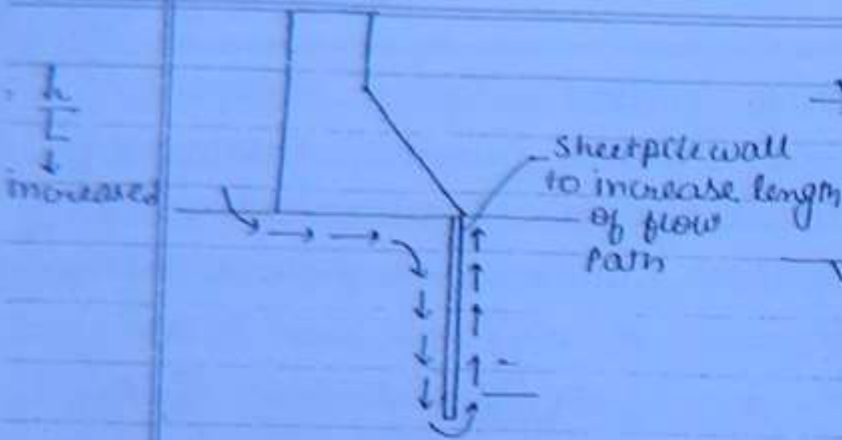
$$L = \frac{1.8}{0.5} = 3.6m$$

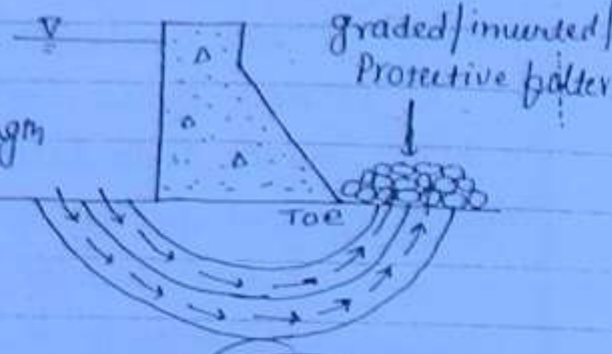
$$L = Z + 1.2m$$

$$3.6 - 1.2 = Z \Rightarrow Z = 2.4\,m$$

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.





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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 $\frac{(D_{15})_{filter}}{(D_{85})_{filter}} \leq 5$

2 $4 < \frac{(D_{15})_{filter}}{(D_{15})_{soil}} \leq 20$

3 $\frac{(D_{50})_{filter}}{(D_{50})_{soil}} \leq 25$

Flownet

see eqn $\nabla^2 H = 0$

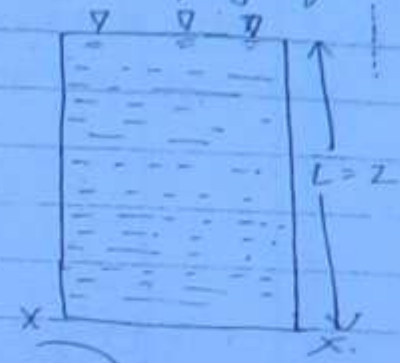
Flownet is graphical representation of solⁿ of laplace eqⁿ

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$$ — 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-istropic Soil

K_x & K_y are permeability Coeffⁿ in X & Y dirⁿ

If soil mass has z vertical depth in c seepage flow is vertically upward. then net eff. pressure at X-X will be



Eff. Stress

$= \bar{\sigma} - p_s$

$= (\sigma - u) - p_s$

$= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - h \cdot \gamma_w$

$= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - \frac{h}{z} \cdot \gamma_w \cdot z$

$= (z \cdot \gamma_{sat} - z \cdot \gamma_w) - i \cdot z \cdot \gamma_w$ 197

$= \gamma_{sub} \cdot z - i \cdot z \gamma_w$

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 c due to vertically upward flow net eff. pressure at any level becomes zero. Under such condition soil particles are floating c may flow c̄ the water such as in case of sand & fine gravels. At Quick Sand Condition

net eff. stress = 0

$$z \cdot \gamma' - i z \gamma_w = 0$$

$$\gamma' = i \gamma_w$$

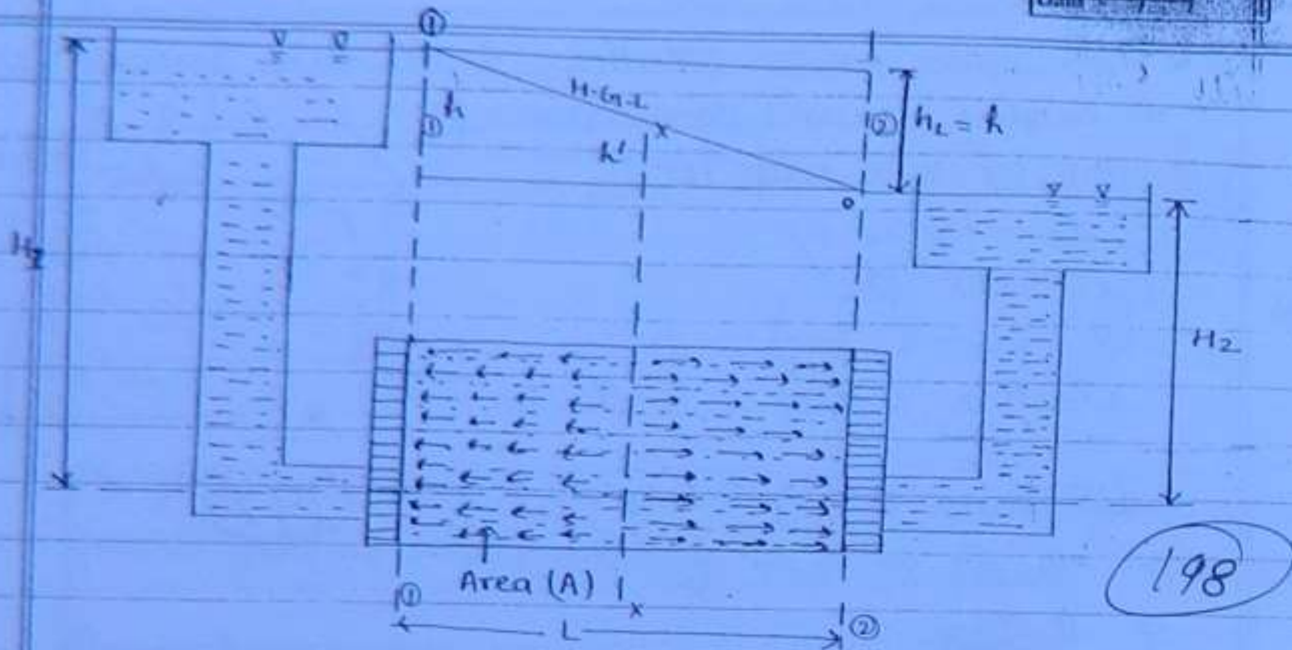
$$i = \frac{\gamma'}{\gamma_w}$$

$$\boxed{i = i_c = \frac{G-1}{1+e}}$$ Critical hydraulic gradient

$$\boxed{i_c \approx 1}$$

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 betn upstream & downstream level). It is also equal to head loss through the soil of length L.

The seepage force is the drag force exerted by the water molecules in the soil molecule c̄ 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 = $p_s \times A = h \cdot \gamma_w \cdot A$

$= \frac{h}{L} \cdot \gamma_w \cdot (A \cdot L) = i \cdot \gamma_w \cdot (\text{Volume})$

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 $= \bar{\sigma} \pm p_s$

+ when flow is downward

− when flow is upward.

Date / /

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

$\Delta H = 18\,cm$ (ultimate settlement)

$$U_1 = \frac{5}{18} = 0.278$$

$$U_2 = \frac{10}{18} = 0.55$$

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$$T_{v_1} = \frac{\pi}{4} U_1^2 = 0.0606 \quad \frac{C_v \cdot t_1}{d_1^2}$$

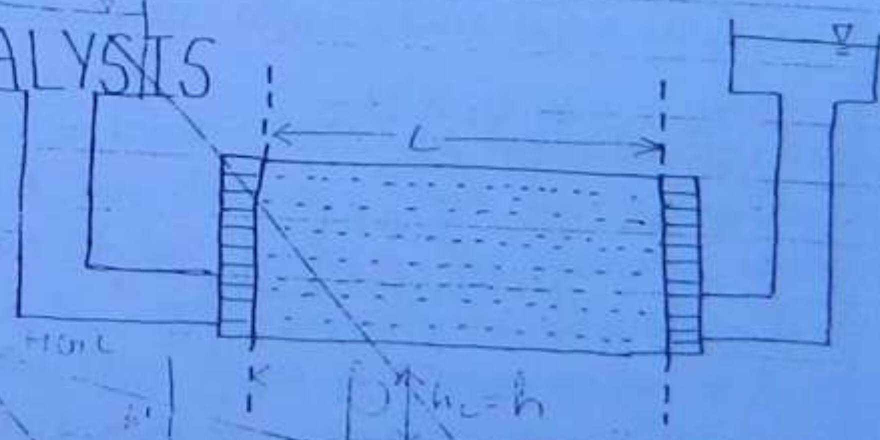
$$T_{v_2} = \frac{\pi}{4} U_2^2 = \frac{C_v \cdot t_2}{d_2^2}$$

$$\frac{T_{v_1}}{T_{v_2}} = \frac{U_1^2}{U_2^2} = \frac{t_1 \cdot d_2^2}{t_2 \cdot d_1^2}$$

$$\frac{t_1}{t_2} = \frac{U_1^2}{U_2^2} =$$

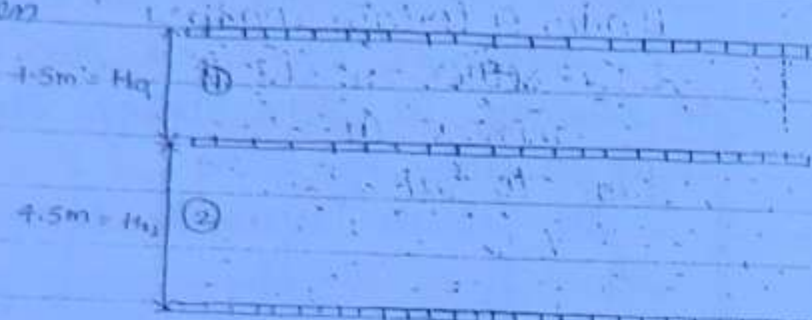
$$t_2 = t_1 \times \frac{U_2^2}{U_1^2} = \frac{0.55^2}{0.278^2} \times 2 = 8\,\text{months}$$

