C.V. RAMAN POLYTECHNIC, BHUBANESWAR

C.V.Raman Polytechnic

Quality Education for the New Millenium

DEPARTMENT OF CIVIL ENGINEERING

LECTURE NOTE
ON

GEOTECHNICAL ENGINEERING, (TH.2)

SEM- 3RD

Prepared by

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(Asst. Prof. in Civil Engineering)

...th Aug,
...ATURDAY

GEOTECHNICAL ENGINEERING
(12 marks)

1. ORIGIN OF SOILS

Soil is a naturally occuring unconsolidated earth material present above the bed rock. – Terzaghy.

Karl Terzaghy – Father of Soil Mechanics.

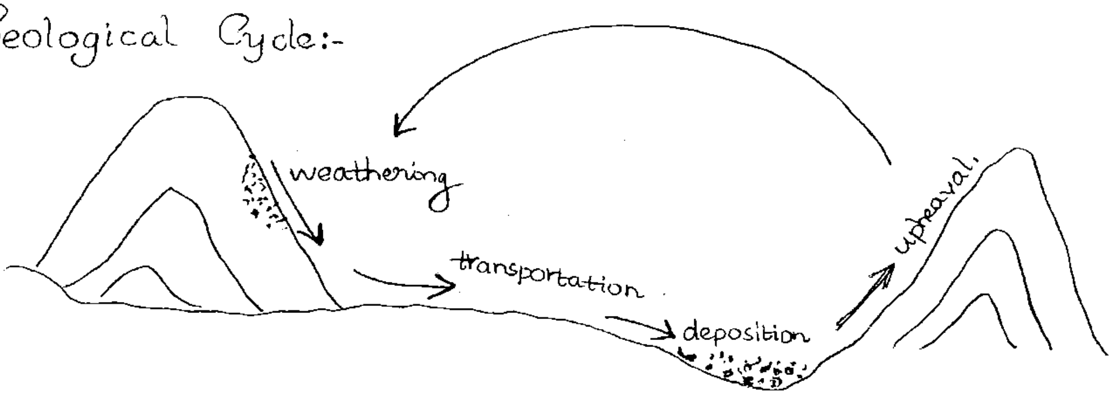
Origin of soils is due to

- Weathering of rocks (disintegration) ('inorganic soil')
  Eg: gravel, sand, some type of silts, clays.
- Decomposition of organic matter ('organic soils' or 'cumulose soils')
  Eg: Humus, muck, peat

→ Weathering of Rocks.

- Physical weathering
  – due to physical effects
  a) temperature change
  b) Abrasion. (grinding action of flowing water & wind)
  c) Splitting action due to ice, rain, penetration of plant roots
  – produces soils like gravel, sand, non plastic silt.
- Chemical weathering
  – due to chemical actions
  a) oxidation
  b) Carbonation
  c) Hydration
  d) Solution
  e) Hydrolysis.
  – produces soils like clay, plastic silt

→ Geological Cycle:-



- Pedogenesis : It is a process of formation of soil.

→ Transportation of Soil :

It is due to -

a) Wind - Aeolian Soil : transported & deposited by wind.
b) Water - Alluvial Soil : transported by water & deposited along river be
c) Glacier - Glacier deposit : transported by glacier
d) Gravity - Colluvial Soil : transported & deposited by Gravity.

Lacustrine Soil :- transported by water & deposited in lakes
Marine Soil :- transported by water & deposited in sea

* Classification of Soils.

a) Residual Soils. :- soil which remains at or near th parent rock.
(Sedentary Soil).

b) Transported Soil :- transported away from parent rock

→ Forces acting on the Soil Particles :

(i) Gravitational Force or Body Force.
(ii) Surface force

Body Force	Surface Force.
(i) It is proportional to mass	(i) It is proportional to Surface are
(ii) Eg : weight.	(ii) Eg : cohesion, electrochemical fo.
(iii) It is predominant in gravel & sand	(iii) It is predominant in clay. (Clay behaviour is mainly controlled by surface force)

In the case of silty soil both body force and surface force are equally important.

→ Popular Field names of Soils.

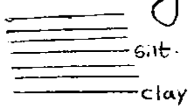
1. Black Cotton soil (BC Soil) :– a residual clayey soil.
   - highly plastic.
   - exhibits high swelling & shrinkage due to presence of "Montmorillonite" clay mineral
   - parent rock is Basalt or 'trap

2. Loam :– a mix of sand, silt & clay

3. Moorum :– a gravel mixed with red clay

4. Bentonite :– a decomposed volcanic ash.
   - a clayey soil, highly plastic, highly water absorb
   - bentonite slurry is called "Drilling Mud"

5. Varved Clay :– contains alternate thin layers of silt & clay
   - lacustrine deposits.



6. Loess :– Aeolian deposit.
   - contains silt sized particles
   - weakly cemented by $CaCO_3$ particles.

7. Sand dunes :– Aeolian deposit.
   - particle size is same.

8. Humus :– half decomposed organic soil
   - amorphous in nature. (amorphous × crystalline)

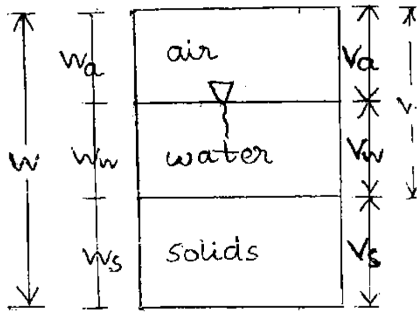
9. Muck :– contains fine inorganic particles with decomposed organic material.
   - black in colour.

10. Peat :– highly decomposed organic matter.
    - fibrous in nature.
    - dark brown to black colour
    - bad odour.

11. Fill :- a manually deposited soil. (a man made deposit)

2. DEFINITIONS & PROPERTIES OF SOIL

* Partially saturated soil : Solids + water + air (3 phase syst[e]
  Fully saturated soil : Solids + water. (2 phase system)
  Dry soil : Solids + air
* Frozen soil : Solids + water + ice + air. (4 phase system)



Phase Diagram
OR
Block Diagram

$V_s \rightarrow$ volume of solids.

$V \rightarrow$ volume of soil.

$V_v \rightarrow$ volume of voids.

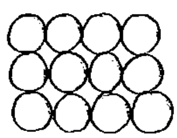
$$V_v = V_w + V_a.$$

$$V = V_s + V_w + V_a.$$

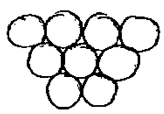
* Void Ratio, e

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

Range :- more than zero, it can have any value (no limit)
- For coarse grained soil, $e < 1$ generally.
- For fine grained soil, $e > 1$ generally
- The 'e' of FG soil is generally more than th[at] of a coarse grained soil.


Cubical array
(loosest state)

$\boxed{e_{max} = 0.91}$


Diagonal array
(densest state)

$\boxed{e_{max} = 0.35}$

* Specific Gravity of <u>Soil Solids</u>, G

$$G = \frac{\gamma_s}{\gamma_w} \quad ; \gamma_w \rightarrow \text{distilled water (pure water)}$$

Also called 'True Specific gravity of <u>Soil</u>'

For soils, G : 2.60 — 2.85 generally.
(inorganic).

* Mass Specific Gravity of Soil. (or) Bulk Sp. Gr. of soil (or) Apparent Specific Gravity of soil, Gm.

For a dry soil, $G_m = \dfrac{\gamma_d}{\gamma_w}$

For a partially saturated soil, $G_m = \dfrac{\gamma}{\gamma_w}$

For a fully saturated soil, $G_m = \dfrac{\gamma_{sat}}{\gamma_w}$.

$$\boxed{G_m < G}$$

For cement :-

True sp. gravity, G = 3.15

Apparent sp. gravity, $G_m = 1.44$

For cement, $\gamma_s = 3150 \text{ kgf/m}^3$

$\gamma_d = 1440 \text{ kgf/m}^3$

$n \approx 44\%$

Important Relationships :

1. $e = \dfrac{wG}{S_r}$

2. $\gamma = \gamma_w \left(\dfrac{G + eS_r}{1+e}\right)$

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

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

5. $\gamma_d = \dfrac{\gamma}{1+w}$

6. $\gamma_d = \dfrac{(1-n_a)\gamma_w G}{1+wG}$

7. $\gamma = \gamma_d + S_r(\gamma_{sat} -$

* Saturated unit weight of Soil, γ_{sat}.

It is the bulk unit weight of soil in a saturated condition. $\Rightarrow \gamma_{sat} > \gamma$

For partially saturated soil, ~~γ_{sat}~~ use γ

For fully saturated soil, use γ_{sat}.

* Dry unit weight of Soil, γ_d.

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

It can be used irrespective of saturation level of soil.

* Unit weight of Solids, γ_s

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

* Submerged Unit Weight of Soil, γ_{sub} or γ'

It is the submerged wt. of soil per unit volume of soil.

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

$\left\{ \begin{array}{l} \gamma_{sat} \downarrow \text{ - gravity force} \\ \gamma_w \uparrow \text{ - buoyant force.} \end{array} \right\}$

7th Aug, MONDAY

Submerged weight of Soil is based on Archimede's Principle.

γ_w = unit weight of water
= 1 g/cc = 1 ton/m³ = 1000 kgf/m³.
= 9.81 kN/m³ ≈ 10 kN/m³.

$$\boxed{\gamma_s > \gamma_{sat} > \gamma_{bulk} > \gamma_{dry} > \gamma'}$$

For a given soil, γ_s remains a constant

* Porosity, n (also called 'Percentage voids')

$$n = \frac{V_v}{V} \times 100$$

Range: $0 < n < 100\%$ ($V_v \neq 0$ for soil, $\therefore n \neq 0$).

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

* Degree of Saturation, S_r

$$S_r = \frac{V_w}{V_v} \times 100.$$

For a dry soil, $S_r = \frac{0}{V_v} \times 100 = 0$ ($V_w = 0$)

For a saturated soil, $S_r = \frac{V_v}{V_v} \times 100 = 100$ ($V_w = V_v$)

Range: $0 \leq S_r \leq 100\%$

* Air content, a_c

$$a_c = \frac{V_a}{V_v}.$$

For a saturated soil, $a_c = \frac{0}{V_v} = 0$ ($V_a = 0$).

For a dry soil, $a_c = \frac{V_v}{V_v} = 1.$ ($V_a = V_v$)

Range: $0 \leq a_c \leq 1$

* % air voids, n_a

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

For a saturated soil, ($V_a = 0$), $n_a = 0$

For a dry soil, $n_a = n$ ($V_a = V_v$).

Range: $0 \leq n_a \leq n$.

$$a_c + S_r = \frac{V_a}{V_v} + \frac{V_w}{V_v}.$$

$$= \frac{V_a + V_w}{V_v} = \frac{V_v}{V_v} = 1.$$

$$\therefore \boxed{a_c + S_r = 1}$$

$$n\,a_c = \frac{V_v}{V} \times \frac{V_a}{V_v} = \frac{V_a}{V} = n_a.$$

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

$W_s \longrightarrow$ weight of solids.

$W \longrightarrow$ weight of soil.

$W = W_s + W_w$ (W_a is negligible)

$W_d \longrightarrow$ total weight of soil in dry condition. ($= W_s$).

* Water Content, ω

$$\omega = \frac{W_w}{W_s} \times 100$$

For dry soil, $\omega = \frac{0}{W_s} \times 100 = 0$

For a saturated soil, $W > 0$.

(i.e, water content can have any value greater than zero sometimes $> 100\%$, sometimes $< 100\%$)

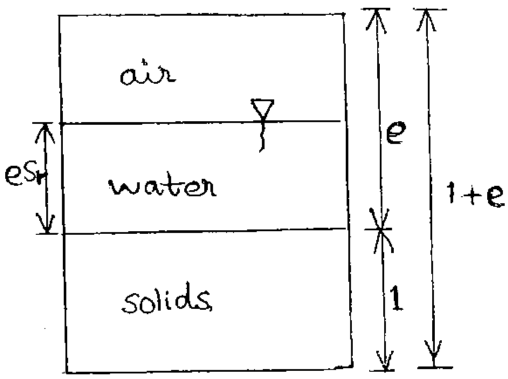
* Bulk unit weight of Soil, γ

– it is the total weight of soil per unit volume of soil.

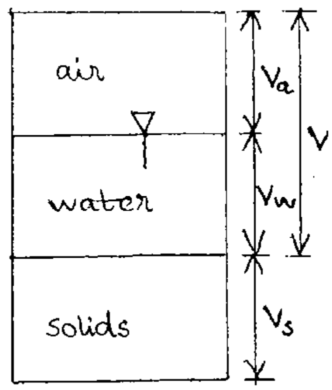
$$\gamma = \frac{W}{V}.$$

In a unit phase diagram,
lets take $V_s = 1$

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



Unit Phase diagram.



Phase Diagram

If $V_s = 1$, then

$$e = V_v.$$

$$S_r = \frac{V_w}{V_v} \Rightarrow V_w = V_v \cdot S_r$$
$$= e \cdot S_r$$

⊙ To derive n & e relationship:

$$n = \frac{V_v}{V} = \underline{\underline{\frac{e}{1+e}}}$$

⊙ To derive $e = \frac{wG}{S_r}$

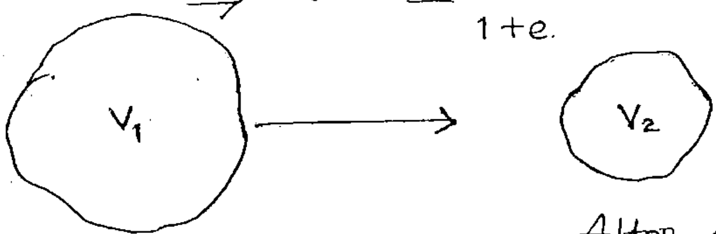
$$w = \frac{W_w}{W_s} = \frac{V_w \gamma_w}{V_s \cdot \gamma_s} = \frac{eS_r}{1 \cdot G}$$

$$\Rightarrow e = \frac{wG}{S_r}$$

⊙ To derive $\gamma = \gamma_w \left(\frac{G + eS_r}{1+e}\right)$

$$\gamma = \frac{W_s + W_w}{V} = \frac{\gamma_s W_s + \gamma_w V_w}{V}.$$
$$= \frac{\gamma_s + \gamma_w \, e . S_r}{1+e.}$$

$$\Rightarrow \gamma = \frac{\gamma_w (G + eS_r)}{1+e.}$$



Before compaction
e_1, γ_{d1}, W_s

After compaction
e_2, γ_{d2}, W_s

$$\boxed{\frac{V_2}{V_1} = \frac{1+e_2}{1+e_1}} \qquad \because \; V \propto (1+e)$$

$$\gamma_d = \frac{W_s}{V} \Rightarrow \gamma_d \propto \frac{1}{V}$$

$$\boxed{\frac{V_2}{V_2} = \frac{\gamma_{d1}}{\gamma_{d2}}}$$

Q. Due to compaction, the void ratio of a soil reduced from 1 to 0.6. What is the % volume loss.

$$\frac{V_2}{V_1} = \frac{1+0.6}{1+1} = \frac{1.6}{2.}$$

$$V_2 = 0.8\, V_1$$

∴ vol. reduced to 80%.

vol. reduced by 20% ⇒ volume loss is 20%.

* To Find Water Content of Soil :

a) Oven drying Method. → most accurate method.

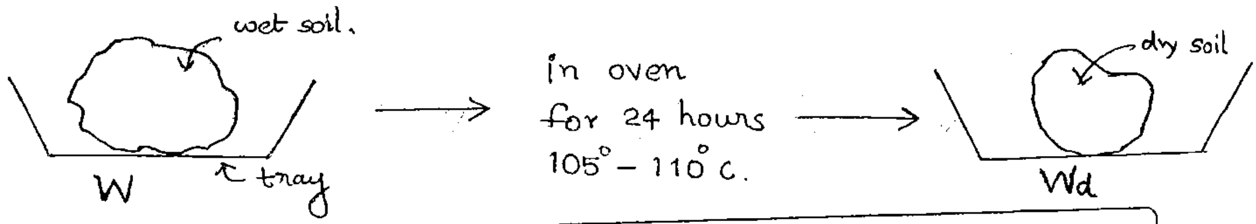
b) Pycnometer Method → can be used only if 'G' is known

c) Sand bath method → quick field method. (approx. value).

d) Calcium Carbide method → quick method.

e) Torsion balance method → to find w.c at different depths below G.L

a) Oven Drying method :

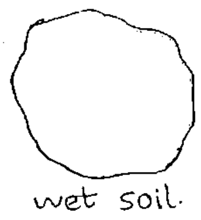


W → wt. of wet soil

W_d = wt. of dry soil = W_s

$$\boxed{w = \frac{W_w}{W_s} \times 100 = \frac{W - W_d}{W_d} \times 100}$$

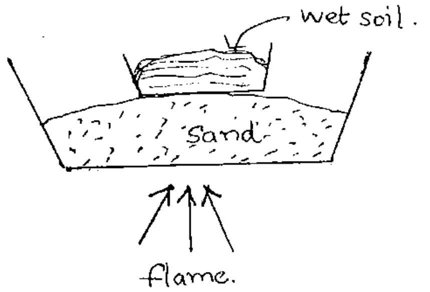
b) Calcium Carbide method.



+ calcium carbide (CaC_2) $\longrightarrow$ acetylene gas $\uparrow$

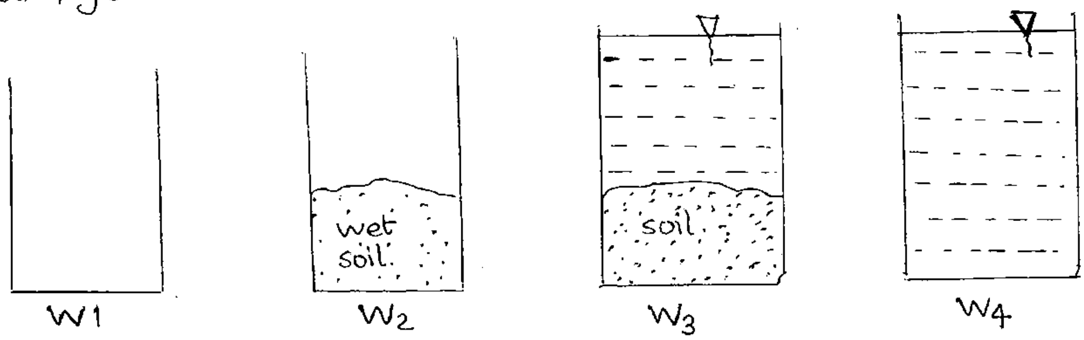
Amount of acetylene gas produced is directly proportional to water content of soil.

c) Sand Bath method.



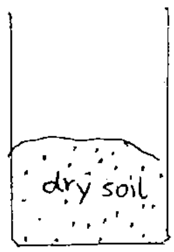
It's a crude method used in the field.

d) Pycnometer Method



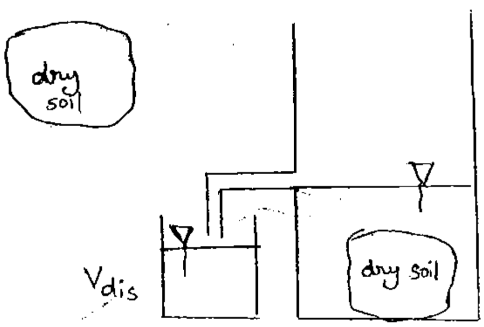
$$\text{Water content, } \omega = \left[\frac{W_2 - W_1}{W_3 - W_4} \left(\frac{G-1}{G} \right) - 1 \right] \times 100$$

⊙ To find Specific Gravity, G, replace second container with dry soil



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

⊙ To find volume:



$V_{dis} \rightarrow$ volume of water displaced

⊙ If dry soil is immersed, then $V_{dis} = V_s$ (vol. of solids).

⊙ If partially saturated soil is immersed, $V_{dis} = V_s + V_w$

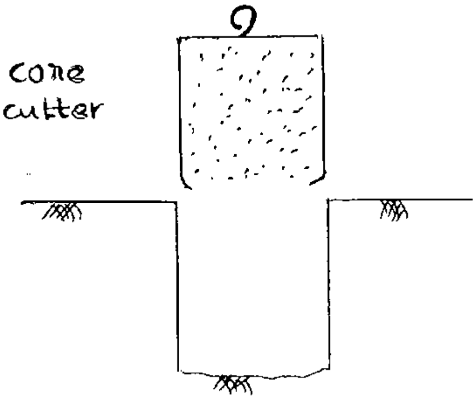
⊙ Partially saturated soil with wax coating, then

V_{dis} = total volume of soil = V + vol. of wa

* To determine in-situ γ_d and e

(i) Core Cutter Method → suitable for clays only (cohesive).

(ii) Sand Replacement method → suitable for any soil



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

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

12th Aug,
TUESDAY

P-8.

1. $V_a = \frac{V}{6}$, $V_w = \frac{V}{3}$

$V_a + V_w = V_v$

$\therefore V_v = \frac{V}{6} + \frac{V}{3} = \frac{V}{2}$

$V_s = V - V_v = \frac{V}{2}$

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

2. $\gamma_1 = 1.8$ g/cc. at $\omega_1 = 5\%$

$\gamma_2 = ?$ at $\omega_2 = 10\%$

'e' remains constant.

$$\gamma_d = \frac{\gamma}{1+\omega} \Rightarrow \gamma = \gamma_d(1+\omega) = \frac{\gamma_w G}{1+e}(1+\omega)$$

$\therefore \gamma \propto (1+\omega)$ (e is constant).

$$\frac{1.8}{\gamma_2} = \frac{1.05}{1.1}$$

$\underline{\underline{\gamma_2 = 1.886}}$ g/cc

4. Volume of soil = vol. of sampler.

$$V = 45 \text{ cc.}$$

Given $V_s = 25$ cc.

$$e = \frac{V - V_s}{V_s} = \frac{20}{25} = \underline{\underline{0.8}}$$

5. Initial wt. of soil = 0.18 kg.

Water added = 0.02.

$\therefore$ Total weight, $W = 0.2$ kg.

Vol. of soil, $V = 10^{-4}\ m^3$. (initial volume assumed to be constant)

$$\gamma = \frac{W}{V} = \frac{0.2}{10^{-4}} = 2000 \text{ kg/m}^3$$

$\gamma_d = 1600$ kg/m^3.

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

$$1600 = \frac{2000}{1+w}$$

$w = 0.25 = \underline{\underline{25\%}}$ (method is valid only if V remains const. after adding water)

OR

Initial weight, $W = 0.18$ kg, $V = 10^{-4}\ m^3$, $\gamma_d = 1600$ kg/m^3

$$\gamma_d = \frac{W_s}{V} \qquad W = W_s + W_w$$

$\therefore W_w = 0.02$ kg.

$1600 = \frac{W_s}{10^{-4}} \Rightarrow W_s = 0.16$ kg. (water present ini

Water added additionally, = 0.02 kg.

$$W_w = 0.02 + 0.02 = 0.04 \text{ kg}.$$

Final water content $= \frac{0.04}{0.16} \times 100 = \underline{\underline{25\%}}$

6. $W = 34.62$ g, $V = 24.66$ cm^3, $W_d = W_s = 20.36$ g

$G = 2.68$, $e = ?$ $S_r = ?$

$$\omega = \frac{W - W_d}{W_d} \times 100 = 70\%$$

$$e = \frac{\omega G}{S_r} = \frac{0.7 \times 2.68}{S_r} \longrightarrow (1)$$

$$\gamma = \frac{W}{V} = \frac{34.62}{24.66} = 1.40 \text{ g/cc.}$$

$$\gamma = \gamma_w\left(\frac{G + eS_r}{1+e}\right) \Rightarrow 1.4 = \frac{1\,(2.68 + 0.7 \times 2.68)}{1+e}$$

$$\therefore\ e = 2.25$$

$$S_r = \frac{0.7 \times 2.68}{2.25} = \underline{\underline{83.4\%}}$$

OR

$$\gamma_d = \frac{W_d}{V} = \frac{20.36}{24.66} = 0.825 \text{ g/cc}$$

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

$$\underline{\underline{e = 2.25}}$$

819. $\omega = 18\%$, $\gamma = 2.05$ g/cc., $G = 2.67$.

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

$$\frac{2.67 \times 1}{1+e} = \frac{2.05}{1.18}$$

$$\underline{\underline{e = 0.54}}$$

$$e = \frac{\omega G}{S_r} \Rightarrow S_r = \frac{0.18 \times 2.67}{0.54} = \underline{\underline{89.52\%}}$$

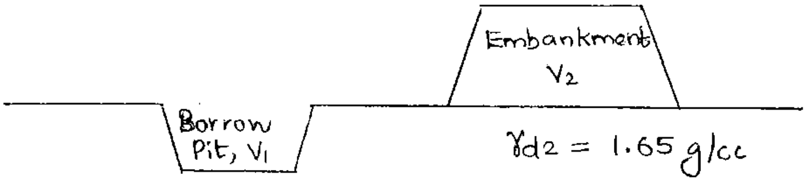
8. $\omega = 39.3\%$, $G_m = \frac{\gamma_{sat}}{\gamma_w} = 1.84$ (Soil is saturated).

$$e = \frac{\omega G}{S_r} = 0.393\,G$$

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

$$1.84 = \frac{G + 0.393\,G}{1 + 0.393\,G} \Rightarrow G = 2.70, \quad e = 0.393\,G = \underline{\underline{1.08}}$$

10.



$\gamma_{d2} = 1.65$ g/cc

$w_2 = 18\%$

$V_2 = 1000\ m^3$.

$\gamma_1 = 1.75$ g/cc

$w_1 = 12\%$

$V_1 = ?$

$$\gamma_{d1} = \frac{\gamma_1}{1+w_1} = \frac{1.75}{1+0.12}$$

$$= 1.57.$$

$$\frac{V_1}{V_2} = \frac{\gamma_{d2}}{\gamma_{d1}}$$

$$V_1 = \frac{1.65}{1.557} \times 1000 = \underline{\underline{1056\ m^3}}$$

$\rightarrow 1\ g/cc = 1\ ton/m$

To raise w.c from $w_1 \rightarrow w_2$:

Weight of water to be added $= \gamma_d V (w_2 - w_1)$

$$= \underset{g/cc}{1.65} \times 1000 \times 1000\ (0.18 - 0.12).$$

$$= 99000\ kg = \underline{\underline{99\ tons}}$$

2. $\gamma_{borrow} = 1.66$

$w_b = 8$

$V_b = ?$

$\gamma_{d1} = 1.54$

$\gamma_{truck} = 1.15$

$w_t = 6.$

$V_t = ?$

$\gamma_{d2} = 1.08$

$\gamma_{fill} = 1.82$

$w_f = 14.$

$V_f = 100\ m^3$

$\gamma_{d3} = 1.25.$

$$\frac{V_2}{V_3} = \frac{\gamma_{d3}}{\gamma_{d2}}.$$

$$V_2 = \frac{1.25}{1.08} \times 100 = 147\ m^3.$$

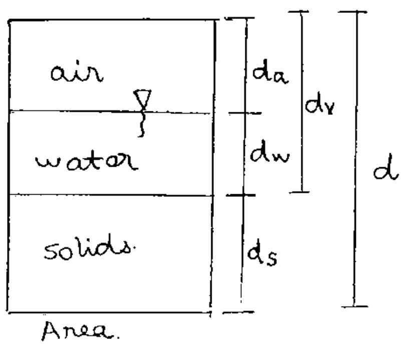
No: of truck loads $= \frac{147}{6} = 24.6$ no.s $= \underline{\underline{25\ \text{no.s}}}$

13. If $d_w = 1$ m, $d = ?$

Given: $e = 0.5$, $S_r = 80\%$

$$S_r = \frac{d_w}{d_v}$$

$$d_v = \frac{1}{0.8} = \underline{\underline{1.25 m}}$$



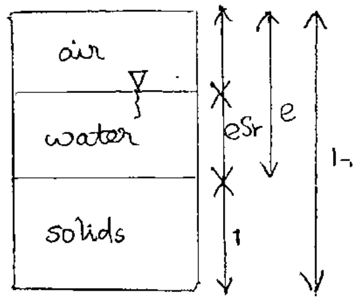
depth of voids, $d_v = 1.25$ m.

$$e = \frac{d_v}{d_s} \Rightarrow d_s = \frac{1.25}{0.5} = \underline{\underline{2.5 \text{ m}}}$$

Total depth of soil, $d = d_s + d_v = \underline{\underline{3.75 \text{ m}}}$

OR based on Unit Phase Diagram.

Given, depth of water $= 1$ m



$$e.S_r = 0.5 \times 0.8 = 0.4 \text{ m}.$$

$$1+e = 1+0.5 = 1.5 \text{ m}.$$

0.4 m depth of water makes 1.5 m depth of soil 80% saturated.

$$\text{Depth of soil} = \frac{1.5}{0.4} \times 1 = \underline{\underline{3.75 \text{ m}}}$$

14. $n = 40\%$, $G = 2.5$, $\omega = 12\%$

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

$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.666.$$

$$e = \frac{\omega G}{S_r} = \omega G \text{ (at full saturation).}$$

$$\omega = \frac{2}{3 \times 2.5} = 26.6\%$$

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

Take $\gamma_w = 1$ ton/m³. $\Rightarrow \gamma_d = 1.5$ t/m³.

Weight of water to be added to achieve full saturation

$$= 1.5 \times 100 \left(\frac{26.6 - 12}{100}\right)$$

$$= \underline{\underline{21.9}} \text{ tons.}$$

16. Let e_2 be void ratio at increased volume of soil

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

$$\frac{1.05 V_1}{V_1} = \frac{1+e_2}{1+0.667} \Rightarrow e_2 = \underline{\underline{0.75}}$$

Let ω_3 be water content at increased volume, 10

$$e_2 = \frac{\omega_3 G}{S_r}$$

$$0.75 = \frac{\omega_3 \times 2.5}{S_r = 1}$$

$$\Rightarrow \omega_3 = \underline{\underline{30\%}}$$

17. $W_d = W_s = 135\ g.$

Wet weight, $W = 195\ g.$

Total vol. of soil, $V = 5^3 = 125\ cm^3.$

$$\omega = \frac{W - W_d}{W_d} \times 100 = \frac{60}{135} \times 100 = \underline{\underline{44.44}}\ \%$$

$$e = \frac{\omega G}{S_r} \Rightarrow e = 0.44\ G$$

$$\gamma_{sat} = \frac{W}{V} = \frac{195}{125} = 1.56\ g/cc.$$

$$\gamma_{sat} = \gamma_w \left(\frac{G+e}{1+e}\right) \Rightarrow 1.56 = 1 \left(\frac{G + 0.44G}{1 + 0.44G}\right)$$

$$= 2.07.$$

$$e = 0.44\ G = \underline{\underline{0.92}}$$

OR

$V = 125\ cm^3.$

Wt. of water added $= 195 - 135 = 60\ g$

$\therefore$ Vol. of water $= \dfrac{W_w}{\gamma_w} = \dfrac{60}{1} = \underline{\underline{60\ cc}}$

Vol. of voids = vol. of water added

$\Rightarrow V_v = 60$ cc. $\quad \therefore V_s = 125 - 60 = 65\ cm^3.$

$$e = \frac{V_v}{V_s} = 0.92.$$

$$\gamma_s = \frac{W_s}{V_s} = \frac{135}{65} = 2.07\ g/cc \Rightarrow G = \frac{\gamma_s}{\gamma_w} = \frac{2.07}{1} = \underline{\underline{2.07}}$$

Q. 100 g of dry soil having $G = 2.7$ is mixed with water and 1 L of soil slurry is prepared. What is the unit weight of soil slurry in g/cc.

$$G = \frac{\gamma_s}{\gamma_w} \Rightarrow \gamma_s = 2.7 \text{ g/cc}$$

$$\gamma_s = \frac{W_s}{V_s} \quad \therefore V_s = \frac{W_s}{\gamma_s} = \frac{100}{2.7} = 37.037 \text{ cc}$$

$V = 1\text{ L} = 1000 \text{ cm}^3$ $\quad \Rightarrow V_w = V - V_s = 1000 - 37.037$
$= 962.963$

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{100 + V_w \times \gamma_w}{V}$$

$$= \frac{100 + 962.963 \times 1}{1000} = \underline{\underline{1.063}} \text{ g/cc}$$

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Q A marine soil has sp.gr. of solids as 2.7 and void ratio as 0.8. If sp.gr. of sea water is 1.03, calculate γ_{sat} of the soil. Take γ_w of fresh water as 9.81 kN/m³.

$G = 2.7$

$\Rightarrow$ ~~γ_{sat}~~ $\gamma_s = 2.7$ g/cc.

$e = 0.8$.

$$\gamma_{sat} = \frac{\gamma_w (G+e)}{1+e} \cancel{=} 19.64$$ (valid only if pure ~~sea~~ water is used)

$$G = \frac{\gamma_s}{\gamma_w \rightarrow \text{pure water}}$$

$\gamma_{sea} = 1.03 \times 9.81$
$= 10.1043$ g/cc

$$\gamma_{sat} = \frac{W_w + W_s}{V} = \frac{V_s \gamma_s + V_w \gamma_{sea\ water}}{V}$$

$$= \frac{1 \times G\gamma_w + e \cdot S_r \times 10.1043}{1+e}$$

$$\left.\begin{array}{l} V_s = 1 \\ V_w = eS_r \\ V = 1+e \end{array}\right\} \text{unit phase.}$$

$$= \frac{9.81 \times 2.7 \times 1 + 0.8 \times 1 \times 10.1043}{1+0.8}$$

$$= \underline{\underline{19.206}} \text{ kN/m}^3$$

Q The mass of an empty pycnometer is 0.498 kg. When completely filled with water, its mass is found to be 1.528 kg. An oven dried soil of mass 0.198 kg is placed in the pycnometer and water is added to fill the pycnometer and t total mass is found to be 1.653 kg. Determine sp. gravity of soil particles

$W_1 = 0.498$, $W_2 = 0.198 - W_1$, $W_3 = 1.653$, $W_4 = 1.528$.

$$G = \frac{W_2 - W_1}{(W_2-W_1)-(W_3-W_4)} = \frac{0.198}{0.198-0.125} = \underline{\underline{2.712}}$$

Wt. of dry soil $= W_2 - W_1 = 0.198$

Q A sample of clay was coated with paraffin wax and the total mass of soil and wax was found to be 700 g. The sample was immersed in water and the vol. of water displaced was found to be 355 mL. The mass of the sample without wax was 690 g. and water content of the soil was 18 %. Determine bulk density, dry density, void ratio and degree of saturation. Take sp. gr. of soil solids as 2.7 and that of wax as 0.89.

355×1 ~~$= \frac{690 x}{2.7} + \frac{10}{0.89}$~~.

Weight of wax $= 700 - 690 = 10$ g.

Density of wax $= 0.89 \times 1 = .89$ g/cc.

Volume of wax $= \frac{10}{0.89} = 11.236$ cc.

Volume of soil = vol. of water displaced − vol of wax

$= 355 - 11.23 = \underline{\underline{343.77 \text{ cm}^3}}$

$$\gamma = \frac{W}{V} = \frac{690}{343.77} = 2.007 \text{ g/cc}.$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{2.007}{1.18} = 1.7 \text{ g/cc}$$

$$\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow \underline{\underline{e = 0.588}}$$

$$e = \frac{wG}{S_r} \Rightarrow S_r = \underline{\underline{82.65\%}}$$

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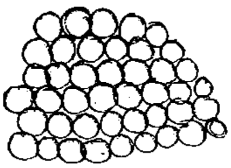
3. SOIL STRUCTURES & CLAY MINERALOGY

↦ Types of Structures

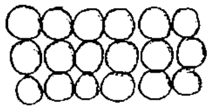
1. Single Grained Structure → in gravel & coarse sand
2. Honey comb structure → in fine sand & silt
3. Flocculent structure → in clays
4. Dispersed structure → in remoulded clays
5. Combined structure → in soil mixtures

due to surface forces

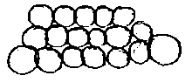


* Single grained structure.

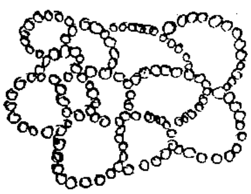
2 extreme cases. ⇒



loosest
(cubical array)
$e_{max} = 0.91$



densest
(diagonal packing)
$e_{min} = 0.35$

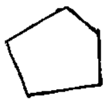


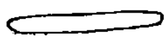
* Honey Comb Structure

- very sensitive to vibrations (decreases in vol. due to vibrations)
- It collapses on wetting (volume decreasing)
- collapsible soils. Eg: loess, fine sand, silt

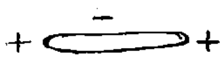
→ Particle Shapes.

(i) Angular :- gravel & sand (ii) Rounded :- gravel & sand (iii) Flaky :- clay soils.





The clay particle is electrically charged as shown below.
(-ve charge on surface)



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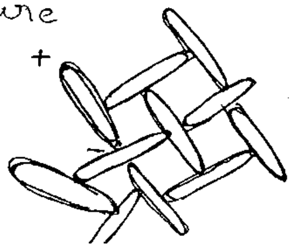
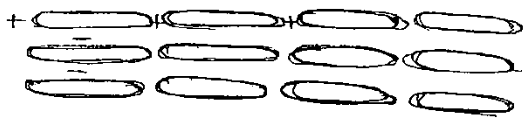
* Flocculent Structure



> relatively more strength
> stable.
> edge to face orientation
> net attraction.

* Dispersed Structure



> relatively low strength
> Unstable
> Face-to-face orientation
> net repulsion

Thixotropy : The phenomenon of regaining of strength with passage of time under const. water content is called Thixotropy

$$\begin{tikzcd} \text{Flocculent Structure} \arrow[r, bend right, "\text{Remoulding}"'] & \text{Dispersed Structure} \arrow[l, bend right, "\text{Thixotropy}"'] \end{tikzcd}$$

In clay,
Due to remoulding, strength decreases.
Due to thixotropy, strength regains.

Marine clay → flocculent structure
(Sea water)
Lake clay → dispersed structure
(Fresh water)

The presence of salts in seawater and due to its alkaline nature, salts acts as flocculating agents. The marine clay has flocculent structure.

→ Minerals.

(i) Rock Minerals.
- no surface activity.
- Eg: Quartz, mica, feldspar.

(ii) Clay Minerals.
- have surface activity. (like cohesion, electrostatic, chemical forces)
- Eg: Kaolinite, Illite, Montmorillonite, Holloysite.

* Kaolinite
- causes no swelling & no shrinkage.
- it is present in china clay (used to make earthenware utensils)

* Illite:
  - causes medium swelling & shrinkage
  - present in most of the clays
* Montmorillonite:
  - causes large swelling and large shrinkage.
  - present, Bentonite clay & B.C soil.
* Holloysite:
  - similar to Kaolinite

NOTE:

- Plasticity of Kaolinite < Plasticity of Illite < Plasticity of Montmorillonite.
- SSA of Kaolinite < SSA of Illite < SSA of montmorillonite

SSA – Specific Surface Area (S.A per unit weight).

→ Specific Surface Area (SSA).

1. It is the surface area per unit weight $\Rightarrow \frac{A}{W}$
2. It is the surface area per unit volume $\Rightarrow \frac{A}{V}$

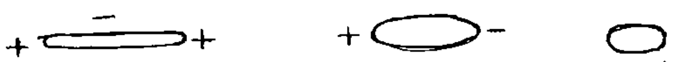
$$SSA = \frac{A}{V} = \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \frac{3}{r}$$

$$\Rightarrow \boxed{SSA \propto \frac{1}{\text{size of soil particle}}}$$

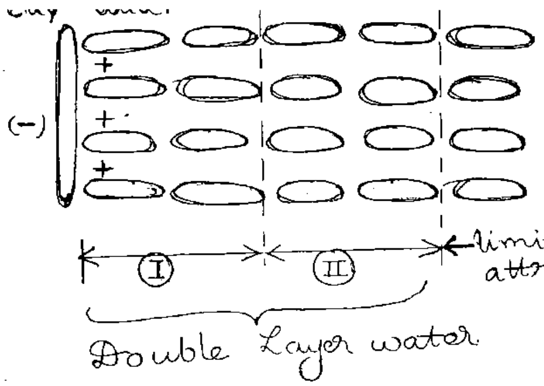
Gravel → least SSA
Sand
Silt
Clay → highest SSA

increasing order ↓

→ Diffuse Double Layer Water (or Adsorbed Water)

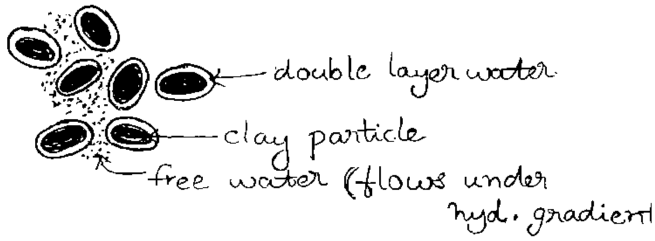


clay	water molecule (Dipolar)	Kerosene molecule (Non polar)



(I) → strongly held layer of water

(II) → loosely held layer of water



Double layer water :

- present only in clays
- causes plasticity property to the clay.

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4. INDEX PROPERTIES OF SOILS

Soil Properties

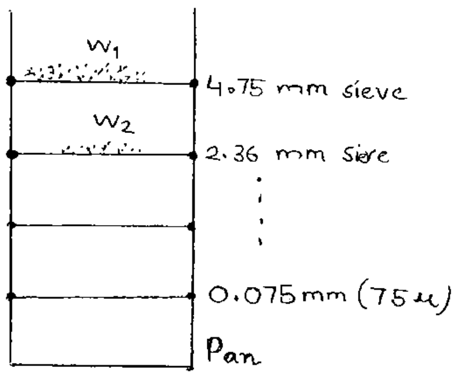
1. Index Properties
   - indicative of behaviour of soil.
   - Eg:- grain size distribution, relative density, atterberg limits.

2. Engineering Properties
   - used for engg. applications
   - Eg:- permeability, shear strengt compressibility.

→ Grain Size Distribution
- by (i) Sieve analysis → used if size > 75 μ
- (ii) Sedimentation analysis. → used if size < 75 μ

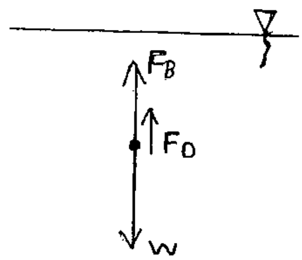
* Sieve Analysis.

size (mm)	% retained.	cumulative %	% finer.
4.75	$P_1 = \frac{W_1}{W} \times 100$	$P_1 = P_1$	$100 - P_1$
2.36	$P_2 = \frac{W_2}{W} \times 100$	$P_2 = P_1 + P_2$	$100 - P_2$
⋮			
0.075			



size vs % finer graph is plotted.

* Sedimentation Analysis
- an indirect method.
- based on "Stoke's Principle"



Settling velocity of particle,

$$V_s = \frac{g}{18}(G-1)\frac{d^2}{\gamma} \longrightarrow \text{Stoke's equation.}$$

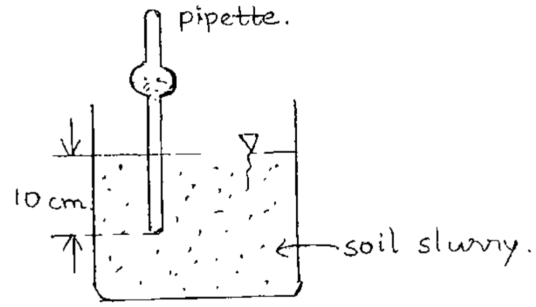
$$\underset{(mm/s)}{V_s} \approx 900 \underset{(mm)}{d^2} \longrightarrow \text{approximate stoke's eqn}$$

Assumptions:

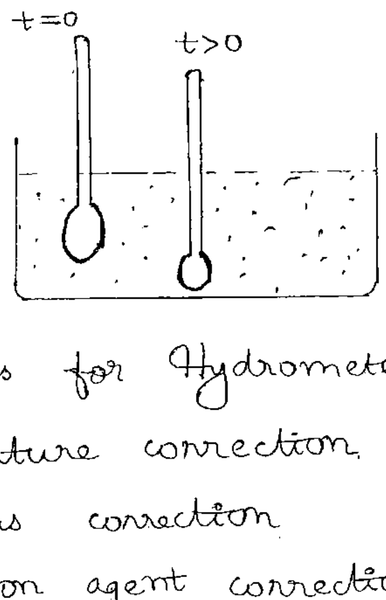
- laminar flow
- particle settle independently without interference.
- Stoke's law is valid only if size is between 0.2μ–0.2mm
  - If size > 0.2 mm, it will cause turbulent condition
  - If size < 0.2 μ, there will be 'Brownian movement' (zig-zag movement).

* Sedimentation Analysis methods:

1. Pipette Method.



2. Hydrometer method.

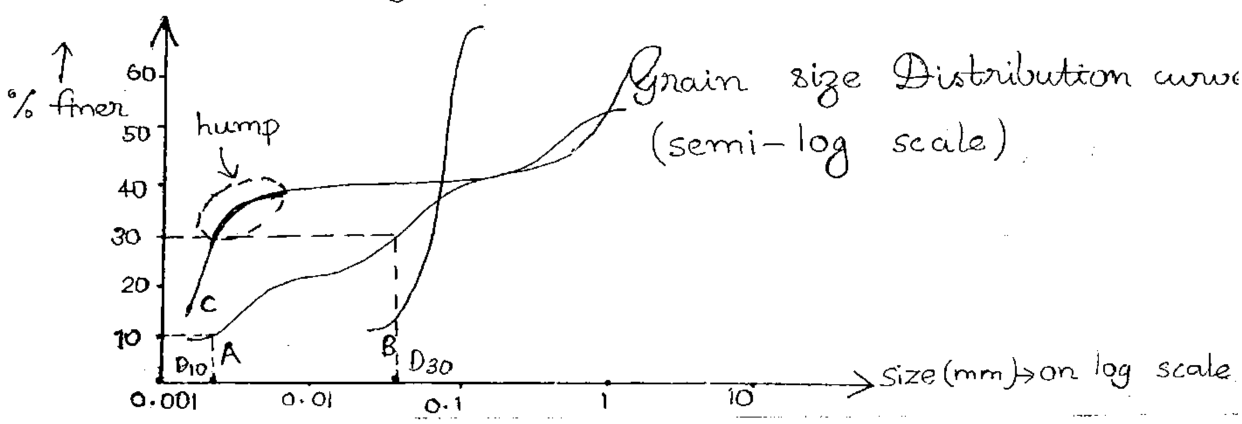


> Corrections for Hydrometer reading.

(i) Temperature correction. (if temp ≠ 27°c)

(ii) Meniscus correction

(iii) Dispersion agent correction (Sodium hexametaphosphate)



'Log Scale' is used because:

a) There is large range of data. ($\frac{10}{0.001} = 10000$ times)

b) to get straight line

A → well graded soil — has different sizes

B → uniformly graded soil — same size of particles. (sand dun

C → gap graded soil :- certain sizes are missing.

B&C → poorly graded soil.

→ Important size of Soil Particle.

a) D_{10} → effective size of soil.

b) D_{30}

c) D_{60}

D : diameter & 10, 30, 60 → % finer

→ Coeffecient of Uniformity (C_u)

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

For well graded gravel, $C_u > 4$

For well graded sand, $C_u > 6$.

If C_u lies b/w 1 & 2, it is called "Uniformly Graded S

→ Coeffecient of Curvature (C_c)

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

> For a well graded soil, $1 < C_c < 3$

> C_c represents shape of curve.

NOTE: Grain size distribution curve is useful only for cohesionless soil (gravel, sand). In the case of clay, grain size distribution curve is not useful, ∵ the clay behaviour is mainly controlled by consistency limits.

→ Relative Density, or Density Index, I_D

$$\boxed{I_D = \left(\frac{e_{max} - e}{e_{max} - e_{min}}\right) \times 100} \qquad e_{min} \leq e \leq e_{max}$$

e_{max} → max. void ratio in the loosest state.

e_{min} → min. void ratio in the densest state.

e → natural or insitu void ratio.

If the soil is the loosest state $(e = e_{max})$, $I_D = 0$.

If the soil is in the densest state $(e = e_{min})$, $I_D = 100\%$

$$0 \leq I_D \leq 100\%$$

- $I_D < 15\%$ → very loose ~~s~~state
- $15 \leq I_D < 35\%$ → loose state.
- $35 < I_D < 65\%$ → medium dense. state.
- $65 < I_D < 85\%$ → dense state
- $I_D > 85\%$ → very dense state.

The more the I_D value, more will be the density,

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

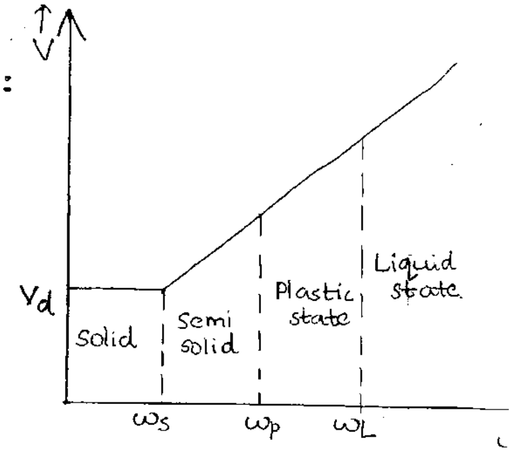
$$\Rightarrow \boxed{I_D = \left[\frac{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_{d\,max}}}\right] \times 100}$$

→ Consistency Limits or Atterberg Limits

- exist only for cohesive soil.
- depending upon water content of soil;
  1. Liquid State.
  2. Plastic state
  3. Semi-solid state
  4. Solid state.

* Atterberg Limits : these are the boundary w/c b/w two different states of soil

* Types of Atterberg Limits :
1. Liquid limit, LL or ω_L
2. Plastic limit, PL or ω_p
3. Shrinkage limit, SL or ω_s



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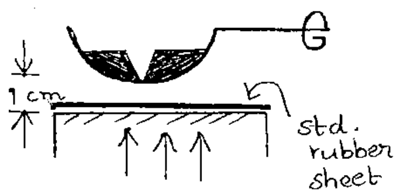
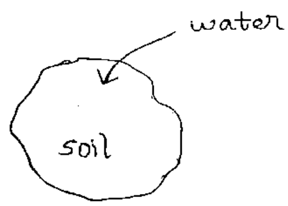
→ To find ω_L

– Casagrande's Liquid limit test.





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...	..
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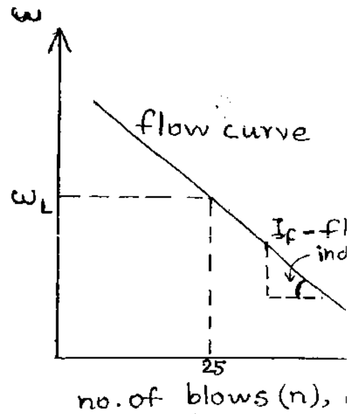
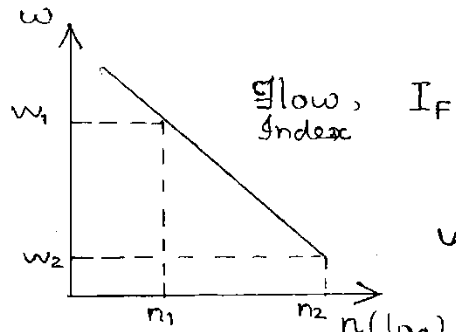
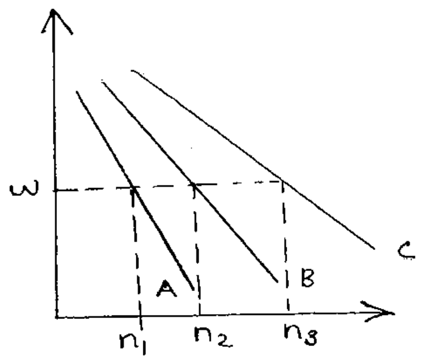




Flow Index, $I_F = \dfrac{w_1 - w_2}{\log_{10}\left(\frac{n_2}{n_1}\right)}$; slope of the flow curve.

w_L – w/c at which 25 no. of blows can close the groove of the apparatus



Flat Flow curve has relatively more shear strength compared to a steep flow curve.

