V.V. Imp.

** Two criteria for designing foundation –

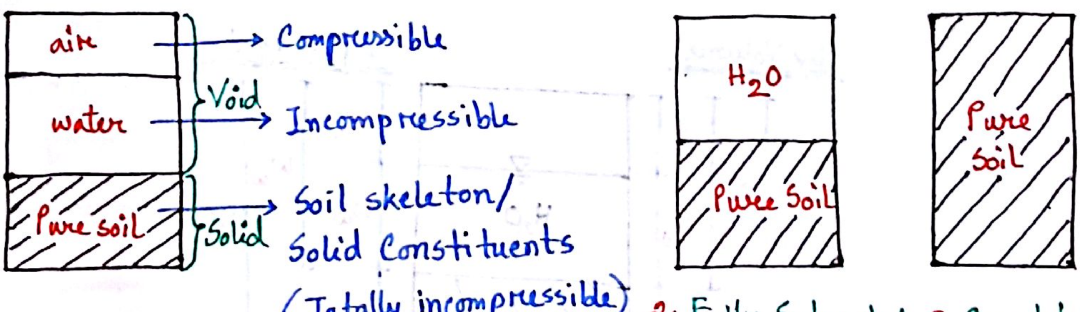
1. Providing adequate factor of safety (F.S.) against bearing capacity failure; for soil F.S. = 2–3.

2. The settlement must be within some tolerable limit and for conventional type of structure, settlement = 1".

Weight-Volume Relationships of Soil Aggregates –

• Diagram of a sample soil – From microscopic point of view, in a soil sample, there are soil particles and voids. These voids may be filled with only air, only water or both air and water.

So the diagrams of a soil sample are as follow →



1. Partially Saturated Soil Sample

2. Fully Saturated Soid Sample

3. Completely Drey Soil Sample

⇓

Soil combination of these 3-phases is known as 3-Phase Structure.

• All the figures (1), (2) and (3) are called the diagramic representation of real soil into its constituents (solid constituents, water and air) ⇒ are called also the **Phase Diagrams.**

Definitions for weight-volume relationship :

1. **Moisture Content or Water content, w** :— This is expressed in %. The water content of soil is defined as —

$$\boxed{w(\%) = \frac{W_W}{W_s} \times 100}$$

where, W_W = the weight of water matter
and W_s = the weight of oven-dry solid matter

• The standard oven temperature is 105 to 115°C

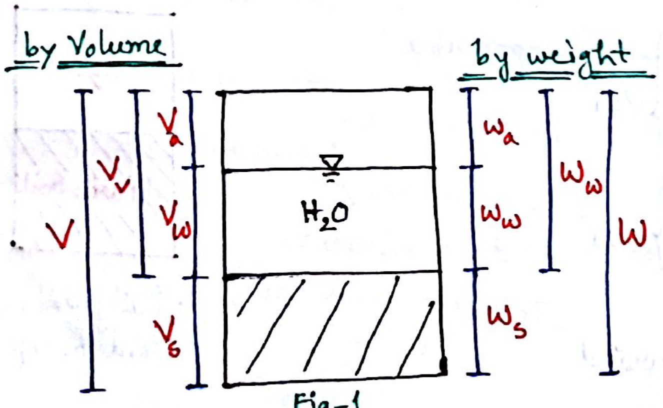


Fig-1

for our case in this course, $W_a = 0$
So total weight of voids = W_w
and total volume of voids = V_v

So, the total weight of soil sample, W is –

$$\boxed{W = W_w + W_s}$$

- This figure-1 is called the phase diagram in water content expression.

- The moisture content value can be more than 100%.

- This is the first parameter determined after collecting the soil sample.

2. Void Ratio (e) :– This is the very important index property of soil. Void ratio is commonly used in settlement computation to refer the void space to an unchanging denominator.

