UNIT - I

SOIL CLASSIFICATION AND COMPACTION

Nature of soil - Phase relationships - Soil description and classification for Engineering purposes, their significance - Index properties of soils - BIS Classification system - Soil Compaction - Theory, Comparison of laboratory and field compaction methods - Factors influencing compaction behaviour of soils.

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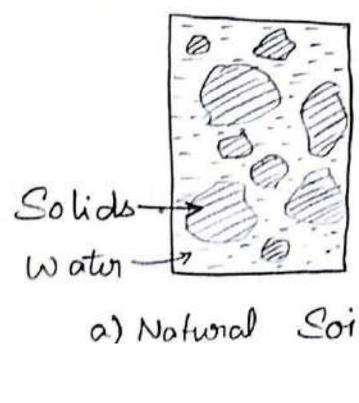
Nature of soil :

A soil mass is a three phase system consisting of solid particles, water and air. The void space between the soil grains is filled partly with water and partly with air.

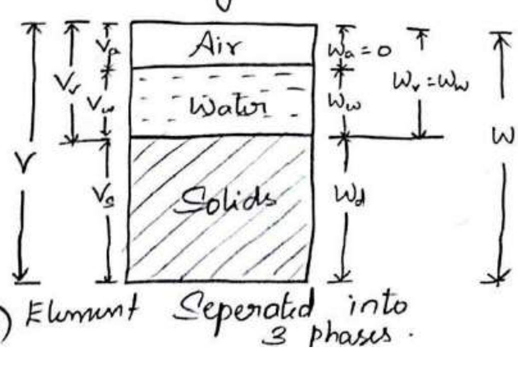
Soil → 3 phase { Solid (Soil grains), Water, Air } Voids (Water + Air)

Dry soil mass → Soil grains + Air

Perfectly saturated soil → Soil grains + water



a) Natural Soil



b) Element Seperated into 3 phases.

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V - Total volume of the soil mass
V_v - Volume of voids
V_s - Volume of solids
V_w - Volume of water
V_a - Volume of air

$$\boxed{\begin{aligned} V_v &= V_w + V_a \\ V &= V_v + V_s \\ &= V_w + V_a + V_s \end{aligned}}$$

W - Total weight of sample
W_d - Weight of solids
W_w - Weight of water
W_a - Weight of air (always zero)
W_v - Weight of voids

$$\boxed{\begin{aligned} W_v &= W_a + W_w \\ W_a &= 0 \\ W_v &= W_w \\ W &= W_d + W_v \\ &= W_d + W_w \end{aligned}}$$

a) Water content (w)

Water content also called moisture content is defined as ratio of weight of water (W_w) to weight of solids (W_d)

$$w = \frac{W_w}{W_d} \times 100$$

w is always expressed as percentage.

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

$$= \left[\frac{W}{W_d} - 1\right] \times 100 .$$

Usual procedure to find the natural water content is to take a mass of about 20 to 30 g. of soil sample in a container and determine its mass M very accurately. The soil

sample is then kept in an oven (105° - 110°C) (2)
for about 24 hours so that it becomes perfectly dry. Its dry mass M_d is then determined and water content is calculated from relation

$$w = \frac{M_w}{M_d} \times 100 = \frac{M - M_d}{M_d} \times 100$$

$$= \left[\frac{M}{M_d} - 1\right] \times 100.$$

b) Density of soil :

It is defined as mass of the soil per unit volume.

i) Bulk density (ρ) / Moist density

It is the total mass of soil per unit of its total volume.

$$\boxed{\rho = \frac{M}{V}} \quad [g/cm^3 \text{ (or) } kg/m^3]$$

ii) Dry density (ρ_d)

It is the mass of solids per unit of total volume of soil mass.

$$\boxed{\rho_d = \frac{M_d}{V}}$$

iii) Density of solids (ρ_s)

It is the mass of soil solids (M_d) per unit volume of solids (V_s)

$$\boxed{\rho_s = \frac{M_d}{V_s}}$$

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iv) Saturated density (ρ_{sat})

When soil mass is saturated, its bulk density is called saturated density. It is the ratio of total soil mass to its total volume.

$$\boxed{\rho_{sat} = \frac{M}{V}}$$

v) Submerged density (ρ') / Buoyant density

It is the submerged mass of soil solids $(M_d)_{sub}$ per unit of total volume V of soil mass

$$\boxed{\rho' = \frac{(M_d)_{sub}}{V}}$$

ρ_w - Density of water

$$\boxed{\rho_w = 1\ g/cm^3}$$

$$\boxed{\rho' = \rho_{sat} - \rho_w}$$

c) Unit weight of soil mass

It is defined as weight of soil per unit volume.

i) Bulk unit weight (γ) / Moist unit weight

It is the total weight (W) of a soil mass per unit of its total volume (V)

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

ii) Dry unit weight (γ_d)

It is the weight of solids per unit of its total volume of soil mass.

$$\boxed{\gamma_d = \frac{W_d}{V}}$$

iii) Unit weight of solids (γ_s)

It is the weight of soil solids per unit of its to volume of solids (V_s)

$$\boxed{\gamma_s = \frac{W_d}{V_s}}$$

$\because V_s$ doesnot alter, γ_s is constant.

γ_d is not constant, it depends on initial volume (V) of soil mass.

iv) Saturated Unit Weight (γ_{sat})

When soil mass is saturated, its bulk unit weight is called saturated unit weight. It is the ratio of total weight of saturated soil sample to its total volume.

$$\boxed{\gamma_{sat} = \frac{W}{V}}$$

v) Submerged unit weight (γ') / Buoyant unit weight

It is the submerged weight of soil solids $(W_d)_{sub}$ per unit volume (V) of soil mass

$$\boxed{\gamma' = \frac{(W_d)_{sub}}{V}}$$

When soil mass is submerged, weight of soil solids is reduced due to buoyancy.

$(W_d)_{sub}$ = Weight of soil solids in air $(-)$ Weight of water displaced by solids

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

$$\boxed{\gamma_w = 9.81\ KN/m^3}$$

γ_w - Unit weight of water

d) Inter conversion b/w density & unit weight

To convert density into unit weight, multiply density by 9.81.

$$\boxed{1 g/cm^3 = 9.81\ KN/m^3}$$

$$\boxed{\gamma = 9.81 \times \rho}$$

Specific Gravity (G)

It is defined as the ratio of weight of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights taken in air.

It is the ratio of unit weight of soil solids to that of water.

$$\boxed{G = \frac{\gamma_s}{\gamma_w}}$$

Indian standard - 27°C - Standard temperature

Soil solids < Permeable voids (may be filled with water) / Impermeable voids

voids are excluded for determining true volume of soils, then it is absolute / true specific gravity

The apparent / mass / bulk specific gravity (G_m) denotes specific gravity of soil mass.

$$\boxed{G_m = \frac{\gamma}{\gamma_m}}$$

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Void ratio (e)

It is the ratio of volume of voids to the volume of soil solids in given soil mass

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

Porosity (n)

It is the ratio of volume of voids to total volume of given mass.

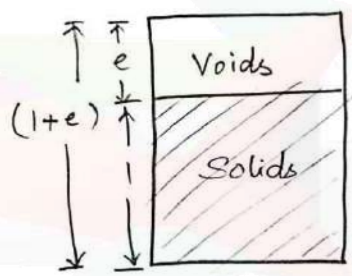
$$\boxed{n = \frac{V_v}{V}}$$

Relation between void ratio and porosity

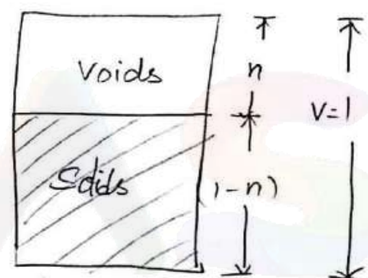
$$e = \frac{V_v}{V_s} \qquad n = \frac{V_v}{V}$$

If $V_s = 1$, $e = V_v$, then $V = V_v + V_s = e + 1 = 1 + e$.

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



a) In terms of e



b) In terms of n

If $V = 1$, $n = V_v$. then $V_s = V - V_v = 1 - n$

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

$$n = \frac{e}{1+e} = e\left[\frac{1}{1+e}\right]$$

$$\cancel{n} = \frac{\cancel{n}}{1-n}\left[\frac{1}{1+e}\right]$$

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

Degree of saturation: (S)

It is defined as ratio of volume of water present in a given soil mass to the total volume of voids in it.

$$\boxed{S = \frac{V_w}{V_v}}$$ → Always expressed as percentage.

Fully saturated soil, $V_w = V_v$, $\boxed{S = 1}$

$$S = \frac{V_w}{V_w} = 1$$

Perfectly dry soil, $V_w = 0$, $\boxed{S = 0}$

Percentage air voids (n_a)

It is defined as the ratio of volume of air voids to total volume of soil mass.

$$\boxed{n_a = \frac{V_a}{V} \times 100}$$ → Expressed as percentage.

Air content (a_c)

It is defined as the ratio of volume of air voids to volume of voids.

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

$V_a = V_v - V_w$,

$$a_c = \frac{V_v - V_w}{V_v} = 1 - \frac{V_w}{V_v}$$

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

Density index (I_D)

Density index (or) relative density (or) degree of density is defined as the ratio of difference between void ratio of soil in its loosest state (e_{max}) and its natural void ratio (e) to the difference between void ratios in the loosest and densest states:

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$$\boxed{I_D = \frac{e_{max} - e}{e_{max} - e_{min}}}$$

e_{max} - void ratio in loosest state

e_{min} - Void ratio in densest state

e - natural voids ratio of the deposit.

It is only used for cohesionless soil.

When cohesionless soil is in its loosest form,

$\boxed{e = e_{max}}$, $\boxed{I_D = 0}$

When cohesionless soil is in its densest state,

$\boxed{e = e_{min}}$, $\boxed{I_D = 1}$

Any intermediate state, $\boxed{I_D = 0 \text{ to } 1}$

I_D in terms of density:

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

$$e_{max} = \frac{G\gamma_w}{\gamma_{d_{min}}} - 1 \qquad e_{min} = \frac{G\gamma_w}{\gamma_{d_{max}}} - 1$$

$$\therefore I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$= \frac{\left[\frac{G\gamma_w}{\gamma_{d_{min}}} - 1\right] - \left[\frac{G\gamma_w}{\gamma_d} - 1\right]}{\left[\frac{G\gamma_w}{\gamma_{d_{min}}} - 1\right] - \left[\frac{G\gamma_w}{\gamma_{d_{max}}} - 1\right]}$$

$$= \frac{\frac{G\gamma_w}{\gamma_{d_{min}}} - \frac{G\gamma_w}{\gamma_d}}{\frac{G\gamma_w}{\gamma_{d_{min}}} - \frac{G\gamma_w}{\gamma_{d_{max}}}} = \frac{\cancel{G\gamma_w}\left[\frac{1}{\gamma_{d_{min}}} - \frac{1}{\gamma_d}\right]}{\cancel{G\gamma_w}\left[\frac{1}{\gamma_{d_{min}}} - \frac{1}{\gamma_{d_{max}}}\right]}$$

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$$I_D = \frac{\dfrac{\gamma_d - \gamma_{dmin}}{\gamma_d \, \gamma_{dmin}}}{\dfrac{\gamma_{dmax} - \gamma_{dmin}}{\gamma_{dmax} \cdot \gamma_{dmin}}}$$

$$\boxed{I_D = \frac{\gamma_d - \gamma_{dmin}}{\gamma_{dmax} - \gamma_{dmin}} \cdot \frac{\gamma_{dmax}}{\gamma_d}}$$

I_D in terms of porosity:

$$\boxed{I_D = \frac{(n_{max} - n)\,(1 - n_{min})}{(n_{max} - n_{min})\,(1-n)}}$$

γ_d - in-situ dry density

γ_{dmax} - maximum dry density (or) dry density at most compact state

γ_{dmin} - Minimum dry density (or) dry density at loosest state.

n - insitu porosity

n_{max} - maximum porosity at loosest state

n_{min} - minimum porosity at densest state.

Relative Density (I_D) (%)	Description
0 - 15	Very loose
15 - 35	Loose
35 - 65	Medium
65 - 85	Dense
85 - 100	Very dense.

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Relative Compaction (R_c)

Degree of compaction expressed in terms of index is called relative compaction.

$$\boxed{R_c = \frac{\gamma_d}{\gamma_{d_{max}}}}$$

$\gamma_{d_{max}}$ - maximum dry density from compaction test.

$\because \gamma_d = \gamma_s (1+e)$

$$\boxed{R_c = \frac{1+e_{min}}{1+e}}$$

In terms of relative density,

$$R_c = \frac{R_o}{1-I_D(1-R_o)} \qquad R_o = \frac{\gamma_{d_{min}}}{\gamma_{d_{max}}}$$

Lee & Singh give approximate relation

$$\boxed{R_c = 80 + 0.2 I_D}$$

When $I_D = 0$, $R_c = 80\%$. $\quad \therefore \boxed{R_c = 80\% \text{ to } 100\%}$

When $I_D = 100$, $R_c = 100\%$.

PHASE RELATIONSHIPS.

i) Relation between e, G, w and S.

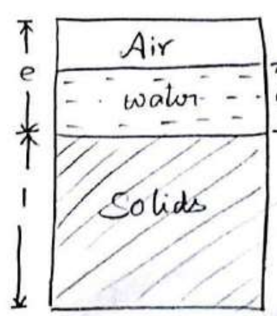
Let Volume of water(V_w) = e_w

Volume of voids(V_v) = e

Volume of solids(V_s) = 1

$$S = \frac{V_w}{V_v} = \frac{e_w}{e}$$

$$e_w = eS \quad \text{—— (a)}$$



In terms of e & e_w

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e_w - Water voids ratio. For fully saturated sample, $e_w = e$.

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

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

$\because V_s = 1,\ W_d = \gamma_s \times 1$

$$= \frac{e_w \gamma_w}{\gamma_s \times 1}$$

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

$\because V_w = e_w,\ W_w = \gamma_w e_w$

$\because G = \frac{\gamma_s}{\gamma_w}$, $\quad w = \frac{e_w}{G}$

$$e_w = wG \quad \text{——— (b)}$$

Equating (a) & (b),

$$eS = wG$$

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

Fully saturated soil,

$S = 1$,

$w = w_{sat}$

$$\underline{e = w_{sat}\, G}$$

ii) Relation between e, S and n_a

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

$V_a = V_v - V_w$

$= e - e_w$

$V = V_s + V_v$

$= 1 + e$

$$n_a = \frac{e - e_w}{1+e}$$

$\because e_w = eS$

$$= \frac{e - eS}{1+e}$$

$$\boxed{n_a = \frac{e[1-S]}{1+e}}$$

iii) Relation between n_a, a_c and n

$$a_c = \frac{V_a}{V_v} \qquad n = \frac{V_v}{V} \qquad n_a = \frac{V_a}{V}$$

$$n_a = \frac{V_a}{V} \times \frac{V_v}{V_v} = \frac{V_a}{V_v} \cdot \frac{V_v}{V}$$

$$= a_c n.$$

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

iv) Relation between γ_d, G and e (or) n

$$\gamma_d = \frac{W_d}{V} \qquad \because W_d = \gamma_s \cdot 1$$

$$\gamma_d = \frac{\gamma_s}{1+e} \qquad V = 1+e$$

$$\because G = \frac{\gamma_s}{\gamma_w}$$

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

For calculating void ratio,

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

We know that, $1-n = \frac{1}{1+e}$

$$\therefore \gamma_d = G\gamma_w(1-n)$$

$$\boxed{\gamma_d = (1-n)\,G\gamma_w}$$

v) Relation between γ_{sat}, G and e (or) n

$$\gamma_{sat} = \frac{W_{sat}}{V} = \frac{W_d + W_w}{V}$$

$$= \frac{\gamma_s V_s + \gamma_w V_w}{V}$$

From fig. $V_s = 1$, $V_w = e_w$, $V = 1+e$.

For fully saturated sample, $e_w = e$. $\therefore V_w = e$

$$\gamma_{sat} = \frac{\gamma_s \cdot 1 + \gamma_w e}{1+e} \qquad \because \gamma_s = G\gamma_w$$

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

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

We know that $(1-n) = \frac{1}{1+e}$ & $n = \frac{e}{1+e}$

$$\gamma_{sat} = G\gamma_w\left[\frac{1}{1+e}\right] + \gamma_w \cdot \left[\frac{e}{1+e}\right]$$

$$\boxed{\gamma_{sat} = (1-n)\,G\gamma_w + n\,\gamma_w.}$$

vi) Relation between γ, G, e and S

$$\gamma = \frac{W}{V} = \frac{W_d + W_w + W_a}{V} \qquad \because W_a = 0.$$

$$\gamma = \frac{\gamma_s V_s + \gamma_w V_w}{V}$$

$V_s = 1$, $V_w = e_w$ & $V = 1+e$.

$$\gamma = \frac{\gamma_s \cdot 1 + \gamma_w e_w}{1+e} \qquad \gamma_s = G\gamma_w.$$

$$= \frac{G\gamma_w + e_w\gamma_w}{1+e} \qquad e_w = eS$$

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

$$\boxed{\gamma = \frac{(G+eS)\,\gamma_w}{1+e}.}$$

If perfectly dry soil, $S = 0$,

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

If perfectly saturated soil, $S = 1$, $\gamma_{sat} = \frac{G+e}{1+e}\gamma_w.$

vii) Relation between γ', G and e

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

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

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

$$\boxed{\gamma' = \frac{(G-1)\,\gamma_w}{1+e}}$$

viii) Relation between γ_d, γ and w

$$w = \frac{W_w}{W_d} \qquad 1+w = 1+\frac{W_w}{W_d}$$

$$= \frac{W_d + W_w}{W_d} = \frac{W}{W_d}.$$

$$W_d = \frac{W}{1+w}$$

$$\therefore \gamma_d = \frac{W_d}{V} = \frac{W}{(1+w)V} \qquad \gamma = \frac{W}{V}.$$

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

ix) Relation between γ', γ_d and n

$$(W_d)_{sub} = (W_d)_{sat} - W_w$$

$$= \gamma_s \cdot 1 - \gamma_w \cdot 1 = G\gamma_w - \gamma_w.$$

$$(W_d)_{sub} = (G-1)\gamma_w.$$

$$\gamma' = \frac{(W_d)_{sub}}{V} = \frac{(G-1)\gamma_w}{V} = \frac{(G-1)\gamma_w}{1+e}.$$

$$\gamma' = \frac{G\gamma_w}{1+e} - \frac{\gamma_w}{1+e}. \qquad (1-n) = \frac{1}{1+e}.$$

$$\boxed{\gamma' = \gamma_d - (1-n)\gamma_w}$$

x) Relation between γ_{sat}, γ, γ_d and S

$$\gamma = \frac{(G+eS)\gamma_w}{1+e}$$

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

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

$$= \gamma_d + S\left[\frac{(G+e)\gamma_w}{1+e} - \frac{G\gamma_w}{1+e}\right]$$

$$\boxed{\gamma = \gamma_d + S\left[\gamma_{sat} - \gamma_d\right]}$$

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xi) Relation between γ_d, G, w and S

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

$$= \frac{G\gamma_w}{1+\frac{wG}{S}}$$

For fully saturated soil, $S = 1$.

$$\boxed{\gamma_d = \frac{G\gamma_w}{1+w_{sat}G}}$$

xiii) Relation between γ_d, G, w and n_a.

$$V = V_a + V_w + V_s$$

$$= V_a + \frac{W_w}{\gamma_w} + \frac{W_d}{\gamma_s}$$

$\div$ by V on both sides.

$\because w = \frac{W_w}{W_d}$.

$W_w = wW_d$.

$$1 = \frac{V_a}{V} + \frac{W_w}{\gamma_w V} + \frac{W_d}{\gamma_s V}$$

$$1 = \frac{V_a}{V} + \frac{w W_d}{\gamma_w V} + \frac{W_d}{\gamma_s V}$$

$\gamma_d = \frac{W_d}{V}$.

$$\left[1 - \frac{V_a}{V}\right] = \frac{w\gamma_d}{\gamma_w} + \frac{\gamma_d}{\gamma_s}$$

$\gamma_s = G\gamma_w$.

$$1 - n_a = w\frac{\gamma_d}{\gamma_w} + \frac{\gamma_d}{G\gamma_w}.$$

$$1 - n_a = \frac{\gamma_d}{\gamma_w}\left[w + \frac{1}{G}\right]$$

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

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

Problems : ①

1. A soil sample has a porosity of 40 %. The specific gravity of solids is 2.7. Calculate.

a) Void ratio

b) Dry density

c) Unit weight if the soil is 50% saturated

d) Unit weight if the soil is completely saturated

$n = 40\% = 0.4 \qquad G = 2.7$

a) $e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$ $\boxed{e = 0.667}$

b) $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{(2.7)(9.81)}{1+0.667}$ $\qquad \gamma_w = 9.81\ kN/m^3$

$\boxed{\gamma_d = 15.89\ kN/m^3}$

c) $\gamma_d = \frac{\gamma}{1+w}$ $\qquad Gw = eS$

$\gamma = \gamma_d(1+w)$ $\qquad S = 0.5$

$w = \frac{eS}{G} = \frac{0.667 \times 0.5}{2.7} = 0.124$

$\gamma = 15.89(1+0.124)$

$\boxed{\gamma = 17.85\ kN/m^3}$

d) $\gamma = \gamma_d(1+w)$ $\qquad w = \frac{eS}{G} = \frac{0.667 \times 1}{2.7} = 0.247.$

$= 15.89(1+0.247)$

$\boxed{\gamma = 19.81\ kN/m^3}$

$\boxed{\gamma_{sat} > \gamma_b > \gamma_d > \gamma'}$

2. An undisturbed sample of soil has a volume of 100 cm^3 and mass of 190g. On oven drying for 24 hours, the mass is reduced to 160g. If the specific gravity of grains is 2.68, determine the water content, void ratio & degree of saturation of soil.

$M = 190g$ After drying $M_d = 160g$.

$M_w = 190 - 160 = 30g$.

$M_d = 160g$

$w = \frac{M_w}{M_d} = \frac{30}{160} = 0.188$.

$\boxed{w = 18.8\%}$

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

$\gamma_d = \frac{\gamma}{1+w}$ $\quad \gamma = 9.81 \times \rho$.

$\rho = \frac{M}{V} = \frac{190}{100} = 1.9 g/cm^3$.

$\gamma = 9.81 \times 1.9 = 18.64 KN/m^3$.

$\gamma_d = \frac{18.64}{1+0.188}$

$\gamma_d = 15.69 KN/m^3$ $\quad G = 2.68$.

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

$\boxed{e = 0.67}$

$eS = wG \implies S = \frac{wG}{e} = \frac{0.188 \times 2.68}{0.67}$

$= 0.744$

$\boxed{S = 74.4\%}$

3. The insitu percentage voids of a sand deposit is 34%. For determining the density index, dried sand from the stratum was first filled loosely in a 1000 cm³ mould and was then vibrated to give a maximum density. The loose dry mass in a mould was 1610g and the dense dry mass at maximum compaction was found to be 1980g. Determine the density index if the specific gravity of the sand particles is 2.67.

Porosity is referred as percentage voids.

$n = 34\% = 0.34$

$V = 1000\ cm^3$

Loose dry mass = 1610g $\qquad G = 2.67$

Dense dry mass = 1980g

$$\text{Density index } (I_D) = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 \qquad \gamma_w = 9.81\ kN/m^3$$

$$e = \frac{n}{1-n} = \frac{0.34}{1-0.34} = 0.515 \qquad \boxed{e = 0.515}$$

$$e_{max} = \frac{G \gamma_w}{(\gamma_d)_{min}} - 1 \qquad e_{min} = \frac{G \gamma_w}{(\gamma_d)_{max}} - 1$$

$$(\gamma_d)_{min} = \frac{1610}{1000} \times 9.81 = 15.79\ kN/m^3 \qquad (\gamma_d)_{max} = \frac{1980}{1000} \times 9.81 = 19.42\ kN/m^3$$

$$e_{max} = \frac{2.67 \times 9.81}{15.79} - 1 = 0.659 \qquad e_{min} = \frac{2.67 \times 9.81}{19.42} - 1$$

$$\boxed{e_{max} = 0.659} \qquad I_D = 0.465 \qquad \boxed{I_D = 46.5\%} \qquad \boxed{e_{min} = 0.349}$$

4. It is required to prepare a compacted cylindrical specimen of 40mm dia and 80mm length from oven dry soil. The specimen is required to have water content of 16% and percent air voids of 18%. Taking G = 2.7, determine the mass of soil and mass of water, required for preparation of above specimen.

Diameter of specimen = 40 mm.

Length of specimen = 80 mm.

$w = 16\%$. $\quad n_a = 18\%$. $\quad G = 2.7$.

Volume of sample $= \frac{\pi}{4} d^2 h. = \frac{\pi}{4}\left[(0.04)^2 \times 0.08\right]$

$V = 1.0053 \times 10^{-4}\ m^3$ — ①

M_d = Mass of solids. $\qquad w = \frac{M_w}{M_d}$.

M_w = Mass of water

$\boxed{M_w = 0.16\, M_d}$. $\qquad \gamma_s = G\gamma_w$.

$V_s = \frac{W_d}{G\gamma_w}$. $\qquad \frac{W_d}{V_s} = G\gamma_w$.

$V_s = \frac{M_d}{G\rho_w}$ $\qquad \frac{1}{V_s} = \frac{G\gamma_w}{W_d}$.

$\rho_w = 1000\ kg/m^3$.

$V_s = \frac{M_d}{2.7 \times 1000} = \frac{M_d}{2700}\ m^3$. — ②.

$V_w = \frac{M_w}{\rho_w} = \frac{0.16 M_d}{1000} = 1.6 \times 10^{-4} M_d\ m^3$ — ③.

$V_a = n_a V = 0.18 \times 1.0053 \times 10^{-4}$

$V_a = 1.8095 \times 10^{-5}\ m^3$.

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

$$1.0053 \times 10^{-4} = \frac{M_d}{2700} + 1.6 \times 10^{-4} M_d + 1.8095 \times 10^{-5}$$

$$(1.0053 \times 10^{-4}) - (1.8095 \times 10^{-5}) = 3.7037 \times 10^{-4} M_d + 1.6 \times 10^{-4} M_d.$$

$$8.2435 \times 10^{-5} = 5.304 \times 10^{-4} M_d.$$

$$M_d = 0.1554 \text{ kg}.$$

$$\boxed{M_d = 155.4g}.$$

$$M_w = 0.16 M_d = 0.16 \times 155.4$$

$$\boxed{M_w = 24.9g}$$

INDEX PROPERTIES

* Water content
* Specific gravity
* Particle size distribution
* Consistency limits
* Insitu density
* Density index.

i) Water content:

Water content of soil sample can be determined by following methods.

a) Oven drying method.
b) Sand bath method.
c) Alcohol method
d) Calcium carbide method.
e) Pycnometer method.
f) Radiation method.
g) Torsion balance method.

a) Oven drying method : (Accurate method).

A specimen of soil sample is kept in a clean container and put in a thermostatically controlled oven with interior of non-corroding material to maintain temperature between 105°C to 110°C. Usually sample is kept for about 24 hours in oven so that complete drying is assured.

Temperature > 110°C - Break crystalline structure

For highly organic soils, lower temperature of 60°C is preferred to prevent oxidation of organic matter.

If gypsum present in soil, sample is dried not more than 80°C but for a long time.

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100$$

M_1 - Mass of container with lid.
M_2 - Mass of container with lid & wet soil
M_3 - Mass of container with lid & dry soil.

b) Sand bath Method : (Field method).

The container with soil is placed on a sand bath. The sand bath is heated over a kerosene stove. The soils becomes dry within 1/2 to 1hr.

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100.$$

c) Alcohol Method : (Crude Field Method)

The wet soil sample is kept in a

evaporating dish and mixed with sufficient quantity of methylated spirit. The dish is then properly covered and the mixture is ignited. It is kept stirred by a wire during ignition. Since there is no control over temperature, it should not be used for soils containing large % of organic matter or gypsum.

d) Calcium Carbide Method : (quick method)

6g of wet soil sample is placed in an air tight container and is mixed with sufficient quantity of fresh calcium carbide powder. The mixture is shaken vigorously.

The acetylene gas produced, exerts pressure on a sensitive diaphragm placed at end of container.

The dial gauge located at diaphragm reads the water content directly.

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

w' - water content based on wet weight.

w - water content based on dry weight.

e) Pycnometer method : (quick method).

It is used for soils whose specific gravity G is accurately known.

Pycnometer - Large size density bottle of 900ml capacity
- Conical brass cap, 6mm dia hole at its top -

- Rubber washer placed b/w conical cap & rim of both so that there is no leakage of water.

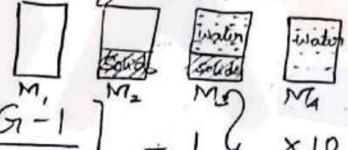
Procedure :

* Take clean, dry pycnometer & find its ma mass with cap & washer (M_1)

* Put about 200g to 400g of wet soil in pycnometer & find mass (M_2).

* Add water & stir it. Fill the pycnometer with water & find its mass (M_3).

* Empty the pycnometer, clean it & fill with clean water to hole of conical cap & find its mass (M_4).



$$w = \left\{ \left[\frac{M_2 - M_1}{M_3 - M_4}\right] \left[\frac{G-1}{G}\right] - 1 \right\} \times 100.$$

f) Radiation Method :

It is useful for determining water content of soil deposit in insitu condition. It uses two steel castings - casting A and casting B which are placed in two bore holes at some distance apart, in soil deposit.

*Device containing some radio-active isotope material is placed in a capsule which is lowered into casing A.

* Detector unit is lowered in steel casing B.

* Small openings are made in A & B, facing each other.

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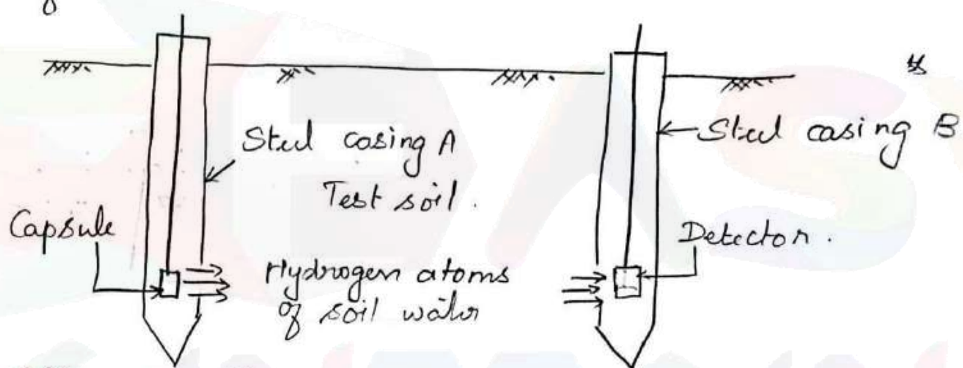
* When radio active device is activated, it (13)
emit neutrons.

* Neutrons strike through hydrogen atoms of water, they loose energy.

* The loss of energy is equal to water content of soil.

* The detector device is calibrated to given directly the water content of subsoil, at level of emission.

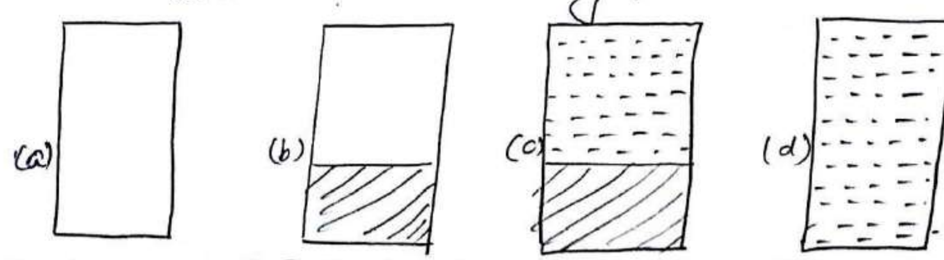


iii) Specific gravity:

It is determined by

i) 50ml density bottle ← Most accurate & suitable for all types of soils.
ii) 500ml flask.
iii) pycnometer. } ← Coarse grained soils.

Density bottle method is the standard method used in laboratory.



Empty bottle. (M_1) Bottle + dry soil (M_2) Bot + dry soil + water (M_3) Bot + water (M_4)

Dry Mass of soil $M_d = M_2 - M_1$

Mass of water in (c) $= M_3 - M_2$.

Mass of water in (d) $= M_4 - M_1$

Mass of water having the same volume as that of soil solids is $= \underline{(M_4 - M_1) - (M_3 - M_2)}$

$$G = \frac{\text{Dry mass of soil}}{\text{Mass of water of equal volume}}$$

$$= \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} = \frac{M_2 - M_1}{M_4 - M_1 - M_3 + M_2}$$

$$\boxed{G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}}$$

$$\boxed{G = \frac{M_d}{M_d - (M_3 - M_4)}}$$

If temperature is $T_t^\circ C$

$$G_{27^\circ C} = G_{T^\circ C} \frac{\text{Sp.gr of water at } T_t^\circ C}{\text{Sp.gr of water at } 27^\circ C}$$

Distilled water is used in flask or pycnometer test

Kerosene is used in density bottle method.

Problems:

Inorder to determine the water content, 370g of a wet sandy sample was placed in a pycnometer. The mass of the pycnometer, sand and water full to the top of conical cap was found to be 2148g. The mass of pycnometer full of clean water was 1932g. Take $G = 2.65$.

Determine water content of sample.

$M_2 = 370g$. $M_3 = 2148g$ $M_4 = 1932g$

$G = 2.65$.

$$w = \left[\frac{M_2 - M_1}{M_3 - M_4} \cdot \frac{G - 1}{G} - 1\right] \times 100$$

$$= \left[\frac{370}{2148 - 1932} \cdot \frac{2.65 - 1}{2.65} - 1\right] \times 100$$

$w = 6.5\%$.

iii) Particle - size distribution.

The percentage of various sizes of particles in a given dry soil sample is found by particle size analysis (or) mechanical analysis.

It is meant for seperation of soil into its different size fractions. It is performed in two stages.

i) Sieve analysis

ii) Sedimentation analysis (or) wet mechanical analysis.

Sieve analysis - Coarse grained soils

Sedimentation analysis - Fine grained soils.

Both stages are necessary.

Sieve Analysis:

In BS & ASTM standards, sieve sizes are given in terms of number of openings per inch.

The number of openings per square inch is equal to square of number of sieve.

Code - IS : 460 - 1962.

Sieve Analysis < Coarse analysis / Fine analysis

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An oven-dried sample of soil is seperated into two fractions by sieving it through 4.75 mm IS sieve.

Portion retained on 4.75 mm sieve – Gravel fraction Coarse Analysis.

Portion passing through 4.75 mm sieve – Fine Analysis.

Set of sieves

Coarse Sieve Analysis	Fine Sieve Analysis.
IS : 100 mm	IS : 2 mm
63 mm	1 mm
20 mm	600 μ
10 mm	425 μ.
4.75 mm.	300 μ
	212 μ
	150 μ
	75 μ.

* It is advisable to wash the soil portion passing through 4.75 mm sieve over 75 μ sieve so that clay and silt particles sticking to sand particles may be dislodged.

* Two grams of sodium hexametaphosphate is added per litre of water used.

* Washing should be continued until the water passing through 75 μ sieve is substantially clean.

* The fraction retained on 75 μ sieve is dried in oven. The dried portion is resieved through fine analysis.

Sedimentation Analysis:

In sedimentation Analysis, the soil fraction finer than 75 micron sieve is kept in suspension

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in a liquid medium. (15)

The analysis is based on Stoke's law, according to which velocity at which grains settle out of suspension, all other factors being equal, is dependent on shape, weight & size of grain.

It is assumed that soil particles are spherical and have same specific gravity. With this assumption, coarser particles settle more quickly than finer ones.

v - terminal velocity of sinking of spherical particle

$$v = \frac{2}{9} r^2 \frac{\gamma_s - \gamma_w}{\eta} \qquad r = \frac{D}{2}$$

$$v = \frac{1}{18} D^2 \frac{\gamma_s - \gamma_w}{\eta} \qquad r^2 = \frac{D^2}{4}$$

r - radius of spherical particle (m)
D - Diameter of spherical particle (m)
v - Terminal velocity (m/sec)
γ_s - Unit weight of particles (KN/m³)
γ_w - Unit weight of water / liquid (KN/m³)
η - Viscosity of water / liquid (KN-s/m²) $= \frac{\mu}{g}$
μ - Viscosity in absolute units of poise
g - acceleration due to gravity.

If water is used as medium for suspension, $\gamma_w = 9.81$ KN/m³, $\gamma_s = G\gamma_w$.

$$v = \frac{1}{18} D^2 \frac{(G-1)\gamma_w}{\eta}$$

If D is in mm, then

$$v = \frac{1}{18}\left(\frac{D}{1000}\right)^2 \frac{(G-1)\gamma_w}{\eta} = \frac{D^2 \gamma_w (G-1)}{18 \times 10^6 \eta}$$

$\gamma_w = 9.81 \ kN/m^3$, $\quad v = \dfrac{D^2 \times (9.81)(G-1)}{18 \times 10^6 \eta}$

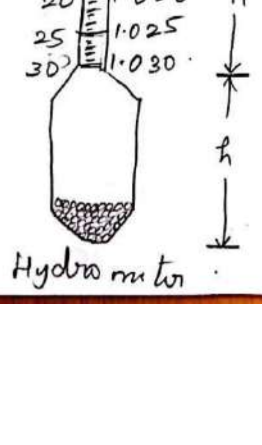
$$= \frac{D^2 (G-1)}{1.835 \times 10^6 \eta}$$

$$D^2 = \frac{18 \times 10^6 \eta \cdot v}{9.81 (G-1)}$$

$$\boxed{D = \sqrt{\frac{18 \times 10^6 \eta \cdot v}{\gamma_w (G-1)}}}$$

Hydrometer Analysis

* The mass M_s per ml of suspension is computed indirectly by reading the density of soil suspension at depth H_e at various time intervals.
* The sampling depth H_e goes on increasing as the particles settle with increase in time intervals.
* It is necessary to calibrate the hydrometer & ~~hydro~~ sedimentation jar before start of sedimentation test, to find relation b/w H_e and density readings of hydrometer.