SEEPAGE ANALYSIS



1st Choice

At the same time t when the overall Consolidation U is 0.5



$$T_{v_1} = \frac{\pi}{4} U_1^2 = \frac{C_v \cdot t}{d_1^2}$$

$$T_{v_2} = \frac{\pi}{4} U_2^2 = \frac{C_v \cdot t}{d_2^2}$$

$$\frac{T_{v_1}}{T_{v_2}} = \frac{U_1^2}{U_2^2} = \frac{d_2^2}{d_1^2}$$

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$$\frac{U_1}{U_2} = \frac{d_2}{d_1} = \frac{H_{o2}/2}{H_{o1}/2} = \frac{2.25}{0.75} = 3$$

$$U_1 = 3U_2 \quad \text{———} \quad (1)$$

Let after time t settlement of top layer is Δh_1

" " " " " bottom " " Δh_2

" " " " Overall settlement is $= \Delta h$

Let ΔH_1 is ultimate settlement of layer 1

$$\Delta H_1 = H_{o1} \cdot m_v \cdot \Delta\bar{\sigma}_1$$

$$= H_{o1} \times K$$

$$U_1 = \frac{\Delta h_1}{\Delta H_1} = \frac{\Delta h_1}{H_{o1} \times K} \Rightarrow \Delta h_1 = K \cdot U_1 \times H_{o1}$$

$$\Delta h_2 = k \cdot U_2 \times H_{o2}$$

$$\Delta h = k \cdot U \times H_o$$

$$\Delta h = \Delta h_1 + \Delta h_2$$

$$U \cdot H_o = U_1 H_{o1} + U_2 \cdot H_{o2}$$

$$0.5 \times 6 = U_1 \times 1.5 + 4.5 \times U_2$$

$$3 = 1.5U_1 + 4.5U_2$$

$$U_1 + 3U_2 = 2 \quad \text{———} \quad (2)$$

$$6U_2 = 2$$

$$U_2 = 2/6 = 1/3 = 0.33$$

$$U_1 = 1$$

$$T_{v_2} = \frac{\pi}{4} \times \frac{1^2}{3} = \frac{4.92 \times 10^{-2} \times 10^{-6} \cdot t}{(2.25)^2}$$

$$t = 103.9 \text{ days}$$

(iii) Ballon
(iv) The Sand Cone

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Methods of feild 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 $\bar{c}$ 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.

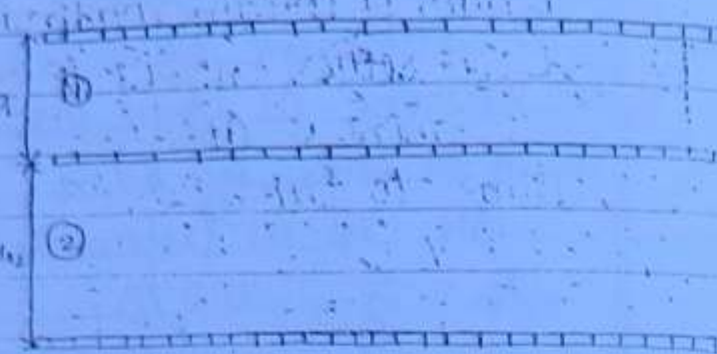
(b) Rammers — These are dropping weights such as frog hammer.

(c) Vibrators — These are suitable for Cohesionless Soil & Concrete work especially when soil is in confined stage

Ques 6 m 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 4.92×10^{-2} mm²/sec. If the structure is to be constructed above the clay layer $\bar{c}$ cause uniform ↑ in clay stress within the thickness of clay. How much time will be require to attain 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.

At the same time t when the overall consolidation U is 0.5



$$T_{v_1} = \frac{\pi}{4} U_1^2 = \frac{C_v \cdot t}{d_1^2}$$

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200

$$\frac{U_1}{U_2} = \frac{d_2}{d_1} = \frac{H_{o2}/2}{H_{o1}/2} = \frac{2.25}{0.75} = 3$$

$$U_1 = 3U_2 \quad \text{——— (1)}$$

Let after time t settlement of top layer is Δh_1

" " " " " bottom " " Δh_2

" " " " overall settlement is $= \Delta h$

Let ΔH_1 is ultimate settlement of layer 1

$$\Delta H_1 = H_{o1} \cdot m_v \cdot \Delta \bar{\sigma}_1$$
$$= H_{o1} \times K$$

$$U_1 = \frac{\Delta h_1}{\Delta H_1} = \frac{\Delta h_1}{H_{o1} \times K} \Rightarrow \Delta h_1 = K \cdot U_1 \times H_{o1}$$

$$\Delta h_2 = k \cdot U_2 \times H_{o2}$$

$$\Delta h = k \cdot U \times H_o$$

$$\Delta h = \Delta h_1 + \Delta h_2$$

$$U \cdot H_o = U_1 \cdot H_{o1} + U_2 \cdot H_{o2}$$

$$0.5 \times 6 = U_1 \times 1.5 + 4.5 \times U_2$$

$$3 = 1.5U_1 + 4.5U_2$$

$$U_1 + 3U_2 = 2 \quad \text{——— (2)}$$

$$6U_2 = 2$$

$$U_2 = 2/6 = 1/3 = 0.33$$

$$U_1 = 1$$

$$T_{v_2} = \frac{\pi}{4} \times \frac{1^2}{3} = \frac{4.92 \times 10^{-2} \times 10^{-6} \cdot t}{(2.25)^2}$$

$$t = 103.9 \text{ days}$$

Date / /

(iii) Ballon

(iv) The Sand Cone

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(b) Rammers - These are dropping weights such as ~~frog~~ frog hammer.

(c) Vibrators - These are suitable for Cohesionless soil & Concrete work especially when soil is in Confined stage

Ques 6m thick Clay layer is located b/w 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 4.92×10^{-2} mm^2/sec. If the structure is to be constructed above the clay layer c̄ cause uniform ↑ in effective stress within the thickness of clay. How much time will be require to ~~produce~~ attain 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.

→ 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

→ 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

$$\frac{(E)_{heavy}}{(E)_{light}} = \frac{(MgH)_{heavy} \times \text{No. of layer} \times \text{No. of blows}}{(MgH)_{light} \times \text{No. of layer} \times \text{No. of blow}}$$

$$\frac{4.9 \times g \times 450 \times 5 \times 25}{2.6 \times g \times 310 \times 3 \times 25} = 4.56$$

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

(1st Choice)

Compaction of Soils

Compaction is an instantenous process in partially saturated soil by c density is increased by ↑ing external pressure. Compaction curve betn 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 $\rho_d = \frac{\rho}{1+\omega}$. The test is repeated

at diffn water content & following graph is found

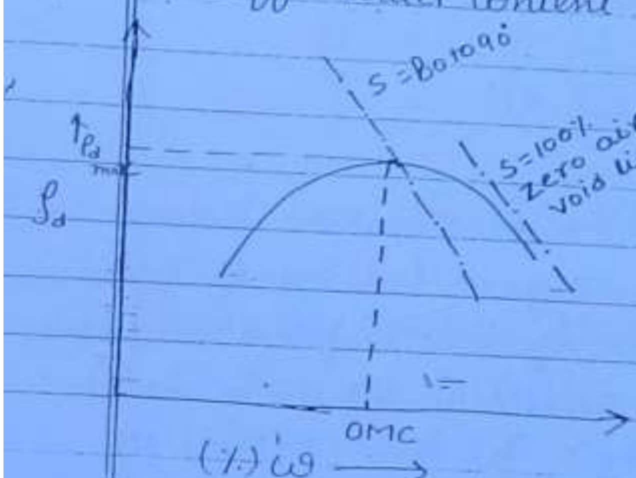


fig 1

When density is max. degree of satturation is 80 to 90%. becoz pratically it is not feasible to expell all the air. However theoretically max-dry density will be atcheived when all the air is expelled out. (i.e S = 100%.) Degree of compactive effect is constant (i.e 25 blows are given)

If compactive effort is ↑ then greater max dry density will be atcheived at smaller optimum moisture content

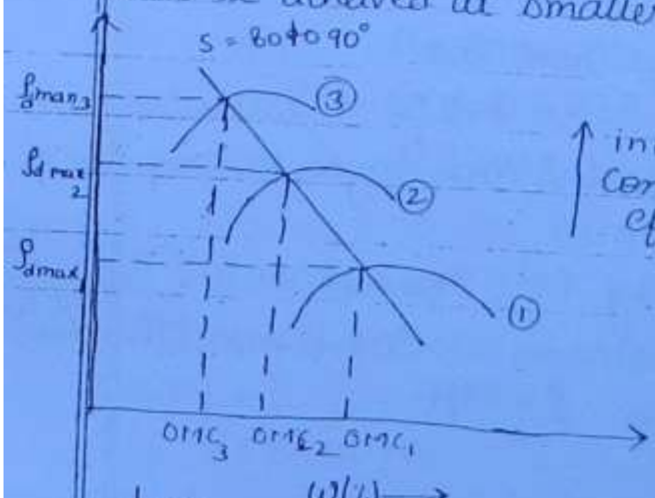


fig 2

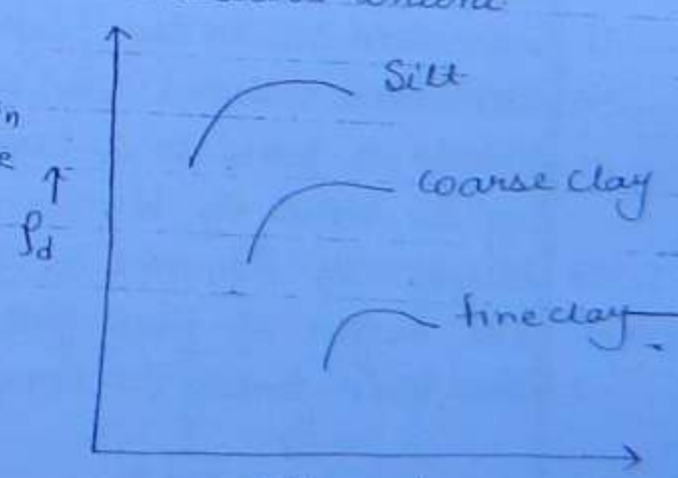
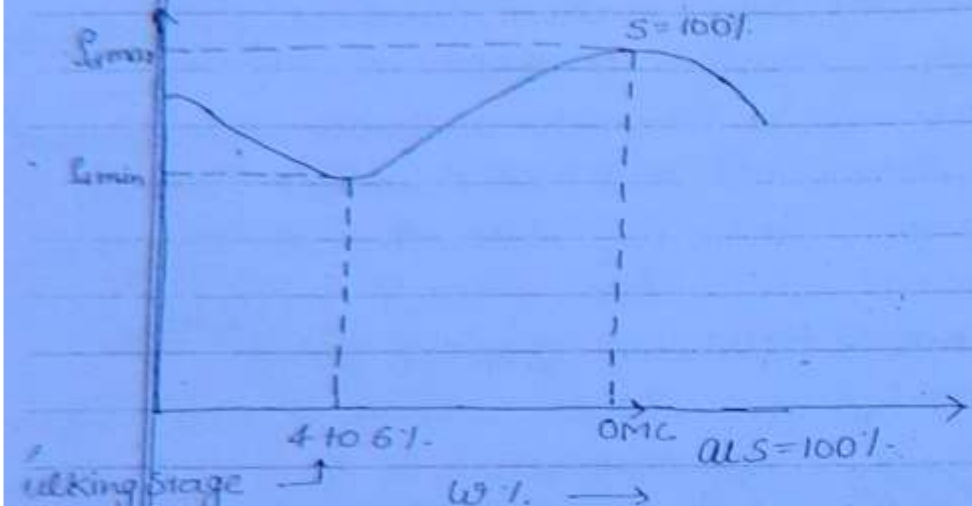


fig 3

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 which results in max. volume such an stage is bulking of sand. At this water content a thin layer of water film is available around the solid particle which have repulsive effect. In sand max. density is acheived 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 2.459 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

at C-C $\bar{\sigma}_0 = \sigma - u$ (total)

effective

$$10 \times \gamma_{sat} + 3 \cdot \gamma_{sat} - 13 \times \gamma_w$$

$$10 \times 19.62 + 3 \times 19.62 - 13 \times 9.81$$

$$= 127.53$$

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CASE II If water table is lowered by six metre then

