Soil Mechanics:

Soil classification and Compaction.

Origin of Soil.

→ Transported Soil

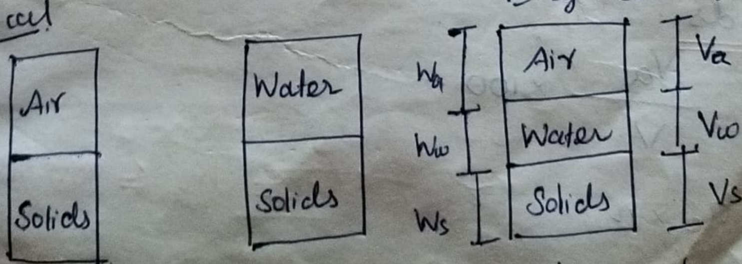
→ Residual soil (Parent rock)

Source of transportation	Types of soil.
1) river	Aluvial
2) Lakes, Pond, Reservoir	Lacustrine
3) Wind	Aeoline (or) Loess (or) → silt; Sand Dune
4) Sea water	Marl (or) Marine
5) Gravity	Colluvial Soil / Talus found in Mountains, valley.
6) Ice	Glacier / Drift, till
7) Decomposition Vegetable →	Peat organic soil
8) Soil consists of organic & Inorganic	Peat & Muck (or) called Cumulose.
9) Soil having high porosity	– Bentonite clay.
10) Sticky soil	– Gumbo.

Properties of Soil.

→ Physical
→ Index
→ Engineering.

(1) Physical



Two phase

Three phase system.

1) Voids ratio : (e)

Voids ratio is more significant than porosity.

$$e = \frac{V_v \text{ void}}{V_s \text{ solids}}$$

$\boxed{e > 0}$ $e \neq 0$.

because V_s is more stable Quantity

2) Porosity (n)

$$n = \frac{V_v}{V_1} \times 100 \quad (\%)$$

$0 < n < 100\%$

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

$\boxed{0 \geq n \leq 1}$

3) Degree of saturation (S_r)

$$S_r = S = \frac{V_w}{V_v} \times 100 \quad (\%)$$

Fully saturated $V_w = V_v$. $S = \frac{V_w}{V_w} \times 100$

$S = 100\%$.

Fully dried $V_w = 0$ $S = \frac{0}{V_v} \times 100 = 0\%$

$\boxed{0 \leq S_r \leq 100\%}$

4) Air content (a_c).

$\boxed{0 \leq a_c \leq 100}$

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

Fully saturated $V_a = 0 \Rightarrow a_c = 0\%$.

Fully dried $V_a = V_v \Rightarrow a_c = 100\%$.

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

5) % air voids. (n_a)

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

$0 \leq n_a \leq 100$.

$n_a = a_c \times n$

$n_a = (1 - S_r)\, n$

6. Water Content (w):

$$w = \frac{W_w}{W_s}$$

Fully Saturated $w_w \geq 0\%$.

Fully dry $W_w = 0$ $w = 0\%$

w for Bentonite clay 500%.

Methods:

1) Oven drying method:

(→ Most accurate method.

– 105 – 110°C

→ should not exceed 110°C (To preserve adsorbed water - maintain crystalline structure

→ Organic soil, not exceed 60°C

→ Soil containing gypsum; not exceed 80°C

$$w = \frac{W_w}{W_s} = \frac{W - W_{dry}}{W_{dry}} \times 100.$$

2) Sand bath method: (~~Fastest method~~).

- Field method

→ takes less time (30 mins)

③ → No control over the temp.

→ Not suitable → organic soil, gypsum.

3) Alcohol method (Field).

③., Not accurate method

4) Calcium carbide method $w = \left(\frac{W_2 - W_1}{W_3 - W_4}\right)\left(\frac{G-1}{G} - 1\right)$.

→ Fast method.

$$\underset{\text{Carbide}}{CaC_2} + \underset{\text{water}}{2H_2O} \rightarrow Ca(OH)_2 + \underset{\text{acetylene gas}}{C_2H_2}$$

5) Pycnometer method.

G is known.

6) Torsion balancing method.

– Mechanical method

Unit wt of water (γ_w)   1 g/cc   1000 kg/m^3

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

9.81 kN/m^3 10 kN/m^3

1 t/m^3.

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

Descending order →

$\gamma_s > \gamma_{sat} > \gamma_{bulk} > \gamma_{dry} > \gamma_{sub}$.

constant

← Ascending order

Specific gravity of soil (G). solid

1) True specific gravity (or) Absolute specific gravity.

$$G_s = \frac{\gamma_s}{\gamma_w}$$

27°C.   Fully saturated 100%.

Sand – 2.65   Inorganic 2.6 to 2.9

clay – 2.7   Organic 1 to 2

$$G_s = \frac{W_s}{W_w} = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)}$$

not including pores

W_1 → Empty wt of pycnometer

W_2 → " + wt of soil

W_3 → wt of pycnometer + wt of soil + wt of water

W_4 → wt of water.

2) Apparent specific gravity (Bulk or mass specific gravity)

$$G_m = \frac{\gamma_b}{\gamma_w}$$

$G_m = 1.4$ to 2.2.

$G_s > G_m$.

$\gamma_s > \gamma_b$

$$\frac{\text{Mass of agg, oven dry}}{\text{Volume of agg including surface pores}}$$

Vol + surface voids

$$\gamma_d = \frac{\gamma_{sat}}{1+w} \qquad \gamma_d = \frac{\gamma_b}{1+w}$$

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

Functional relation:

① $$\gamma_{bulk} = \frac{G + Se}{1+e}\gamma_w$$

Dry $S = 0$

② $$\gamma_d = \frac{G\gamma_w}{1+e}$$

Saturate $S = 100\%$ (or) 1

③ $$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w.$$

④ $$\gamma' = \gamma_{sat} - \gamma_w$$

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

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

⑤ $$wG_s = eS_r$$

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

⑥ $$\gamma_{bulk} = \frac{G\gamma_w(1+w)}{1+e}$$

$$W_s = \frac{W_T}{1+w}.$$

⑦ $$\gamma_d = \frac{\gamma_{bulk}}{1+w}$$

8) $$\gamma_{bulk} = \gamma_d + S_r(\gamma_{sat} - \gamma_d)$$

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

10) $$i_c = \frac{G-1}{1+e} \quad (or) \quad \frac{G-1}{1-n} \quad (G-1)(1-n)$$

Critical hydraulic gradient:

$$i_c > i$$

In a wet soil mass air occupies $\frac{1}{4}$th of its volume & water occupies 1/2 of its volume. The voids ratio will be?

$$e = \frac{V_v}{V_s} = \frac{V_a + V_w}{V - V_a - V_w} = \frac{\frac{V}{4} + \frac{V}{2}}{V - \frac{V}{4} - \frac{V}{2}}$$

$$= \frac{3/4}{1/4} = 3//$$

$\boxed{e = 3}$

specific volume of soil is 1.9, the porosity =?

Specific volume $= 1+e$

$V = V_s + V_v$

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

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

$\boxed{e = V_v}$

$1+e = 1.9$

$e = 0.9$

$$n = \frac{e}{1+e} = \frac{0.9}{1.9} = 47.37\%$$

Index properties of soil.

1) Water content } Both 7.0, C.S.
2) Specific gravity → Density bottle, pycnometer
3) Particle size distribution — C.S
4) Consistency limits — Fine grained soil.
5) Density index / Relative density — coarse

$$I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100\%$$

$\boxed{0 \le I_D \le 100\%}$

e_{max} → Voids ratio in loosest state (0.91)

e_{min} → Voids ratio in densest state (0.36)

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

$$I_D = \left[\left(\frac{1}{\gamma_d}\right)_{min} - \frac{1}{\gamma_d}\right] \Big/ \left[\left(\frac{1}{\gamma_d}\right)_{min} - \left(\frac{1}{\gamma_d}\right)_{max}\right]$$

Particle size distribution:

Brownian movement effect 0.2 mm - 0.0002 mm Turbulent.

Sieve Analysis

>75 μ.

Coarse Sieve

Fine Sieve.

Dry Sieve

Wet Sieve

Sedimentation Analyses

< 75 μ.

Principle - Stokes law.

0.2 mm - 0.2 μ.

$v \propto d^2$

$v = 900d^2$

Suspension agent

(or)

Deflocating agent

33gm Sodium hexa meta phosphate +

7g sodium carbonate

Pipette — direct measurement

Hydrometer — Indirect

Accurate method.

Corrections:

1) Miniscus +ve

2) Dispersion agent corr -ve

3) Temp +ve con -ve.

T > 27°C C +ve

T < 27°C C -ve.

Size 0.2mm - 0.2 μ.

Soil

Well graded Soil

Poorly graded Soil.

Uniform

Gap/skip

$C_c = C_u = 1$

$C_c = 1 - 3$

$C_u > 6$ for sand

$C_u > 4$ for gravel.

(i) d_{10} → effective size

(ii) Uniformity coeffi.

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

(iii) Coeff of curvature

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

(1 to 3).

particle size range.

75μ - 4.75mm - Sand

4.75 - 80mm - Gravel

80 - 300mm - Cobble

> 300mm - boulder.

75μ - 2μ → Silt

< 2μ → Clay

} Fine.

4.75mm - 2mm Coarse sand

2mm - 0.425mm Medium sand.

0.425 mm - 75μ Fine sand

} Sand

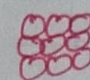
$$I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100\%$$

(i) when $e_{nat} = e_{min} \Rightarrow I_D = 100\%$. Very dense soil.

(ii) when $e_{nat} = e_{max} \Rightarrow I_D = 0\%$. Very loose soil.

I_D	Type of soil
0-15 %	Very loose
15-35	loose
35-65	Medium dense
65-85	Dense
>85 %	Very dense

For uniformly graded spherical particle.



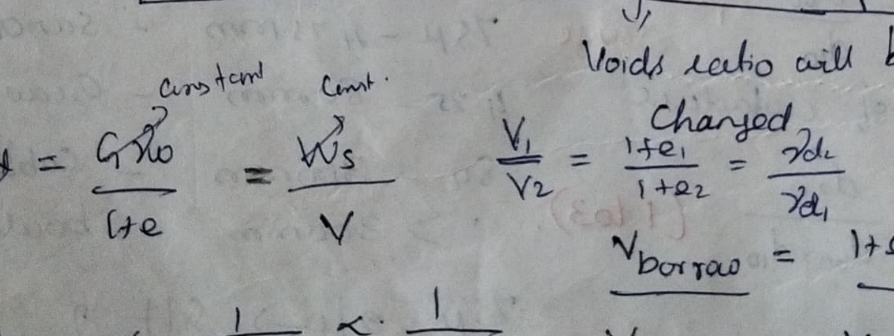
$e_{max} = 0.91$

$e_{min} = 0.35$

Interms of γ_d.

$$I_D = \left[\frac{\gamma_{d\,nat} - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right] \times \frac{\gamma_{d\,max}}{\gamma_d}$$

Borrow pit :



Voids ratio will be changed

$$\gamma_d = \frac{\overset{\text{constant}}{G\gamma_w}}{1+e} = \frac{\overset{\text{Const.}}{W_s}}{V} \qquad \frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}$$

$$\gamma_{d\,borrow} \propto \frac{1}{1+e} \propto \frac{1}{V} \qquad \frac{V_{borrow}}{V_{embank}} = \frac{1+e_{borrow}}{1+e_{embank}}$$

Angle of internal friction is increased by compaction.

A field having a vol. of 150000 cc is said to be constructed at a void ratio of 0.8. The borrow pit soil has a void ratio of 1.4. The vol. of soil required to be excavated from the borrow pit will be :-

$$\frac{V_{borrow}}{V_{ex}} = \frac{1+e_{bor}}{1+e_{ex}}$$

$$\frac{V_{borr}}{150000} = \frac{1+1.4}{1+0.8}$$

$$V_{borrow} = 200000 \text{ cc.}$$

Consistency limit: (Atterberg's limit)

- Relative ease to which we work.

$W_s = 22\%$. $W_p = 85\%$. $W_L = 125\%$.

Solid | Semi Solid | Plastic | Liquid.

Max w/c. Min w/c

Shear strength @ L.L = 0

$\tau_f = 0$ $W_n > W_L$

Volume constant — Resistance to pressure — Resistance to flow.

No change in the volume below the shrinkage limit.

$W_L > W_P > W_S$.

For moorum soil $W_P > W_L$

As water content increases

(i) Volume increases

(ii) Fluidity / flowability increases.

(iii) Shear strength decreases.

Shear strength

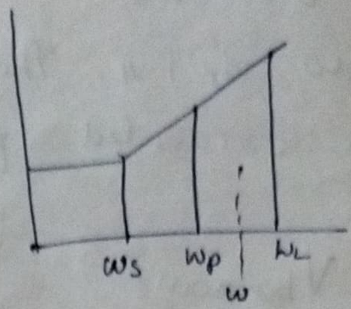
Fine grained soil.

$\tau_f = c + \sigma \tan\phi$ ($\phi = 0$)

$\boxed{\tau_f = c}$

$c = F = \frac{G m_1 m_2}{d^2}$

$c \propto \frac{1}{d^2}$



$I_p = w_L - w_P$

$I_s = w_P - w_S$

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

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

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

@ $w = w_s$ $S_r = 100\%$.

$w < w_s$ $S_r = 0-100\%$

$w > w_s$ $S_r = 100\%$.

Shrinkage limit:

- Min. water content with which a soil can be completely saturated is called Shrinkage limit

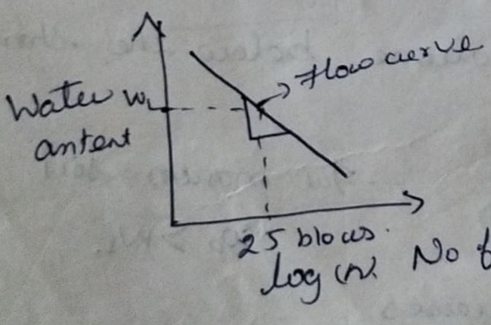
Determination of liquid limit

- ① Casagrande liquid limit
- Sieve $425\,\mu$
- → groove dimension - 12 mm
- → cup move to height of 1 cm.

② One point method $w_L = w\left(\frac{N}{25}\right)^x$

$x = 0.068$ to 1.21

③ Cone penetrometer Test (20mm)



(slope of curve) $= \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$

Rate of loss of shear strength.

(i) Flow index is the slope of the flow curve

$$I_f = \frac{dy}{dx} = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

$I_{f\,metal} > I_{f\,rubber} >$

$I_{f\,soft}$...

Plasticity index I_p (or) PI.

$$\boxed{I_p = LL - PL}$$

$(I_p)_{organic} < (I_p)_{silt} < I_{p\,clay}$

* I_p cannot be -ve.
* indicates plasticity of soil ; soil can be deformed without rupture.
* clay fraction ↑, I_p ↑
* Particle size ↓ I_p ↑
* For organic soil, LL ↑, PL ↑ But I_p ↓.
* LL & I_p is higher — fat clay
* LL & I_p is lower — lean clay.
* Non-plastic soil where LL is less than 20%.

Plasticity Index (I_p) ($w_L - w_p$)	Types of soil.
(Even if -ve) zero	Non-plastic (Sand & Gravel)
< 7 %	Low plastic
7 – 17 %	Medium plastic
> 17 %	Highly plastic (Fat clay)

Silt – 10 – 15
clay – 15 – 100

Liquidity Index (Water plasticity ratio) I_L

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

* Important role in determination of consistency of clay.

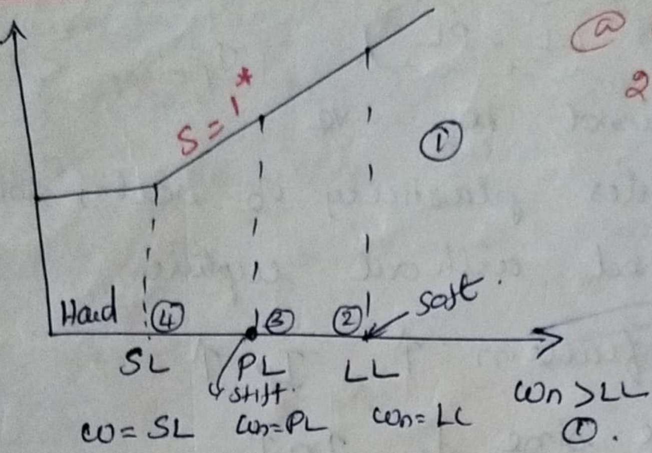
  >1, =1, 0, -ve (<0).

A fine grained soil is found to be plastic in the water content range of 26–48%. As per IS system,

CL – ML.

Significance:

Shear strength @ LL 2.7 kN/m



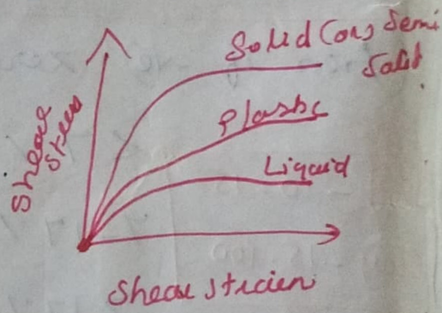
Case (i) $I_L = \dfrac{\omega - \omega_P}{\omega_L - \omega_P}$

$\omega_n > LL$

$I_L = \dfrac{\omega_L - \omega_P}{\omega_L - \omega_P} > 1$ Liquid.

(ii) $\omega_n = LL$

$I_L = \dfrac{\omega_L - \omega_P}{\omega_L - \omega_P} = 1$ very soft.

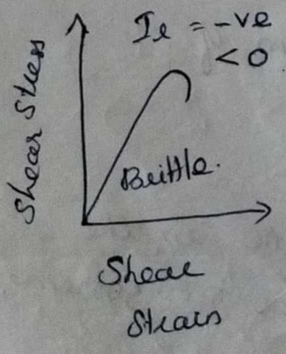


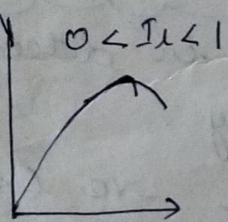
(iii) $\omega_n = PL$

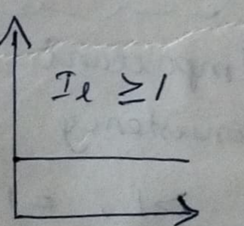
$I_L = \dfrac{\omega_P - \omega_P}{\omega_L - \omega_P} = 0$ Stiff.

(iv) $\omega_n = SL$

$I_L = \dfrac{<PL - \omega_P}{\omega_L - \omega_P} = -ve < 0$ Semi Solid state Brittle.







