

LEARNING MATERIAL

SEMESTER & BRANCH: 3RD SEMESTER CIVIL ENGINEERING

THEORY SUBJECT: GEOTECHNICAL ENGINEERING (TH-2)

NAME OF THE FACULTY: Er. SIBANI SAHU

> Soil is originated from the latin word
"solium"

> solium means disintegration of ~~rock~~ 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 engineer - The formation of soil
takes place from the decomposition of
vegetative 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.



* Physical weathering -

(i) It occurs due to corrosion or disinte-
gration 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

27 Aug, 2020

* Chemical Weathering :- $\rightarrow$ when rocks

come in to contact with acid or alkali, some chemical reaction occur, due to which, rock disintegrates into smaller particles.

$\rightarrow$ Here property of parent rock don't same as smaller particles.

$\rightarrow$ Here we get smaller size of particle i.e. called colloidal particle

(10^{-9} m)

Ex:- clay

$\rightarrow$ Fine bond between each particle is strong.

Scope of soil mechanics :-

$\rightarrow$ Soil mechanics is used for studying or investigating the soil before any construction that is made on the soil.

⇒ If we don't analyse the soil then it may be defined on any observation can make the superstructure & substructure damaged.

Types of soil

Residual

⇒ when the soil is located at its origin place i.e. at the foot of parent rock that soil is known as residual soil.

Transported

⇒ 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

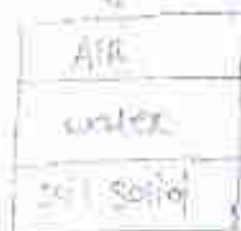
- (1) Alluvial soil → River
- (2) Lacustrine soil → Lake
- (3) Marine soil → Sea water
- (4) Aeolian → Air or wind
- (5) Glacial soil → Glacier or ice

2nd chapter

20/11/2020

Soil & water Relationships

⇒ soil phase system is only an idealized representation of soil sample & it is not particularly possible.



(partly 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 $(w) = V_a + V_w$

④ void ratio (e) :-

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

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

→ It is calculated in fraction.

→ 0.5 - 0.95 — coarse grain soil

0.5 - 1.7 — fine grain soil

sieve - 4.75 mm

> 0.075 mm



Sieve

pass - fine grain

Retain - coarse grain

(ii) Porosity :- (n)

porosity is defined as the ratio of volume of voids of the soil sample to the total volume.

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

It is expressed in percentage.

Range - $0\% < n < 100\%$

↓
Dry soil

↓
Fully saturated soil

70% = partially saturated soil

(iii) Air Content :- (a_c)

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

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

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

$$a_c = 1 - S$$

% air void :- percentage of air void is the ratio of volume of air to the total volume

→ It is expressed in percentage.

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

21 Aug 2020

(6) Water Content (w) :-

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

$$w = \frac{\text{weight of water}}{\text{weight of soil solid}} = \frac{w_w}{w_s}$$

→ Range :- 0 to 100

→ It is expressed in percentage.

unit weight & Density :-

→ unit weight is the ratio of weight per volume.

→ unit weight is symbolised as γ .

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

→ Density is the ratio of mass per volume.

→ It is symbolised as ρ .

unit :- kg/m^3

(7) Specific Gravity (G) :-
It is defined as the ratio of the weight of soil to the weight of water.

$$G = \frac{\text{weight of soil}}{\text{weight of water}} = \frac{W_s}{W_w}$$

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 :-
$$\gamma_d = \frac{\text{weight of soil solid particles in a volume}}{\text{volume of soil mass}}$$
$$= \frac{W_s}{V}$$

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

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

(6) Unit weight of soil solids :-
$$\gamma_s = \frac{\text{weight of soil solid particles in a volume}}{\text{volume of soil solid}}$$
$$= \frac{W_s}{V_s}$$

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 :-
$$\rho_d = \frac{\text{mass of soil solid particles in a volume}}{\text{volume of soil mass}}$$
$$= \frac{M_s}{V}$$

(4) Saturated Density :-
$$\rho_{sat} = \frac{\text{mass of saturated soil mass}}{\text{volume of soil mass}}$$
$$= \frac{M_{sat}}{V}$$

(5) Submerged Density :-
$$\rho' = \frac{\text{submerged mass of soil mass}}{\text{total volume of soil mass}}$$
$$= \frac{M_{sub}}{V}$$

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

(6) Density of soil solids :-
$$\rho_s = \frac{\text{mass of soil solid particles in a volume}}{\text{volume of soil solid}}$$
$$= \frac{M_s}{V_s}$$

$$\gamma_w = 9.81 \text{ kN/m}^3$$

$$S_w = 2.9 \text{ g/cm}^3$$

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).

$$G_s = \frac{\gamma_s}{\gamma_w} \text{ or } \frac{S_s}{S_w}$$

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

$$\rightarrow G_s = 2.6 - 2.8 \text{ (soil)}$$

Bulk specific gravity :-

$$G_m = \frac{\gamma}{\gamma_w} \text{ or } \frac{S}{S_w}$$

Types of soil

G_s

(1) sand

$$2.64 - 2.67$$

(2) silt

$$2.68 - 2.70$$

(3) clay

$$2.70 - 2.80$$

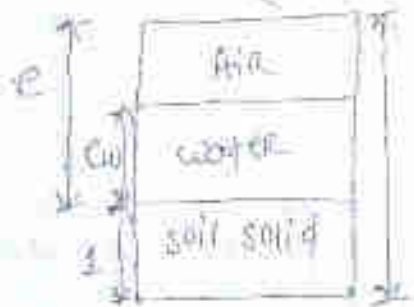
(4) soil containing iron

$$2.85 - 2.90$$

(5) organic soil

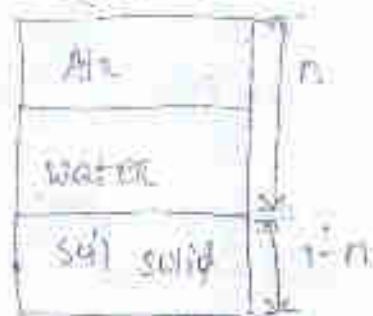
$$1.26 - 2.30$$

Relations :- (Fig-I)



(soil element is expressed in terms of e)

(Fig-II)



(expressed in terms of n)

Relation betⁿ e & n :-

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

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

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

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

Relation betⁿ e , w , G & S :-

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

$$\Rightarrow e_w = eS \quad \dots \dots \dots \text{eqn (i)}$$

$$w = \frac{w_w}{w_s} = \frac{V_w \cdot \gamma_w}{V_s \cdot \gamma_s}$$

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

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

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

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

$$\Rightarrow w_w = V_w \gamma_w$$

$$w_s = V_s \gamma_s$$

$$\Rightarrow \omega = \frac{e v_s}{\hbar}$$

$$\boxed{\omega_{L_1} = e v_s}$$

(2) Relation betⁿ e mass :-

$$\text{In fig (i)} \Rightarrow n a = \frac{V_a}{V}$$

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

$$= e - e w$$

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

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

$$n a = \frac{e - e s}{1 + e}$$

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

(3) r_d, r_g, e, m, π :-

$$s_L = \frac{M + s_L}{N} = \frac{s_L - s_L}{N} = \frac{w_M}{N} = \frac{p_M}{N} = p_L$$

$$\left[m g_L = \frac{m}{r_L} \leq \frac{s_L}{r_L} = \frac{p_L}{r_L} \right] (s_M = p_M)$$

$$\frac{\partial H}{\partial p_L} = \frac{m g_L}{r_L} = p_L$$

$$\boxed{r_L = \frac{m g_L}{p_L}}$$

(5) γ_d in air :-

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

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

24 Aug 2020

(6) $\gamma_{sat}, e, w(n)$:-

Let fully saturated soil :-

In fig-11 :- $\gamma_{sat} = \frac{w_{sat}}{V} = \frac{w_d + w_w}{V}$

$$= \frac{m_s \cdot m_d + m_w \cdot V_w}{V_s + V_v} \quad (w_d = w_s)$$

$$= \frac{\gamma_s \cdot V_s + m_w \cdot V_w}{V_s + V_v} \quad \left[\begin{array}{l} e = \frac{V_v}{V_s} \\ \gamma_s = \gamma_w V_w \end{array} \right]$$

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

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

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

γ_{sat}, e, w

In fig (ii)

$$\gamma_s = \gamma_s + \gamma_w V_w$$

$$V_s + V_v$$

$$= \frac{\gamma_{sat} (V_s + V_v) + \gamma_w V_w}{1}$$

$$1$$

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

(i) $\gamma, \gamma_s, \gamma_w, e$:-

Let partially saturated soil :-

$$\text{In 1 g of soil} = \frac{1}{\gamma} = \frac{V_s + V_w + V_a}{1}$$

$$\begin{aligned} &= \gamma_s \cdot V_s + \gamma_w \cdot V_w + 0 \\ &\text{If } e \end{aligned}$$

$$= \gamma_s \cdot m_s + \gamma_w \cdot e m_s$$

$$= \gamma_s \cdot m_s (1 + e)$$

$$\gamma = \frac{\gamma_s \cdot m_s (1 + e)}{1 + e}$$

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

$$\boxed{\gamma = \frac{(\gamma_s + e \gamma_w)}{1 + e} \gamma_w}$$

← partially saturated soil

(ii) $\gamma, \gamma_s, \gamma_w, e$:-

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

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

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

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

(a) $\gamma_1, \gamma_2, \omega$:-

$$\omega = \frac{\omega_0}{\omega_1}$$

$$\Rightarrow 1 + \omega = 1 + \frac{\omega_0}{\omega_1}$$

$$\Rightarrow 1 + \omega = \frac{\omega_1 + \omega_0}{\omega_1} = \frac{\omega_0}{\omega_1}$$

$$\Rightarrow 1 + \omega = \frac{\omega_0}{\omega_1}$$

$$\Rightarrow \omega_1 = \frac{\omega_0}{1 + \omega}$$

$$\gamma_1 = \frac{\omega_0}{1 + \omega}$$

$$\gamma_2 = \frac{\omega_0}{1 + \omega}$$

$$\boxed{\gamma_2 = \frac{\omega_0}{1 + \omega}}$$

(b) $\gamma_1, \gamma_2, \omega$:-

$$\gamma_1 = \frac{\gamma_2(1 - \omega)}{1 + \omega}$$

$$= \frac{\gamma_2(1 - \omega)}{1 + \omega}$$

$$= \gamma_2 \cdot \frac{1 - \omega}{1 + \omega} = \frac{1}{1 + \omega}$$

$$= \gamma_2 \cdot \frac{1 - \omega}{1 + \omega} = \frac{1}{1 + \omega}$$

$$\gamma_2 = \frac{1}{1 + \omega}$$

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

$$\boxed{\gamma_1 = \frac{\omega}{1 + \omega}}$$

(11) $r_{th} = r_s + r_{th}$

$$r = \frac{(r_s + r_{th}) r_o}{1 + \beta}$$

$$= \frac{r_s r_o}{1 + \beta} + \frac{\beta r_{th} r_o}{1 + \beta}$$

$$= r_s + \left[\frac{(r_s + r_{th}) r_o}{1 + \beta} - \frac{r_s r_o}{1 + \beta} \right] \beta$$

(12) $r_{th} = r_s + r_{th}$

$$r_{th} = r_s + r_{th}$$

$$= \frac{r_s r_o}{1 + \beta}$$

$$r_{th} = \frac{r_s r_o}{1 + \beta}$$

partially saturated
sol

(1 = 100% saturated sol)

(13) $r_{th} = r_s + r_{th}$

$$V = V_s + V_{th} + V_o$$

$$\Rightarrow V = V_s + \frac{V_o}{\beta} + V_o$$

$$\Rightarrow \frac{V}{V_o} = \frac{V_s}{V_o} + \frac{1}{\beta} + 1$$

$$\Rightarrow 1 = \frac{V_s}{V_o} + \frac{1}{\beta} + 1$$

$$\Rightarrow 1 = \frac{V_s}{V_o} + \frac{1}{\beta} + 1 \quad (w_s = w_{th})$$

$$r_{th} = \frac{w_{th}}{w_s}$$

$$V_{th} = \frac{w_{th}}{w_s} V_o$$

$$w_s = \frac{w_{th}}{w_s}$$

$$w_s = w_{th}$$

$$\Rightarrow 1 = \frac{v_a}{V} + \frac{w \cdot w_d}{v \cdot r_w} + \frac{r_d}{r_s}$$

$$\Rightarrow 1 = \frac{v_a}{V} + \frac{w \cdot r_s}{r_w} + \frac{r_d}{r_s}$$

$$\Rightarrow 1 - v_a = \frac{r_d}{r_s} \left(\frac{w \cdot r_s}{r_w} + 1 \right)$$

$$\Rightarrow r_d = \frac{(1 - v_a) r_w}{w + \frac{1}{G_s}}$$

$$= r_d = \frac{r_w (1 - v_a)}{1 + \frac{w}{G_s}}$$

$$\boxed{r_d = \frac{(1 - v_a) r_w G_s}{1 + w G_s}}$$

27 Aug 2020 :-

Ques A soil sample in its undisturbed state was found to have volume of 105 cm³ & mass of 201 gm after oven drying the mass got reduced to 168 gm. Calculate.

Sol Given data = Total volume (V) = 105 cm³

Total mass (M) = 201 gm.

Dry mass (M_d) = 168 gm.

$$(i) \text{ Water content (W)} = \frac{W_w}{W_s} \left[W_s = W_d \right]$$

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

$$= \frac{\text{Wet wt}}{\text{Dry wt}} \%$$

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

$$= 0.196 \times 100$$

$$= 19.6\%$$

(ii) void Ratio (e) :-

$$e = \frac{V_{\text{Sol.}} - V_s}{V_s}$$

$$S_w = 1g/cm^3, S_d = \frac{m_d}{V}$$

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

$$e = \frac{2.7 \times 1}{1.6} = 1.687 \approx 0.69$$

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

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

(iv) Degree of saturation (S) :-

$$S = \frac{w \times G}{e} \quad [e_s = w \times G]$$

$$= \frac{9.6 \times 2.7}{100 \times 0.69}$$

$$= 0.77$$

$$= 77\%$$

(v) Air Content (a_c) :-

$$a_c = 100\% - S$$

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

$$= 23\%$$

20 For a soil sample the specific gravity of soil mass is 1.7 & specific gravity of soil particles is 2.7. Determine the void ratio in case :-

- (i) assuming soil sample is dry
(ii) soil sample has a water content of 12%.

Specific gravity is unit less

Soil Given data :- $G_m = 1.7$
 $G_s = 2.7$

(i) soil sample is dry :-

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

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

$$\therefore \gamma_d = G_m \cdot \gamma_w$$

$$= 1.7 \times 1$$

$$= 1.7$$

Relationship :-

unit weight

density

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

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

$$(2) \gamma_d = (1-n) G_s \gamma_w$$

$$(2) \gamma_d = (1-n) G_s \gamma_w$$

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

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

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

$$(5) \quad \gamma = \gamma_d - (1-n)\gamma_w \quad (5) \quad S = S_d - (1-n)S_w$$

$$(6) \quad \gamma_d = \frac{\gamma_w}{\frac{G_m + 1}{G_m}}$$

$$(7) \quad S_d = \frac{G_m S_w}{\frac{G_m + 1}{G_m}}$$

$$(8) \quad \gamma = \frac{\gamma_w}{\frac{G_m + 1}{G_m}} - 1$$

$$(9) \quad e = \frac{G_m S_w}{S_d} - 1$$

$$(10) \quad \gamma_{sat} = \frac{(G_m + e)\gamma_w}{1+e}$$

$$(11) \quad S_{at} = \frac{(G_m + e)S_w}{1+e}$$

$$(12) \quad \gamma = \frac{(G_m + e)\gamma_w}{1+e}$$

$$(13) \quad S_{at} = \frac{(G_m + e)S_w}{1+e}$$

$$(14) \quad \gamma = \frac{\gamma}{1+e} = \gamma_d$$

$$(15) \quad S_d = \frac{S}{1+e}$$

$$(16) \quad \gamma = \gamma_d + S(\gamma_{sat} - \gamma_d)$$

$$(17) \quad S = S_d + S(S_{at} - S_d)$$

$$(18) \quad \gamma_d = \frac{\gamma_w (G_m + 1)}{1 + w G_m}$$

$$(19) \quad S_d = \frac{S_w (G_m + 1)}{1 + w G_m}$$

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

(a) void ratio

is dry unit weight

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

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

Given data :-

$$n = 40\%$$

$$G_s = 2.7$$

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

$$ii) \gamma_d = \frac{\gamma_{sw}}{1+e} \\ = \frac{2.7 \times 9.81}{1+0.66} = 15.9560 \text{ kN/m}^3$$

iii) if the soil is 50% saturated

$$S = 50\% = 0.50$$

$$\gamma = \frac{(\gamma_d e_s) \gamma_w}{1+e} = \frac{(2.7 \times 0.66 \times 0.5) \times 9.81}{1+0.66}$$

$$= 17.90 \text{ kN/m}^3$$

iv) if the soil is completely saturated

$$S = 100\% = 1$$

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

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

$$= 19.85 \text{ kN/m}^3$$

29 Aug 2020

20

A soil has a bulk unit weight of 20.11 kN/m^3 & water content of 15%. Calculate the water content if the soil partially dries to a unit weight of 19.142 kN/m^3 & the void ratio remains unchanged.

Bulk unit weight γ_1
 $= 20.11 \text{ kN/m}^3$
 $w = 15\% = 0.15$
 $\gamma_2 = 19.42 \text{ kN/m}^3$

void remain unchanged

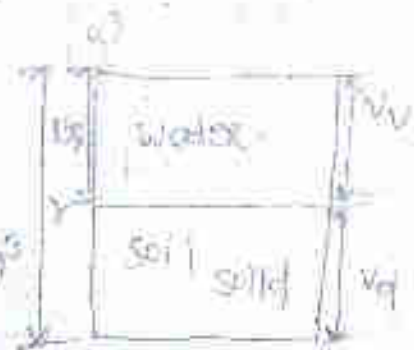


Fig-(i)

$T_1 = T_2$

$$\gamma_1 = \gamma_2$$

$$V_d = V_d$$

$$V_d = V_d$$



Fig-(ii)

Soil diagram (i) $\gamma_1 = 20.11 \text{ kN/m}^3$
 $w = 15\%$

$$\gamma_{sat} = \gamma_{d(1)} = \gamma_{d(2)} = 17.48 \text{ kN/m}^3$$

$$\gamma_{d(1)} = \frac{\gamma_{sat}}{1+w}$$

$$= \frac{20.11}{1.15} = 17.48 \text{ kN/m}^3$$

Soil diagram (ii) $\gamma_2 = \gamma_{d(2)}$

$$= 17.48 = 19.42$$

$$\Rightarrow \gamma_{d(2)} = 19.42$$

$$\gamma_{sat} = 19.42$$

$$= 0.11$$

$$= 11\%$$

66. The in-situ percentage void of a sand deposit is 34%. For determining the density index, dried sand from the stratum was first filled loosely in a 1000 cm³ mould & 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_s) = 2.67.

Given data:- $n = 34\%$

$$V = 1000 \text{ cm}^3$$

$$E_{\text{maximum}} = \text{loose soil mass} = 1610 \text{ gm} \quad (S_{\text{min}})$$

$$E_{\text{minimum}} = \text{dense soil mass} = 1980 \text{ gm} \quad (S_{\text{max}})$$

$$I D = \frac{E_{\text{max}} - e}{E_{\text{max}} - e_{\text{min}}}$$

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

$$e_{\text{min}} = \frac{G_s \omega}{S_{\text{min}}} - 1 \quad \left[\begin{array}{l} \omega = 0 \\ G_s = 2.67 \end{array} \right]$$

$$S_{\text{min}} = \frac{M}{V} = \frac{1610}{1000} = 1.61 \text{ gm/cm}^3$$

$$S_{\text{max}} = \frac{1980}{1000} = 1.98 \text{ gm/cm}^3$$

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

$$e_{\text{min}} = \frac{2.67 \times 0}{1.98} - 1 = 0.36$$

$$I.D = \frac{0.57 - 0.515}{0.67 - 0.35} = 0.5 = 50\%$$

(NEW CHAPTER)

31 Aug 2020

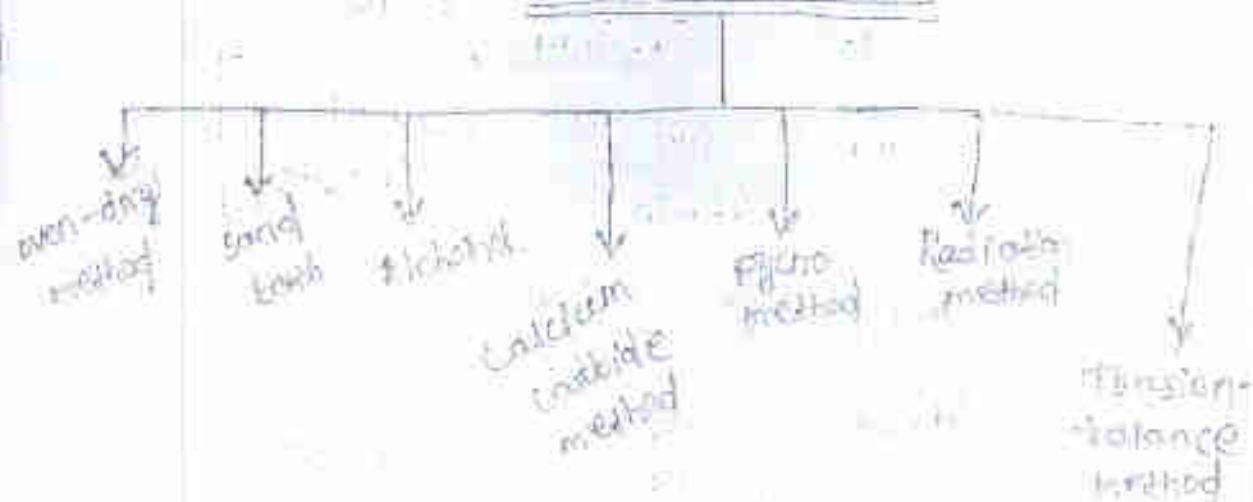
Index properties of soil

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



(ii) 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 exposure of non-conducting 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 hours 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)



m_3

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

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

Ex:- gypsum is suspected in the soil, then oven temperature should not more than 80°C .

→
= sign-
tance
cting

ad of
is to

(2) 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.

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

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

(3) Alcohol method :-

→ This is a field method.

→ The wet soil sample is kept in a evaporation dish & mixed with sufficient quantity of methylated spirit.

→ When dish is properly covered & the mixture is ignited.

→ The mixture is kept stirred by a wire during ignition.

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

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

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

(2) Calcium Carbide method:-

→ In this method 5 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 dial gauge is such that it gives the water content (%) of the wet weight of the sample.

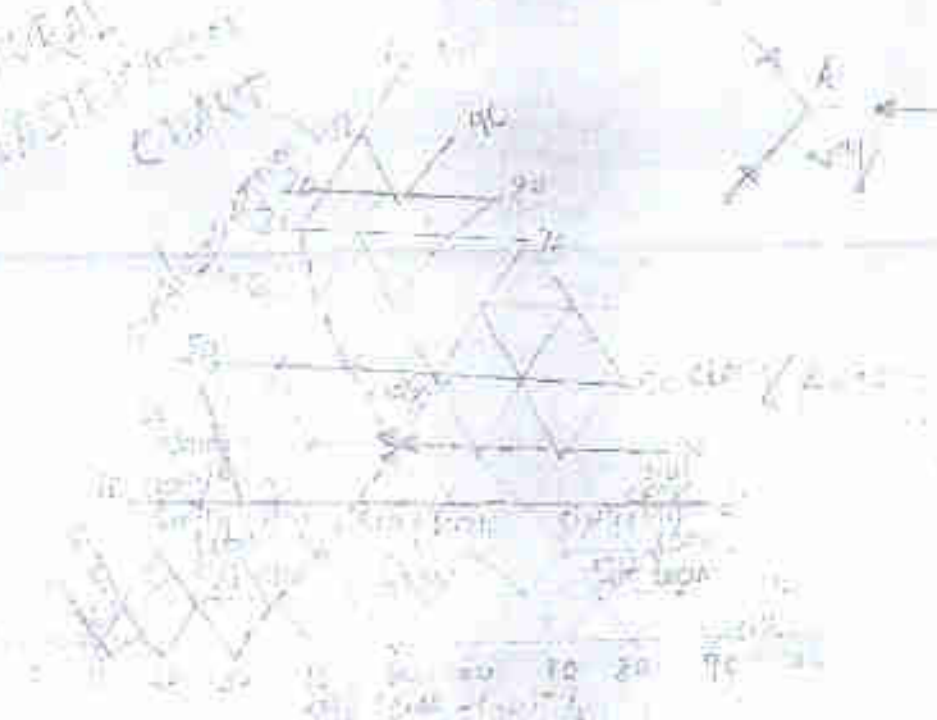
→ So actual $w = \frac{wt}{1 - wt}$

		0.075 mm	0.075 mm to 2 mm	2 mm to 4.75 mm	4.75 mm to 20 mm	20 mm to 60 mm	60 mm to 200 mm
			Fine	medium	coarse	Fine	coarse
clay	silt	sand			gravel		cobbles
							boulders

(I.S classification)

(II) Textural classification :- 19 SEP 2020

TEXTURAL CLASSIFICATION



- 1) Soil occurring in nature are composed of different clay, sand, silt sizes material.
- 2) Soil classification is based on the particle size distribution is known as textural classification.
- 3) This classification is a triangular classification and is known as "US public Road administration".
- 4) This classification is based on the percentage of sand, silt & clay size making the soil.
- 5) such classification is suitable for designing concrete.

- This is measured by a quadrilateral triangle.
- For example - If a soil is composed of 30% sand, 50% silt & 40% clay, then the three lines are drawn and intersect at point 'e'.
- This point is situated near clay - so that this type of clay is known as clayey soil.

(iii) 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 divided 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 m 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.

Content plastic limit is amount of
 Liquid passing the 75- μ m sieve

$$\text{Group Index} = 0.2a + 0.00075a^2 + 0.01b$$

where a = portion of percentage
 passing 75 μ m sieve, lies
 between $(25 \leq a < 75)\%$ (or)
 in no between "0" to "40"

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

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

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

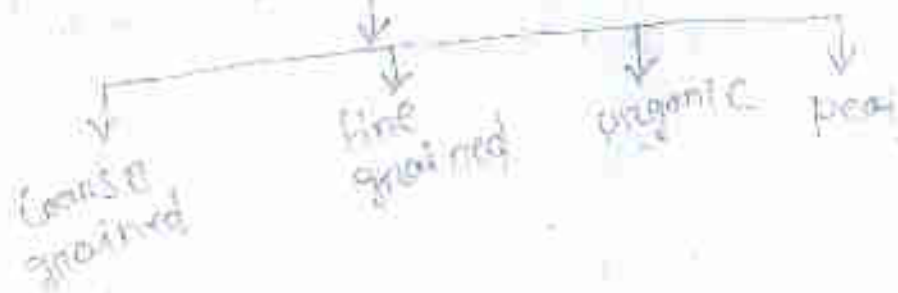
35% >

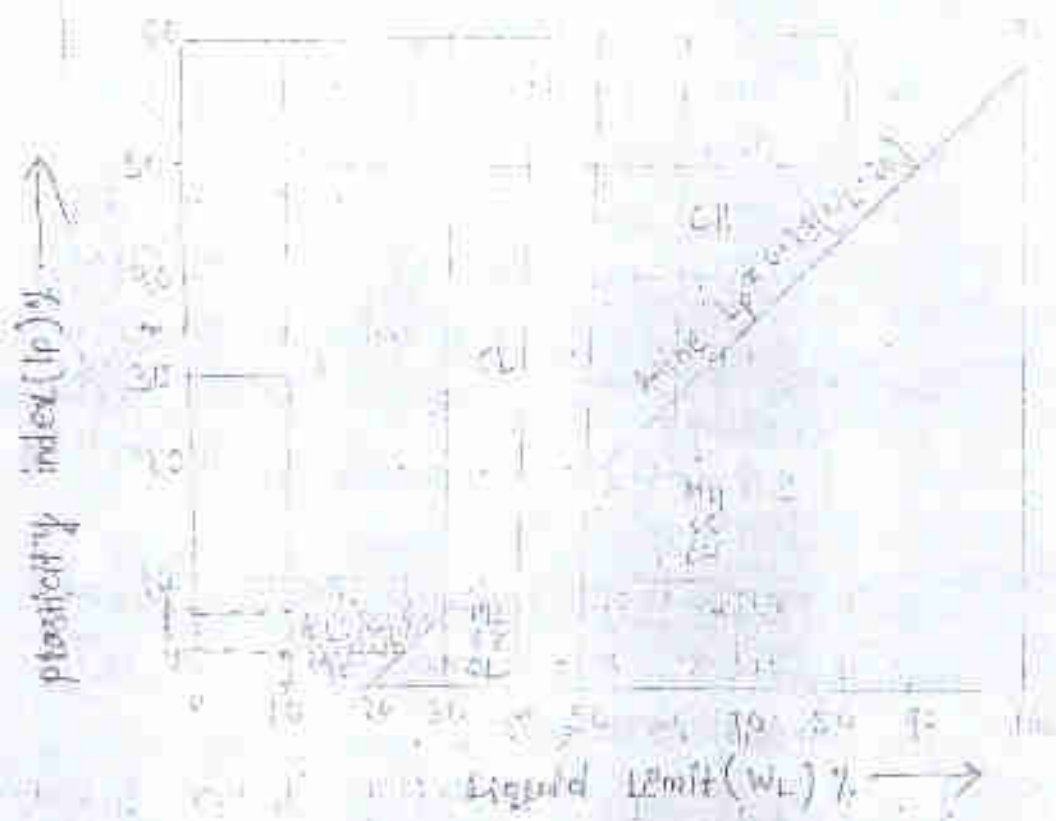
A-1, A-3, A-2

35% <

A-4, A-5, A-6, A-7

(4) unified soil classification system :-

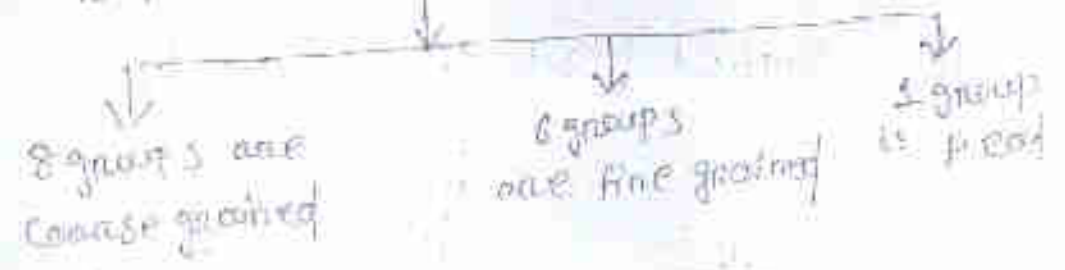




CASAGRANDE'S PLASTICITY CHART
(USCS)

22 Sep 2020

Based on the symbol soils are divided
is to 15 groups:



Symbol	Soil type	Prefix	Subgroup	Suffix
(1)	Gravel	G	(1) well graded	(1)
(2)	Sand	S	(2) poorly	(2)
(3)	Silt	M	(3) silty	(3)
(4)	Clay	C	(4) clayey	(4)

(5) organic ϕ (3) $w_L < 50\%$ U
 (6) peat pt (4) $w_L > 50\%$ H

(1) Coarse-grained -

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

→ A coarse grained soil is designated as "Gravel (G)" when 50% (a) 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 (b) 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 termed as fine grained if the soil is (a) more than 75 μ sieve that passing through the 75 μ sieve.

