LEARNING MATERIAL

SEMESTER & BRANCH : 3rd SEMESTER CIVIL ENGINEERING

THEORY SUBJECT : GEOTECHNICAL ENGINEERING (TH – 2)

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1st chapter   14 Aug 2020   Geotechnical Engineering

⇒ Soil is originated from the latin word "solum"

⇒ Solum means disintegration of ~~rok~~ rocks.

* **To An agriculturalist :-** soil means loose material lying on the earth's surface which is formed by disintegration of rocks & mixed with organic matter. soil gives life to plant.

* **To Geologist :-** soil means disintegration of rocks overlying the parent rock.

* **To the civil engg :-** The formation of soil takes place from the decomposition of vegitative matters & weathering of rocks.

* **weathering :-** weathering is the process where surface particles of rock breaking down by various agencies such as water, air and gravity.

Weathering → Physical, Chemical

* **Physical weathering :-**

(i) It occurs due to corrosion or disintegration of rocks by the natural resources.

(ii) The soil particles formed by this process are of bigger size.

(iii) The soil particles have same property with their parent rock.

(iv) The soil particles which are formed they don't have any bonding between them.

(v) Between them only friction acts.

Ex:- Sand

17 Aug, 2020

* Chemical weathering :- → when rocks come in to contact with acid or alkali some chemical reaction occur due to which rock disintegrate into smaller particle.

→ Here property of parent rock and some as smaller particles.

→ Here we get smaller size of particle i.e called colloidal particle (10^{-19} mt)

Ex:- clay

→ The bond between each particle is strong.

Scope of soil mechanics :-

→ Soil mechanics is used for studying or investigating the soil before any construction that is made one the soil.

⇒ If we don't analyse the soil then it may be deformed or any obstruction can made the superstructure & substructure damaged.

Types of soil

Residual	Transported
⇒ when the soil is located at it's origin place i.e at the foot of parent rock that soil is known as residual soil.	⇒ when the soil is transported from one place to another place with the help of various transporting medium such as water, wind, ice gravity etc.

* various types of transported soil :-

Soil Name	Transported medium
(1) Alluvial soil →	River
(2) Lacustrine soil →	Lake
(3) Marine soil →	Sea water
(4) Aeolian →	Air or wind
(5) Glacier soil →	Glacier or ice

2nd chapter

20-11-2020

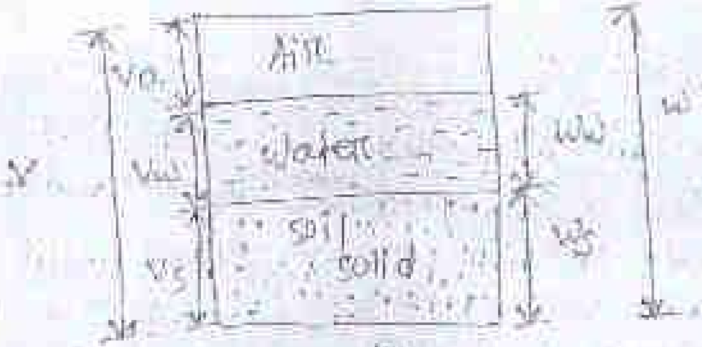
Soil & water Relationships

⇒ soil phase system is only an idealised representation of soil sample & it is not practically possible.

(partially saturated soil) (dry soil) (Fully saturated soil)

→ also 4- phase system is possible
soil solid → ice → water → Air

* Some Terminology :-



Total volume $V = V_a + V_w + V_s$
Total weight $W = W_w + W_s$
where V_a = volume of air
V_w = volume of water
V_s = volume of soil solid
W_w = weight of water
W_s = weight of soil solid
volume of void $(V_v) = V_a + V_w$

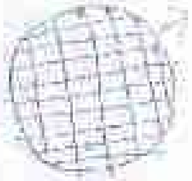
(1) Void Ratio (e) :-

→ Void ratio is defined as the ratio of volume of voids of the soil sample to the volume of soil solid.

$$\boxed{e = \frac{\text{volume of voids}}{\text{volume of solid}} = e = \frac{V_v}{V_s}}$$

→ It is calculated in fraction.
→ 0.5 - 0.95 — coarse grain soil
0.3 - 1.3 — fine grain soil

$\boxed{\text{Sieve - 4.75 mm}}$



φ 0.075 mm

pass - fine grain   Retain - coarse grain

(3) Porosity - (n)

→ Porosity is defind as the ratio of volume of voids of the soil sample to the total volume

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

It is expressed in percentage.

Range - 0% < n < 100%

0% → Dry soil

100% → Fully saturated soil

n% = partially saturated soil

(4) Air Content - (a_c)

Air content is the ratio of volume of air present in soil void to the volume of void

$$\boxed{a_c = \frac{\text{volume of air}}{\text{volume of void}} = \frac{V_a}{V_v}}$$

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

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

(5) % air void - percentage of air void is the ratio of volume of air to the total volume

→ It is expressed in percentage.

$$\boxed{n_a = \frac{\text{volume of air}}{\text{total volume}} = \frac{V_a}{V}}$$

21 May 2021

(6) Water content (w) :-

Water content is defined as the ratio of weight of water to the weight of soil solid.

$$\boxed{w = \frac{\text{weight of water}}{\text{weight of soil solid}} = \frac{W_w}{W_s}}$$

→ Range :- 0 - ∞

→ It is expressed in percentage.

Unit weight & Density :-

→ Unit weight is the ratio of weight per volume.

→ Unit weight is symbolised as γ.

$$\boxed{\gamma = \frac{W}{V}} \quad kN/m^3 \rightarrow \text{unit}$$

→ Density is the ratio of mass per volume.

→ It is symbolised as ρ.

unit -: kg/m^3

→ ... is symbolised as S

(7) Degree of saturation ... It is defined as the ratio of the volume of water to the volume of voids.

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

Unit weight

(1) Unit weight of water :-

$$\gamma_w = \frac{\text{weight of water}}{\text{volume of water}} = \frac{W_w}{V_w}$$

(2) Bulk unit weight :-

$$\gamma = \frac{\text{weight of soil mass}}{\text{volume of soil mass}} = \frac{W}{V}$$

(3) Dry unit weight :-

$$\gamma_d = \frac{\text{weight of soil solid}}{\text{volume of soil mass}} = \frac{W_s}{V}$$

(4) Saturated unit weight :-

$$\gamma_{sat} = \frac{\text{weight of saturated soil mass}}{\text{volume of soil mass}} = \frac{W_{sat}}{V}$$

(5) Submerged unit weight :-

$$\gamma' = \frac{\text{submerged weight of soil mass}}{\text{total volume of soil mass}} = \frac{W_{sub}}{V}$$

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

(6) Unit weight of soil solid :-

$$\gamma_s = \frac{\text{weight of soil solid}}{\text{volume of soil solid}} = \frac{W_s}{V_s}$$

Density

(1) Density of water :-

$$\rho_w = \frac{\text{mass of water}}{\text{volume of water}} = \frac{M_w}{V_w}$$

(2) Bulk density :-

$$\rho = \frac{\text{mass of soil mass}}{\text{volume of soil mass}} = \frac{M}{V}$$

(3) Dry density :-

$$\rho_d = \frac{\text{mass of soil solid}}{\text{volume of soil mass}} = \frac{M_s}{V}$$

(4) Saturated density :-

$$\rho_{sat} = \frac{\text{mass of saturated soil mass}}{\text{volume of soil mass}} = \frac{M_{sat}}{V}$$

(5) Submerged density :-

$$\rho' = \frac{\text{submerged mass of soil mass}}{\text{total volume of soil mass}} = \frac{M_{sub}}{V}$$

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

(6) Density of soil solid :-

$$\rho_s = \frac{\text{mass of soil solid}}{\text{volume of soil solid}} = \frac{M_s}{V_s}$$

$\boxed{\gamma_w = 9.81\, kN/m^3}$ $\boxed{\rho_w = 1 gm/cm^3}$

Specific gravity :- 22 Aug 2020

→ specific gravity is defined as the ratio of weight or mass of a volume to the weight or mass reference material at a standard temperature (27°C)

$$\boxed{G = \frac{\gamma_s}{\gamma_w} \text{ or } \frac{\rho_s}{\rho_w}}$$

→ It is used for to known either object is sink or float.

→ G = 2.6 - 2.8 (soil)

→ Bulk specific gravity :-

$$\boxed{G_m = \frac{\gamma}{\gamma_w} \text{ or } \frac{\rho}{\rho_w}}$$

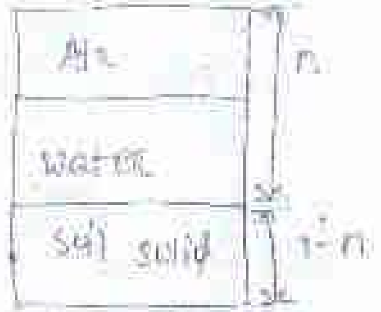
Types of soil	G
(1) sand	2.64 - 2.67
(2) silt	2.68 - 2.70
(3) clay	2.70 - 2.80
(4) soil containing iron	2.85 - 2.90
(5) organic soil	1.26 - 2.20

Relations :-

(Fig-i)

(soil element is expressed in terms of e)

(Fig-ii)



(expressed in terms of n)

Relation betn e & n :-

In fig(i) $n = \frac{V_v}{V} = \frac{e}{1+e}$

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

In fig(ii) $e = \frac{V_v}{V_s} = \frac{n}{1-n}$

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

Relation betn e, w, G & S :-

In fig(i) $S = \frac{V_w}{V_v} = \frac{e_w}{e}$

$\Rightarrow e_w = eS$ .... eqn (i)

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

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

$= \frac{\gamma_w \cdot eS}{\gamma_s}$

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

$\Rightarrow W_w = V_w \times \gamma_w$

$W_s = V_s \times \gamma_s$

$w = \frac{\gamma_w eS}{\gamma_s} = \frac{eS}{\frac{\gamma_s}{\gamma_w}}$

$\Rightarrow \quad w = \frac{e \times S}{G}$

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

(2) Relation betn e, n_a & S :-

In Fig(i) :- $n_a = \frac{V_a}{V}$

$$= \frac{V_v - V_w}{V_v + V_s}$$

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

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

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

$$\Rightarrow \boxed{n_a = \frac{e - eS}{1 + e}}$$

(3) γ_d, G, e or n :-

$$\gamma_d = \frac{W_d}{V} = \frac{W_s}{V} = \frac{\gamma_s \cdot V_s}{V_s + V_v} = \frac{\gamma_s}{1 + e}$$

$$(W_d = W_s) \quad \left[G = \frac{\gamma_s}{\gamma_w} \Rightarrow \gamma_s = G\gamma_w \right]$$

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

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

(5) γ_d in G, n :-

$$\gamma_d = \frac{W_s}{V} = \frac{\gamma_s \cdot V_s}{V_s + V_v} = \frac{G\gamma_w(1-n)}{1}$$

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

24 Aug 2020

(6) γ_{sat}, G, e or (n) :-

Let fully saturated soil :-

In fig-(1) :- $\gamma_{sat} = \frac{W_{sat}}{V} = \frac{W_d + W_w}{V}$

$$= \frac{\gamma_d \cdot V_d + \gamma_w \cdot V_w}{V_v + V_s} \quad (W_d = W_s)$$

$$= \frac{\gamma_s \cdot V_s + \gamma_w \cdot V_w}{V_v + V_s}$$

$$G = \frac{\gamma_s}{V_w}$$

$$\gamma_s = G V_w$$

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

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

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

γ_{sat} in G, n

In fig (ii) $\quad \gamma = \frac{\gamma_s \cdot V_s + \gamma_w V_w}{V_s + V_v}$

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

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

(7) γ, G, e & s :-

Let partially saturated soil :-

In fig (i) $= \frac{W}{V} = \frac{W_s + W_w + W_a}{V_s + V_v}$

$$= \frac{\gamma_s \cdot V_s + \gamma_w \cdot V_w + 0}{1+e}$$

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

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

$$\gamma = \frac{G\gamma_w + \gamma_w \cdot es}{1+e}$$

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

$$\boxed{\gamma = \frac{(G+es)\gamma_w}{1+e}}$$ ← partially saturated soil

(8) γ', G & e :-

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

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

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

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

(9) γ_d, γ, w :-

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

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

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

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

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

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

$$= \frac{W}{(1+w)V}$$

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

(10) γ', γ_d & n :-

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

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

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

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

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

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

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

(11) γ_{sat}, γ, γ_d & S

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

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

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

(12) γ_d, G, w & S

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

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

$$\boxed{\gamma_d = \frac{G\gamma_w}{1+wG/S}}$$ ← partially saturated soil

(fully saturated soil S = 1)

(13) γ_d, γ_s, w & na

$V = V_s + V_w + V_a$

$\Rightarrow V = V_a + \frac{W_w}{\gamma_w} + \frac{W_s}{\gamma_s}$

$$\left[\gamma_w = \frac{W_w}{V_w},\quad V_w = \frac{W_w}{\gamma_w}\right]$$

$\Rightarrow \frac{V}{V} = \frac{V_a}{V} + \frac{W_w}{\gamma_w \cdot V} + \frac{W_s}{\gamma_s \cdot V}$

$$\left[w = \frac{W_w}{W_d},\quad \Rightarrow W_w = w \cdot W_d\right]$$

$\Rightarrow 1 = \frac{V_a}{V} + \frac{W_w}{\gamma_w \cdot V} + \frac{W_s}{\gamma_s \cdot V}$

$\Rightarrow 1 = \frac{V_a}{V} + \frac{w \cdot W_d}{V \cdot \gamma_w} + \frac{W_s}{\gamma_s \cdot V}$ $(W_s = W_d)$

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

$$\Rightarrow 1 = \frac{V_a}{V} = \frac{w \cdot \gamma_d}{\gamma_w} + \frac{\gamma_d}{\gamma_s}$$

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

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

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

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

27 Aug 2020

(Q) A soil sample in it's undisturbed state was found to have volume of 105 cm^3 & mass of 201 gm after ovendrying the mass get reduced to 168 gm. Calculate.

Sol Given data = Total volume (V) = 105 cm^3

Total mass (M) = 201 gm.

Dry mass (M_d) = 168 gm.

(i) Water content (w) $= \frac{W_w}{W_s}$ $[W_s = W_d]$

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

$$= \frac{M_w \times g}{M_d \times g}$$

$= \frac{201 - 168}{168}$

$= 0.196 \times 100$

$= 19.6\ \%$

(ii) void Ratio(e) :-

$$e = \frac{G\,\delta_w}{\delta_d} - 1$$

$$\delta_w = 1g/cm^3\ ,\ \delta_d = \frac{M_d}{V}$$

$$= \frac{168}{105} = 1.6\ g/cm^3$$

$$e = \frac{2.7 \times 1}{1.6} - 1$$

$$= 0.687 \simeq 0.69$$

(iii) Porosity (n) :- $\frac{e}{1+e}$

$$= \frac{0.69}{1+0.69} = 0.408 = 40.8\%$$

(iv) Degree of saturation (S) :-

$$S = \frac{w \times G}{e} \quad \boxed{eS = wG}$$

$$= \frac{19.6 \times 2.7}{100 \times 0.69}$$

$$= 0.77$$

$$= 77\%$$

(v) Air content (a_c) :-

$$a_c = 100\% - S$$

$$= 100\% - 77\%$$

$$= 23\%$$

10 For a soil sample the specific gravity of soil mass (1.7) & specific gravity of soil particle (2.7). Determine the void ratio in case :-

(i) assuming soil sample is dry

(ii) soil sample has a water content of 12%

$\boxed{\text{Specific gravity is unit less}}$

Sol: Given data :- $G_m = 1.7$

$G = 2.7$

(i) soil sample is dry :-

$$G_m = \frac{\gamma}{\gamma_w} \text{ or } \frac{\gamma}{\gamma_w} \quad (\gamma_w = 1)$$

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

$$\Rightarrow \gamma_d = G_m \cdot \gamma_w$$

$$1.7 \times 1$$

$$= 1.7$$

Relationship :-

unit weight

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

(2) $\gamma_d = (1-n) G \cdot \gamma_w$

(3) $\gamma_{sat} = G\gamma_w(1-n) + \gamma_w n$

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

Density

(1) $\rho_d = \frac{G \cdot \rho_w}{1+e}$

(2) $\rho_d = (1-n) G \rho_w$

(3) $\rho_{sat} = G \rho_w(1-n) + \rho_w n$

(4) $\rho' = \frac{(G-1)\rho_w}{1+e}$

(5) $\gamma' = \gamma_d - (1-n)\gamma_w$ (5) $\rho' = \rho_d - (1-n)\rho_w$

(6) $\gamma_d = \dfrac{G\gamma_w}{1+\frac{wG}{S}}$ (6) $\rho_d = \dfrac{G\rho_w}{1+\frac{wG}{S}}$

(7) $e = \dfrac{G\gamma_w}{\gamma_d} - 1$ (7) $e = \dfrac{G\rho_w}{\rho_d} - 1$

(8) $\gamma_{sat} = \dfrac{(G+e)\gamma_w}{1+e}$ (8) $\rho_{sat} = \dfrac{(G+e)\rho_w}{1+e}$

(9) $\gamma = \dfrac{(G+eS)\gamma_w}{1+e}$ (9) $\rho_{sat}\ \dfrac{(G+e)\rho_w}{1+e}$

(10) $\gamma_d = \dfrac{\gamma}{1+w}$ (10) $\rho_d = \dfrac{\rho}{1+w}$

(11) $\gamma = \gamma_d + S(\gamma_{sat} - \gamma_d)$ (11) $\rho = \rho_d + S(\rho_{sat} - \rho_d)$

(12) $\gamma_d = \dfrac{(1-n_a)G\gamma_w}{1+wG}$ (12) $\rho_d = \dfrac{(1-n_a)G\rho_w}{1+wG}$

10 A soil sample has a porosity of 40%. The specific gravity of solid 2.7 calculate.

(a) void ratio

(b) dry unit weight

(c) unit weight if the soil is 50% saturated

(d) unit weight if the soil is completely saturated.

Given data :-

$n = 40\%$

$G = 2.7$

(i) $e = \frac{n}{1-n} = \frac{0.40}{1-0.40} = 0.66$

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

$= \frac{2.7 \times 9.81}{1+0.66} = 15.9560\ kN/m^3$

(iii) If the soil is 50% saturated

$S = 50\% = 0.50$

$\gamma = \frac{(G+eS)\gamma_w}{1+e} = \frac{(2.7+0.66\times0.50)\times9.81}{1+0.66}$

$= 17.90\ KN/m^3$

(iv) If the complet saturated

$S = 100\% = 1$

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

$= \frac{(2.7+0.66)\,9.81}{1+0.66}$

$= 19.85\ kN/m^3$

29 Aug 2020

Q8 A soil has a bulk unit weight of 20.11 kN/m³ & water content of 15%. Calculate the water content if the soil partially dries to a unit weight of 19.42 kN/m³ & the void ratio remains unchanged.

Bulk unit weight γ_1
$= 20.11\ KN/m^3$
$W = 15\% = 0.15$
$\gamma_2 = 19.42\ KN/m^3$

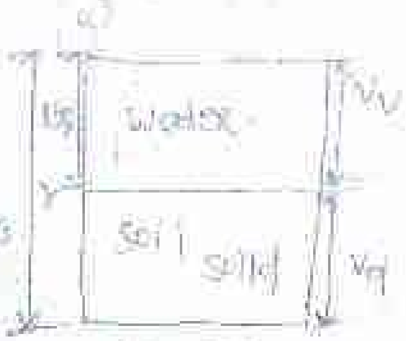


fig-(i)

void remain unchanged

$\gamma v_1 = \gamma v_2$
$vd_1 = vd_2$
$vd_1 = \gamma d_2$

fig-(ii)

In diagram (i) $= \gamma_1 = 20.11\ KN/m^3$
$\omega = 15\%$
$vd_1 = vd_2 = 17.48\ KN/m^3$

$\gamma d_1 = \frac{\gamma_1}{1+\omega_1}$

$= \frac{20.11}{1+0.15} = 17.48\ KN/m^3$

In diagram (ii) $\gamma d_2 = \frac{\gamma_2}{1+\omega}$

$\Rightarrow 17.48 = \frac{19.42}{1+\omega}$

$\Rightarrow 1+\omega = \frac{19.42}{17.48}$

$\Rightarrow \omega = \frac{19.42}{17.48} - 1$

$= 0.11$

$= 11\%$

Q) The in-situ percentage void of a sand deposite is 34%. for determining the density index, dried sand from the stratum was first filled loosely in a 1000 cm³ mould & was then vibrated to give a maximum density. The loose dry mass in the mould was 1610 gm. & the dense dry mass at maximum compaction was found to be 1980 gm. determine the density index if the specific gravity (G) = 2.67.

Given data :- $n = 34\%$

$V = 1000\,cm^3$

$e_{maximum}$ = loose soil mass = 1610 gm (γ_{min})

$e_{minimum}$ = dense soil mass = 1980 gm. (γ_{max})

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

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

$$e_{max} = \frac{G\gamma_w}{\gamma_{d\,min}} - 1 \qquad \begin{bmatrix} \gamma_w = 1 \\ G = 2.67 \end{bmatrix}$$

$$\gamma_{d\,min} = \frac{M_d}{V} = \frac{1610}{1000} = 1.61\,gm/cm^3$$

$$\gamma_{d\,max} = \frac{1980}{1000} = 1.98\,gm/cm^3$$

$$e_{max} = \frac{2.67 \times 1}{1.61} - 1 = 0.67$$

$$e_{min} = \frac{2.67 \times 1}{1.98} - 1 = 0.36$$

$$I_D = \frac{0.67 - 0.515}{0.67 - 0.36} = 0.5 = 50\%$$

(NEW - CHAPTER)

31 Aug 2020 Index properties of soil

Those properties which are used for soil classification is known as Index property.

(i) Water Content
(ii) specific gravity
(iii) particle size distribution
(iv) Consistency Limit
(v) In-situ density
(vi) Density Index

(i) Water Content

- oven-dry method
- sand bath
- Alcohol
- Calcium carbide method
- Pycnometer method
- Radiation method
- Torsion balance method

(1) oven drying :-

→ This is the most accurate method of determining the water content & is used in Laboratory.

=> A specimen of soil sample is kept in a clean container & put it in a thermostatically controlled oven with interior of non-corroding material to maintain the temperature between 105° C to 110° C.

=> For complete drying sandy soil takes 4 hours & for clays take about 14 to 16 hours.

=> usually the sample kept 24 hour for complete drying.

=> After drying the container is removed from the oven & allowed to cool.

mass of container with lid (M_1)

Mass of container with lid + wet soil (M_2)

$$\boxed{\text{water content } (w) = \left(\frac{M_2 - M_3}{M_3 - M_1}\right) \times 100}$$

Note :- soil oven temperature should not cross 110° C because it break the crystalline structure of clay particle.

=> If gypsum is suspected in the soil, then oven temperature should not more than 80° C.

=sion-
tance
ethod

od of
ination

(B) Sand bath method :-

-> This is a field method, where the facility of an oven is not available

-> The container with the soil is placed on a sand bath is heated over a kerosene stove.

-> The soil becomes dry within 1/2 to 1 hour.

-> This method is not suitable for organic soil & soil having gypsum

$$\boxed{\text{So} \quad w = \left(\frac{M_2 - M_3}{M_3 - M_1}\right) \times 100}$$

where M_1 = mass of empty container with lid

M_2 = mass of container with lid + wet soil

M_3 = mass of container with lid + dry soil

(C) Alcohol method :-

-> This is a field method.

-> The wet soil sample is kept in a evaporating dish & mixed with sufficient quantity of methylated spirit

-> When dish is properly covered & the mixture is ignited.

-> The mixture is kept stirred by by a wire during ignition.

→ Since there is no control over the temperature it sould not be used for soils containing large percentage of organic matter & gypsum.

$$\boxed{w = \left(\frac{m_2 - m_3}{m_3 - m_1}\right) \times 100}$$

where w = water content

m_1 = mass of empty dish,

m_2 = mass of empty dish + wet soil

m_3 = mass of empty dish + dry soil

(8). Calcium carbide method :-

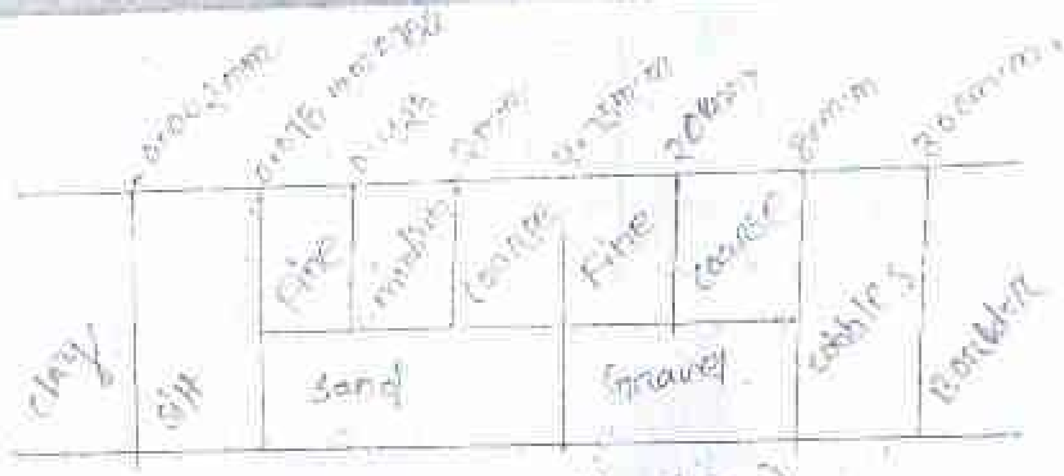
→ In this method 6 gm. of wet soil sample is placed in an air-tight container called moisture tester & is mixed with sufficient quantity of fresh calcium carbide powder.

→ The mixture is taken vigorously.

→ The acetylene gas produced by the reaction of the moisture of the soil & the calcium carbide exerts pressure on diaphragm placed at the end of the container & it reads the water content directly.

→ The calibration of the diagauge is such that it gives the water content (w') of the wet weight of the sample.

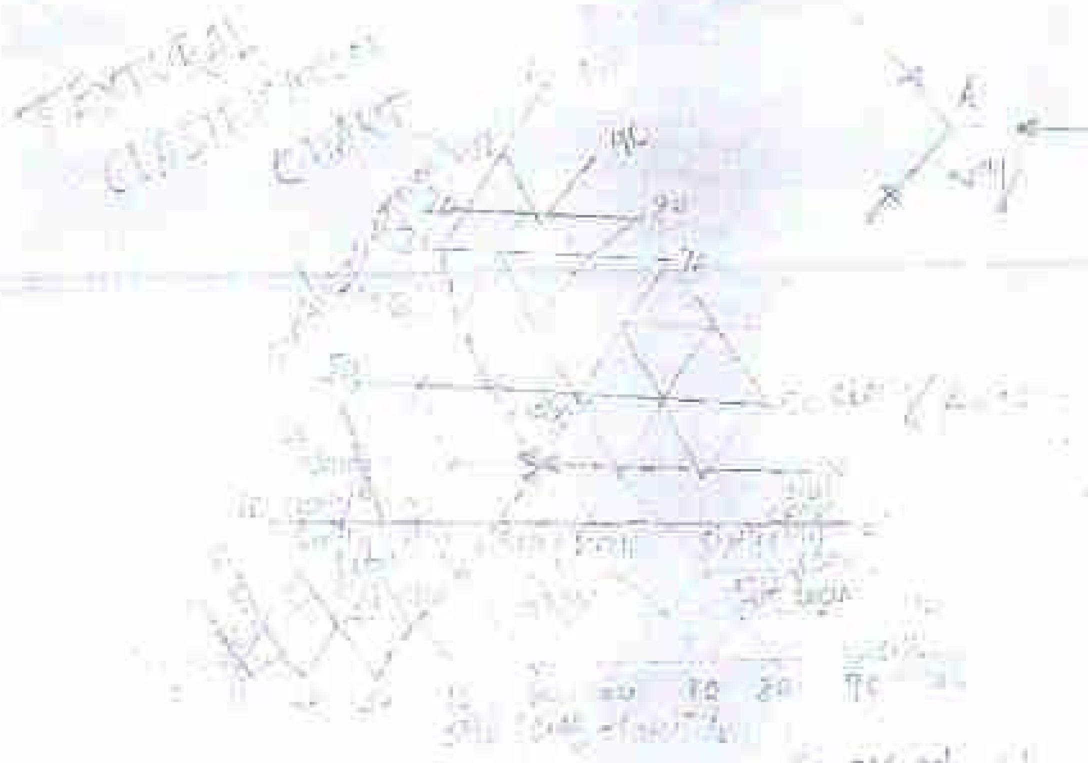
→ $\boxed{\text{So actual } w = \frac{w'}{1-w'}}$



(I.S classification)

19 sep 2020

(ii) Textural classification :-



1) Soil occurring in nature are composed of differ clay, sand, silt sizes mixtures.

2) Soil classification is based on the particle size distribution is known as textural classification.

3) This classification is a triangular classification, use by the "U.S. Public Road administration".

4) This classification is based on the percentage of sand, silt & clay size making the soil.

5) This classification is suitable for designing coarse

=> This represent by a equilateral ~~triangular~~ triangle.

=> For example:- If a soil is composed of 30% sand, 30% silt & 40% clay, then the three dimension drawn and intersect at point 'z'.

=> This point is situated near clay - so that this type of clay is known as clayey soil.

(3) Highway Research Board classifications

=> Highway research board classification system, also known as public road administration.

=> It is based on the particle size composition & plasticity characteristics.

=> This system is mostly used for permanent construction.

=> The soil is devided into 7 groups i.e. A-1, A-2, A-3 .... A-7

=> A characteristic group Index is used to describe the performance construction.

=> Group index actually means of rating the values of soil of a subgrade material with in it's own group.

=> Higher the value of group index poorer the quality of material.

=> The group index depends upon the liquid limit, plastic limit & amount of material passing the 75 μ IS sieve values of soil of a subgrade material within in it's own group.

=> Higher the value of group index poorer the quality of material.

=> The group index depends upon the liquid

{ limit, plastic limit & amount of
material passing the 75-micron sieve

$$\boxed{\text{Group Index} = 0.2a + 0.005ac + 0.01bd}$$

where a = portion of percentage passing 75 μ sieve, lies between (35 < a < 75)% or in no between "0" to "40"

b = portion of percentage passing through 75 μ sieve, lies between (15 < b < 55)%, in no "0" to "40"

c = portion of liquid limit greater than 40 & not exceeding 60 in whole no (0 to 20).

d = portion of plasticity Index greater than 10 & not exceeding 30 in whole number (0 to 20)

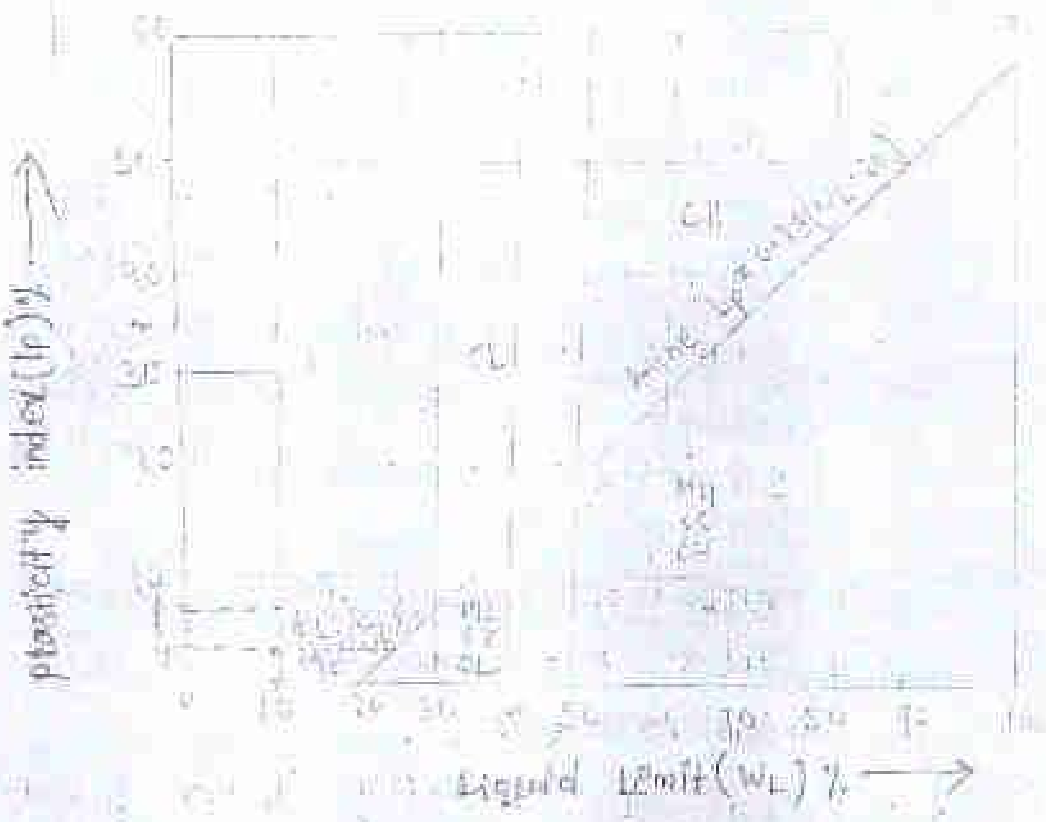
35% >
↓
A-1, A-3, A-2

35% <
↓
A-4, A-5, A-6, A-7

(4) unified soil classification system :-

- coarse grained
- fine grained
- organic
- peat

(1)



CASKARANDE'S PLASTICITY CHART (USCS)

22 Sep 2020

→ Based on the symbol soils are devided is to 15 groups.

- 8 groups are coarse grained
- 6 groups are fine grained
- 1 group is peat

Sl.No	soil type	prefix	subgroup	suffix
(1)	Gravel	G	(1) well graded	W
(2)	sand	S	(2) poorly	P
(3)	Silt	M	(3) silty	M
(4)	clay	C	(4) clayey	C

(5) organic O (5) $w_L < 50\%$ L

(6) peat Pt (6) $w_L > 50\%$ H

(1) Coarse - grained :-

=> If more than 50% of the soil particle retain on 75µ sieve then it is known as coarse grained soil.

=> A coarse grained soil is designated as "Gravel (G)" when 50% or more of the soil fraction is retained on the sieve 4.75 mm.

=> If the soil particle passing through the 4.75 mm sieve then it is called sand.

=> Coarse grained soil containing less than 5% fines, are designated by the symbol "GW" & "SW" when they are well graded or used "GP" & "SP" when they are poorly graded.

=> When % fine is more than 12% for coarse grained soil, then it is designated by the symbol :- GM, SM, GC & SC

=> If coarse grained soil % fine is in between 5% - 12% then it is designated by dual symbol i.e :- GW - GM, SP - SM

(2) Fine grained -

=> soil is designed as fine grained if the soil is 50% or more than that passing through the 75µ sieve

=> Fine grained soil is devided in to 2 types :- (i) silt (ii) clay

⇒ This fine grained soil is based on liquid limit (W_L) & plasticity Index (I_p).
⇒ organic soil is also included in the fine grained soil
⇒ To represent the "USCS" casagrande used a plasticity chart plotted between liquid limit & plasticity Index.

* ~~Indian standard soil classification~~ :- (IS)

⇒ Based on liquid limit fine grained soil sub devided into 2 groups-
(i) W_L < 50% = low plasticity soil (L)
(ii) W_L > 50% = High plasticity soil (H)
⇒ when the liquid limit decreased to 30% or more than it is said to be organic soil, otherwise it is said to be inorganic soil.

* Indian standard soil classification (IS) :-

⇒ IS soil classification system is first developed in 1959 & in 1970 it is revised (casagrande
⇒ The revised version is based on "USCS" with the modification grouped (i.e low compressibility, medium compressibility & high compressibility)
⇒ Instead of using 15 groups from "USCS" system, 18 groups are used.
⇒ soils are classified into 3 major groups :- (i) coarse grained
(ii) fine grained

(iii) Highly organic soil

23 Sep 2020

(i) Coarse grained soil :-

> In these soils, more than half of the total mass of material is larger than 75 µ sieve

> Coarse grained soils are subdevided in to 2 types :- (a) Gravel (G)

(b) Sand (S)

(A) Gravel :- If 50% or more than that is larger than the 4.75 mm sieve is called gravel & it is simbolised by "G"

(B) Sand :- If 50% or more than that is smaller than the 4.75 mm sieve is called sand & it is simbolised by "S"

NOTE :-

> Each of the subdivision devided in to 4 groups depend upon grading that are :-

W = well graded

C = well graded with excellent clay

P = poorly graded

M = containing fine material

(ii) Fine grained soil :- In these soils 50% or more than that material by mass is smaller than 75 µ sieve.

Fine grained soils are further devided into 3 sub-division that are :-

(a) Inorganic silt & very fine sand (M)

(b) Inorganic clay (C)

(c) organic silt or clay or organic matter (O)

⇒ Based on liquid limit it is again devided into 3 types.

(<35) (a) Silt & clays of low compressibility

⇒ When the liquid limit is less than 35% then that soil is said to be low compressibility (L).

(35-50) (b) Silt & clay of medium compressibility

⇒ when the liquid limit of sample is greater than 35% and smaller than 50% then it is said to be medium compressibility (I)

(50) (c) silt & clay of high compressibility

⇒ when the liquid limit is greater than 50% then it is said to be high compressibility (H).

∴ With the help of plasticity chart i.e A line using for separating inorganic clay from silt & organic soils

$$\boxed{I_p = 0.73\,(w_L - 20)}$$

28 Sep 2020

Permeability

-> Permeability is defined as the property of a porous material which permits the passage or seepage of water through it's interconnecting voids.

-> A material having continuous voids is known as permeable.

-> Gravel are highly permeable which stiff clay is least permeable so it is called impermeable for all purpose.

-> The flow of water through soils may either laminar flow or turbulent flow.

-> In laminar flow each fluid particle flow through them or travels along a definite path in which no particle cross each others path.

-> In turbulent flow particles are irregularly flow with crossing the each other's path.

-> For practical purpose mostly consider laminar flow.

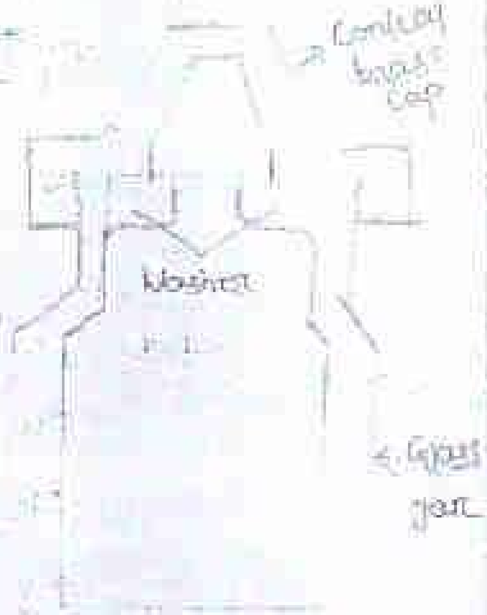
29 Sep 2020

Darcy's Law

-> The law of flow through soil was first studied by French scientist Henry Darcy & this law is known as "Darcy's law"

→ This method is very quick & gives the result in 5 to 10 minutes.

(E) pycnometer method :-



→ This is also a quick method for determining the water content of those soils whose specific gravity (G) is known.

→ Take a clean, dry pycnometer & find its mass with its cap & washer (M_1)

→ put about 200 gm to 400 gm wet soil sample in the pycnometer & find its mass with cap & washer (M_2)

→ Fill the pycnometer to half its height & mix it thoroughly with the glass rod.