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

$$= 6 \times \gamma_{sat} + 4 \times \gamma_{sat} + 3 \times \gamma_{sat} - 7\gamma_w$$

$$= 186.39 \text{ kN/m}^2$$

Increase in eff stress at C-C

$$\Delta\bar{\sigma} = 186.39 - 127.53 = 58.89 \text{ kN/m}^2$$

$$m_v = 0.02 \text{ cm}^2/\text{kg}$$

$$= \frac{0.02 \times 10^{-4}}{9.81 \times 10^{-3}} = \frac{m^2}{kN} = 2.038 \times 10^{-4}$$

Settlement Of Clay

$$\Delta H = H_0 m_v \cdot \Delta\bar{\sigma}$$

$$= 6m \times 2.038 \times 10^{-4} \times 58.89 = .0720 \text{ m}$$

$$= 72 \text{ mm}$$

$e = 0.28$ A saturated soil has compression index Of 0.25 if voids ratio [illegible] 205 [illegible] mm/sec
Compute (i) [illegible]
[illegible] 81.6 kN/m² & [illegible]
is [illegible]

1st Choice

In the laboratory test on a clay sample of thickness 25mm drained at top only 50% consolidation occured in 11 minutes. Find the time required for the same clay layer in the field having 2m thickness & drained at top & bottom both. To undergo 70% consolidation given that time factor $T_{50} = 0.197$ & $T_{70} = 0.405$

Soln Lab test $d = 25mm = H_0$ (single drainage)

$t = 11$ minutes for 50% consolidation

$T_{50} = 0.197$ $\quad U = 0.5$

$$T_{50} = \frac{C_v \cdot t_{50}}{d^2} \Rightarrow C_v = \frac{0.197 \times 25^2}{11} \; mm^2/min$$

Field Condition $H_0 = 2m$

$$d = \frac{2}{2} = 1m \text{ (double drainage)}$$

$$T_{70} = 0.405$$

$$T_{70} = \frac{C_v \, t_{70}}{d^2}$$

$$t_{70} = \frac{T_{70} \times d^2}{C_v} \Rightarrow \frac{0.405 \times 1^2 \times (1000)^2 \times 11}{.197 \times 25^2} \; \frac{mm^2 \cdot \cancel{min}}{mm^2/min} \; min$$

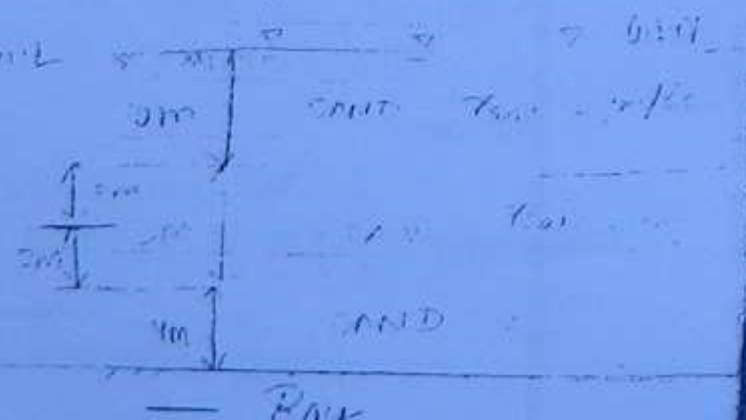
$$min \xrightarrow{\div 24 \times 60} days \quad = 25.13 \text{ days.}$$

At a given site the soil profile is shown ... water table is lowered six meter ...

$\gamma_w = 1 \, gm/cc$

$\gamma_{sat} = 2 \times 9.81 = 19.62 \; KN/m^3$

$\gamma_w = 9.81 \; KN/m^3$



A layer of soft clay 5m thick lies under newly constructed building. The eff pressure due to overlying strata on the ~~centre of~~ clay layer is 3kg/cm² & new construction increases the over burden pressure by 1.2 kg/cm². If compression index of the clay is 4.5 then compute the settlement of the clay assuming natural water content of clay 43% & G = 2.7, degree of saturation = 100%.

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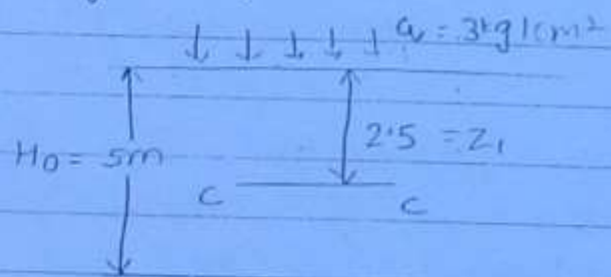


Initial eff over burden pressure at centre of clay layer

$$\bar{\sigma}_0 = q + \gamma_{sub} \cdot Z_1$$

$S \cdot e = w \cdot G$

$e = 1.161$

$$\gamma_{sub} = \frac{G-1}{1+e}\gamma_w \Rightarrow \frac{2.7-1}{1+1.161} 9.81$$

$$= 7.72 \ kN/m^3$$

$$q = 3kg/cm^2 = \frac{3 \times 9.81 \times 10^{-3}}{10^{-4}} = 294.3 \ kN/m^2$$

$$\bar{\sigma}_0 = q + \gamma_{sub} \cdot Z_1$$

$$= 294.3 + 7.72 \times 2.5$$

$$= 313.6 \ kN/m^2$$

$$\Delta\bar{\sigma} = 1.2 \ kg/cm^2 = 1.2 \times 9.81 \times 10^{-3} / \times 10^{-4}$$

$$= 117.72 \ kN/m^2$$

$C_c = 4.5$

$e_0 = 1.161$

$$S_c = \Delta H = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

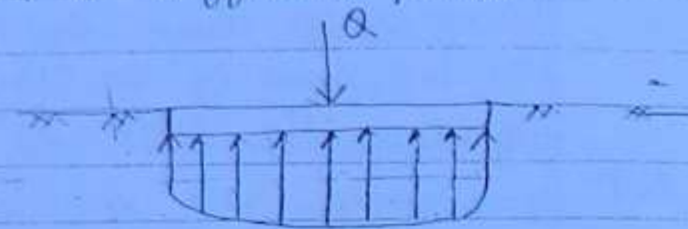
$$= \frac{4.5 \times 5}{1+1.161} \log_{10}\left(\frac{313.6 + 117.7}{313.6}\right)$$

= 1.44 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

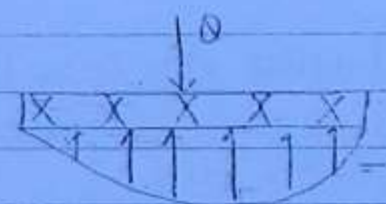
Flexible foundation
(for all soils)



2. Rigid foundation Such as Raft - In Rigid foundation the settlement is uniform but pressure distribution depends upon type of soil

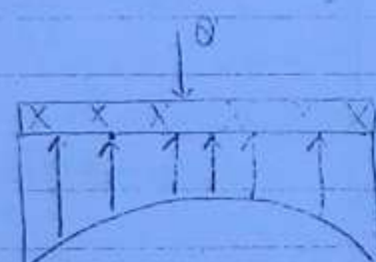
(1) Cohesionless soil (sand)

zero at edge



Rigid foundation in Sand

Max at edge



Rigid foundation in Clayey soils

Max at

$$S_c = \overbrace{\frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 - \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}^{S_{c_1}} + \overbrace{\frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_1 - \Delta\bar{\sigma}'}{\bar{\sigma}_1}\right)}^{S_{c_2}}$$

where $\bar{\sigma}_1 = \bar{\sigma}_0 - \Delta\bar{\sigma}$

$(\bar{\sigma}_0$ to $\bar{\sigma}_1$ $(\bar{\sigma}_0 + \Delta\bar{\sigma}))$ → over Consolidated

$\bar{\sigma}_1$ to $\bar{\sigma}_1 + \Delta\bar{\sigma}'$ → 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 pratical 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.

$$\boxed{S_s = \frac{C_s H_{100}}{1+e_{100}} \log_{10}\left(\frac{t}{t_{100}}\right)}$$

(100% Consolidation)

here H_{100} = thickness of soil layer after primary 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 which secondary consolidation is required.

$t >>> t_{100}$

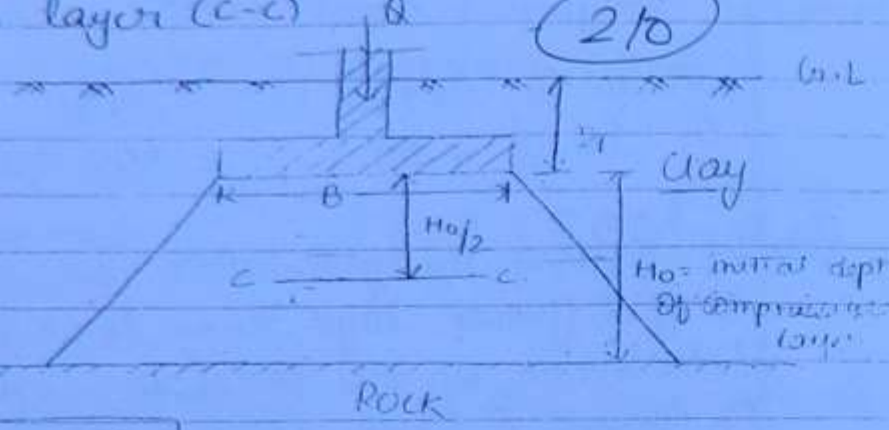
2 If m_v is coefficient of volume compressibility at the centre of compressible layer & $\Delta\bar{\sigma}$ = increase in effective stress at the centre of compressible layer (c-c)

(210)

Find $\bar{\sigma}_0$ at c-c

" $\Delta\bar{\sigma}$ at c-c

" m_v at c-c



$$\boxed{\Delta H = S_c = H_0 \cdot m_v \cdot \Delta\bar{\sigma}}$$

III Since $\Delta\bar{\sigma}$ & m_v vary c̄ 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

$$\Delta H = \Sigma H \, m_v \cdot \Delta\bar{\sigma}$$

H = height of any layer

If Coefficient of Compression then

$$\boxed{\Delta H = S_c = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

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.