Liquid State

I_l	Description	$I_c = 1 - I_l$
>1	liquid state	<0
0.75 – 1	Very soft (plastic)	0 – 0.25
0.75 – 0.5	Soft (Plastic) plastic.	0.25 – 0.5
0.5 – 0.25	Medium stiff	0.5 – 0.75
0.25 – 0	Stiff	0.75 – 1
<0	Semi Solid Hard & Brittle.	> 1

Consistency Index : I_c

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

$$I_c + I_L = 1$$

Toughness Index : I_T (Shear strength @ PL

$$I_T = \frac{I_p}{I_f} = \frac{w_L - w_P}{\dfrac{w_2 - w_1}{\log_{10}\left(\frac{N_2}{N_1}\right)}}$$

I_T = 0 to 3 for clay.

$I_T < 1$ → Soil is friable at plastic limit.
easily break.

$I_f \uparrow$, Shear strength ↓, $I_T \downarrow$

Shrinkage Index. I_s

$$I_s = PL - SL = w_p - w_s$$

→ Indicates max presence of clay fraction

Linear shrinkage $LS = \left\{ 1 - \left(\frac{100}{100 + V.S}\right)^{1/3} \right\} \times 100$

Activity number: Ac.

$$A_c = \frac{I_p}{\text{\% finer than } 2\mu \text{ (clay)}}$$

$\boxed{I_p = w_L - w_p}$

Ac	Quality.
less than 0.75	Inactive soil
0.75 – 1.25	Normal soil
More than 1.25	Active soil (Black cotton) Montmorillonite (Highly Expansion & Shrinkage)

Sensitivity: St

It indicates the loss of shear strength due to remoulding of soil without chang in water content.

$$S_t = \frac{q_u \text{ (undistured) shear strength}}{q_u \text{ (remoulded) distubed shear streng.}}$$

St	Type.
1 – 4	Normal
4 – 8	Sensitive
8 – 16	ES Extra Sensitive
> 16	Quick clay.

≤ 1 = Insensitive

2–4 = low Honeycomb.

4–8 = Medium HC(or) flocculated.

8–16 → ES (flocculated)

>16 = Quick

Degree of shrinkage:

% decrease in Volume w.r to original volume

$$D.S = \frac{V_o - V_{dry}}{V_o} \times 100.$$

DS	Quality.
< 5%	Good
5 – 10	Medium
10 – 15	Poor
> 15	Very poor

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

$$VS = \frac{V_0 - V_{dr}}{V_d} \times w$$

$$SR = \frac{\frac{V_o - V_d}{V_d}}{w_L - w_s}$$

$$DS = \frac{V_o - V_d}{V_o}$$

Volumetric Shrinkage :

$$VS = \frac{V_0 - V_{dry}}{V_{dry}} \times 100 \qquad w_s =$$

Shrinkage ratio (SR).

Because of less change of deformation is less value is preferred,

$$SR = \frac{VS}{W_L - W_S} = \frac{\frac{V_0 - V_d}{V_d}}{w_l - w_s} \qquad \frac{\text{decrease in Volume}}{\text{Change in w/c}}$$

$$\frac{\gamma_d}{\gamma_w} = G_m \qquad \text{Shrinkage limit} = \frac{1}{SR} - \frac{1}{G}$$

Thixotropy :

It takes place due to reorientation of its grain to achieve a part of its original strength.

-> Pile driving.

Liquid limit is an indicator of compressibility.

If LL is more, the soil will settle more.

$LL \propto$ compressibility.

Types of clay :	Activity
1) Kaolinite	0.4 to 0.5
2) Illite	0.5
3) Montmorillonite	1 - 7.

$w_L = 35\%$ $\quad w_p = 20\%$ $\quad w_s = 10\%$. $\quad w = 25\%$.

1) $I_L = \frac{w - w_p}{w_L - w_p} = \frac{25-20}{35-20} = \frac{5}{15} = 0.33$

2) $I_p = w_L - w_p = 35 - 20 = 15\%$.

$\quad w_p - w_s = 20 - 10 = 10\%$.

Classification of soils.

- $d > 75\mu$ — Coarse grained soil
  - Gravel: 4.75 mm to 80 mm. % of retained at 4.75 mm Sieve more than 50%.
  - Sand: 4.5 mm to 75 μ. % of passing 4.75 mm Sieve & retained on 75 μ more than 50%.
- $d < 75\mu$ — Fine grained soil
  - Silt: 75 μ to 2 μ
  - Clay: < 2 μ

(1) Sedimentation Analysis.

(2) Consistency limits.

Gravel
- fines content < 5% is clean Gravel
- " 5–12% Dual symbol
- " > 12% is gravel with fines

Sand
- fine content < 5% is clean sand
- " > 12% is sand with fines.

Symbol

Gravel	– G	Well graded Gravel	GW
Sand	– S	Well graded Sand	SW
Silt	– M	Poorly graded Gravel	GP
Clay	– C	Poorly graded Sand	SP
		Organic soil	O

Gravel 40%
Sand 30%
Clay 20%
} clayey sandy gravel.

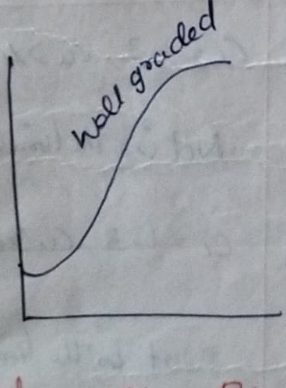
Types of classification:

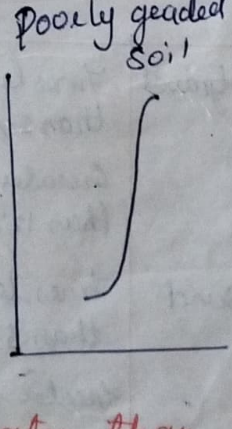
1) Unified soil classification system (US).
2) Indian Standard soil classification system
3) Highway research board classification system
4) Textural soil classification system

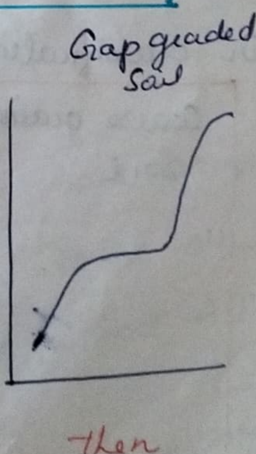
$I_{P(std)}$ = I_p for black cotton soil
Black cotton soil has high swelling & shrinkage.

I_p = 4-7 means, symbol is CL-ML.

Clay	< 2μ (0.002 mm)
Silt	2μ - 75μ (0.002 - 0.075 mm)
Fine sand	75μ - 425μ (0.075 - 0.425 mm)
Medium sand	425μ - 2mm
Coarse sand	2mm - 4.75mm
Fine gravel	4.75mm - 20mm
Coarse gravel	20mm - 80mm
Cobble	80mm - 300mm
Boulders	> 300mm.





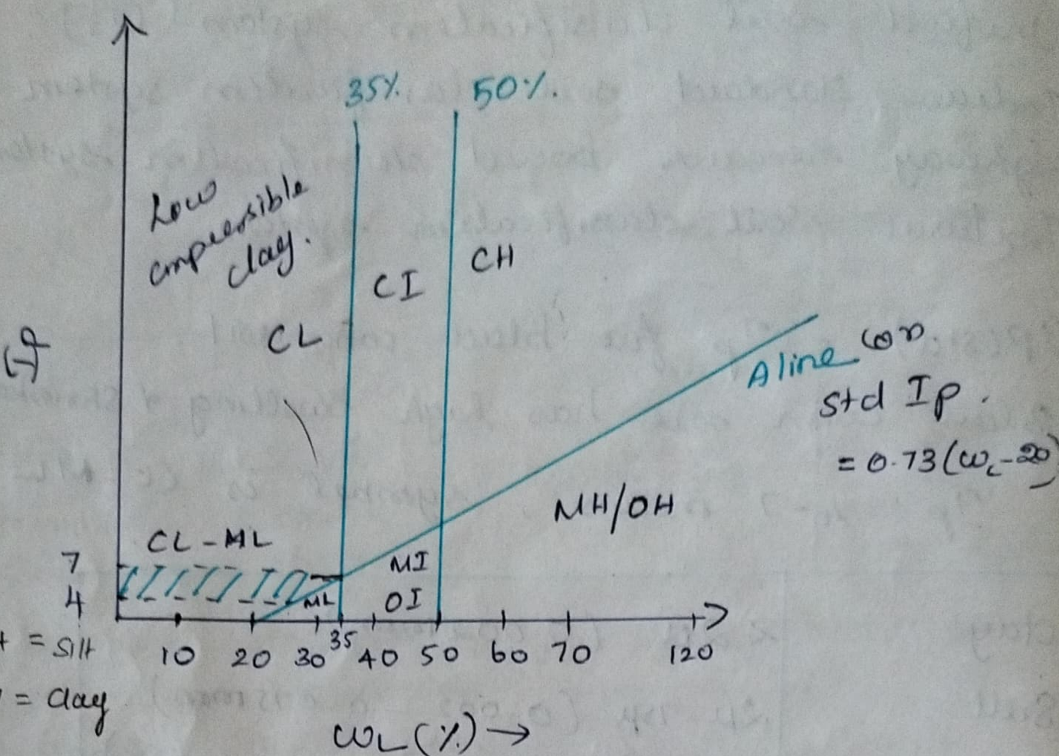


When the PL is greater than LL, then the plasticity index is reported as Zero

$w_L - w_P$ = Answer will be (-ve) (Non plastic)

It is reported as Zero.

Plasticity chart (or) Arthur Casagrande's chart



$I_p < 4$ = silt

$I_p > 7$ = clay

Above A line – clay (C). Inorganic clay.

Below A line – Silt (M), organic (O). Inorganic silt

wL

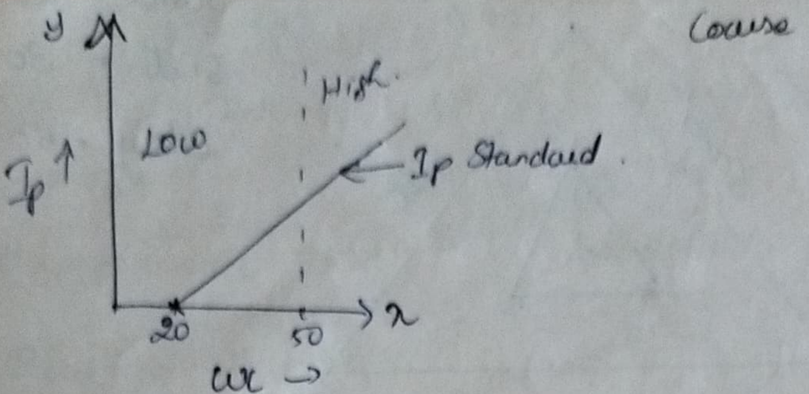
Low compressible	Intermediate	High
$\boxed{w_L < 35\%}$	$\boxed{w_L = 35-50\%}$	$\boxed{w_L > 50\%}$

Silt ⊖ $I_{P(Std)} > I_P$ | clay $I_{P(Std)} < I_P$.

Orgin classification

Coarse grained soil	Gravel	Fines less than 5%.	GW	$C_c = 1-3$, $C_u > 4$
		Greater than 12%.	GP	Not in the limit.
	Sand	Fines less than 5%.	SW	$C_c = 1-3$ $C_u > 6$
		Greater than 12%.	SP	Not in the limit
Fine grained soil	Silt Clay			Refer Plasticity chart

US classification:



Highway Research Board classification.

- Deals with coarse grained soil

Group index $= 0.2a + 0.005ac + 0.01bd$

GI value = 0 to 20

a → % of passing 75 μ sieve greater than 35 $(x-35)$

b → % of passing 75μ sieve greater than 15 $(x-15)$

c → Liquid limit greater than 40 $(w_L - 40)$

d → plasticity Index greater than 10 $(I_p - 10)$

If GI value is minimum (ie) 0, the soil is good condition.

If GI value is maximum (ie) 20, the soil is bad condition. $\boxed{GI\ 0-20}$

The laboratory test results of a soil sample are given below: % of passing 75 μ sieve is 60%, $w_L = 53\%$, $I_p = 20$. Find GI?

$a = x - 35 = 60 - 35 = 25$

$b = x - 15 = 60 - 15 = 45$

$c = w_L - 40 = 60 - 40 = 20$

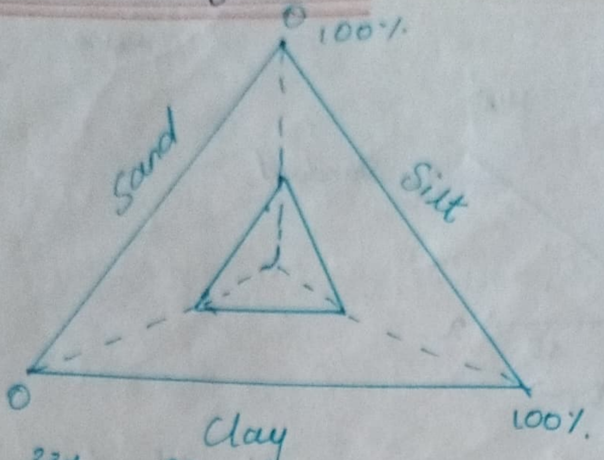
$d = I_p - 10 = 20 - 10 = 10$

$GI = 0.2a + 0.005ac + 0.01bd$

$GI = 11.125$

$= 12$

Textural classification.



Sand = 20%
Silt = 30%
Clay = 50%

Sand (33%), silt (33%), clay (33%) in equal proportion called as loam

A soil is having $w_L = 80\%$, $w_P = 35\%$. to classify the soil.

$w_L > 50\% \Rightarrow$ High compressible soil.

$I_P = w_L - w_P = 80 - 35 = 45\%$.

$I_{P(std)} = 0.73\,(w_L - 20) = 0.73 \times 60 = 43.8$.

$I_{P(std)} < I_P . \Rightarrow$ clay.

Highly compressible clay. **CH Soil**

Compaction:

→ To decrease the void ratio, porosity

→ To increase density.

SBC ↑ $\propto \gamma \uparrow \propto \gamma \uparrow \propto \frac{1}{1+e} \downarrow$

For compaction, we require.

(i) Amount of energy

(ii) Amount of water

(iii) How to impact energy?

Energy

JS: Light energy

γd_1

Std. proctor test

JS: Heavy energy.

γd_2

Modified proctor test.

Std Proctor test:

Heavy energy / MPT energy = 4.9 kg (wt of hammer) * 45 cm (Ht of fall) × 25 × 5 layer.

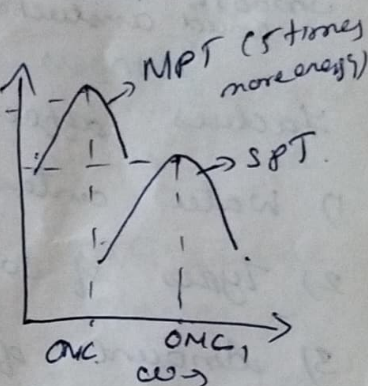
$$\frac{MPT}{SPT} = \frac{4.9 \times 45 \times 25 \times 5}{2.6 \times 30.5 \times 25 \times 3} = \underline{\underline{5}}$$

With increase in compaction energy,

(i) $\gamma d \rightarrow$ Max.

(ii) OMC $\rightarrow$ Decreased.

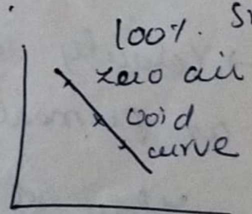
(iii) density curve gets shifted to left upwards.



Theoretical dry density / Zero air voids density $\gamma_d = \dfrac{(1-\eta_a) G \gamma_w}{1+\omega G}$

$\eta_a = \dfrac{V_a}{V}$; $\eta_a = 0$ $V_a = 0$.

$$\gamma_{d\,max} = \frac{G \gamma_w}{1+\omega n}.$$



$(\gamma_{d\,max})_{field} < (\gamma_{d\,max})_{lab} < (\gamma_{d\,max})_{theoretical}$

Field density < Proctor density < ZAVL

Relative compaction $R.C = \frac{(\gamma_d)\ field}{(\gamma_d)\ lab}$

$RC = 0.95 - 0.96$

The water content to the site after compaction is called placement water content.

Vibrator roller - Coarse grained soil, sand. Embankment

Sheep foot roller - plastic clay soil, cohesive soil

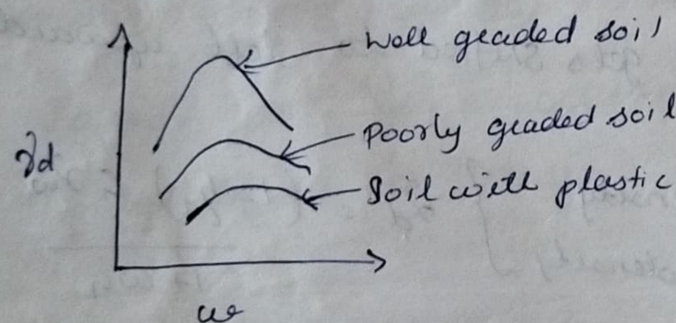
Pneumatic roller - Cohesive & cohesionless soil
Highways, earth dam

Smooth wheeled roller - Used for levelling on compact coarse soil
Drum cor
road construction
To drain away the water.

Tampers - Retaining wall.

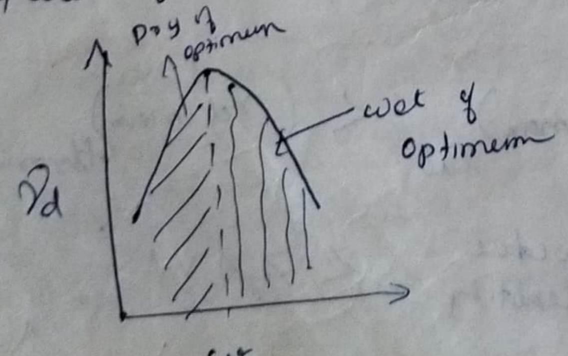
Factors affecting compaction.

1) Water content.
2) Types of soil
3) Amount of energy



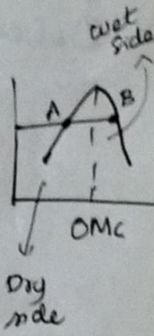
→ Velocity of water inside the soil is known as permeability (K)

→ Flow of water is known as seepage.



To achieve shear strength soil should be compacted at OMC.

Property of Soil	Dry of optimum	wet of optimum
1. Shear strength	more	Less
2. Permeability	more	less
3. Swellability	more	less.
4) Shrinkage	Less	More
5) Compressibility	Less	More.



Types of project	Compaction under content zone	Reason.
1) Embankment / Slope Stability	Dry of optimum	To have adequate Shear Strength.
2) Highway subgrade	wet of optimum	Min. volume changes.
3) Core of an earthen dam	wet of optimum	To have less permeability.
4) Shell of an earthen dam	Dry of optimum	To have more Shear Strength.

Specific gravity ②.

Wet soil

$$G = \left[\frac{W_2 - W_1}{W_3 - W_4} \left(\frac{G-1}{G} \right) - 1 \right]$$

Dry soil

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

1) The porosity of a certain soil sample was found to be 80% & Specific gravity 2.7 Determine the critical hydraulic gradient (i_c).