→ Add more water & stir it to remove the air from it & then take its mass (M_3)

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

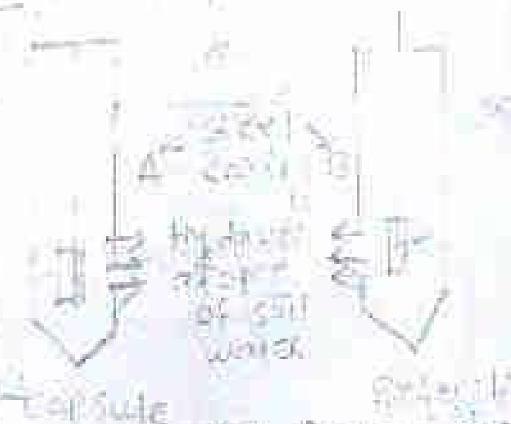
So $\boxed{\text{water content} = \left[\left(\frac{M_2-M_1}{M_3-M_4}\right)\left(\frac{G-1}{G}\right)-1\right]\times 100}$

Empty pyc (M_1) pyc + wet soil (M_2) pyc + soil + water (M_3) pyc + water (M_4)

Note:- This method is suitable for large grained soil only.

(F) Radiation Method:-

=> It uses two steel casing A & casing B which are placed in two bore holes at some distance apart.



=> A device containing capsule some radio-active isotope material (cobalt-60) is placed in a capsule which in turn is lowered into casing A.

=> Similarly a detector unit is lowered in steel casing B.

=> Small openings are made in both casing A & B facing each other.

=> Radio-active device is activated it emits neutrons & these neutron strike with the hydrogen atoms of water in the soil & they lose energy.

& this loss of energy is evidently equal to water content in the soil & which is detected by detector.

(G) Torsion Balance method:-

=> The equipment has two main parts.

(i) Infra-red lamp

(ii) Torsion balance.

=> The infra-red radiation is provided by 250 watt lamp built in the balance for use with alternating current 220-230V, 50 cycle single phase main supply.

⇒ The test specimen is kept in a suitable container so that the water content to be determined is not affected.

⇒ normal time varies between 15 to 30 minutes & temperature between 110° & 50°C mentioned in thermometer.

⇒ from the test water content of wet mass is w'

$$\boxed{\text{actual water content, } w = \frac{w'}{1-w'}}$$

2nd sep 2020

Q. The in-situ density of an embankment compacted at a water content of 12% was determined with the help of a core cutter. The empty mass of the cutter was 1286 g and the cutter full of soil had a mass of 3195 g. the volume of the cutter being 1000 cm3. Determine the bulk density, dry density and the degree of saturation of the embankment. If the embankment becomes fully saturated during rains, what would be its water content and saturated unit weight? Assume no volume change in soil on saturation. The specific gravity of the soil as 2.70.

Given data :- volume of cutter = 1000 cm³

mass of cutter = 1286 gm

mass of cutter soil sample = 3195 gm.

$w = 12\%$

$G = 2.7$

case(i) :- $\rho = \dfrac{\text{mass of soil sample}}{\text{volume of soil sample}}$

$$= \frac{3195 - 1286}{1000}$$

$$= 1.90 \text{ g/cm}^3$$

(ii) $$\rho_d = \frac{\rho}{1+w}$$

$$= \frac{1.90}{1+0.12} = 1.69 \text{ g/cm}^3$$

(iii) $$\rho_d = \frac{G\rho_w}{1 + \frac{wG}{S}}$$

$$\Rightarrow 1.69 = \frac{2.7 \times 1}{1 + \frac{0.12 \times 2.7}{S}}$$

$$\Rightarrow 1 + \frac{0.32}{S} = \frac{2.7}{1.69}$$

$$\Rightarrow \frac{0.32}{S} = \frac{2.7}{1.69} - 1$$

$$\Rightarrow \frac{0.32}{S} = 0.597$$

$$\Rightarrow S = \frac{0.32}{0.597} = 0.536$$

$$= 53.6\%$$

$$eS = wG$$

$$\Rightarrow w = \frac{eS}{G} = \frac{0.597 \times 1}{2.7}$$

$$= 0.221 \times 100\%$$

$$= 22.1\%$$

(i) $eS = wG$

$\Rightarrow w = \frac{eS}{G} = \frac{0.597\times 1}{2.7}$

$= 0.2211 \times 100\%$

$= 22.11\%$

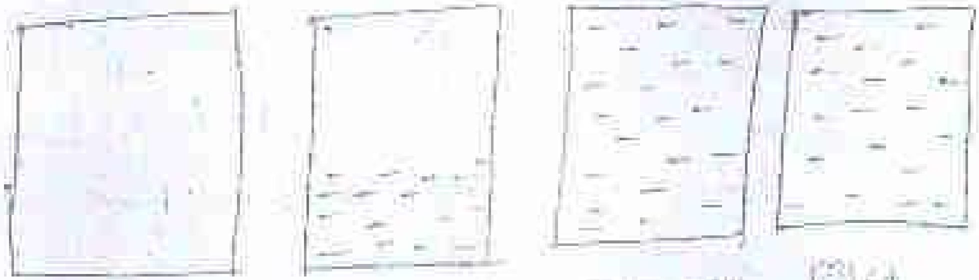
(ii) $\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$

$= \frac{(2.7+0.597)\times 9.81}{1+0.597}$

$= 20.25\ kN/m^3$

2 Sep 2020

(1) Specific Gravity :-



=> The specific gravity of soil solids is determined by

(i) sand density bottle

(ii) 500 ml flask

(iii) a pycnometer

=> The density bottle method is the most accurate method, which is used for all type of soil.

=> pycnometer method is only suitable for coarse grained soil.

=> First take a empty pycnometer with its mass (M_1)

=> Then take wet soil sample & pour it in the pycnometer & take the mass of pycnometer & wet soil sample i.e (M_2)

=> After that add little water at a gap & stir the sample with a wire to remove the air voids.

=> Then completely fill the pycnometer & soil sample & water with water & take its mass (M_3)

=> Then clean the pycnometer & only fill up the pycnometer with water, take its mass (M_4)

=> After observing all the mass then the specific gravity of that soil sample is :-

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

(3) Consistency limit :- 24/sep/2020

=> Consistency denotes degree of firmness of the soil i.e either the soil is soft or hard or stiff.

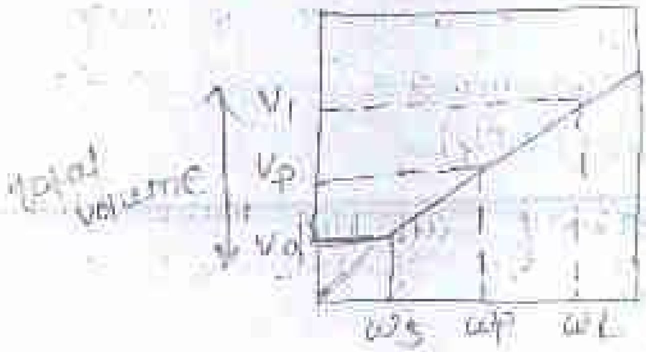
=> This consistency limit is derived by Albert Atterberg so it is also known as atterberg limit.

=> Atterberg devide the soil state from liquid to solid in to 4 stages.

(i) Liquid state
(ii) plastic state
(iii) semi-solid state
(iv). solid state

-> This atterberg limit is very much well for engg purpose.

Atterberg set limits based on the ~~based~~ found state of soil i.e
(i) Liquid limit
(ii) plastic limit
(iii) shrinkage limit



(i) Liquid limit W_L

=> Liquid limit is the water content which arbitarily between liquid & plastic state.

=> It is defind as the minimum water content at which soil is still in liquid state, but has small shear strength against flowing.

=> Liquid limit is found out by using two instrument i.e:-
(a) casagrande
(b) cone tool

(ii) Plastic Limit (w_P) :-

→ Plastic limit lies between plastic state and semi-solid state.

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

(iii) Shrinkage Limit :-

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

(iv) Plasticity Index (I_P) :-

The range of consistency within a soil exhibits plastic properties is called plasticity range & it is indicated by plasticity Index.

→ Plasticity Index is the difference between liquid limit and plastic limit.

$$\boxed{I_P = W_L - W_P}$$

Liquid limit - plastic limit = Plasticity Index.

→ When plasticity Index can not be determined then that soil is said to be non-plastic.

→ When plastic limit is equal to or greater than liquid limit the I_P will be zero.

(v) Consistency Index :-

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

=> It is ratio of difference between liquid limit & natural water content to the plasticity Index.

I_p = plasticity Index

w_L = Liquid limit

w = Natural water

=> Consistency Index is useful for study the field behaviour of fine grained soil.

Note :- (1) I_c = 1 → plastic limit

(2) I_c = 0 → Liquid limit

(3) I_c > 1 → Semi-soil state

(4) I_c = -ve → the state of soil is in liquid state.

(vi) Liquidity Index (I_L) :-

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

It is ratio of difference between natural water content & plastic limit to the plasticity Index

w = natural water content

w_p = plastic limit

I_p = plasticity Index

11 Sep 2020

FLOW INDEX :- Flow index or slop of the curve can be determined by :-

$$\boxed{\dfrac{w_1 - w_2}{\log_{10} \dfrac{n_2}{n_1}}}$$

Where w_1 = water content at 'n_1' no of blow.

w_2 = water content at 'n_2' no of blow.

Toughness Index :- (I_T)

Toughness index is defined as the ratio of plasticity Index to the flow Index

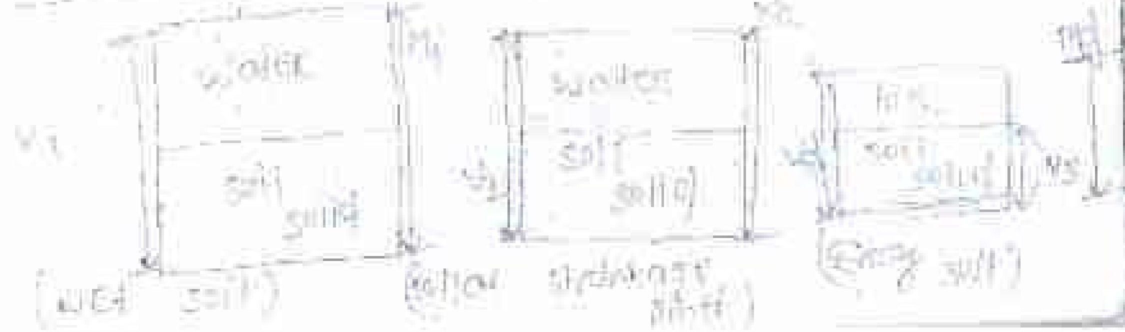
$$\boxed{I_T = \dfrac{I_P}{I_F}}$$

NOTE :-

(1)

Ip%	Plasticity
(a) 0	Non - plastic
(b) < 7	Low plastic
(c) 7-17	medium plastic
(d) > 17	Highly plastic

Shrinkage limit :-



$$\boxed{w_s = \frac{(M_1 - M_d) - (V_1 - V_d)\,\gamma_w}{M_d}}$$

where (M_1) = mass of wet soil

(M_d) = mass of dry soil

(V_1) = volume of wet soil

(V_d) = volume of dry soil (found out from mercury displacement method)

(or) w_s can be determine by alternative method, when using G is known as :-

$$w_s = \frac{\text{Mass of water at Shrinkage limit}}{\text{Mass of soil solid}}$$

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

where V_1 = volume of soil mass at any water content

V_d = volume of dry soil.

Shrinkage ratio :- (SR)

⇒ when a wet soil mass with its water content above shrinkage limit is dried to a water content - then there will be reduction in water content & it is known as volumetric shrinkage.

→ So shrinkage ratio is the ratio between "V_s" to the reduction in water content.

$$\boxed{SR = \frac{V_s}{w_1-w_2} = \frac{\frac{V_1-V_d}{V_d}\times 100}{w_1-w_2}}$$

$$V_s = SR(w_1-w_2)$$

Q. The mass & volume of a saturated clay specimen is 29.8 g & 17.7 cm³ on oven drying the mass get reduce to 19 gm & the volume to 8.9 cm³ calculate.

(i) Shrinkage Limit

(ii) Shrinkage ratio

(iii) volumetric shrinkage

(iv) G_s

Sol. wet soil mass = 29.8 g = M_1

of wet soil = (M_d) = 19 g

volume

V_d = 8.9

(i) $w_s = \frac{(M_1 - M_d) - (V_1 - V_d)\gamma_w}{M_d}$

$= \frac{(29.8 \times 19) - (17.7 \times 8.9) \times 1}{19}$

$= \frac{0.1052}{V_s} = 10.52\%$

(iii) $w_1 - w_2 = \frac{w_1 \omega_1}{w_d \omega_1} - \frac{w_2}{w_d \omega_2}$

$= \frac{w_1}{w_d} - \frac{w_2}{w_d}$

$$= \frac{(W_{w_1} - W_{w_2})}{W_d}$$

$$= \frac{V_{w_1} \times \gamma_w - V_{w_2} \times \gamma_w}{W_d}$$

$$= \frac{(V_1 - V_2)\,\gamma_w}{W_d} = \frac{(V_1 - V_d)\,\gamma_w}{W_d} \quad \text{--- eq (i)}$$

$$SR = \frac{\frac{V_1 - V_d}{V_d} \times 100}{\frac{V_1 - V_d}{W_d} \times \gamma_w}$$

$$= \frac{W_d}{V_d \times \gamma_w} = \frac{M_d}{V_d \times g_w} = \frac{19}{8.9 \times 1}$$

$$= 2.13$$

(iii) $$V_S = \frac{V_1 - V_d}{V_d} \times 100$$

$$= \frac{17.7 - 8.9}{8.9} \times 100$$

$$= 98.8\%$$

(iv) $$\gamma_f = w_s = \frac{V_d \times g_w}{M_d} - \frac{1}{G_i}$$

$$\Rightarrow \frac{1}{G_i} = \frac{V_d \times g_w}{M_d} \times w_s$$

$$\Rightarrow \frac{8.9 \times 1}{19} - 0.103 = 0.363$$

$$\frac{1}{G_i} = 0.363$$

$$G_i = \frac{1}{0.363} = 2.75$$

12 sep 2020

NOTE

Degree of shrinkage %	Quality of soil
(i) ≤ 5	Good soil
(ii) 5 - 10	medium
(iii) 10 - 15	poor
(iv) > 15	very poor soil

(5)

particle size distribution (5)

mechanical analysis distribution

Sieve analysis

> 75 μ - 4.75 mm

Sedimentation analysis

≤ 75 μ ≥ 0.2 mm

particle size distribution :-

=> The percentage various size of particle is found out by particle size analysis or mechanical analysis.

=> This means separation of soil into its different size fraction.

=> This is performed by 2 method s. they are (i) sieve analysis

(ii) sedimentation analysis

Sieve analysis is meant for coarse grained soil & sedimentation analysis ... are ... fine grained soil

(ii) Sieve analysis :-

→ This method only used for coarse grained soil that is from 4.75 mm - 75 µ size of particle.

→ In BS (British standard) & ASTM (American society of testing & material) the sieve sizes are given in terms of no of opening.

→ In IS (Indian standard) sieve size are designated by the size of aperture in mm.

→ The complete sieve analysis devide-d into 2 parts, one is coarse & other one will be fine.

→ The following sets of sieve are used for the test are :- IS = 100, 63, 20, 10, 4.75

IS = 2mm, 1mm, 600µ, 425µ, 300µ, 212µ, 150µ & 75µ.

Fine

→ Then sieving is performed by arranging the various size of sieves one over other in of their mesh opening.

→ The largest aperture sieve is placed at the top & smaller aperture is at the bottom.

→ At the end pan or receiver will place & at top cap.

⇒ The soil sample is put on the top sieve & the whole assembly is fitted on mechanical sieve shaker.

⇒ The amount of shaking depends on the shape & no of particle.

⇒ But generally it took 10 minutes.

⇒ The portion of the soil sample retained on each sieve is weighted.

⇒ The percentage of soil retained on each sieve is calculated on the basis of total mass of soil sample taken & from these results.

⇒ If silt & clay particles stick to the sieve then use a brush to remove the particles from each sieve.

⇒ If some soil still stick with the sieve then use dispersing agent sodium hexametaphosphate of 2g mixed with 1 litre of water & used is mixture for washing till the soil particles pass through this.

$$\boxed{\% \text{ soil retained} = \frac{\text{mass of soil retained}}{\text{Total mass}} \times 100}$$

(2) Sedimentation Analysis :-

⇒ In the wet mechanical analysis or sedimentation analysis the soil fraction finer than 75µ size is kept in suspension.

⇒ The analysis is based on "Stokes" law

→ According to "Stokes law" the soil particles set the in the suspension with a velocity.

→ It is assumed that all the particles are spherical & the coarse particles sets quickly than the finer particle.

→ This velocity from "Stokes" law, known as setting velocity (or) terminal velocity (v)

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

$$\boxed{\frac{2}{9} \times R^2 \times \frac{\gamma_s - \gamma_w}{\eta}}$$

where :- v = terminal velocity

R = Radius of the spherical particle (m)

D = Diameter of the spherical particle (m)

γ_s = unit wt of soil solid

γ_w = unit wt of water

η = viscosity of water (kN-s/m²)

(ETA)

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

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

15 Sep 2020

Limitation of sedimentation analysis :-

=> The analysis is based on the following assumptions :-

(i) soil particles are spherical.

(ii) particles settle independent of other particles & they do not affect the other particles.

(iii) The way of jar in which suspension is kept also donot affect the settlement.

These 3 assumptions practually not possible because all the particles are not in spherical shape. when the particle settle down walls are affect & also each particle strike with other while setting.

There are 2 methods used to distribute the particles less than 75μ are:-

(a) Pipette method

(b) Hydrometer method

(i) pipette method

fig. 1

(sampling pipette)

(pipette sampling apparatus)

→ This method is a standard sedimentation method used in the laboratory.
→ The equipment consist of a pipette, jar & sampling bottles & boiling tube of 500ml capacity.
→ The particles finer than 75µ size are distributed by this method.
→ About 10 to 20gm of oven-dried soil is mixed with distilled water in a beaker & form a thin paste.
→ To have proper dispersion of soil, a dispersing agent (such as sodium hexametaphosphate & sodium carbonate) of 2 spoon mixed in the solution & mixture is mixed for 5 minutes.
→ Then the sample transformed to the 500ml capacity of boiling tube & close the tube & shaken several times.
→ Then step with stored soil sample are collection at various time interval. Such as:- $\frac{1}{2}$ min, 1min, 2min, 4min, 8min, 15min, 30min, 1 hour ... 24 hour.
→ At the time of pipette should be inserted in the boiling tube about 25 seconds before the selected time interval & time taken for sucking the sample should not be more than 10 to 20 seconds.
→ When the collection sample by pipette is 10ml of volume is placed in sampling bottle & kept it for oven drying.

Calculation :-

$$\boxed{\text{Mass per cc of suspension } (M_D) = \frac{\text{Dry mass of sample}}{V_P}}$$

where V_P = volume of pipette = 10ml.

$$\boxed{\text{percentage finer } (N') = \frac{M_D - \frac{M}{V}}{M_D/N} \times 100}$$

where M = mass of dispersing agent (i.e. 7gm of sodium carbonate & 33gm sodium hexametaphosphate)

$\Rightarrow \frac{33+7}{1000} \times 25 = \frac{1}{...}$ gm mass (25ml dispersing agent solution)

V = volume of suspension = 500ml

(ii) Hydrometer method :-

=> It differs from pipette analysis
=> first calibrate the hydrometer.
=> the reading on the hydrometer give the density of the soil suspension situated at the centre of the bulb at any time.
=> For convenience hydrometer readings are recorded after subtracting 1 & multiplying the remaining digits by 1000.
=> such reduce reading is "R_h"

$$\boxed{H_e = H_1 + \frac{1}{2}\left(h + \frac{V_h}{2A}\right) - \frac{V_h}{A} = H_1 + \frac{1}{2}\left(h - \frac{V_h}{A}\right)}$$

=> In this volume of suspension is 1000ml.
quantity of dry soil & dispersing agent is taken double.
=> stopwatch simultaneously started & readings are taken at 1/2, 1 & 2 minutes
..... 1, 2, 4 .... taken.
=> To take the readings hydrometer is inserted about 30 seconds before the interval.
=> then take the reading by doing some correction due to meniscus, temperature & dispersing agent.

Calculation :- Corrected hydrometer reading

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

where R_h' = observed hydrometer reading
C_m = corrected meniscus
C_t = corrected temperature
C_d = corrected dispersing agent

% soil

$$\boxed{N = \frac{R}{1000}\left(\frac{G}{G-1}\right)}$$

where G = specific gravity of soil solid (v = 1000ml)

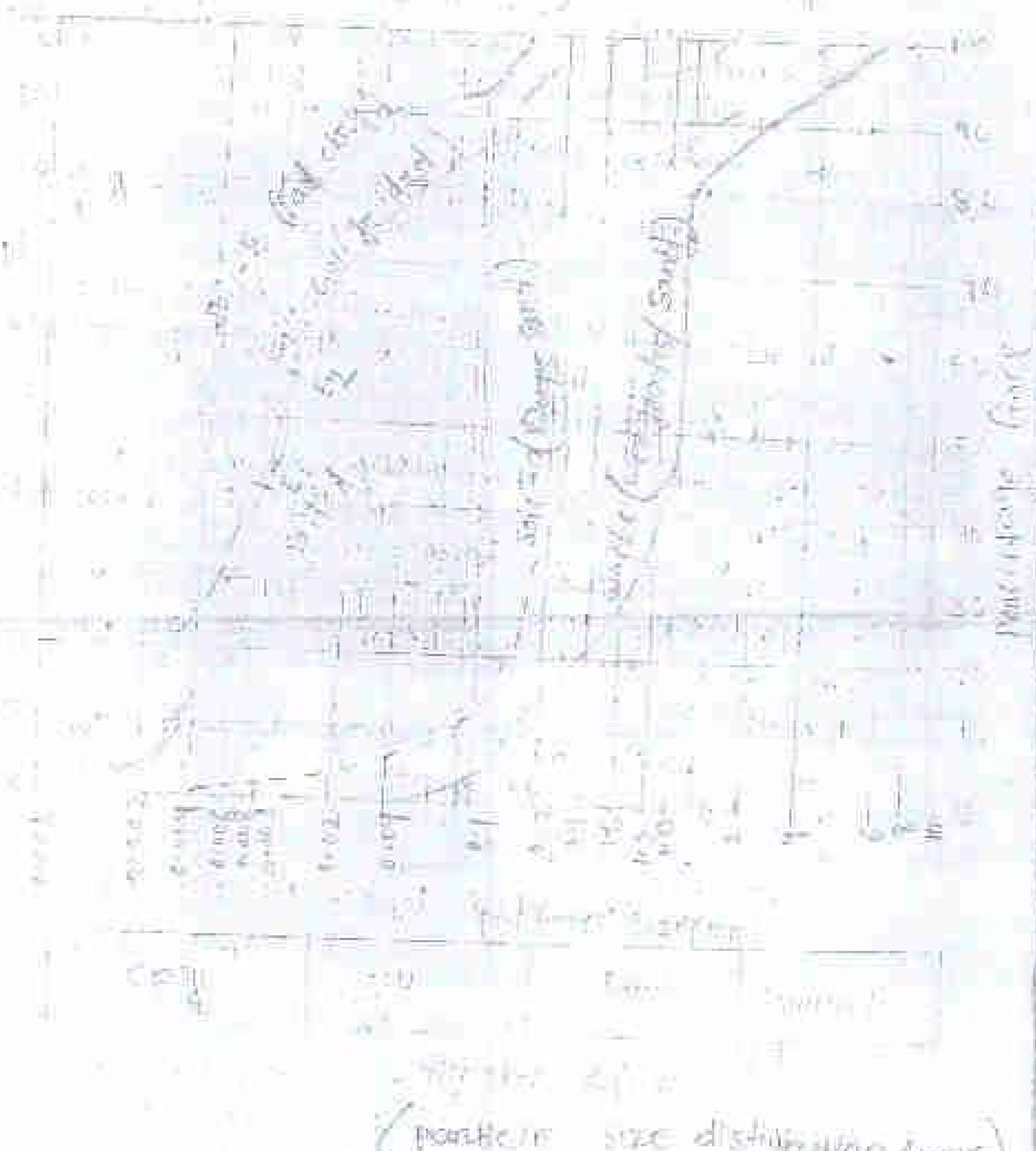
$$N' = \frac{R}{1000} \times \left(\frac{G}{G-1}\right) \times \frac{100}{M_d/V}$$

$$\boxed{N' = \frac{100\,G\,R}{M_d\,(G-1)}}$$

where N' = percentage finer.

Particle size distribution curve :

Soil Mechanics and Found



(particle size distribution curve)

⇒ The results from mechanical analysis are plotted on a graph which is known as particle size distribution curve.

⇒ It is the graph plotted between particle size & percentage finer (N)

⇒ A particle size distribution curve gives idea about different size or gradation of various soils

⇒ A soil may be either well graded or poorly graded.

⇒ well graded soil is the any combination of one type of soil particles.

⇒ In the above figure soil-A curve represent well graded soil while soil-B represent poorly graded soil or uniformity grading soil.

⇒ A curve with a flat portion represent a soil in which intermediate size of particles are absent. that soil is known as gap graded or skip graded soil.

⇒ For coarse grained soil D_{10}, D_{30}, D_{60} size particles are important :

D_{10} = particle size in mm, in which 10% of particle finer than this size.

D_{30} = particle size in mm, in which 30% of particle finer than this size

D_{60} = particle size in mm, in which 60% of particle finer than this size

⇒ D_{10} size also called effective size or effective diameter.

(i) Co-efficient uniformity or uniformity co-efficient (C_u)

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

where D_{60} = particle size in mm, in which 60% of particle finer than this size.

D_{10} = particle size in mm in which 10% of particle finer than this size.

(ii) Co-efficient of curvature (Cc) :-

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

where D_{30} = particle size in mm, in which 30% of particle finer than this size.

D_{10} = particle size in mm, in which 10% of particle finer than this size.

D_{60} = particle size in mm, in which 60% of particle finer than this size.

NOTE :-

(i) Poorly graded or uniformly graded :- $C_u = 1$

(ii) well graded soil :- $1 < C_c < 1-3$

$\rightarrow C_u > 4$:- gravel

$C_u > 6$:- sand

Activity of clay :-

⇒ The properties of clay & their behaviour is influenced by presence of certain clay minerals in small quantities.

⇒ The plasticity of clay depends upon the (1) nature of clay mineral

(2) amount of clay mineral.

⇒ On the basis of experiment observed for a given soil & found the plasticity index is directly proportional to the percentage of clay size fraction present in the sample (i.e. the size should less than ...)

→ The ratio of plasticity index to the percentage clay size particle by weight :

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

where I_p = plasticity index

C_w = % by weight clay size

Activity	Classification
(i) < 0.75	Inactive
(ii) 0.75 - 1.40	Normal
(iii) > 1.40	Active

18 Sep 2020

Note :- (i) Clay containing kaolinite slow down the activity & has low activity.

(ii) Clay containing montmorillonite higher the activity.

(iii) Kaolinite - 0.4 - 0.5

Illite - 0.5 - 1.0

Montmorillonite - 1.0 - 7.0

Q1. A natural soil sample has a bulk density of 2gm/cm³ with 8% water content. Calculate the amount of water required to be added to one cubic meter of soil to raise the water content to 15% while the void ratio remain constant. Also find the degree of saturation ?

Sol^n $\rho = 2gm/cm^3$ $w_2 = 15\%$

$w_1 = 6\%$

$G = 2.67$

Stage-1 $V_{w_1} = \frac{M_w}{\rho_w}$

$w_1 = 6\% = 0.06$

$\Rightarrow \frac{M_{w_1}}{M_{d_1}} = 6\%$

$\Rightarrow M_{w_1} = 6\% \times M_{d_1}$ ——— ①

$M_{d_1} = \rho_{d_1} \times V_d$

$= 1.88 \times 1 \times 10^6$ $\left[\rho_d = \frac{\rho}{1+w} = \frac{2}{1+0.06} = 1.88 g/cm^3 \right]$

$= 1.88 \; gm \times 10^6$

By putting the value of 'm_d' in eq^n ① we get

$M_{w_1} = 6\% \times m_{d_1}$

$= 0.06 \times 1.88 \times 10^6$

$= 112800 \; gm$

$V_{w_1} = \frac{M_{w_1}}{\rho_w} = \frac{11280}{1} = 112800 cm^3$

Stage-2

$w_2 = 15\% = 0.15$

$V_{w_2} = \frac{M_{w_2}}{\rho_w}$

$(m_{d_1} = m_{d_2})$

$M_{w_2} = 15\% \times M_{d_2}$

$= 0.15 \times 1.88 \times 10^6$

$= 282000 \; gm$

$$V_{w_2} = \frac{M_{w_2}}{\gamma_w} = \frac{282000}{1}$$

$= 282000\ cm^3$

volume of water required to make the water is $(V_w) = (V_{w_2} - V_{w_1})$

$= 282000 - 112800$

$= 169200\ cm^3$

Degree of saturation is

$$Se = wG$$

$$\Rightarrow S = \frac{w \times G}{e}$$

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

$$= \frac{2.67 \times 1}{1.88} - 1 = 0.420$$

$$S = \frac{0.15 \times 2.67}{0.420} \times 100$$

$= 95.357\ \%$

name	unit
Length	metre
mass	kilogram
Time	second
Electric current	ampere
Thermodynamic Temperature	kelvin
Amount of substance	mole
Luminous intensity	candela
plain angle	radian
solid angle	steradian
volume	liter
Density	kilogram/cubic metre

1

1.88

$88\ g/cm^3$

Energy ⟶ Joule

Area ⟶ square meter (m^2)

Frequency ⟶ hertz (s^{-1})

Acceleration ⟶ m/s^2

Angular Acceleration ⟶ rad/s^2

Resistance ⟶ ohm

Inductance ⟶ henry

Force ⟶ Newton

Pressure ⟶ pascal

Sound ⟶ Decibel

humidity ⟶ Hygrometer

Torque ⟶ Newton meter

Illuminance ⟶ Lux

Depth of ocean ⟶ fathometer

Magnetic Flux ⟶ Weber

New chapter

① ~~Particle size~~ classification of soil

⟹ The purpose of soil classification is to arrange various types of soil into group according to their engineering & agricultural properties.

⟹ Soil possessing same characteristic placed under same group.

⟹ Soil survey & soil classification are carried out by several agencies or organisations.

⟹ Soil may be classified into following types:-

(i) particle size classification
(ii) Textural classification
(iii) Highway Research Board (HRB) classification
(iv) unified soil classification & I.S classification

19-09-20

> In this system soils are arranged according to the grain size.
> Terms such as gravel sand, silt & clay are used for grain size.
> It is preferable to use the word clay size or "silt size" instead of simply clay & silt.
> There are various grain size classification in use from these some systems are:-
(i) U.S (united state) Bureau of soil classification.
(ii) International soil classification.
(iii) M.I.T (Massachusetts Institute of Technology)

soil classification.

(iv) I.S (Indian standard) soil classification
> soil occurring in nature are composed of differ clay, sand, silt size material.
> soil classification is based on the particle size distribution is known as [illegible] [illegible] of classification.
> this classification is a [illegible] classification in of U.S Public Road administration
> this classification is based on the percentages of sand silt & clay size making the soils

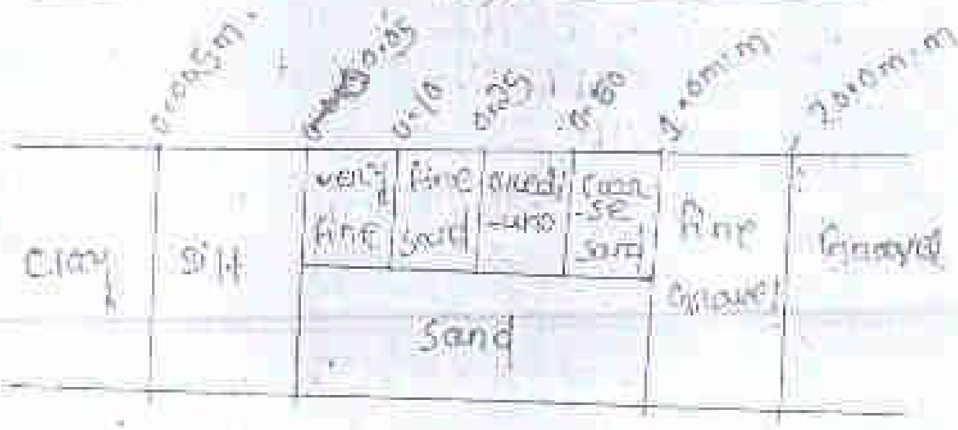
> Such classification is suitable for describing coarse grained soil.

> This represent by a equilateral triangle.

> For example :- If a soil is composed of 30% sand, 30% silt & 40% clay, then the three lines are drawn and intersect at point 'X'.

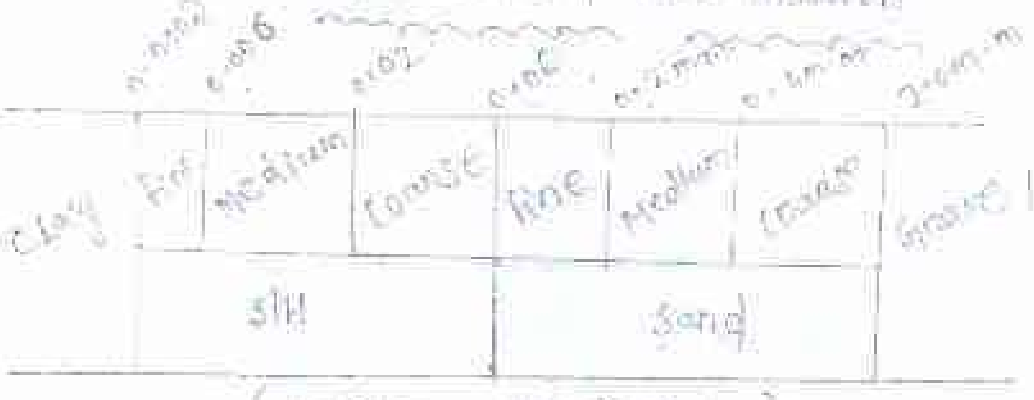
> This point is situated near clay, so that.

(i) Particle size classification :- (diagram)



U.S Bureau of soil

International classification



(M.I.T classification)

etc. assumed (i) flow is laminar to through the soil

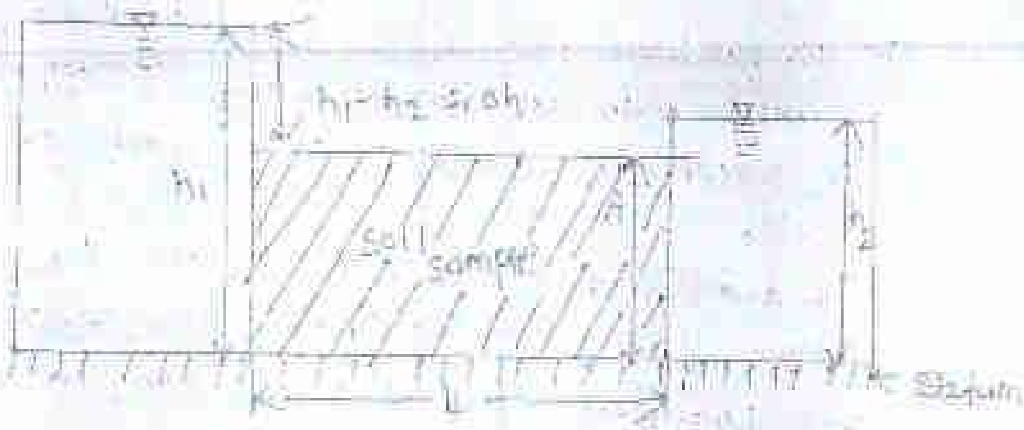
(ii) soil is fully saturated

∴ According to Darcy's law velocity of flow is directly proportional to the hydraulic gradient i.e. $\boxed{V \propto i}$ ← eqn

V = velocity of flow

= $\dfrac{\text{discharge per unit time}}{\text{Area of cross section}}$

$= \boxed{V = \dfrac{q}{A}}$



$V \propto i$

$\Rightarrow V = Ki$

$\Rightarrow \dfrac{q}{A} = Ki$

$\boxed{q = KiA}$ where K = co-efficient of permeability

$\Rightarrow q = KiA$

$q = K\left(\dfrac{h_1 - h_2}{L}\right)A$

$$\boxed{q = K\left(\frac{\Delta h}{L}\right)A}$$

where Δh = difference in head

head (unit m)

L = Length of soil sample

i = hydraulic gradient = $\frac{\Delta h}{L}$ (unit less)

q = discharge per unit time

A = Area of cross-section

Soil type	k (cm/sec)
(1) Gravel	1 or > 1
(2) Coarse sand	1 - 1 × 10^{-2}
(3) Sand (mixture)	1 × 10^{-2} - 5 × 10^{-2}
(4) Fine sand	5 × 10^{-2} - 1 × 10^{-3}
(5) Silty sand	2 × 10^{-3} - 1 × 10^{-4}
(6) Silt	5 × 10^{-4} - 1 × 10^{-5}
(7) Clay	1 × 10^{-6} or smaller

Note

unit: of v = m/sec or cm/sec

q = m^3/sec

A = m^2

L = m

i = unitless

30 Oct 25 -

Discharge velocity & Seepage velocity -

⇒ The darcy's law is no way describes state of flow with in individual pores.

⇒ Darcy's law represent the statistical macroscopic equivalent of the fluid in ground flow.

⇒ The velocity of flow (v) is the rate of discharge of water (q) per unit of total cross-sectional area (A).

⇒ This total cross sectional area is composed of area of solid (A_s) & area of voids (A_v).

⇒ Since the flow takes place through the voids, the actual or true velocity of flow will be more than the discharge velocity.

⇒ The actual velocity is called the seepage velocity (V_s)

⇒ Seepage velocity (V_s) is defined as the rate of discharge of percolating water per unit cross-sectional area of voids perpendicular to the direction of flow.

$$q = VA = V_s A_s$$

$$\Rightarrow VA = V_s A_s$$

$$\Rightarrow V_s = \frac{VA}{A_v} \times \frac{L}{L}$$

$$v = \frac{v}{V_v}$$

$$\therefore v = \frac{1}{\frac{V_v}{V}} \qquad \left(n = \frac{V_v}{V}\right)$$

$$\Rightarrow V_s = v \cdot \frac{1}{n}$$

$$\Rightarrow \boxed{V_s = \frac{v}{n}}$$

⇒ Seepage velocity (V_s) is directly proportional to the hydraulic gradient.

$$V_s \propto i$$

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

Where K_p = co-efficient of percolation

⇒ from Darcy's law $\boxed{K_p = \frac{K}{n}}$

Factors affecting permeability :-

From Darcy's law $q = KiA$, we get

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

i) Effect of size of particles :-

⇒ Permeability varies approximately as the square of the grain size.

⇒ Allen Hazen based on his experiments on filter sands of particle size between 0.1 & 3mm, found that the permeability can be expressed as:-

$$\boxed{K = C D_{10}^2}$$

where K = co-efficient of permeability (cm/sec)

D_{10} = effective diameter (cm)
C = constant (approx = 100)

Effect of viscosity of pore fluid :-

⇒ It indicates that the permeability is directly proportional to the unit weight of water & inversely proportional to its viscosity.