→ Fine grained soil is divided into 2 types - : (i) silt (ii) clay

- This fine grained soil is based on Liquid Limit (WL) & Plasticity Index (Ip).
- organic soil is also included in the fine grained soil
- To represent the "uses" Casagrande used a plasticity chart plotted between Liquid Limit & Plasticity Index.

* Indian standard soil classification :- (IS)

- Based on Liquid Limit fine grained soil sub divided into 2 groups -
 - (I) $WL < 36$ = low plasticity soil (L)
 - (II) $WL > 50$ = high plasticity soil (H)
- When the Liquid Limit decreased to 30% - @ 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 "uses" with the modification 1 groups (i.e. low compressibility, medium compressibility & high compressibility).
- Instead of using 10 groups from "uses" system, 12 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 subdivided into 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 symbolised by "G"

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

NOTE :-

→ Each of the subdivision divided into 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

(a) more than 75 μ sieve

(b) smaller than 75 μ sieve

Fine grained soils are further divided 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 divided into 3 types.

(10) (i) silt & clays of low compressibility

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

(25-30) (ii) 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).

(60) (iii) 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 soil

$$I_p = 0.73 (w_L - 20)$$

28 Sep 2020

Permeability

- > Permeability is defined as the property of a porous material which permits the passage or seepage of water through its interconnected voids.
- > A material having continuous voids is known as permeable.
- > Gravel are highly permeable while stiff clay is least permeable so it is called impermeable for all purpose.
- > The flow of water through soils may be either laminar flow or turbulent flow.
- > In laminar flow each fluid particle flows through them or travels along a definite path in which no particle cross each other's 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.

(E3) pycnometer method :-

→ This is also a quick method for determining the water content of those soils whose specific gravity G_s 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.

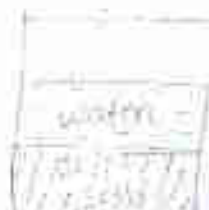
→ Add more water & stir it to remove the air from it & then take its mass (M_3)

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

$$\therefore \text{Water content} = \left[\left(\frac{M_2 - M_1}{M_3 - M_4} \right) \left(\frac{G_s - 1}{G_s} \right) + 1 \right] \times 100$$



Empty pyc (M_1)



pyc with soil (M_2)



pyc with water (M_3)

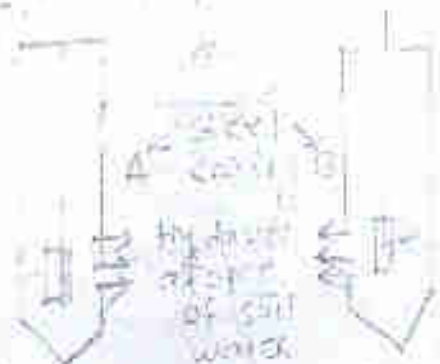


pyc with water (M_4)

NOTE:- This method is suitable for loose
unconsolidated 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 a 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 neutrons 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) Evaporation balance method:-

→ The equipment has two main parts.

(i) Infra-red lamp.

(ii) Evaporation balance.

→ The infra-red radiation is provided by
250 watt lamp bulb in the balance
box. 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 10°C to 50°C mentioned in thermometer.
- from the test water content of wet mass is $w\%$

$$\Rightarrow \text{Actual water content } w = \frac{w_1}{1 - w_1}$$

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 cm^3 . 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 cm^3
 mass of cutter = 1286 gm
 mass of cutter soil sample = 3195 gm

$$G_s = 2.7 \quad w = 12\%$$

$$\text{Case (i)} \therefore \gamma = \frac{\text{mass of soil sample}}{\text{Volume of soil sample}}$$

$$= \frac{319.5}{128.6}$$

$$= 1.96 \text{ g/cm}^3$$

$$(ii) \gamma_d = \frac{\gamma}{1 + \frac{w}{G}}$$

$$= \frac{1.96}{1 + 0.12} = 1.69 \text{ g/cm}^3$$

$$(iii) \gamma_d = \frac{G \gamma_w}{1 + \frac{w}{G}}$$

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

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

$$= \frac{0.32}{G} = \frac{2.7}{1.69} - 1$$

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

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

$$= 53.6\%$$

$$e_s = w \cdot G$$

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

$$= 0.221 \times 100\%$$

$$= 22.1\%$$

$$(i) e.s = w G$$

$$\Rightarrow w = \frac{e.s}{G} = \frac{0.59721}{2.7}$$

$$= 0.221 \times 100\%$$

$$= 22.1\%$$

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

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

$$= 20.25 \text{ kN/m}^3$$

2 SEP 2020

(A) SPECIFIC GRAVITY :-



Empty pyc
(m)



pyc + wet
soil (m)



Pyc + water
(m)



Pyc +
water
(m)

$\Rightarrow$ The specific gravity of soil solids is determined by

(i) Sand density bottle

(ii) Brom flask

(iii) a pycnometer

$\Rightarrow$ 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 it the sample with a wire to remove the air voids.

→ then completely fill the pycnometer with water & soil sample & 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 :-

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

(3) Consistency Limit - 20/5/2020

→ 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 divide the soil state from liquid to solid into 4 stages.

(I) Liquid state

(II) Plastic state

(III) Semi-solid state

(IV) Solid state

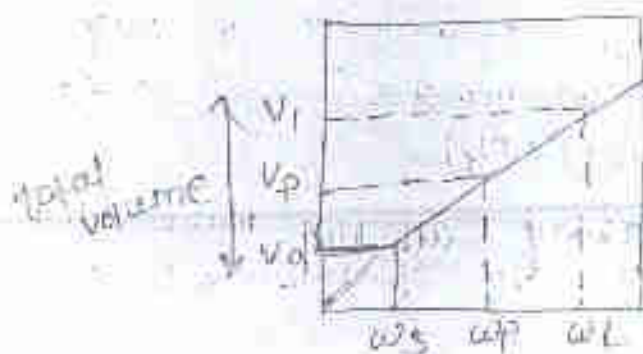
→ this Atterberg limit is very much well for egg purpose.

Atterberg, set limits based on the ~~liquid~~ ~~liquid~~ 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 arbitrarily between liquid & plastic state.

→ It is defined 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) Fellenius

(iv) 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.

(v) Shrinkage Limit :-

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

(vi) 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.

$$I_p = w_L - w_p$$

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 then I_p will be zero.

(V) Consistency Index:-

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

→ 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-solid state

④ $I_c = -ve$ → the state of soil is

in liquid state.

(VI) Liquidity Index (I_L):-

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

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 slope of the curve can be determined by :-

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

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)

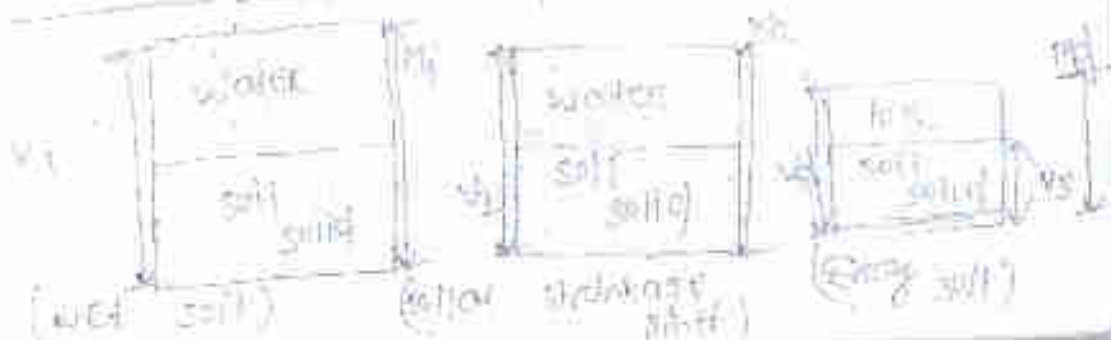
Toughness index is defined as the ratio of plasticity index to the flow index.

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

NOTE :-

(1) $I_p\%$	plasticity
(a) 0	Non-plastic
(b) < 7	Low plastic
(c) 7-17	medium plastic
(d) > 17	Highly plastic

Shrinkage Limit :-



$$W_s = \frac{(M_u - M_d) - (V_u - V_d) \gamma_w}{M_d}$$

where (M_u) = Mass of wet soil

(M_d) = Mass of dry soil

(V_u) = volume of wet soil

(V_d) = volume of dry soil

(found out from mercury displacement method)

(or) W_s can be determined by using alternative method, when G is known as =

$$W_s = \frac{\text{Mass of water at Shrinkage Limit}}{\text{Mass of soil solid}}$$

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

where V_d = volume of soil mass at any water content

V_d = volume of dry soil

Shrinkage ratio = $\left(\frac{V_d}{V_u} \right)$

→ 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 & is known as volumetric shrinkage.

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

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

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

Q10 The mass & volume of a saturated clay specimen is 29.8 g & 17.7 cm³. on oven drying the mass got reduced to 19 g & the volume to 8.9 cm³. calculate.

(i) Shrinkage Limit

(ii) Shrinkage ratio

(iii) volumetric shrinkage

(iv) γ_g

Soln

wet soil mass 29.8 g = M_1

volume of wet soil = (M_d) = 19 g

$$V_d = 8.9$$

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

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

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

$$(ii) w_1 - w_2 = \frac{w_1 w_1}{w_1 d_1} = \frac{w_1 w_2}{w_1 d_2}$$

$$= \frac{w_1 w_1}{w_1 d_1} = \frac{w_1 w_2}{w_1 d_2}$$

$$= \frac{(Wd_1 - Wd_2)}{Wd}$$

$$= \frac{Vw_1 \times Yw - Vw_2 \times Yw}{Wd}$$

$$= \frac{(V_1 - V_2) \times Yw}{Wd} = \frac{(V_1 - V_d) \times Yw}{Wd} \quad \text{--- eq (1)}$$

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

$$= \frac{V_1 - V_d \times Yw}{Wd}$$

$$= \frac{Wd}{V_d \times Yw} = \frac{Md}{V_d \times Yw} = \frac{19}{8.9 \times 1}$$

$$= 2.13$$

$$(iii) \quad V_3 = \frac{V_1 - V_d}{V_d} \times 100$$

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

$$= 98.8\%$$

$$(iv) \quad \frac{1}{G_f} = w_s = \frac{V_d \times Yw}{Md} = \frac{1}{G_f}$$

$$\Rightarrow \frac{1}{G_f} = \frac{V_d \times Yw}{Md} \times w_s$$

$$\Rightarrow \frac{8.9 \times 1}{19} - 0.105 = 0.363$$

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

$$G_f = \frac{1}{0.363} = 2.76$$

12 sep 2020

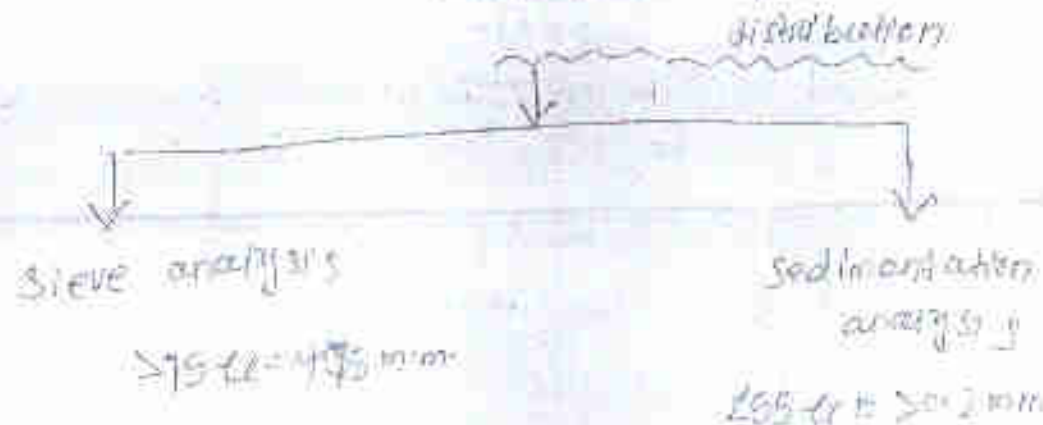
NINE

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

particle size distribution - ②

mechanical analysis

disturbance



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 i.e. that

are (i) sieve analysis

(ii) sedimentation analysis

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

1) sieve analysis :-

→ This method is used for coarse grained soil that is from 4.75 mm - 75 μ m 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 divide -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, 600 μ m, 425 μ m,

300 μ m, 212 μ m, 150 μ m & 75 μ m -

coarse & 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 & smallest aperture is at the bottom.

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

→ the soil sample is put on the 4 sieve & the whole assembly is placed on mechanical sieve shaker.

→ the amount of shaking depends on the shape & no of particle.

→ But generally it take 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 sil & 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 as mixture for washing till the soil particles pass through it.

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

(2) Sedimentation Analysis:-

→ In the wet mechanical analysis (2) sedimentation analysis the soil particles finer than 75 μ m size kept in suspension.

→ the analysis is based on "Stokes" law.

> According to "Stokes law" the solid 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 settling velocity (or) terminal velocity (v)

$$v = \frac{1}{18} \frac{D^2 (\rho_s - \rho_w)}{\eta} \quad (or) \quad (1)$$

$$\frac{2}{9} \times r^2 \times \frac{\rho_s - \rho_w}{\eta}$$

where :- v = terminal velocity

r = radius of the spherical particle (m)

D = Diameter of the spherical particle (m)

ρ_s = unit wt of soil/solid

ρ_w = unit wt of water

η = viscosity of water (kg-m/s/m²)
(ETA)

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

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

15 SEP 2020

Limitation of sedimentation analysis :-

⇒ The analysis is based on the following assumptions :-

- (i) all particles are spherical
- (ii) particles settle independent of other particles & they do not affect the other particles
- (iii) The way of fall in which suspension is kept also don't affect the settlement

These 3 assumptions are actually not possible because all the particles are not in spherical shape, when the particle settle down walls are affected & also each particle strike with other while settling.

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

(a) pipette method

(b) hydrometer method

(a) pipette method



- This method is a standard sedimentation method used in the laboratory.
- The equipment consist of a pipette, jar & sampling bottles & boiling tube of 50ml capacity.
- The particles finer than 75 μ size are distributed by this method.
- About 20 gms of oven-dried soil is mixed with distilled water in a beaker & form a thin paste.
- Then have proper dispersion of soil, a dispersing agent (such as sodium hexametaphosphate or sodium carbonate) of 2 spoon mixed in the solution & mixture is stirred for 5 minutes.
- Then the sample transferred to the 50ml capacity of boiling tube & close the tube & shaken several times.
- Then stop with stated soil sample are collection at various time interval. Such as - $\frac{1}{2}$ min, 1min, 2min, 4min, 8min, 15min, 30min, 1 hour, ... 24 hours.
- At the time of pipette should be inserted in the boiling tube about 25 seconds before selected time interval & time taken for sucking the sample should not be more than 10 to 20 seconds.
- Then the collection sample by pipette is 50ml of volume is placed in sampling bottle & kept it for oven drying.

Calculating :-

$$\text{mass per cent} = \frac{\text{Dry mass of sample}}{\text{Suspension (ml)}} \times \frac{V_p}{V}$$

where V_p = volume of pipette = 40 ml

$$\text{percentage fixed (pf)} = \frac{m \cdot \frac{M}{V} \times 100}{m_0 / V}$$

where m = mass of dispersing agent (i.e. 1gm of sodium carbonate & 3gm sodium hexametaphosphate)

$\Rightarrow \frac{239}{1000} \times 2.5 = 598.75$ (dispersing agent solution)

V = volume of suspension = 50ml

(ii) hydrometer method:-

- It differs from pycnometer method.
- 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. readings are for convenience hydrometer readings are recorded after subtracting 1 & multiplying the remaining digits by 1000.
- such reduced reading is "R"

$$H_c = H \left(\frac{1}{2} + \frac{V_h}{2A} \right) - \frac{V_h}{A} = H \left(\frac{1}{2} - \frac{V_h}{A} \right)$$

- In this volume of suspension is 100 ml. Quantity of dry soil & dispersing agent is taken double.
- stopwatch simultaneously started & readings are taken at 1/2, 1 & 2 minutes ... 1, 2, 4 hours.
- to take the readings hydrometer is inserted about 30 seconds before the transmittance.
- then take the reading by doing some correction due to meniscus, temperature & dispersing agent.

Calculation:-

Corrected hydrometer reading

$$R = R_h + C_m + C_t + C_d$$

Eq. Sep 2020

where R_h = observed hydrometer reading

C_m = corrected meniscus

C_t = corrected temperature

C_d = corrected dispersing agent

$$M = \frac{R}{1000} \left(\frac{G_s}{G - 1} \right)$$

where G_s = specific gravity of soil solid

$$N = \frac{M}{V} \times \left(\frac{G_s}{G - 1} \right) \times 100 \quad (V = 100 \text{ ml})$$

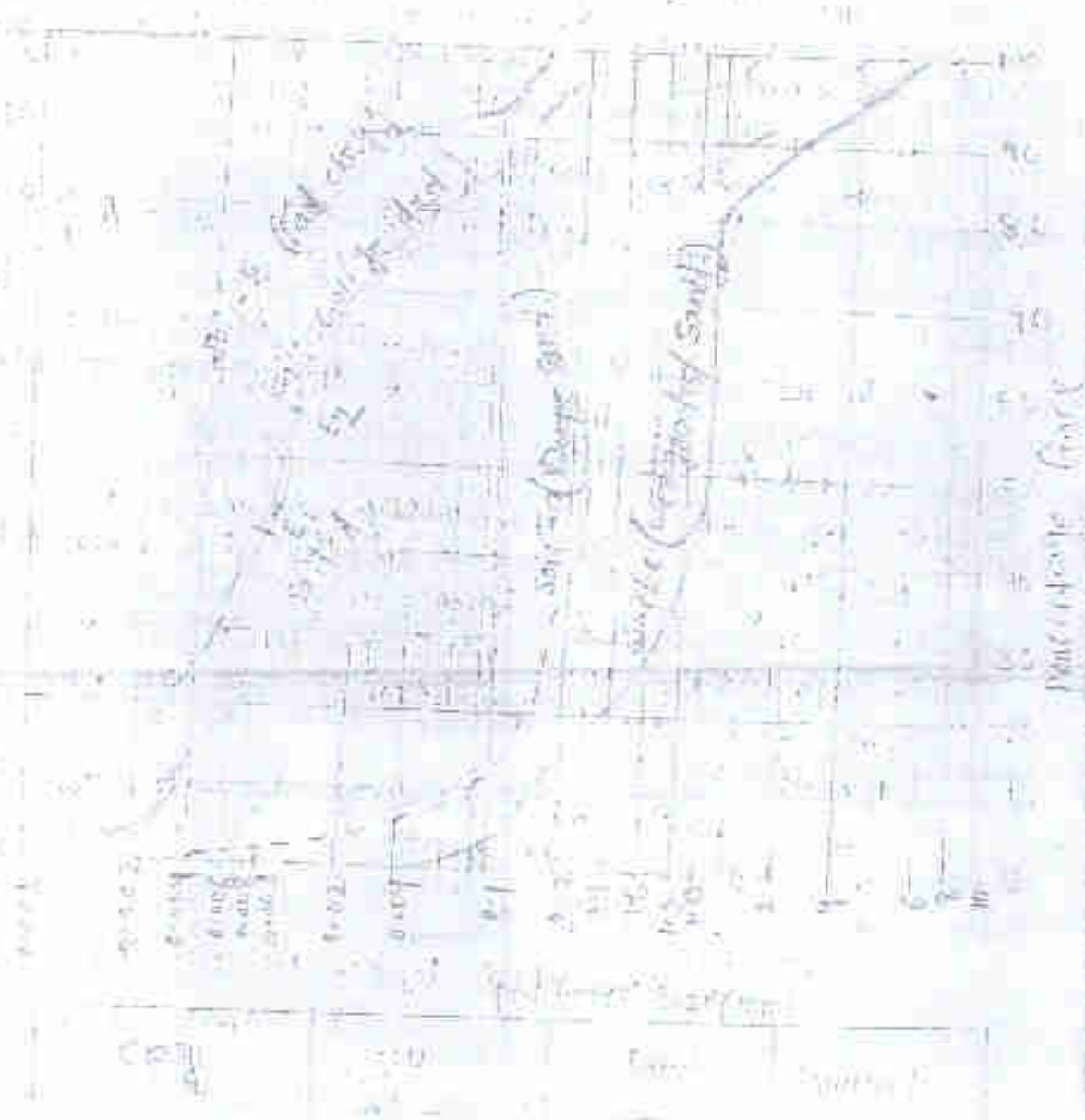
$$N = \frac{M \times G_s}{M \times G_s + 1000(G - 1)}$$

where N = percentage fines.

17 SEP 2020

PARTICLE SIZE DISTRIBUTION CURVE

Soil mechanics and foundations



(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 (%)
- A particle size distribution curve gives idea about different size of particles

of various soils

A soil may be either well graded or poorly graded.

→ well graded soil is the mix 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 & soil-C represent uniformly graded 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 graded 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.

(A) Co-efficient of uniformity or uniformity co-

efficient (C_u)

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

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.

(b) Co-efficient of curvature (C_u) :-

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

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 :- $1 < C_u < 5$

$C_u > 4$:- 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 (i) nature of clay mineral
(ii) amount of clay mineral.

= On the basis of "empirical" observed for a given soil, its liquid & plasticity index is directly proportional to the percentage of clay size fraction present in the sample (for the size

should less than 0.002 mm)

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

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

where I_p = plasticity index

C_w = % by weight clay size

Activity	Classification
(i) ≤ 0.75	Inertive
(ii) $0.75 - 1.40$	Medium
(iii) > 1.40	Active

18 Sep 2020

note (i) clay containing kaolinite slow down the activity (ii) has low activity :

(iii) clay containing montmorillonite higher the activity :

(iii) kaolinite - : 0.1 - 0.5

illite - : 0.5 - 1.0

montmorillonite - : 1.0 - 1.6

Q1 A natural soil sample has a bulk density of 29 kN/m^3 with by water content calculate the amount of water required to be added to the cube meter of soil to raise the water content to 15% while the void ratio remain constant. Also find out degree of saturation.

$\underline{\text{Sol}^n}$ $\omega_1 = 29\% = 0.29$ $\omega_2 = 15\%$
 $\omega_1 = 6\%$ $G = 2.67$

Stage-1 $V_{w1} = \frac{M_{w1}}{\rho_{w1}}$

$\omega_1 = 6\% = 0.06$

$\Rightarrow \frac{M_{w1}}{M_{d1}} = 6\%$

$\Rightarrow M_{w1} = 6\% \times M_{d1}$ ——— ①

$M_{d1} = S_{d1} \times V_{d1}$

$= 1.88 \times 1 \times 10^6$ [$S_d = \frac{\rho}{\rho_{w1}} = \frac{2}{1.88} = 1.06$]
 $= 1.88 \text{ gm} \times 10^6$ [$\rho_{w1} = 1.88 \text{ gm/cm}^3$]

By putting the value of M_{d1} in eqn ① we get

$M_{w1} = 6\% \times M_{d1}$
 $= 0.06 \times 1.88 \times 10^6$
 $= 112800 \text{ gm}$

$V_{w1} = \frac{M_{w1}}{\rho_{w1}} = \frac{112800}{1} = 112800 \text{ cm}^3$

Stage-2

$\omega_2 = 15\% = 0.15$

$V_{w2} = \frac{M_{w2}}{\rho_{w2}}$

$M_{w2} = 15\% \times M_{d2}$

($M_{d1} = M_{d2}$)

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

$= 282000 \text{ gm}$

$$V_{w2} = \frac{M_{w2}}{\rho_w} = \frac{282000}{1}$$

$$= 282000 \text{ cm}^3$$

Volume of water required to fill
the water (S(W)) & (W₂ - V_{w1})

$$= 282000 - 112800$$

$$= 169200 \text{ cm}^3$$

Degree of saturation :-

$$S_e = \frac{w G}{e}$$

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

$$\Rightarrow e = \frac{G S}{S_d} - 1$$

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

$$S = \frac{0.15 \times 267}{0.420} \times 100$$

$$= 95.357 \%$$

name	unit
Length	metre
Mass	Kilogram
Time	second
Electric current	ampere
Thermodynamic Temperature	Kelvin
Amount of substance	mole
Luminous intensity	candela
Plane angle	radian
Solid angle	steradian
Volume	liter
Density	Kilogram per cubic metre

Energy $\longrightarrow$ Joule
 Area $\longrightarrow$ square meter (m^2)
 Frequency $\longrightarrow$ hertz (s^{-1})
 Acceleration $\longrightarrow$ m/s^2
 Angular Acceleration $\longrightarrow$ rad/s^2
 Resistance $\longrightarrow$ ohm
 Inductance $\longrightarrow$ henry
 Force $\longrightarrow$ Newton
 Pressure $\longrightarrow$ pascal
 Sound $\longrightarrow$ Decibel
 Humidity $\longrightarrow$ Hygrometer
 Torque $\longrightarrow$ Newton meter
 Illuminance $\longrightarrow$ lux
 Depth of ocean $\longrightarrow$ fathometer
 Magnetic Flux $\longrightarrow$ Weber

new chapter

① ~~Particle size~~ classification of soil

- The purpose of soil classification is to arrange various types of soil into groups according to their engineering & agricultural properties.
- Soil possessing same characteristic is placed under same group.
- Soil survey & soil classification are carried out by several agencies or organisations.
- Soil may be classified into following types:-

→ this point is situated near clay, so that

1) Particle size classification - (diagram)

	0-5mm	5-10mm	10-20mm	20-60mm	60-200mm	200-600mm	600-2000mm
Clay	Silt	very fine sand	fine sand	coarse sand	very coarse sand	fine gravel	coarse gravel
		Sand					

[illegible]

International classification							
0-0.002		0.002-0.006		0.006-0.02		0.02-0.075	
Clay		Fine medium		Coarse fine		Medium coarse	
Silt				Sand			

(MIT Classification)

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

(ii) soil is fairly saturated

2. According to Darcy's law velocity of flow is directly proportional to the hydraulic gradient i.e. $[v \propto i]$ $\leftarrow$ eqn

v = velocity of flow

= discharge per unit time

Area of cross section

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



$$v \propto i$$

$$\Rightarrow v = Ki$$

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

$$\left[q = KiA \right]$$

Where K = coefficient of permeability

$$\Rightarrow q = Kih$$

$$q = K(h_1 - h_2) \frac{A}{L}$$

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

where dh = difference in head
head (unit m)

L = Length of soil sample

i = hydraulic gradient = $\frac{dh}{L}$ (unitless)

q = discharge per unit time

A = Area of cross-section

Soil type

k (cm/sec)

(1) Gravel : $1.00 > 1$

(2) Coarse sand : 1×10^{-2}

(3) Sand (medium) : 1×10^{-2} to 5×10^{-2}

(4) Fine sand : 5×10^{-2} to 1×10^{-3}

(5) Silty sand : 2×10^{-3} to 1×10^{-4}

(6) Silt : 5×10^{-4} to 1×10^{-5}

(7) Clay : 1×10^{-6} to 1×10^{-8} (smaller)

NOTE

unit of v = m/sec or cm/sec

q = m³/sec

A = m²

L = m

i = unitless

- The Darcy's law is a 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.

$$Q = VA = V_s A_v$$

$$\Rightarrow VA = V_s A_v$$

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

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

$$= V \cdot \frac{1}{\frac{V_0}{V}} \quad \left(n = \frac{V_0}{V} \right)$$

$$\Rightarrow V_0 = V \cdot n$$

$$\Rightarrow \boxed{V_0 = \frac{V}{n}}$$

$\Rightarrow$ Seepage velocity (V_0) is directly proportional to the hydraulic gradient.

$V_0 \propto i$

$$\Rightarrow \boxed{V_0 = K_p i}$$

Where K_p = co-efficient of permeability

$\Rightarrow$ From Darcy's law $\boxed{K_p = \frac{K}{n}}$

Factors affecting permeability :-

From Darcy's law $q = K i A$, we get

$$\boxed{K = \frac{Q}{A} \cdot \frac{L}{h} \cdot \frac{1}{i} \cdot \frac{1}{C}}$$

c) Effect of Size of particles:-

$\Rightarrow$ permeability varies approximately as the square of the grain size.

$\Rightarrow$ Allen Hazen based on his experiments on filter sands of particle size between 0.1 & 2mm found that the permeability can be expressed as:-

$$\boxed{K = C \cdot d_{10}^2}$$

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

d_{10} = effective diameter (cm) :

C = constant, (approx = 100)

② Effect of parameters is mentioned :-

→ 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 $\gamma \uparrow \rightarrow K \uparrow$

$\mu \downarrow \rightarrow K \uparrow$

ie
$$\frac{K_1}{K_2} = \frac{\gamma_{w1}}{\gamma_{w2}} \times \frac{\mu_2}{\mu_1}$$

$$K \propto \gamma \text{ \& } K \propto \frac{1}{\mu}$$

→ ALSO $K \propto T$

where T = Temperature

γ : unit weight

μ = viscosity

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

③ Effect of structural arrangement of fracture & discontinuity :-

→ The structural arrangement of faulting may vary, at the same void ratio depending upon the method of deposition @ compaction the soil mass.

→ The permeability parallel to the stratification is always greater than the perpendicular to the stratification.

② 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.

③ Degree of saturation & entrapped air & other foreign matter :-

→ The permeability is greatly reduced if air is entrapped in the voids, thus reducing the 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 & plug them up, thus ~~also~~ decreasing the permeability.

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

try
it
portion
change
etc

and of

of path
void, not
of of
the soil

(6) Specific Surface area :-

- ⇒ specific surface area is defined as the ratio of area to unit weight or area to unit volume.

$$SSA = \frac{\text{Area}}{\text{weight}} \quad \text{or} \quad \frac{\text{Area}}{\text{Volume}}$$

- ⇒ specific surface area is inversely proportional to the size & 'K' is directly proportional to size.

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

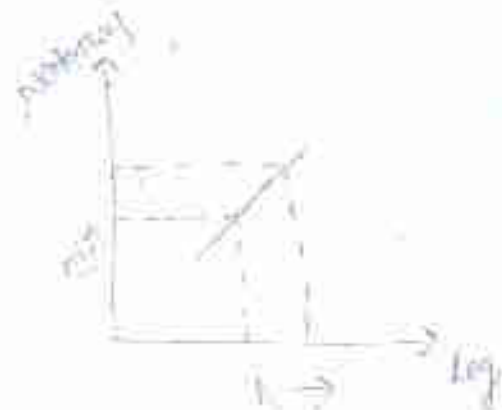
- ⇒ As more surface is contact with the water greater is the resistance & co-efficient of permeability is low.

(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.

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

$$\text{i.e. } e \propto \log K$$



(i) 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.

(ii) Coefficient 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, thickness of a liquid & its density.