Hydrometer

* The readings on hydrometer stem gives the density of soil suspension situated at centre of bulb at any time.
* For convenience, hydrometer readings are recorded after subtracting 1 & multiplying remaining digits by 1000.
* Such a reduced reading is R_h.

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If reading is 1.010. $R_h = (1.010-1)\times 1000 = 10$ (h).
If reading is 0.995, $R_h = (0.995-1)\times 1000 = -5$.

R_h increase in downward direction towards hydrometer bulb.

$$H_e = \left[H + \frac{h}{2} + \frac{V_h}{2A}\right] - \frac{V_h}{A}.$$

$$= H + \frac{h}{2} + \frac{2V_h - V_h}{2A}.$$

$$= H + \frac{h}{2} + \frac{V_h}{2A}$$

$$\boxed{H_e = H + \frac{1}{2}\left[h + \frac{V_h}{A}\right]}.$$

Hydrometers are calibrated @ 27°C, If temperature of soil suspension is not 27°C, temperature correction C_t is applied.

If $T > 27°C$, Reading is less, correction (+ve)
If $T < 27°C$, Correction (−ve)

i) Temperature correction
ii) Meniscus correction
iii) Dispersing agent correction.

Meniscus correction - Always (+ve)

The adding of dispersing agent in water increase it's density, C_d is always (−ve).

$$\boxed{R = R_h' + C_m \pm C_t - C_d}$$

3 corrections can be combined into 1 correction, composite correction ($\pm C$).

$$\boxed{R = R_h' \pm C}$$

Particle Size Distribution Curve :

The results of the mechanical analysis are plotted to get a particle size distribution curve with the percentage finer N as ordinate and particle diameter as abscissa, diameter being plotted on logarithmic scale.

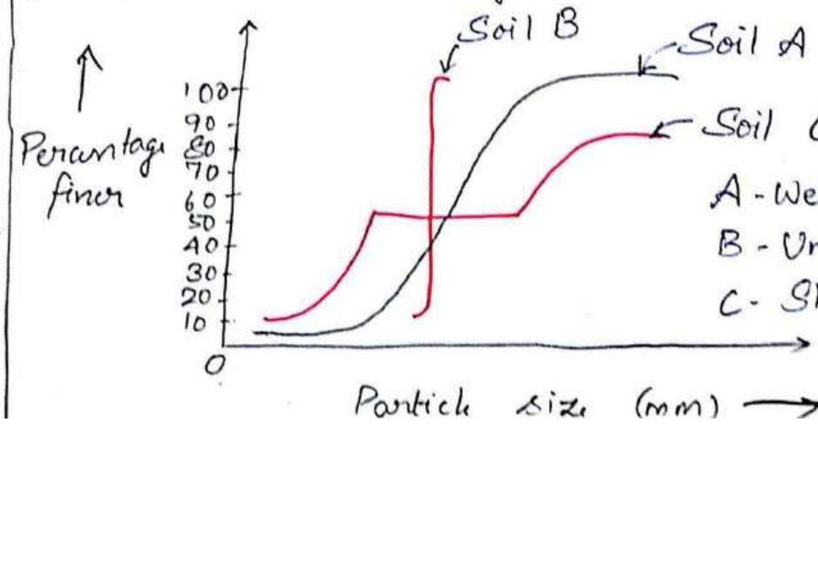
It gives an idea about type and gradation of soil.

A curve situated higher up or to the left represents a relatively fine grained soil while a curve situated to the right represents coarse grained soil.

Soil
- Well graded ← Particles of all sizes.
- Poorly graded (Uniformly graded) ← Excess of certain particles and deficiency of other.

Uniformly graded - Particles of about same size.

A curve with flat portion represent a soil in which some intermediate size particles are missing. Such a soil is known as gap graded or skip graded.



A - Well graded soil
B - Uniform soil
C - Skip/gap graded soil.

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For coarse grained soil, certain particle (17)
sizes such as D_{10}, D_{30}, D_{60} are important.

D_{10} - Size, in mm such that 10% of particles are finer than this size.

D_{60} - 60% of total mass of sample are finer.

D_{10} - Effective size or effective diameter

Uniformity Coefficient (C_u) - Measure of particle-size range and is given by ratio of D_{60} and D_{10} sizes.

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

Coefficient of Curvature (C_c)

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

Gravels - $C_u > 4$.
Sands - $C_u > 6$.

Uniformly graded soil, $C_u = 1$.
Well graded soil, $C_c = 1$ to 3.

Problem:

A soil sample, consisting of particles of size ranging from 0.5mm to 0.01mm, is put on the surface of still water tank 5 meters deep. Calculate the time of settlement of the coarsest and finest particles of the sample, to the bottom of the tank. Assume average specific gravity of soil particles as 2.66 and viscosity of water as 0.01 poise.

$G = 2.66 \qquad \eta = 0.01 \times 10^{-4}\ kN{\cdot}s/m^2$

$D = 0.5mm$ to $0.01mm. \qquad h = 5m$

$$v = \frac{D^2 \gamma_w (G-1)}{18 \times 10^6 \eta}$$

$$= \frac{D^2 \times 9.81 (2.66-1)}{18 \times 10^6 \times 0.01 \times 10^{-4}}$$

$$v = 0.905 D^2$$

If $D = 0.5$ mm, $v = 0.905 (0.5)^2$
$= 0.2263$ m/sec.

$$t = \frac{h}{v} = \frac{5}{0.2263}$$

$$\boxed{t = 22.1 s}$$

If $D = 0.01$ mm, $v = 0.905 (0.01)^2$
$= 9.05 \times 10^{-5}$ m/sec.

$$t = \frac{h}{v} = \frac{5}{9.05 \times 10^{-5}} = 55249 \text{ sec.}$$

$$\boxed{t = 15 \text{ hrs } 20 \text{ min } 49 \text{ sec}}$$

—— × ——

CONSISTENCY OF SOILS.

Consistency denotes degree of firmness of soil which may be termed as soft, firm, stiff or hard.

Atterberg divided the entire range from liquid to solid state into four stages.

Solid	Semi solid	Plastic	Liquid. $(\tau = 0)$

w_s — w_p (Working test) — w_L (Flow test.)

τ - Shear strength $= 0$.

w_L - Liquid limit

w_p - Plastic limit.

w_s - Shrinkage limit

$w > w_L$ - Liquid state

$w < w_s$ - Solid state..

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Liquid limit (w_L)

It is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing.

It is the minimum water content at which a part of soil cut by a groove of standard dimensions, will flow together for a distance of 12mm (1/2 inch) under an impact of 25 blows in the device.

Plastic limit (w_p)

It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3mm in diameter.

Shrinkage limit (w_s)

It is defined as the maximum water content at which a reduction in water content will not cause a decrease in volume of soil mass.

Plasticity index (I_p)

The range of consistency within which soil exhibits plastic properties is called plastic range and is indicated by plasticity index.

It is defined as the difference b/w liquid limit and plastic limit of soil.

$\boxed{I_p = w_L - w_p}$. If $\boxed{w_p \geq w_L, I_p = 0}$.

Plasticity :

It is defined as the property of soil which allows it to be deformed rapidly, without rupture, without elastic rebound and without volume change.

Consistency Index (I_c) :/ Relative Consistency

It is defined as the ratio of liquid limit minus natural water content to the plasticity index of soil.

$$\boxed{I_c = \frac{w_L - w_c}{I_p}}$$

w - Natural water content.

$I_c = 1 \Rightarrow$ Plastic limit

$I_c = 0 \Rightarrow$ Liquid limit.

$I_c > 1 \Rightarrow$ Semi-solid state.

$I_c < 1 \Rightarrow w > w_L$, behaves like liquid.

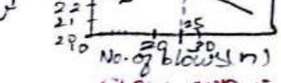
Liquidity Index (I_L) / water-plasticity ratio.

It is the ratio, expressed as percentage, of the natural water content of a soil minus its plastic limit to its plasticity index.

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

Determination of Liquid limit & Plastic limit:

The liquid limit is determined in lab with the help of standard liquid limit apparatus designed by Casagrande.



FLOW CURVE

$$w_1 - w_2 = I_f \log_{10} \frac{n_2}{n_1}$$

Flow index :

Flow index / Slope of curve is determined

$$\boxed{I_f = \frac{w_1 - w_2}{\log_{10} \frac{n_2}{n_1}}}$$

Selecting the values of n_2 & n_1 corresponding to number of blows over one log-cycle difference $\log_{10} \frac{n_2}{n_1}$ becomes equal to unity, I_f becomes equal to difference b/w corresponding water contents.

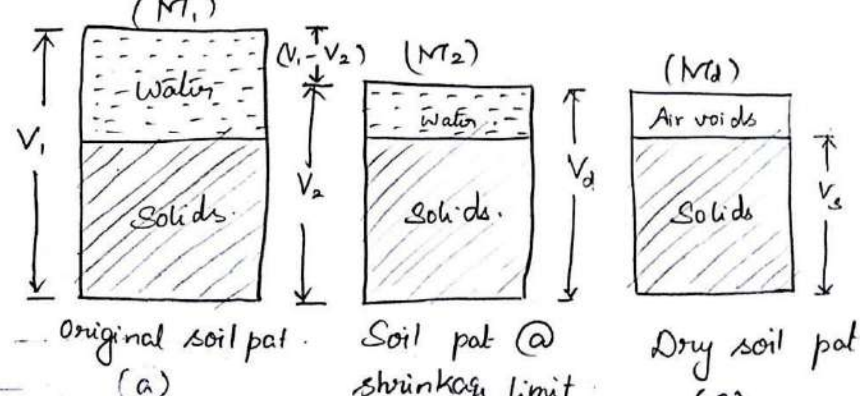
Toughness Index :

It is defined as the ratio of plasticity index to flow index.

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

Shrinkage limit :

If a saturated soil sample is taken, and allowed to dry up gradually, its volume will go on reducing till a stage will come after which the reduction in soil water will not result in further reduction in total volume of soil sample. The water content at this stage is shrinkage limit.



Original soil pat (a) | Soil pat @ shrinkage limit (b) | Dry soil pat (c)

Mass of water in (a) $= M_1 - M_d$.

Loss of water from (a) to (b) $= (V_1 - V_2)\,\rho_w$

Mass of water in (b) $= (M_1 - M_d) - (V_1 - V_2)\,\rho_w$.

$$w_s = \frac{(M_1 - M_d) - (V_1 - V_2)\,\rho_w}{M_d} \times 100.$$

$\because V_2 = V_d$

$$\boxed{w_s = \left[w_1 - \frac{(V_1 - V_d)\,\rho_w}{M_d}\right] \times 100.}$$

$$w_s = \left[w_1 - \frac{(V_1 - V_d)\,\gamma_w}{W_d}\right] \times 100.$$

w_1 - Water content of original saturated sample of volume V_1.

V_d - dry volume of soil sample.

W_d - dry weight of soil sample.

M_d - Dry mass of soil sample.

Value of G from shrinkage limit test:

$$\gamma_s = G\gamma_w = \frac{M_d \times 9.81}{V_s}.$$

$$\text{So, } G = \frac{M_d}{V_s}.$$

$$V_s = V_1 - \frac{M_1 - M_d}{\rho_w}$$

$$G = \frac{M_d}{V_1 - \frac{M_1 - M_d}{\rho_w}}.$$

$$G = \frac{M_d\,\rho_w}{V_1\rho_w - (M_1 - M_d)}$$

$\rho_w = 1\,g/cc$.

$$\boxed{G = \frac{M_d}{V_1 - (M_1 - M_d)}}$$

$$\boxed{G = \frac{1}{\frac{\gamma_w}{\gamma_d} - \frac{w_s}{100}}}$$

Shrinkage Ratio (SR)

It is defined as the ratio of given volume change expressed as a percentage of dry volume, to the corresponding change in water content above the shrinkage limit expressed as a percentage of weight of oven dried soil

$$\boxed{S.R = \frac{\frac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2}}$$

V_1 : Volume of soil mass at water content w_1

V_2 - Volume of soil mass at water content w_2.

V_d - Volume of dry soil mass.

At shrinkage limit,

$$\boxed{SR = \frac{\left[\frac{V_1 - V_d}{V_d}\right] \times 100}{w_1 - w_s}}$$

Shrinkage ratio of soil is equal to mass specific gravity of soil in dry state.

Volumetric Shrinkage (VS)

It is defined as the decrease in volume of soil mass, expressed as percentage of dry volume of soil mass, when the water content is reduced from a given percentage to shrinkage limit.

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

$$\because SR = \frac{\left[\frac{V_1 - V_d}{V_d}\right] \times 100}{w_1 - w_s}.$$

But $\frac{V_1 - V_d}{V_d} \times 100 = (w_1 - w_s) SR$

$$\boxed{VS = (w_1 - w_s) SR}$$

Linear Shrinkage (L_s)

It is defined as the decrease in one dimension of soil mass expressed as percentage of original dimension, when water content is reduced from a given value to shrinkage limit.

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

ACTIVITY OF CLAYS

It is defined as the ratio of plasticity index to the percent by weight of soil particles of diameter smaller than two microns present in the soil.

$$\boxed{A_c = \frac{I_p}{C_w}}$$

C_w - Percentage, by weight of clay sizes (i.e. particles of size less than 2 microns).

Activity	Classification.
<0.75	Inactive
0.75 - 1.4	Normal
>1.4	Active.

Kaolinite - 0.4 - 0.5

Illite - 0.5 - 1

Montmorillonite - 1 - 7.

SENSITIVITY OF CLAYS

The degree of disturbance of undisturbed clay sample due to remoulding is expressed by Sensitivity (S_t).

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It is defined as the ratio of its unconfined compression strength in the natural or undisturbed state to that in the remoulded state, without change in water content ;

$$\boxed{S_t = \frac{q_u \text{ (undisturbed)}}{q_u \text{ (remoulded)}}} \qquad S_t = 1 \text{ to } 8.$$

Sensitivity	Classification	Structure.
1	Insensitive.	
2 to 4	Normal / less sensitive	Honey Comb structure.
4 to 8	Sensitive	Honey comb or flocculent structure.
8 to 16	Extra sensitive	Flocculent structure
>16	Quick	Unstable.

THIXOTROPY OF CLAYS

The phenomenon of 'strength loss - strength gain' with no change in volume or water content is called thixotropy.

It is defined as an isothermal, reversible, time dependent process which occurs under constant composition and volume, thereby a material softens, as a result of remoulding and then gradually returns to its original strength when allowed to rest.

Sensitivity larger, Thixotropic hardening larger.

CLASSIFICATION OF SOILS

Soils may be classified by following system

1. Particle Size classification.
2. Textural classification
3. Highway Research Board (HRB) classification
4. Unified Soil classification and
   IS classification system

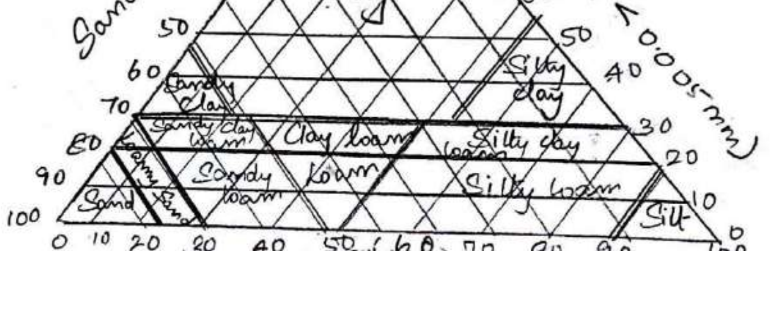
Particle Size Classification:

In this system, soils are arranged according to grain size. Terms such as gravel, sand, silt and clay are used to indicate grain sizes.

Textural Classification:

Soil classification of composite soils exclusively based on the particle size distribution is known as textural classification.

The best known is the triangular classification of U.S. Public Roads Administration. The classification is based on percentages of sand, silt and clay sizes making up the soil.



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Highway Research Board (HRB) Classification (22)

- HRB classification system, also known as Public Road Administration (PRA) classification system, is based on both particle size composition as well as plasticity characteristics.

  * Used for pavement construction.

  * Soils are divided into 7 primary groups, A-1, A-2, A-3, A-4, A-5, A-6, A-7.

Group index = $0.2a + 0.005\,a \cdot c + 0.01\,b \cdot d$.

a - % passing 75 μ greater than 35, not exceeding 75 (0 to 40)

b - % passing 75 μ greater than 15, not exceeding 55. (0 to 40)

$c = w_L - 40$ [0 to 20]

$d = I_p - 10$ [0 to 20].

GI differs b/w 0 to 20.

GI = 0 - Good soil

GI = 20 - Poor soil.

Unified Soil Classification System (USCS).

Soils are classified into four major groups.

i) Coarse grained
ii) Fine grained.
iii) Organic Soils
iv) Peat.

Gravel - G	Well graded - W
Sand - S	Poorly graded - P
Silt - M	Silty - M
Clay - C	Clayey - C
Organic - O	$w_L < 50\%$ - L
Peat - Pt	$w_L > 50\%$ = H

Soil → Organic ← Visual classification (Peat, Muk, Humus)
Soil → Inorganic.

Inorganic — 75 μ:
- Coarse grained. (> 50% retained)
- Fine grained. (> 50% passing)

Coarse grained — 4.75 mm:
- Gravels (> 50% retained)
- Sand. (> 50% passing)

Gravels (> 50% retained):
- Clean gravel (fines < 5%)
  - Well graded gravel.
    - * $C_u > 4$
    - * $C_c = 1-3$
  - Poorly graded gravel.
- Gravel with fines. (fines > 12%)
  - Clayey gravel, Silty gravel. — Based on plasticity chart

Sand. (> 50% passing):
- Clean sand.
  - Well graded sand.
    - * $C_u > 6$
    - * $C_c - 1-3$
  - Poorly graded sand.
- Sand with fines
  - Clayey sand, Silty sand — Based on Plasticity chart

Fine grained:
- Silt, Clay. — Based on plasticity chart.

Plasticity Chart:

C_c - Compression index.

$w_L \propto C_c$.

$w_L > 50\%$. - Highly Compressible (H)

$w_L < 35\%$. - Low compressible (L)

w_L - 35-50%. - Intermediate compressibility (I).

$I_{p\,max}$ - Clayey $\quad$ $I_{p\,min}$ - Silty.

This chart is devised by Casagrande. (1948).

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

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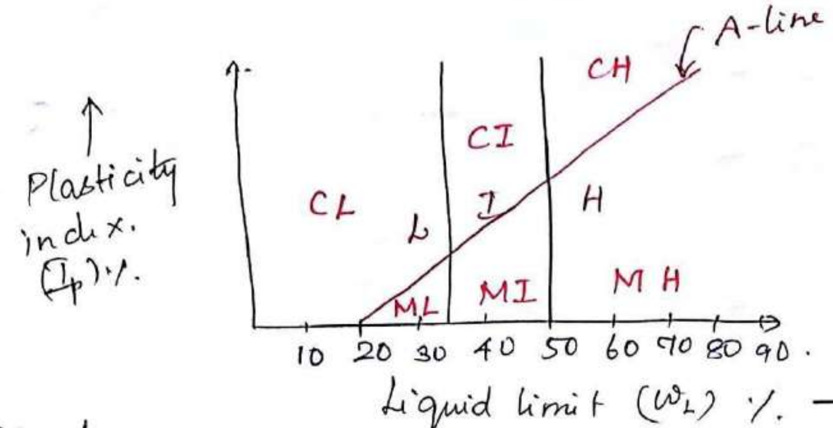

CL - Low compressibility clay
CI - Intermediate compressibility clay
CH - High Compressibility clay
ML - Low compressibility Silt
MI - Intermediate compressibility silt
MH - High Compressibility Silt.

Indian Standard Classification System (ISCS)

Code : IS : 1498 - 1970.

Soil is divided into
- USCS - 15 groups.
- IS CS - 18 groups.

Broadly classified into 3 groups.
1. Coarse grained.
2. Fine grained
3. Highly Organic.

Gravel
- Clean gravel: GW, GP
- Gravel with fines: GC, GM

Sand
- SW, SP, SC, SM

Major divisions				Symbol	Classification Criteria
Coarse grained	Gravel	Clean gravel	Well graded gravel	GW	$C_u > 4$, $C_c = 1-3$
			Poorly graded gravel	GP	-
		Gravel with fines	Clayey gravel	GC	Plasticity chart
			Silty gravel	GM	Plasticity chart
	Sand.	Clean sand	Well Graded sand	SW	$C_u > 6$, $C_c = 1-3$
			Poorly Graded sand	SP	—
		Sand with fines.	Clayey Sand	SC	Plasticity chart
			Silty sand.	SM	Plasticity chart
Fine grained	Silt	Highly Compressible silt		MH	Plasticity chart - $w_L > 50$
		Intermediate Compressible silt		MI	" $w_L = 35-50\%$
		Low Compressible silt		ML	" $w_L < 35\%$
	Clay.	High Compressible clay		CH	" $w_L > 50\%$
		Intermediate Compressible clay		CI	" $w_L = 35-50\%$
		Low compressible clay		CL	" $w_L < 35\%$
Organic soil	High Compressible organic clay			OH	" w
	Intermediate compressible Organic clay & silt			OI	"
	Low Compressible organic silt.			OL	"
	High Organic soil			P_t	Peat

COMPACTION

(2A)

Compaction is a process by which the soil particles are artificially rearranged and packed together into a closer state of contact by mechanical means inorder to decrease the porosity of soil and thus increase its dry density.

It may be accomplished by rolling, tamping or vibration.

At maximum dry density, "optimum water content" is reached.

Laboratory Compaction Methods

The tests are based on any one of the types of compaction : dynamic or impact, kneading, static and vibration.

Usual compaction methods used in laboratory to determine water-density relationships

* Standard Proctor test
* Modified Proctor test.
* Harward Miniature Compaction Test.
* Abbot compaction test.
* Jodhpur-mini Compactor test.

Standard Proctor test :

It was developed by R.R. Proctor (1933) for construction of Earth fill dams in state of California. The test equipment consists of

i) Cylindrical metal mould, having an int. diameter of 4 inches, an effective height of 4.6 in (11.7 cm) & a capacity of 1/30 cu.ft

ii) detachable base plate.

iii) Collar 2 in in effective height

iv) Rammer 5.5 lb (2.5 kg) in mass falling through a height of 12 in (30.5 cm).

* The test consists in compacting soil at various water contents in the mould, in three equal layers, each layer being given 25 blows of the 5.5 lb rammer dropped from a height of 12 in.

* Dry density obtained in each test is determined by knowing the mass of compacted soil and its water content.

* The compactive energy used for this test is 6065 kg cm per 1000 ml of soil.

IS: 2720 (Part VII) 1980

Mould - 1000 ml capacity

Internal dia - 100 mm.

Int. Effective height - 127.5 mm.

Rammer mass - 2.6 kg.

Drop - 310 mm.

* About 3 kg of air dried soil, passing 4.75 mm sieve, is mixed throughly with water.

* The quantity of water to be added may be taken 4% for coarse grained and 10% for fine grained soils.

* The empty mould attached to base plate is weighed without collar.

* The collar is then attached to mould.

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* The mixed and mature soil is placed in the mould and compacted by giving 25 blows of rammer uniformly distributed over surface, such that compacted height of soil is about 1/3 of height of mould.

* The second and third layers are similarly compacted. each layer being 25 blows.

* The last compacted layer should project not more than 6mm into collar.

* The collar is removed and excess soil is trimmed off to make it level with top of mould.

* Weight of mould, base plate and compacted soil is taken.

* A representative sample is taken from centre of compacted specimen and kept for water content determination.

* Bulk density (ρ) & dry density (ρ_d) for compacted soil is calculated from:

$$\boxed{\rho = \frac{M}{V} \ (g/cm^3)} \qquad \boxed{\rho_d = \frac{\rho}{1+w} \ (g/cm^3)}$$

M - Mass of wet compacted specimen

V - Volume of mould

* The compacted soil is taken out of the mould, broken with hand and remixed with raised water content (by 2 to 4 %).

* The test is repeated on soil samples

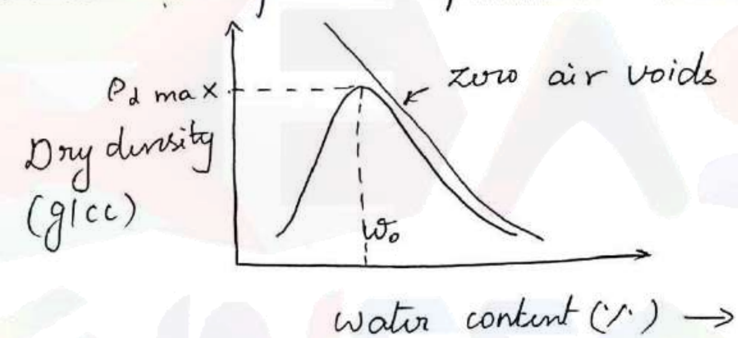
with increasing water contents & corresponding dry density ρ_d obtained is determined.

* A compaction curve is plotted b/w. water content as abscissa and dry densities as ordinate.

γ increases, w increases. till maximum dry density.

<u>Optimum water content</u> :

The water content corresponding to maximum density is optimum water content w_0.



Zero air voids line :

A line shows the water content dry den relation for compacted soil containing a constan percentage air voids, is known as zero air void line.

$$\rho_d = \frac{(1-n_a)\, G\, \rho_w}{1 + wG}$$

n_a - percentage air voids.

G - Specific gravity.

The line showing dry density as a function of water content for soil containing no air voids is called zero air voids line or saturation line. $\rho_d = \dfrac{G\rho_w}{1+wG}$

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Modified Proctor test / AASHO test

* It was developed to give higher standard of compaction.

* In this test, soil is compacted in Standard Proctor mould, but in five layers, each layer being given 25 blows of 10 lb (4.5 kg) rammer dropped through height of 18 inches.

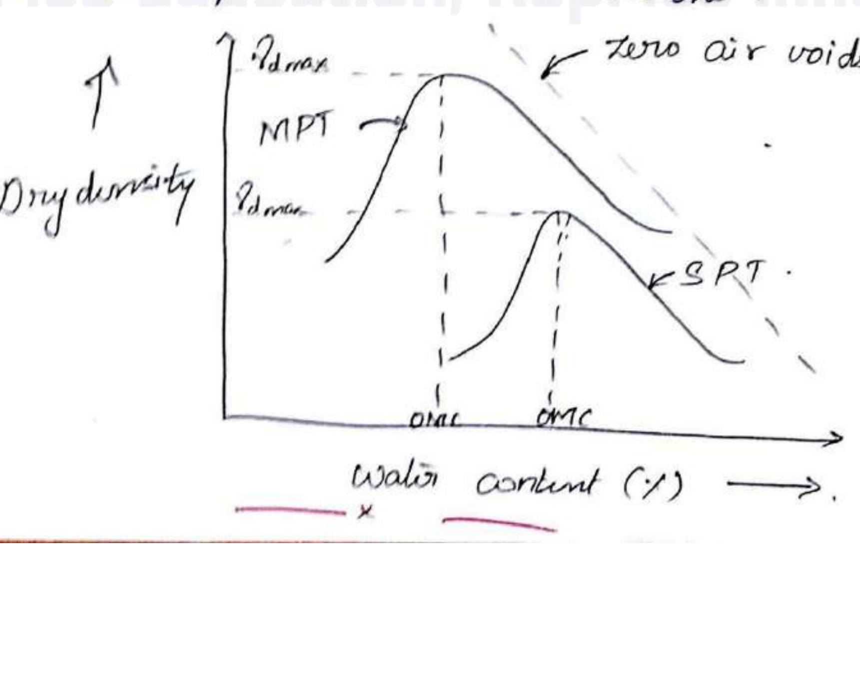
* Compactive energy - 27260 kg-cm/1000 cm^3 of soil which is 4½ times that of SPT.

IS: 2720 (Part VII) - 1980/87

- Rammer - 4.9 kg.
  Drop - 45 cm.

Water content dry density curve lies above SPT curve & has its peak placed towards left.

∴ For same soil, effect of heavy compaction is to increase in maximum dry density & to decrease optimum water content.



Field Compaction Methods :

* Rolling ~~(Rammers~~ (Rollers)
* Ramming (Rammers)
* Vibration. (Vibrators).

Rolling equipments →
- Smooth wheel rollers.
- Pneumatic tyred rollers
- Sheep foot rollers.
- Lorries & pneumatic tyred construction plant
- Track laying vehicles.

Ramming equipments →
- Dropping weight type
- Internal Combustion type
- Pneumatic type.

Vibrating equipment →
- Dropping weight type.
- Pulsating hydraulic type

Suitability of various compaction equipments :

Smooth wheel rollers → Crushed rock, hard core, mechanically stable gravel
→ Cohesive soils.

Vibrating type equipment / rubber tyred rollers / Crawler tractors → Cohesionless sands & gravels.

Sheepfoot rollers → Cohesive soils.

Pneumation tyred rollers < Cohesive soils / cohesionless soils.

Rammers → Confined places.

Rollers → cohesionless soils.

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Factors affecting Compaction:

i) Water content
ii) Amount and type of compaction
iii) Method of compaction
iv) Type of soil
v) Addition of admixtures.

i) Water Content

When Water content increased, compacted density goes on increasing, till a maximum dry density is achieved after which further addition of water decreases density.

Upto $\gamma_{d_{max}}$, $w\uparrow$, $\gamma_d\uparrow$
After $\gamma_{d_{max}}$, $w\uparrow$, $\gamma_d\downarrow$.

ii) Amount of Compaction:

It affects $\gamma_{d_{max}}$ & OMC.

The effect of increasing compactive energy results in increasing maximum dry density & decreasing OMC.

Compacting energy $\uparrow$; $\gamma_{d_{max}}\uparrow$, OMC $\downarrow$.

iii) Method of compaction

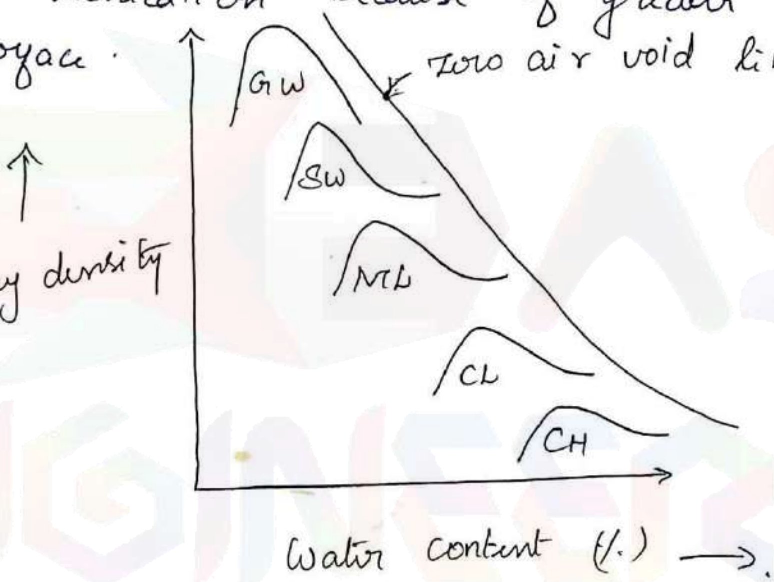
Density obtained depends on type of compaction or manner in which compactive effort is applied. variables in this aspect are

i) Weight of compacting equipment
ii) Manner of operation - Dynamic or impact, Static, kneading or rolling

iii) Time and area of contact b/w compacting element and soil.

iv) Type of soil :

$\gamma_{d_{max}}$ achieved depends on type of soil.
Well graded soils - higher density & lower optimum moisture content than
fine grained soil which require more water for lubrication because of greater specific surface.



v) Addition of Admixtures

Compaction properties of soil can be modified by number of admixtures other than soil material. Certain chemicals are added to soil cement to reduce cement consumption.

Clays & silts → Lime & Calcium chloride
→ Sodium Carbonate & Sulphate

Fly ash - Additive - Stabilisation of dune sand.
Fly ash - Filler - increasing density.

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Unit - II

SOIL WATER AND WATER FLOW

Soil water - Static Pressure in water - Effective stress concepts in soils - Capillary stress - Permeability measurement in the laboratory and field pumping in pumping out tests - Factors influencing permeability of soils - Seepage - Introduction to flow nets - Simple problems (Sheet Pile and weir).

SOIL WATER

Water present in voids of soil mass is called soil water.

Soil water

- Broad classification
  - Free water (or) Gravitational water.
  - Held water
    - Structural water
    - Adsorbed water
    - Capillary water
- Phenomenological basis
  - Ground water
  - Capillary water
  - Adsorbed water
  - Infiltered water.
- Structural aspect.
  - Pore water
  - Solvate water
  - Adsorbed water
  - Structural water

Ground water :

It is the subsurface water that fill the

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voids continuously and is subjected to no forces other than gravity.

It is also known as gravitational water or free water. Ground water fills up the voids in the soil upto ground water table and translocates through them.

Capillary water:

It is the water which is lifted up by surface tension above the free ground water surface. The water is in suspended. condition within the Interstices and pores of capillary size of soil.

The capillary water fills all the pores in the soil to a certain distance above water table - this distance being known as zone of capillary saturation.

Adsorbed water: < Hygroscopic water / Film water.

Hygroscopic water / Contact moisture / surface bound moisture is that water which the soil particles freely adsorb from atmosphere by physical forces of attraction and is held by forces of adhesion. It is affected neither by gravity nor by capillary forces.

Film moisture is attached to the surface of soil particles as a film on the layer of hygroscopic film.

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Infiltered water :

It is that portion of surface precipitation which soaks into ground, moving downwards through air-containing zones. subject to capillary forces.

Pore water :

Pore water, which is essentially free of strong soil attractive forces.

Pore water < Capillary water, Gravitational water

Solvate water :

It is the water which forms a hydration shell around soil grains.

Structural water

It is the water chemically combined in crystal structure of soil mineral. It refers to hydroxyl groups that constitute parts of crystal lattice.

CAPILLARY WATER.

Capillary water is the soil moisture located within interstices and voids of capillary size of soil.

Capillary action / capillarity is the phenomenon of movement of water in interstices of a soil due to capillary forces.

The minute pores of soil serve as capillary tubes through which moisture rises above ground water table.

The capillary forces depends on various factors such as surface tension of water, pressure in water related to atmospheric pressure, size & conformation of soil pores.

Surface tension: (T_s)

It is the property which exists in the surface film of water tending to contract the contained volume into a form having a minimum superficial area possible.

T_s @ 20°C = 72.8 dynes / 0.728×10^{-6} KN/cm.

For actone, benzene, petrol, etc.

T_s @ 20°C = 0.29×10^{-6} KN/cm.

T_s (water) > 2× T_s (other liquids).

T_s (Mercury) = 2.45×10^{-6} KN/cm.

The formation of curved meniscus around other material inserted in water is due to surface tension.

Capillary rise:

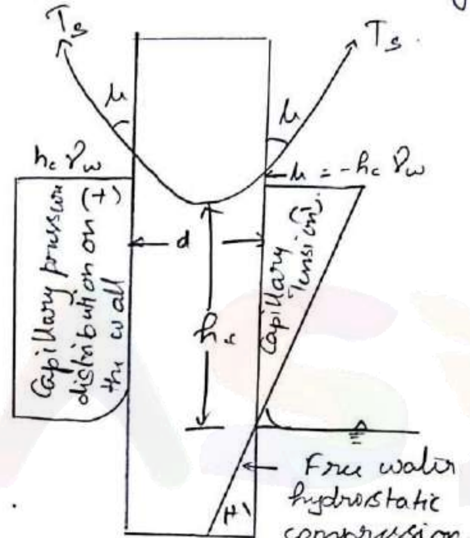
The rise of water in capillary tubes, or fine pores of soil is due to existence of surface tension which pulls the water up against gravitational force.

The height of capillary rise above ground water surface depends upon diameter of capillary tube & value of surface tension.

The formation of concave meniscus will take place only if the inner walls of the tube are initially wet.

If the walls are dry before insertion, a convex meniscus depressed below water surface is formed.

∵ The soil pores always carry adsorbed water, meniscus formation in soil will always be concave.



The vertical component $T_s \cos\alpha$ of the surface tension force depends upon angle of incidence α between meniscus and tube.

d - inner dia of tube

h_c - height of capillary rise.

When the capillary tube is inserted in water, the rise of water will take place. When equilibrium has reached, water will stop moving further.

At this equilibrium position, when height of rise is h_c.

Weight of column of water $= \frac{\pi d^2}{4} \times h_c \times \gamma_w$

Vertical component of reaction of meniscus against inside circumference of tube, supporting the above weight of column $= \pi d T_s \cos\alpha$.

$$\frac{\pi d^2}{4} h_c \gamma_w = \pi d T_s \cos\alpha.$$

$$\boxed{h_c = \frac{4 T_s \cos\alpha}{\gamma_w d}}$$

If tube is perfectly clean & wet, semi spherical meniscus will be formed. In this case. $\alpha = 0$, maximum capillary rise takes place.

$$\boxed{(h_c)_{max} = \frac{4T_s}{\gamma_w d}}$$

At 4°C, $(h_c)_{max} = \dfrac{0.3084}{d}$ cm.