$i_c = (G-1)(1-n)$ $\qquad \left(\frac{G-1}{1+e}\right) = i_c$

$= (2.7-1)(1-0.8)$

$i_c = 0.34$

2) The void ratio is equal to unity, what will be the porosity.

$n = \frac{e}{1+e} = \frac{1}{2} = 0.5$ $\qquad \left(e = \frac{n}{1-n}\right)$

3) The natural water content of the soil sample was found to be $w = 40\%$. $G_s = 2.7$ $e = 1.2$, Determine degree of saturation.

$wG = S_r e$ $\qquad \frac{10}{25} = \frac{2}{5} = 0.4$

$\cancel{8}\ 0.4 \times 2.7 = S_r \times 1.2$

$\boxed{S_r = 0.9}$

4) The volume of soil in natural State is 1300 due to heavy compaction its volume becomes 1000 cc. The volume of the same soil in loosest condition is 1700 cc. Determine I_D.

$$I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}}$$

$$e_{max} = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{1700 - V_s}{V_s}$$

$$e_{min} = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{1000 - V_s}{V_s}$$

$$e_{nat} = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{1300 - V_s}{V_s}$$

$$I_D = \frac{\frac{1700 - V_s}{V_s} - \frac{1800 - V_s}{V_s}}{\frac{1700 - V_s}{V_s} - \frac{1000 - V_s}{V_s}} = \frac{400}{700}$$

$I_D = 0.57$ 57% (35% to 65%
Medium sand

6) The mass of dry sand was 198.6 gm. The mass of the flask filled with water was 1508.2 gm. the mass of the flask, water, sand was 1632.6 gm. Determine G.

$w_2 - w_1 = 198.6$ g

$w_4 = 1508.2$ gm

$w_3 = 1632.6$ gm

$$G = \frac{w_2 - w_1}{(w_4 - w_1) - (w_3 - w_2)}$$

$$= \frac{w_2 - w_1}{w_4 - w_1 - w_3 + w_2}$$

$$= \frac{w_2 - w_1}{w_2 - w_1 + w_4 - w_3} = \frac{198.6}{198.6 - 1632.6 + 1508.2}$$

$$\boxed{G = 2.7}$$

7) Determine the value of γ_d for spherical soil grains arranged in cubical array having $G_s = 2.65$.

$$\gamma_d = \frac{G \gamma_w}{1+e} = \frac{2.65 \times 10}{1 + 0.91} = 13.87 \ kN/m^3$$

$e_{min} = 0.35$ (Face centered) $e_{max} = 0.91$ (Cubical array)

Unit - II

Effective stress and permeability.

Effective pressure in soil:

(1) Total pressure (or) Total stress (σ)

It is the total wt of soil mass including water present in soil pore (or) water above the soil mass per unit of surface area of soil mass.

$$\sigma = \frac{W}{A} = \frac{\gamma \times V}{A} = \frac{\gamma \times A \times h}{A}$$

$$\boxed{\sigma = \gamma h}$$ Load cell.

2) Pore water pressure (or) Neutral Pressure (u)

It is the pressure developed by water available in soil pores due to presence of water table. → compressive / tensile

$u = \gamma_w h_w$ (only for pond, lake)

Measured by pitot tube.

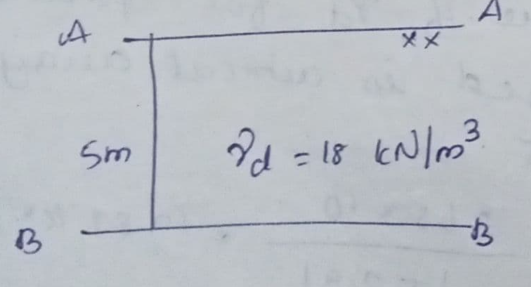
3) Effective pressure (σ'). Intergranular pressure

→ Cannot be measured directly.

$\sigma' = \sigma - u.$ transmitted through point of contact

case i)

To find effective stress @ point A & B



Layer A-A.

$\sigma = 0\ kN/m^2$

$u = 0\ kN/m^2$

$\sigma' = \sigma - u = 0\ kN/m^2$

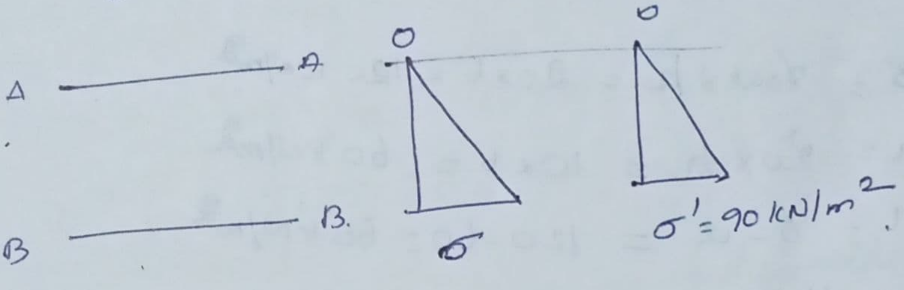
- WT above GL, will not affect eff. stress.
- WT below GL will affect the eff. stress.

Layer B-B

$\sigma = \gamma_d \times h = 18 \times 5 = 90\ kN/m^2$

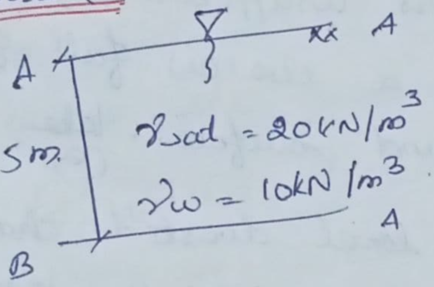
$u = 0$

$\sigma' = \sigma - u = 90\ kN/m^2$.



case (ii)

2)



Layer A A

$\sigma = 0$, $u = 0$, $\sigma' = 0$.

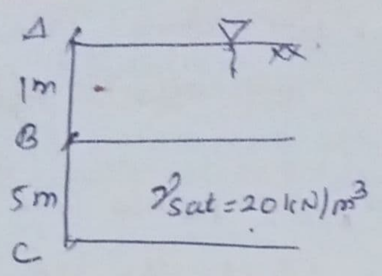
Layer B.B

$\sigma = \gamma_{sat} \times h = 20 \times 5 = 100\ kN/m^2$

$u = \gamma_w h_w = 10 \times 5 = 50\ kN/m^2$

$\sigma' = \sigma - u = 100 - 50 = 50\ kN/m^2$.

Case iii)



A - A $\quad \sigma = u = \sigma' = 0$

B . B $\quad \sigma = \gamma_{sat} \times h_1 = 20 \times 1 = 20 \ kN/m^2$

$u = \gamma_w \times h_1 = 10 \times 1 = 10 \ kN/m^2$

$\sigma' = \sigma - u = 20 - 10 = 10 \ kN/m^2$

C - C $\quad \sigma = \gamma_{sat} \times h = 20 \times 6 = 120 \ kN/m^2$

$u = \gamma_w \times h = 10 \times 6 = 60 \ kN/m^2$

$\sigma' = \sigma - u = 120 - 60 = 60 \ kN/m^2$

Effective (σ') stress remains unaffected (or) constant when there is a rise (or) fall of water above the ground surface. ; lake (eg).

Assuming the river bed level doesn't change and depth of water in river was 10m, 15m, & 8m during the month of February, July, Dec in the respective year $\gamma_{sat} = 20 kN/m^3$ $\gamma_w = 10 \ kN/m^3$, σ' at a depth of 10m below ground surface is 100, 100, 100 kN/m^2
F J D.

$\sigma' = \sigma - u$

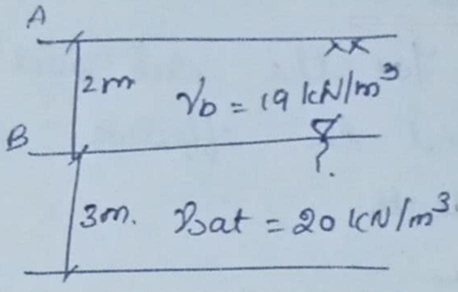
$= \gamma_{sat} h - \gamma_w h_w$

$= 20 \times 10 - 10 \times 10$

$\sigma' = 200 - 100 = 100 \ kN/m^2$

Case (iv)

Layer B.B



$\sigma = \gamma_b \times h_1 = 19 \times 2 = 38\ kN/m^2$

$u = 0$

$\sigma' = 38 - 0 = 38\ kN/m^2$

Layer CC

$\sigma = \gamma_b \times 2 + \gamma_{sat} \times 3$

$= 19 \times 2 + 20 \times 3 =$

$\sigma = 98\ kN/m^2$

$u = \gamma_w \times H_2 = 10 \times 3 = 30\ kN/m^2$

$\sigma' = 98 - 30 = 68\ kN/m^2$.

* Effective stress increases when there is a decrease in level of WT (dewatering) below the ground surface.
* Eff Stress decreases when there is a rise in water table below the ground surface.

Below Ground level, WT $\uparrow$ $\sigma' \downarrow$
WT $\downarrow$ $\sigma' \uparrow$

Effect of capillarity:

Surface tension is responsible for capillarity

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

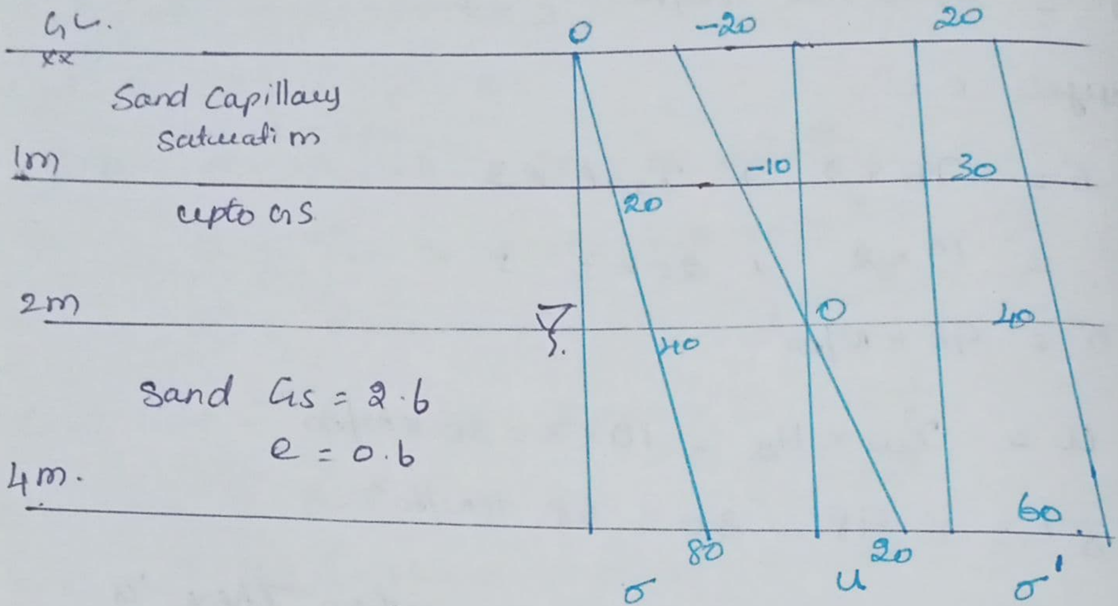
σ – surface tension
$\theta = 0°$ for distilled water
d – dia of tube.

$$h_c \propto \frac{1}{d}$$

According to h_c,
Gravel $\leq$ Sand $<$ Silt $<$ clay $<$ colloid

Case III

For the sub soil conditions shown in fig, find the effective stress values @ 1m, 2m and 4m depths.



$$\gamma_{sat} = \frac{(G+e)\,\gamma_w}{1+e} = \frac{(2.6+0.6)\,10}{1+0.6}$$

$$\gamma_{sat} = 20\ kN/m^3.$$

1) At depth $z = 0m$.

$\sigma = 0\ kN/m^2$

$u = -\gamma_w h_w = -10\times 2 = -20\ kN/m^2$.

$\sigma' = \sigma - u = 20\ kN/m^2$

2) @ depth $z = 1m$.

$\sigma = \gamma_{sat} \times 1 = 20\ kN/m^2$

$u = -\gamma_w h_w = -10\times 1 = -10\ kN/m^2$.

$\sigma' = \sigma - u = 30\ kN/m^2$.

3) @ depth $z = 2m$

$\sigma = \gamma_{sat} \times 2 = 20\times 2 = 40\ kN/m^2$

$u = 0$ @ WT $= 0$ $\sigma' = 40\ kN/m^2$

@ $z = 4m$.

$\sigma = \gamma_{sat} \times 4 = 20 \times 4 = 80 \; kN/m^2$

$u = \gamma_w h_w = 10 \times 2 = 20 \; kN/m^2$

$\sigma' = 80 - 20 = 60 \; kN/m^2$

Capillary fringe $\sigma' > \sigma$

capillary causes

* Bulking of sand
* Capillary siphoning
* freezing & thawing in back fill.
* Capillary increases the shear strength, but it is not reliable.

soil water

- Structural water
  * crystalline structure
  * can't be removed by oven drying method
- Adsorbed water
  * water held by electrochemical forces.
  * Impt for clay
  * Coarse grained soil — negligible
  * Hygroscopic water ← depend on size of particle. fine grain > Coarser. clay > sand. (More specific surface area)
  * Can be removed by oven drying method.
- Capillary water
  $h_c = \frac{0.3}{d}$
  Clay - Max
  More than 25 m
  $h_c \uparrow$ for c.g.s.
  $h_c \downarrow$, fine soil

Frost heave:

- water rises from capillary fringe may freeze
→ temp below freezing point & ice formed.
→ Causes increase in volume of soil. → uplift of structure
→ Silt & fine sand (eg)

Frost boil :

→ Process of softening of soil due to increase in water content caused by melting (or thawing) of ice formed in the capillary fringe.

→ silt & fine sand.

To prevent

* Replace by coarse grained sand (15 to 30 cm thick).

1) Capillary rise (or) fringe.

$\sigma' = \sigma + u$ $\quad u = -\gamma_w h_w$.

$\sigma' \uparrow$

2) Due to rise of water Table

$w = 100\%$

$e\hat{S} = wG$

$\gamma_{sat} \rightarrow \gamma_{sub}$. $\quad \sigma' \downarrow$

3) Due to lower water table.

$\gamma_{sub} \rightarrow \gamma_{bulk}$ $\quad \sigma' \uparrow$

A soil sample is found to have the following

Soil type	Coeff. of permeability (cm/sec)
Gravel	1 to 100
Sand	10^{-3} to 1
Silt	10^{-6} to 10^{-3}
Clay	$< 10^{-6}$.

Permeability : Hydraulic conductivity.

It is defined as the ease due to which the liquid (or) water passes through the interconnecting voids of the soil mass.

Principle : Darcy law (1856).

$\boxed{V_{av} \propto i}$ (laminar flow). seepage flow

Average velocity $\propto$ hydraulic gradient.

$V = k\,i$ — V → Superficial velocity, Discharge velocity

× A

$V \cdot A = k\,i\,A$ — cm/sec (LT^{-1}).

$Q = k\,i\,A$.
(seep)

Re < 1. $V = \frac{Q}{t}$

$\boxed{i = \frac{\Delta H}{L}}$.

Coeff. of permeability (or) Hydraulic conductivity (k).

$\boxed{V = k\,i}$

$k = \frac{V}{i}$ — permeability coarse > fine

1) Porous soil → k – high $k > 10^{-4}$ cm/s
gravel, coarse & medium sand

2) Semi pervious soil : k 10^{-4} cm/s to 10^{-6} cm/s.
Fine sand & silt

3) Impervious soil : $k < 10^{-6}$ cm/s.
clay. 10^{-6} to 10^{-8} cm/s.

$k_{gravel} > k_{\text{coarse sand}} > k_{medium} > k_{fine} > k_{silt} > k_{clay}$.

	Gravel	Sand	silt	clay	
		1 to 10^{-3}	10^{-3} to 10^{-7}		10^{-7}
cm/sec k cm/sec	1	10^{-2}	10^{-4}	10^{-6}	

Factors affecting permeability:

① $\boxed{k \propto D_{10}^2}$ ← ② Allan Hazen eqn

$k_{coarse} > k_{fine}$ $k = C D_{10}^2$ $\boxed{C = 100}$

$\boxed{k = 100 D_{10}^2}$

③ Terzaghi eqn:

$k = 200 e^2 D_e^2$

$(S_s)_{particle} = (S_s)_{sphere} = \frac{6}{D_e}$

② $k = \frac{e^3}{1+e}$ $k\uparrow \rightarrow e\uparrow$ Indirect method.

① Kozeny Karman eqn

$k = \frac{1}{k_k}\left(\frac{\gamma_w}{\mu}\right)\left(\frac{e^3}{1+e}\right)\left(\frac{1}{S_s^2}\right)$ ($\frac{1}{S_s^2} \to D_{10}^2$)

fluid medium

③ $k \propto \frac{1}{\mu}$ $k\uparrow$ $\mu\downarrow$

$S_s \rightarrow$ Specific surface area

④ $k \propto \gamma_w$ $k \propto \frac{\gamma_w}{\mu}$

Spherical: $\frac{1}{S_s^2} = d^2$, $S_s \propto \frac{1}{d}$

Not sphere: $S_s = \frac{6}{\sqrt{A \cdot B}}$

A - passing
B - Retaining

$S_s = \frac{\text{Surface area}}{\text{Unit volume}}$

$S_s = \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \frac{3}{r}$

$S_s = \frac{6}{d}$

⑤ Temp: $T\uparrow$ $\mu\downarrow$ $k\uparrow$

⑥ Soil structure:

Flocculated structure → $k\uparrow$

Dispersed structure → $k\downarrow$

⑦ Pattern of seepage flow

Seepage flow $\parallel^{el}$ to the bedding plane → $k\uparrow$

Seepage flow $\perp^{re}$ to the bedding plane → $k\downarrow$

⑧ Due to entrapped air in voids, $k\downarrow$

9) Due to adsorbed water, the void ratio gets decreased by 10%. $(e - 0.1) \rightarrow k\downarrow$

10) Degree of saturation.

$k \propto S_r^3$

11) $k \propto \frac{1}{\text{Specific surface area}}$

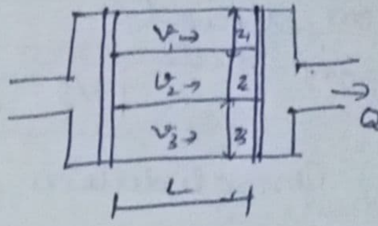
$K_{angular} < K_{rounded}$ particle

⑪ Loudon's eqn: $\log_{10}(k S_s^2) = a + bn$

a = 13
b = 5.1

case (i) Horizontal flow

Seepage flow $\parallel^{elto}$ to bedding plane



$$Q = Q_1 + Q_2 + Q_3$$

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

loss of head is same

case ii' Vertical flow

Seepage flow $\perp^{re}$ to beddingplane

$$k_v = \frac{z_1 + z_2 + z_3}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}}$$

$h_f = h_1 + h_2 \ldots$

$k_H > k_V$

$$k_{equivalent} = \sqrt{k_V \times k_H}.$$

Capillary pressure is more in fine grained soil.