IV (1) If soil is over consolidated then use C_r instead of C_c

(2) If for initial part soil is over consolidated or for later part soil is normally consolidated then

$S_c \quad S_{c_1} + S_{c_2}$!

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
$C_s - 1.5(C_r/\bar{\sigma}_0)$ — $C_r \rightarrow$ Static cone pore resistance in kN/m²
H_0 = Initial depth of soil layer

(211)

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2. 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 due to deformation of clay & squeezing of water.

Immediate settlement below a rectangular flexible foundation is given by

$$S_i = \frac{q \cdot B}{E_s}(1-\mu^2) I_t$$ Schleicher's eq^n

q = Uniform pressure at the base of foundation
B = Width of foundation elastic (Plastic condition)
μ = Poission ratio of soil ($0.2 - 0.5$)
For soft saturated clays under undrained condition → 0.5
E_s = Young's modulus of soil
I_t = Influence factor or shape factor depends upon shape & rigidity of foundation.

rangular ndation & a smaller nension circular ning B = Dia the circular tion.

Determination of Consolidation Settlement (S_c)

1. Let $\Delta H = S_c$ is settlement of soil layer due to applied eff. stress which causes change in volume which results into change in void ratio.

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

Δe = Change in void ratio
H_0 - initial depth
e_0 - initial void ratio

$$\Delta H = S_c = \left(\frac{\Delta e}{1+e_0}\right) H_0$$

n organic soil — S_s assumes great significance ... proportion of total settlement.

(2) Primary Compression ration (r_p)

$$r_p = \frac{R_0 - R_{100}}{R_i - R_f}$$

(3) Secondary Compression ratio (r_s)

$$r_s = \frac{R_{100} - R_f}{R_i - R_f}$$

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NOTE THAT $\boxed{r_i + r_p + r_s = 1}$

SETTLEMENT ANALYSIS

Total settlement $S = S_i + S_c + S_s$

S_i = immediate settlement

S_c = Primary Consolidation settlement or Simply Consolidation settle

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 $\frac{1}{1000}$

(... independent)

Determination of Immediate settlement (take place immediately or less than a 1 day after the load is placed)

1. For Cohesionless soil (sands) – To determine immediate settlement Standard penetration test data may be used

$$S_i = \frac{H_0}{C_s} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

1st Choice | K = m/sec or cm/sec
T_v = dimensionless

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Determination Of Cv (Coefficient Of Consolidation)

Cv depends on type of soil & ↓ c̄ ↑ in liquid limit.
For most of the soils its value lies betn 5×10^{-4} mm²/sec to 2×10^{-2} mm²/sec

Thou Through value of Cv Changes c̄ Change in effective stress but for the purpose of Calculation its avg. value is taken

There are two methods to determine Cv

1. Square root of time fitting method

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 c̄ have high secondary Consolidation such as highly plastic Clays

2. Logarithm of time fitting Method

It was given A Cassagrande

$$C_v = \frac{T_{50} \cdot d^2}{t_{50}}$$

T_{50} = 0.196 or 0.197

Compression Ratios

Let R_i is initial reading of dial gauge, R_0 is reading of dial gauge at the beginning of primarily 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

$$r_i = \frac{R_i - R_0}{R_i - R_f}$$

at $t = 0$, $e = e_0$
at $t \Rightarrow \infty$ $e = e_{100}$

Time factor (T_v)

It is the parameter which realates 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 which time factor is T_v
d = length of drainage path

Note: 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 $\frac{H_0}{2}$ (double drainage)
d_f = final length of drainage path at the time of completion of consolidation $\left[d_f = \frac{H_0 - \Delta H}{2}\right]$ (double drainage)

T_v is realated to degree of Consolidation $\Rightarrow$ $\boxed{T_v = \frac{\pi}{4}(U)^2}$
valid — when $U \le 0.6$

$\boxed{T_v = -0.9332 \log_{10}(1-U) - 0.0851}$ when $U > 0.6$

(1) If degree of consolidation is 50% then $U = 0.5$ then T_{50} $\boxed{T_v = \frac{\pi}{4}(0.5)^2 = 0.196}$

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 90% consolidation is assumed to be computed.

$$m_v = \frac{-\left(\Delta H/H_0\right)}{\Delta\bar{\sigma}} = \frac{-\left(\Delta e/1+e_0\right)}{\Delta\bar{\sigma}} = \frac{a_v}{1+e_0}$$

a_v = Coefficient of Compressibility

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Degree of Consolidation

It is the fraction of Consolidation stage completed at any time to the total primarily Consolidation

$0 \le U \le 1$

$0\% \le U \le 100\%$

At beginning $t = 0$ $U = 0\%$

At Completion $U = 100\%$

Theoritically 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 $\boxed{\frac{\Delta h}{\Delta H} \times 100 = \% U}$

2. Let u_i ($= \Delta\bar{\sigma}$) is initial excess pore pressure developed due to ↑ in eff. stress ($\Delta\bar{\sigma}$). Let u is excess pore pressure at any stage of time then degree of Consolidation is given as $\boxed{\frac{u_i - u}{u_i} \times 100 = u\%}$

At Begining ($t=0$) $u = u_i \Rightarrow \% u = 0$

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

$$\% u = \frac{e_0 - e}{e_0 - e_{100}} \times 100$$

$$\boxed{\frac{\partial U}{\partial t} = C_v \cdot \frac{\partial^2 U}{\partial z^2}}$$

$\frac{\partial U}{\partial t}$ = Rate of change of pore pressure $\bar{c}$ the time i.e rate of consolidation

C_v = Coefficient of consolidation $\underline{c}$ depends upon type of Soil & is defined as $\boxed{C_v = \frac{K}{\gamma_w \cdot m_v}}$

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 $\bar{c}$ the depth

Inverse of m_v is called Compression modulus

$$\boxed{E_c = \frac{1}{m_v}}$$

Coefficient of volume Compressibility is defined as Ratio of unit vol. Change to the Corresponding Change in effective Stress.

$$\boxed{m_v = \frac{-\left(-\frac{\Delta V}{V}\right)}{\Delta \bar{\sigma}}} \quad \frac{m^2}{KN}$$

$m_v \uparrow \bar{c} \downarrow$ in $\Delta\bar{\sigma}$

IE Since Soil mass is Consider Semi infinite (area is large & Constant & depth is finite $\underline{c}$ may change) Hence $-\Delta V/V = \frac{\Delta H}{H_0}$

$H = H_0$ = initial height

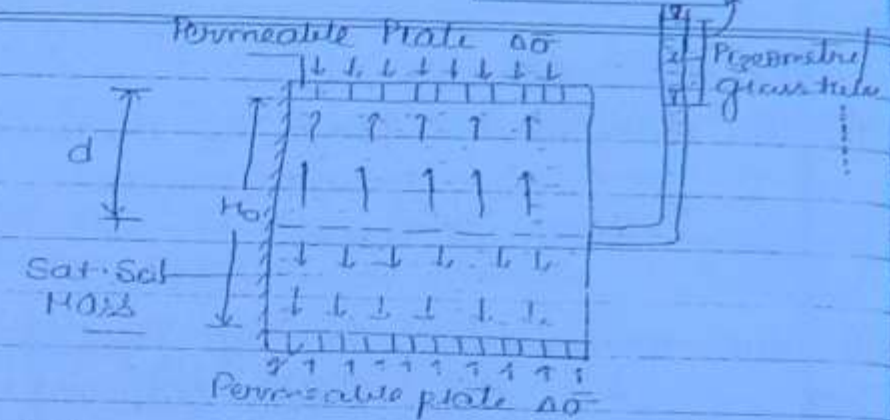
$$\frac{\Delta V}{V} = \frac{A \Delta H}{A H_0} \rightarrow \frac{\Delta V}{V} = \frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

e_0 = initial void Ratio

Δe = Change in void Ratio due to Change in effective Stress

$$\boxed{\frac{\Delta V}{V} = \frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}}$$

(217)



Let top & bottom both plates are permeable hence Soil is in double drainage system. If only ~~p~~ 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

$$u = u_i = \Delta\bar{\sigma} \quad \text{(At time } T = 0)$$

After completion of primary consolidation excess pore pressure reduces to 0. So At time $t = \infty$

Theoritically Consolidation will be Completed in time $t = \infty$ but practically Consolidation will Complete in finite time & generaly when degree of ~~Consolod~~ Consolidation is $\geq$ to 90%. it is assumed to be Completed

The laboratory Conditions are designed analogous to feild Condition (i.e if feild soil is in double drainage then Consolidometre has top & bottom face permeable & if ~~fet~~ feild Soil is in Single drainage then either top or bottom plate is permeable in Consolidometre

The rate of Consolidation $\propto \frac{1}{D^2}$ $\qquad \frac{\partial V}{\partial t} \propto \frac{1}{D^2}$

D = length of drainage path.

The one dimensional eq^n of Consolidation given by terzaghi is

Determination of C_c

① For undisturbed clays of medium sensitivity

$$C_c = 0.009 (\omega_L - 10)$$

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ω_L = % liquid limit

② For remoulded soil of medium to low sensitivity

$$C_c = 0.007 (\omega_L - 10)$$

③ If natural moisture content is given then

$$C_c = 0.115\, \omega_n$$

ω_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

Terzaghi 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 $\Delta\bar{\sigma}$)

The hydrodynamic lag is consider but plastic lag is ignored where as plastic lag is known to exists. It means theory is valid for primary consolidation not secondary consolidation ratio of

Over Consolidation ratio. - It is the max. eff. Stress upto soil is stressed in the past to the present applied Effective stress

$$O.C.R = \frac{\text{Max applied eff. Stress in past}}{\text{Present applied eff. Stress.}}$$

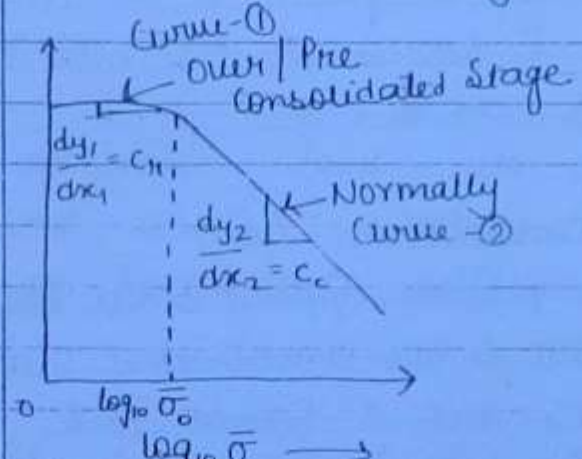
For O.C soil O.C.R will be greater than 1.