⇒ Unit weight of water does not change with the change in temperature

⇒ So when $\nu \uparrow \longrightarrow K \downarrow$
$\mu \downarrow \longrightarrow K \uparrow$

i.e. $$\boxed{\frac{K_1}{K_2} = \frac{\gamma_{w_1}}{\gamma_{w_2}} \times \frac{\mu_2}{\mu_1}}$$

$$\boxed{K \propto \gamma \; \& \; K \propto \frac{1}{\mu}}$$

⇒ Also $\boxed{K \propto T}$

where T = Temperature
γ = unit weight
μ = viscosity

So $\boxed{T \propto \frac{1}{\mu}}$

Effect of structural arrangement of particle & stratification :-

⇒ The structural arrangement of particle may vary, at the same void ratio depending upon the method of deposition or compacting the soil mass.

⇒ The permeability parallel to the stratification being always greater than the perpendicular to the stratification.

(4) Shape of particles :-

⇒ There are 3 types of shape of particles → (i) Rounded } Coarse-grained soil
(ii) Angular
(iii) flaky ⇒ fine grained soil

⇒ fine grained soil have more void ratio but bonding betⁿ the particles are poor, so its permeability is low.

⇒ Rounded shape particles have more permeability than the angular shape particle.

(5) Degree of saturation & entrapped air & other foreign matter :-

⇒ The permeability is greatly reduced if air is entrapped in the voids, thus reducing its degree of saturation.

⇒ The dissolved air in the pore fluid may get liberated, thus changing the permeability.

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

⇒ permeability is more in fully saturated soil than the partially saturated soil.

(6) Specific Surface area :-

⇒ Specific surface area is defined as the ratio of area to unit weight (or) area to unit volume.

$$\boxed{S.S.A = \frac{\text{Area}}{\text{weight}} \text{ (or) } \frac{\text{Area}}{\text{volume}}}$$

⇒ Specific surface area is inversely proportional to the size & 'k' is directly proportional to size.

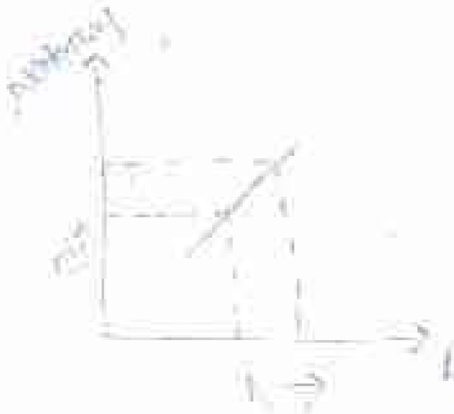
$$\boxed{\text{So } k \propto \text{size} \propto \frac{1}{\text{specific surface area}}}$$

⇒ As more surface is contact with the water greater is the resistance & co-efficient of permeability is low.

(7) Effect of void ratio :-

⇒ If a curve is plotted with void ratio on natural scale & co-efficient of permeability on logarithmic scale (abscissa) the curve will be a linear curve.

$$\frac{k_1}{k_2} = \frac{\frac{e_1^3}{1+e_1}}{\frac{e_2^3}{1+e_2}}$$



i.e. $\boxed{e \propto \log k}$

log

(c) Effect of adsorbed water :-

→ The adsorbed water surrounding the fine - soil particles is not free to move, and reduces the effective pore space available for the passage of water.

→ Fine particles interlock water in their surface so permeability is lesser.

Co-efficient of permeability :-

→ It describes how easily a liquid will move through a soil.

→ It is also commonly referred to as the hydraulic conductivity of a soil.

→ This factor can be affected by the viscosity or thickness of a liquid & it's density.

→ There is some emperical formula for determination of Co-efficient of permeability are:-

(i) Jaky's :

$$\boxed{K = 100 D_m^{2}}$$

where D_m = grain size (mm) that occurs with greatest frequency

(ii) Allen Hazen's

$$\boxed{K = C D_{10}^{2}}$$

where C = constant (approx = 100 in cm).

(3) Terzaghi's formula :-

$$\boxed{k = 200\, D_e^2 e^2}$$

where D_e = effective grain size

e = void ratio

(4) Kozeny's formula

$$\boxed{k = \frac{1}{K_k \eta S_s^2} \times \frac{n^3}{(1-n)^2}}$$

where S_s = specific surface of particles (cm^2/cm^3)

η = viscosity ($g\text{-}sec/cm^2$)

K_k = constant (equal to 5 for spherical particle)

(5) Loudon's formula :-

$$\boxed{\log_{10}(k S_s^2) = a + bn}$$

where a, b = constant i.e. 1.365 & 5.15 at $10°C$

n = porosity

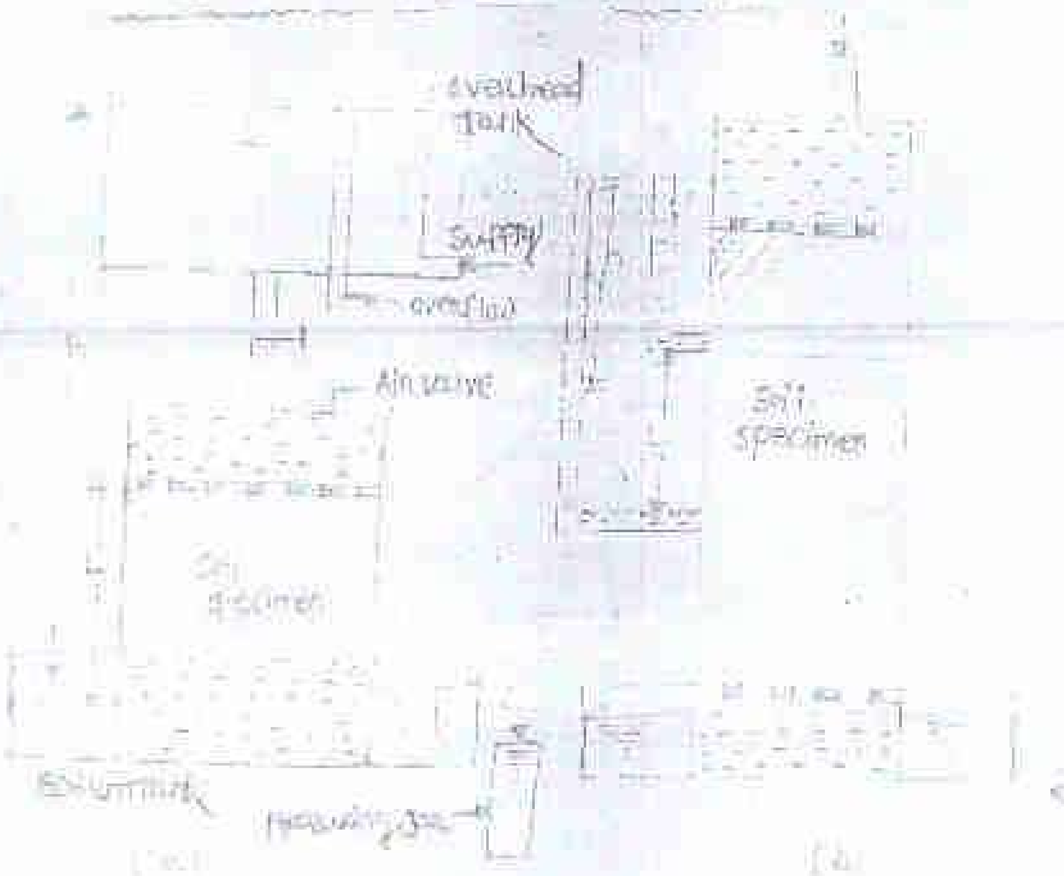
5 oct 2020

Determination of coefficient of permeability by laboratory method :-

There are 2 methods of determining the 'K' that are ① constant head permeability test

② Falling head permeability test.

① Constant head permeability test :-



⇒ Water flows from the overhead tank consisting of 3 tubes that are inlet tube, overflow tube & the outlet tube.

⇒ The constant hydraulic gradient 'i' causing the flow is the head 'h' (i.e. it is different in the water levels of the overhead & bottom tanks) divided by the length of the sample.

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

⇒ If 'Q' is the total quantity of flow in a time interval t, we have from Darcy's law :-

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

$$\Rightarrow k = \frac{Q}{t} \times \frac{1}{iA}$$

$$\Rightarrow \boxed{k = \frac{Q \times L}{Aht}}$$

where A = cross-sectional area of the sample.

t = time interval

Q = total quantity of water of flow

L = length of the sample

h = head

(2) Falling head permeability test

⇒ Constant head permeability test is used for coarse grained soil only where a reasonable discharge can be collected in a given time.

⇒ However the falling head test is used for relatively less permeable soils where the discharge is small.

⇒ A stand pipe of known cross-sectional area 'a' is fitted over the permeameter & water is allowed to run down.

⇒ The water level in the stand pipe constantly falls as water flows.

⇒ Observations are started after steady state of flow has reached.

⇒ The head at any time instant 't' is equal to the difference in the water level in the stand pipe & the bottom tank.

⇒ Let h_1 & h_2 be heads at time interval t_1 & t_2 ($t_2 > t_1$) respectively.

⇒ Let h be the head at any intermediate time interval & $-dh$ be the change in the head in a smaller time interval dt.

⇒ Hence from Darcy's law, the rate of flow 'q' is:

$$q = \frac{-dh \cdot a}{dt} = kiA$$

by integrating the equation we get

$$\boxed{k = \frac{aL}{At} \log_e \frac{h_1}{h_2} = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}}$$

where a = stand pipe area

A = Area of sample

t = time interval i.e. $t_2 - t_1$

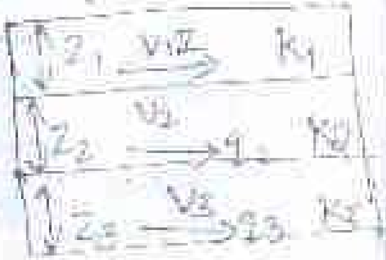
h_1 = head at time t_1

h_2 = head at time t_2

Average permeability parallel to the bedding plane :-

⇒ Let $z_1, z_2 \cdots z_n$ = thickness of layer

$k_1, k_2 \cdots k_n$ = permeability of layer



⇒ For flow to be parallel to the bedding planes, the hydraulic gradient (i) will be same for all the layers.

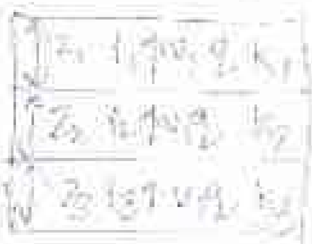
⇒ Avg permeability of soil :-

$$\boxed{k_h = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3 + \cdots + k_n z_n}{z}}$$

where $z = z_1 + z_2 + z_3 \cdots + z_n$

Average permeability perpendicular to the bedding plane :-

⇒ In this case velocity of flow & discharge is same through each layer.



⇒ Head loss through each layer is different.

$\Rightarrow$ Avg. permeability (k_z)

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

where $Z = Z_1 + Z_2 + Z_3 \dots + Z_n$

2023
Ex A stratified soil deposite consist of 4 layers of equal thickness. The co-efficient of permeability of the 2nd, 3rd & 4th layer is $\frac{1}{3}$rd, $\frac{1}{2}$ & twice the co-efficient of permeability of top layer. Compute the average co-efficient of permeability of the deposite parallel & perpendicular to the direction of stratification?

Ans parallel to the bedding plane :-

$$K = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3 + k_4 z_4}{Z}$$

$$= \frac{kz + \frac{1}{3}kz + \frac{1}{2}kz + 2kz}{4z}$$

$$= \frac{\cancel{z}(k + \frac{1}{3}k + \frac{1}{2}k + 2k)}{4\cancel{z}}$$

$$= \frac{\frac{6k + 2k + 3k + 12k}{6}}{4} = \frac{23k}{24}$$

(b) Perpendicular to the bedding plane:

$$K = \frac{Z}{\frac{Z_1}{k_1} + \frac{Z_2}{k_2} + \frac{Z_3}{k_3} + \frac{Z_4}{k_4}}$$

$$= \frac{4Z}{\frac{Z}{k} + \frac{Z}{3k} + \frac{Z}{2k} + \frac{Z}{2k}}$$

$$= \frac{4Z}{\frac{2Z+6Z+4Z+Z}{2k}}$$

$$= \frac{8kZ}{13Z}$$

$$= \frac{8k}{13}$$

7-02-2019

16. Calculate the coefficient of permeability of a soil sample 6 cm in hight & 50 cm2 cross sectional area. If the a quantity of water equal to 420 ml passed down in 10 minute under an effective constant head of 40 cm. On overdrying the test specimen has mass of 498 gm. Taking the specific gravity of soil solid as 2.65. Calculate the seepage velocity of water during test.

Soln Given data

Seepage velocity $(v_s) = \frac{v}{n}$

n = porosity

v = Total velocity [average]

t = 10 minute = 600 sec

Q = 420 ml

G = 2.65

$A = 50\,cm^2$ $\quad L = 6\,cm$ $\quad h = 40\,cm$

$M_d = 498\,gm$

$v = Ki$

where $K = \frac{QL}{Aht}$

$= \frac{430 \times 6}{50 \times 40 \times 600} = 2.15 \times 10^{-3}\ cm/sec$

$i = \frac{\Delta h}{L} = \frac{h}{L} = \frac{40}{6} = 6.6$

$v = Ki = 2.15 \times 10^{-3} \times 6.6$

$= 1.43 \times 10^{-2}\ cm/sec$

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

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

$\gamma_d = \frac{M_d}{V} = \frac{498}{50 \times 6} = 1.66\ g/cm^3$

$e = \frac{G\gamma_w}{\gamma_d} - 1 = 0.596$

$n = \frac{e}{1+e} = \frac{0.596}{1+0.596} = 0.373$

$v_s = \frac{v}{n} = \frac{1.43 \times 10^{-2}}{0.373}$

$= 0.0383\ cm/sec$

$= 0.3835 \times 10^{-3}\ cm/sec$

Q. In a falling head permeability test, the initial head [illegible] cm [illegible] test [illegible] initial [illegible] in [illegible] minutes. [illegible] sample [illegible] in [illegible] the cross section [illegible] the coefficient of permeability [illegible] standpipe area of [illegible] cm^2

Given data :- $h_1 = 40\ cm$

$h_2 = 40 - 5 = 35\ cm$

$a = 0.5\ cm^2$, $t = 10\ min = 600\ sec$

$L = 6\ cm$, $A = 50\ cm^2$

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

$$= 2.3 \times \frac{0.5 \times 6}{50 \times 600} \times \log_{10} \frac{40}{35} =$$

1.33×10^{-5} cm/sec

Book 202[illegible]

Soil Structure

> The term soil structure in general, referred to the arrangement of particles of soil mass based upon their composition & shape.

Type of soil structure

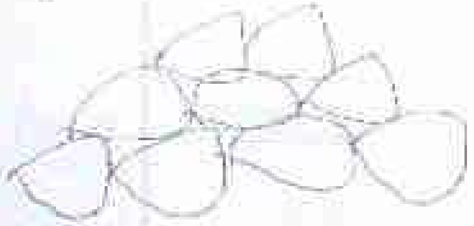
- Single grained structure
- Honeycomb structure
- Flocculated & dispersed structure
- Structure of composite soil

(i) Single grained structure :-

> This type of structure will be found in case of coarse grain soil.

> When such soil settle out of suspension in water, the particles settle independently of each other.

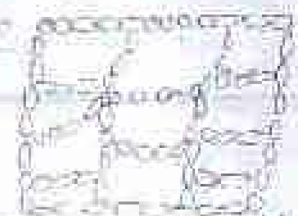
⇒ The major force causing their deposi-tion is gravitational & the surface forces.
⇒ There will be particle-to-particle contact in the deposite.
⇒ The void ratio depends upon the relative grain size of particle.



(single grained structure)

② Honeycomb structure -

⇒ This type of structure is associated with silt deposit.



⇒ When silt particles settle out of suspension in addition to gravitational forces & the surface forces also play a significant role.
⇒ When particles approach the lower region of suspension they will be attracted by particles already deposited as well as the neighbou-ring particles leading to the formation.
⇒ The combination of a no of of this type of particles leads to honey-comb structure.
⇒ It has high void ratio.

(5) Flocculated & dispersed structure:-

(Flocculated structure) (dispersed structure)

⇒ There are two types of structure in clay that are flocculated & dispersed.

⇒ In case of flocculated structure soil particle contact to each other by edge-to-edge & edge-to-face.

⇒ This type of particle formation is due to net electrical forces between the adjacent particles at the time of deposition.

⇒ It has very high void ratio & poor connection between particle so that it is less permeable.

⇒ In case of dispersed structure the particles arranged face-to-face.

⇒ This type of formation is due to net electrical forces between the particles during soil deposite.

⇒ This type of soil is commonly for water deposite.

Structures of Composite Soils :-

Coarse grained skeleton structure

Cohesive matrix structure

⇒ Their formation depend on the relative proportions of Coarse grained & fine grained fractions.

(i) Coarse grained skeleton structure:-

* The coarse grained skeleton structure can be found in case of composite soils in which the coarse grained fraction is greater in proportion compared to fine grained fraction.

* The coarse grained particles form the skeleton with particle to particle contact & the voids between these particles will be occupied by the fine grained particles.

(ii) Cohesive matrix Structure:-

* Cohesive matrix structure can be found in composite soils in which the fine grained fraction is more in proportion compared to coarse-grained fraction.

* In this case the coarse-grained particles will be embedded in fine-grained fraction & will be prevented from having particle-to-particle contact.

* This type of structure is relatively more compressible compared to the more stable coarse-grained skeleton structure.

9 oct 2020

New chapter

Soil water & Effective stress

* Soil water -
  - Free water or Gravitational water
  - Held water
    - Structural water
    - Adsorbed water
    - Capillary water

→ Water present in soil mass is known as soil water.

⇒ It is broadly divided into 2 types:-
(i) Free water or gravitational water
(ii) Held water

(i) Free water - Water i.e free to move through the soil mass under the influence of gravity is known as free water or gravitational water.

Held water - Held water is the water which is held with in a soil mass by soil particles.

⇒ It is again devided in to 3 types -:

(i) Structural water:-

⇒ Structural water is the water chemically combined in the crystal structure of the soil particle cannot be removed without breaking the structure of soil particle.

⇒ The structure of soil particle cannot be removed by load applied & by ovendrying of temperature 110°c.

⇒ Generally this type of water is not considered in civil engg.

(ii) Adsorb water :- (surface se lagi rahti)

⇒ Adsorb water is the water which is held by the grain soil particles due to electro chemical forces of adhesion.

⇒ A colloidal soil particle carries a net negative charge on its surface & water molecule is a permanent dipole.

⇒ Therefore water molecules adjacent to soil particle get attracted by the water molecule.

⇒ Because of net negative charge on its surface a soil particle get attracted a number of ~~other~~ other exchangeable cations like those of sodium, calcium, magnesium, potassium & these in turn attract them by dipolar water molecules.

Colloidal particle

⇒ Thus water in the vicinity of a soil particle is subjected to:-

(i) Attraction by residual negative charge on surface of soil particle.

(ii) Attraction by cations held by soil particle.

⇒ The thickness of adsorbed layer depends upon mineralogical composition of soil particle, specific surface of soil particle & this environment.

⇒ The physical properties of water in the adsorbed layer will be different from those of normal water.

⇒ It is difficult to precisely define the thickness of complete adsorbed layer.

⇒ Adsorbed water has significant effect on the cohesion & plasticity characteristics of fine grained soils.

Capillary water :-

⇒ Capillary water is the water which is held in a soil mass due to capillary forces.

⇒ Capillary action is the ability of a liquid to flow in narrow spaces without the assistance of

(iii) any opposition due to any external forces other gravity.

$$\boxed{\text{height of Capillary rise} = \frac{4\sigma_s \cos\alpha}{\gamma_w d}}$$

where σ_s = surface tension

=> Generally capillary rise in natural soil suggested by Allen Hazen is:

$$\boxed{h_c = \frac{C}{e D_{10}}}$$

where h_c = Capillary rise in cm

e = void ratio

D_{10} = effective diameter of soil in cm

C = Constant (taken between 0.1 to 0.5)

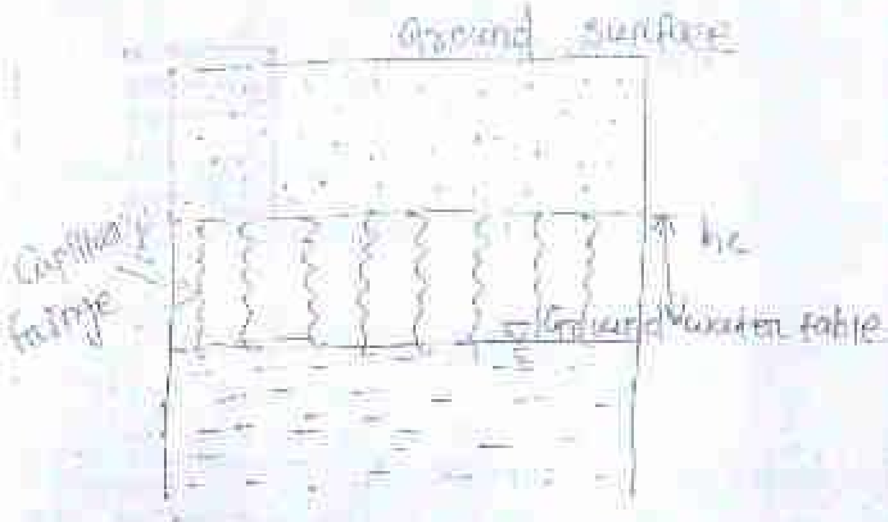
* Capillary phenomenon in soils:-

=> The voids in natural soil formations act as capillary tubes & water rises in the continuous voids to a certain height above ground water table (or) free water surface.

=> The height to which water rises, the capillary rise, depends on size of voids (particle size & void ratio).

=> All other things being equal the capillary rise is more in a fine grained soil deposits than that in a coarse grained soil deposits, as the size of voids is much less.

=> The zone of soil strata saturated with capillary water is called the capillary fringe.



(Capillary phenomenon in soil formation)

10 oct 2020

Effective stress, pore pressure and Total stress:-

* Effective stress :-

⇒ When the pressure is transmitted through soil mass by soil particles through ~~~~ their point of contact is called effective stress.

⇒ It is symbolised by "σ'"

⇒ It is also called intergranular pressure.

* Pore water pressure :-

⇒ The pressure transmitted by the pore water in a soil is called pore water pressure @ pore pressure.

⇒ It is symbolised by u & also referred as neutral pressure.

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

where γ_w = unit weight of water

h_w = piezometric head (height of water)

effective stress = Total stress − pore water pressure

Seepage pressure

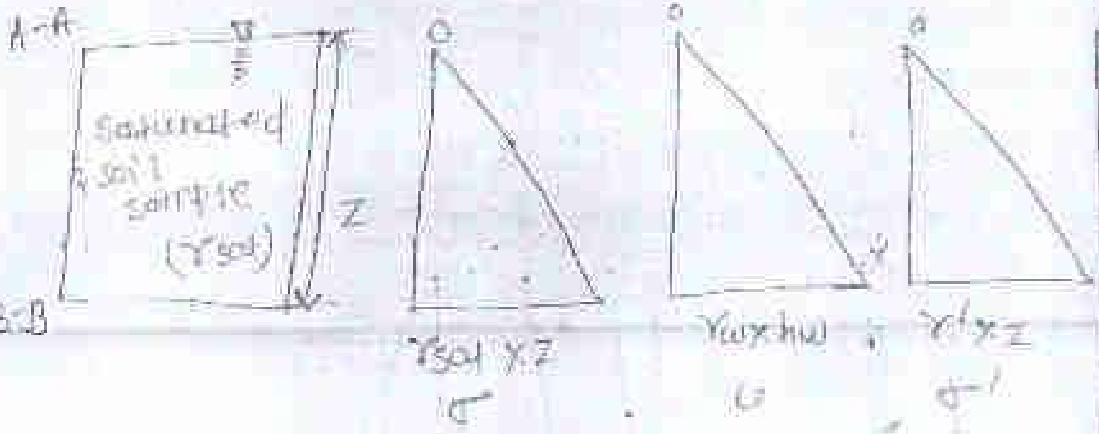
Total Stress

→ It is denoted by σ

→ At any point total stress is the sum of effective stress & pore water pressure.

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

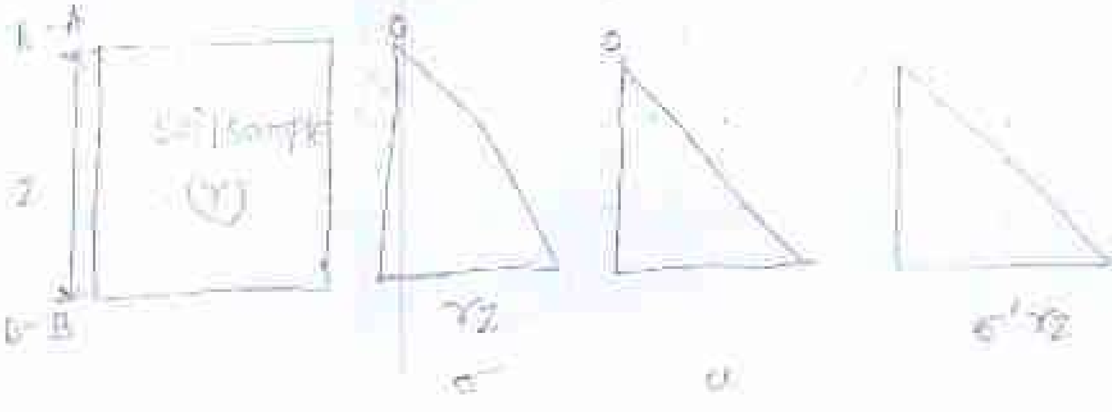
CASE → 1 Submerged soil mass with water table at the surface



Layer	σ	u	σ'
A-A	0	0	0
B-B	$\gamma_{sat} \times Z$	$\gamma_w Z$	$(\gamma_{sat} - \gamma_w) Z = \gamma' Z$

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

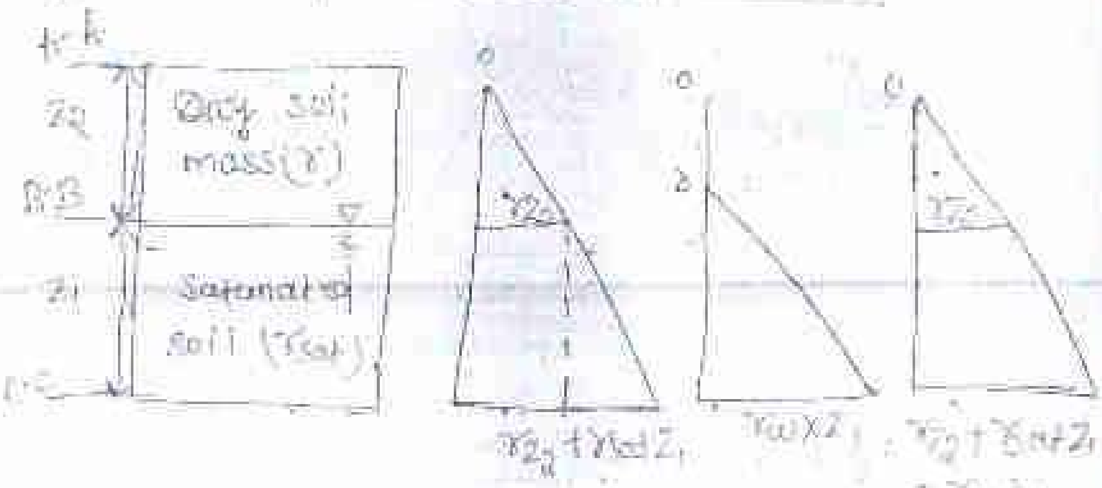
Case 2 → Dry soil mass



Layer	σ	u	σ'
A-A	0	0	0
B-B	γz	0	γz

Case-II 12.01.2026

Partly submerged soil mass:-



σ $\qquad$ u $\qquad$ σ'

$: \gamma z_2 + \gamma_{sat} z_1 - \gamma_w z_1$

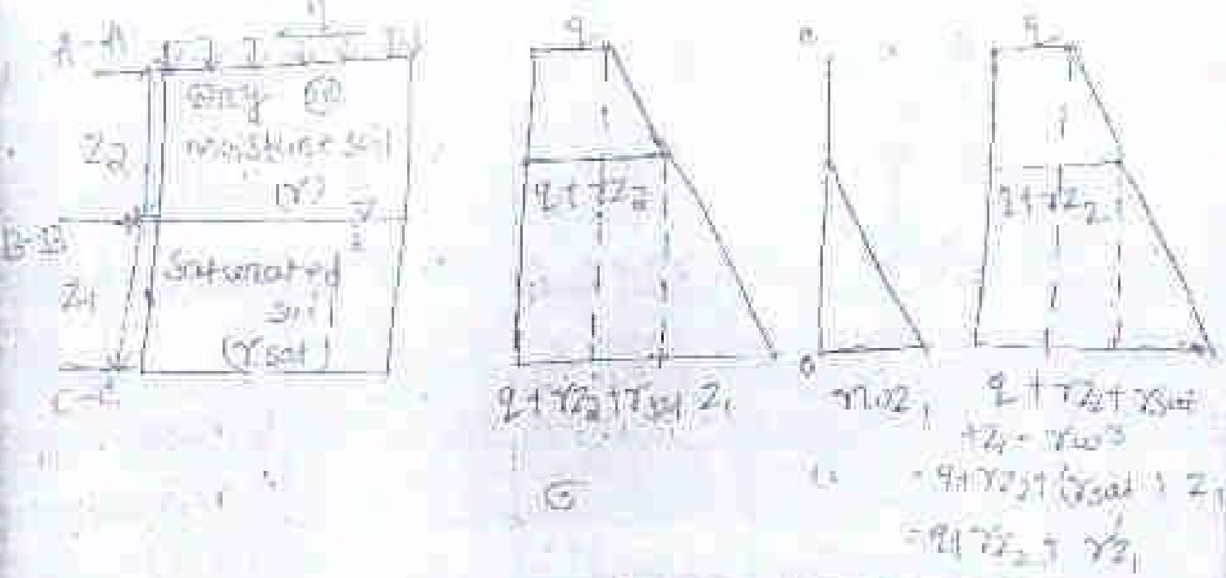
$\Rightarrow \gamma z_2 + (\gamma_{sat} - \gamma_w) z_1$

$\Rightarrow \gamma z_2 + \gamma' z_1$

Layer	σ	u	σ'
A-A	0	0	0
B-B	γz_2	0	γz_2
C-C	$\gamma z_2 + \gamma_{sat} z_1$	$\gamma_w z_1$	$\gamma z_2 + (\gamma_{sat} - \gamma_w) z_1 = \gamma z_2 + \gamma' z_1$

Case - IV

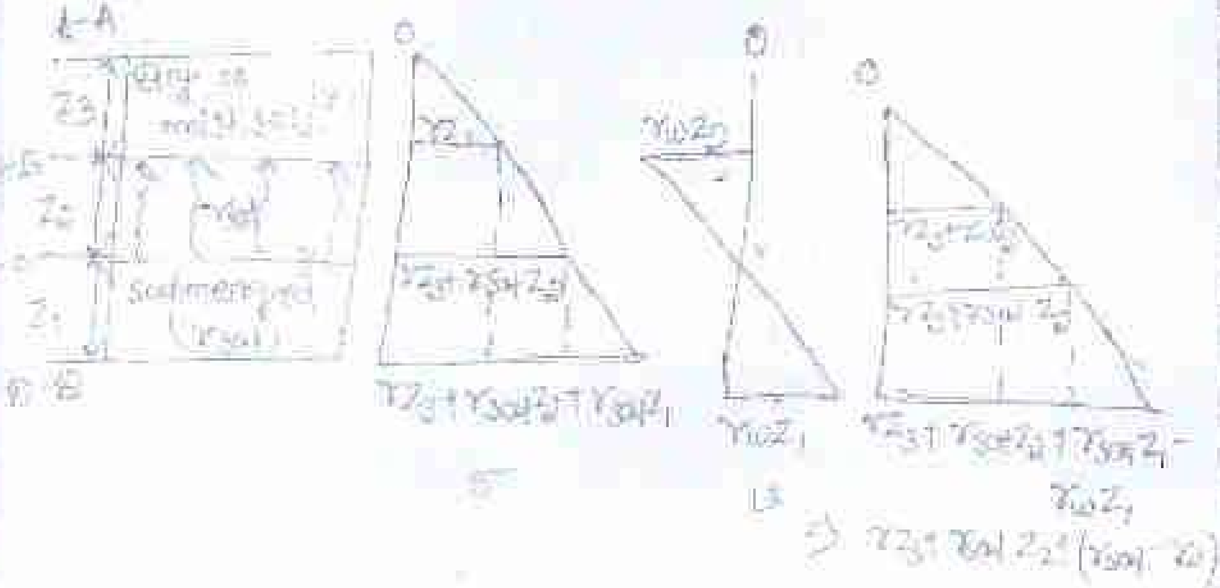
Partially submerged soil with surcharge



Layer	σ	u	σ'
A-A	q	0	q
B-B	$q+\gamma Z_2$	0	$q+\gamma Z_2$
C-C	$q+\gamma Z_2+\gamma_{sat} Z_1$	$\gamma_w Z_1$	$q+\gamma Z_2+(\gamma_{sat}-\gamma_w) Z_1$ $= q+\gamma Z_2+\gamma' Z_1$

18 Oct 2020

Case - V :- Soil with capillary fringe :-

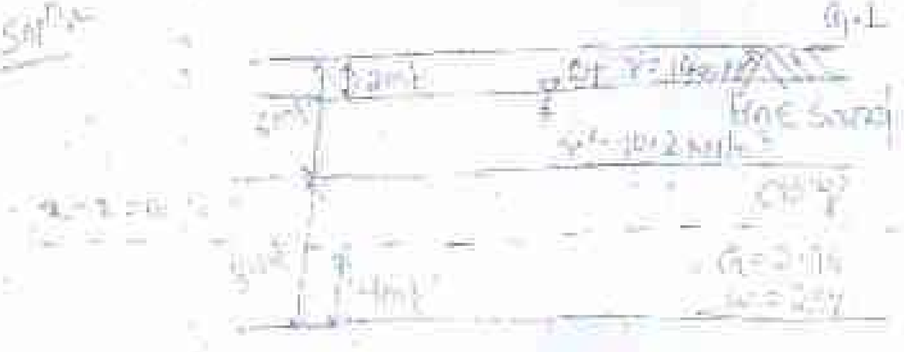


$\Rightarrow \gamma z_3 + \gamma_{sat} z_2 + \gamma' z_1$

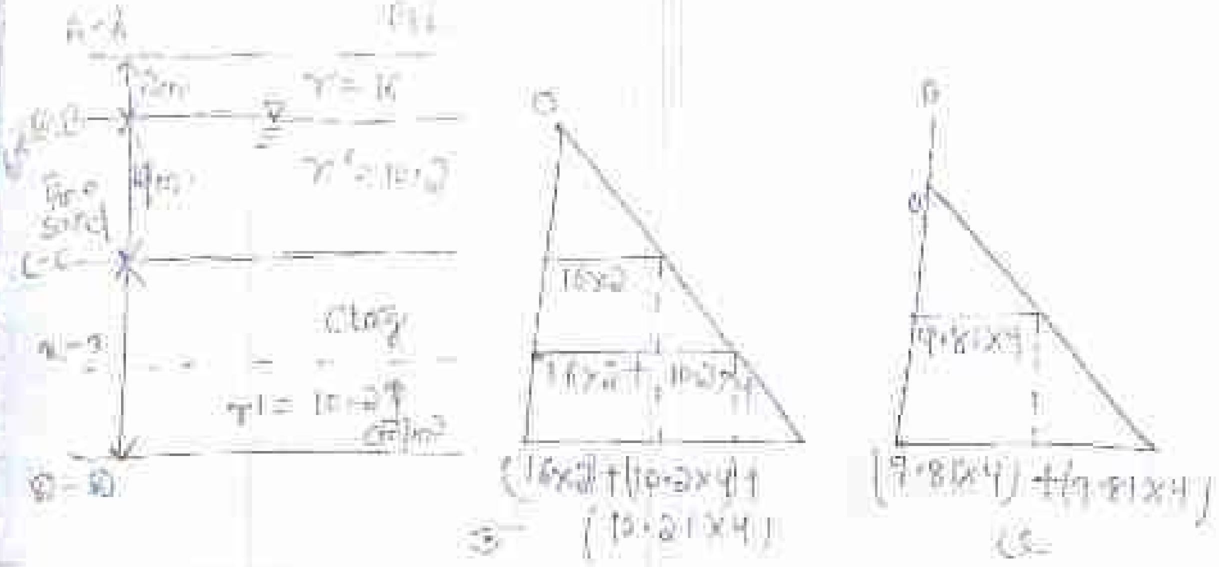
Layer	σ	u	σ'
A - A	0	0	0
B - B	γz_3	$-\gamma_w z_2$	$\gamma z_3 + \gamma_w z_2$
C - C	$\gamma z_3 + \gamma_{sat} z_2$	0	$\gamma z_3 + \gamma_{sat} z_2$
D - D	$\gamma z_3 + \gamma_{sat} z_2 + \gamma_{sat} z_1$	$\gamma_w z_1$	$\gamma z_3 + \gamma_{sat} z_2 + \gamma_{sat} z_1 - \gamma_w z_1$ $\Rightarrow \gamma z_3 + \gamma_{sat} z_2 + \gamma' z_1$

Q) A clay stratum of thickness 8mts is located at a depth of 6mt below the ground surface. It is overlain by fine sand. The water table is located at a depth of 2mt below ground surface. For fine sand submerged unit weight is 10.2 kN/m³. The moist unit wt of sand located above water table is 18 kN/m³. For clay layer G = 2.78 & w = 25%. Compute the effective stress at the middle of clay layer.

Sol:



17 oct 2020



$$\sigma'_{x-x} = \sigma_{xx} - u_{xx} = \left[(16\times2) + (10.2\times4) + (10.21\times4)\right] - \left[(9.81\times4) + (9.81\times4)\right]$$

$$112.84 \ \text{kN/m}^2$$

w.r.t full saturated $S_r = 1$

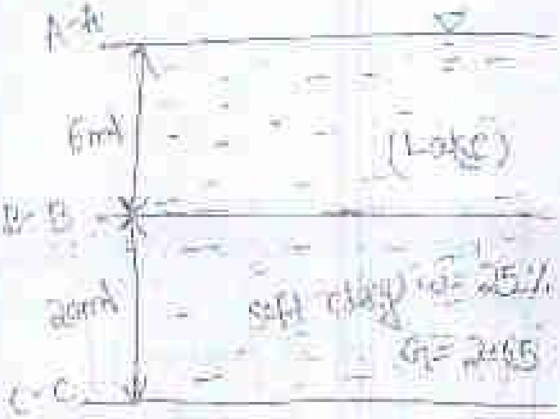
$$S_r \times e = wG$$

$$e = \frac{wG}{S_r} = \frac{0.25\times2.76}{1} = 0.69$$

$$\gamma' = \frac{(G-1)\gamma_w}{1+e} = \frac{(2.76-1)\times9.81}{1+0.69}$$

$$= 10.21 \ \text{kN/m}^3$$

10. Compute the total, effective & pore water pressure at a depth of 20m below the bottom of a lake 6m. The bottom consists of soft clay with a thickness 20m. The average water content of clay is 25% & the specific gravity of the soil is assumed 2.65 ?