→ There is some empirical formula for determination of coefficient of permeability one -

(1) Jaeky's

$$[K = 100 D_m^2]$$

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

(2) Allen Hazen's

$$[K = C D_{10}^2]$$

where C = constant (approx = 100 in cm)

(ii) Derzagili's formula :-

$$k = 300 \bar{d}_c^2 e^2$$

where $\bar{d}_c$ = effective grain size
 e = void ratio

(iii) Kozeny's formula

$$k = \frac{1}{K_R \eta s_s^2} \times \frac{e^3}{1+e}$$

where s_s = specific surface of particles
 $(\text{cm}^2/\text{cm}^3)$

η = viscosity $(\text{g} \cdot \text{sec}/\text{cm}^2)$

K_R = constant (equal to 5 for spherical particle)

(iv) Logden's formula :-

$$\log_{10}(k \eta) = a + b \eta$$

where a, b = constant i.e. 1.363×10^{-5} at 10°C

η = porosity

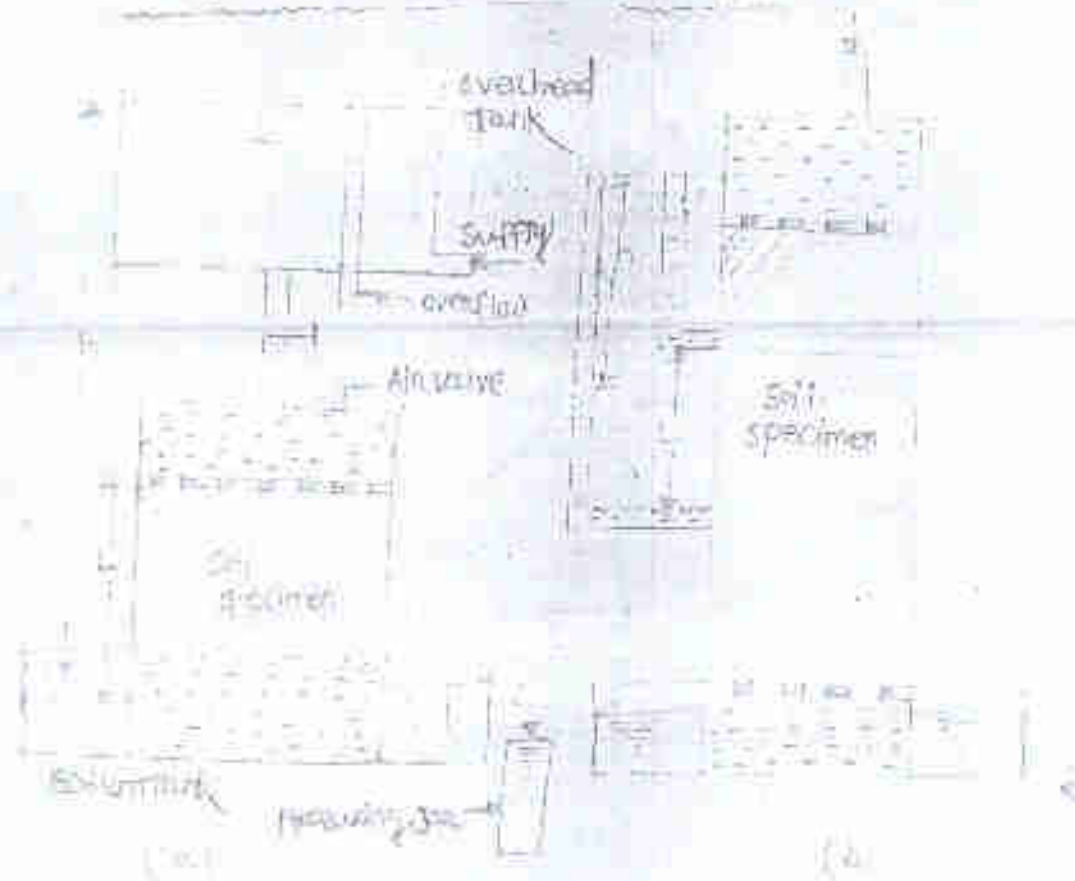
Sept 2020

Determination of Coefficient of permeability by Laboratory method:-

There are 2 methods of determining the k that are (i) constant head permeability test

(ii) falling head permeability test.

(i) Constant head permeability test:-



Water flows from the overhead tank, existing it strikes the inlet tube, overflow tube & the outlet tube.

The constant hydraulic gradient $\frac{h}{L}$ causing the flow is the head h i.e. it is difference 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 -

$$Q = \frac{q}{L} = KA \Rightarrow \frac{Q}{A} = k \frac{h}{L}$$

$$\Rightarrow K = \frac{Q}{A} \times \frac{L}{h}$$

$$\Rightarrow K = \frac{Q \times L}{A \times h}$$

where A = Cross-sectional area of the sample.

t = time interval

Q = total quantity of water @ flow

L = length of the sample

h = head

(2) For a head permeability test



1 by
3th
effy

→ Constant head permeability test is used for water saturated soil using 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 permeation & 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.

of the
en (2)

→ 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 t & $-dh$ be the change in the head in a smaller time interval dt .

→ Hence, from Darcy's law, the rate of flow $q = 1.375$

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

by integrating the equation we get

$$K = \frac{a}{At} \log \frac{h_1}{h_2} = 2.3 \frac{a}{At} \log_{10} \frac{h_1}{h_2}$$

where a = Stand pipe area

A = Area of sample

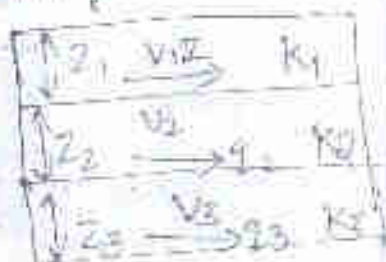
t = time interval is $t_2 - t_1$

h_1 = head at time t_1

h_2 = head at time t_2

Average permeability is defined as the
bedding plane:-

Let $z_1, z_2, \dots, z_n$ = thickness of layer
 $k_1, k_2, \dots, k_n$ = permeability
of layer



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

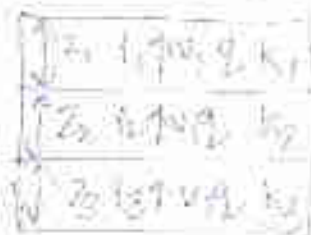
→ Avg permeability of soil:-

$$k_a = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3 + \dots + k_n z_n}{Z}$$

where $Z = z_1 + z_2 + z_3 + \dots + z_n$

→ Average permeability is perpendicular to the
bedding plane.

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



→ Head loss through each layer is
different.

⇒

Avg permeability (k_{av})

$$= \frac{Z}{\frac{Z_1}{k_1} + \frac{Z_2}{k_2} + \dots + \frac{Z_n}{k_n}}$$

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

Ex 2025

Q4

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

Ans

parallel to the bedding

Plane :-

$\frac{1}{4}Z$	$k_1 = k$
$\frac{1}{4}Z$	$k_2 = \frac{k}{3}$
$\frac{1}{4}Z$	$k_3 = \frac{k}{2}$
$\frac{1}{4}Z$	$k_4 = 2k$

$$K = \frac{k_1 \bar{z}_1 + k_2 \bar{z}_2 + k_3 \bar{z}_3 + k_4 \bar{z}_4}{Z}$$

$$= \frac{kZ + \frac{1}{3}kZ + \frac{1}{2}kZ + 2kZ}{4Z}$$

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

$$= \frac{k(5 + \frac{2}{3} + \frac{2}{3} + 6)}{4} = \frac{23k}{24}$$

$$= \frac{23k}{24}$$

(b) Permeability to the bedding planes:

$$K = \frac{Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3} + \frac{Z_4}{K_4}}$$

$$= \frac{42}{\frac{2}{K} + \frac{62}{3K} + \frac{42}{4K} + \frac{2}{2K}}$$

$$= \frac{42}{\frac{22 + 62 + 42 + 2}{2K}}$$

$$= \frac{842}{132}$$

$$= \frac{84}{13}$$

7 Oct 2011

26 Calculate the coefficient of permeability of a soil sample 60 cm in height & 50 cm² cross sectional area. If the quantity of water equal to 420 ml passed down in 10 minutes under an effective constant head of 40 cm. On overdrying the soil 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

$$\text{Seepage velocity } (v_s) = \frac{Q}{A}$$

Q = Porosity

v = Total velocity (average)

$$t = 10 \text{ minutes} = 600 \text{ sec}$$

$$Q = 420 \text{ ml}$$

$$G_s = 2.65$$

$$A = 50 \text{ cm}^2 \quad L = 6 \text{ cm} \quad h = 40 \text{ cm}$$

$$M_d = 998 \text{ g/cm}^3$$

$$v = ki$$

$$\text{where } k = \frac{eL}{A\eta i}$$

$$i = \frac{430 \times 6}{50 \times 40 \times 500} = 2.15 \times 10^{-3} \text{ cm/sec}$$

$$e = \frac{A}{L} = \frac{h}{L} = \frac{40}{6} = 6.6$$

$$v = ki = 2.15 \times 10^{-3} \times 6.6 = 1.43 \times 10^{-2} \text{ cm/sec}$$

$$\eta = \frac{F}{LTe}$$

$$e = \frac{GJ\omega}{T\theta} = 1 \quad (J\omega = 1)$$

$$\theta = \frac{M_d}{V} = \frac{998}{50 \times 5} = 1.56 \text{ g/cm}^3$$

$$e = \frac{GJ\omega}{T\theta} = 1 = 0.596$$

$$\eta = \frac{F}{LTe} = \frac{0.596}{140 \times 596} = 0.373$$

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

$$= 0.0383 \text{ cm/sec}$$

$$= 0.383 \times 10^{-2} \text{ cm/sec}$$

Ex. To determine the permeability of a material by the steady state method. A cylindrical specimen of length 6 cm and cross-sectional area 50 cm² is placed in a desiccator. The desiccator is filled with a gas of known pressure. The specimen is exposed to the gas for a certain time and the change in weight is measured. The permeability of the material is then calculated.

Given data :- $h_1 = 40 \text{ cm}$

$$h_2 = 40 \times 10^{-5} = 350 \mu$$

$$a = 0.5 \text{ cm}^2, \quad t = 10 \text{ min} = 600 \text{ sec}$$

$$L = 6 \text{ cm}, \quad K = 50 \text{ cm}^2$$

$$K = 0.3 \times \frac{a}{L} \log_{10} \frac{h_1}{h_2}$$

$$= 0.3 \times \frac{0.5 \times 6}{50 \times 600} \times \log_{10} \frac{40}{35}$$

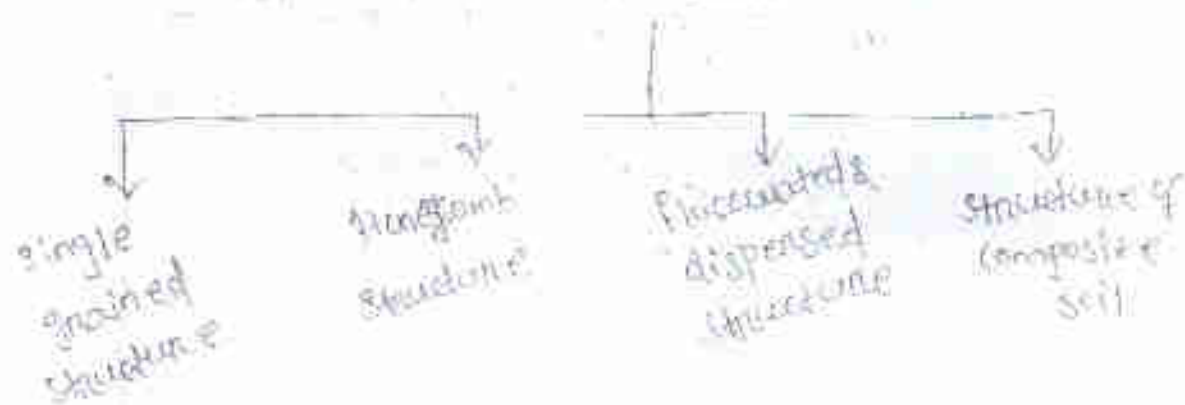
$$1.53 \times 10^{-5} \text{ cm/sec}$$

8 Oct 2021

Soil Structure

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

Types of soil structure



(i) Single grained structure -

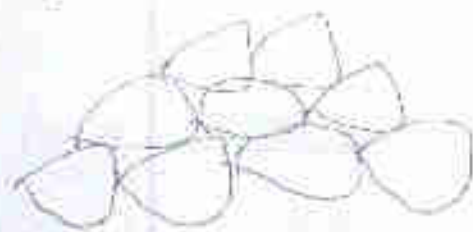
> This type of structure will be found in case of coarse grain soil.

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

> The major force causing their deposition is gravitational & the surface forces.

> There will be particle-to-particle contact in the deposit.

> 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 neighbouring particles leading to the formation.

> The combination of a no. of of this type of particles leads to honeycomb structure.

> It has high void ratio.

(3) 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 particles 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 particles so that it is less permeable.

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

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

⇒ This type of soil is commonly for water depositing.

Structure of Composite Solids =



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

(i) Coarse grained skeleton structures:-

* The coarse grained skeleton structure can be found in case of composite solids 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 structures:-

* Cohesive nature 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 enclosed in fine-grained matrix & 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.

and
rest of
power

Notes
cont.
ng
very
the

⇒ It is broadly divided into 2 types:-

- (i) Free water or gravitational water
- (ii) Held water

(i) Free water - water is 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 within a soil mass by soil particles.

⇒ It is again divided into 3 types:-
(a) Structural water:-

⇒ Structural water is the water chemically combined or 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 or by overdrying of temperature 110°C .

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

(b) Adsorb water:- (adsorb in jagiti khaba)

⇒ Adsorb water is the water which is held by some granular soil particles due to electrical chemical forces of adhesion.

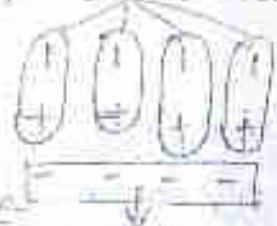
applied
power

known

→ 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 it. water molecule

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



→ 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 and subbed 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

① any opposition due to any external forces like gravity.



$$\text{height of capillary rise} = \frac{2\sigma \cos \theta}{\rho g r}$$

where σ = surface tension

⇒ Generally capillary rise in natural soil suggested by Allen Hazen is:-

$$h_c = \frac{c}{e d_{10}}$$

where h_c = capillary rise in cm

e = void ratio

d_{10} = effective diameter of soil in mm

c = constant (taken between 0.1 to 0.5)

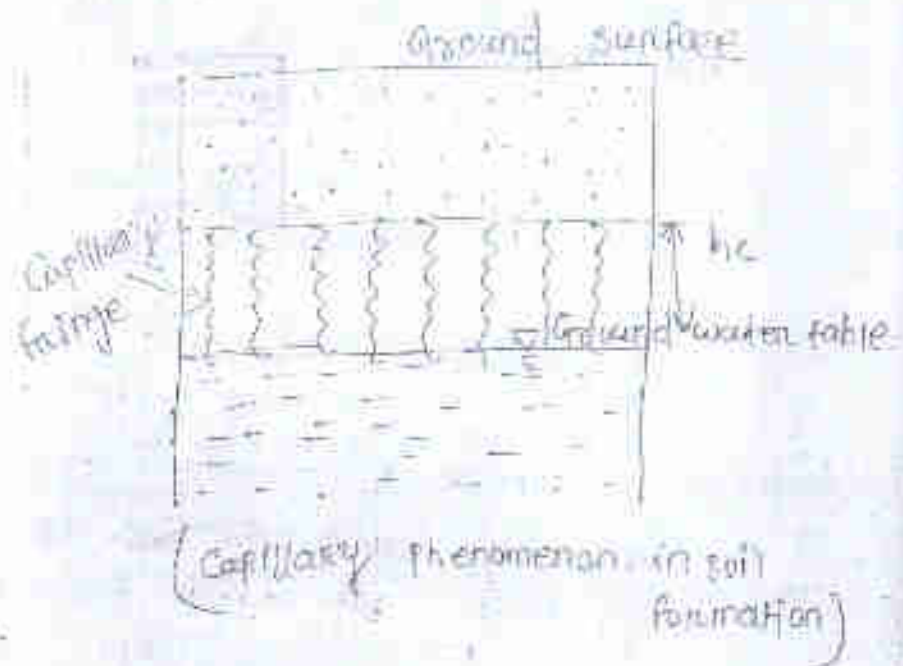
* Capillary phenomenon in soils:-

⇒ The voids in natural soil formations act as capillary tubes & water rises in the continuous voids to a certain height above ground water table @ 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 deposit than that in a coarse grained soil deposit, as the size of voids is much less.

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



10 Oct 2020

Effective stress = pore pressure and Total stress:-

→ Total 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 or 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.

$$u = \gamma_w h_w$$

Where γ_w = unit weight of water

h_w = piezometric head or height of water.

Effective stress = Total stress - pore water pressure
seepage pressure

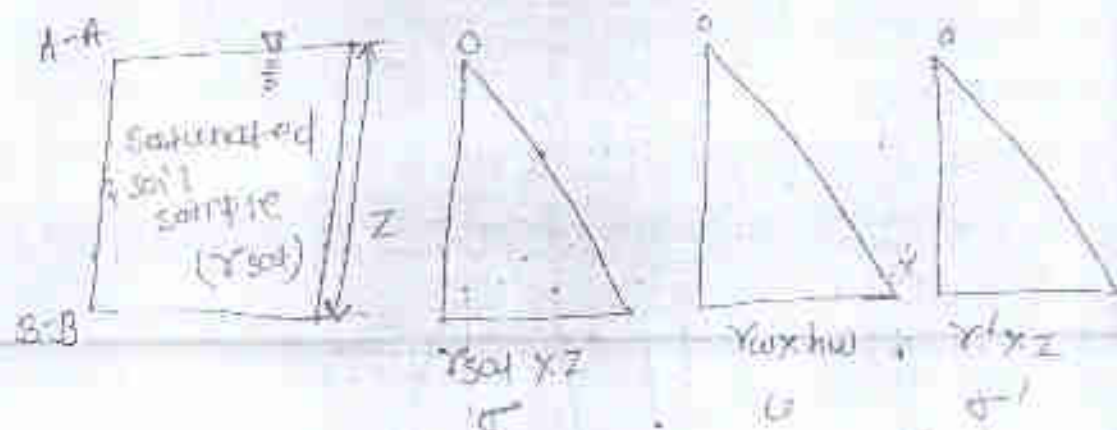
Total Stress

It is denoted by σ

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

$$\sigma = \sigma' + u$$

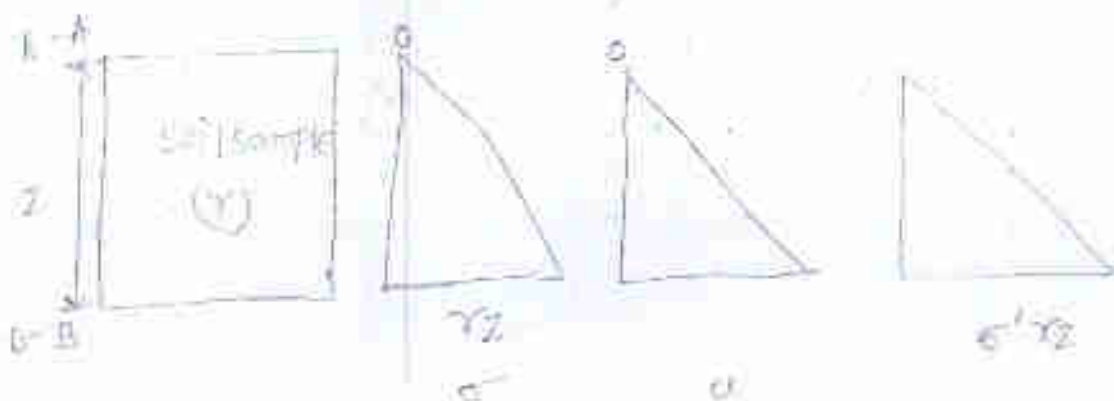
CASE - 1 Submerged soil mass with water table at the surface



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

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

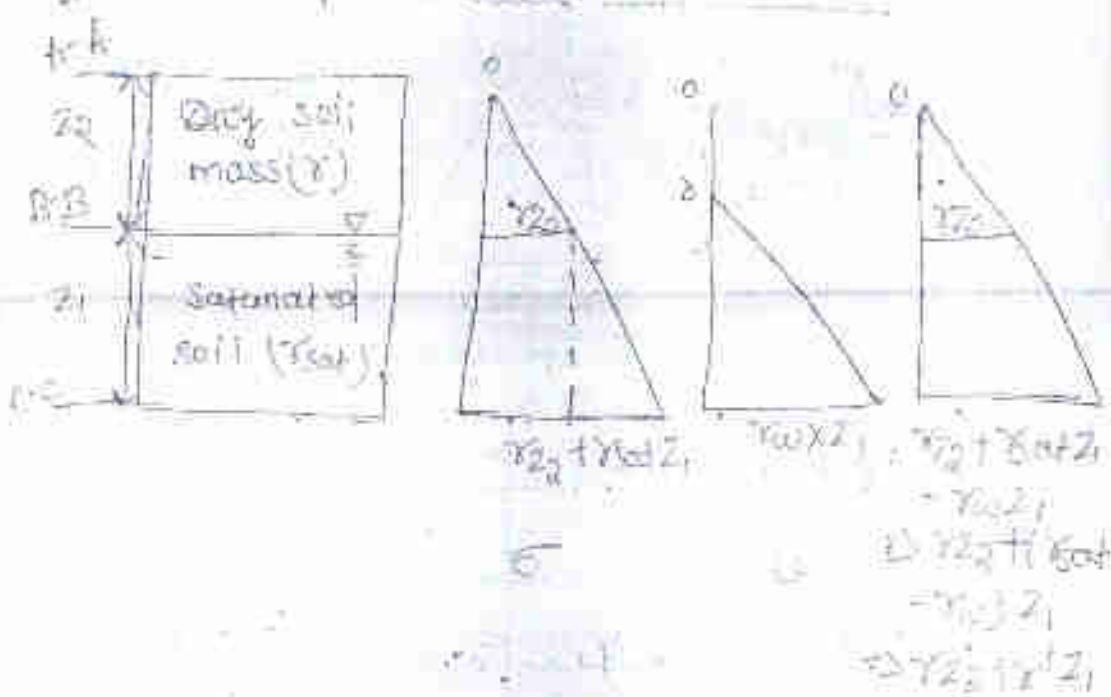
CASE 2 Only soil mass



Layer	σ	u	σ'
A-A	0	0	0
B-B	γ_2	0	γ_2

Example 10 12 Oct 2020

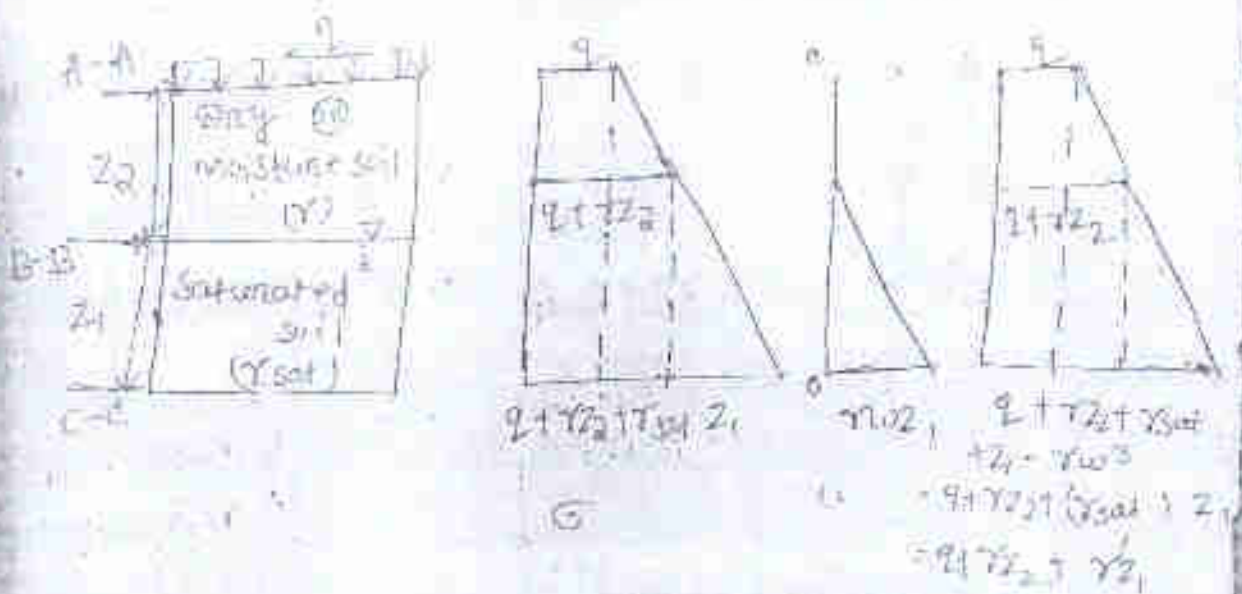
Partially submerged soil mass:-



Layer	σ	u	σ'
A-A	0	0	0
B-B	γ_2	0	γ_2
C-C	$\gamma_2 + \gamma_{sat} Z_1$	$\gamma_w Z_1$	$\gamma_2 + (\gamma_{sat} - \gamma_w) Z_1$

Case - IV

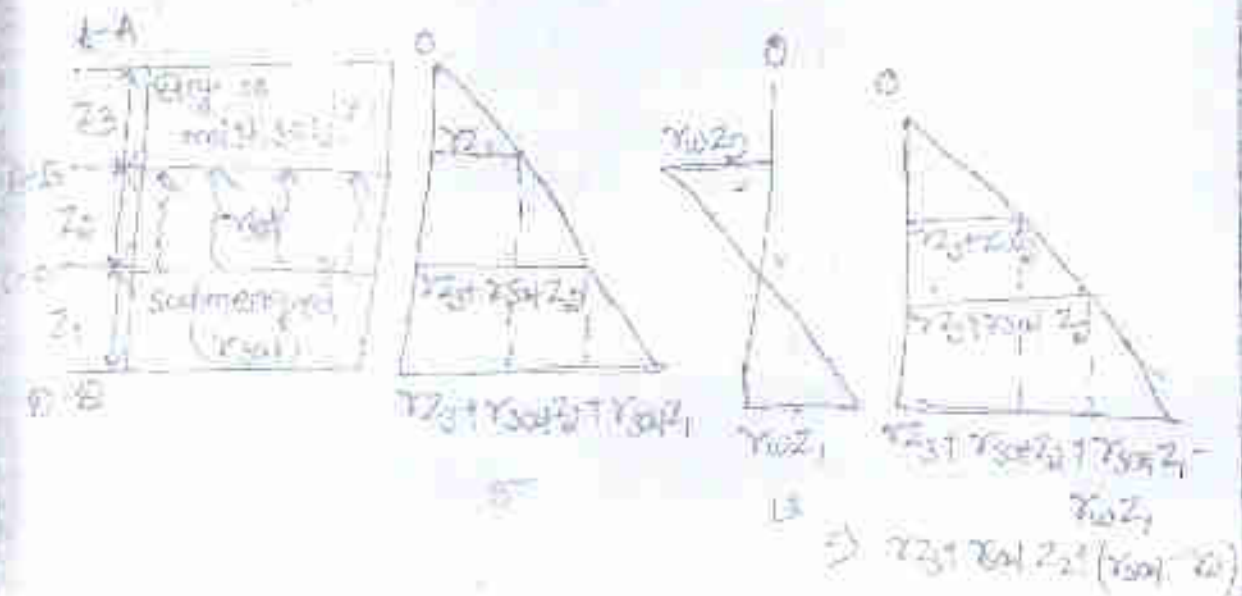
Rectangular section of soil with surface water



Layer	A	B	C
A-A	q	0	q
B-B	$q + \gamma_2 z_2$	0	$q + \gamma_2 z_2$
C-C	$q + \gamma_2 z_2 + \gamma_{sat} z_1$	$\gamma_w z_1$	$q + \gamma_2 z_2 + (\gamma_{sat} - \gamma_w) z_1$ $= q + \gamma_2 z_2 + \gamma_2 z_1$

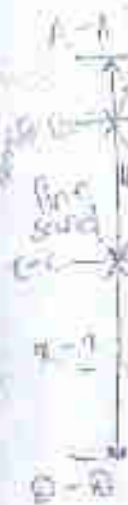
18 oct 2020

Case - V : Soil with capillary fringe

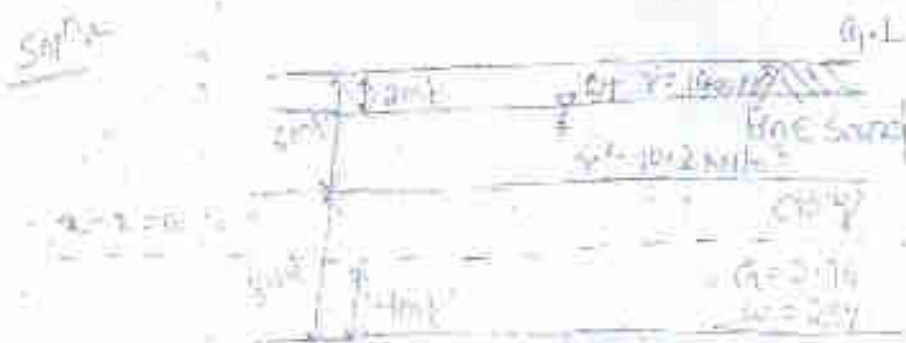


2) $\sigma_{v2} + \gamma_{sat} z_2 + \gamma' z_1$

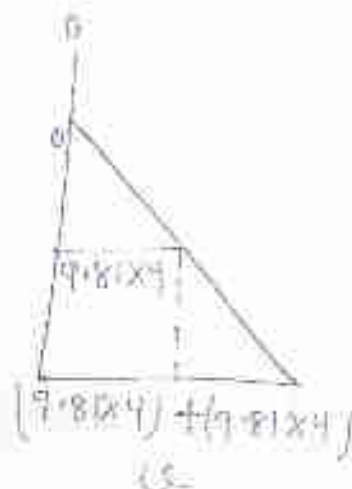
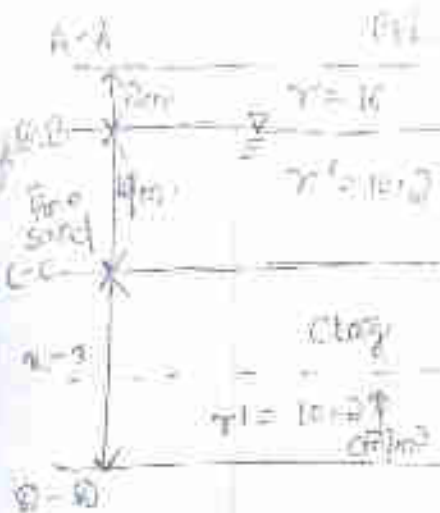
Layer	σ	u	σ'
A-A	0	0	0
B-B	γz_2	$-\gamma_w z_2$	$\gamma z_2 + \gamma_w z_2$
C-C	$\gamma z_2 + \gamma_{sat} z_2$	0	$\gamma z_2 + \gamma_{sat} z_2$
D-D	$\gamma z_2 + \gamma_{sat} z_2 + \gamma' z_1$	$\gamma_w z_1$	$\gamma z_2 + \gamma_{sat} z_2 + \gamma' z_1 - \gamma_w z_1$



16) A clay stratum of thickness 8m is located at a depth of 6m below the ground surface. It is overlain by fine sand. The water table is located at a depth of 2m below ground surface. For fine sand submerged unit weight is 10.2 kN/m^3 . The moist unit wt of sand located above water table is 16 kN/m^3 . For clay layer $G = 2.75$ & $w = 25\%$. Compute the effective stress at the middle of clay layer.