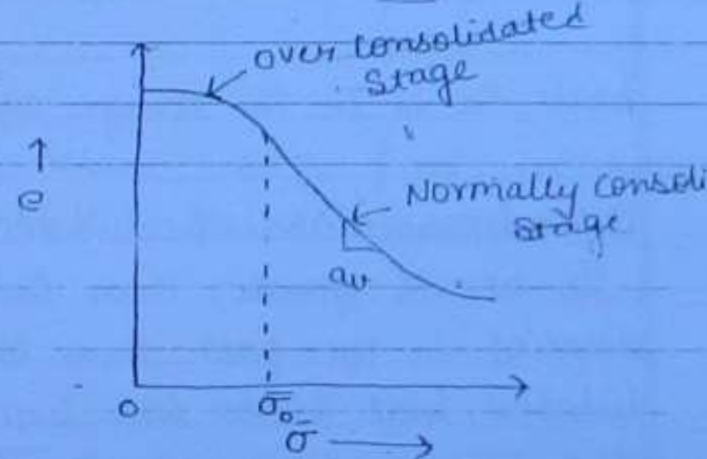
Heavily pre consolidated soils have O.C.R >> 1

OTC 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

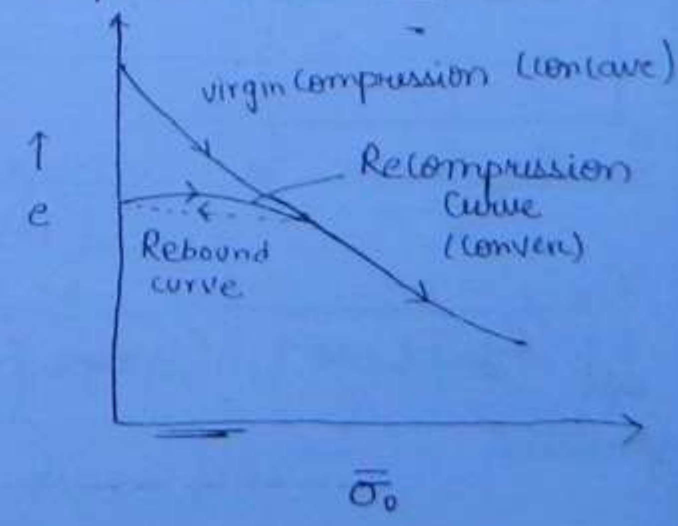




Slope of Curve 1 is Called C_r

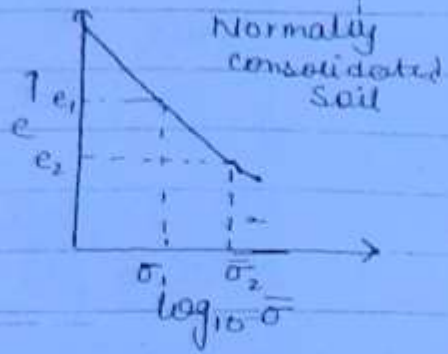
Slope of Curve 2 is Called C_c

Cyclic loading (Eff Stress v/s void ratio Curve)



a_v - Coefficent of Compressibility depends upon type of soil & magnitude of load

If ↑ in eff. stress is taken in log scale then curve is found to be straight line & slope is called coeff. of compression



coeff of compression $\boxed{C_c = \dfrac{-\Delta e}{\log_{10}(\bar{\sigma}_2/\bar{\sigma}_1)}}$

$$\boxed{C_c = \dfrac{-\Delta e}{\log_{10}\dfrac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}}}$$

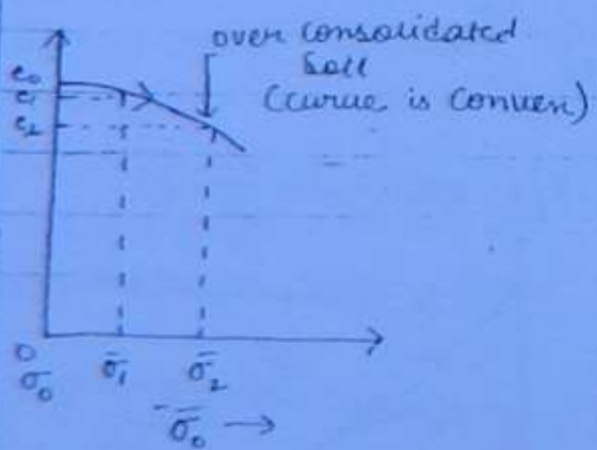
C_c does not change $\bar{c}$ change in eff. Stress but it depends on type of soil. Greater the mag. of C_c greater is the Compressibility & Settlement. For most of the soils C_c is in the range of 0.1 to 0.8.

Over Consolidated Soils / Reconsolidated Soils

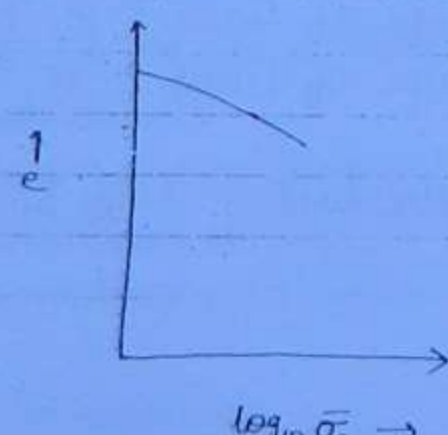
If stress greater then the present applied stress has been applied in the past then soil is over consolidated. Over consolidated soil show less settlement $\bar{c}$ slower rate.

Eff. Stress v/s void ratio Curve

numeric scale



log scale



$$C_r = \frac{-\Delta e}{\log_{10}\left(\dfrac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)} = \frac{-\Delta e}{\log_{10}\left(\dfrac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)}$$

coeff of recompression

$$\boxed{C_r = \left(\frac{1}{5} \text{ to } \frac{1}{10}\right) \text{ of } C_c}$$

From load settlement curve Q_{up} is known

hence $Q_{ap} = \frac{Q_{up}}{F.O.S.}$ FOS - 2.5 to 3.

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 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) -

$$\boxed{Q_{up} = (400N)A_b + (2\bar{N})A_s}$$

FOS = 4.0 for driven piles

2. For bored piles :-

$$\boxed{Q_{up} = (133N)A_b + \left(\frac{2}{3}\bar{N}\right)A_s} \quad \boxed{FOS - 2.5}$$

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/cm²)

Let q_c = cone resistance at base of pile

$\bar{q_c}$ = Avg. cone resistance in kg/cm² over the depth of pile

then ultimate load on pile in KN is given as -

$$\boxed{Q_{up} = q_c A_b + \frac{\bar{q_c}}{2} A_s}$$

where A_s & A_b are in m²

& Q_{up} in KN.

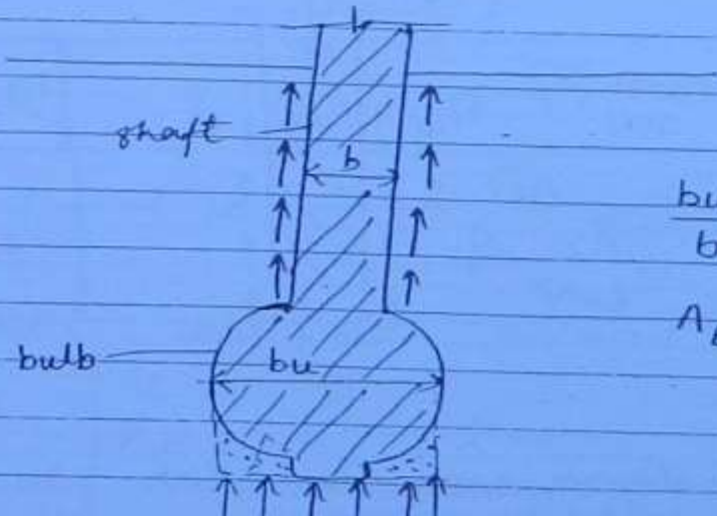
⇒ For non displacement Pile / H Pile Q_{up} is taken as 50% of Q_{up} of displacement pile.

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 is 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.

Under reamed piles cannot be driven hence these are essentially bored piles



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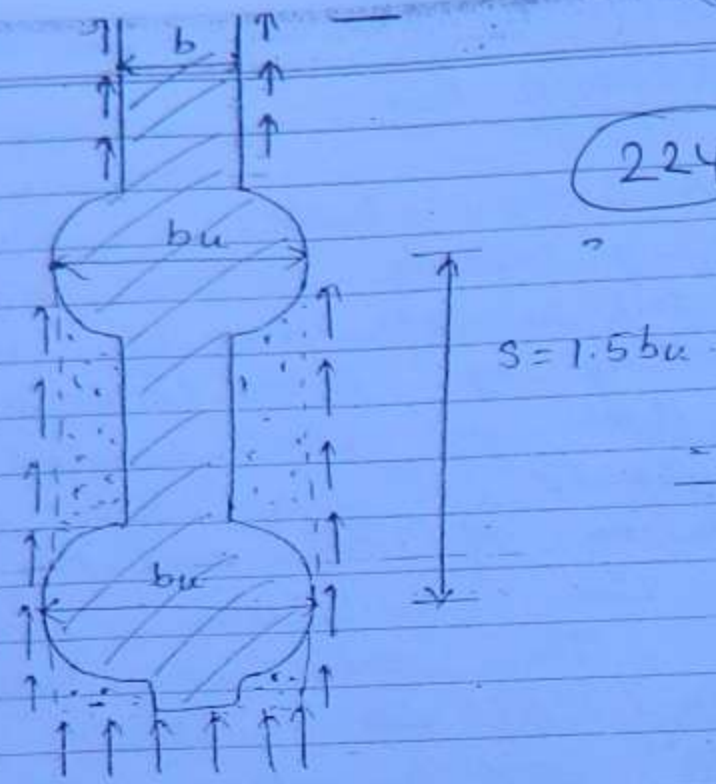
$$\frac{b_u}{b} = 2 \text{ to } 3$$

$$A_b = \frac{\pi}{4}(b_u)^2$$

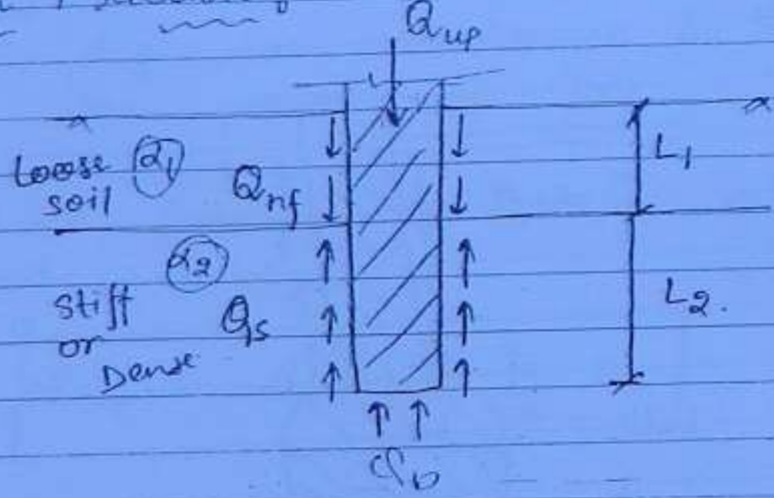
(Single bulb U/R Pile)

$$Q_u = c \underbrace{N_c}_{9} \overbrace{A_b}^{\text{Area of bulb}} + \underbrace{\alpha}_{0.4} c A_s$$



Multibulb U/R Pile.