Determination of coefficient of permeability)

Laboratory test :

(c) Capillarity Permeability Test – partially saturated soil.

(a) Constant Head permeameter test

($\rightarrow$ Coarse grained soil.

$$k = \frac{QL}{Aht}$$

$Q = k i A$ $\quad i = \frac{h}{l}.$

$Q = k \frac{h}{l} \frac{\pi}{4} D^2$ $\quad Q = \frac{V}{t}$

D – Dia of Sample.

l – length of Sample

(b) Falling head permeameter test.

$\rightarrow$ Fine grained soil.

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

A $\rightarrow$ Area of Sample.

a $\rightarrow$ Area of Standpipe $\quad = \frac{2.3\,aL}{A(t_2-t_1)} \log_{10}\left(\frac{h_1}{h_2}\right)$

Seepage velocity (or actual (or) true $\boxed{V_s = \frac{V}{n}}$ discharge velocity. $n = \frac{e}{1+e}$

$\boxed{V_s > V}$.

Consolidation test → clay $k < 10^{-6}$ cm/sec

Capillary - permeability test → partially saturated soil

pumping out test → Large engg. projects. expansive.

pumping in test → used for testing small area rocks & individual stratum.

$V_s = k_p\, i$

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

$V = k\, i$

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

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

If the Volume of voids is equal to the volume of solids in a soil mass, then the values of porosity & voids ratio.

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

$e = \frac{V_v}{V_s} = 1$ $\quad e = \frac{n}{1-n} = \frac{1}{1}$

$n = \frac{e}{1+e} = \frac{1}{2} = 0.5$

$n, e.\ 0.5, 1$ //

If the water content of a fully saturated soil mass is 100%, then the voids ratio of the sample.

$S_r = 100\%$.

$w = 100\%$.

$wG = S_r e$

$\boxed{e = G}$

Seepage (Casagrande)

Seepage through an eathen dam takes place due to hydraulic gradient & permeability of the soil where darcy law is applied.

$$\text{Seepage Force} = \underset{\text{Seepage pressure (Ps)}}{i\ \gamma_w} \times \text{Volume}$$

hydraulic head

$= \gamma_w \times h_w \times A$

$= \gamma_w \times \left(\frac{h_w}{L}\right) \times A \times L$

$= \gamma_w\, i\, \underline{A \times L}$

Stream line, path line, phreatic line (Parabolic curve)

Total head = pressure head + velocity head + datum head (or) elevation head.

Velocity head is neglected.

Total head = Pressure head + datum head

$$\boxed{H = h_w \pm z}$$

Note:

1) Seepage pressure always acts in the direction of flow.
2) Vertical effective pressure increased due to seepage pressure. depends on direction of flow.
3) Effective stress increases if flow is downwards.
4) Effe stress decreases if flow is upwards.

phreatic line +ve pressure / top line pressure.

Quick sand condition (or Boiling condtn.)

If upward seepage pressure becomes equal to submerged wt of soil, effective pressure reduces to zero.

In this case sand looses all its shear strength & soil particles move in upward direction.

.. Occurs in cohesionless soil. (Silts & fine sands)
($i_c = 1$)

$i \rightarrow i_c$ @ quick sand condn

$\boxed{i_c = \frac{\gamma'}{\gamma_w}}$ $\boxed{i_c = \frac{G-1}{1+e} \text{ (or } (G-1)(1-n))}$

i_c not affected by depth of water.

Factor of safety against quick sand or piping

$F = \frac{i_c}{i}$ exit or permissible gradient.

1) Foundation soil at the toe of a dam has a voids ratio of 0.62, $G = 2.62$. $F = 5$. permissible exit gradient?

$F = \frac{i_c}{i}$ $i_c = \frac{(G-1)}{1+e} = \frac{1.62}{1.62} = 1$

$5 = \frac{1}{i}$ .. $i = \frac{1}{5} = 0.2$

$i = 0.2$

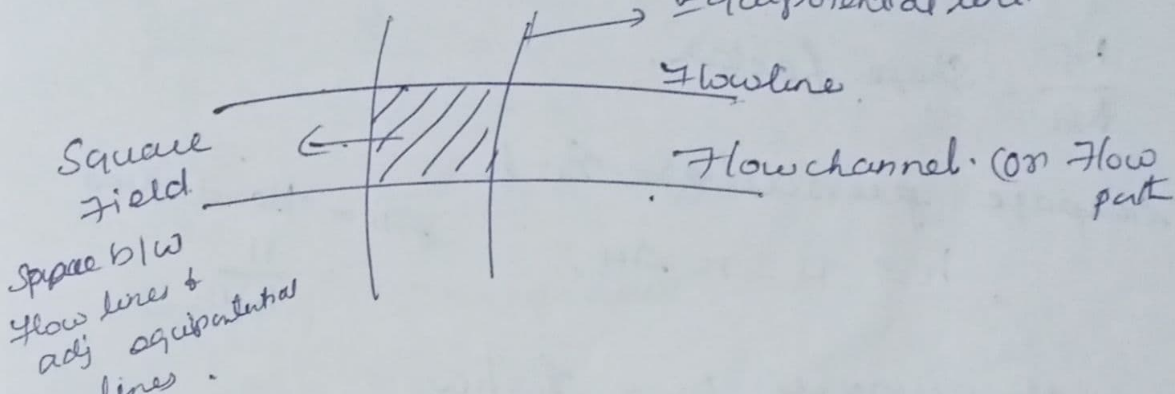
Flow net :

→ Combination of flow lines and equipotential lines.

Flow line → along water movement (stream line)

Equipotential line → line joining points having equal total head.

@ EL, - Total head is constant, But pressure head, elevation heads are different.



Characteristics of flow net :

1) FL ⊥^{re} EL
2) Quantity of seepage in each flow is same.
3) Drop in head b/w adjacent EL is same.
4) Two flow lines (or) two EL never meet each other
5) Flow net depends on boundary conditions only.

Not depend on K, H.

Uses: To find

1) Seepage loss
2) Seepage pressure
3) Uplift pressure
4) Exit gradient.

Seepage quantity:

$$q = kH\left(\frac{N_f}{N_d}\right)$$

$$k = \sqrt{k_x k_y}$$ k_x = permeability in h2l dirⁿ, k_y = " " Vert "

H → Difference b/w U/S & D/S water level.

N_f → No. of flow channel

N_d → No of potential drop.

$\frac{N_f}{N_d}$ = Shape factor.

seepage pressure (P_s) = $\gamma_w h$

$h = H - n\Delta H$. ΔH = Head drop $= \frac{H}{N_d}$.

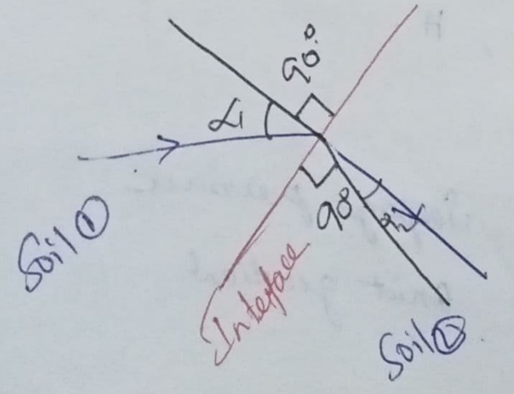
Uplift pressure $p_u = \gamma_w h_w$

h_w = total head - elevation head.

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

For safety, $\boxed{i_{exit} < i_c}$

Filters (graded (or) inverted) are provided at exit point to prevent escape of soil particles & to check piping.



$$\frac{k_1}{k_2} = \frac{\tan\alpha_1}{\tan\alpha_2}$$

Laplace equation: (Seepage flow).

Assumption:

1) Soil is homogeneous, isotropic, fully saturated and incompressible
2) Seepage water (or) pore water is incompressible
3) Darcy law is applied
4) Law of conservation of mass.

Applying continuity eqn,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0.$$

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0 \quad —①.$$

Applying Darcy law,

$$v = ki$$

$$v_x = k_x i_x \qquad v_z = k_z i_z$$

$$v_x = k_x \frac{\partial h}{\partial x} \qquad v_z = k_z \frac{\partial h}{\partial z}$$

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial h}{\partial z}\right) = 0.$$

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0.$$

For isotropic soil, $k_x = k_z$.

$$\boxed{\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0.}$$

Laplace eqn interms of head of water

$$\frac{\partial^2 (k_x h)}{\partial x^2} + \frac{\partial^2 (k_z h)}{\partial z^2} = 0.$$

$k \cdot h$ = Velocity potential (ϕ)

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0.$$ Laplace eqn of 2D flow

For anisotropic soil $k_x \neq k_z$

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0$$

$$\frac{k_x}{k_z} \frac{\partial^2 h}{\partial x^2} + \frac{k_z}{k_z} \frac{\partial^2 h}{\partial z^2} = 0.$$

$$\frac{1}{k_z/k_x} \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

$$\frac{\partial^2 h}{\partial \left(x \sqrt{\frac{k_z}{k_x}}\right)^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

$$\frac{\partial^2 h}{\partial x_t^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

$x_t \rightarrow$ Transformed section of 'x' direction

$$= x \sqrt{\frac{k_z}{k_x}}$$

$$x = \sqrt{\frac{k_z}{k_x}} \qquad z = \sqrt{\frac{k_x}{k_z}}$$

FOS against uplift $= \dfrac{W_{soil}}{\text{Uplift pressure}}$ (Submerged)

$$= \frac{\gamma_{sub} \times h_{soil}}{\gamma_w \times h_w'}$$

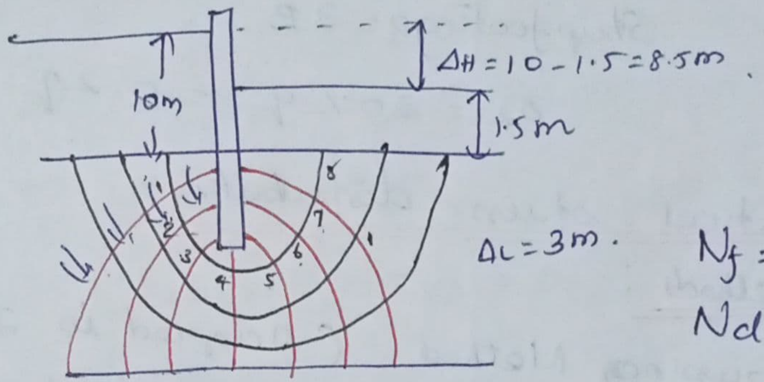
Sheet pile:

The flow net along a sheet pile is shown in fig. $k = 0.09$ m/day $G_s = 2.7$

$e = 0.85$

(i) Determine seepage loss in $m^3/d/m$ length

ii) FOS against piping.



$\Delta L = 3m.$

$N_f = 4$

$N_d = 8$

$$q = k \frac{\Delta H}{N_d} \times N_f = 0.09 \times \frac{8.5}{8} \times 4$$

$$q = 0.3825 \ m^3/day/m$$

$$F.OS = \frac{i_c}{i} = \frac{\frac{G-1}{1+e}}{\Delta h/\Delta L} = \frac{\frac{2.7-1}{1+0.85}}{\frac{\Delta H/N_d}{\Delta l}}$$

$$F.OS = 2.59.$$

If the angle made by the seepage line with the normal, $\alpha_1 = 60^\circ$, $\alpha_2 = 30^\circ$ what will be the coeff. of permeability, if $k_1 = 0.01$ cm/sec

$$\frac{k_1}{k_2} = \frac{\tan \alpha_1}{\tan \alpha_2}$$

$$k_2 = 3.33 \times 10^{-3} \ cm/sec.$$

Unit - III Stress distribution & Settlement

The depth of soil below the base of the footing were the max consolidation of the soil occurs is known as significant depth. $D_S = 1.5\,B$

Strip footing $D_S = 3\,B$.

$\sigma_z = 20\%\; q = 0.2 q$.

Vertical stress distribution:

Methods:

1) Boussinesq Method (Adopted in India)
2) Westerguard method (Stratified soil)
3) Newmark's chart (Applicable for all shapes)
4) 2 : 1 (2V : 1H) method (Stress is uniform which is not true)

1. Boussinesq method:

Assumption.

1) Homogeneous, isotropic, weightless. Semi-infinite.
2) Load is vertical.
3) ~~Load~~ Load is independent of E & μ.
4) Vertical stress depends on lineal elastic property of the soil.
5) $\sigma_z \propto \frac{1}{z^2}$
6) $\sigma_z \propto Q$

$$\sigma_z = \frac{Q}{z^2} \times k_B .$$

Bousinesq stress coeff $k_B = \frac{3}{2\pi} \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$

1) when $z = 0$ top $\sigma_z = \infty$

centre line of loading $r = 0$ $\sigma_z = 0.477 \frac{Q}{z^2}$ (Max).

2) $r = \infty \Rightarrow \sigma_z = 0$

$z = \infty \Rightarrow \sigma_z = 0.$

3) $r = 0$ Shear stress $\tau_{rz} = 0$

$$\tau_{rz} = \frac{3Qz^2}{2\pi(r^2+z^2)^{5/2}} \times r.$$

For circular footing:

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

(or)

$\sigma_z = q\left[1 - \cos^3\theta\right]$

For line load, Railway line.

$$\sigma_z = \frac{q'}{z} \times \frac{2}{\pi} \times \frac{1}{\left[1+\left(\frac{x}{z}\right)^2\right]^2}.$$

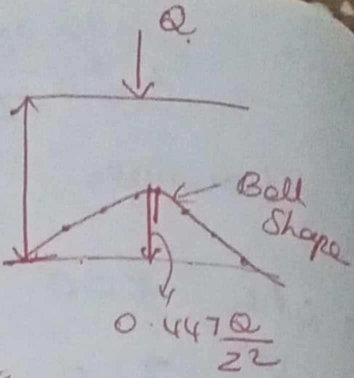
For strip footing

$$\sigma_z = \frac{q}{\pi}\left[2\theta + \sin 2\theta\right]$$

$q \rightarrow$ kPa, $\theta \rightarrow$ radian

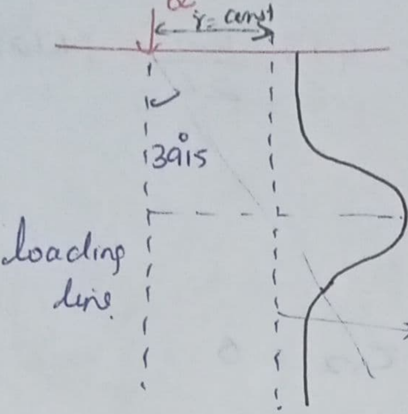
$\tan\theta = \frac{b}{z} = \frac{B/2}{z}$

Case (i) Depth constant.



Case ii) r = Const.

D = varies.



$0.133 \times \frac{Q}{Z^2}$

(σ_z)

$\sigma_{z\,max} = \frac{0.133 Q}{Z^2}$ $\quad \frac{r}{Z} = 0.817$ maximum

$\sigma = Q$

→ radial distance constant line.

Case iii) Vertical compressive stress due to rectangular / square load.

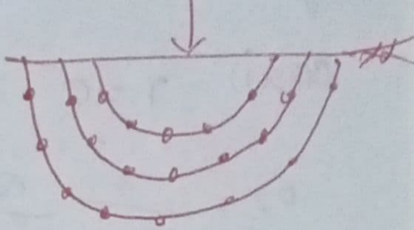
$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\,(m^2+n^2+1)^{1/2}}{m^2n^2+m^2+n^2+1} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\left[\frac{2mn\,(m^2+n^2+1)^{1/2}}{m^2+n^2+1-m^2n^2}\right]\right]$$

$x = m^2+n^2+1$ $\qquad m = \frac{L}{Z}$

$y = mn$ $\qquad n = \frac{B}{Z}$

Stress contour ~~or~~ Isobar (or) pressure bulb:

The line joining the points of same stress is known as stress contour



* Pressure intensity max in inside stress contour
* Pressure intensity min. in the outside stress contour.

Newmarks Influence chart:

* Irregular shape.
* This chart is based on ~~Newmarks~~ Boussinques ~~Influe~~ theory.
* No. of circles - 10
* No of radial line - 20.
* radius of 10th circle is infinity.

$$\sigma_z \propto n q \qquad I \rightarrow \text{Influence factor}$$

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

$$I = \frac{1}{\text{no. of circle} \times \text{no of radial lines.}}$$

$n \rightarrow$ Number of sectors (or) area units occupied by footing

1) A concentrated load of 2000kN is applied at the ground surface. Determine the vertical stress at a point P. which is 6m directly below the load. Also calculate vertical stress at a point R which is at a depth

of 6m at a horizontal distance of 5m from the load.

case (i) $r = 0 \quad z = 6m$

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

$$= \frac{3}{2\pi} \times \frac{2000}{(6)^2} \left[\frac{1}{1 + 0} \right]^{5/2}$$

$$\sigma_z = 26.53 \text{ kN/m}^2$$

case (ii) $r = 5m ; \; z = 6m$

$$\sigma_z = \frac{3}{2\pi} \times \frac{2000}{(6)^2} \left[\frac{1}{1 + \left(\frac{5}{6}\right)^2} \right]^{5/2}$$

$$\sigma_z = 7.09 \text{ kN/m}^2$$

Westergaard's theory:

Assumptions:

* Soil is heterogeneous
* Soil is sedimentary deposit / layered deposit. (eg) varied clay, stratified soil.
* No be consider the poisson's ratio and youngs modulus.

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

$$\sigma_z = K_w \frac{Q}{z^2}$$

k_w = Westergaard's coeff.

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

* lateral strain is negligible.
* Fenske's chart based on We used to find vertical stress based on westergaard's theory.

1) If the soil sample $n = 50\%$. $S_r = 90\%$.
% air voids.

$S_r = 1 - \eta_a$.

$\eta_a = \frac{V_r}{V}$

$a_c = 1 - S_r$

$n_a =$

$a_c + S_r = 1$

$a_c = 1 - S_r$

$a_c = 1 - 0.9 = 0.1$

$n_a = n\, a_c = 0.5 \times 0.1 = 5\%$.

$n_a = 5\%$.

2) $LL = 65\%$. $PL = 30\%$. $SL = 25\%$. $w = 45\%$.

$$I_d = \frac{w - w_p}{w_L - w_p} = \frac{45-30}{65-30} = \frac{15}{35} = 0.428.$$

3) $e = 0.9$ $n = \frac{e}{1+e} = \frac{0.9}{1.9} = 0.47$

4) % air voids $= 30\%$. $n = 0.4$ air content = ?