17 Oct 2020



$$\sigma_{1-2} = \sigma_{1-2} = \left[(16 \times 2) + (10.2 \times 4) + (18.6 \times 4) \right] - \left[(9.81 \times 4) + (9.81 \times 4) \right]$$

$$113.64 \text{ kN/m}^2$$

with full saturation $sr = 1$

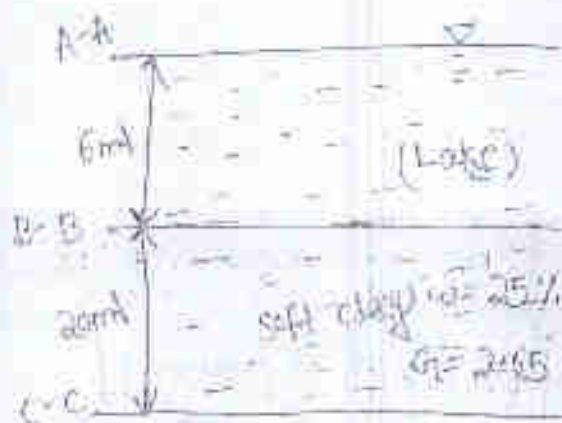
$$sr \times e = w \times G$$

$$e = \frac{w \times G}{sr} = \frac{0.25 \times 2.76}{1} = 0.69$$

$$\gamma' = \frac{[G(1-e)] \times \gamma_w}{1+e} = \frac{2.76(1-0.69)}{1+0.69}$$

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

20. Compute the total, effective & pore water pressure at a depth of 2m below the bottom of a lake 6m. The bottom consists of soft clay with a thickness 2m. The average water content of clay is 35% & the specific gravity of the soil is assumed 2.65?



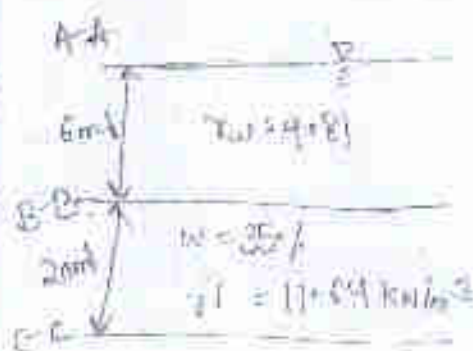
$$\gamma_{sat} = \frac{(G + e) \gamma_w}{1 + e}$$

$$Se = w G [S = 1]$$

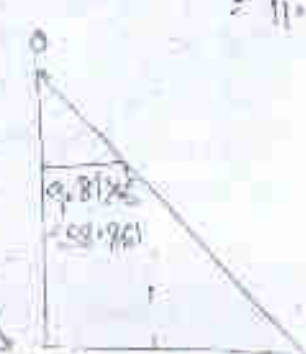
$$\begin{aligned} e &= w G \\ &= 0.35 \times 2.65 \\ &= 0.93 \end{aligned}$$

$$\gamma' = \frac{(G - 1) \gamma_w}{1 + e} = \frac{(2.65 - 1)}{1 + 0.93}$$

$$= 11.64 \text{ kN/m}^3$$



$$\begin{aligned} \sigma &= \gamma_{sat} \times z \\ &= 291.66 \end{aligned}$$



$$\begin{aligned} u &= \gamma_w \times z \\ &= 255.06 \end{aligned}$$



$$\sigma' = 36.6$$

Layers

A-A

B-B

C-C

σ

0

57.86

291.66

u

0

57.86

255.06

σ'

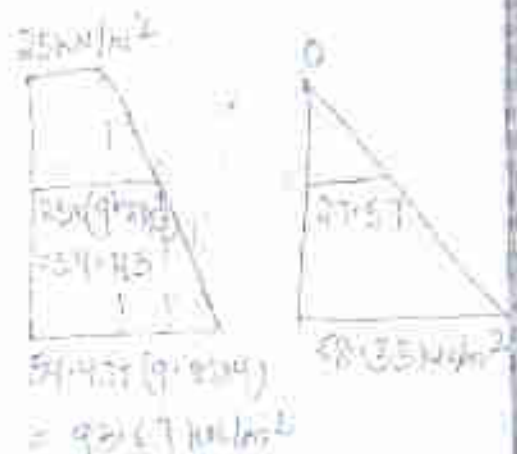
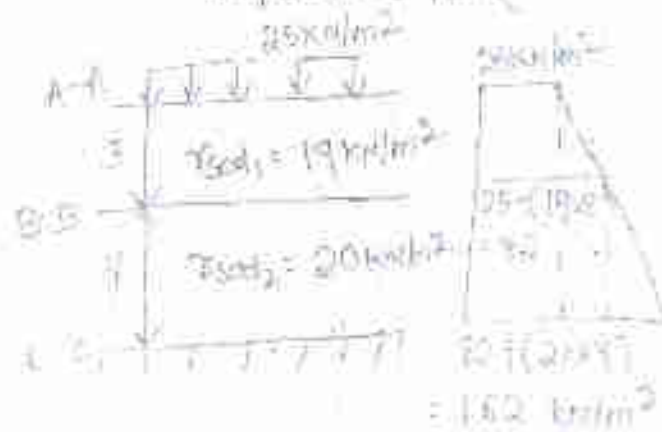
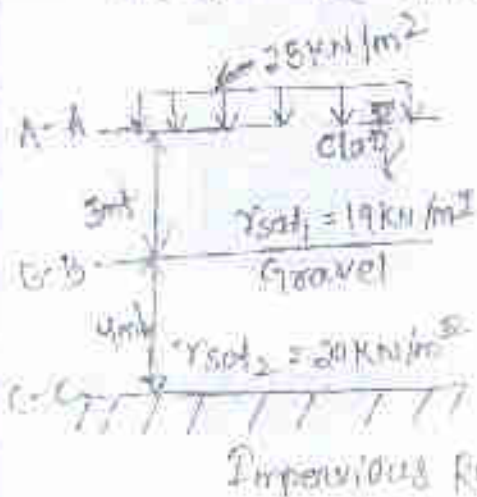
0

0

36.6

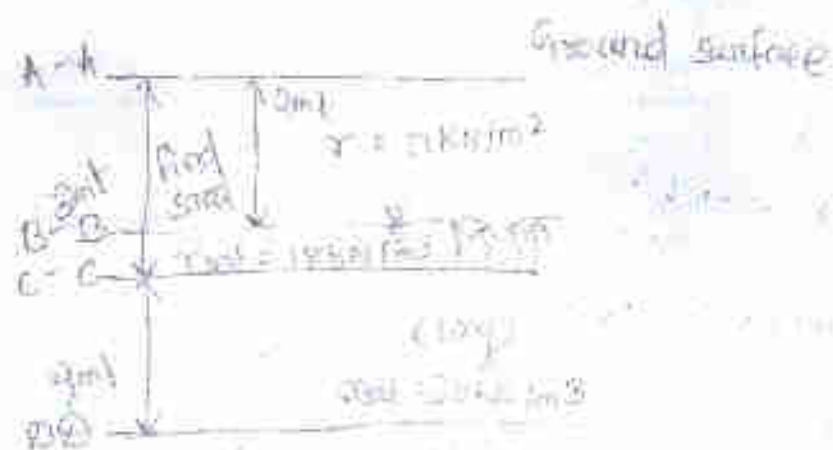
Hence the total stress, effective stress & pore water pressure at the bottom of the soft clay layer is 291.66 kN/m^2 & 36.6 kN/m^2 & 255.06 kN/m^2 .

36. At a construction site 3 m thick clay layer is followed by a 4 m thick gravel layer which is resting on impervious rock. A load of 25 kN/m^2 is applied suddenly at the surface. The saturated unit weight of the soil are 19 kN/m^3 & 20 kN/m^3 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?



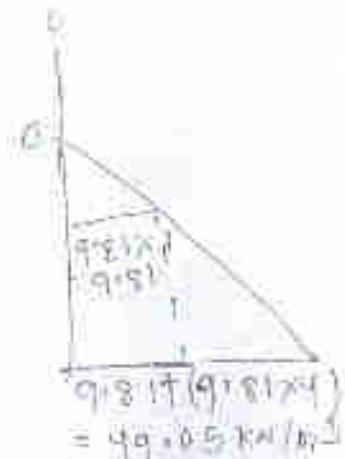
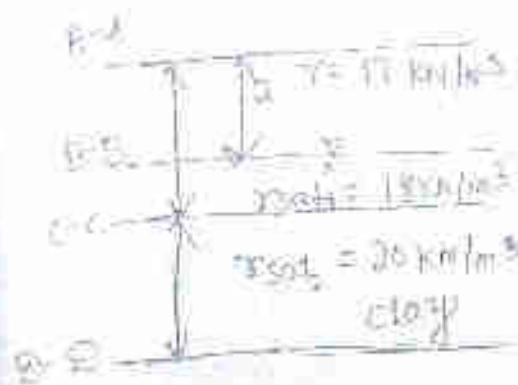
Layers	5'	13'	5'
A-A	25	25	0
B-B	80	54.53	27.51
C-C	160	93.67	68.33

23 Oct 2020



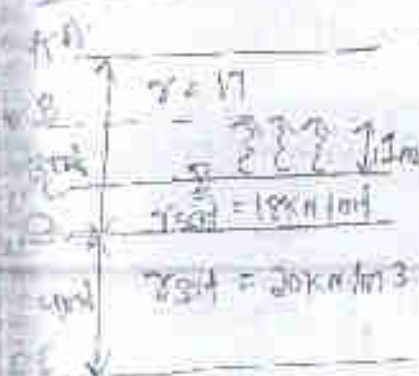
Find out the different ineffective stress at 7m below the ground surface when in and out of capillary occur from water table to 2m height.

Case-1

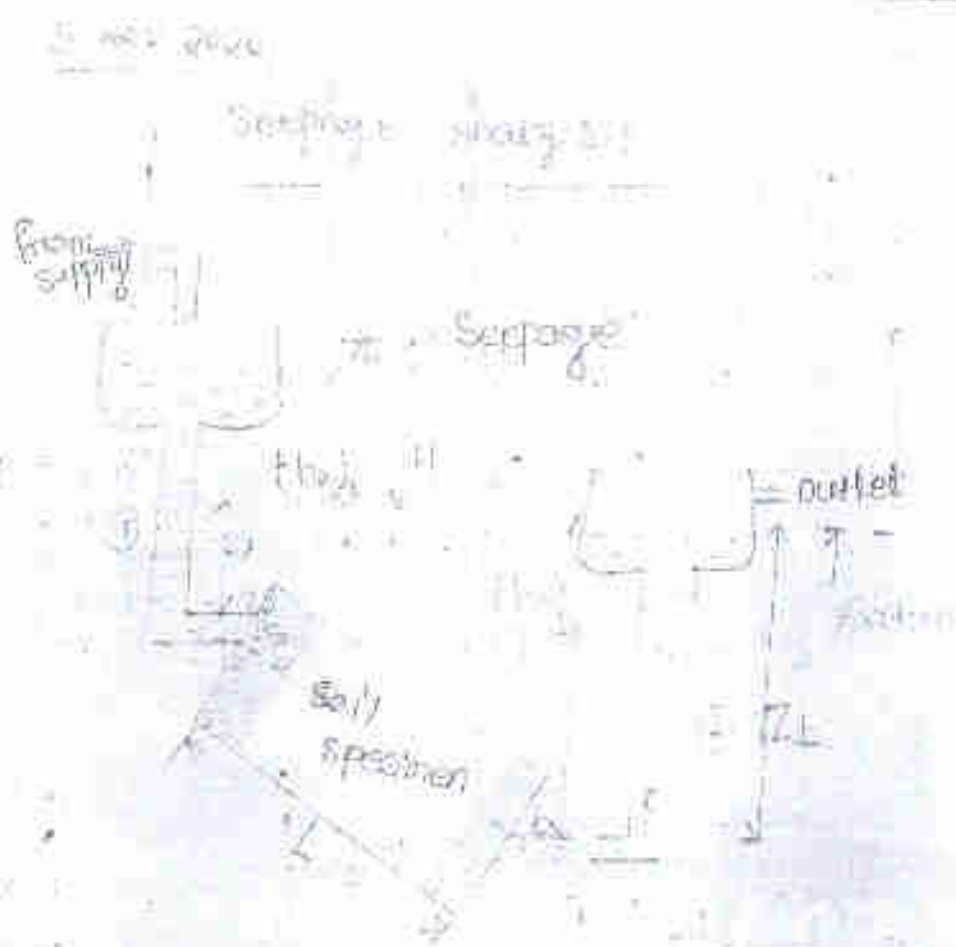


$$\sigma' = 132 - 39.24 = 92.76 \text{ kN/m}^2$$

Case-2



$$\text{difference in effective stress} = 92.76 - 74.14 = 18.62 \text{ kN/m}^2$$



→ 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 soil mass in which the flow is taking place is the algebraic sum of pressure head (h_w) and datum head (z)

$$h = h_w + z$$

$h_w \rightarrow$ water height

$z \rightarrow$ datum height

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

Datum line above filter (true)

Datum line above filter (true)

$$\text{Total } h_a = (h_w)_a - z_a$$

$$\text{Total } h_b = (h_w)_b - z_b = 0$$

⇒ In the above figure :-

$$\text{Total head at a, } h_a = (h_w)_a - z_a$$

$$\text{Total head at b, } h_b = (h_w)_b - z_b = 0$$

∴ No loss of head between a & b

$$H = h_a - h_b$$

$$= (h_w)_a - z_a - 0$$

$$= (h_w)_a - z_a$$

Example 2.20

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

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

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

Seepage pressure :-

⇒ 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 transmitted effected

Due to frictional drag between water & soil particles.

⇒ The seepage pressure at any point in the soil mass is:

$$P_s = \gamma_w \times h$$

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

$$P_s = \gamma_w h$$

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

$$\Rightarrow P_s = \gamma_w \times \frac{1}{2} \times z \quad \left[i = \frac{h}{L} \right]$$

$$\Rightarrow P_s = i \gamma_w z$$

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

$$J = P_s A$$

where J = seepage force

P_s = seepage pressure

A = cross-sectional area

$$\Rightarrow J = i \gamma_w z A$$

⇒ 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.

$$\sigma' = \sigma \pm u$$

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

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

NOTE - flow occurs in downward direction
+ve sign is used & in upward
direction -ve sign is used

Quick sand condition - 9 Nov 2020

→ 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 become
equal to the downward pressure due
to submerged weight of soil.

→ When this happens in case of a fine
coarse 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 particles
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 another
name for quick condition.

→ This quick condition occurs when -

$$\tau' = 0$$

$$\Rightarrow \tau' / z - \rho_s = 0$$

$$\Rightarrow \tau' / z = \rho_s$$

$$\Rightarrow \tau' / z = i \rho_w z \quad (i = \text{critical})$$

$$\Rightarrow \tau' / z = i \rho_w z$$

$$\Rightarrow i_c = \tau' / z \rho_w$$

$$\Rightarrow i_c = \tau'$$

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

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

from
eqn

→ 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 is gradually increased until quick condition is reached in the soil.

→ At this condition upward force of the water on soil mass becomes equal to downward force due to weight of soil above first level.

⇒ If A is the cross sectional area of well then we have:

$P_1 = P_2 \Rightarrow$ quicksand condition.

$$\Delta P_A = P_2 A$$

$$\Delta P, T_w \times h \times A + T_w \times 2 \times A = T_{sat} \times 2A$$

$$\Rightarrow T_w h = T_{sat} \times 2A - T_w \times 2A$$

$$\Rightarrow T_w h = (T_{sat} - T_w) \times 2A$$

$$\Rightarrow h = \frac{T_{sat}}{T_w} \times 2A$$

$$\Rightarrow \frac{h}{2} = \frac{T_{sat}}{T_w}$$

$$\left[i.e. = \frac{h}{2} \right]$$

⇒ Hence we get :-

$$\left[\frac{i.e. \times T_{sat}}{T_w} = \frac{G \times 1}{i.e.} = \frac{h}{2} \right]$$

NOTE

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

$i.e.$ will become unity ($i.e. > 1$)

(b) For most coarse less soil $i.e.$ will be less than unity ($i.e. < 1$)

EXERCISE

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

$$\boxed{v = K i.c.} \quad i.e. \propto K$$

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

Solⁿ Data given $G = 2.67$

$$e = 0.7$$

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

Ex A soil sample taken in a laboratory & placed in a container has a length of 12.5 cm. The coefficient of permeability of soil is 1.5×10^{-3} cm/sec with void ratio of 0.7. Water is made to flow through the soil sample in parallel direction & the rate of flow is 0.04 cm³/sec. Assume the surface area of the bottom a constant surface as the water flows. Find $G = 2.67$.

Solⁿ Data given $G = 2.67$

$$q = 0.04 \text{ cm}^3/\text{sec}$$

$$\text{Container diameter} = 8 \text{ cm}$$

$$\text{Container length} = 12.5 \text{ cm}$$

$$e = 0.7$$

$$K = 1.5 \times 10^{-3} \text{ cm/sec}$$

$$\sigma' = \sigma'_{1/2} \pm p_s$$

$$\sigma'_{1/2} = \sigma'_{1/2} - p_s$$

$$\sigma'_{1/2} = 12 \text{ cm}$$



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

$$= 9.52 \text{ kN/m}^3$$

From Darcy's law $q = k i A$

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

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

$$\Rightarrow i = 0.53$$

The effective stress at middle of the section

$$\text{i.e. A-A layer } \sigma' = \gamma' z - 12 \gamma_w$$

$$= \left[(9.52) \times (0.0625) \right] -$$

$$\left[0.53 \times (0.0625) \times 9.81 \right]$$

$$= 0.27 \text{ kN/m}^2$$

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

$$\sigma' = \gamma' z - 12 \gamma_w$$

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

$$= 0.54 \text{ kN/m}^2$$

Quick Sand Condition

Upward condition in down
ward condition is same here
Quick sand condition occurs
here.

* Definition :- 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 line of joining points of equal potential 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 flow net :-

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

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

(2) the rate of flow through such flow channel is same.

(3) the same potential drop occurs between two successive equipotential lines.

(4) In a homogeneous soil, every transition in the slope of the ϕ 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 retain the salient features in mind.

(2) About 10-15 flow channels should be sufficient for the first trial, as too many flow channels will distract attention from essential features.



(iv) After initial sketching the flow net should be observed at a whole while adjusting the finer details.

(v) All transitions should be made such 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 gradients.

(i) Quantity of seepage :-

The q : rate of discharge through each flow channel.

$$q = \text{head drop per field} = \frac{H}{N_d}$$

H = Head causing flow through entire flow net or quantity of seepage.

$$q = kH \frac{H}{N_f}$$

Where k = coefficient of permeability.

$$p = h \gamma_w$$

$$\gamma_w = 9.81$$

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

$$H = n \Delta h$$

$$\Delta h = \frac{H}{n}$$

$$n = 3$$

(ii) Determination of seepage pressure at any point :-

→ Seepage pressure at any point

$$p_s = h \gamma_w$$

where h = total head at that point
 $= (H - n \Delta h)$

Δh = potential head drop per

$$\text{field} = \frac{H}{N_f}$$

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

$$u = h' \gamma_{sat}$$

where h_{avg} = hydraulic head $\pm (h/2)$

h = total head of that point
is $(p = \rho gh)$

z = datum head of that point

(iv) Exit gradient =

$$i_{exit} / i_e = \frac{zh}{\Delta L_e}$$

where zh = 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 impermeable stratum. The impermeable stratum lies at a depth of 12 m below the ground surface. A sheet pile wall penetrates 3 m into the permeable soil stratum. Water is a height of 5 m on left side & 12 m on right side. Find the flow rate in direction.

7-7)

(i) quantity of seepage

(ii) The seepage pressure at a point 'p' located 8 m below the surface of soil stratum & 9 m away from the sheet pile.

(iii) pore pressure at point 'p'

(iv) Maximum exit gradient.

Solⁿ (i) quantity of seepage (q) = $kH \frac{NF}{NS}$

$$H = H_1 - H_2$$

$$= 9 - 1.5 = 7.5 \text{ m}$$

$$NF = 4$$

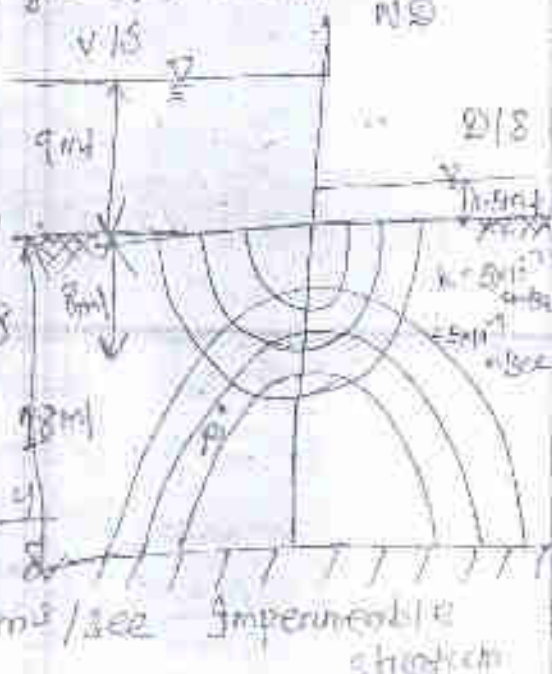
$$NS = 8$$

$$K = 5 \times 10^{-3}$$

$$= 5 \times 10^{-3} \times 7.5 \times \frac{4}{8}$$

$$= 1.875 \times 10^{-2} \text{ m}^3/\text{sec}$$

$$= 18.75 \times 10^{-3} \text{ m}^3/\text{sec}$$



(ii) Seepage pressure at point 'p'

$$P_s = \gamma_w h$$

$$h = H \cdot \frac{m}{n}$$

$$H = 7.5 \text{ m}$$

$$n = \frac{H}{h} = \frac{7.5}{8} = 0.937 \text{ m}^3/\text{sec}$$

$$h = (7.5 - 2.5 \times 0.937)$$

$$= 5.16 \text{ m}$$

$$P_s = h_{sao} = 5.16 \times 9.81$$

$$= 50.62 \text{ kN/m}^2$$

(iii) hydrostatic or pore pressure at point = $h - z$

$$h_{so} = h - z = 5.16 - (-9.5)$$

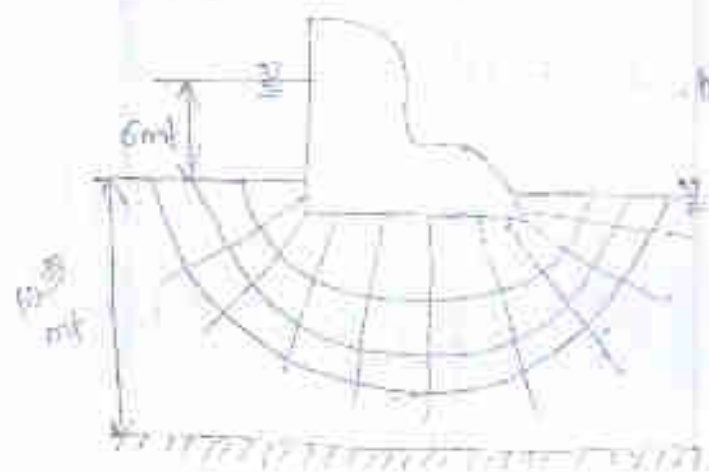
$$= 14.66 \text{ m}$$

$$p_s = 14.66 \times 9.81$$

$$= 143.81 \text{ kN/m}^2$$

(iv) $\frac{0.937}{2.8} = 0.334 \text{ m/sec}$

Q3 A soil stratum with permeability $K = 5 \times 10^{-8} \text{ cm/sec}$ overlies an impermeable stratum. The impermeable stratum lies at a depth of 15 m below the ground surface. A sand pile wall penetrates 8 m into the permeable soil stratum where stands to a height of 9 m on its side & find or draw a sketch the flow net to determine



$$K = 5 \times 10^{-8} \text{ cm}$$

Soln

$$q = KH \times \frac{NF}{NB}$$

$$NF = 4$$

$$NB = 12$$

$$KH = \frac{NF}{NB}$$

$$H = H_1 - H_2$$

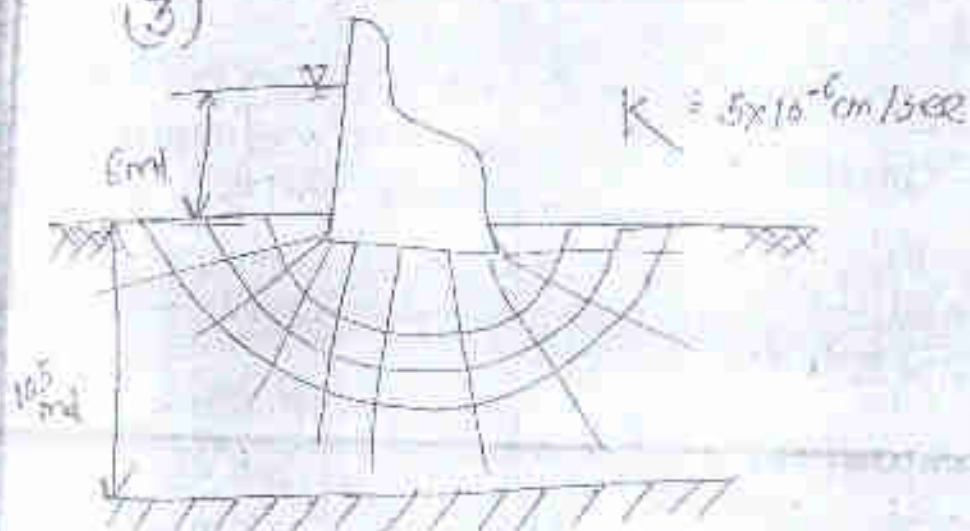
$$= 6 - 0$$

$$= 6 \text{ m}$$

$$q \Rightarrow 5 \times 10^{-8} \times 6 \times \frac{4}{12}$$

$$q = 1 \times 10^{-7} \text{ m}^2/\text{sec}$$

(3)



$$q = K \times \frac{NF}{NB}$$

$$H = H_1 - H_2 = 6 - 0 = 6$$

$$NF = 4$$

$$NB = 14$$

$$K \times \frac{NF}{NB}$$

$$= 5 \times 10^{-3} \times 6 \times \frac{4}{14}$$

$$= 8.571 \times 10^{-8}$$

1 Dec 2020

Compaction

- Compressibility of soil mass is an engineering property by virtue of which the soil mass is capable of undergoing compression @ 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 ramming, 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 & rearrangement of particles resulting in their closer packing.
- Compaction of a soil mass results in increase in dry density.

→ The dry density depends on way.

→ The amount & type of compaction determined the compaction effort.

→ For a specific amount of compaction 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 :-

1. The main aim of compacting a soil is to improve some desirable properties of the soil.

2. There is reduction in compressibility, water absorption & permeability, increase in soil strength & bearing capacity.

3. There is change in shrinkage & swelling characteristics.

4. Following are the factors due to effect of compaction.

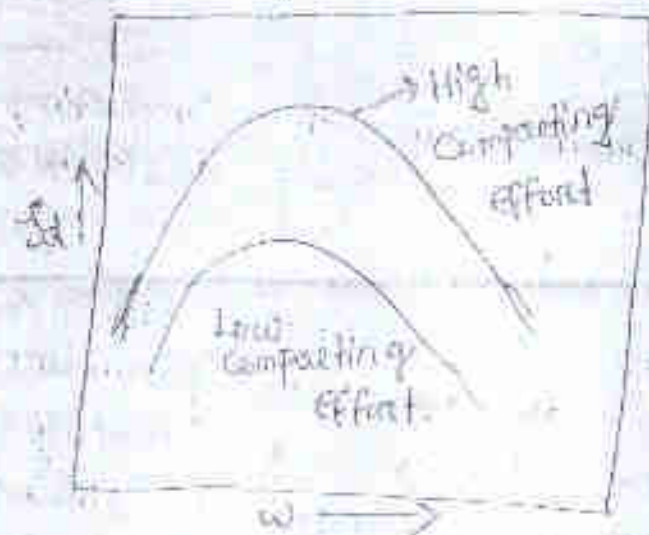
① change in structure of soil :-

→ 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, Finely 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.



(3) 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.

(iv) For a given void ratio, greater the size of individual pores, greater permeability.

(v) As the compaction 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 soil 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 & exerts more swelling pressure & swell to a 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 :-

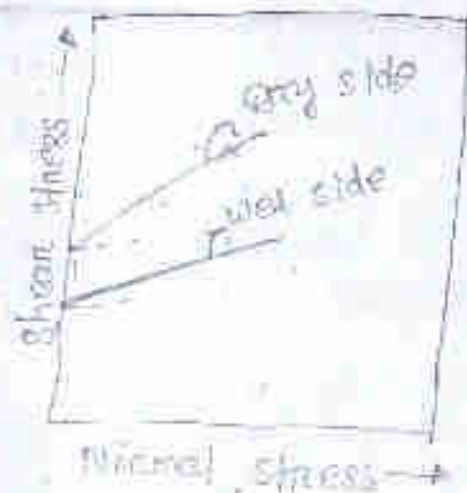
⇒ For a given soil, a sample Compacted dry side of optimum has a steeper stress strain curve & high modulus of elasticity.

⇒ 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:- 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.



Saturated Proctor test:-

- ⇒ The standard proctor test was developed by R.R. Proctor.
- ⇒ The test equipment consists of:-
 - ① Cylindrical actual mould having internal diameter 10 cm & height 11.7 cm.
 - ② detachable base plate.

(ii) Collar

(iii) Hammer (2.5 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.

⇒ When the Collar is at two half the mixed nature soil is placed in 2 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 $\frac{1}{3}$ 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.

→ Pure soil compacted = largest shrinkage
→ suggest the shrinkage than before sample the
cotton.

→ The cotton 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 ~~sum~~ of the compacted
specimen & kept the water content
determination.

→ The bulk density γ & dry density
 γ_d for the compacted soil are
calculated from:-

$$\gamma = \frac{W}{V}, \quad \gamma_d = \frac{W}{V \cdot w} \quad (\text{g/cm}^3)$$

→ 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)

Ex: 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100

→ A line which shows the
water content vs dry density
relation for the compacted
soil showing a constant
percentage of air voids is
known as an air void
line represented by:-



$$S_d = \frac{(1 - \frac{w}{G_s}) \cdot \rho_w}{1 + wG_s} \cdot S_w$$

where w = percent air voids

w = water content of compacted soil

G_s = specific gravity

S_d = dry density

ρ_w = density of water = 1 g/cm^3

- * 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 @ the saturation line.

$$S_d = \frac{G_s \rho_w}{1 + wG_s}$$

* 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 AASHTO test.

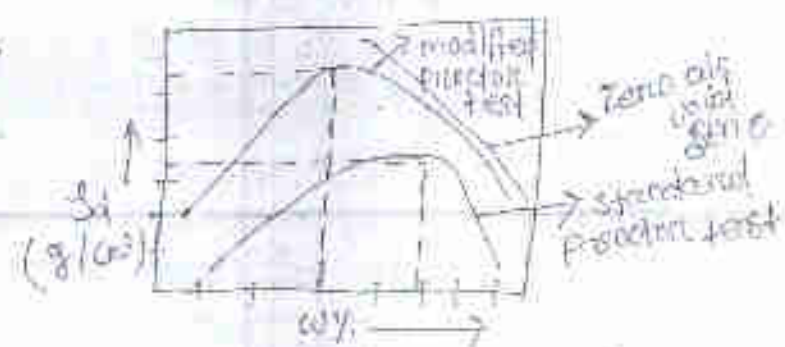
⇒ In this test the soil is compacted in five standard layers, each layer being given 25 blows.

of (4.9 kg) hammer dropped through a height of 45 cm .