At 20°C, $(h_c)_{max} = \dfrac{0.2975}{d}$ cm.

———— × ————

STRESS CONDITIONS IN SOIL :

$$\text{Total stress} = \frac{\text{Total load}}{\text{Area}}$$

It may be due to

i) Self-weight of soil

ii) Overburden of soil.

Total pressure

↓ Intergranular (or) Effective pressure.

↓ Neutral pressure (or) Pore pressure.

Effective pressure (σ') is the pressure transmitted from particle through their point of contact through soil mass above plane. It is effective in decreasing the void ratio of soil mass and in mobilising shear strength.

Neutral pressure or pore pressure (u) is the pressure transmitted through pore fluid.

This pressure, equal to water load per unit (4)
area above plane, doesnot have measurable influence on void ratio or Shearing resistance.

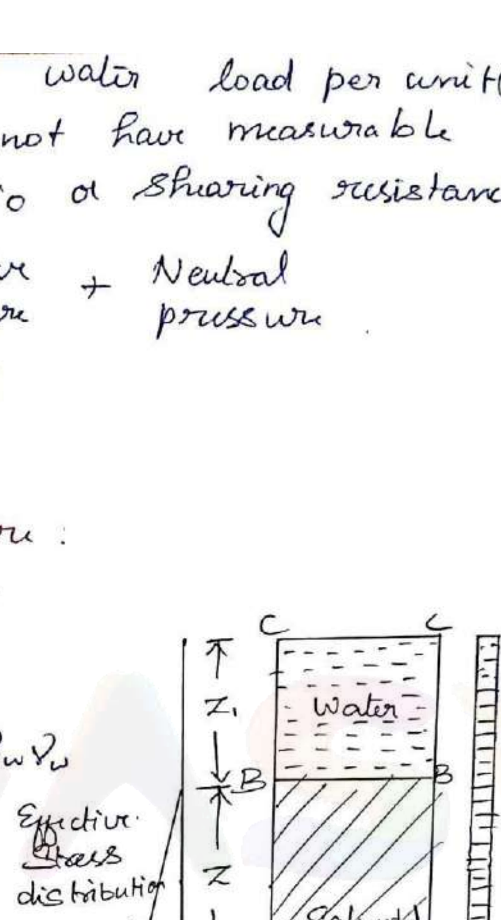
Total pressure = Effective pressure + Neutral pressure.

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

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

To find effective pressure:

i) Submerged soil mass:



$\sigma = Z\gamma_{sat} + Z_1\gamma_w$

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

$\sigma' = Z\gamma_{sat} + Z_1\gamma_w - h_w\gamma_w$

$h_w = Z + Z_1$

$\sigma' = Z\gamma_{sat} + Z_1\gamma_w - Z\gamma_w - Z_1\gamma_w$

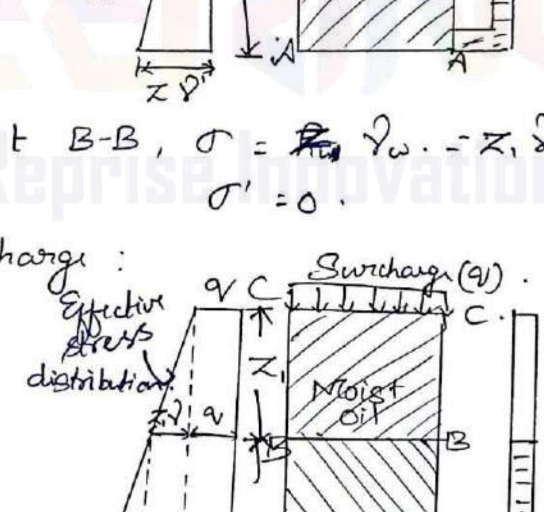
$= Z(\gamma_{sat} - \gamma_w)$.

$$\boxed{\sigma' = Z\gamma'}$$

At B-B, $\sigma = \cancel{h_w}\gamma_w . - Z_1\gamma_w$

$\sigma' = 0$.

ii) Soil mass with surcharge:



At level AA.

$\sigma = q + Z_1\gamma + Z\gamma_{sat}$.

$u = h_w\gamma_w = Z\gamma_w$.

$\sigma' = \sigma - u$

$= q + Z_1\gamma + Z\gamma_{sat} - Z\gamma_w$

$= q + Z_1\gamma + Z(\gamma_{sat} - \gamma_w)$

$$\boxed{\sigma' = q + Z_1\gamma + Z\gamma'}$$

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At plane BB,

$$\sigma = q + z_1 \gamma.$$

$$u = h_w \gamma_w = 0.$$

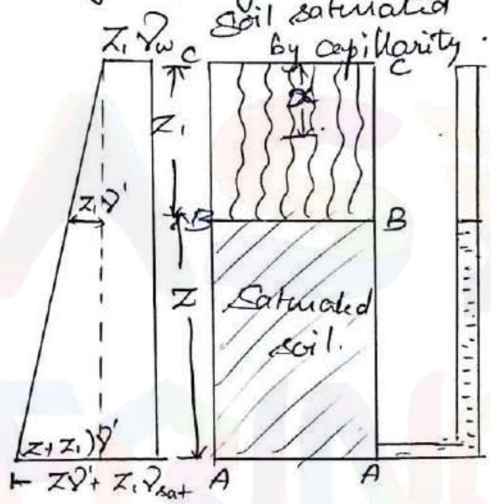
$$\sigma' = \sigma - u = q + z_1 \gamma$$

$$\boxed{\sigma = q + z_1 \gamma}$$

At plane CC,

$$\boxed{\sigma = q}$$

iii) Saturated soil with capillary fringe:



At level A-A,

$$\sigma' = (Z + Z_1)\gamma' + Z_1 \gamma_w$$

$$= Z\gamma' + Z_1 \gamma' + Z_1 \gamma_w$$

$$= Z\gamma' + Z_1(\gamma' + \gamma_w)$$

$$\boxed{\sigma' = Z\gamma' + Z_1 \gamma_{sat}}.$$

At level BB,

$$\sigma' = Z_1 \gamma' + Z_1 \gamma_w$$

$$\boxed{\sigma' = Z_1 \gamma_{sat}}.$$

At CC,

σ' = Capillary pressure = $Z_1 \gamma_w$

The effect of capillarity of height Z_1, is analogous to surcharge $\boxed{q = Z_1 \gamma_w}$

At any depth x below level CC,

$$\boxed{\sigma' = x\gamma' + Z_1 \gamma_w}$$

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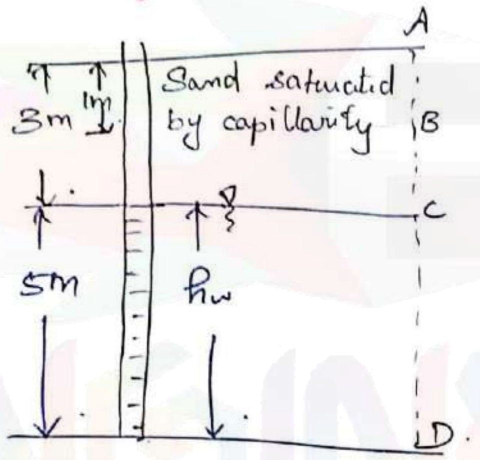
Problems: (5)

The water table in a deposit of sand 8m thick, is at a depth of 3m below the surface. Above water table, the sand is saturated with capillary water. The bulk density of sand is 19.62 KN/m³. Calculate the effective pressure at 1m, 3m and 8m below surface. Hence plot the variation of total pressure, neutral pressure and effective pressure over the depth of 8m.



$\gamma_{sat} = 19.62 \text{ KN/m}^3$

a) Stresses at D, 8 m below ground.

$\sigma = (3+5)\,\gamma_{sat} = 8\,\gamma_{sat}$
$= 8 \times 19.62$
$= 156.96 \text{ KN/m}^2$.

$u = h_w \gamma_w = 5 \times 9.81 = 49.05 \text{ KN/m}^2$

$\sigma' = \sigma - u = 156.96 - 49.05 = 107.91 \text{ KN/m}^2$

$\boxed{\sigma' = 107.91 \text{ KN/m}^2}$

b) Stresses at C, 3m below ground

$\sigma = 3\,\gamma_{sat} = 3 \times 19.62 = 58.86 \text{ KN/m}^2$.

$u = 0$

$\sigma' = \sigma - u = 58.86 \text{ KN/m}^2$.

$\boxed{\sigma' = 58.86 \text{ KN/m}^2}$

c) Stresses at A, @ ground level

$\sigma = 0.$

$u = -h_c \gamma_w$

$= -3 \times 9.81 = -29.43\ kN/m^2.$

$\sigma' = \sigma - u = 29.43\ kN/m^2$

$\boxed{\sigma' = 29.43\ kN/m^2}$

Stresses at B, 1m below ground level

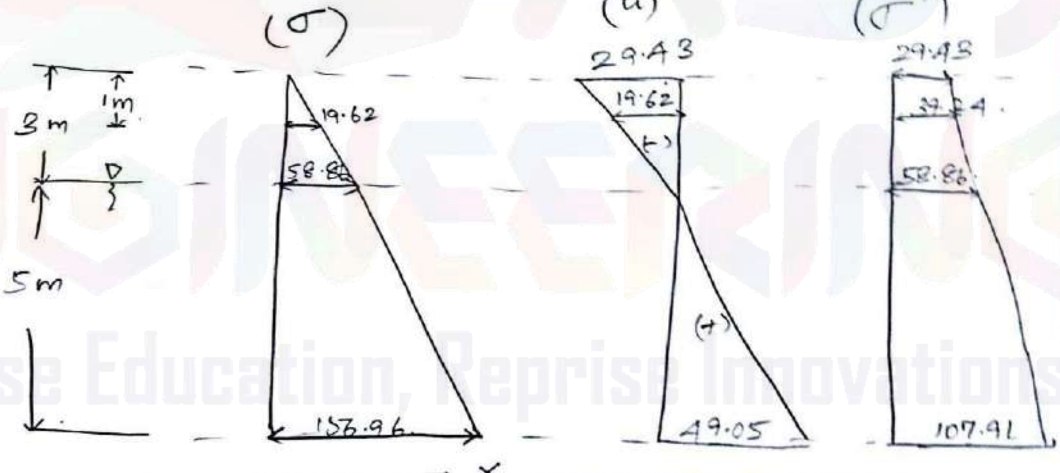
$\sigma = 1 \times \gamma_{sat} = 19.62\ kN/m^2.$

$u = -2\gamma_w = -2 \times 9.81$

$= -19.62\ kN/m^2.$

$\sigma' = \sigma - u = 19.62 + 19.62$

$\boxed{\sigma' = 39.24\ kN/m^2}$



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PERMEABILITY.

It is defined as the property of a porous material which permits the passage or seepage of water through its interconnecting voids.

A material having continuous voids is called permeable.

Gravels - Highly permeable

Stiff clay - Least permeable.

Clay - Impermeable

The flow of water through soils may be either laminar flow & turbulent flow.

Darcy's Law:

The rate of flow or discharge per unit time is proportional to hydraulic gradient.

$$\boxed{q = kiA}$$

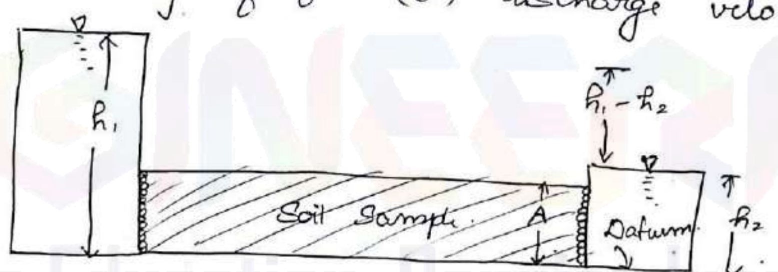
q - Discharge per unit time.

A - Total c/s area of soil mass ⊥ direction of flow

i - hydraulic gradient

k - Darcy's coefficient of permeability

v - Velocity of flow (or) discharge velocity.



$$v = \frac{q}{A} = ki$$

Hydraulic gradient $i = \frac{h_1 - h_2}{L}$.

$$\boxed{q = k \frac{h_1 - h_2}{L} A}$$

If $i = 1$, $k = v$

Coefficient of permeability is defined as average velocity of flow that will occur through total c/s area of soil under unit hydraulic gradient.

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Discharge velocity and seepage velocity

The velocity of flow v is the rate of discharge of water per unit of total cross-sectional are A of soil. This total cross sectional area is composed of area of solids A_s and area of voids A_v.

Since the flow takes place through voids, the actual or true velocity of flow will be more than discharge velocity. This actual velocity is called seepage velocity (V_s) and is defined as the rate of discharge of percolating water per unit cross-sectional area of voids ⊥ to direction of flow.

$$q = vA = V_s A_v$$

$$V_s = v\frac{A}{A_v} \qquad \because \frac{A_v}{A} = \frac{V_v}{V} = n.$$

$$V_s = v \cdot 1/n.$$

$$\boxed{V_s = \frac{v}{n}} = \frac{1+e}{e}v \quad \Rightarrow \frac{V_s}{v} = 1/n.$$

Seepage velocity $V_s \propto$ hydraulic gradient i,

$V_s = k_p i$ (K_p - Coefficient of percolation).

From Darcy's law, $v = ki$

$$\frac{V_s}{v} = \frac{k_p}{k} \qquad \boxed{k_p = \frac{k}{n}}$$

$$\frac{k_p}{k} = 1/n.$$

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Factors affecting Permeability (1)

$q = KiA$.

$K = D_s^2 \frac{\gamma_w}{\eta} \frac{e^3}{1+e} C$

Factors affecting permeability are

* Grain size
* Properties of pore fluid.
* Void ratio of soil
* Structural arrangement of soil particles.
* Entrapped air & foreign matter.
* Adsorbed water in clayey soils.

1. Effect of size and shape of particles:

Permeability varies as square of grain size.

$K = CD_{10}^2$

K - Coefficient of permeability (cm/sec)

D_{10} - Effective diameter (cm).

C - Constant = 100.

2. Effect of properties of pore fluid:

Permeability $\propto$ Unit weight of water

$\propto 1/\text{Viscosity}$.

Though the unit weight of water does not change much with change in temperature, there is great variation in viscosity with temperature

$\frac{K_1}{K_2} = \frac{\eta_2}{\eta_1}$ $\quad\quad K_{27} = K\frac{\eta}{\eta_{27}}$

Standard temperature 27°C.

However, change in unit weight of water due to temperature is also taken into account

$$\frac{K_1}{K_2} = \frac{\eta_2}{\eta_1} \frac{\gamma_{w_1}}{\gamma_{w_2}}$$

K_{27} - Permeability @ 27°C.

η_{27} - Viscosity @ 27°C

K - Permeability @ test temperature.

3. Effect of voids ratio:

$$\frac{K_1}{K_2} = \left[\frac{C_1 e_1^3}{1+e_1}\right] \Big/ \left[\frac{C_2 e_2^3}{1+e_2}\right]$$

Laboratory experiments have shown that factor C changes very little with change in voids ratio of un-stratified soil samples.

For coarse grained soils,

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1} \div \frac{e_2^3}{1+e_2}$$

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1} \frac{1+e_2}{e_2^3}$$

Based on another concept of mean hydraulic radius for soils, the following relationship is obtained.

$$\frac{K_1}{K_2} = \frac{e_1^2}{e_2^2}$$

Semi-logarithmic plot of void ratio Vs permeability is approximately a straight line for both coarse grained & fine grained soils.

4. Effect of structural arrangement of particles and stratification

The effect of structural disturbance on

permeability is much pronounced in fine grained soils.

Stratified soil mass have marked variations in their permeabilities in direction parallel & perpendicular to stratifaction, permeability parallel to stratification being always greater.

5. Effect of degree of saturation

Air in voids, $S_r \downarrow$, Permeability reduces.

The dissolved air in water may get liberated, thus changing permeability.

Organic foreign matter have the tendency to move towards critical flow channels & choke them up, thus decreasing permeability.

Determination of Coefficient of permeability:

a) Laboratory methods
   i) Constant head permeability test
   ii) Falling head permeability test.

b) Field methods.
   i) Pumping-out tests
   ii) Pumping-in tests.

c) Indirect methods.
   i) Computation from grain size or specific surface.
   ii) Horizontal capillary test
   iii) Consolidation test data.

Permeability can be determined in lab by direct measurement with help of permeameters, by allowing water to flow through soil sample either under constant head or variable head.

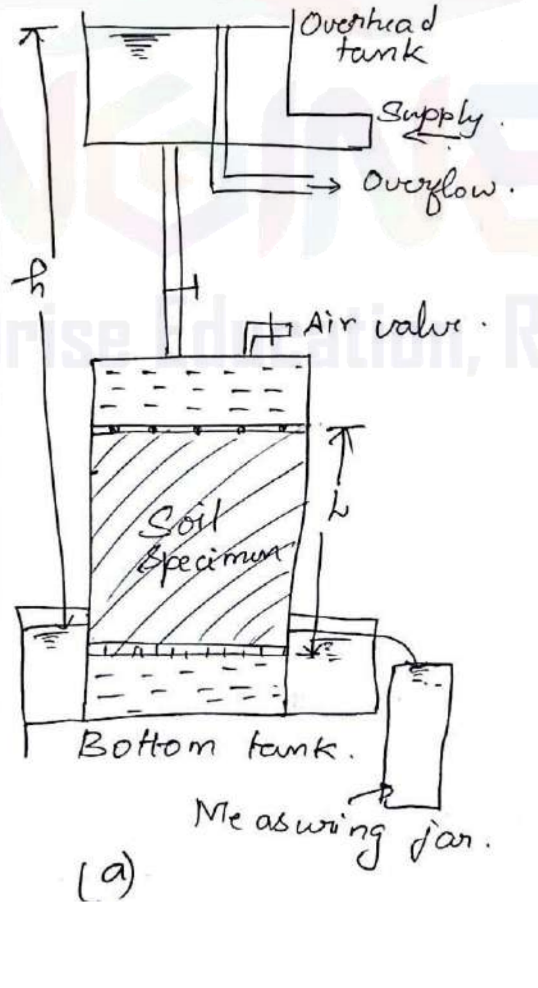
LABORATORY Tests

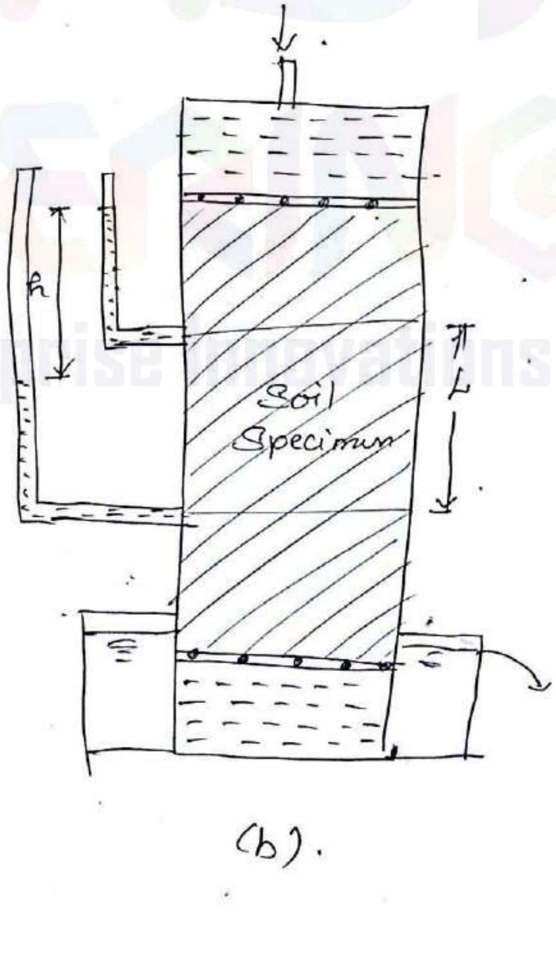
i) Constant head Permeability test.

Overhead tank → Inlet tube
→ Overflow tube
→ Outlet tube.

The constant hydraulic gradient i causing the flow is the head h (diff. b/w water levels of overhead & bottom tanks). divided by length of sample.

If length of sample is large, the head lost over length of specimen is measured by inserting piezometric tubes.





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

$$i = h/L$$

$$K = \frac{Q}{tiA}$$

$$\boxed{K = \frac{QL}{thA}}$$

ii) Falling Head Permeability tests

Constant Head - Coarse grained soil, discharge easy.
Falling head - less permeable soils, discharge small.

A standard pipe of known c/s area is fitted over permeameter & water is allowed to run down. Water level in stand pipe constantly falls as water flows.

Observations are started after steady state of flow has reached.

The head at any time is difference in water level in stand pipe & bottom tank

h_1, h_2 - heads at time intervals t_1 & t_2.

h - head @ any intermediate time interval

$-dh$ - change in head in smaller time dt.

$$q = \frac{-dh\,a}{dt} = KiA.$$

$i = h/L$

$$K\frac{h}{L}A = \frac{-dh}{dt}a$$

$$\frac{AK}{La}\,dt = -\frac{dh}{h}$$

$$\frac{AK}{La}\int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2}\frac{dh}{h} = \int_{h_2}^{h_1}\frac{dh}{h}$$

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

$$\boxed{k = \frac{aL}{At}\log_e \frac{h_1}{h_2}}$$

$$k = 2.3\,\frac{aL}{At}\log_{10}\frac{h_1}{h_2}$$

Problem :

In a falling head permeameter test, initial head ($t=0$) is 40 cm. The head drops by 5 cm in 10 minutes. Calculate the time required to run the test for the final head to be at 20 cm. If the sample is 6 cm in height & 50 cm² in C/S area. Calculate the coefficient of permeability, taking area of stand pipe = 0.5 cm².

$$k = 2.3\,\frac{aL}{At}\log_{10}\frac{h_1}{h_2}.$$

$a = 0.5\ cm^2$ $\quad A = 50\ cm^2$

$h_1 = 40\ cm$ @ $t = 10$ min

$h_2 = 35\ cm$

Final head 20 cm @ $t = ?$

$$t = 2.3\,\frac{aL}{Ak}\log_{10}\frac{h_1}{h_2}$$

$$m = 2.3\,\frac{aL}{Ak}.$$

$$t = m\log_{10}\frac{h_1}{h_2}.$$

$$10 = m\log_{10}\frac{40}{35}.$$

$$\boxed{t = 172.5\ \log_{10}\frac{h_1}{h_2}}$$

$m = 172.5$ units

At time t, $h_2 = 20$ cm.

$$t = 172.5 \log_{10} \frac{40}{20}$$

$$\boxed{t = 51.9 \text{ min}}$$

$$m = \frac{2.3aL}{Ak} = 172.5 \text{ units.}$$

$$k = 2.3 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$

$$= \frac{2.3 \times 0.5 \times 6}{50 \times 172.5 \times 60} \text{ cm/sec}$$

$$\boxed{k = 1.335 \times 10^{-5} \text{ cm/sec}}$$

Permeability of stratified soil deposits.

In nature, soil mass may consists of several layers deposited one above other. Their bedding planes may be horizontal, inclined or vertical.

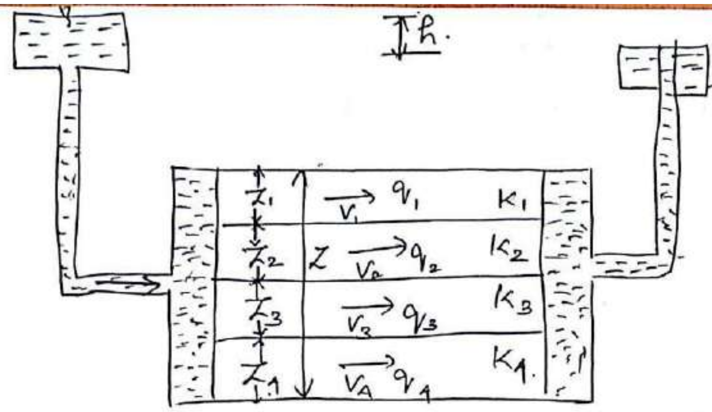
Each layer, assumed to be homogeneous, isotropic, has its own value of coefficient of permeability.

Average permeability depends on direction of flow with relation to direction of bedding planes.

i) Parallel to bedding planes
ii) Perpendicular to bedding plane.

i) Average permeability parallel to bedding plane

$Z_1, Z_2, Z_3, \ldots Z_n$ - Thickness of layers.
$K_1, K_2, K_3, \ldots K_n$ - Permeability of layers.



K_x - average permeability of soil deposit parallel to bedding plane

$q = q_1 + q_2 + \dots q_n$.

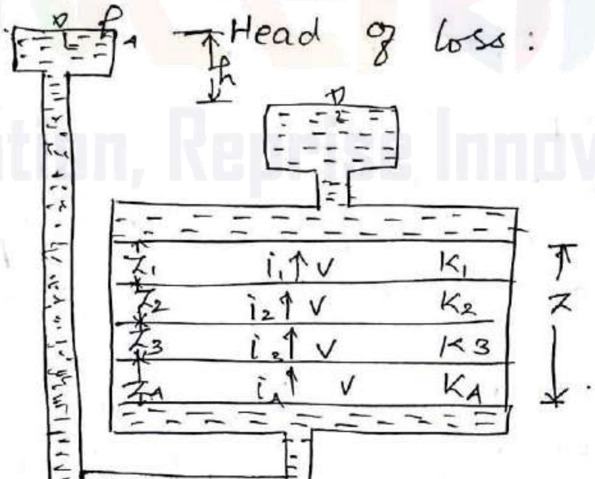
$q = K_x i Z$

$= K_1 i Z_1 + K_2 i Z_2 + K_3 i Z_3 + \dots K_n i Z_n$.

$$\boxed{K_x = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3 + \dots K_n Z_n}{Z}}$$

ii) Average permeability perpendicular to bedding plane :

$h_1, h_2, h_3, \dots h_4$ - Head of loss :



$i = h/L$

$h = h_1 + h_2 + \dots h_n$.

$\therefore h_n = i Z_n$

$h = i_1 Z_1 + i_2 Z_2 + \dots i_n Z_n$

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K_z - average permeability perpendicular to bedding plane.

$$v = k_z i = k_z \frac{h}{z} \qquad h = \frac{vz}{K_z}$$

$$i = \frac{v_1}{K_1}, \quad i_2 = \frac{v}{K_2} \quad \dots$$

$$\frac{vz}{K_z} = \frac{vz_1}{K_1} + \frac{vz_2}{K_2} \quad \dots\dots$$

$$\boxed{K_z = \frac{z}{\frac{z_1}{K_1} + \frac{z_2}{K_2} \dots + \frac{z_3}{K_3}}}$$

$$k_x > k_z$$

$K_1 = 2, \; k_2 = 1, \; k_3 = 4.$

Field Tests :

Pumping Out tests :

It gives value of permeability with minimum disturbance. The value of coefficient so obtained is an overall average for large area.

In this test, drawdowns, corresponding to a steady discharge q, are observed at number of wells.

Pumping must continue @ uniform rate for a sufficient time to approach steady state condition.

Steady state condition is one in which drawdown changes negligibly with time.

a) In unconfined aquifer :

$$K = \frac{q}{\pi(H^2 - h^2)} \log_e \frac{R}{r}.$$

$$K = \frac{q}{1.36(H^2 - h^2)} \log_{10} \frac{R}{r}$$

$$\boxed{K = \frac{q}{1.36\ h_2^2 - h_1^2)} \log_{10} \frac{r_2}{r_1}}$$

b) In confined aquifer

$$K = \frac{q}{2\pi b(H-h)} \log_e \frac{R}{r}.$$

$$\boxed{K = \frac{q}{2.72 b(h_2 - h_1} \log_{10} \frac{r_2}{r_1}}$$

Coefficient of transmissibility (T) :

S_1 - drawdown in observation well $1 = (H - h_1)$

S_2 - drawdown in observation well $2 = (H - h_2)$

$h_2 - h_1 = (H - S_2) - (H - S_1)$

$$\boxed{h_2 - h_1 = S_1 - S_2.}$$

$$T = \frac{q}{2.72(S_1 - S_2)}$$

$$\boxed{T = \frac{q}{2.72 \Delta S}}$$

Pumping in tests :

US Bureau of Reclamation.

Pumping in Open-end tests

Packer tests.

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a) Open end tests : (12)

* An open end pipe is sunk in strata and the soil is taken out of the pipe just to the bottom.

* Clean water, having temperature higher than ground water, is added through metering system to maintain gravity flow under constant head.

* Permeability calculated from electrical analog experiments.

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

h - differential head of water.
r - radius of casing
q - constant rate of flow.

b) Packer tests :

* An uncased portion of drill hole or a perforated portion of casing is used for performing test.

* A top packer is placed inside or below casing. Water is pumped in lower portion of hole.

* To perform test after completion of hole, which can stand without casing, two packers are set on pipe keeping perforated portion of pipe b/w plugs.

L - length of portion of hole tested.

$$K = \frac{q}{2\pi L h} \log_{10} \frac{L}{r} \; ; \; L \geq 10r.$$

$$K = \frac{q}{2\pi L h} \sin^{-1} \frac{L}{2r} \; ; \; 10r > L \geq r$$

SEEPAGE ANALYSIS

Head, Gradient and Potential

When water flows through saturated soil mass, total head at any point in soil mass consists of

i) Piezometric head or pressure head
ii) Velocity head.
iii) Position head.

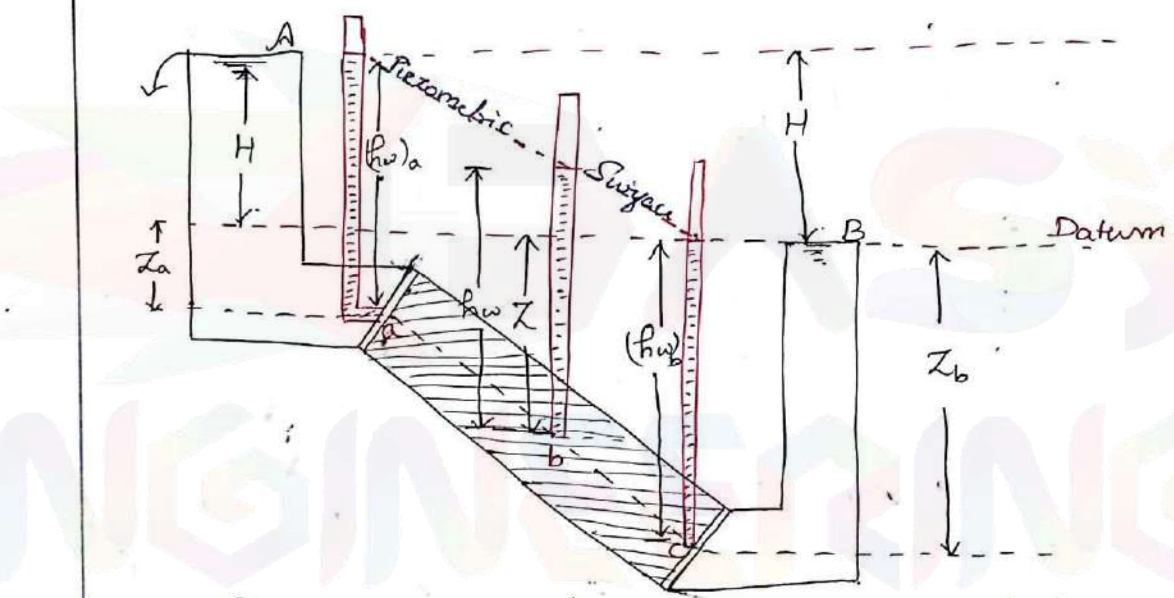

Figure represents flow of water through saturated soil sample of length L, due to difference in elevation of free water surface at A and B.

$(h_w)_a$ - Piezometric head @ upper point a.
$(h_w)_b$ - Piezometric head @ lower point b.
h_w - Piezometric head at any intermediate point.

Piezometric surface is the line joining water levels in the piezometers.

Position (or) elevation head at any point is the elevation of that point with respect to any arbitrary datum. (13)

The position head z is taken positive if it is situated above the datum and negative if below datum.

- Z_a - Position head @ a
- Z_b - Position head @ b.

The total head at any point may be regarded as potential energy per unit weight of water measured w.r. to datum.

The velocity head $v^2/2g$ is negligibly small for flow of water through soil & is usually neglected.

Total head = Piezometric head + Position head.

Potential at b $= (h_w)_b - Z_b = 0$.

Potential at a $= (h_w)_a - Z_a = H$ - Initial hydraulic head.

Hydraulic head - hydraulic potential.

Hydraulic potential @ any point,

$$\boxed{h = h_w \pm Z}$$

When water level is the datum,

Total head & hydraulic potential are equal.

The loss of head or dissipation of hydraulic head per unit distance of flow through soil is called hydraulic gradient.

$$\boxed{i = h/L}$$

Seepage Pressure

By virtue of viscous friction exerted on water flowing through soil pores, an energy transfer is effected b/w water & soil. The force corresponding to this energy transfer is called seepage force or seepage pressure.

Seepage pressure is the pressure exerted by water on soil through which it percolates. It is the seepage pressure that is responsible for phenomenon known as quick sand and it is important in stability analysis of earth structures subjected to seepage.

h - hydraulic head

z - thickness

p_s - Seepage pressure. $\quad i = h/z$.

$$p_s = h\gamma_w$$

$$= \frac{h}{z} \cdot z\gamma_w$$

$$p_s = iz\gamma_w$$

Seepage force (J) transmitted to soil mass of total cross-sectional area A is

$$J = p_s A = iz\gamma_w A.$$

Seepage force per unit area $j = \frac{iz\gamma_w A}{zA}$

$$= i\gamma_w.$$

Seepage pressure always acts in direction of flow.

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(1A)

Effective pressure in soil mass subjected to seepage pressure is given by

$$\sigma' = z\gamma' \pm p_s = z\gamma' \pm iz\gamma_w.$$

Effective pressure - Increase = + sign.
- decrease = (-) sign.

Upward Flow : Quick Condition

When flow takes place in an upward direction, seepage pressure also acts in the upward direction & effective pressure is reduced.

If Seepage pressure = Pressure due to submerged weight of soil, Effective pressure reduced to zero.

In this case, cohesionless soil loses all its shear strength, & soil particles have the tendency to move up in direction of flow.

This phenomenon of lifting of soil particles is called quick condition, boiling condition or quick sand.

During quick condition,

$$\sigma' = z\gamma' - p_s = 0.$$

$$p_s = z\gamma' \qquad iz\gamma_w = z\gamma'$$

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

Hydraulic gradient at such a critical state is critical hydraulic gradient.

$\boxed{i_c \text{ - Critical gradient}}$

Problem:

A coarse-grained soil has void ratio of 0.78 and specific gravity as 2.67. Calculate the critical gradient at which quick sand condition will occur.

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.78}$$

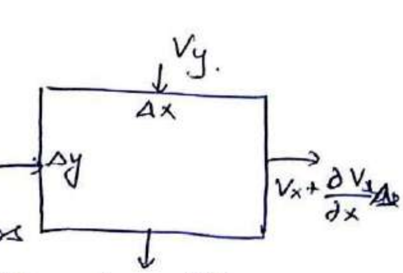
$$\boxed{i_c = 0.94}$$

TWO DIMENSIONAL FLOW: LAPLACE EQUATION.

The quantity of water flowing through a saturated soil mass, as well as the distribution of water pressure can be estimated by theory of flow of fluids through porous medium.

Assumptions:

1. The saturated porous medium is incompressible.
2. Darcy's law for flow through porous medium is valid.
3. The hydraulic boundry conditions @ entry and exit are known.
4. Water is incompressible.
5. There is no change in degree of saturation in zone of soil through which water seeps and the quantity of water flowing into any element of volume is equal to quantity which flows out in same length of time.



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Consider an element of soil of size $\Delta x . \Delta y$ (19)
and of unit thickness $\perp$ to plane of paper.

V_x, V_y - Entry velocity components in x & y

$\left[V_x + \frac{\partial V_x}{\partial x}\Delta x\right], \left[V_y + \frac{\partial V_y}{\partial y}\Delta y\right]$ - Velocity components @ exit.

Quantity of Water entering element = Quantity of Water leaving element

$$V_x(\Delta y \cdot 1) + V_y(\Delta x \cdot 1) = \left[V_x + \frac{\partial V_x}{\partial x}\Delta x\right] \cdot (\Delta y \cdot 1) + \left[V_y + \frac{\partial V_y}{\partial y}\Delta y\right] \cdot (\Delta x \cdot 1).$$

$$\cancel{V_x \Delta y} + \cancel{V_y \Delta x} = \cancel{V_x \Delta y} + \frac{\partial V_x}{\partial x}\Delta x \Delta y + \cancel{V_y \Delta x} + \frac{\partial V_y}{\partial y}\Delta y \Delta x.$$

$$\frac{\partial V_x}{\partial x}\Delta x \Delta y + \frac{\partial V_y}{\partial y}\Delta x \Delta y = 0.$$

$$\boxed{\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} = 0}$$ ← Continuity Equation.

$$V_x = K_x i_x = K_x \cdot \frac{\partial h}{\partial x}$$

$$V_y = K_y i_y = K_y \frac{\partial h}{\partial y}$$

h - hydraulic head under which water flows

K_x & K_y - Coefficients of permeability in x & y directions.

Substitute V_x & V_y,

$$\frac{\partial^2 (K_x h)}{\partial x^2} + \frac{\partial^2 (K_y h)}{\partial y^2} = 0.$$

For isotropic soil, $K_x = K_y = K$.