$\eta_a = n\, a_c$

$a_c = \frac{\eta_a}{n} = \frac{0.30}{0.4} = 0.75$

$\frac{I_P}{I_f}$ $w_L - w_P$

$e = 0.67$ $w = 0.188,$ $G = 2.68$ $S = ?$

$wG = S_r e$

$0.188 \times 2.68 = S \times 0.67$ $\frac{V_v}{V}$

$S = 75.2\%$

If the volume of voids is equal to the volume of solids in a soil mass, then the values of porosity & voids ratio respectively are.

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

$n = \frac{e}{1+e} = \frac{1}{2} = 0.5$

Relative compactness: R_c.

→ Cohesionless soil & cohesive soil.

$R_c = \frac{\gamma_d}{\gamma_{max}} \times 100$ (%).

$\gamma_d = \frac{G\gamma_w}{1+e}$ $\gamma_{dmax} = \frac{G\gamma_w}{1+e_{min}}$

$R_c = \left(\frac{1+e_{min}}{1+e}\right) \times 100$

$R_c = 80 + 0.2 I_D.$

$LL = 20\%$, $PL = 25$. $I_P = ?$

$I_P = W_L - W_P = 20 - 25 = -5$

But the answer is 0%

Bulking of Sand:

If the depth of moist sand in a cylinder is 15 cm & the depth of the sand when fully inundated with water is 12 cm, what is the bulking of sand?

$V_m = 15\text{ cm}$ $\qquad V_s = 12\text{ cm}.$

$$\text{Bulking of sand} = \frac{V_m - V_s}{V_s} = \frac{15-12}{12} = 0.25.$$

pumping out test. → large area.
More reliable test

Unconfined aquifer

$$k = \frac{2.303\, q \log_{10} \frac{r_2}{r_1}}{\pi (h_2^2 - h_1^2)}.$$

Confined aquifer

$$k = \frac{2.303\, q \log_{10} \left(\frac{r_2}{r_1}\right)}{2\pi D (h_2 - h_1)}$$

Pumping in test – More economical one.

(1) open end test
(2) Packer Test.

If the voids ratio & V_a for soil is 0.5 & 6×10^{-7} m/s resply, what is the value of seepage velocity (m/s).

$e = 0.5$ $\qquad V = 6 \times 10^{-7}$ m/s

$$V_s = \frac{V}{n} \qquad n = \frac{e}{1+e} = \frac{0.5}{1.5} = \frac{1}{3}$$

$$V_s = \frac{6 \times 10^{-7}}{1/3} = 18 \times 10^{-7}\text{ m/s}.$$

Consolidation: — Terzaghi

Compressibility:
↓ function of effective stress (principle of σ')

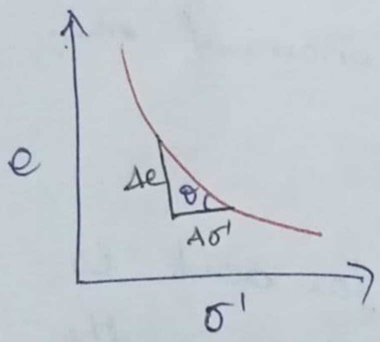
- Reduction in volume occurs under compression.

In soil compression takes place due to
(a) expulsion of air in the voids (Compaction)
(b) expulsion of water in the voids. (Consolidation)

$$\boxed{\frac{\Delta H}{H} = \frac{\Delta e}{1+e} \qquad \Delta H = H \times \frac{\Delta e}{1+e_0}}$$

① **Coeff. of compressibility (a_v)** $\frac{1}{kPa}$ (or) $\frac{m^2}{kN}$

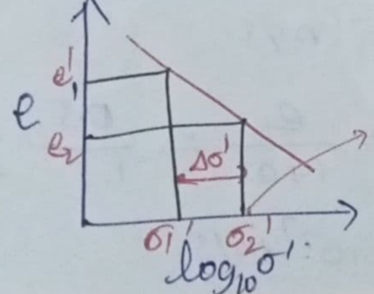
Decrease in the void ratio per unit increase in effective stress applied on the soil sample.



$$\tan\theta = \frac{\Delta e}{\Delta\sigma'} = \frac{e_2 - e_1}{\sigma_2' - \sigma_1}$$

$$\boxed{a_v = \frac{\Delta e}{\Delta\sigma'}}$$

② **Compression Index: C_c**



$$\boxed{\Delta\sigma = \Delta\bar{\sigma}\uparrow + u_i\downarrow}$$

$$C_c = \frac{e_1 - e_2}{\log_{10}\sigma_2' - \log_{10}\sigma_1'} = \frac{e_1 - e_2}{\log_{10}\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

$$C_c = \frac{e_1 - e_2}{\log_{10}\left(\frac{\sigma_1' + \Delta\sigma'}{\sigma_0'}\right)}.$$

$$\boxed{C_c = \frac{\Delta e}{\log_{10}\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_0'}\right)}}$$ No unit.

③ Primary consolidation (Consolidation Settlement)

$$S_p = \frac{H\, C_c \log_{10}\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right)}{1 + e_0}$$

$C_c = 0.009\,[LL - 10\%]$ Natural soil Undisturbed Site condition

$C_c = 0.007\,[LL - 10] \rightarrow$ Remoulded soil

Organic $C_c = 0.0115\, w_n$

④ Coefficient of volume change (Volume Compressibility) (m_v).

→ It is the reciprocal of bulk modulus of soil which is constant.

$$m_v = \frac{\epsilon_v}{\Delta\sigma'} = \frac{\frac{\Delta V}{V_0}}{\Delta\sigma'} = \frac{\frac{\Delta e}{1+e_0}}{\Delta\sigma'} = \frac{\Delta e}{\Delta\sigma'} \times \frac{1}{1+e_0}$$

$$\boxed{m_v = \frac{a_v}{1+e_0}}$$

$C_c = 1.15\,(e_0 - 0.35)$

$C_c = 0.54\,(e_0 - 0.30)$

$$m_v = \frac{\Delta v}{V} \times \frac{1}{\Delta \sigma'} = \frac{\Delta H \times A}{H \times A} \times \frac{1}{\Delta \sigma'}$$

$$\boxed{S_P = \Delta H = m_v \times H \times \Delta \sigma'}$$

Terzaghi One dimensional Consolidation Theory

1) Soil mass is homogeneous & isotropic, fully saturated
2) Load is applied in one dirn (Vert)
3) Drainage of pore water due to applied load takes place along the dirtn of load.
4) Primary consolidation governed by expulsion of pore water

At initial, $u = 100\%$. consolidation 0%. $(\sigma' = 0)$

At final, $u = 0\%$. Consolidation 100%.

5) Secondary consolidation is infinity.
6) $\Delta e = a_v \times \Delta \sigma'$.
7) Darcy law is valid.

$$v = k i \qquad\qquad u = \gamma_w h$$

$$v = k \frac{\partial h}{\partial z} \qquad\qquad h = \frac{u}{\gamma_w}$$

$$v = k \frac{\partial}{\partial z}\left(\frac{u_w}{\gamma_w}\right)$$

$$\boxed{v = \frac{k}{\gamma_w} \frac{\partial u_w}{\partial z}}$$

eqn of 1D consolidation theory,

$$\frac{\partial u_w}{\partial t} = \frac{k}{m_v \gamma_w} \times \frac{\partial^2 u_w}{\partial z^2}$$

$$m_v = \frac{a_v}{1+e_0} \quad \text{or} \quad \frac{\Delta e}{\Delta \sigma'} \frac{1}{1+e}.$$

Coeffi of consolidation C_v depends on types of soil & change in effective stress

$$C_v = \frac{k}{m_v \gamma_w}$$

$k \rightarrow$ m/sec

$m_v \gamma_w \rightarrow$ kN/m³.

$C_v \rightarrow$ m²/sec $[M^0 L^2 T^{-1}]$.

$w_L \uparrow \rightarrow C_v \downarrow$

Compressibility, a_v, m_v $\uparrow$.

Time of consolidation.

$T_v \rightarrow$ Time factor.

$$T_v = \frac{C_v t}{d^2}.$$

T_v depends on degree of consolidation.

$$T_v = \frac{\pi}{4}\left[\frac{U}{100}\right]^2 \leftarrow U \le 60\%$$

$$T_v = -0.9332 \log_{10}\left(1 - \frac{U}{100}\right) - 0.0851 \leftarrow U > 60\%$$

Degree of consolidatn.

Degree of Consolidatn $U = \frac{S_t}{S_f} \times 100$ [U ratio of settlement of soil at any time to the total primary settlement of the soil.

① Settlement at any stage t.

$$U = \frac{\Delta h}{\Delta H} \times 100.$$

Ultimate.

(2) $\boxed{U = \dfrac{u_i - u_t}{u_i - u_f \to 0} \times 100}$

$u_i \to$ Excess pore pr. @ begening

$u_t \to$ " after time

If $U = 50\%$ $\quad T_v = \frac{\pi}{4}\left[\frac{50}{100}\right]^2 = 0.196$

If $u = 90\%$ $\quad T_v = -0.9332 \log_{10}\left(1 - \frac{90}{100}\right) - 0.0851$

$T_v = 0.848$

If $u = 100\%$ $\quad T_v = \infty$

(3) when e is known

$U = \dfrac{e_0 - e}{e_0 - e_{100}} \times 100$

$e_0 \to$ Initial

$e \to$ Void rah @ 't'

$e_{100} \to$ final void

single drainage	$d = H$
Double drainage	$d = H/2$

$t_1 = 4 t_2$

$t_2 = \frac{1}{4} t_1$

$t_1 \to$ Single drainage

$t_2 \to$ Double drainage

Determination of coeff. of consolidation in the lab:

(1) square root of time fitting method.

(2) Logrithmic time filling method.

(1) $\sqrt{t}$ method. (Height consolidation settlement soil)

→ Taylor.

→ Oedometer

→ Graph plotted b/w $\sqrt{t}$ & dial gauge reading.

→ consider 90% of primary consolidation & the time required for 90% consolidation

$T_v = \dfrac{C_v t}{d^2}$ $\quad d = \dfrac{H_i + H_f}{2}$ (Avg)

(2) Logarithmic time method:

→ Casagrande

→ graph plotted b/w log of time of consolidation & dial gauge

→ Consider 50% of primary consolidation & the time required for 50% consolidation

$$C_v = \frac{T_v d^2}{t} \qquad T_v = \frac{\pi}{4}\left(\frac{u}{100}\right)^2$$

Immediate settlement (or) Elastic Compression / Initial settlement.

- Independent of pore water pressure.

$$S_i = S_e = \frac{qB(1-\mu^2)}{E} \times I_f$$

$\mu = 0.35$ to 0.5

Permissible settlement of soil.

→ depend on

(a) Type of soil - Hard clay, soft clay

(b) Type of footing - Single footing (Isolated footing) (or) Raft.

(c) Material used for footing - RCC (or) Steel.

Isolated RCC footing	Sand	- 50 mm
	Hard clay	- 50 mm
	Plastic clay	- 75 mm
RCC Raft foundation	Sand	- 75 mm
	Hard clay	- 75 mm
	Plastic clay	- 100 mm

$$S = S_i + \underset{1^\circ}{S_c} + \underset{2^\circ}{S_s}.$$

Problem

A soil sample which has been subjected to consolidation test has an area of $50 cm^2$. Initial dry weight of sample is 190.24 g. Initial height of the sample is 25 mm, $G = 2.67$. Determine the height of solids & the initial voids ratio of the soil.

$A = 50 cm^2$ $\quad W_d = 190.24 g$ $\quad H_0 = 25 mm$

Height of solids $H_s = \dfrac{W_d}{G \gamma_w A} = \dfrac{190.24}{2.7 \times 1 \times 50} = 1.425 cm$

Saturated & consolidated

$e_0 = \dfrac{H_0 - H_s}{H_s} = \dfrac{25 - 14.25}{14.25} = 0.75$

Voids ratio at any stage $e = \dfrac{H_1 - H_s}{H_s}$

$H_1 = H_0 - \Delta H$

An undisturbed sample of soil as a volume of $100 cm^3$ & mass of 190 gm. On over drying for 24 hr a mass is reduced to 160 g. If the specific gravity of grains is 2.68. Determine the water content, void ratio, degree of saturation.

Given:

$V = 100 cm^3$ $\quad M = 190 g$

$G = 2.68$ $\quad M_d = 160 g$.

soln:

1) water content

$$\omega = \frac{M_w}{M_d}$$

$$M_w = M - M_d = 190 - 160$$

$$M_w = 30g$$

$$\omega = \frac{30}{160} = 0.188$$

$$\omega = 18.8\%$$

2) Void ratio (e)

$$\gamma = \gamma_d (1+\omega)$$

$$\rho = \frac{M}{V} = \frac{190}{100}$$

$$\rho = 1.9 \ g/cm^3 = 1.9 \times 9.81$$

$$\rho = 18.64 \ kN/m^3$$

$$\gamma = 18.64 \ kN/m^3$$

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

$$= \frac{18.64}{1+\frac{18.8}{100}}$$

$$\gamma_d = 15.69 \ kN/m^3$$

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

$$= \frac{2.68 \times 9.81}{15.69} - 1$$

$$e = 0.67$$

3) Degree of Saturation.

$Se = wG$

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

$S = \frac{0.188 \times 2.68}{0.67}$

$S = 0.75$

$S = 75\%$

(Bulk) unit weight (Total unit wt / Moist unit weight

→ existing unit wt.

1) $\gamma = \gamma_{bulk} = \frac{W}{V} = \frac{W_s + W_s}{V_s + V_v}$ 14 to 18 kN/m^3.

2) $\gamma_d = \frac{W_{dry}}{V} = \frac{W_s}{V_s + V_v}$ 16 to 17 kN/m^3.

3) $\gamma_{sat} = \frac{W_{sat}}{V} = \frac{W_s + W_w}{V_s + V_v}$ completely filled with water

The value of γ_{sat} = γ_{bulk}.

Every γ_{sat} is γ_{bulk} but every bulk is not γ_{sat}.

γ_{sat} = 20 kN/m^3.

Double of γ'

4) γ_{sub} (or) γ'

Based on Archimedes principle.

$\gamma' = \frac{W_{sub}}{V} = \frac{W_{sat} - W_{w}}{V}$

$\gamma_{sub} = \gamma_{sat} - \gamma_w$ [8.5 – 11 kN/m^3]

Soil mass is capillary zone is in saturation condition only.

5) Unit wt of soil solids :

$$\gamma_s = \frac{W_s}{V_s}$$ [26.5 kN/m^3 – 27 kN/m^3]

Descending order

$\gamma_s > \gamma_{sat} > \gamma_{bulk} > \gamma_{dry} > \gamma_{sub}$

$$\boxed{\gamma' = \frac{1}{2}\gamma_{sat}}$$

In an earthen embankment under construction the bulk unit wt is 16.5 kN/m^3 at water content of 11%. If the water content is to be raised to 15%, compute the quantity of water required to added per cubic meter of soil? Assume no change in the void ratio.

$\gamma_1 = 16.5$ kN/m^3

$w_1 = 11\%$

$w_2 = 15\%$ $\qquad V = 1$ m^3.

$$\gamma_w = \frac{W_{w_1}}{V_{w_1}}$$

$$V_{w_1} = \frac{W_w}{\gamma_w} \Leftarrow \frac{1.631}{9.81} = \cancel{0.227}\ 0.167 \text{ m}^3 = 167 \text{ l}$$

$$w_1 = \frac{W_{w_1}}{W_s}$$

$$W_{w_1} = w_1 \times W_s \Leftarrow 0.11 \times 14.86 = 1.635 \text{ kN}$$

$$\frac{W_{s_1}}{V_1} = \gamma_{d_1}$$

$$W_{s_1} = \gamma_{d_1} \times V_1 = 14.86 \times 1 = 14.86 \text{ kN}$$

$$\gamma_{d_1} = \frac{\gamma_1}{1+w_1} = 14.86 \text{ kN/m}^3$$

Illy $V_{w_2} = \frac{W_{w_2}}{\gamma_w} = \frac{w_2\, W_s}{\gamma_w} = .227 \text{ m}^3$

$= 227$ litre.

Required quantity of water to be added per cubic m of soil

= 227 − 167

= 60 litres.

$\frac{100}{1000} \times 100$

$\frac{350}{1000} \times 100$

Fineness modulus:

→ Index number which represents the mean size of the particles in sand.

$$\text{Fineness modulus} = \frac{\text{cumulative \% retained on each sieve is added}}{100}$$

Fine agg means the agg which passes through 4.75 mm Sieve.

Sieve size	wt retained.	cumulative retained.	% cumul.
		0	0
4.75	0		
		100	10
2.36	100		
		350	35
1.18	250	700	70
0.6	350		
0.3	200	900	90
0.15	100	1000.	100.

250
300
550
2
2.75

$$F.M = \frac{275}{100} = 2.75.$$

Fine sand 2.2 − 2.6
Medium 2.6 − 2.9
Coarse sand 2.9 − 3.2

SR

The liquid limit & Shrinkage limit of a soil sample are 49% & 16% respectively. If the volume of a specimen of this soil decreases, on drying, from 37.2 cm³ @ liquid limit to 22.4 cm³ at shrinkage limit, compute the specific gravity of soil particles.

$w_L = 49\%$ $\quad$ $w_s = 16\%$

Vol. of soil specimen @ liquid limit $V_L = 37.2\ cm^3$

Vol. of " @ Shrinkage limit $V_d = 22.4\ cm^3$

Soln:

$$SR = \frac{\left(\frac{V_L - V_d}{V_d}\right) \times 100}{w_L - w_s}$$

$$= \frac{\frac{37.2 - 22.4}{22.4} \times 100}{49 - 16}$$

$$SR = 2$$

$$SR = \frac{W_d}{V_d \gamma_w}$$

$$\frac{W_d}{V_d} = (SR)\gamma_w$$

$$\frac{W_d}{V_d} = 2\gamma_w$$

$W = \frac{W}{\ }$

$W_d = \gamma_d V_d$

$= \frac{W_d}{V_d} \cdot V_d$

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

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

$SR = \frac{\gamma_d}{\gamma_w}$

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

$\gamma_d = \frac{G\gamma_w}{V}$

$\gamma_d = \frac{G\gamma_w}{\frac{W_d}{\gamma}}$

$$SR = \frac{\frac{V - V_d}{V_d} \times 100}{w_L - w_s}$$

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

$$SR = \frac{W_d}{V_d \gamma_w} \quad \text{or} \quad \frac{\gamma_d}{\gamma_w}$$

$$w_s = \frac{G\gamma_w - \gamma_d}{G\gamma_d}$$

$$w_s = \frac{V_d \gamma_w}{W_d \gamma_w} - \frac{1}{G}$$

$$V_S = \frac{V - V_d}{V_d}$$

1) 80, 40, 20, 10, 4.75, 2.36, 1.18, 0.6, 0.3, 0.15

10 9 8 7 6 5 4 3 2 1

If finess moduluss of coarse agg is 7.7, which means, the avg size of particle of C.A is in 7^{th} & 8^{th} sieve, (ie) 10mm to 20mm.