→ The compactive energy given to the soil in this test is 2766 kg/cm^2 per 100 cm^3 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 some 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 achieved after which further addition of water decreases the density.

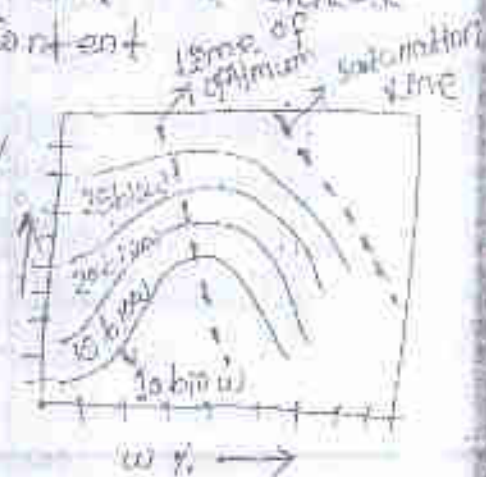
It is also true that voids due to water & air combination go on increasing with increase of water content beyond the optimum & hence the dry density of the soil falls.

(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.

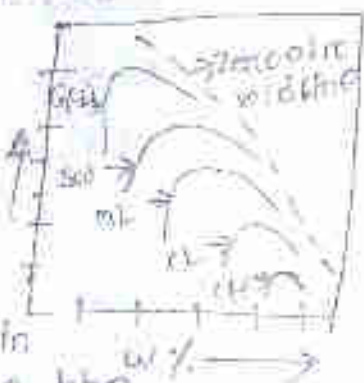
> well graded coarse-grained soil attain a much higher density & lower optimum water content than fine grained soils which require more water for lubrication because of the greater specific surfaces.

> 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.

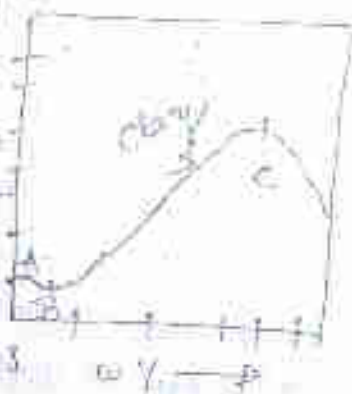
> For such soils the dry density decreases with an increase in water content in the initial slope of the curve. This is due to the "bulging of sand", where the capillary tension developed in the sandy soil is not fully counteracted by the compactive effort & this capillary tension holds the particles in a loose state resisting compaction.



(iii) Shrinkage curve for clay soils.

⇒ Here, there is initially decrease of dry density or 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.

Effects of Compaction on soil properties:-

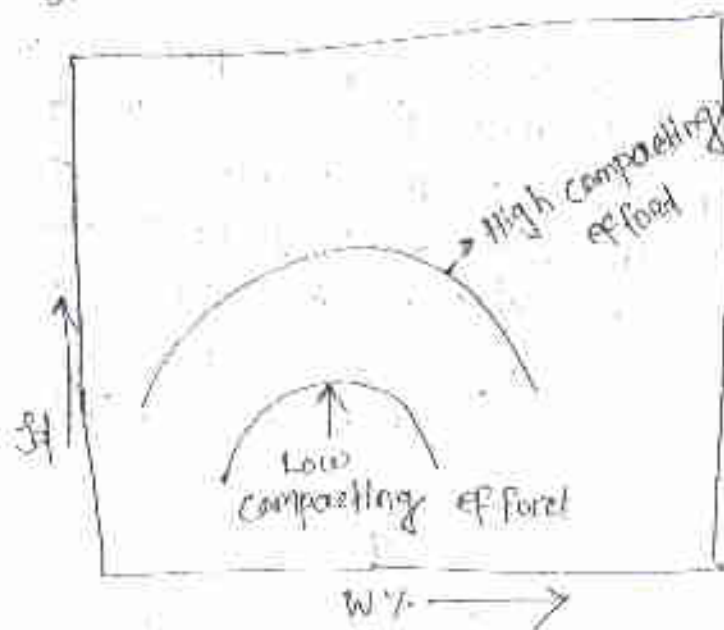
- * The main aim of Compacting a soil is to improve some desirable properties of the soil.
- * There is reduction in compressibility, water absorption & permeability, increase in soil strength & bearing capacity.
- * There is change in shrinkage & swelling characteristic.
- * Following are the factors due to effect of compaction:
 - (i) 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.



(ii) Permeability:-

The following points are noted:-

- (a) 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, greatest permeability.

(iv) As the compacting effort increases, the permeability of decrease.

(5) 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.

(6) Swelling A clayey soil sample compacted dry of optimum water content has high water deficiency & exert more swelling pressure & swell to higher water content than the same density of soil obtained from wet of optimum.

(7) Pore pressure :- Saturated sample of clay compacted dry of optimum tend to develop low pore pressure and water content compacted wet of optimum.

(8) Compressibility :-

→ Saturated sample of clay compacted wet of optimum is more compressible than another sample of same soil.

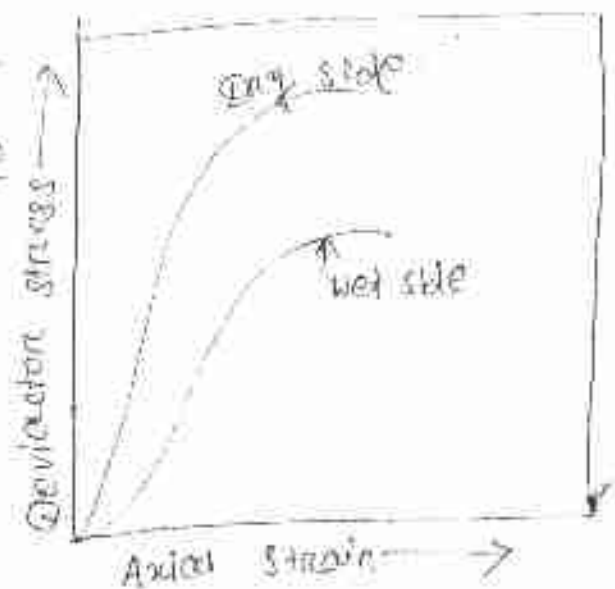
→ This is so because sample compacted dry of optimum has flocculated structure & requires extra pressure to parallel orientation of particles.

→ In high pressure range a sample compacted dry of optimum is more com. than wet of optimum.

(9) Stress - strain characteristics -

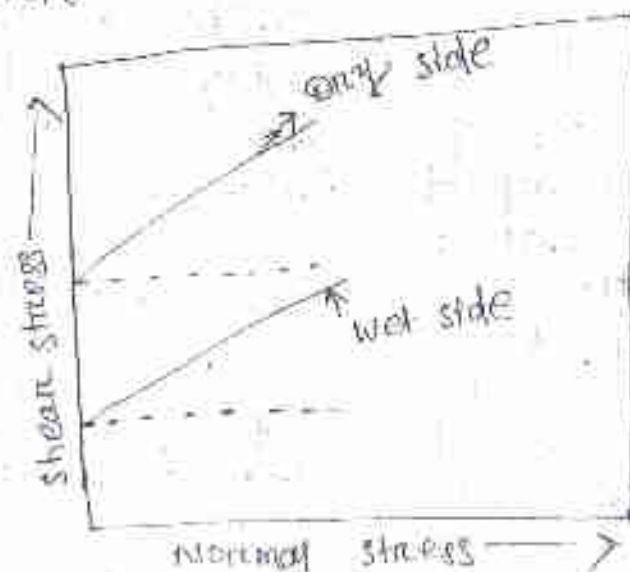
→ For a given soil a sample compacted dry side of optimum has a steeper stress-strain curve & high modulus of elasticity.

→ With same density of soil, compacted wet of optimum have brittle failure & curve is lower than dry side.



(3) 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

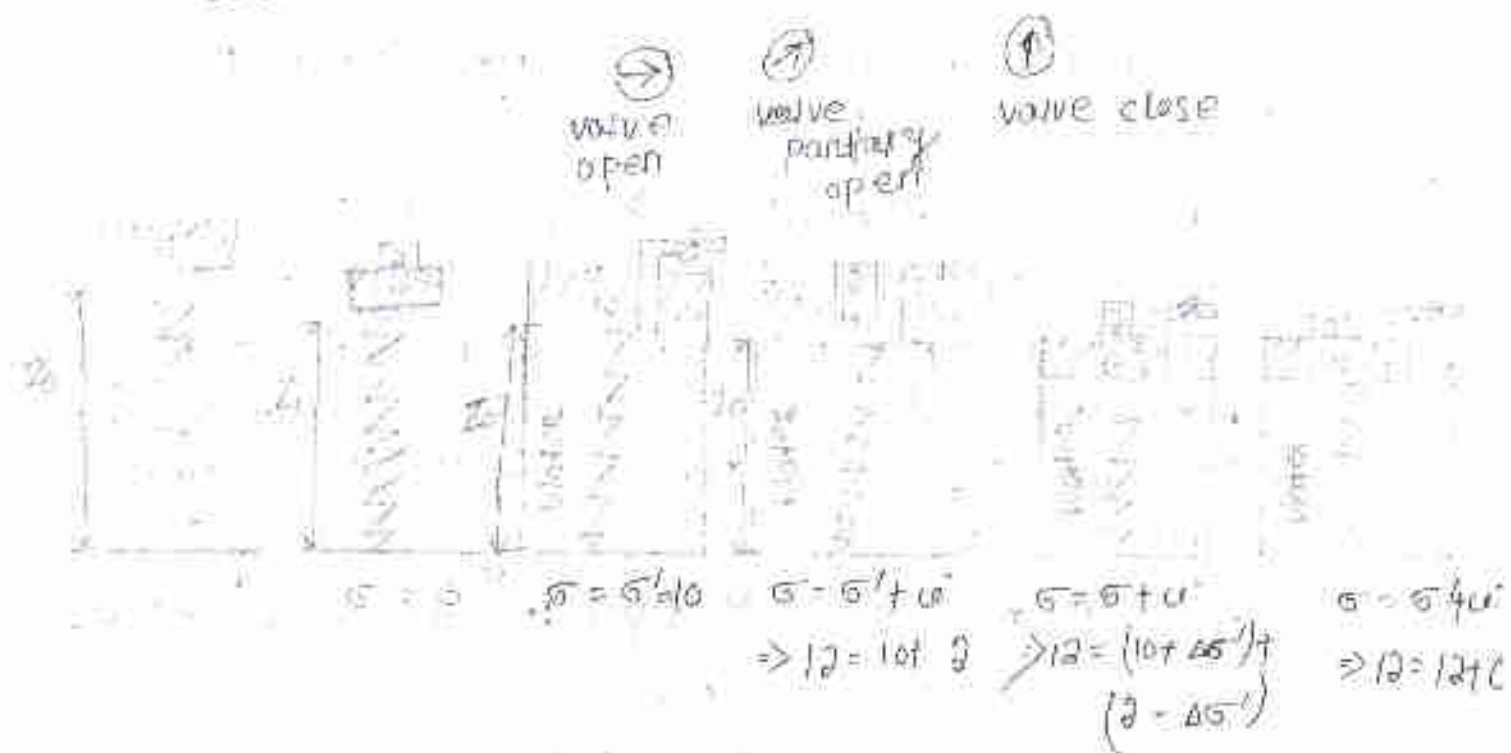
Consolidation

- * When a compressive load is applied to soil mass a decrease in its volume takes place. The decrease in volume of soil mass under stress is known as compression.
- * For the voids removing the air by compressibility 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. ② Dynamic loads such as tamping, rolling, vibrating are applied for a small interval in soil compaction. ③ In compaction soil volume is reduced by removing air void from the saturated & dry soil. ④ Compaction is mainly used for sandy soil.	① Consolidation is a process where steady & static pressure causes compression. ② Static & sustained loading is applied for a long interval in soil consolidation. ③ In consolidation process soil volume is reduced by squeezing out pore water from the saturated soil. ④ Consolidation is used for clayey soil.

Consolidation process by Spring Analogy :-



Grain structure $\longrightarrow$ Spring
 voids with water $\longrightarrow$ Cylinder
 Valve opening $\longrightarrow$ Permeability

9 Dec 2020

- $\rightarrow$ Terzaghi demonstrated the mechanics of Consolidation by Spring Analogy.
- $\rightarrow$ A saturated soil mass taken in a container consists of soil particles forming the skeleton of soil mass & voids filled with water.
- $\rightarrow$ The skeleton frame of soil particles can be assumed to be replaced by a member of springs & the water filling voids in soil mass by the water filling the cylinder.
- $\rightarrow$ The compressive stress is caused by load applied on piston placed on top of the springs.
- $\rightarrow$ An outlet with valve is provided to control drainage of water from out of the cylinder.

1) > Let z_0 be the length of spring under a pressure 10 units.

> Let the length decrease to 2, when the pressure is increased by 2 units.

> In fig (ii) (iii) (iv) (v) 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 :- $\sigma = \sigma' + u$

where σ = total stress

σ' = effective stress

u = pore water pressure.

> In fig (iv) additional pressure of 2 unit acts & the valve is closed. Because water is incompressible the spring are prevented from under going any further compression & there fore 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. $\sigma = \sigma' + u$
 $\Rightarrow \sigma = \sigma' = 10$

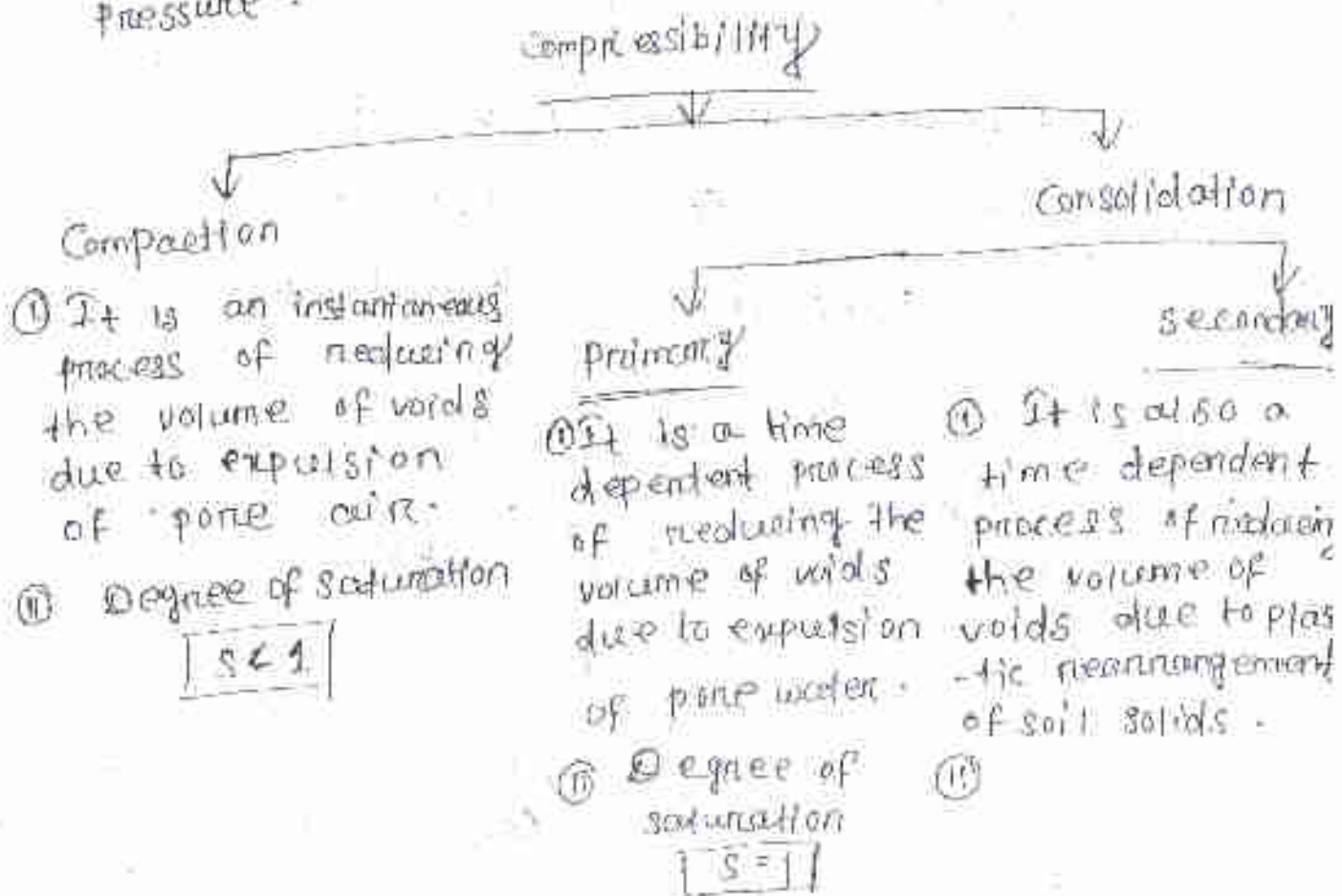
1. Soil mass

→ Under an applied pressure the soil mass will have reached a particular value of void ratio when the primary consolidation is complete.

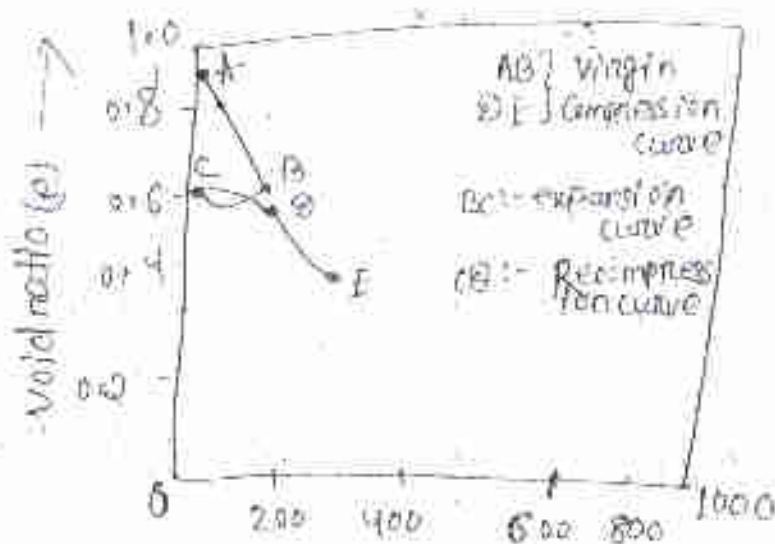
→ This value is referred to as final equilibrium void ratio.

→ 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.



Consolidation of Undersaturated Clay



$$e_c = \frac{e_0 - e}{\log_{10} \frac{\sigma'}{\sigma_0'}}$$

e_0 = initial void ratio

σ_0' = initial stress

$$C_c = \frac{\Delta e}{\Delta \log_{10} \sigma'}$$

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 condition. The soil specimen is sand with "bed" 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 σ' 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 pressure are found. The void ratio is plotted as ordinate against effective stress σ' 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 σ'_B and soil specimen expansion curve BCs as indicated by

Curve BC however the soil specimen will not attain again the original void ratio corresponding to beginning of test because it will have undergone some permanent compression which can be attributed to irreversible orientation under 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



AB } virgin
DE } compression
curve

BC :- Expansion
curve

CD :- Recompression
curve

specimen is recompressed and the test continued the curves CD and DE are obtained. The curves AB and DE correspond to

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 though both recompression to 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.



According to Terzaghi, the virgin compression curve can be defined by the following empirical relation.

$$e = e_0 - C_c \log_{10} \frac{\sigma'}{\sigma'_0}$$

where e_0 = Initial void ratio corresponding to initial effective stress σ'_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

$$C_c = \frac{e_0 - e}{\log_{10} \frac{\sigma_1}{\sigma_0}} = \frac{\Delta e}{\Delta \log_{10} \sigma}$$

The expansion curve on semi-log plot is defined by the following relation.

$$e_0 = e + C_s \log_{10} \frac{\sigma_1}{\sigma_0}$$

C_s denotes expansion index or swelling index. It is the slope of straight line portion of 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 α 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.

$$\Delta v = \frac{-\Delta e}{\Delta \sigma'} = \frac{-(e - e_0)}{\sigma' - \sigma'_0}$$

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.

$$m_v = \frac{-\Delta v}{v_0} \cdot \frac{1}{\Delta \sigma'}$$

$$m_v = \frac{\Delta v}{1 + e_0 \Delta \sigma'}$$

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 ~~under~~ overburden pressure acting on the soil. The preconsolidation ^{condition} 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 first step in the theoretical analysis of the consolidation process.

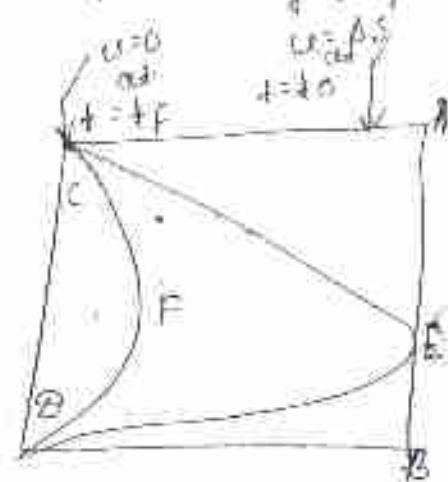
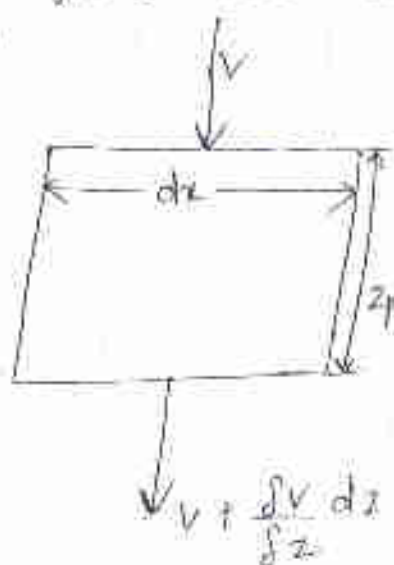
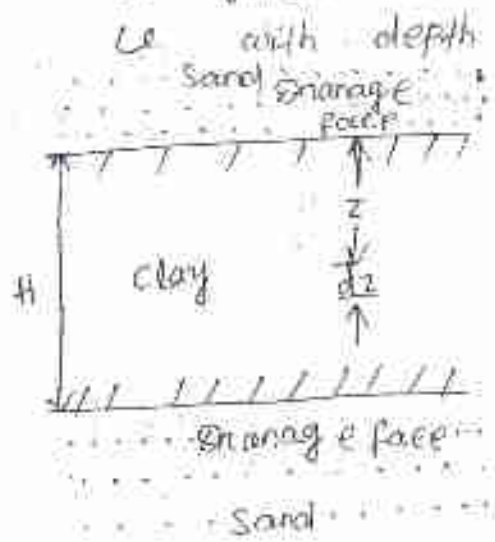
➤ Following are the assumptions made in Terzaghi's one-dimensional consolidation theory.

- (i) The soil mass is homogeneous and fully saturated.
- (ii) The soil particles and water are incompressible.

- 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 p increment Δp the pressure increment is first borne by pore water so that at initial time to the excess pore pressure $u = \Delta p$ 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 GE . At any intermediate time, betⁿ to and to part of consolidating pressure ΔT is transferred to soil particles so that $\Delta T = \Delta T' + u$. The distribution of excess pore pressure u at any intermediate time t is represented by a curve such as CDE . A number of such curves by representing excess pore pressure distribution along the depth of clay layer at different instants of time $t = t_1, t_2, \dots$ can be drawn and 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.



(b) Isochrones

(a) Consolidating layer

At any time t , the hydraulic head h corresponding to the excess pore pressure u is given by

$$h = \frac{u}{\gamma_w} \quad \text{--- (1)}$$

The hydraulic gradient i is given by

$$i = \frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \cdot \frac{\partial u}{\partial z} \quad \text{--- (2)}$$

Applying Darcy's law, the velocity of flow of pore water due to this hydraulic gradient is given by

$$v = kv = \frac{k}{\gamma_w} \cdot \frac{\partial u}{\partial z} \quad \text{--- (3)}$$

The rate of change of velocity along the depth of the layer is given by

$$\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} \quad \text{--- (9)}$$

Let us consider a soil element of side dx , dy and of width dz 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.

$$\Delta q = \left(v + \frac{\partial v}{\partial z} dz\right) dx dy - v dx dy$$

$$\Delta q = \frac{\partial v}{\partial z} dx dy dz$$

The decrease in the volume of soil element is equal to the volume of water squeezed out also, we have

$$\Delta v = -m_v V_0 \Delta \sigma'$$

Where V_0 = volume of soil element at time $t_0 = dx dy dz$

∴ change in volume per unit time is given by

$$\frac{\partial}{\partial t} (\Delta v) = -m_v (dx dy dz) \frac{\partial (\Delta \sigma')}{\partial t}$$

Comparing eq (9) and (10), we get

$$\frac{\partial v}{\partial z} = -m_v \frac{\partial (\Delta \sigma')}{\partial t}$$

$$\text{Now } \Delta \sigma = \Delta \sigma' + u$$

(vi) $\Delta T' = \Delta T - u$ where ΔT is Constant

$$\therefore \frac{\partial}{\partial t} (\Delta T') = - \frac{\partial u}{\partial t}$$

Substituting in eq (viii)

$$\frac{\partial v}{\partial z} = m_v \cdot \frac{\partial u}{\partial t}$$

Comparing eq (iii) and eq (ix) we get

$$\frac{\partial u}{\partial t} = \frac{k}{m_v \cdot \gamma_w} \cdot \frac{\partial^2 u}{\partial z^2}$$

(x) $\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2}$

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

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 rate of volume change, i.e. k is expressed in ml/sec , m_v in m^2/kg and γ_w in kg/m^3 , unit of C_v will be m^2/sec .

The mathematical steps involved in obtaining the solution by means of further series, of the differential equation of consolidation is presented 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 P_f denotes final settlement under pressure increment Δp and p the settlement at any intermediate time t , then the degree of consolidation attained at that time t is given by

$$U(\%) = \frac{p}{P_f} \times 100$$

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

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

where d = drainage path. The drainage path represents the maximum distance a water particles has to travel within the layer. Each 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 $T_v = \frac{C_v t}{d^2} = \frac{k}{\gamma_v \alpha} \cdot \frac{t}{d^2}$

* We notice that the time factor, and hence the degree of consolidation, depends upon (i) coefficient of permeability k (ii) Coefficient of volume compressibility, α (iii) thickness of layer and (iv) number of

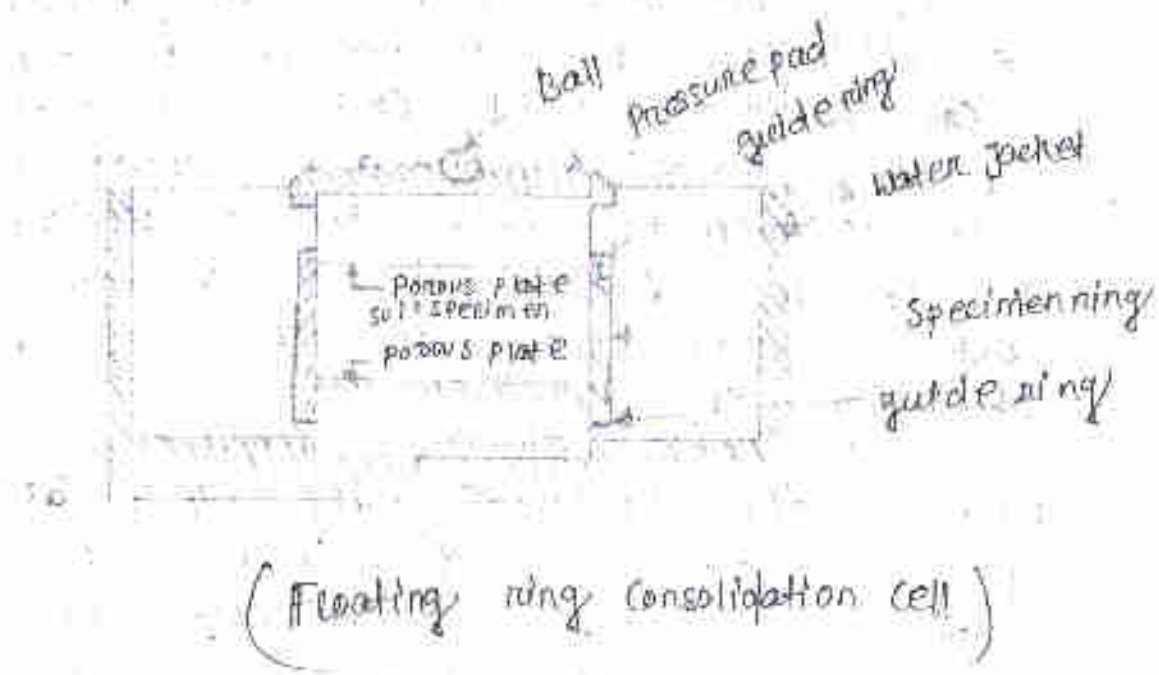
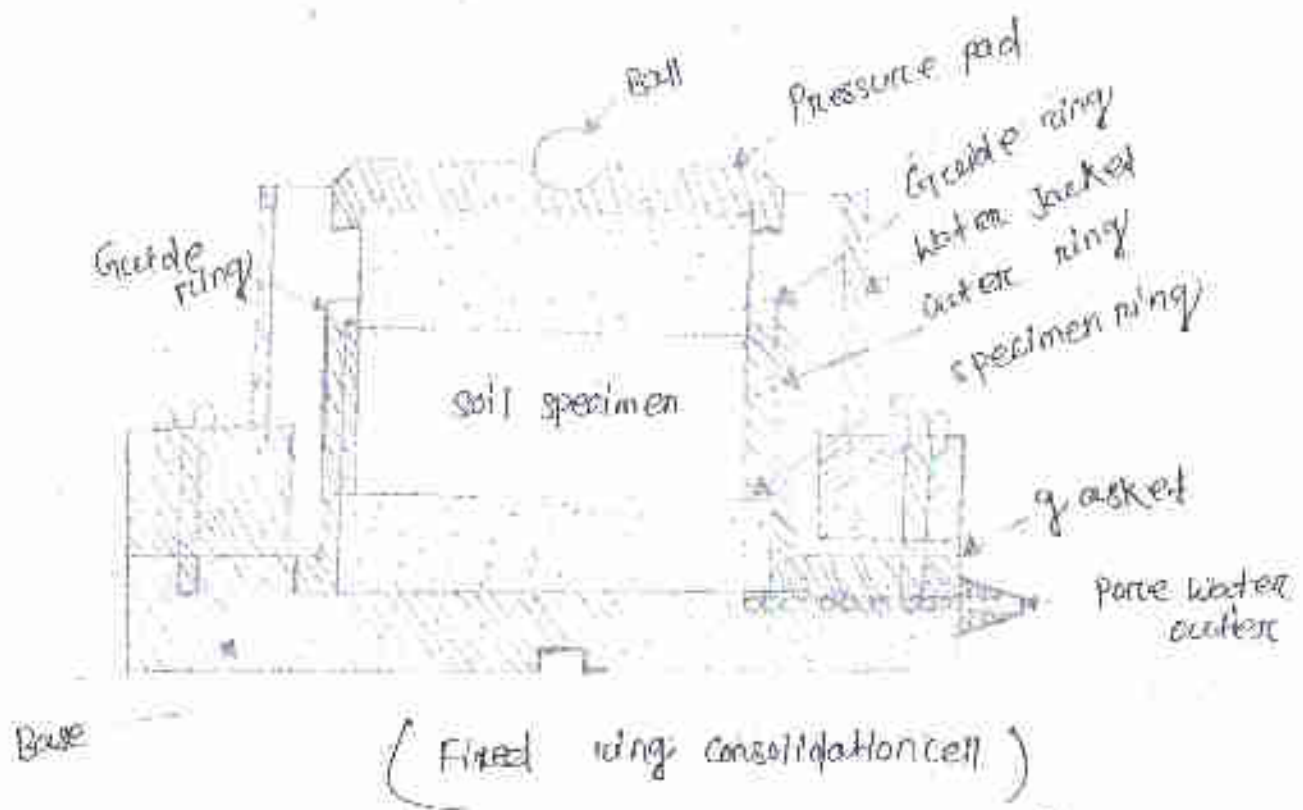
drainage faces. In addition it is found to depend upon the consolidating pressure and its manner of distribution across the depth of layer. The time factor T_v corresponding to various values of degree of consolidation U for the two types of drainage conditions and different distributions of consolidating pressure are presented in the following approximate expressions may be used to compute T_v , in the absence of the tables.

$$\text{When } U < 60\% \text{, } T_v = \frac{\pi}{4} \left(\frac{U}{100} \right)^2$$

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

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.