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

$Se = wG \; [S=1]$

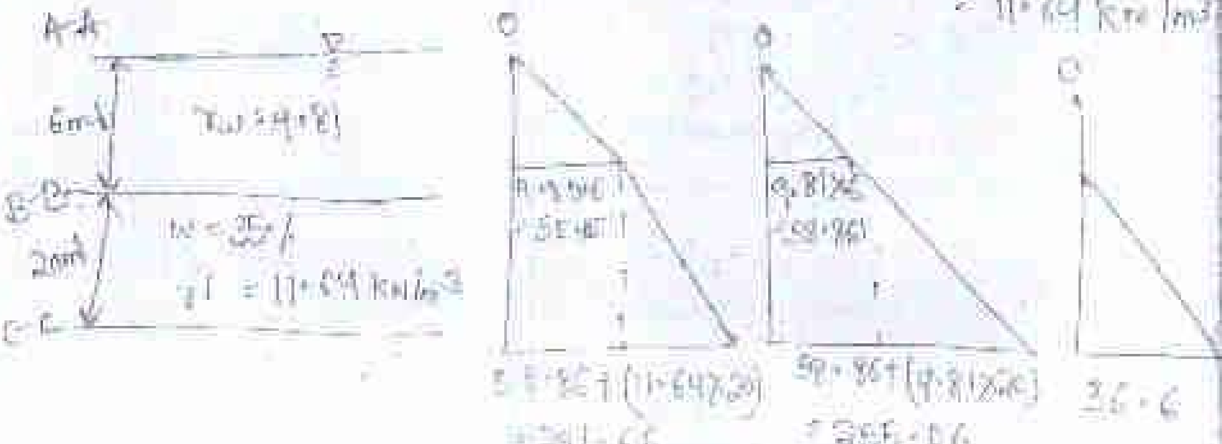
$\Rightarrow e = wG$

$= 0.15 \times 2.65$

$= 0.39$

$\gamma' = \frac{(G-1)\gamma_w}{1+e} = \frac{(2.65-1)\,9.81}{1+0.39}$

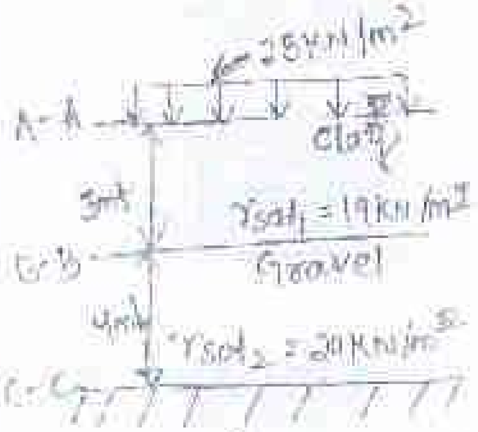
$= 11.64 \; kN/m^3$

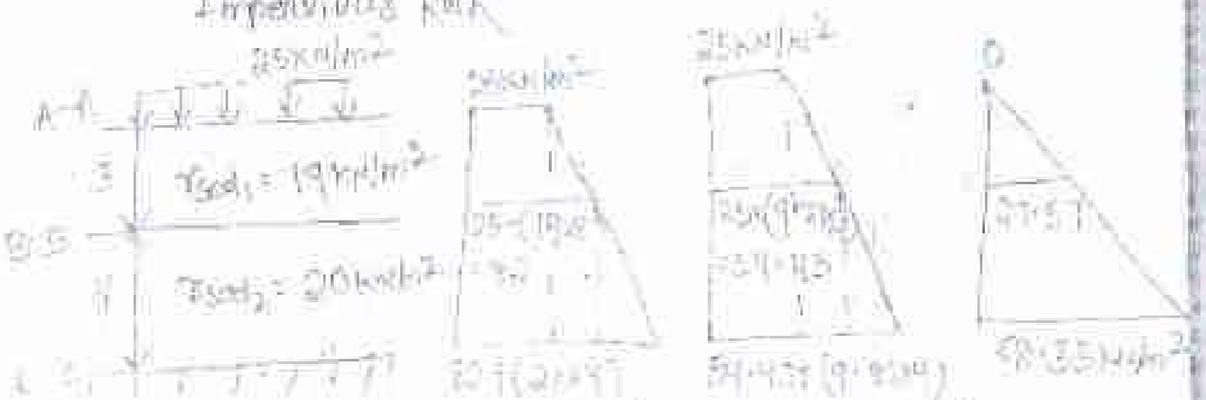


Layers	σ	u	σ'
A-A	0	0	0
B-B	58.86	58.86	0
C-C	291.66	255.06	36.6

Hence the total stress, effective stress & pore water pressure at the bottom of the soft clay layer is 291.66 KN/m2 & 36 kN/m2 & 225.06 kN/m2.

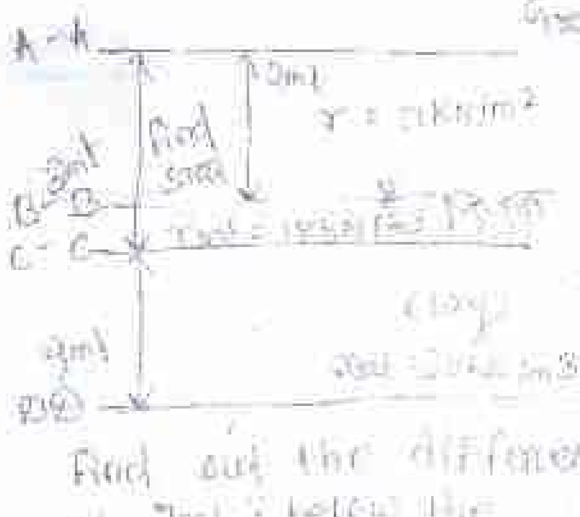
3) At a construction site 3m thick clay layer is formed by a 4m thick gravel layer which is resting on impervious rock. a load of 25 KN/m2 is applied suddenly at the surface. The saturated unit weight of the soil are 19 KN/m3 & 20 KN/m3 for clay and gravel. The water table is at the surface. Draw the diagram showing the variation with depth of total, neutral/pore water & effective stress?





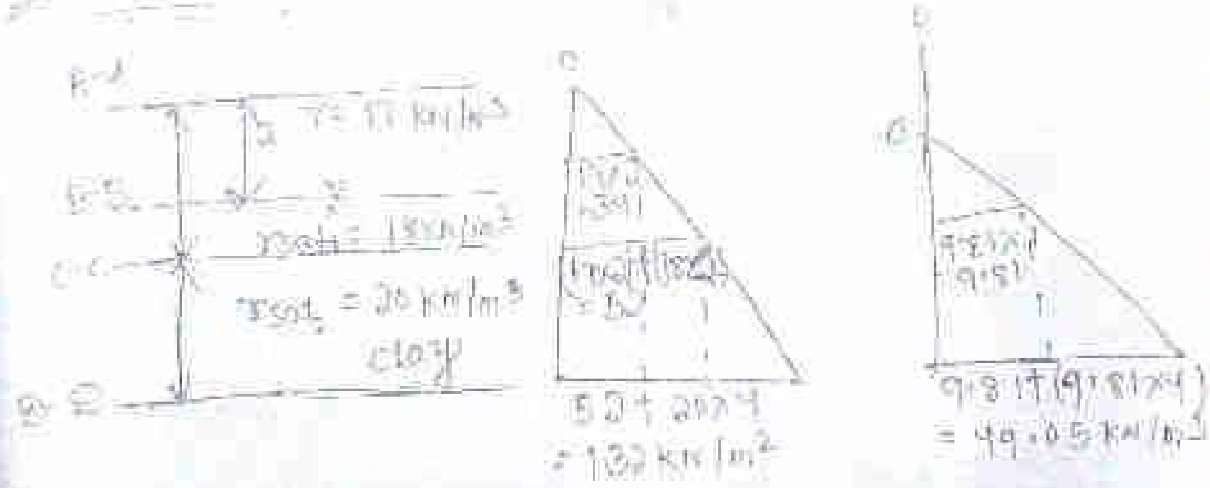
Layers	σ	u	σ'
A-A	25	25	0
B-B	82	54.53	27.51
C-C	162	93.67	68.33

<u>23 oct 2020</u>



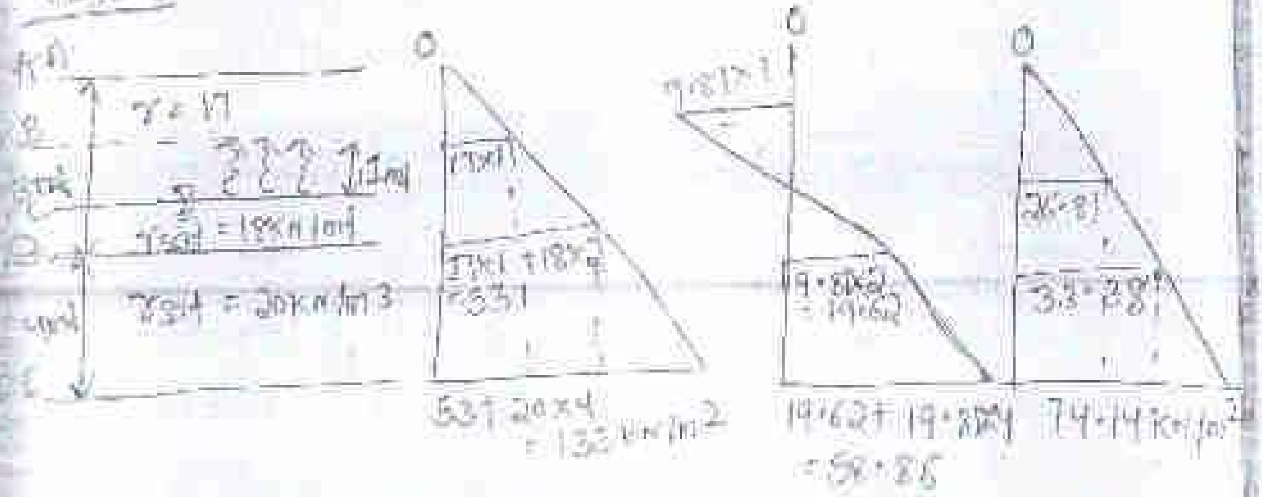
Find out the different ineffective stress at 7m below the ground surface when in 2nd case capillary occur from water table to 2m height?

Case - 1



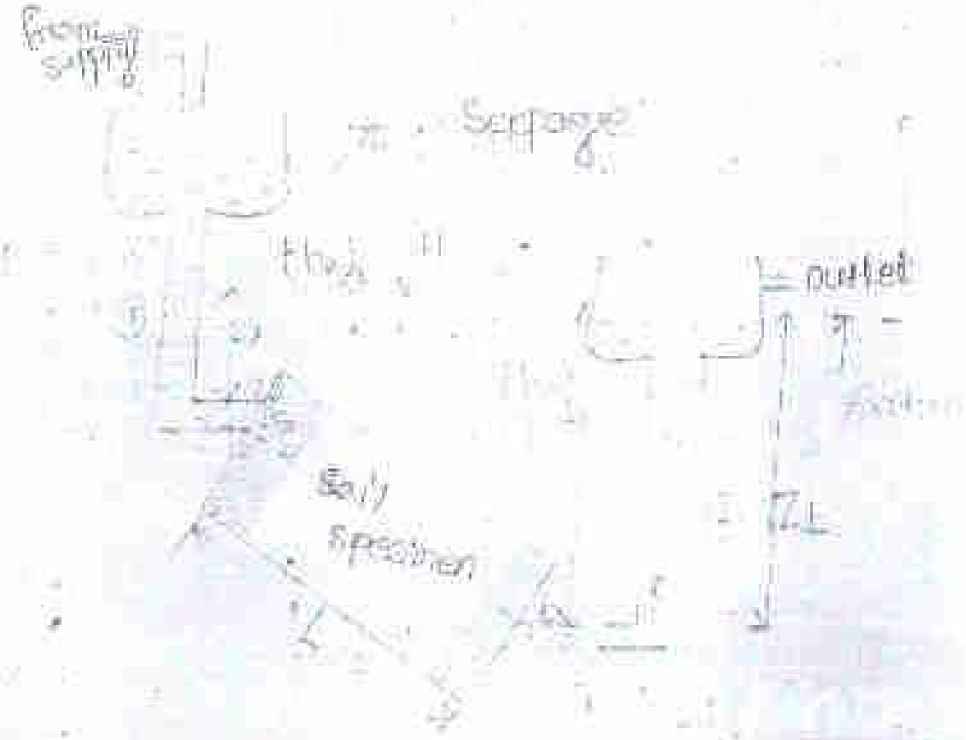
$\sigma' = 132 - 49.05 = 82.95\,kN/m^2$

Case - 2



difference in effective stress $= 82.95 - 74.14$
$= 8.81\,kN/m^2$

Seepage analysis



⇒ Seepage or flow of water through a soil mass occurs when there is difference in total head between two points.

⇒ Total head (h) also known as effective head at any point in the soilmass in which the flow is taking place is the algebraic sum of pressure head (h_w) and datum head (z)

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

$h_w \Rightarrow$ water height

$z \longrightarrow$ datum height

Note :- z will have positive value if the point is above datum & negative if the point is below the datum.

Datum line below line (-ve)
Datum line above line (+ve)

Total $h_a = (h_w)_a - Z_a$
Total $h_b = (h_w)_b - Z_b = 0$

⇒ In the above figure :-
Total head at a, $h_a = (h_w)_a - Z_a$
Total head at b, $h_b = (h_w)_b - Z_b = 0$
∴ So loss of head between a & b
is $H = h_a - h_b$
$= (h_w)_a - Z_a - 0$
$= (h_w)_a - Z_a$

⇒ The loss of head per unit distance of flow through soil mass is called the hydraulic gradient (i)

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

⇒ The hydraulic head or total head (h) at any point in the soil mass, being the sum of pressure head (h_w) & datum head or piezometric head (Z).

⇒ The seepage pressure is the pressure exerted by flowing water on the section of soil mass in the direction of flow.

⇒ It is caused by the force corresponding to energy transferred effected due to frictional drag between water & soil particles.

⇒ The seepage pressure at any point in the soil mass =

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

⇒ If z is the length of flow over which h is lost & 'i' is hydraulic gradient, then we can write

$$P_s = \gamma_w h$$

$$\Rightarrow P_s = \gamma_w h \times \frac{z}{z}$$

$$\Rightarrow P_s = \gamma_w \times \frac{h}{z} \times z \quad \left[i = \frac{h}{z} \right]$$

$$\Rightarrow \boxed{P_s = i \gamma_w z}$$

⇒ The seepage force over total cross-sectional area 'A' of flow in the soil mass is

$$\boxed{J = P_s A}$$

where J = seepage force

P_s = seepage pressure

A = cross sectional area

$$\Rightarrow \boxed{J = i \gamma_w z A}$$

⇒ In case of vertical flow through the soil mass the effective stress at a section will be increased or decreased according to the flow in the downward or upward direction

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

$$\Rightarrow \sigma' = \gamma' z \pm P_3$$

$$\Rightarrow \boxed{\sigma' = \gamma' z \pm i\gamma_w z}$$

<u>Note</u> :- flow occurs in downward direction '+ve' sign is used & in upward direction '-ve' sign is used

9 Nov 2020

=> Quick sand condition :-

-> In case of upward flow of water through a soil mass, the seepage pressure acts in the upward direction causing reduction in effective stress.

-> In case of submerged soil mass, the upward seepage pressure may becomes equal to the ~~the~~ downward pressure due to submerged weight of soil.

-> When this happen in case of a cohesionless soil at that level loses all its shear strength as the effective stress become zero.

-> Because of this soil particles have the tendency to be carried away by flowing water.

-> This phenomenon of lifting of soil particle by flowing water is called quick sand condition or quick condition or boiling condition.

-> It should be noted that quick sand is not a type of sand but is condition name for quick condition.

⇒ Thus quick condition occurs when :-

$$\sigma' = 0$$

$$\Rightarrow \gamma' z - P_s = 0$$

$$\Rightarrow \boxed{\gamma' z = P_s}$$

$(i = i_c \text{ critical})$

$$\Rightarrow \gamma' z = i \gamma_w z$$

$$\Rightarrow \gamma' \cancel{z} = i_c \gamma_w \cancel{z}$$

$$\Rightarrow i_c = \gamma' / \gamma_w$$

$$\Rightarrow i_c = \frac{\gamma'}{\gamma_w}$$

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

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

from safety

where i_c = critical hydraulic gradient

⇒ The critical hydraulic gradient is the hydraulic gradient at which quick sand condition occurs.

⇒ An experimental set up to demonstrate quick condition in which water flows through soil mass of thickness z under hydraulic head (h) :-

⇒ This head can be adjustable by moving the supply tank up or down.

⇒ This head (h) gradually increased until quick condition is noticed in the soil.

⇒ In this condition upward force of the water of soil mass becomes equal to submerged force due to section of soil above that level.

⇒ If A' is the cross sectional area of mass then we have .

$F_1 = F_2$ ⇒ quicksand condition .

⇒ $P_1 A = P_2 A$

⇒ $\gamma_w \times h \times A + \gamma_w \times Z \times A = \gamma_{sat} Z A$

⇒ $\gamma_w h A = \gamma_{sat} Z A - \gamma_w Z A$

⇒ $\gamma_w h \cancel{A} = (\gamma_{sat} - \gamma_w) Z \cancel{A}$

⇒ $\gamma_w h = \gamma' Z$

⇒ $\frac{h}{Z} = \frac{\gamma'}{\gamma_w}$

$\boxed{i_c = \frac{h}{Z}}$

⇒ Hence we get :- $\boxed{i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} = \frac{h}{Z}}$

Note

(a) If we put $G = 2.67$ & $e = 0.67$

i_c will become unity ($i_c \simeq 1$)

(b) for most cohesionless soil i_c will be less than unity (i.e ≤ 1)

Note

(c) The velocity of flow V_c required to maintain the critical hydraulic gradient i_c is directly proportional to the coefficient of permeability

$\boxed{v = K i_c}$ $v_c \propto K$

1Q. Calculate the critical hydraulic gradient for a coarse grained soil deposit with void ratio of 0.7 & G = 2.67 ?

Sol^n Data given $G = 2.67$

$e = 0.7$

$$i_c = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.7} = 0.98$$

2Q. A soil sample taken in a container of internal diameter 8 cm has a length of 12.5 cm. The coefficient of permeability of soil is 1.5 × 10⁻⁷ [illegible] with void ratio 0.7. If water is made to flow through the soil sample in upward direction & the rate of discharge is [illegible] the [illegible] at the bottom & [illegible] condition of the [illegible] take G = 2.65 ?

Sol^n Data given $G = 2.65$

$q = 0.04\ cm^3/sec$

Container diameter = 8 cm

Container length = 12.5 cm

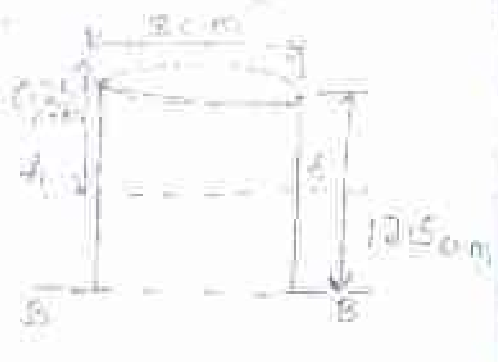
$e = 0.7$

$K = 1.5 \times 10^{-7}\ cm/sec$

$\sigma' = \gamma' z \pm P_S$

$\Rightarrow \sigma' = \gamma' z - P_S$

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



$$\gamma' = \frac{(G-1)\gamma_w}{1+e} = \frac{(2.65-1)\times 9.81}{1+0.7}$$

$$= 9.52\ kN/m^3$$

From darcy's law $q = kiA$

$$\Rightarrow 0.04 = 1.5\times 10^{-3} \times i \times \frac{\pi}{4}\times 8^2$$

$$\Rightarrow i = \frac{0.04\times 4}{1.5\times 10^{-3}\times\pi\times 8^2}$$

$$\Rightarrow i = 0.53$$

The effective stress at middle of the section i.e A-A layer $\sigma' = \gamma' z - iz\gamma_w$

$$= [(9.52)\times(0.0625)] - [0.53\times(0.0625)\times 9.81]$$

$$= 0.27\ kN/m^2$$

The effective stress at the bottom of soil sample i.e B-B layer.

$$\sigma' = \gamma' z - iz\gamma_w$$

$$[(9.52)\times 0.125] - [0.52\times 0.125\times 9.81]$$

$$= 0.54\ kN/m^2$$

Quick sand condition

up word condition is down word condition to same here quick sand condition occure i.e.

FLOW NET

13 NOV 2020

* Flow net :- Flow lines & equipotential lines together constitute a flow net.

=> A flow line represents the path traced by an individual water particle.

=> An equipotential line is a contour or line of joining points of equal potential or head.

=> The flow lines & equipotential lines cut each other at right angles i.e they are mutually orthogonal.

=> The space between any two adjacent flow lines is called flow channel.

=> The space enclosed between two adjacent flow lines & two successive equipotential lines is called a Field.

* Properties of flow net :-

=> Following are the properties of flownet :-

(i) Flow lines & equipotential lines cut each other at right angles i.e. they are mutually orthogonal

(2) Each field is an approximate square & in a well-constructed flow net one should be all the four sides.

(3) The rate of flow through such flow channel is same.

(4) The same potential drop occurs between two successive equipotential lines.

(5) In a homogeneous soil, every transition in the ~~spee~~ slope of the 2 types of curves will be smooth being either elliptical or parabolic in shape.

* Flow net by Graphical method :-

⇒ The graphical method of flow net construction involves sketching by trial & error.

⇒ The hydraulic boundary conditions are examined & keeping in mind the properties of flow net initial sketching is done & by trial & error the flow net is improved to make it acceptable for practical applications :-

(1) Well constructed flow nets should be studied & effort should be made to reproduce the salient features in mind.

(2) About four or five flow channels should be are sufficient for the first trial, as too many flow channels will distract attention from essential features.

(iii) After initial sketching the flow net should be observed as a whole while adjusting the flow details.

(v) All transitions should be made smooth being either elliptical or parabolic in shape.

Application of flow net :-

A flow net can be used to determine the (i) quantity of seepage
(ii) determination of seepage pressure at any point
(iii) hydrostatic pressure at a point
(iv) Exit gradient.

(i) Quantity of seepage :-

The Δq = rate of discharge through each flow channel.

Δh = head drop per field = $\frac{H}{N_d}$

low
whole
g.
ε

H = Head causing flow through entire flow net or quantity of seepage.

$$\boxed{q = kH \frac{N_f}{N_d}}$$

$$\boxed{\begin{aligned} p &= h\gamma_w \\ \gamma_w &= 9.81 \\ h &= H - n\Delta h \\ H &- n\Delta h \\ \Delta h &= \frac{H}{9} \\ n &= 3 \end{aligned}}$$

Where k = coefficient of permeability.

determine

(ii) Determination of seepage pressure at any point :-

point
a point

→ seepage pressure at any point

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

where h = total head at that point
$= (H - n \cdot \Delta h)$

Δh = potential head drop per field $= \frac{H}{N_d}$

H = total head causing flow

n = no of potential drop up to the points.

(iii) Hydrostatic pressure at a point :-

Hydrostatic pressure at a point

through

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

ii

where $h_{w} = $ hydrostatic head $= (h - z)$

h = total head of that point

$= (h = cm)$

Z = datum head at that point.

(iv) Exit gradient =

$$\boxed{\text{Exit } i_e = \frac{\Delta h}{\Delta L_e}}$$

where Δh = potential drop for each field.

ΔL_e = length of smallest exit field.

10 A soil stratum with permeability, $k = 5 \times 10^{-7}$ cm/sec overlies an impervious stratum. The impermeable stratum lies at a depth of 12 m below the ground surface. A sheet pile wall penetrates 8 m into the permeable soil stratum. Water stands to a height of 5 m on u/s side & 1.5 m on d/s side. Sketch the flow net to determine ...

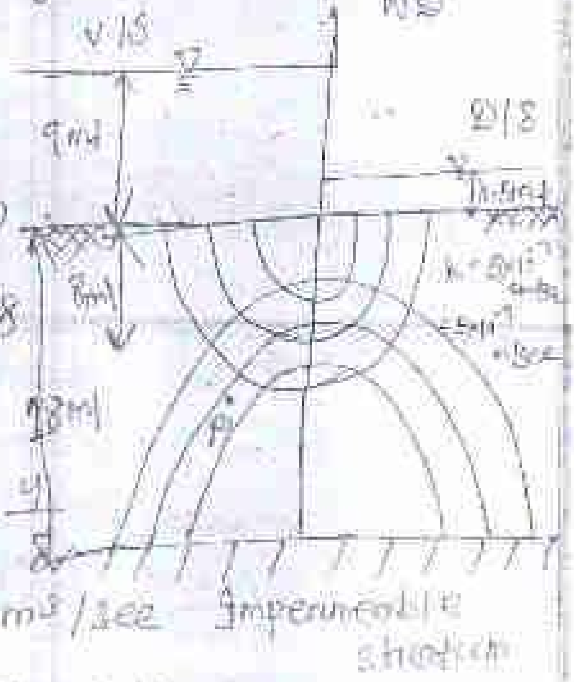
(i) quantity of seepage

(ii) The seepage pressure at a point p located 8 m below the surface of soil stratum & 4 m away from the sheet pile.

(iii) pore pressure at point p.

(iv) Maximum exit gradient.

Sol^n (i) quantity of seepage $(q) = kH \frac{N_f}{N_d}$



$H = H_1 - H_2$

$= 9 - 1.5 = 7.5\,m$

$N_f = 4 \quad N_d = 8$

$K = 5\times10^{-9}$

$= 5\times10^{-9} \times 7.5 \times \frac{4}{8}$

$= 1.875\times10^{-8}\ m^3/sec$

$= 18.75\times10^{-9}\ m^3/sec$

(ii) Seepage pressure at point 'p'

$P_s = h\gamma_w$

$h = H - n\Delta h$

$H = 7.5\ m$

$n \cdot \Delta h = \frac{H}{N_d} = \frac{7.5}{8} = 0.937\ m/sec$

$h = (7.5 - 2.5 \times 0.937)$

$= 5.16\ m$

$P_3 = h_3 \gamma_w = 5.16 \times 9.81$

$= 50.62 \ kN/m^2$

(iii) hydrostatic or pore pressure at point $= h - z$

$h_w = h - z = 5.16 - (-9.5)$

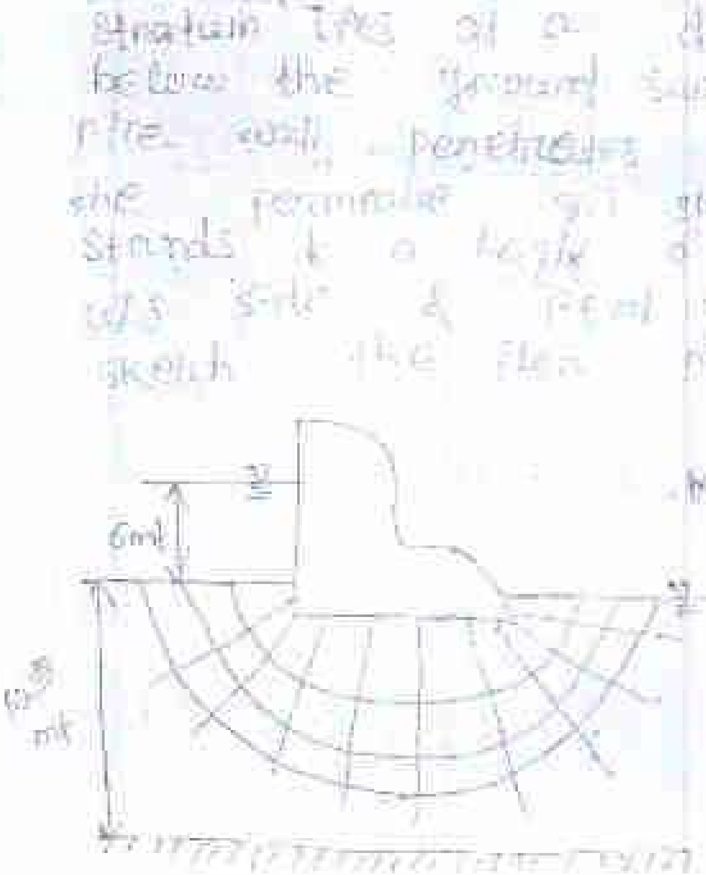
$= 14.66 \ mt$

$P = 14.66 \times 9.81$

$= 143.81 \ kN/m^2$

(iv) $\dfrac{0.937}{2.8} = 0.334 \ mt/sec$

<u>Q4</u> A soil stratum with permeability $K = 5 \times 10^{-?} \ cm/sec$ overlies an impermeable stratum. The impermeable stratum lies at a depth of 18 mt below the ground surface. A sheet pile wall penetrates 8 mt in to the permeable soil stratum. Water stands to a height of 9 mt on u/s side & 1 mt on d/s side. Sketch the flow net & determine



$K = 5 \times 10^{-3} \ cm$

Solⁿ $q = KH \times \frac{NF}{ND}$

$NF = 4$ $H = H_1 - H_2$

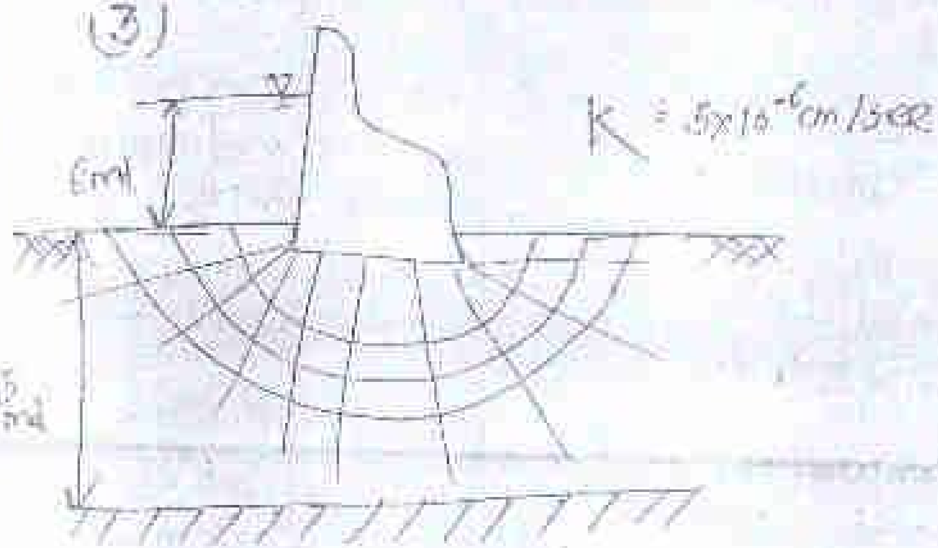
$N\theta = 12$ $= 6 - 0$

$KH = \frac{NF}{N\theta}$ $= 6\,mt$

$q \Rightarrow 5 \times 10^{-8} \times 6 \times \frac{4}{12}$

$q = 1 \times 10^{-7}\, m^2/sec$

(3)



$q = KH \times \frac{NF}{N\theta}$

$H = H_1 - H_2 = 6 - 0 = 6$

$NF = 4$

$N\theta = 14$

$KH \times \frac{NF}{N\theta}$

$= 5 \times 10^{-8} \times 6 \times \frac{4}{14}$

$= 8.571 \times 10^{-8}$

1 Dec 2020

Compaction

> Compressibility of soil mass is an engineering property by virtue of which the soil mass is capable of undergoing compression (or) decrease in volume when subjected to compressive load.

> The two process namely compaction & consolidation involve in reduction in volume but in practice only consolidation is associated with compressibility.

> Compaction is the process in which rapid reduction in volume takes place due to sudden application of loads as caused by running, tamping, rolling & vibrations.

> Consolidation is the process in which gradual reduction in volume takes place due to sustained loading.

> Compaction :- During compaction the reduction in volume is mainly due to expulsion of pore air & rearrange-ment of particles resulting in their closer packing.

> Compaction of a soil mass results in increase in dry density.

⇒ The dry density depends on ary.
⇒ The amount & type of compaction determined the compacting effort.

⇒ For a specific amount of compacting energy applied on soil, the mass attains maximum dry density at a particular water content. This water content is referred as optimum water content.

5 Dec 2020

Effects of compaction on soil properties :-

* The main aim of compacting a soil is to improve some desirable properties of the soil.
* There is reduction in compressibility, water absorption & permeability, increase in soil strength & bearing capacity.
* There is change in shrinkage & swelling characteristics.
* Following are the factors due to effect of compaction.

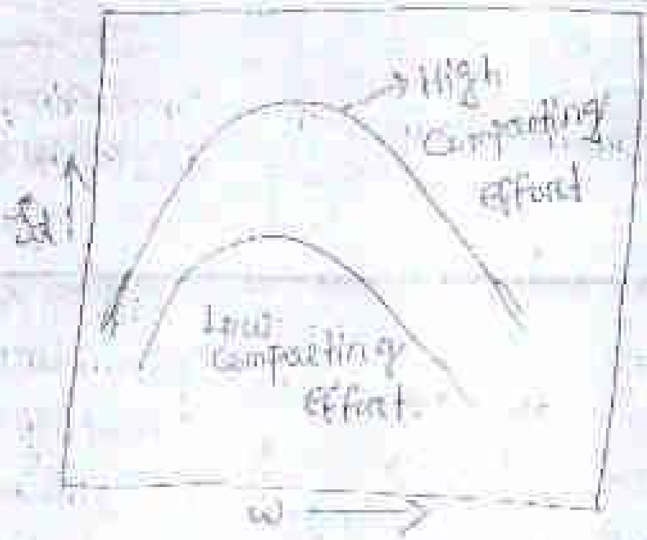
① Change in structure of soil :-

⇒ The structure of soil during compaction depends upon :- type of soil, water content, type & amount of compaction.

⇒ Normally the soil is divided into 3 types - Coarse grained soil, Composite soil, Purely cohesive soil (i.e clay)

⇒ The soil of first type maintain a single grain structure & Composite soil is the combination of both fine & coarse grained.

⇒ The structure of compacted clay is complicated.



(8) Permeability

⇒ The following points are noted :-

① As the dry density increases due to compaction the voids go on reducing & hence permeability decreases.

② For the same density, fine grained sample

(ii) For the same density, fine grained samples compacted dry of optimum are more permeable than those compacted wet of optimum.

(iii) For a given void ratio, greater the size of individual pores, greater permeability.

(iv) As the compacting efforts increases the permeability is decreases.

(3) Shrinkage :-

=> For the same density soil sample compacted dry of optimum shrinks less than the wet of optimum.

=> This is so because soils particles are dispersed structure have nearly parallel with each other.

(4) Swelling :- A clayey soil sample compacted dry of optimum water content has higher water deficiency & exert more swelling pressure & swell to water content than the same density of soil obtained from wet of optimum.

(5) Pore pressure :-

=> Saturated sample of clay compacted dry of optimum tend to develop low pore pressure than same soil of the same density & water.

Content compacted wet of optimum.

(6) Compressibility :-

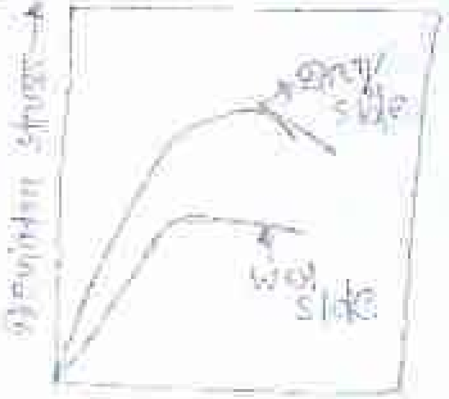
=> Saturated sample of clay compacted wet of optimum is more compressible than another sample of same soil.

=> This is so because sample compacted.

(7) Stress-strain characteristics :-

=> For a given soil a sample compacted dry side of optimum has a steeper stress strain curve & high modulus of elasticity.

=> With same density of soil compacted wet of optimum have brittle failure & curve is lower than dry side.



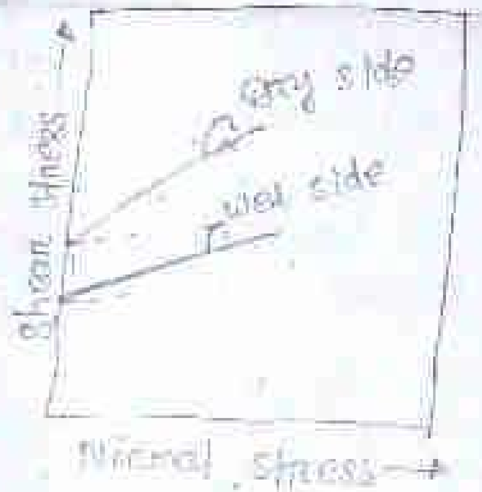
(7) Shear Strength-

⇒ Shear strength of soil is depending upon - i dry density, water content, method of Compaction, type of soil structure, drainage in soil sample.

⇒ In low strain the strength of Cohessive soil Compacted dry of optimum is higher than the Compacted wet of optimum.

⇒ In higher strain, the strength of Compacted dry of optimum is lower than the Compacted wet of optimum.

⇒ The failure envelope of wet side is lower than the dry side.



Saturated Procter test :-

⇒ The standard procter test was developed by R.R procter.

⇒ The test equipment consists of :-

① Cylindrical metal mould having internal diameter 10 cm & height 11.7 cm.

② detachable base plate.

(ii) collar

(iii) hammer (2.6 kg)

=> About 3kg of air dried soil, passing a 4.75mm sieve is mixed thoroughly with a small quantity of water.

=> About 3kg of air dried soil, passing a 4.75mm sieve, is mixed thoroughly with a small quantity of water.

=> The mixture is covered with wet cloth & left for a maturing time of about absorption of water.

=> The initial water content may be taken 4% for coarse grained soil & 8% for fine grained soil.

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

=> Then the collar is attached the mixed nature soil is placed in 3 layers in the mould & each layer is compacted by giving 25 blows of the hammer uniformly distributed over the surface, such that the compacted height of soil is taken about 1/3 height of the mould.

=> Before putting the second layer of soil the top of the first compacted layer is scratched with the help of any sharp edge.

=> The last compacted layer should project not more than 6mm above the collar.

=> The collar is removed & the excess soil is trimmed off to make it level with top of mould.

=> The weight of the mould, base plate & compacted soil is taken.

=> A representative sample is taken from the centre of the compacted specimen & kept for water content determination.

=> The bulk density 'γ' & dry density 'γ_d' for the compacted soil are calculated from :-

$$\gamma = \frac{W}{V}, \quad \gamma_d = \frac{\gamma}{1+w} \ (g/cm^3)$$

=> Take samples with different water content & plot it in a curve.

=> This compaction curve is plotted between the water content & dry density.

=> The water content corresponding to the maximum density is called optimum water content (w_o)

Air void line :-

=> A line which shows the water content dry density relation for the compacted soil containing a constant percentage air voids is known as an air void line established by —

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

where n_a = percent air voids

w = water content of compacted soil

G = specific gravity.

γ_d = dry density

γ_w = density of water = 1 g/cm³

* The theoretical maximum compaction for any given water content corresponds to zero air void condition. The line showing the dry density as a function of water content for soil containing no air voids is called the zero air voids line or the saturation line.

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

* Modified proctor test :-

-> Higher compaction is needed for heavier transport & military air craft.

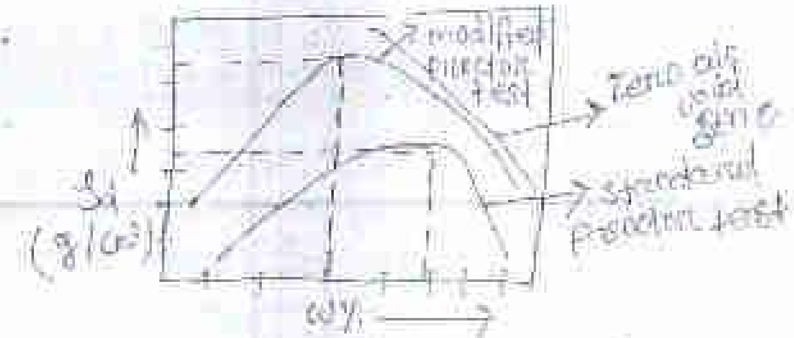
-> This test was standardised by the American Association of State Highway Officials & is known as modified by AASHO test.