FM = 4.3. ⇒ Sample b/w 1.18 mm to 2.36mm.

2) $G = 2.65$ $\rho = 1000\ kg/m^3$. Zero air voids, $n_a = 0$

$w = 10\%$. $\gamma_d = ?$

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

$1\ g/cm^3$

$$\frac{1 \times 10^{-3}}{(10^{-2})^3}$$

$$\gamma_d = \frac{G\gamma_w}{1 + wG} = \frac{2.65 \times 10^3}{1 + 0.1 \times 2.65}$$

1kg = 1000g
1000g = 1kg
$1g = \frac{1}{1000}kg$

$$\gamma_d = 2.094 \times 10^3 kg/m^3$$

100cm = 1m
$1cm = \frac{1}{10^2}$

$$\gamma_d = 2094\ kg/m^3$$

$1 \times 10^{-3} \times 10^6$

3) $V_d = 50cm^3$ $W_d = 88g$ $G = 2.71$ $\rho = 1g/cc$ 10^3

$w_s = ?$

$$w_s = \frac{V_d \gamma_w}{W_d} - \frac{1}{G} \Rightarrow \frac{50 \times 1}{88} - \frac{1}{2.71}$$

$= 19.91\%$

Unit - IV

Shear Strength

Shear strength :

The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles (or) on masses upon the action of shear stress.

Shear resistance consists of three components

* Structural resistance due to interlocking
* Frictional resistance due to translocation
* cohesion (or) adhesion.

In granular materials, the shear strength is due to intergranular friction & interlocking of particles.

In pure clay → cohesion b/w clay particles.

In all other soil → friction & cohesion.

Shear strength of cohesive and cohesionless soil.

Cohesive soil :

* It depends upon whether the soil is normally consolidated (or) over consolidated.

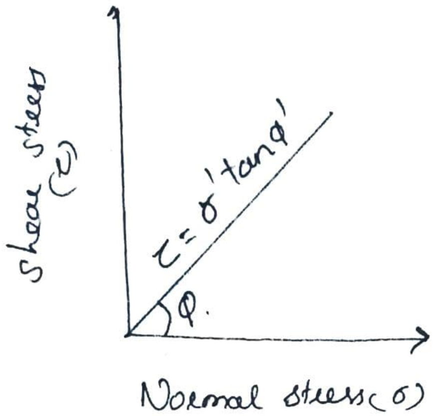
$$\tau = c' + \sigma \tan \phi'$$

For a normally consolidated soil (or) clay,

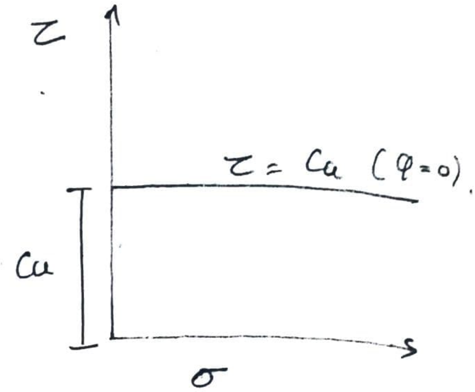
$$c' = 0$$

For pure clay soil, $\phi = 0$

$$\boxed{\tau = C_u}$$

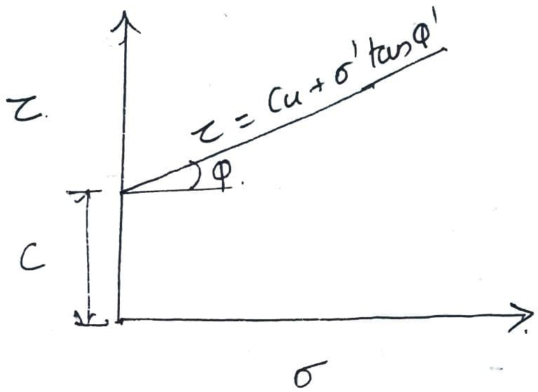


Normally consolidated clay ($C = 0$)



Pure clay ($\varphi = 0$)

C – φ soil.



φ = Max angle of obliquity = Internal angle of friction = angle of repose.

Shear strength of cohesionless soil : (Sand).

Due to

(i) Internal frictional resistance b/w soil (Translocation b/w the individual soil particles at their contact point)

(ii) Structural resistance to displacement of the soil because of the interlocking of the particles.

$C = 0$.

The following parameters are affects the shear strength of cohesionless soils.

Shape of particles, Gradation, Density, Confining pressure, Deviator stress, Loading, Type of minerals, Moisture.

Shear strength:

Mohr- Coulomb failure theory:

* Materials fails by shear
* The ultimate strength of the material is determined by the stresses in the failure plane.
* when the material subjected to three principal stresses, the intermediate principal stresses does not have any influence on the strength of material.

$$\tau_f = S = F(\sigma).$$

$\tau_f \to$ Shear resistance of material.

$F(\sigma)$ = Funtion of normal stress

$$S \text{ on } \tau = c + \sigma \tan \varphi.$$

c - cohesion

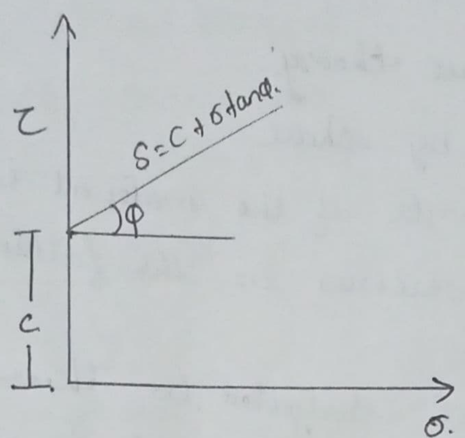
φ - Angle of internal friction (or) Angle of shearing resistance.

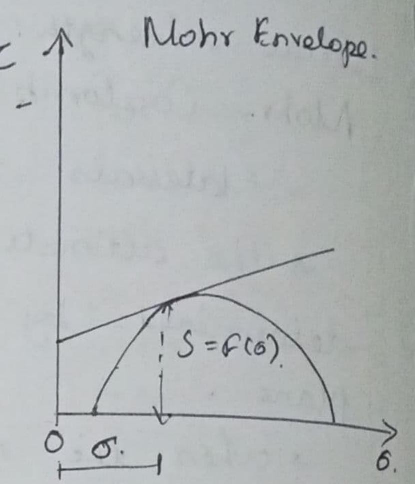
Cohesion:

The shearing strength in which a soil posses by virtue of its pressure, when loads are applied to a cohesive soil attraction (or) bond comes in to action to resist.

Angle of internal friction :

The angle b/w the resultant force and the perpendicular to the surface.





Measurement of Shear Strength :

1) Direct shear strength
2) Triaxial Shear test
3) Unconfined compression test.
4) Vane shear test.

Based on drainage condition, the shear test are classified as.

* Undrained test - UU test - Unconsolidated
* Consolidated undrained test (CU test) - consolidated.
* Drained test (CD test) - Consolidated

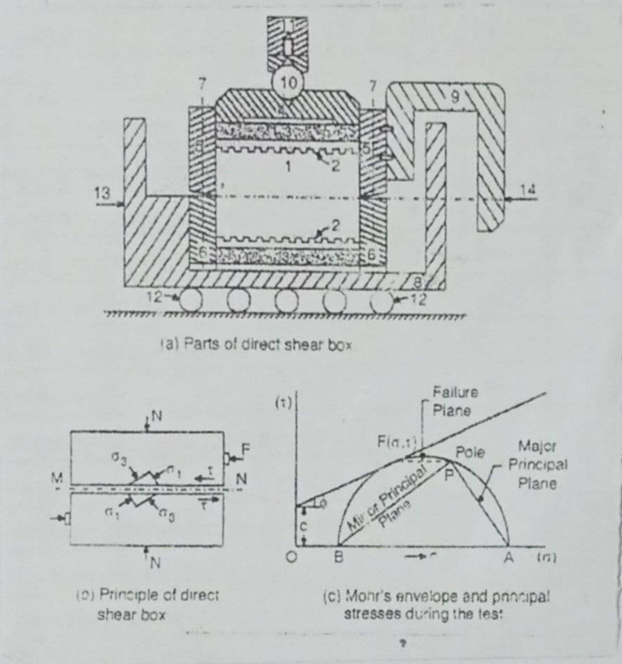
Direct shear test :

* Simple test and is performed in a shear box of 60 mm x 60 mm square size.

* It consists of two piece of square section.

* The lower half of the box is rigid. which is driven by motor on hand.

* The upper half of box is ~~rig~~ connected with proving ring .



(a) Parts of direct shear box

(b) Principle of direct shear box

(c) Mohr's envelope and principal stresses during the test

1. Soil specimen
2. Metal Grids.
3. Porous stone
4. Loading pad
5. Upper part
6. Lower part
7. Screws .
9. U ARM
10. Steel ball
11. Loading yoke.
12. Rollers.
13. Shear force applied by Jack
14. Shear resistance. measuring proving ring.

* Normal load is applied on specimen through loading yoke.

* The shearing force is applied to a lower by means of geared Jack.

* The change in volume shear displacement shear force, vertical deformation is measured by dial gauge.

* If test continuous beyond 20% strain, It is used to stop the test & defined failure point as corresponding to any desired level of strain upto 20%.

* A graph is plotted b/w normal stress (x axis) & shear stress (y axis).

Advantages:

1) Sample preparation is easy.
2) Drainage is quick.
3) It is ideally suited for cohesion less soils.
4) It is cheap.

Disadvantages:

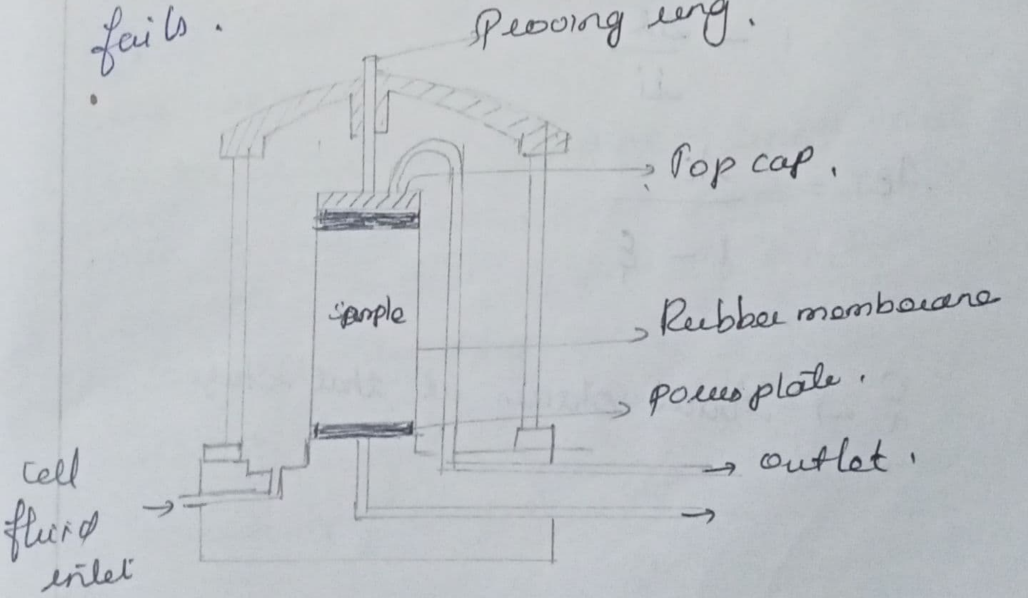
1) The Mohr Circle cannot be drawn.
2) The stress distribution is not uniform.
3) The failure plane is predetermined. ∴ the specimen is not allowed to fail along it weakest plane.
4) Measurement of pore pressure is not possible.

Triaxial compression test:

In this test, the specimen is compressed by applying all the three principal stresses, σ_1, σ_2 & σ_3.

Length of sample = 2 to 2.5 m dia.

* The tri axial cell consists of a high pressure cylindrical cell, made up of transparent material like perspex, fitted b/w base & top cap.
* At the top an air release valve to expel air from the cell & a steel plunger for applying axial force on specimen are provided.
* Loading cap is placed on top porous plate.
* The specimen is enclosed in a rubber membrane to prevent its contact with the cell fluid.
* Cell pressure is applied.
* The additional axial force called the deviator force is applied through the plunger & the deviator force corresponding to different axial deformations at regular intervals are noted.
* The test is continued until the specimen fails.



Triaxial test :

$\sigma_1 = \sigma_d + \sigma_3$.

$\sigma_d = \sigma_1 - \sigma_3$.

$\sigma_d \rightarrow$ deviator stress.

Initial Volume = Vol. at any Stage of compress.

$A_i L_i = A_c L_c$

$$A_c = \frac{V_i + \Delta V}{L_i - \Delta L}.$$

$A_c \rightarrow$ Corrected area of specimen when axial compression is ΔL.

$V_i \rightarrow$ Initial volume $= A_i L_i$

$A_i \rightarrow$ Initial C/s area of specimen

$L_i \rightarrow$ Initial length of specimen

ΔV - change in volume.

Incase of undrained test, on saturated soil sample $\Delta V = 0$.

$$A_c = \frac{V_i + \Delta V}{L_i - \Delta L}$$

$V = AL$

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

$= \frac{V + \Delta V}{L - \Delta L}$

$$= \frac{A_i}{1 - \frac{\Delta l}{l_i}}$$

$$A_c = \frac{A_i}{1 - \varepsilon}.$$

$\varepsilon \rightarrow$ Axial strain at that stage.

Merits:

* Complete control of the drainage condition is possible.
* The possibility to vary the cell pressure (or) confining pressure.
* Stress distribution is uniform.
* It shows the behaviour of field condition. (3 stress).

Demerits:

* The apparatus is costly & bulky.
* It takes long period.
* It is not possible to determine the c/s area of the specimen accurately.

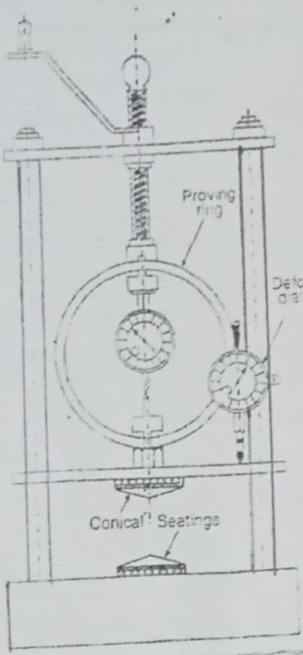
Unconfined compression test: (UCC)

* It is used to determine the unconfined compressive strength of the clayey soil.
* The spilit mould is oiled lightly from inside, the mould is pushed into the soil.
* Measure the initial length and dia of specimen. and place the specimen on the bottom plate of loading.
* Set the loading dial gauge & the strain dial gauge to zero.

$$\text{Sensitivity of clay} = \frac{Q_u \text{ (undisturbed)}}{Q_u \text{ (disturbed or remoulded)}}$$

$\sigma_2 = \sigma_3 = 0$. (due to the absence of confining pressure)

Applicable for saturated clay soil ($\varphi = 0$)



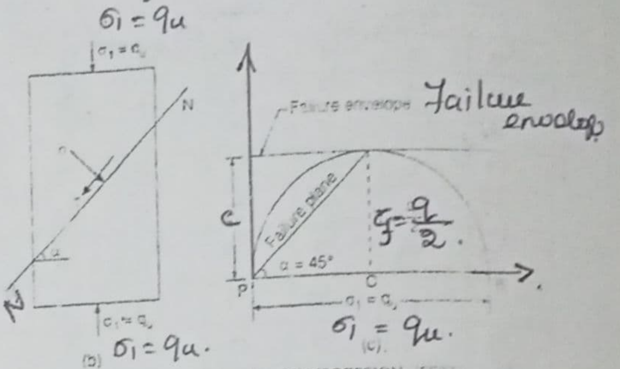


FIG 18.12 UNCONFINED COMPRESSION TEST

Unconfined compression Test :

WKT.

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c \tan\alpha.$$

$$\sigma_3 = 0.$$

$$\sigma_1 = 2c \tan\alpha. \qquad \left(\alpha = 45^\circ + \frac{\varphi}{2}\right)$$

$$\sigma_1 = 2c \tan\left(45^\circ + \frac{\varphi}{2}\right) \qquad (\varphi = 0)$$

$$\sigma_1 = 2c \tan 45^\circ.$$

$$\sigma_1 = 2c.$$

$$c = \frac{\sigma_1}{2}.$$

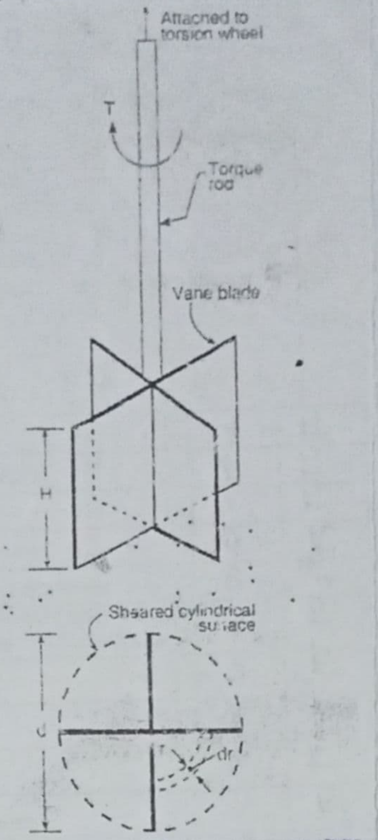
But

$$\tau = c + \sigma \tan\varphi \qquad (\varphi = 0)$$

$$\tau = c = \frac{\sigma_1}{2}. \qquad \tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = c$$

$q_u \rightarrow$ Unconfined compressive strength.

Vane shear test :



Soft clay
saturated clay.

* Quick test, used in situ (or) in the lab, to determine the undrained shear strength of cohesive soils.
* It consists of four thin steel plates called vanes.
* A torque measuring arrangement is attached to the rod which is rotated by a wheel arrangement.
* After pushing the vanes ~~genitally~~ gently into the soil, the torque rod is rotated at a uniform speed.
* The torque 'T' is then calculated by multiplying the dial reading with the spring constant.

(4). Vane Shear Test

Partially Submerged Vane — bottom.

$$T = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{12}\right]$$

$$\tau_f = \frac{T}{\pi d^2 \left[\frac{H}{2} + \frac{d}{12}\right]}.$$

Fully Submerged vane. top + bottom

$$\tau_f = \frac{T}{\pi d^2 \left[\frac{H}{2} + \frac{d}{6}\right]}.$$

Skempton's pore pressure parameters:

During undrained shear a change in applied stress causes change in pore pressure & the relation b/w the two is expressed in terms of empirical coefficients called pore pressure parameters.