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 vertical pressure on the soil specimen in convenient increments. During the test the specimen is allowed to consolidate fully under different vertical pressure such as 10, 20, 50, 100, 200, 400, 800, 1600 KN/m^2 . 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 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 used 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 4-6%. This indicates that secondary consolidation comes into play even before the primary consolidation ends and continues thereafter. The secondary consolidation is represented by a series of straight lines with different slopes on e versus $\log t$ plot. It is of much significance in the case of highly organic 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.

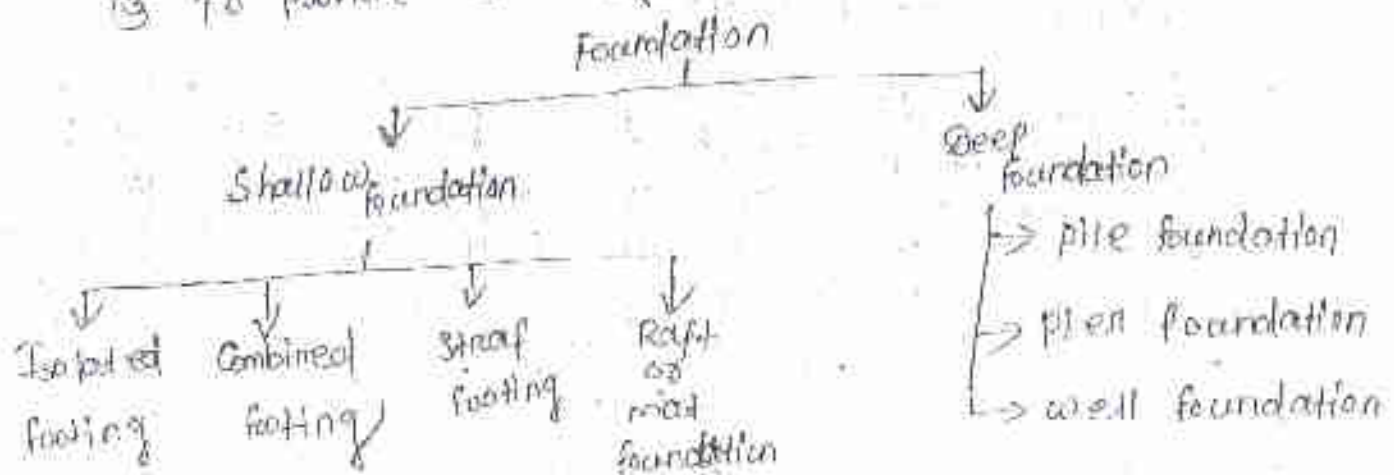
Footing:- A footing is a portion of the foundation of a structure that transmits loads directly to the soil.

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.



① Shallow foundation:-

→ Shallow foundation is also known as a stepped foundation.

→ If the depth of foundation is less than the width of foundation then it is known as shallow @ general consideration.

⇒ It can be used where the bearing capacity of soil in which the structure is to be constructed is maximum.

⇒ The depth of foundation = 800 mm. $\leq$ 24 in.

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 picestair. The step or picestair 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 concentrated, 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 center we can see that this footing is in the shape of a trapezoid.

12] Wall footing

Wall footings are used to support structural walls that carry loads for other floors or to support nonstructural walls.

Wall footing are further classified into 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 10 centimeters 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.

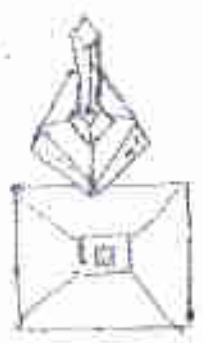
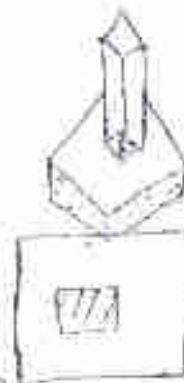
(4) Inverted arch footing :-

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.



(5) Spread footing :-

This is also a type of footing. In this spread foundation, 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 $(t + 100)$ mm here $(t = \text{thickness of wall})$. 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 $(t + 400)$ mm. The width of the base foundation is $2(t + 100)$ mm.



(SPREAD FOOTING)

(6) Raft Footing or mat Footing:-

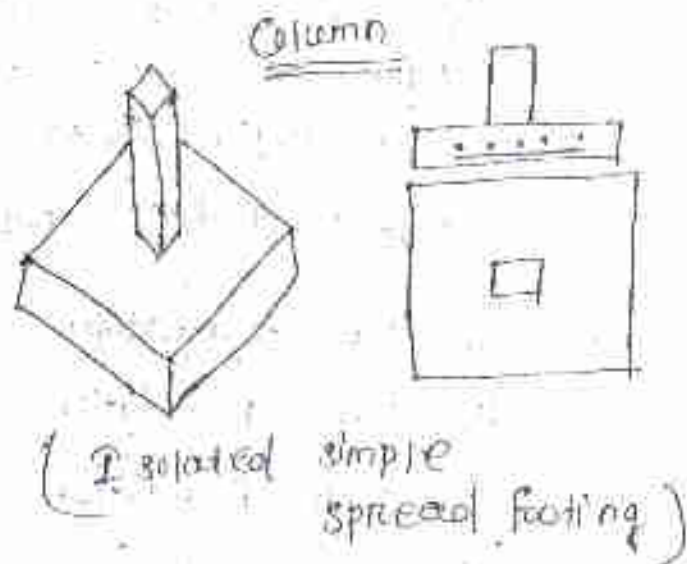
This is also known as combined type footing or foundation. It covers the whole structure. It provides the stability and strength to the structural members like R.C. wall and columns. Above the soil surface, a base is created of any thickness. It is just done to create a base for raft foundation. On that base, this raft foundation is constructed. When bearing capacity of a soil is less than, raft or mat footing is used.

(7) Combined footing:-

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 footings are done when the load coming from two or more column is not equal or same.

(8) Strap 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 uses 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.



Classification 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

Concrete 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 in-situ

These piles are manufactured in the factory which is further transported to the construction site where ever it is required. These piles can bear 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 pre cast 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 bear 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 50 cm.

Length of a pile is taken 20 times that of its diameter. For (Example - 25 cm is its diameter. Then, $L = 20 \times 25 = 500 \text{ cm}$). 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 T 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 sink in the soil due to structural load.

4. Composite pile :-

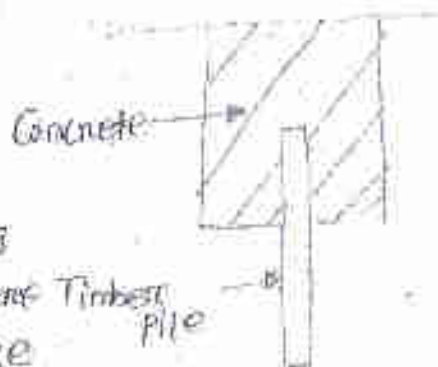
When the piles are made from more than one material they are known as composite pile.

These piles are made from

Concrete and wood. These piles

are used in those areas where Timber

the water table is up. These piles are used in such conditions just because concrete and wood both are good water absorbers.



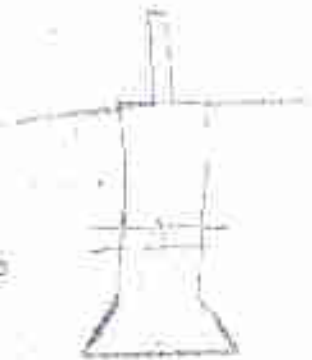
Advantages of pile foundation :-

- Use of these piles can save time
- They are very much economical
- By using this pile system it reduces the need of 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 cross-section 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 Battered caissons
- ③ 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 superstructure.

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.

② Drilled Caissons :-

They are masonry in a cylindrical shape so they are also known as cylindrical Piers. Distribution 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 which 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 cone or a rod is fixed which gives more stability to the structure as the weight taking capacity increased due to the steel cone.

* Types of Bearing Capacity Failures :-

→ When a footing fails due to insufficient bearing capacity, failure patterns are developed, depending upon type of failure.

→ There are three types of failure :-

- General shear failure
- Local shear failure
- Punching shear failure

(a) General shear failure :-

* In case of general shear failure, continuous failure surfaces develop between the bottom 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 upto ground surface.

(ii) Failure is by of the footing.

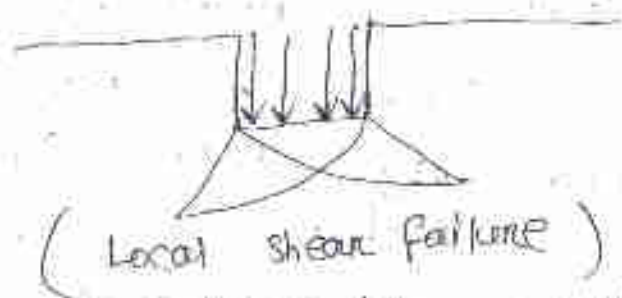
(iii) Failure is sudden

(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 plastic equilibrium

* Due to this reason, the failure surfaces do not the ground surface & only



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

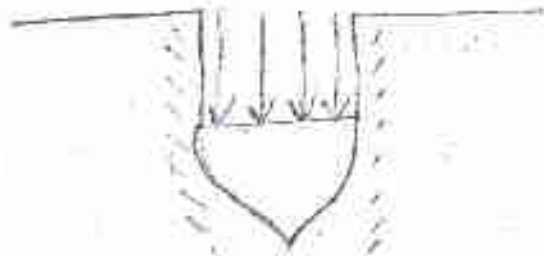
(ii) Failure surfaces is ground surface.

(iii) There is physical soil around of footing.

- (iv) Failure is not defined by settlements.
- (v) ultimate bearing capacity is not well defined.

(c) Punching shear failure:

It occurs 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 occur in relatively loose soil with density less than 5%.
- * Shear failure may also occur in relatively low compressibility soil, if the foundation is at depth.
- * Following are the characteristics of shear failure.
 - (i) No failure is observed.
 - (ii) The failure surface, which is vertical or inclined of the base.



Bearing Capacity :-

→ The of a soil or rock is referred 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 excess pressure or the difference in intensity of the gross pressure after the construction of the structure & the original overburden pressure.

$$q_n = q - \sigma = q - \gamma D$$

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 min^m 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 min net pressure intensity causing shear failure of soil

$$q_{nf} = q_s - \bar{\sigma}$$

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.

$$q_{ns} = \frac{q_{nf}}{F}$$

where F = factor of safety

q_{nf} = net ultimate bearing capacity.

safe bearing capacity (q_s) :-

The max pressure which the soil can carry safely without risk of shear failure is called the surface bearing capacity.

$$q_s = q_{ns} + \bar{\sigma} = \frac{q_{nf}}{F} + \bar{\sigma}$$

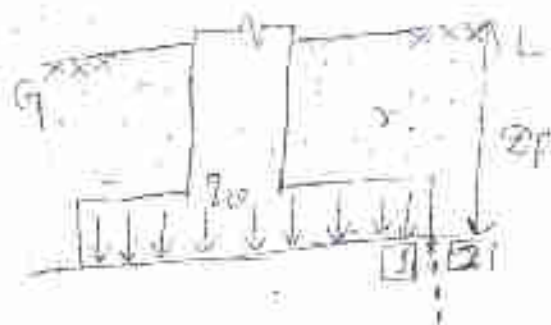
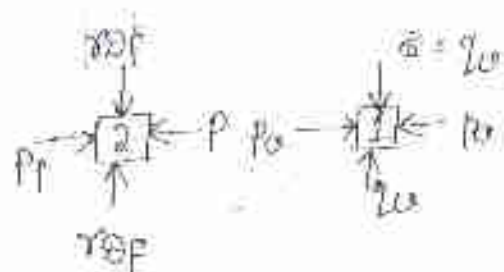
(or)

$$q_s = \frac{q_f}{F}$$

28 Dec 2020

Rankine's 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

$$\Rightarrow q_u = \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right)^2 \gamma D_f \text{ kN/m}^2$$

where ϕ :- angle of internal friction.

$$\textcircled{m} \quad D_f = \frac{q_u}{\gamma} \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)^2$$

where q_u = ultimate bearing capacity of soil.

Terzaghi's Bearing Capacity Equation :-

$$q_u = C N_c + \bar{\sigma} N_q + \frac{1}{2} B \gamma N_\gamma$$

where q_u = ultimate bearing capacity
 c = cohesion (kN/m²)

$\bar{\sigma}$ = effective overburden pressure (kN/m²)

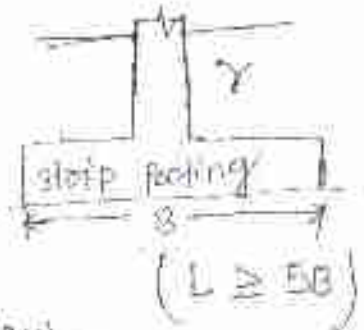
$\bar{\sigma} = \gamma D_f$ (no weight reduction)
 $\bar{\sigma} = \gamma_{\text{sub}} D_f$ (fully submerged)

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 factors

(According to Terzaghi Analysis these factors depend upon the angle of internal friction of the soil)

	Terzaghi	Priestly 1/3
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 failure. $\left[\phi > 36^\circ, I_p > 70\% \right]$
- (iii) Terzaghi equation is given for strip footing $\left[L \geq 5B \right]$

Modified Terzaghi Equation for other shape of footing :-

Case-I Strip footing

$$q_u = (C_u + \bar{\sigma} N_q + \frac{1}{2} B \gamma N_q) (G.S.F)$$

$$q_u = \frac{2}{3} c u_c' + \bar{\sigma} N_q' + \frac{1}{2} B \gamma N_q' (L.S.F)$$

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.

$$q_u = \tan^2 \left[\frac{3}{2} \tan \phi \right]$$

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) :-

$$q_u = \left(1 + 0.3 \frac{B}{L} \right) c u_c' + \bar{\sigma} N_q + \frac{1}{2} \left(1 - 0.2 \frac{B}{L} \right) B \gamma N_\gamma$$

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 :- ($B=L$)

$$q_u = 1.3 c u_c' + \bar{\sigma} N_q + 0.4 B \gamma N_\gamma$$

29 Dec 2020

10

A rectangular footing ($2\text{m} \times 3\text{m}$) rests on a $c-\phi$ soil with its base at 1.5m 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 $\gamma = 18\text{ kN/m}^3$.

$$c = 10\text{ kN/m}^2 ; \phi = 30^\circ ; N_c = 37.2$$

$$N_q = 22.5$$

$$N_\gamma = 19.7$$

(Use Terzaghi Analysis)

solⁿ

$$q_{u0} = \left[\left(1 + 0.3 \times \frac{2}{3} \right) 10 \times 37.2 \right] + (22 \times 22.5) + \left[\frac{1}{3} \times \left(1 - 0.2 \times \frac{2}{3} \right) \times 2 \times 18 \times 19.7 \right]$$

$$= 1361.2\text{ kN/m}^2$$

$$q_{nu} = q_{u0} - \gamma D = 1361.2 - 18 \times 1.5$$

$$\downarrow \text{net ultimate} \quad = 1361.2 - 27$$

$$\text{bearing capacity} = 1334.2\text{ kN/m}^2$$

11 Jan 2021

Determining 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 $c = 10 \text{ kN/m}^2$ & $\phi = 30^\circ$. Use Terzaghi analysis.

$$N_c = 37.2$$

$$N_q = 22.5$$

$$N_\gamma = 19.7$$

Solⁿ For circular footing

$$q_u = 1.3 C N_c + \bar{\sigma} N_q + 0.3 B \gamma N_\gamma$$

$$[\bar{\sigma} = \gamma D_f]$$

given data :- N_c, N_q, N_γ

$$\phi = 30^\circ$$

$$c = 10 \text{ kN/m}^2$$

footing diameter = 2m

$$F.O.S = 3$$

safe load = 1600 kN

$$D_f = ?$$

$$q_{us} = \frac{\text{load}}{\text{Area}} = \frac{1600}{\frac{\pi}{4} \times 2^2} = 509.29 \approx 509.30 \text{ kN/m}^2$$

$$q_u = q_s$$

$$\Rightarrow 1.3 C N_c + \gamma D_f N_q + 0.3 B \gamma N_\gamma = q_s$$

$$\Rightarrow (1.3 \times 10 \times 37.2) + (18 \times D_f \times 22.5) + (0.3 \times 2 \times 18 \times 19.7)$$

$$= 509.30$$

$$\Rightarrow D_f = 0.4$$

5 June 2021

Effect of water table 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.

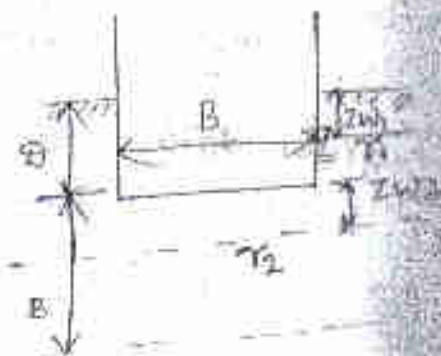
1st method:-

$$q_f = C N_c + \gamma_1 D N_q R_{w1} + \frac{1}{2} \gamma B N_\gamma R_{w2}$$

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:-

$$q_f = C N_c + \bar{\sigma} N_q + \frac{1}{2} \gamma B N_\gamma R_w$$

$$\text{where } R_w = R_{w2} = 0.5 \left(1 + \frac{Z_{w2}}{B} \right)$$

3rd Method:-

$$q_f = C N_c + \bar{\sigma} N_q + \frac{1}{2} \gamma_e B N_\gamma$$

where γ_e = effective unit weight of soil
in wedge zone.

7th Jan 2021

New chapter

Shear strength of soil

Shear strength

The shear strength is attributed by the maximum shearing resistance that is mobilised on the potential failure plane & is equal to the ultimate shear stress.

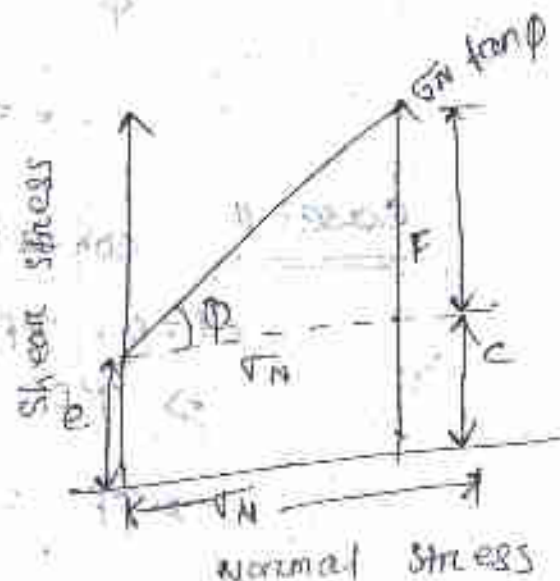
→ The shear strength is attributed by

- Interlocking of particles
- Cohesion & adhesion of particle
- frictional resistance

Mohr Coulomb

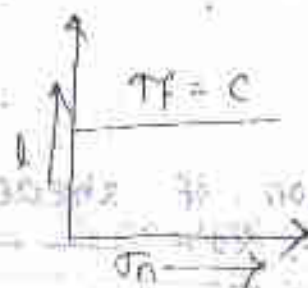
$$\tau_f = c + \sigma_n \tan \phi$$

→ This case is for
 $c = 0$ soil



Cohesionless soil

(c=0)



purely cohesive soil (φ=0)

where c_f = shear stress at failure

ϕ = Angle of internal friction

σ_n = Normal stress

c = cohesion

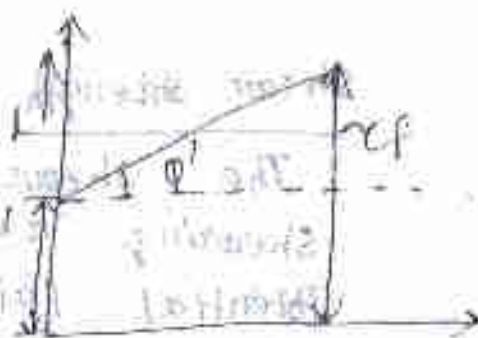
Modified Mohr-Coulomb Theory :-

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

where c' & ϕ' = effective cohesion & effective

angle of shearing resistance

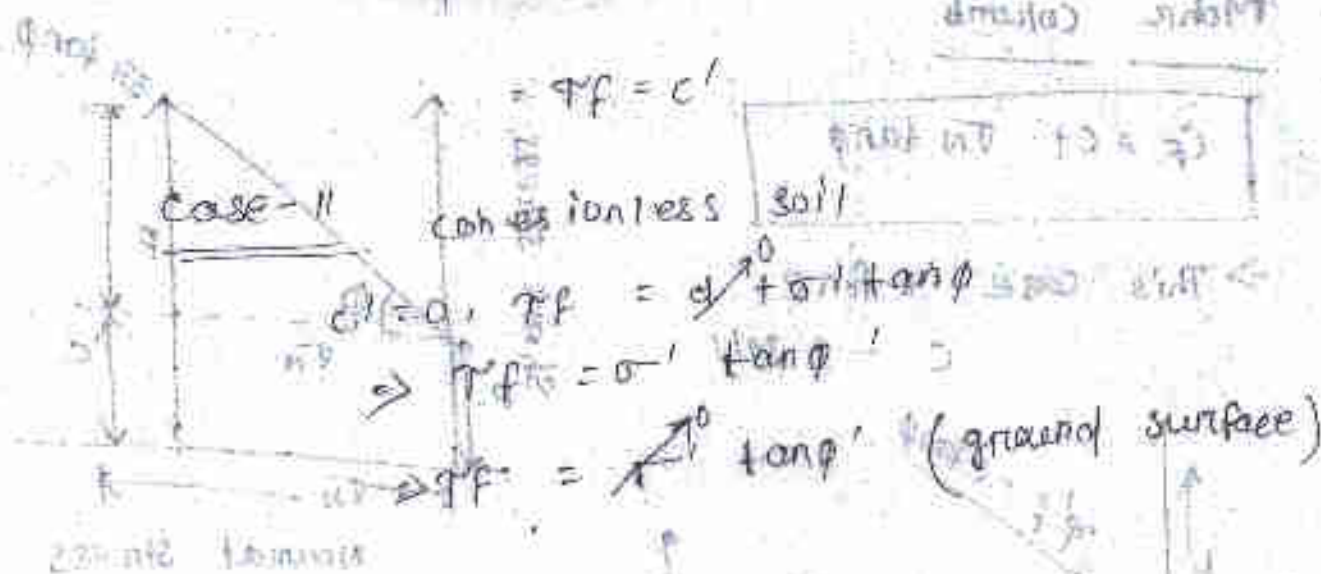
σ' = effective stress



Limitations :-

Case - I :- pure cohesive soil (clay)

$$\phi = 0, \tau_f = c' + \sigma' \tan \phi$$



9 Jan 2021

Determination of shear strength parameters :-

Load Application

- (i) Direct shear stress test
- (ii) Triaxial compression test
- (iii) Unconfined compression test

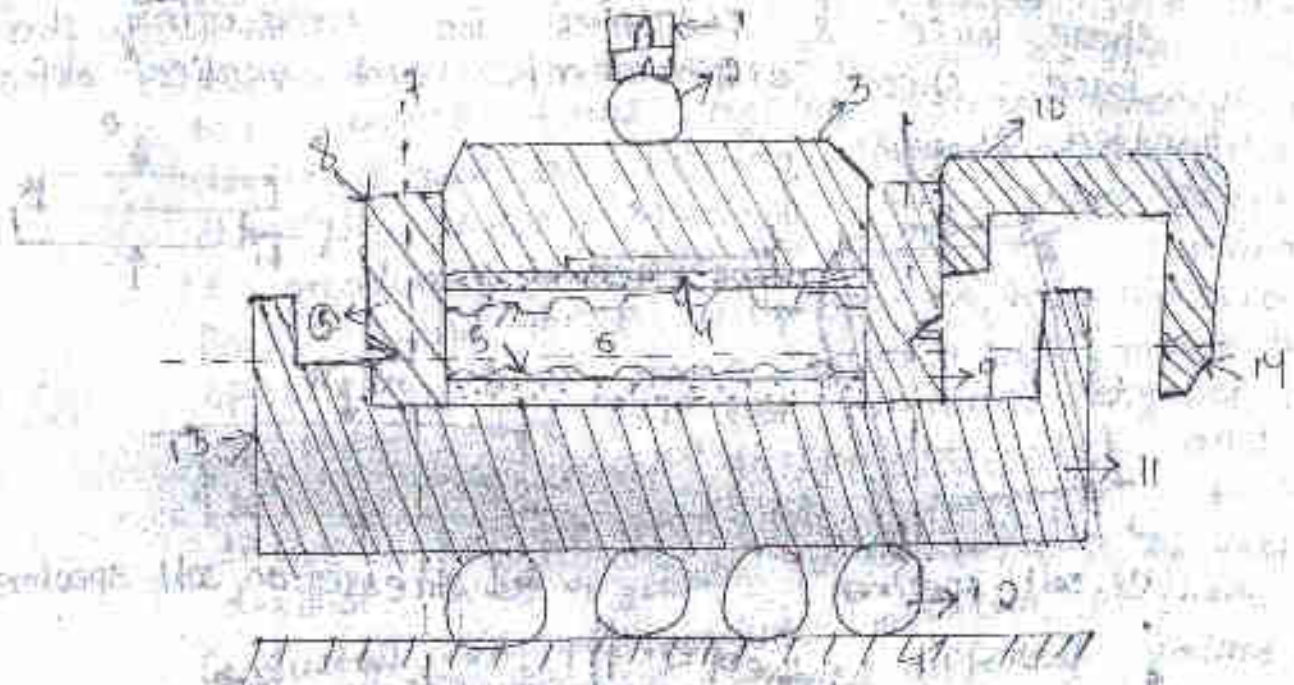
Drainage Application

- (i) Undrained test (UU test)
- (ii) Consolidated undrained test (CU)
- (iii) Consolidated drained test (CD)

① Vane shear test

11 Jan 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 fix
two valves

⑧ → Upper part of
shear box

⑨ → Lower part of shear box

⑩ → U-rim

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 $60\text{ mm} \times 60\text{ mm}$ and thickness about 20 to 25 mm. The direct shear test equipment essentially consists of

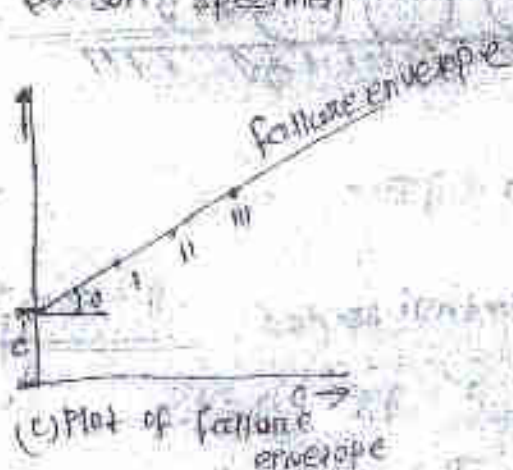
- (i) shear box, (ii) loading jack for applying normal force, (iii) geared jack for applying shear force & facilities for measuring shear force, shear displacement and vertical deformation change.



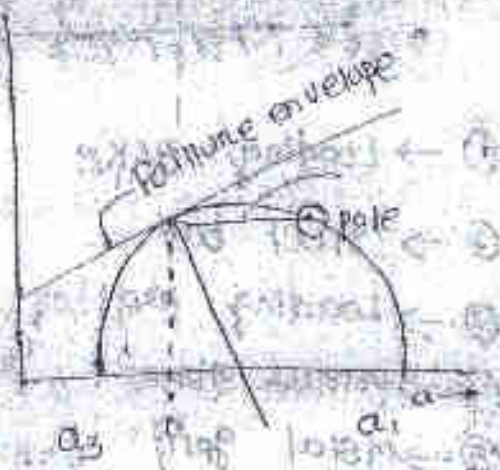
(a) soil specimen



(b) stresses on soil specimen



(c) Plot of failure envelope



(d) Mohr's circle of failure

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 central 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). Through the soil specimen, the shear stress 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 specimens subjected to three different values of σ . By plotting σ and τ the failure envelope is obtained and 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 average σ is kept constant and the shear stress at constant rate and measuring the shear strain.

Advantages of direct shear test :-

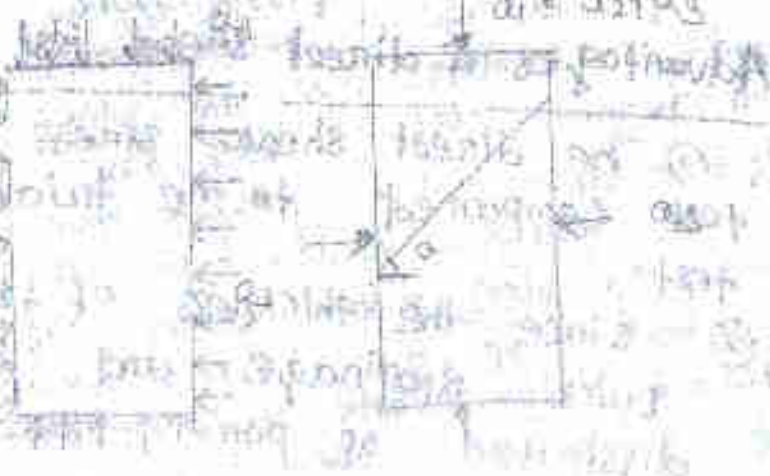
- ① The direct shear stress test is a simple test compared to the triaxial compression test.
- ② Since the thickness of the sample is small quick drainage and hence rapid dissipation of pore pressure is possible.

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. Connected 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 expressed by applying all the three principal stresses.