-> In this test the soil is compacted in the standard proctor mould in five layers, each layer being given 25 blows of (4.9 kg) rammer dropped through a height of 45 cm.

⇒ The compactive energy given to the soil in this test is 2700 kg-cm per 1000 cm³ of soil.

⇒ In modified proctor test, the water content dry density curve lies above the standard proctor test curve & has its peak relatively placed towards the left.

⇒ Thus for same soil, the effect of heavier compaction is to increase in the maximum dry density & to decrease the optimum water content.



Factors Affecting Compaction :-

⇒ The various factors which affect the compacted density are as follows :-

① Water content :-

※ It has been seen by in the lab that as the water content is increased the compacted density goes on increasing, till a maximum dry density is obtained after which further addition of water decreases the density.

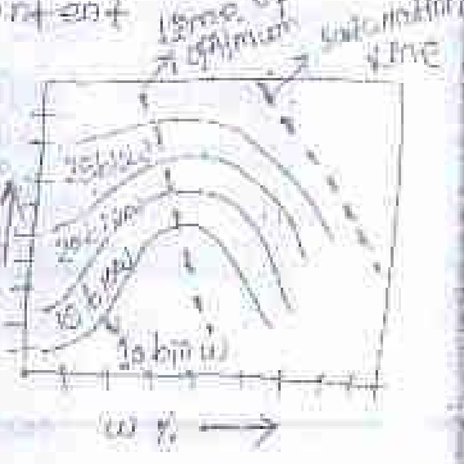
※ The total voids due to water & air with increasing combination go on increasing beyond the optimum of water content & hence the dry density of the soil falls.

(2) Amount of Compaction :-

* The amount of compaction greatly affects the maximum dry density & optimum water content of a given soil.

* The effect of increasing the compactive energy results in an increase in the maximum dry density & decrease in optimum water content.

* The increase in maximum dry density does not have a linear relationship with increase of compactive effort.



(3) Method of compaction :-

* The density obtained during compaction for a given soil, greatly depends upon the type of compaction or the manner in which the compactive effort is applied.

* The variables in this aspect are :-
(i) weight of compacting equipment.
(ii) manner of operation such as dynamic impact, static kneading & rolling.
(iii) time & area of contact between compacting element & soil.

4) Type of soil :-

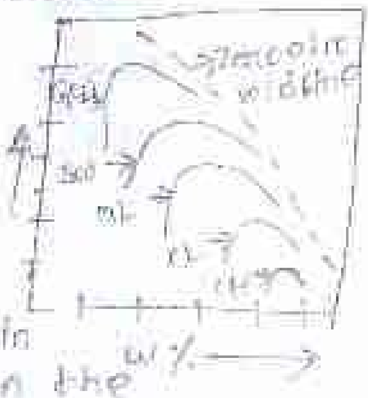
⇒ The maximum dry density achieved corresponding to a given compactive energy largely depends upon the type of soil.

⇒ Well graded coarse-grained soil attain a much higher density & lower optimum water content than fine grained soils which require more water for lubrication because of the greater specific surfaces.

⇒ In general coarse grained soils can be compacted to higher dry densities than fine grained soil.

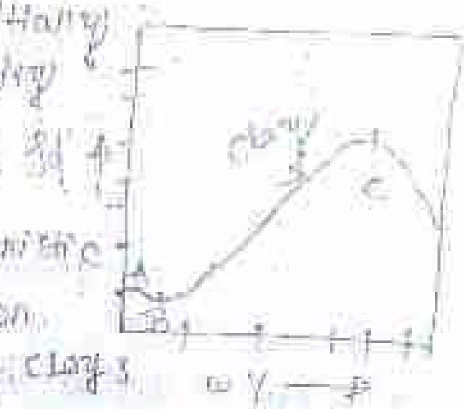
① Compaction curve for cohesionless sands :-

⇒ In case of cohesionless soils which are devoid of fines the water content has very little influence on the compacted density.



⇒ For sandy soils the dry density decreases with an increase in water content in the initial stage of the curve. This is due to the "bulking of sand", where the capillary tension developed in the sandy soil is not fully counteracted by the compactive effort & this capillary tension holds the particles in a loose state resisting compaction.

⇒ Here there is initially decrease of dry density at lower water contents.



⇒ This is the characteristic feature of black cotton soil, highly swelling clays & some fat clays.

⇒ The optimum water content for such soils range between 20 to 25 %.

Addition of admixtures :-

* The compaction properties/characteristics of a soil can be modified by a number of admixtures other than soil materials.

* These admixtures have special application in stabilised soil construction.

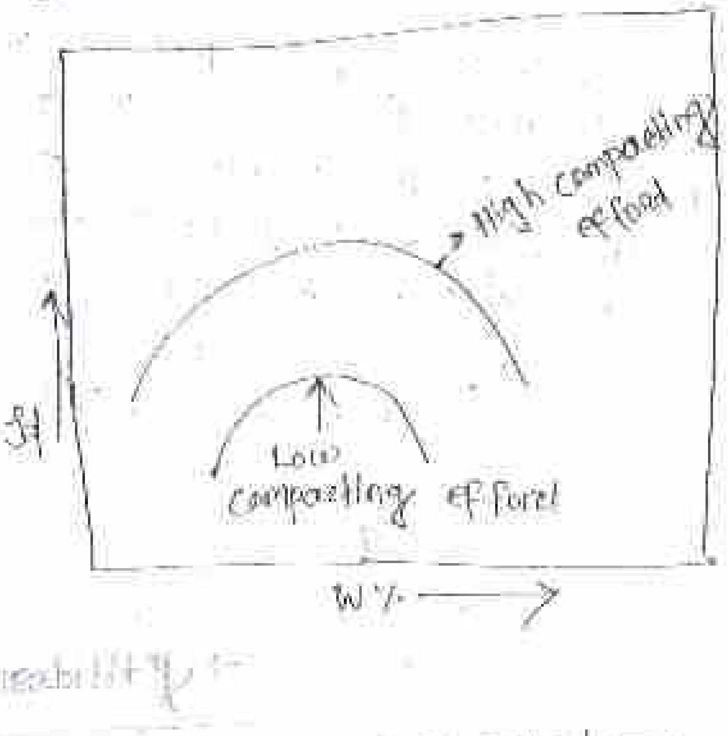
3 [illegible] 2018

Effects of Compaction on soil properties:-

* The main aim of Compacting a soil is to improve some desirable properties of the soil.
* There is reduction in compressibility water absorption & permeability, increase in soil strength & bearing capacity.
* There is change in shrinkage & swelling characteristic due to effect of compaction
* following are the factors

(1) change in structure of soil :-

* The structure of soil during Compaction depends upon-; types of soil, water content, type and amount of Compaction.
* Normally the soil is divided into 3 type:- Coarse grained soil, composite soil, purely cohesive soil (i.e clay)
* The soil of first type maintain a single grained structure & composite soil is the combination of both fine & coarse grained.
* The structure of compacted clay is complicated.



(2) permeability :-

The following points are noted -;

① As the dry density increase due to Compaction, the voids go on reducing & hence permeability decreases.

(ii) For the same density, fine grained sample compacted dry of optimum are more permeable than those compacted wet of optimum.

(iii) For a given void ratio, greater permeability.

(iv) As the compacting effort increases, the permeability of decrease.

(3) Shrinkage -

→ For the same density soil sample compacted dry of optimum shrink less than the wet of optimum.

→ This is so because soil particles are dispersed structure have nearly parallel with each other.

(4) Swelling - A clayey soil sample compacted dry of optimum water content has high water deficiency & more swelling pressure & sweel to higher water content than the same density of soil obtained from wet of optimum.

(5) Pore pressure :- Saturated sample of clay compacted dry of optimum tend to develop low pore pressure and water content compacted wet of optimum.

(6) Compressibility :-

→ Saturated sample of clay compacted wet of optimum is more compressible than another sample of same soil. compacted dry of optimum

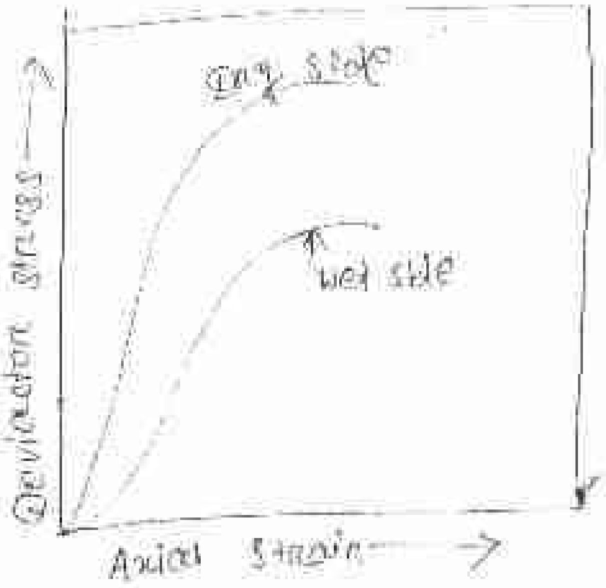
→ This is so because sample has flocculated structure & requires extra pressure to be parallel orientation of particles.

→ In high pressure range a sample compacted dry of optimum is more com than wet of optimum.

(7) Stress - Strain characteristics -

→ For a given soil a sample compacted dry side of optimum has a steeper stress - strain curve & high modulus of elasticity.

⇒ with same density of soil compacted wet of optimum have brittle failure & curve is lower than dry side.



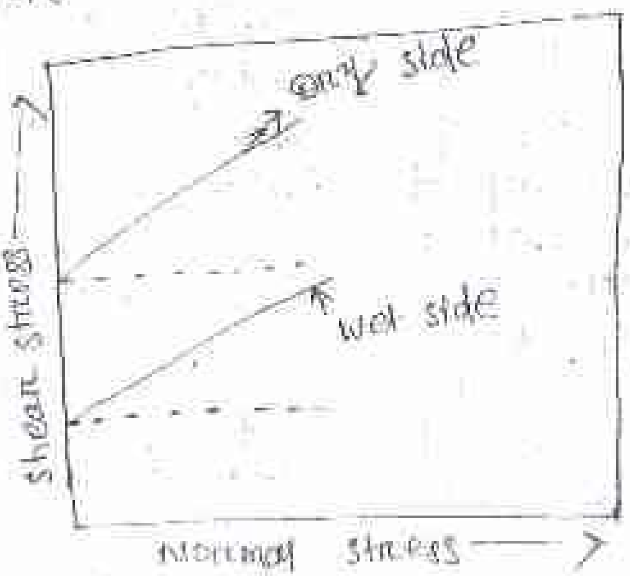
(V) Shear strength :-

⇒ shear strength of soil is depending upon :- dry density, water content, method of compaction, type of soil structure, drainage in soil sample.

⇒ In low strain, the strength of cohesive soil compacted dry of optimum is higher than the compacted wet of optimum.

⇒ In higher strain, the strength of compacted dry of optimum is lower than the compacted wet of optimum.

⇒ The failure envelope of wet side is lower than the dry side.



7 Dec 2020

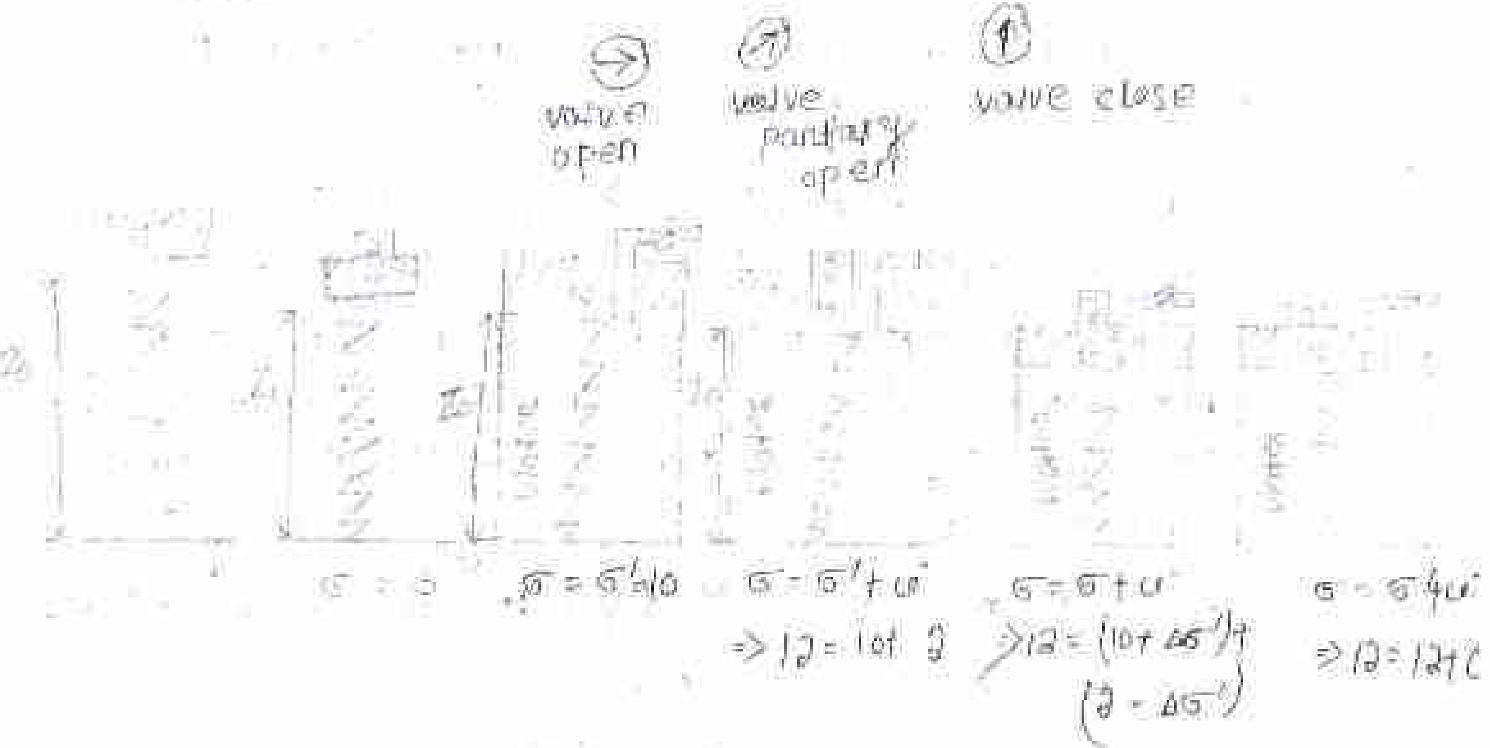
Consolidation

* When a compressive load is applied to soil mass a decrese in it's volume takes place. The decrese in volume of soil mass under stress is known as compression.
* For the voids removing the air by compresibility is known as compaction.
* According to "Terzaghi" Every process involving a decrease in the water content of saturated soil without replacement of water by air is called process of consolidation".

Difference between compaction & consolidation :-

Compaction	Consolidation
① Compaction is a process where a mechanical pressure is used to compress the soil mass.	① Consolidation is a process where steady & static pressure causes compression
② Dynamic loads such as tamping, rolling, vibrating are applied for a small interval in soil compaction.	② Static & sustained loading is applied for a long interval in soil consolidation.
③ In compaction soil volume is reduced by removing air void from the saturated & dry soil.	③ In consolidation process soil volume is reduced by squeezing out pore water from the saturated soil.
④ Compaction is mainly used for sandy soil.	④ Consolidation is used for clayey soil.

* Consolidation process by spring Analogy :-



Grainstructure ⟶ Spring

voids with water ⟶ Cylender

valve opening ⟶ Permeability

9 Dec 2020

-> Terzaghi demonstrated the mechanics of Consolidation by spring Analogy

-> A saturated soil mass taken in a container consists of soil particles forming the skelton of soil mass & voids filled with water.

-> The skelton forme of soil particles can be assumed to be replaced by a member of springs & filling voids in soil mass by the the water filling the water filling the cylinder.

-> The compressive stress is caused by load applied on piston placed on top of the springs.

-> An outlet with valve is provided to control drainage of water from out of the cylinder.

=> Let z_0 be the length of spring under a pressure 10 units.

=> Let the length decrease to z_1 when the pressure is increased by 2 units.

=> In fig (iii), (iv), (v), (vi) spring with piston is placed in a container with water.

=> In fig (iii) the valve is open but no drainage in take place as the entire pressure of 10 units is borne by the spring & the pressure in water is zero.

=> For soil mass by analogy :- $\boxed{\sigma = \sigma' + u}$

Where σ = Total stress

σ' = effective stress

u = pore water pressure.

=> In fig (iv) additional pressure of 2 unit acts & the valve is closed. Beacause water is incompressible the spring are prevented from under going any further compression & there force the additional pressure will have to be borne by water. i.e $\sigma = \sigma' + u$

$\Rightarrow 12 = 10 + 2$

=> In fig - (v) the valve is partly open & as the water starts flowing out transfer of additional pressure from water to spring commences & at any intermediate stage, we have $\sigma = \sigma' + u$

$\Rightarrow 12 = (10 + \Delta\sigma') + (2 - \Delta\sigma')$

where $\Delta\sigma'$ = additional pressure transferred to spring.

=> In fig (vi) the valve is fully open & the rate of drainage of water increases & finally drainage stops when all additional pressure is transfer from water to the spring

⇒ This is similar to the condition when the excess pore pressure has fully dissipated in case of soil mass i.e $\sigma = \sigma' + u$

$\Rightarrow \sigma = \sigma' + 0$

⇒ under an applied pressure the soil mass will have reached a particular value of void ratio when the primary consolidation is complete.

⇒ This value is referred to as final equilibrium void ratio.

⇒ Thus as the pressure is incremented in stages and full primary consolidation allowed at each stage.

⇒ The pressure increment that causes consolidation to take place at any stage is called consolidation pressure.

Compressibility

- Compaction
- Consolidation
  - Primary
  - Secondary

Compaction

(i) It is an instantaneous process of reducing the volume of voids due to expulsion of pore air.

(ii) Degree of saturation $\boxed{S < 1}$

Primary

(i) It is a time dependent process of reducing the volume of voids due to expulsion of pore water.

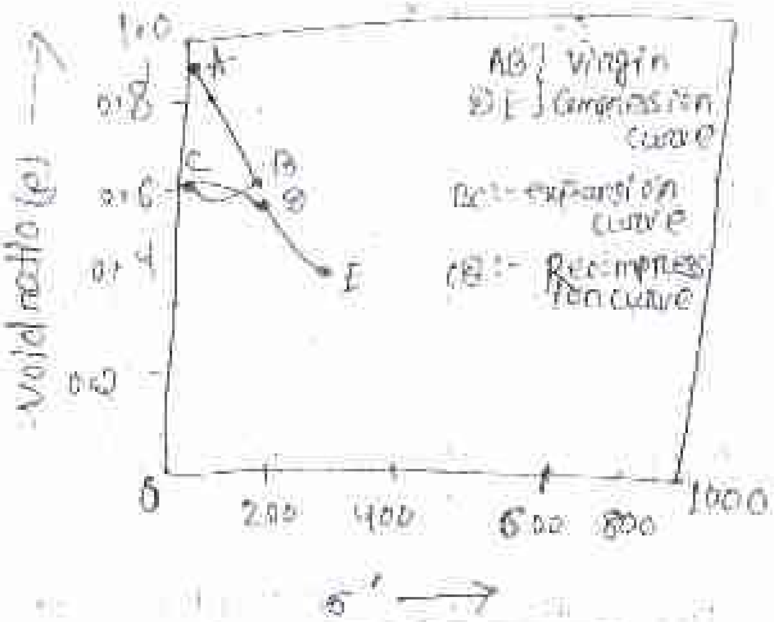
(ii) Degree of saturation $\boxed{S = 1}$

Secondary

(i) It is also a time dependent process of reducing the volume of voids due to plastic rearrangement of soil solids.

(ii)

Consolidation of laterally Confined soil



$$\boxed{C_c = \frac{e_0 - e}{\log_{10}\frac{\sigma'}{\sigma_0'}}}$$

e_0 = initial void ratio

σ_0' = initial stress

$$\boxed{C_c = \frac{\Delta e}{\Delta \log_{10}\sigma'}}$$

Skempton :- $C_c = 0.007(w_L - 10)$

$C_c = 0.009(w_L - 10)$

Coefficient of compressibility :-

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

Consolidation of laterally confined soil specimen (one Dimensional consolidation)

If a soil specimen is laterally confined and subjected to vertical pressure, compression or consolidation takes place in the vertical direction in the laboratory, Consolidation tests can be conducted both on remoulded soil specimen - the drainage condition in the field is simulated by using tube porous plate and a non porous plate for single drainage conditio-n. The soil specimen is sand witched betn the two plates and pressure applied in increment top plate. Under any applied pressure excess pore pressure builds up and as the pore water drains out compression the vertical direction proceeds and after sometime when excess pore pressure is fully dissipated i.e. $u = 0$, the equilibrium state, is reached. At this stage the effective stress $\bar{\sigma}'$ in soil specimen become equal to applied pressure. The final equilibrium void ratio, can be computed. During the progress of test the equilibrium void ratio obtained under different applied pressu-re are found. The void ratio is plotted as ordinate against effective stress $\bar{\sigma}'$ as abscissa to obtain the relation betn the two.

* In typical curves illustrate the relation betn void ratio and effective stress for a laterally confined remoulded soil specimen are shown. The curve AB is obtained by increasing the applied pressure in increment allowing equilibrium stage to be reached under each pressure. If at stage corresponding to point B, the applied pressure is (soil specimen expan-)... soil specimen expan- as indicated by curve BC. Curve BC Rebound the soil specimen will not attain again the ori-ginal void ratio corresponding to beginning to test because it will have undergone some permanent compression which can be attributed to irreversible orientation undr grain by soil particles. If the specimen is recompressed and the test continued the curves CD and DE are obtained. The curves AB and DE correspond to ... of specimen during

AB } virgin
DE } compression curve

BC :- Expansion curve

CD :- Recompress-ion curve

which at any stage, the applied pressure is greater than any pressure to which the soil specimen has been subjected to in the past, they are referred to as virgin compression curves. The curve BC is called expansion curve and the curve CD, the recompression curve. It is to be observed that point D of recompression curve does not coincide with B even through both correspond-ed same effective stress, clearly D lies below B indicating that void ratio attained during recompression is less than attained during virgin compression under the same applied pressure.

If void ratio e is plotted as ordinate on natural scale against effective stress σ' as abscissa on logarithmic scale, the virgin compression curve and the ~~virgin~~ expansion curve become nearly straight lines.

(Effective Stress)

According to Terzaghi, the virgin compression curve can be defined by the following empirical relation.

$$\boxed{e = e_0 - C_c \log_{10} \frac{\bar{\sigma}'}{\bar{\sigma}'_0}}$$

where e_0 - Initial void ratio corresponding to initial effective stress $\bar{\sigma}'_0$

e = void ratio corresponding to increased effective stress σ'

C_c denotes compression index and it is the slope of straight line portion of virgin compression curve and it found to remain constant within a fairly large range of pressure.

$$\boxed{C_c = \frac{e_0 - e}{\log_{10}\frac{\sigma'}{\sigma'_0}} = \frac{\Delta e}{\Delta \log_{10}\sigma'}}$$

The expansion curve on semi-log plot is defined by the following relation.

$$e_0 = e + C_s \log_{10}\frac{\sigma'}{\sigma'_0}$$

C_s denotes expansion index or swelling index. It is the slope of straight line portion of the expansion curve and is a measure of the increase in volume that occurs on removal of pressure.

Skempton (1944) has given the following equation for estimating C_c for remoulded clay sample

$$C_c = 0.007(w_L - 10)$$

For undisturbed clay of medium to low sensitivity the value of C_c is roughly equal to 1.3 times that corresponding to remoulded sample and therefore can be estimated by.

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

In the value of w_L to be substituted is that expressed as a percentage.

Coefficient of Compressibility

The coefficient of compressibility denoted by a_v is defined as the decrease in void ratio unit increase in pressure.

$$a_v = \frac{-\Delta e}{\Delta \sigma'} = \frac{-(e - e_0)}{\sigma' - \sigma'_0}$$

When e_0 = void ratio under pressure σ'_0

e = void ratio under pressure σ'

The minus sign indicates decrease in void ratio for any given difference in pressure it is found that the coefficient of compressibility is not constant for different pressure ranges but increases with increasing values of initial pressure σ'_0.

Coefficient of volume change:-

The coefficient of volume change, also known as coefficient of volume compressibility, is denoted by m_v and is defined as the decrease in volume of soil mass per unit volume due to unit increase in pressure.

$$m_v = \frac{-\Delta V}{V_0} \cdot \frac{1}{\Delta \sigma'}$$

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

When the soil mass is laterally confined, the decrease in volume ΔV is proportional to decrease in thickness ΔH and the initial volume is proportional to initial thickness H_0. Therefore we can write.

$$m_v = \frac{\Delta H}{H_0} \cdot \frac{1}{\Delta \sigma'}$$

The compression ΔH due to pressure increment $\Delta \sigma'$ is given by

$$\Delta H = m_v H_0 \cdot \Delta \sigma'$$

Depending on state of consolidation soil deposit are divided into three types :-

(i) Preconsolidated deposit
(ii) Normally Consolidation deposit
(iii) Under Consolidation deposit

A soil deposit is said to be preconsolidated, precompressed or overconsolidated if it has in the past been fully consolidated under a pressure greater than present overburden pressure acting on the soil. The preconsolidation loading may have been caused by a geologic overburden in the past or structural load which has been subsequently removed.

A soil deposit is said to be normally consolidated if it has never been subjected to a pressure greater than the present overburden pressure and has been fully consolidated under the presently acting pressure.

An underconsolidated soil deposit is one which is still not fully consolidated under the existing overburden pressure.

Terzaghi's Theory of one Dimensional Consolidation :-

Terzaghi (1923) derived the basic differential equation of consolidation which represents the firststep in the theoritical analysis of the consolidation process.

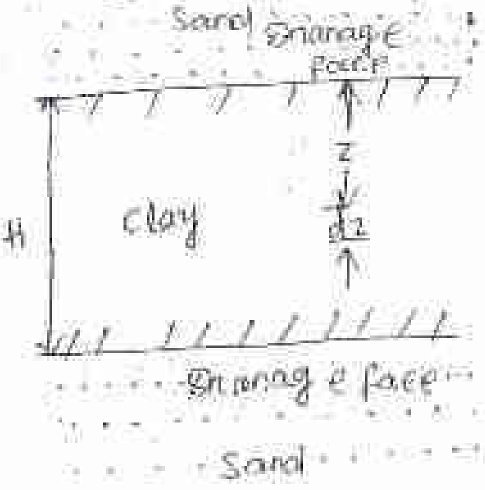
> Following are the assumptions made in Terzaghi's one-dimensional consolidation theory.

(i) The soil mass is homogeneous and fully saturated.

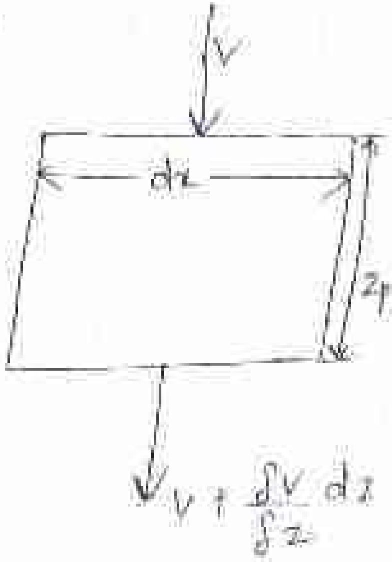
(ii) The soil particles and water are incompressible i.e.

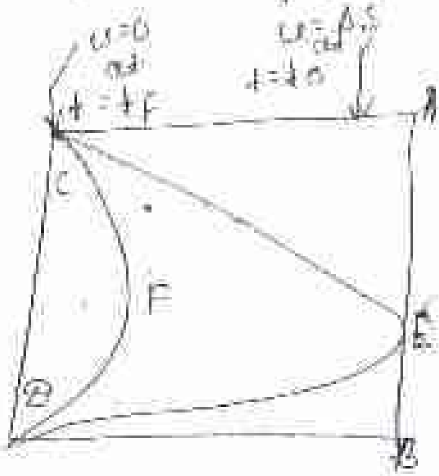
(3) Darcy's law flow of water through soil mass is applicable during consolidation.
(4) Coefficient of permeability is constant during consolidation.
(5) Load is applied in one direction only and deformation occurs only in the direction of load application.
(6) The deformation is due entirely to decrease in one direction.
(7) The drainage of pore water occurs only in one direction.
(8) A boundary drainage face offers no resistance to flow of water from soil.
(9) During consolidation the change in thickness is continuous but final value of compression is related to initial thickness only.
(10) The time lag in consolidation is due entirely to permeability of soil. Any secondary time effect is disregarded.

Let a saturated clay layer of thickness H lie betn two layers of sand which serve as two drainage faces. When the clay layer is subjected to a pressure increment $\Delta\sigma'$ the pressure borne by pore increment is first borne by pore water so that at initial time to the excess pore pressure $u = \Delta\sigma$ at all points along the depth of clay layer and is plotted as line AB. Drainage of pore water into the sand layer starts and the excess pore pressure at the top and bottom boundaries of clay layer drops down to zero and remains so at all time, during the consolidation process. At the end of consolidation process, say, at $t = t_f$ the excess pore pressure will have been completely dissipated so that $u = 0$ at all points and is represented by the line C . At any intermediate time bet^n t_0 and t_f part of consolidating pressure $\Delta\sigma$ is transferred to soil particles so that $\Delta\sigma = \Delta\sigma' + u$. The distribution of excess pore pressure u at any intermediate time t is represented by a curve such as CFE . A number of such curves by a representing excess pore pressure distribution along the depth of clay layer at different instants of time $t = t_1, t_2, \ldots$ can be drawn and they are they are known as isochrones. The slope of an isochrone at any point at a given time gives the rate of change of u with depth.



ⓐ consolidating layer





ⓑ Isochrones

At any time t, the hydraulic head h corresponding to the excess pore pressure u is given by

$$h = \frac{u}{\gamma_w} \quad \text{——} \quad (1)$$

The hydraulic gradient i is given by

$$i = \frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \cdot \frac{\partial u}{\partial z} \quad \text{——} \quad (2)$$

Applying Darcy's law, the velocity of flow of pore water due to this hydraulic gradient is given by

$$v = ki = \frac{k}{\gamma_w} \cdot \frac{\partial u}{\partial z} \quad \text{——} \quad (3)$$

The rate of change of velocity along the depth of the layer is given by

$$\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} \quad \text{---} (V)$$

Let us consider a soil element of side dx, dz and of width dy perpendicular to the plane of figure. If v is the velocity of water at entry, the velocity at exit will be $\left(v + \frac{\partial v}{\partial z} \cdot dz\right)$ as indicated the quantity of water entering the soil element in unit time $= v \cdot dx \cdot dy$ the quantity of water leaving the soil element in unit time $= \left(v + \frac{\partial v}{\partial z} \cdot dz\right) dx \, dy$

Hence the net quantity of water squeezed out of the soil element in unit time is given by.

$$\Delta q = v + \frac{\partial v}{\partial z} dz \Big) dx\, dy - v\, dx\, dy \Big)$$

$$\Delta q = \frac{\partial v}{\partial v} dx\, dy \quad dz$$

The decrease in the volume of soil element is equal to the volume of water squeezed out also, we have

$$\Delta V = -m_v \, V_0 \, \Delta\bar{\sigma}'$$

where V_0 = volume of soil element at time t_0 = $dx \, dy \, dz$

∴ change in volume per unit time is given by

$$\frac{\partial}{\partial t}(\Delta V) = -m_v \left(dx\, dy\, dz\right) \frac{\partial(\Delta\bar{\sigma}')}{\partial t}$$

Comparing eq (V) and (VI), we get

$$\frac{\partial v}{\partial z} = -m_v \frac{\partial(\Delta\bar{\sigma}')}{\partial t}$$

Now $\Delta\bar{\sigma} = \Delta\bar{\sigma}' + u$

(xi) $\Delta T' = \Delta T - u$ where ΔT is Constant

$\therefore \frac{\partial}{\partial t}(\Delta T') = -\frac{\partial u}{\partial t}$

Substituting in eq (viii)

$$\frac{\partial v}{\partial z} = m_v \frac{\partial u}{\partial t}$$

Comparing eq (vi) and eq (x) we get

$$\frac{\partial u}{\partial t} = \frac{k}{m_v \gamma_w} \cdot \frac{\partial^2 u}{\partial z^2}$$

(xi) $\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2}$

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

C_v denotes coefficient of consolidation.

The coefficient of consolidation C_v as defined in eq 11.16 indicates the combined effects of permeability and compressibility of soil on the rate of volume change. If k is expressed in m/sec, m_v in m^2/kN and γ_w in kN/m^3, unit of C_v will be m^2/sec.

The mathematical steps involved in obtaining the solution by means of fourier series, of the differential equation of consolidation is present in Appendix. However, the following points can be understood even without going through the detailed solution.

The hydraulic boundary condition to be satisfied by the solution of the differential equation of consolidation are:-

(i) at $t = 0$, at any distance z, $u = u_0 = \Delta T$

(ii) at $t = \infty$, at any distance z, $u = 0$

(ii) at any intermediate time t, at z = 0, u = 0 and at z = H, u = 0

If ρ_f denotes final settlement under pressure increment $\Delta\sigma$ and ρ the settlement at any intermediate time t, then the degree of consolidation attained at that time t is given by

$$U(\%) = \frac{\rho}{\rho_f} \times 100$$

The degree of consolidation is a function of time factor T_v.

$$U(\%) = f(T_v)$$

The time factor T_v is a dimensionless parameter defined by the following equation

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

where d = drainage path. The drainage path represents the maximum distance a water particles has to travel within the layer to reach a drainage face. when a clay layer bound by two drainage faces, double drainage occurs. when the clay layer is bound by a drainage face at one end and single drainage occurs.

For the case of double drainage, $d = \frac{H}{2}$

For the case of single drainage $d = H$

where H = thickness of layer.

The time factor $T_v = \frac{C_v t}{d^2} = \frac{k}{m_v \gamma_w} \cdot \frac{t}{d^2}$

* We notice that the time factor, and hence the degree of consolidation, depends upon (i) coefficient of permeability, k (ii) coefficient of volume compressibility, m_v (iii) number of thickness of layer and (iv)

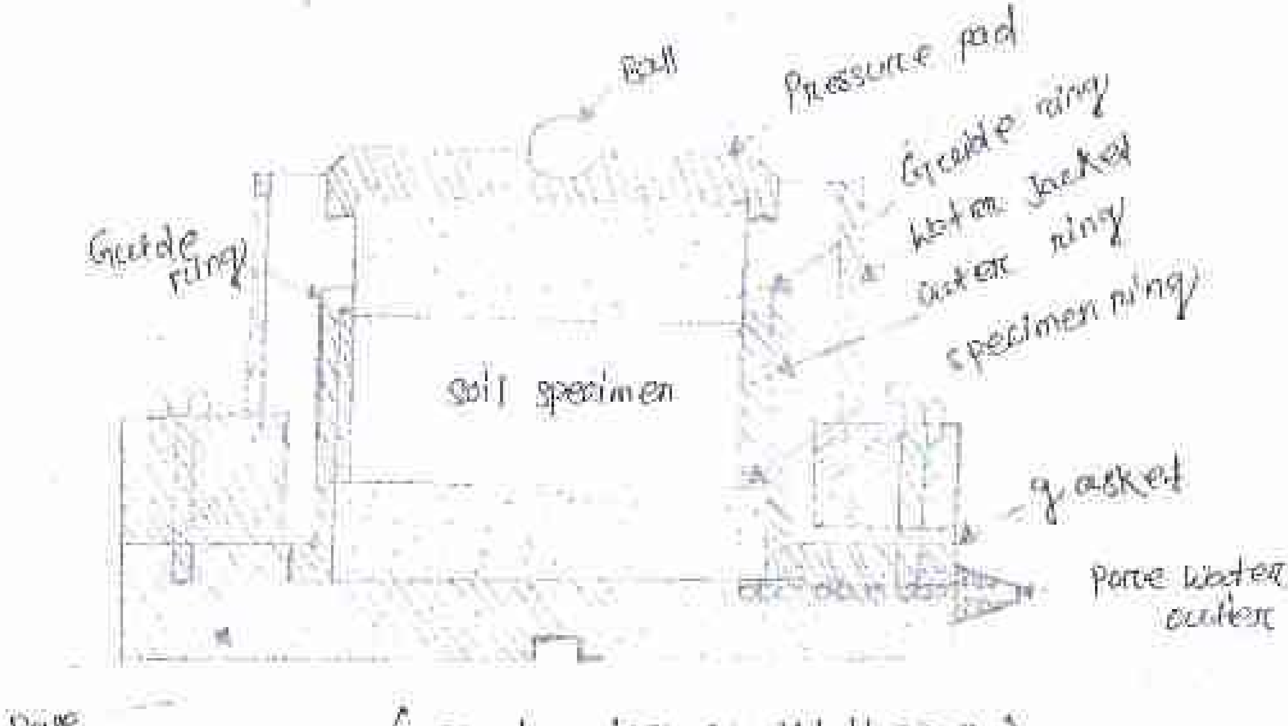
drainage faces. In addition it is found to depend upon the consolidating pressure and its manner of distribution across the depth of layer. The time factor T_v correspon-ding to various value of values of degree of consolidation U for the two types of drainage conditions and different distributions of consolidating pressure are presented in the following approximate expressions may be used to computer T_v, in the absence of the tables.

when $U < 60\%$, $T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$

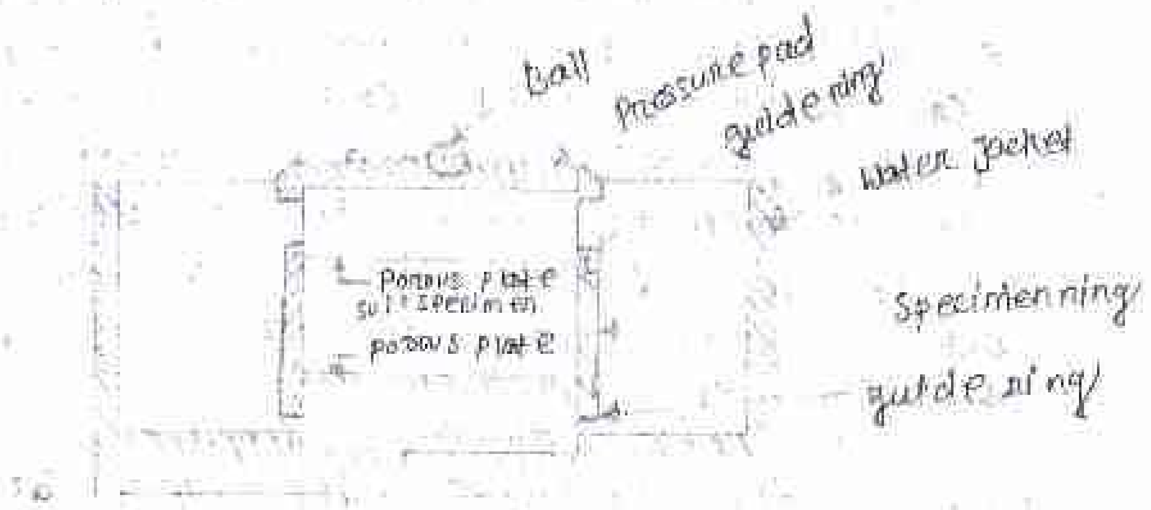
and when $U > 60\%$, $T_v = -0.9332 \log_{10}\left(1-\frac{U}{100}\right) - 0.0851$

Consolidation Test :-

The apparatus used in the laboratory consolidation test is known as consolidometer (or oedometer). It consists essentially of a loading frame and a consolidation cell. The soil specimen is kept in the consolidation cell. To simulate double drainage condition two porous plates, one on top and the other at bottom of specimen are used. In the case of single drainage condition only one porous plate is used, the other being replaced by a non-porous plate. In the fixed ring cell the bottom porous plate is fixed relative to the top plate and only the top of plate is free to move downward and compress the specimen. In the floating ring cell, both top and bottom porous plate are relatively free to compress the specimen towards the middle.