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

φ - velocity potential $= kh$.

$$\boxed{\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial y^2} = 0} \leftarrow \text{Laplace equations.}$$

Velocity Potential (φ)

It is defined as scalar function of space and time such that its derivative w.r. to any direction gives fluid velocity in that direction.

$$\varphi = kh.$$

$$\frac{\partial \varphi}{\partial x} = k\frac{\partial h}{\partial x} = k i_x = V_x.$$

$$\frac{\partial \varphi}{\partial y} = k\frac{\partial h}{\partial y} = k i_y = V_y.$$

The solution of Laplace equation can be obtained by

i) Analytical method

ii) Graphical method.

iii) Experimental methods.

The solution gives two sets of curves known as 'equipotential lines' and 'stream lines', mutually orthogonal to each other.

Equipotential lines - Contours of equal head.

Direction of seepage $\perp$ equipotential lines.

Path along which water seep through soil - Flow lines/

Stream Function

(16)

Laplace equation is satisfied by conjugate harmonic function φ and ψ & that the curve $\varphi(x, y) = \text{constant}$ are the orthogonal trajectories of the curve $\psi(x, y) = \text{constant}$.

It is defined as a scalar function of space and time such that a partial derivative of this function w.r. any direction gives the velocity component in a direction $+90°$ to original direction.

$$\frac{\partial \psi}{\partial y} = V_x \qquad \frac{\partial \psi}{\partial x} = -V_y.$$

$$\frac{\partial \varphi}{\partial x} = \frac{\partial \psi}{\partial y} \quad \& \quad \frac{\partial \varphi}{\partial y} = -\frac{\partial \psi}{\partial x}$$

Continuity equation:

$$\frac{\partial}{\partial x}\left[\frac{\partial \psi}{\partial y}\right] + \frac{\partial}{\partial y}\left[-\frac{\partial \psi}{\partial x}\right] = 0.$$

$$\frac{\partial^2 \psi}{\partial x \partial y} - \frac{\partial^2 \psi}{\partial x \partial y} = 0.$$

$\therefore$ Stream function satisfies continuity equation.

Laplace equation

$$\frac{\partial}{\partial x}\left[\frac{\partial \varphi}{\partial x}\right] + \frac{\partial}{\partial y}\left[\frac{\partial \varphi}{\partial y}\right] = 0.$$

$$\frac{\partial}{\partial x}\left[\frac{\partial \varphi}{\partial y}\right] + \frac{\partial}{\partial y}\left[-\frac{\partial \varphi}{\partial x}\right] = 0.$$

$$\frac{\partial^2 \psi}{\partial x \, \partial y} - \frac{\partial^2 \psi}{\partial x \, \partial y} = 0.$$

$\therefore$ Stream function also satisfies Laplace equation.

Complex potential (w)

A combination of function ϕ & ψ is called complex potential.

$$w = \phi + i\psi.$$

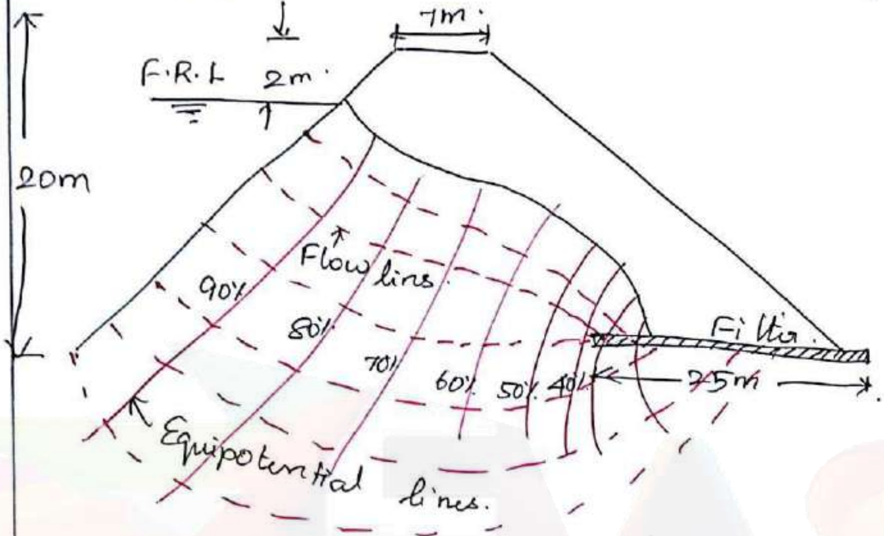
FLOW NET :

The hydraulic boundary conditions have a great effect on general shape of flow net.

Properties of flow net :

* Flow lines and equipotential lines meet at right angles to each other.
* Fields are approximately squares, so that circle can be drawn touching all four sides of square.
* The quantity of water flowing through each flow channel is the same. Similarly, the same potential drop occurs b/w two successive equipotential lines.
* Smaller the dimensions of field, greater will be hydraulic gradient & velocity of flow through it.

* In a homogeneous soil, every transition in the shape of curves is smooth, being either elliptical or parabolic in shape. (17)



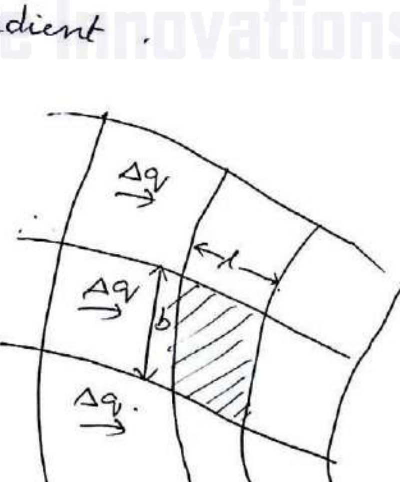
APPLICATIONS OF FLOW NET

Flow net can be utilised for following purposes.

i) Determination of seepage
ii) Determination of hydrostatic pressure.
iii) Determination of seepage pressure.
iv) Determination of exit gradient.

1) Determination of seepage :

b - width of field.
l - length of field.
Δh - head drop through field.
Δq - Discharge passing through flow channel.
H - Total hydraulic head causing flow
H = difference b/w upstream & downstream heads.



Portion of flow net

$$\Delta q = k \cdot \frac{\Delta h}{l} (b \times 1) \quad \text{(unit thickness)}$$

N_d - Total No. of potential drops in complete flow net.

$$\Delta h = \frac{H}{N_d}.$$

$$\Delta q = k \cdot \frac{H}{N_d} \left(\frac{b}{l}\right)$$

The total discharge through complete flow net

$$q = \Sigma \Delta q = k \frac{H}{N_d} \left(\frac{b}{l}\right) . N_f .$$

$$\boxed{q = kH \frac{N_f}{N_d} \left[\frac{b}{l}\right]} \leftarrow \text{Rectangle.}$$

$$\boxed{q = kH \frac{N_f}{N_d}} \leftarrow \text{Square.}$$

ii) Determination of hydrostatic pressure.

$$u = h_w \gamma_w .$$

u = hydrostatic pressure

h_w - piezometric head.

$$h_w = h - z.$$

h - hydraulic potential at point under consideration.

z - Position head of point above datum

iii) Determination of Seepage Pressure:

The hydraulic potential h at any point located after n potential drops, each of value Δh is given by

$$h = H - n\Delta h .$$

The seepage pressure at any point equals the hydraulic potential or balance hydraulic head multiplied by unit weight of water. (19)

$$p_s = h\gamma_w$$
$$= (H - n\Delta h)\gamma_w.$$

d) Determination of exit gradient:

Exit gradient is the hydraulic gradient at the downstream end of flow line where the percolating water leaves the soil mass and emerges into free water at the downstream.

$$i_e = \frac{\Delta h}{l}.$$

i_e - Exit gradient.

Problems:

For a homogeneous earth dam 52m high & 2m free board, a flow net was constructed, & following results were obtained.

Number of equipotential drops = 25

Number of flow channels = 4.

The dam has a horizontal filter of 40m length at its downstream end. Calculate the discharge per unit length of dam if the coefficient of permeability of dam material is 3×10^{-3} cm/sec.

$$q = K H \frac{N_f}{N_d} \qquad H = 52 - 2 = 50\,m.$$

$k = 3 \times 10^{-3}$ cm/sec $= 3 \times 10^{-5}$ m/sec.

$N_f = 4 \quad N_d = 25.$

$$q = 3 \times 10^{-5} \times 50 \times \frac{4}{25}$$

$$= 24 \times 10^{-5} \; m^3/sec/m.$$

$= 0.00024\; m^3$/metre length.

--- x ---

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UNIT - III

STRESS DISTRIBUTION AND SETTLEMENT

Stress Distribution - Soil media - Boussinesq theory - Use of Newmarks influence chart - Components of settlement - immediate and consolidation settlement - Terzaghi's one dimensional consolidation theory - Computation of rate of settlement - $\sqrt{t}$ and log t methods - e-log p relationship - Factors influencing compression behaviour of soils.

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STRESS DISTRIBUTION :

Consider stresses within a soil mass due to its own weight. Stresses due to self weight are sometimes known as geostatic stresses.

Let us take soil mass to be bounded by horizontal plane (ground surface) xy and the z-axis be directed downwards. Under this condition, soil mass is said to be semi-infinite.

When there is no external loading, the ground plane becomes a principal plane since it is devoid of any shear loading.

Within soil mass,

$$\tau_{xy} = \tau_{xz} = \tau_{yz} = 0.$$

$$\boxed{\sigma_z = \gamma z} \leftarrow \text{From equilibrium equation}$$

γ - Unit Weight of soil

σ_z - Vertical stress at a point within and soil mass, situated at a depth z below ground surface.

From compatibility equations,

$$\sigma_x = \sigma_y = \frac{\mu}{1-\mu}\gamma z.$$

$$\boxed{\sigma_x = \sigma_y = K_0 \gamma z} \qquad K_0 = \frac{\mu}{1-\mu}$$

μ - Poisson ratio

K_0 - Coefficient of lateral pressure @ rest.

At certain point within soil mass, the stresses are caused due to both surface loadings as well as due to self weight of soil above it.

Total stress = Stress due to Self weight + Stress due to surface loads.

Stress tensor:

$$\begin{bmatrix} \sigma_x & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z \end{bmatrix}$$

σ - Normal stress

τ - Shear stress.

Strain Tensor:

$$\begin{bmatrix} \varepsilon_x & \gamma_{yx}/2 & \gamma_{zx}/2 \\ \gamma_{xy}/2 & \varepsilon_y & \gamma_{zy}/2 \\ \gamma_{xz}/2 & \gamma_{yz}/2 & \varepsilon_z \end{bmatrix}$$

ε - direct strain

γ - Shearing strain

(2)

BOUSSINESQ EQUATIONS.

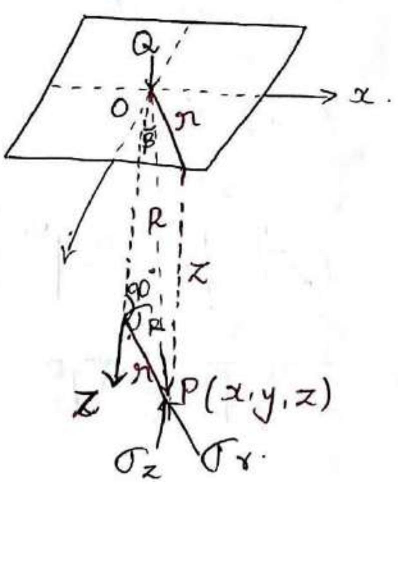
CONCENTRATED FORCE:

Assumptions:

* Soil mass is an elastic medium, for which modulus of elasticity E is constant.
* Soil mass is homogeneous, it has identical properties at every point in it in identical directions.
* Soil mass is isotropic, it has identical elastic properties in all directions through any point of it.
* Soil mass is semi-infinite, it extends infinitely in all directions below level surface.

Let a point load Q act at the ground surface, at a point O which may be taken as origin of x, y & z axis.

Let us find stress components at a point P in soil mass, having coordinates x, y, z, having radial distance r and vertical distance z from point O.



σ_R - Polar Radial Stress.

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Using logarithmic stress function, Boussineq showed that polar radial stress

$$\sigma_R = \frac{3}{2}\frac{Q}{\pi}\frac{\cos\beta}{R^2}$$

R - Polar radial coordinate of point P

$$R = \sqrt{r^2 + z^2} = \sqrt{x^2+y^2+z^2} \qquad r = \sqrt{x^2+y^2}$$

$$\cos\beta = \frac{z}{R}$$

σ_z - Vertical stress.

τ_{rz} - Tangential stress.

$$\sigma_z = \sigma_R \cos^2\beta$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos^3\beta}{R^2} \qquad \because \cos\beta = \frac{z}{R}$$

$$\sigma_z = \frac{3}{2}\frac{Q}{\pi}\frac{z^3}{R^5}$$

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

Multiply & divide by z^2.

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^2}{z^2}\frac{z^3}{(r^2+z^2)^{5/2}}.$$

$$= \frac{3Q}{2\pi z^2}\frac{z^5}{(r^2+z^2)^{5/2}}$$

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

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

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

$$\tau_{rz} = \frac{1}{2}\sigma_R \sin 2\beta \qquad \because \sin 2\beta = 2\sin\beta\cos\beta$$

$$= \frac{1}{\cancel{2}}\sigma_R(\cancel{2}\sin\beta\cos\beta)$$

$$= \frac{3}{2}\frac{Q}{\pi}\frac{\cos\beta}{R^2}\sin\beta\cos\beta \qquad \sin\beta = \frac{r}{R}, \quad \cos\beta = \frac{z}{R}.$$

$$= \frac{3Q}{2\pi R^2}\sin\beta\cos^2\beta.$$

$$= \frac{3Q}{2\pi R^2}\,\frac{r}{R}\cdot\frac{z^2}{R^2}$$

$$= \frac{3Q\,r\,z^2}{2\pi R^5}$$

$$= \frac{3Q}{2\pi}\,\frac{r z^2}{(r^2+z^2)^{5/2}}$$

$$= \frac{3Qr}{2\pi}\,\frac{\cancel{z^2}}{(z^2)^{5/2}\left[1+\frac{r^2}{z^2}\right]^{5/2}}.$$

$$\boxed{\tau_{rz} = \frac{3Qr}{2\pi z^3}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}}.$$

Bousinesq influence factor (K_B)

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

$$= K_B\frac{Q}{z^2}$$

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

K_B - Boussinesq influence factor.

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Influence factor - dimensionless.

Intensity of vertical pressure, directly below point load ($r=0$),

$$\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2} \quad \because r=0.$$

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

$$\boxed{\sigma_z = 0.4775\frac{Q}{z^2}}$$ ← At point directly below point load.

PRESSURE DISTRIBUTION DIAGRAMS:

By means of Boussinesq's stress distribution theory, the following vertical pressure distribution diagrams can be prepared

* Stress isobar or isobar diagram
* Vertical pressure distribution on a horizontal plane.
* Vertical pressure distribution on a vertical line.

Isobars :

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

It is a spatial, curved surface of the shape of a bulb, because vertical pressure on a given horizontal plane is same in all directions at points located at equal radial distances around axis of loading. (A)

The zone in a loaded soil mass bounded by an isobar of given vertical pressure intensity is called a pressure bulb.

The vertical pressure at every point on surface of pressure bulb is same.

Suppose an isobar of value $\sigma_z = 0.25\,Q$ per unit area is to be plotted.

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

$$K_B \frac{Q}{z^2} = 0.25\,Q$$

$$\boxed{K_B = 0.25\,z^2}$$

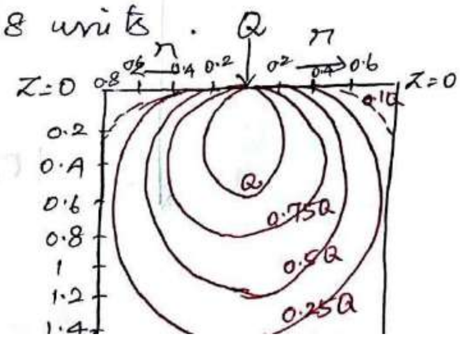
r/z values can be calculate from Table 13.1 in Soil Mechanics by B.C. Punmia Book.

ISOBAR DATA : ($\sigma_z = 0.25\,Q$).

z (units)	K_B	r/z	r (units)
0.2	0.01	1.92	0.38
0.4	0.04	1.3	0.52
0.6	0.09	0.97	0.58
0.8	0.16	0.74	0.59
1	0.25	0.54	0.54

When $r = 0$, $K_B = 0.4775$.

$$z = \sqrt{\frac{0.4775}{0.25}} = 1.38 \text{ units}.$$



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Vertical pressure distribution on a horizontal Plane :

The vertical pressure distribution on any horizontal plane @ a depth z below ground surface, due to concentrated load is given by

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

z is known.

r - horizontal distance

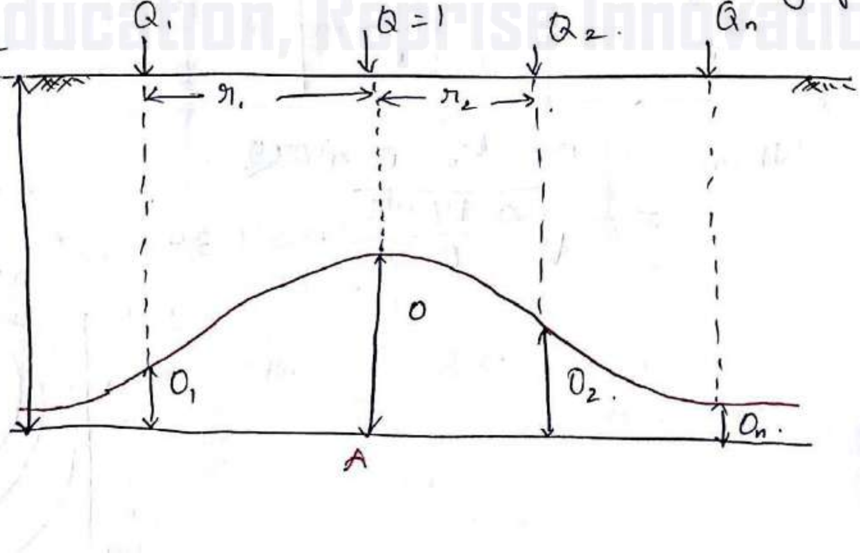
Below the load, i.e $r = 0$,

$$\sigma_z = 0.4775 \frac{Q}{z^2}$$

r/z	K_B	σ_z
0	0.4775	Maximum.
0.5	0.2733	57% of maximum.
1	0.0844	17.7% of maximum
2	0.0085	1.8% of maximum

When $r = 2z$, the vertical pressure is negligible.

Influence Diagram



Such a diagram is helpful in (5) computing the vertical stress σ_z at A due to number of concentrated loads $Q_1, Q_2, \dots Q_n$ situated at radial distances $r_1, r_2 \dots r_n$ from vertical axis through point A.

The vertical stress is then given by.

$$\sigma_z = \Sigma Q \cdot O.$$

$$= Q_1 O_1 + Q_2 O_2 + \dots \dots Q_n O_n \cdot$$

where $O, O_1, O_2 \dots O_n$ are ordinates of influence diagram plotted for σ_z at A.

Vertical pressure distribution on vertical line

σ_z decreases with increase in depth z.

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

If $r = 1$ unit.

z (units)	$\frac{r}{z}$	K_B	$\frac{K_B}{z^2}$	σ_z
0	∞	—	—	Intermediate
0.2	5	0.0001	0.0025	0.0025Q.
0.5	2	0.0085	0.0340	0.0340Q.
1	1	0.0844	0.0844	0.0844Q.
2	0.5	0.2733	0.0683	0.0683Q.
4	0.25	0.4103	0.0256	0.0256Q.

The vertical stress increases first, attains a maximum value and then decreases. It can be shown that maximum value of σ_z on a vertical line is obtained at the point of

intersection of the vertical plane with a radial line at $\beta = 39°15'$ through point load.
Corresponding value of $r/z = 0.817$.

$$z = \frac{r}{0.817}$$

If $r = 1$, $$z = \frac{1}{0.817}$$

$$z = 1.225$$

$$K_B = 0.1332$$

$$(\sigma_z)_{max} = \frac{0.1332\,Q}{(1.225)^2}$$

$$\boxed{(\sigma_z)_{max} = 0.0888\,Q}$$

VERTICAL PRESSURE UNDER A UNIFORMLY LOADED CIRCULAR AREA

Boussinesq equation for vertical stress due to single concentrated load can now be extended to find vertical pressure on any point on vertical axis passing through the centre of uniformly loaded circular area.

The figure shows a uniformly loaded circular area of radius a and load intensity q per unit area.

Consider an elementary ring of radius r and width δr on the loaded area. If elementary ring is further divided into small parts, each of area δA, the load on each elementary area will be $q\delta A$.

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This load may be considered as point load. Vertical pressure at point P, situated at depth z on vertical axis through centre of area, is given by.

$$\delta \sigma_z = \frac{3}{2\pi}(q\,\delta A)\frac{z^3}{(r^2+z^2)^{5/2}}.$$

Integrating over entire ring of radius r, vertical stress $\Delta\sigma_z$ is given by

$$\Delta\sigma_z = \frac{3q}{2\pi}(\Sigma\,\delta A)\frac{z^3}{(r^2+z^2)^{5/2}}.$$

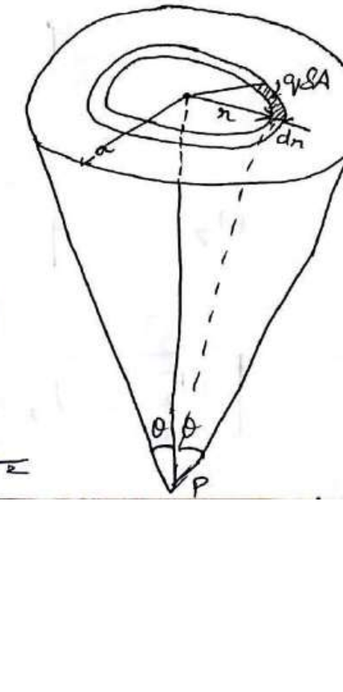
$$= \frac{3q}{2\pi}\,\Sigma\,\delta(\pi r^2)\frac{z^3}{(r^2+z^2)^{5/2}}.$$

$$= \frac{3q}{2\pi}\,\frac{2\pi r\,\delta r}{(r^2+z^2)^{5/2}}\,z^3$$

$$= 3qr\,\delta r\,\frac{z^3}{(r^2+z^2)^{5/2}}.$$

The total vertical pressure σ_z due to entire loaded area is given by integrating above expression b/w limits $r=0$ to $r=a$.

$$\sigma_z = 3qz^3\int_0^a \frac{r\,dr}{(r^2+z^2)^{5/2}}.$$



Put $r^2+z^2 = n^2$.

Differentiate w.r.to r.

$$2r\,dr + 0 = 2n\,dn.$$

$$r\,dr = n\,dn.$$

when $r=0$, $n=z$. when $r=a$, $n=\sqrt{a^2+z^2}$

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$$\sigma_z = 3q z^3 \int_z^{\sqrt{a^2+z^2}} \frac{n\,dn}{(n^2)^{5/2}}$$

$$= 3q z^3 \int_z^{\sqrt{a^2+z^2}} \frac{\cancel{n}\,dn}{n^{\cancel{5}}}$$

$$= 3q z^3 \int_z^{\sqrt{a^2+z^2}} \frac{dn}{n^4}$$

$$= -\frac{\cancel{3}}{\cancel{3}} q z^3 \left[\frac{1}{n^3}\right]_z^{\sqrt{a^2+z^2}}$$

$$= -q z^3 \left[+\frac{1}{n^3}\right]_z^{\sqrt{a^2+z^2}}$$

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

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

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

$$= q \left[1 - \frac{\cancel{z^3}}{\cancel{z^3}\left[1+\frac{a^2}{z^2}\right]^{3/2}}\right]$$

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

$$\boxed{\sigma_z = K_B\, q}$$

Differentiation

x^n

$n x^{n-1}$

n^{-4}

$-4\, n^{-4-1}$

Integration.

$x^n = \frac{x^{n+1}}{n+1}$

$n^{-4} = \frac{n^{-4+1}}{-4+1}$

$= \frac{n^{-3}}{-3}$

K_B - Boussinesq influence factor for uniformly distributed circular load (7)

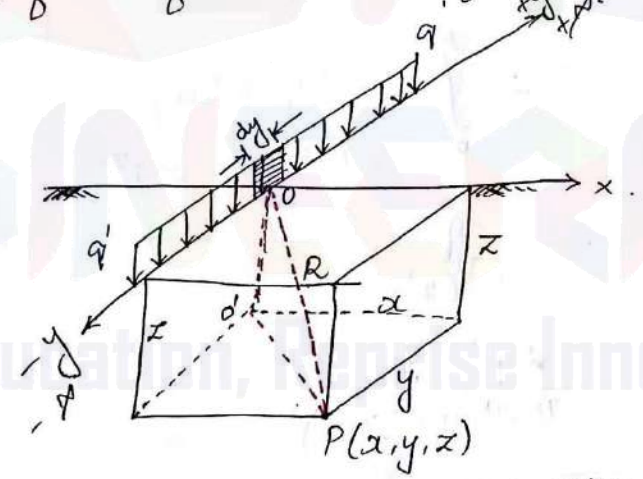
$$\boxed{K_B = 1 - \left[\frac{1}{1+\left(\frac{a}{z}\right)^2}\right]^{3/2}}$$

If θ is the angle which the line joining the point P makes with outer edge of loading.

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

VERTICAL PRESSURE DUE TO A LINE LOAD

Let us consider an infinitely long line load of intensity q per unit length, acting on the surface of a semi-infinite elastic medium.



Let us find the expression for vertical stress at any point P having coordinates (x,y,z)

The radial distance of point $P = r$

$$r = \sqrt{x^2+y^2}$$

The polar distance of point $P = R$

$$R = \sqrt{r^2+z^2} = \sqrt{x^2+y^2+z^2}$$

Consider a small length dy along line load. The elementary load in this length will be equal to $q'dy$, which may be considered as concentrated load.

Vertical stress

$$\Delta\sigma_z = \frac{3(q'dy)z^3}{2\pi R^5}$$

$$= \frac{3q'dy\ z^3}{2\pi\ (x^2+y^2+z^2)^{5/2}}.$$

$$\sigma_z = \int_{-\infty}^{+\infty} \frac{3q'z^3dy}{2\pi\ (x^2+y^2+z^2)^{5/2}}.$$

$$= 2\int_0^{\infty} \frac{3q'z^3\,dy}{2\pi(x^2+y^2+z^2)^{5/2}}.$$

$$= \frac{2q'z^3}{\pi\ (x^2+z^2)^2}$$

$$= \frac{2q'\cancel{z^3}}{\pi \cancel{z^4}\left(1+\frac{x^2}{z^2}\right)^2}$$

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

When P is situated vertically below line load, at depth z, we have $x=0$.

$$\boxed{\sigma_z = \frac{2q'}{\pi z}}$$

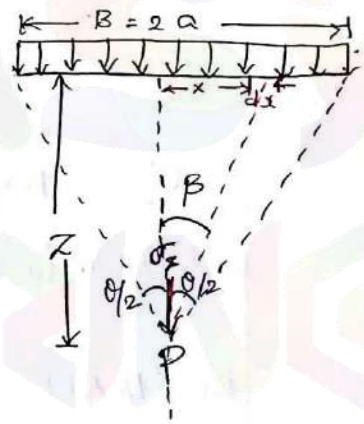
VERTICAL PRESSURE UNDER STRIP LOAD (8)

Consider a strip load of width dx, at distance x from centre. The elementary line load intensity along this elementary strip of width dx will be $q.dx$.

The vertical pressure P due to this elementary line load is given by

$$\Delta\sigma_z = \frac{2(q\,dx)}{\pi z}\left[\frac{1}{1+\left(\frac{x^2}{z}\right)}\right]^2$$

Total vertical pressure due to whole strip is given by



$$\sigma_z = \frac{2q}{\pi z}\int_{-B/2}^{B/2}\frac{dx}{\left[1+(x/z)^2\right]^2}$$

$$= \frac{2q}{\pi z}\cdot 2\int_{0}^{B/2}\frac{dx}{\left[1+(x/z)^2\right]^2}$$

$x/z = \tan\beta$.

$x = z\tan\beta$.

$dx = z\sec^2\beta \cdot d\beta$.

$$\sigma_z = \frac{4q}{\pi \cancel{z}}\int_{0}^{\theta/2}\frac{\cancel{z}\sec^2\beta\, d\beta}{(1+\tan^2\beta)^2}$$

$$= \frac{4q}{\pi}\int_{0}^{\theta/2}\cos^2\beta\, d\beta$$

$$\boxed{\sigma_z = \frac{q}{\pi}(\theta+\sin\theta)}$$

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VERTICAL PRESSURE UNDER A UNIFORMLY LOADED RECTANGULAR AREA

Let us take the case of rectangular load area of length 2a & width 2b

A more common form of vertical stress under corner of rectangular area of size a.b is as follows.

$m = a/z$, $n = b/z$

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

$\sigma_z = Kq$.

K - Influence factor.

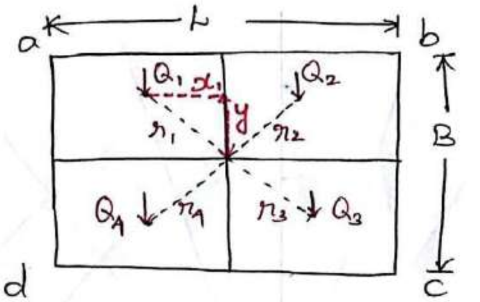
———— x ————

EQUIVALENT POINT LOAD METHOD:

(Approximate method)

- To find vertical stress at any point due to any loaded area.

The entire area is subdivided into a number of small area units & total distributed load over a unit area is replaced by a point load of same magnitude acting at centroid of area unit.



$$\sigma_z = \frac{1}{z^2}\left[Q_1 K_{B_1} + Q_2 K_{B_2} + \dots Q_n K_{B_n}\right].$$

If all point are of equal magnitude Q'

$$\boxed{\sigma_z = \frac{Q'}{z^2}\,\Sigma K_B}$$

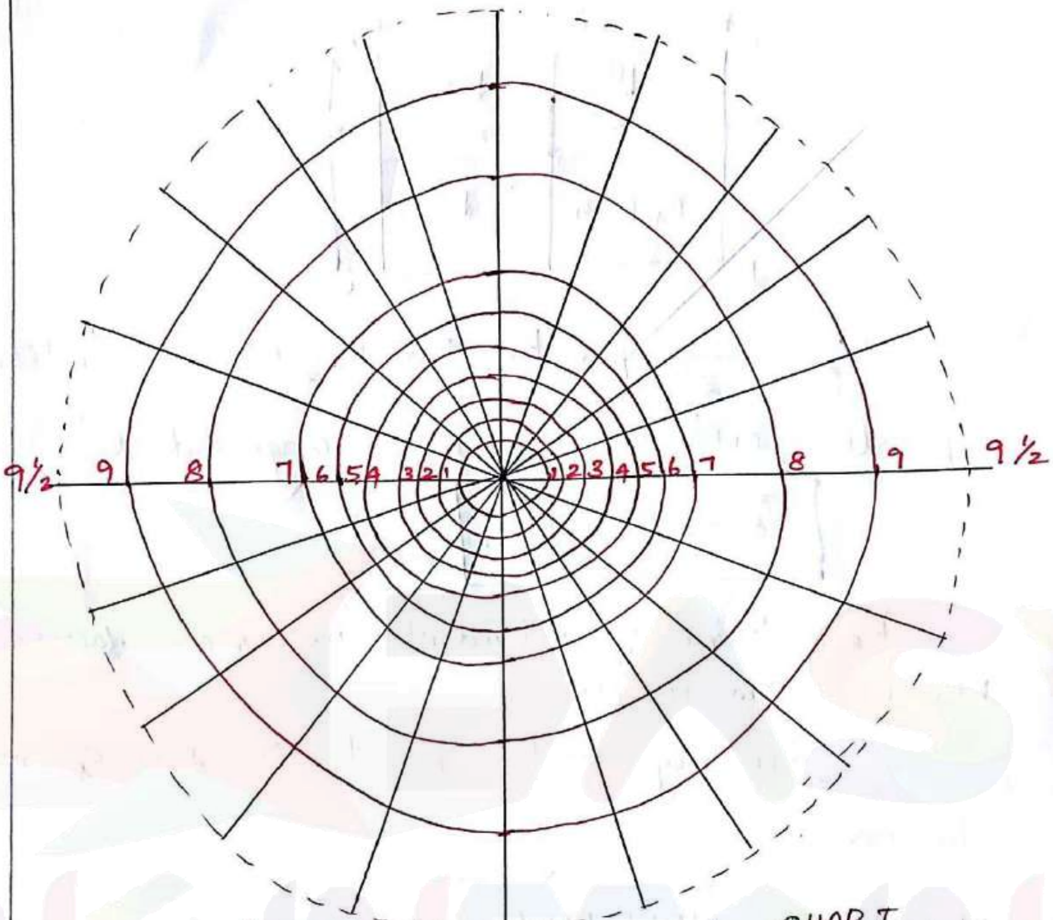
ΣK_B = Sum of individual influence factors for various area units.

Accuracy will depend on size of area unit choosen.

NEWMARK'S INFLUENCE CHART: [Accurate Method]

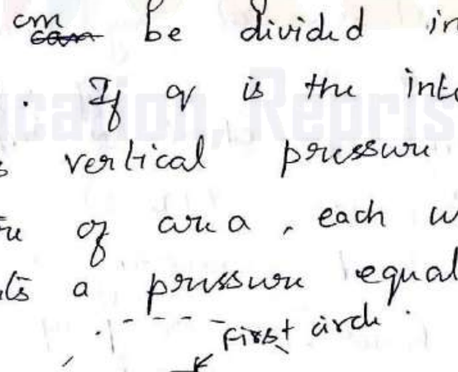
A more accurate method of determining the vertical stress at any point under a uniformly loaded area of any shape is with the help of influence chart or influence diagram suggested by Newmark (1942).

A chart, consisting of number of circles & radiating lines, is so prepared that the influence of each area unit is the same at centre of circles, i.e. each area unit causes the equal vertical stress at the centre of diagram.



NEWMARK'S INFLUENCE CHART

Let a uniformly loaded circular area of radius r_1 cm be divided into 20 sectors (area units). If q is the intensity of loading and σ_z is vertical pressure at depth z below centre of area, each unit such as OA_1B_1 exerts a pressure equal to $\sigma_z/20$ at centre.



From formula of σ_z from uniformly loaded circular area, (10)

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

Here $a = r_1$,

$$\frac{\sigma_z}{20} = \frac{q}{20}\left[1-\left\{\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right\}^{3/2}\right]$$

$$\boxed{\frac{\sigma_z}{20} = i_f\, q}$$

i_f - influence value.

$$i_f = \frac{1}{20}\left[1-\left\{\frac{1}{1+(r_1/z)^2}\right\}^{3/2}\right].$$

If i_f be made equal to an arbitrary fixed value, say 0.005, we have.

$$\frac{\sigma_z}{20} = i_f\, q$$

$$= 0.005\, q.$$

$$i_f = \frac{1}{\text{No. of radial lines} \times \text{No. of concentric circles}}$$

$$= \frac{1}{20\times 10}$$

$$\boxed{I_f = 0.005}$$

$$\frac{q}{20}\left[1-\left\{\frac{1}{1+(r/z)^2}\right\}^{3/2}\right] = 0.005\, q.$$

r_1 can be found from this equation, when z is known.

The pressure due to first concentric circle OA_1B_1 is $0.005q$.

The pressure due to second concentric circle $A_1B_1\,A_2B_2$ is $0.005q$.

Total pressure $OA_2B_2 = 2\times 0.005q$.

Illrly, the radii of 3rd, 4th, 5th, 6th, 7th, 8th, 9th circles can be calculated.

The radius of 10th circle is given by equation,

$$\frac{q}{20}\left[1-\left\{\frac{1}{1+\left(\frac{r_{10}}{z}\right)^2}\right\}^{3/2}\right] = 10 \times 0.005 q = q/20.$$

which means r_{10} = infinity.

Vertical pressure $\boxed{\sigma_A = 0.005 q \times N_A}$

N_A - Number of area units under loaded area.

ONE DIMENSIONAL CONSOLIDATION

When a compressive load is applied to soil mass, a decrease in its volume takes place. The decrease in volume of soil mass under stress is known as compression.

The property of soil mass pertaining to its susceptibility to decrease in volume under pressure is known as compressibility.

Voids - air - compressible - escape easily from void.

Saturated soil mass -

Voids - incompressible water - compression - water expelled out of voids.

Such a compression results from long term static load & consequent escape of pore water is termed as consolidation

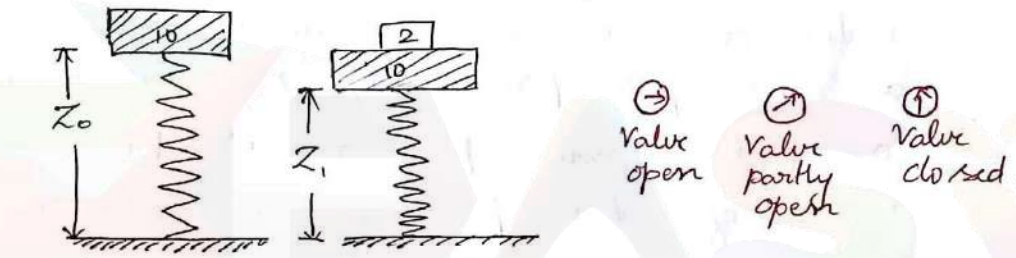
According to Terzaghi : "Every process involving a decrease in water content of a saturated soil without replacement of water by air is called a process of consolidation".