The soil specimen used in the test is cylindrical in shape with length 2.1025 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 in 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 outlet 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 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 it may be stopped and failure point defined at observed strain level upto 20%. The

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 = 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 = 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 $(V_1 + \Delta V) = A (L_1 - \Delta L)$

$$A = \frac{V_1 + \Delta V}{L_1 - \Delta L}$$

In the case of undrained test on saturated soil sample $\Delta V = 0$ and

$$A = \frac{A_1 L_1}{L_1 - \Delta L} = \frac{A_1}{1 - e}$$

where e is the axial strain at that stage



- ① Axial load
- ② Loading ram
- ③ Air release valve
- ④ Top Cap
- ⑤ Perspex cylinder
- ⑥ Sealing ring
- ⑦ pore water outlet
- ⑧ Additional pore water outlet
- ⑨ Cell fluid index
- ⑩ Soil specimen
- ⑪ Porous disc

After finding deviator stress σ_d at failure, we have major principal stress at failure $\sigma_1 = (\sigma_3 + \sigma_d)$ with this set of (σ_1, σ_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 σ_3 are read out from the plot. For the benefit of student it is repeated that deviator stress

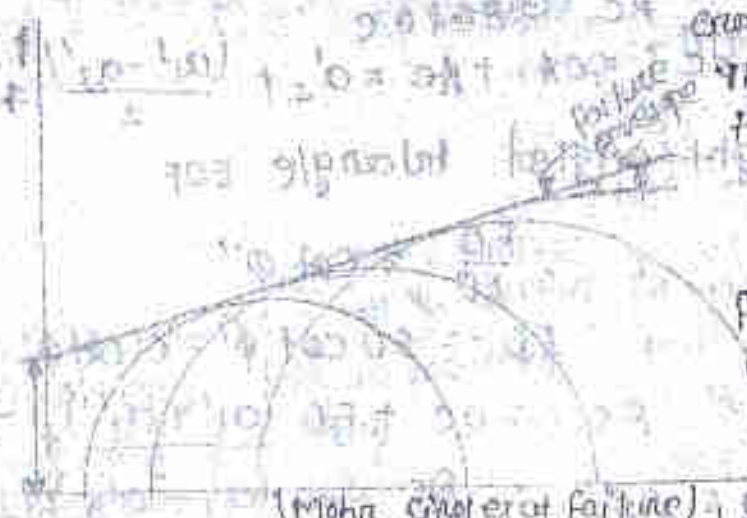
on the specimen through the plunger. This is because the cell pressure σ_3 and not only acts on the sides of the specimen but also acts on top of the specimen and there will be equal reaction at the base. This is due to the

fact that area of plunger is smaller than area of cross section of specimen. Thus σ_3 acts all round the specimen.

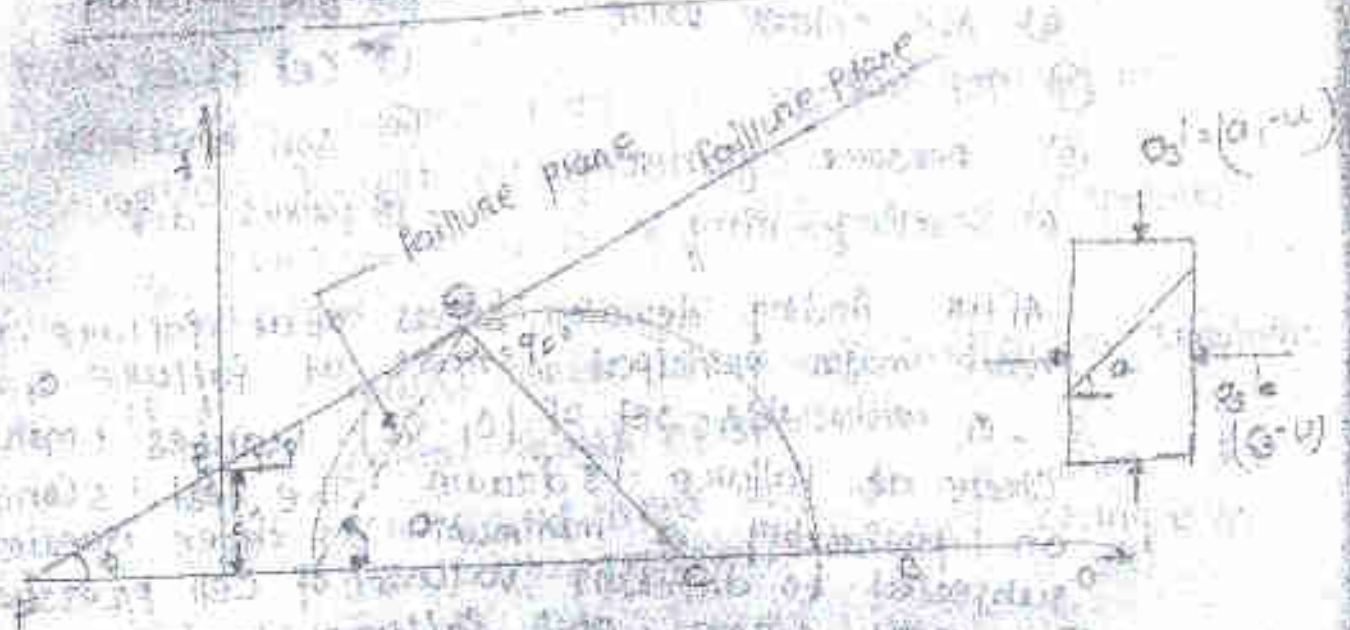
Since $\sigma_1 = (\sigma_3 + \sigma_d)$
we have $\sigma_1 = (\sigma_3 + \sigma_d)$

Further, in the triaxial compression test on cylindrical soil specimen we have $\sigma_2 = \sigma_3$ specimens

(Mohr circles at failure) developed; to test soil we have triaxial cell has been developed; to test soil specimen, cubical in shape, with separate chambers for applying σ_2 and σ_3 separately.



Relation between shear strength parameters and principal stresses at failure



Mohr circle of failure is drawn for specimen subjected to triaxial compression test. Point A is the pole of Mohr circle. At the failure plane and $\angle AOC = \phi$ is the angle made by failure plane with the major principal plane. Let the failure envelope be produced backwards to intersect σ -axis at c .

In the right angled triangle OFC

$$\sin \phi = \frac{OC}{FC}$$

$$OC = \text{radius of Mohr circle} = \frac{(\sigma_1' - \sigma_3')}{2}$$

$$FC = FO + OC$$

$$OC = OA + AC = \sigma_3' + \frac{(\sigma_1' - \sigma_3')}{2} = \frac{(\sigma_1' + \sigma_3')}{2}$$

In the right angled triangle EOF

$$\frac{FO}{EO} = \cot \phi'$$

$$FO = EO \cot \phi' = c' \cot \phi'$$

$$FC = OC + FO = \frac{(\sigma_1' + \sigma_3')}{2} + c' \cot \phi'$$

$$\text{Hence } \sin \phi' = \frac{OC}{FC} = \frac{(\sigma_1' - \sigma_3')/2}{(\sigma_1' + \sigma_3')/2 + c' \cot \phi'}$$

$$\text{simplifying, } \frac{(\sigma_1' - \sigma_3')}{2} = (\sigma_1' \cos \phi' + c' \sin \phi')$$

Further, we can write

$$a_1' - a_3' = (a_1 + a_3) \sin \phi' + 2c' \cos \phi'$$

Combining with

$$a_1' (1 - \sin \phi') = a_3' (1 + \sin \phi') + 2c' \cos \phi'$$

$$\cos \phi'$$

$$a_1' = \left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right) a_3' + 2c' \left(\frac{\cos \phi'}{1 - \sin \phi'} \right)$$

It can be proved by trigonometry that

$$\left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right) = \tan^2 \alpha \text{ and } \left(\frac{\cos \phi'}{1 - \sin \phi'} \right) = \tan \alpha$$

$$\text{So } a_1' = \tan^2 \alpha a_3' + 2c' \tan \alpha$$

where

Finally we get the following relation

$$a_1' = a_3' \tan^2 \alpha + 2c' \tan \alpha$$

$$\text{Not 1: } a_1' = a_3' N_p + 2c' \sqrt{N_p}$$

$$\text{Not 2: } a_1' = a_3' \tan^2 \alpha + 2c' \tan \alpha$$

Not 2: we can show that

Referring right angled triangle CDF, $\angle DCF = (90 - \phi')$

$$\angle CAD = \angle CDA = \alpha$$

$$\angle CAD + \angle CDA + \angle ACD = 180^\circ$$

$$\alpha + \alpha + (90 - \phi') = 180^\circ$$

$$2\alpha = 90^\circ + \phi'$$

$$\alpha = 45^\circ + \frac{\phi'}{2}$$

is found very useful for analytical solution of problems. In undrained test, pore pressure is measured by means of pore pressure measuring apparatus a_1' and a_3' are given by

$$a_1' = a_1 - p$$

$$a_3' = a_3 - p$$

In drained test $\dot{\epsilon}_v = 0$ so that $a_1' = a_1$ and $a_3' = a_3$

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

$$\frac{(\sigma_1' - \sigma_3')}{2} = \frac{(\sigma_1' + \sigma_3')}{2} \tan \psi + d$$

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 $\frac{(\sigma_1' - \sigma_3')}{2}$ we get a set 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.

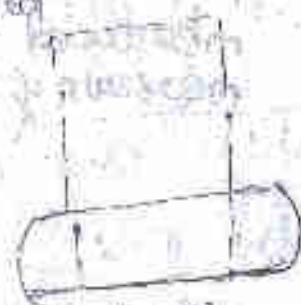
Depending on the soil type and its physical properties a soil specimen can exhibit one of three failure patterns indicated



(a)



(b)



(c)

① It is an example of a brittle failure with a well defined failure plane and little lateral bulging.

② It is shown a semi-plastic failure with shear cones and some lateral bulging.

③ It 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 up to 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.

① 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.

② 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.

③ 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.

④ Precise measurements of pore pressure and volume change are possible during the test.

⑤ The stress measurements at any stage of plane within the specimen can be determined.

⑥ 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 therefore in the laboratory shear test the drainage condition expected to the field for a particular problem should be simulated. Based on drainage conditions the shear tests are classified as

(i) consolidated undrained or simply undrained test (UU test)

(ii) consolidated undrained test (CU test)

(iii) consolidated drained test or simply drained test (CD test)

(i) Undrained test

Drainage is not permitted throughout the test in the case of undrained 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 or simple initial or dissipation of pore pressure in later stage the test is also called quick test.

(ii) Consolidated undrained test

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.

$$q_u = \frac{F}{A_f}$$

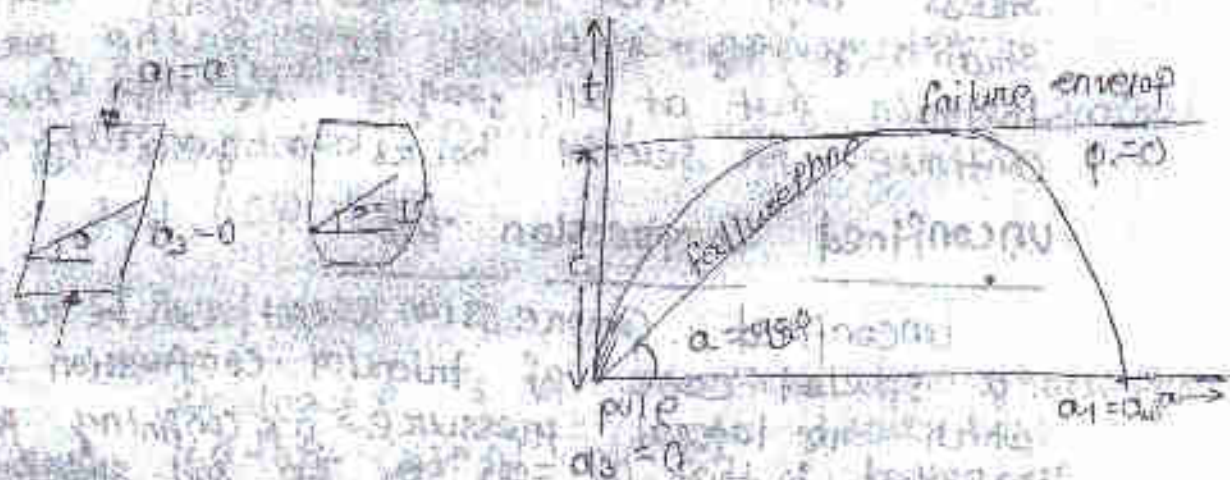
F = axial compressive force at failure

A_f = corrected area of cross section of specimen at failure = $\frac{A_0}{1 - e}$

A_0 = initial area of cross section of specimen.

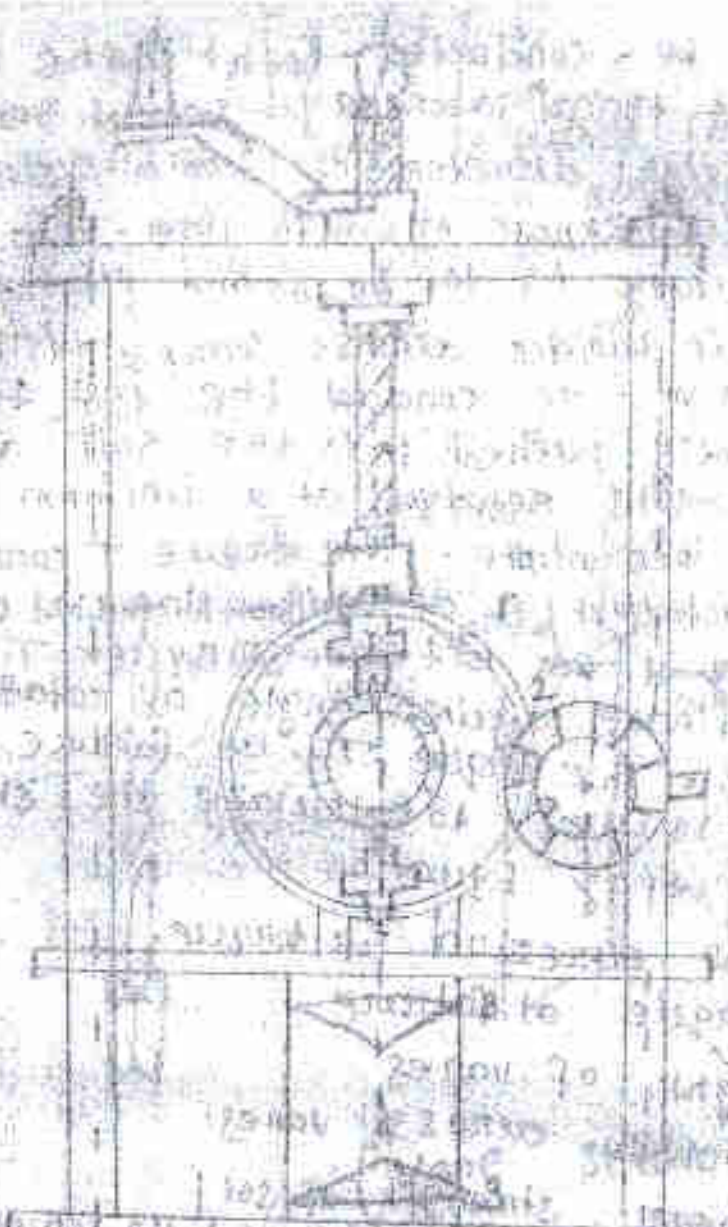
$$\epsilon = \frac{\Delta l}{l} = \text{axial strain at failure point}$$

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 unconfined 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.



The angle α which the failure plane makes with the horizontal is measured after carefully sketching 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 load versus deformation can be plotted.



- ① proving ring dial gauge
- ② Deformation dial gauge
- ③ Conical seatings
- ④ soil specimen

(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 at 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 rod which is rotated by a combination worm gear and crown wheel. The vanes

shear test can be conducted both in the laboratory and in field. A typical laboratory set of vanes has 80 mm height diameter of 12 mm across vanes with blade thickness of 0.5 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 noted. Torque T is plotted as ordinate against angle of rotation θ as abscissa. The torque T at failure in soil is used to calculate the shear strength τ using Equation

Derivation of expression for torque.

Let T_f = Torque at failure

H = height of vanes

d = diameter across vanes

τ = 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 rotate 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

$$T_f = (\pi d H \tau) \frac{d}{2} + 2 \int_0^{H/2} (2\pi r d \tau) r dr$$

$$= \frac{\pi d^2 H \tau}{2} + 4\pi d \tau \int_0^{H/2} r^2 dr$$

$$= \frac{\pi d^2 H \tau}{2} + 4\pi d \tau \left[\frac{r^3}{3} \right]_0^{H/2}$$

$$= \frac{\pi d^2 H \tau}{2} + \frac{\pi d^3 \tau}{3}$$

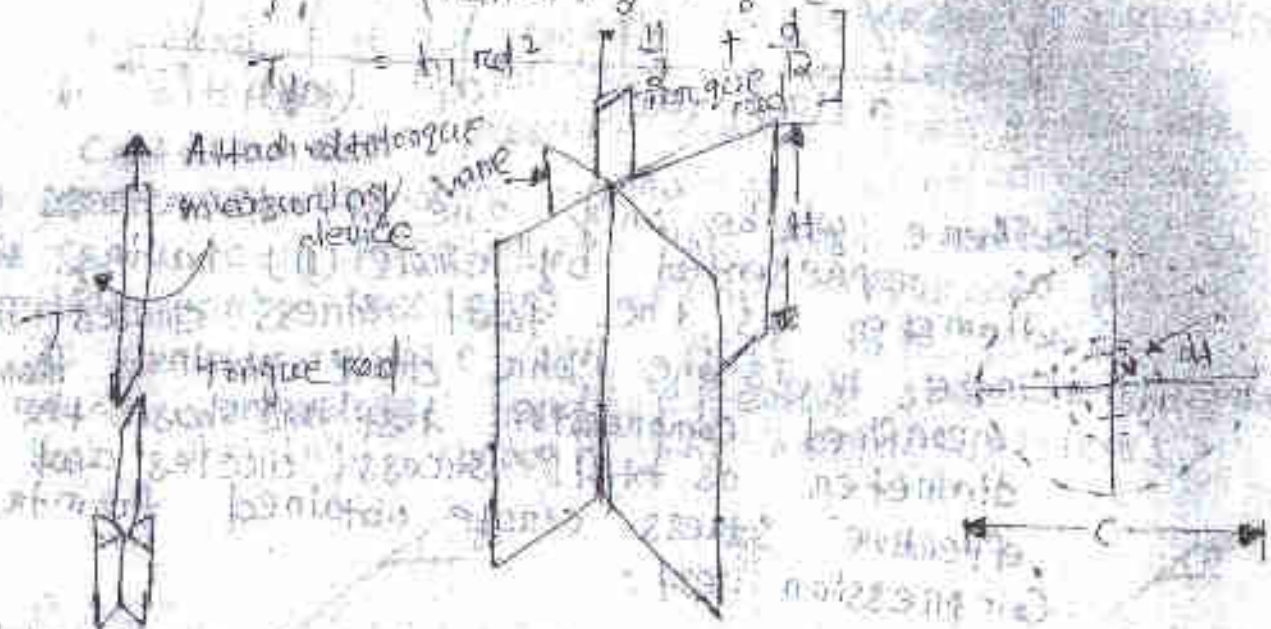
$$= \pi d z \left[\frac{u}{2} + \frac{q}{6} \right]$$

Assail 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

$$T_1 = (\pi d H t_1) \frac{d}{2} + \int_0^H (\pi r d t_1) r dr$$

$$T_1 = \pi r d^2 \left[\frac{H}{2} + \frac{d}{6} \right]$$

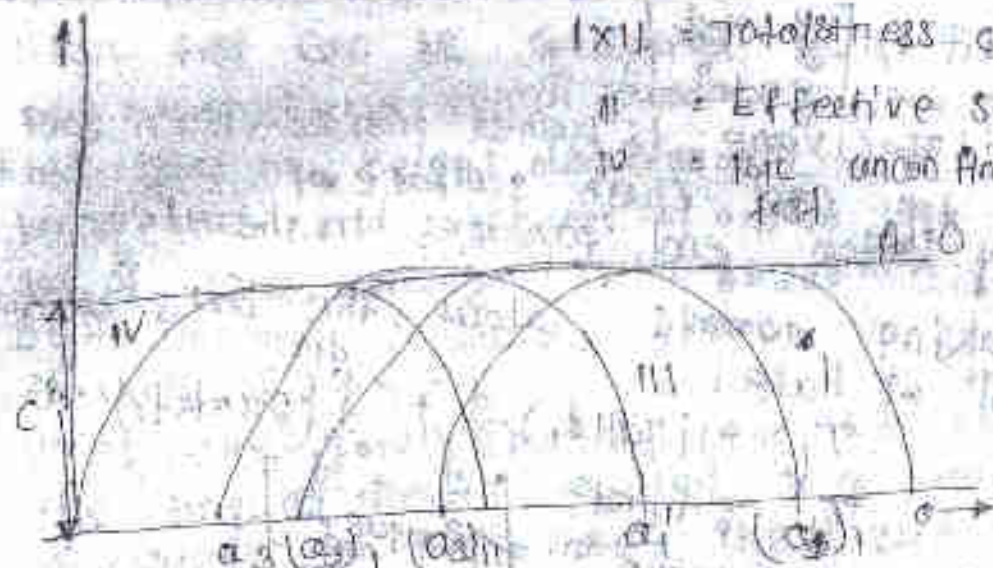


(Vane shear test)

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 $\phi_a = 0$ and $c_1 = c_2$. It is clear that $(\sigma_1)_1 - (\sigma_3)_1 = (\sigma_1)_2 - (\sigma_3)_2$



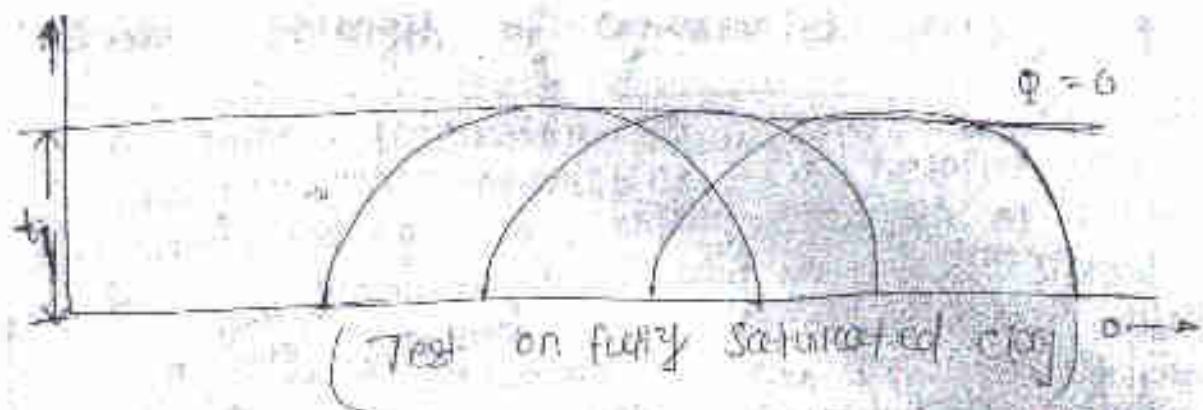
I, II = Total stress circles

III = Effective stress circle

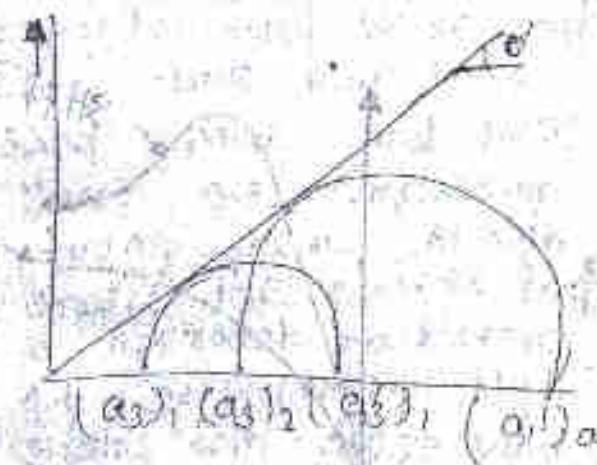
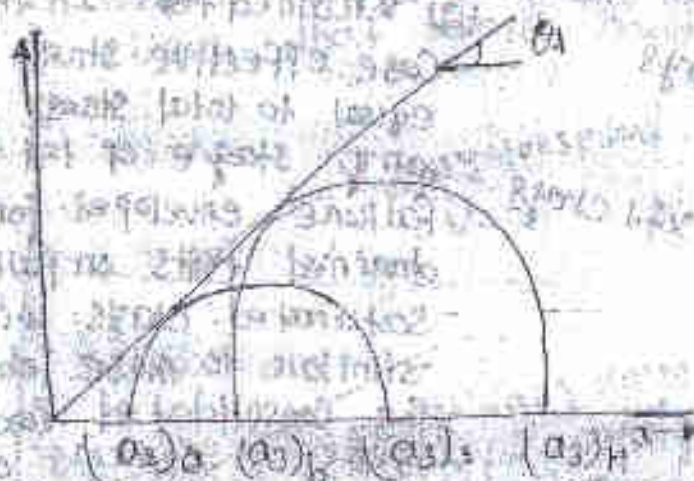
IV = Total unconfined compression test

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, fully saturated clay specimen initially consolidated with some 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 $c_u = c_a$. Further if the tests are conducted with specimen initially consolidated with same but increased cell pressure and then sheared with different cell pressure the failure envelope obtained is still horizontal with $\phi_u = 0$ but c_u will be greater than in the previous case.



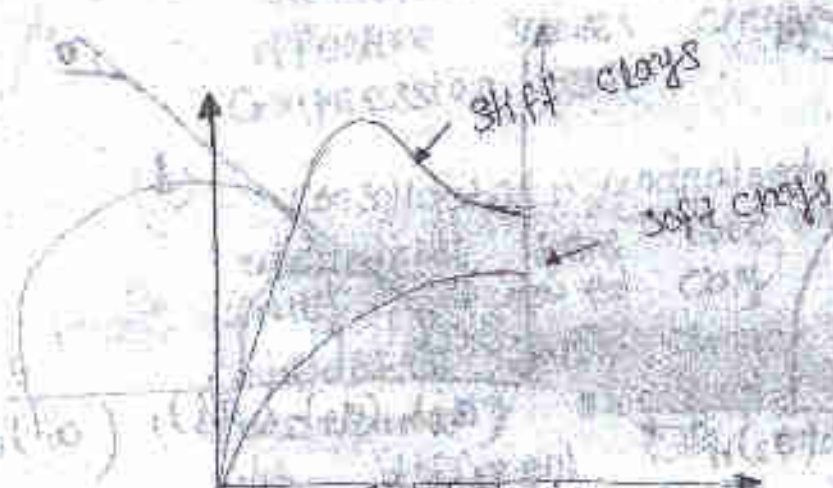
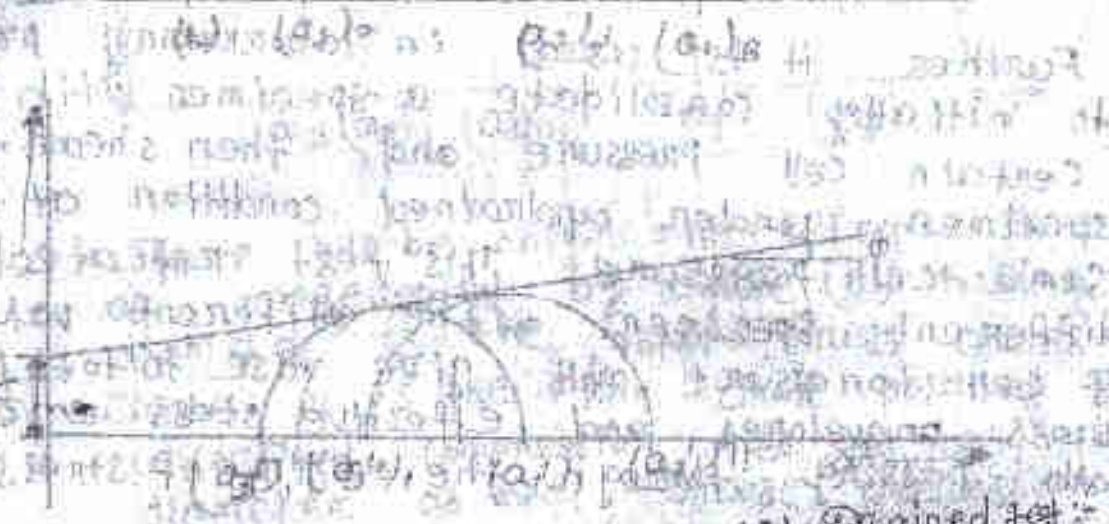
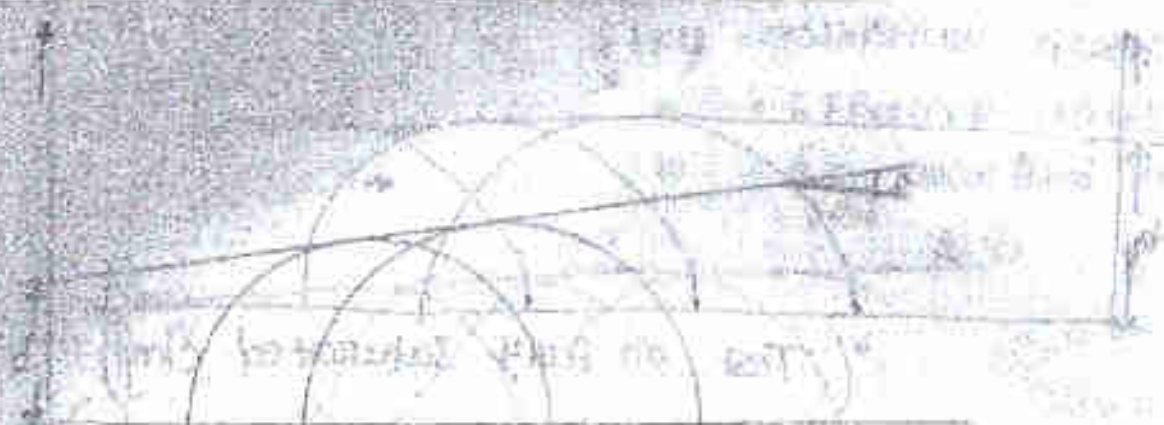
Further it is usual in laboratory practice to initially consolidate a specimen with a certain cell pressure and then shear the specimen under undrained condition at the same cell pressure. This test repeated for different specimens with the different value of cell pressure will give rise to total stress envelopes and effective stress envelopes both passing through the origin of stress.



NOTE: - $\phi' > \phi$

Effective stress circle is shifted to left.

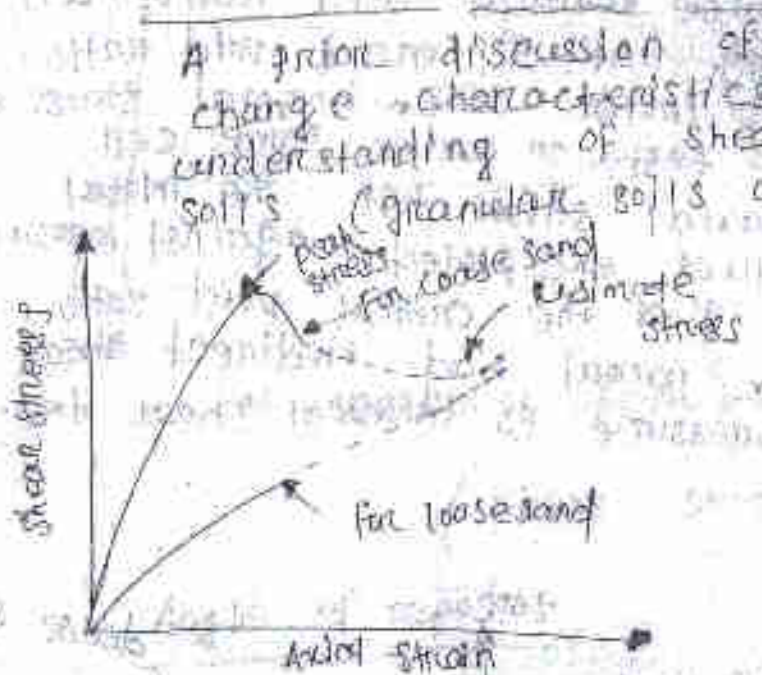
When consolidated undrained tests are conducted on preconsolidated fully saturated clay specimens the failure envelope will have cohesion intercept for both total stress and effective stress plotting s , with apparent cohesion c_a greater than effective cohesion c' . ϕ may be slightly greater or smaller than ϕ' corresponding to any total stress circle, the effective stress circle will be found shifted to right.