Negative Skin Friction :-



It is the phenomenon in which surrounding soil to the pile settles more than the settlement of pile. This condition occurs when surrounding soil is very loose or soft & pile rest over stiff strata. Following condition may cause negative skin friction

1) increase in surcharge over surrounding soil.
2) lowering of GWT
3) disturbance due to earthquake & other dynamic effects.

The negative skin friction reduces the load carrying capacity of pile

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NSF $\boxed{Q_{nf} = \bar{q}_{s_1} (\pi D) L_1}$

$\bar{q}_{s_1}$ = avg. unit skin resistance over depth L_1

For cohesive soils $\boxed{\bar{q}_{s_1} = \alpha_1 \bar{c}_1}$

SWR $Q_s = \bar{q}_{s_2} (\pi D) L_2$

$\bar{q}_{s_2} = \alpha_2 \bar{c}_2$

$Q_b = q_b \cdot A_b$

$Q_b = (9c) \frac{\pi}{4} D^2$

The F.O.S will be reduced and it is given by

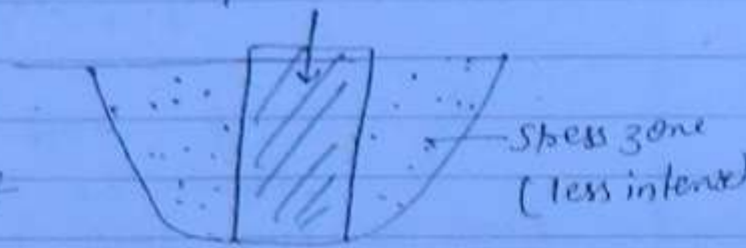
$$\boxed{F.O.S = \frac{Q_{up}}{\text{working load} + Q_{nf}}}$$

Stress Zone in loaded Piles :-

1. End bearing Resistance :-

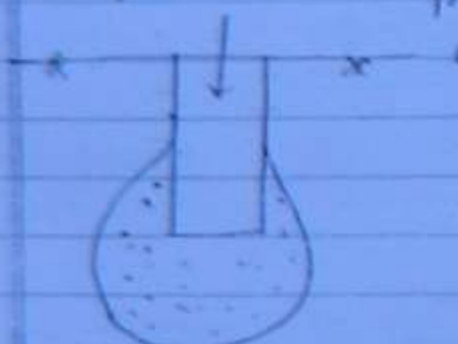
= (end bearing resistance).

+



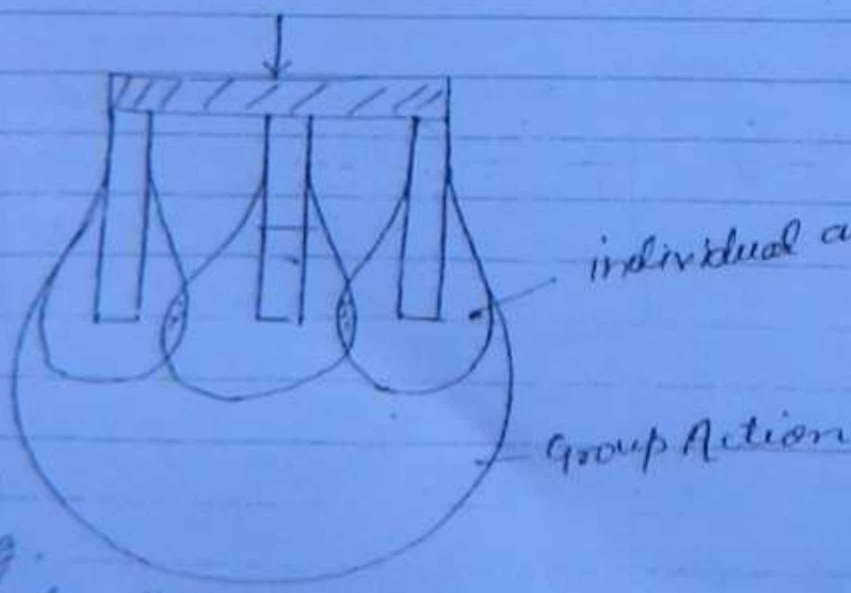
Friction action

equivalent/eff stress zone in bearing & friction



Group action of pile :-

If load applied is larged & more number of piles are used then piles may act individually or in group action depending upon the spacing.



If spacing is less than 4d (d = dia) the soil may get

compacted b/w piles & these may be group action.
In group action all piles & compacted soil b/w piles act at 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 prefered.

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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 then the end bearing piles.

Determination of ultimate ~~bea~~ load carrying capacity of pile load :-

$$Q_{ug} = Q_b + Q_s$$

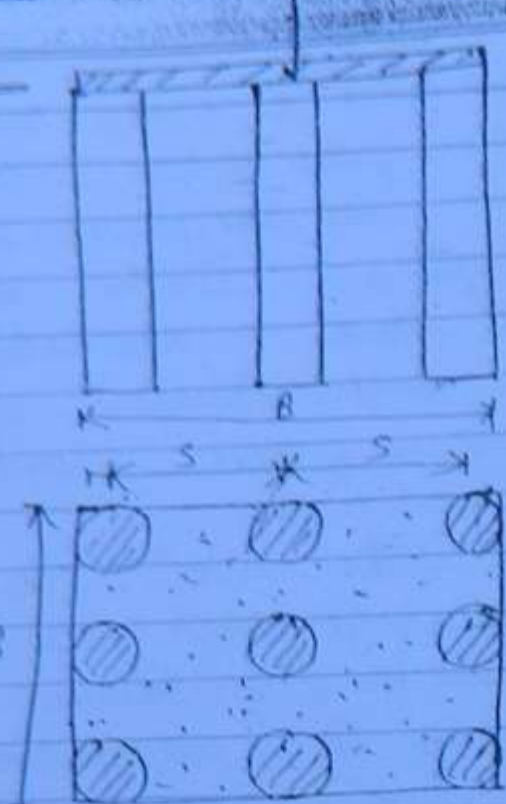
$$= q_b A_b + q_s A_c \rightarrow 1$$

$$Q_{ug} = (9c) b^2 + (\alpha c)(4B \cdot L)$$

= 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 pipes
S = c/c spacing
B = Size of pile group



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 -

α = 0.6 to 0.9 (loose clay)
α = 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 :- c/c 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 $\boxed{\eta_g = \frac{Q_{ug}}{n \cdot Q_{up}}}$

6) No. of Piles :- Generally a square pile group is prefered 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 :-

$$S_r = \frac{\text{settlement of pile group } (S_g)}{\text{settlement of individual pile } (S_i)}$$

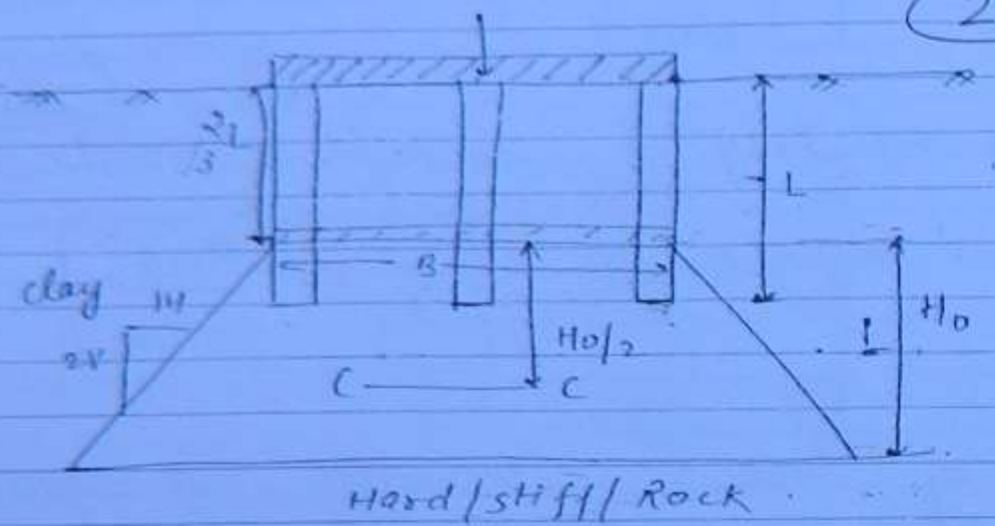
$$S_r = \frac{S_g}{S_i}$$

> 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 :-

1. When piles are embaded on a uniform clay deposit and piles act as friction piles.



It is assumed that applied load acts on equivalent raft of size (B×B) at a depth $\frac{2}{3}L$ from the GL. The load below the raft is assumed to be distributed with 2V : 1H distribution. The compressible soil layer has initial thickness H_0 located below the raft.

Let C–C is the centre of the compressible soil layer at the depth of $H_0/2$ below the raft.

Let $\bar{\sigma}_0$ = initial eff. overburden pr. at C–C

$\overline{\Delta\sigma}$ = increased in eff. stress at C–C due to load on pile group which is assumed to act at the equivalent raft.

Hence settlement of pile group = consolidation settlement of clay layer below the raft.

$$\boxed{S_g = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma_0} + \bar{\Delta\sigma}}{\bar{\sigma_0}}\right)}$$

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 :- .