Consolidation - decrease in water content

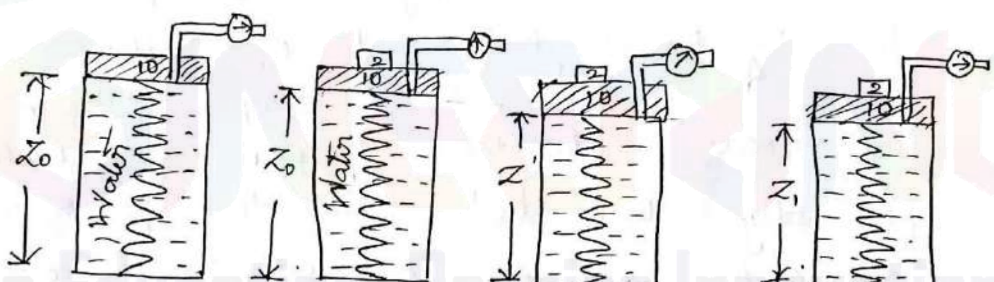
Swelling - increase in water content.

Compaction - Expulsion of air

SPRING ANALOGY.



a) $\sigma = \sigma'$ b) $\sigma = \sigma'$



c) $\sigma = \sigma' = 10$ d) $\sigma = \sigma' + \bar{u}$ e) $12 = (10 + \Delta\sigma') + (2 - \Delta\sigma')$ f) $\sigma = \sigma' + \bar{u}$

Total pressure = Pressure in spring + Pressure in water.

$$\sigma = \sigma' + u.$$

When there is a pressure increment, the whole of pressure is first taken by water. As water escapes out of system, the load transfer takes place from water to spring till spring is deformed by full amount

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corresponding to applied stress increment.

This analogy can be applied to consolidation process of soil mass consisting of soil water system.

Grain Structure - Spring.

Voids filled with water - cylinder.

Valve opening - Permeability of soil mass.

<u>Pore Pressure</u> ($\bar{u}$)

The pressure that builds up in pore water due to load increment on soil is termed as excess pore pressure / excess hydrostatic pressure or hydrodynamic pressure ($\bar{u}$). because it is in excess of initial pressure in water under static condition.

The excess hydrostatic pressure forces the water to drain out of voids.

As water starts escaping from voids, the excess hydrostatic pressure in water gets gradually dissipated and the pressure increment is shifted as an increase in effective pressure on soil solids and soil mass decrease in volume.

When whole of pressure increment or consolidation pressure is carried as an increase in effective pressure on solids, no more water escapes from voids and a condition of equilibrium is attained.

Hydrodynamic lag:
The delay caused in consolidation by the slow drainage of water out of saturated soil mass is called hydrodynamic lag.

Primary Consolidation:
The reduction in volume of soil which is due principally due to squeezing out of water from voids is termed primary consolidation primary compression or primary time effect.

Secondary Consolidation:
Even after reduction of all excess hydrostatic pressure to zero, some compression of soil takes place at very slow rate. This is known as secondary consolidation, secondary compression or secondary time effect.

During secondary compression, some of highly viscous water between the points of contact is forced out from b/w particles.

CONSOLIDATION OF LATERALLY CONFINED SOIL

If a remoulded soil is laterally confined in a consolidometer, consisting of a metal ring and porous stones are placed both at its top & bottom faces, the compression or consolidation of soil sample takes place under a vertical pressure applied on the top of porous stones. The porous stones provide free drainage

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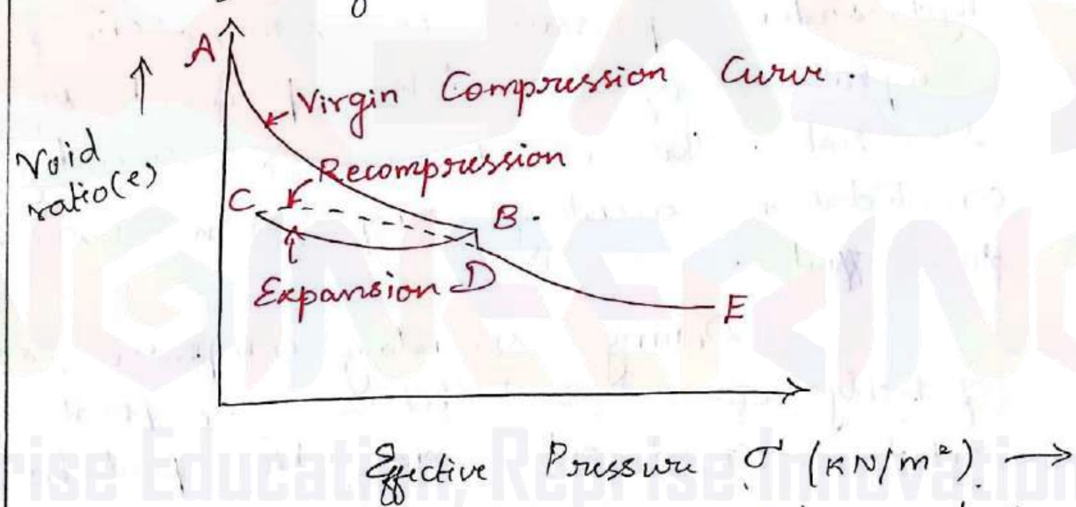
of water and air from or into soil sample.

Under a given applied pressure, a final settlement & equilibrium voids ratio is attained after certain time.

At the equilibrium stage, the applied pressure naturally becomes effective pressure σ' on soil. The pressure can then be increased and a new equilibrium voids ratio is attained.

Thus, a relation can be obtained b/w effective pressure σ' & equilibrium voids ratio e. in the form of curve.



At any intermediate stage at B, the pressure is completely removed, the the sample expands as represented by the expansion curve BC.

During expansion, the sample never attains the original void ratio, because of some permanent compression mainly due to some irreversible orientation undergone by soil particles under compression. (13)

If soil is again put under compression a recompression curve such as CD is obtained. the void ratio at D being always less than that at B. at the same pressure.

On further pressure increments, the curve DE is obtained.

The portion AB of the curve represents the compression of soil which has not been subjected in past to pressures greater than those which are being applied for the present compression. Such a curve is called virgin compression curve & curve DE is Virgin curve.

σ' - abscissa. } Semi - log plot.
e - ordinate } Virgin compression curve - Straight line.

It can be expressed by

$$\boxed{e = e_0 - C_c \log_{10} \frac{\sigma'}{\sigma_0'}}$$

e_0 - initial voids ratio corresponding to initial pressure σ_0'

e - void ratio at increased pressure σ'

C_c - Compression index (dimensionless).

The compression index represents slope of linear portion of the pressure - voids ratio curve & remains constant within a fairly large range of pressure.

$$\boxed{C_c = \frac{e_0 - e}{\log_{10} \frac{\sigma'}{\sigma_0'}}} = \frac{\Delta e}{\Delta \log_{10} \sigma'}$$

$$\boxed{\Delta e = C_c \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}}.$$

The expansion curve is also a fairly straight line on semilog plot & is expressed as

$$e_0 = e + C_s \log_{10} \frac{\sigma'}{\sigma_0'}$$

C_s - Expansion/ Swelling index. It is a measure of volume increase due to removal of pressure -

Skempton conducted consolidation tests on number of clays, & gave following equation.

$$\boxed{C_c = 0.007\,(w_L - 10\%)}.$$

C_c of remoulded sample,

$$\boxed{C_c = 0.009\,(w_L - 10\%)}$$

Hough gave equation for precompressed soils.

$$\boxed{C_c = 0.3(e_0 - 0.27)}$$

e_0 - insitu void ratio.

Coefficient of Compressibility (a_v) :

It is defined as the decrease in void ratio per unit increase of pressure :

$$\boxed{a_v = \frac{-\Delta e}{\Delta\sigma'} = \frac{e_0 - e}{\sigma' - \sigma_0'}}.$$

For a given difference in pressure, value of coefficient of compressibility decreases as the pressure increases.

Coefficient of volume Change (m_v)

It is defined as the change in volume of a soil per unit of initial volume due to given unit increase in the pressure:

$$\boxed{m_v = -\frac{\Delta e}{1+e_0}\,\frac{1}{\Delta\sigma'}}$$

$$-\frac{\Delta e}{\Delta\sigma'} = a_v \implies \boxed{m_v = \frac{a_v}{1+e_0}}$$

When soil is laterally confined, change in volume is proportional to change in thickness ΔH and initial volume is proportional to initial thickness H_0

$$\boxed{m_v = -\frac{\Delta H}{H_0}\,\frac{1}{\Delta\sigma'}}$$

SETTLEMENT:

The two types of settlement are.

i) Initial settlement

ii) Consolidation settlement

Initial Settlement:

The change in thickness, ΔH due to pressure increment is given by

$$\boxed{\Delta H = -m_v H_0 \Delta\sigma'}$$

(-) sign - decrease of void ratio/thickness with increase in pressure.

Consolidation Settlement:

a) Using coefficient of volume change (m_v)

b) Using void ratio.

1) Final settlement using coefficient of volume change (m_v).

ρ_f - Consolidation settlement.

$\rho_f = m_v H \Delta \sigma'$

This is on assumption that pressure increment is transmitted uniformly over thickness H.

In practical cases, under a finite surface loading, intensity of $\Delta\sigma'$ decreases with depth of layer in a non-linear manner. In such circumstances, consolidation settlement $\Delta\rho_f$ of an element of thickness dz is calculated under an average effective pressure increment $\Delta\sigma'$.

$\Delta\rho_f = m_v \, \Delta\sigma' \, dz$

Integrating for total thickness H of layer.

$$\rho_f = \int_0^H m_v \, \Delta\sigma' dz.$$

m_v & $\Delta\sigma'$ are variables.

The numerical integration can be performed by dividing the total thickness H into no. of thin layers & $\Delta\sigma'$ at mid-height of each layer may be considered to represent a constant average pressure increment for the layer. Settlement of each layer can then be calculated.

Total Settlement = Sum of individual settlements of various thin layers.

$$\frac{\Delta H}{H} = \frac{e_0 - e}{1+e_0}$$

$$\rho_f = \Delta H = \frac{e_0 - e}{1+e_0} H.$$

i) Normally consolidated soils :

Compression index for normally consolidated soil is constant.

$$\rho_f = H \frac{C_c}{1+e_0} \log_{10} \frac{\sigma'}{\sigma_0'}$$

$$\sigma' = \sigma_0' + \Delta\sigma'$$

$$\boxed{\rho_f = H \frac{C_c}{1+e_0} \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}}$$

ii) Preconsolidated soils :

Swelling index $C_s < C_c$.

$$\boxed{\rho_f = H \frac{C_s}{1+e_0} \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}}$$

CONSOLIDATION OF UNDISTURED SPECIMEN :

OCR - Over Consolidation Ratio

Soil deposits may be divided into 3 classes.

* Pre-consolidated or overconsolidated - OCR > 1, U > 100
* Normally consolidated - OCR = 1, U = 100
* Under consolidated - OCR < 1, U < 100

Clay - Precompressed / Preconsolidated / Overconsolidated if it has been subjected to pressure in excess of its present overburden pressure

The temporary overburden pressure is known as preconsolidation pressure.

Normally Consolidated soil: is one which has never been subjected to an effective pressure greater than existing overburden pressure & which is completely consolidated by existing overburden.

Under. Consolidated soil:

A soil which is not fully consolidated under the existing overburden pressure is called an under-consolidated soil.

——— × ———

TERZAGHI'S THEORY OF ONE DIMENSIONAL CONSOLIDATION:

Assumptions:

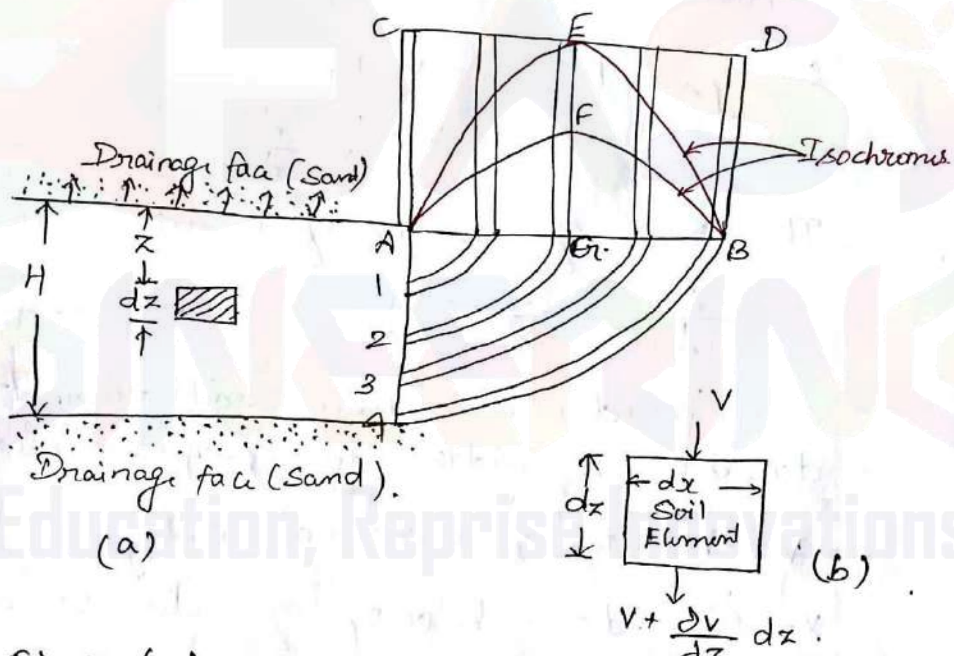
* The soil is homogeneous and fully saturated.
* Soil mass & water are incompressible
* Deformation of soil is entirely due to change in volume
* Darcy's law for velocity of flow of water through soil is perfectly valid.
* Coefficient of permeability is constant during consolidation.
* Load is applied in one direction only & deformation occurs only in direction of load application. i.e Soil is restrained against lateral deformation

* Excess pore water drains out only in vertical direction. (16)

* Boundary is free surface offering no resistance to flow of water from soil.

* Change in thickness during consolidation is insignificant.

* Time lag in consolidation is entirely due to permeability of soil & thus secondary consolidation is disregarded.



(a) (b)

Figure (a) Shows a clay layer of thickness H sandwitched b/w two layers of sand which serves as drainage faces.

When layer is subjected to pressure increment $\Delta\sigma$, excess hydrostatic pressure is set up in clay layer.

At time t_0 — $\Delta\sigma = \bar{u}$ — CED

At time t_f : $\bar{u} = 0$. – AFB

At any time t, $\Delta\sigma = \Delta\sigma' + \bar{u}$ – AFB.

$\bar{u} = h\gamma_w$.

Hydraulic head, $h = \frac{\bar{u}}{\gamma_w}$ —— ①

Hydraulic gradient $i = \frac{dh}{dz} = \frac{d}{dz}\left(\frac{\bar{u}}{\gamma_w}\right)$.

$$i = \frac{1}{\gamma_w}\frac{d\bar{u}}{dz} \quad \text{—— ②}$$

Thus, rate of change of $\bar{u}$ along depth of layer represents hydraulic gradient.

Darcy's law $v = ki$

$$v = \frac{k}{\gamma_w}\frac{d\bar{u}}{dz} \quad \text{—— ③}$$

The rate of change of velocity is

$$\frac{dv}{dz} = \frac{k}{\gamma_w}\frac{d^2\bar{u}}{dz^2} \quad \text{—— ④}$$

Consider a small soil element of size dx, dz & width dy perpendicular to xz plane.

v - Velocity of water @ entry

$v + \frac{dv}{dz}.dz$ - Velocity of water @ exit.

Quantity = Velocity × Area.

Quantity of water entering soil = $v\,dx\,dy$

Quantity of water leaving soil = $\left[v + \frac{dv}{dz}dz\right]dx\,dy$.

Net quantity of water squeezed out of soil

= Quantity leaving soil (−) Quantity entering soil.

$$\Delta q = -v\, \cancel{dx}\, dy + v\, dx\, dy + \frac{\partial v}{\partial z}\, dx\, dy\, dz$$ (19)

$$\Delta q = \frac{\partial v}{\partial z}\, dx\, dy\, dz \qquad — (5).$$

Decrease in volume of soil is equal to volume of water squeezed out.

$$\Delta V = -m_v V_0\, \Delta\sigma' \qquad — (6)$$

$$V_0 = dx\, dy\, dz.$$

Change of volume per unit time

$$\frac{\partial(\Delta V)}{\partial t} = -m_v\, dx\,dy\,dz\, \frac{\partial(\Delta\sigma')}{\partial t}. \qquad — (7)$$

Equating (5) & (7) $\Delta q = \dfrac{\partial(\Delta V)}{\partial t}$

$$\frac{\partial v}{\partial z} = -m_v \frac{\partial(\Delta\sigma')}{\partial t} \qquad — (8).$$

Combining (A) & (8)

$$\frac{K}{\gamma_\omega}\frac{\partial^2 \bar{u}}{\partial z^2} = -m_v \frac{\partial(\Delta\sigma')}{\partial t}$$

$$\Delta\sigma = \Delta\sigma' + \bar{u} \implies \Delta\sigma \text{ is constant.}$$

$$\frac{\partial(\Delta\sigma')}{\partial t} = -\frac{\partial \bar{u}}{\partial t}$$

$$\frac{K}{\gamma_\omega}\frac{\partial^2 \bar{u}}{\partial z^2} = -m_v \frac{\partial \bar{u}}{\partial t}$$

$$\boxed{\frac{\partial \bar{u}}{\partial t} = \frac{-K}{m_v \gamma_\omega}\cdot\frac{\partial^2 \bar{u}}{\partial z^2}}$$

$$\boxed{\frac{\partial \bar{u}}{\partial t} = C_v \frac{\partial^2 \bar{u}}{\partial z^2}}$$

C_v - Coefficient of consolidation

$$\boxed{C_v = \frac{K}{m_v \gamma_w}} = \frac{K(1+e_0)}{a_v \gamma_\omega}$$

C_v - cm^2/sec.

This is the basic differential equation of consolidation which relates rate of change of excess hydrostatic pressure to rate of expulsion of excess pore water from unit volume of soil during same time interval.

The solution of consolidation equation is

$$\boxed{T_v = \frac{K}{m_v \gamma_w} \frac{t}{d^2}} \Longrightarrow \boxed{T_v = \frac{C_v t}{d^2}}$$

$$\boxed{T_v = \frac{K(1+e_0)}{a_v \gamma_w} \frac{t}{d^2}}$$

$\frac{T_v}{C_v}$ = constant

$t \propto d^2$

T_v - Time factor

t - Thickness of clay layer.

Degree of consolidation

$$U = \frac{\text{Excess pore pressure dissipated}}{\text{Initial excess}} \times 100$$

$$= \frac{\text{Initial excess} - \text{Present excess}}{\text{Initial excess}} \times 100.$$

If initial excess & Present excess is same, then $U = 0$ - No consolidation.

If Present excess is 0,

$U = 100\%$. - Full consolidation.

$$\boxed{U = \frac{\text{Present Settlement}}{\text{Ultimate settlement}} \times 100.}$$

If $\underline{U < 60\%}$, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

If $\underline{U > 60\%}$, $T_v = 1.7813 - 0.9332 \log_{10}(100 - U\%)$

LABORATORY CONSOLIDATION TEST

Apparatus - Consolidometer.

* Consists of loading frame & Consolidation cell.
* Porous stones are put on top & bottom ends of specimen.

- Floating ring cell
- Fixed ring cell.

	Fixed ring cell	Floating ring cell
1.	Only top porous stone is permitted to move downwards as specimen compresses.	Both top & bottom porous stones are free to compress the specimen towards middle.
2.	Permeability of specimen at any stage can be directly measured.	Permeability of specimen at any stage cannot be directly measured. Smaller effects of friction b/w specimen ring & soil specimen

Vertical compression of specimen is measured by means of dial gauge.

After completion of consolidation under desired maximum vertical pressure, specimen is unloaded & allowed to swell. Final dial reading is recorded & specimen is taken out. Test data are used to determine

1. Voids ratio & coefficient of volume change.
2. Coefficient of consolidation
3. Coefficient of permeability.

1. Determination of voids ratio & coefficient of volume change:

Two methods : i) Height of solids method.
ii) Change in voids ratio method

Change in voids ratio - Only for fully saturated specimens.

Height of solids - Both saturated as well as unsaturated specimens.

i) Height of solids method:

Height of solids $H_s = \frac{M_d}{G A \rho_w} = \frac{W_d}{G A}$

H_s - Height of solids (cm)
M_d - Mass of dried specimen (g)
W_d - Weight of dried specimen (g)
A - C/S area of specimen (cm^2)
G - Specific gravity of soil.

void ratio $\boxed{e = \frac{H - H_s}{H_s}}$

H = Specimen height @ equilibrium
$H = H_0 + \Sigma \Delta H = H_1 + \Delta H$
H_0 - Initial height of specimen
ΔH = Change in specimen thickness under any pressure increment
H_1 - Height of specimen at beginning of load increment.

ii) Change in voids ratio method : (19)

$e_f = w_f G$ $\quad$ $eS_r = wG$. If $S_r = 1$, $e = wG$

e_f - Final void ratio

w_f - Final water content.

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

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

H_f - Final height of specimen.

Coefficient of volume change :

$$\boxed{m_v = -\frac{\Delta e}{1+e_0} \frac{1}{\Delta \sigma'}}$$

$$\boxed{m_v = -\frac{\Delta H}{H_0} \frac{1}{\Delta \sigma'}}$$

2. Determination of coefficient of consolidation :

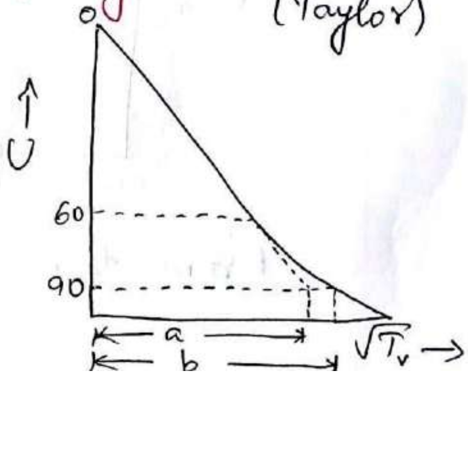
Two methods :

i) Square root of time fitting method.

ii) Logarithm of time fitting method.

i) Square root of time fitting method : (Taylor)

Figure shows theoretical characteristic curve b/w degree of consolidation & $\sqrt{T_v}$



Upto $U = 60\%$, curve is straight.

T_v Abscissa @ $U = 90\% = 1.15$ (Abscissa @ $U = 60\%$).

Abscissa $= \sqrt{t}$

Ordinate $= R$

R - dial reading

t - Time.

R_0 - Initial dial reading

R_c - Corrected zero reading.

Consolidation b/w R_0 & R_c - Initial consolidation.

From R_c, B is drawn,

$B = 1.15A$.

Intersection of B with consolidation curve = U-90%.

$$\boxed{(T_v)_{90} = 0.848}$$

$$\boxed{C_v = \frac{0.848\, d^2}{t_{90}}}$$

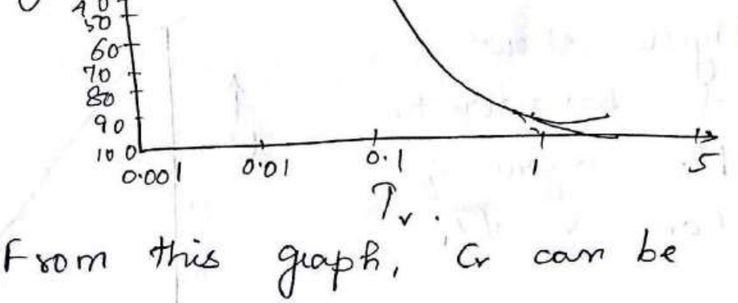
From this C_v can be calculated.

ii) Logarithm of time fitting method: (Casagrande).

Abscissa - $\log_{10} T_v$

Ordinate - $U\%$.

Semi log plot of lab time-consolidation curve.



From this graph, C_v can be calculated.

3. Determination of coefficient of permeability

A falling head permeability test can be performed on consolidation specimen attaching a stand pipe to fixed ring consolidometer, when the consolidation of specimen is complete under a particular pressure increment.

Coefficient of permeability

$K = C_v m_v \gamma_\omega$

$$\boxed{K = \frac{C_v a_v \gamma_w}{1+e_0}}$$

Knowing C_v & m_v, K can be calculated.

PROBLEMS:

1. An undisturbed sample of clay 24mm thick, consolidated 50% in 20 minutes, when tested in laboratory with drainage allowed at top & bottom. The clay layer, from which sample was obtained, is 4m thick in field. How much time will it take to consolidate 50%, with double drainage? If clay stratum has only single drainage, calculate the time to consolidate 50%. Assume uniform distribution of consolidation pressure.

Same degree of consolidation, T_v - same.

Both soils are same, C_v - Same.

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

$$t \propto d^2$$

a) Double drainage :

$$t \propto d^2$$

$$\frac{t_2}{t_1} \propto \left(\frac{d_2}{d_1}\right)^2$$

Field - 2
Lab - 1

$t_1 = 20$ min

$U = 50\%$

$t_2 = ?$

Double drainge $= d/2$.

$d_1 = 24\,mm/2$

$= 12\,mm$

$= 0.012\,m$.

$d_2 = 4\,m/2 = 2\,m$.

$$t_2 = t_1\left(\frac{d_2}{d_1}\right)^2 = 20\left[\frac{2}{0.012}\right]^2 = 555,566.67 \text{ min.}$$

$$= \frac{555566.67}{24 \times 60} \text{ days}$$

$$\boxed{t_2 = 386 \text{ days}}.$$

b) Single drainage :

$$\boxed{\begin{array}{l} d = H - \text{Single} \\ d = H/2 - \text{Double} \end{array}}$$

$$t \propto d^2$$

$$t_2 = t_1\left(\frac{d_2}{d_1}\right)^2.$$

$t_1 = 20$ mins.

$d_1 = 24\,mm/2 = 12\,mm = 0.012\,m$. [Double]

$d_2 = 4\,m$ [Single].

$$t_2 = 20\left[\frac{4}{0.012}\right]^2 = 2,222,220 \text{ min}$$

$$\boxed{t_2 = 1544 \text{ days}}$$

$T_{\text{single drainge}} = 4 \times T_{\text{double drainage}}$.

$$\boxed{T_S = 4\,T_D}$$

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2. An undisturbed sample of clay stratum 2m thick, was tested in laboratory & average value of coefficient of consolidation was found to be 2×10^{4} cm^2/sec. If a structure is built on clay stratum, how long it will take to attain half the ultimate settlement under load of structure? Assume double drainage:

$C_v = 2\times10^{4}\ cm^2/sec$.

$H = 2\ m$.

$U = 50\%$. Double drainage

$t = ?$

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

$d = H/2 = 2/2 = 1m. = 100\ cm$

$$U < 60\%,\quad T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$T_v = \frac{\pi}{4}\left(\frac{50}{100}\right)^2$$

$$\boxed{T_v = 0.197}$$

$$t = \frac{0.197\times(100)^2}{2\times10^{4}}$$

$t = 9850000$ secs$/24\times60\times60$

$$\boxed{t = 114 \text{ days}}$$

Time to attain half the ultimate settlement $= 114$ days.

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3. Two clay specimens A & B of thickness 2cm & 3cm, have equilibrium void ratio 0.68 & 0.72 respectively under a pressure of 200 KN/m². If the equilibrium void ratios of the two soils reduced to 0.5 & 0.62 respectively when pressure was increased to 400 KN/m², find ratio of coefficient of permeability of two specimens. The time required by specimen A to reach 40% degree of consolidation is 1/4 of that required by specimen B for reaching 40% degree of consolidation.

$U = 40\%, \quad t_A = \frac{1}{4} t_B.$

$H_A = 2cm = d_A \quad H_B = 3cm = d_B$

$\sigma = 200 \ kN/m^2, \quad e_A = 0.68, \quad e_B = 0.72.$

$\sigma = 400 \ kN/m^2, \quad e_A = 0.5, \quad e_B = 0.62.$

$$C_v = \frac{k}{m_v \gamma_\omega} \qquad \gamma_{\omega A} = \gamma_{\omega B}.$$

$$\frac{C_{v_A}}{C_{v_B}} = \frac{\dfrac{K_A}{m_{v_A} \cancel{\gamma_{\omega A}}}}{\dfrac{K_B}{m_{v_B} \cancel{\gamma_{\omega B}}}} = \frac{K_A}{K_B} \cdot \frac{m_{v_B}}{m_{v_A}}.$$

Coefficient of permeability - k $\quad \frac{K_A}{K_B} = ?$

$$\frac{K_A}{K_B} = \frac{C_{v_A}}{C_{v_B}} \cdot \frac{m_{v_A}}{m_{v_B}}.$$

$$m_v = \frac{\frac{\Delta e}{1+e_0}}{\Delta \sigma'}$$

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

$$\frac{T_{vA}}{T_{vB}} = \frac{C_{vA} t_A}{C_{vB} t_B} \cdot \left(\frac{d_B}{d_A}\right)^2$$

$U = 40\% \quad - \quad T_{vA} = T_{vB}$.

$t_B = 4 t_A$

$$\frac{C_{vA}}{C_{vB}} = \frac{t_B}{t_A}\left(\frac{d_B}{d_A}\right)^2$$

$$= \frac{4\cancel{t_A}}{\cancel{t_A}}\left(\frac{3}{2}\right)^2$$

$$\boxed{\frac{C_{vA}}{C_{vB}} = 9}$$

$$\cancel{m_{vA} = \frac{0.72 - 0.68}{1+0.}}$$

$$m_{vA} = \frac{0.68 - 0.5}{1+0.68} / (400 - 200)$$

$$= 5.36 \times 10^{-4} \ m^2/kN.$$

$$m_{vB} = \frac{0.72 - 0.62}{1+0.72} / 200$$

$$= 2.91 \times 10^{-4} \ m^2/kN.$$

$$\boxed{\frac{m_{vA}}{m_{vB}} = 1.842}$$

$$\frac{m_{vA}}{m_{vB}} = \frac{5.36 \times 10^{-4}}{2.91 \times 10^{-4}} = 1.842$$

$$\frac{K_A}{K_B} = \frac{C_{vA}}{C_{vB}} \cdot \frac{m_{vA}}{m_{vB}} = 9 \times 1.842$$

$$\boxed{\frac{K_A}{K_B} = 16.58}$$

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4. The loading period for a new building extended from July 1980 to July 1982. In July 1985, the average measured settlement was found to be 6.78 cm. If it is known that ultimate settlement will be about 25cm, Estimate settlement in July 1991. Assume double drainage to occur.

July 1981 - July 1985 - 4 years.

$t_1 = 4$ yrs $= 6.78$ cm.

Degree of consolidation Ultimate settlement = 25cm

$$U = \frac{6.78}{25}$$

4 yrs, $U = 27.12\%$.

July 1981 - July 1991 - 10 yrs.

$$U_{10} = \frac{\rho_{10}}{25} \qquad \rho \text{ - Settlement}.$$

$$U_{10} = 0.04\,\rho_{10}$$

$$T_v = \frac{C_v t}{d^2}, \quad T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$\frac{\frac{\cancel{C_{v_1}} t_1}{\cancel{d_1^2}}}{\frac{\cancel{C_{v_2}} t_2}{\cancel{d_2^2}}} = \frac{\cancel{\frac{\pi}{4}}\left(\frac{U_1}{\cancel{100}}\right)^2}{\cancel{\frac{\pi}{4}}\left(\frac{U_2}{\cancel{100}}\right)^2}$$

Settlement in July 1991 = 10.72 cm

$$\frac{t_1}{t_2} = \left(\frac{U_1}{U_2}\right)^2$$

$$\frac{4}{10} = \left(\frac{0.2712}{0.04\,\rho_{10}}\right)^2$$

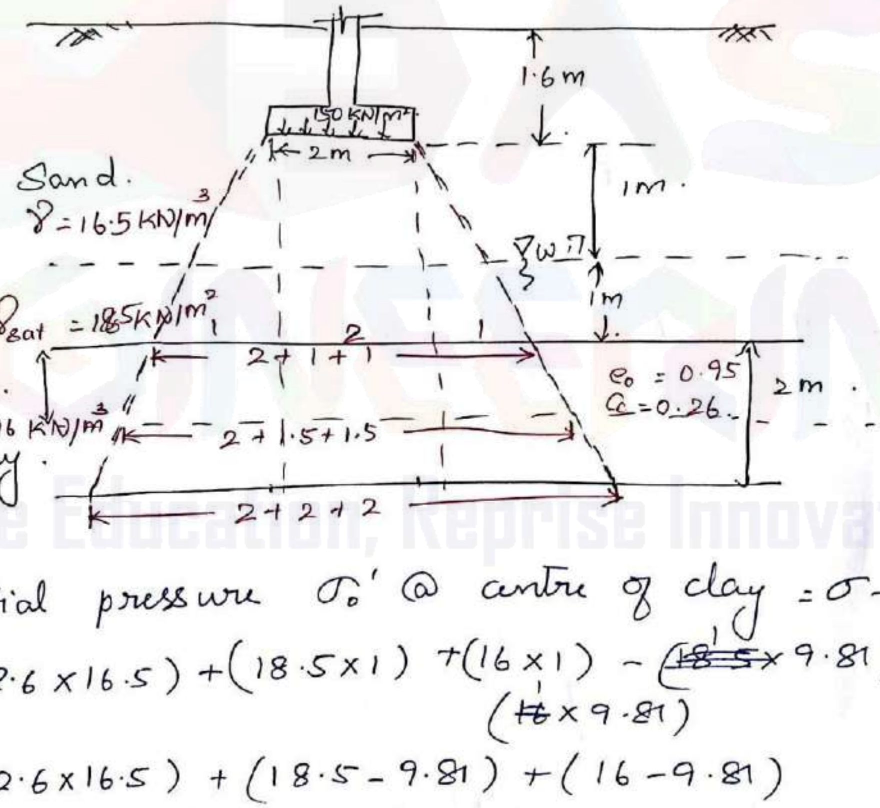
$$\rho_{10}^2 = \left(\frac{0.2712}{0.04}\right)^2 \times \frac{10}{4}$$

$$\boxed{\rho_{10} = 10.72\text{ cm}}$$

5. A building column has a footing area of 2m × 3m & transmits a pressure increment of 150 KN/m² at its base embedded 1.6 m below ground level. Assuming pressure distribution of 2 vertical to 1 horizontal, Determine consolidation settlement @ middle of clay layer. Consider pressure variation across thickness of clay layer also. Given the following. Q-3

i) For sand, $\gamma = 16.5\,KN/m^3$ & $\gamma_{sat} = 18.5\,KN/m^3$.

ii) For clay, $\gamma_{sat} = 16\,KN/m^3$, $e_0 = 0.95$, $C_c = 0.26$.



Initial pressure σ_0' @ centre of clay $= \sigma - u$

$= (2.6 \times 16.5) + (18.5 \times 1) + (16 \times 1) - (\cancel{18.5}\,1 \times 9.81) - (\cancel{16}\,1 \times 9.81)$

$= (2.6 \times 16.5) + (18.5 - 9.81) + (16 - 9.81)$

$= 57.78\ KN/m^2$.

$$\boxed{\sigma_0' = 57.78\ KN/m^2}$$

Pressure increase @ top, middle & bottom of clay layer.

Area = 2 x 3 m.

$$(\Delta \sigma)_t = \frac{150 \times 2 \times 3}{(2+2)\ (3+2)} = 45\ KN/m^2.$$

$$(\Delta \sigma)_m = \frac{150 \times 2 \times 3}{(2+3)(3+3)} = 30\ KN/m^2.$$

$$(\Delta \sigma)_b = \frac{150 \times 2 \times 3}{(2+4)(3+4)} = 21.43\ KN/m^2.$$

Average pressure is found by Sympson's rule:

$$\Delta \sigma = \frac{1}{6}\left[\Delta\sigma_t + 4\Delta\sigma_m + \Delta\sigma_b\right]$$

$$= \frac{1}{6}\left[45 + (4 \times 30) + 21.43\right]$$

$$= 31.07\ KN/m^2.$$

Settlement @ middle of clay layer:

$$\rho_f = \frac{C_c}{1+e_o} H \log_{10} \frac{\sigma_o' + \Delta\sigma}{\sigma_o'}$$

$$= \frac{0.26}{1+0.95} \times 2 \times \log_{10} \frac{57.78 + 31.07}{57.78}$$

$$= 0.0498\ m$$

$$\boxed{\rho_f = 49.8\ mm}.$$

——— x ———

Secondary Consolidation:

When excess pore pressure due to consolidation has been dissipated, the change in void ratio continues, but at reduced rate. This phenomenon is secondary consolidation. It is very small, so it is neglected.

——— x ——— x ———

UNIT - IV

SHEAR STRENGTH

Shear strength of cohesive and cohesion less soils - Mohr - Coulomb failure theory - Measurement of shear strength - Direct shear - Triaxial Compression, UCC & Vane shear tests - Pore pressure parameters Cyclic mobility - Liquefaction.

SHEAR STRENGTH:

When soil is loaded, shear stresses are induced in it. When shear stress reaches its limiting value, shear deformation takes place, leading to failure of the soil mass.

The failure may be in the form of

- Sinking of a footing
- Movement of a wedge of soil behind retaining wall forcing it to move out.
- Slide in Earth Embankment.

Definition: The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles upon the action of shear stress. The failure conditions for a soil may be expressed in terms of limiting shear stress, called shear strength.

The shearing resistance of soil is constituted basically of following components.

* Structural resistance to displacement of soil because of interlocking of particles.
* Frictional resistance to translocation b/w individual soil particles at their contact points.
* Cohesion or adhesion b/w surface of soil particles.

Shear strength of cohesionless soils:

The shear strength in cohesionless soil results from intergranular friction alone, while in all other soils it results from both internal friction as well as cohesion. However, plastic undrained clay doesnot possess internal friction.

MOHR'S STRESS CIRCLE:

Through a point in loaded soil mass, innumerous planes pass & stress components on each plane depends upon direction of plane.