The change in pore pressure is a fraction of the change in applied stress. The dimensionless quantity representing that fraction is called pore pressure parameter.

Let $\Delta\sigma_1$, $\Delta\sigma_2$ & $\Delta\sigma_3$ be the increase in the three principle stresses acting on a soil element.

ΔV - Change in volume

Δu - Increase in pore pressure.

The increase in effective principle stresses

$$\Delta\sigma_1' = \Delta\sigma_1 - \Delta u$$

$$\Delta\sigma_2' = \Delta\sigma_2 - \Delta u$$

$$\Delta\sigma_3' = \Delta\sigma_3 - \Delta u$$

The strain in the three directions are given by

$$\epsilon_1 = \frac{1}{E}\left[\Delta\sigma_1' - \mu(\Delta\sigma_2' + \Delta\sigma_3')\right]$$

$$\epsilon_2 = \frac{1}{E}\left[\Delta\sigma_2' - \mu(\Delta\sigma_1' + \Delta\sigma_3')\right]$$

$$\epsilon_3 = \frac{1}{E}\left[\Delta\sigma_3' - \mu(\Delta\sigma_1' + \Delta\sigma_2')\right]$$

Volumetric strain ϵ_v

$$\epsilon_v = \epsilon_1 + \epsilon_2 + \epsilon_3$$

$$= \frac{1}{E}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3' - \mu\Delta\sigma_2' - \mu\Delta\sigma_3' - \mu\Delta\sigma_1' - \Delta\sigma_3' - \mu\sigma_1' - \mu\Delta\sigma_2'\right]$$

$$= \frac{1}{E}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3' - 2\mu\Delta\sigma_1' - 2\mu\Delta\sigma_2' \cancel{- 2\mu\Delta\sigma_1' - 2\mu\Delta\sigma_2'} - 2\mu\Delta\sigma_3'\right]$$

$$= \frac{(1-2\mu)}{E}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right]$$

$$\epsilon_v = \frac{\Delta V}{V} = \frac{1-2\mu}{E}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right]$$

$$\frac{\Delta V}{V} = \frac{3(1-2\mu)}{E}\,\frac{1}{3}(\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3')$$

$$C_c = \frac{3(1-2\mu)}{E}.$$

Compressibility of soil.

$$\frac{\Delta v}{V} = C_c\left\{\frac{1}{3}(\Delta\sigma_1 + \Delta_2 + \Delta\sigma_3) - \Delta u\right\}.$$

$$\Delta V = V C_c\left\{\frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) - \Delta u\right\}.$$

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

Volume of voids (or) volume of pore fluid is nV.

$$\Delta V_w = C_v n V \Delta u$$

$C_v \rightarrow$ coeff. of volume compressibity.

The decrease in the volume of soil element is due to decrease in volume of voids.

ΔV eqn to ΔV_w

$$C_v n V \Delta u = \frac{1}{3}\left\{(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) - \Delta u\right\} V C_c$$

$$C_v n \Delta u + \Delta u C_c = \frac{1}{3} C_c(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$$

$$\Delta u\,(C_v n + C_c) = C_c\,\frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$$

$$\Delta u = \frac{C_c}{C_c + C_v n}\left[\frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)\right].$$

$$\Delta u = \frac{1}{1 + n\frac{C_v}{C_c}}\left[\frac{1}{3}\{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3\}\right].$$

In the conventional triaxial, $\Delta\sigma_2 = \Delta\sigma_3$

$$\Delta u = \frac{1}{1 + n\frac{C_v}{C_c}}\left[\frac{1}{3}(\Delta\sigma_1 + 2\Delta\sigma_3)\right].$$

$$\Delta u = \frac{1}{1 + n\frac{C_v}{C_c}}\left[\frac{1}{3}(\Delta\sigma_1 - \Delta\sigma_3) + \Delta\sigma_3\right].$$

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right]$$

$A, B \rightarrow$ Skemptons pore pressure parameters.

Undrained saturated clay,

Poisson ratio $= \frac{\sigma_3}{\sigma_1 + \sigma_3}$.

Youlme change due to confining pressure is 0

$\varepsilon_3 = 0$.

$$\frac{\sigma_3}{E} - \frac{\mu}{E}[\sigma_1 + \sigma_3] = 0$$

$$\boxed{\mu = \frac{\sigma_3}{\sigma_1 + \sigma_3}}$$

Unit - V

Slope Stability:

Slope failure mechanism:

Slope:

It is the angle of inclination of the soil surface from the horizontal.

Natural slopes & Man made slopes:

They are exist in nature & are formed cuttings, the slope of embankments constructed for taking loads, railway lines etc.

Sliding soil mass:

A slope failure involves downward and outward movement of a portion of slope called sliding soil mass.

Land slide:

A slope failure occurring in the case of a natural slope is refered to as land slide.

Slip surface:

In a slope failure the sliding soil mass slips along a surface called the slip surface.

Causes of Slope failure:

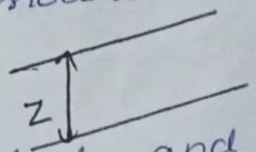
- Erosion
- Steady seepage
- Sudden drawdown
- Rainfall
- Earthquake
- External loading
- Construction activities at the toe of the slope.

Types of slopes:

a) Infinite slope:

It is used to designate as a constant slope of infinite extent in very large.

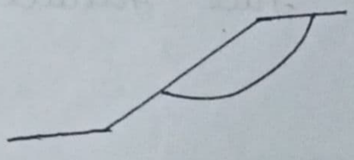
(eg) Long slope of a mountain.



b) Finite slope:

It is limited in extent and the properties of soil will not be the same identical depths so that the slip surface will be curved.

(eg) The inclined face of earth dam, embankment.



Types of failure:

(1) Face failure $D_f < 1$

(2) Toe failure $D_f = 1$

(3) Base failure $D_f > 1$.

$$D_f = \frac{H + D}{H}$$

1. Face failure:

This type of failure occurs when the slope angle is large & when the soil at the toe portion is strong.

2. Toe Failure:

In this case, the failure surface passes through the toe. This occurs when the slope is steep & homogeneous.

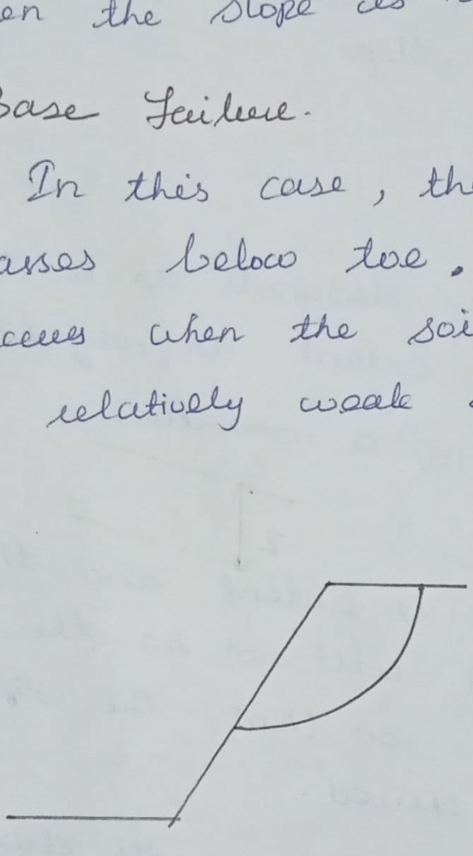
3. Base Failure.

In this case, the failure surface passes below toe. This generally occurs when the soil below the toe is relatively weak & soft.

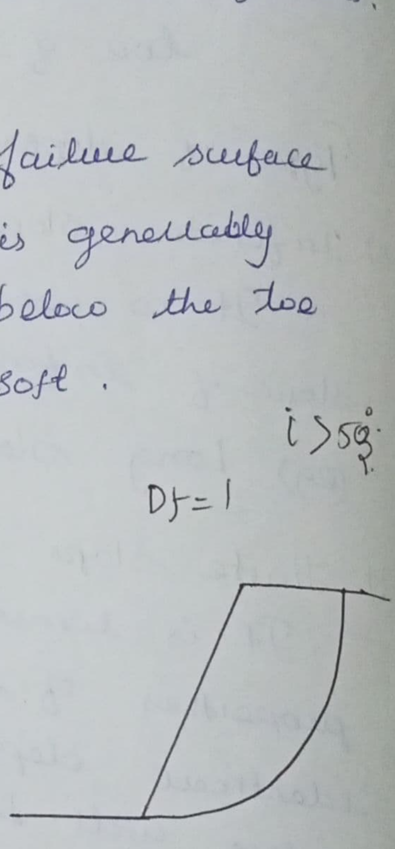
$i > 53°$

$D_f = 1$

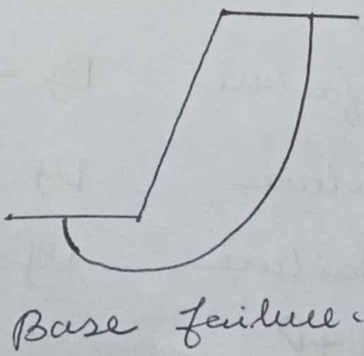
$D_f < 1$



Face Failure.



Toe failure.



$D_f > 1$

$\phi = 0$ $i < 53°$

Base failure.

Stability analyses of Finite slope:

1) Swedish circle method.

2) Friction circle method :

3) By use of Taylor stability number

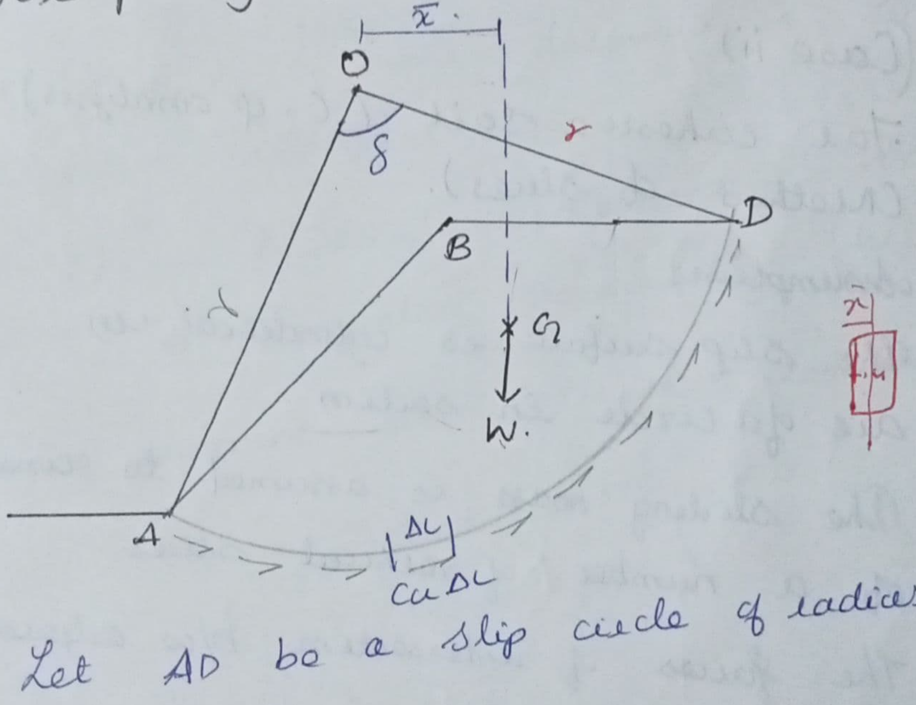
4) Bishop's method :

1) Swedish circle method:

In this method, the slip surface is assumed to be cylindrical (or) arc of a circle in section.

Case(1).

For purely cohesive soil ($\varphi_u = 0$).



Let AD be a slip circle of radius 'r' and centre O.

$\angle AOD = \delta$

W → weight of sliding soil mass ABDA acting vertically downward through its centre of gravity 'G' and at distance $\bar{x}$ from O.

Taking moment about centre of rotation O.

Driving moment $M_D = W\bar{x}$

Resisting moment $M_R = \Sigma C_u \cdot \Delta L \cdot r$

$= C_u \cdot r \, \Sigma \Delta L$

$= C_u \cdot r \cdot \hat{L}$

$\hat{L} \rightarrow$ Length of arc AD.

Factor of safety against sliding (F)

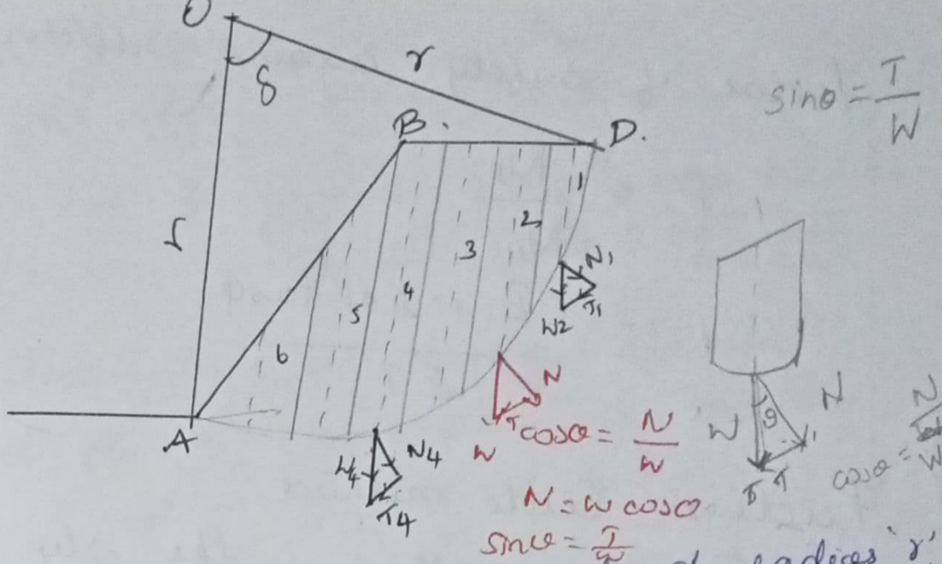
$$F = \frac{M_R}{M_D} = \frac{C_u \cdot r \hat{L}}{W\bar{x}}.$$

(Case ii)

For cohesive soil (C-φ analysis).
(Method of slices).

Assumptions:

1) The slip surface is cylindrical (i.e). arc of a circle in section.
2) The sliding mass is assumed to consists of a number of vertical slices.
3) The forces of intersection b/w adjacent slices are neglected.



Let AD be a slip circle of radius 'r', centre O, & central angle $\angle AOD = \delta$.

Let the sliding soil mass ABDA is divided into number of vertical slices 1, 2..

The $W_1, W_2 ..$ = acts through G. of respective slides.

$N_1 \rightarrow$ Normal component $= W\cos\theta$

$T_1 \rightarrow$ Tangential component. $= W\sin\theta$.

Taking moment about centre of rotation 'O'.

$$M_D = T_1 r + T_2 r + \cdots\cdots$$

$$= r(T_1 + T_2 + \cdots)$$

$$M_D = r\,\Sigma T$$

Restoring moment

$$M_R = \Sigma c\,\Delta L\,r + (N_1 \tan\varphi + N_2 \tan\varphi \ldots)\,r.$$

$$M_R = c\,r\,\Sigma \Delta L + (N_1 + N_2 + \cdots)\,r\tan\varphi$$

$$M_R = r\,[c\hat{L} + \Sigma N \tan\varphi]$$

Factor of safety against sliding.

$$F = \frac{M_R}{M_D}$$

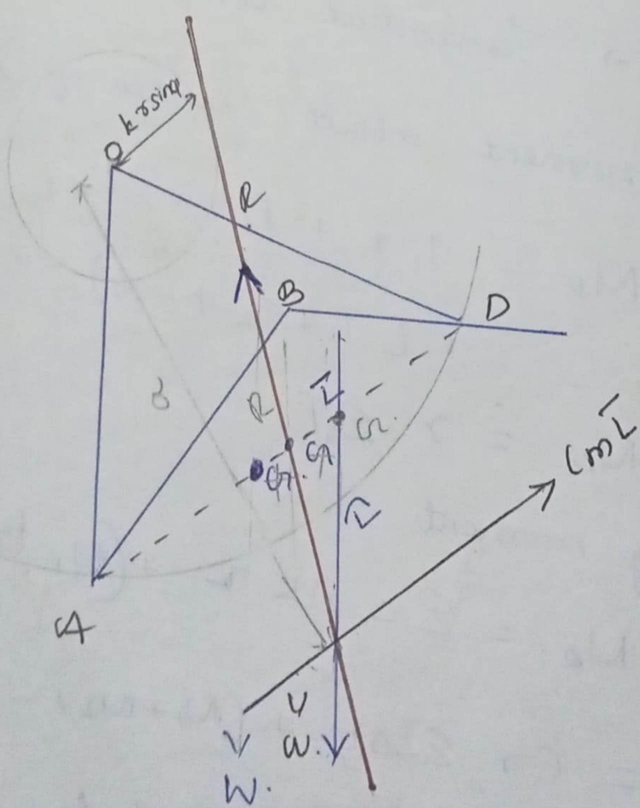
$$F = \frac{c\hat{L} + \Sigma N \tan\varphi}{\Sigma T}.$$

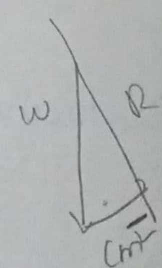
Friction Circle method :

In this method, the slip surface is assumed to be cylindrical.

Three forces ~~acting~~ are considered in equilibrium of the sliding mass.

* Weight of soil mass. W
* The resultant cohesive force C ($C_m \hat{L}$)
* The resultant reaction 'R'.





Triangle force diagram

1) With centre O and radius r, the slip circle AD is drawn.

2) With the O centre and $kr \sin\phi$ radius the friction circle is drawn. $\boxed{k = 1}$

3) A vertical line is drawn through centroid of section ABDA, to get the line of action of weight W.

4) Chord AD is drawn. $AD = \bar{L}$

5) A line is drawn parallel to chord AD and at a distance $a = r \frac{\hat{L}}{\bar{L}}$ from O, to get the line of action of resultant cohesive force $C_m \bar{L}$.

Length of arc $= \hat{L} = \frac{\pi r \delta}{180}$.

6) Through the point of intersection of the lines of action of forces W & $C_m \bar{L}$, a line is drawn tangential to the friction circle, to get the line of action of resultant reaction R.

7) The weight W of the sliding soil mass ABDA is computed & plotted to scale as shown in fig.

8) Through the ends of the vector representing W, lines are ~~po~~ drawn parallel to the line of action of forces $C_m \bar{L}$ and R to complete the

triangle of forces.

9. The value of $C_m \bar{L}$ is obtained from the force triangle and divided by value of $\bar{L}$ to obtain the value of mobilised cohesion C_m.

Factor of safety w.r.to Cohesion F_c

$$F_c = \frac{C}{C_m}$$

C - Ultimate cohesion.

C_m → Mobilised cohesion.