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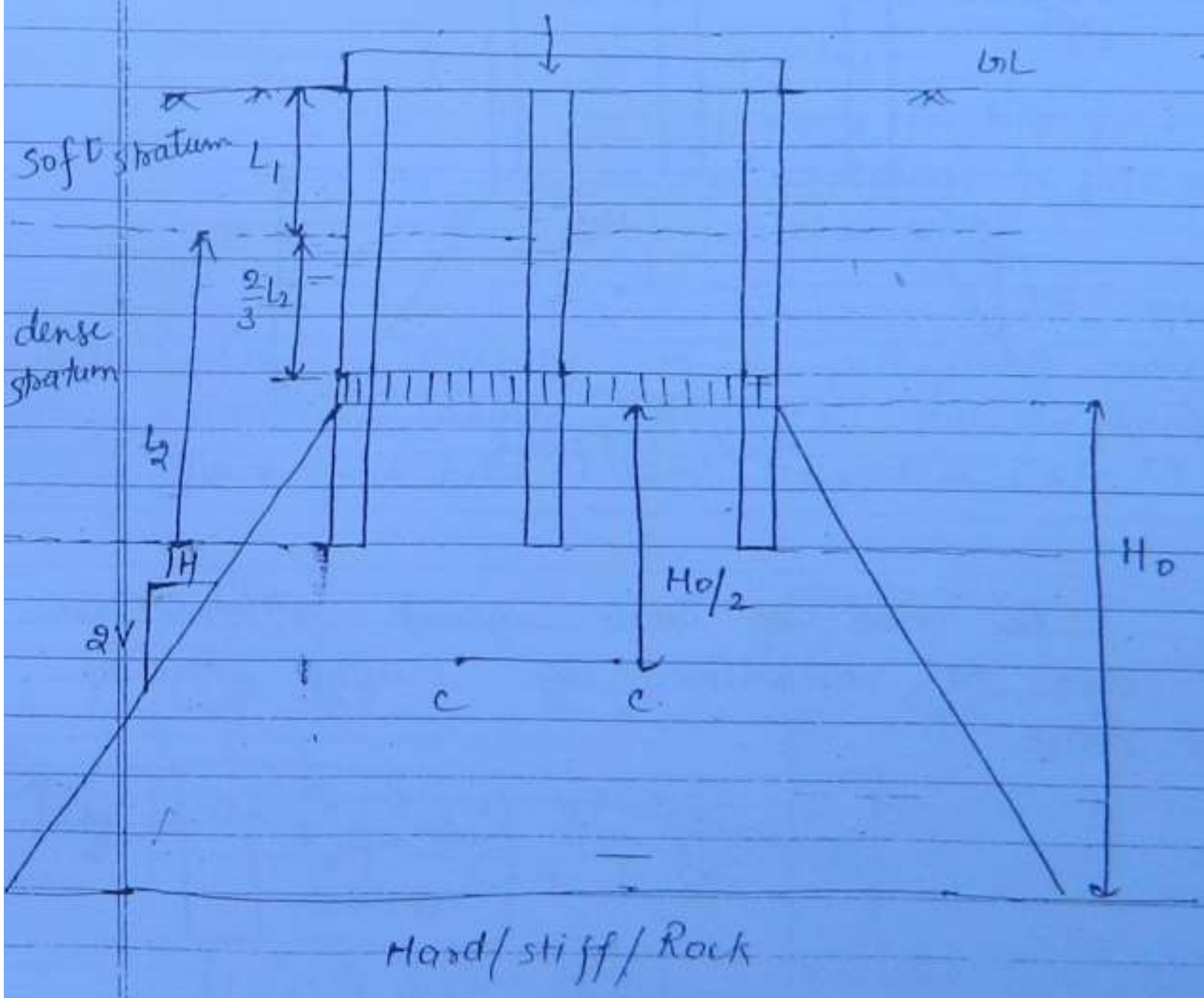

The equivalent raft is assumed at $\frac{2}{3}L_2$ below the soft stratum and rest of the procedure is similar to previous case.

Case 3. If Pile group is end bearing resting on a firm clay the equivalent raft is assumed at the base of pile i.e on the top of firm clay and the rest procedure is same.

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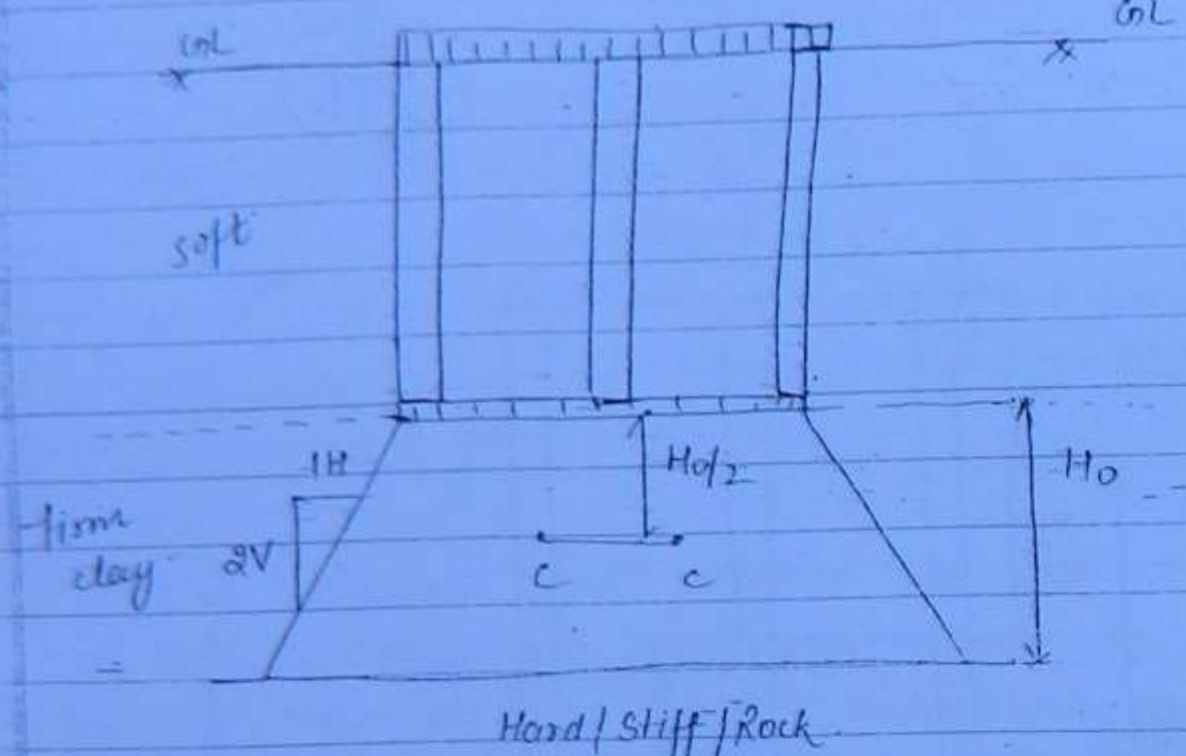


Case 4. In case of Sands group settlement ratio can be determine by using size of pile group.

$$S_R = \frac{S_g}{\ } = \left[\frac{4B + 2.7}{\ }\right]^2$$

Converse

Converse - Labbare Formula to determine group efficiency :-

$$\% \eta_g = \frac{Q_{ug}}{n \cdot Q_{up}} \times 100$$

$$\boxed{\% \eta_g = \left\{ 1 - \frac{\phi}{90} \left[\frac{m(n-1) + n(m-1)}{m \cdot n} \right] \right\} \times 100}$$

where ϕ = arc tan value

m = no. of rows in pile group

n = no of column in pile group

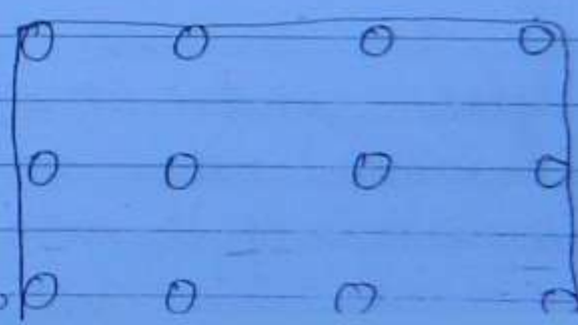
m & n are interchangeable.

Ques. Find efficiency of a pile group using converse Labbare formula for a square group of 16 piles. arc tan value $(\phi) = 18$

$m = 4$

$n = 4$

$\phi = 18$



$$\% \eta_g = 1 - \frac{18}{90} \left\{ 4(4-1) + 4(4-1) \right\} \times 100$$

Methods of soil stabilization :-

Soil stabilization is the process of improvement of shearing str 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 rods.

Sodium Silicate ($NaSiO_2$) :- $NaSiO_2$ with H_2O & $CaCl_2$ is known as <u>water glass</u>. 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 :-

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 pr. 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 width 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 stabilized highly plastic clays such as black soils. 4-6% volume of soil is the lime requirement.

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Bitumin Stabilization :- Bitumin, tar & Emmulsion 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.

Geo reinforcement :- Geomambrane and Geotextiles are poly proplene material which are available in synthetic format in open market.
These are transparent, permeable & colourless material. There 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.

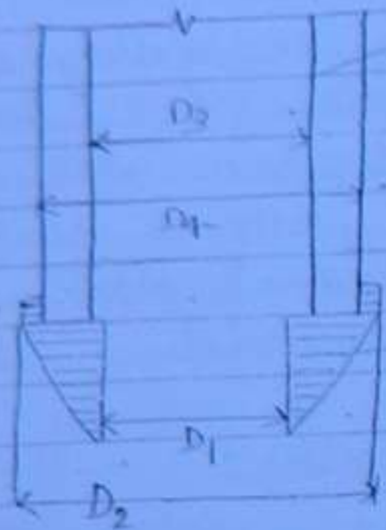
2. **Undisturbed Samples :-** If original soil str., mineral content & w.c donot change while sampling then it is called undisturbed sampling. Such samples are practically difficult to obtain. however small disturbance may be neglected.

For the purpose of consistency limit, sp. gr., grain size distribution, representative sample & undisturbed samples may be used. But for coff. of permeability, cosolidation parameter, shear str. parameter undisturbed samples will 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) :-

$$\boxed{\% \, C_i = \frac{D_3 - D_1}{D_1} \times 100}$$

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) :-

$$\boxed{\% C_o = \frac{D_2 - D_4}{D_4} \times 100}$$

It will help to reduce friction while sampler is driven & withdrawn.
It's value b/w 1% to 2%.

Area Ratio :-

$$\% A_r = \frac{\cancel{\pi/4}\,(D_2^2 - D_1^2)}{\cancel{\pi/4}\,D_1^2} \times 100$$

$$\boxed{\% A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100}$$

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 $L_R = 1$ (Good Recovery).

if $L_R < 1$ (Sample is shrunk)

if $L_R > 1$ (Sample is swelled).