On every plane, there will be three typical planes, mutually orthogonal to each other, on which stress is wholly normal & no shear stress acts. These planes are called principal planes & normal stress acting on these planes are called principal stresses.

The plane which is devoid of shear is principal plane.

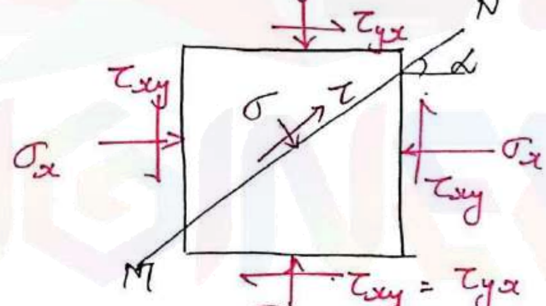
In the order of decreasing magnitude ②
of normal stress, these plane are called

* Major principal planes
* Intermediate Principal planes
* Minor principal planes.

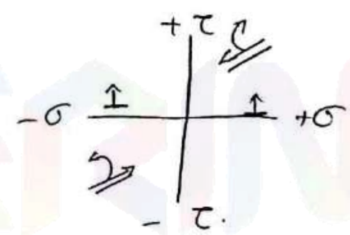
The corresponding normal stress on them are

* Major principal stress (σ_1)
* Intermediate principal stress (σ_2)
* Minor principal stress (σ_3).

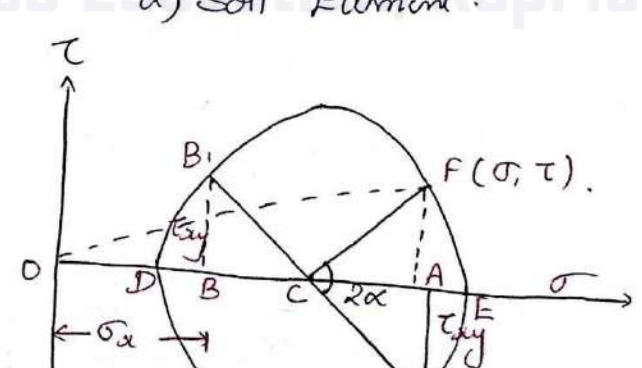
Many problems can be approximated by considering 2D stress conditions.



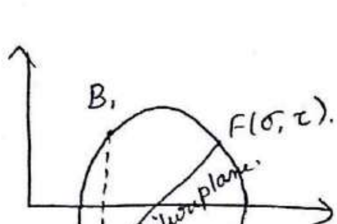
a) Soil Element.



b) Sign Convention



c) Mohr Circle.



P – Pole (or) Origin of plane.

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Figure (a) shows a soil element subjected to 2D stress system.

From equilibrium of element, the following expressions were found for normal stress σ & shearing stress τ on any plane MN inclined at α with x-direction.

$$\sigma = \frac{\sigma_x + \sigma_y}{2} + \left[\frac{\sigma_y - \sigma_x}{2}\right] \cos 2\alpha + \tau_{xy} \sin 2\alpha \quad \text{—(1)}$$

$$\tau = \frac{\sigma_y - \sigma_x}{2} \sin 2\alpha - \tau_{xy} \cos 2\alpha \quad \text{(2) when } (\sigma_y > \sigma_x)$$

σ_x - Normal stress on plane $\perp$ to x axis.

σ_y - Normal stress on plane $\perp$ to y-axis.

$\tau_{xy} = \tau_{yx}$ - Shear stress on these two planes

From (1),

$$\sigma - \frac{\sigma_y + \sigma_x}{2} = \left[\frac{\sigma_y - \sigma_x}{2}\right] \cos 2\alpha + \tau_{xy} \sin 2\alpha \quad \text{—(3)}$$

Squaring & adding (3) & (2).

$$\left[\sigma - \frac{\sigma_y + \sigma_x}{2}\right]^2 = \left[\frac{\sigma_y - \sigma_x}{2} \cos 2\alpha + \tau_{xy} \sin 2\alpha\right]^2$$

$$= \left[\frac{\sigma_y - \sigma_x}{2} \cos 2\alpha\right]^2 + \left[\tau_{xy} \sin 2\alpha\right]^2 + \cancel{2}\left[\frac{\sigma_y - \sigma_x}{\cancel{2}} \cos 2\alpha \cdot \tau_{xy} \sin 2\alpha\right]$$

$$\left(\sigma - \frac{\sigma_y + \sigma_x}{2}\right)^2 = \left(\frac{\sigma_y - \sigma_x}{2}\right)^2 \cos^2 2\alpha + \tau_{xy}^2 \sin^2 2\alpha + (\sigma_y - \sigma_x)\, \tau_{xy} \sin 2\alpha \cos 2\alpha.$$

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$$\tau^2 = \left[\frac{\sigma_y - \sigma_x}{2} \sin 2\alpha - \tau_{xy} \cos 2\alpha\right]^2 \qquad (3)$$

$$= \left(\frac{\sigma_y - \sigma_x}{2}\right)^2 \sin^2 2\alpha + \tau_{xy}^2 \cos^2 2\alpha -$$

$$\tau_{xy}(\sigma_y - \sigma_x) \sin 2\alpha \cos 2\alpha .$$

Adding .

$$\left[\sigma - \frac{\sigma_y + \sigma_x}{2}\right]^2 + \tau^2 = \left(\frac{\sigma_y - \sigma_x}{2}\right)^2 \left[\underbrace{\sin^2 2\alpha + \cos^2 2\alpha}_{1}\right] +$$

$$\tau_{xy}^2 \left[\underbrace{\sin^2 2\alpha + \cos^2 2\alpha}_{1}\right] +$$

$$(\sigma_y - \sigma_x)\, \tau_{xy} \cancel{\sin 2\alpha \cos 2\alpha} -$$

$$(\sigma_y - \sigma_x)\, \tau_{xy} \cancel{\sin 2\alpha \cos 2\alpha}$$

$$\sin^2\theta + \cos^2\theta = 1$$

$$\left[\sigma - \frac{\sigma_y + \sigma_x}{2}\right]^2 + \tau^2 = \left[\frac{\sigma_y - \sigma_x}{2}\right]^2 + \tau_{xy}^2 \quad \text{——} (4)$$

Equation 4 is the equation of circle like

$x^2 + y^2 = R^2$ $\qquad (x-a)^2 + (y-b)^2 = R^2$.

Coordinates (a,b) $= \left[\frac{\sigma_y + \sigma_x}{2}, 0\right]$

Radius $R^2 = \left[\frac{\sigma_y - \sigma_x}{2}\right]^2 + \tau_{xy}^2$

$$R = \sqrt{\left[\frac{\sigma_y - \sigma_x}{2}\right]^2 + \tau_{xy}^2}$$

Coordinates of points on circle represent normal & shearing stress on inclined planes at a given point. This circle is known as Mohr's circle of stress.

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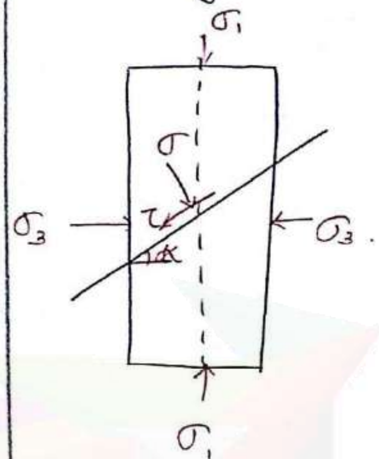
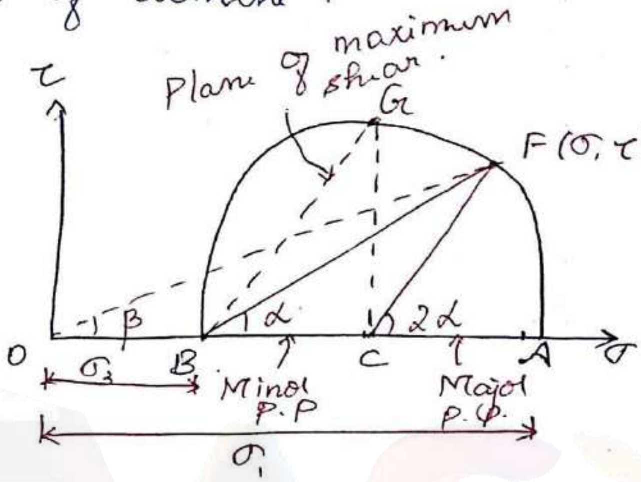
Let us take the case of soil element whose sides are principal planes - consider the state of stress where only normal stresses are acting on faces of element.





Expression for σ, τ are.

$$\boxed{\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha. \qquad \tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha.}$$

Resultant stress $= \sqrt{\sigma^2 + \tau^2}$

Angle of obliquity $\beta = \tan^{-1}\left(\frac{\tau}{\sigma}\right)$

Max. Shear stress

$$\boxed{\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}}$$

It occurs on plane $\alpha = 45°$.

At max. shear stress,

Normal stress $= \dfrac{\sigma_1 + \sigma_3}{2}$.

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MOHR - COULOMB FAILURE THEORY : (4)

Essential points of Mohr's strength theory are:

* Material fails essentially by shear. The critical shear stress causing failure depends upon the properties of material as well as on normal stress on failure plane.

* The ultimate strength of material is determined by stresses on potential failure plane.

* When the material is subjected to 3D principal stress ($\sigma_1, \sigma_2, \sigma_3$) the intermediate principal stress does not have any influence on strength of material. i.e: the failure criterion is independent of intermediate principal stress.

The theory can be expressed algebraically by equation.

$$\boxed{\tau_f = S = F(\sigma)} \longleftarrow \text{Mohr.}$$

τ_f = s = Shear stress on failure plane @ failure
= Shear resistance of material.

$F(\sigma)$ = Function of normal stress.

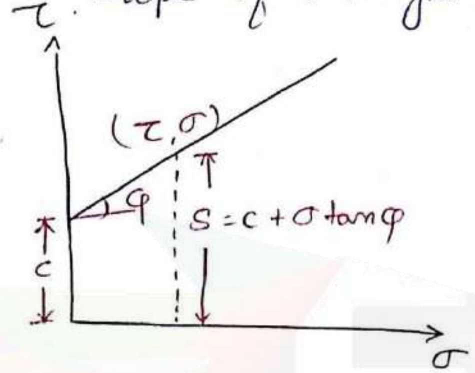
If normal stress & shear stress corresponding to failure are plotted, then a curve is obtained. That curve is called strength envelope.

Coulomb defined the function $F(\sigma)$ as a linear function of σ & gave strength equation : $\boxed{S = c + \sigma \tan\varphi}$ ← Coulomb.

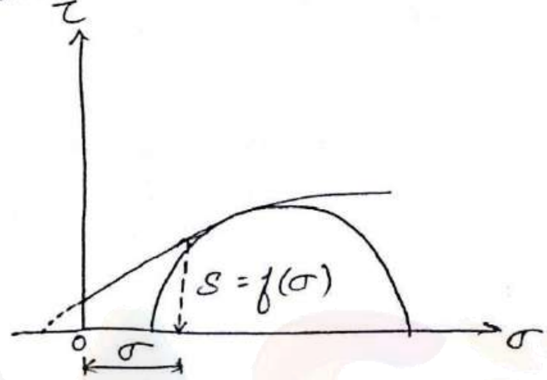
c & φ - Empirical constants.

c - cohesion - Intercept on shear axis.

φ - Angle of internal friction/shearing resistance - Slope of straight line.



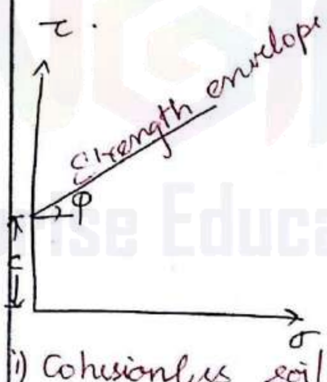
a) Coulomb Envelope (Straight line)



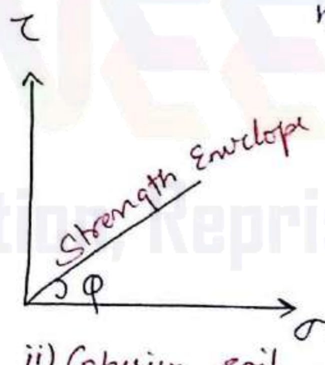
b) Mohr's Envelope. (Curve).

Shear strength $\propto$ Normal stress

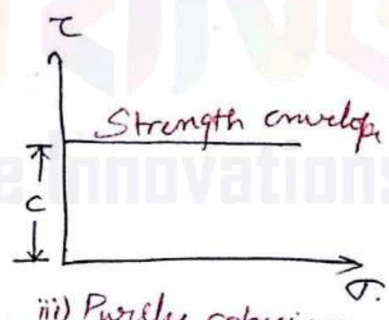
Shear stress & normal stress relation is not linear.



i) Cohesionless soil



ii) Cohesive soil (Pure friction).



iii) Purely cohesive soil.

EFFECTIVE STRESS PRINCIPLE :

In equation $S = c + \sigma \tan\varphi$, it is assumed that total normal stress governs shear strength of soil. This assumption is not always correct.

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Extensive tests on remoulded clays have (c)
sustained beyond doubt Terzaghi's early concept that effective normal stresses control shearing resistance of soils.

$$\tau_f = c' + \sigma' \tan\varphi'$$

$$\boxed{\tau_f = c' + (\sigma - u)\tan\varphi'}$$

c' - Effective cohesion intercept.
φ' - Effective angle of shearing resistance.
In terms of total stresses,

$$\boxed{\tau_f = c_u + \sigma \tan\varphi_u}.$$

c_u - Apparent cohesion
φ_u - Apparent angle of shearing resistance.

Normal stress σ' & shear stress τ on any plane inclined @ an angle α to major principal plane can be expressed by

$$\sigma' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2}\cos 2\alpha.$$

$$\tau = \frac{\sigma_1' - \sigma_3'}{2}\sin 2\alpha.$$

σ_1' - Effective Major principal stress.
σ_3' - Effective minor principal stress.
Substituting values of σ'

$$\tau_f = c' + \tan\varphi'\left[\frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2}\cos 2\alpha\right].$$

Most dangerous plane - Failure occurs - $(\tau_f - \tau)$ b/w shear strength & shear stress is minimum.

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$$\tau_f - \tau = c' + \frac{\sigma_1' + \sigma_3'}{2}\tan\varphi' + \frac{\sigma_1' - \sigma_3'}{2}\cos 2\alpha \tan\varphi' - \frac{\sigma_1' - \sigma_3'}{2}\sin 2\alpha .$$

Differentiate w.r. to α,

$$\frac{d}{d\alpha}(\tau_f - \tau) = \frac{\sigma_1' - \sigma_3'}{\cancel{2}}\tan\varphi' \left[-\cancel{2}\sin 2\alpha\right] - \frac{\sigma_1' - \sigma_3'}{\cancel{2}}\left[\cancel{2}\cos 2\alpha\right].$$

$$= -(\sigma_1' - \sigma_3')\tan\varphi' \sin 2\alpha - (\sigma_1' - \sigma_3')\cos 2\alpha .$$

For minimum, $(\tau_f - \tau)$,

$$\frac{d}{d\alpha}(\tau_f - \tau) = 0 .$$

$$\cancel{(\sigma_1' - \sigma_3')}\cos 2\alpha = -\cancel{(\sigma_1' - \sigma_3')}\tan\varphi' \sin 2\alpha .$$

$$\tan\varphi = -\frac{\cos 2\alpha}{\sin 2\alpha}.$$

$$-\tan\varphi = \cot 2\alpha .$$

$$\cot(90^\circ + \varphi) = \cot 2\alpha .$$

$$2\alpha = 90^\circ + \varphi$$

$$\alpha = \frac{90^\circ + \varphi}{2}$$

$$\boxed{\alpha_f = \alpha = 45^\circ + \frac{\varphi}{2}}$$

It can also be derived from Mohr's circle.

MEASUREMENT OF SHEAR STRENGTH: ⑥

The measurement of shear strength of soil involves certain test observations at failure, with the help of which failure envelope or strength envelope can be plotted.

Laboratory tests :

1. Direct shear test.
2. Triaxial shear test
3. Unconfined Compression test
4. Vane Shear test.

Depending on drainage conditions, 3 types of shear tests.

* Undrained test (or) quick test
* Consolidated drained test.
* Drained Test.

Undrained test :

No drainage of water is permitted. There is no dissipation of pore pressure during entire test

Direct Shear test - Drainage not permitted during application of both normal stress & shear stress.

Triaxial compression test : Drainage not permitted during period of both pore pressure & deviated pressure.

Drained test :

Drainage is permitted throughout the test during application of both normal & shear stresses so that full consolidation occurs & no excess pore pressure is set up at any stage of test.

Consolidated undrained test

Drainage is permitted under initially applied normal stress only & full primary consolidation or softening is allowed to take place. No drainage is allowed afterwards.

C & φ - vary with drainage conditions.

Direct Shear test - Allowed to consolidate fully under applied normal stress & shear for high rate of strain to prevent dissipation of pore pressure during shearing.

Triaxial compression test : Allowed to consolidate fully under applied cell pressure & then pore water outlet is closed & specimen subjected to increasing deviator stress at high rate of strain.

———— × ————

1. DIRECT SHEAR TEST :

- Simple & commonly used test.

Apparatus: Shear box Apparatus.

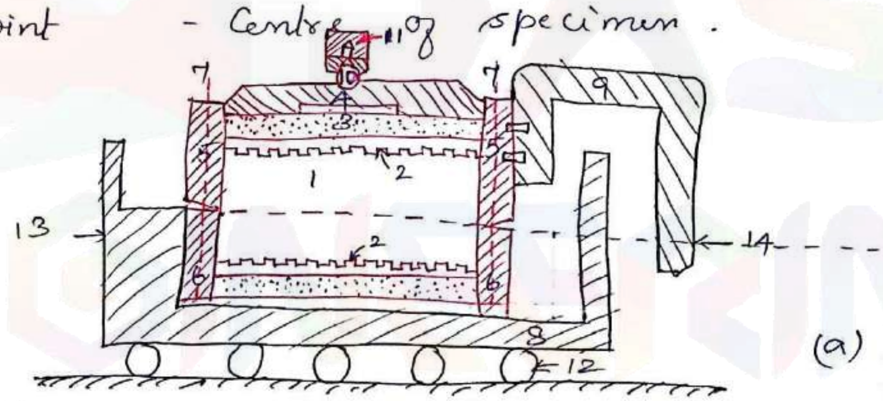
The apparatus consists of two piece shear box of square or circular cross-section.

The lower half of the box is rigidly held in position in a container which rests over slides or rollers and can be pushed forward at a constant rate by geared jack, driven either by electric motor or by hand.

The upper half of the box butts against a proving ring.

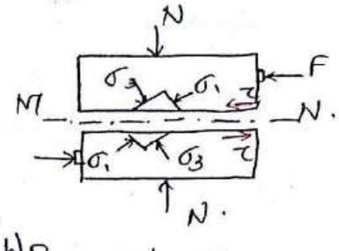
The soil sample is compacted in the shear box & is held b/w metal grids and porous stones.

Upper box - Upper half of specimen
Lower box - Lower half of specimen
Joint - Centre of specimen.

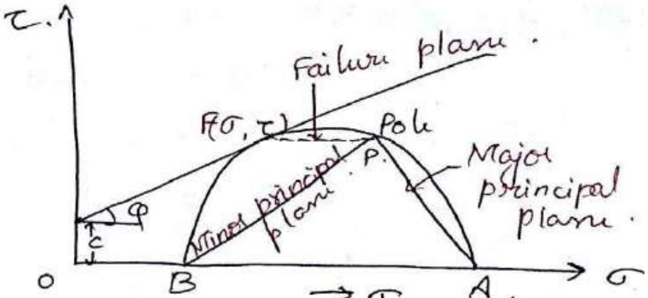


(a)

1. Soil specimen
2. Metal grids
3. Porous stones
4. Loading pad
5. Upper part
6. Lower part
7. Screws to fix two halves of shear box
8. Container for shear box
9. U-Arm
10. Steel ball
11. Loading yoke
12. Rollers
13. Shear force applied by jack
14. Shear resistance Measured by proving ring.



b) Principle of direct shear box.



c) Mohr's envelope.

Procedure:

Normal pressure is applied on the specimen from loading yoke bearing upon steel ball of pressure pad.

When shearing force is applied to lower box through geared jack, the movement of lower part of box is transmitted through specimen to upper box ~~of~~ and hence on proving ring.

Deformation of proving ring indicates shear force.

The volume change during consolidation & during shearing process is measured by mounting a dial guage at top of box.

The soil specimen can be compacted in shear box by clamping both the parts together with the help of two screws.

These screws are removed before shearing force is applied.

Metal grids placed above top & below bottom of specimen may be <u>perforated</u> if <u>drained</u> test is required or <u>plain</u> if <u>undrained</u> test is required.

Serrations in grids are $\perp$ to direction of shear force.

* Strain controlled test
* Stress controlled test.

Strain controlled Test:

Shear strain is made to increase at constant rate. Fig(b) shows strain controlled shear test. Fig(c) shows failure envelope plotted as a function of shear stress & normal stress.

Stress controlled test:

There is an arrangement to increase the shear stress at a desired rate and measure the shearing strain.

Test can be performed under all three conditions of drainage.

* Undrained test - Plain grids are used.
* Drained test (Slow test) - Perforated grids are used. Consolidated under normal load & then sheared slowly so that complete dissipation of pore pressure takes place. (2-5 days).

Consolidated undrained test - Perforated grids are used. Consolidated under normal load & sheared quickly in about 5-10 minutes.

Advantages:

* Simple test.
* The relatively thin thickness of sample permits quick drainage & quick dissipation of pore pressure developed during test.

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Disadvantages:

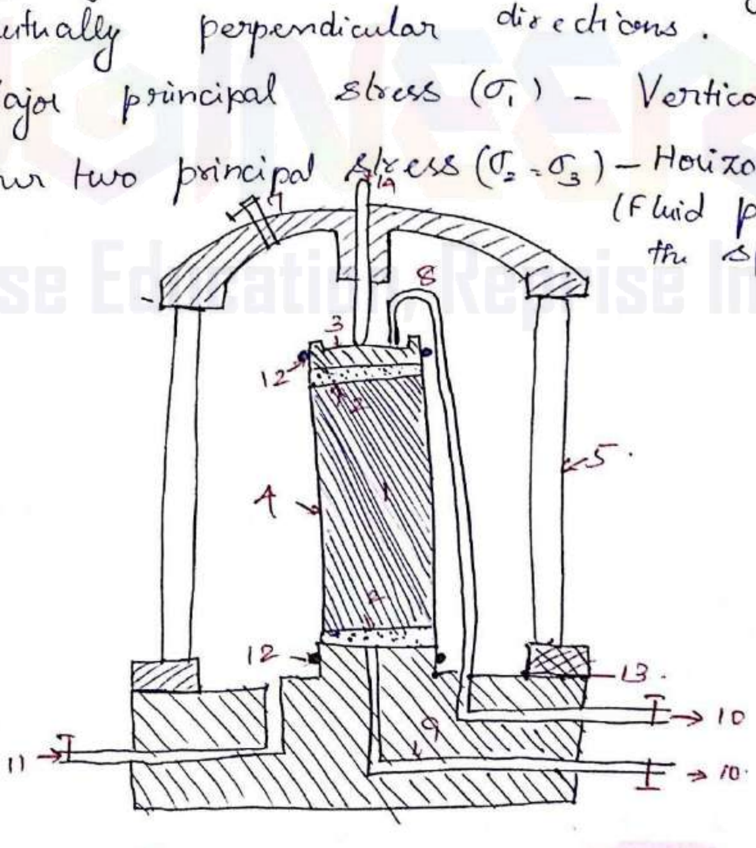
* Stress conditions across soil sample are very complex.
* As test progresses, the area under shear gradually decreases.
* As compared to triaxial test, there is little control on drainage of soil.
* The plane of shear failure is predetermined, which may not be weakest one.
* There is effect of lateral restraint by side walls of shear box.

TRIAXIAL COMPRESSION TEST:

The solid specimen, cylindrical in shape, is subjected to direct stresses acting in three mutually perpendicular directions.

Major principal stress (σ_1) - Vertical direction.

Other two principal stress ($\sigma_2 = \sigma_3$) - Horizontal direction. (Fluid pressure round the specimen)



(9)

1. Soil specimen
2. Porous disc
3. Top cap
4. Rubber membrane
5. Perspex cylinder
6. Loading ram
7. Air Release Valve
8. Top Drainage Tube
9. Bottom Drainage Tube
10. Connections for drainage (or) Pore pressure measurement.
11. Cell fluid inlet
12. Rubber rings
13. Sealing ring
14. Axial load through proving ring.

Apparatus :

- High pressure cylindrical cell - Perspex or other transparent material - fitted b/w base & top cap.

- 3 outlet connections are provided through base :
  * Cell fluid inlet
  * Pore water outlet from bottom of specimen
  * Drainage outlet from top of specimen.

- A seperate compressor is used to apply fluid pressure in the cell.

- Pore pressure developed in specimen during test can be measured with help of seperate pore pressure measuring equipment, such as Bishop's apparatus.

- A stainless steel piston running through centre of top cap applies the vertical compressive load (deviator stress) on specimen under test.

- The load is applied through a proving ring, with the help of mechanically operated load frame.

- Depending on drainage conditions of test, solid nonporous or porous discs are placed on top & bottom & rubber membrane is sealed on to the end caps

Length of specimen = 2 to $2\frac{1}{2}$ times its diameter.

Cell pressure σ_3 ($=\sigma_2$) acts all round specimen; it also acts on top of specimen as well as vertical piston meant for among deviator stress.

Vertical stress $= (\sigma_1 - \sigma_3)$

Total stress on top $= (\sigma_1 - \sigma_3) + \sigma_3$

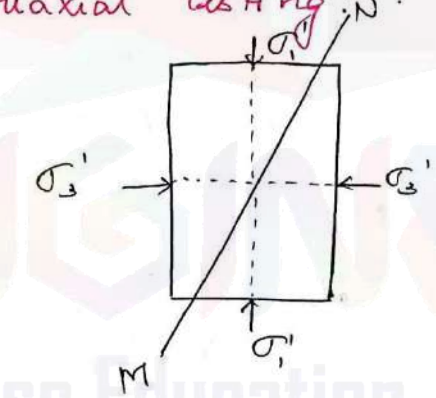
$= \sigma_1$ – Major principal stress

Stress difference $(\sigma_1 - \sigma_3)$ – Deviator stress.

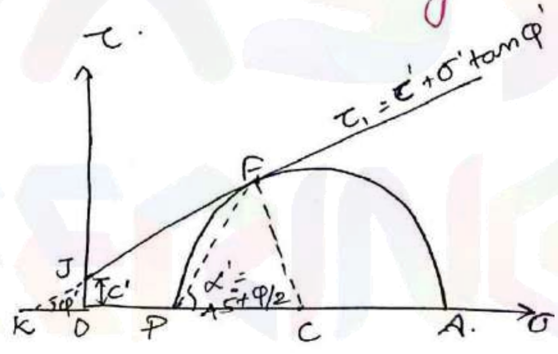
Record on proving ring dial

Another dial – Vertical deformation.

Stress condition in soil specimen during triaxial testing :



a) Stress conditions



b) Failure Envelope in Triaxial Compression tests.

Minor principal stress = Intermediate principal stress.

Effective Minor principal stress = Cell pressure (–) Pore pressure.

Major Principal stress = Deviator stress + Cell pressure

Failure plane inclined at an angle α' to major principal plane

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$$\alpha' = 45^\circ + \phi'/2$$

FC - Radius of Mohr's circle $= \frac{1}{2}(\sigma_1' - \sigma_3')$.

$$OC = \frac{1}{2}(\sigma_1' + \sigma_3')$$

$$OK = c' \cot \phi'$$

$$\sin\phi' = \frac{FC}{KC} = \frac{FC}{KO + OC}$$

$$= \frac{\frac{\sigma_1' - \sigma_3'}{2}}{c'\cot\phi' + \frac{\sigma_1' + \sigma_3'}{2}}$$

$$\sin\phi' = \frac{\sigma_1' - \sigma_3'}{2c'\cot\phi' + (\sigma_1' + \sigma_3')}$$

$$(\sigma_1' - \sigma_3') = 2c'\cot\phi'\sin\phi' + (\sigma_1' + \sigma_3')\sin\phi'$$

$$= 2c'\frac{\cos\phi'}{\cancel{\sin\phi'}}\cancel{\sin\phi'} + (\sigma_1' + \sigma_3')\sin\phi'$$

$$\sigma_1' - \sigma_3' = 2c'\cos\phi' + (\sigma_1' + \sigma_3')\sin\phi'$$

$$\sigma_1' - \sigma_1'\sin\phi' = 2c'\cos\phi' + \sigma_3'\sin\phi' + \sigma_3'$$

$$\sigma_1'(1 - \sin\phi') = 2c'\cos\phi' + \sigma_3'(1 + \sin\phi')$$

$$\sigma_1' = 2c'\frac{\cos\phi'}{(1 - \sin\phi')} + \sigma_3'\frac{1 + \sin\phi'}{1 - \sin\phi'}$$

$$\sigma_1' = 2c'\tan\left(45^\circ + \frac{\phi'}{2}\right) + \sigma_3'\tan^2(45^\circ + \phi'/2)$$

$$\boxed{\sigma_1' = \sigma_3'\tan^2\alpha' + 2c'\tan\alpha'}$$

$$\sigma_1' = \sigma_3' N_\phi' + 2c'\sqrt{N_\phi'}$$

$$\boxed{N_\phi' = \tan^2\alpha' = \tan^2[45^\circ + \phi'/2]}$$

In terms of total stress,

$$\boxed{\sigma_1 = \sigma_3 \tan^2\alpha + 2c_u \tan\alpha.}$$

$$\boxed{\alpha = 45^\circ + \frac{\varphi_u}{2}}$$

The calculation of deviator stress must be done on basis of changed area of c/s at failure, or during any stage of test. The area A_2 at failure, or during any stage of test can be found by relation -

$$\boxed{A_2 = \frac{V_1 \pm \Delta V}{L_1 - \Delta L}}$$

V_1 - Initial volume of specimen

L_1 - Initial length of specimen

ΔV - change in volume of specimen

ΔL - change in length of specimen

Deviator stress σ_d

$$\sigma_d = \frac{\text{Additional axial load.}}{A_2.}$$

σ_3 = Fluid pressure

$\sigma_1 = \sigma_3 + \sigma_d$.

Advantages:

* Shear tests under all three drainage conditions can be performed with complete control.
* Precise measurement of pore pressure & volume change during test are possible
* Stress distribution on failure plane is uniform.
* State of stress within specimen during any stage of test, as well as @ failure is completely determinate.

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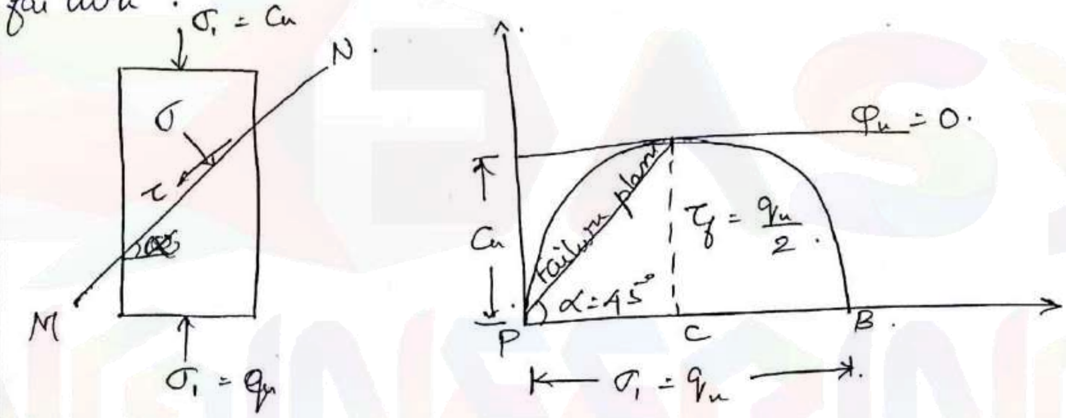
UNCONFINED COMPRESSION TEST

It is a special case of biaxial compression test in which $\sigma_2 = \sigma_3 = 0$.

Cell pressure = Confining pressure.

Absence of confining pressure - Uniaxial test - Unconfined compression test.

Cylindrical specimen of soil is subjected to major principal stress σ_1 till specimen fails due to shearing along a critical plane or failure.



Apparatus:

* Small load frame fitted with proving ring to measure vertical stress applied to soil specimen.
* Deformation is measured with dial gauge.

$\sigma_3 = 0$, Mohr's circle passes through origin which is also a pole.

$$\boxed{\sigma_1 = 2C_u \tan\alpha = 2C_u \tan\left(45^\circ + \frac{\varphi_u}{2}\right)}$$

Unknowns - C_u & φ_u - can't be found by unconfined test.

It gives the value of σ_1.

This test is generally applicable to saturated clays for which the apparent angle of shearing resistance φ_u is zero.

$$\sigma_1 = 2C_u$$

$$\boxed{\sigma_1 = 2C_u}$$

When Mohr's circle is drawn, its radius is equal to $\sigma_1/2 = C_u$.

$$\boxed{\sigma = \frac{\sigma_1}{2} = \frac{q_u}{2}} \qquad \tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = C_u.$$

$$\boxed{\tau_f = C_u = \frac{q_u}{2}}$$

q_u = Unconfined compressive strength @ failure.
Compressive stress is calculated on the basis of changed C/S area A_2 at failure.

$$A_2 = \frac{V}{L_1 - \Delta L} = \boxed{\frac{A_1}{1 - \frac{\Delta L}{L_1}} = A_2}$$

V - Initial volume of specimen
L_1 - Initial length of specimen
ΔL - Change in length at failure.

VANE SHEAR TEST (Quick test).

- Used in laboratory & in field.
- To determine undrained shear strength of cohesive soil.
- The tester consists of 4 thin steel plates, called vanes, welded orthogonally to steel rod.
- A torque measuring arrangement, such as

a calibrated torsion spring, is attached to (12)
the rod which is rotated by worm gear
and worm wheel arrangement.

After pushing the vanes gently into soil, the torque rod is rotated at uniform speed

The rotation of spring in degrees is indicated by a pointer moving on graduated dial attached to worm wheel shaft.

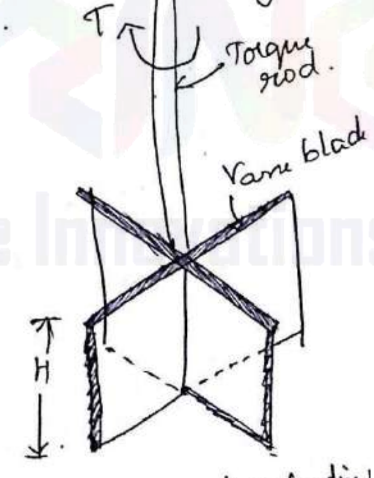
The torque T is then calculated by multiplying dial reading with spring constant.

A typical laboratory vane is 20mm high & 12mm in diameter with blade thickness from 0.5 to 1mm.

Blades - High tensile steel.

Field shear vane - 10 to 20cm in height.

Blade thickness - 2.5m.



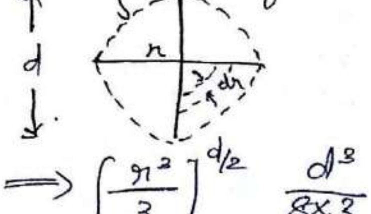
τ_f - Unit strength of soil

H - Height of vane

d - diameter of vane.

Case I:

Vane is pushed in soil with its top end below surface of soil so that both top & bottom ends partake in shearing of soil.



$$T_f = \pi d H \tau_f \frac{d}{2} + 2\int_0^{d/2} (2\pi r dr)\tau_f r.$$

$$= \pi d H \tau_f \frac{d}{2} + 4\pi \tau_f \int_0^{d/2} r^2 dr \implies \left[\frac{r^3}{3}\right]_0^{d/2} \quad \frac{d^3}{8\times 3}$$

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$$T = \pi d H \tau_f \frac{d}{2} + \cancel{4}\pi \tau_f \frac{d^3}{\cancel{8}_2 \times 3}$$

$$= \pi \tau_f \left[\frac{d^2 H}{2} + \frac{d^3}{6}\right]$$

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

Case II :

The vane is pushed inside the soil with its top end flushed with surface of soil so that only bottom end partakes in shearing the soil.

$$T_f = (\pi d H \tau_f)\, d/2 + \int_0^{d/2} 2\pi r\, dr\, \tau_f\, r.$$

$$= \frac{\pi d^2}{2} H \tau_f + \cancel{2}\pi \tau_f \frac{d^3}{\cancel{8}_4 \times 3}$$

$$= \frac{\pi d^2}{2} H \tau_f + \pi \tau_f \frac{d^3}{12}$$

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

Knowing T_f, H & d, Shear strength τ_f can be determined.

PROBLEMS:

1. A direct shear test was carried out on cohesive soil sample under the following results are obtained.

Normal Stress (kN/m^2)	Shear stress (kN/m^2)
150	110
250	120

What would be deviated stress at failure (13)
if the triaxial shear test was carried on
the same soil with a cell pressure of
150 KN/m^2.