($n_1 < n_2 < n_3$, more no. of blows required to close the groove.
∴ Shear strength of A < B < C)

$$\boxed{\text{Shear strength} \propto \frac{1}{I_F}}$$

* Instead of standard rubber sheet, if hard rubber sheet is used, then ω_L decreases. If soft rubber sheet is used, then ω_L increases.

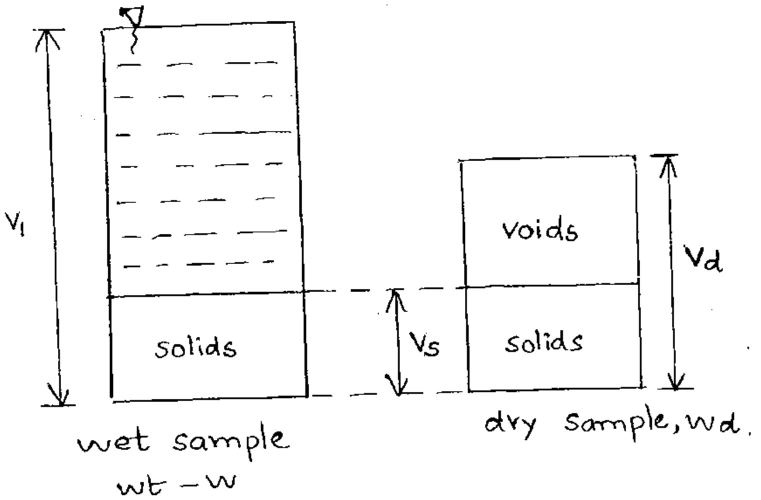
* At liquid limit condition, shear strength of soil is 2.7 kN/m² and is same for all soils. (Shear strength is zero for all soils in the liquid state)

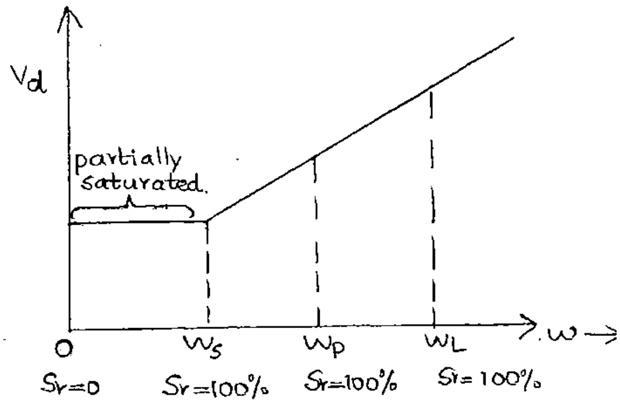
→ Plastic Limit Test. (or Thread Test)

Plastic limit is the minimum w/c at which soil can be rolled into a thread of <u>3mm diameter</u> without crumbling.

→ Shrinkage Limit.

Shrinkage limit is the minimum w/c that can make the soil fully saturated ($S_r = 100\%$). or it is the min. w/c that can be filled in the voids of soil with 100% desaturation





1. $$\omega_s = \frac{(V_d - V_s)\gamma_w}{W_d} \times 100$$

$$\omega_s = \left(\frac{1}{\gamma_d} - \frac{1}{\gamma_s}\right)\gamma_w \times 100$$

$$= \left(\frac{1}{G_m} - \frac{1}{G}\right) \times 100$$

2. $$\omega_s = \left(\frac{1}{G_m} - \frac{1}{G}\right) \times 100$$

$$e = \frac{\omega G}{S_r}$$

$$e = \frac{\omega_s G}{1}$$

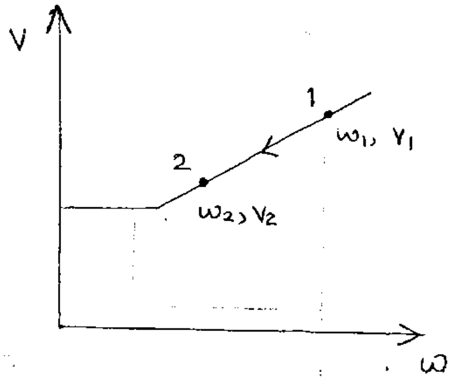
3. $$\omega_s = \frac{e}{G} \times 100$$; $e \rightarrow$ void ratio at saturation level or dry condition.

4. $$\omega_s = \left(\omega_1 - \frac{(V_1 - V_d)\gamma_w}{W_d}\right) \times 100$$

$\omega_1 \rightarrow$ initial water content $= \dfrac{\omega - \omega_d}{\omega_d}$

→ Shrinkage Ratio (SR).

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



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

$$SR = \frac{(V_1 - V_d)/V_d \times 100}{\omega_1 - \omega_s}$$

→ Volumetric Shrinkage (VS)

$$VS = \frac{V_1 - V_d}{V_d} \times 100$$

$$\therefore \boxed{SR = \frac{VS}{\omega_1 - \omega_s}}$$

$$\left\{ SR = \frac{\frac{(V_1 - V_d)}{V_d}}{\frac{(V_1 - V_d)\gamma_w}{W_d}} = \frac{\gamma_d}{\gamma_w} \right.$$

Also $SR = \dfrac{\gamma_d}{\gamma_w}$; $\gamma_d = \dfrac{W_d}{V_d}$

SR is the mass specific gravity in dry condition.

→ Plasticity Index, I_P

$$\boxed{I_P = w_L - w_P}$$

If $I_P = 0$; non plastic soil.

$I_P < 7$; low plastic soil.

$7 - 17$; medium plastic soil.

> 17 ; high plastic soil.

→ Shrinkage Index, I_S

$$\boxed{I_S = w_P - w_S}$$

→ Toughness Index, I_T

$$\boxed{I_T = \frac{I_P}{I_f}}$$

Shear strength $\propto I_T$

→ Consistency Index, I_c

$$\boxed{I_c = \frac{w_L - w}{I_P} \times 100}$$

→ Liquidity Index, I_L

$$\boxed{I_L = \frac{w - w_P}{I_P} \times 100}$$; $w \rightarrow$ natural w/c of soil.

$$\boxed{I_c + I_L = 1 \text{ or } 100\%}$$

- If $I_L > 1$, the soil is in liquid state.
  ($w > w_L \rightarrow$ liquid state)
- If I_L is -ve, the soil is either in semi-solid or solid sta
  ($w < w_P$).

→ Activity Number, A

$$A = \frac{I_P}{C} \quad ; \quad C = \% \text{ of clay particles.}$$

- For a given soil, A is constant.

'A' indicates swelling and shrinkage characteristics.

If A < 0.75 ; inactive soil.

0.75 – 1.25 ; normal active soil.

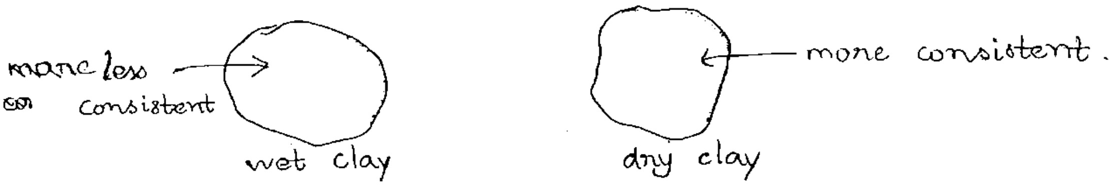
> 1.25 ; active soil. (Bentonite & BC soil)

→ Effect of Size of Particle.

- If size of particle decreases, w_L increases
  w_p ~~de~~ increases ⇒ I_p increa

- If $\underbrace{\text{silt or fly ash}}_{\text{non-plastic}}$ is added to clay, w_L decreases
  w_p decreases ⇒ I_p decrea

- consistency : resistance against deformation, depends o water content. More the w/c, less the consiste



- plasticity : property due to which soil deforms plastically without rupture is called plasticity.

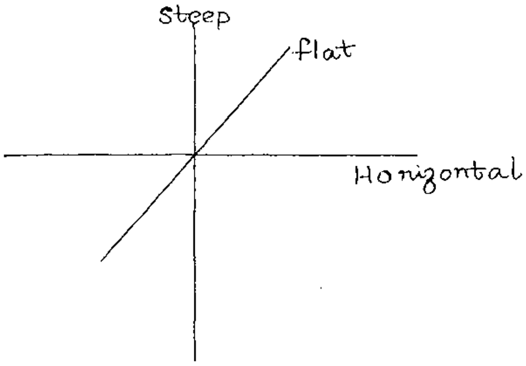
- If lime is added to clay, w_L decreases, w_p increases and I_p decreases.

18 (16) (17)

P-17

2. Fat Clay → highly compressible clay : $w_L > 50\%$

22



29th Aug,
FRIDAY

P-19.

1. $\gamma = 1746\ kg/m^3$; $w = 8.6\%$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{1746}{1.086} = 1607\ kg/m^3 \quad ; \quad \gamma_s = 2.6\ g/cc$$

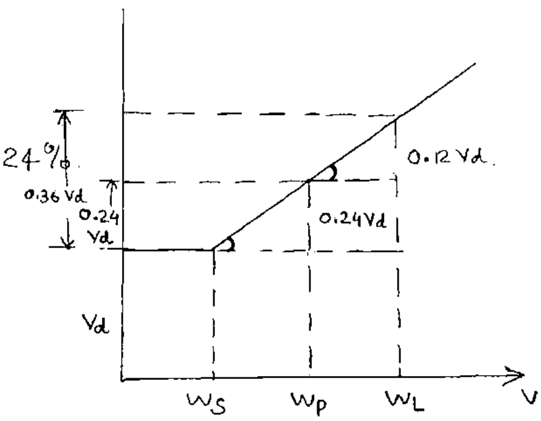
$\therefore G = 2.6.$

$$\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow e = 0.617.$$

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.642 - 0.617}{0.642 - 0.462} = \underline{\underline{13.9\%}}$$

2. $w_L = 45\%$; $w_p = 33\%$

$$\frac{V_L - V_d}{V_d} \times 100 = 36\% \quad ; \quad \frac{V_p - V_d}{V_d} \times 100 = 24\%$$



$$\frac{0.36 V_d - 0.24 V_d}{w_L - w_p} = \frac{0.24 V_d}{w_p - w_s}$$

$$\frac{0.12}{45-33} = \frac{0.24}{33 - w_s}$$

$$\therefore w_s = \underline{\underline{9\%}}$$

3. $$SR = \frac{\dfrac{V_L - V_d}{V_d} \times 100}{w_L - w_s} = \frac{36}{45-9} = \underline{\underline{1}}$$

4. $G_m = \frac{\gamma_{sat}}{\gamma_w} = 1.88$

$G_m = \frac{\gamma_d}{\gamma_w} = 1.74$

$e = \frac{wG}{S_r} = 0.46$

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

$1.88 = \frac{G+0.46}{1+0.46}$

$\underline{\underline{G = 2.9}}$

5. $w_s = \left(\frac{1}{G_m} - \frac{1}{G} \right) 100 = \left(\frac{1}{1.74} - \frac{1}{2.9} \right) 100 = \underline{\underline{23\%}}$

6. $SR = \frac{\gamma_d}{\gamma_w} = \underline{\underline{1.74}}$

7 $W = 95.6$ gm ; $V_1 = 68.5$ cc

$W_d = 43.5g$; $V_d = 24.1$ cc.

Initial water content of soil, $w_1 = \left(\frac{W - W_d}{W_d} \right) \times 100$

$= 119.77\%$

Shrinkage limit, $w_s = \left(w_1 - \frac{(V_1 - V_d)\gamma_w}{W_d} \right) 100$

$= \left(1.197 - \frac{(68.5 - 24.1)1}{43.5} \right) 100$

$= \underline{\underline{17.7\%}}$

8. $w_s = \left(\frac{1}{G_m} - \frac{1}{G} \right) 100$

$G_m = \frac{\gamma_d}{\gamma_w}$; $\gamma_d = \frac{W_d}{V_d} = 1.804.$

$G_m = 1.804$

$17.7\% = \left(\frac{1}{1.804} - \frac{1}{G} \right) 100$

$G = \underline{\underline{2.65}}$

9. To find initial void ratio, e_1 :

$$e_1 = \frac{w_1 G}{S_r} = \frac{1.197 \times 2.65}{1} = \underline{\underline{3.15}}$$

OR

$$\gamma_{d_1} = \frac{W_s}{V_1} = \frac{43.5}{68.5} = 0.635 \text{ g/cc}$$

$$\gamma_{d_1} = \frac{\gamma_w G}{1+e_1} \Rightarrow e_1 = \underline{3.15}$$

OR

$$\gamma_{sat} = \gamma_w \left(\frac{G+e_1}{1+e_1}\right)$$

$$\frac{W}{V_1} = \left(\frac{2.65+e_1}{1+e_1}\right) = \frac{95.6}{68.5}$$

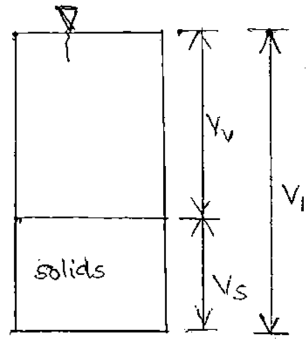
$$e_1 = \underline{\underline{3.15}}$$

OR

$$V_s = \frac{W_s}{\gamma_s} = \frac{43.5}{G\gamma_w} = 16.42 \text{ cm}^3.$$

$$V_v = V_1 - V_s = 52.08 \text{ cm}^3.$$

$$e_1 = \frac{V_v}{V_s} = \frac{52.08}{16.42} = \underline{\underline{3.17}}$$



To find final void ratio, e_2 :

$$e_2 = w_s G$$

$$= \frac{17.7}{100} \times 2.65 = 0.\underline{\underline{47}}$$

OR

$$\gamma_{d2} = \frac{W_s}{V_2} = 1.804 \text{ g/cc}$$

$$\gamma_{d2} = \frac{\gamma_w G}{1+e_2}$$

$$e_2 = \underline{\underline{0.47}}$$

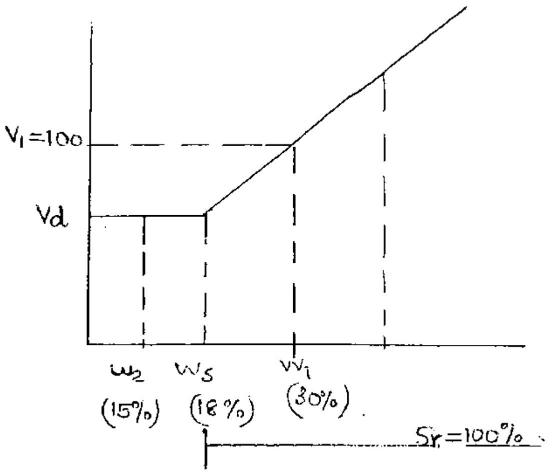
(OR)

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

$$\frac{24.1}{68.5} = \frac{1+e_2}{1+3.15}$$

$$e_2 = \underline{\underline{0.47}}$$

10. Let e_1 be void ratio of w_1 of 30%.



$$e_1 = \frac{w_1 G}{Sr} = \frac{0.3 \times 2.72}{1} = 0.816$$

$V_1 = 100$ cc.

Let e_2 be void ratio at w_s.

$$e_2 = w_s G = 0.489.$$

$$\frac{V_1}{V_d} = \frac{1+e_1}{1+e_2}.$$

$$\frac{100}{V_2} = \frac{1+0.816}{1+0.489} \Rightarrow V_2 = \underline{\underline{82 \text{ cc}}} = V_d$$

For all $w < w_s$, $\underline{\underline{V_2 = V_d}}$

(OR)

$$w_s = w_1 - \frac{(V_1 - V_d)\gamma_w}{w_d.} \quad (\text{lengthy})$$

11. $\gamma_d = \frac{W_d}{V} = \frac{890}{225} = 1.733$ g/cc.

$$\gamma_d = \frac{\gamma_w G}{1+e_1} \Rightarrow e_1 = 0.56.$$

Let e_2 be void ratio at increased volume.

$$\frac{V_2}{V_1} = \frac{1+e_2}{1+e_2} \Rightarrow \frac{1.08\, V_1}{V_1} = \frac{1+e_2}{1+0.56} \qquad \therefore e_2 = \underline{\underline{0.684}}$$

$$e_2 = \frac{\omega_2 G}{S_r} \Rightarrow 0.47 = \omega_2 \times 2.7$$

$$\omega_2 = \underline{\underline{25.4\%}}$$

12. $$C_u = \frac{D_{60}}{D_{10}} = 4$$

$$D_{60} = 4 D_{10}.$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = 1$$

$$\frac{D_{30}^2}{4 D_{10}^2} = 1 \Rightarrow \frac{D_{30}}{D_{10}} = \underline{\underline{2}}$$

13. $$I_p = w_L - w_p$$

$$15 = 56 - w_p$$

$$\therefore w_p = 41\%$$

$\because$ given natural water content ($w = 45\%$) lies b/w liquid limit & plastic limit, the soil is in plastic state.

5. SOIL CLASSIFICATION

→ IS Particle Size Classification System

* Grain Size only - criteria

2μ		75μ		4.75mm		80mm		300 mm

Clay	Silt.	Sand.	Gravel.	Cobble	Boulo

The colloidal soil particle size is less than the clay size. However in our Indian code of practise, colloidal soil is not recognized.

But the Indian classification based on size alone is not always true. For eg:- Rock dusts or rock powder's particle size is less than 2μ and hence belong to clay as per Indian system. But rock dust has no plasticity.

→ HRB Soil Classification System.

- HRB – Highway Research Board.

* Criteria : a) Grain size distribution.

b) Consistency or Atterberg limits

- This system is more useful for pavement design.
- In this system, soils are given group numbers like A_1, A_2, A_3 . . . . A_7, A_8.
- The smaller the group number, the better the soil for pavement purpose. ie A_1 is better than A_2. A_2 is bette than A_3 and so on.

A_8 group – highly organic soil (worst soil for constructo

A_7 group – black cotton soil.

- Group Index, GI : an index value calculated by an emporical equation. GI value depends on –
  a) % of soil passing 75 μ sieve.
  b) ω_L
  c) ω_P or I_P

GI values ranges from 0 to 20

If GI value is found to be negative from equation, it is reported as 'zero'.

GI indicates quality of soil within its own group.

The smaller the GI value, the better the soil for pavement purpose.

- Symbol : A5 (6)
  - 6 ← GI
  - A5 ← Group no.

→ **Unified Soil Classification System.**

- most popular in European countries.
- criteria : a) grain size distribution data.
  b) consistency limits
  c) compressibility characteristics.

→ **IS Soil Classification System.**

- followed by all engineering departments in India.

Soil.

- Coarse grained Soil.
  - if >50% retained on 75 μ IS sieve.
  - Gravel (G)
  - Sand (S)
- Fine grained Soil.
  - if >50% passes 75 μ IS Sieve
  - Silt (M)
  - Clay (C)
  - Organic Soil. (O)
- Highy Organic Soil(P
  - dark colours (black or dark brow
  - bad odour
  - Eg: Peat, muck, humus.

(Mo = silt in european langua

Gravel – if more than 50% of coarse fraction retained on 4.75 mm sieve

Sand – if > 50% coarse fraction passes 4.75 mm sieve.

- Gravel :
  - a) Well graded Gravel – GW
  - b) Poorly graded Gravel – GP
  - c) Silty gravel. – GM
  - d) Clayey gravel – GC

- Sand :-
  - a) Well graded sand – SW
  - b) Poorly graded sand – SP
  - c) Silty ~~gravel~~ sand – SM
  - d) Clayey ~~gravel~~ sand – SC

⊙ Clayey sand
$\Rightarrow$ clay qty < sand qty

⊙ Siltey Clayey gravel
$\Rightarrow$ silt qty < clay qty < gravel qty

⊙ Sandy clay.
$\Rightarrow$ sand qty < clay qty.

- Silt :-
  - a) Low compressible silt – ML
  - b) Intermediate compressible silt – MI
  - c) Highly compressible silt. – MH

- Clay :-
  - a) CL
  - b) CI
  - c) CH

- Organic Soil
  - a) OL
  - b) OI
  - c) OH

- Total group symbols $= \underset{\text{(coarse)}}{8} + \underset{\text{(fine)}}{9} + \underset{\text{(Pt)}}{1} = 18$ groups.

- If $\omega_L < 35\%$ – Low compressible

  $35\% < \omega_L < 50\%$ – Intermediate compressible.

  $\omega_L > 50\%$ – Highly Compressible.

30th Aug, SATURDAY

→ Plasticity Chart.

- to classify the fine grained soils.

* To identify Organic Soils:

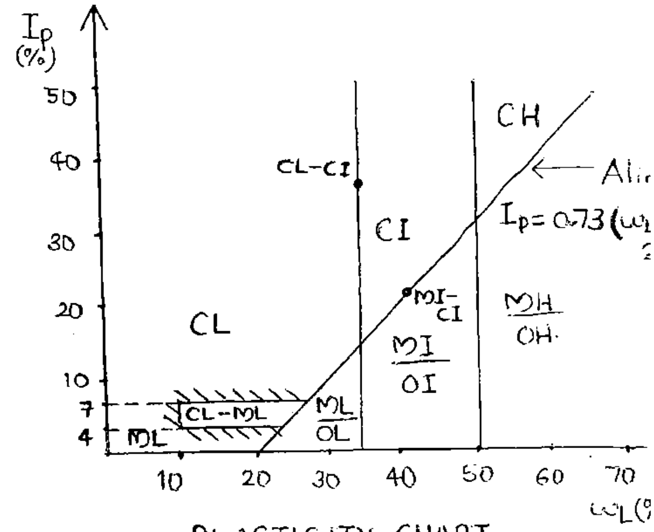
a) Colour test

Dark colours (black or dark brown)

b) Odour test.

Bad odour.

c) w_L test before and after oven drying.



PLASTICITY CHART.

For organic soils, w_L decreases (by more than one fourth of initial value) after oven drying.

* Boundary Classifications:

(i) CL – CI, CI – CH

(ii) ML – MI, MI – MH

(iii) OL – OI, OI – OH

(iv) MI – CI, MH – CH. (coarser particle should be given preference)

∴ MI – CI ✓
CI – MI ✗

Q.1 Classify the fine grained soil with $w_L = 60\%$, $w_p = 20\%$.

I_p of soil $= 60 - 20 = 40\%$

I_p of A-line $= 0.73(w_L - 20) = 29.2\%$

Since I_p of soil is $>$ I_p of A line, the point plots above A line ⇒ clay.

$w_L > 50$ ⇒ highly compressible.

∴ Soil is CH.

Q. $w_L = 20\%$, $w_p = 15\%$

$I_p = 5\%$ (b/w 4 & 7)

$w_L = 20\%$

$\Rightarrow$ CL − ML

— Equation of I_p vs w_L is called A-line because A is the surname of A. Casagrande.

→ GW : (i) if fines <5% (ii) $C_u > 4$ (iii) C_c lies b/w 1 & 3

GP : if (i) fines < 5% (ii) not meeting above gradation requirements (C_c & C_u).

GM : if (i) fines > 12% (ii) I_p value < 4% or Atterberg limits fall below A-line.

GC : if (i) fines >12% (ii) I_p > 7% with Atterberg limit fall above A-line.

If fines lies between 5% & 12%, it is a border line case requiring dual symbol.

For eg: GW − GC, GP − GM, GP − GC, GW − GM

If I_p lies b/w 4 & 7%, it is a border line case requiring dual symbol. For eg:- GM − GC.

Q. Fines = 15%, $I_p = 6\%$

$\Rightarrow$ GM − GC.

Q. Fine = 3%, $C_u = 5$, $C_c = 2$

$\Rightarrow$ GW.

Q. Fines = 15%, $I_p = 2\%$ $\Rightarrow$ GM.

Q. Fines = 10%, $C_u = 5$, $C_c = 2$, $I_p = 5\%$

$\Rightarrow$ GW − GM

In case of border line cases, the coarser one is to be favour (or) the coarser one is given priority.

Between organic soil and clay, the organic one is coarser on

CI – OI ✗
OI – CI ✓

→ In the case of sand, all the conditions are same except $C_u > 6$.

→ Single Clay Particle } not visible to
Single silt particle } naked eye

P-24.

2.

Size	% Retained	Cumulative %	% Finer
600 μ	$\frac{240}{600} \times 100 = 40\%$	40%	60%
500 μ	$\frac{300}{600} \times 100 = 50\%$	90%	10%
425 μ	10%	100%	0

D_{60} = diameter corresponding to 60% finer
= 600 μ.

$D_{10} = 500\ \mu$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{600}{500} = 1.2$$

3. $C_u < 6$
$\Rightarrow$ SP

4. $W_L = 42\%$ ie b/w 35% & 50% $\Rightarrow$ intermediate, I

MI

5. Fines $= \frac{250}{1000} \times 100 = 25\%$

Coarse fraction = 100 – 25 = 75% ⇒ coarse grained soil.

$I_p = 42 - 20 = 22\% \quad > 7\%$

I_p of A line $= 0.73(w_L - 20) = 16.06\%$

$I_p > I_p$ of A line. $\therefore$ pt. lies above A line.

$\therefore$ SC

6. Fines $= 30\%$ (silt + clay)

$\therefore$ Coarse fraction $= 100 - 30 = 70\%$ (gravel + sand).

$\therefore$ It is coarse grained soil.

Gravel fraction $= 100 - 60 = 40\%$ (more than 50% of 70%)

$\left\{ \begin{array}{l} \text{Gravel + sand} = 70\%. \\ \therefore \text{Sand fraction} = 30\%. \end{array} \right\}$

$\therefore$ soil is gravel. ($\because$ gravel % > sand %)

I_p of soil $= 35 - 27 = 8\%$

I_p of A line $= 0.73(w_L - 20) = 10.95\%$

$\therefore$ point falls below A-line. $\Rightarrow$ GM

7. $\left. \begin{array}{l} C_u = \dfrac{D_{60}}{D_{10}} = 1.78. \\ C_c = \dfrac{D_{30}^2}{D_{60} \cdot D_{10}} = 0.95 \end{array} \right\}$ poorly graded.

$D_{60} = 0.41$ mm. $\Rightarrow$ 60% passing 0.41 mm (or 4.75 mm)

$\therefore$ sand

SP

8.

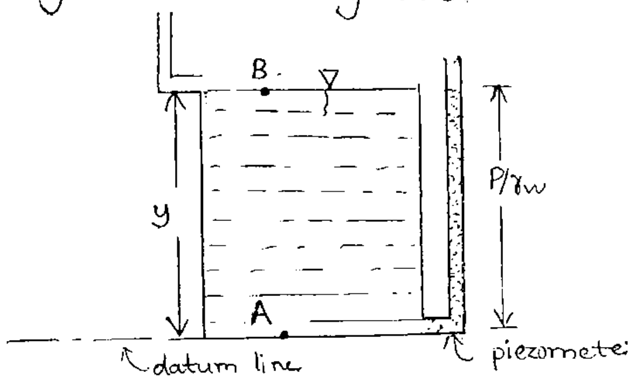
Sept,
ONDAY

6. PERMEABILITY

- Flow occurs only when there is difference in the total heads b/w the two points.

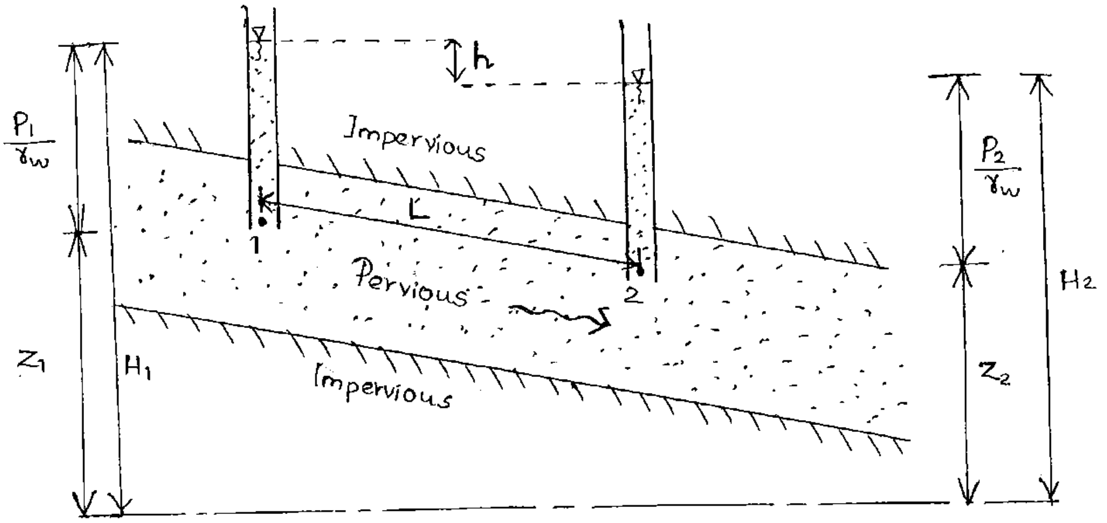
- Pressure head difference alone or elevation difference alone may not cause the flow.

- In the case of soils the velocity head is neglected (negligible),



At point A	At point B
$\frac{P}{\gamma_w} = y$	$\frac{P}{\gamma_w} = 0$
$z = 0$	$z = y$
Total head, $H = y$	Total head, $H = y$

$\because$ total head is same $(=y)$ at both A & B, no flow occurs b/w A B.



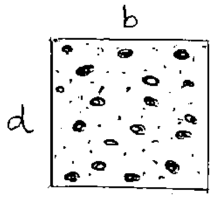
$\therefore$ Total head difference or total head loss, $h = H_1 - H_2$

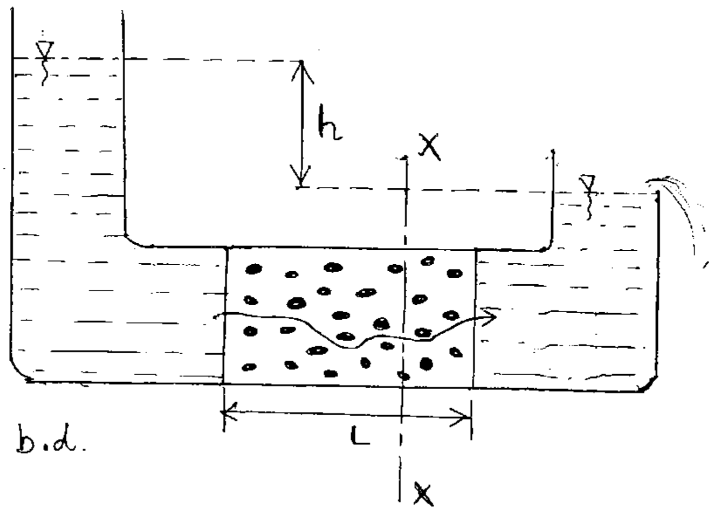
The total head loss b/w any two points in a soil mass is equal to the difference in the elevations of water in the two piezometers kept at those two points.

Hydraulic gradient, $i = \frac{h}{L}$; $L \rightarrow$ seepage length.

Hydraulic gradient is the head loss per unit seepage length.

c/s at XX :





Total c/s area of soil, $A = b.d.$

Area of voids or Actual area of flow $\}$ $= A_v = nA$. ; $n \rightarrow$ porosity of soil.

Discharge velocity or Apparent velocity $\}$ $V = \frac{Q}{A}$.

Actual velocity or Seepage velocity $\}$ $V_s = \frac{Q}{A_v}$.

$$Q = AV = A_v V_s$$

$$\boxed{V_s = \frac{V}{n}}$$

$n < 1$ always, $\Rightarrow V_s > V$.

→ Darcy's Law

Discharge velocity $\propto$ Hydraulic Gradient.

$$V \propto i$$

DARCY'S EQUATION $\boxed{V = ki}$

$K \rightarrow$ Coefficient of Permeability of soil.

If $i = 1$, $v = K \Rightarrow$ Coeffecient of ~~velocity~~ permeability is the discharge velocity occuring under unit hydraulic gradient.

Units of k : cm/s. or m/s or m/hour.

If $k > 10^{-1}$ cm/s $\Rightarrow$ 'Permeable Soil'

$k < 10^{-7}$ cm/s $\Rightarrow$ 'Impermeable Soil'

Permeable soil — gravel, coarse sand.

Impermeable soil — stiff clays.

Soil	Gravel	Sand	Silt	Clay.
K (cm/s)	10^{0} ($10^{-1} - 10^{1}$)	10^{-2} ($10^{-1} - 10^{-3}$)	10^{-4} ($10^{-3} - 10^{-5}$)	10^{-6} ($10^{-5} - 10^{-7}$)

⊙ Darcy's Law is valid for Laminar flow only

In soils, if reynold's number, $Re \leq 1$, it is laminar.

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

$$Re \propto v \propto K$$

∴ as $K \downarrow$, $Re \downarrow$ and flow becomes laminar.

⊙ Clay, silt & fine sand — flow is laminar.

$$V = Ki$$

$$\boxed{V_s = K_p\, i}$$

$K_p \rightarrow$ coeffecient of percolation.

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

$$\boxed{K = C \cdot D_{10}^{2}\, \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\mu}}$$

$C \rightarrow$ shape constant ($C = 1$ for perfectly spherical particle)

$\mu \rightarrow$ dynamic viscosity.

→ Factors affecting K of Soil:

- shape of particle
- size of particle
- void ratio
- properties of fluid
- degree of saturation
- organic matter
- specific surface area
- stratification etc.

* Effect of Size, D_{10}

$$K \propto D_{10}^2$$

* Effect of Specific Surface Area

$$K \propto \frac{1}{SSA}$$

* Effect of shape.

K of rounded particles is more compared to angular particle since SSA of rounded particle is less.

* Effect of void ratio

$$K \propto \frac{e^3}{1+e} \quad ; \quad \text{also} \quad K \propto e^2$$

$$\text{also} \quad \log K \propto e \ \text{(latest one)}$$

The more the void ratio, more will be permeability. But this cannot explain the reason for clay's low permeability and clay has low permeability though it has high void ratio.

$$K \propto \frac{Ce^3}{1+e}$$

Clay has lowest value of C. So the product of C & e will be low.

$$K \propto \gamma_w$$

$$K \propto \frac{1}{\mu} \propto \text{temperature} \qquad \mu \propto \frac{1}{\text{temp}}$$

$$\Rightarrow K \propto \text{temperature}$$

$$\therefore K \propto \frac{\gamma_w}{\mu} \quad \Rightarrow \quad \boxed{\frac{K_2}{K_1} = \frac{\gamma_{w2}}{\gamma_{w1}} \cdot \frac{\mu_1}{\mu_2}}$$

During summer season days, permeability will be more.

* Effect of Degree of Saturation

k of partially saturated soil is relatively less compared to fully saturated soil. (air blocking)

* Effect of Organic matter

Organic matter decreases k of soil. Due to low specific gravity of organic matter, it flows along with water and fills the voids.

$$k = \overbrace{C \cdot D_{10}^{2} \cdot \frac{e^3}{1+e}}^{\text{soil property}} \cdot \underbrace{\frac{\gamma_w}{\mu}}_{\text{fluid properties}}$$

$$k = k_o \cdot \frac{\gamma_w}{\mu}$$

$k_o \rightarrow$ intrinsic permeability of soil (Inherent property)

Units of k_o : cm^2 or m^2 or darcys

$$\boxed{1 \text{ darcy} = 9.87 \times 10^{-13} \ m^2}$$

→ Tests to Determine k of soil.

1. Constant Head Test
2. Variable Head test
3. Capillarity Permeability test

} lab test.

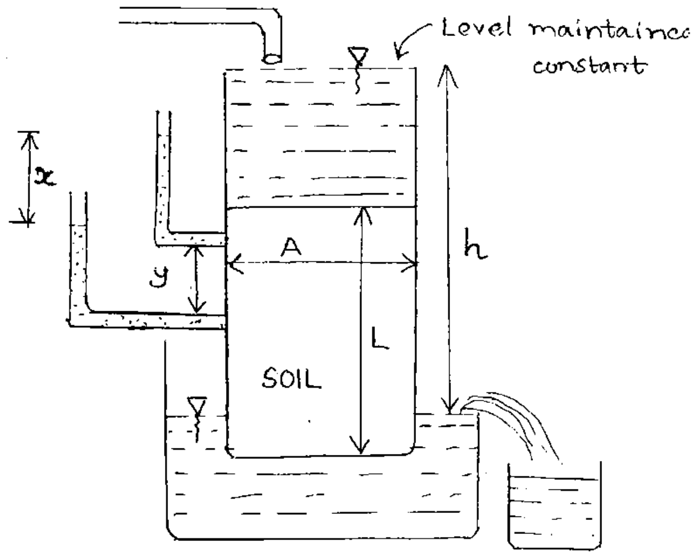
4. Consolidation test
5. Pumping out test
6. Pumping in test.

} Field test.

Pumping out test : most accurate, used for large engg. projects.
Consolidation test : suitable for impermeable clays.
Capillarity Permeability test : for partially saturated soils.
Constant head test : for coarse grained soils
Variable head test : for fine grained soils
Pumping in test : to find k of individual layer of soil.

2nd Sept,
TUESDAY

→ Constant Head Test



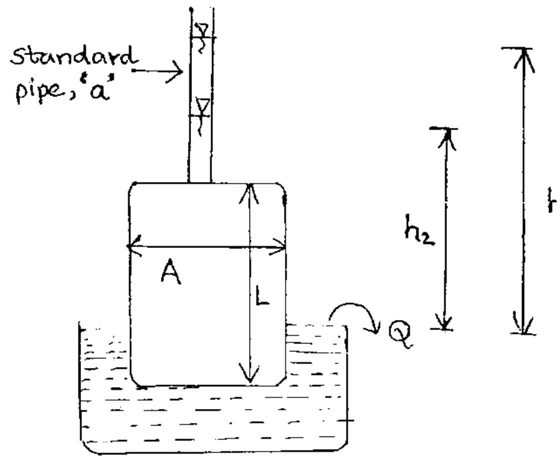
$$Q = k\,i\,A$$

$$Q = k\,\frac{h}{L}\,A$$

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

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

→ Variable Head Test



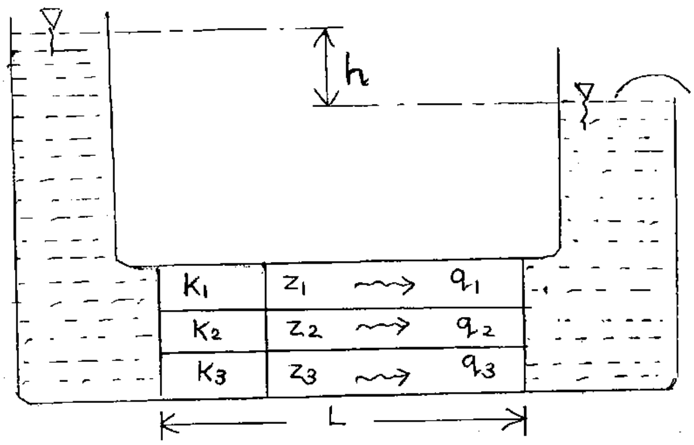
$$a\,dh = Q \cdot dt$$

$$= k\,\frac{h}{L}\,A \cdot dt$$

$$\int_{h_2}^{h_1} \frac{dh}{h} = \frac{kA}{La} \int_0^t dt$$

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

→ Flow Parallel to Bedding Plates.



Total head loss = h

Let h_1, h_2, h_3 → head losses in layers 1, 2 & 3 resptly.

$$h_1 = h_2 = h_3 = h.$$

$$\therefore \; i_1 = i_2 = i_3 = i = \frac{h}{L}$$

$$Q = q_1 + q_2 + q_3$$

$q_1 = k_1 i_1 A_1$; $q_2 = k_2 i_2 A_2$; $q_3 = k_3 i_3 A_3$

$= k_1 \frac{h_1}{L} \cdot (z_1 \times 1)$; $= k_2 \cdot \frac{h_2}{L} \cdot (z_2 \times 1)$; $= k_3 \cdot \frac{h_3}{L} \cdot (z_3 \times 1)$

Let K_H be average permeability for entire soil as a whole.

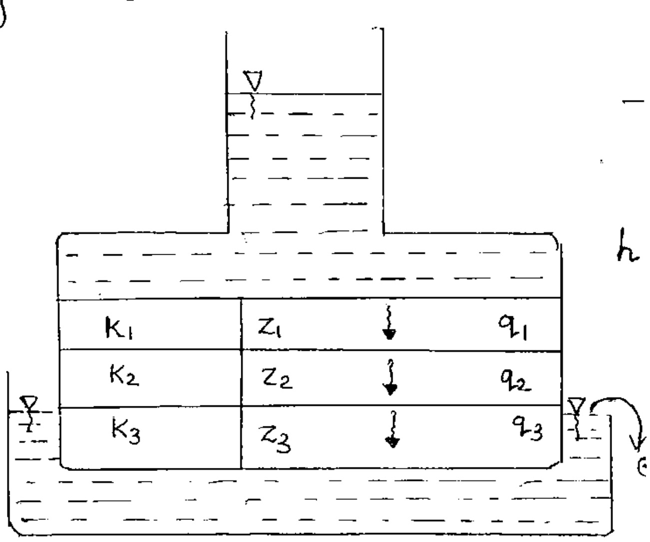
$$Q = k_H \cdot i \cdot A$$
$$= k_H \cdot \frac{h}{L} \cdot (z_1 + z_2 + z_3) \times 1$$

$$k_H \cdot i \cdot A = k_1 \cdot i_1 \cdot A_1 + k_2 \cdot i_2 \cdot A_2 + k_3 i_3 A_3.$$

$$k_H \cdot (z_1 + z_2 + z_3) \cdot 1 = k_1 \cdot z_1 \cdot 1 + k_2 z_2 \cdot 1 + k_3 z_3 \cdot 1.$$

$$\therefore \boxed{k_H = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3}{z_1 + z_2 + z_3}}$$

→ Flow Perpendicular to Bedding Plane.



$h \rightarrow$ total head loss.

$$h = h_1 + h_2 + h_3$$

$$q_1 = q_2 = q_3 = Q.$$

Let K_V be average permeability for entire soil.

$$q_1 = k_1 \cdot i_1 \cdot A_1$$
$$= k_1 \cdot \frac{h_1}{z_1} \cdot A.$$

$$q_2 = k_2 i_2 A_2 \quad ; \quad q_3 = k_3 \cdot \frac{h_3}{z_3} \cdot A.$$
$$= k_2 \cdot \frac{h_2}{z_2} \cdot A$$

$$Q = k_v \cdot i \cdot A$$
$$= k_v \cdot \frac{h}{z_1 + z_2 + z_3} \cdot A$$

$$\therefore \boxed{K_V = \frac{z_1 + z_2 + z_3}{\frac{z_1}{K_1} + \frac{z_2}{K_2} + \frac{z_3}{K_3}}}$$

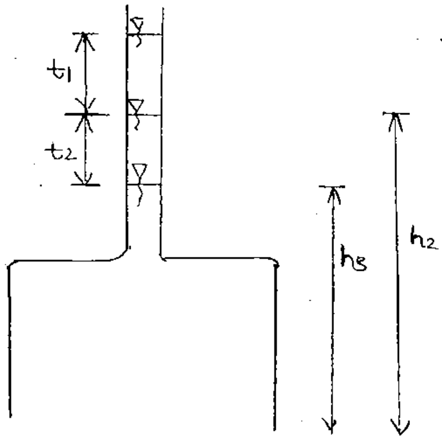
$$\boxed{K_H > K_V}$$

P-30

1. $$\frac{k_2}{k_1} = \frac{\gamma_{w2}}{\gamma_{w1}} \cdot \frac{\mu_1}{\mu_2}$$

$$= \frac{0.9\,\gamma_{w1}}{\gamma_{w1}} \cdot \frac{\mu_1}{0.75\,\mu_1} = 1.2$$

$k_2 = 1.2\ k_1 \implies 20\%$ increase.



2. Given $t_1 = t_2$.

$$\frac{aL}{AK} \log_e \frac{h_1}{h_2} = \frac{aL}{A \cdot K} \log_e \frac{h_2}{h_3}$$

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

$$h_2^2 = h_1 h_3$$

3. $k_1 = 2 \qquad z_1 = 2$

$k_2 = 3 \qquad z_2 = 1$

$k_3 = 1 \qquad z_3 = 2.$

$$K_v = \frac{z_1 + z_2 + z_3}{\frac{z_1}{K_1} + \frac{z_2}{K_2} + \frac{z_3}{K_3}} = \frac{5}{1 + \frac{1}{3} + 2} = \underline{\underline{\frac{3}{2}}}$$

4. $$k_v = \frac{6 + 6 + 6}{\frac{6}{30 \times 10^{-5}} + \frac{6}{4 \times 10^{-4}} + \frac{6}{6 \times 10^{-4}}} = 4.667 \times 10^{-4}\ cm/s$$

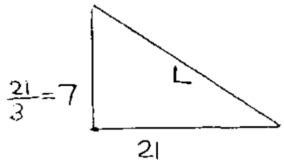
$$t = \frac{aL}{A \cdot K_v} \log_e \frac{h_1}{h_2} = \frac{2 \times 18}{22 \times 4.667 \times 10^{-4}} \log_e \frac{25}{10}$$

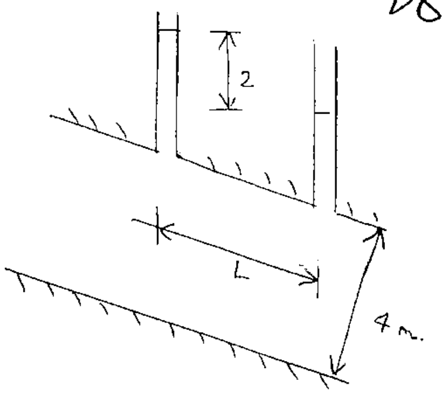
$$= 3748.46\ s$$

$$= \underline{\underline{62.474}}\ min$$

05. $L = \sqrt{21^2 + 7^2}$

$= 22.135$ m.





$Q = kiA$

$= k\frac{h}{L} \cdot d \times 1$

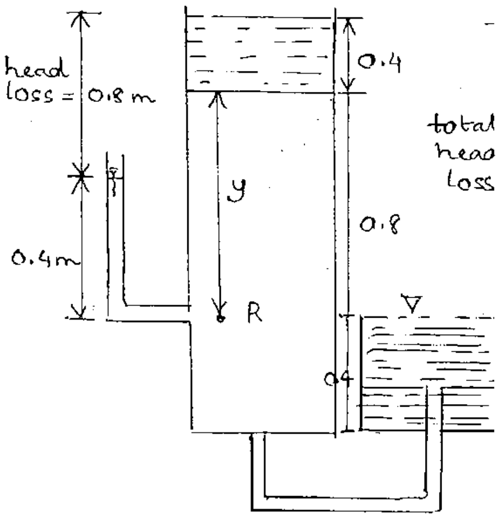
$\frac{5}{1000} = k \cdot \frac{2}{22.135} \times 4 \times 1$

$k = 0.0138$ m/hr $= \underline{\underline{3.85 \times 10^{-6}}}$ m/s

06. Total head loss $= 0.8 + 0.4 = 1.2$ m.

Total seepage length $= 0.8 + 0.4 = 1.2$ m.

$i = \frac{h}{L} = \frac{1.2}{1.2} = 1$



For a seepage length of y,

head loss $= i \times y = 1 \times 0.8 = \underline{\underline{0.8 \text{ m}}}$

$\therefore$ Pressure head at R $= 1.2 - 0.8 = \underline{\underline{0.4 \text{m}}}$

$\because$ datum line is not given, let as assume,

datum line at the d/s water surface

If d/s water surface is datum, then datum head at R = 0.

Total head at R = Pressure head + datum head

$= 0.4 + 0 = \underline{\underline{0.4 \text{ m}}}$

* If datum is taken to be bottom of soil,

datum head at R = 0.4 m.

Total head at R = pressure head + datum head

$= 0.4 + 0.4 = \underline{\underline{0.8 \text{ m}}}$

07 Discharge velocity, $V = ki = k$.

Seepage velocity, $V_s = \frac{V}{n} = \frac{k}{0.5} = \underline{\underline{2k}}$

→ Allen Hazen's Equation :

$$K \approx 100\ D_{10}^{2} \qquad D \to cm \quad K \to cm/s$$

Q The hydraulic conductivity of a soil at a void ratio of 0.8 is 0.047 cm/s. Estimate the hydraulic conductivity at a void ratio of 0.5.

At $e_1 = 0.8$, $K_1 = 0.047$

$e_2 = 0.5$, $K_2 = ?$

$$K \propto \frac{e^3}{1+e} \Rightarrow \frac{K_1}{K_2} = \frac{e_1^3/1+e_1}{e_2^3/1+e_2}$$

$$\frac{0.047}{K_2} = \frac{0.8^3/1.8}{0.5^3/1.5}$$

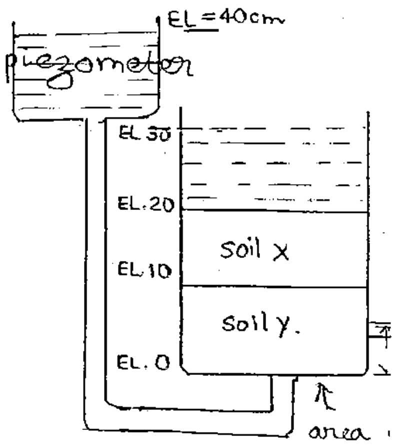
$K_2 = \underline{0.0137}$ cm/s

If $K \alpha e^2$, $K_2 = 0.\underline{01835}$ cm/s

Q. In fig. shown below, the soil X has a permeability of 4×10^{-3} cm, and the head loss in soil Y is 9 times the head loss in soil X a) What is the permeability of the soil Y?

b) What is seepage rate per hour?

c) To what elevation would water rise in a piezometer inserted in soil Y at elevation 5 cm ?



a) Total head loss, $h = 40 - 30 = 10$ cm.

$h_x + h_y = h. = 10$ cm ⟶ ①

Given, $h_y = 9\ h_x$.

∴ $h_x = 1$ cm

$h_y = 9$ cm

$$Q_x = Q_y$$

$$k_x \cdot \frac{h_x}{Z_x} \cdot A_x = k_y \cdot \frac{h_y}{Z_y} \cdot A_y$$

$$4 \times 10^{-3} \times \frac{1}{10} \times 10 = k_y \cdot \frac{9}{10} \cdot 10$$

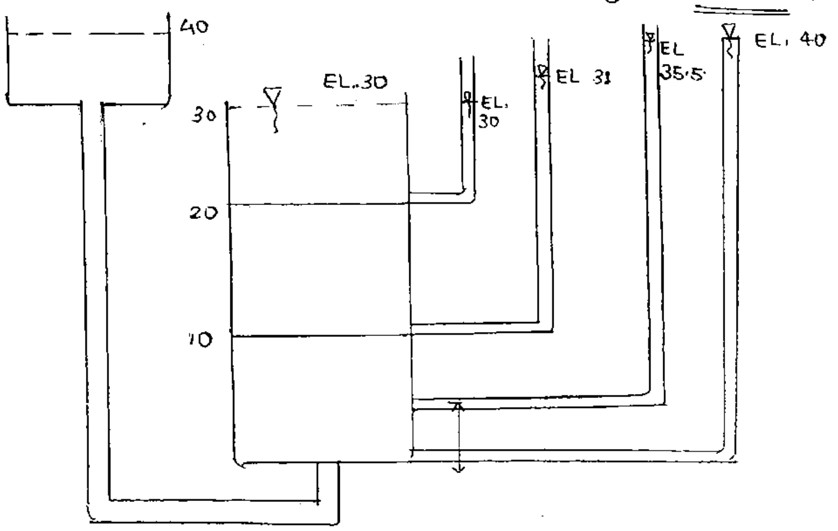
$$k_y = 4.4 \times 10^{-4} \text{ cm/s}$$

b) $$Q_x = k_x \cdot \frac{h_x}{Z_x} \cdot A_x = Q$$

$$= 4 \times 10^{-3} \times \frac{1}{10} \times 10 = 4 \times 10^{-3} \text{ cm}^3/\text{s}$$

$$= 4 \times 10^{-3} \times 60 \times 60 = \underline{14.4 \text{ cm}^3/\text{hr}}$$

c)



Pressure head = height of water column in piezometer.

H

3rd Sept,
WEDNESDAY

7. EFFECTIVE STRESS

→ Total Stress (σ)

It is the stress due to total load.

* Neutral Stress or Pore water Pressure (u)

It is the pressure in the water.

* Effective Stress (σ')

It is the total stress minus neutral stress.