(Fixed ring consolidation cell)



(Floating ring consolidation cell)

The Floating ring cell has the advantage of
having smaller effects of friction betn the specimen
ring and the soil specimen where as direct
measurement of permeability of the specimen at
any stage of loading can be made only in
the fixed ring cell.

The loading frame is equipped to apply
on the soil specimen in
vertical pressure . During the test the
convenient increments to consolidate fully
specimen is allowed
under different vertical pressure such as
10, 20, 50, 100, 200, 400, 800, 1600 kN/m² Each
pressure increment is maintained constant until the
compression ceases, generally for 24 hours
The vertical compression of specimen is measured
with the help of a dial gauge and dial
gauge readings are taken after application of
each pressure increment, at the end of elapsed
time intervals of 0.25, 1.00, 2.25, 4.00, 6.25
9.00, 12.25, 16.00, 20.25, 25, 36, 49, 60 minutes
and 2, 4, 8 and 2, 4, 8 and 24 hours. Attention
is paid to record the final compression under
each pressure increment. After completion of
consolidation under the desired maximum vertical
pressure, the specimen is unloaded and
allowed to swell. The completion of swelling
is recorded. The specimen is taken out
and dried to determine its water content
and the weight of soil solids. The consolidation
test data are used to determine the
following -

1. void ratio and coefficient of volume change
2. coefficient of consolidation
3. coefficient of permeability

Secondary consolidation :-

The primary consolidation under a pressure increment ceases when the excess pore pressure caused by the applied pressure increment is fully dissipated. But some compression is observed even after the primary consolidation has ceased. It is referred to as secondary consolidation and is due to highly viscous water between the points of contact of soil particles being forced out, change in orientation of soil particles and possible fracture of some of the particles due to creep.

In many inorganic soil deposits the magnitude of secondary compression is much less than that of primary compression and is often neglected. Terzaghi's and theory of consolidation is not applicable to secondary consolidation as it is not governed by dissipation of excess pore pressure. It can be observed that any experimental time-compression curve will be in agreement with Terzaghi's theoretical curve only upto about $U = 60\%$. This indicates that secondary consolidation comes into play even before the primary consolidation ends and continues there after. The secondary consolidation is represented by a series of straight lines with different slopes one versus log t plot. It is of much significance is the case of highly organics soils, micaceous soils and some loosely deposited clays.

Referring the straight line representing secondary compression on e-log t plot may have equation of the following form :

$\Delta e = -C_\alpha \log_{10} \frac{t_2}{t_1}$

$\Delta e = e_1 - e_2$

where C_α = Coefficient of secondary compression.

(Time on log scale)

(Secondary Consolidation)

FOUNDATION

* A foundation is that part of the structure which is in direct contact with and transmits loads to the ground.

Footing:- A footing is a portion of the foundation of a structure that transmits loads directly to the soil.

foundation soil:- It is the upper part of the earth mass carrying the load of the structure.

Functions of foundation :-

Following are the main functions of foundations

1. To transmit & distribute the total load of the structure to a large area of underlying.
2. To prevent differential settlement of the structure.
3. To provide stability to the foundations.

Foundation
- Shallow foundation
  - Isolated footing
  - Combined footing
  - Strap footing
  - Raft or mat foundation
- Deep foundation
  - → pile foundation
  - → pier foundation
  - → well foundation

① Shallow foundation :-

→ In shallow foundation i.e also known as a stepped foundation.

→ If the depth of foundation is less than the width of foundation then it is known as shallow or stepped foundation.

⇒ It can be used where the bearing capacity of soil in which the structure is to ~~the~~ be constructed is maximum.

⇒ The depth of foundation = 80 cm to 1 m.

Types :-

Following are the types of shallow foundation.

(1) Isolated footing or column footing :-

This type of footing is used for an individual column. This isolated footing is further classified into three types. They are as follow :-

① stepped footing In this type of footing on a base foundation a step is raised, which is also known as pedestal. The step or pedestal is further followed by a column. This type of footing is generally used where a heavy load is coming from a superstructure.

② simple spread footing :-

In this type of footing, only a base foundation is constructed which is further followed by a column. This type of footing is used for a small structure where that type of heavy load is not coming from the structure as in case of stepped footing are concerned. there these type footings are used.

(3) sloped footing :- In this type of footing also only base foundation is constructed which is further followed by a column. But when we cut a section from the centre we can see that this footing is in the shape of a trapezoid.

(2) Wall footing

Wall footings are used to support structural walls that carry loads for other floors or to support nonstructural walls.

Wall footing for further classified in to two types
- simple wall footing
- stepped wall footing

(1) simple wall footing :-

In this type of footing, one base foundation, ~~two~~ more steps is constructed which is further followed with a wall. This type of footing is used for load-bearing structures but with less amount of structural load.

(2) stepped wall footing :-

In this type of footing on a base of foundation two more steps are constructed which are further followed with a wall. In this, the projections of the steps are taken to centimet- ers for either side. The width of the foundation has to be twice or more than that of the wall. This type of footing is used where the structural load is very heavy.

(3) Continuous footing :-

In this type of footing a single slab type footing is done when more than one column is in a row. This footing transfers load to a bigger area.

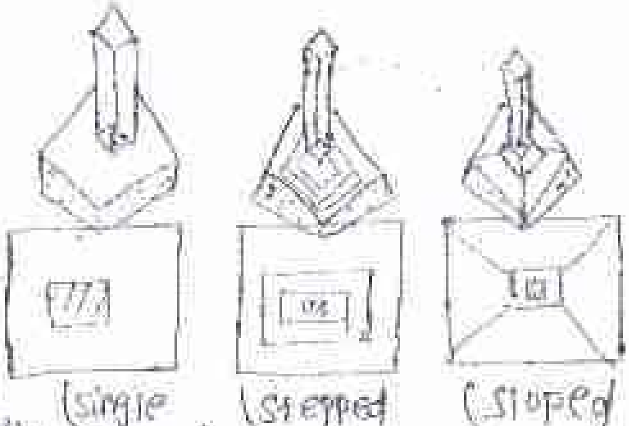
(4) Inverted arch footing :-

This type of footing is not used commonly. The inverted arch footing is used to satisfy the special condition. when the bearing capacity of the soil is very less in that condition we make use of this footing. Also if deep excavation is not possible then this type of footing is done.

(Inverted Arch footing)

(5) Spread footing :-

This is also spread foundation. In this type of footing, a base foundation is created which is an RCC member. Above which three steps are created which are done by brickwork. these three steps are ~~created~~ not RCC members. which are further followed by a wall ? in this type of footing. ground level is maintained above all the steps. The projection of the first step below the wall is (t + 150) mm here (t = thickness of wall). projection of the second step below the first step is (t + 200) mm followed by the third step it is (2t) mm. Lastly the projection between the third step and base foundation is (t + 450) m.m. The width of the base foundation is 2 (t + 100) mm.



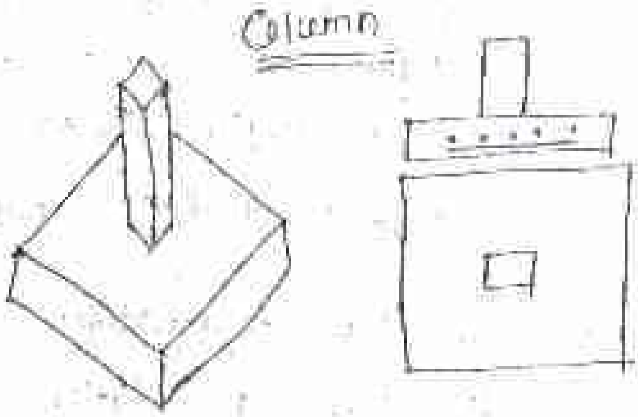
(single footing) (stepped footing) (sloped footing)

(SPREAD FOOTING)

(6) Raft footing or mat footing :-

This is also known as combined type footing or foundation. It covers the whole structure. It provides the stability and strength to the structural member like R.C.C wall and columns. Above the soil surface, a base is created of any thickness, it is just done to create a base for raft foundation. On that base, this raft foundation is constructed. When bearing capacity of a soil is less than, raft or mat footing is used.

(7) Combined footing :-



(Isolated simple spread footing)

When two or more than two columns come in a row then this type of footing is constructed. In this there are two types of footing they are.

① Rectangular shaped combined footing.

② Trapezoidal shaped combined footing.

The rectangularly shaped footing is done when the load coming from two or more columns is uniform or the same.

Trapezoidal shaped combined footing

Trapezoidal shape footing's are done when the load coming from two or more column is not equal or same.

(8) Strip footing or cantilever footing :-

In this, if two more columns are in a row, and these columns are interconnected by a beam. These types of footings are known as strap or cantilever footings.

Deep foundation :-

If the depth of footing greater or equal to the width of footing. It is known as the deep foundation. Deep foundation is used where the bearing capacity of the soil is very low. The load coming from the superstructure is further transmitted vertically to the there are Three major types of deep foundation, and their users in construction discussed below.

Types of deep foundation :-

1. Pile foundation
2. Pier foundation
3. Well foundation

(1) Pile foundation

In this type of foundation the load is transmitted by a vertical member. This vertical member is known as a pile. These piles are generally made of steel, concrete and wooden. These days precast members are used but we can create these members on site as well.

Classified of pile foundation

① According to foundation

② According to material

① According to foundation

It is subdivided into two types they are as follow

① Bearing piles

② Friction piles

Bearing piles

They are driven till hard strata or layer of Rock beds. The load is transmitted by columns to the hard layer of soil.

Friction piles

These piles are used where the soil is soft at a considerable depth. The load is transferred to the soft soil due to the friction produced between the soft soil which is in contact with these piles.

According to material

It is further divided into four types they are as follows

① Concrete pile

② Wooden pile or Timber pile

③ Steel pile

④ Composite pile

Concret pile

The piles which are made with the help of Concrete are known as concrete piles. The diameter of these pile varies from 30 to 50 cm. minimum length of these pile is not taken less than 20 meters and maximum it can be taken till 30 meters.

① precast (Ready Made) ② Cast insitu

These piles are manufactured in the factory, which is further transported to the construction site where ever it is required. These piles can bare load up to 800 kN.

Advantages of precast piles :-

① It saves our time as these piles are ready to install.
② By using these piles the construction is done at greater speed.
③ For these piles deep excavation is not required.

Disadvantages of precast piles :-

① The concrete piles are costly.
② As these precast members are prepared in a factory and then they are transported to the construction site the transportation charges are also added which increases the ultimate cost of these piles.

(2) Cast-in-situ :- These piles are made or manufactured on site where it is to be installed. So it saves money as the transportation cost is reduced. These piles bare load up to 750 kN.

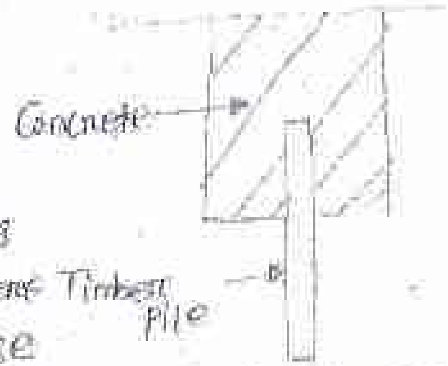
(2) Wooden or Timber piles :-

As the name suggests these piles are made up of wooden & they are known as wooden or Timber piles. For these piles seasonal Timber wood is used. The diameter of the timber pile varies in between 20 to 50cm. Length of a pipe is taken 20 times that of its diameter. for (Example - 25 cm is its diameter. Then, L = 20 x 25 = 500 cm). The maintenance cost of these piles is more because it is wood if it comes in contact with water then it can be damaged by fungus or white ants. So care has to be taken.

(3) Steel piles :- These piles are generally in shape of I or hollow section. It can be easily driven in the soil because it has a very small cross-sectional area. These piles can be used as a bearing pile but cannot be used as friction piles because if we use them as a friction pile it can sunk to the soil due to structural load.

4. Composite pile :-

When the piles are made from more than one material they are known as composite pile. These piles are made from concrete and wood. These piles are used in those areas where the water table is up. These piles are used in such conditions just because concrete and wood both are good water absorbers.



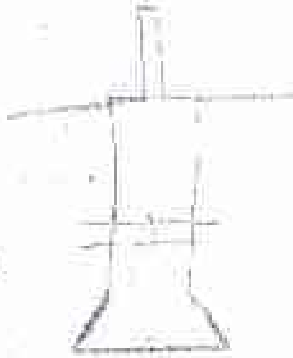
Advantages of pile foundation :-

- use of these piles can save time
- They are very much economical
- By using this pile system it reduces the needed excavation.

- pumping of water is not required as we are not excavating much in soil.

(2) Pier Foundation :-

A pier foundation is a vertical column of relatively larger crosssection than a pile. The load coming from the superstructure is carried to the hard strata through these vertical columns.



(Pier Foundation)

They are generally cast on site. A pier is installed in dry area by excavating a cylindrical hole. If the diameter is greater than 0.6 m or equal to 0.6 meters then it is termed as a pier.

Types of pier foundation

① masonry or concrete pier
② Drilled caissons

①. masonry or concrete pier

This type of footing is chosen when the depth of the hard strata is at 5 meters or less than 5 meter. Also, this type of footing is done when not much heavy load is coming from the super structure.

The masonry work is done by brick or concrete. The size of excavation depends upon the level at which hard strata exists. The size and shape of these masonry or concrete pier depend upon the level of hard strata is present.

(3) Drilled Caissons :-

They are mainly in a cylindrical shape so they are also known as cylindrical foundation. Construction for drilled caissons is generally carried out by drilling process. This foundation work as a compression member. The load acted on top of these members so we can say that they are subjected to axial load and which further transferred to hard layer of soil.

The masonry work is done by brick or concrete. The size of ~~excavation~~ depends upon ~~the level at which hard strata exists.~~ ~~The size and shape of these~~ masonry or ~~concrete pier depend upon the level of hard strata is present.~~

Drilled caissons are classified into three categories

(1) Concrete caissons with enlarged bottom
(2) Caissons of steel pipe with concrete filled.
(3) Caissons of steel pipe with concrete and steel core.

(1) In this at top which is at ground level of cap is provided. Above that cap brickwork is carried out. Below this cap a pier is ~~carried out~~ constructed. this is further following by the enlarged bottom which is also known as bell. The angle of this bell at bottom is 60 degree.

(2) In this also at the top which is at ground level a cap is provided. Below this cap, at both extreme ends, a steel shell is created. This steel shell is the outer portion. Inside this steel shell concrete is filled.

(3) The assembly of this type is also the same as the caissons of steel pipe with concrete - filled. But the only change is that in the central portion of the steel core as a rod is fixed which gives more stability to the structure as the weight taking capacity increased due to the steel core.

* Types of Bearing capacity failures :-

> When a footing fails due to insufficient bearing capacity, failures patterns are developed, depending upon type of failure

> There are three types of failure :-
(a) General shear failure
(b) Local shear failure
(c) punching shear failure

(a) General shear failure :-

* In case of general shear failure, continuous failure surfaces develop between the edges of the footing & the ground surface.

(General shear failure)

* When the pressure the value of ultimate bearing capacity the state of plastic equilibrium is initially in the soil around the edges of the & it generally

* The failure is of failure surfaces & by considerable

* Such a failure occurs for soil in low compressibility
i.e on soil & the pressure settlement
curve is of the general form.

* Following are the

(i) It has defined failure surfaces up to ground surface.

(ii) Failure is by of the footing.

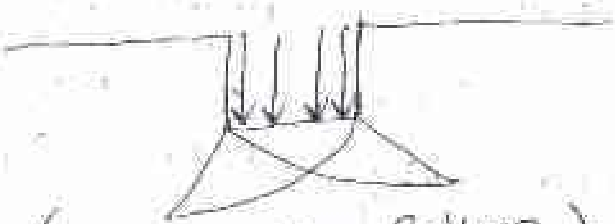
(iii) Failure is sudden

(iv) Ultimate bearing capacity is well defined.

(b) Local shear failure :-

In local shear failure, there is compression of the soil the footing & only development of of plastic equilibrium

* Due to this reason, the failure surfaces do not the ground surface & only



(Local shear failure)

Local shear failure is with soils of high compressibility and in sands relative density lying.

* Following are typical characteristics of local shear failure :-

(i) Failure is defined

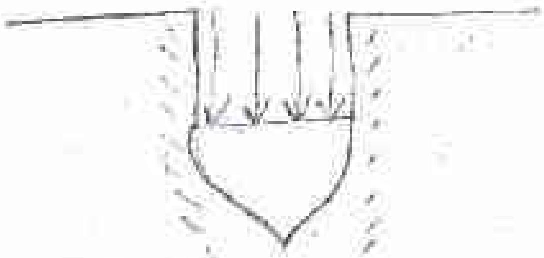
(ii) Failure surfaces is ground surface.

(iii) There is physically soil around of footing.

(iv) failure is not & there is no of footing.
(v) failure defined by settlements.
(vi) ultimate bearing capacity is not well defined.

(c) Punching shear failure :-

It where there is relatively high compression of soil under the footing, in the vertical direction around the edges of the footing.



(punching shear failure)

* shear may occure in relatively base with density less than 5%.

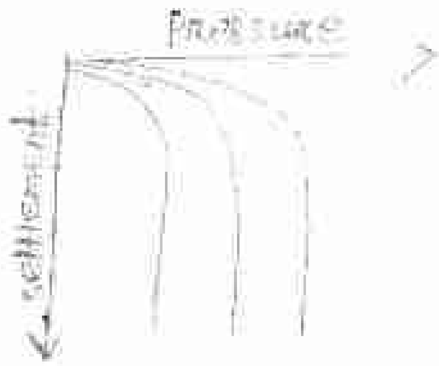
* shear failure may also is in relatively low compressibility, if the foundation is at depth.

* Following are the characteristics of shear failure. is observed.

(i) No failure

(ii) The failure surface, which is vertical or inclined

of the base.



Bearing capacity -

-> The of a soil or rock is prefere to as its bearing capacity.

Gross pressure intensity (q)

The gross pressure intensity 'q' is the total pressure at the base of the footing due to the weight of superstructure, self weight of the footing & the weight of the earth fill.

Net pressure intensity (q_n)

It is defined as the gross pressure on the difference in intensity of the gross pressure after the construction of the structure & the original overburden pressure.

$$\boxed{\therefore q_n = q - \sigma' = q - \gamma D}$$

where q = Gross pressure intensity
D = Depth of footing
γ = average unit weight of soil above foundation base.

Ultimate bearing capacity (q_f)

It is defined as the minm gross pressure intensity at the base of the foundation at which the soil fails in shear.

Net ultimate bearing capacity (q_{nf})

It is the minm net pressure intensity causing shear failure of soil

$$\boxed{q_{nf} = q_f - \bar{\sigma}}$$

where $\bar{\sigma}$ = effective at the base level of foundation.

q_f = ultimate bearing capacity

Net safe bearing capacity (q_{ns}) :-

The net safe bearing capacity is the net ultimate bearing capacity divided by a factor of safety.

$$\boxed{q_{ns} = \frac{q_{nf}}{F}}$$

where F = factor of safety

q_{nf} = net ultimate bearing capacity.

Safe bearing capacity (q_s) :-

The maxm pressure which the soil can carry safety without risk of shear failure is called the surface bearing capacity.

$$\boxed{q_s = q_{ns} + \gamma D = \frac{q_{nf}}{F} + \gamma D}$$

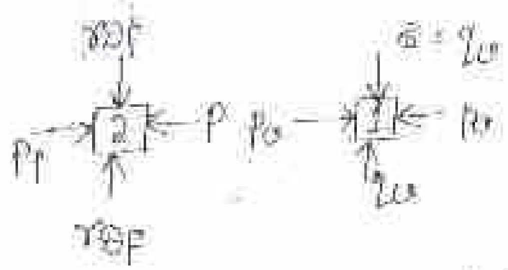
(or)

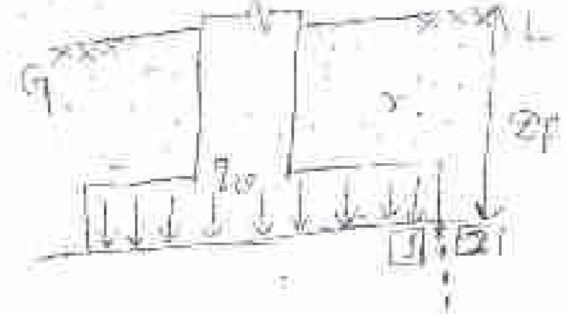
$$\boxed{q_s = \frac{q_f}{F}}$$

28 Dec 2023

Rankine Bearing capacity Equation :-

⇒ It is given for cohesionless soil.





where p_a = active earth pressure

p_p = passive earth pressure

γ = unit wt of soil

D_f = depth of foundation

$(\sigma_n)_2 = (\sigma_n)_1$

$$\Rightarrow \boxed{q_u = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2 \gamma D_f} \ kN/m^2$$

where = ϕ :- angle of internal friction.

(or)

$$\boxed{D_f = \frac{q_u}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2}$$

where q_u = ultimate bearing capacity of soil.

Terzaghi Bearing capacity Equation :-

$$\boxed{q_u = cN_c + \bar{\sigma} N_q + \frac{1}{2} B\gamma N_\gamma}$$

where q_u = ultimate bearing capacity

c = cohesion (kN/m^2)

$\bar{\sigma}$ = Effective overburden pressure (kN/m²) → $\bar{\sigma} = \gamma D_f$ (No water table)

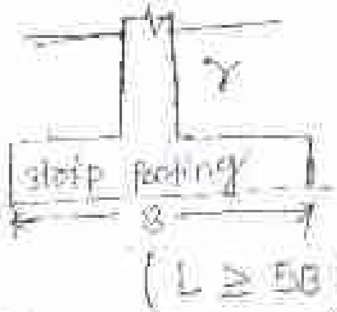
→ $\bar{\sigma} = \gamma_{sub} D_f$ (fully submerged)

B = Width of the footing

γ = unit wt. of soil (Below the base of footing)

↓

(3rd term)



γ = unit wt. of soil (above the base of foundation)

C = C is the cohesion of soil below the base of footing

N_c, N_q, N_γ = bearing capacity factor (According to Terzaghi Analysis these factors depend upon the angle of internal friction of the soil)

	Terzaghi	prandtl's
clayey soil ($\phi = 0°$):-	$N_c = 5.7$	$N_c = 5.14$
	$N_q = 1$	$N_q = 1$
	$N_\gamma = 0$	$N_\gamma = 0$

Assumption :-

(i) Terzaghi equation is given for all types of soil.

(ii) Terzaghi has considered general shear failure. [$\phi > 36°$, $I_D > 70\%$]

(iii) Terzaghi equation is given for strip footing [$L \geq 5B$]

Modified Terzaghi equation for other shape of footing :-

Case-I Strip footing

$\rightarrow q_u = cN_c + \bar{\sigma} N_q + \frac{1}{2} B\gamma N_\gamma$ (G.S.F)

$\rightarrow q_u = \frac{2}{3} cN_c' + \bar{\sigma} N_q' + \frac{1}{2} B\gamma N_\gamma'$ (L.S.F)

where N_c', N_q', N_γ' = bearing factor in local shear failure (L.S.F) depend on angle of internal friction of L.S.F .

$$\boxed{q_L = \tan^{-1}\left[\frac{2}{3}\tan\phi\right]}$$

where ϕ = angle of internal friction at L.S.F

ϕ = angle of internal friction at G.S.F

Case-II :- Rectangular footing (G.S.F) :-

$$\boxed{q_u = \left(1+0.3\frac{B}{L}\right)cN_c + \bar{\sigma} N_q + \frac{1}{2}\left(1-0.2\frac{B}{L}\right)B\gamma N_\gamma}$$

where B = width of footing

L = Length of footing

shape factor :- $\left(1+0.3\frac{B}{L}\right)$ &

$\left(1-0.2\frac{B}{L}\right)$.

Case-III :- Square footing :- (B = L)

$$\boxed{q_u = 1.3cN_c + \bar{\sigma} N_q + 0.4B\gamma N_\gamma}$$

29 Dec 2020

10 A rectangular footing (2m x 3m) rests on a C - φ soil with it's base at 1.5 mt - below the ground surface calculate the safe bearing capacity, using a factor of safety 3 on (i) Net ultimate bearing capacity

(ii) ultimate bearing capacity.

The following parameters of soil are γ = 18kN/m.

$c = 10 kN/m^2$; $\phi = 30^0$; $N_c = 37.2$

$N_q = 22.5$

$N_\gamma = 19.7$.

(use Terzaghi Analysis)

Sol^n

$$q_u = \left[\left(1+0.3\times\frac{2}{3}\right)10\times37.2\right] + (27\times22.5) +$$

$$\left[\frac{1}{2}\times\left(1-0.2\times\frac{2}{3}\right) \times2\times18\times19.7\right]$$

$= 1361.2 \ kN/m^2$

$q_{nu} = q_u - \gamma D = 1361.2 - 18\times1.5$

net ultimate bearing capacity

$= 1361.2 - 27$

$= 1334.2 \ kN/m$

11 Jan 2021

Q. Determine the depth at which a circular footing of 2m diameter be founded to provide a factor of safety of 3, if it has to carry a safe load of 1600 kN. The foundation soil has c = 10 kN/m², $\phi = 30°$ & $\gamma = 18$ kN/m³. Use Terzaghi analysis.

$N_c = 37.2$

$N_q = 22.5$

$N_\gamma = 19.7$

Solⁿ For circular footing

$q_u = 1.3 c N_c + \bar{\sigma} N_q + 0.3 B \gamma N_\gamma$ $\quad \left[\bar{\sigma} = \gamma D_f\right]$

given data :- N_c, N_q, N_γ

$\phi = 30°$

$c = 10$ kN/m²

footing diameter = 2m.

F.O.S = 3

safe load = 1600 kN.

$D_f = ?$

$$q_{us} = \frac{\text{load}}{\text{Area}} = \frac{1600}{\frac{\pi}{4} \times 2^2} = 509.29 \simeq 509.30 \text{ kN/m}^2$$

$q_u = q_s$

$\Rightarrow 1.3 c N_c + \gamma D_f N_q + 0.3 B \gamma N_\gamma = q_s$

$\Rightarrow (1.3 \times 10 \times 37.2) + (18 \times D_f \times 22.5) + (0.3 \times 2 \times 18 \times 19.7)$

$= 509.30$

$\Rightarrow D_f = 0.4$

5 June 2021

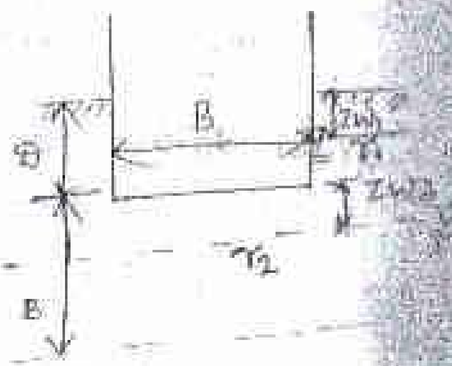
Effect of watertable on bearing capacity of soil :-

→ When the water table is above the footing, the submerged weight "γ" should be used for the soil below the water table for computing the effective surcharge.

1st method :-

$$\boxed{q_f = CN_c + \gamma_1 D N_q R_{w1} + \frac{1}{2} B \gamma N_\gamma R_{w2}}$$

R_{w1}, R_{w2} = Reduction factor for water table



$$R_{w1} = 0.5 \left(1 + \frac{Z_{w1}}{D}\right)$$

$$R_{w2} = 0.5 \left(1 + \frac{Z_{w2}}{B}\right)$$

where : D -: Depth of footing

B -: Breadth of footing

2nd Method :-

$$\boxed{q_f = CN_c + \bar{\sigma} N_q + \frac{1}{2} \gamma B N_\gamma R_w}$$

where $R_w = R_{w2} = 0.5 \left(1 + \frac{Z_{w2}}{B}\right)$.

3rd method :-

$$\boxed{q_f = CN_c + \bar{\sigma} N_q + \frac{1}{2} \gamma_e B N_\gamma}$$

where γ_e = effective unit weight of soil in wedge zone.

7th June 2021 New chapter

Shear strength of soil

Shear strength

The shear strength is ~~attributes~~ by the maximum shearing resistance that is mobilised on the potential failure plane & is equal to the ultimate shear stress

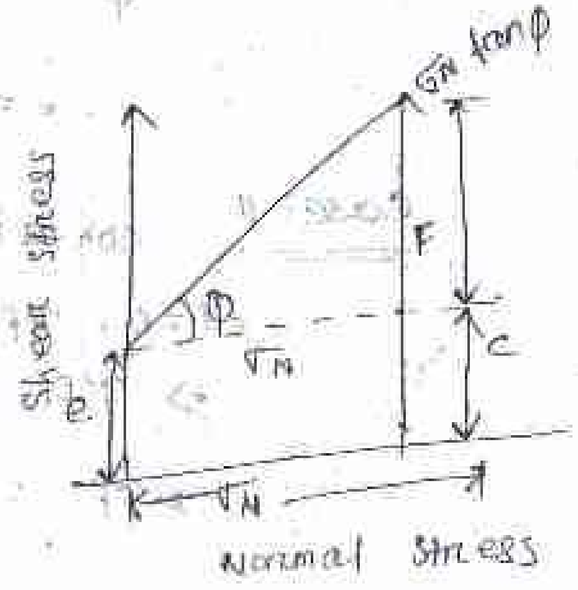
> The shear strength is attributed by

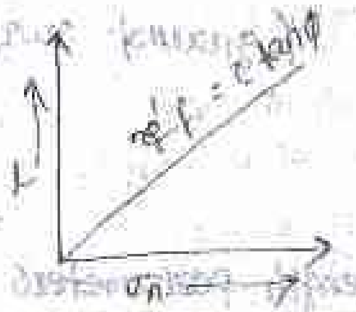
- Interlocking of particles
- Cohesion & adhesion of particle
- Frictional resistance.

Mohr columb

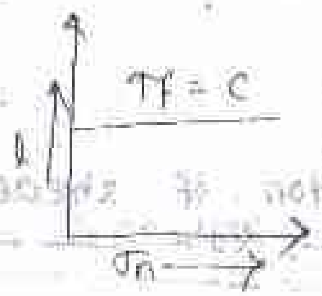
$$\boxed{\tau_f = c + \sigma_N \tan\phi}$$

> This case is for $c - \phi$ soil





Cohesionless soil ($c = 0$)



pure cohesive soil ($\phi = 0$)

where τ_f = shear stress at failure

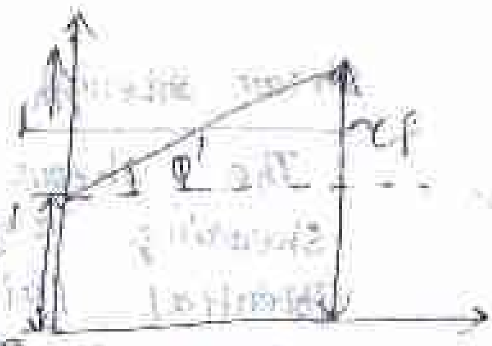
ϕ = Angle of internal friction

σ_N = normal stress

c = cohesion

Modified Mohr Coulomb Theory :-

$$\boxed{\tau_f = c' + \sigma' \tan \phi'}$$



where c' & ϕ' = effective cohesion & effective angle of shearing resistance

σ' = effective stress

$= (\sigma - u)$

Limitations :-

Case - I :- pure cohesive soil (clay)

$\phi = 0$, $\tau_f = c' + \sigma' \cancel{\tan\phi}$

$= \tau_f = c'$

Case - II cohesionless soil

$c' = 0$, $\tau_f = \cancel{c'} + \sigma' \tan\phi$

$\Rightarrow \tau_f = \sigma' \tan\phi'$

$\tau_f = \cancel{\sigma'} \tan\phi'$ (ground surface)

9 June 2021

Determination of shear strength parameters :-

Load Application

(i) Direct shear stress test
(ii) Triaxial compression test
(iii) Unconfined Compression test

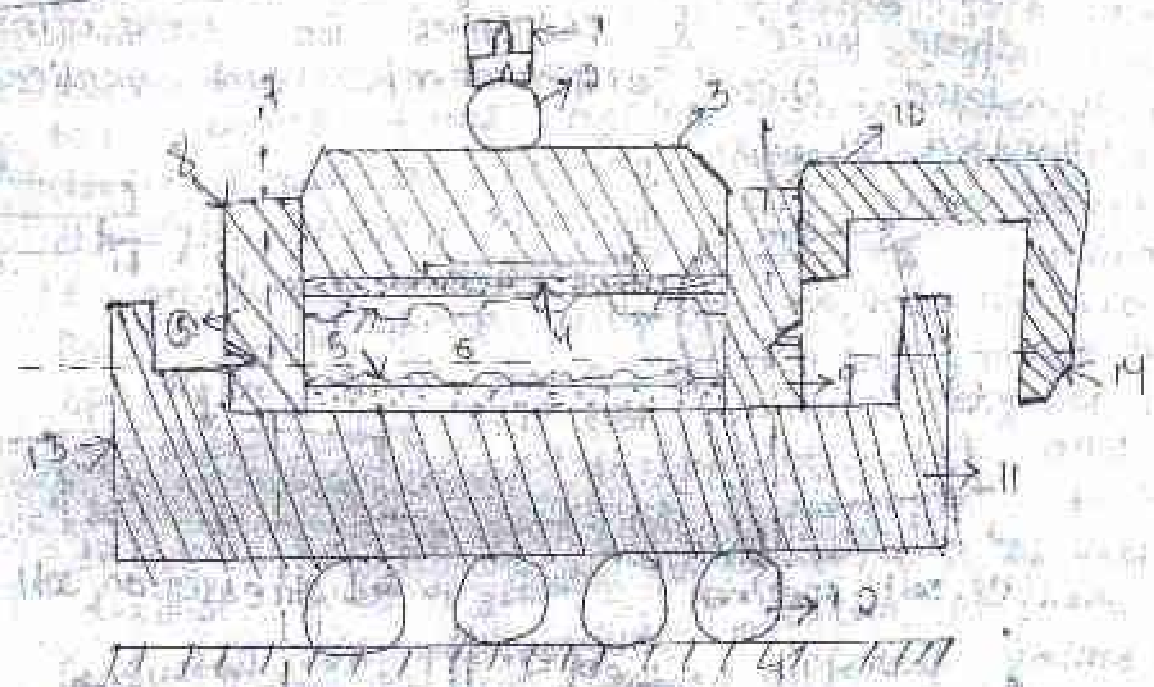
Drainage Application

(i) Undrained test (UU test)
(ii) Consolidated undrained test (CU)
(iii) Consolidated drained test (CD)

(4) vane shear test

18 June 2024

Direct shear test



1 → Loading yoke

2 → steel ball

3 → Loading pad

4 → porous stone

5 → metal grid

6 → soil specimen

7 → Pins to the two valves

8 → upper part of shear box

9 → Lower part of shear box

10 → U arm

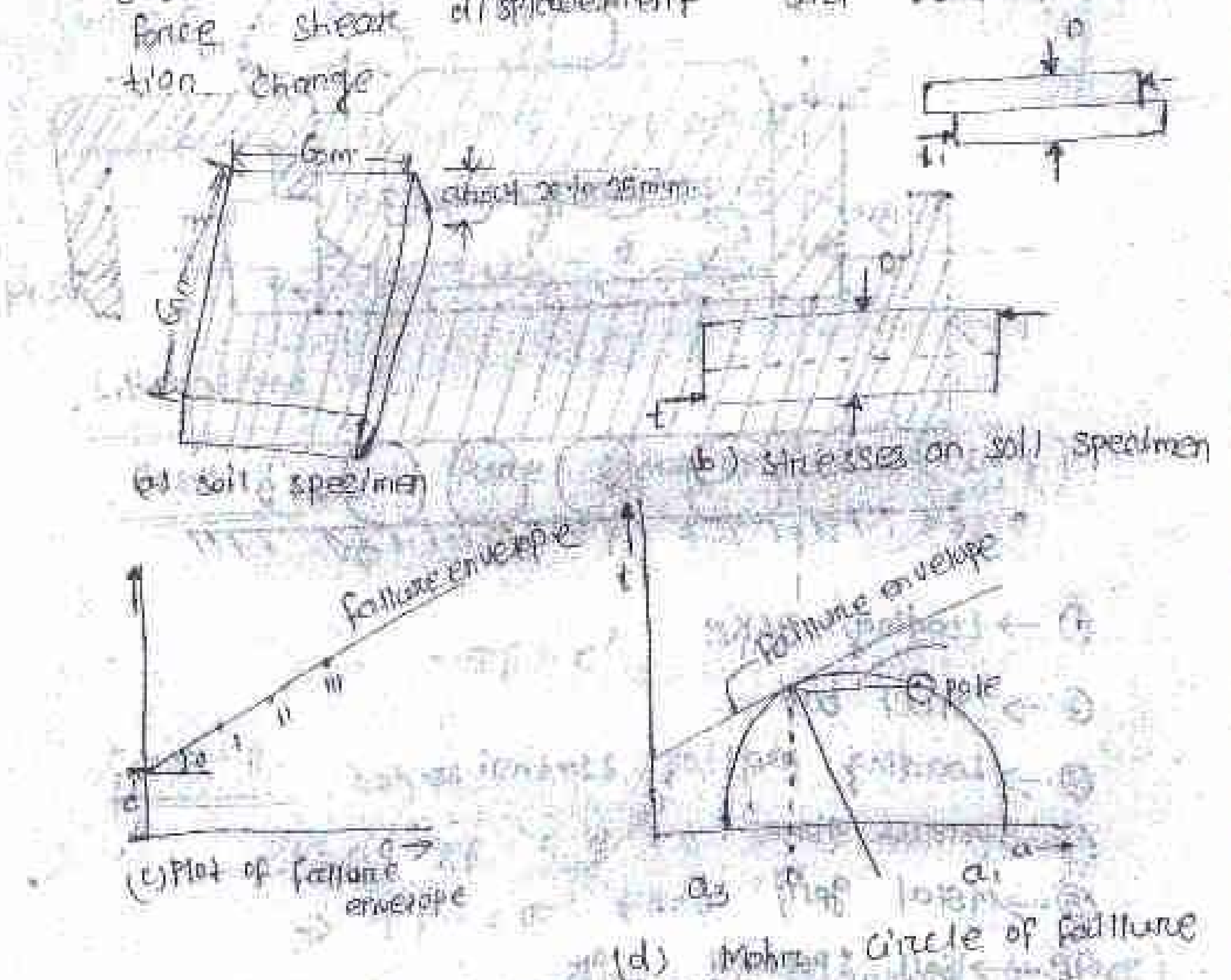
11 → Container for shear box

12 → Roller

13 → shear force

14 → shear resistance

The soil specimen used in the test is usually square in plane of size 60 m.m. x 60 m.m. and thickness about 20 to 25 m.m. The direct shear test equipment essentially consists of (i) shear box, (ii) loading yoke for applying normal force, (iii) geared jack for applying shear force & facilities for measuring shear force, shear displacement and vertical deformation change.



The shear box consists of two halves, the lower half is in contact with the shear box container which freely slides on rollers and to which the shear force is applied by means of geared jack. The soil specimen is placed in the shear box such that it gets sheared on a horizontal plane exactly at its mid height. The specimen is sandwiched between a pair of metal grid plates and a pair of porous plates (or non-porous plates). The grid plates provided with serrations are placed which serrations at right angles to direction of shearing to provide grip on the specimen. For conducting drained test perforated grid plates and porous stones are used.