Typical stress-strain curves for clays

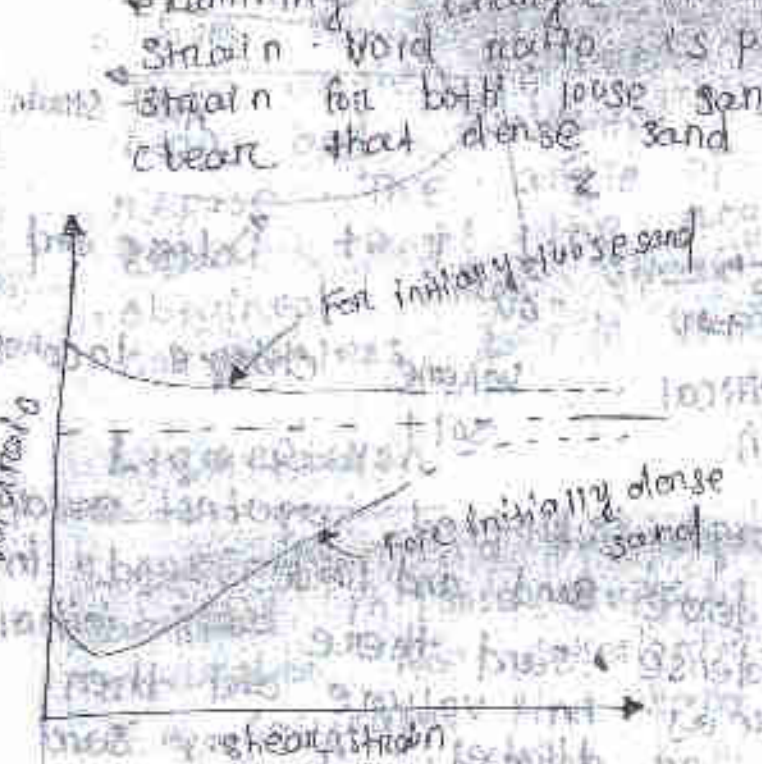
(c) Drained test:- In this case effective stress is equal to total stress at any stage of test. The failure envelopes for drained tests on fully saturated clays will be similar to those obtained for consolidated undrained tests in terms of effective stress. For normally consolidated condition the failure envelope passes 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 soils (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.

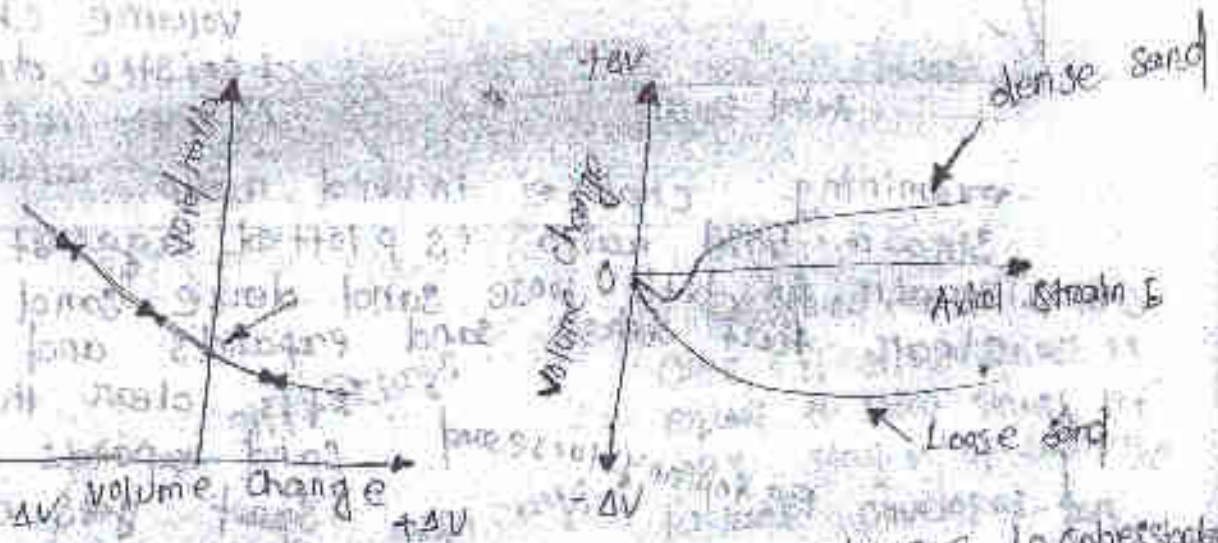
Volume change characteristics during shear is best understood by examining change in void ratio with increasing strain.



Void ratio is plotted against shear strain for both loose sand dense sand. It is clear that dense sand expands and loose sand contracts 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 indirect shear test or cell pressure is triaxial shear test.

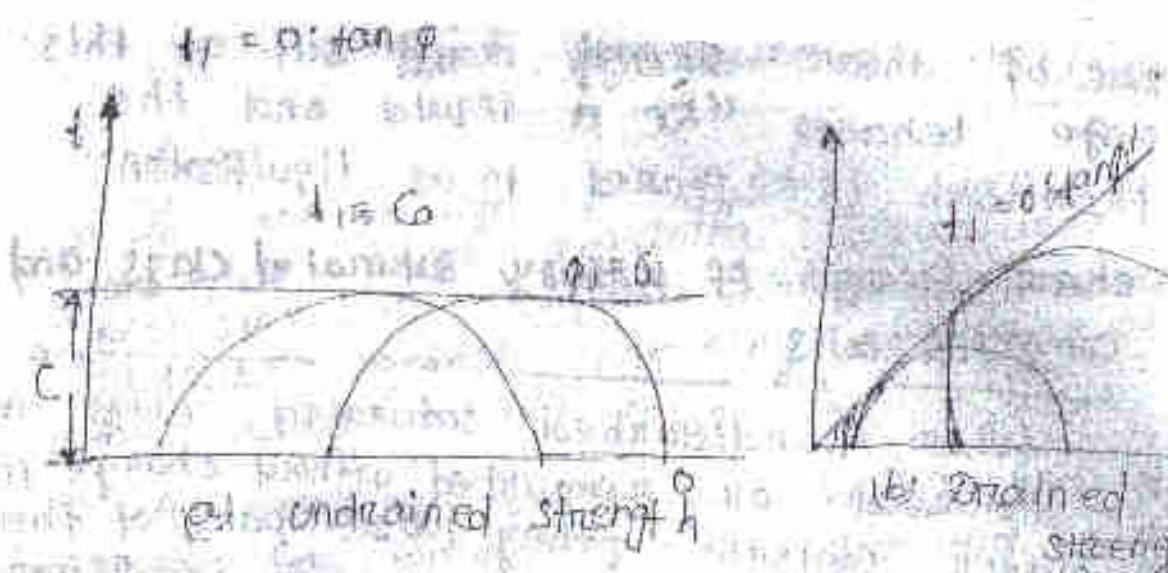


Determination of critical void ratio

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 sand expands or dilates. A loose sand gets compressed when sheared.

Undrained strength of saturated sand

If drained tests are conducted on saturated sand specimen, initially at the same density index, the failure envelope will be approximately a straight line passing through the origin of stresses, with effective stresses being equal to total stresses, i.e., $u = 0$. The undrained shear strength is given by



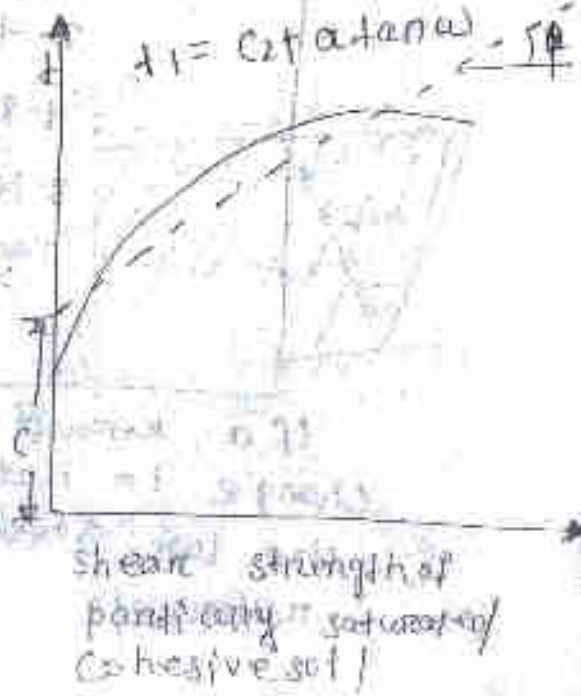
(Shear strength of cohesionless soils)

Angle of repose

When a cohesionless soil is poured from a small height it forms a heap. The angle between the sloping side of the heap and horizontal is referred to as 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 occur 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



loss of shear strength. The soil at this stage behaves like a liquid and the phenomenon is referred to as liquefaction.

shear strength of partially saturated clays and composite soils

When undisturbed saturated clays are disturbed or remoulded without change in water content, they lose 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 undisturbed condition.

$$\text{Sensitivity} = \frac{t_i (\text{undisturbed})}{t_i (\text{remoulded})} = \frac{c_u (\text{undisturbed})}{c_u (\text{remoulded})}$$

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.

> 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 at different elevations on either side of it.

> The material retained or supported by the structure is called back fill, which may have its 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 & its 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

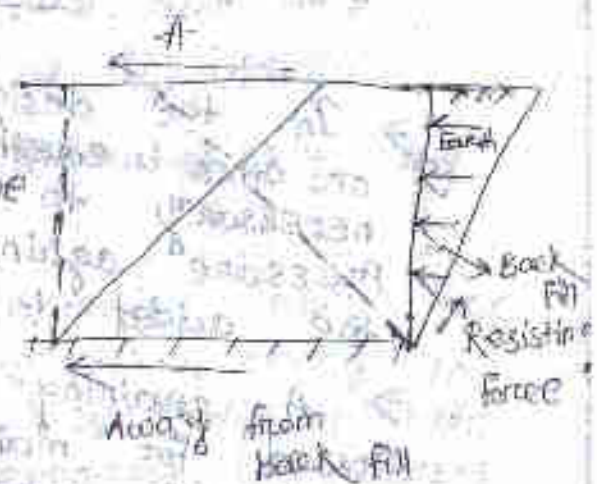
(A) Earth pressure at rest

> The pressure at which there is no deformation in the retaining wall, the pressure due to earth is known as earth pressure at rest.



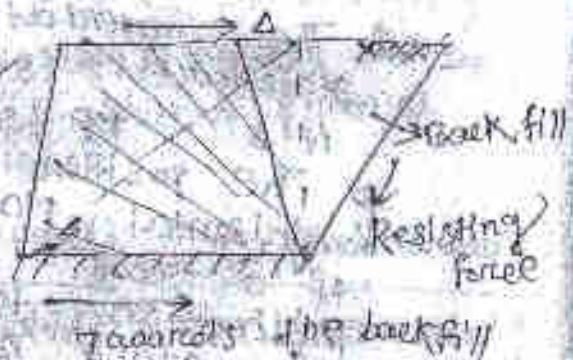
1.1 Active Earth Pressure:-

The minimum pressure by which the retaining wall deformed away from the back fill, that pressure is known as Active earth pressure (p_a)



2. 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 (p_p)



19 Jan 2021

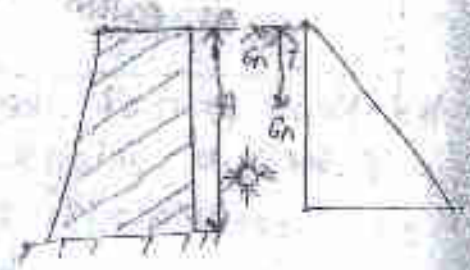
Calculation of Earth Pressure:-

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)

- ⇒ There will be no shear stress.
- ⇒ The lateral strain (ϵ_h) in the horizontal direction is given by:-



$$\epsilon_h = \frac{1}{E} [\sigma_h - \mu (\sigma_h + \sigma_v)] \rightarrow \text{eqn (1)}$$

The earth pressure at rest corresponding to this condition having zero lateral strain

$$\epsilon_h^f = \frac{1}{E} [\sigma_h - \mu (\sigma_h + \sigma_v)]$$

$$\Rightarrow \sigma_h - \mu (\sigma_h + \sigma_v) = 0$$

$$\sigma_h = \mu (\sigma_h + \sigma_v)$$

or

$$\frac{\sigma_h}{\sigma_v} = \frac{\mu}{1-\mu} = K_0$$

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 / σ_v) = $K_0 \sigma_v$

$$\Rightarrow p_0 = K_0 \sigma_v \quad (\sigma_v = \gamma z)$$

$$\Rightarrow p_0 = K_0 \gamma z$$

$\Rightarrow$ The total pressure of γH (at the base $\Rightarrow p_0 = K_0 \gamma H$ of retaining wall)

The total pressure p_0 per unit length for vertical height H is given by:-

$$p_0 \text{ per unit length} = \frac{1}{2} K_0 \gamma H^2$$

$$\text{where } K_0 = 1 - \sin \phi$$

where ϕ = Angle of internal friction

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	0.9

A rigid retaining wall 6m height is restrained from yielding. The backfill consist of cohesionless soil having $\phi = 26^\circ$, $\gamma = 19 \text{ kN/m}^3$. Compute the total earth pressure per meter length of wall?

Solⁿ

Data given: $H = 6 \text{ m}$

$$\phi = 26^\circ$$

$$\gamma = 19 \text{ kN/m}^3$$

$$P_0 = ?$$

$$P_0 = \frac{1}{2} K_0 \gamma H^2$$

$$K_0 = 1 - \sin \phi = 1 - \sin(26^\circ) = 0.561$$

$$P_0 = \frac{1}{2} \times 0.561 \times 19 \times 6^2$$

$$= 191.86 \text{ kN/m}^2$$

Active Earth pressure - Rankine's Theory:-

- ⇒ Rankine's Theory of lateral earth pressure is applied to uniform cohesionless soils only.
- ⇒ Lateral 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

(a) submerged backfill

(c) Backfill with uniform surcharge

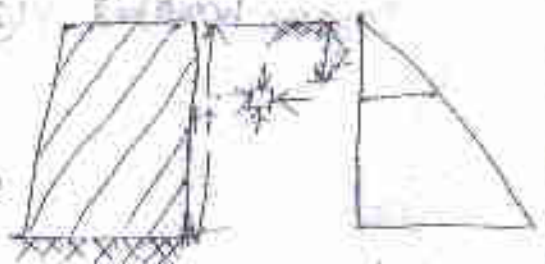
(d) Backfill with sloping surface

(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 $z = H$

$$P_a = K_a \gamma H$$

where H = Height of retaining wall

γ = unit weight of dry soil

K_a = Coefficient of active earth pressure.

$$K_a = \frac{\sigma_h}{\sigma_v} = \tan^2 \left(45^\circ - \frac{\phi}{2} \right) = \frac{1 - \sin \phi}{1 + \sin \phi}$$

⇒ Total earth pressure per unit length

$$P_a = \frac{1}{2} K_a \gamma H^2$$

⇒ 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 up of 2

Categories

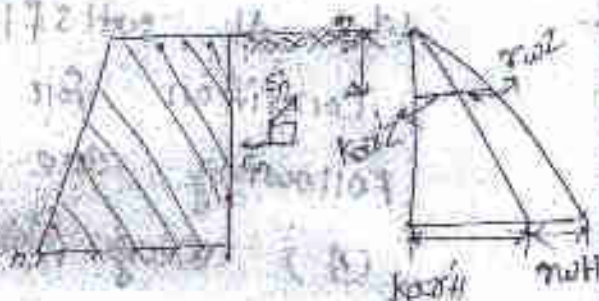
(i) submerged weight (γ') of soil

(ii) unit weight of water

⇒ Active earth pressure of submerged

backfill ($z = H$)

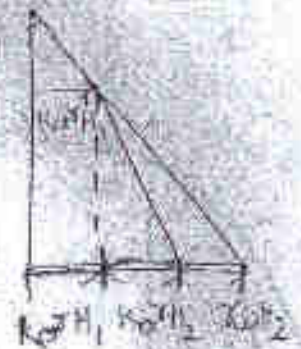
$$P_a = K_a \gamma' H + \gamma_w H$$



21 Jan 2021

⇒ If the free water stands to both sides of the wall, the water pressure need not be considered & the net lateral earth pressure is given by :-

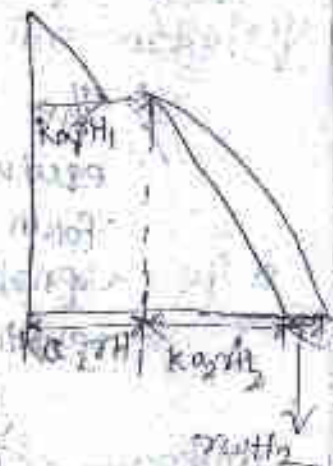
$$p_a = K_a \gamma H$$



⇒ 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

$$p_a = K_a \gamma H_1 + K_a \gamma' H_2 + \gamma_w H_2$$

⇒ If there is different in ϕ_1 & ϕ_2 respectively, the earth pressure coefficient is also different i.e. K_{a1} & K_{a2} .



⇒ The lateral earth pressure intensity at the base of the wall is given

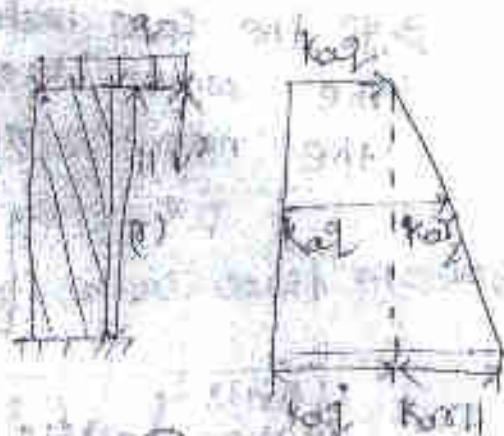
$$p_a = K_{a2} \gamma_2 H_2 + K_{a2} \gamma_1' H_2 + \gamma_w H_2$$

NOTE

As ϕ decreases K_a will be increases.

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 kaq . Hence the lateral earth pressure at a depth z will be :- $P_a = kaq + k\sigma_z$



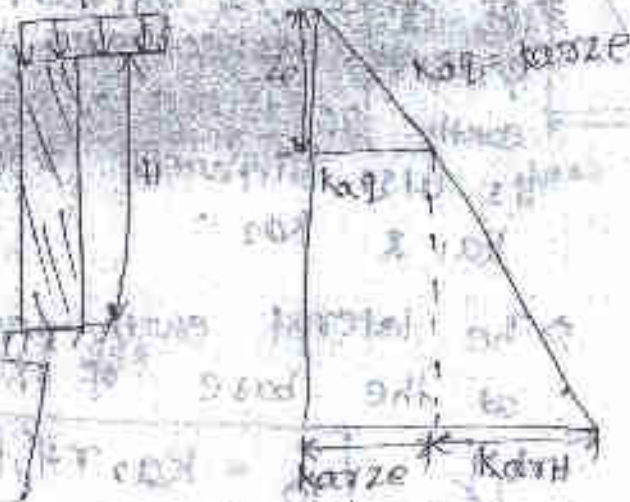
(or)



> The lateral earth pressure at a depth H will be ($z=H$) :- $P_a = kaq + k\sigma_H$

The height of fill z_e equivalent to the unit form surcharge intensity q is given by the relation :- $kaq = k\sigma_{ze}$

$$z_e = \frac{kaq}{k\sigma_z} = \frac{q}{\gamma}$$



NOTE

Effect of surcharge due to q = pressure due to a fill of earth z_e above the ground surface.

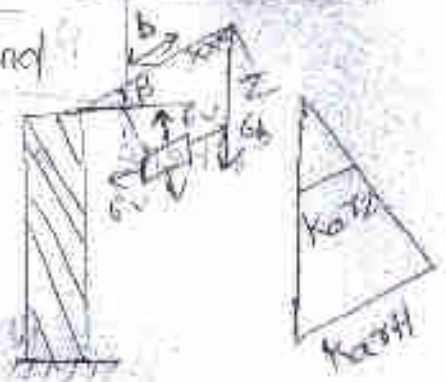
The lateral earth pressure at the base of the wall:-

$$p_a = K_a \gamma z + K_a \gamma H$$

$$p_a = K_a \gamma (z + H)$$

(E) Back fill with sloping surface- 22 Jan 2021

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 active earth pressure at depth z is:-

$$p_a = K_a \gamma z$$

pressure of back fill soil is γ (unit weight of soil).
 Total earth pressure per unit length, of a height H is -

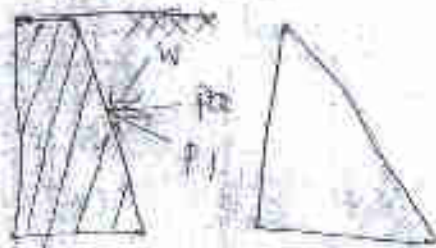
$$P_a = \frac{1}{2} K_a \gamma H^2$$

 It acts at $H/3$ above the base in direction parallel to the surface.

$$K_a = \frac{\cos \beta}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

10) Inclined back & surcharge - Retained soil

If the retaining wall with an inclined back with horizontal ground surface then the total earth pressure (P_a) will be -



$$P_a = \sqrt{P_1^2 + W^2}$$

Where P_1 = horizontal pressure

$$= \frac{1}{2} \times K_a \times \gamma H^2$$

W = weight

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

If there is inclined back with surcharge & inclined ground surface then the total earth pressure



$$P_a = \sqrt{P_1^2 + W^2}$$

$$P_1 = \frac{1}{2} K_a \gamma H^2$$

$$K_a = \frac{\cos \phi - \cos \phi'}{\cos \phi + \cos \phi'}$$

11) 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/m^3 . What will be the water table rise upto ground level? Take the saturated unit weight of sand is 22 kN/m^3 .

3/1

Given -

$$\phi = 30^\circ, \quad \gamma = 18 \text{ kN/m}^3$$

$$\gamma' = 22 \text{ kN/m}^3$$

$$H = 8 \text{ m}$$

25 June 2021

Case - I

Dry Soil:-

$$p_a = K_a \gamma H$$

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$= \frac{1 - \sin(30^\circ)}{1 + \sin(30^\circ)} = \frac{1}{3}$$

$$p_a = \frac{1}{3} \times 18^6 \times 8$$

$$= 48 \text{ kN/m}^2$$



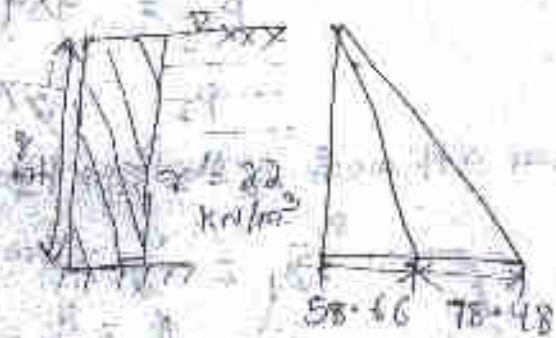
Case - II Submerged Soil:-

$$p_a = K_a \gamma' H + \gamma_w H$$

$$= \left(\frac{1}{3} \times 23 \times 8 \right) + (9.81 \times 8)$$

$$= 58.66 + 78.48$$

$$= 137.14 \text{ kN/m}^2$$



Q.20

A retaining wall 4m high has a smooth vertical back. The backfill has a horizontal surface in level with top of the wall. There's uniformly distributed surcharge load of 12 kN/m² intensity over the backfill. The unit weight of the backfill is 18 kN/m³ & cohesion 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 $\gamma_w = 9.81$.

Solⁿ

Given

$$\gamma = 18 \text{ kN/m}^3$$

$$\phi = 30^\circ$$

$$H = 4 \text{ m}$$

$$q = 12 \text{ kN/m}^2$$

Ex-1

$$K_0 = \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$= \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ}$$

$$= \frac{1 - 0.5}{1 + 0.5} = \frac{0.5}{1.5} = \frac{1}{3}$$

min $P_0 = K_0 \gamma H + k_a q$

$$= \left(\frac{1}{3} \times 18 \times 4 \right) + \left(\frac{1}{3} \times 12 \right)$$

$$= 24 + 4$$

$$= 28 \text{ kN/m}^2$$

⇒ Total earth pressure per unit length is :-

$$P_1 = 4 \times 4 = 16 \text{ kN/m}^2$$

$$P_2 = \frac{1}{2} \times 24 \times 4 = 48 \text{ kN/m}^2$$

$$P = P_1 + P_2 = 16 + 48 = 64 \text{ kN/m}^2$$

$$\bar{Z}_1 = \frac{4}{2} = 2 \text{ m}$$

$$\bar{Z}_2 = \frac{1}{3} \times \frac{4}{2} = 1.33 \text{ m}$$

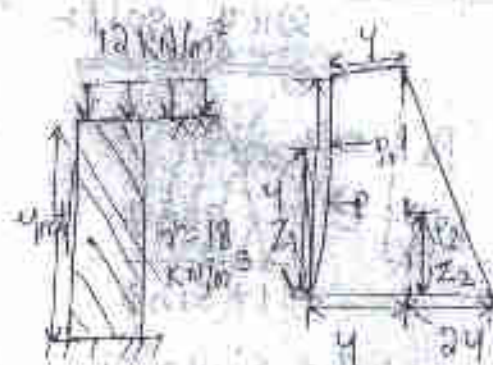
$$P_{total} \bar{Z} = P_1 \bar{Z}_1 + P_2 \bar{Z}_2$$

$$= (16 \times 2) + (48 \times 1.33)$$

$$= 32 + 64$$

∴ Total earth pressure = 96 kN/m

∴ Resultant force acts at a distance of 1.33 m from the base.

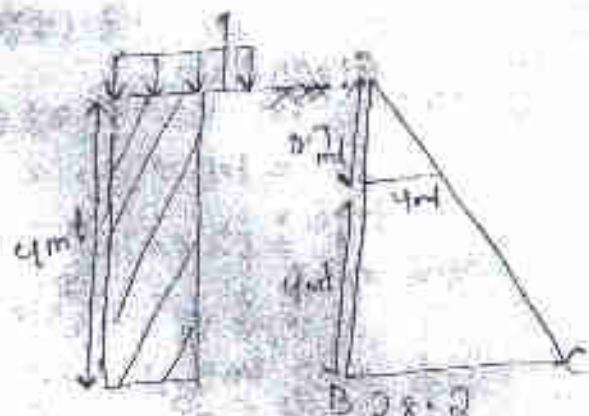


Case-II

$$\gamma_{zo} = k \gamma$$

$$\Rightarrow z_e = \frac{2}{3}$$

$$\Rightarrow z_e = \frac{2}{3} \times \frac{12}{18} = 0.66 \text{ m}$$



Active earth pressure at the base of the wall is :-

$$P_a = k_a \gamma (z_e + H)$$

$$= \frac{1}{3} \times 18 \times (0.66 + 4)$$

$$= 6 \times 4.66$$

$$= 28.2 \text{ kN/m}^2$$

Total active earth pressure per unit length is :-

$$P = \frac{1}{2} (4 \times 28.2) \times 4$$

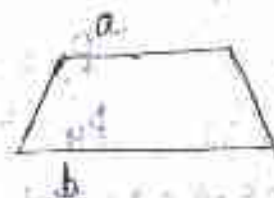
$$= 64.4 \text{ kN/m}$$

The distance of point of application from the base is :-

$$\bar{x} = \frac{1}{3} \left(\frac{2a + b}{a + b} \right) \times \frac{2}{3}$$

$$= \frac{1}{3} \times \frac{2 \times 4 + 28.2}{4 + 28.2} \times \frac{2}{3}$$

$$= 1.49 \text{ m above the base of wall}$$



Q In the above problem if the water table rise behind the wall to an elevation of 1.4 m below the ground surface determine the total active earth pressure & its point of application. Take submerged unit weight of sand is 12 kN/m³. Assume there is no change in angle of shearing resistance due to submergence?

Given data:

$$H = 1.5 \text{ m}$$

$$q = 12 \text{ kN/m}^2$$

$$\gamma = 18$$

$$\phi = 30^\circ$$

Case-1

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1 - 0.5}{1 + 0.5} = \frac{0.5}{1.5} = \frac{1}{3}$$

$$p_1 = (\text{due to surcharge})$$

$$p_2 = (\text{due to moist soil/dry soil})$$

$$p_a = K_a q + K_a \gamma H_1$$

$$= \left(\frac{1}{3} \times 12 \right) + \left(\frac{1}{3} \times 18 \times 1.5 \right)$$

$$= 4 + 9 \quad p_a = K_a \gamma H_1 + \frac{1}{3} \times 18 \times 1.5 = 9 \text{ kN/m}^2$$

$$= 13 \quad p_1 = K_a q = \frac{1}{3} \times 12 = 4 \text{ kN/m}^2$$

$$p_3 = 2.5 \quad p_4 = K_a \gamma H_2 (\text{due to submerged soil})$$

$$= \frac{1}{3} \times 12 \times 2.5$$

$$= 10 \text{ kN/m}^2$$

$$p_5 = (\text{due to water})$$

$$= 9.81 \times 2.5$$

$$= 24.525$$

$$\text{Total active earth pressure} = p_1 + p_2 + p_3 + p_4 + p_5$$

$$P_{\text{total active earth pressure}} = 4 + 9 + 2.5 + 10 + 24.525 = 49.525$$

$$P_1 = P_1 H = 4 \times 4 = 16 \text{ kN/m}^2$$

$$P_1 = P_1 H$$

$$= 4 \times 4$$

$$= 16 \text{ kN/m}^2$$

(From Page)

$$P_2 = \frac{1}{2} \times P_2 \times H_1 = \frac{1}{2} \times 9 \times 1.5$$

$$= 6.75 \text{ kN/m}^2 \text{ at } Z_2 = 2.5 + \frac{1.5}{3}$$

$$= 3 \text{ m}$$

$$P_3 = 9 \times 2.5 = 22.5 \text{ kN/m}^2 \text{ at } Z_3 = \frac{2.5}{2} = 1.25 \text{ m from base}$$

$$P_4 = \frac{1}{2} \times 10 \times 2.5 = 12.5 \text{ kN/m}^2 \text{ at } Z_4 = \frac{2.5}{3} = 0.833 \text{ m from base}$$

$$P_5 = \frac{1}{2} \times 24.52 \times 2.5 = 30.65 \text{ kN/m}^2, \text{ at } Z_5 = \frac{2.5}{3} = 0.833 \text{ m from base}$$

$$P = P_1 + P_2 + P_3 + P_4 + P_5$$

$$= 16 + 6.75 + 22.5 + 12.5 + 30.65$$

$$= 88.4 \text{ kN/m}$$

$$\bar{Z} = \frac{P_1 Z_1 + P_2 Z_2 + P_3 Z_3 + P_4 Z_4 + P_5 Z_5}{P_1 + P_2 + P_3 + P_4 + P_5}$$

$$= \frac{(16 \times 2) + (6.75 \times 3) + (22.5 \times 1.25) + (12.5 \times 0.83) + (30.65 \times 0.83)}{88.4}$$

$$= 1.314 \text{ m from base}$$