It's also called intergranular pressure. It is the stress which controls the behaviour of soil, shear strength of soil and volume change of soil.

$$\boxed{\sigma = u + \sigma'}$$

$$\sigma' = \sigma - u$$

$$\sigma = \frac{W}{A}$$

where W → total load (external load + self wt. of soil)

A → total c/s area of soil

$$u = \gamma_w h$$

where h → pressure head = depth of water in piezometer

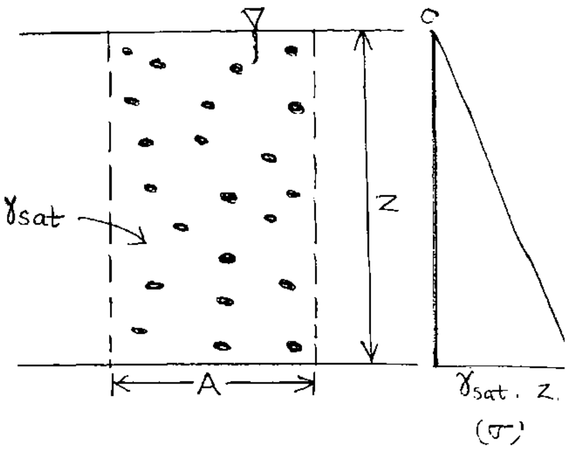
∴ σ & u can be measured, but σ' can't be measure

However it can be computed using σ & u values.

$$\sigma = \frac{W}{A}$$

$$= \frac{A \cdot z \cdot \gamma_{sat}}{A}$$

$$\therefore \boxed{\sigma = z \cdot \gamma_{sat}}$$



$$u = \gamma_w h$$

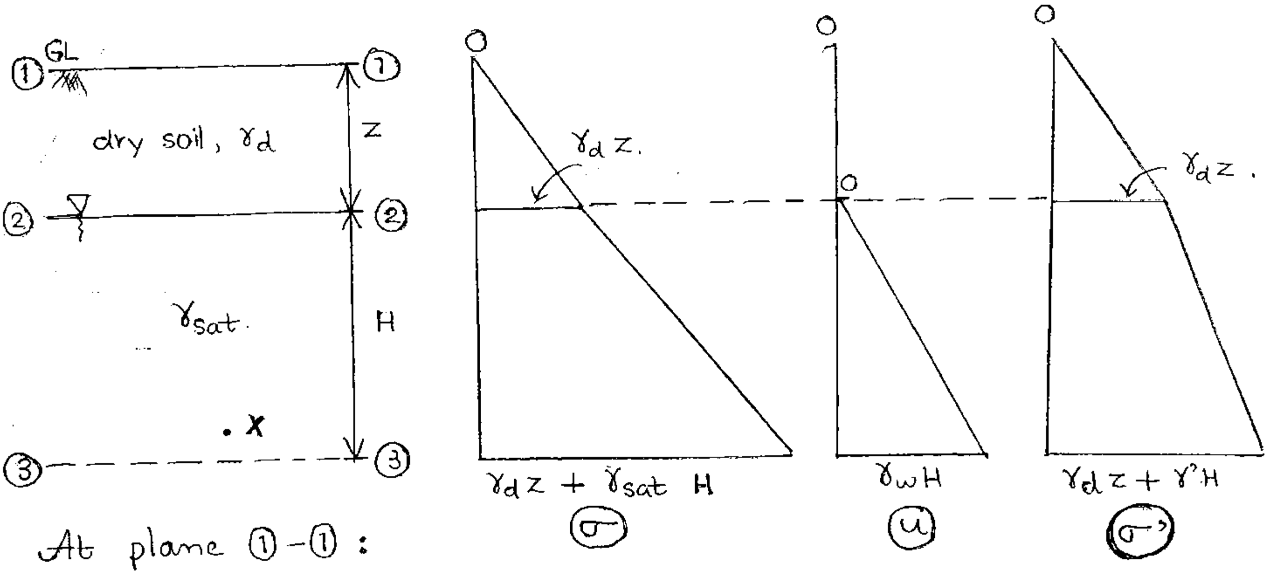
$$\therefore \boxed{u = \gamma_w \cdot z}$$

$$\sigma' = \sigma - u$$

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

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

$$\therefore \boxed{\sigma' = \gamma' z}$$



At plane ①-① :

$\sigma = 0$; $u = 0$; $\sigma' = 0$

At plane ②-② :

$\sigma = \gamma_d \cdot z$; $u = 0$; $\sigma' = \gamma_d z$.

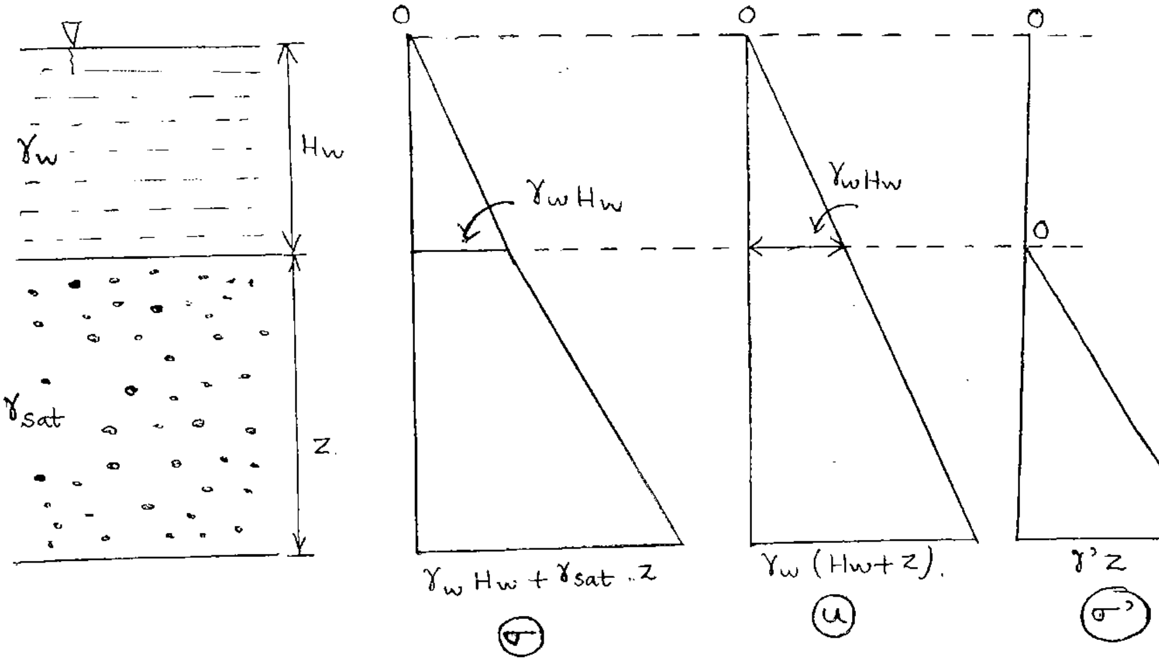
At plane ③-③ :

$\sigma = \gamma_d \cdot z + \gamma_{sat} \cdot H$

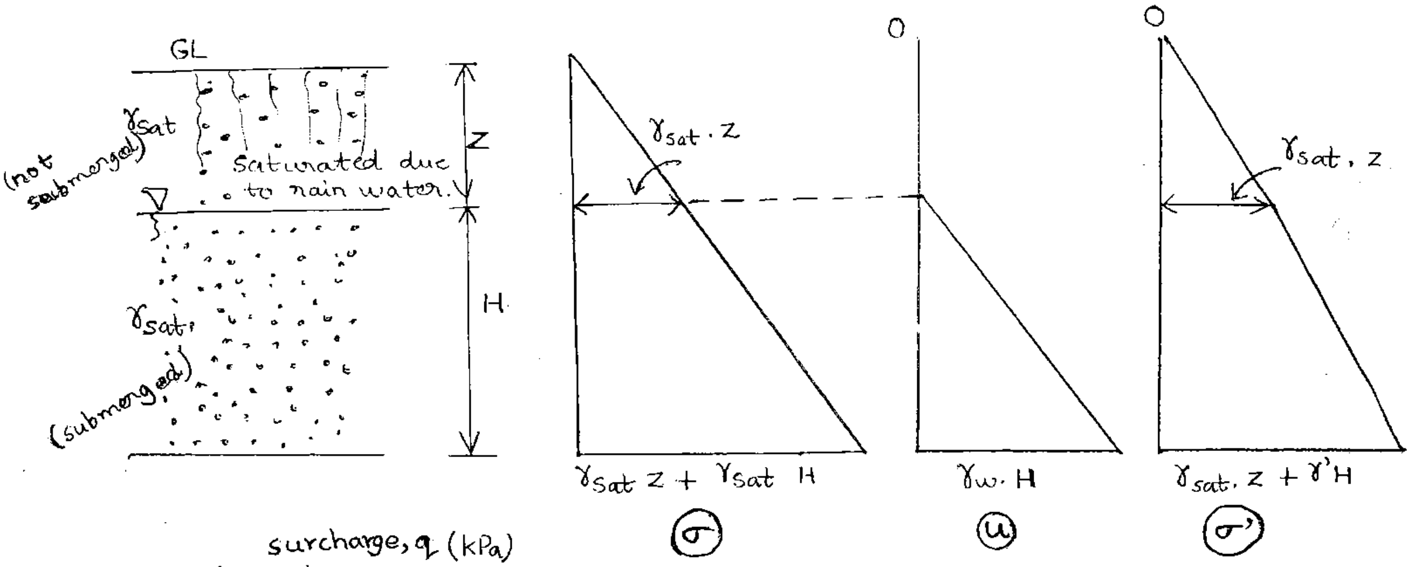
$u = \gamma_w \cdot H$

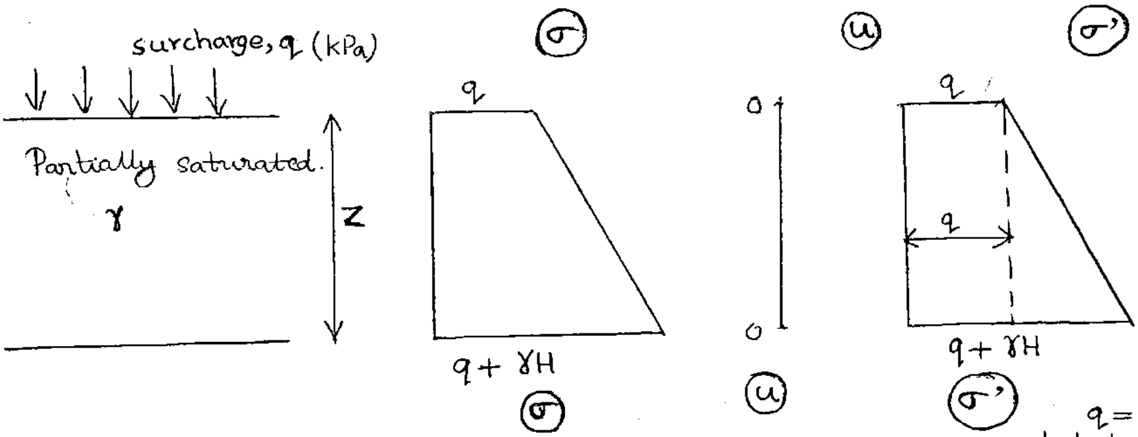
$\sigma' = \gamma_d z + \gamma' H$

- If WT raises, then σ & u increase, but σ' decreases.
- If WT falls, then σ & u decrease but σ' increases.

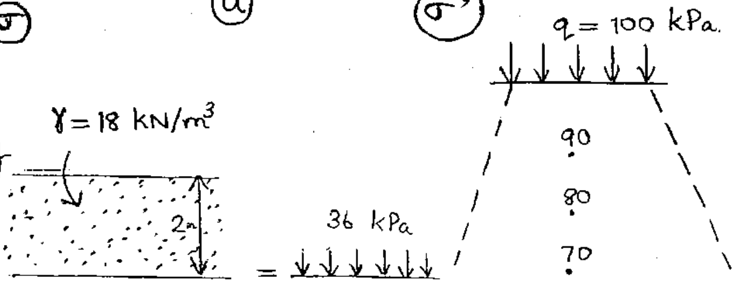


⊙ Effective stress below the bed level is totally independent of depth of water (H_w) above bed level.





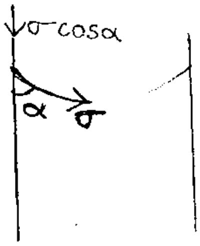
Above stress distribution diagrams are possible only if surcharge is infinite

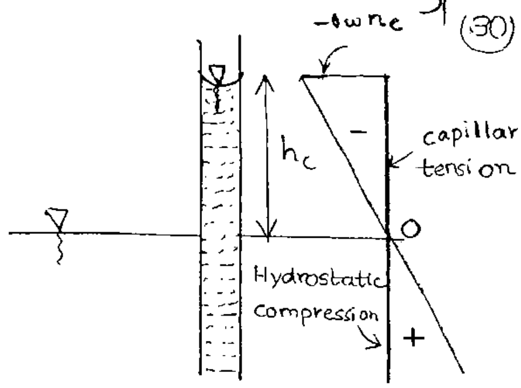


→ Capillarity

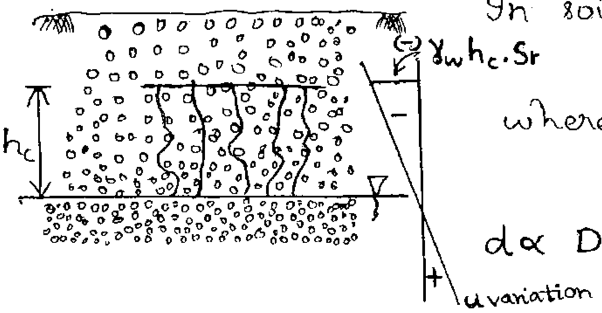
Due to capillarity, the capillary water is under tension whereas the wall of capillary tube is under compression.

'$\sigma \cos\alpha$' causing compression on the walls.





th Sept, THURSDAY



In soils, capillarity rise, $h_c \approx \frac{0.3}{d}$

where d — diameter of void (cm)

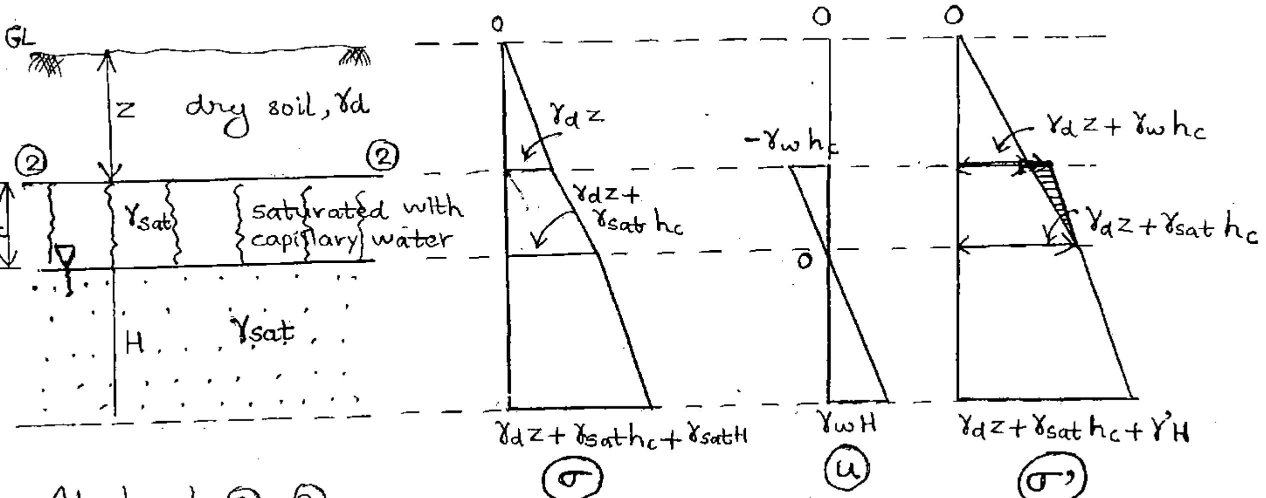
h_c — capillary height (cm)

$d \propto D_{10} \Rightarrow h_c \propto \frac{1}{D_{10}}$

- For gravel & coarse sand, h_c is negligible.
- h_c is highest for clay.

$$d \propto e D_{10} \Rightarrow h_c \propto \frac{1}{e D_{10}}$$

For clay, effect of size of particle on capillary height is more than that of void ratio.



At level ②-②:

a) Just above

$\sigma = \gamma_d z$

$u = 0$

$\sigma' = \gamma_d z - 0 = \gamma_d z$

b) Just below

$\sigma = \gamma_d z$

$u = -\gamma_w h_c$

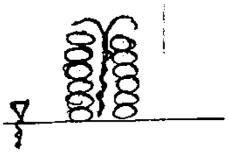
$\sigma' = \gamma_d z - (-\gamma_w h_c) = \underline{\underline{\gamma_d z + \gamma_w h_c}}$

$- \sigma' = \sigma - u.$

$\sigma' = \sigma$ if $u = 0$ $\rightarrow$ dry & partially saturated soil

$\sigma' < \sigma$ if u is +ve. $\rightarrow$ below WT

$\sigma' > \sigma$ if u is −ve $\rightarrow$ in capillary zone



Soil molecules act like walls of the capillary tube and they are under compression. ∴ σ' incre in capillary zone.

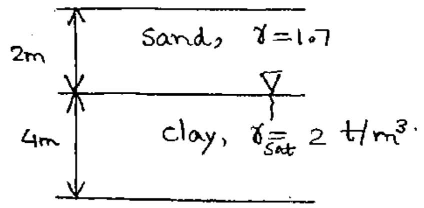
Frost heave (night time) & Frost boil (day time) are disadvantages of capillary actn

But capillary increases the shear strength of soil. ($s = c + \sigma' \tan\phi$)

2. $\sigma' = \gamma' z$

$100 = \gamma' z.$

Taking $\gamma' = 10$, $z = \underline{10\ m}$



3. At centre of clay,

$\sigma = 2\gamma + 2\gamma_{sat} = 7.4\ t/m^2$ 8.2

$u = 2\gamma_w = 2\ t/m^2$

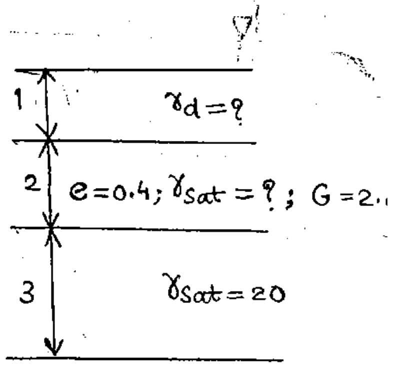
$\sigma' = \sigma - u = 7.4 - 2 = \underline{5.4}\ t/m^2$

OR

$\sigma' = 2\gamma + 2\gamma'$

$= 2 \times 1.7 + 2(2-1) = \underline{5.4}\ t/m^2$

4. $\sigma' = 3 \times 18 + 7 \times 10 = \underline{124}\ t/m^2$



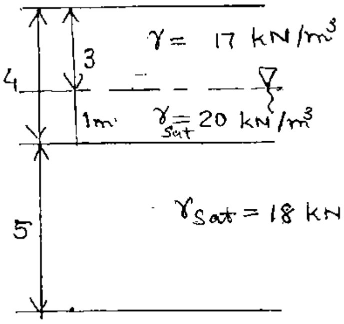
5. $\gamma_d = \dfrac{G\gamma_w}{1+e} = 18.93\ kN/m^3$

$\gamma_{sat} = \gamma_w \left(\dfrac{G+e}{1+e}\right) = 21.78\ kN/m^3$

$\sigma' = 1 \times 18.93 + 2(21.78 - 10) + 3(20-10) = \underline{72.49}\ kN/m^2$

P-35

Q. 7 b) Calculate σ' at a depth of 2.4 m below GL for the above capillarity case.



6. $\sigma'_{(9m)} = 17 \times 3 + (20 - 9.81) \times 1 + (18 - 9.81)\, 5 = \underline{\underline{102.14}}\ kN/m^2$

7. a) After capillary rise,

At 9 m depth,

$\sigma = 2 \times 17 + 20 \times 2 + 5 \times 18 = 164$

$u = 6\gamma_w = 58.86$

$\sigma' = \sigma - u = 105.14\ kPa$

$\Delta\sigma' = 105.14 - 102.14 = \underline{\underline{3\ kPa}}$

b) At 2.4 m depth,

$\sigma = 2\gamma_d + 0.4\gamma_{sat}$

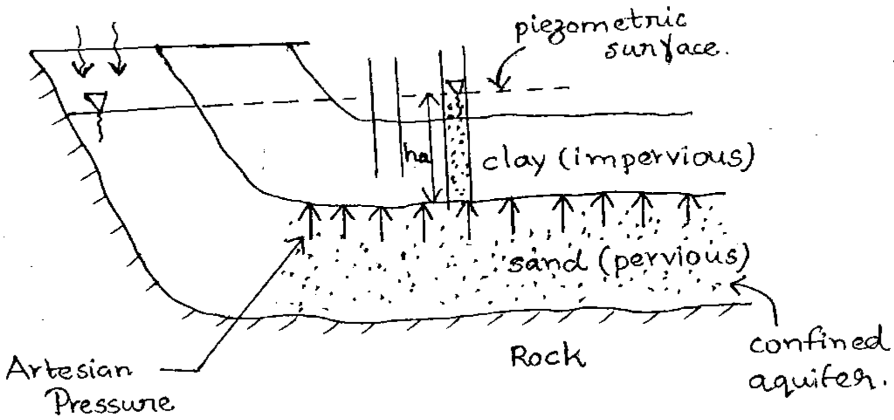
$= 2\gamma_d + 0.4 \times 20 = \underline{\underline{42\ kPa}}$

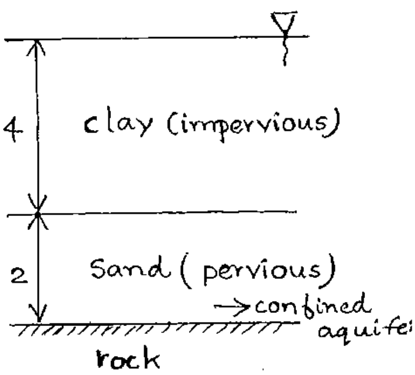
$\sigma' = \sigma - (-u) = 42 + (3 - 2.4)\, 9.81$

$= \underline{\underline{47.886}}\ kPa$

Sept, URDAY

8.





ha → artesian pressure head.

$\boxed{\text{Artesian pressure} = \gamma_w h_a}$

Springs are developed only when piezometric surface is above GL.

8\. a) Effective stress at a depth of 6m $= \gamma'_{clay} \times 4 + \gamma'_{sand} \times 2 - \gamma_w h_a$

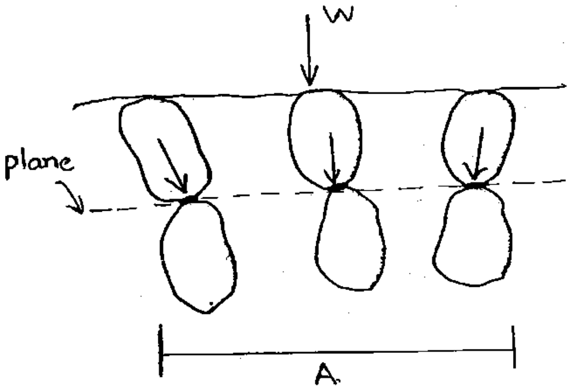
$$= (19.5-10)4 + (18.5-10)2 - 10\times 2$$

$$= \underline{\underline{35 \text{ kPa}}}$$

b). When $h_a = 1$,

$$\sigma' = 55 - 10\times 1 = 45 \text{ kPa}.$$

$$\Delta\sigma' = \underline{\underline{\cancel{35}\ 10 \text{ kPa}}}$$



$A_c \rightarrow$ area of contact.

$A_w \rightarrow$ area of water

$A \rightarrow$ total area of soil.

A_c is very small.

$$A = A_w + A_c \approx A_w.$$

$$W = uA_w + \Sigma N_v.$$

Dividing by A,

$$\frac{W}{A} = u\frac{A_w}{A} + \frac{\Sigma N_v}{A}$$

$$\Rightarrow \sigma = u + \frac{\Sigma N_v}{A}$$

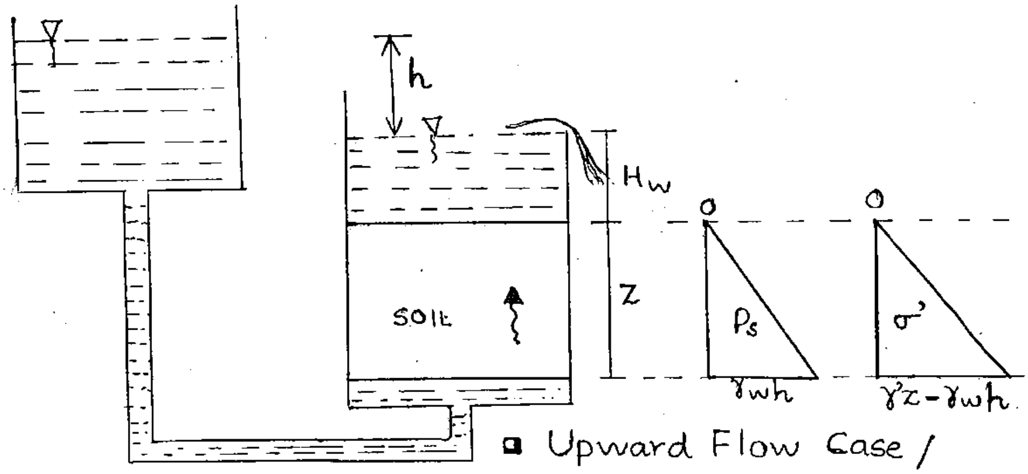
Comparing above equation with, $\sigma = u + \sigma'$

$$\therefore \sigma' = \frac{\Sigma N_v}{A}.$$

It is equal to the total vertical reaction force transmit at the points of contact of soil grains divided by the total area, including that occupied by water.

It is much smaller than actual contact stress. $\left(\frac{\Sigma N_v}{A_c}\right)$.

8. SEEPAGE PRESSURE & CRITICAL HYDRAULIC GRADIENT



- Upward Flow Case / Upward Seepage

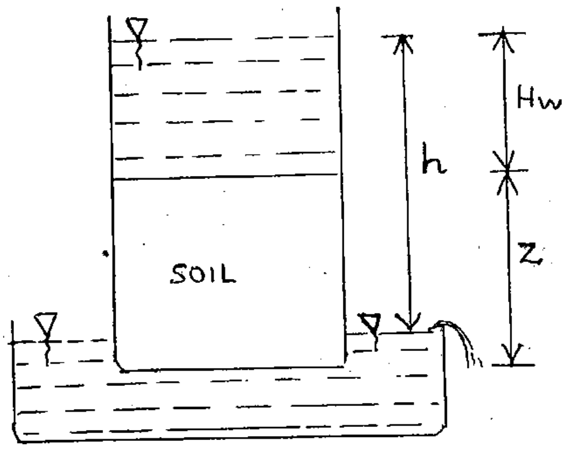
At bottom of soil:

$$\sigma = \gamma_w H_w + \gamma_{sat} \cdot z$$

$$u = \gamma_w (z + H_w + h)$$

$$\sigma' = \sigma - u$$

$$= \gamma' z - \gamma_w h$$



At bottom of soil:

$$\sigma = \gamma_w H_w + \gamma_{sat} z$$

$$u = \gamma_w (z + H_w - h)$$

$$\sigma' = \sigma - u$$

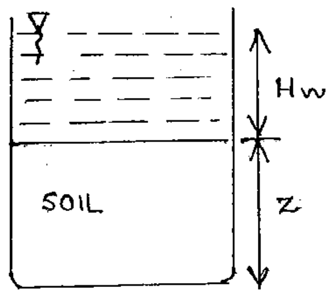
$$= \gamma' z + \gamma_w h$$

- Downward Seepage.

$$\sigma = \gamma_w H_w + \gamma_{sat} \cdot z$$

$$u = \gamma_w (H_w + z)$$

$$\sigma' = \sigma - u = \underline{\underline{\gamma' z}}$$



$\therefore$ when there is seepage, $\sigma' = \gamma' z \pm \gamma_w h$

use <u>-ve sign</u> for <u>upward</u> seepage
<u>+ve</u> sign for <u>downward</u> seepage

$$\boxed{\text{Seepage pressure, } P_s = \gamma_w h.}$$

The pressure caused by the seepage water on the soil partic is called seepage pressure.

The seepage pressure always acts in the direction of flow.

Upward flow: $\gamma' z' \downarrow$ $\quad \gamma_w H \uparrow$

Downward flow: $\gamma' z \downarrow$ $\quad \gamma_w H \downarrow$

Hydraulic Gradient, $i = \frac{h}{z}$.

$$\therefore P_s = \gamma_w h$$
$$= \gamma_w i z$$

Seepage force, $P_s = P_s \cdot A.$
$$= \gamma_w \cdot i \cdot z \cdot A.$$

$A \rightarrow$ area at bottom of soil.

$\therefore$ Seepage force per unit volume of soil $= \underline{\underline{\gamma_w i}}$

$\rightarrow$ Critical Hydraulic Gradient; i_c.

It is the hydraulic gradient at critical condition. $(\sigma' = 0)$.

In an upward seepage, $\sigma' = \gamma' z - \gamma_w h$.

At critical condition $(\sigma' = 0)$; $\gamma_w h = \gamma' z$

$$\Rightarrow \frac{h}{z} = \frac{\gamma'}{\gamma_w}$$

$$\therefore \boxed{i_c = \frac{\gamma'}{\gamma_w}}$$

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

For soils, $i_c \approx 1$. ($G = 2.6 - 2.85$ & $e = 0.6 - 0.85$)

→ Quick Sand & Quick Condition or Boiling Condition.

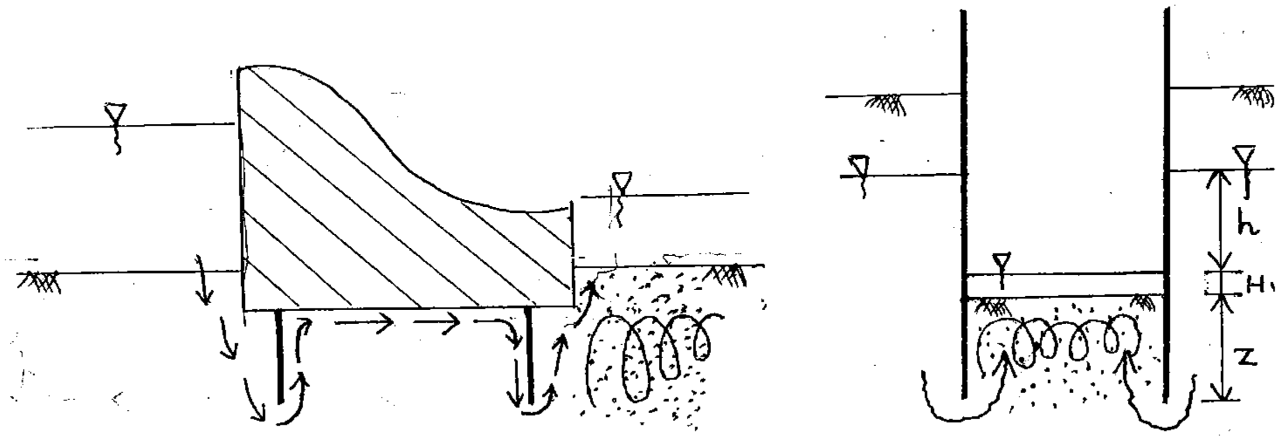
Shear strength, $s = c' + \sigma' \tan\phi'$

For cohesionless soils, $s = \sigma' \tan\phi'$

In an upward seepage, $\sigma' = \gamma' z - \gamma_w h$.

At critical condition ($\sigma' = 0$), shear strength of cohesionless soil becomes zero and the soil behaves like a boiling liquid. This phenomenon is called Quick Condition. It occurs only in cohesionless soils.

Quick condition is generally observed in fine sand and silts. In the case of gravel and coarse sand, though they are cohesionless, quick sand condition is not common, sin these are highly permeable.



Practically, quick sand condition occurs at the bottom or d/s side of hydraulic structures. This is also experienced during construction activities in regions where WT is closer to GL.

* To prevent Quick Condition

- Provide more depth of sheet piles and reduce the hydraulic gradient.
- Keep some depths of water (H_w) in the trench without completely dewatering.
- Lower down the surrounding WT.
+ Apply some surcharge load intensity (q) on top of soil, at D/S of hydraulic structures.

Let i be actual hydraulic gradient $(= \frac{h}{z})$

i_c be critical hydraulic gradient of soil $(= \frac{G-1}{1+e})$.

If $i \geq i_c$, quick condition occurs.

To avoid quick condition, i must be kept less than i_c.

$\therefore$ FOS against quick condition, $\boxed{F = \frac{i_c}{i}}$

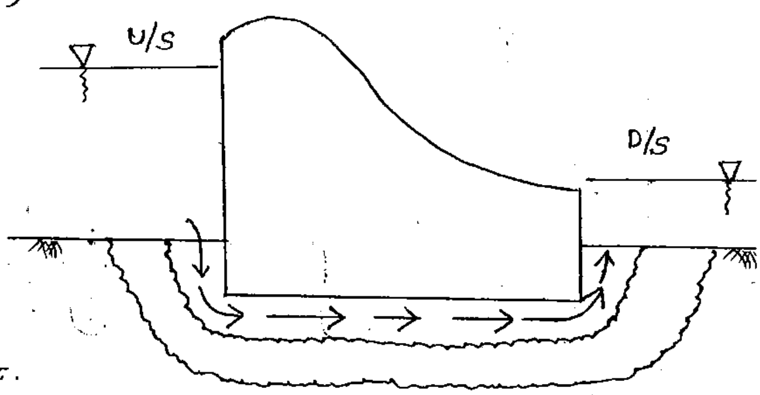
* The minimum head required to cause quick condition,

$$h = i_c \cdot z.$$

9th Sept,
TUESDAY

→ Piping: (undermining)

- gradual erosion of soil particles.
- It occurs when $\sigma' = 0$ in case of cohesionless soils like fine sand and silt.



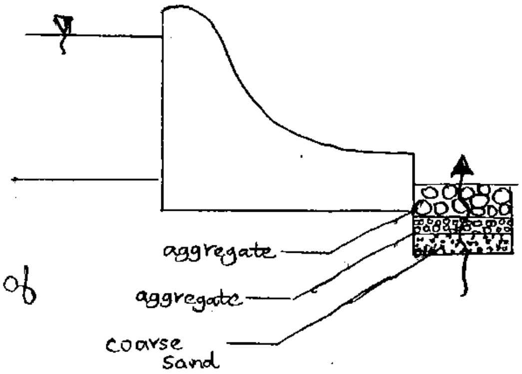
Let i_{exit} be hydraulic gradient at exit point.

$$\text{FOS against piping} = \frac{i_c}{i_{exit}}$$

* To prevent Piping:

– provide sheet piles in the foundation to reduce the hydraulic gradient.

– provide inverted filter on D/S.

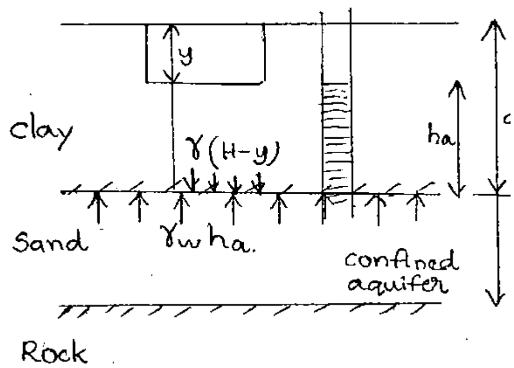


■ Terzaghi's criteria for design of filter:

(i) $\frac{(D_{15})_{filter}}{(D_{15})_{base}} \geq 5$; to allow escape of water

(ii) $\frac{(D_{15})_{filter}}{(D_{85})_{base}} \leq 5$; to prevent escape of soil particles.

Q-38



1. At critical condition,

downward pr. = uplift pressure

$\gamma(H-y) = \gamma_w h_a$

$20(9-y) = 10 \times 6.$

$y = 9-3 = \underline{\underline{6 m}}$

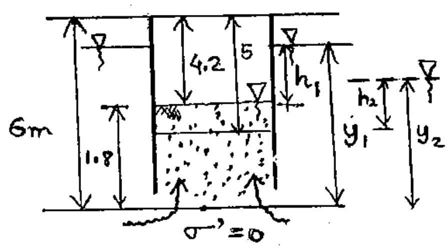
2. If $y = 7 m$,

$\gamma(H-y) = \gamma_w h_a$

$20(9-7) = 10 \; h_a$

$\therefore h_a = 4 m.$

∴ Water is to be lowered by $6-4 = \underline{\underline{2 m}}$



3. $\sigma' = \gamma' z - \gamma_w h_1$

$0 = 11 \times 1.8 - 10 \times h_1$

$\therefore h_1 = 1.98 m$

$y_1 = 1.8 + 1.98 = \underline{\underline{3.78 m}}$

When depth of excavation increased to 5m,

$$0 = 11 \times 1 - 10 h_2$$

$$h_2 = 1.1 \text{ m}$$

$$y_2 = 1 + 1.1 = 2.1 \text{ m}$$

$$y_1 - y_2 = 3.78 - 2.10 = \underline{\underline{1.68 \text{ m}}}$$

$$\Delta z . \gamma_{sat} = \Delta h . \gamma_w$$

$$(1.8-1)(11+10) = \Delta y . 10$$

$$0.8 \times 2.1 = \Delta y$$

$$\therefore \underline{\underline{\Delta y}} = 1.68 \text{ m}$$

04. $e = 0.8$, $G = 2.65$, $z = 10$ cm.

$$I_c = \frac{G-1}{1+e} = \frac{1.65}{1.8} = 0.916$$

$$h = I_c . z = \underline{\underline{9.16 \text{ cm}}}$$

05. $Q = kiA$.

$$0.04 = 2 \times 10^{-3} \times i \times 45$$

$$i = 0.44.$$

$$h = iz = 0.44 \times 10 = \underline{\underline{4.4 \text{ cm}}}$$

06. $\sigma' = \gamma' z - \gamma_w h$.

$$= (1.93 - 1) \times 10 - 1(4.4)$$

$$= \underline{\underline{4.86}} \text{ g/cm}^2$$

9th Sept,
TUESDAY

9. SEEPAGE ANALYSIS

* Flow Line or Stream line.

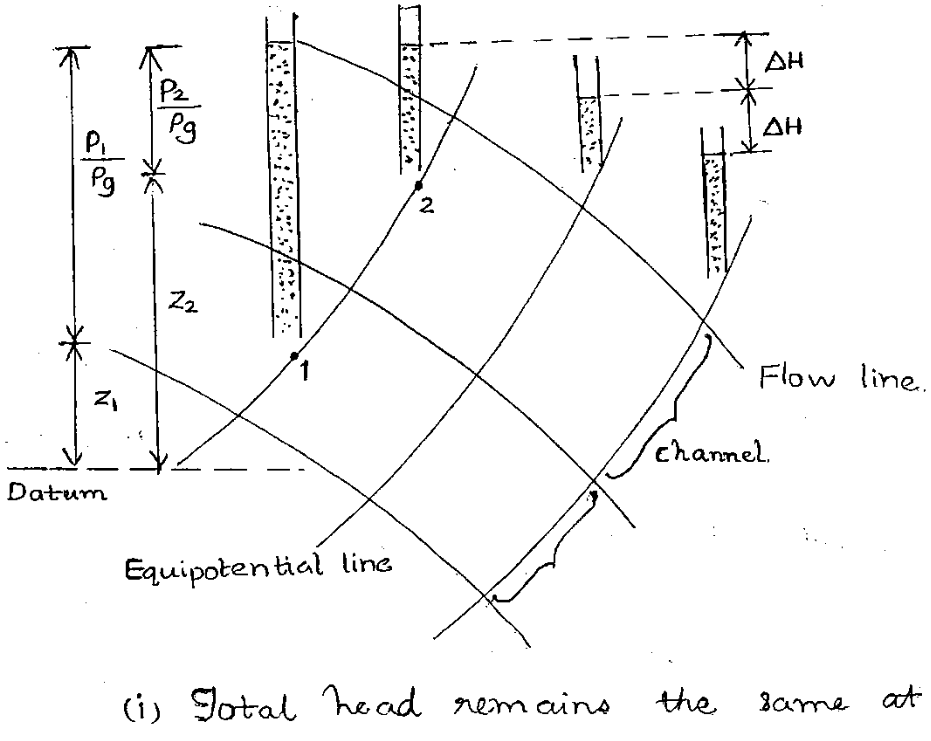
Line which shows the direction of seepage on flow.

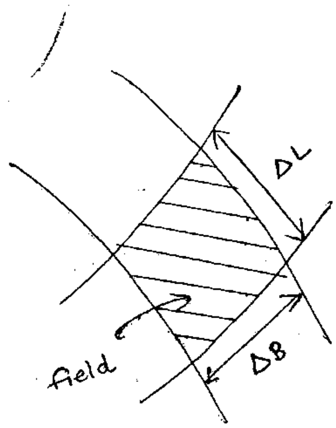
* Equipotential Line

Total Head or potential remains the same at all points in an equipotential line.

* Flow net

Network of equipotential lines and flow line.





(i) Total head remains the same at all points in an equipotential line.

(ii) Head loss remains the same b/w two adjacent equipotential ~~flow~~ line. (ΔH is same)

(iii) ΔQ remains the same for channel.

(iv) For every field, $\frac{\Delta L}{\Delta B}$ ratio must be same.

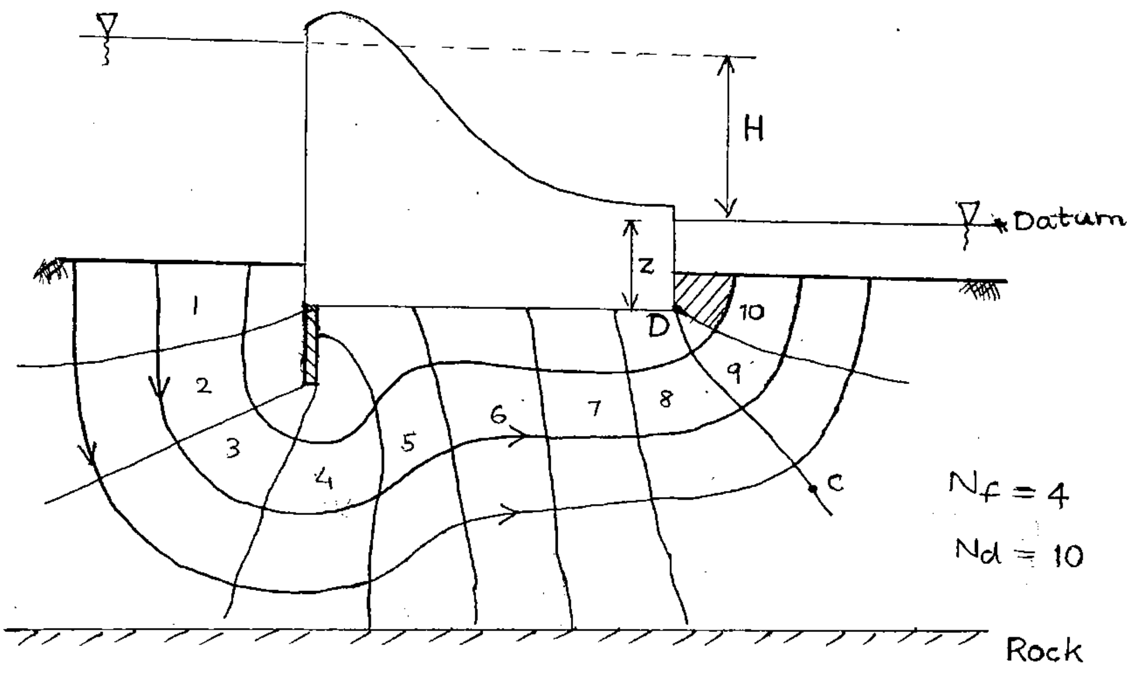
* Applications of Flow net.

(i) To find seepage loss rate.

(ii) To find seepage pressure.

(iii) To find uplift pressure.

(iv) To find exit gradient



$N_f = 4$

$N_d = 10$

H = seepage head (or) head causing flow. (or) total head los

(i) Seepage Loss Rate, Q

important

$$\boxed{\begin{aligned} Q &= kH\,\frac{N_f}{N_d}\left(\frac{\Delta B}{\Delta L}\right) \\ &= kH\cdot\frac{N_f}{N_d} \end{aligned}}$$

; for rectangular fields

; for square fields.

$N_f \rightarrow$ No: of flow channels.

$N_d \rightarrow$ No: of potential drops.

$\frac{N_f}{N_d}$ = shape factor of flow net (a constant)

(ii) Seepage Pressure, P_s

$$\boxed{P_s = \gamma_w h.}$$

$h \rightarrow$ balance seepage head at point c.

$$h = \text{total seepage head} - \text{head loss upto point c.}$$
$$= H - h_f$$

$h_f = n\,\Delta H$; $n \rightarrow$ no: of potential drops upto c.

$\Delta H \rightarrow$ head loss b/w two adjacent equipotential lines.

$$\Delta H = \frac{H}{N_d} \Rightarrow h = H - \frac{nH}{N_d}$$

(iii) Uplift Pressure, P_u

$$\boxed{P_u = \gamma_w h_w}$$

$h_w \rightarrow$ pressure head at point D.

Total head = Pressure head + Elevation head.

$$h = h_w + z$$

$$h_w = h - (-z)$$

$$h_w = h + z$$

(iv) Exit Gradient, i_{exit}

$$\boxed{i_{exit} = \frac{\Delta H}{\Delta L}}$$

$\Delta L \rightarrow$ length of field at exit. (hatched field)

Q. A flow net is shown in the fig. If coefficient of permeability of soil is 2×10^{-3} cm/s, determine the seepage loss rate in m³/day per m length of the weir.

$N_f = 4$ $H = 10.5 - 0.5 = 10$ m, $z = 0.5 + 0.8 = 1.3$ m

$N_d = 14$ $K = 2 \times 10^{-3}$ cm/s, $\Delta L = 1.2$ m.

Determine the seepage pressure and uplift pressure at point D shown in the fig, take depth of foundation as 0.8 m. Also determine the exit gradient if length of the field at the exit point is 1.2 m.

Seepage loss rate, $Q = KH \frac{N_f}{N_d}$

$$= 2 \times 10^{-3} \times 10^{-2} \times 10 \times \frac{4}{14} \times 60 \times 60 \times 24.$$

$K = \underline{\underline{1.728}}$ m/day & $Q = 4.937$ m³/day.m

Seepage pressure, $P_s = \gamma_w h.$

$$= \gamma_w \left(H - \frac{nH}{N_d}\right)$$

$$= 10\left(10 - \frac{9 \times 10}{14}\right) = \underline{\underline{35.714}} \text{ kPa}$$

Uplift pressure, $P_u = \gamma_w h_w$

$$= \gamma_w (h + z).$$

$$= 10\,(3.5714 + 1.3) = \underline{\underline{48.714}} \text{ kPa}$$

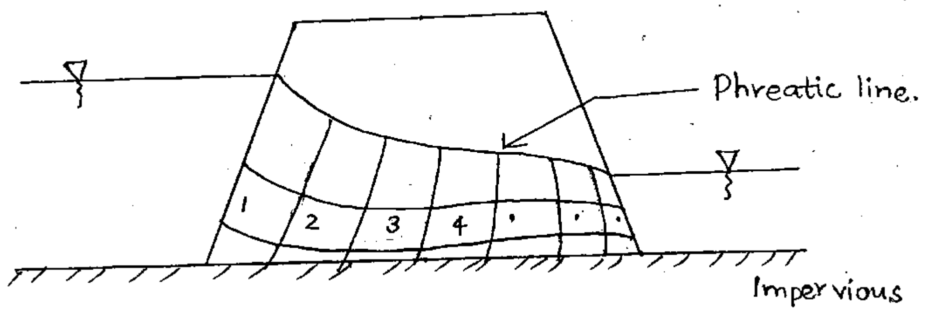
Exit gradient; $i_{exit} = \frac{\Delta H}{\Delta L} = \frac{10/14}{1.2} = 0.595$ m.

→ Earthen Embankment

$N_f = 3$

$N_d = 7$

$Q = KH \frac{N_f}{N_d}$.



* Phreatic Line:

- topmost flow line
- On the phreatic line, pressure head is zero.
- Parabolic shape

(37)

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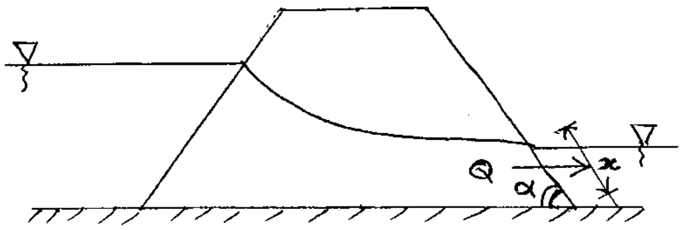
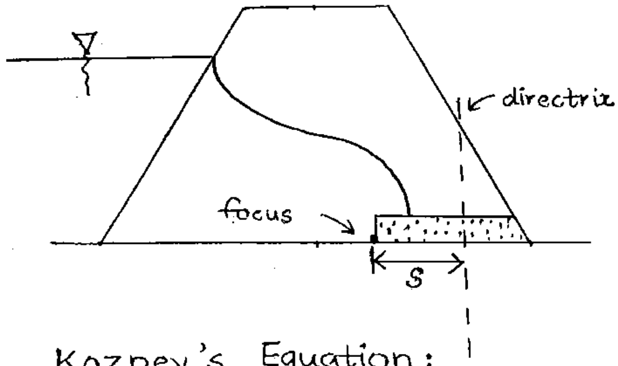
- If $\alpha < 30^\circ$,

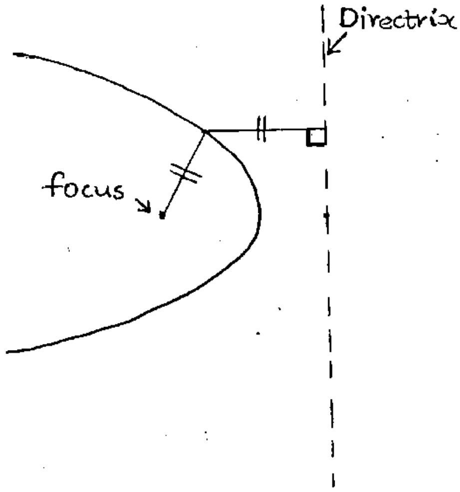
$$Q = k.x.\sin\alpha.\tan\alpha$$

If $30^\circ \leq \alpha \leq 60^\circ$,

$$Q = k.x.\sin^2\alpha$$





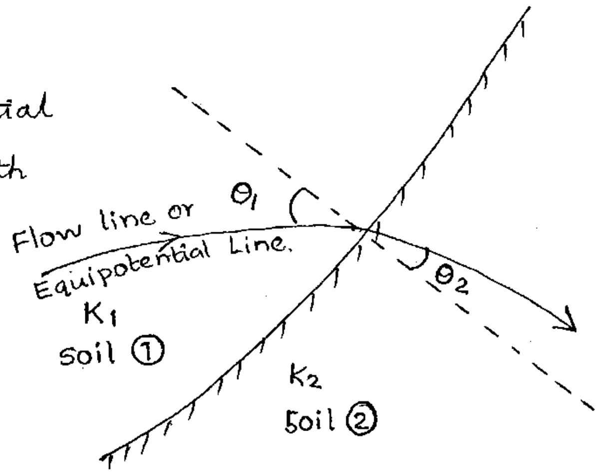


Kozney's Equation:

$$Q = K.S.$$

→ Anisotropic Soil:

Flow lines or equipotential lines are generally smooth lines. But whenever permeability changes, there will be deflection.