A Normal stress σ is applied on the specimen and is kept constant throughout the test. The shear stress is caused by application of shear force through geared jack and is transmitted to the top half of the shear box which bears against shear force measuring device (such as proving ring dial gauge). The shear stress through the soil specimen is gradually increased until the specimen fails and there will be no transmission of shear force from lower half to top half of shear box. If test continues beyond 20% strain is usual to stop the test and define failure point as corresponding to any desired level of strain up to 20%. The test is conducted on preferable minimum of three specimen subjected to three different values of σ. By plotting the failure envelope is obtained c and φ are obtained by measurement from the plot.

The shear box test can be either strain controlled or stress controlled. In the strain controlled shear box test the shear strain is made to increase at a constant rate and the shear stress is measured. In the stress controlled shear box test the arrangement is for increasing the shear stress at constant rate and measuring the shear strain.

Advantages of direct shear test :-

1) The direct shear stress test is a simple test compared to the triaxial compression test.

2) Since the thickness of the sample is small quick drainage and hence rapid dissipation of pore pressure is possible,

Disadvantages of direct shear test

1. The shear stress is not uniformly distributed being more at the edge than at the Center. Because of this the entire shear strength is not mobilized simultaneously at all points on the failure plane and this leads to progressive failure of the specimen.
2. The failure plane is predetermined. Therefore the specimen is not allowed to fail along its weakest plane.
3. Shear displacement causes reduction in area under shear. Corrected area should be used in computing normal and shear stress.
4. The side walls of the shear box can cause lateral restraint on the edges of the specimen.
5. There is little control on drainage of pore water as compared with triaxial compression test.
6. Measurement of pore pressure is not possible.

Triaxial Compression Test

The triaxial compression test was introduced by Casagrande and Terzaghi in 1936 and to this day is the most extensively used type of shear test. As the name indicates, in this test the specimen is stressed by applying all the three principal stress.

The soil specimen used in the test is cylindrical in shape with length 2 to 2.5 times the diameter. The triaxial compression test equipment essentially consists of (i) triaxial cell, (ii) loading frame with accessories for applying gradually increasing axial load on specimen at constant rate of strain (iii) provision for measuring axial force and axial displacement, (iv) constant pressure system to apply and maintain constant cell pressure (v) pore pressure measuring apparatus and (vi) volume change gauge.

The triaxial cell consists of a high pressure cylindrical cell, made of a transparent material like perspex, fitted between base and top cap and is provided at the base with inlet for cell fluid, outlets for drainage of pore water from specimen and measurement of pore pressure. At the top an air release valve to expel air from the cell and a steel plunger for applying axial force on specimen are provided.

The soil specimen is kept inside the triaxial cell with porous plates (or non porous plates for undrained test) at top and bottom. The loading cap is placed on top porous plate. The specimen is enclosed in a rubber membrane to prevent its contact with the cell fluid. After filling the cell with fluid (usually water) required cell pressure (σ_3) is applied by means of constant pressure system. The additional axial force called the deviator force is applied through the plunger and the deviator force corresponding to different axial deformation at regular intervals are noted. The test is continued until the specimen fails. If the test continues even after 20% strain and failure point it may be stopped and defined at desired strain level upto 20%. The deviator stress (σ_d only) stage of the test is given by:

$$\sigma_1 = \frac{F}{A}$$

where F = deviator force i.e. additional axial force applied through plunger.

A_f = corrected area of cross section of specimen at that stage

If A_1 = initial area of cross section of specimen.

L_1 = initial length of specimen

A_f = corrected area of specimen when the axial compression is ΔL and change in volume is ΔV.

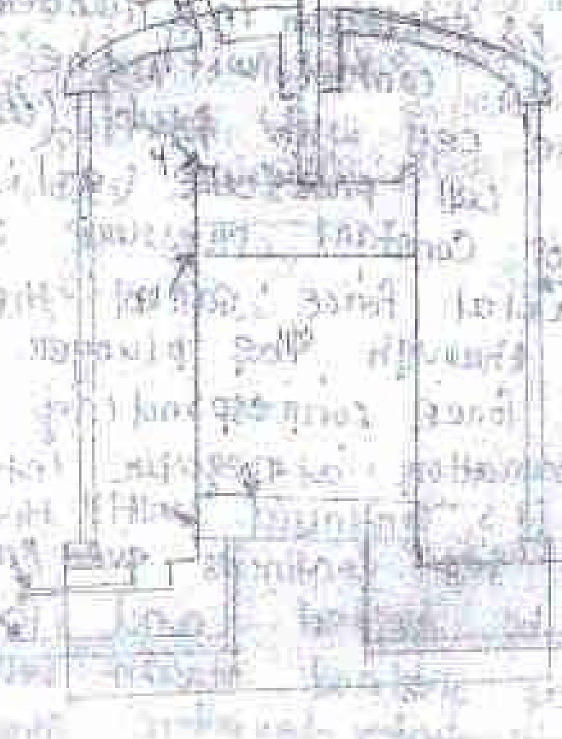
we have, initial volume $V_1 = A_1 L_1$ and volume at any stage of compression $(V_1 + \Delta V) = A_1 (L - \Delta L)$

$$A = \frac{V_1 + \Delta V}{L_1 - \Delta L}$$

In the case of undrained test on saturated soil sample $\Delta V = 0$ and

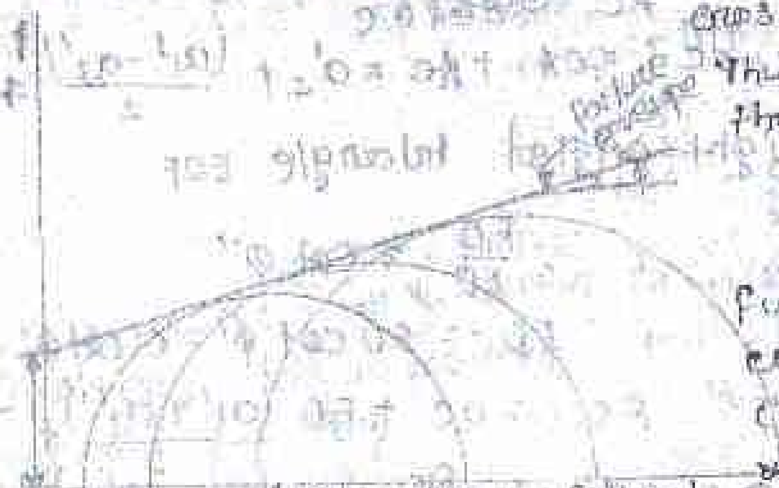
$$A = \frac{A_1 L_1}{L_1 - \Delta L} = \frac{A_1}{1 - \frac{\Delta L}{L_1}} = \frac{A_1}{1-\epsilon}$$

where ϵ is the axial strain at that stage.



③ Axial load
④ Loading ram
⑤ Air release valve
⑥ Top cap
⑤ perspex cylinder
⑥ sealing ring
⑦ pore water outlet
⑧ additional pore water outlet
⑨ cell fluid index
⑩ soil specimen
⑤ porous disc

After finding deviator stress σ_d at failure, we have major principal stress at failure $\sigma_1 = (\sigma_d + \sigma_3)$ with this set of $(\sigma_1 - \sigma_3)$ values mohr circle at failure is drawn. The test is conducted on preferably a minimum of three specimens subjected to different values of cell pressure σ_3. The mohr circle at failure is drawn for each specimen and the common tangent touching all the circles will be failure envelope and c are read out from the plot. for the benefit of readers it is repeated that deviator stress is the additional axial stress applied on the specimen through the plunger. This is because the cell pressure σ_3 not only acts on the sides of the specimen but also acts on top of the specimen and there will be equal reaction at the base. This is due to the fact that area of plunger is smaller than area of cross section of specimen. Thus σ_3 acts all round the specimen

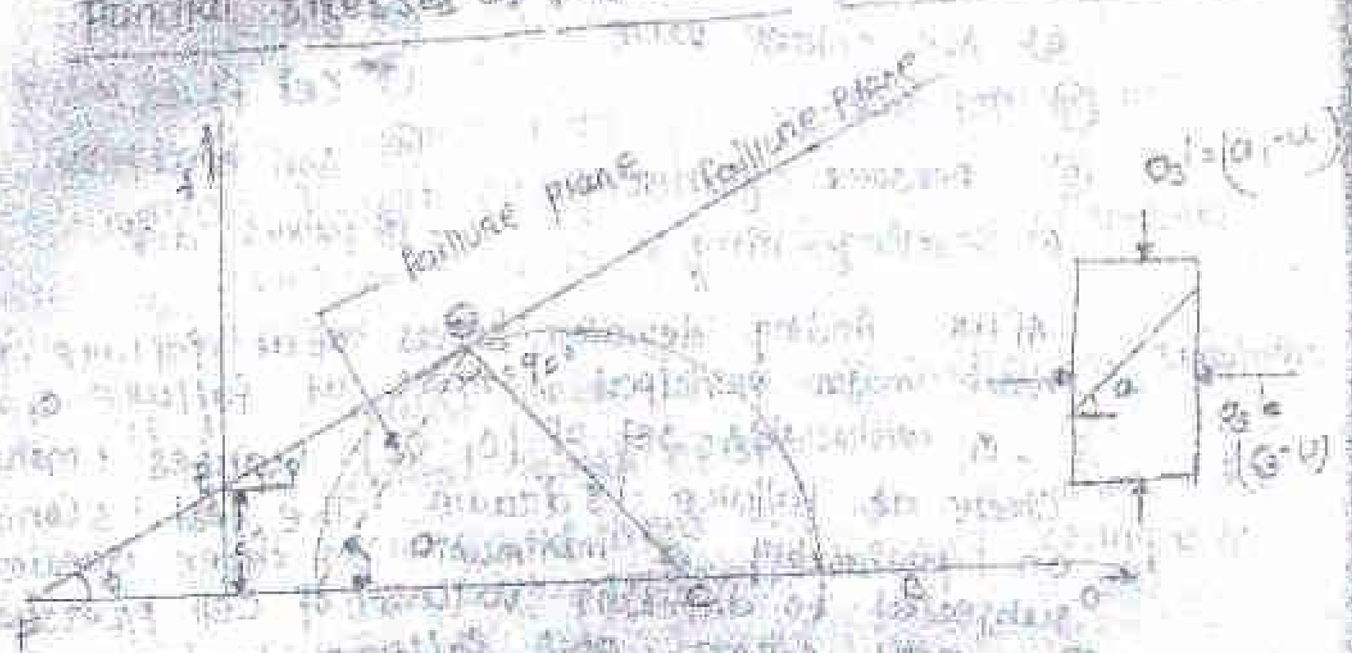


(Mohr circles at failure)

since $\sigma_1 = (\sigma_d + \sigma_3)$
we have $\sigma_d = (\sigma_1 - \sigma_3)$
Further, In the triaxial compression test on cylindrical soil specimen we have $\sigma_2 = \sigma_3$. specimen developed, to test soil

we here triaxial cell has been cubical in shape, with separate specimen chambers for applying σ_2 and σ_3 separately.

Relation between shear strength parameters and Principal stresses at failure



Mohr circle at failure is drawn for specimen subjected to triaxial compression test. point A is the pole of mohr circle. AD the failure plane and $\angle DAC = \alpha$ the angle made by failure plane with the major principal plane. Let the failure envelope be produced backwards to intersect σ-axis at F.

In the right angled triangle DFC

$$\sin\phi = \frac{DC}{FC}$$

but DC = radius of mohr circle $= \frac{(\sigma_1' - \sigma_3')}{2}$

$$FC = FO + OC$$

$$OC = OA + AC = \sigma_3' + \frac{(\sigma_1' - \sigma_3')}{2} = \frac{(\sigma_1' + \sigma_3')}{2}$$

In the right angled triangle EOF

$$\frac{FO}{EO} = \cot\phi'$$

$$FO = EO\cot\phi' = c'\cot\phi'$$

$$FC = OC + FO = \frac{(\sigma_1' + \sigma_3')}{2} + c'\cot\phi'$$

Hence

$$\sin\phi' = \frac{DC}{FC} = \frac{(\sigma_1' - \sigma_3')/2}{(\sigma_1' + \sigma_3')/2 + c'\cot\phi'}$$

Simplifying

$$\frac{(\sigma_1' - \sigma_3')}{2} = \frac{(\sigma_1' + \sigma_3')}{2}\sin\phi' + c'\cos\phi'$$

Further, we can write

$$(\sigma_1' - \sigma_3') = (\sigma_1' + \sigma_3')\sin\phi' + 2c'\cos\phi'$$

$$\sigma_1'(1-\sin\phi') = \sigma_3'(1+\sin\phi') + 2c'\cos\phi'$$

$$\sigma_1' = \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)\sigma_3' + 2c'\left(\frac{\cos\phi'}{1-\sin\phi'}\right)$$

It can be proved by trigonometry that

$$\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right) = \tan^2\alpha \text{ and } \left(\frac{\cos\phi'}{1-\sin\phi'}\right) = \tan\alpha$$

where $\alpha = 45° + \frac{\phi'}{2}$

Finally we get the following relation

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

$$= \sigma_3' N_\phi + 2c'\sqrt{N_\phi}$$

$$N_\phi = \tan^2\alpha$$

$$\alpha = 45° + \frac{\phi'}{2}$$

Not 2: we can show that

Referring right angled triangle CDF, $\angle DCF = (90 - \phi')$

$\angle CAD = \angle CDA = \alpha$

In Δ^t CAD $\angle CAD + \angle CDA + \angle DCA = 180°$

$\angle CAD + \angle CDA + (90 - \phi') = 180°$

$$2\alpha = 90° + \phi'$$

$$\alpha = 45° + \frac{\phi'}{2}$$

Is found very useful for analytical solution of problems. In undrained test, pore pressure is measured by means of pore pressure measuring apparatus σ_1' and σ_3' are given by

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

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

In drained test $u = 0$ so that $\sigma_1' = \sigma_1$ and $\sigma_3' = \sigma_3$

Modified failure envelope:

we have the following relation between shear strength parameters and principal stresses at failure

$$\frac{(\sigma_1' - \sigma_3')}{2} = \frac{(\sigma_1' + \sigma_3')}{2} \sin\phi' + c' \cos\phi'$$

we rewrite the above equation as

$$\frac{(\sigma_1' - \sigma_3')}{2} = \left(\frac{\sigma_1' + \sigma_3'}{2}\right) \tan\psi + d$$

where $d = c' \cos\phi'$ and $\tan\psi = \sin\phi'$

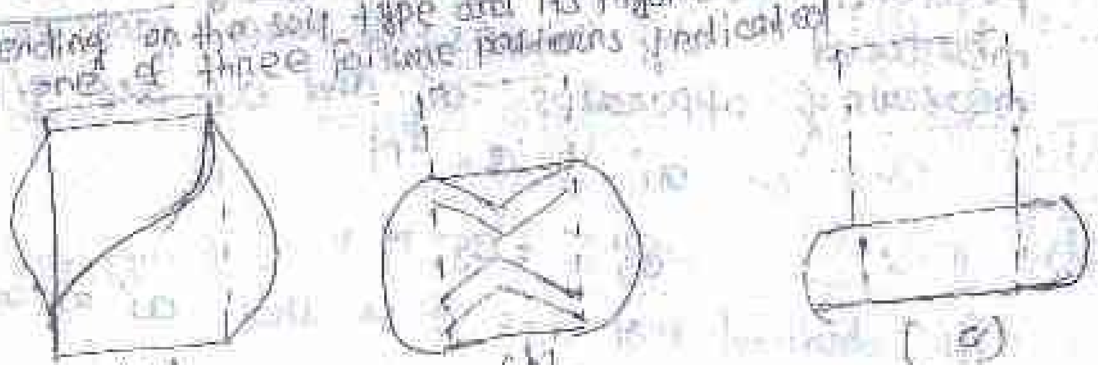
If we plot $\frac{(\sigma_1' - \sigma_3')}{2}$ against $\frac{(\sigma_1' + \sigma_3')}{2}$ for different sets of observation $(\sigma_1' - \sigma_3')$ we get a test of plotted points. The best fitting straight line through the points will be represented by and is called the modified failure envelope. The intercept of on the vertical axis and angle are measured and the shear strength parameters computed from the relations $\phi' = \sin^{-1}(\tan\psi)$

$$c' = \frac{d}{\cos\phi'}$$

This modified procedure was introduced by Lambe and Whitman (1969) and provides a means for averaging scattered data when tests are conducted ~~from the relations~~ on a large number of samples with wide range of cell pressures.

Types of failure of soil specimens in the triaxial compression test

Depending on the soil type and its physical properties a soil specimen can exhibit one of three failure patterns indicated



(a) (b) (c)

(i) Part (a) is an example of a brittle failure with a well defined failure plane and little lateral bulging.

(ii) (b) is shown a semi-plastic failure with shear cones and some lateral bulging.

(iii) (c) is a typical plastic failure with excessive lateral bulging and absence of failure plane. If this type of failure is noticed, it is usual to stop the test when the strain exceeds 20% and the failure point is defined corresponding to a strain level upto 20% depending on practical consideration.

Advantages of triaxial compression test

The advantages of triaxial compression test particularly when compared with direct test are outlined below.

(i) The specimen is free to fail along the weakest plane unlike in the direct shear test in which the specimen is forced to fail along a predetermined plane.

(ii) The stress distribution on the failure plane is uniform. The shear strength is mobilized uniformly at all points on the failure plane unlike in the direct shear test in which progressive failure takes place.

(iii) There is complete control of drainage condition. This enables better simulation of field drainage conditions during the test as compared with the direct shear test.

(iv) The precise measurements of pore pressure and volume change are possible during the test.

(v) The stress measurements involved on any plane within the specimen at any stage of the test can be determined.

(vi) As failure usually occurs near the middle of sample the effect of end restraint is not a serious disadvantage.

Types of shear tests based on drainage conditions

The shear strength parameters in the case of saturated soils depend very much upon the drainage conditions and their fore in the laboratory shear test the drainage condition expected to the field for a particular problems should be simulated based on drainage conditions the shear tests are classified as

1. Unconsolidated undrained or simply undrained test (UU Test)
2. Consolidated undrained test (CU Test)
3. Consolidated drained test or simply drained test (CD Test)

1. Undrained test

Drainage is not permitted throughout the test in the case of direct shear test drainage is not permitted during the application of both normal stress and shear stress in the case of triaxial compression test drainage is not permitted during the application of both cell pressure and deviator stress. Since the test is conducted fast allowing no time for either consolidation of sample initially or dissipation of pore pressure in later stage. The test is also called quick test.

(ii) Consolidated undrained

In this type of shear test the soil specimen is allowed to consolidate fully under initially applied stress and then sheared quickly without allowing dissipation of pore pressure in the case of direct shear test the specimen is allowed to consolidate fully under applied normal stress and then sheared at high rate of strain to prevent dissipation of pore pressure during shearing in the case of triaxial compression test the specimen is allowed to consolidate fully under applied cell pressure and then the pore water outlet is closed and the specimen subjected to increasing deviator stress at high rate of strain.

(iii) Drained test

In this type of shear test drainage is allowed throughout the test. The specimen is allowed to consolidate fully under the applied initial stress and then sheared at low rate of strain giving sufficient time for the pore water to drain out at all stages. The test may continue for several hours to several days.

Unconfined compression test

Unconfined compression test can be regarded as a special case of triaxial compression test in which no lateral pressure or confining pressure is applied so that $\sigma_2 = \sigma_3 = 0$. The soil specimen is cylindrical in shape with length about 2 to 2.5 times its diameter. The laboratory equipment for conducting unconfined compression test has facilities for compressing the specimen at uniform rate of strain and measuring the axial deformation and corresponding axial compressive force. The maximum compressive stress resisted by specimen before failure is called unconfined compressive strength. It is denoted by q_u and computed as shown below:

$$q_u = \frac{F}{A_f}$$

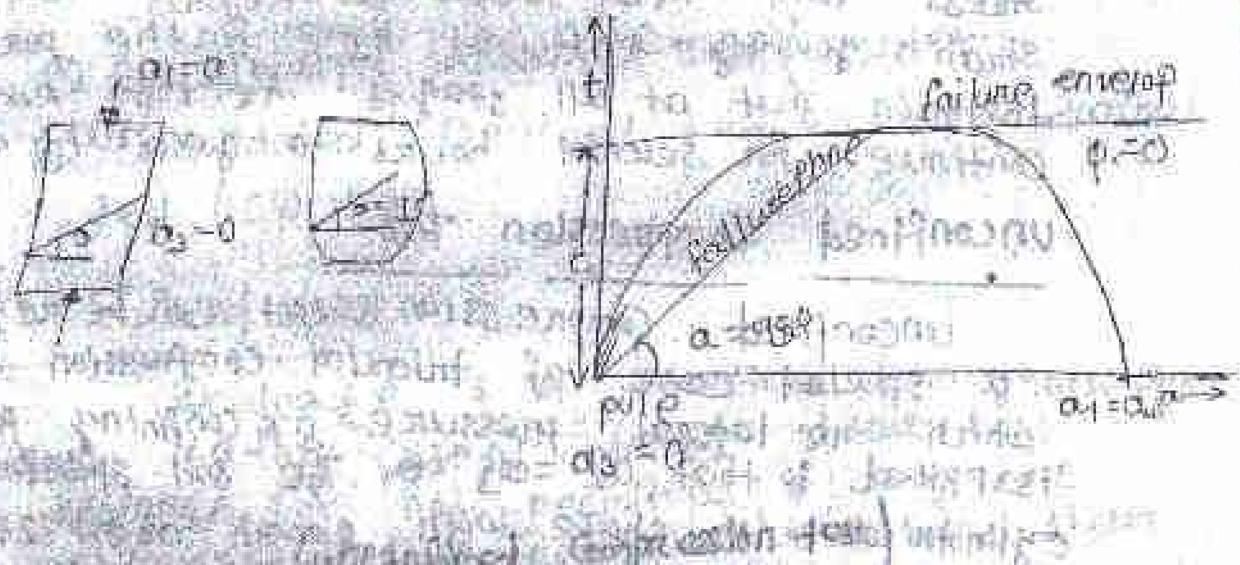
F = axial compressive force at failure

A_f = corrected area of cross section of specimen at failure $= \frac{A_0}{1-\epsilon}$

A_0 = initial area of cross section of specimen.

$\epsilon = \frac{\Delta l}{l}$ = axial strain at failure point

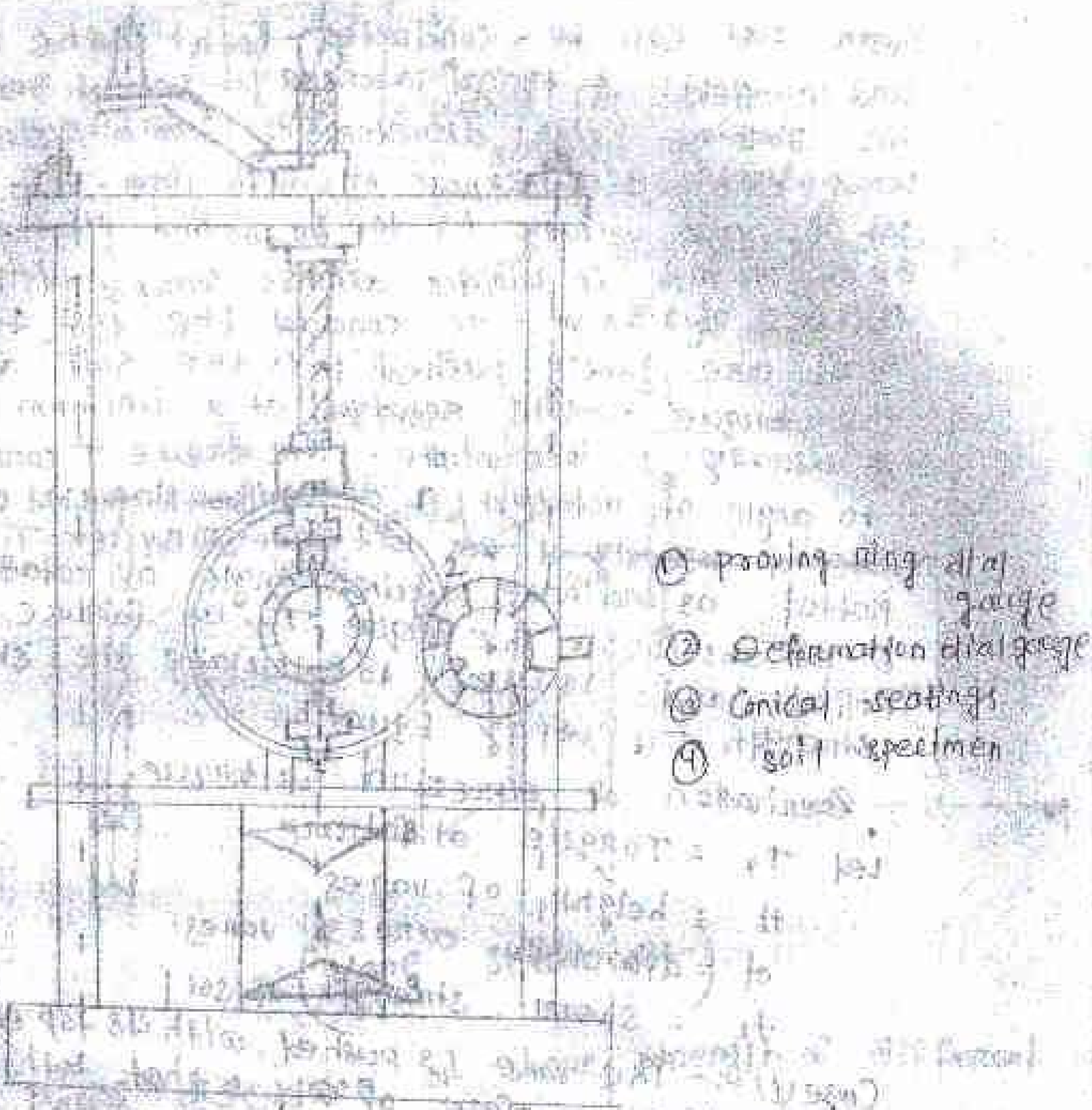
The unconfined compression test is a quick test in which no drainage is allowed. The test is conducted on saturated clay and volume change is assumed to be zero. The undrained shear strength parameters obtained are denoted by c_u and ϕ_u. The test results are acceptable for soil having no friction or little friction. The failure envelopes for the two cases are shown



(unconfined compression test)

The angle α which the failure plane makes with the horizontal is measured after carefully sketch in the failed specimen.

Unconfined compression test can also be conducted in field. The soil specimen is placed between two conical seatings attached to two metal plates. The soil specimen is loaded through a calibrated spring by manually operated screw jack at top of the machine. Then a graph of the load versus deformation can be plotted.



(Unconfined Compression test)

Vane Shear test :- Vane shear test is a quick test used to determine undrained shear strength of cohesive soils. The equipment essentially consists of four high tensile steel plates called vanes which are welded orthogonally to the bottom end of a steel rod called the torque rod with an arrangement to measure the torque and rotation. A typical arrangement consists of a calibrated torsion spring attached to the top of torque end which is rotated by a combination worm gear and worm wheel. The vanes

- Shear test can be conducted both in the laboratory and in field. A typical laboratory set of vanes has 20 mm height, diameter of 12 mm across vanes with blade thickness of 0.05 to 1 mm. The field set of vanes usually is 100 to 200 mm in height, 50 to 100 mm in diameter across vanes with blade thickness of 2.5 mm. To conduct the test the vanes are gently pushed into the soil and the torque rod is rotated at a uniform rate of usually 1° per minute. The torque T corresponding to angle of rotation θ at uniform interval are ~~rate of usually~~ noted. Torque T is plotted as ordinate against angle of rotation θ as abscissa. The torque T, at failure is found and is used to calculate the shear strength τ_f using equation.

Derivation of expression for torque.

Let T_f = Torque at failure

H = height of vanes

d = diameter across vanes

τ_f = shear strength of soil

Case (i) - The vane is pushed with its top end below the surface of soil so that both top and bottom ends take in shearing.

We note that shearing takes place along cylindrical surface of diameter d and height H. Taking moments about the axis of torque rod, we have

$$T_f = (\pi d H \tau_f)\frac{d}{2} + 2\int_0^{d/2}(2\pi r\,dr\,\tau_f)$$

$$= \frac{\pi d^2 H \tau_f}{2} + 4\pi\tau_f\int_0^{d/2} r^2\,dr$$

$$= \frac{\pi d^2 H \tau_f}{2} + 4\pi\tau_f\left[\frac{r^3}{3}\right]_0^{d/2}$$

$$= \frac{\pi d^2 H \tau_f}{2} + \frac{\pi d^3}{6}\tau_f$$

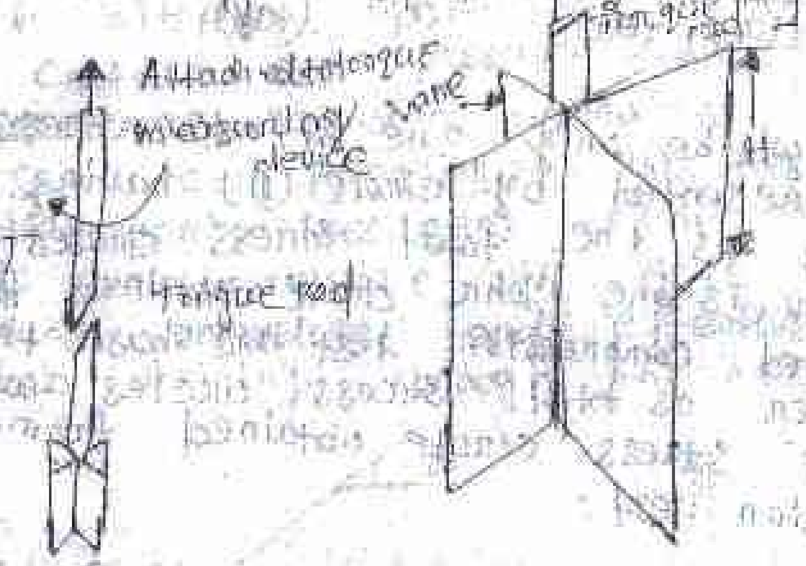
$$\pi d^2 \tau_1 \left[\frac{H}{2} + \frac{d}{6}\right]$$

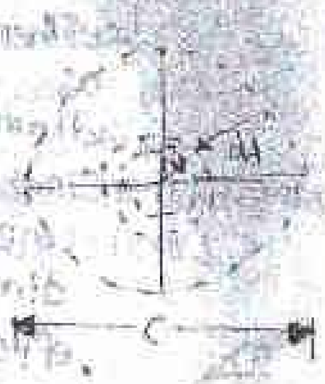
Case (ii) The vane is pushed inside the soil with its top end flush surface of soil so that only bottom end partakes in shearing.

Taking moments about the axis of torque rod we have

$$T_1 = (\pi d H \tau_1)\frac{d}{2} + \int_0^{d/2} (2\pi r\, dr\, \tau_1)\, r$$

$$T_1 = \tau_1 \pi d^2 \left[\frac{H}{2} + \frac{d}{12}\right]$$





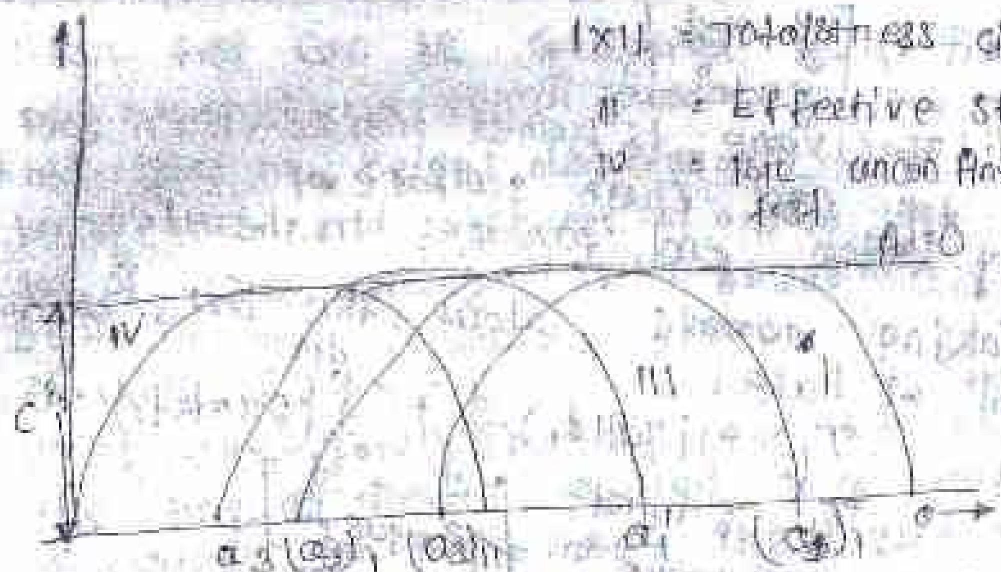
(Vane shear test)

Brief discussion on shear strength of different soil types

(i) Shear strength of fully saturated cohesive soils.

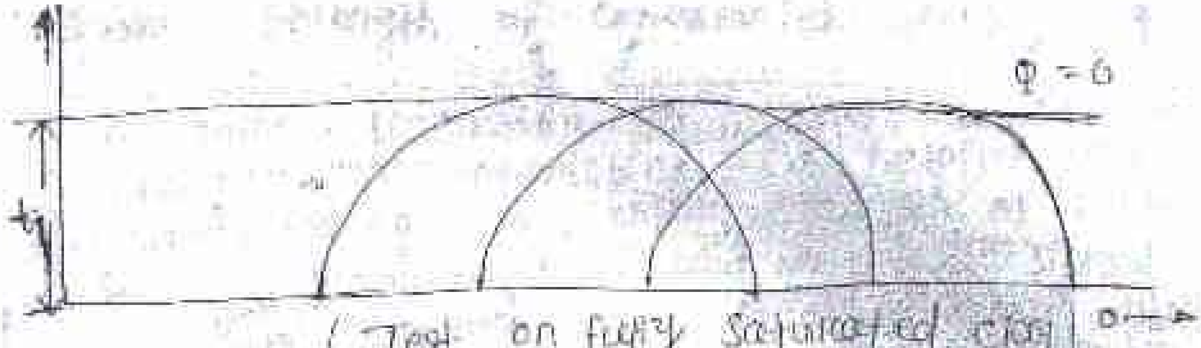
(a) Undrained test :- When undrained tests are conducted on identical specimens of a fully saturated clay with different cell pressure the mohr circles obtained will all be of same diameter. The failure envelope is horizontal giving $\phi_u = 0$ and $\tau_f = c_u$. It is clear that $(\sigma_1)_I - (\sigma_3)_I = (\sigma_1)_{II} - (\sigma_3)_{II}$

I & II = Total stress circles
III = Effective stress circle
IV = Total unconfined compression test



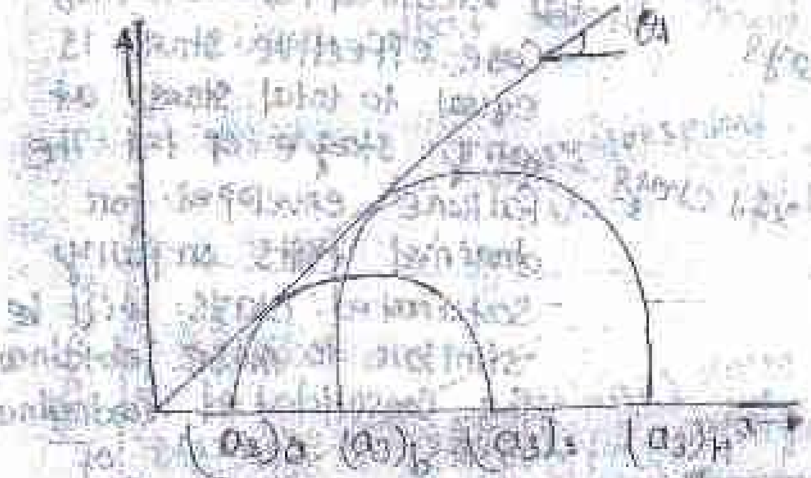
There will be only one effective stress circle as represented by circle (III) having same diameter as the total stress circles. The circle IV is the Mohr circle obtained from unconfined compression test and has the same diameter as total stress circles and effective stress circle obtained from triaxial compression test.

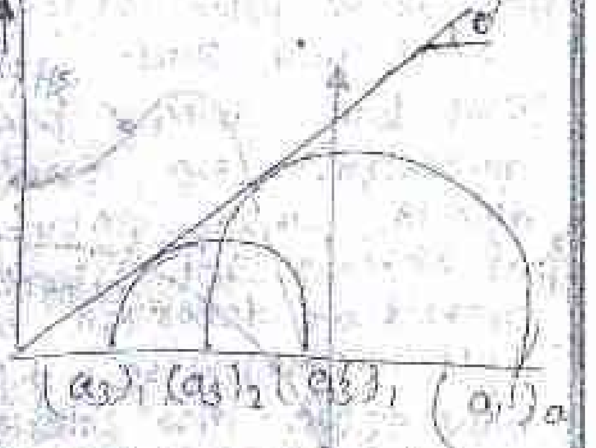
(b) Consolidated undrained test :- If consolidated undrained tests are conducted on remoulding, initially fully saturated clay specimen and consolidated with same cell pressure and subsequently sheared under undrained condition with different cell pressure the Mohr circles obtained will be of same diameter. The failure envelope is horizontal with $\phi_u = 0$ and f_i = C_u. Further if the tests are conducted with same but specimen initially consolidated with increased cell pressure and then sheared with different cell pressure as the failure envelope obtained is still horizontal with $\phi_u = 0$ but C_u will be greater than in the previous case.



(Test on fully saturated clay)

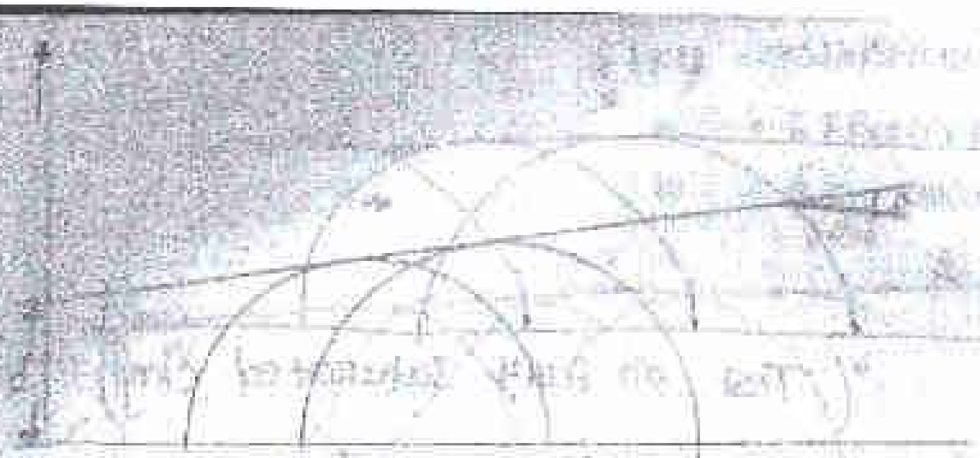
Further it is usual in laboratory practice to initially consolidate a specimen with a certain cell pressure and then shear the specimen under undrained condition at the same cell pressure. this test repeated for different specimens with the different value of cell pressure will give rise to total stress envelopes and effective stress envelopes both passing through the origin of stress.