Coulomb's shear strength equation is
given by

$$\tau_f = c + \sigma \tan\varphi$$

$$110 = c + 150 \tan\varphi$$

$$120 = c + 250 \tan\varphi$$

Solving, $\boxed{c = 95 \ KN/m^2}$

$$\tan\varphi = 0.1$$

$$\boxed{\varphi = 5.71^\circ}$$

Deviated stress at failure,

$$\sigma_d = \sigma_1 - \sigma_3$$

$$\sigma_1 = \sigma_3 \tan^2(45 + \varphi/2) + 2c \tan(45 + \varphi/2)$$

$$= 150 \tan^2(45 + 5.71/2) + 2 \times 95 \tan\left(45 + \frac{5.71}{2}\right)$$

$$\sigma_1 = 183.146 + 209.95$$

$$\boxed{\sigma_1 = 393.09 \ KN/m^2}$$

$$\sigma_d = \sigma_1 - \sigma_3$$

$$= 393.09 - 150$$

$$\boxed{\sigma_d = 243.1 \ KN/m^2}$$

---

2. A consolidated undrained test was conducted on a dry sample on following results were obtained.
Determine the shear strength parameters w.r.to i) Total stress concept ii) Effective stress concept

Cell pressure (kN/m^2)	Deviator stress at failure (kN/m^2)	Pore water pressure at failure (kN/m^2)
200	118	110
400	240	220
600	352	320.

Solution :

σ_3 (kN/m^2)	σ_d (kN/m^2)	u (kN/m^2)	$\sigma_1 = \sigma_3 + \sigma_d$ (kN/m^2)	$\sigma_1' = \sigma_1 - u$ (kN/m^2)	$\sigma_3' = \sigma_3 - u$ (kN/m^2)
200	118	110	318	208	90
400	240	220	640	420	180
600	352	320	952	632	280.

$$\sigma_1 = \sigma_3 \tan^2(45° + \varphi/2) + 2c \tan(45 + \varphi/2)$$

$$318 = 200 \tan^2\alpha + 2c \tan\alpha$$

$$640 = 400 \tan^2\alpha + 2c \tan\alpha.$$

$$322 = 200 \tan^2\alpha$$

$$\tan^2\alpha = 1.61$$

$$\boxed{\alpha = 51.76°}$$

$$45 + \varphi/2 = 51.76$$

$$\varphi/2 = 6.76$$

$$\boxed{\varphi = 13.52°}$$

$$318 = (200 \times 1.61) + 2c \tan 51.76$$

$$\boxed{c = -1.5}$$

$\sigma_1' = \sigma_3' \tan^2(45+\varphi/2) + 2C' \tan(45+\varphi/2)$ (1A)

$208 = 90 \tan^2\alpha' + 2c' \tan\alpha'$

$420 = 180 \tan^2\alpha' + 2c' \tan\alpha'$

$212 = 90 \tan^2\alpha'$

$\boxed{\alpha' = 56.91^\circ}$

$45 + \varphi/2 = 56.91$

$\boxed{\varphi' = 23.83^\circ}$

$208 = 90 \times \tan^2(56.91) + 2c' \tan(56.91)$

$\boxed{C' = -1.3}$

3. An unconfined compression test was conducted on an undisturbed sample of clay. The sample has a diameter of 38mm & length 76mm. Load at failure was 30N. & axial deformation of sample is 11mm. Determine the undrained shear strength parameter if failure plane makes an angle of 50° with horizontal.

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Initial length of sample = 76mm.

Diameter of sample = 38 mm.

Initial area of C/S.

$A = \frac{\pi d^2}{4} = \frac{\pi \times 38^2}{4} = 1134.11 \, mm^2$

change in length $\Delta L = 11mm$.

Axial strain at failure $= \frac{\Delta L}{L} = \frac{11}{76} = 0.145$.

Area of c/s at failure.

$$A_f = \frac{A}{1-\epsilon} = \frac{1134.11}{1-0.145} = 1326.44\ mm^2.$$

$$q_u = \frac{P}{A_f} = \frac{30}{1326.44} = 0.023\ N/mm^2.$$

$$\boxed{q_u = 0.023\ N/mm^2}$$

$$\alpha = 50^\circ$$

$$45 + \phi/2 = 50^\circ \qquad \sigma_3 = 0$$

$$\phi = 10^\circ \rightarrow 0$$

$$\sigma_1 = \cancel{\sigma_3 \tan^2\left(45 + \frac{\phi_u}{2}\right)} + 2c \tan\left(45 + \frac{\phi_u}{2}\right).$$

$$\sigma_1 = q_u = 2c \tan(45^\circ + \phi/2)$$

$$2c = \frac{0.023}{\tan 50}$$

$$2c = 0.0193$$

$$\boxed{c = 0.0096\ N/mm^2}$$

A. An unconfined compression test is conducted on a saturated clay specimen of 40mm diameter & 90mm length measured on its sides. The specimen has cored edge & its length b/w apex of cone is 80mm. The specimen fails under an axial compression load of 460N with axial deformation of 10mm. Calculate the unconfined compressive strength of clay.

Length of sample on its sides = 90mm.
Length b/w apex of cone = 80 mm.
Diameter of sample = 40mm.

$$\text{Actual Length} = 90 - \frac{2 \times 5}{3} = 86.67\ mm. \qquad \text{Area} = \frac{\pi d^2}{4} = 1256.64\ mm^2$$

$\Delta L = 10.$

$$\text{Strain} = \frac{\Delta L}{L} = \frac{10}{86.67} = 0.1154$$

$$A_f = \frac{A}{1-\varepsilon} = \frac{1256.64}{1-0.1154}$$

$$A_f = 1420.5\,mm^2.$$

$P = 460\,KN.$

$$q_u = \frac{P}{A_f} = 0.324\,N/mm^2$$

$$= \frac{0.324 \times 10^6}{10^3}\,KN/m^2$$

$$= 324\,KN/m^2.$$

$$C_u = \frac{q_u}{2} = \frac{324}{2}$$

$$\boxed{C_u = 162\,KN/m^2}$$

5. In a vane shear test conducted in a soft clay deposit, failure occured at a torque of 42N m. Afterwards, the vane was allowed to rotate rapidly and the test was repeated in the rebounded soil. The torque at failure in the remoulded soil was 17N m. Calculate the sensitivity of soil in both cases. In both cases, the vane was pushed completely inside the soil. The height & diameter of vane was 100mm & 80mm respectively.

$$\text{Sensitivity of clay} = \frac{\text{cohesion in undisturbed state}}{\text{cohesion in remoulded state}}.$$

$$S = \frac{C_{undisturbed}}{C_{remoulded}} \qquad S = \frac{q_u/2\,(u)}{q_u/2\,(r)}$$

Height of vane $(H) = 100mm$.
Diameter $(d) = 80mm$.

$T_f = 17Nm = 17000 Nmm$.

In this problem, both top & bottom ends particle in shearing the soil.

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

Case - I

For natural soil,

$T = 42Nm = 42000 Nmm$.

$$42000 = \pi \times 80^2 \times \tau_f \left[\frac{100}{2} + \frac{80}{6}\right].$$

$$\boxed{\tau_f = 0.033 N/mm^2.}$$

Case - II

For remoulded state.

$$17000 = \pi \times 80^2 \times \tau_f \left[\frac{100}{2} + \frac{80}{6}\right]$$

$$\boxed{\tau_f = 0.0134 N/mm^2}$$

$$\text{Sensitivity} = \frac{0.033}{0.0134} = 2.54. \qquad \boxed{S = 2.47}$$

Hyxotrophic Effect:

If the soil is disturbed, its shear strength decreases. If we leave the soil, it regain its shear strength after sometimes. This is known as hyxotrophic effect.

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SKEMPTON'S PORE PRESSURE PARAMETERS (16)

The change in the pore pressure due to change in the applied stress, during an undrained shear, may be explained in terms of empirical coefficients called pore pressure parameters.

A pore pressure parameter may be defined as a dimensionless number that indicates the fraction of total stress increment that show up an excess pore pressure for condition of no drainage.

Let us consider a soil mass subjected to increase in 3 principal stresses, $\Delta\sigma_1$, $\Delta\sigma_2$ & $\Delta\sigma_3$. resulting in volume decrease ΔV and a consequent increase in pore pressure of Δu.

The increase in effective stresses

$$\Delta\sigma_1' = \Delta\sigma_1 - u$$
$$\Delta\sigma_2' = \Delta\sigma_2 - u$$
$$\Delta\sigma_3' = \Delta\sigma_3 - u.$$

$\varepsilon_1, \varepsilon_2, \varepsilon_3$ - Strains in 3 directions.

Youngs modulus $E = \frac{\text{Stress}}{\text{Strain}}$.

$$E\,\varepsilon_1 = \Delta\sigma_1' - \mu\,(\Delta\sigma_2' + \Delta\sigma_3')$$
$$E\,\varepsilon_2 = \Delta\sigma_2' - \mu\,(\Delta\sigma_3' + \Delta\sigma_1')$$
$$E\,\varepsilon_3 = \Delta\sigma_3' - \mu\,(\Delta\sigma_1' + \Delta\sigma_2').$$

$$E(\varepsilon_1 + \varepsilon_2 + \varepsilon_3) = E\cdot\varepsilon_v = E\,\frac{\Delta V}{V}$$

$$= \Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3' - \mu\,[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_2' + \Delta\sigma_3' + \Delta\sigma_3' + \Delta\sigma_1']$$

$$= \Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3' - 2\mu\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right]$$

$$= (1-2\mu)\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right].$$

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

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

Multiply & divide by 3.

$$\frac{\Delta V}{V} = \frac{3(1-2\mu)}{E}\cdot\frac{1}{3}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right].$$

$\frac{3(1-2\mu)}{E} = C_c$ = Compressibility of soil skeleton

Substituting values of effective stress in terms of total stresses. $\Delta\sigma_1' = \Delta\sigma - u$.

$$\frac{\Delta V}{V} = \frac{C_c}{3}\left[\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3 - 3\Delta u\right]$$

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

n - Porosity

Volume of voids $= nV$.

If the pore fluid is assumed to show a linear relationship b/w volume change & stress, and its coefficient of volume compressibility is represented by C_v, the change in volume of pore fluid ΔV_w due to increase in pore pressure Δu under the condition of no drainage is given by

$$\Delta V_w = nVC_v\Delta u.$$

The decrease in volume of soil skeleton (17) is almost entirely due to decrease in volume of voids.

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

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

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

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

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

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

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

In a conventional triaxial test,

$$\Delta\sigma_2 = \Delta\sigma_3$$

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

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

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

The equation can be written in the form

$$\boxed{\Delta u = B [\Delta\sigma_3 + A (\Delta\sigma_1 - \Delta\sigma_3)]} \leftarrow (2)$$

A & B - Skempton's pore pressure parameters

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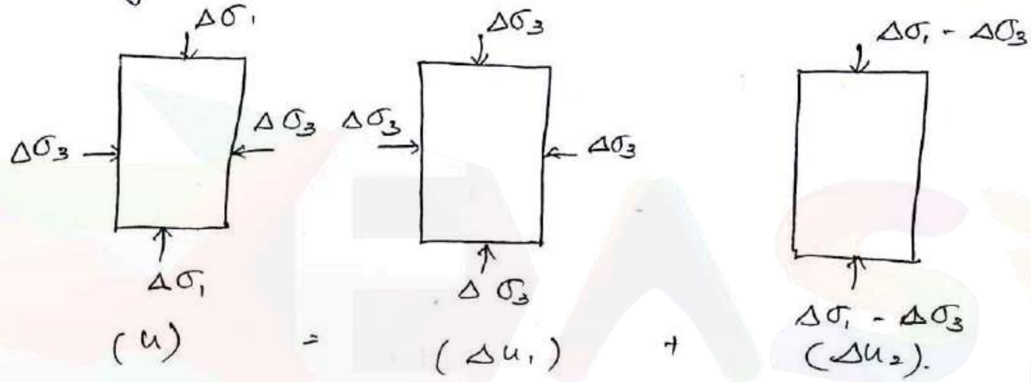
These paramdtors are to be determined experimentally.

In an undrained triaxial test, stress changes are usually made in two stages.

i) an increase in cell pressure $\Delta\sigma_3$ resulting in an all round change in stress.

ii) an increase in axial load resulting in change in deviator stress $\Delta\sigma_d = (\Delta\sigma_1 - \sigma_3)$



Let

Δu_1 - Change in pore pressure during first stage of test when cell pressure is applied.

Δu_2 - change in pore pressure when deviator stress is applied.

$$\Delta u = \Delta u_1 + \Delta u_2 \quad \text{—— (3)}.$$

Comparing (2) & (3).

$$\Delta u_1 + \Delta u_2 = B\,\Delta\sigma_3 + AB\,(\Delta\sigma_1 - \Delta\sigma_3)$$

$$\Delta u_1 = B\,\Delta\sigma_3 \qquad \Delta u_2 = AB(\Delta\sigma_1 - \Delta\sigma_3)$$

$$= \bar{A}\,(\Delta\sigma_1 - \Delta\sigma_3)$$

$$\boxed{B = \frac{\Delta u_1}{\Delta\sigma_3}} \qquad \boxed{\bar{A} = \frac{\Delta u_2}{(\Delta\sigma_1 - \Delta\sigma_3)}}$$

From equation (1) & (2), $B = \dfrac{1}{1 + \dfrac{n\,C_v}{C_c}}$

Factors affecting A & B

Fully Saturated Soil - $C_v <<< C_c$

$\frac{C_v}{C_c} = 1 \quad \therefore B = 1.$

Perfectly dry soil - $\frac{C_v}{C_c} = \infty, \quad \therefore B = 0.$

Partially saturated soils $= 0 < B < 1.$

Proctor's optimum water content & density,

$B = 0.1$ to 0.5

The coefficient A varies with stresses & strains. It depends on whether total stresses are increasing or decreasing.

Preconsolidation reduces A.

Other factors affecting A - Type of shear
Sample disturbance
Environment (Temp & nature of fluid)

Determination of parameters A & B:

B is determined in lab by measuring change in pore pressure Δu_1 due to change in cell pressure $\Delta\sigma_3$, in first part of test.

$$\boxed{B = \frac{\Delta u_1}{\Delta\sigma_3}}.$$

$\bar{A}$ is measured during second stage of test when deviator stress $(\Delta\sigma_1 - \Delta\sigma_3)$ is applied at constant cell pressure, when Δu_2 is measured.

$$\bar{A} = A \cdot B = \frac{\Delta u_2}{\Delta\sigma_1 - \Delta\sigma_3}.$$

$$\boxed{A = \frac{\Delta u - \Delta \sigma_3}{\Delta \sigma_1 - \Delta \sigma_3}}$$

For usual undrained triaxial test, $\Delta \sigma_3 = 0$, when deviator stress is applied.

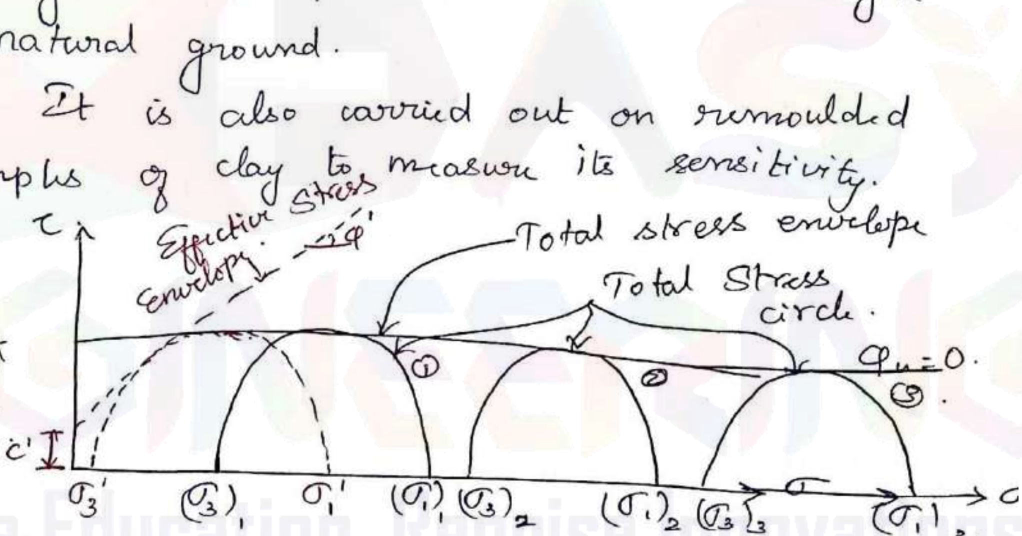
$$\boxed{A = \frac{\Delta u}{\Delta \sigma_1}}$$

SHEAR STRENGTH OF COHESIVE SOILS:

a) Undrained test on saturated cohesive soils:

It is carried on undisturbed sample of clay, silt & peat to determine strength of natural ground.

It is also carried out on remoulded samples of clay to measure its sensitivity.



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For 1st circle,

Diameter $= (\sigma_1)_1 - (\sigma_3)_1$

u - Pore pressure measured @ failure.

Diameter of effective stress circle $= (\sigma_1')_1 - (\sigma_3')_1$

$$= ((\sigma_1)_1 - u) - ((\sigma_3)_1 - u)$$

$$= (\sigma_1)_1 - (\sigma_3)_1$$

Diameter of effective stress circle $\}$ = Diameter of total stress circle.

Major effective principal stress doesnot change

$B = 1$, both Major principal Effective stress σ_1' & minor principal Effective stress σ_3' are independent of magnitude of cell pressure applied.

$\therefore$ We get only one Mohr Circle in terms of effective stress, for all identical specimens tested under increased pressure.

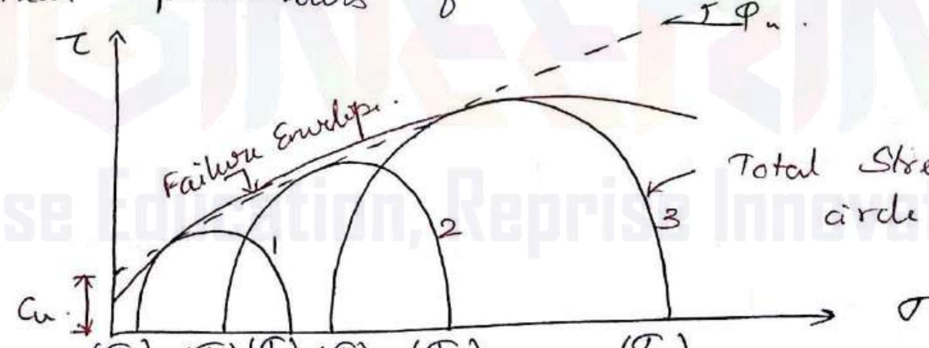
$$\boxed{\begin{aligned}\varphi_u &= 0\\ C_u &= d\end{aligned}}$$

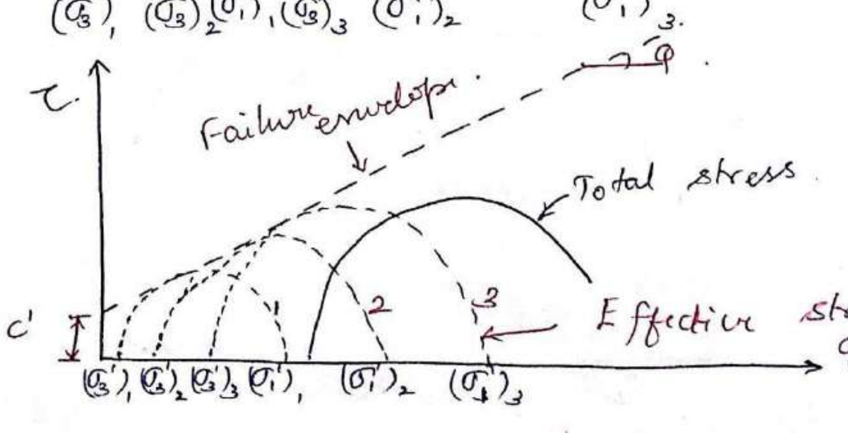
Deviator stress $\sigma_d = \sigma_1' - \sigma_3'$

Effective stress envelope cannot be obtained from this test.

b) Undrained test on partly saturated cohesive soil:

In case of Earth Embankments, which are compacted at optimum water content, the soil remains partly saturated and it is necessary to conduct undrained test to determine Shear parameters of soil.





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As all pressure increased, deviator stress @ failure also increases, though this increase in deviator stress becomes smaller as the air in soil voids is compressed & dissolved.

The increase in deviator stress later ceases when large stresses cause full saturation.

Due to this, failure envelope in terms of total stress is non-linear.

Failure envelope in terms of effective stress is very closely a straight line over wide range of pressure.

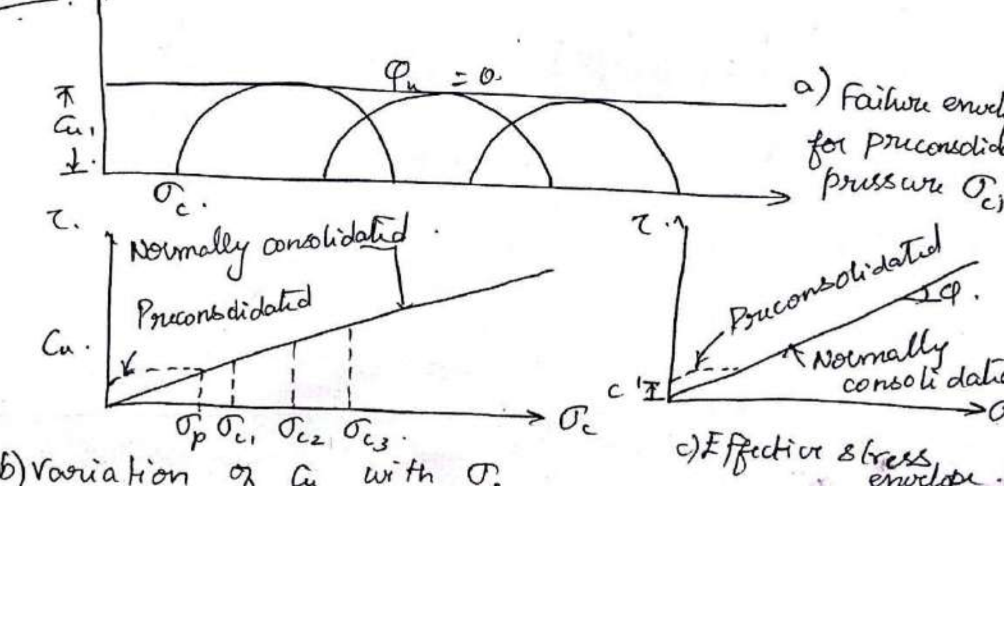
c) Consolidated undrained test on saturated cohesive soils:

Performed by two methods.

i) Moulded specimens are first consolidated under same cell pressure & then sheared under undrained conditions with different cell pressure by increasing axial stress.

ii) Remoulded specimens are sheared under a cell pressure equal to consolidation pressure.

I-Method.



a) Failure envelope for preconsolidation pressure σ_{c1}.

b) Variation of c_u with σ.

c) Effective stress envelope.

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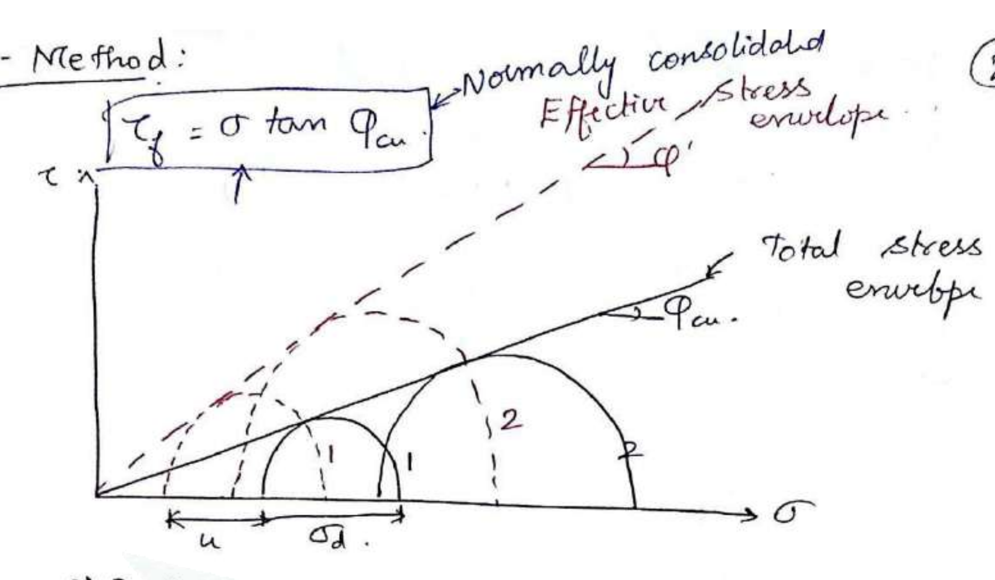
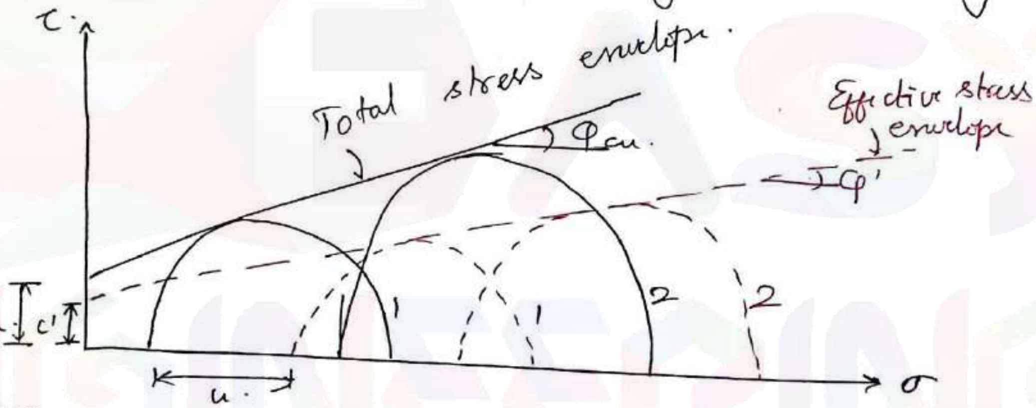
II - Method:

$$\boxed{\tau_f = \sigma \tan \varphi_{cu}}$$



a) Failure envelope for normally consolidated clay.



b) Failure envelopes for preconsolidated clay.

$$\boxed{\tau_f = C_{cu} + \sigma \tan \varphi_{cu}} \leftarrow \text{Preconsolidated.}$$

d) Consolidated undrained test on partly saturated cohesive soils:

– To examine the effect on c' & φ' of flooding foundation strata & earth fill materials, by applying back pressure to pore space to ensure full saturation.

Shear strength - independent of change in cell pressure.

Incomplete saturation will mean that unique value of C_{cu} & ϕ_{cu} will only be obtained if no change in cell pressure is made after consolidation stage, before sample is sheared.

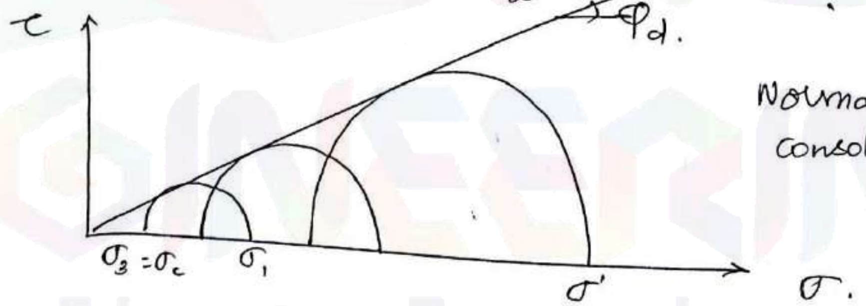
C' & ϕ' are determined by measuring pore pressure & getting value of effective stress @ failure.

e) Drained tests :

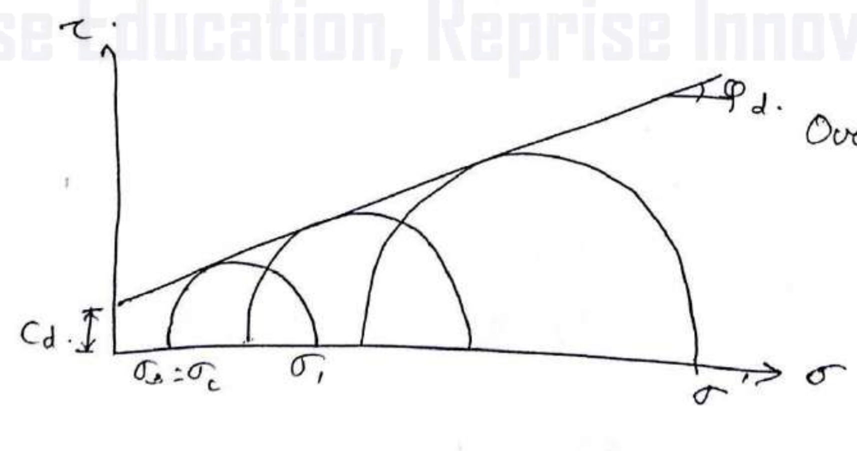
Specimen is first consolidated under cell pressure ($\sigma_c = \sigma_3$) & is then sheared slowly so that pore pressure developed during shearing is dissipated.

$\sigma_3' = \sigma_c$ σ_c' - Axial stress.

$u = 0$, Total stress = Effective stress.



Normally consolidated clay.



Overconsolidated clay.

LIQUEFACTION: (21)

* It is a phenomenon in which loose saturated sand loses a large percentage of its shear strength & develops characteristics similar to those of a liquid.

* It is usually induced by cyclic loading of relatively high frequency, resulting in undrained conditions in sand. Cyclic loading may be caused by vibrations from machinery & by earth tremors.

* Loose sand tends to compact under cyclic loading. The decrease in volume causes an increase in pore water pressure which cannot dissipate under undrained conditions.

* Indeed, there may be cumulative increase in pore water pressure under successive cycles of loading. If pore water pressure = Maximum total stress component, normally overburden pressure, value of effective stress will be zero. & sand will exist in a liquid state with negligible shear strength.

* Even if effective stress doesnot fall to zero, the reduction in shear strength may be sufficient to cause failure.

* Liquefaction may develop at any depth in sand deposit where a critical combination of insitu density and cyclic deformation occurs.

- Higher void ratio, Lower confining pressure - Liquefaction
- Larger strains produced by cyclic loading, Lower no. of cycles required for liquefaction

Initiation of liquefaction

The fact that soil deposit is susceptible to liquefaction does not mean than liquefaction will necessarily occur in given Earthquake. It's occurrence requires a disturbance that is strong enough to initiate or trigger it. Evaluation of nature of that disturbance is one of the most critical parts of liquefaction hazard evaluation.

Cyclic mobility is an earthquake related phenomena, flow liquefaction can be initiated in variety of ways.

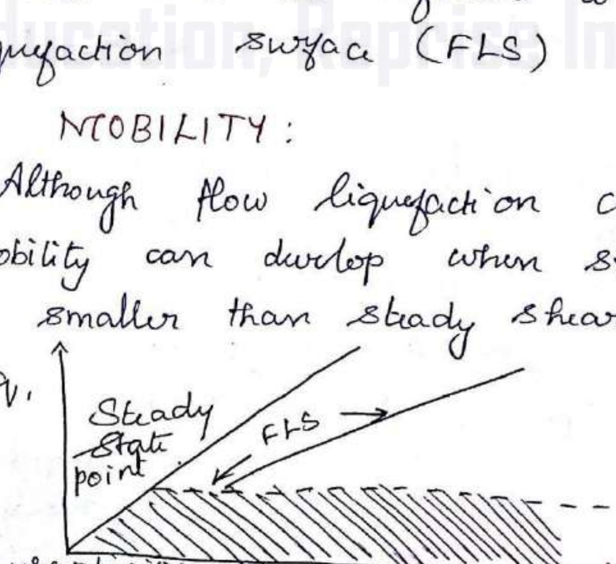
Static Liquefaction - Monotonic loading.

Flow liquefaction Surface:

The effective stress conditions at the ~~intimation~~ initiation of flow liquefaction can be described in stress path space by a 3D surface that will be referred to as flow liquefaction surface (FLS)

CYCLIC MOBILITY:

Although flow liquefaction cannot occur, cyclic mobility can develop when static shear stress is smaller than steady shear strengths.

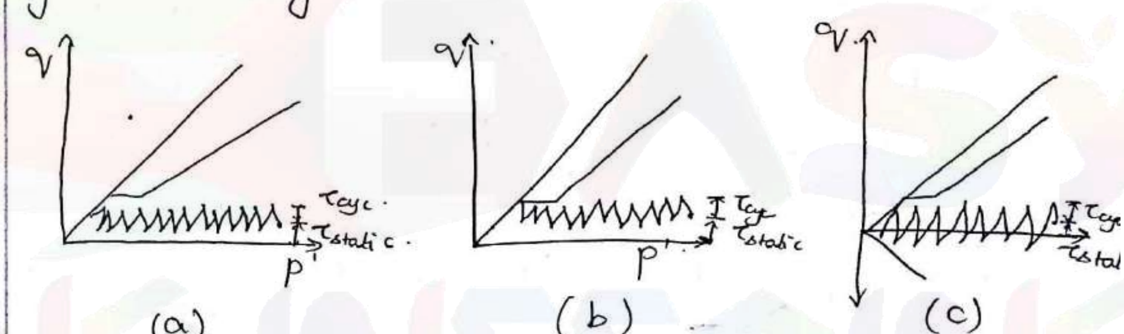


Zone of susceptibility to cyclic mobility.

If initial conditions plot within shaded zone, cyclic mobility can occur. (22)

Cyclic mobility can occur in both loose & dense soils [the shaded region extends from very low to very high effective confining pressures and corresponds to states that would plot both above & below SSL].

Three combinations of initial conditions & cyclic loading conditions generally produce cyclic mobility.



a) No stress reversal and no exceedance of steady state strength.

b) No stress reversal with momentary periods of steady state strength exceedance.

c) Stress reversal with ~~no~~ exceedance of steady state strength.

First - when $\tau_{static} - \tau_{cyc} > 0$ [No stress reversal] & $\tau_{static} + \tau_{cyc} > S_{su}$ (Steady state strength is exceeded momentarily).

Again cyclic loading will cause effective stress path to move to left, when it touches FLS momentary periods of instability will occur.

Significant permanent strain may develop during these periods, particularly τ_{static} is greater than quasi-static shear strength, but straining will generally cease at end of cyclic loading when shear stress returns to τ_{static}.

Final condition: $\tau_{static} - \tau_{cyc} > 0$ (Stress reversal occurs).

$\tau_{static} + \tau_{cyc} > S_{su}$ (Steady state strength is not exceeded).

In this case, the direction of shear stress changes so that each cycle includes both compressional & extensional loading.

Effective stress path moves relatively quickly to left [because excess pore pressure builds up quickly – Rate of pore pressure generation increases with increasing degree of stress reversal] and eventually oscillates along compression & extension portions of drained failure envelope.

Each time, effective stress path passes through origin, the specimen is in an instantaneous state of zero effective stress.

State of zero effective stress – Initial liquefaction – No shear strength.

If monotonic loading is applied at state of initial liquefaction, specimen will dilate until steady state strength is mobilized. Significant permanent strains may accumulate during cyclic loading, but flow failure cannot occur.

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Note that initial liquefaction can only (23)
occur when stress reversals occur.

Incontrast to flow liquefaction, there is no clear cut point at which cyclic mobility is initiated.

Permanent strains & permanent deformation, they produce accumulate incrementally. Their magnitude depends on static shear stress & duration at nearly level sites, permanent deformations may be small.

For moderately sloping sites or gently sloping sites subjected to ground motions of long duration, cyclic mobility can produce damaging levels of soil deformation.

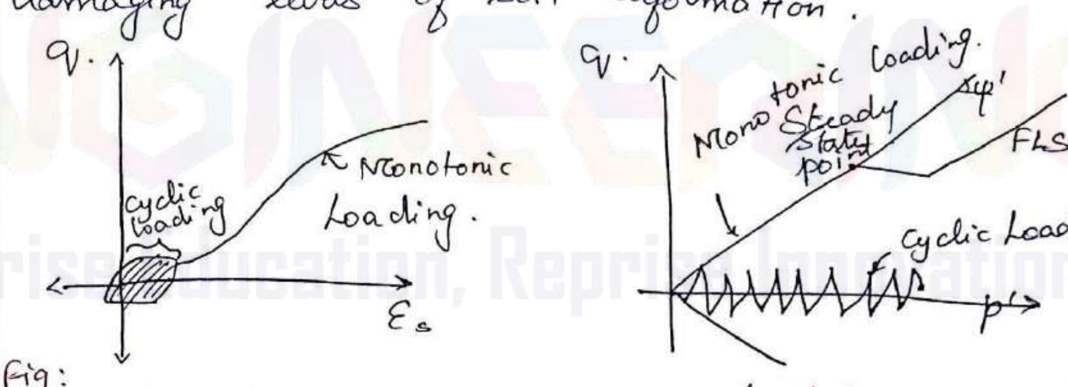

Fig: Dilative behavior of specimen loaded monotonically after occurrence of cyclic mobility. Cyclic loading with stress reversal causes effective confining pressure to decrease rapidly, eventually reaching momentary values of zero. Subsequent monotonic loading, causes dilation as steady state strength is mobilized.

UNIT - V

SLOPE STABILITY

Slope failure mechanisms - Types - Infinite Slopes - Finite Slopes - Total stress analysis for saturated clay - Fellenius method - Friction circle method - Use of stability number - Slope protection measures.

STABILITY OF SLOPES:

Earth embankments are commonly required for railways, roadways, earth dams, levees & river training works. The stability of these embankments or slopes, as they are commonly called, should be very thoroughly analysed since their failure may lead to loss of human life as well as collosal economic loss.

Failure of mass of soil located beneath a slope is called slide. It involved a downward and outward movement of entire mass of soil that participates in failure.

Failure of slopes takes place mainly due to

i) Action of gravitational forces

ii) Seepage forces within the soil.

iii) Due to excavation or undercutting of its foot

IV) Due to gradual disintegration of structure of soil.