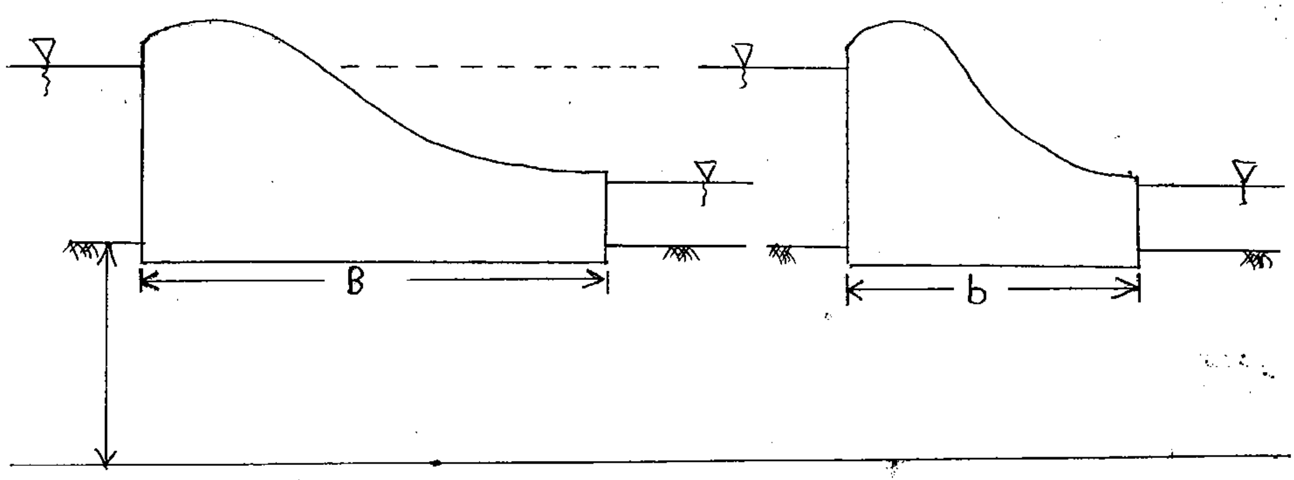
$$\boxed{\frac{\tan\theta_1}{\tan\theta_2} = \frac{k_1}{k_2}.}$$

In the case of anisotropic soils ($kx > ky$), the flow net is to be drawn to the transformed section which is obtained by reducing the horizontal dimensions and keeping vertical dimensions unchanged. The horizontal dimension is reduced by multiplying with a reduction coeffecient of

$\sqrt{\dfrac{k_y}{k_x}}$. The seepage loss rate is computed by taking average permeability (k') as follows:

$$Q = k' H \frac{N_f}{N_d}.$$

where $k' = \sqrt{k_x \cdot k_y}$



1. $b = B\sqrt{\dfrac{k_y}{k_x}} = 65.8\ \text{m}.$

Scale factor = 1:25

$$= \frac{65.8}{25} = \underline{\underline{2.63\ \text{m}}}$$

2. $\underset{(\text{cm/s})}{K} = 100\ \underset{(\text{cm})}{D_{10}^2} = 100 \times (0.01)^2 = 10^{-4}\ \text{cm/s} = 10^{-6}\ \text{m/s}$

$$Q = kH \cdot \frac{N_f}{N_d} = \underline{\underline{1.5 \times 10^{-4}}}\ \text{m}^3/\text{s} \text{ per metre length}$$

3. $k = 3.8 \times 10^{-6}$; $H = 6.3\ \text{m}$; $N_f = 3$; $N_d = 10$

$$Q = kH\frac{N_f}{N_d} = 7.18 \times 10^{-6}\ \text{m}^3/\text{s} \text{ per m length}$$

$$= 7.18 \times 10^{-6} \times 10^{6}\ \text{cm}^3/\text{s} \text{ per m length}$$

$$= \underline{\underline{7.18}}\ \text{cm}^3/\text{s}$$

4. $\Delta H = \frac{H}{N_d} = \frac{18}{9} = 2\,m$

$n = 3.$

$\therefore h_f = n\,\Delta H = 6\,m$

$h = H - h_f = 18 - 6 = \underline{\underline{12\,m}}$

11th Sept,
THURSDAY

10. STRESS DISTRIBUTION

→ Boussinesq's Theory

* Assumptions :

- Soil is homogenous.
- Isotropic soil.
- Semi infinite.
- Elastic medium.
- Point load.

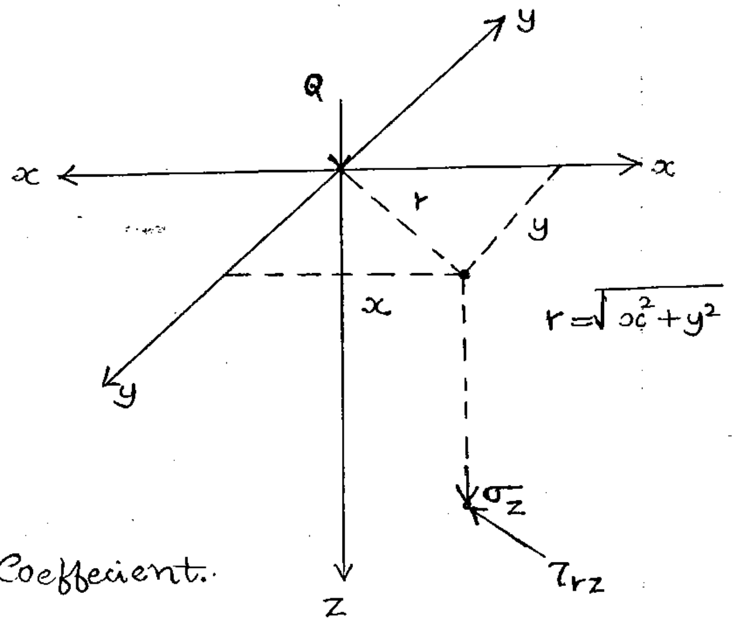
Homogenous means at different locations, soil has same elastic properties in same direction. (same E, μ)

Isotropic means at a single point, soil has same elastic properties in different directions.

Semi-infinite means material bounded by a horizontal plan and extending to infinite length in all directions to one side of horizontal plane.

Vertical stress,

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi} \left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$



$$\sigma_z = \frac{Q}{z^2} \cdot K_B.$$

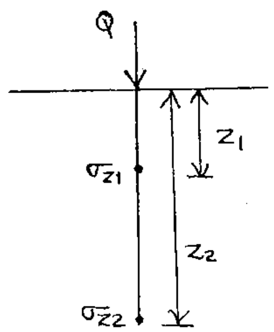
K_B → Boussinesq Influence Coeffecient.

- If r = 0 (vertically below the load).

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi}.$$

$$\Rightarrow \sigma_z \propto \frac{1}{z^2}$$

$$\frac{\sigma_{z1}}{\sigma_{z2}} = \left(\frac{z_2}{z_1}\right)^2$$



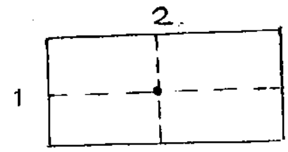
- Radial Shear Stress,

$$\tau_{rz} = \sigma_z \cdot \frac{r}{z}$$

Vertically below the load, $\tau_{rz} = 0$.

Q. A rectangular footing 1 m x 2m size has a load intensity of 10 t/m² on the ground surface. Determine the vertical stress at 3 m below ground level a) below CG of the footing b) below the corner of footing using Boussinesq's Theory.

a) below CG of footing,



$Q = 10 \times 1 \times 2 = 20$ t. (acting at CG)

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi}$$

$$= \frac{20}{3^2} \cdot \frac{3}{2\pi} = 1.06 \ t/m^2$$

b) below corner of footing,

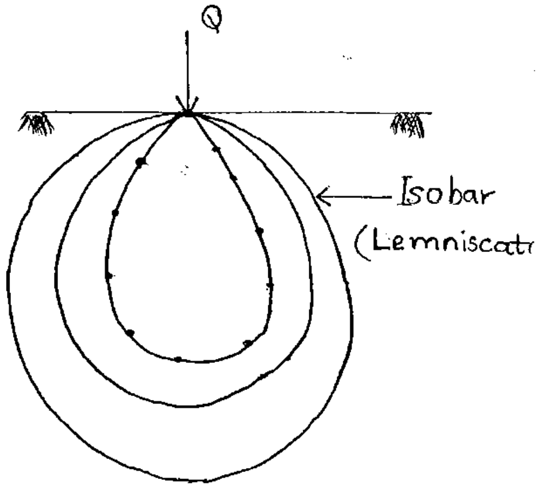
$$r = \sqrt{0.5^2 + 1^2} = 1.11 \ m.$$

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{3}{2\pi} \left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2} = \frac{20}{9^2} \cdot \frac{3}{2\pi} \left(\frac{1}{1+\left(\frac{1.118}{3}\right)^2}\right)^{5/2}$$

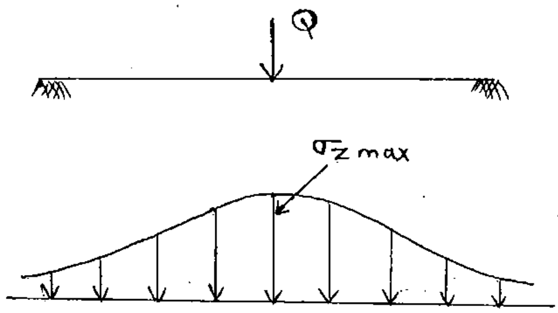
$$= \underline{\underline{0.7665 \ t/m^2}}$$

→ Isobar

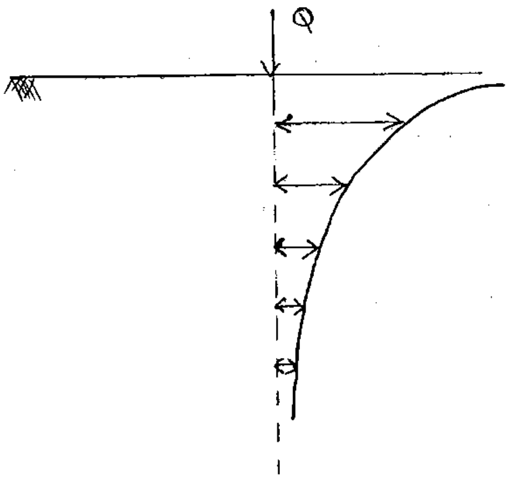
It is a curve or contour connecting all points below the ground surface of equal vertical stress.

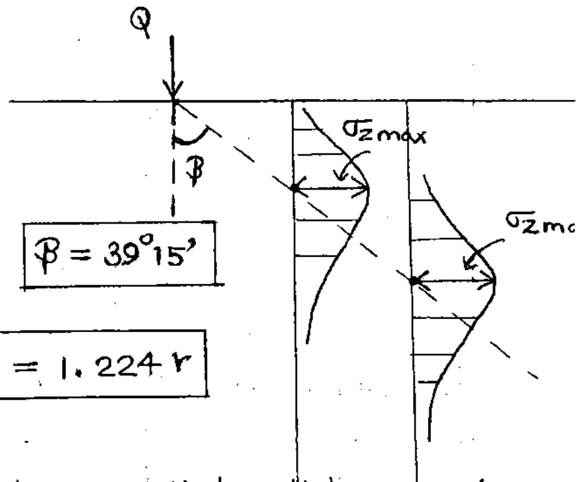


* σ_z variation on a Horizontal Plane.



* σ_z variation on a Vertical Plane Passing through Load

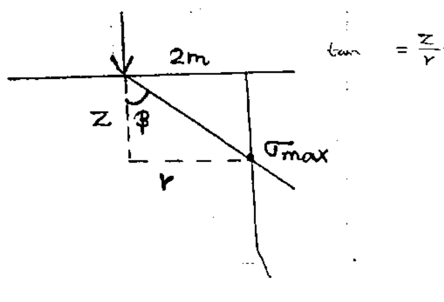




$\boxed{\beta = 39°15'}$

$\boxed{z = 1.224\, r}$

Q If a vertical plane is drawn at a radial distance of 2 m away from a vertical load at what depth max σ_z occurs.



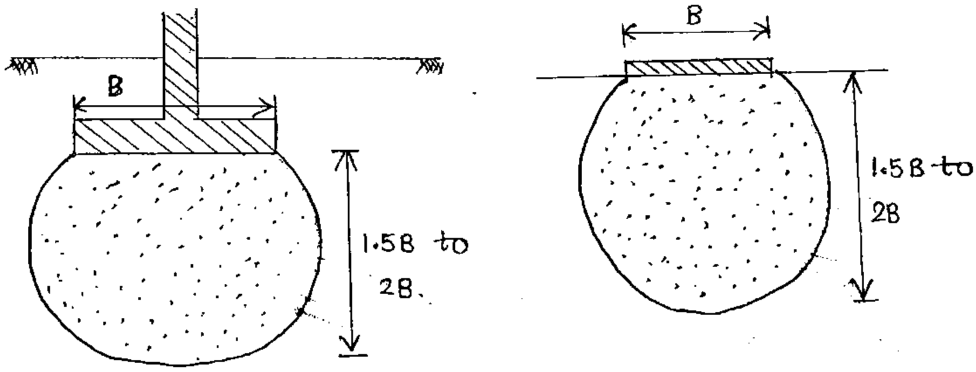
$$\tan\beta = \frac{r}{z}$$

$$z = \frac{r}{\tan\beta} = 1.224\, r$$

$$= 1.224 \times 2 = 2.447$$

→ Pressure Bulb.

It is the zone of the soil in which there is significant stress. Beyond the pressure bulb, stress in the soil is negligible. In the case of footings, the depth of the pressure bulb is taken as 1.5 B to 2B (as shown in the fig) below the footing

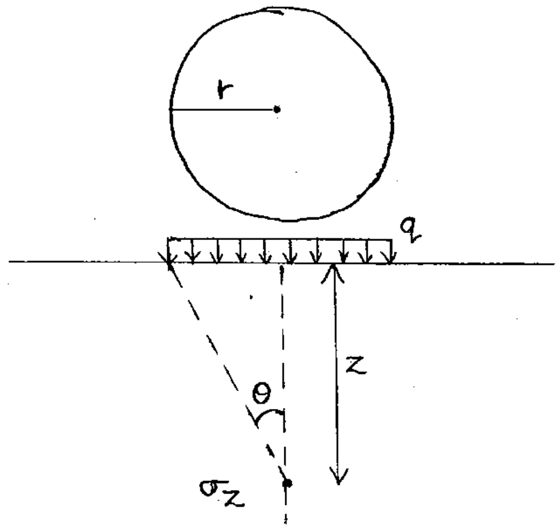


→ Circular Loaded Areas.

$$\sigma_z = q\left[1 - \left\{\frac{1}{1+\left(\frac{r}{z}\right)^2}\right\}^{3/2}\right]$$

OR

$$\sigma_z = q\left(1 - \cos^3\theta\right)$$

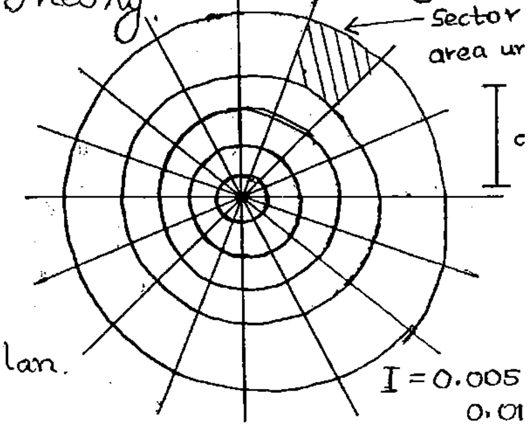


Sept, → Newmark's Influence Chart
DAY

- to find σ_z at any point under any shape of loading.
- prepared based on Boussinesq's Theory.
- Each sector causes equal stress at the centre of the chart.

$$\boxed{\sigma_z = I n q}$$

I → influence coeffecient of chart.
n → no. of sectors covered by footing plan.
q → load intensity of footing.



Q. In a Newmarks influence chart depth line is 5cm. If the stress is required at a depth of 10m, what scale is to be used to draw the fig on the tracing paper?

Scale : Depth line = Z.

$$5\text{ cm} = 10\text{ m}$$

$$1\text{ cm} = 2\text{ m}$$

$$1\text{ cm} = 200\text{ cm}$$

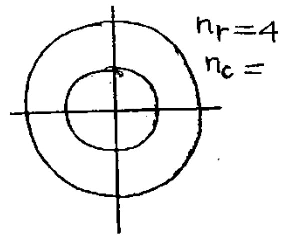
or $\underline{\underline{1 : 200}}$

$$\ast \quad I = \frac{1}{\text{Total no. of sectors of chart}}$$

Total no. of sectors of chart = No. of concentric circles × no. of radial lines.

If no. of circles = 10 & no. of radial lines = 20

$$I = \frac{1}{10 \times 20} = \underline{\underline{0.005}}$$



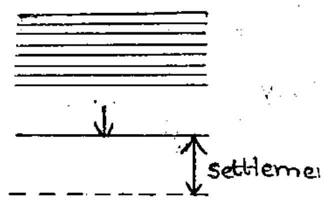
→ Westergaard's Method :

* Assumptions :-

(i) Point Load.

(ii) Soil consists of no. of thin layers.

(iii) Applicable for <u>stratified soils</u> (or) sedimentary soils. or <u>varved clay</u>



$$\boxed{\sigma_z = \frac{Q}{z^2} \cdot \frac{1}{\pi} \left[\frac{1}{1 + 2\left(\frac{r}{z}\right)^2} \right]^{3/2}}$$

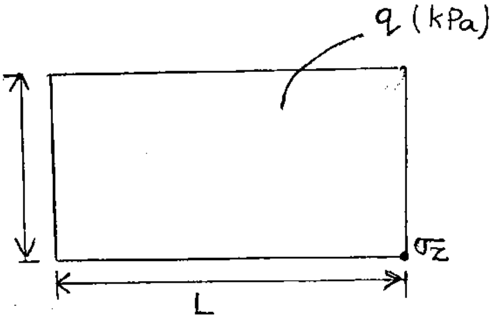
– For $\frac{r}{z} < 1.5$, Boussinesq's eqn gives higher stresses compar to Westergaard's eqn.

- For $\frac{r}{z} = 1.5$ both equations give the same stress value
- For $\frac{r}{z} > 1.5$, Westergaard's eqn gives slightly higher values compared to Boussinesq eqn.

→ Newmark's Method.

- to find σ_z at corner of rectangular loaded area

$$\boxed{\sigma_z = I q}$$

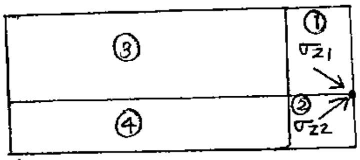


I → influence coeffecient which depends on m & n coeffecients.

$m = \frac{L}{z}$ & $n = \frac{B}{z}$.

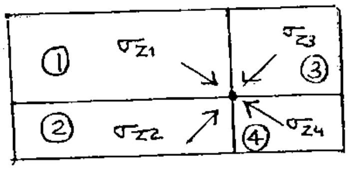
m & n values are available in the form of charts or tables

Pt. outside footing:



$\sigma_z = \sigma_{z3} + \sigma_{z4} - \sigma_{z1} - \sigma_{z2}$

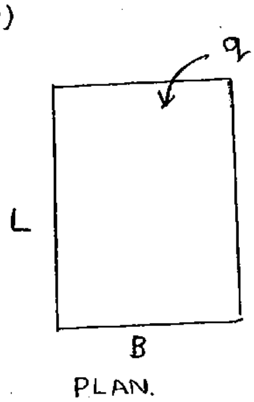
Pt. inside footing:

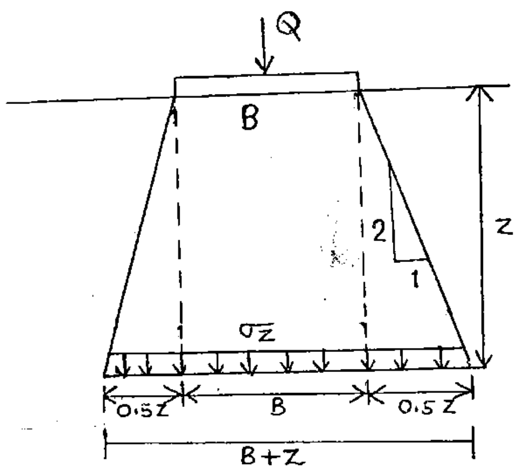


$\sigma_z = \sigma_{z1} + \sigma_{z2} + \sigma_{z3} + \sigma_{z4}$.

→ Approximate Method:

- Load dispersion angle is assumed to be as 2V to 1H or 1:2 load dispersion. (H:V)





$Q = LBq$.

$\sigma_z = \frac{Q}{(B+z)(L+z)}$; for rectangular footing

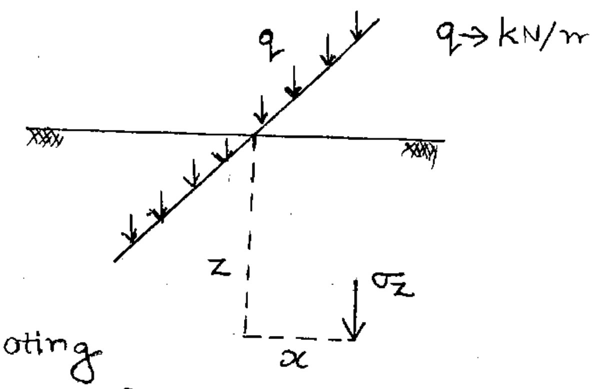
$\sigma_z = \frac{Q}{(B+z)^2}$; for square footing

$$\sigma_z = \frac{Q}{(B+z)\,1} \; ; \text{ for continuous footing}$$

$$Q = (B \times 1)\,q \; ; \; Q \to \text{load per unit length.}$$

→ Vertical Stress due to Line Load.

$$\sigma_z = \frac{q}{z}\,\frac{2}{\pi}\left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2$$

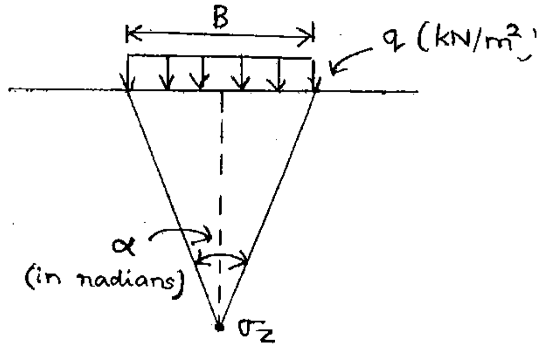


Eg :- Railway lines, sewer pipes etc.

→ Vertical Stress due to Strip footing
(Continuous footing)

$$\sigma_z = \frac{q}{\pi}\,(\alpha + \sin\alpha)$$

(below CG of loading)

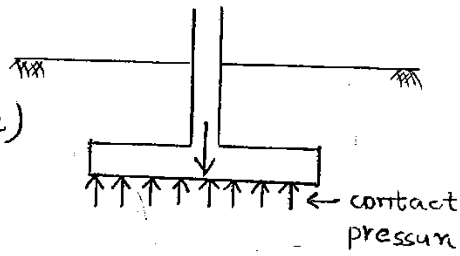


→ Contact Pressure

Variation depends upon :-

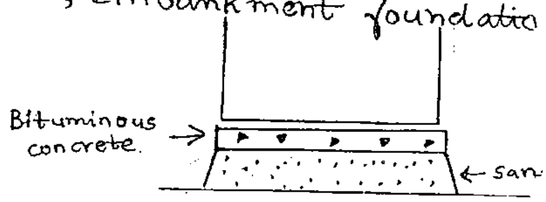
(i) Type of footing (rigid or flexible)

(ii) Type of soil.



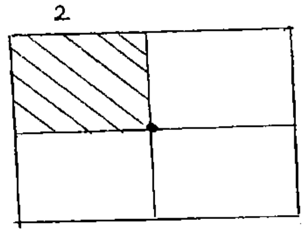
<u>Rigid Footing</u> (Eg: RCC footing)	<u>Flexible Footing</u>
(i) uniform settlement.	(i) Non uniform settlement.
(ii) non-uniform contact pressure	(ii) Uniform contact pressure.

Flexible footing :- oil tank foundation, embankment foundatio



2. For the hatched rectangle,

$l = 2m, \; b = 1m.$



$$\left.\begin{aligned} m &= \frac{l}{z} = 0.4 \\ n &= \frac{B}{z} = 0.2 \end{aligned}\right\} \quad I = 0.0328$$

$$\sigma_z = 4 \cdot Iq = 4 \times 0.0328 \times 8 = \underline{\underline{1.05}} \; t/m^2$$

To find σ_z below the corner,

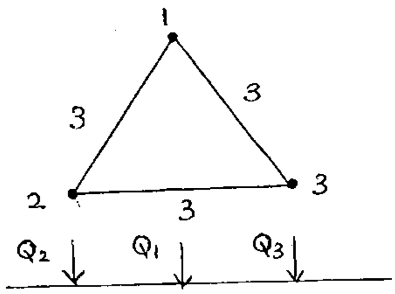
$$\left.\begin{aligned} m &= \frac{L}{z} = \frac{4}{5} = 0.8 \\ n &= \frac{2}{5} = 0.4 \end{aligned}\right\} \quad I = 0.0931.$$

$$\sigma_z = Iq = 0.0931 \times 8 = \underline{\underline{0.74}} \; t/m^2$$

3. σ_{z1} = stress under column ①. due to q_1

σ_{z2} = stress under column ① due to q_2

σ_{z3} = " due to q_3.



$$Q_1 = \frac{200}{3} = 66.67 = Q_2 = Q_3.$$

$z = 2\,m, \; r = 0.$ (for σ_{z1}).

$$\sigma_{z1} = \frac{66.67}{4} \cdot \frac{3}{2\pi} = 7.96 \; t/m^2$$

$$\sigma_{z2} = \frac{Q_2}{z^2} \cdot \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2}$$

$$= \frac{66.67}{4} \cdot \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{3}{2}\right)^2}\right)^{5/2}$$

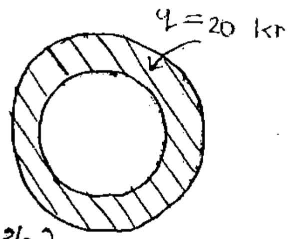
$$= \underline{\underline{0.418}} \; t/m^2 = \sigma_{z3}$$

$$\sigma_z = \sigma_{z1} + \sigma_{z2} + \sigma_{z3} = 7.96 + 0.418 \times 2$$

$$= \underline{\underline{8.796}} \; t/m^2$$

4.

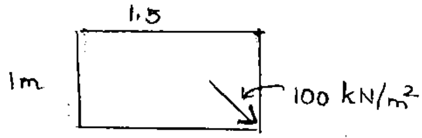
$$\sigma_z = q\left\{1-\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{3/2}\right\}$$

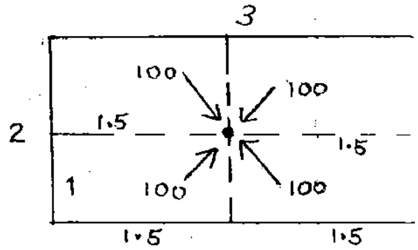


$$\sigma_z = 20\left\{1-\left(\frac{1}{\left(1+\frac{4}{10}\right)^2}\right)^{3/2}\right\} - \left[20\left\{1-\left(\frac{1}{1+\left(\frac{3}{10}\right)^2}\right)^{3/2}\right\}\right.$$

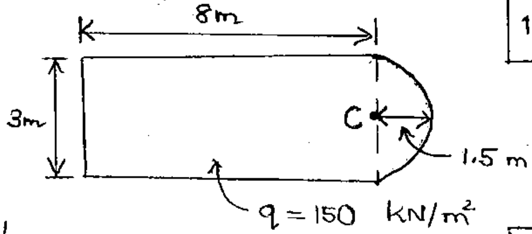
$$= \underline{\underline{1.56}} \text{ kN/m}^2$$

5. $\sigma_z = 4 \times 100 = \underline{\underline{400}} \text{ kN/m}^2$



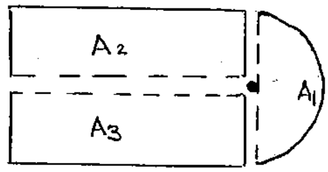


Q. A footing is shown in fig below. Determine the vertical stress at the point c shown in the fig. at a depth of 3 m. Use the following coefficients.



$m = 0.5 \quad n = 2.67 \quad I = 0.1365$

$m = 1 \quad n = 2.67 \quad I = 0.2028.$



Stress due to area A_1: (semicircular area)

$$\sigma_z = \frac{1}{2}\cdot q\left(1-\left\{\frac{1}{1+\left(\frac{r}{z}\right)^2}\right\}^{3/2}\right) = \frac{1}{2}\times 150\left(1-\left(\frac{1}{1+\left(\frac{1.5}{3}\right)^2}\right)^{3/2}\right)$$

$$= 21.33 \text{ kN/m}^2$$

Stress due to A_2:

$$\left.\begin{aligned} m &= \frac{L}{z} = \frac{8}{3} = 2.67 \\ n &= \frac{B}{z} = \frac{1.5}{3} = 0.5 \end{aligned}\right\} \quad I = 0.1365$$

$$\sigma_{z2} = Iq = 20.47 \text{ kN/m}^2 = \sigma_{z3}$$

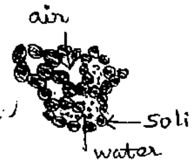
$$\sigma_z = \sigma_{z1} + \sigma_{z2} + \sigma_{z3} = \underline{\underline{62.28}} \text{ kN/m}^2$$

11. CONSOLIDATION

→ Compression

Compressibility is the property of soil due to which compression occurs. Clay has relatively more compressibility compared to gravel, sand and silt.

Compression of soil is due to :-

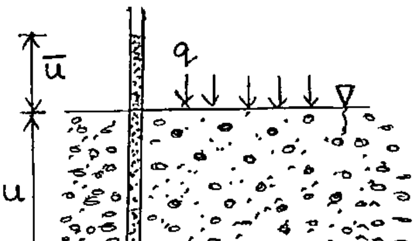


- compression and escape of air from voids → compaction.
- escape of pore water; whereas compression of solid grains and water is negligible. → consolidation

∴ compression depends upon volume of voids. More the vol. of voids, more will be the compression.

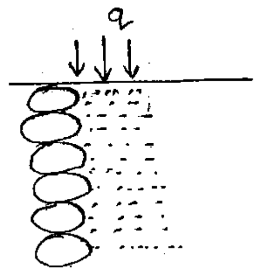
→ Consolidation

- It is the compression of soil due to expulsion of water under static long term loading.
- It is a slow process.
- It occurs in low permeable soil.



u = porewater pressure (or) Hydrostatic pressure.

$\bar{u}$ = excess pore water pressure (or) hydrodynamic pressure.



Hydro static → due to self wt.

Hydrodynamic → due to external load.

k_w → stiffness of water under confined condition

k_s → stiffness of soil grain structure.

$$k_w >> k_s.$$

At beginning, $\bar{u} = q$

At end of consolidation, $\bar{u} = 0$

Immediately after Loading	During Consolidation	At the end of consolidation
$\bar{u} = q$	$\bar{u} < q$	$\bar{u} = 0$
$\sigma' = 0$	$\sigma' > 0$	$\sigma' = q$
$\bar{u} + \sigma' = q$.	$\bar{u} + \sigma' = q$	$\bar{u} + \sigma' = q$

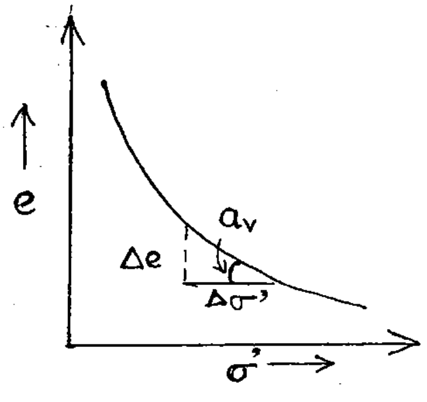
During consolidation, excess pore pressure ($\bar{u}$) decreases, effective stress (σ') increases, but total stress (σ) remains constant.

th Sept,
TURDAY

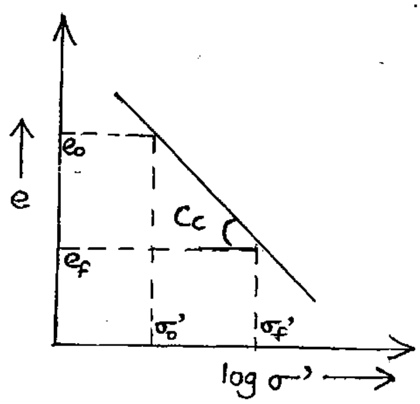
During Consolidation,

Properties which decrease.	Properties which increase	Properties which remains same
$\bar{u}$, Permeability, Compressibility, water content, void ratio.	σ', γ_d, settlement, shear strength.	σ, $S_r (= 100\%)$

→ $e - \sigma'$ curve.



→ $e - \log \sigma'$ curve



* Coefficient of Compressibility, a_v

$$\boxed{a_v = \frac{\Delta e}{\Delta \sigma'}}$$

* Compression Index, C_c

Coefficient of compression index, $C_c = \dfrac{e_0 - e_f}{\log\left(\dfrac{\sigma_f'}{\sigma_0'}\right)}$

$$\boxed{C_c = \frac{\Delta e}{\log_{10}\left(\frac{\sigma_f'}{\sigma_0'}\right)} \quad \Rightarrow \Delta e = C_c \log_{10}\left(\frac{\sigma_f'}{\sigma_0'}\right)}$$

$C_c = 0.007 (w_L - 10)$; for remoulded clay

$C_c = 0.009 (w_L - 10)$; for field consolidation
OR
normally consolidated clay

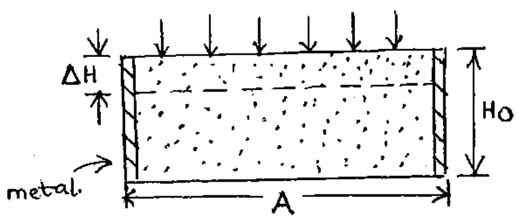
* Coeffecient of Volume Change (or)
Coeffecient of Volume compressibility

$$\boxed{m_v = \frac{\Delta V}{V_0 \cdot \Delta\sigma'}}$$

It is the volumetric strain per unit change of effective st

If soil is laterally confined (area remains same),

$$\frac{\Delta V}{V_0} = \frac{\Delta H}{H_0}$$



We have, $V \propto 1+e$.

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0}$$

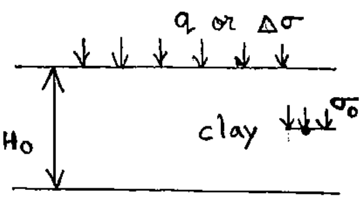
$$\Rightarrow \frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

$$m_v = \frac{\Delta V}{V_0 \cdot \Delta\sigma'} = \frac{\Delta e}{(1+e_0)\,\Delta\sigma'}$$

$$= \frac{a_v}{1+e_0} = \frac{\Delta H}{H_0 \cdot \Delta\sigma'}$$

* To find ultimate or final consolidation settlement, S_f or ΔH

(i) $\dfrac{\Delta H}{H_0} = \dfrac{\Delta e}{1+e_0}$



$\therefore$ ΔH or $S_f = H_0 \left(\dfrac{\Delta e}{1+e_0}\right)$

(ii) ΔH or $S_{f_1} = H_0 \cdot \dfrac{C_c}{(1+e_0)} \log_{10}\left(\dfrac{\sigma_f'}{\sigma_0'}\right)$

$\sigma_f' = \sigma_0' + \Delta\sigma'$

$\sigma_0' \rightarrow$ original or initial effective stress in the clay (due to self weight) at the centre of clay

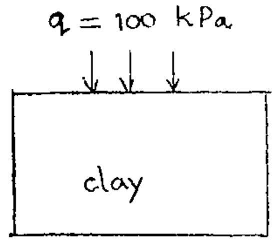
(iii) $m_v = \dfrac{\Delta H}{H_0 \cdot \Delta\sigma'}$

$\therefore$ ΔH or $S_f = m_v \cdot H_0 \cdot \Delta\sigma'$

(ii)nd equation is always preferred. because for a given soil, a_v & m_v are not constant; these decrease with increase in σ'. But C_c is always constant.

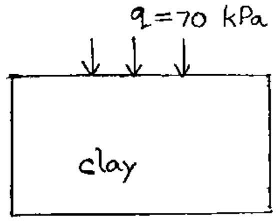
→ Depending upon Stress history, the States of Soil are:

- Normally Consolidated soil. (NC Soil)
- Over Consolidated Soil (OC soil)
- Under Consolidated soil (UC soil)



At the end of consolidation, $\sigma' = 100$ kPa

(Pre consolidated stress, $\sigma_c' = 100$ kPa)



Overconsolidated soil (or) preconsolidated soil.

Overconsolidated soil :- if the soil has ever been subjected to a pressure greater than existing pressure.

Normally consolidated soil :- if the soil has never been subjected to a pressure more than existing pressure.

Under consolidated soil :- when the soil is under consolidation.

→ Over Consolidation Ratio, OCR

$$OCR = \frac{\sigma_c'}{\sigma'}$$

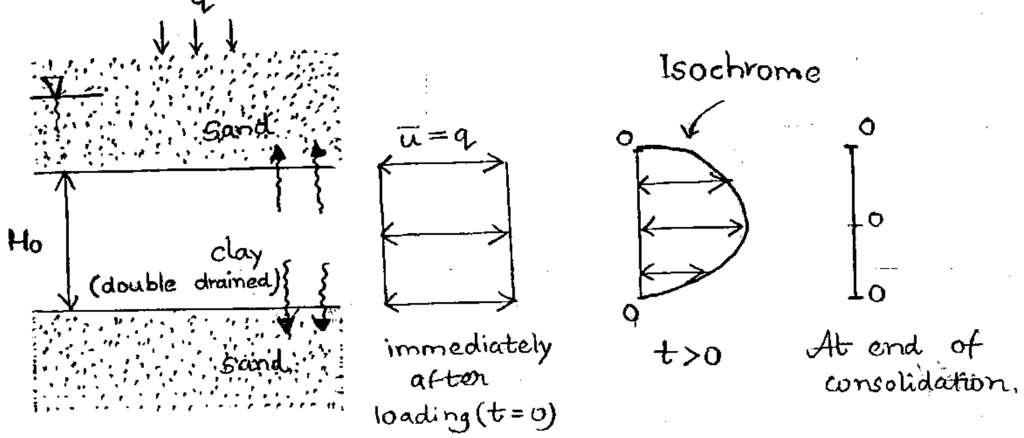
It is the ratio of preconsolidation stress to present effective stress in the soil.

For NC soil, OCR = 1.

For OC soil, OCR > 1

For UC soil, OCR < 1

→ Terzaghi's Theory of 1-D Consolidation :



Water tends to escape in vertical direction (to sand hav higher permeability), one direction, ∴ called 1D consolidation.

Water near the sand escapes first (lesser drainage distance) and $\bar{u}=0$. Since there is sand in both sides of clay, it is called double drained clay.

* Slope of Isochrome $= \frac{\bar{u}}{z} = \frac{h}{z} = i =$ hydraulic gradie.

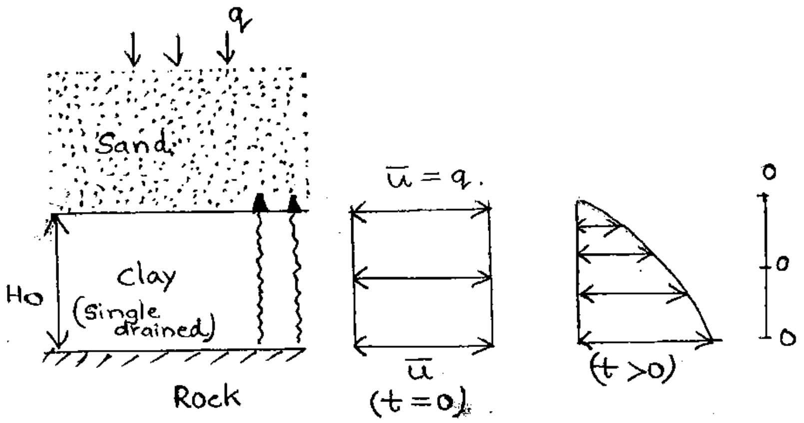
But $\bar{u}$ varies with time and depth. ∴ slope is not constant.

* Drainage path, d

$d = \frac{H_o}{2}$; for double drained condition

$d = H_o$; for single drained condition

Drainage path is the max distance that the water has to travel to escape.



$$\frac{\partial \bar{u}}{\partial t} = \frac{k}{m_v \gamma_w} \frac{\partial^2 u}{\partial z^2}$$

DIFFERENTIAL EQUATION OF 1D CONSOLIDATION
$$\boxed{\frac{\partial \bar{u}}{\partial t} = C_v \cdot \frac{\partial^2 \bar{u}}{\partial z^2}}$$

→ Coefficient of Consolidation, C_v

$$C_v = \frac{k}{m_v \gamma_w}$$

Units: m^2/sec , cm^2/sec

$$\frac{\partial \bar{u}}{\partial t} = C_v \frac{\partial^2 \bar{u}}{\partial z^2}$$

Solution is given in terms of (i) Time factor, T_v
(ii) Degree of consolidation, U

* Time factor, $T_v = \frac{C_v t}{d^2}$

$t \rightarrow$ time of consolidation.

* Degree of consolidation, $U = \frac{S}{S_f} \times 100$

$S \rightarrow$ settlement occured upto certain time, t
$S_f \rightarrow$ final settlement.

At beginning, $S = 0$; $U = 0$ $\quad 0 \leq U \leq 100\%$
At end, $S = S_f$; $U = 100\%$

Also, $U = \frac{\text{dissipated excess pore pressure}}{\text{initial excess pore pressure}} \times 100$

$$= \frac{\bar{u}_i - \bar{u}}{\bar{u}_i} \times 100$$

$\bar{u}_i \rightarrow$ Initial excess pore pressure.
$\bar{u} \rightarrow$ excess pore pressure after certain time, t

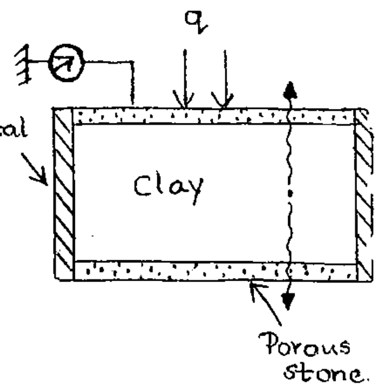
or $U = \frac{\sigma'}{\bar{u}_i} \times 100$; $\sigma' = \bar{u}_i - \bar{u}$

* Relation b/w U & T_v

(i) $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$; $U \leq 60\%$

(ii) $T_v = 1.781 - 0.933 \log_{10}(100 - U\%)$; $U > 60\%$

→ Consolidation Test (Oedometer Test)



- undisturbed sample is used
- diameter of sample ≥ 3 × thickness.

* To find 'e' in Consolidation test:

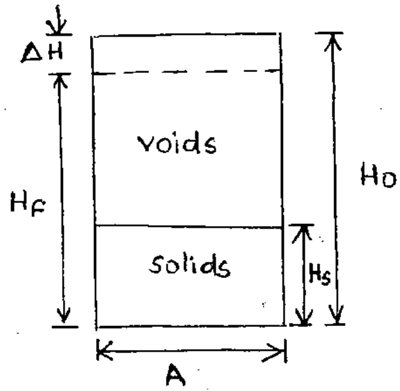
(i) Change in void ratio method:-

Final most void ratio, $e_f = w_f G$

$$\frac{\Delta e}{1+e_f} = \frac{\Delta H}{H_f}$$

Initial void ratio, $e_0 = e_f \pm \Delta e$

(ii) Height of Solids method:-



H_s → Height of solids

$$H_s = \frac{W_d}{G\gamma_w \cdot A}$$

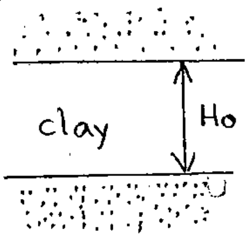
$$e_0 = \frac{H_0 - H_s}{H_s}$$

$$H_f = H_0 - \Delta H.$$

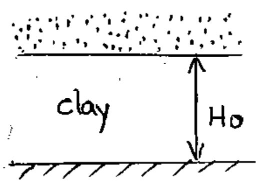
$$e_f = \frac{H_f - H_s}{H_s}$$

NOTE :

- For same loading and same clay properties, the ultimate consolidation settlement remains the same for both double drain clay condition and single drained clay condition.



$d = \frac{H_0}{2}$; S_{f1}



S_{f2} ; $d = H_0$

$S_{f1} = S_{f2}$

$$\Delta H = H_0\left(\frac{\Delta e}{1+e_0}\right) \Rightarrow \Delta H \propto H_0$$

- For a given clay, C_v is constant (assumed).

For a given clay, to undergo same degree of consolidation the time required for a double drained condition is $\frac{1}{4}$th time required for a single drained condition

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

If C_v & U (or T_v) are same, $t \propto d^2 \Rightarrow \frac{t_1}{t_2} = \left(\frac{d_1}{d_2}\right)^2$

$$\boxed{t \propto \frac{d^2 m_v}{k}}$$

$$t = \frac{T_v d^2}{C_v} = \frac{T_v d^2 . m_v \gamma_w}{k}$$

$\therefore$ For same degree of consolidation, $t \propto \frac{d^2 m_v}{k}$.

$$\frac{t_2}{t_1} = \left(\frac{d_2}{d_1}\right)^2 \left(\frac{m_{v2}}{m_{v1}}\right) \frac{k_1}{k_2}$$

15th Sept, MONDAY

P-58

01 From 1 to 2 kg/cm^2, $\Delta H = 1\,cm \Rightarrow \Delta\sigma' = 2-1 = 1$; $\frac{\sigma_f'}{\sigma_0'} = \frac{2}{1} = 2$

From 2 to 4 kg/cm^2, $\Delta H = ?$ $\Rightarrow \Delta\sigma' = 4-2 = 2$; $\frac{\sigma_f'}{\sigma_0'} = \frac{4}{2} = 2$

$$\Delta H = H_0 . \frac{C_c}{1+e_0} \log_{10}\left(\frac{\sigma_f'}{\sigma_0'}\right)$$

Since $\frac{\sigma_f'}{\sigma_0'}$ ratio is same, ΔH is also same.

$$\therefore \Delta H = 1 \text{ cm}$$

02. In $t_1 = 4$ years ; $S_1 = 80$ mm

$t_2 = 9$ years ; $S_2 = ?$

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

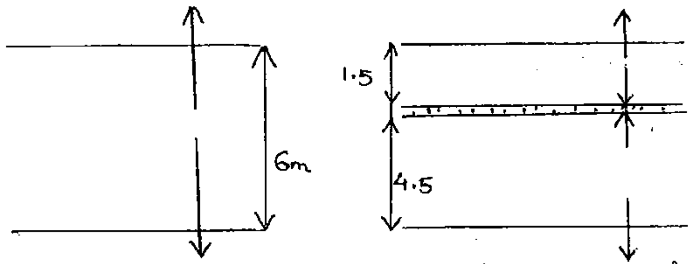
$$\frac{\pi}{4} u^2 = \frac{C_v t}{d^2} \Rightarrow t \propto u^2$$

$$u = \frac{S}{S_f} \Rightarrow u \propto S.$$

$$\therefore \boxed{t \propto s^2}$$

$$\frac{t_1}{t_2} = \left(\frac{S_1}{S_2}\right)^2 \quad \text{(valid only when } u < 60\%\text{)}$$

$$\frac{4}{9} = \left(\frac{80}{S_2}\right)^2 \Rightarrow S_2 = \frac{80 \times 3}{2} = \underline{\underline{120 \text{ mm}}}$$

3. 

Two drainage paths. 4 drainage paths

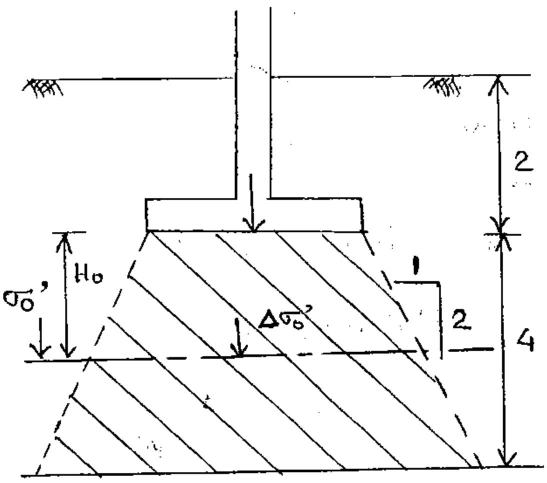
But ultimate settlement remains the same

4. $H_0 = 4\,m$, $C_c = 0.36$

$e_0 = 0.92$

$$\sigma_0' = \left(2 + \frac{4}{2}\right)\gamma = \underline{\underline{37.2}} \text{ kPa}$$

$$\Delta\sigma' = \frac{Q}{(B+z)^2}$$



z : distance b/w pt. where load is acting & pt. where $\Delta\sigma$ is required.

$$\Delta\sigma' = \frac{500}{(B+2)^2}$$

$$S_f = H_0 \cdot \frac{C_c}{1+e_0} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right)$$

$$0.12 = 4 \times \frac{0.36}{1+0.92} \log_{10}\left(\frac{37.2 + \frac{500}{(B+2)^2}}{37.2}\right)$$

$$\Rightarrow B = \underline{\underline{3.5m}}$$

05. $H_0 = 4m$;

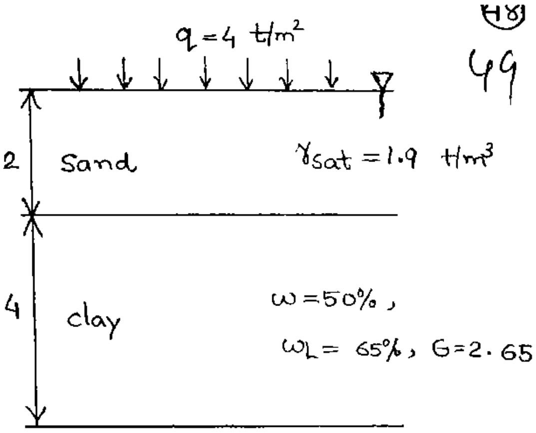
$C_c = 0.009(w_L - 10) = 0.495$

$e_0 = \frac{wG}{S_r} = 0.5 \times 2.65 = \underline{\underline{1.325}}$

$\sigma_0' = 2\gamma' + 2\gamma'$

$= 2(1.9-1) + 2(1.71-1)$

$= \underline{\underline{3.2}}\ t/m^2$



For clay, $\gamma_{sat} = \gamma_w \frac{(G+e_0)}{1+e_0} = \underline{\underline{1.71}}\ t/m^3$.

$\Delta\sigma' = q = 4\ t/m^2$.

$$S_f = 4 \times \frac{0.495}{1+1.325} \log_{10}\left(\frac{3.2+4}{3.2}\right) = \underline{\underline{29.8\ cm}}$$

06. $u_1 = 25\%$, $t_1 = 10$ min

$u_2 = 50\%$, $t_2 = ?$

$$\frac{t_1}{t_2} = \left(\frac{25}{50}\right)^2 \qquad (t \propto u^2)$$

$$\frac{10}{t_2} = \frac{1}{4} \Rightarrow t_2 = \underline{\underline{40}}\ min$$

07. $$U = \frac{\overline{u_i} - \overline{u}}{\overline{u_i}} \times 100 = \frac{2-0.6}{2} \times 100 = \underline{\underline{70\%}}$$

$$U = \frac{S}{S_f} \times 100.$$

$$70 = \frac{S}{20} \times 100 \Rightarrow \underline{\underline{S = 14}}\ mm$$

08. $$t \propto \frac{d^2 m_v}{k}$$

$$\frac{t_2}{t_1} = \left(\frac{d_2}{d_1}\right)^2 \left(\frac{m_{v2}}{m_{v1}}\right) \cdot \left(\frac{k_1}{k_2}\right)$$

$$= \left(\frac{2d_1}{d_1}\right)^2 \left(\frac{4m_{v1}}{m_{v1}}\right)\left(\frac{k_1}{3k_1}\right) = \frac{16}{3} \Rightarrow t_2 = \frac{16}{3} \times 15 = \underline{\underline{80\ ye}}$$

9. **Lab specimen** | **Field Clay**

$H_1 = 20$ mm | $H_2 = 10$ m

$d_1 = \frac{H_1}{2} = 10$ mm. | $d_2 = \frac{H_2}{2} = 5000$ mm

$U_1 = 50\%$ | $U_2 = 50\%$

$t_1 = 45$ min. | $t_2 = ?$

For same C_v & u, $t \propto d^2$

$$\frac{t_2}{t_1} = \left(\frac{d_2}{d_1}\right)^2$$

$$\frac{t_2}{45} = \left(\frac{5000}{10}\right)^2$$

$$t_2 = \underline{\underline{21.4}} \text{ years}$$

10. If field clay is single drained = 4 times of 21.4 years
= $\underline{\underline{85.6}}$ years

Q. 11. In the laboratory ..... Find the time required to undergo 70% consolidation of a field clay of 3m thick and double drained?