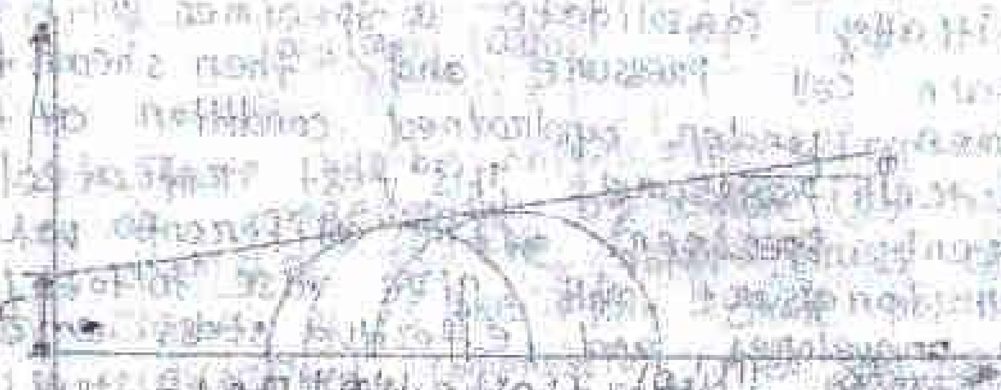


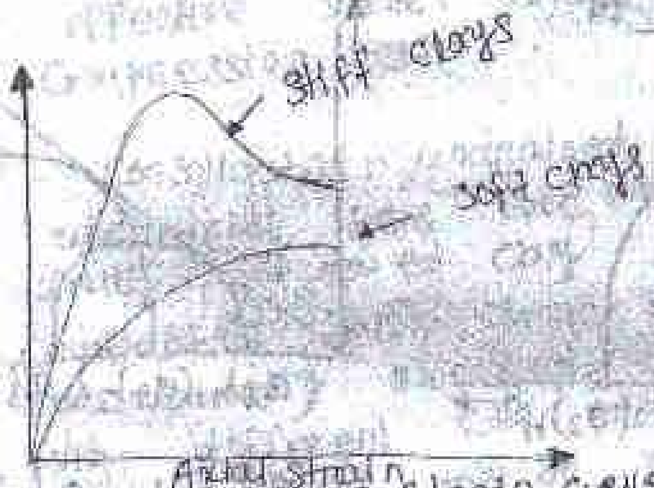


NOTE :- $\phi_1' > \phi$
Effective stress circle is shifted to left.

When consolidated undrained tests are conducted on preconsolidated fully saturated clay specimens the failure envelope will have cohesion intercept for both total stress and effective stress plottings, with apparent cohesion Ca greater than effective cohesion c' ϕ may be slightly greater of smaller than ϕ' corresponding to any total stress circle, the effective stress circle will be found shifted to right.





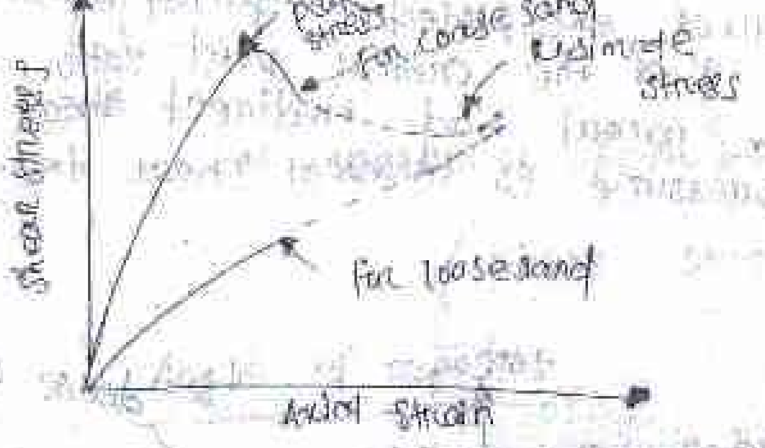


Typical stress-strain curves for clays

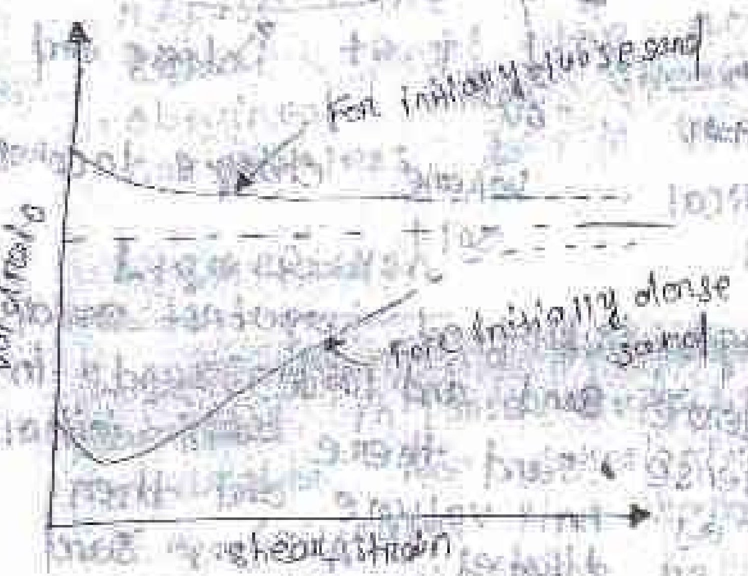
(c) Drained test :- In this case effective stress is equal to total stress at any stage of test. The failure envelopes for drained tests on fully saturated clays will be similar to those obtained for consolidated undrained tests in terms of effective stress. For normally consolidated condition the failure envelope passage through origin giving $c_d = 0$, For preconsolidated condition the failure envelope gives effective cohesion intercept c'. The angle ϕ' will be slightly different for the two cases.

Shear strength of Cohesionless soils

A prior discussion of stress - strain and volume change characteristics is helpful for proper understanding of shear strength of Cohesionless soil's (granular soils and non plastic silts). The stress - strain relation can be easily obtained from direct shear test under drained condition on saturated specimen or alternatively on dry specimen.

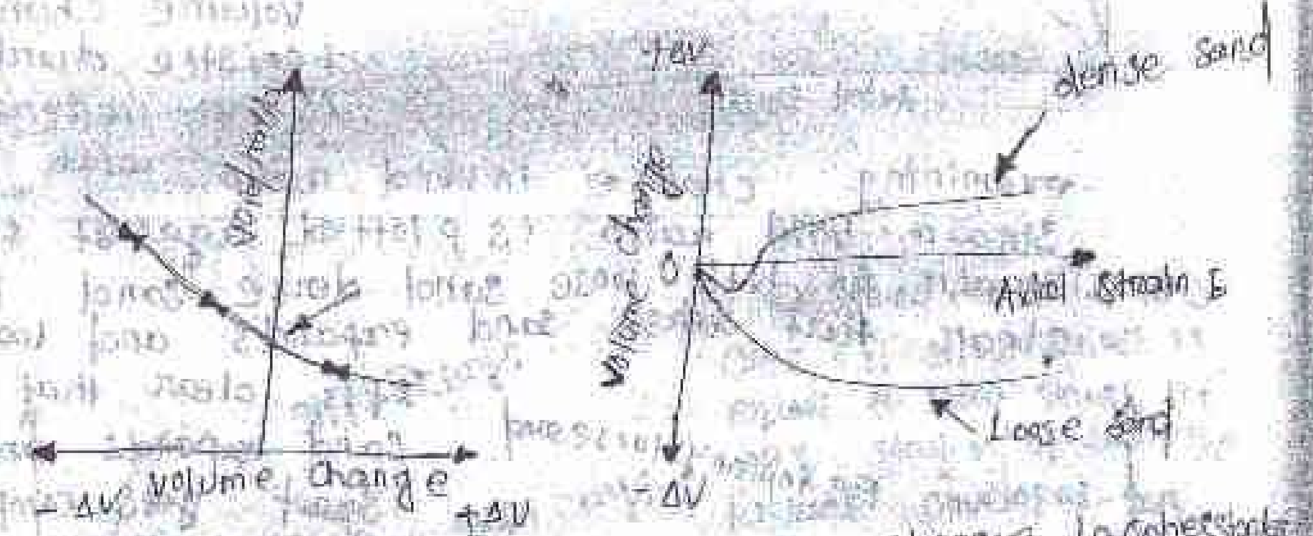


Volume change characteristic during shear is best understood by examining change in void ratio with increasing strain. Void ratio is plotted against shear strain for both loose sand dense sand. It is clear that dense sand expands and loose sand. It is clear that dense sand expands and loose sand gets compressed during shear. At high strains the curves tend to approach each other. At sufficiently high strains both dense sand and loose sand may be thought of as attaining the same void ratio at which there will be no further change in volume with increase in shear strain. Such a void ratio is termed critical void ratio. If a cohesionless soil is having its initial void ratio equal to critical void ratio, shear deformation can be expected to take place at constant volume. If initial void ratio is higher than critical void ratio, shear deformation will cause reduction in volume. If initial void ratio is lower than critical void ratio, increase in volume will accompany



shear deformation.

To determine critical void ratio, soil specimen initially at different void ratio are sheared under same normal stress in direct shear test or under same cell pressure in triaxial shear test. The initial void ratio values are plotted against measured volume change gives the critical void ratio for the particular normal load in direct shear test or cell pressure is triaxial shear test.



Determine of critical void ration

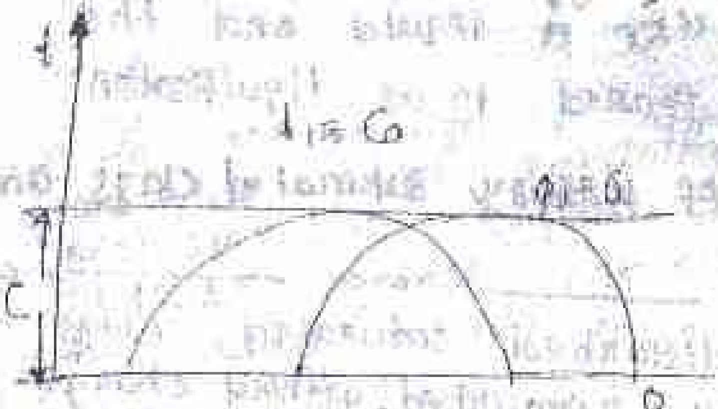
volume change in cohessionless soil

Volume change is plotted against axial strain for both dense sand and loose sand. In shearing of dense sand there will initially be slight decrease in volume and then expands or dilates. A loose sand gets compressed when sheared.

Drained strength of saturated sand

If drained tests are conducted on saturated sand specimen initially at the same density will be approximately the failure envelope through the origin by a straight line passing with effective stresses being of stresses equal to total stress, we have $c' = 0$. The drained shear strength is given by

$\tau_f = \sigma' \tan\phi'$



(a) undrained strength

(b) Drained strength

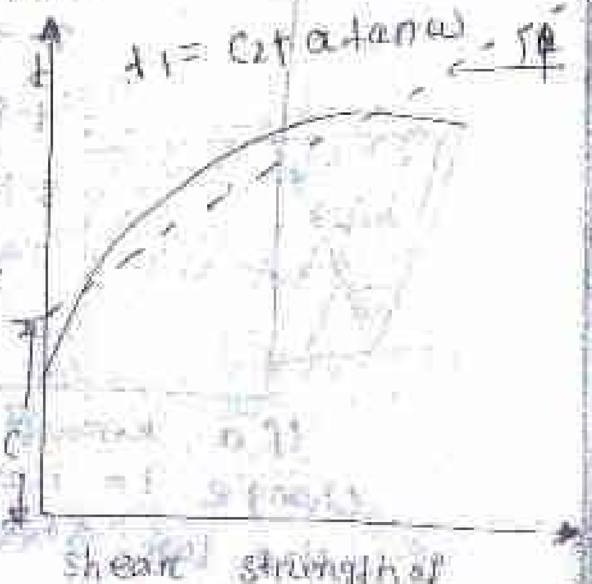
(Shear strength of cohesionless soils)

Angle of repose

When a cohesionless soil is poured from a small height it forms a heap. The angle between the sloping side of heap and horizontal is referred to angle of repose. The angle of repose will be nearly equal to ϕ' the angle of shearing resistance obtained from shear tests on the same cohesionless soil in loose state.

Liquefaction

Liquefaction is a phenomenon which can in loose deposits of saturated fine cohesionless soils. If a saturated fine sand deposit is subjected to a sudden disturbance as caused by vibrations of heavy machinery blasting or earthquake, rapid decrease in volume takes place and the pore pressure may increase to such an extent that effective stresses become zero leading to complete



Shear strength of partially saturated cohesive soil loss of shear strength. The soil at this stage behaves like a liquid and the phenomenon is referred to as liquification

shear strength of partially saturated clays and composite soils

When undisturbed saturated clays are disturbed or remoulded without change in water content, they loose part of their shear strength. This phenomenon is referred to as sensitivity. The degree of sensitivity is given by the ratio of undisturbed shear strength to the remoulded shear strength under unconfined condition.

$$\text{sensitivity} = \frac{\tau_f\,(\text{undisturbed})}{\tau_f\,(\text{remoulded})} = \frac{C_u\,(\text{undisturbed})}{C_u\,(\text{remoulded})}$$

The sensitivity of clays is found to vary from about 1 to over 100. A typical classification of clays based on sensitivity is given in the following table

Sensitivity	Classification of clay
<2	Insensitive
2-4	Medium sensitive
4-8	Sensitive
8-16	Extra sensitive
>16	Quick

If a remoulded soil is allowed to rest without change in water content, it regains a part of the lost shear strength. This phenomenon is referred to as thixotropy.

Earth pressure 18.1.2021

> In the design of retaining wall sheet pile or other earth retaining structures, it is necessary to compute the lateral earth pressure against retaining walls is one of the added in civil engineering.

> A retaining wall or retaining structure is used for maintaining the ground surface of different elevations on either side of it.

> The material retained or supported by the structure is called back fill, which may have it's top surface horizontal or inclined.

> The position of back fill lying above the horizontal plane at the elevation of the top of a wall is called the surcharge & it's inclination to the horizontal is called surcharge angle β.

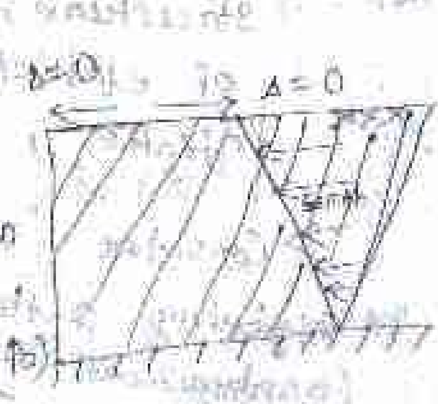
Type of earth pressure:-

This is generally divided into 3 types:-

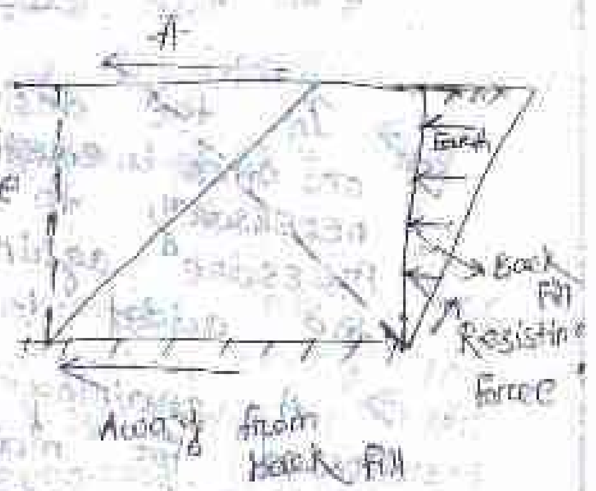
(A) Earth pressure at Rest
(B) Active Earth pressure
(C) passive Earth pressure

(A) Earth pressure at rest :-

> The pressure at which there is no deformation occur in the retaining wall, the pressure due to earth is known as earth pressure at rest.

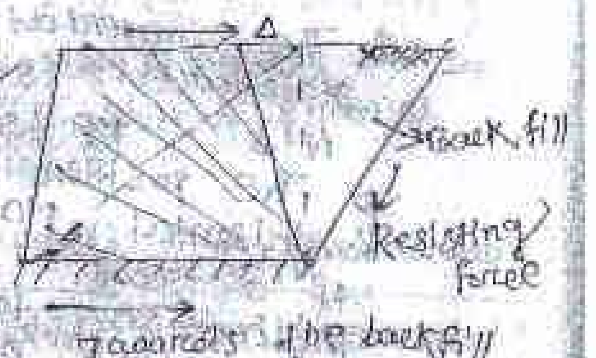


(4) Active earth pressure:-



⇒ The minimum pressure by which the retaining wall deformed away from the back fill, that pressure is known as active earth pressure (pa)

(5) Passive earth pressure:-



⇒ The maximum pressure by which the retaining wall is deformed towards the back fill that pressure is known as passive earth pressure (pp)

19 Jan 2021

Calculation of Earth pressure:

(1) Earth pressure at rest:-

⇒ The earth pressure at rest, exerted on the back of a rigid, unyielding retaining structure, can be calculated using theory of elasticity, assuming the soil is semi infinite, homogeneous, elastic & isotropic

⇒ Consider an element of soil at a depth 'z' being acted upon by vertical stress (σ_v) & horizontal stress (σ_h)

⇒ The will be no shear stress.

⇒ The lateral strain (ϵ_h) in the horizontal direction is given by :-

$$\epsilon_h = \frac{1}{E}\left[\sigma_h - \mu(\sigma_h + \sigma_v)\right] \longrightarrow eq^n (1)$$

The earth pressure at rest corresponding to this condition having zero lateral strain

$$\epsilon_h^0 = \frac{1}{E}\left[\sigma_h - \mu(\sigma_h + \sigma_v)\right]$$

$$\Rightarrow \sigma_h - \mu(\sigma_h + \sigma_v) = 0$$

$$\boxed{\sigma_h = \mu(\sigma_h + \sigma_v)}$$

or

$$\boxed{\frac{\sigma_h}{\sigma_v} = \frac{\mu}{1-\mu} = K_0}$$

where σ_h = horizontal stress
σ_v = vertical stress
μ = poisson's ratio
K_0 = Co-efficient of earth pressure at rest

Lateral earth pressure (σ_h / p_0) at rest $= K_0 \sigma_v$

$$\Rightarrow p_0 = K_0 \sigma_v \quad (\sigma_v = \gamma z)$$

$$\Rightarrow \boxed{p_0 = K_0 \gamma z}$$

$\Rightarrow$ ~~The total~~ ~~pressure~~
If $z = H$ (at the base of retaining wall) $\Rightarrow p_0 = K_0 \gamma H$

The total pressure 'P_0' per unit length for vertical height 'H' is given by:-

$$\boxed{P_0 \text{ (per unit length)} = \frac{1}{2} K_0 \gamma H^2}$$

where $\boxed{K_0 = 1 - \sin\phi}$ where ϕ = Angle of internal friction

Note:-

Soil type	K_0
(a) Loose sand	0.4
(b) Dense sand	0.6
(c) Sand compacted in layer	0.8
(d) Soft clay	0.6
(e) Hard clay	0.5

Q. A rigid retaining wall 6m height is restrained from yielding. The backfill consist of cohesionless soil having $\phi = 26^\circ$, $\gamma = 19\ kN/m^3$. Compute the total earth pressure per meter length of wall?

Sol^n

Data given: $H = 6m$

$\phi = 26^\circ$

$\gamma = 19\ kN/m^3$

$P_0 = ?$

$$P_0 = \frac{1}{2} K_0 \gamma H^2$$

$$K_0 = 1 - \sin\phi = 1 - \sin(26^\circ) = 0.561$$

$$\boxed{P_0 = \frac{1}{2} \times 0.561 \times 19 \times 6^2 = 191.86\ kN/m^2}$$

20 अगस्त 2021

Active Earth pressure :- Rankine's Theory :-

=> Rankine's Theory of lateral earth pressure is applied to uniform cohesionless is applied to uniform cohesionless soils only.

=> Later it was extended to include cohesive soils, by Resal.

=> Following are the assumptions of the Rankine's theory :-

(a) The soil mass is semi-infinite, homogeneous, dry & cohesionless.

(b) The ground surface is a plane which may be horizontal or inclined.

(c) The back of the wall is vertical & smooth. In other words there are no shearing stress between the wall & the soil.

(d) It satisfies the plastic equilibrium condition for deformation.

Following are the cases for cohesionless soil :-

(A) Dry or moist backfill with no surcharge.

(B) Submerged backfill.

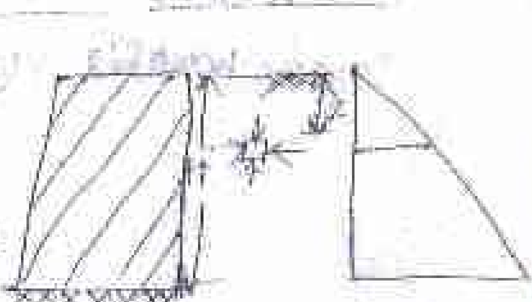
(C) Backfill with uniform surcharge.

(D) Backfill with sloping surface.

(E) Inclined back & surcharge.

(A) Dry or moist Backfill with no surcharge :-

=> Consider an element at a depth (z) below the ground surface.



=> When the wall is at the point of moving outwards the active state of plastic equilibrium is established.

Active earth pressure at a depth $z = H$

$$\boxed{p_a = k_a \gamma H}$$

where H = Height of retaining wall
γ_d = unit weight of dry soil
k_a = Co-efficient of active earth pressure.

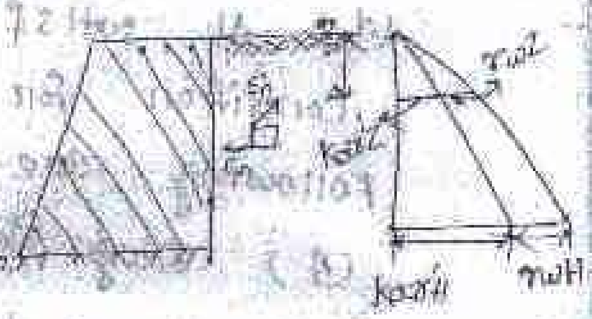
$$\boxed{k_a = \frac{\sigma_h}{\sigma_v} = \cot^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi}}$$

⇒ Total earth pressure per unit length

$$\boxed{P_a = \frac{1}{2} k_a \gamma H^2}$$

⇒ This total pressure per unit length is acting at $H/3$ above the base of the wall.

(B) Submerged backfill :-



⇒ In case the sand fill behind the retaining wall is saturated with water, then the lateral earth pressure is made upto 2 categories

(i) submerged weight (γ') of soil

(ii) unit weight of water

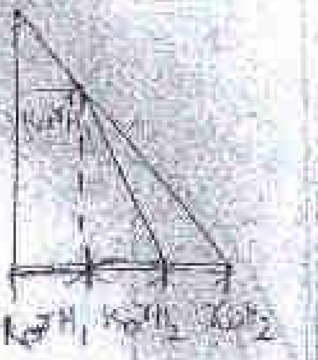
⇒ Active earth pressure of submerged backfill ($z = H$)

$$\boxed{p_a = k_a \gamma' H + \gamma_w H}$$

21 Jan 2021

⇒ If the free water stands to both sides of the wall, the water pressure need not to be considered & the net lateral earth pressure is given by :-

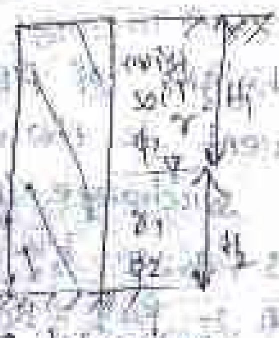
$$\boxed{p_a = k_a \gamma' H}$$

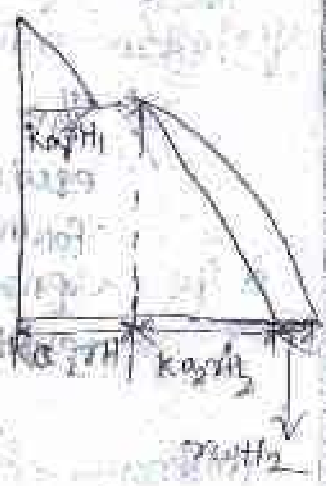


If the backfill is partly submerged i.e. the backfill is moist to a depth of H_1 below the ground level & then it is submerged, the lateral earth pressure intensity at the base of wall is given by :-

$$\boxed{p_a = k_a \gamma H_1 + k_a \gamma' H_2 + \gamma_w H_2}$$

⇒ If there is different in ϕ_1 & ϕ_2 respectively the earth pressure co-efficient is also different i.e. k_{a1} & k_{a2}



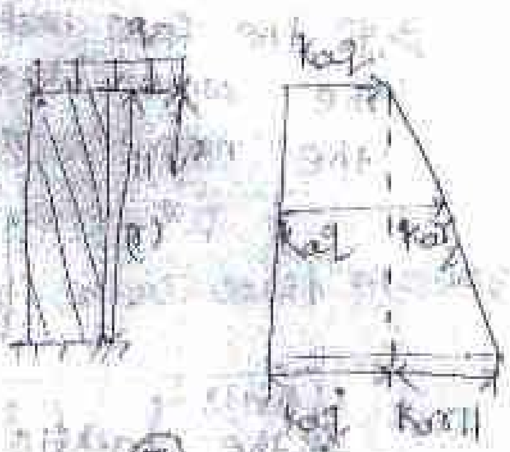


⇒ The lateral earth pressure intensity at the base of the wall is given by

$$\boxed{p_a = k_{a1} \gamma H_1 + k_{a2} \gamma' H_2 + \gamma_w H_2}$$

NOTE :- As ϕ decreases k_a will be increases.

(ii) Backfill with uniform surcharge :-



→ If the backfill is horizontal & carries a surcharge of uniform intensity 'q' per unit area, the vertical pressure increment at any depth 'z' will increase by q, the increase in lateral earth pressure due to this will be $K_a q$. Hence the lateral earth pressure at a depth 'z' will be :-

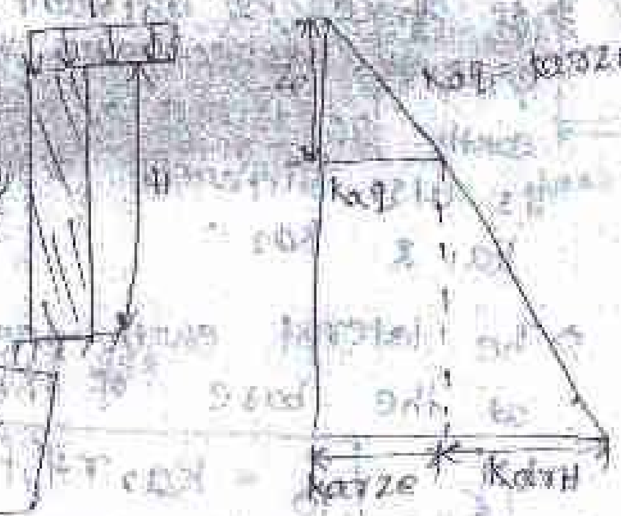
$$p_a = K_a q + K_a \gamma z$$

→ The lateral earth pressure at a depth H will be (z = H) :-

$$\boxed{p_a = K_a q + K_a \gamma H}$$

→ The height of fill Z_e equivalent to the uniform surcharge intensity 'q' is given by the relation :- $K_a q = K_a \gamma Z_e$

$$\boxed{Z_e = \frac{K_a q}{K_a \gamma} = \frac{q}{\gamma}}$$



NOTE Effect of surcharge due to q = pressure due to a fill of earth 'Z_e' above the ground surface.

The lateral earth pressure at the base of the wall :-

$$P_a = K_a \gamma z_e + K_a \gamma H$$

$$\boxed{P_a = K_a \gamma (z_e + H)}$$

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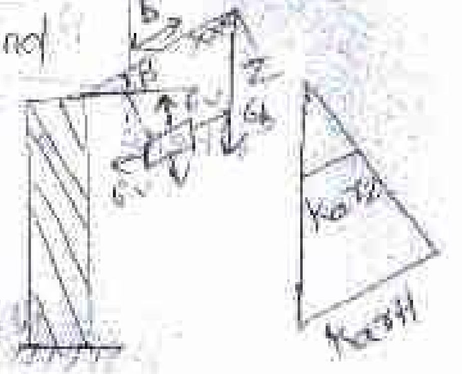
(2) Back fill with sloping surface :-



⇒ Let the sloping surface behind the wall will be inclined at angle 'β' with the horizontal, 'β' is called the surcharge angle

⇒ In finding out the active earth pressure for this case by Rankine's theory, an additional assumption that the vertical & lateral stresses are conjugate is made.

⇒ Consider an element at a depth 'z' from the inclined ground surface. The activity earth pressure at depth 'z' is :-

$$\boxed{P_a = K_a \gamma z}$$

⇒ Total earth pressure per unit length of a height 'H' is :-

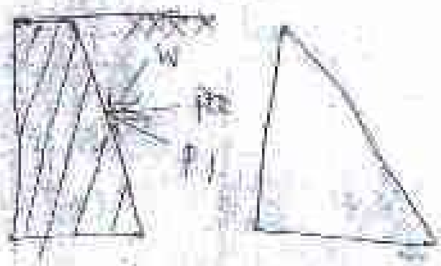
$$\boxed{P_a = \frac{1}{2} K_a \gamma H^2}$$

It acts H/3 above the base in direction parallel to the surface

$$\boxed{K_a = \cos\beta \, \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}}$$

(c) Inclined back & surcharge :-

If the retaining wall with an inclined back with horizontal ground surface then the total earth pressure (P_a) will be :-



$$\boxed{P_a = \sqrt{P_1^2 + W^2}}$$

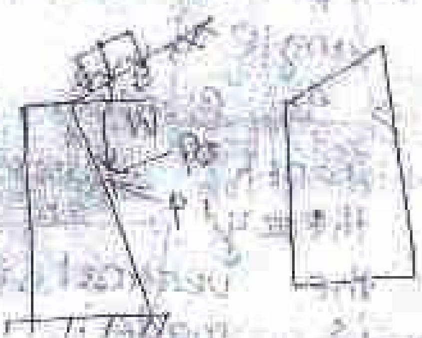
Where P_1 = horizontal pressure

$= \frac{1}{2} \times K_a \times \gamma H^2$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

W = weight

If there is inclined back with surcharge & inclined ground surface then the total earth pressure



$$\boxed{P_a = \sqrt{P_1^2 + W^2}}$$

$P_1 = \frac{1}{2} K_a \gamma H^2$

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Q) Compute the intensity of active earth pressure at a depth of 8m in dry cohesionless sand with an angle of internal friction of 30° & unit weight of 18 kN/m³ ? What will be the water table rises upto ground level ? Take the ~~saturated~~ submerged unit weight of sand is 22 kN/m³ ?

Sol Data given :- $\phi = 30°$, $\gamma = 18$ kN/m³

$\gamma' = 22$ kN/m²

$H = 8$ m

25 Janu 2021

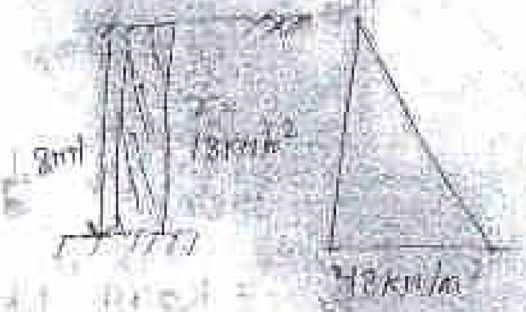
Case - I

Dry soil :-

$P_a = K_a \gamma H$

$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

$= \frac{1 - \sin(30^\circ)}{1 + \sin(30^\circ)} = \frac{1}{3}$



$P_a = \frac{1}{3} \times 18 \times 8$

$= 48 \ kN/m^2$

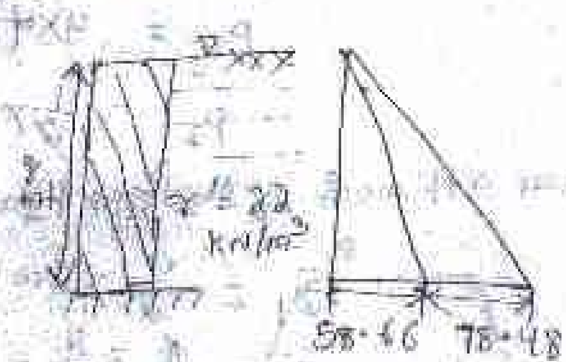
Case - II Submerged backfill :-

$P_a = K_a \gamma' H + \gamma_w H$

$= \left(\frac{1}{3} \times 22 \times 8\right) + (9.81 \times 8)$

$= 58.66 + 78.48$

$= 137.14 \ kN/m^2$



Q) A retaining wall 4mt high has a smooth vertical back. The backfill has a horizontal surface in level with top of the wall. There is uniformly distributed surcharge load of 12 kN/m² intensity over the backfill. The unit weight of the backfill is 18 kN/m³. It's angle of shearing resistance is 30° & cohesion is zero. Determine the magnitude & point of application of active earth pressure per meter length of the wall. In case II 2e = 2mt.

Sol^n Data given

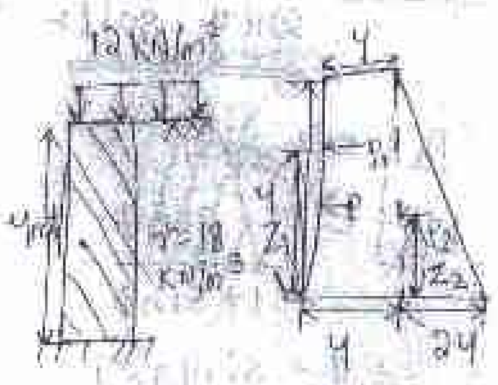
$\gamma = 18 \ kN/m^3$

$\phi = 30^\circ$

$H = 4 mt$

$q = 12 \ kN/m^2$

Case-I



$K_a = \frac{1-\sin\phi}{1+\sin\phi}$

$= \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3}$

$p_a = K_a \gamma H + K_a q$

$= (\frac{1}{3} \times (18\times 4)) + (\frac{1}{3} \times 12)$

$= 24 + 4$

$= 28 \text{ kN/m}^2$

⇒ Total earth pressure per unit length is :-

$P_1 = 4\times 4 = 16 \text{ kN/m}^2$

$P_2 = \frac{1}{2} \times 24 \times 4 = 48 \text{ kN/m}^2$

$P = P_1 + P_2 = 16 + 48 = 64 \text{ kN/m}^2$

$\bar{Z}_1 = \frac{4}{2} = 2\text{m}$

$\bar{Z}_2 = \frac{H}{3} = \frac{4}{3} = 1.33\text{m}$

$\bar{Z} = \frac{P_1 \bar{Z}_1 + P_2 \bar{Z}_2}{P_1 + P_2}$

$= \frac{(16\times 2) + (48\times 1.33)}{16 + 48}$

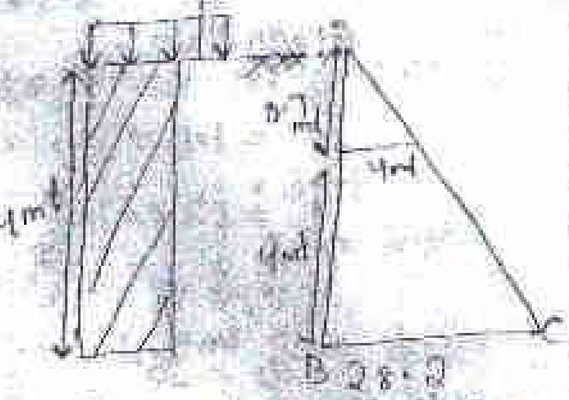
Case-II

$\gamma z_e = k q$

$\Rightarrow z_e = \frac{q}{\gamma}$

$\Rightarrow z_e = \frac{12}{18} = 0.66$ mt

$= 0.67$ mt



Active earth pressure at the base of the wall is :-

$P_a = k a \gamma (z_e + H)$

$= \frac{1}{3} \times 18 \times (0.7+4)$

$= 6 \times 4.7$

$= 28.2$ kN/m²

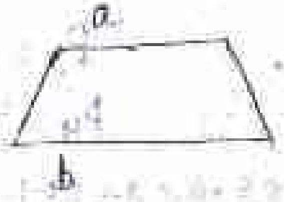
Total active earth pressure per unit length is:-

$P = \frac{1}{2}(4 \times 28.2) \times 4^2$

$= 64.4$ kN/m

The distance of point of application from the base is :-

$\bar{z} = \frac{H}{3} \frac{(2a+b)}{a+b}$



$= \frac{4}{3} \cdot \frac{2 \times 4 + 28.2}{4 + 28.2}$

$= 1.49$ mt above the base of wall

Q In the above problem if the water table rise behind the wall to an elevation of 1.4 mt below the ground surface. determine the total active earth pressure & its point of application Take submerged unit weight of sand is 12 kN/m³. Assum there is no change in angle of shearing resistance due to submergence ?

Given data:

$H = 1.5\,m$

$q = 12\,kN/m^2$

$\gamma = 18$

$\phi = 30^\circ$

Case- 1

$$k_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$= \frac{1-30^\circ}{1+30^\circ} = \frac{1}{3}$$

P_1 = (due to surcharge)

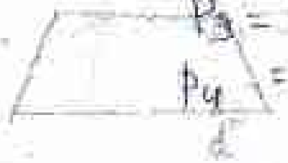
P_2 = (due to moist soil dry soil)

$P_a = k_a q + k_a \gamma H$

$$= \left(\frac{1}{3} \times 12\right) + \left(\frac{1}{3} \times 18 \times 1.5\right)$$

$$= 4 + 9 \qquad P_2 = k_a \gamma H_1 = \frac{1}{3} \times 18 \times 1.5 = 9\,kN/m^2$$

$$= 13 \qquad P_1 = k_a q = \frac{1}{3} \times 12 = 4\,kN/m^2$$



$P_3 = 2.5$

$P_4 = k_a \gamma' H_2$ (due to submerged soil)

$$= \frac{1}{3} \times 12 \times 2.5$$

$$= 10\,kN/m^2$$

$P_5 = \gamma_w H_2$ (due to water)

$$= 9.81 \times 2.5$$

$$= 24.524$$

Total active earth pressure

Total active earth pressure unit length $= P_1 + P_2 + P_3 + P_4 + P_5$

$P_1 = P_1 H = 4 \times 4 = 16\,kN/m^2$ and $z_1 = \frac{4}{2} = 2$

(from base)

$P_2 = \frac{1}{2} \times P_2 \times H_1 = \frac{1}{2} \times 9 \times 1.5$

$= 6.75 \text{ kN/m}^2$ act at $Z_2 = 2.5 + \frac{1.5}{3}$

$= 3 \text{ mt}$

(from base)

$P_3 = 9 \times 2.5 = 22.5 \text{ kN/mt}$ act at $\frac{Z_3}{}$

$= \frac{2.5}{2} = 1.25 \text{ mt}$ from base

$P_4 = \frac{1}{2} \times 10 \times 2.5 = 12.5 \text{ kN/mt}$ act at $Z_4 = \frac{2.5}{3} = 0.833 \text{ mt}$ from base

$P_5 = \frac{1}{2} \times 24.52 \times 2.5 = 30.65 \text{ kN/mt}$, act at $Z_5 = \frac{2.5}{3}$

$= 0.833 \text{ mt}$ from base

$P = P_1 + P_2 + P_3 + P_4 + P_5$

$= 16 + 6.75 + 22.5 + 12.5 + 30.65$

$= 88.4 \text{ kN/mt}$

$$\bar{Z} = \frac{P_1 Z_1 + P_2 Z_2 + P_3 Z_3 + P_4 Z_4 + P_5 Z_5}{P_1 + P_2 + P_3 + P_4 + P_5}$$

$$= \frac{(16 \times 2) + (6.75 \times 3) + (22.5 \times 1.25) + (12.5 \times 0.83) + (30.65 \times 0.83)}{88.4}$$

$= 1.314 \text{ mt}$ from base