Analysis of stability of slope consists of two parts

i) Determination of the most severely stressed internal surface and the magnitude of shearing stress to which it is subjected.

ii) Determination of shearing strength along this surface.

The shearing stress to which any slope can be subjected depends upon unit weight of material & geometry of slope, while shear strength which can be mobilised to resist shearing stress depends on character of soil, its density & drainage conditions.

Slops

Infinite Slope

Boundary surface of semi-infinite soil mass.

↓

Soil properties for all identical depths below surface are constant.

↓

Slope extending to infinity donot exist in nature.

Finite Slope

Slope of limited extent.

↓

Inclined faces of Earth dam, embankments and cuts, etc.

↓

All man made slopes.

STABILITY ANALYSIS OF INFINITE SLOPES:

Figure shows an infinite slope AB, inclined at angle i to the horizontal.

Soil properties & the soil stress on any

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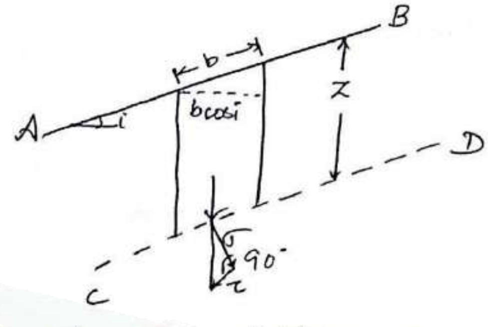
plane ∥ to slope surface are identical & ② failure of slope involves a sliding of soil mass along a plane ∥ to slope at some depth.

CD - Failure plane at depth z below surface.



Consider a prism of soil, of inclined length b along slope & depth z upto critical surface.

Horizontal length of prism $= b\cos i$

Volume / unit length of prism $= z\, b \cos i$

Weight of prism $= W = \gamma z\, b \cos i$

Vertical stress σ_z on surface CD is given by

$$\sigma_z = \frac{W}{b} = \gamma z \cos i$$

σ - Stress component normal to surface CD.

τ - Stress component tangential to surface CD.

$$\boxed{\begin{aligned} \sigma &= \sigma_z \cos i = \gamma z \cos^2 i \\ \tau &= \sigma_z \sin i = \gamma z \cos i \sin i \end{aligned}}$$

τ - Shear stress which is resisted by shear strength (τ_f).

Factor of safety against sliding due to shear

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

τ_f consists of both cohesion & internal friction.

Two cases : i) Cohesion less soil
ii) Cohesive soil.

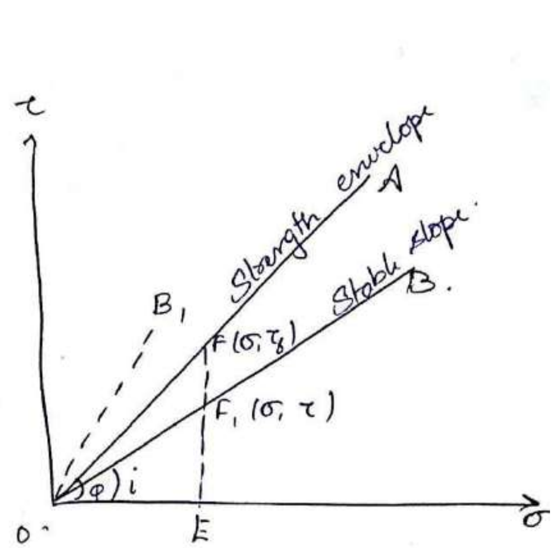
i) Cohesion less soil:

$\tau_f = \sigma \tan \varphi$.

$\frac{\sigma}{\tau} = \frac{\gamma z \cos^2 i}{\gamma z \cos i \sin i}$

$= \frac{\cos i}{\sin i}$

$\frac{\sigma}{\tau} = \cot i$ = constant.



$\sigma = \tau \cot i$

$\tau = \frac{\sigma}{\cot i}$

$\boxed{\tau = \sigma \tan i}$

$\tau < \tau_f$ (or) $i < \varphi$ - Failure not occur.

Factor of safety

$F = \frac{\tau_f}{\tau} = \frac{\tan\varphi}{\tan i}$

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

Submerged Slope:

If slope is submerged, the bulk unit weight γ should be replaced by submerged unit weight γ'

$\sigma = \gamma' z \cos^2 i$

$\tau = \gamma' z \cos i \sin i$

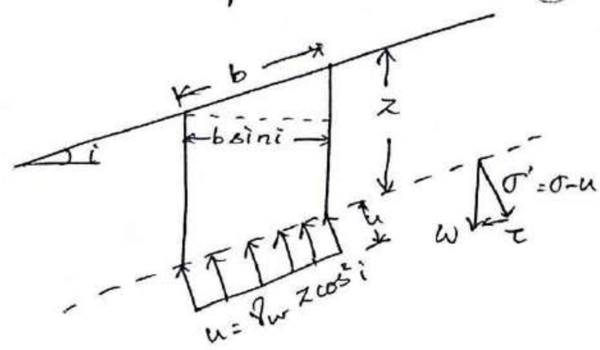
$\tau_f = \sigma \tan\varphi$

$= \gamma' z \cos^2 i \tan\varphi$.

$F = \frac{\tau_f}{\tau} = \frac{\cancel{\gamma' z} \cos^2 i \tan\varphi}{\cancel{\gamma' z} \cos i \sin i}$

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

Steady seepage along the slope : (3)



$W = \gamma_{sat} \cdot z b \cos i$

$\sigma_z = \frac{W}{b}$

$= \gamma_{sat} \cdot z \cos i .$

$\sigma = \sigma_z \cos i = \gamma_{sat} z \cos^2 i$

$\tau = \sigma_z \sin i = \gamma_{sat} z \cos i \sin i$

In addition to these, there is an upward force u due to seeping water,

$u = \gamma_w z \cos^2 i$

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

$\tau_f = \sigma' \tan\varphi$

$\sigma' = \sigma - u$

$= \gamma_{sat} z \cos^2 i - \gamma_w z \cos^2 i$

$\sigma' = \gamma' z \cos^2 i$

$\tau_f = \gamma' z \cos^2 i \tan\varphi .$

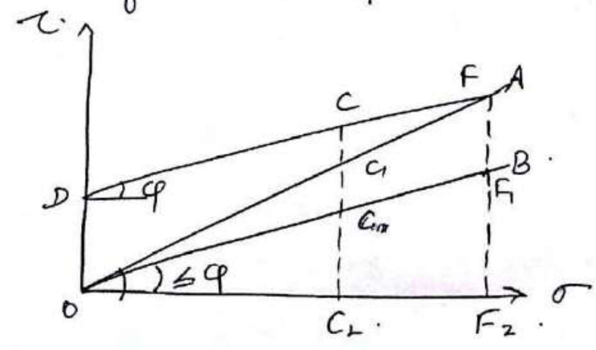
$\tau = \gamma_{sat} \cdot z \cos i \sin i$

$F = \frac{\tau_f}{\tau} = \frac{\gamma' z \cos^2 i \tan\varphi}{\gamma_{sat} z \cos^2 i \sin i}$

$$\boxed{F = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\varphi}{\tan i}}$$

ii) Cohesive soil :

$\tau_f = c + \sigma \tan\varphi .$

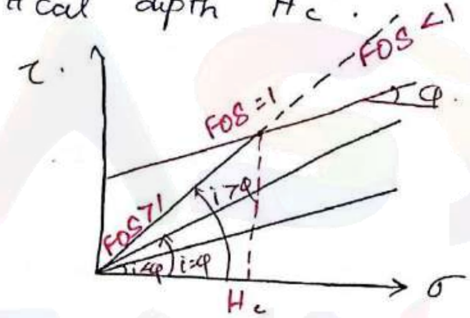


Slope angle (i) $\leq \varphi$, No critical state of stress - Slope Stable.

If $i > \varphi$ - it will cut strength envelope at some point F and a state of incipient failure is reached because shear stress corresponding to depth represented by point F equals to shear strength τ_f.

For any depth, $z < F$, $\tau < \tau_f$ - Slope - stable

$i > \varphi$ - Slope is stable upto limited depth known as critical depth H_c.



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

$$= \frac{c + \sigma \tan\varphi}{\tau}$$

$$\sigma = \gamma z \cos^2 i$$

$$\tau = \gamma z \cos i \sin i$$

$$F = \frac{c + \gamma z \cos^2 i \tan\varphi}{\gamma z \cos i \sin i}$$

$$= \frac{c}{\gamma z \cos i \sin i} + \frac{\cancel{\gamma z} \cos^2 i \tan\varphi}{\cancel{\gamma z} \cos i \sin i}$$

$$\boxed{F = \frac{c}{\gamma z \cos i \sin i} + \frac{\tan\varphi}{\tan i}}$$

For non-cohesive soil, $c = 0 \Rightarrow F = \frac{\tan\varphi}{\tan i}$

For critical depth $z = H_c$, $\tau_f = \tau$

$$c + \gamma H_c \cos^2 i \tan\varphi = \gamma H_c \cos i \sin i$$

$$c = \gamma H_c \cos i \sin i - \gamma H_c \cos^2 i \tan\varphi.$$

$$= \gamma H_c \cdot \frac{\cos i}{\cos i} \cos i \sin i - \gamma H_c \cos^2 i \tan\varphi.$$

$$c = \gamma H_c \cos^2 i \tan i - \gamma H_c \cos^2 i \tan\varphi$$
$$= \gamma H_c \cos^2 i \,(\tan i - \tan\varphi)$$

$$\boxed{H_c = \frac{c}{\gamma \cos^2 i \,(\tan i - \tan\varphi)}}$$

For given values of i & φ,
$H_c \propto$ cohesion.

$$\frac{c}{\gamma H_c} = \cos^2 i \,(\tan i - \tan\varphi).$$

Stability number $(S_n) = \frac{c}{\gamma H_c}$ - Dimensionless quantity.

$$\boxed{S_n = \frac{c}{\gamma H_c}}$$

F_c - Factor of Safety w.r.to cohesion.
C_m - Mobilised cohesion at depth H.

$$\boxed{C_m = \frac{c}{F_c}}$$

$$S_n = \frac{c}{\gamma H_c} = \frac{C_m}{\gamma H}.$$

$$\boxed{S_n = \frac{c}{F_c \gamma H}} = (\tan i - \tan\varphi)\cos^2 i$$

$$\boxed{F_c = \frac{c}{C_m}} \qquad \boxed{F_c = \frac{H_c}{H}}$$

Submerged Slope:

$$F = \frac{c + \gamma' z \cos^2 i \tan\varphi}{\gamma' z \cos i \sin i}.$$

$$H_c = \frac{c}{\gamma'} \frac{1}{(\tan i - \tan\varphi)\cos^2 i}$$

Steady Seepage along the slope :

$$F = \frac{c + \gamma' z \cos^2 i \tan\varphi}{\gamma_{sat} z \cos i \sin i}$$

$$\boxed{F = \frac{c}{\gamma_{sat} z \cos i \sin i} + \frac{\gamma'}{\gamma_{sat}} \frac{\tan\varphi}{\tan i}}$$

For critical height, $z = H_c$. $F = 1$.

$$\gamma_{sat} H_c \cos i \sin i = c + \gamma' H_c \cos^2 i \tan\varphi.$$

$$c = \gamma_{sat} H_c \cos i \sin i - \gamma' H_c \cos^2 i \tan\varphi.$$

$$= \gamma_{sat} H_c \cos^2 i \tan i - \gamma' H_c \cos^2 i \tan\varphi.$$

$$\boxed{H_c = \frac{c}{\cos^2 i \left[\gamma_{sat} \tan i - \gamma' \tan\varphi\right]}}$$

PROBLEMS :

A long natural slope of cohesionless soil is inclined at 12° to horizontal. Taking $\varphi = 30°$, Determine FOS of slope. If slope is completely submerged, what will be FOS?

$$F = \frac{\tan\varphi}{\tan i} \qquad \varphi = 30° \quad i = 12°.$$

$$F = \frac{\tan 30}{\tan 12} = 2.72 \qquad \boxed{F = 2.72}$$

Submergence :

$$F = \frac{\tan\varphi}{\tan i} = 2.72.$$

2. A long natural slope of sandy soil ($\varphi = 25°$) is inclined at 10° to horizontal. The water table is at the surface & seepage is parallel to slope. If saturated unit weight of soil is 19.5 KN/m³, determine factor of safety of slope. (5)

$$F = \frac{\gamma' \tan\varphi}{\gamma_{sat} \tan i} = \frac{(19.5 - 9.81) \tan 25°}{19.5 \times \tan 10°}$$

$$\boxed{F = 1.31}$$

3. A long natural slope in a c-φ soil is inclined at 12° to horizontal. The W.T is at the surface and the seepage is parallel to slope. If a plane slip has developed at a depth of 4 m. determine FOS. Take $c = 8\,KN/m^2$, $\varphi = 22°$ & $\gamma_{sat} = 19\,KN/m^3$

$$F = \frac{c + \gamma' z \cos^2 i \tan\varphi}{\gamma_{sat}\, z \cos i \sin i}$$

$$= \frac{8 + (19 - 9.81) \times 4 \cos^2 12° \tan 22°}{19 \times 4 \cos 12° \sin 12°}$$

$$\boxed{F = 1.44}$$

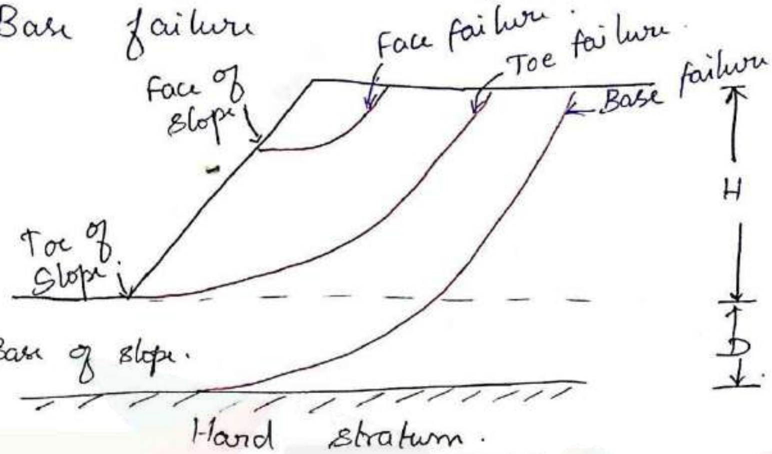
———— x ————

STABILITY ANALYSIS OF FINITE SLOPES:

Failure of finite slopes occurs along a surface which is a curve. In stability computations, the curve representing real surface of sliding is usually replaced by an arc of a circle.

Two basic types of failure

* Slope failure < Face Failure / Toe failure.
* Base failure



Depth factor $\boxed{D_f = \frac{H + D}{H}}$

$D_f > 1 \rightarrow$ Base failure

$D_f = 1 \rightarrow$ Toe failure

$D_f < 1 \rightarrow$ Face failure.

Face failure:

When slope angle β is very high. Soil in the upper part of slope is relatively weaker.

Toe failure:

* It occurs in steep slopes.
* It happens when soil mass above and below base is homogeneous.

Base failure:

* Soil below toe is relatively weak & soft.
* The slope is flat.

Types of slip surfaces or failure surfaces:

The rupture of a finite slope may takes

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place along one of following failure surfaces. 6

* Planar failure surface
* Circular failure surface
* Non-circular failure surface.

Planar failure surface - Soil deposit or embankment with a specific plane of weakness.

In composite Earth dams with sloping cores, planes of weakness within the bank may consists of 2 or 3 planar surfaces.

Circular failure surface -

In most cases, actual failure surfaces are curved. The rupture mass slide down a sliding surface, in a definite pattern resembling that of cycloid. Generally, the failure surfaces have arcs somewhat flatter at ends & sharper at centre. For simple idealised problems, the assumption of a circular failure surface is sufficiently accurate.

Non-circular failure surface:

It occurs in many practical cases. It may arise in homogeneous dams having one or more of the following.

i) Foundation of infinite depth.
ii) Rigid boundary planes of maximum or zero shear.
iii) Presence of relatively stronger or weaker layer.

Non-homogeneous earth dams.

i) Presence of soft layer in foundation.
ii) Use of different type of soil or rock in dam

with varying strength & pore pressure condition

iii) Use of drainage blankets to facilitate dissipation of pore pressures.

Methods of Analysis:

Stability of finite slope can be investigated by no. of methods.

1. Fellenius Method (or) Swedish circle method (or) Slip circle Method.
2. Friction circle Method
3. Culmann's method.
4. Bishop's method.

Swedish Slip Circle Method / Fellenius Method:

This method was developed at Swedish Geotechnical Commission headed by Fellenius.

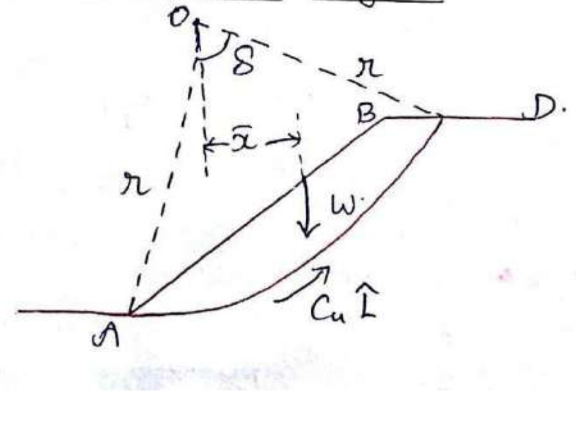
In this method, slip surface is assumed to be cylindrical, i.e: arc of circle in section.

Two cases:

i) Analysis of purely cohesive soil ($\phi_u = 0$ analysis)

ii) Analysis of a soil possessing both cohesion and friction ($c - \phi$ analysis).

i) <u>Cohesive Soil ($\phi_u = 0$ Analysis)</u>



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The method consists in assuming no. of trial slip circles & finding FOS of each. (7)

The circle corresponding to minimum FOS is called critical slip circle.

AD - Trial slip circle.

r - radius

O - centre of rotation.

W - Weight of soil of wedge ABDA of unit thickness, acting through its centroid.

$\bar{x}$ - Distance of line of action of W from vertical line passing through centre of rotation.

C_u - Unit cohesion.

$\hat{L}$ - Length of slip arc AD.

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

$$\hat{L} = \frac{2\pi r \delta}{360}$$

Shear resistance developed along slip surface $= C_u\hat{L}$

Resisting moment $\boxed{M_R = r \cdot C_u \hat{L}}$

FOS, $\boxed{F = \frac{M_R}{M_D} = \frac{r C_u \hat{L}}{W\bar{x}}}$

C_m - Mobilised shear resistance of soil

$F = 1$

$W\bar{x} = r C_m \hat{L}$.

$\boxed{C_m = \frac{W\bar{x}}{\hat{L}} 1/r}$ $\qquad F = \frac{C_u}{C_m} = \frac{C_u \hat{L} r}{W\bar{x}}$

$\bar{x}$ from O can be determined by dividing wedge into no. of vertical slices & dividing algebraic sum of moment of weight of each slice by weight of wedge.

Tension crack :

If a tension crack of depth $z_0 = \frac{2c}{\gamma}$ develops, water enter into crack, exerting hydrostatic pressure force P_w acting on the portion DE at the height $z_0/3$ from E. That portion will be ineffective in resisting slide.

ii) <u>C-φ analysis</u>:

It is known as Swedish method of slices.

i) The slip surface is cylindrical

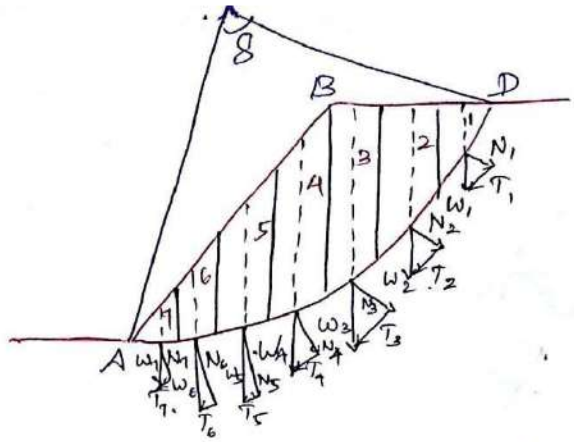
ii) The sliding soil mass is assumed to consist of a number of vertical slices.

iii) The forces of interaction b/w adjacent slices are neglected.

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

Let sliding soil mass ABDA be divided into no. of vertical slices 1, 2, ...

The weights W_1, W_2 . . . . . of slices 1, 2, ... acting through centre of gravity of respective slices are resolved into normal components N_1, N_2 . . . & tangential components T_1, T_2, . . .



Taking moments about centre of rotation O.

Driving moment

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

$$= r\left[T_1 + T_2 + \ldots\ldots\right]$$

$$\boxed{M_D = r\ \Sigma T}$$

Restoring Moment,

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

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

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

$\hat{L}$ - length of arc AD.

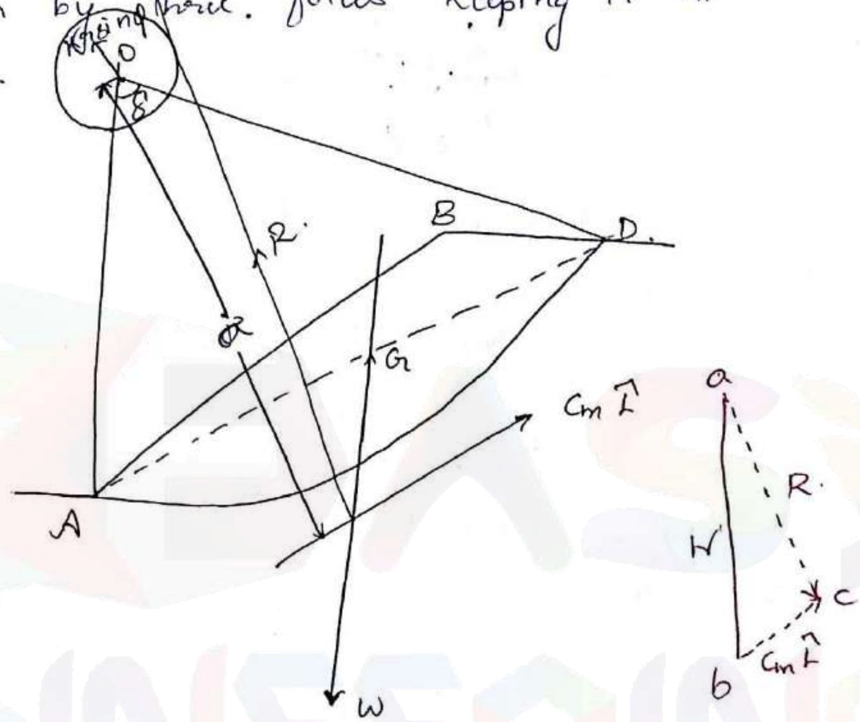
FOS against sliding, $F = \dfrac{M_R}{M_D}$

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

This method is not only applicable to homogeneous soils but also to stratified soils, fully or partially submerged soils with considerations of seepage forces and pore pressures that may exist, and also non-uniform slopes.

Friction Circle Method :

* The slip surface is assumed to be cylindrical . i.e arc of circle in section.
* The sliding soil mass is assumed to be acted upon by three. forces keeping it in equilibrium.



* Weight . W of the sliding soil mass ABDA, acting vertically through its centre of gravity.
* The resultant cohesive force, $C_m \hat{L}$, acting parallel to chord AD and at distance a from centre of rotation O, where

$\boxed{a = r\,\frac{\hat{L}}{L}}$ ⇒ $\hat{L}$ - Length of arc AD.
L - Length of chord AD.

* The resultant reaction R passing through point of intersection of the above two forces and tangential to the friction circle.

Procedure :

1. With centre O & radius r, the slip circle AD is constructed. The friction circle is drawn with centre O & radius $K r \sin\varphi$.

$K = 1$.

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

3. Chord AD is drawn, A line is drawn parallel to chord AD and at distance $a = r\frac{\hat{L}}{L}$ from O, to get line of action of resultant cohesive force $C_m \hat{L}$. The length of arc AD, $\hat{L}$ is computed using equation $\hat{L} = \frac{\pi r \delta}{180}$

The length of chord AD, L is obtained by measurement.

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

5. Weight (W) of sliding soil mass ABDA is computed & plotted to scale. Through the ends of vector representing W, lines are drawn parallel to lines of action of forces $C_m L$ & R to complete triangle of forces.

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

FOS : $\boxed{F_c = \frac{c}{C_m}}$ c - Ultimate cohesion.

USING TAYLOR STABILITY NUMBER :

Taylor Stability number is a dimensionless quantity denoted by S_n.

$$\boxed{S_n = \frac{C_m}{\gamma H}}$$

C_m = Mobilised cohesion on slip surface

γ = Unit weight of soil

H - Height of slope.

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

$$\boxed{S_n = \frac{C}{F_c \gamma H}}$$ c = Unit Ultimate cohesion.

$F_H = F_c$.

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

$F_H H = H_c$.

$$\boxed{S_n = \frac{C}{\gamma H_c}}$$

H_c - Critical height of slope.

S_n varies w.r.to slope angle i & angle of shearing resistance φ.

FOS is applicable to both cohesion & friction, we have mobilised shear resistance given by. $\tau_m = \frac{\tau_f}{F} = \frac{c + \sigma \tan\varphi}{F}$

While obtaining S_n from chart, mobilised angle of shearing resistance φ_m should be used.

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

$$\varphi_m = \tan^{-1}\left(\frac{\tan\varphi}{F}\right).$$

$$\boxed{\varphi_m \simeq \frac{\varphi}{F}}$$

For cohesionless soil ($c = 0$), Taylor stability number $S_n = 0$. Taylor's chart is not applicable. (10)

$$F = \frac{\tan\varphi}{\tan i}$$ & independent of height of slope.

For cohesive soils, c & φ can be obtained from drained test should be used. Use of Taylor's stability number gives an approximate idea of long term stability, if seepage effect can be neglected and no change in water content can be assumed.

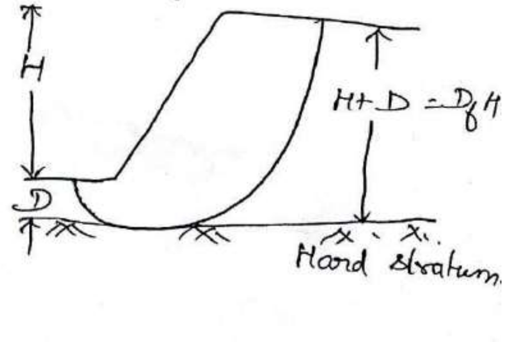
In case of fully submerged slopes, γ' should be used in expression for S_n.

When slope is saturated by capillary water, γ_{sat} should be used in expression S_n. S_n can be obtained from Taylor's chart, corresponding to weighted frictional angle φ_w

$$\varphi_w = \frac{\gamma'}{\gamma_{sat}} \varphi .$$

Taylor also determined stability number S_n for different values of slope angle i and depth factor D_f.

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



PROBLEMS:

1. Stability analysis by Swedish method of slices gave following values per running metre for a 10m high embankment.

i) Total shearing force = 480 kN.

ii) Total normal force = 1950 kN.

iii) Total neutral force = 250 kN.

iv) Length of arc = 22 m.

If the properties of soil are $c = 24\ kN/m^2$ & $\varphi = 6°$, calculate factor of safety w.r. to shear strength

$$\Sigma T = 480\ kN$$
$$\Sigma N = 1950\ kN \qquad C = 24\ kN/m^2$$
$$\Sigma U = 250\ kN \qquad \varphi = 6°.$$
$$\bar{L} = 22\ m$$

$$F = \frac{C\bar{L} + \Sigma(N-U)\tan\varphi}{\Sigma T}$$

$$= \frac{24 \times 22 + (1950 - 250)\tan 6°}{480}$$

$$\boxed{F = 1.47.}$$

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

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

Calculate i) FOS w.r. to cohesion.

ii) Critical height of slope.

If i is slope angle $\tan i = 1/2$

$$\boxed{i = 26.6°}$$

From Taylor stability chart, for $i = 26.6°$ (11)

$\varphi = 10°$, $S_n = 0.064$.

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

$$F_c = \frac{c}{S_n \gamma H} = \frac{28}{(0.064)(18)(8)}$$

$$\boxed{F_c = 3.04}$$

$$F_c = \frac{H_c}{H} \qquad H_c = F_c H = 3.04 \times 8$$

$$\boxed{H_c = 24.32\,m}$$

3. A 5m deep canal has side slopes of 1:1. Properties of soil are $C_u = 20\,KN/m^2$, $\varphi_u = 10°$, $e = 0.8$ & $G = 2.8$. If Taylor stability number is 0.108, determine FOS w.r.to cohesion, when the canal runs full. Also find the same in sudden drawdown if Taylor's stability number for the condition is 0.137.

$C_u = 20\,KN/m^2$, $\varphi_u = 10°$, $G = 2.8$.

$$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w = \frac{(2.8+0.8)}{(1+0.8)} 9.81$$
$$= 19.62\,KN/m^3.$$

$$\gamma' = \gamma_{sat} - \gamma_w = 9.81\,KN/m^3.$$

Case (i): When canal runs full the side slopes are submerged.

$$S_n = \frac{c}{F_c \gamma' H} \qquad \boxed{F_c = 3.8}$$

$$F_c = \frac{c}{S_n \gamma' H} = \frac{20}{(0.108)(9.81)(5)} = 3.8.$$

Case (ii) Sudden drawdown condition, $S_n = 0.137$.

$$F_c = \frac{c}{S_n \gamma' H} = 1.5 \qquad \boxed{F_c = 1.5}$$

SLOPE PROTECTION MEASURES:

Slopes that are susceptible to sliding should be protected so that the area will be safe. Slopes which have failed recently are likely to fall under long-term condition.

Slope have been protected by adopting some successful techniques. In general, protective measures involves.

i) Reducing the mass or loading which contributes to sliding.

ii) improving the shearing strength along the anticipated zone of failure.

iii) Providing certain materials which will provide resistance to movement.

The protective measure to be adopted depends on different field conditions, type of soil in slope, the volume or depth of soil involving in sliding, groundwater conditions, assessment of complete area which may require stabilization, the space available to undertake corrective measures, topographical conditions prevailing in the area & the possible changes that could due to vibratory measure undertaken.

* When base failure is anticipated, a berm may be provided near the toe.

* If a zone near the toe is susceptible (19) to erosion, a protective rock-fill blanket followed by a rip-rap can be provided.

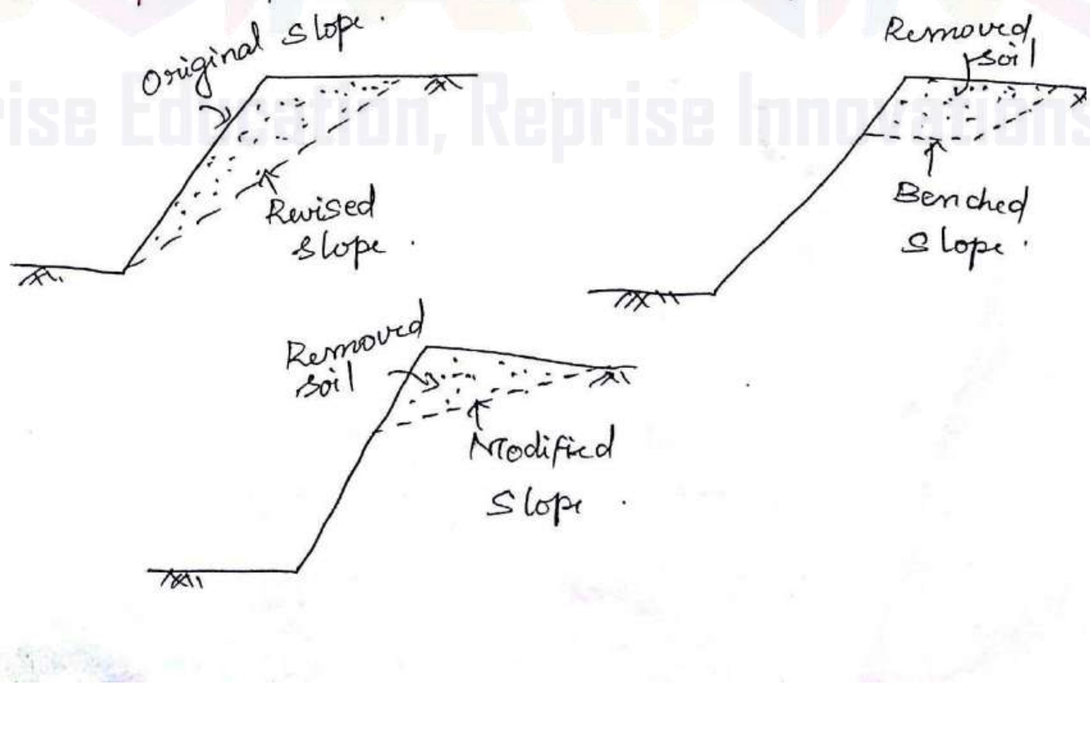
* Soil shearing resistance of soil is reduced due to high groundwater & excess pore-water pressure.

* This could be avoided by lowering the groundwater or intercepting the surface water.

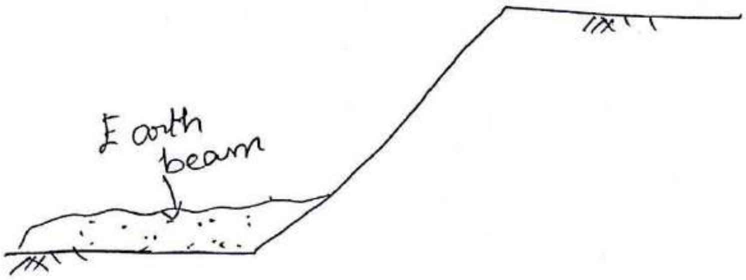
* Driven piles are sometimes used to keep the moving part intact with the original ground. Sometimes driven piles, sheet piling and construction of retaining wall help by providing lateral support and increasing the resistance of slopes to sliding.

* If it ~~is~~

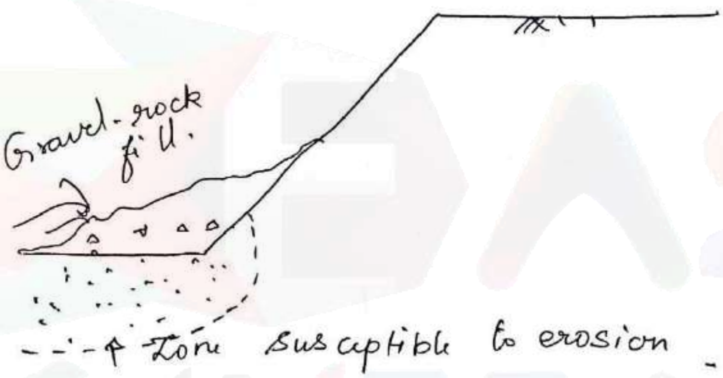
a) Slopes flattened or benched :



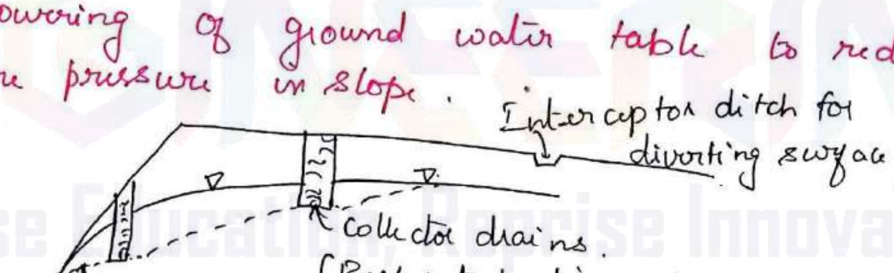
b) Beam provided at toe.



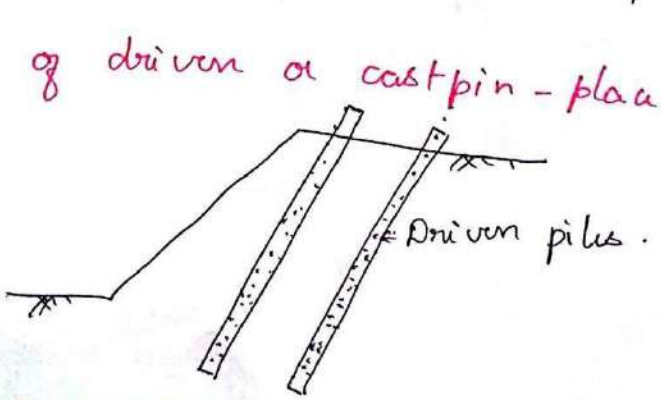
c) Protection against erosion provided at toe.



d) Lowering of ground water table to reduce pore pressure in slope.



e) Use of driven or cast-in-place piles.



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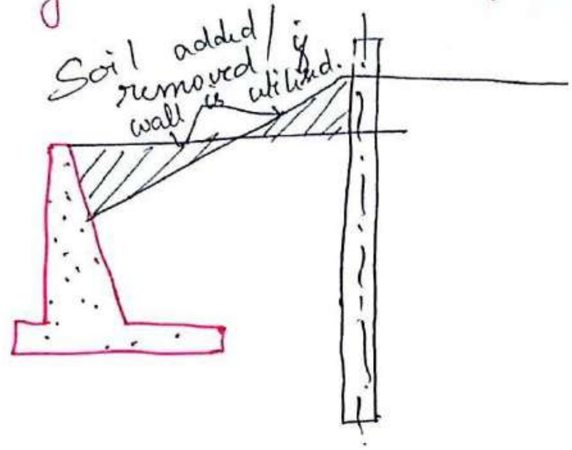
Retaining wall or sheet piling



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