Lab Specimen:

$H_1 = 25$ mm

$d_1 = H_1 = 25$ mm (single drained)

$U_1 = 50\%$

$t_1 = 11$ min.

Field Clay:

$H_1 = 3$ m

$d_1 = \frac{H_1}{2} = 1.5\text{m} = 1500$ mm (double drai

$U_2 = 70\%$

$t_2 = ?$

For same C_v, $t \propto d^2 T_v$.
(same soil)

$$\frac{t_2}{11} = \frac{0.405}{0.197} \times \left(\frac{1500}{25}\right)^2$$

$$t_2 = \underline{\underline{56.5}} \text{ days}$$

12. Double drainage | Single drainage

Double drainage	Single drainage
H	H
$d_1 = \frac{H}{2}$	$d_1 = H$.
$t_1 = 5$ years.	$t_2 = 5$ years.
$S_1 = 9$ cm.	$S_2 = ?$
$S_f = 45$ cm	$S_f = 45$ cm.

$$T_v = \frac{\pi}{4} U^2$$

$$\frac{C_v t}{d^2} = \frac{\pi}{4} U^2 \Rightarrow U \propto \frac{1}{d}$$

$$U \propto S \Rightarrow \boxed{S \propto \frac{1}{d}}$$ (when $u < 60\%$).

$$\frac{S_1}{S_2} = \frac{d_2}{d_1}$$

$$\frac{9}{S_2} = \frac{2H}{\cancel{2}H} \Rightarrow S_2 = \underline{\underline{4.5}} \text{ cm}$$

13. For $t_1 = 4$ years.; $S_1 = 80$ mm.

For $t_2 = 9$ years; $S_2 = ?$

$$U_1 = \frac{S_1}{S_f} \times 100 = 26.67\%$$

$$T_{v1} = \frac{\pi}{4}\left(\frac{U_1}{100}\right)^2 = \underline{\underline{0.0558}}$$

$$T_v = \frac{C_v t}{d^2} \Rightarrow t \propto T_v$$

$$\frac{t_2}{t_1} = \frac{T_{v2}}{T_{v1}}$$

$$\frac{9}{4} = \frac{T_{v2}}{0.0558} \Rightarrow T_{v2} = \underline{\underline{0.1255}}$$

From the table given, $T_{v2} = \frac{\pi}{4}\left(\frac{U_2}{100}\right)^2$

$$0.125 = \frac{\pi}{4} \times \left(\frac{U_2}{100}\right)^2 \Rightarrow U_2 = 40\%$$

$$U_2 = \frac{S_2}{S_f}$$

$$0.4 = \frac{S_2}{300} \Rightarrow S_2 = \underline{\underline{120}} \text{ mm}$$

14. $$\frac{T_{v2}}{T_{v1}} = \frac{t_2}{t_1}$$

$$\frac{T_{v2}}{0.0558} = \frac{25}{4} \Rightarrow T_{v2} = 0.348$$

$\therefore U_2 = 65\%$ (from table)

$$U_2 = \frac{S}{300} \Rightarrow S = 300 \times 0.65 = \underline{\underline{195 \text{ mm}}}$$

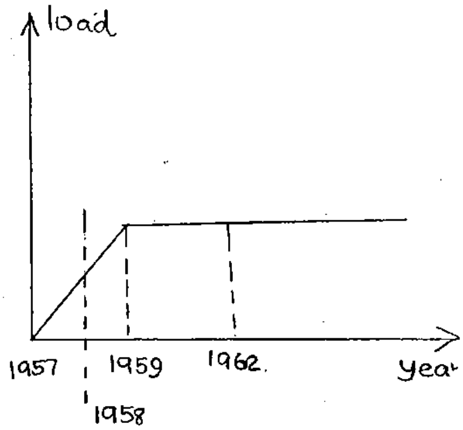
15 $$U_2 = \frac{S_2}{S_f} \times 100 = 70\%$$

$\therefore T_{v2} = 0.403$ (from table)

$$\frac{t_2}{t_1} = \frac{T_{v2}}{T_{v1}} \Rightarrow \frac{t_2}{4} = \frac{0.403}{0.0558}$$

$\therefore t_2 = \underline{\underline{28.8}}$ years

16. Consolidation time is measured from the middle of construction period. (1958)



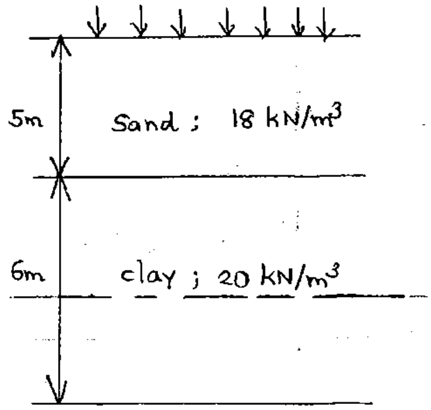
$t = 1962 - 1958 = 4$ years ; $S_1 = 90$ mm.

$t = 1967 - 1958 = 9$ years ; $S_2 = ?$

$$\frac{t_2}{t_1} = \left(\frac{S_2}{S_1}\right)^2$$

$$\frac{9}{4} = \left(\frac{S_2}{90}\right)^2$$

$$S_2 = \underline{\underline{135 \text{ mm}}}$$



17. **Immediately after loading**

$\sigma = q + 5\gamma_{sat} + 3\gamma_{sat}$

$= 76 + 5 \times 18 + 3 \times 20$

$= 226$ kPa.

Total $U = U_{st} + \bar{U}$

$= (5+3)10 + 76$

$= 156$ kPa

After many years (at end of consolidation)

$\sigma = 226$ kPa.

Total $\bar{U} = U_{static}$

$= 80$ kPa.

$\sigma' = 226 - 80 = \underline{\underline{146 \text{ kPa}}}$

σ' (immediately after loading) $= 5\gamma' + 3\gamma'$

$= 5 \times (18-10) + 3(20-10)$.

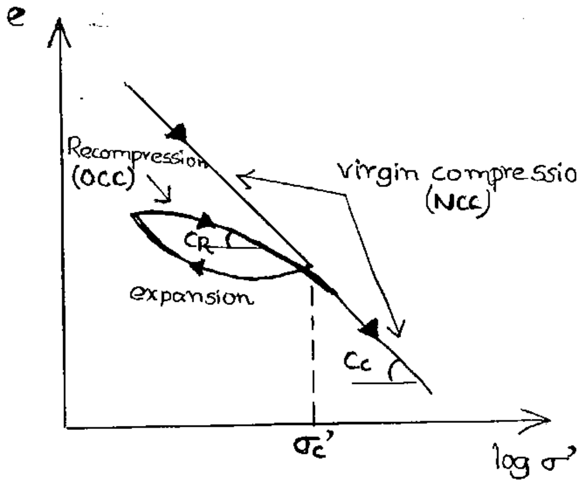
$= \underline{\underline{70 \text{ kPa}}}$

σ' (after many years) $= \sigma_0' + \Delta\sigma = 70 + 76 = \underline{\underline{146 \text{ kPa}}}$

→ Recompression Index, C_R

Slope of recompression curve (OCC) is called C_R.

$C_R \approx \frac{1}{5} C_c$

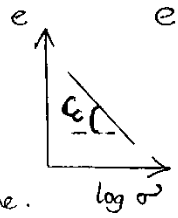


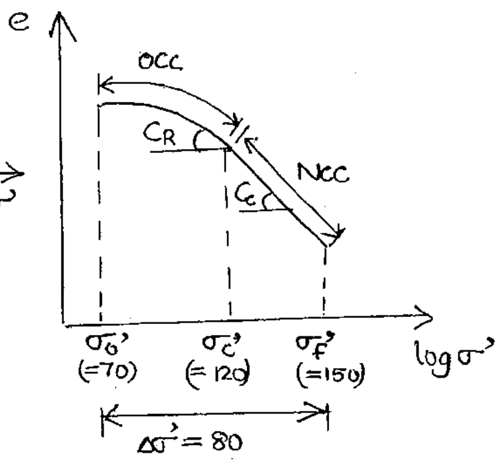
Straight line portion is called Virgin compression or NCC.

UCC cannot be marked in 'e-log σ'' as eqbm void ratios are used (ie void ratio at the end of consolidation)

20. $S_f = H_0 \times \frac{C_c}{1+e_0} \times \log_{10}\left(\frac{\sigma_f'}{\sigma_0'}\right)$

Above eqn is used only when 'e-log σ'' curve is a straight line.





$\therefore S_f = H_0 \cdot \frac{C_R}{1+e_0} \log_{10} \frac{\sigma_c'}{\sigma_0'} +$

$H_0 \cdot \frac{C_c}{1+e_0} \log_{10} \frac{\sigma_f'}{\sigma_c'}$

$= 5 \times \frac{0.03}{(1+0.9)} \log_{10}\left(\frac{120}{70}\right) + 5 \times \frac{0.27}{(1+0.9)} \times \log_{10}\left(\frac{150}{120}\right)$

$= 0.087 \text{ m} = \underline{\underline{87.34 \text{ mm}}}$

→ Stages of Consolidation

(i) Initial Consolidation.

Settlement that occurs immediately after loading due to elastic nature and escape of air.

(ii) Primary Consolidation

Due to expulsion of pore water

(iii) Secondary Consolidation (Creep)

due to plastic readjustment of particles and escape of some double layer water or highly viscous water.

→ Elastic (or Immediate) Settlement :

$$S_i = \frac{q_n}{E_s} \cdot B \cdot (1-\mu^2) \cdot I$$

where q_n → net pressure intensity.

E_s → Young's Modulus of Soil.

μ → Poisson's Ratio of Soil.

B → Characteristic linear dimension (usually width of footing or diameter of footing)

I → Influence factor

Influence factor, (I) depends on stiffness, shape, $\frac{L}{B}$ ratio of footing and location of point.

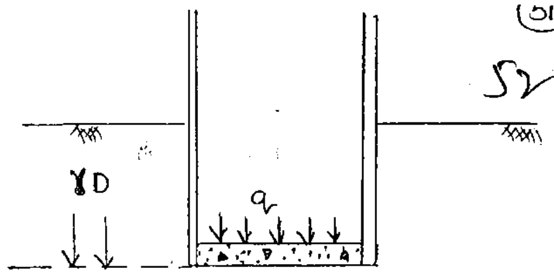
NOTE : The immediate settlement of a rigid footing is about 0.8 times the maximum settlement of an equal flexible footing (at the centre).

For a circular footing,

	Centre	Corner	Average
– Flexible	1.0	0.64	C
– Rigid	0.8		

52

$$\boxed{q_n = q - \gamma D}$$



Q21. Oil tank foundation → flexible footing.

$I_{(centre)}$ for circular flexible footing $= 1$.

$$q_n = q - \gamma D = 250 - 22 \times 3 = 184 \text{ kN/m}^2$$

$$B = \text{diameter} = 20 \text{ m}.$$

$$\mu = 0.45$$

$$\Rightarrow S_i = \frac{q_n B (1-\mu^2) I}{E_s} = \frac{184 \times 20 (1-0.45^2) \times 1}{6 \times 10^4}$$

$$= 0.04891 \text{ m} = \underline{48.9 \text{ mm}}$$

Q. The average effective overburden pressure on a 10m thick saturate clay layer is 150 kPa. Consolidation test on an undisturbed soil sample taken from clay layer showed that void ratio decreased from 0.6 to 0.5, By increasing the stress intensity from 100 kPa to 300 kPa. Determine the initial void ratio of clay layer. Also determine the total consolidation settlement of the clay layer due to construction of a structure imposing additional stress intensity of 200 kPa.

0.0209

e_0

$\frac{\Delta e}{1+e_0} = \frac{\Delta H}{H_0}$

$$C_c = \frac{\Delta e}{\log \frac{\sigma_f'}{\sigma_0'}} = \frac{0.6 - 0.5}{\log_{10}\left(\frac{300}{100}\right)} = \underline{0.209}.$$

$$C_c = \frac{0.6 - e}{\log_{10} \frac{150}{100}} \Rightarrow e = \underline{0.563}$$

$$\Delta H = H_0 \cdot \frac{C_c}{1+e_0} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right) = 10 \times \frac{0.209}{1+0.563} \log\left(\frac{150+200}{150}\right)$$

$$= \underline{0.492 \text{ m}}$$

16th Sept,
TUESDAY

12. COMPACTION

- Compaction of soil is due to compression and escape of air.
- It is a quick process.
- under short term loading, moving loads etc

→ Effect of Compaction:

- shear strength increases.
- compressibility decreases
- permeability decreases

⊙ During compaction, some amount of water is generally added to have a lubrication effect b/w the particles to facilita easy compaction.

→ Compaction Tests:

The purpose of compaction test is :
- to find OMC
- to find compactive energy.

1. IS Light Compaction Test (Std. Proctor's test)

This test is performed for ordinary roads, earthen dams.

2. IS Heavy Compaction Test (Modified Proctor's test

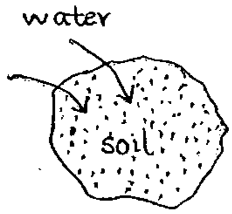
This test is performed for expressways, runways e

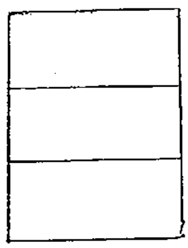
	Wt. of Rammer	Ht. of fall.	No: of blow per each layer	No: of Layers
Heavy test	4.90 kg	45 cm	25	5
Light test	2.60 kg	31 cm	25	3

- Total energy ratio $= \dfrac{4.90 \times 45 \times 25 \times 5}{2.60 \times 31 \times 25 \times 3} = \underline{\underline{4.55}}$

- Mould Capacity = 1 L generally or 2.25 L. (if soil has % reta on 4.75 mm > 20%)

$$\left.\begin{array}{l}\text{For 1 L mould} \rightarrow \text{25 blows per each layer} \\ \text{For 2.25 L mould} \rightarrow \text{56 blow per each layer}\end{array}\right\} \text{for both tests.}$$





Mould.

W = wt. of soil

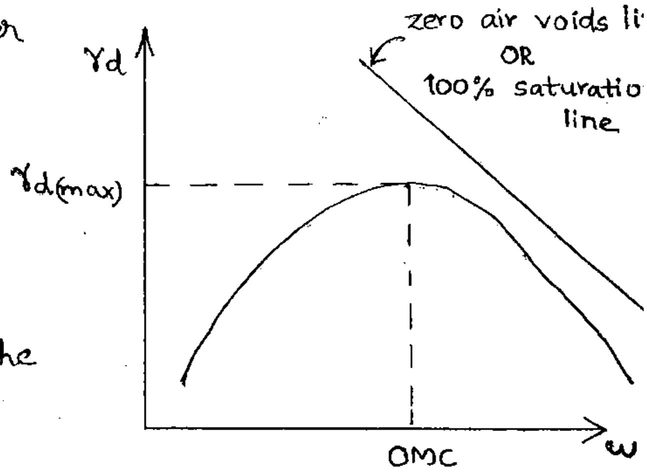
V = vol. of soil.

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

$\omega \rightarrow$ obtained by over drying.

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

The test is repeated for diff water contents and γ_d is obtained in each case. A graph is plotted b/w γ_d & ω.



- compaction curve

OMC:— Optimum Moisture Content is the water content at which max. dry density is obtained.

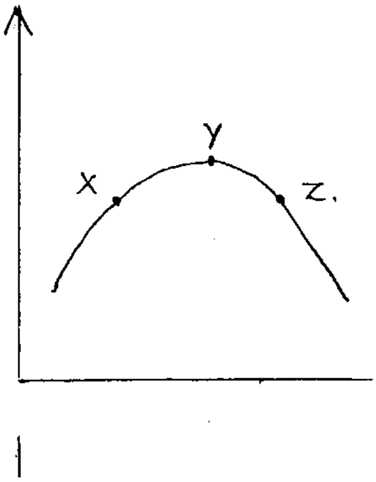
$\gamma_d = \dfrac{(1-n_a) G \gamma_w}{1+\omega G}$; n_a = % air voids $\rightarrow$ Equation of Compaction cur

$(\gamma_{dmax})_{theo.} = \dfrac{G \gamma_w}{1+\omega G} \rightarrow$ Equation of Zero Air Voids line

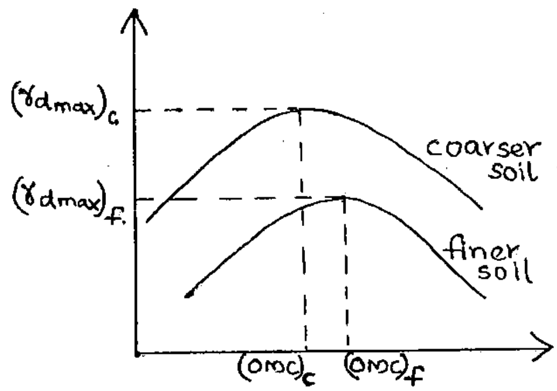
- Zero air voids line is used to compare and understand the level of compaction.

- K at Y is least (as least no. of voids due to optimum compaction)

- K at Z is relatively less than K at X.



* Same Test but Soils are Different

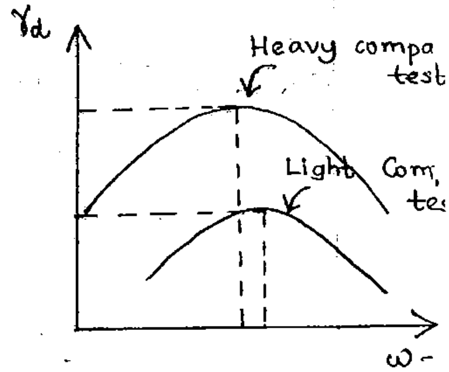


Finer soil has relatively more omc and less $(\gamma_d)_{max}$ compared to coarser soil.

Finer soils have more surface area and hence more omc. Also, $\gamma_d = \frac{G\gamma_w}{1+e}$ $\Rightarrow$ finer soils having larger void ratio (e) will be having lesser $(\gamma_d)_{max}$.

* Same soil but tests are Different.

As the compactive energy increases, γ_{dmax} increases but omc decreases.

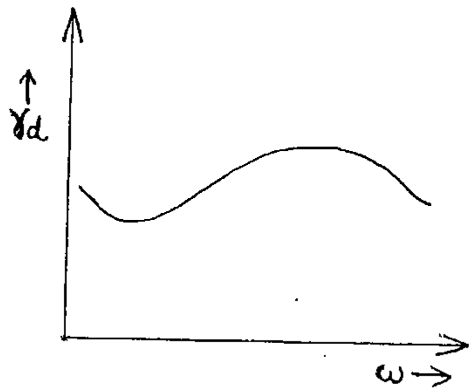


→ Factors affecting Compaction:

- water content (w.c)
- compactive energy
- type of soil.

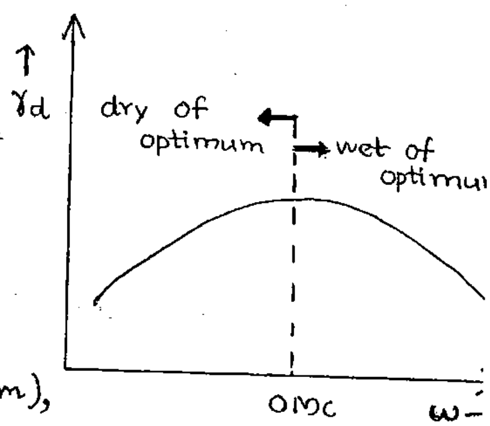
In the case of pure sand without fines, there is no well defined OMC and the curve is shown below.

∴ For the above soil, the compaction curve is not useful.



Relative density (or density index) is used to indicate the level of compaction achieved.

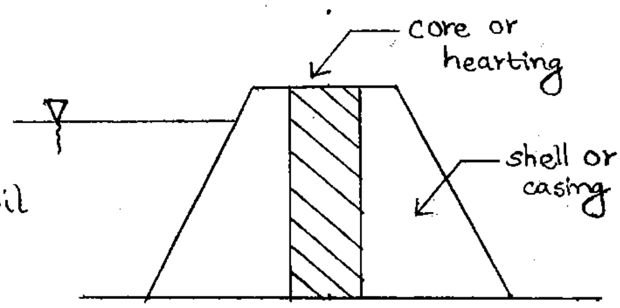
Dry of Optimum	Wet of Optimum
- Flocculent struct.	- dispersed struct.
- more shear strength	- less shear strength
- more swelling type	- less swelling type.



- To avoid swelling of soil (below the floors, pavements, core of earthen dam), soil is compacted wet of optimum.
- To have more strength (road embankments & casing of earthen dams), the soil is compacted dry of optimum.
- Placement water content :- w/c actually used at site

→ Earthen Dam:

* Core :-
  - to check seepage
  - made up of impermeable soil (clay)
  - wet of optimum



* Shell :-
  - to provide stability
  - made up of soil other than clay.
  - dry of optimum.

→ Compaction Equipment :

- Tampers → manual compaction (for inaccessible areas trenches, behind retain walls
- Smooth wheel roller → to have smooth surface.
- Pneumatic Tyred roller → for all soils
- Sheep foot roller. → best suitable for clays
- vibratory roller → best suitable for sands

Kneading action : best for compaction of clay.

Vibratory : best for compaction of sand.

→ Relative Compaction

$$\text{Relative compaction} = \frac{\gamma_d \text{ of field}}{\gamma_{d\,max} \text{ in lab}} \times 100$$

Generally 90-95% is acceptable.

→ Proctor's Needle

To measure in-situ w.c and γ_d (approximate)

P-64

01. $\gamma_d = 1.8$ g/cc ; $\omega = 16\%$; $G = 2.65$

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

$$1.8 = \frac{2.65}{1+e} \Rightarrow e = 0.472$$

$$e = \frac{wG}{S_r} \Rightarrow 0.472 = \frac{0.16 \times 2.65}{S_r}$$

$$\therefore S_r = \underline{\underline{89.7\%}}$$

$$a_c + S_r = 100\% \qquad n_a = n a_c$$

$$\therefore a_c = \underline{\underline{10.3\%}} \qquad = \frac{e}{1+e} \cdot 10.3 = \underline{\underline{3.27\%}}$$

$$(\gamma_{d\,max})_{theo.} = \frac{G\gamma_w}{1+wG} = \underline{\underline{1.86\ g/cc}}$$

2. Vol. of soil = volume of cutter

$$= \frac{\pi}{4} d^2 h =$$

wt. of soil = 30.10 − 10.8 = 19.30 N.

= 0.0193 kN

$$\gamma_d = \frac{\gamma}{1+w} = \frac{W/V}{1+w} = \frac{13.65}{1+0.122}$$

$$= 12.16\ kN/m^3.$$

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

$$12.16 = \frac{2.65 \times 9.81}{1+e} \Rightarrow e = \underline{\underline{1.136}}$$

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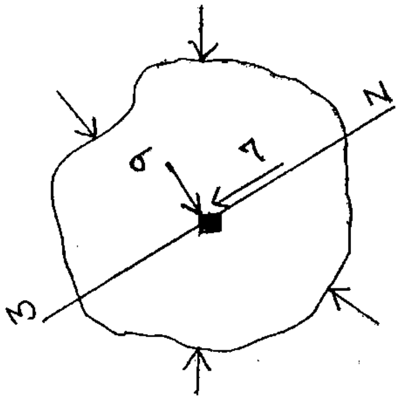
13. ⊙ 100% (S_r) saturation line & 0% (n_a) air voids line are same.

⊙ 95% (S_r) saturation line & 5% (n_a) air voids line are different.

$$\underline{\underline{S_r + a_c = 100\%}}$$

16th Sept,
TUESDAY

13. SHEAR STRENGTH



σ or $\sigma_n \rightarrow$ Normal Stress.

$\tau \rightarrow$ Shear Stress

3 – Principal planes, $\perp^{r}$ to each other

3 – Principal Stresses, $\perp^{r}$ to each other.

($\tau = 0$ in principal plar

$\sigma_1, \sigma_2, \sigma_3 \rightarrow$ Principal stresses.

$$\sigma_1 \geq \sigma_2 \geq \sigma_3$$

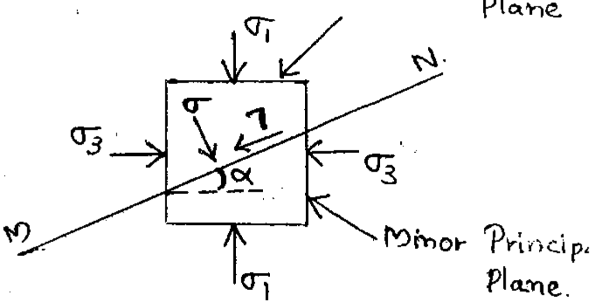
$\sigma_1 \rightarrow$ Major Principal Stress

$\sigma_2 \rightarrow$ Intermediate Principal Stress

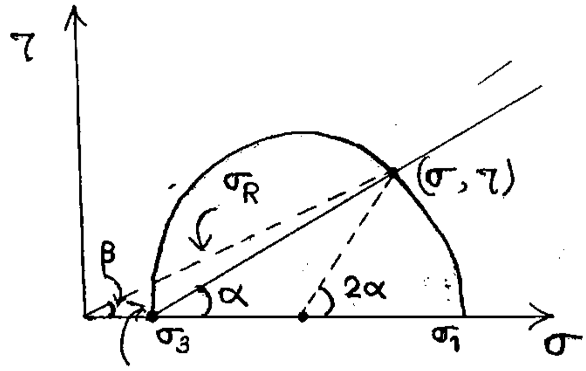
$\sigma_3 \rightarrow$ Minor Principal Stress

Analytical :

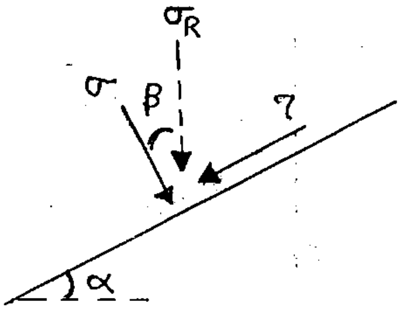
$$\boxed{\begin{aligned} \sigma &= \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\alpha \\ \tau &= \frac{\sigma_1 - \sigma_3}{2}\sin 2\alpha \end{aligned}}$$



Graphical : (Mohr Circle)



Pole (origin of planes)



σ_R = Resultant stress = $\sqrt{\sigma^2 + \tau^2}$

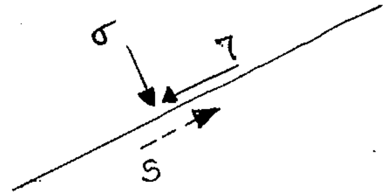
β = angle of obliquity (angle b/w σ & σ_R)

- For failure plane, angle of obliquity will be <u>maximum</u>

$S \rightarrow$ shear strength or shear resistance

$\tau \rightarrow$ shear stress

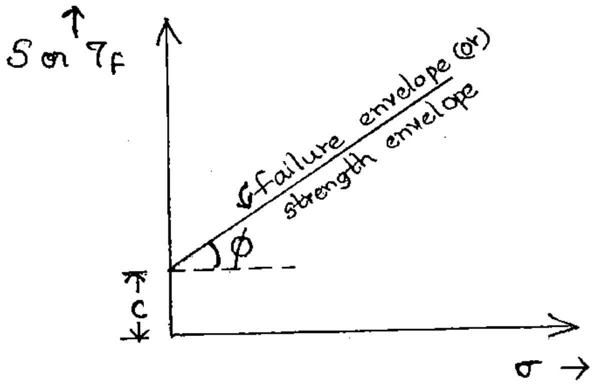
$\tau_f \rightarrow$ failure shear stress.

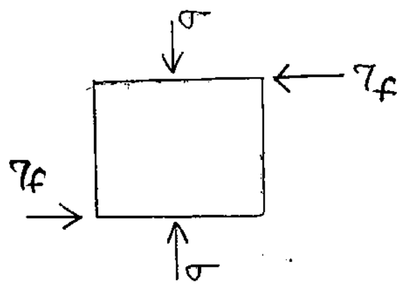


- If $\tau < s$, no shear failure.

$$\boxed{\tau_f = S}$$

→ Coulomb's Law





COULOMB'S EQUATION $\Longrightarrow$ $\boxed{S = c + \sigma \tan\phi}$

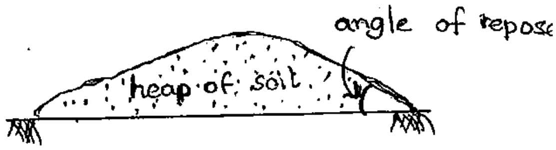
$\sigma \rightarrow$ Normal stress on the plane

$c \rightarrow$ Cohesion

$\phi \rightarrow$ angle of internal friction (or) angle of shearing resistance.

- Angle of Repose :- Natural slope of a soil heap.

Angle of repose & angle of shearing resistance are not equal to each other.



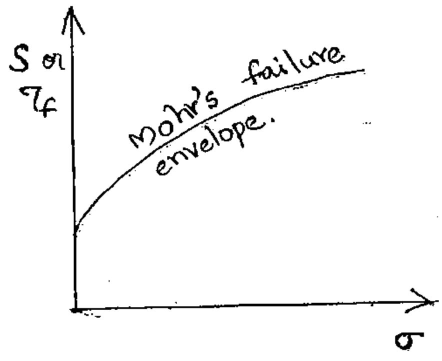
However, for a loose sand, these two are nearly equal

Angle of Shearing Resistance : angle of inclination of failure envelope.

→ Mohr's Theory

- Soil fails essentially due to shear.
- Failure shear stress depends on σ.
- σ_2 has no effect on the behaviour of soil

$$S = f(\sigma)$$



- As per Mohr's theory,

Mohr's failure envelope is curvilinear

* Nowadays,

$$S = c + \sigma \tan\phi \; ; \quad \boxed{\text{Mohr – Coulomb Equation.}}$$

→ Terzaghi's Concept.

- S depends on effective normal stress (σ')

$$\Rightarrow \quad S = c' + \sigma' \tan\phi'$$

c' → effective cohesion (or drained cohesion)

ϕ' → effective angle of internal friction.
(or drained angle of internal friction)

- Another equation :-

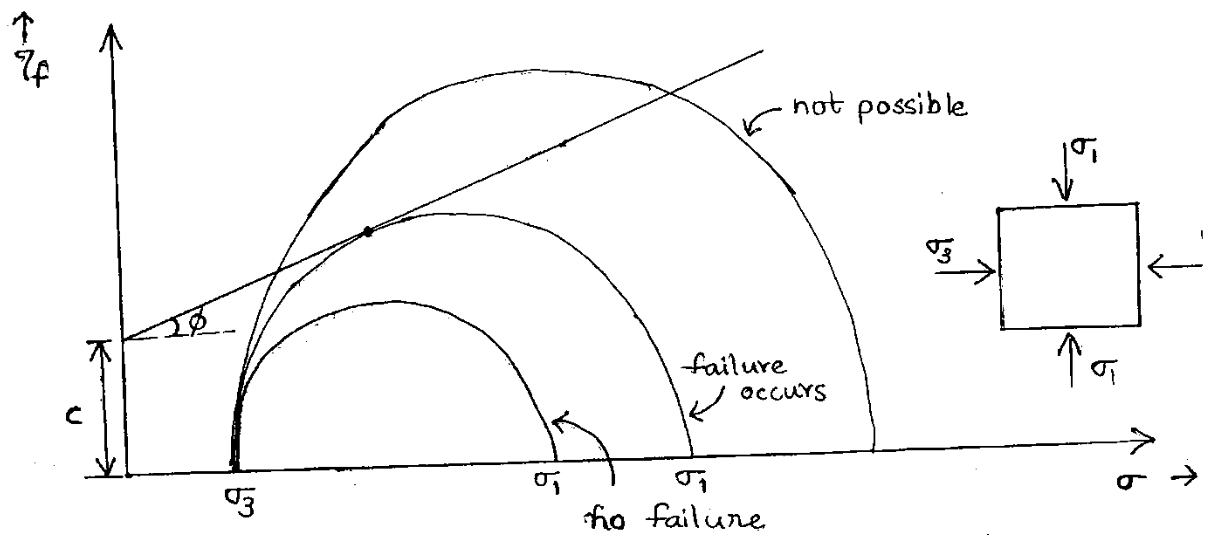
$$S = C_u + \sigma \tan\phi_u$$

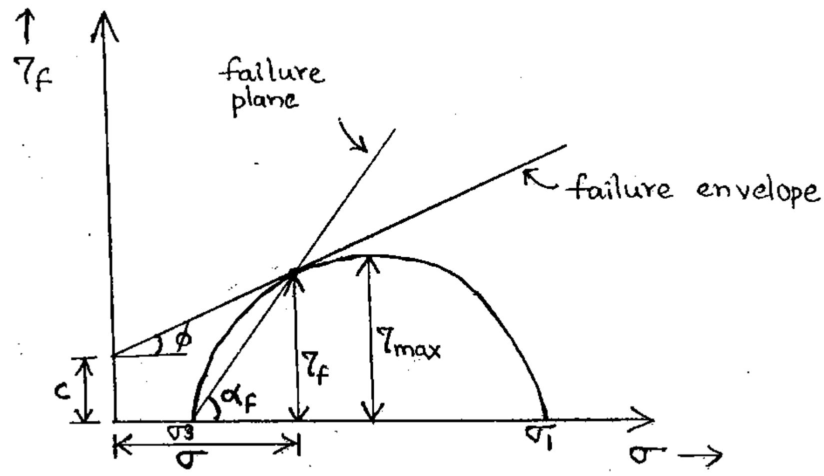
σ → total normal stress.

C_u → apparent cohesion (or) total cohesion (or) undrained cohesion.

ϕ_u → apparent angle of int. friction (or)
total angle of int. friction (or)
undrained angle of int. friction.

$S = c + \sigma \tan\phi$	$S = c' + \sigma' \tan\phi'$	$S = c_u + \sigma \tan\phi_u$ 57
- general equation - not used nowadays.	- interms of effective stress. - c' & ϕ' are called 'effective shear strength parameters' - used for drained conditions of soil	- interms of total stress. - c_u & ϕ_u are called 'total shear strength parameters' - used for undrained conditions of soil.



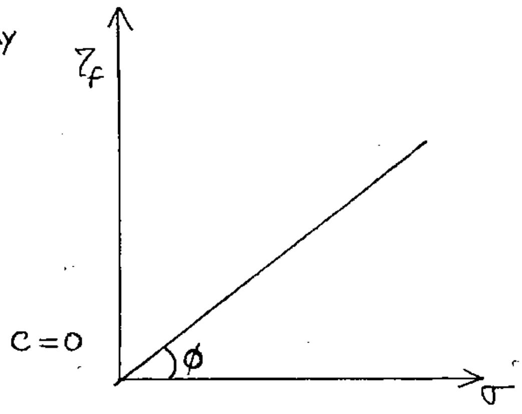


$\alpha_f \rightarrow$ failure plane inclination with major principal plane

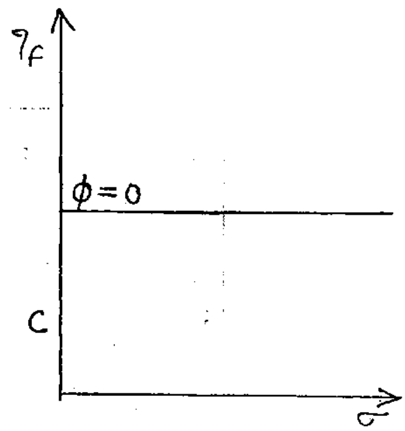
$$\boxed{\alpha_f = 45 + \phi/2}$$

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2} \; ; \quad \tau_f \leq \tau_{max}.$$

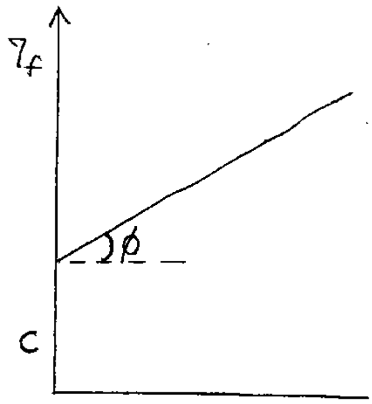
17 Sept
WEDNESDAY



Cohesionless soil (or)
Granular Soil.
Eg :- Dry sand.



Cohesive Soil.
Eg: Plastic ~~soil~~ clay.

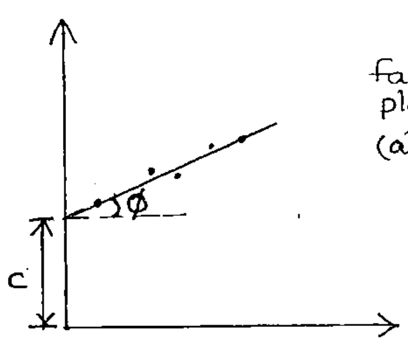


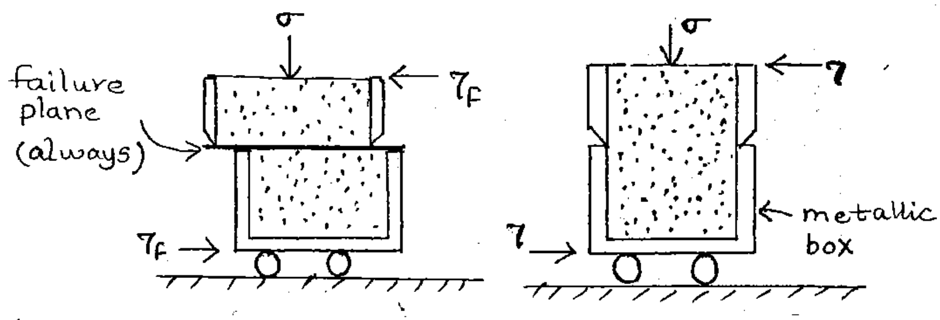
$C-\phi$ soil.
Eg : Clayey sand
Clayey gravel
Silty clay.

→ Tests to find c & ϕ Parameters

1. Direct Shear Test.
2. Triaxial Shear Test.
3. Unconfined Compression Test.
4. Vane Shear Test.

* Direct Shear Test (or) Box Shear Test.





σ	τ_f
10	8
20	15
30	22
40	29

To find analytically c & ϕ :

$$\tau_f = c + \sigma \tan\phi$$

$$8 = c + 10\tan\phi \rightarrow (1)$$

$$15 = c + 20\tan\phi \rightarrow (2)$$

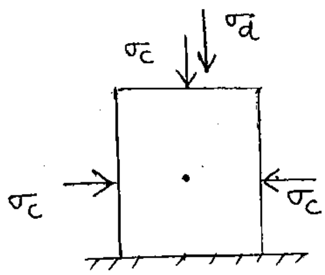
Solving we get c & ϕ values.

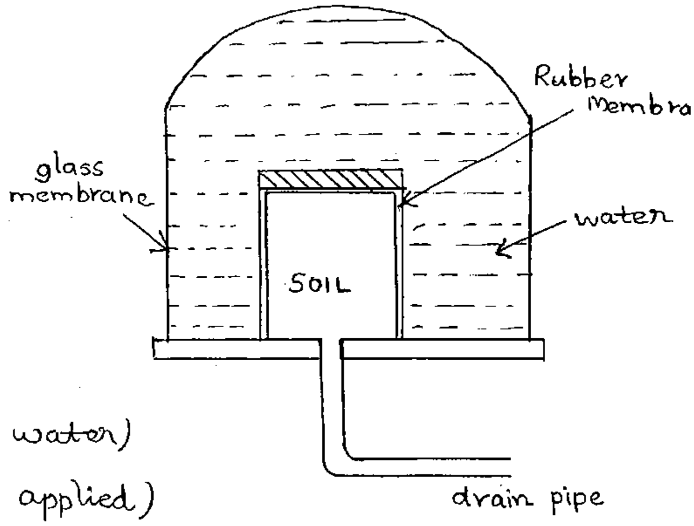
– This test is generally used for cohesionless soils.

– The main disadvantage of the test is that pore pressure cannot be measured directly.

– Depending on the drainage conditions provided, value of cohesion and angle of shearing resistance may be c' or c_u and ϕ' or ϕ_u.

* Triaxial Shear Test.



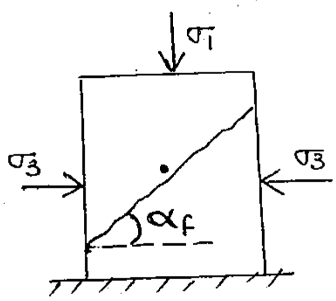


σ_c : confining pressure (due to water)

σ_d : Deviator stress (externally applied)

Water and wind always exerts normal force.

σ_c = confining pressure or cell pressure or chamber pressure or consolidation pressure or alround pressure



$\sigma_3 = \sigma_c$

$\sigma_1 = \sigma_c + \sigma_d$

$\sigma_2 = \sigma_c$

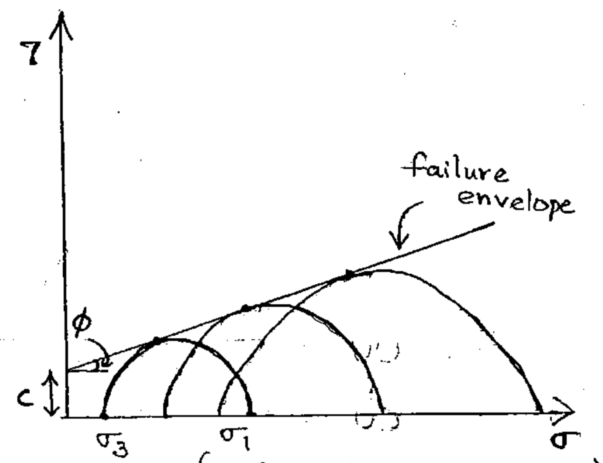
σ_3	σ_1	u
10	19	5
20	36	9
30	54	13
40	70	18

Keeping σ_3 constant, determine the deviator stress required to cause shear failure. Using σ_d, determine σ_1

– Open the valve of drain pipe to drain out pore water which gives c' & ϕ'

– Close the drain pipe valve to obtain c_u & ϕ_u (undrained)

When the test is performed under undrained conditions, pore water pressure can be measured.



(TOTAL STRESS CIRCLES)

$\sigma_3' = \sigma_3 - u$ & $\sigma_1' = \sigma_1 - u$.

- using σ_3' & σ_1', draw the effective stress circles. A failure envelope tangential to the effective stress circles is drawn and c' & ϕ' are obtained.
- Pore pressure developed in the case of drained test is zero

→ Plastic Equilibrium

A material is said to be in plastic equilibrium if every point of soil is at verge of failure.

* Plastic Equilibrium equations:

$$\boxed{\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2C_u \tan\left(45 + \frac{\phi}{2}\right)}$$

$$\boxed{\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2C' \tan\left(45 + \frac{\phi'}{2}\right)}$$

→ Depending upon drainage condition, types of SHEAR TESTS:

(i) Unconsolidated Undrained Test (UU test)
(ii) Consolidated Undrained Test (CU test)
(iii) Consolidated Drained Test. (CD test)

* Stages of Shear test:

- 1^{st} stage or consolidation stage (application of σ_c)
- 2^{nd} stage or shearing stage. (application of σ_d)

⊙ When the valve of drain pipe is opened. when σ_c is applied (1^{st} stage), pore water escapes and soil gets consolidate Otherwise, it remains unconsolidated.

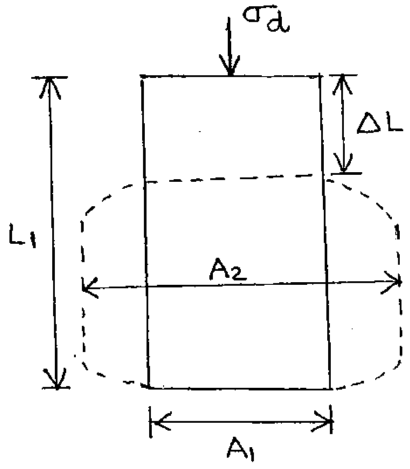
⊙ When the valve is open during application of σ_d, drained condition is obtained.

⊙ UU test : Drain valve is always kept closed. (Quick test)

⊙ CU test : Valve is open in 1^{st} stage & closed in 2^{nd} stage

⊙ CD test : Valve is always kept open (Slow test)

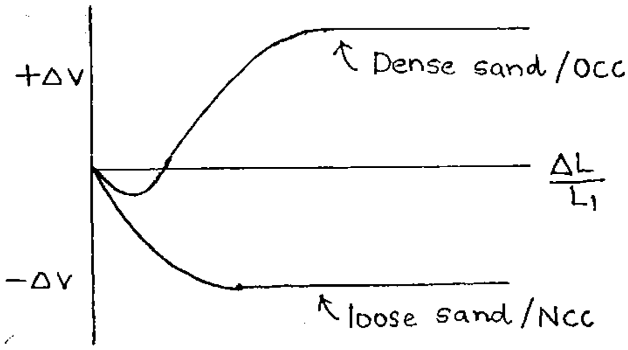
- c & ϕ are shear strength parameters and not the property of soil. It varies with drainage conditions and test performed.

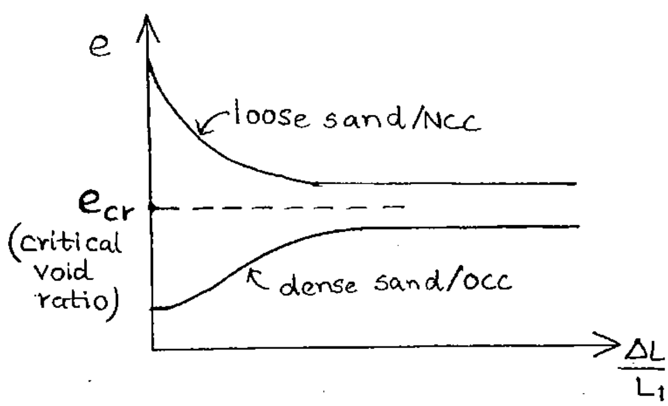


$$\sigma_d = \frac{\text{axial load}}{\text{area at failure}} = \frac{Q_a}{A_2}$$

$$A_2 = \frac{V_1 \pm \Delta V}{L_1 - \Delta L}$$

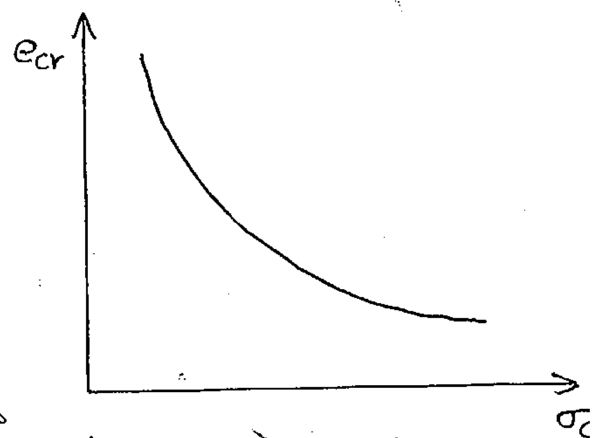
$\Delta V = 0$; undrained test



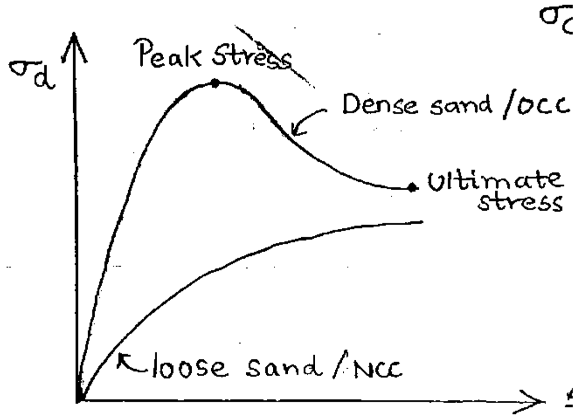


- Dilation :- The phenomenon of increase in volume of soil during shearing is called dilation. This is exhibited by dense sand and OCC.

- Critical void ratio (e_{cr}) decreases with increase in confining pressure (σ_c)

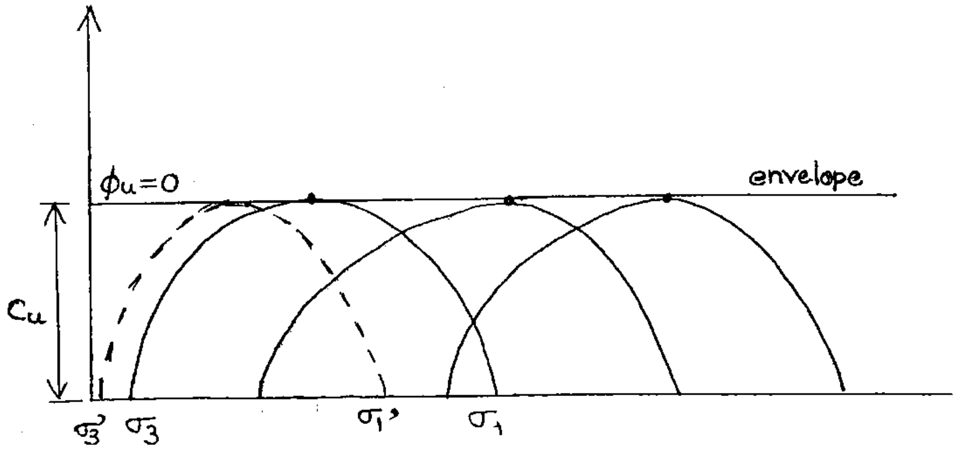


- Peak stress and ultimate stress



20th Sept,
SATURDAY

→ Triaxial test Curves:



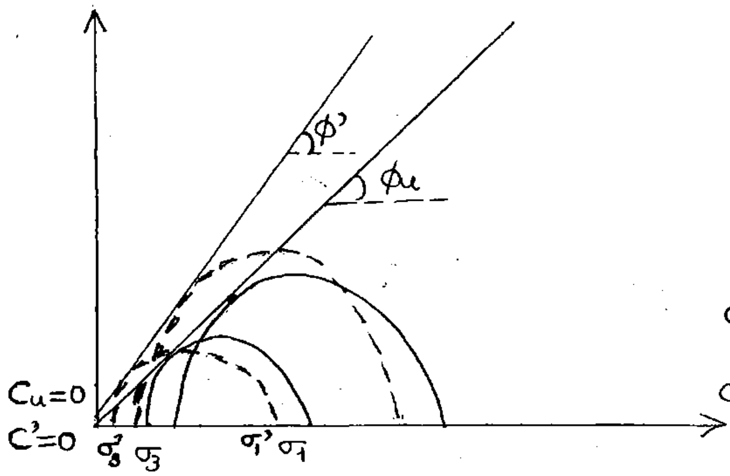
$$\sigma_1' = \sigma_1 - u$$

$$\sigma_3' = \sigma_3 - u.$$

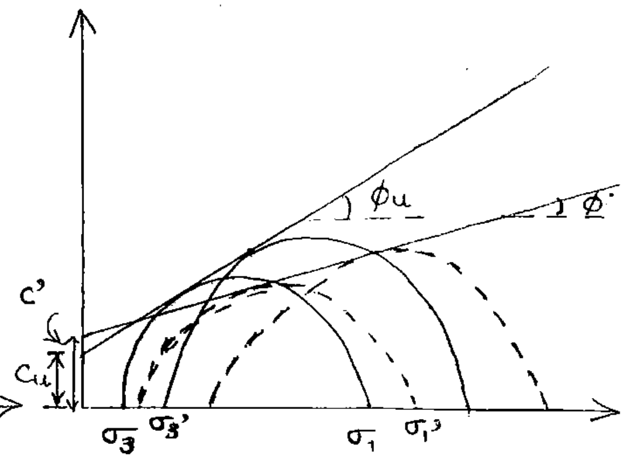
$$\sigma_3 - \sigma_1 = \sigma_{dv}$$

$$= \text{diameter}$$

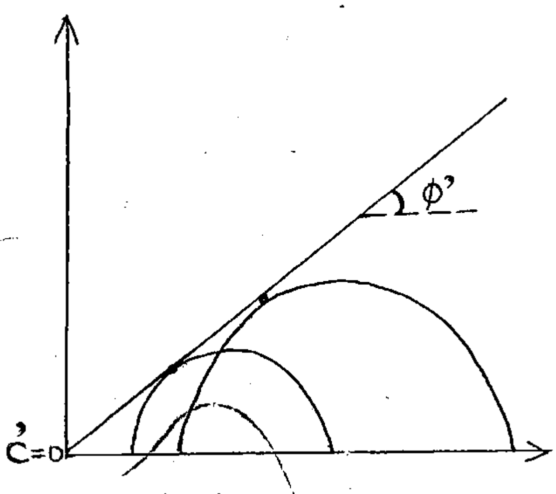
- UU test on NCC & OCC



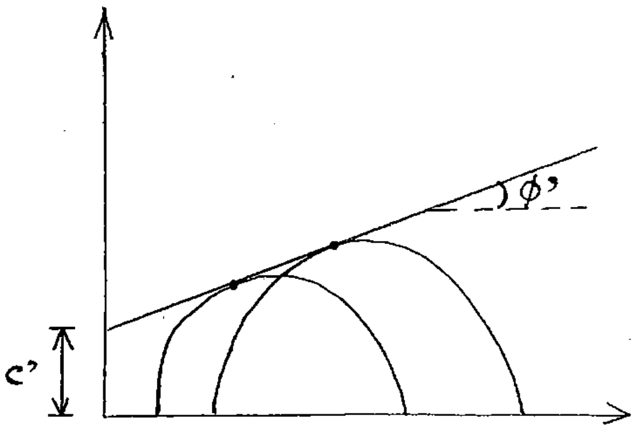
- CU test on NCC



- CU test on OCC.