Factor of safety used in stability analysis of slopes:

1. Factor of safety w.r.to Cohesion

$$F_c = \frac{C}{C_m}$$

C → Ultimate cohesion

C_m → mobilised cohesion.

2. Factor of safety w.r.to friction.

$$F_\varphi = \frac{\tan\varphi}{\tan\varphi_m} = \frac{\varphi}{\varphi_m}$$

φ = Ultimate angle of shearing resistance

φ_m → Mobilised angle of shearing resistance.

3. Factor of safety w.r. to shear strength

$$F = \frac{\tau_f}{\tau}$$

ultimate shear strength $\tau_f = c + \sigma \tan\varphi$.

mobilised shear strength $\tau_{\varphi} = c_m + \sigma \tan\varphi_m$

4. Factor of safety w.r. to height

$$F_H = \frac{H_c}{H.}$$

$H_c \rightarrow$ Critical height of slope

$H \rightarrow$ actual height of slope

$F_H = F_c$ (Assuming cohesion to be fully mobilised)

1. A slope of very large extent of soil with properties $c' = 0$ & $\varphi' = 32°$ is likely to be subjected to seepage parallel to the slope with water level at the surface. Determine the maximum angle of slope for a factor of safety 1.5 treating it as an infinite slope. For this angle of slope what will be the factor of safety if the water level were to come down well below the surface? The saturated unit wt of soil $20\ kN/m^3$.

Soln:

$c' = 0 \qquad \varphi' = 32^\circ \qquad \gamma_{sat} = 20\ kN/m^3$.

Case(i) When water level is at surface and seepage occurs parallel to surface

$$F = \left(\frac{\gamma'}{\gamma_{sat}}\right)\left(\frac{\tan\varphi'}{\tan i}\right).$$

$$\tan i = \frac{(20-9.81)}{20} \times \frac{\tan 32^\circ}{1.5}$$

$$\tan i = 0.2122$$

Slope angle, $i = 12^\circ$.

(case ii)

When the water table goes well below the surface,

$$F = \frac{\tan\varphi'}{\tan i}$$

$$F = \frac{\tan 32^\circ}{\tan 12^\circ}$$

$$F = 2.94$$

2) A slope 1 in 2 with a height of 8m has the following soil properties.

$c = 28\ kN/m^2, \quad \varphi = 10^\circ, \quad \gamma = 18\ kN/m^3$.

Calculate

(1) FOS w.r.to cohesion

2) Critical height of slope

Soln:

(i) If 'i' is the slope angle.

$$\tan i = \frac{1}{2}$$

$$i = 26.6^\circ.$$

From Taylor stability chart.

$$i = 26.6^\circ, \quad \phi = 10^\circ, \quad S_n = 0.064$$

$$S_n = \frac{c}{F_c \gamma H}$$

$$F_c = \frac{c}{S_n \gamma H}$$

$$F_c = \frac{28}{0.064 \times 18 \times 8}$$

$$F_c = 3.04$$

$$F_c = \frac{H_c}{H}$$

$$H_c = F_c \cdot H$$

$$H_c = 24.32m$$

Slope Protection measures.

* Slope flattening reduces the weight of the mass tending to slide
* Providing a berm below the toe of the slope increases the resistance to movement.

* Drainage helps in reducing the seepage forces.
* Densification (or compaction increases the shear strength of soils).
* Consolidation by external load, increases the stability.
* Grouting & injection of cement help in increasing the stability.
* Retaining wall can be provided for lateral support. This method is costlier.
* Stabilization of the soil improves the stability of slopes.

Updation.

Consistency Index (I_c)

$$I_c = \frac{W_L - W_n}{W_L - W_p}$$

liquid $I_c < 0$ $\quad w_n > w_L$

solid (or Semi) $I_c > 1$ $\quad w_n < w_p$

Plastic $0 < I_c < 1$ $\quad w_p < w_n < w_L$.

Liquidity Index (I_L)

$$I_L = \frac{W_n - W_p}{W_L - W_p}$$

$I_L > 1$ $\quad w_n > w_L$

$I_L < 0$ $\quad w_n < w_p$

$0 < I_L < 1$ $\quad w_p < w_n < w_L$

$$I_c + I_L = 1$$

Isobar:

An isobar is a line or contour joining points inside soil mass at which the vertical stress have same value:

The Boussinesq eqn for vertical stress due to point load is

$$\sigma_z = \frac{3Q}{2\pi} \frac{z^3}{(r^2+z^2)^{5/2}}$$

For a single concentrated load 1000 kN acting on the ground surface construct an isobar for $\sigma_z = 40 \text{ kN/m}^2$.

$$\sigma_z = \frac{3Q}{2\pi} \frac{z^3}{(r^2+z^2)^{5/2}}.$$

$$\frac{\sigma_z\, 2\pi}{3Q z^3} = \frac{1}{(r^2+z^2)^{5/2}}$$

$$5.3 = \frac{1}{(r^2+z^2)^{5/2}}$$

$$1.9 = \frac{1}{r^2+z^2}$$

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

$$r^2+z^2 =$$

$$r^2+z^2 = 0.52$$

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

$$r^2 = \left(\frac{3Q z^3}{\sigma_z\, 2\pi}\right)^{2/5} - z^2.$$

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

$$r = \sqrt{\left(\frac{3Q z^3}{\sigma_z\, 2\pi}\right)^{2/5} - z^2}.$$

0.510.

$z = 0.25$ $= 0.7147 - 0.25$

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

$$40 = \frac{3\times 1000}{2\pi}\left(\frac{z^3}{(r^2+z^2)^{5/2}}\right)$$

$$\frac{40}{477.465} = \frac{z^3}{(r^2+z^2)^{5/2}}$$

$$0.0838\,(r^2+z^2)^{5/2} = z^3$$

$$(r^2+z^2)^{5/2} = \frac{z^3}{0.0838}$$

$$(r^2+z^2) = \left(\frac{z^3}{0.0838}\right)^{2/5}$$

$$r^2 = \sqrt{\left(\frac{z^3}{0.0838}\right)^{2/5} - z^2}$$

$$= \sqrt{\left(\frac{0.25^3}{0.0838}\right)^{2/5} - (0.25)^2}$$

$$= \sqrt{0.511 - 0.0625}$$

$$r = \sqrt{0.25}\left(\sqrt{\left(\frac{3\times 1000}{2\pi\,(0.25)^2\,40}\right)^{2/5} - 1}\right)$$

$=$

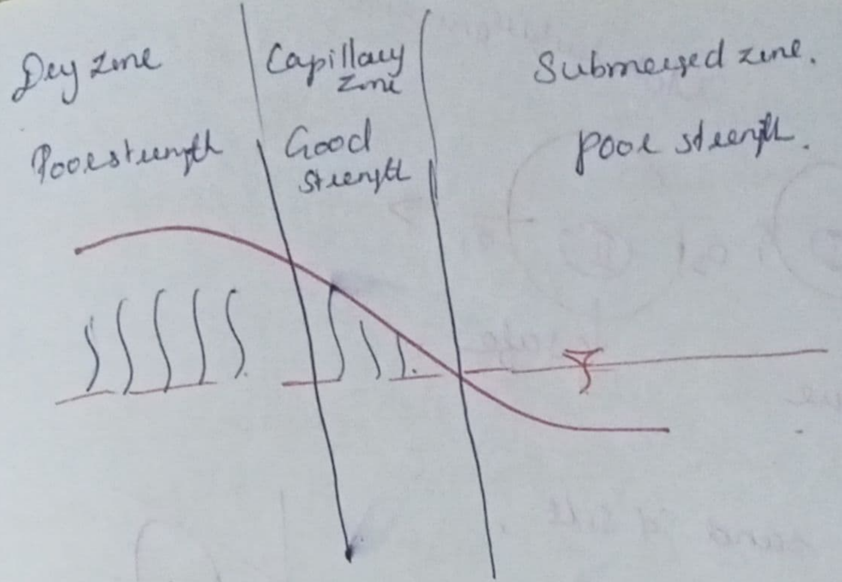
Preconsolidation

over con : $\frac{\bar{\sigma}_0}{\bar{\sigma}} > 1$

present

Normal $\frac{\bar{\sigma}_0}{\bar{\sigma}} \equiv 1$

Under : $\frac{\bar{\sigma}_0}{\bar{\sigma}} < 1$



$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e_0} = \frac{\Delta V}{V}$$

			Unit
coeff. compressibility	a_v	$\frac{\Delta e}{\Delta \bar{\sigma}}$	m^2/kN
coeff. of compression or Compression Index	C_c	$\frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}$	-
Recompression	C_R $C_R = \frac{1}{5}$ to $\frac{1}{10}$ C_c.	$\frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}$	-
Volume Compressibility or modulus of volume change	m_v	$\frac{a_v}{1+e_0}$	m^2/kN.
coeff of consolidation	C_v	$\frac{k}{\gamma_w m_v}$	$m^2/sec.$
Time of consolidation	T_v	$\frac{C_v t}{d^2}$	

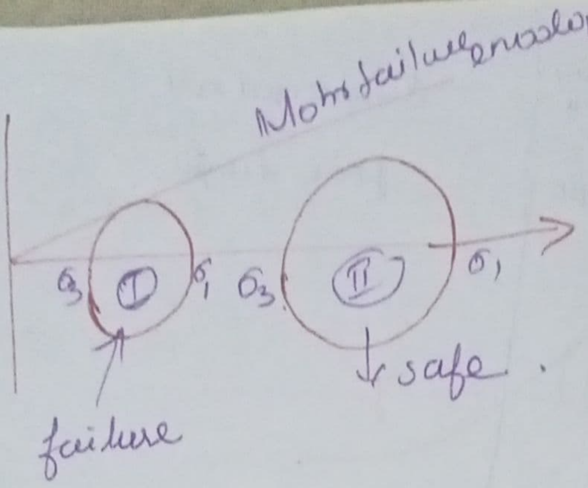
Single drainage $d = H$

Double drainage $d = \frac{H}{2}$

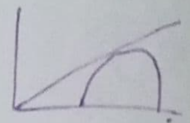
T_v depends on degree of consolidation (U).

$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \quad U \le 60\%$

$T_v = -0.9332 \log_{10}\left(1 - \frac{U}{100}\right) - 0.0851 \quad U > 60\%$

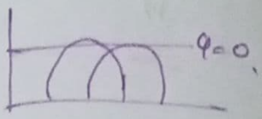


Incease of sand & silt,



$\tau_f < \tau_{max}$

Shear strength.



Clay → $\tau_f = \tau_{max}$.

* Dense sand have greater friction angle.
* Normal stress on critical plane (or) failure plane which increases with depth in soil.

c & ϕ are not fundamental properties of soil, these are depend on the types of test, water content, & drainage conditions.

Undrained shear parameter. c_u ϕ_u

Drained shear parameters. c' ϕ'
(effective pressure)

Saturated soil mass → Shear load → pore pressure will developed. (Unconsolidated undrained test

Clay → total stress used Consolidated Undrained test

If fast loading is done on sat. clay, - Short term condition → Undrained shear parameters should be determined. Ex the (eg) sand.

Drained Shear

If loading rate is low - expulsion of pore water is complete; then pore pressure will become zero.

Consolidated drained test - Effective stress be used.

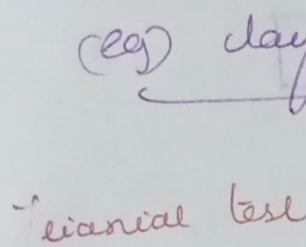
Long term

(eg) clay

Triaxial test:

(i) Confining pressure stage / cell pressure / (σ_3) Consolidation stage.

(ii) Deviator stage (or) Shear stage (or) Back pressure stage. (σ_d). confining constant & add additional axial load.



(i) Deviator stress at failure is termed as " confined compres strength

$$CCS = (\sigma_d)_f = (\sigma_1 - \sigma_3) = \frac{P}{A_f}.$$

peak eff stress → Pre consolidation test

Over consolidation = $\dfrac{\text{Max past stress}}{\text{Present existing stress}}$

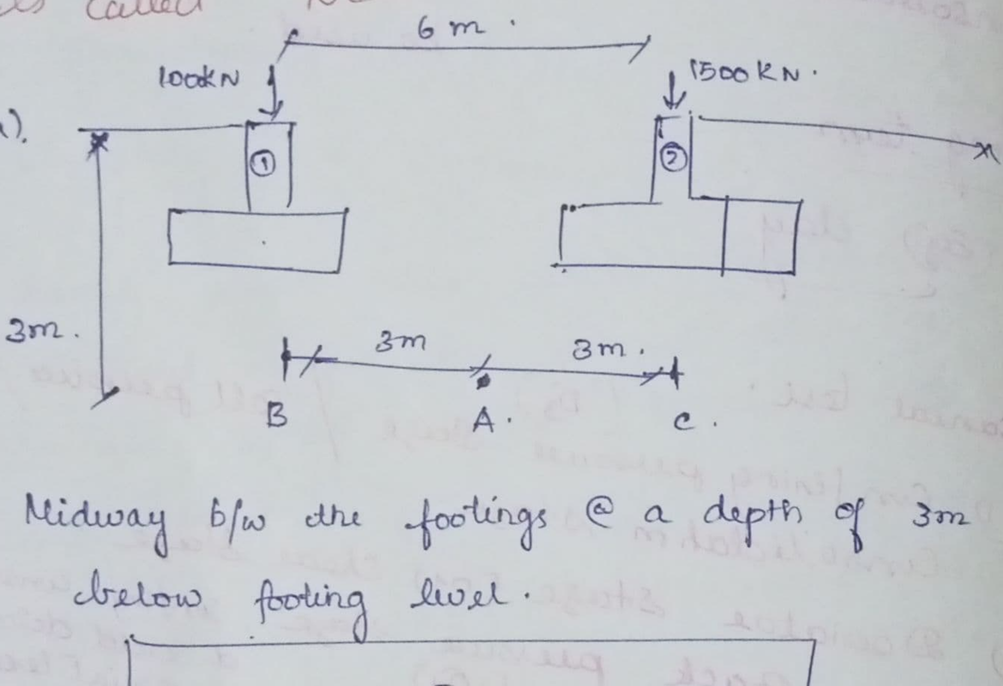
Present stress is less than past stress

Normally consolidated clay:

Never experienced eff. stress when compared to existing eff stress..

The soil below the earth surface is called NCC.

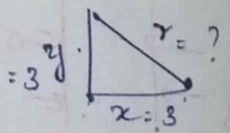
13. a).



a) Midway b/w the footings @ a depth of 3m below footing level.

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

Here $z = 3$; $r = \sqrt{3^2+3^2} = 4.24$



$$\therefore \frac{3}{2\pi \times 3^2}\left[\frac{1000}{1+\left(\frac{4.24}{3}\right)^2}\right]^{5/2} + \frac{3\times 1500}{2\pi \times 3^2} \times \left[\frac{1}{1+\left(\frac{4.24}{3}\right)^2}\right]^{5/2}$$

$\sigma =$

b) Vertically below footing 1. ∴ At point B.

$$\sigma = \frac{3 \times 1000}{2\pi \times 3^2}\left[\frac{1}{1+(0/2)^2}\right]^{5/2} + \frac{3 \times 1500}{2\pi \times 3^2}\left[\frac{1}{1+\left(\frac{6.708}{3}\right)^2}\right]^{5/2}$$

$r_1 = 0$ $\quad r_2 = \sqrt{3^2+6^2} = 6.708$

=

Vertically below footing 2 ∴ At point c.

$$\sigma = \frac{3 \times 1000}{2\pi \times 3^2}\left[\frac{1}{1+\left(\frac{6.708}{3}\right)^2}\right]^{5/2} + \frac{3 \times 1500}{2\pi \times 3^2}\left[\frac{1}{1+(0/2)^2}\right]^{5/2}$$

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\alpha$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2}\sin 2\alpha$$

resultant $= \sqrt{\sigma^2 + \tau^2}$

Max Shear Stress $= \sqrt{\cancel{\tau^2 + \sigma^2}}$ $\quad \dfrac{\sigma_1 - \sigma_3}{2}$.

Boussinesq equations:

Concentrated load as force.

Assumptions:

1) The soil mass is homogeneous.

2) The soil mass is isotropic.

3) The soil mass is semi-infinite.

4) The soil mass is an elastic medium.

E is constant.

Let a point load 'Q' act at the ground surface, at a point O. Stress component at a point P in the soil mass, having a radial distance 'r' & vertical distance z from point O.

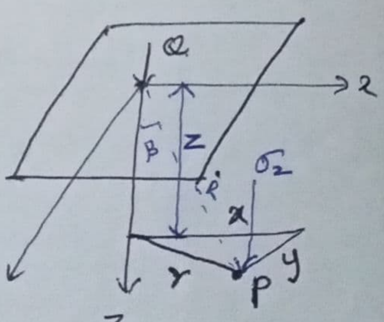
Boussinesq polar radial stress (σ_R)

$$\sigma_R = \frac{3}{2} \frac{Q}{\pi} \frac{\cos\beta}{R^2}$$

R = polar coordinate of point P

$$R = \sqrt{r^2 + z^2} = (x^2 + y^2 + z^2)^{\frac{1}{2}}$$

$$\cos\beta = \frac{z}{R}$$



Vertical stress due to point load

Vertical stress (σ_z)

$$\sigma_z = \sigma_R \cos^2\beta$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos^2\beta}{R^2} \qquad \cos\beta = \frac{z}{R}.$$

$$\sigma_z = \frac{3}{2}\frac{Q}{\pi}\frac{z^3}{R^5}$$

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}} \qquad R = (r^2+z^2)^{1/2}$$

Tangential stress (or) Shear Stress. (τ_{rz})

$$\tau_{rz} = \frac{1}{2}\sigma_R \sin 2\beta$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos^2\beta \sin\beta}{R^3}$$

$$\cancel{\sigma}_{rz} = \frac{3}{2}\frac{Q}{\pi}\frac{z^2 r}{R^5}$$

$$\cancel{\sigma}_{z} = \frac{3Q}{2\pi}\frac{z^2 r}{(r^2+z^2)^{5/2}}$$

$$\tau_z = \frac{3Qr}{2\pi z^3}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}.$$

x —— x

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

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

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

$K_B \rightarrow$ Boussinesq influence factor

1) Single grained structure → gravitational force
(dense or loose) → 0.02 mm greater.

2) Honey comb structure - GF & surface electric force.
0.0002 mm – 0.02 mm. (SEF)

3) Flocculated → Net force (attractive)
→ 0.0002 mm. (fine clay)
→ SEF
→ edge - face orientation.

→ Opposite
4) Dispersed structure:
→ Face - Face orientation.

High compressibility, ↓ strength, ↓ permeability,
→ Net force (Repulsive).

plasticity of mineral:
Increasing order:
Silica < kaolinite < illite < Montmorillonite.

Pressure bulb: or isobar: → vertical pressure is same
forms in the width wise direction.

Rigid footing - settlement uniform
Flexible " - contact pressure is uniform