Field methods used for subsurface investigation:

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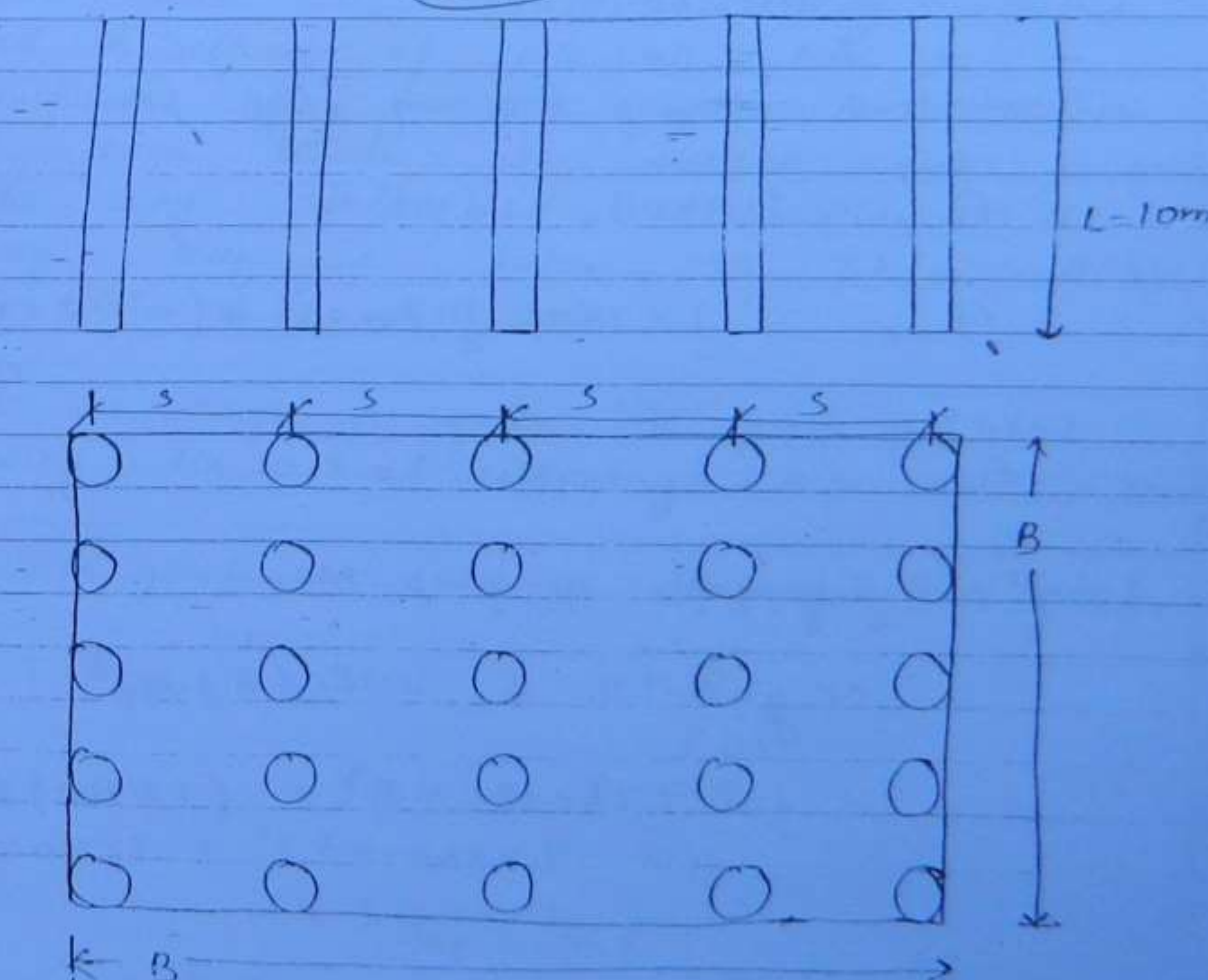
Penetration Test — SPT
CPT

Plate load Test
Vane Shear Test
Pr. 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 0.3m and c/c spacing b/w piles is 0.9m. length of pile is 10m. The shear strength of clayey soil at the base of pile is 180 kN/m² & avg. shear strength over the depth of 10m is 110 kN/m² & 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

$$B = 4S + D$$
$$= 4 \times 0.9 + 0.3$$
$$= 3.9 m$$

shear strength at base = 18.0 kN/m^2

$$S = c + \bar{\sigma}_n \tan\phi$$
$$S = c$$
$$c = 18.0 \; kN/m^2$$

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$\bar{c}$ = avg cohesion over depth of pile

$$\bar{c} = \bar{S} = 110 \; kN/m^2$$

ultimate load carrying cap of single pile

$$Q_{up} = 9c\, A_b + (\alpha \bar{c}) A_s$$

$$Q_{up} = 9 \times 180 \times \frac{\pi}{4}(0.3)^2 + (0.45 \times 110) \times \pi \times 0.3 \times 10$$

$$Q_{up} = 581.03 \; kN$$

Load carrying cap of group pile

$$Q_{ug} = 9c\, A_b + (\alpha \bar{c}) A_s$$
$$= 9 \times 180 \times B^2 + (1 \times 110)(4 B \cdot L)$$
$$= 9 \times 180 \times 3.9^2 + 1 \times 110 \times 4 \times 3.9 \times 10$$
$$= 41800.2 \; kN$$

$$\eta_g = \frac{Q_{ug}}{n Q_{up}} = \frac{41800.2}{25 \times 581.03} = 2.81$$

$$\frac{Q_{ug}}{FOS} \nless \frac{n\, Q_{up}}{FOS}$$

Q_{ag} = allowable load on pile group

$$= \frac{n\, Q_{up}}{FOS} = \frac{25 \times 581.03}{2.5}$$

$$Q_{ag} = 5810.3 \text{ kN}$$

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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 = 3×0.4 = 1.2m.

The pile group is always designed for $\eta_g \geq 1$

$$Q_{ag} = Q_{safe} = 300t = \frac{n\, Q_{up}}{FOS}$$

$$300t = \frac{25 \times Q_{up}}{3}$$

$$Q_{up} = 36\ t$$

For friction pile

$$Q_{up} = \overset{0}{Q_b} + Q_s$$

$$\bar{c} = 24/2 = 7/2 = 3.5$$

$$Q_{up} = q_s \cdot A_s$$

$$= \alpha \bar{c} \cdot \pi D L$$

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$$36\,t = 0.6 \times 3.5 \times \pi \times 0.4 \times L$$

$$L = 13.64\ m$$ (for friction pile $L = 10$ to 20)

Check for group efficiency.

$$B = 4s + D = 4 \times 1.2 + 0.4 = 5.2$$

$$Q_{ug} = \overset{0}{Q_b} + Q_s$$

$$= q_s \cdot A_s$$

$$= (\alpha \bar{c})(4BL)$$

$$= 1 \times 3.5 \times 4 \times 5.2 \times 13.64$$

$$= 992.99\ t$$

$$\eta_g = \frac{Q_{ug}}{n\, Q_{up}} = \frac{992.99}{25 \times 36} = 1.1 > 1$$

$$\eta_g > 1 \quad \text{ok.}$$

Design parameters are

no of piles = $5 \times 5 = 25$

dia = $0.4\,m$

spacing = $3d = 3 \times 0.4 = 1.2\,m$

length = $13.64\,m$

η_g = 1.1

In this end bearing effect is negligible b/c design of friction pile

$$Q_{ag} = \frac{n \times Q_{up}}{FOS}$$

$$= \frac{9 \times 371.100}{2.5}$$

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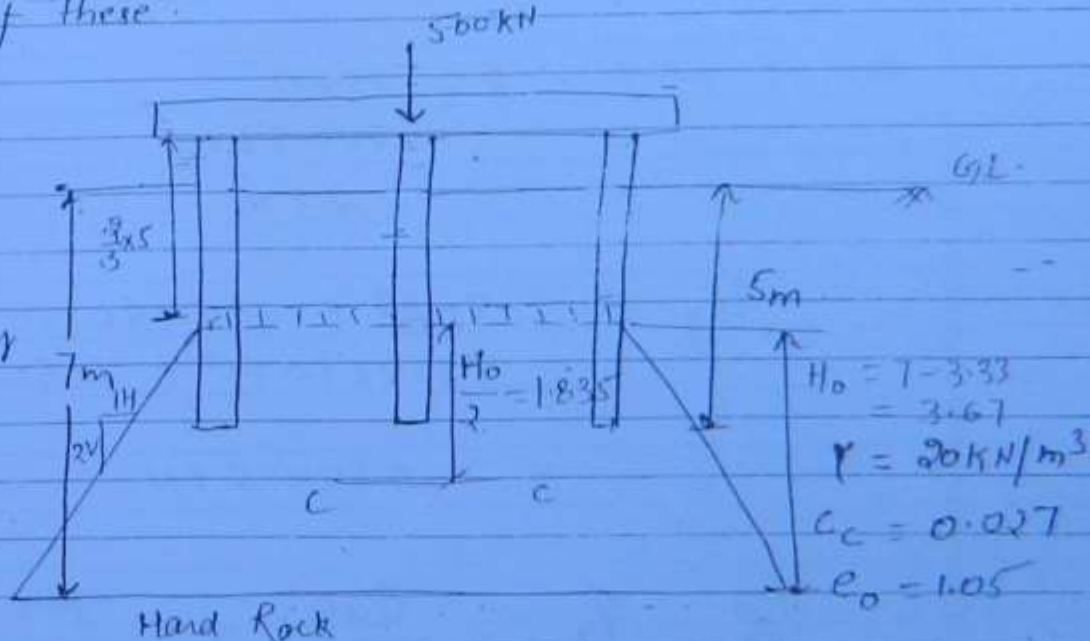
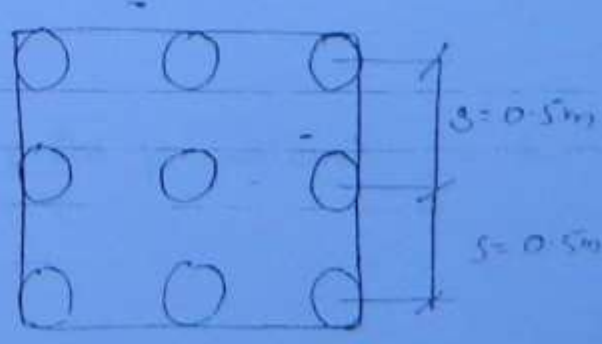
$$Q_{ag} = 1335.96 \text{ KN}$$

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 = 0.3m.

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 deposit of medium stiff clay (q_u = 100 KN/m²) c/c spacing = 0.9 m. & α = 0.9

calculate the ~~pile~~ load capacity of pile group. FOS = 2.5

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$$Q_{up} = 9 c A_b + \alpha \bar{c} A_s$$
$$= 9 \times \left(\frac{100}{2}\right) \times \frac{\pi}{4}(0.3)^2 + 0.9 \times \frac{100}{2} \times \pi \times 0.3 \times$$

$$= 371.100 \text{ KN}$$

size of group = $B = 2s + D$
$$= 2 \times 0.9 + 0.3$$
$$= 2.1 \text{ m}$$

$$Q_{ug} = 9 c B^2 + \alpha \bar{c} (4BL)$$
$$= 9 \times 50 \times (2.1)^2 + 1 \times 50 (4 \times 2.1) \times 8$$

$$= 5344.5 \text{ KN}$$

$\cancel{\frac{Q_{up}}{FOS}}$ $\eta_g = \frac{Q_{ug}}{n\, Q_{up}} = \frac{5344.5}{9 \times 371.00}$
$$= 1.6$$

safe load = smaller of $\frac{n Q_{up}}{FOS}$ & $\frac{Q_{ug}}{FOS}$