- CD test on NCC

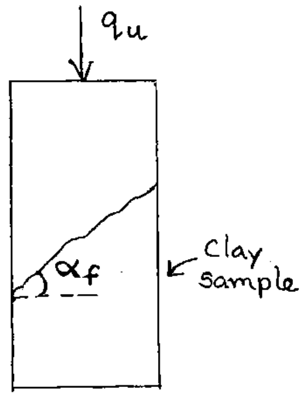


- CD test on OCC

* Unconfined Compression Test (UCC test)

- It is a special type of triaxial test with $\sigma_c = 0$
- conducted quickly to have undrained condition.
- Suitable for undrained, saturated clays. ($\phi_u = 0$)

q_u : unconfined compressive stress.



$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2 C_u \tan\left(45 + \frac{\phi_u}{2}\right)$$

In UCC test, $\sigma_3 = 0$ & $\sigma_1 = q_u$

$$\therefore \boxed{q_u = 2 C_u \tan\left(45 + \frac{\phi_u}{2}\right)}$$

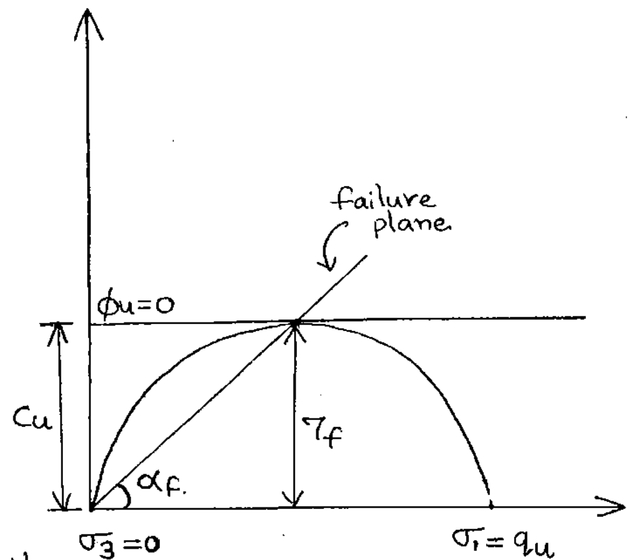
If soil is undrained saturated clay, then $\phi_u = 0$

$$\Rightarrow \boxed{C_u = \frac{q_u}{2}}$$

If $\phi = 0 \Rightarrow \alpha = 45^\circ$

$\tau_f = \tau_{max}$

- UCC is the only test in which Mohr's Circle passes through the origin.



⊙ Sensitivity = $\dfrac{\text{undisturbed strength}}{\text{remoulded strength}}$

If sensitivity $= 1$; soil is called "insensitive"
If sensitivity > 16; soil is called "quick soil" (Eg:- marine clay)

Quick soil is a type of soil whereas quick sand is a hydraulic condition.

- UCC is not suitable for fissured clay (cracked)
Soil will fail along the existing crack and does not exhibit the real strength.

* Vane Shear Test :

- can be conducted in lab or field.
- suitable for undrained saturated clays for which

$\phi_u = 0 \quad (\Rightarrow \; s = c)$

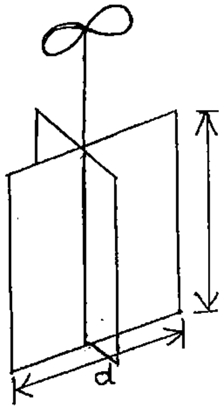
$$\boxed{T = \pi d^2 . C_u \left(\frac{H}{2} + \frac{d}{6}\right)}$$

when both bottom & top part take in shear

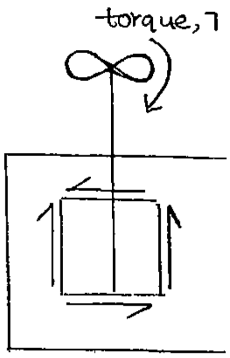
OR

$$\boxed{T = \pi d^2 . C_u \left(\frac{H}{2} + \frac{d}{12}\right)}$$

when bottom face takes part in shear



vane

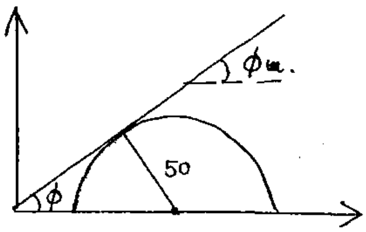


clay

P-73.

01. $\sin\phi = \frac{50}{150} = \frac{1}{3}$

$\phi = \sin^{-1}\left(\frac{1}{3}\right)$



02. $\tau_f = \frac{\sigma_1 - \sigma_3}{2} . \sin 2\alpha$.

$0.9 = \frac{5 - 3.2}{2} \sin 2\left(45 + \frac{\phi}{2}\right)$.

$\phi = 0$

OR

$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$

Since $\tau_f = \tau_{max}$, $\phi = 0$

03. $\sigma_1 = 200$, $\sigma_3 = 60$, $c' = 5 \text{ kN/m}^2$, $\phi' = 25^\circ$, $u = 20$

$\sigma_1' = \sigma_1 - u = 200 - 20 = 180$ kPa

$\sigma_3' = \sigma_3 - u = 60 - 20 = 40$ kPa

If $\sigma_3' = 40^\circ$, the required σ_1' to cause failure:

$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2 c' \tan\left(45 + \frac{\phi'}{2}\right)$.

$= 40 \tan^2\left(45 + \frac{25}{2}\right) + 2 \times 5 \tan\left(45 + \frac{25}{2}\right)$

$$\sigma_1' = 114 \text{ kPa}.$$

Since the proposed effective stress (180 kPa) is more than 114 kPa, the sample will definitely fail.

04. Compressive strength = deviator stress. = σ_d.

$c' = 15 \text{ kN/m}^2$, $\phi' = 20^\circ$, $\sigma_3' = 60$

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi}{2}\right) + 2c' \tan\left(45 + \frac{\phi}{2}\right).$$

$$= 60 \tan^2\left(45 + \frac{20}{2}\right) + 2\times15 \tan\left(45 + \frac{20}{2}\right).$$

$$= \underline{\underline{165.2}} \text{ kPa}$$

$$\sigma_d = \sigma_1' - \sigma_3' = 165.2 - 60 = \underline{\underline{105.2 \text{ kPa}}}$$

05. $q_u = 1.2 \text{ kg/cm}^2$.

$\alpha_f = 50^\circ$.

$$45 + \frac{\phi}{2} = \alpha_f$$

$$\phi = (50 - 45)2 = \underline{\underline{10^\circ}}$$

06. $q_u = 2c_u \tan\alpha_f$

$$\therefore \quad C_u = \frac{1.2}{2\tan 50} = \underline{\underline{0.503}} \text{ kg/cm}^2$$

07. $\sigma = 20 \text{ kN/m}^2$; $\tau_f = 16 \text{ kN/m}^2$.

$c = 8 \text{ kN/m}^2$; $\phi = 20^\circ$

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha_f$$

$$20 = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\left(45 + \frac{20}{2}\right) \rightarrow (1)$$

$$\tau_f = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha_f$$

$$16 = \frac{\sigma_1 - \sigma_3}{2} \sin 2\left(45 + \frac{20}{2}\right) \rightarrow (2)$$

Solving (1) & (2),

$$\sigma_1 = 42.85 \text{ kPa} \quad \& \quad \sigma_3 = 8.7 \text{ kPa.}$$

$\sigma_1 - \sigma_3 = 34.05$

$\sigma_1 + \sigma_3 = 51.65$

08. For NCC soil, in CU test, $C_u = 0$ & $c' = 0$.

To find ϕ_u,

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2C_u \tan\left(45 + \frac{\phi_u}{2}\right).$$

$$200 + 150 = 200 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 0$$

$$\underline{\underline{\phi_u = 15.826^0}}$$

To find ϕ',

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right).$$

$$(200 + 150 - 75) = (200 - 75) \tan^2\left(45 + \frac{\phi'}{2}\right) + 0$$

$$\underline{\underline{\phi' = 22^0}}$$

10. $\sigma_d = 200 \text{ kN/m}^2$, $\sigma_3 = 100 \text{ kN/m}^2$

In an undrained test, $\phi_u = 0$.

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2C_u \tan\left(45 + \frac{\phi_u}{2}\right).$$

$$\sigma_1 = \sigma_3 + 2C_u.$$

$$C_u = \frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_d}{2} = \frac{200}{2} = \underline{\underline{100 \text{ kPa}}}$$

11. For dry sand, $c = 0$

$\sigma_3 = 50$ kPa, $\sigma_d = 100$ kPa.

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2C_u \tan\left(45 + \frac{\phi_u}{2}\right).$$

$$\sigma_1 = 50 \tan^2\left(45 + \frac{\phi_u}{2}\right)$$

$$150 = 50 \tan^2\left(45 + \frac{\phi_u}{2}\right) \Rightarrow \phi = \underline{\underline{30^0}}$$

12 $\phi = 37^\circ$, $\sigma_3 = 200\,kN/m^2$, $c = 0$.

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right).$$

$$200 + \sigma_d = 200\tan^2\left(45+\frac{37}{2}\right) + 0.$$

$$\sigma_d = \underline{\underline{604.56\ kPa}}$$

13. $\sigma_3' = 150\ kN/m^2$

Principle effective stress ratio, $\dfrac{\sigma_1'}{\sigma_3'} = 4.2$.

$$\sigma_1' = \sigma_3' \tan^2\left(45+\frac{\phi'}{2}\right) + 2c\tan\left(45+\frac{\phi'}{2}\right).$$

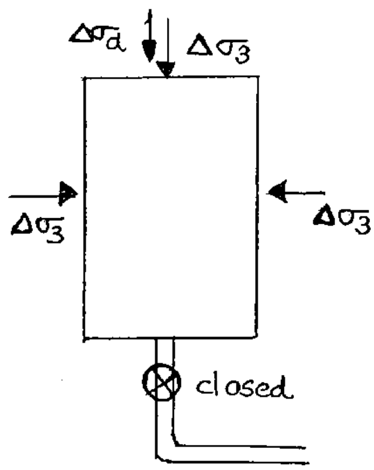
$$4.2 = \tan^2\left(45+\frac{\phi'}{2}\right).$$

$$\Rightarrow \phi' = \underline{\underline{37.97^\circ}}$$

14 $$\sigma_d = \sigma_1' - \sigma_3' = \sigma_3'\left(\frac{\sigma_1'}{\sigma_3'} - 1\right).$$

$$= 150\,(4.2-1) = \underline{\underline{480\ kPa}}$$

→ Skempton's Pore Pressure Parameters (A & B)



$$\boxed{\Delta u_3 = B\,\Delta\sigma_3}$$

$B = 0$; for dry soil.

$B = 1$; for saturated soil.

$$0 \leq B \leq 1$$

$$\boxed{\Delta u_d = AB\,\Delta\sigma_d}$$

At failure, $A \rightarrow A_f$.

- A_f is −ve for dense sand / OCC.
- A_f is +ve for loose sand / NCC

$$\text{Total, } \Delta u = \Delta u_3 + \Delta u_d$$

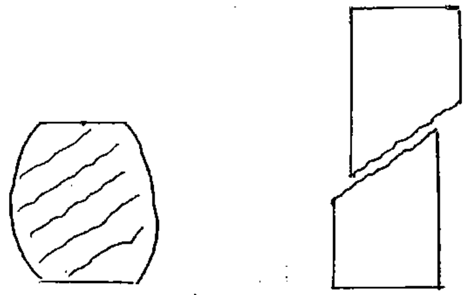
$$= B\Delta\sigma_3 + AB\,\Delta\sigma_d.$$

$$\therefore \quad \boxed{\Delta u = B(\Delta\sigma_3 + A\,\Delta\sigma_d)}$$

→ Shear Failures :

(i) Brittle Failure.

(ii) Plastic Failure.



Plastic Failure. Brittle Failure

■ Quick sand is due to upward seepage; liquefaction is due to compaction caused by vibrations

09. $\Delta\sigma_3 = 100 - 0 = 100$ kN/m².

$\Delta u_3 = 10 - (-60) = 70$ kN/m².

$$\Delta u_3 = B\,\Delta\sigma_3.$$

$$\therefore B = \frac{70}{100} = \underline{\underline{0.7}}$$

-60 } 1st stage (Consolidatio
10
2nd stage (Shearing) { -70

$\Delta\sigma_d = 500$ kN/m².

$\Delta u_d = -70 - 10 = -80$ kN/m².

$$\Delta u_d = AB\,\Delta\sigma_d.$$

$$-80 = A \times 0.7 \times 500$$

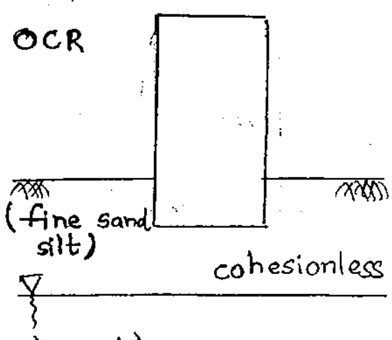
$$\therefore A = \underline{\underline{-0.23}}$$

B → depends on degree of saturation, S_r

A → depends on over consolidation ratio, OCR

→ Liquefaction:

$$S = \sigma' \tan\phi' = (\sigma - u)\tan\phi'$$



Due to vibrations caused by earthquakes or pile drivings, soil gets compacted and u↑.

If $u = \sigma$, $S = 0$ (soil behaves as a liquid)

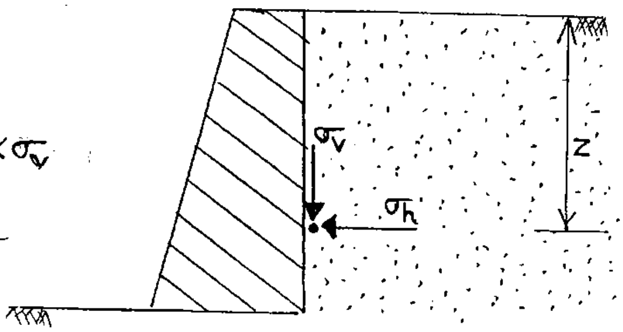
20th Sept,
SATURDAY

14. EARTH PRESSURE

$\sigma_v = \gamma z$

Lateral Earth Pressure, $\sigma_h = k\sigma_v$

where K → coeffecient of lateral earth pressure.



$$\boxed{K = \frac{\sigma_h}{\sigma_v}}$$

→ Types of Lateral Earth Pressures.

1. At-rest Earth pressure (P_o)
2. Active Earth pressure. (P_a)
3. Passive Earth pressure. (P_p)

* At-rest Earth Pressure :-

- It arises when there is no movement of wall.
- No yielding of soil.
- elastic equilibrium. ; theory of elasticity is used to find σ_h.

At rest earth pressure, $P_o = k_o \cdot \sigma_v$

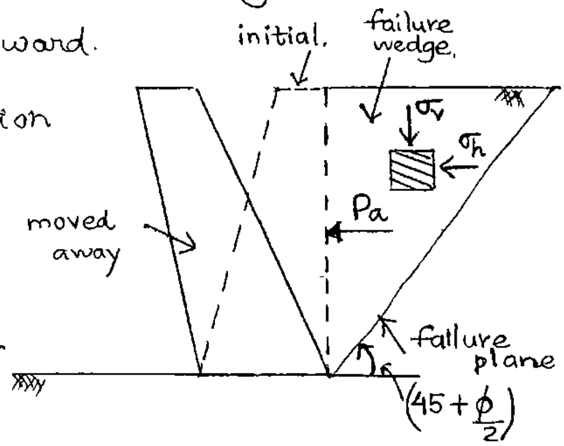
where K_o → coeffecient of at-rest earth pressure.

$$K_o = \frac{\mu}{1-\mu} \; ; \; \mu \rightarrow \text{poisson's ratio of soil.}$$

also $K_o = 1 - \sin\phi$; for cohesionless soils.

* Active Earth Pressure.

- It arises when the wall moves away from backfill.
- Failure wedge moves downward.
- It is a plastic eqbm condition

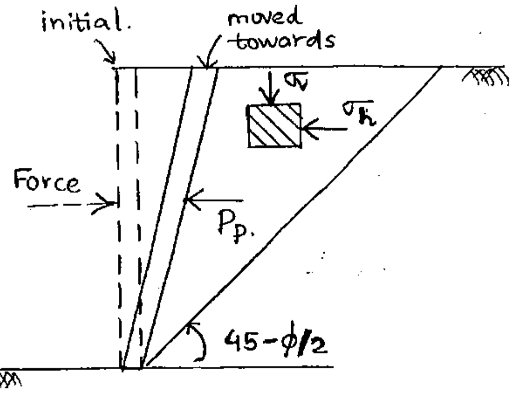


Here $\sigma_v = \sigma_1$ & $\sigma_h = \sigma_3$

Failure plane makes an angle of $(45+\phi/2)$ with Major principal plane.

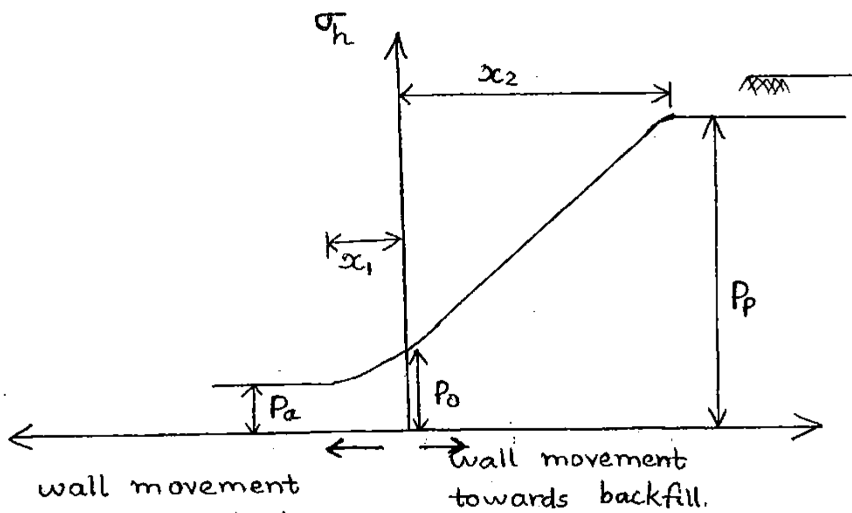
* Passive Earth Pressure

- It arises when the wall moves towards the backfill.
- Failure wedge moves upwards.
- It is a plastic eqbm condition.



Here $\sigma_v = \sigma_3$ $\sigma_h = \sigma_1$

22nd Sept, MONDAY

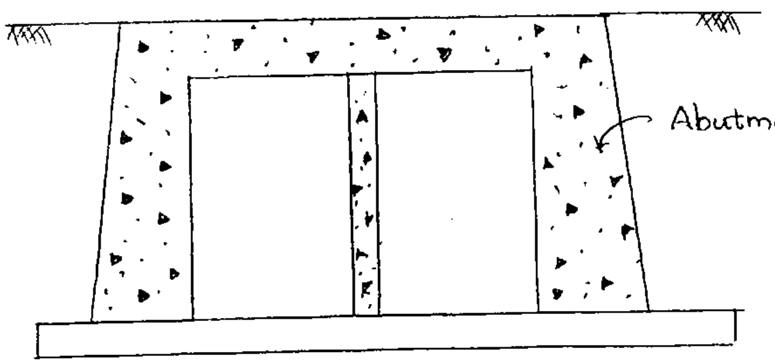


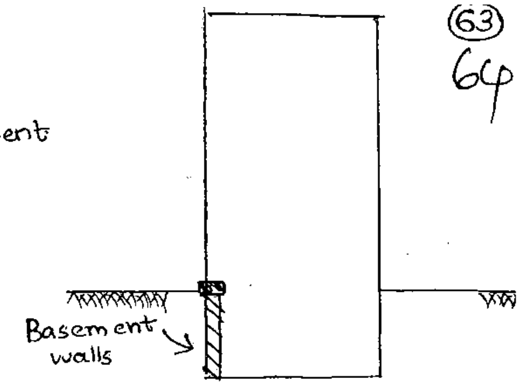
- $P_p > P_o > P_a$
- $x_1 < x_2$

→ Practical Applications:

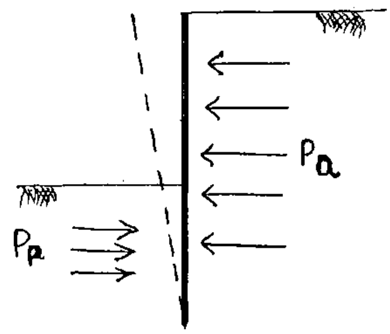
(i) For design of ordinary retaining wall. – active pressure is used

(ii) For design of bridge abutments and basement walls – at rest earth pressure is used





(iii) For design of sheet piles – both active & passive pre. used.



→ Rankine's Theory :

– to find P_a & P_p.

* Assumptions :-

(i) Soil is dry and cohesionless.

(ii) The back of the wall is vertical and smooth.

(iii) Plastic equilibrium.

Plastic equilibrium equation :-

$$\sigma_1 = \sigma_3 \tan^2\alpha_f + 2c\tan\alpha_f.$$

for cohesionless soil,

$$\sigma_1 = \sigma_3 \tan^2\alpha_f.$$

In active case, $\sigma_1 = \sigma_v$ & $\sigma_3 = \sigma_h$.

$$\therefore \sigma_v = \sigma_h \tan^2\alpha_f.$$

$$\sigma_h = \frac{\sigma_v}{\tan^2\alpha_f}$$

or $$\boxed{P_a = K_a \sigma_v}$$

where $K_a \rightarrow$ coeffecient of active earth pressure. $\left(= \frac{1}{\tan^2\alpha_f} \right)$

$$K_a = \frac{1}{\tan^2\alpha_f} = \frac{1}{\tan^2(45+\frac{\phi}{2})} = \tan^2(45-\phi/2)$$

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

Similarly,

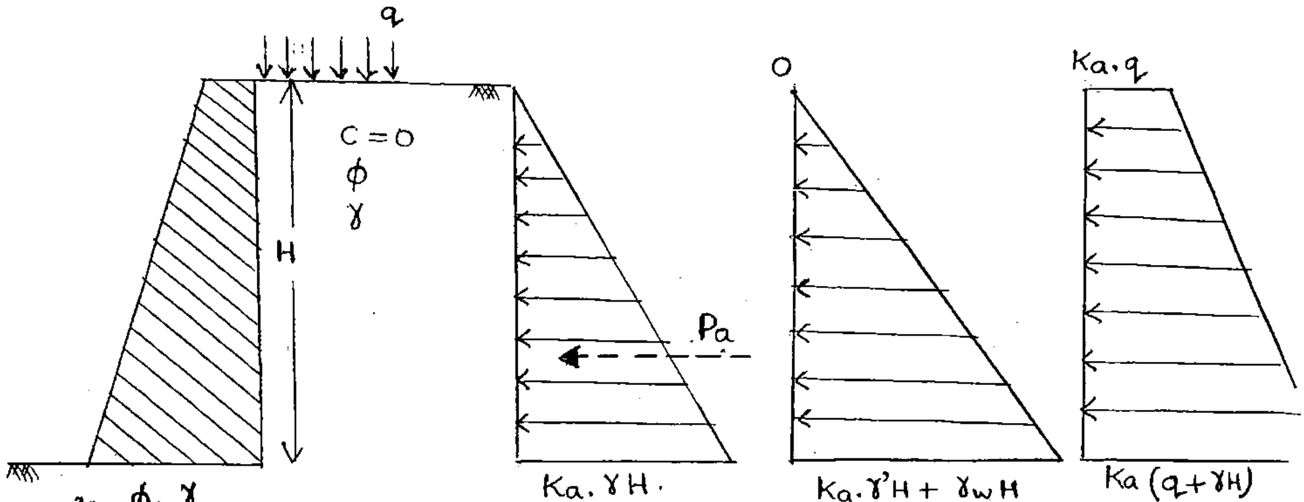
$$\boxed{P_p = K_p \cdot \sigma_v}$$

$K_p \rightarrow$ coeffecient of passive earth pressure.

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

$P_o = k_o \, \sigma_v \rightarrow$ for both cohesive & cohesionless soils

$$\left.\begin{aligned} P_a &= k_a \, \sigma_v \\ P_p &= k_p \, \sigma_v \end{aligned}\right\} \rightarrow \text{cohesionless soils}$$



case 1: ϕ, γ

■ P_a – diagram

At top; $\sigma_v = 0$

$P_a = K_a \sigma_v = 0.$

At bottom, $\sigma_v = \gamma H.$

$P_a = K_a \gamma H$

Let P_a = total active force.

= area of pressure diagram

$P_a = \dfrac{K_a \cdot \gamma H^2}{2}$; at $\dfrac{H}{3}$ from base

case 2: ϕ, γ_{sat}, <u>WT on ground</u>.

At top; $P_a = 0$

At bottom; $\sigma_v' = \gamma' H$

$$\boxed{P_a = K_a \sigma_v' + \gamma_w H}$$

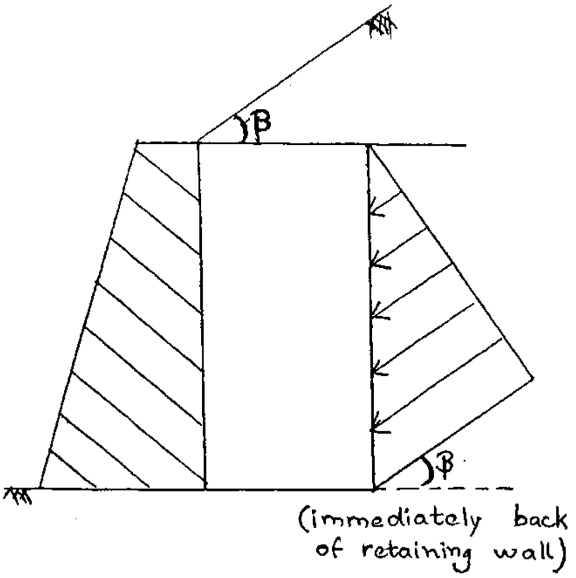
Case 3: surcharge loading, q

At top; $\sigma_v = q$.

$$P_a = K_a \sigma_v = K_a \cdot q.$$

At bottom; $\sigma_v = q + \gamma H$

$$P_a = K_a \sigma_v = K_a (q + \gamma H).$$



$$P_a = k_a \cdot \sigma_v$$
$$P_p = k_p \cdot \sigma_v.$$

$K_a = \cos\beta \left(\dfrac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right)$
$K_p = \cos\beta \cdot \left(\dfrac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right)$

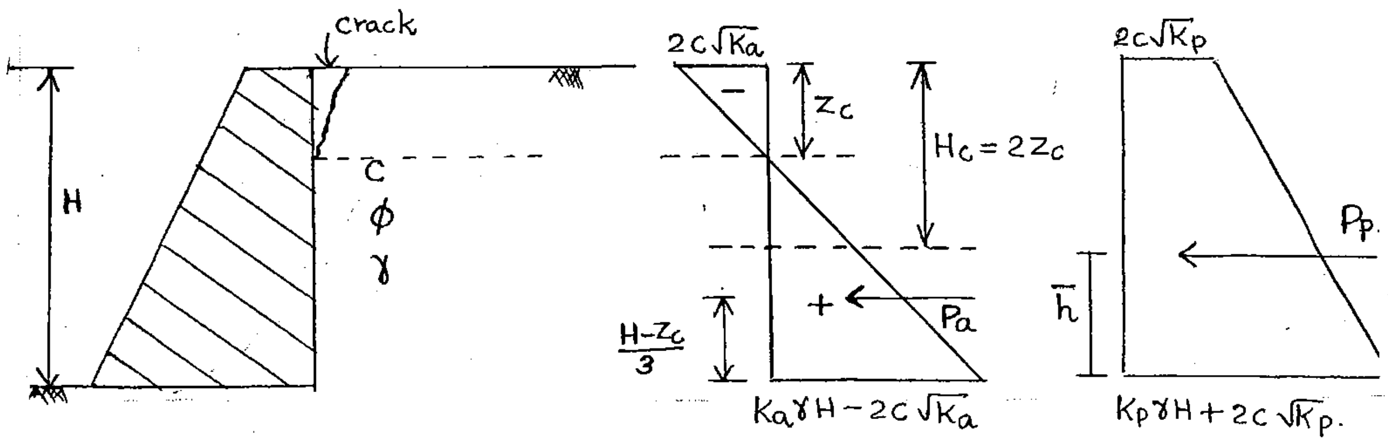
Here $K_p \neq \dfrac{1}{K_a}$.

→ $C-\phi$ Soils

$P_a = K_a \cdot \sigma_v - 2c\sqrt{K_a}$
$P_p = K_p \cdot \sigma_v + 2c\sqrt{K_p}$

⇒ BELL'S EQUATIONS

- Cohesion decreases active pressure but increases passive pressure



At top; $\sigma_v = 0$

$$P_a = K_a \sigma_v - 2c\sqrt{K_a}$$
$$= -2c\sqrt{K_a} \text{ (tension)}$$

At bottom; $\sigma_v = \gamma H$.

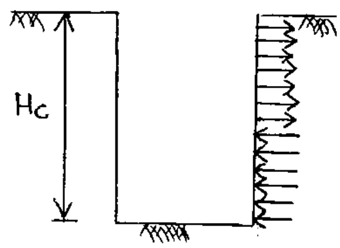
$$P_a = K_a \gamma H - 2c\sqrt{K_a}$$

Z_c : depth of tension zone (or) depth of tension crack

* To find Z_c :-

At a depth of Z_c,

$$P_a = K_a \gamma Z_c - 2c\sqrt{K_a}$$
$$0 = K_a \gamma Z_c - 2c\sqrt{K_a}$$
$$\therefore \quad Z_c = \frac{2c}{\gamma\sqrt{K_a}}$$
$$= \frac{2c}{\gamma}\tan(45 + \phi/2)$$
$$= \frac{2c}{\gamma} \text{ ; for pure clay } (\phi = 0)$$



H_c : critical height or depth of unsupported vertical trench.

$$* \quad H_c = 2Z_c = \frac{4c}{\gamma\sqrt{K_a}} = \frac{4c}{\gamma}\tan\left(45 + \frac{\phi}{2}\right)$$
$$= \frac{4c}{\gamma} \text{ ; for pure clay}$$

* To find total active force, P_a

ⓐ. Before formation of crack.

$$P_a = \int_0^H p_a \cdot dz = \text{total algebraic sum of area of pressure diagram.}$$

$$\boxed{P_a = K_a \cdot \frac{\gamma H^2}{2} - 2c\sqrt{K_a} \cdot H}$$

ⓑ After formation of crack.

$$P_a = \int_{Z_c}^{H} p_a \cdot dz = \text{area of +ve portion only.}$$

$$\boxed{P_a = K_a \cdot \frac{\gamma H^2}{2} - 2c\sqrt{K_a}\,H + \frac{2c^2}{\gamma}}$$

* To find total Passive fore, P_p.

$P_p = K_p \sigma_v + 2c\sqrt{K_p}$.

At top; $\sigma_v = 0$. $\therefore P_p = 2c\sqrt{K_p}$

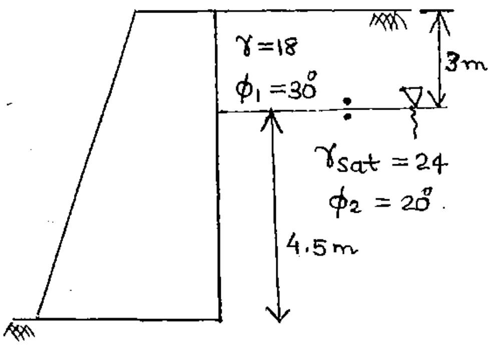
At bottom; $\sigma_v = \gamma H$. $\therefore P_p = K_p \gamma H + 2c\sqrt{K_p}$

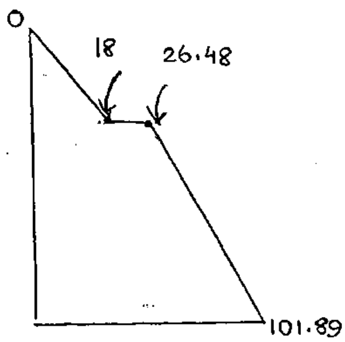
Total passive pressure force, P_p = area of pressure diagram.

$$\boxed{P_p = K_p \gamma \frac{H^2}{2} + 2c\sqrt{K_p}\,H}$$

Q-82

Sol. $H = 7.5\,m$;





$K_{a_1} = \frac{1-\sin\phi_1}{1+\sin\phi_1} = \underline{\underline{0.33}}$

$K_{a_2} = \frac{1-\sin 20}{1+\sin 20} = \underline{\underline{0.49}}$

At top; $\sigma_v = 0$

$P_a = K_{a_1} \cdot \sigma_v = 0$.

At 3m depth, $\sigma_v = 18 \times 3 = 54$

a) Just above 3m depth

$P_a = K_{a_1} \sigma_v = \underline{\underline{18}}$ kPa

b) Just below 3m depth,

$P_a = K_{a_2} \sigma_v = 0.49 \times 54$

$= 26.48$ kPa

At bottom; $\sigma_v = 18 \times 3 + (24 - 9.81)\,4.5$

$= 117.855$ kPa

$P_a = K_{a_2} \sigma_v' + \gamma_w h$

$= 0.49 \times 117.855 + 9.81 \times 4.5$

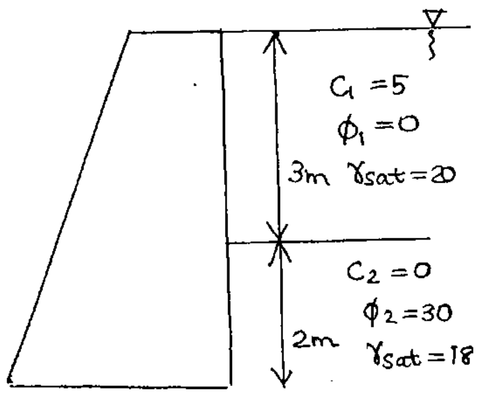
$= \underline{\underline{101.89}}$ kPa

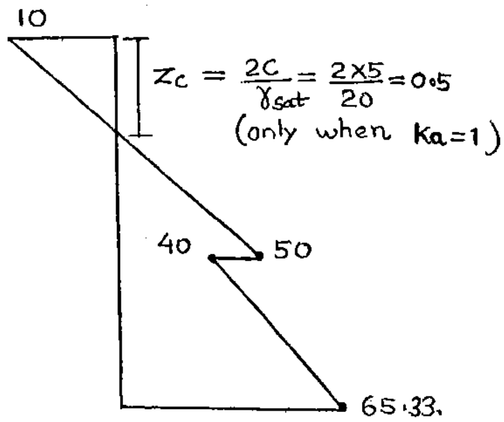
$$\text{Total force, } P_a = \frac{1}{2} \times 18 \times 3 + \left(\frac{26.46 + 101.89}{2}\right) \times 4.5$$

$$= \underline{\underline{315.832}} \text{ kN/m.}$$

- Area of Pressure force diagram = Force per unit length.

02.





$$Ka_1 = \frac{1 - \sin\phi}{1 + \sin\phi} = \underline{\underline{1}}$$

$$Ka_2 = \frac{1 - \sin 30}{1 + \sin 30} = \underline{\underline{\frac{1}{3}}}$$

At top;

$\sigma_v = 0$

$P_a = Ka_1 \sigma_v - 2c_1 \sqrt{Ka_1}$.

$= 0 - 2 \times 5 \times 1 = -10$ kPa. (tension).

At 3m depth;

$\sigma_v' = (20 - 10) 3 = 30$ kPa.

a) Just above 3m depth,

$P_a = \sigma_v' Ka_1 - 2C_1 \sqrt{Ka_1} + \gamma_w h$.

$= 30 \times 1 - 2 \times 5 + 10 \times 3 = 50$ kPa.

b) Just below 3m depth,

$P_a = \sigma_v' Ka_2 - 2C_2 \sqrt{Ka_2} + \gamma_w h$.

$= 30 \times \frac{1}{3} - 0 + 30 = 40$ kPa.

At bottom; $\sigma_v' = 10 \times 3 + 8 \times 2 = 46$.

$P_a = 46 \times \frac{1}{3} + 10 \times 5$.

$= \underline{\underline{65.33}}$ kPa.

To find Z_c :

At a depth of Z_c, $P_a = 0$.

$$P_a = K_{a1}\,\gamma' Z_c - 2C_1\sqrt{K_{a1}} + \gamma_w Z_c$$

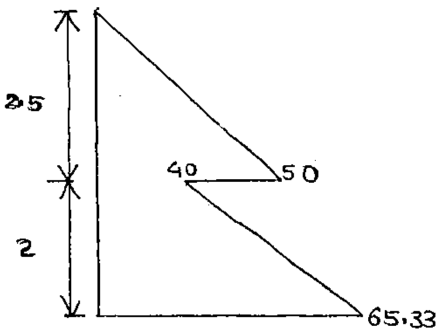
$$0 = 1 \times 10\,Z_c - 2 \times 5 \times \sqrt{1} + 10\,Z_c.$$

$$Z_c = \frac{10}{20} = \underline{\underline{0.5}}\ m$$

OR

From similar triangles,

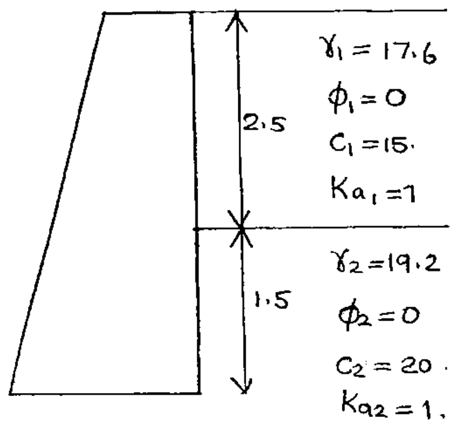
$$\frac{10}{Z_c} = \frac{50}{3 - Z_c} \Rightarrow Z_c = \underline{\underline{0.5\,m}}$$

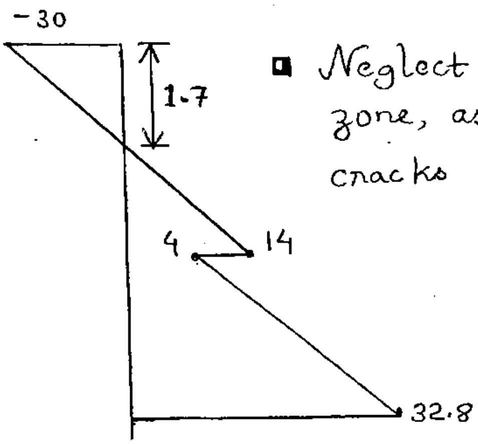


Neglecting tension zone :

$$P_a = \frac{1}{2} \times 50 \times 2.5 + 2\left(\frac{40 + 65.33}{2}\right) = \underline{\underline{167.8}}\ kN/m$$

03.





- Neglect tension zone, as tension cracks develop.

At top : $\sigma_v = -2C\sqrt{K_{a1}} = \underline{\underline{-30}}$

$$P_a = K_{a1}\,\sigma_v = -30\ kPa.$$

At 2.5 m depth :

$$\sigma_v^* = \gamma^* \times 2.5$$

$$= 17.6 \times 2.5 = 44\ kPa.$$

a) Just above,

$$P_a = K_{a1}\sigma_v^* - 2C_1\sqrt{K_{a1}}$$

$$= 44 - 2 \times 15 = 14\ kPa.$$

b) Just below,

$$P_a = 1 \times 44 - 2 \times 20 = 4\ kPa.$$

At bottom :

$$\sigma_v = 17.6 \times 2.5 + 19.2 \times 1.5.$$

$$= 72.8.$$

$$P_a = 72.8 - 2 \times 20$$

$$= \underline{\underline{32.8}}\ kPa.$$

Total active force, P_a

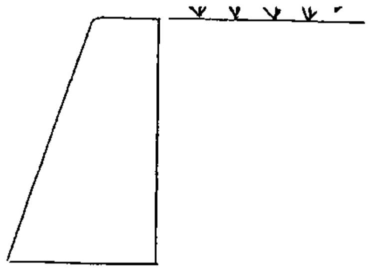
$$= (2.5 - 1.7) \times 14 \times 0.5 + 0.5(4 + 32.8) \times 1.5$$

$$= \underline{\underline{33.168}}\ kPa$$

04. $P_a = K_a \sigma_v - 2c\sqrt{K_a}$.

$0 = K_a . q - 2c\sqrt{K_a}$.

$q = \frac{2c}{\sqrt{K_a}} = 2c \tan\alpha_f$.

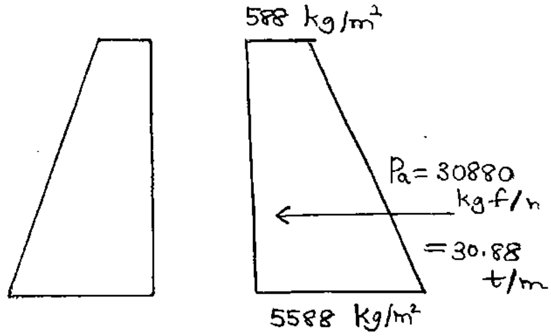


05. When there is no surcharge,

at bottom, $P_a = K_a \gamma H$.

$5000 = K_a . 1700 \times 10$.

$K_a = 0.294$.



If there is surcharge,

at ~~bottom,~~ top $P_a = K_a . \sigma_v$.

$= K_a . q = 0.294 \times 2000$

$= \underline{588}$ kg/m².

at bottom, $\sigma_v = q + \gamma H$.

$P_a = K_a . \sigma_v$.

$= K_a (q + \gamma H) = \underline{\underline{5588}}$ kg/m².

Maximum earth pressure $= \underline{\underline{5588}}$ kg/m²

Resultant force on the wall $= \frac{1}{2}(5588 + 588) \times 10$

$= 30880$ kgf/m

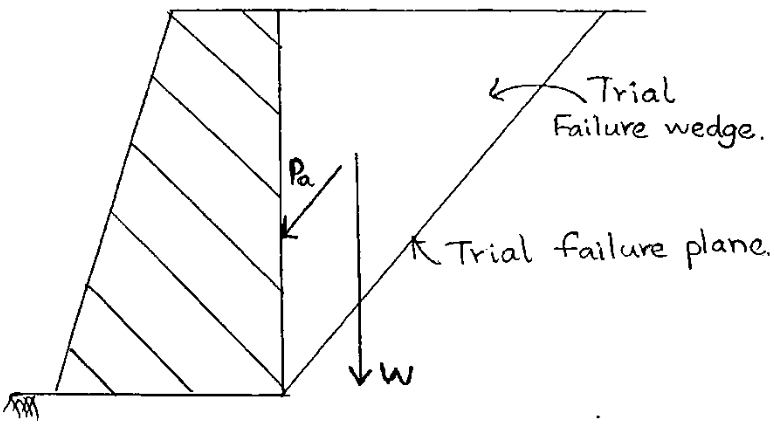
$= \underline{\underline{30.880}}$ t/m

→ Coulomb's Theory

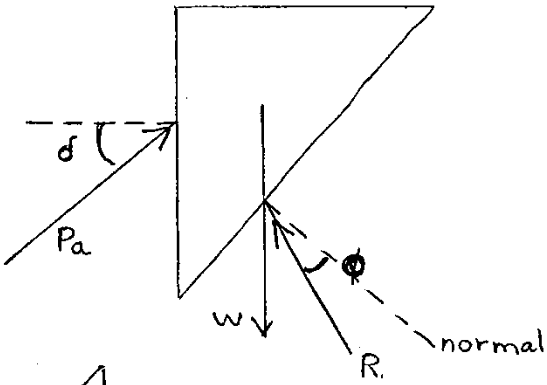
* Assumptions:

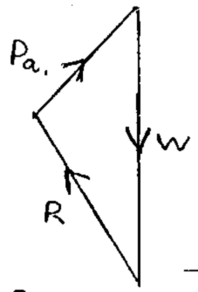
(i) Soil is dry and cohesionless.

(ii) Back of the wall is <u>rough</u>

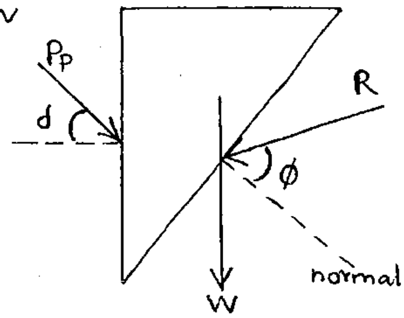


FBD of Failure wedge.





force triangle



$\delta \rightarrow$ angle of wall friction.

Coulomb's theory is a graphical trial & error method of computing lateral earth pressures (P_a & P_p).

NOTE: Effect of Wall friction :-

- The wall friction reduces active pressure but increases passive pressure; both are advantageous.

- For RCC retaining walls, Rankine's Theory can be used. For stone masonry retaining walls, Coulomb's Theory is used.

* Rebhan's Method & Culman's Method – graphical method of computing P_a & P_p using Coulomb's theory.

8-81

16. When soil is compacted, $\phi \uparrow \Rightarrow K_a \downarrow$

$$P_a = K_a \gamma H \quad (\downarrow).$$

$K_a \downarrow > \gamma \uparrow$

26. Cohesive soils are poor for backfilling as they cause more lateral pressure. due to following reasons:

(i) for clays ϕ is less, Hence K_a is more.

(ii). Swelling of clays.

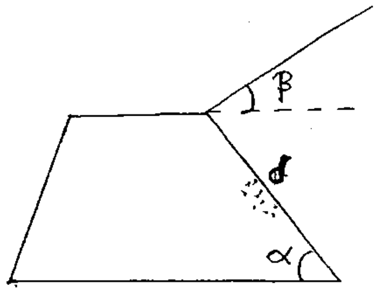
(iii) Clays have poor drainage properties.

(iv) Compaction of clays behind the wall is difficult.

For backfilling behind the walls, cohesionless soils like gravel and sand are best.

↑ K_a due to above factors is more than the ↓ K_a due to cohesion. ($P_a = K_a \sigma_v - 2c\sqrt{K_a}$).

→ Solution for Coulomb's Theory.



Force, $P_a = K_a \dfrac{\gamma H^2}{2}$.

where K_a depends on $\alpha, \beta, \delta, \phi$

- For vertical wall and horizontal backfill and if $\delta = \phi$,

then, $K_a = \dfrac{\cos\phi}{(1+\sqrt{2}\sin\phi)^2}$; $K_p = \dfrac{\cos\phi}{(1-\sqrt{2}\sin\phi)^2}$

23rd Sept,
TUESDAY

15. STABILITY OF SLOPES

→ Forces which cause failure of Slopes:

(i) Gravitational Force.

(ii) Seepage force.

(iii) Earthquake Force.

(iv) Construction equipment loads.

→ Types of Slopes:

(i) Infinite slope. Eg: mountain slope.

(ii) Finite slope Eg: embankment of roads, earthen dams, canals etc.

→ Types of Slope Failures:

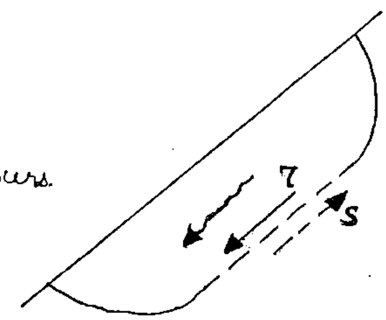
(i) Translational failure.

(ii) Rotational Failure.

(iii) Wedge Failure.

(iv) Compound Failure.

* Translational Failure.



If $\tau > s$; translational failure occurs.

∴ to avoid failure, τ must be kept less than s.

$$\boxed{FOS = \frac{s}{\tau}}$$

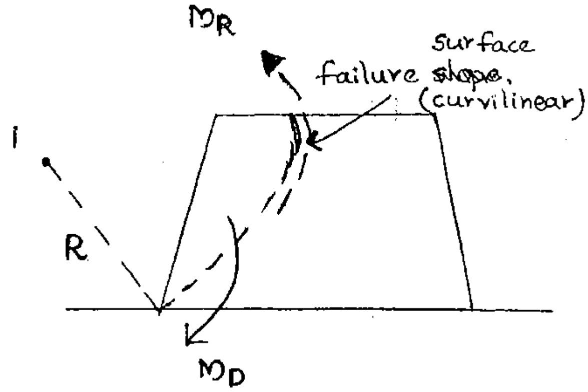
if

FOS > 1 ; it is safe.

FOS < 1 ; unsafe.

FOS = 1 ; critical

* Rotational Failure



M_D = driving moment.

M_R = resisting moment.

If $M_D > M_R$, failure occurs.

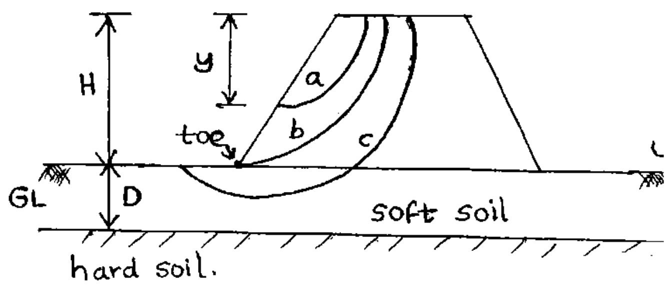
$$\therefore \boxed{FOS = \frac{M_R}{M_D}}$$

– types of rotational failures :-

a) Face failure

b) Toe failure.

c) Base failure.



Depth factor, $D_F = \frac{H+D}{H}$

For base failure, $D_F > 1$ (when there is soft soil)

For toe failure, $D_F = 1$ (when there is no soft soil).

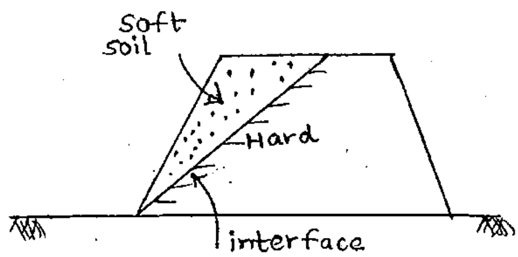
For face failure, $D_F < 1$

Depth factor, $D_F = \frac{y}{H}$.

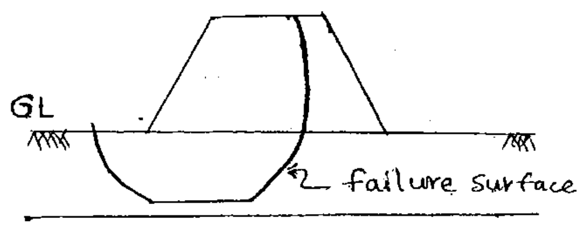
where $y \rightarrow$ vertical depth of point where failure surface passes as shown.

* Wedge Failure

The soft soil above the interface b/w soft soil and hard soil will fail as a wedge.



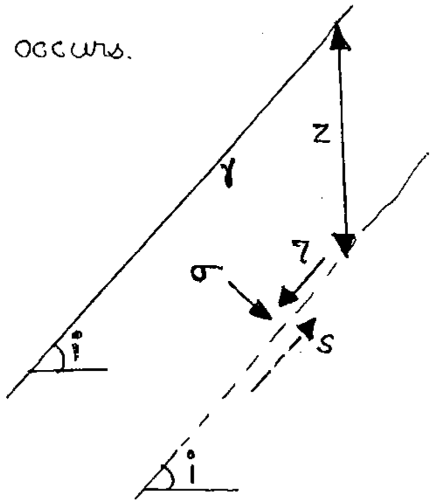
* Compound Failure.



→ Infinite Slope:

- generally, translational failure occurs.

$$Fos = \frac{S}{\tau}$$



$$S = c + \sigma \tan\phi$$

$$\sigma = \gamma z \cos^2 i$$

$$\tau = \gamma z \cos i . \sin i.$$

* Infinite Slope in c-φ soil:-

a) Dry or Partially Saturated Soil.

$$Fos = \frac{c + \sigma \tan\phi}{\tau} = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i . \sin i};$$ (at a depth

b) Fully submerged soil. (mountain in ocean).

$$FOS = \frac{c' + \gamma' z \cos^2 i . \tan\phi'}{\gamma' z \cos i . \sin i}$$

c) If there is seepage parallel to slope. (rain water seeping).

$$FOS = \frac{c' + \gamma' z \cos^2 i . \tan\phi'}{\gamma_{sat.} z . \cos i . \sin i}$$

* Infinite slope in Cohesionless soils (c = 0)

imp a) Dry or Partially Saturated Soil.

$$FOS = \frac{\tan\phi}{\tan i}$$ (If $i > \phi$; it fails)

b) For fully submerged slope.

$$FOS = \frac{\tan\phi'}{\tan i}$$

c) Seepage parallel to slope,

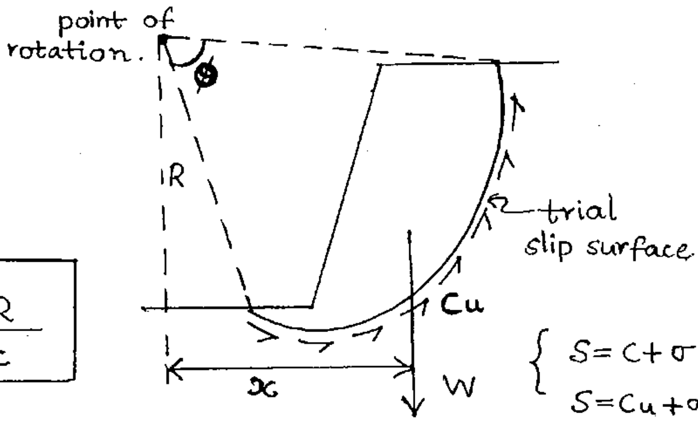
$$FOS = \left(\frac{\gamma'}{\gamma_{sat}}\right) \frac{\tan\phi'}{\tan i}$$

→ Finite Slope

- generally, rotational failure occurs.
- methods of analysis:
  1. $\phi_u = 0$ Analysis.
  2. Method of slices.
  3. Bishop's method.
  4. Friction circle method.
  5. Stability number method.

* $\phi_u = 0$ Analysis :

- suitable only for undrained saturated clays.
- graphical method based on trial & error.



$\therefore M_D = W \cdot x$

$M_R = C_u \cdot \hat{L} \times 1 \times R$

$$\boxed{FOS = \frac{M_R}{M_D} = \frac{C_u \hat{L} R}{W \cdot x}}$$

$\left\{ \begin{array}{l} S = C + \sigma \tan\phi \\ S = C_u + \sigma \tan \\ \Rightarrow S = C_u \end{array} \right\}$

$\hat{L} \rightarrow$ arc length.

$$\hat{L} = R\theta \cdot \frac{\pi}{180}$$

$W \rightarrow$ weight of trial failure wedge.

$W = a \times 1 \times \gamma$; $a \rightarrow$ area of trial failure wedge.

↳ use γ' for submerged slope. (canal running full)

use γ_{sat} for sudden drawdown conditi

$FOS \propto \frac{1}{W}$; but $W \propto \gamma$

$$\Rightarrow FOS \propto \frac{1}{\gamma}$$

$\therefore$ for a canal slope, the critical condition is sudden drawdown condition.

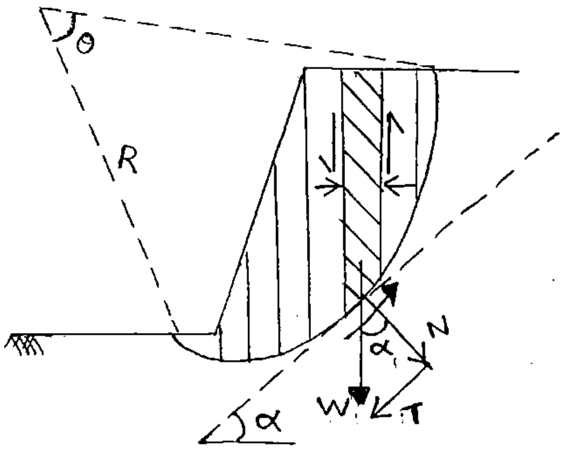
- Canal will have more FOS when running full compard to sudden drawdown condition.

During sudden drawdown, γ increases to γ_{sat} and $\therefore$ FOS decreases.

- Among the various trials, the trial slip surface which gives min. factor of safety is called '<u>Critical Slip Surface</u>' and that min. FOS is taken as the FOS of the slope.

* Method of Slices:

- used for all soils.
- trial and error.
- forces acting on the sides of slices <u>are neglected</u>.



$$\boxed{F = \frac{c\hat{L} + \Sigma N \tan\phi}{\Sigma T}}$$

$N = W\cos\alpha$

$T = W\sin\alpha$

- When there is seepage,

$$\boxed{F = \frac{c'\hat{L} + \Sigma (N-U)\tan\phi'}{\Sigma T}}$$

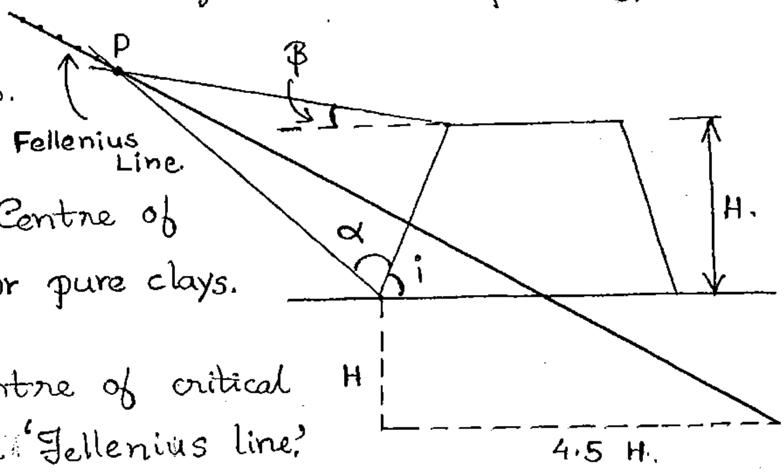
$\Sigma U \rightarrow$ sum of neutral forces.

- Fellenius method to identify Critical Slip Circle:-

$\alpha, \beta \rightarrow$ direction angles. (depend on i)

Point 'P' represents Centre of Critical Slip circle for pure clays.

For c-ϕ soils, the centre of critical slip circle lies on the 'Fellenius line.'



* Bishop's Method.

- forces acting on the sides of slices are also <u>considered</u>
- trial and error

* Friction Circle Method.

- trial and error

c = cohesion

C_m = mobilised cohesion.

ϕ = angle of internal friction.

ϕ_m = mobilised angle of internal friction.

S = shear strength.

S_m = mobilised shear strength.

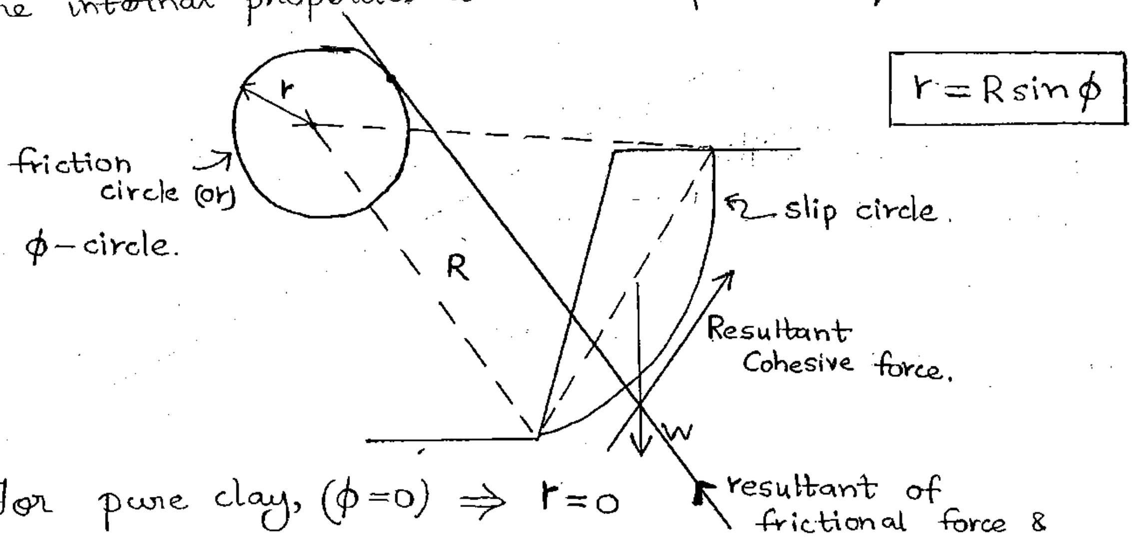
⊙ 'Mobilised' means actually developed to keep system (wedge) in eqbm.

FOS wrt cohesion, $F_c = \dfrac{c}{C_m}$

FOS wrt friction, $F_\phi = \dfrac{\tan\phi}{\tan\phi_m}$

FOS wrt shear strength, $F = \dfrac{S}{S_m} = \dfrac{c + \sigma\tan\phi}{C_m + \sigma\tan\phi_m}$

σ remains the same as its related to weight, whereas c & ϕ are internal properties which develops as required.



$$\boxed{r = R\sin\phi}$$

⊙ For pure clay, $(\phi = 0) \Rightarrow r = 0$

⊙ Resultant of frictional force and normal reaction will be tangential to the friction circle.

* Taylor's Stability Number Method.

- developed based on Friction Circle method.
- $\boxed{\text{Stability number, } S_n = \frac{C}{F_c \cdot \gamma H}}$

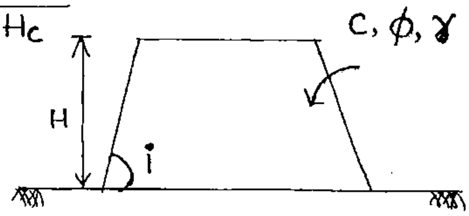
where $F_c = \frac{C}{C_m} = \frac{H_c}{H}$; $H_c \rightarrow$ critical height.

$H_c \rightarrow$ max. permitted height which can provided for given soil against failure.

$H \rightarrow$ safe or actual height.

$$\Rightarrow S_n = \frac{C_m}{\gamma H} = \frac{C}{\gamma H_c}$$

$$\therefore F_c = \frac{C}{S_n \gamma H}$$



⊙ S_n depends on i & ϕ_m (or) i & depth factor

Knowing F_ϕ & ϕ value, ϕ_m can be calculated by:-

$$F_\phi = \frac{\tan\phi}{\tan\phi_m}.$$

Max. value of $S_n = 0.261$ (for $i = 90^\circ$ & $\phi_m = 0$)

⊙ Except for cohesionless soil, this method is suitable for all soils. (for cohesionless, $c = 0 \Rightarrow F_c = 0$; meaningless)

⊙ For submerged slope, use γ'

For sudden draw down condition, use γ_{sat} & $\phi_m = \phi_w$

$$\phi_w = \text{weighted friction angle} = \frac{\gamma'}{\gamma_{sat}} \cdot \phi$$

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P-89

01. $F = \dfrac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i . \sin i} \rightarrow$ for $c-\phi$ soil.

i can be greater than ϕ.

03. $S_n = \dfrac{c}{F_c \gamma H}$ $\Rightarrow F_c \propto \dfrac{1}{\gamma}$

04. $F = \dfrac{\gamma'}{\gamma_{sat}} \dfrac{\tan\phi'}{\tan i}$

$1.5 = \dfrac{19-9.81}{9.81} \dfrac{\tan 36}{\tan i}$

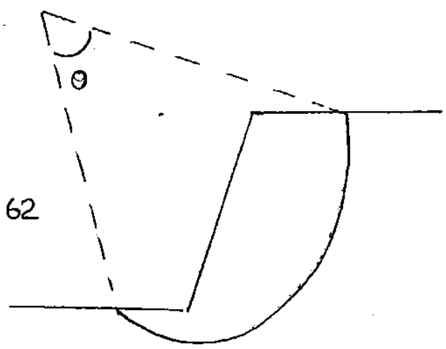
$i = 13.18^0$

If no seepage,

$F = \dfrac{\tan\phi}{\tan i} \Rightarrow F = 3.10$

05. Ignoring the crack,

a) $\theta = 109 + 12 = 121^0$



$\hat{L} = R\theta \dfrac{\pi}{180} = 14.5 \times 121 \times \dfrac{\pi}{180} = 30.62$

$w = a \times 1 \times \gamma$

$= 110 \times 1 \times 18 = 1980$

$\bar{x} = 3.75$ m.

$F_{os} = \dfrac{c\hat{L}R}{wx} = \dfrac{27 \times 30.62 \times 14.5}{1980 \times 3.75} = 1.61$

b). $\theta = 109^0$,

$L = R\theta \times \dfrac{\pi}{180} = 27.585$

$w = a \times 1 \times \gamma$ $\quad \bar{x} = 3.75$ m.

$= (110 - 1.5) \times 18$

$= 1953$

$F = \dfrac{C_u \hat{L} R}{w\bar{x}}$

$= \dfrac{27 \times 27.585 \times 14.5}{1953 \times 3.75} = 1.47$

6. $H = 25m$, $C = 35$, $\phi = 15^\circ$

a) $\gamma = 20$.

$$F_\phi = \frac{\tan\phi}{\tan\phi_m}$$

$$1.5 = \frac{\tan 15}{\tan\phi_m} \Rightarrow \phi_m \approx 10^\circ$$

For $\phi_m = 10^\circ$, $S_n = 0.06$.

$$S_n = \frac{c}{F_c \gamma H}.$$

$$0.06 = \frac{35}{F_c \times 20 \times 25} \Rightarrow F_c = \underline{\underline{1.167}}$$

b) If $F_c = 1.5$, $F_\phi = ?$

$$S_n = \frac{c}{F_c \gamma H} = 0.05.$$

$\phi_m = 12.5^\circ$ (From table given).

$$F_\phi = \frac{\tan\phi}{\tan\phi_m} = \frac{\tan 15^\circ}{\tan 12.5^\circ} = \underline{\underline{1.208}}$$

7. Canal running full, (submerged condition)

a) $$S_n = \frac{c}{F_c \gamma' H}.$$

Since F_ϕ is not given, take $\phi_m = \phi$.

$\therefore \phi_m = 15^\circ \longrightarrow S_n = 0.083.$

$$S_n = \frac{c}{F_c \gamma' H}.$$

$$0.083 = \frac{1.4}{F_c \times 0.945 \times 5}$$

$\Rightarrow F_c = \underline{\underline{3.56}}$

b) For suddenly draw down,

$$S_n = \frac{C}{F_c\, \gamma_{sat} \cdot H}$$

$$\phi_m = \phi_w$$

$$\phi_w = \frac{\gamma'}{\gamma_{sat}} \phi \approx 7.5^0 \Rightarrow S_n = 0.122$$

$$\Rightarrow \quad 0.122 = \frac{1.4}{F_c \times 1.945 \times 5}$$

$$\therefore F_c = \underline{\underline{1.18}}$$

8. $$F_c = \frac{C}{C_m} = \frac{30}{22} = \underline{\underline{1.36}}$$

$$F_\phi = \frac{\tan\phi}{\tan\phi_m} = \frac{\tan 15}{\tan 12} = \underline{\underline{1.26}}$$

$$F = \frac{S}{S_m} = \frac{c' + \sigma' \tan\phi'}{C_m + \sigma' \tan\phi_m} = \frac{62.17}{47.5} = \underline{\underline{1.308}}$$

* To find F_ϕ when $F_c = 1$:

$$S_m = C_m + \sigma \tan\phi_m *$$

$$47.5 = \frac{c}{F_c} + 120 \frac{\tan\phi'}{F_\phi}$$

$$= \frac{30}{1} + \frac{120 \tan 15}{F_\phi}$$

$$\Rightarrow \quad F_\phi = \underline{\underline{1.829}}$$

Similarly, to find F_c when $F_\phi = 1$.

$$47.5 = \frac{c}{F_c} + \frac{120 \tan\phi'}{1.}$$

$$F_c = \underline{\underline{1.95}}$$

Q A granular soil possesses $\gamma_{sat} = 20\ kN/m^3$. If $\phi' = 35°$, and the designed FOS is 1.5, what is safe angle of slope for this soil when seepage occurs parallel to slope surface

$$F = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\phi'}{\tan i}$$

$$1.5 = \frac{10}{20} \frac{\tan 35}{\tan i}$$

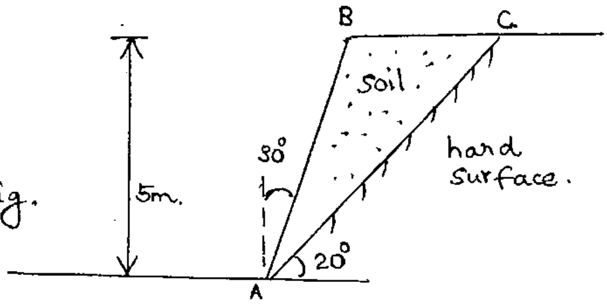
$$\Rightarrow i = \underline{\underline{13.14°}}$$

Q $c = 15\ kN/m^2$

$\phi = 20°$

$\gamma = 18\ kN/m^3$.

A slope is shown in the fig. If the properties of soil are as above, find the FOS against possible wedge failure at the inter face.



$$\boxed{F = \frac{CL + N\tan\phi}{T}}$$

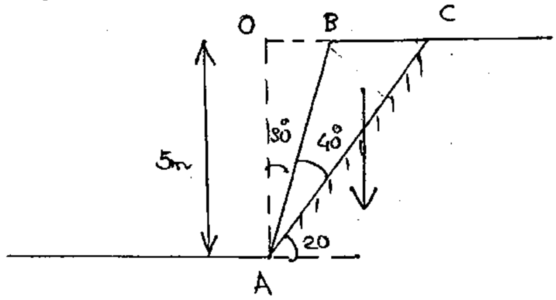
L = length of AC

N = normal component of weight, w ; $N = W\cos 20$

W → weight of wedge, ABC. $T = W\sin 20$

T = Tangential component of weight, w.

$\tan 70° = \frac{OC}{5}$.



OC = 13.73 m.

$\tan 30° = \frac{OB}{5}$

OB = 2.88 m.

BC = OC − OB = 13.73 − 2.88 = 10.85 m.

$$AC = \sqrt{5^2 + 13.73^2} = 14.61 \text{ m}$$

$$W = \underbrace{\left(\frac{1}{2} \times 10.85 \times 5\right)}_{\text{area of wedge}} \gamma = 488.25 \text{ kN}$$

$$N = 488.25 \cos 20 = 458.805$$

$$T = 488.25 \sin 20 = 166.99$$

$$F = \frac{CL + N\tan\phi}{T} = \frac{15 \times 14.61 + 458.805 \tan 20}{167}$$

$$= \underline{\underline{2.31}}$$

25th sept,
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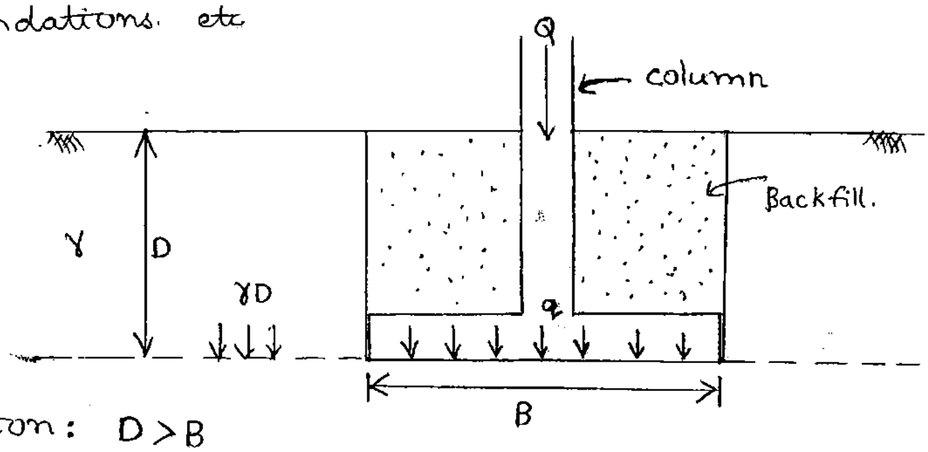
16. BEARING CAPACITY

* Shallow Foundation : $D \leq B$
  Eg: Spread footings
  Raft foundations. etc



* Deep Foundation: $D > B$
  Eg: Pile foundation
  Well foundation.

* Original overburden pressure due to self weight of soil
$$= \gamma D$$

Gross Pressure $= q$

Net Pressure, $q_n = q - \gamma D$

* Gross Ultimate BC of soil or Ultimate BC of soil $\Big\}$, q_u

Min. gross pressure required to cause shear failure of soils

* Net Ultimate BC of soil, $q_{nu} = q_u - \gamma D$.

Min. net pressure required to cause shear failure of soils.

* Net safe BC of soil, $q_{ns} = \frac{q_{nu}}{F}$ $(F = 3)$

* Gross safe BC of soil or Safe BC of soil } $q_s = q_{ns} + \gamma D$

* Net safe settlement pressure, q_{np}.

It is the max. net pressure which the ~~the~~ soil can carry without exceeding allowable settlement.

* Net Allowable BC of soil, q_{na} = Smaller of q_{ns} or q_{np}

$q_{ns} \rightarrow$ based on shear failure criteria

$q_{np} \rightarrow$ based on settlement criteria.

It is the net pressure at which soil neither fails in shear nor undergoes excessive settlement.

→ Condition to be satisfied for Design of Foundation

The external pressure on soil ≤ net allowable BC of so

$$\boxed{q_n \leq q_{na}}$$

* If footing is backfilled,

$$q_n \approx \frac{Q}{A}$$

where Q = column load.

A = area of footing

* If footing is not backfilled, (raft)

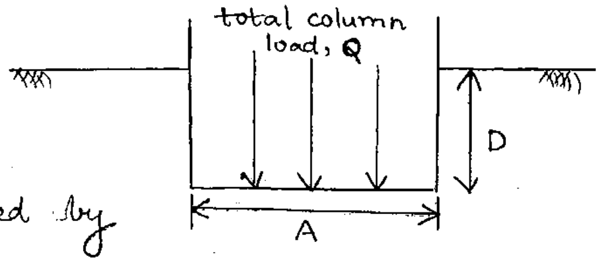
$$q_n \fallingdotseq \frac{Q}{A} - \gamma D$$

It is based on the assumption that self weight of concrete is equal to unit weight of soil ($\gamma_c = 25$; $\gamma_s = 20$)

→ Compensated Raft Foundation. (Floating Raft)

$$q_n = \frac{Q}{A} - \gamma D.$$

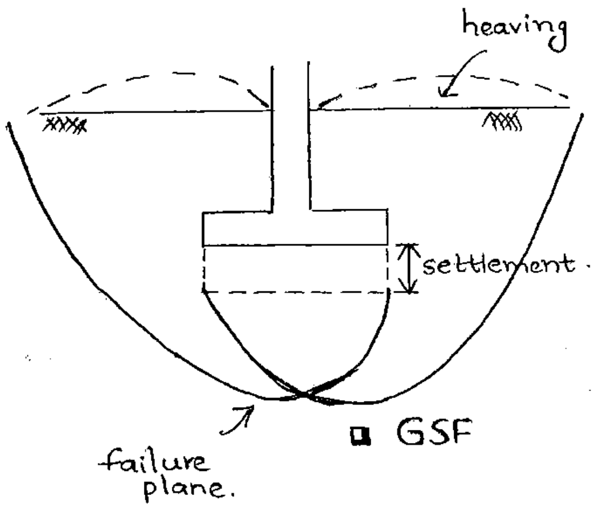
If $\gamma D = \frac{Q}{A}$; $q_n = 0$



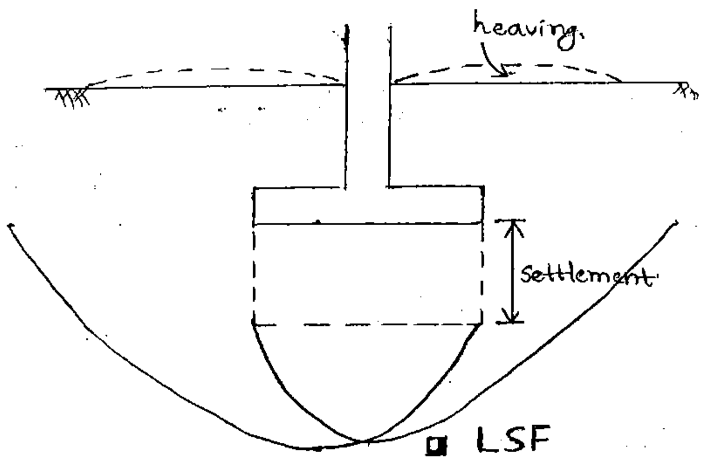
Pressure applied is just balanced by pressure released.

→ Types of Shear Failure:

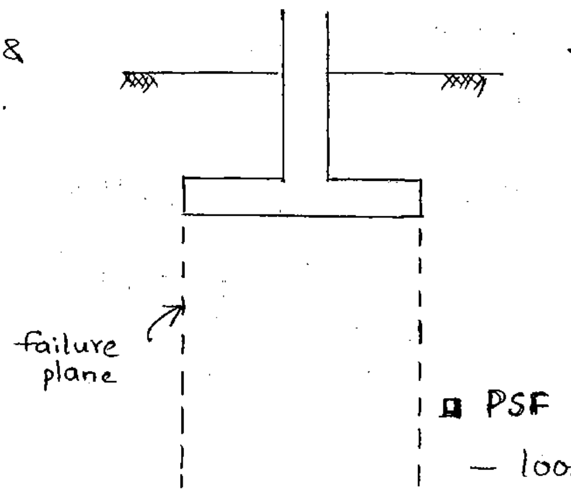
1. General Shear Failure (GSF)
2. Local Shear Failure (LSF).
3. Punching Shear Failure (PSF).



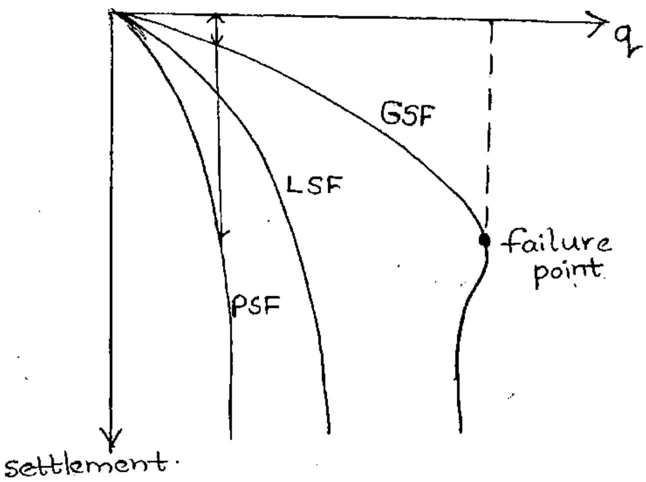
- dense sand & stiff clays.



- medium dense sand & medium clays.



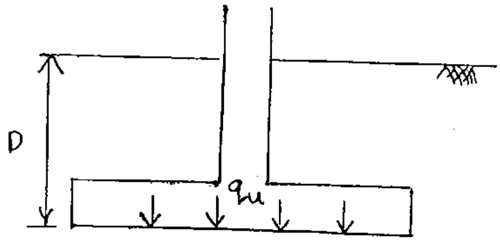
- loose sand & soft clays



- For GSF, there will be a definite failure point.
- For the same load intensity, $(\text{Settlement})_{PSF} > (\text{settlement})_{GSF}$

→ Rankine's Theory

- Soil is cohesionless.
- Footing base is smooth.
- Plastic equilibrium.



$$q_u = \gamma D \left(\frac{1+\sin\phi}{1-\sin\phi} \right)^2$$

To avoid shear failure of soil, the min. depth of foundation required,

$$D_{min} = \frac{q}{\gamma} \left(\frac{1-\sin\phi}{1+\sin\phi} \right)^2$$

$$\boxed{D_{min} = \frac{q}{\gamma} k_a^2}$$

However, as per this equation, as $D=0$, $q_u = 0$; which is not possible. So this equation is not used to calculate bearing capacity.

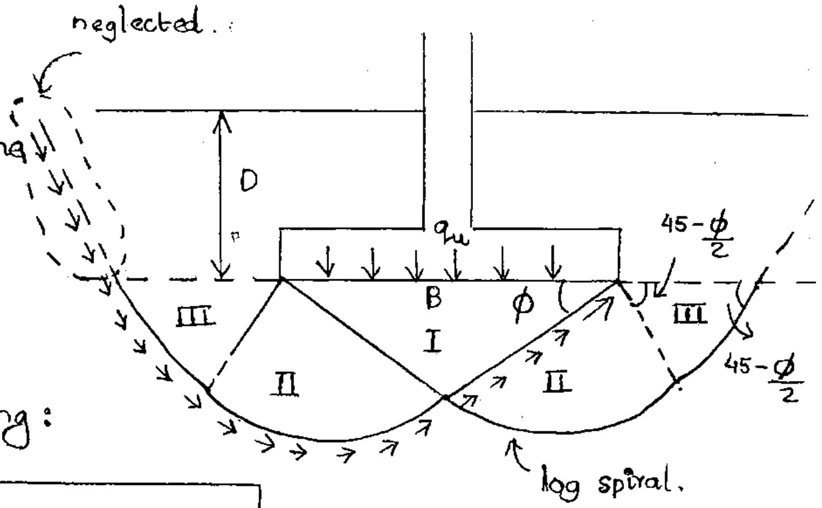
→ Terzaghi's Theory

- Footing base is rough.
- Shallow foundation
- Continuous footing (Strip footing, $L >> B$)
- General Shear Failure.

Zone I: Elastic Zone

Zone II: Radial Shear Zone

Zone III: Rankine Passive Zone



* for Continuous footing:

$$\boxed{q_u = CN_c + \gamma D N_q + 0.5\,\gamma B N_\gamma}$$

N_c, N_q, N_γ → Bearing Capacity Factors of Soil. (depends on ϕ-value only)

ϕ	N_c	N_q	N_γ

If $\phi = 0$ (Pure Clay),

$N_c = 5.7$

$N_q = 1$

$N_\gamma = 0$

∴ For pure clay,

$$\boxed{q_u = 5.7\,C + \gamma D}$$

$$q_{nu} = q_u - \gamma D.$$

$$\Rightarrow \boxed{q_{nu} = 5.7\,C}$$

q_{nu} is independent of B & D of foundation for pure clay.

$$q_u = CN_c + \gamma D N_q + 0.5\,\gamma B N_\gamma$$

$$q_{nu} = q_u - \gamma D.$$

$$= CN_c + \gamma D(N_q - 1) + 0.5\,\gamma B N_\gamma$$

$$\Rightarrow q_{ns} = \frac{q_{nu}}{F}$$

$$\therefore \quad q_s = q_{ns} + \gamma D$$

* for circular footing

$$\boxed{q_u = 1.3\, CN_c + \gamma D N_q + 0.3\, \gamma B N_\gamma}$$

where, B → diameter of footing

* for square footing

$$\boxed{q_u = 1.3\, CN_c + \gamma D N_q + 0.4\, \gamma B N_q}$$

0.3, 0.5, 1.3, 0.4 are called 'shape factors'

* for rectangular footing

$$\boxed{q_u = \left(1 + 0.3\frac{B}{L}\right) CN_c + \gamma D N_q + \left(1 - 0.2\frac{B}{L}\right) 0.5\, \gamma B N_\gamma}$$

All the above equations are for GSF

– For LSF, use C_m & ϕ_m to find BC of soil.

$$C_m = \frac{2}{3} C \quad \& \quad \tan\phi_m = \frac{2}{3}\tan\phi$$

$$\therefore\ q_u = C_m N_c' + \gamma D N_q' + 0.5\, \gamma B N_\gamma'$$

N_c', N_m', N_γ' are based on ϕ_m value.

– If $\phi > 36^\circ \Rightarrow$ GSF
  $\phi < 28^\circ \Rightarrow$ LSF

– If failure strain < 5% ⇒ GSF
  failure strain 10 to 20% ⇒ LSF

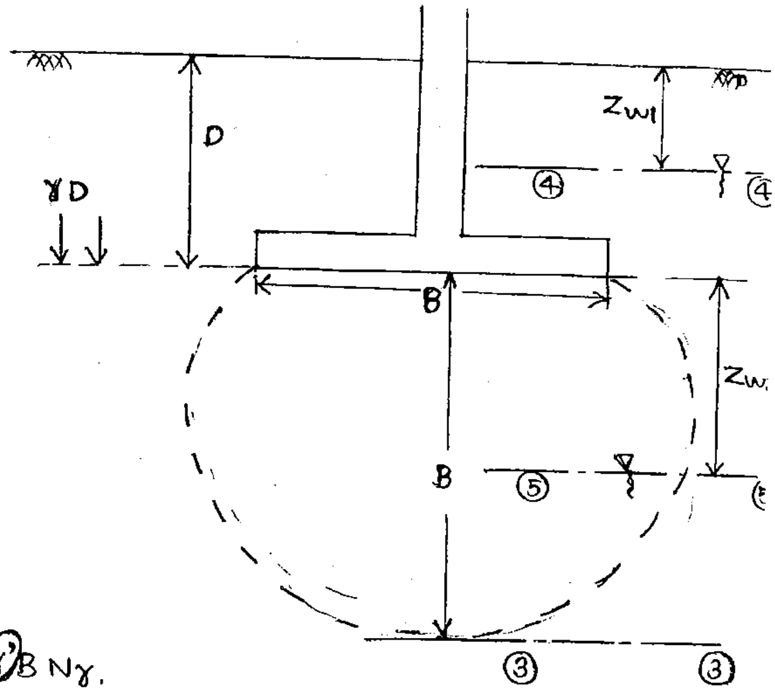
20th sept, SATURDAY

→ Effect of Water Table on Bearing Capacity of Soil:

$$q_u = \underbrace{CN_c}_{\text{I}} + \underbrace{\gamma D N_q}_{\text{II}} + \underbrace{0.5\, \gamma B N_\gamma}_{\text{III}}$$

I → cohesion effect
II → Depth effect
III → width effect.

1. When water table is at or above GL



$$q_u = c' N_c + \gamma' D N_q + 0.5 \gamma' B N_\gamma$$

2. When water table is at footing level.

$$q_u = c' N_c + \gamma D N_q + 0.5 \gamma' B N_\gamma.$$

3. When water table is at a level ③-③.

⊙ No effect of water table.

4. When water table is at level ④-④

$$q_u = c' N_c + \gamma_a D N_q + 0.5 \gamma' B N_\gamma$$

$$\gamma_a = \frac{Z_{w1} \gamma + (D - Z_{w1}) \gamma'}{D}.$$

5. When water table is at level ⑤-⑤

$$\gamma_a = \frac{Z_{w2} \times \gamma + (B - Z_{w2}) \gamma'}{B}.$$

$$q_u = c N_c + \gamma D N_q + 0.5 \gamma_a B N_\gamma$$

* Approximate Method:

$$q_u = c N_c + \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2}$$

R_{w1} & R_{w2} → water table correction factors

$$R_{w1} = 0.5 \left(1 + \frac{Z_{w1}}{D}\right) \qquad R_{w2} = 0.5 \left(1 + \frac{Z_{w2}}{B}\right)$$

$$\boxed{0.5 \leq R_w \leq 1}$$

If WT is at GL:

$Z_{w1} = Z_{w2} = 0$

$R_{w1} = R_{w2} = 0.5$

If WT at footing level:

$Z_{w1} = D$; $R_{w1} = 1$

$Z_{w2} = 0$; $R_{w2} = 0.5$

If WT is below the footing:

$R_{w1} = 1$ (no correction reqd.)

If WT is above footing:

$Z_{w2} = 0$

$R_{w2} = 0.5$

If WT is at a depth B below footing:

$R_{w1} = R_{w2} = 1$ (no correction reqd)

For cohesionless soils,

$$q_u = \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma \cdot R_{w2}$$

If WT is at or above GL, $R_{w1} = R_{w2} = 0.5$

$$q_u = \gamma D N_q \underbrace{(0.5)}_{R_{w1}} + (0.5 \gamma B N_\gamma) \underbrace{(0.5)}_{R_{w2}}.$$

∴ for cohesionless soils, bearing capacity reduces by 50% when WT raises to GL.

In case of cohesive soils, the effect of WT on the bearing capacity is negligible.

$$q_{nu} = 5.7\, C \quad \text{(no '}\gamma\text{' included)}$$

→ Skempton's Theory :

- For cohesive soils only ($\phi = 0$)

$$\boxed{q_{nu} = C N_c}$$

- For strip footing :

$$N_c = 5\left(1 + 0.2 \frac{D}{B}\right) \qquad \boxed{5.14 \leq N_c \leq 7.50}$$

- For rectangular footing:

$$N_c = 5\left(1 + 0.2 \frac{D}{B}\right)\left(1 + 0.2 \frac{B}{L}\right) \qquad \boxed{6.2 \leq N_c \leq 9}$$

→ Plate Load Test.

– to find BC and settlements.

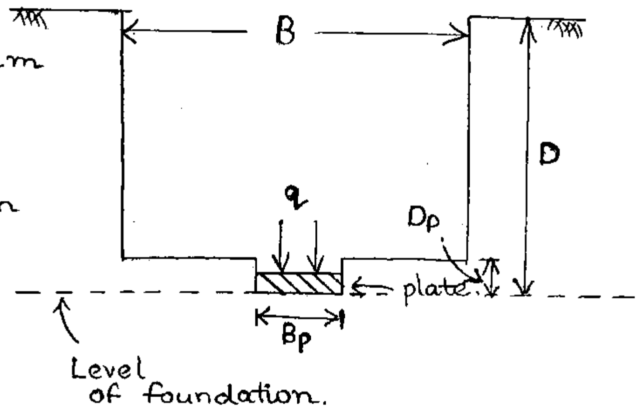
* Specifications:

Min. size of plate
= 30 cm x 30 cm

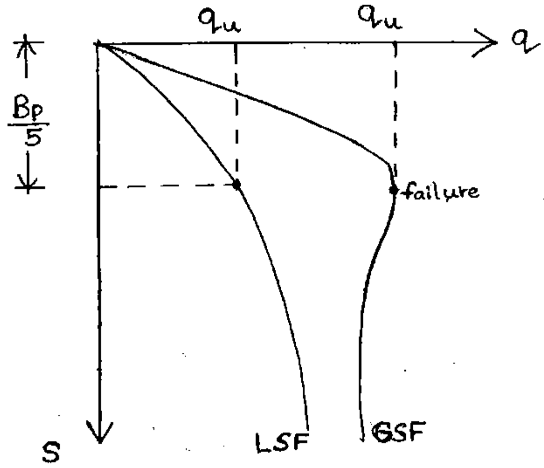
Max. size of plate
= 75 cm x 75 cm

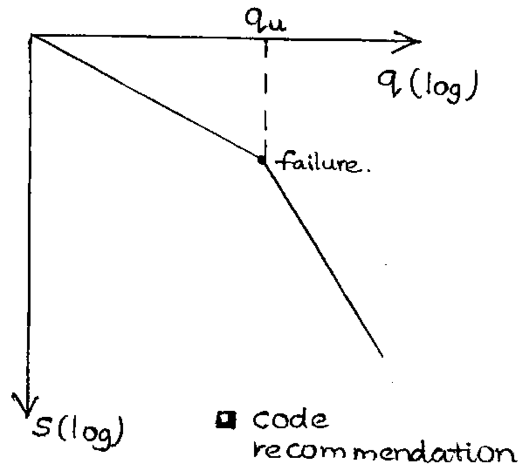
Min. thickness = 25 mm (1")

$\frac{B}{B_p} = 5$ & $\frac{D}{D_p} = 5$



– Initially, a seating pressure of 7.5 kPa applied





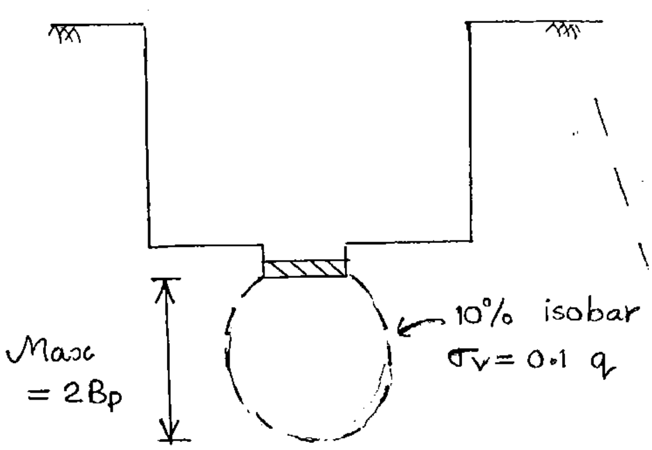
Safe bearing capacity, $q_s = \frac{q_u}{F}$

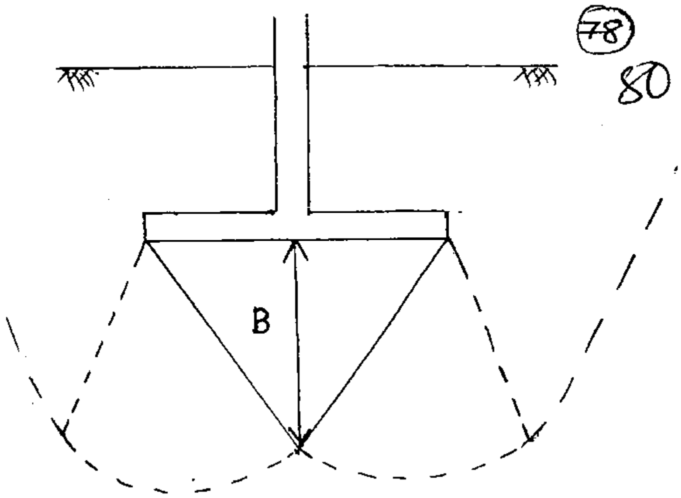
* Limitations of Plate Load test:

(i) It is a short duration test. Hence not reliable for pure clays. (consolidation settlement occurs for pure clays)

(ii) There is a width effect. (q_u depends on B in Terzaghi's theory & Skemptons theory)

(iii) Depth effect. (max. depth of pressure bulb in plate load test = $2B_p$)





→ Corrections for Plate Load test Results:

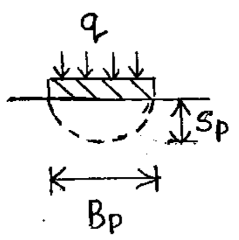
* Correction for Settlements

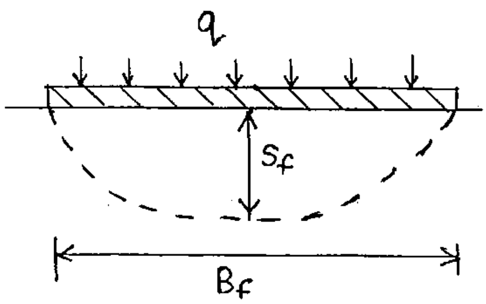
(i) For Clays

$$\frac{S_F}{S_P} = \frac{B_F}{B_P}. \Rightarrow S \propto B$$

(ii) For sands.

$$\frac{S_F}{S_P} = \left[\frac{B_F\,(B_P + 0.3)}{B_P\,(B_F + 0.3)}\right]^2 \quad B_F \text{ \& } B_P \text{ in } \underline{\text{metres}}$$





- q is same in both plate & footing

* Correction for Bearing Capacity

(i) For clays

$$q_f = q_p \quad (q_u \text{ independent of } B)$$

(ii) For sands

$$\frac{q_f}{q_p} = \frac{B_F}{B_P}$$

→ Meyerhof's Theory :

– For both shallow and deep foundations.

$$q_u = C N_c \underline{S_c d_c i_c} + \gamma D N_q . \underline{S_q . d_q . i_q} + 0.5 \gamma B N_\gamma . \underline{S_\gamma . d_\gamma . i_\gamma}$$

S → shape factor

D → depth factor

i → load inclination factor

* For (DL + LL) → BC obtained by formulae can be used directly.

For $\left.\begin{matrix} DL + LL + WL \\ DL + LL + EL \end{matrix}\right\}$ → above BC is increased by 25%

→ Loads for Settlement Calculation :

* For sands : DL + LL + WL or EL

For clays : Permanent loads (DL + 50% LL)

→ Settlements

* Uniform Settlement : equal settlement everywhere.

* Differential Settlement : more detrimental to struct (additional moments are created)

Differential settlement ≈ 75% of total uniform settleme

* Permissible limits :

	Isolated Foundations	Raft Foundations. (differential settlements.)
Sand & Hard clay	50 mm	75 mm
Plastic clay (settlement occurs slowly for plastic clay)	75 mm.	100 mm.

01. $q_{ns} = \frac{1}{F}(cN_c)$; for clay

Usually, $F \approx 3$. & $N_c = 5.7$ for clays.

$q_{ns} = \frac{1}{3}(c \times 5.7) \approx 2c \Rightarrow$ unconfined compressive strength.

02. $B = 3\,m$, $\gamma_{sat} = 20\ kN/m^3$, $\phi = 35^0$, $c = 0$, $N_q = 33$, $N_\gamma = 34.0$
$D = 2m$, $\gamma = 18\ kN/m^3$

(i). WT at GL

$q_u = 1.3\,c'N_c + \gamma' D N_q + 0.4\,\gamma' B N_\gamma$.

$= 0 + 10 \times 2 \times 33 + 0.4 \times 34 \times 3 \times 10 = \underline{1068}$ kPa

(ii) WT at footing level

$q_u = \gamma D N_q + 0.4\,\gamma' B N_\gamma$.

$= 18 \times 2 \times 33 + 0.4 \times 10 \times 3 \times 34 = \underline{1596}$ kPa

(iii) at 1 m below footing

$R_{w2} = 0.5\left(1 + \frac{1}{3}\right) = 0.666$

$q_u = 18 \times 2 \times 33 + 0.4 \times \cancel{10}^{20} \times 3 \times 34 \times 0.66$ {use accurate method}

$= \underline{\cancel{1460}}\ 1672$ kPa

(iv) at 1m below GL.

$\gamma_a = \frac{\gamma Z_w + \gamma'(D - Z_w)}{D} = \frac{18 \times 1 + 10 \times (2-1)}{2} = 14$

$q_u = 14 \times 2 \times 33 + 0.4 \times 10 \times 3 \times 34 = \underline{1332\ kPa}$

03. Strip footing → WT at footing

$q_{ns} = \frac{1}{F}\left(cN_c + \gamma D(N_q - 1) + 0.5\,\gamma' B N_\gamma\right)$.

For short term condition: use C_u & ϕ_u

For long term condition : use c' & ϕ'

a) Short term:

$$q_{ns} = \frac{1}{2}\left(80 \times 6 + 16 \times 1\,(1-1) + 0\right)$$

$$= \underline{\underline{240 \text{ kPa}}}$$

b) Long term:

$$q_{ns} = \frac{1}{2}\left(0 \times 37.2 + \overset{16}{\cancel{20}} \times 1\,(22.5-1) + 0.5 \times 10 \times 2 \times 19.7\right)$$

$$= \underline{\underline{270.5}}$$

4. For clays:

$$q_s = \frac{1}{F}(CN_c) + \gamma D$$

$N_c = 5.7$ for rough base (Terzaghi)

$= 5.14$ for smooth base (Prandtl)

$$C = \frac{1}{2} \times q_u = 10 \text{ t/m}^2.$$

$$q_s = \frac{1}{2}(10 \times 5.14) + 2 \times 1 = \underline{\underline{27.7}} \text{ t/m}^2$$

5. For design purpose, the condition to be satisfied:

$$q_n \leq q_{na}.$$

↳ smaller of q_{ns} & q_{np}

Since q_{np} is not given, $q_{na} = q_{ns}$.

$$\therefore \quad q_n \leq q_{ns}$$

$$q_n = \frac{Q}{A} = \frac{1000}{B^2} \text{ kN/m}^2$$

$$q_{ns} = \frac{1}{F}\left(1.3\,CN_c + \gamma D(N_q - 1) + 0.4\,\gamma B N_\gamma\right).$$

$$= \frac{1}{2.5}\left(0.4 \times 19 \times B \times 42\right).$$

$$\Rightarrow \quad \frac{1000}{B^2} = (0.4 \times 19 \times B \times 42) \times \frac{1}{2.5}$$

$$\Rightarrow B = 1.98\text{m} \approx \underline{\underline{2\text{m}}}$$

06. Since q_{np} not given,

$$q_n \leq q_{ns}.$$

or $$q \leq q_s$$

$$q = \frac{300}{B \times 1} \text{ kN/m}^2$$

$$q_n = q - \gamma D = \frac{300}{B} - 18 \times 1$$

$$q_{ns} = \frac{1}{F}(CN_c) = \frac{1}{3}(60 \times 5.7) = 114$$

$$\frac{300}{B} - 18 = 114.$$

$$\therefore B = \underline{\underline{2.27}} \text{ m}$$

* For cohesionless soil $\longrightarrow$ use ϕ to decide GSF & LSF
  For cohesive soil. $\longrightarrow$ use C to decide GSF & LSF
  For c$-\phi$ soil $\longrightarrow$ use <u>strain</u> to decide GSF & LSF

07 Elastic settlement, $S_i = \frac{q_n}{E_s} B(1-\mu^2) I.$

$$\boxed{S_i \propto q_n}$$

$$\therefore \frac{S_2}{S_1} = \frac{q_2}{q_1}$$

$$\frac{10}{25} = \frac{q_2}{7.2/0.3^2}$$

$$\Rightarrow q_2 = \underline{\underline{32}} \text{ t/m}^2$$

08. $q_n = q - \gamma D.$

$$0 = 150 - 20D$$

$$\Rightarrow D = \underline{\underline{7.5}} \text{ m}$$

9. $B_P = 0.3\,m$, $B_F = 1.5\,m$

$q_P = 6\ t/m^2$, $q_F = ?$

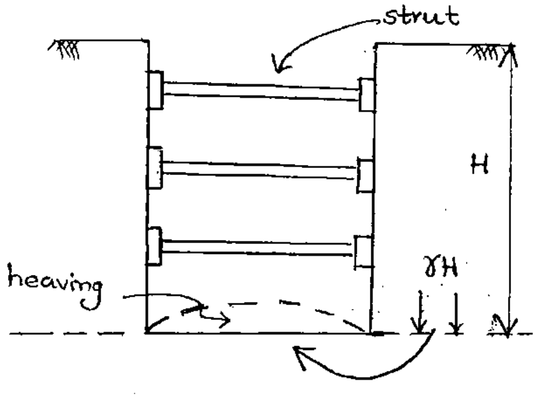
For sands,

$$\frac{q_F}{q_P} = \frac{B_F}{B_P}.$$

$$\frac{q_F}{6} = \frac{1.5}{0.3} \Rightarrow q_F = 30\ t/m^2.$$

Load = area x q_F = $1.5^2 \times 30$ = $\underline{\underline{67.5\ \text{tons}}}$

→ Braced Excavations – Heave Failure of Bottom



Factor of safety against heave failure, $F = \dfrac{CN_c}{\gamma H}$

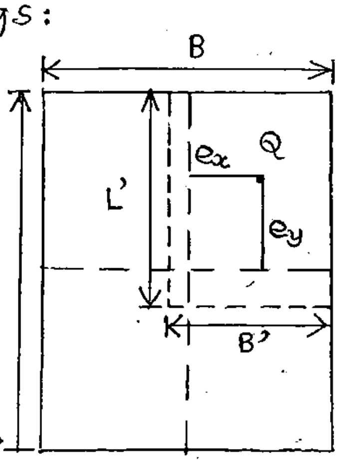
$N_c = 5.7$ (Terzaghi).

$= 5\left(1 + 0.2\dfrac{D}{B}\right)$; Skempton's theory

10. $N_c = 5\left(1 + 0.2 \times \dfrac{5}{2.5}\right) = \underline{\underline{7}}$ $\quad (D = H)$

$$FOS = \frac{CN_c}{\gamma H} = \frac{20 \times 7}{20 \times 5} = \underline{\underline{1.4}}$$

Bearing Capacity of Eccentric Footings:



For eccentric footings, modified dimensions (reduced) are to be taken to calculate the bearing capacity. The reduced dimensions are taken in such a manner that the load acting point should become the CG of modified dimensions.

$$B' = 2\left(\frac{B}{2} - e_x\right) \quad \& \quad L' = 2\left(\frac{L}{2} - e_y\right).$$

Accordingly the modified dimensions B', L', A' are shown below: 83

$$B' = B - 2e_x$$

$$L' = L - 2e_y$$

$$A' = B'L'$$

$$\therefore \quad q_u = \left(1 + 0.3\frac{B'}{L'}\right)cN_c + \gamma D N_q + 0.5\gamma B' N_\gamma \left(1 - 0.2\frac{B'}{L'}\right)$$

Safe load capacity, $Q_{safe} = A' q_s$.

PILE FOUNDATIONS

- Deep Foundation.
- Punching shear failure occurs

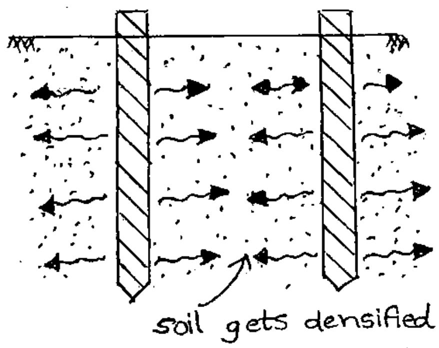
→ Necessity of Pile foundation :

- if loads are heavy and soil is poor.
- in expansive soils (like black cotton soil)
- to transfer loads onto a hard stratum.
- to resist uplift loads, horizontal loads etc.
- to reduce settlements.

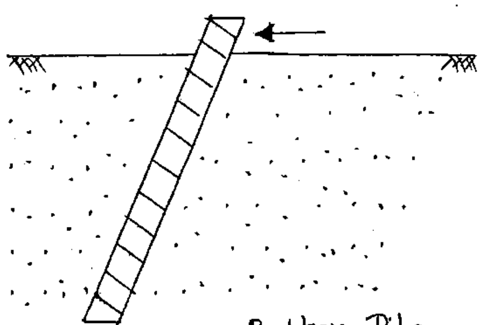
→ Classification of Piles :

* Based on Function (or purpose)

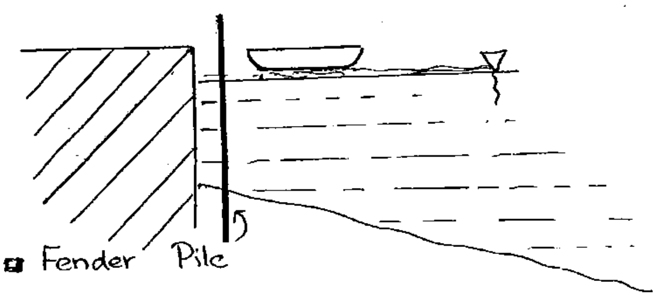
- compaction pile : to compact the soil (loose & medium sand)
- tension pile : to resist uplift loads.
- batter pile : inclined pile to resist lateral load.
- anchor pile : to anchor the structure
- Fender or dolphin pile : for protection of water front structures



Compaction pile.



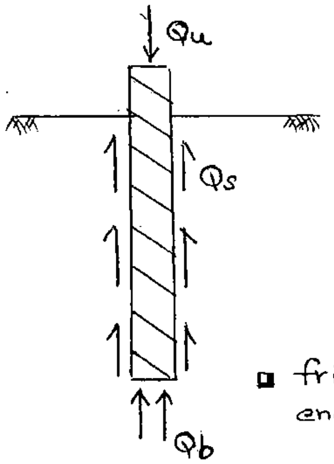
Batter Pile.



Fender Pile

* Based on Load Transfer.

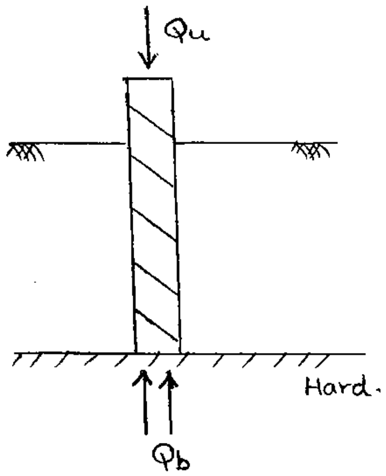
- Friction Pile : generally in clays.
- End bearing Pile : pile resting on hard stratum
- Friction cum end bearing pile : generally in sands



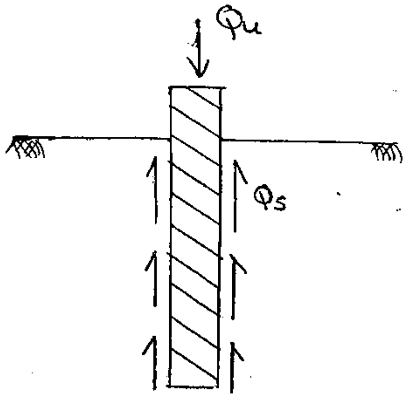
$Q_b \rightarrow$ end bearing resistance.

$Q_s \rightarrow$ skin frictional resistance.

▪ friction cum end bearing piles



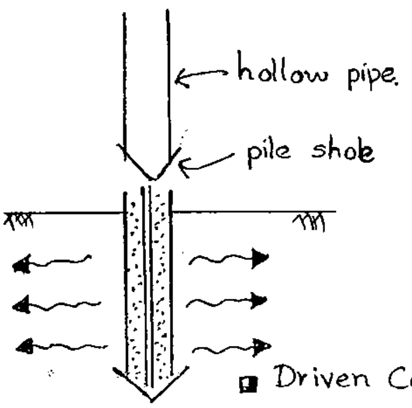
▪ End bearing piles



▪ friction piles

* Based on Construction

- Precast driven pile : in loose sand & medium dense sand
- Driven cast-in-situ pile.: in loose sand & "
- Bored cast-in-situ pile.: in clays.



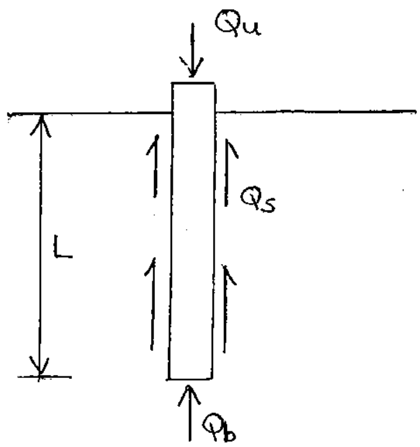
- Hollow pipe is driven into soil
- Pile is casted in site using steel rft. & concrete in the hollow pipe.
- Hollow pipe is removed using cranes immediately after casting the piles.

▪ Driven Cast in-situ pile.

→ Pile driving Equipment:
- Simple drop hammer
- Single acting steam hammer.
- Double acting steam hammer
- Diesel hammer
- vibratory driving system: least noise. (no hammers used in loose soils).

→ Load Carrying Capacity of Pile:
1. Static Formulae
2. Dynamic Formulae.
3. Pile load tests.
4. N-Value method (N-SPT value)

* Static Formulae:



$$Q_u = Q_b + Q_s$$
$$= A_b f_b + A_s f_s$$

A_b → area of pile at pile base ($= \pi/4\, d^2$).

f_b → bearing capacity of soil at pile base level.

A_s → skin surface area of pile. ($= \pi d L$)

f_s → shear resistance of soil, surrounding the pile shaft

a) For clays: ($\phi = 0$)

$$f_b = C_1 N_c$$

C_1 : cohesion at pile base level.

$N_c = 9$ (for $\phi = 0$)

$$f_s = \alpha C_2$$

$S = c + \sigma \tan\phi$

$S = c + 0.$

C_2 : cohesion of soil along the pile shaft.

α : shear mobilisation factor (or) adhesion factor

$\alpha = 1$; for soft clay

$\alpha < 1$; for stiff clay.

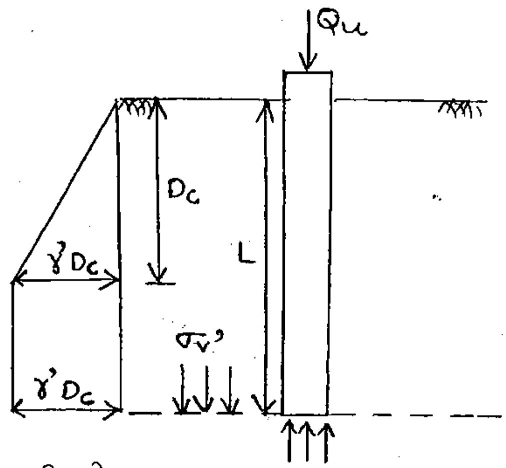
$$\therefore \boxed{Q_u = A_b C_1 N_c + A_s \alpha C_2}$$

$$Q_{safe} = \frac{Q_u}{F}$$

29th Sept, MONDAY

b) Single Pile in Sand ($c=0$)

$$Q_u = A_b f_b + A_s f_s$$



To find f_b:

$$f_b = \sigma_v' N_q$$

σ_v' = effective vertical stress at pile base level.

$D_c \rightarrow$ critical depth : depth upto which σ_v' increases and then remains a constant.

$D_c = 10d$ to $20d$; depending on soil type

$$\therefore f_b = \sigma_v' N_q$$

$$\Rightarrow \boxed{\begin{aligned} \sigma_v' &= \gamma' D_c \quad ; \text{ if } L \geq D_c \\ &= \gamma' L \quad ; \text{ if } L < D_c \end{aligned}}$$

To find f_s:

$$f_s = k \cdot \sigma_a' \cdot \tan\delta$$

$\sigma_a' \rightarrow$ average effective vertical stress along the pile shaft.

$K \rightarrow$ coeffecient of lateral earth pressure.

'K' is found out from field in-situ test for pile.

$\boxed{K = 1 \text{ to } 3}$; depending on soil type.

$\delta \rightarrow$ angle of friction b/w pile and soil

$\boxed{\delta = \phi}$; as per BIS

For Clay:

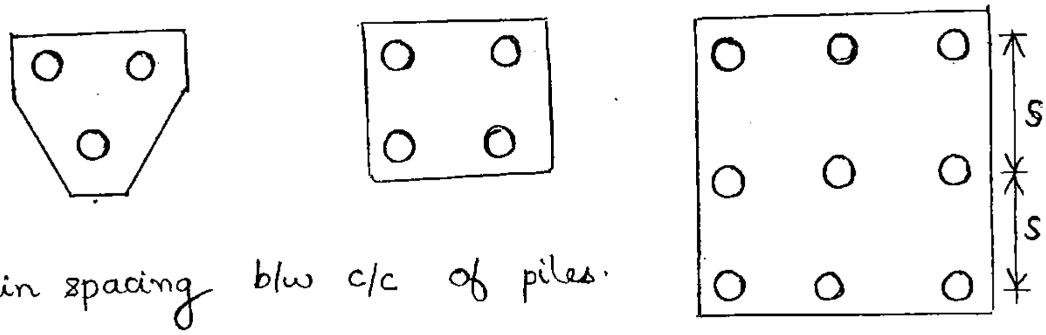
$$\boxed{Q_u = A_b C_1 N_c + A_s . \alpha C_2}$$

For Sand:

$$\boxed{Q_u = A_b \sigma_v' N_q + A_s K . \sigma_a' \tan\delta}$$

→ Group Piles

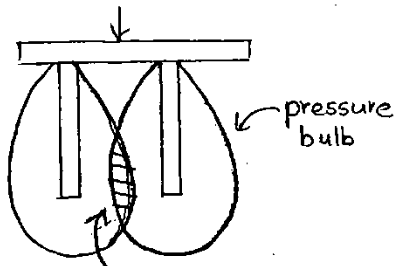
- under a column, a min. of 3 piles shall be used.



$S \rightarrow$ min spacing b/w c/c of piles.

$S = 3d$; for friction piles.

$= 2.5d$; for end bearing piles.



- Benefits of group piling are:
  - a) Increases reliability.
  - b) eccentricity is avoided.
- Min spacing is recommended to avoid "stress overlap"

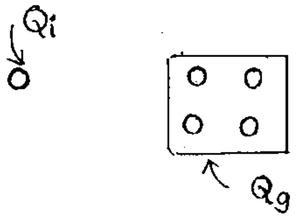
(Due to stress overlap, settlement increases)

* Pile Group Efficiency, $\eta_g = \dfrac{Q_g}{n . Q_i} \times 100$

$Q_g \rightarrow$ total group capacity.

$n \rightarrow$ no. of piles in the group.

$Q_i \rightarrow$ capacity of single, in isolation.



η_g is the ratio of average capacity of a single pile in a group action to the single capacity of a pile in isolation. (OR) is the ratio of total capacity of a group pile to the sum of capacities of individual piles in isolation.

$\eta_g > 100\,\%$; for loose & medium dense sands

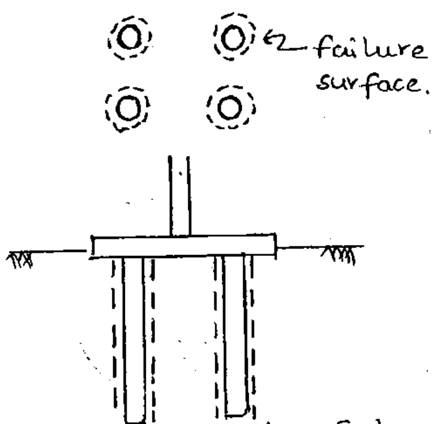
$\eta_g < 100\,\%$; for dense sand & clays

Loose and medium dense sands gets compacted upon group piling and $\therefore\ \eta_g > 100\,\%$.

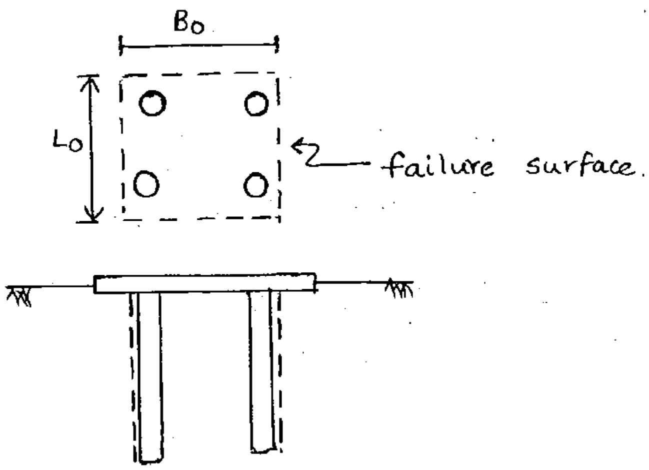
* Modes of failure of Pile Group:

(i) Individual Pile Failure.

(ii) Block Failure.



▫ individual pile failure



▫ Block Failure

* Pile Group Capacity based on Individual Failure mode

$$Q_{gi} = n\,Q_i$$

$$= n\left(A_b . C_1 N_c + A_s\, \alpha\, C_2\right) \longrightarrow \text{for clay}$$

$$= n\left(A_b\, \sigma_v' N_q + A_s\, K . \sigma_a' \tan\delta\right) \rightarrow \text{for sand}$$

* Pile Group Capacity based on Block Failure mode:

$$Q_{gb} = A_B \cdot C_1 N_c + A_s C_2 \quad \rightarrow \text{for clay}$$

$$= A_B \cdot \sigma_v' N_q + A_s \, K \cdot \sigma_a' \tan\delta \rightarrow \text{for sand}$$

$A_B \rightarrow$ area of block $= B_o \cdot L_o$

$A_s \rightarrow$ perimeter of block $\times L = 2(B_o + L_o) \cdot L$

α is adhesion factor.

$\alpha = 1$ for block failure because contact is b/w soil & soil.

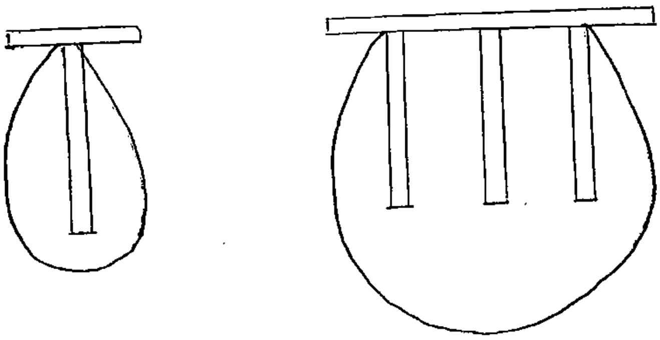
$\alpha < 1$ for individual pile as contact is b/w soil & pile.

$\therefore$ Pile group capacity, Q_g = Smaller of Q_{gi} & Q_{gb}

Safe capacity, $= \dfrac{Q_g}{F}$

NOTE:

Settlement of a group pile is always more than that of a single pile. (due to larger size of pressure bulb)

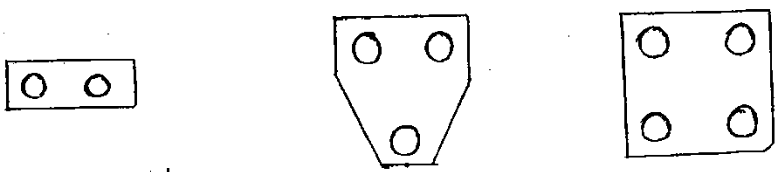


larger pressure bulb
$\Rightarrow$ more soil gets compressed, and
$\therefore$ more settlement.

→ Empirical Formulae to find η_g:

1. Feld's Rule.

In this rule, for every nearby pile, $\frac{1}{16}$th capacity is reduced. (for all types of soil)



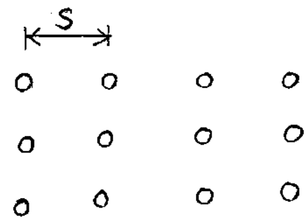
$\eta_g = 1 - \frac{1}{16} = \frac{15}{16}$ //

$\eta_g = 1 - 2 \times \frac{1}{16} = \frac{14}{16}$ //

$\eta_g = 1 - 3 \times \frac{1}{16} = \frac{13}{16}$ //

2. Converse – Lebarre Formula.

$$\eta_g = 1 - \frac{\theta^\circ}{90}\left[\frac{m(n-1)+n(m-1)}{m.n.}\right]$$



$$\theta^\circ = \tan^{-1}\left(\frac{d}{s}\right)$$

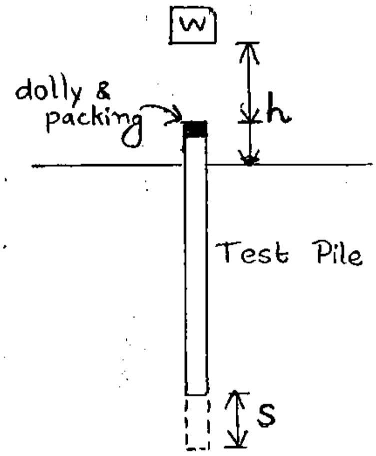
m = no: of rows of piles.

n = no: of piles in each row.

d → diameter of pile ; s → spacing b/w piles

→ Dynamic Formulae:

1. Engineering News Formula



S → set value or settlement per blow.

$$\boxed{Q_{safe} = \frac{W h \eta_h}{F(s+c)}}$$

c → a constant

C = 2.54 cm for drop hammer

= 0.254 cm for steam hammer

F → factor of safety (min. of 6)

2. Hiley's Formula.

$$\boxed{Q_{safe} = \frac{W h \eta_h \cdot \eta_b}{F(s + c/2)}}$$

η_h → efficiency of hammer

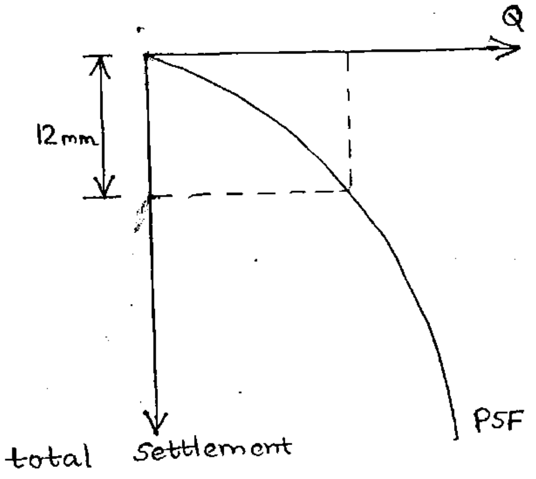
η_b → efficiency of blow.

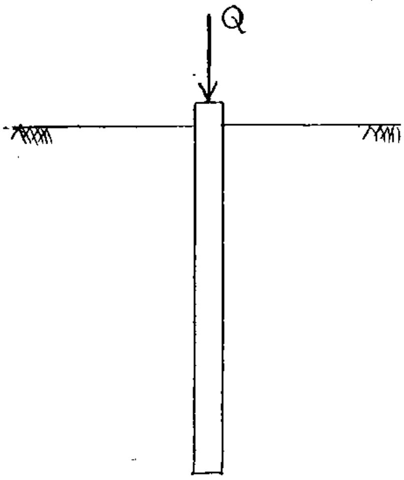
C → total temporary elastic compression of soil, pile & packing.

$$C = C_1 + C_2 + C_3$$

As this test is based on 'short term loading', it is suitable only for sand and not for clays.

→ Pile Load Test





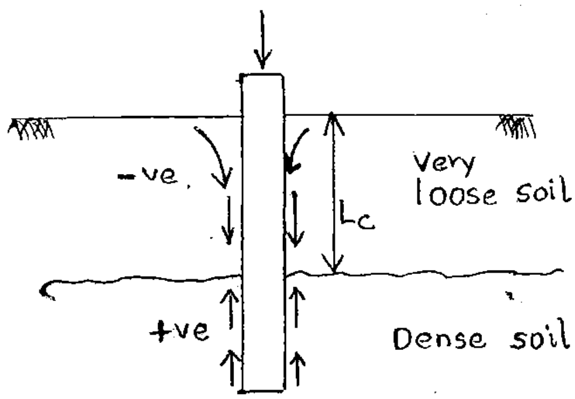
Safe load is taken as the smaller of the following:

(i) $\frac{2}{3}$ of load corresponding to a total settlement of 12 mm.

(ii) $\frac{2}{3}$ of load corresponding to a net settlement of 6 mm.

(iii) 1/2 of load corresponding to a total settlement of 10% d.

→ Negative Skin Friction:



When loose soil compacts, it drags the pile along with it. Negative skin friction occurs when settlement of loose soil due to compaction is more than the settlement of pile due to external loading.

* Negative Skin Friction occurs in the case of:

(i) Recently filled upsoil.

(ii) Very loose sand.

(iii) Soft clay.

(iv) Due to lowering of WT (σ' ↑ and settlement increases)

(v) Pile driving operations nearby.

* Negative Skin Friction can be reduced by:

(i) Keeping the surface of pile smooth in areas of loose sand.

(ii) Providing a sleeve to the pile and isolating it from surrounding loose sand.

* To calculate negative skin friction:

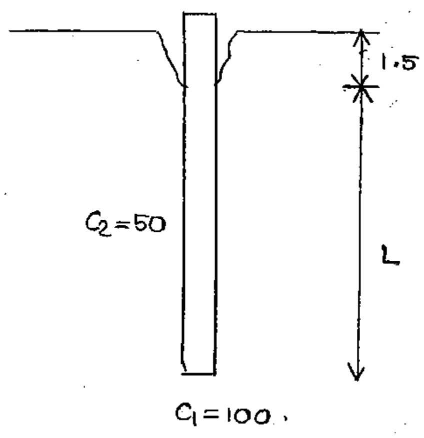
(i) In Clays:-

$$\boxed{Q_{nf} = \pi d L_c \alpha C.} \quad (\text{usually } \alpha = 1)$$

(ii) In Sands:-

$$\boxed{Q_{nf} = \pi d L_c \, k . \sigma_a' \tan \delta}$$

1st Oct,
WEDNESDAY. → B



1. $Q_u = A_b C_1 N_c + A_s \alpha . C_2$.

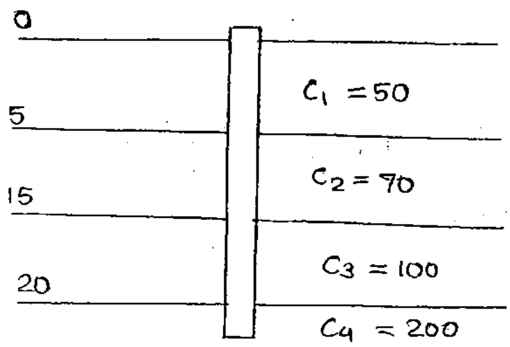
$= \frac{\pi}{4} \times 0.3^2 \times 100 \times 9 + \pi \times 0.3 \times 5 \times 0.3 \times 50$

$= \underline{\underline{134}}$ kN

2. $Q_{safe} = \frac{1}{F}\left(A_b C_1 N_c + A_s \alpha C_2\right)$.

$400 = \frac{1}{2.5}\left(0.45^2 \times 100 \times 9 + 4 \times 0.45 \times L \times 0.75 \times 100\right)$.

$\Rightarrow L = \underline{\underline{6.05}}$ m



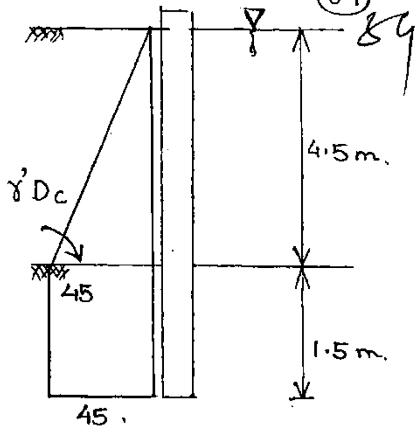
3. $Q_{safe} = \frac{1}{F}\left(A_b C_1 N_c + A_s \alpha C_2\right)$

$= \frac{1}{2.5}\Big(\frac{\pi}{4} \times 0.5^2 \times 200 \times 9 + \pi \times 0.5 \times 0.4 \left(50 \times 5 + 70 \times 5 + 100 \times 5 + 200 \times 5\right)\Big)$

$= \underline{\underline{669.16}}$ kN

04. NOTE :

If critical depth is not mentioned in the case of cohesionless soils, then assume that D_c is more than length of piles and take linear vertical stress distribution to estimate capacity of pile.



$$\gamma_{sat} = 2 g/cc = 19.613 \ kN/m^3$$
$$\approx 20 \ kN/m^3$$

$$\gamma_w = 10 \Rightarrow \gamma' = 10 \ kN/m^3$$

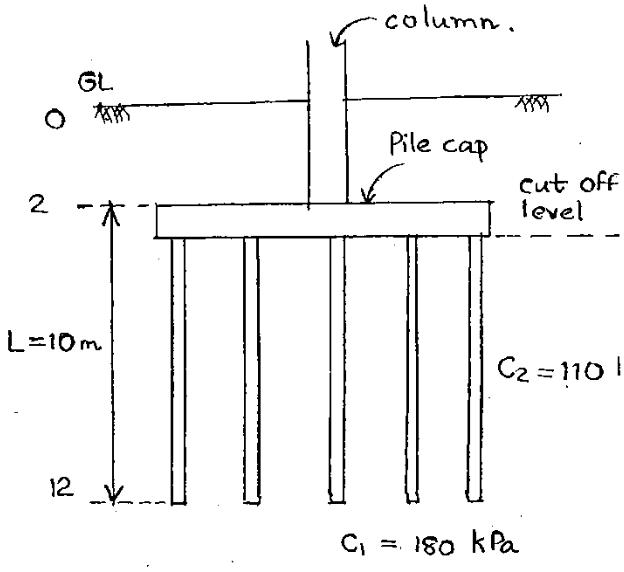
$$Q_u = A_b \sigma_v' N_q + A_s \ k . \sigma_a' \tan\delta$$

$$= \frac{\pi}{4} \times 0.45^2 \times 45 \times 18 + \pi \times 0.45 \times 4.5 \times 1.2 \left(\frac{0 + 4.5}{2}\right) \tan 20^\circ +$$

$$\pi \times 0.45 \times 1.5 \times 1.2 \left(\frac{45 + 45}{2}\right) \tan 20^\circ$$

$$= \underline{\underline{233 \ kN}}$$

05. $B_o = L_o = 4s + d = 4.5 \, m$



$$Q_{gi} = n \ Q_i$$
$$= 25 \left(\frac{\pi}{4} d^2 . C_1 N_c + \pi d L \alpha C_2\right)$$
$$= 27390 \ kN .$$

$$Q_{gb} = A_B C_1 N_c + A_s C_2$$
$$= B_o L_o C_1 N_c + 2 (B_o + L_o) L \ C_2$$
$$= 52605 \ kN .$$

Q_g = Smaller of Q_{gi} & Q_{gb}.

$$= \underline{\underline{27390 \ kN .}}$$

06. $B_o = L_o = (3s + d)$.

For $\eta_g = 100\%$ (or for optimum condition).

$$Q_{gi} = Q_{gb}.$$

$$n(\pi d L \alpha c) = 4 \times B_0 \times L\ C.$$

$$16(\pi \times d \times L \times 0.6\,C) = 4\,(3s + d)\,LC$$

$$\Rightarrow s = \underline{\underline{2.18d}}$$

07. Safe capacity of single isolated pile,

$$Q_i = \frac{1}{F}\left(\frac{\pi}{4} d^2 \times C_1 N_c + \pi d L \alpha C_2\right).$$

$$= \frac{1}{2.5}\left(\frac{\pi}{4} \times 0.3^2 \times 150 \times 9 + \pi \times 0.3 \times 10 \times 0.57 \times 100\right).$$

$$= \underline{\underline{253.05\ \text{kN}}}$$

For $\eta_g = 1$,

$$Q_g = n\,Q_i$$

$$\text{or}\quad n = \frac{Q_g}{Q_i} = \frac{5000}{253.05} = 19.75\ \text{no.s} \approx \underline{\underline{20}}$$

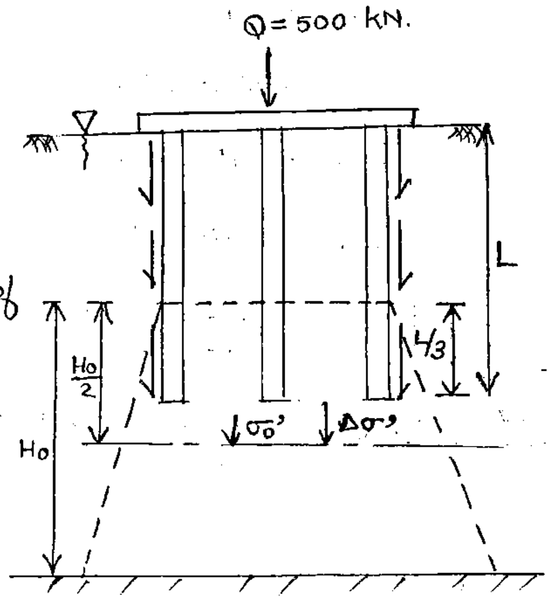
09. Rated energy, $Wh = 3500$ kN cm.

$$Q_{safe} = \frac{Wh\,\eta_h\,\eta_b}{F(S + C/2)}$$

$$S = \frac{25.4}{6} = 4.23\ \text{mm} = \underline{\underline{0.423\,\text{cm}}}$$

$$\therefore\ Q_{safe} = \frac{3500 \times 0.8 \times 0.476}{4\,(0.423 + 1.8/2)} = \underline{\underline{251\ \text{kN}}}$$

08. The given pile group is assumed to be friction pile group and the total load is assumed to be acting at lower $\frac{1}{3}$ rd of length of pile for settlement calculations



$$S_f = H_o \cdot \frac{C_c}{1+e_o} \log_{10}\left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'}\right)$$

$H_0 = 3.667$ m

$\sigma_0' = \left(3.33 + \frac{3.667}{2}\right)\gamma' = 51.63$

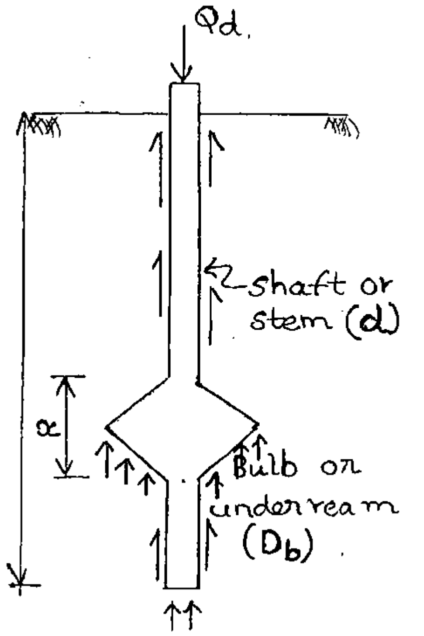
$\Delta\sigma' = \frac{Q}{(B_0+z)^2}$; $B_0 = 2s+d$

$z = \frac{H_0}{2}$.

$= \frac{500}{(1.2+1.8335)^2} = 54.335$ kN/m^2.

$S_f = 3.667 \times \frac{0.027}{(1+1.05)} \times \log_{10}\left(\frac{51.63+54.335}{51.63}\right) = \underline{\underline{15.08}}$ mm

→ Under-reamed Piles.



- The bulb provides anchorage against uplift caused by expansive soils.
- Generally used in B.C soil.
- Bulb diameter, $D_b \approx 2.5\,d$.
- Bored cast-in-situ piles.

Ultimate downward load carrying capacity, $Q_d = \frac{\pi}{4} d^2 C_1 N_c +$

$\frac{\pi}{4}\left(D_b^2 - d^2\right) C_2 N_c +$

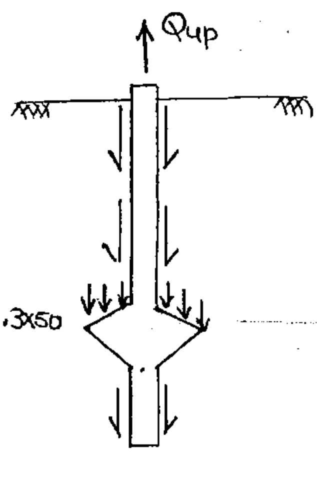
$\pi d (L-x) \alpha C_a$

C_a → avg. cohesion along pile length.

Ultimate uplift resisting capacity,

$Q_{up} = \frac{\pi}{4}\left(D_b^2 - d^2\right) C_2 N_c +$

$\pi d (L-x) \alpha C_a +$ self weight.



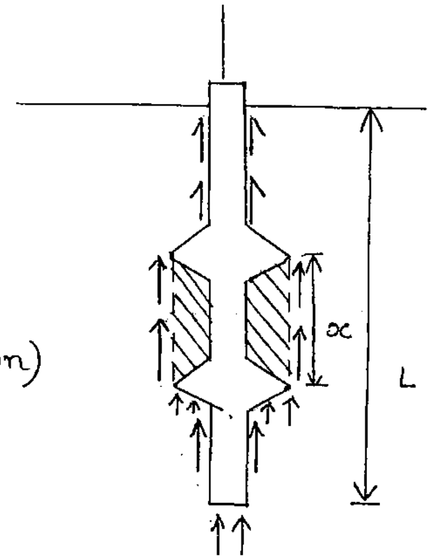
10. $Q = \frac{\pi}{4}\left(0.75^2 - 0.35^2\right) . 50 \times 9 + \pi \times 0.35 (8-0.4) \times 0.3 \times 50$

$+ 20$

$= \underline{\underline{300.8}}$ kN

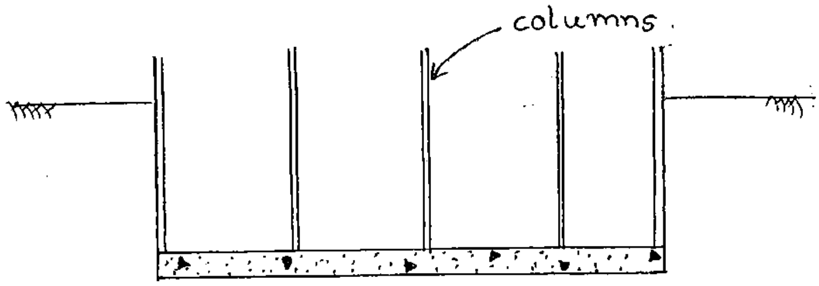
* Double Under-reamed Pile.

$$Q_d = \frac{\pi}{4} d^2 c_1 N_c + \frac{\pi}{4}\left(D_b^2 - d^2\right) c_2 N_c + \pi d (L - x)\, \alpha\, c_a + \pi D_b \cdot x \cdot c_a$$



→ **Raft Foundations: (Mat Foundation)**

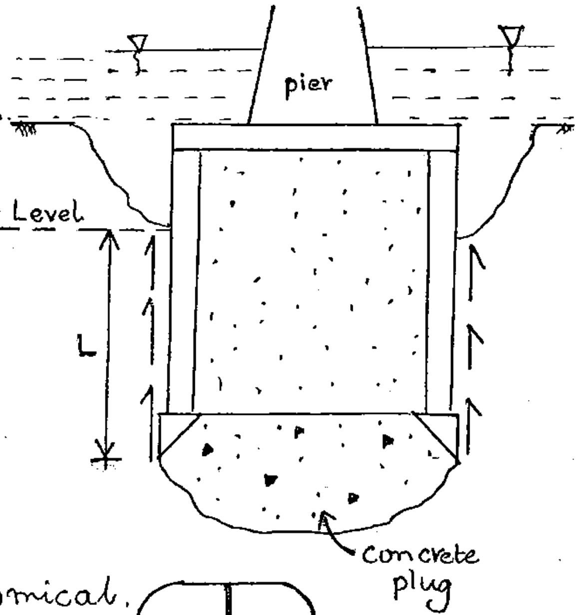
- common footing for all columns.



- shallow foundation
- to avoid differential settlement.
- also used in poor soils if loads are heavy.

→ **Well Foundation:**

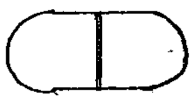
- used for bridges across rivers.



Max Scour level is depth at which max erosion occurs.

Depth of the well foundation below the max scour level is the 'Grip Length'

- double D shape is most economical.



- As per IS code of practise, the allowable tilt & shift are 1 in 60 & 1% of depth of sunk.

18. SOIL EXPLORATION

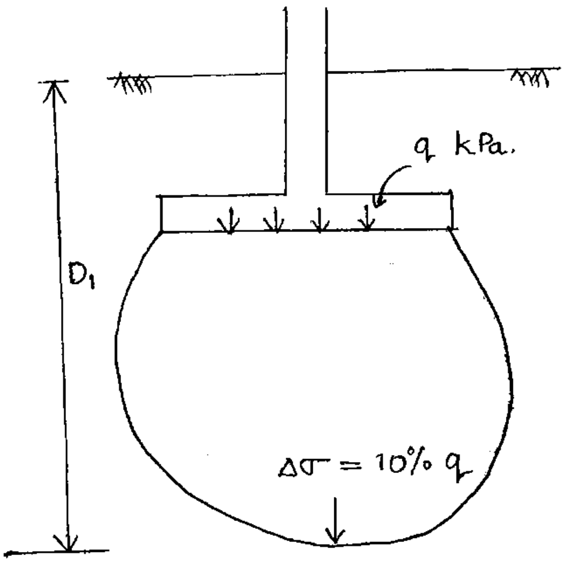
→ Soil Investigation

* For small sites → 1 hole at centre.

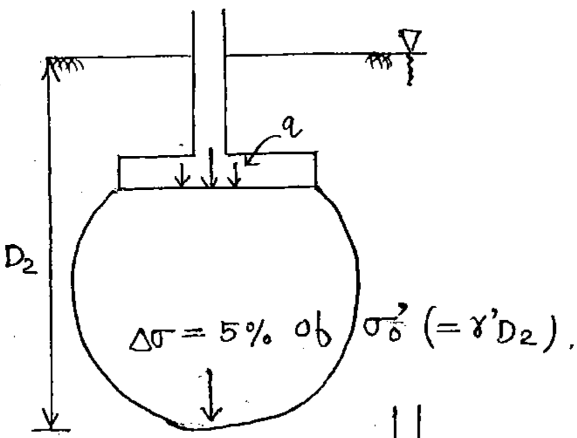
For areas upto 0.4 ha with important buildings } 5 holes → 1 @ centre, → 4 @ corners

* The min. depth of exploration is equal to 'significant depth'.

Significant depth is the depth at which increase in stress is equal to 10% of intensity of load applied. ($=D_1$).

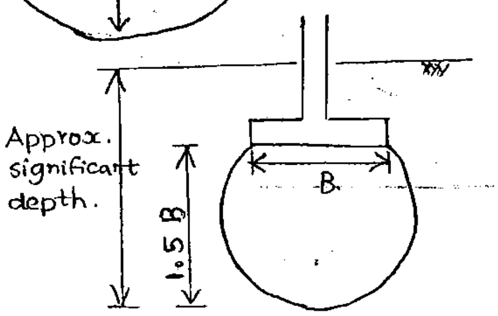


It can also be explained as the depth (D_2) at which increase in stress is equal to 5% of overburden pressure at that point.



Significant depth = Higher of D_1 & D_2

* Approximately significant depth = 1.5 B to 2 B, below the footing. As per IS code, its ≈ 1.5 B.



* Methods of Exploration:

(i) Open trial pits. → upto 3m depth

(ii) Auger boring – suitable for clays & shallow depths (6m) for highways, railways etc.

(iii) Wash boring – for deep holes, not suitable for hard stratum, can be used even below WT

(iv) Rotary drilling – for any soil, including rock.

(v) Percussion boring – for hard stratums & rocks

(vi) Core drilling – to obtain rock samples.

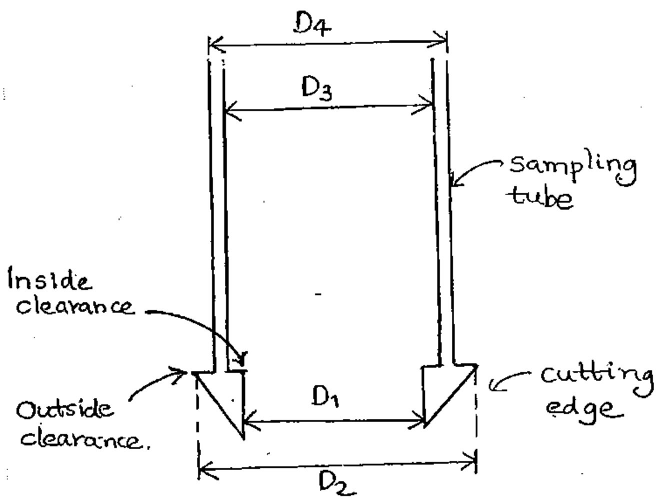
* Sampler :-

An equipment to collect soil sample.

– Types of Samplers:

(i) Thin walled Sampler – used to collect undisturbed samples. (engineering properties)

(ii) Thick walled Sampler – used to collect disturbed samples (index properties)



* Area ratio, $A_r = \dfrac{D_2^2 - D_1^2}{D_1^2} \times 100$

◉ To obtain undisturbed sample, $A_r \leq 10\%$

* Inside clearance, $= \frac{D_3 - D_1}{D_1} \times 100$

⊙ To obtain undisturbed sample, it should be 0.5 to 3%

* Outside clearance $= \frac{D_2 - D_4}{D_4} \times 100$

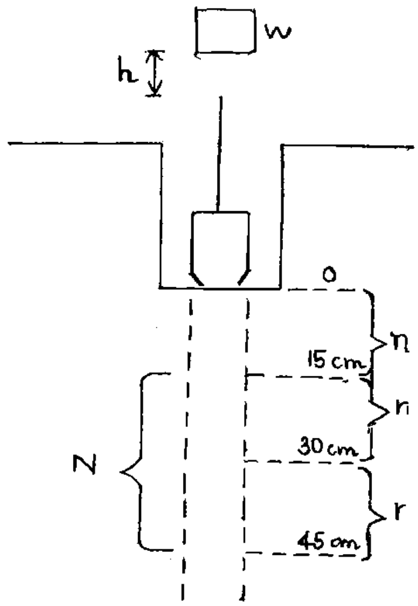
⊙ It shall be 0 to 2%

→ Standard Penetration Test (SPT)

- an insitu test.
- best suited to cohesionless soils.
- conducted by using split spoon sampler.

* SPT value or N-value of soil :

- It is the no: of blows to cause a penetration of 30 cm.



- N value (at field) $= n_2 + n_3$

- The more the N-value, the more the strength of soil.

- Weight of hammer, $w = 65$ kg
- Height of fall, $h = 75$ cm

* Corrections for N-value.

(i) Overburden Pressure Correction

- to report N-value ~~at~~ to a std overburden pressure. ($96 \, kN/m^2 \approx 100 \, kN/m^2$)

- If N_F is N-value measured at field, the corrected value N' for overburden pressure.

Peck, Hansen & Thornburn (by IS code also) :-

- $$N' = N_F * 0.77 \log_{10}\left(\frac{1905}{\sigma_0'}\right) \quad \text{for } \sigma_0' \geq 24\ kN/m^2$$

where $\sigma_0' \rightarrow$ effective overburden pressure at the depth. (in kN/m^2).

Correction factor, $\frac{N'}{N_F}$ shall be b/w 0.45 & 2.

- $$N' = N_F * \left(\frac{350}{\sigma_0' + 70}\right) \text{ ; by Gibbs \& Holtz}$$

(ii) Dilatancy Correction.

- It is required only if the soil is fine sand or silt and if present below the WT.
- If $N' > 15$;

  Corrected value, $N'' = 15 + \left(\frac{N' - 15}{2}\right)$
- If $N' \leq 15$;

  Corrected value, $N'' = N'$ (no correction required)

→ Cone Penetration Test (CPT)

- insitu test for cohesionless soil.
- types :
  - (i) Static
  - (ii) Dynamic

→ Pressuremeter Test

- to measure in-situ stress-strain curve of soil.

→ Geophysical Methods

(i) Seismic Refraction method. – for civil engg. investigation

(ii) Seismic Reflection method. – for petroleum investigation

→ Soil Stabilisation Methods:

(i) Mechanical Stabilisation.

(ii) Cement Stabilisation.

(iii) Lime stabilisation.

(iv) Chemical stabilisation.

→ Ground Improvement Techniques:

(i) Electro osmosis.

(ii) Vibrofloatation method

(iii) Terraprobe method.

(iv) Lime piles.

(v) Stone columns

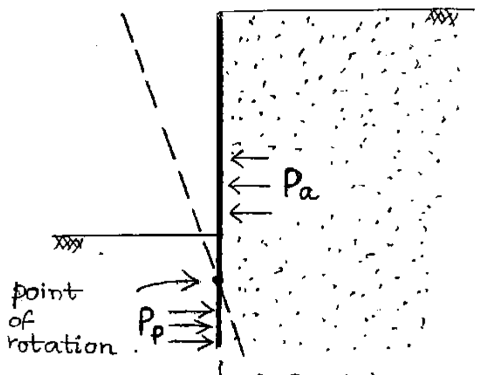
(vi) Geotextiles.

~~SOIL EXPLORATION~~

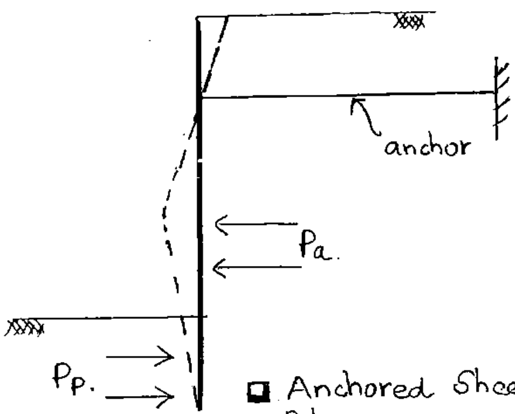
19 SHEET PILES

- used to retain soil.

- Point of rotation for a cantilever sheet pile is just above the bottom point. (used for heights upto (5m))



Cantilever Sheet Piles.



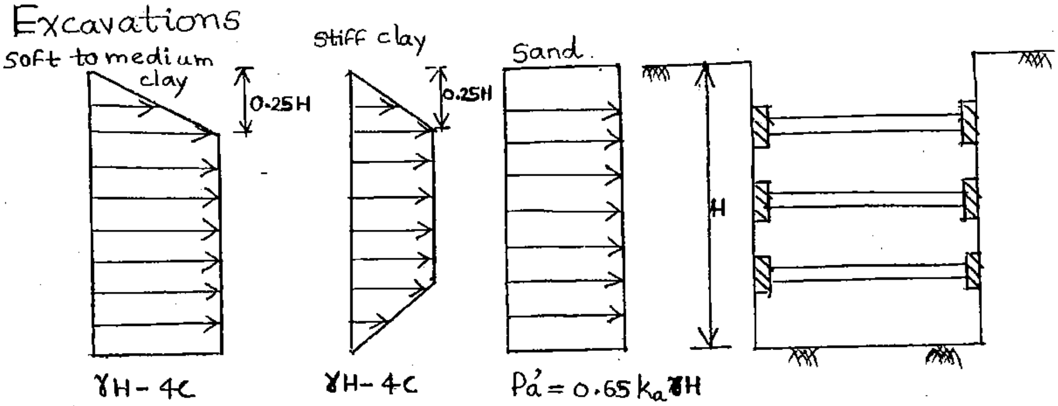
Anchored Sheet Piles

→ Types of Anchored Sheet Piles

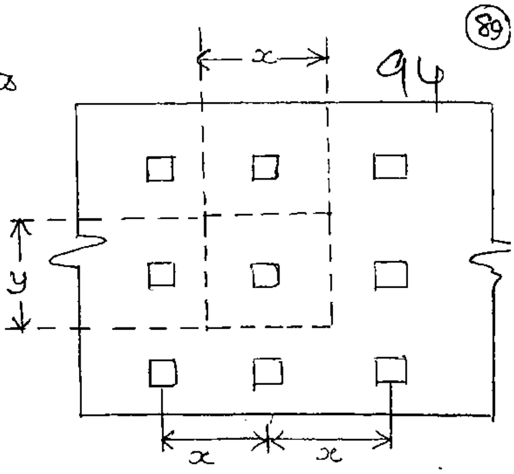
(i) Free Earth Support Type: if depth of penetration shallo

(ii) Fixed Earth Support Type: depth of penetration deep

→ Braced Excavations



$x \rightarrow$ c/c horizontal spacing of struts
$y \rightarrow$ c/c vertical spacing of struts.



Max. compressive force induced in the strut,

P = Influencing area x pressure.

$= x \cdot y \cdot P_a'$

Q. A vertical trench 3m wide & 6.5m deep is proposed in a cohesionless deposit ($\phi = 36°$ & $c = 0$). Assuming first row of struts to be placed at 0.5m below ground surface and spacing b/w the struts as 1.5m in the vertical direction and 3m spacing in the horizontal direction, calculate max strut compressive load. Take $\gamma = 20\ kN/m^3$

$$P_a' = 0.65\ K_a\ \gamma H$$

$$= 0.65 \left(\frac{1-\sin 36}{1+\sin 36}\right) 20 \times 6.5 = 21.94\ kN/m^2$$

Max strut compressive load $= x\, y\, P_a'$

$= 3 \times 1.5 \times 21.94$

$= \underline{\underline{98.72\ kN}}$