Void ratio is defined as –

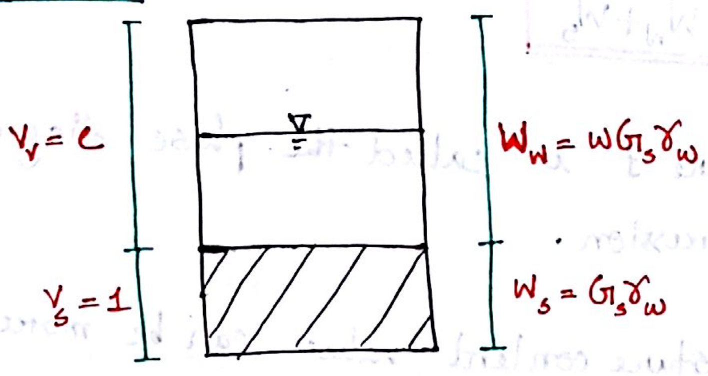
$$\boxed{e = \frac{V_v}{V_s}}$$ → It is expressed in decimal and could have value more than 1.

- for our convenience, we assume –

$V_s = 1$

So the modified diagram is –

by volume   by weight



Here, $V_s = 1$

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

$\therefore V_v = e \times 1$

$\Rightarrow \underline{V_v = e}$

and we know, weight = vol^m × unit weight.

here, G_s = specific gravity of soil

$= \underline{2.6 - 2.7}$

and γ_w = unit weight of water

$\therefore$ unit weight of soil $= G_s \gamma_w$

$\therefore$ Weight of solid, $W_s = V_s \times G_s \gamma_w = 1 \times G_s \gamma_w$

or, $\underline{W_s = G_s \gamma_w}$

Again, moisture content, $\omega = \frac{W_w}{W_s}$

$\Rightarrow W_w = W_s \omega$

$\Rightarrow \underline{W_w = \omega G_s \gamma_w}$

So, total volume, $\boxed{V = 1 + e}$ → It is a proportionate value.

So we can write, $\boxed{V \propto (1+e)}$

- Determination of proportionality constant –

We know $V \propto (1+e)$

or, $V = k(1+e)$; where, k = proportionality constant --- (i)

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

or, $V = V_s(1+e)$ --- (ii)

So from theses two eq^{n}s –

$\boxed{k = V_s}$

- Uses of void ratio –
  1. Settlement computation, permeablity determination
  2. Indicates looseness or compactness of soil (sandy type soil).
  3. Compressibility or incompressibility of soil (for clay).

3. Porosity (Percent Void) :– Porosity of a soil sample is defined by –

$$n = \frac{\text{Vol}^m \text{ of voids}}{\text{Total Volume}}$$

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

- for our convenience let us assume –

$V = 1$

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

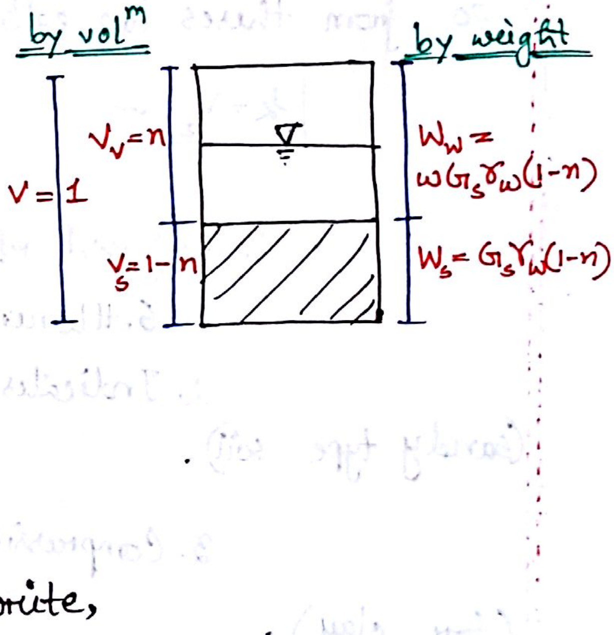
or, $\underline{V_v = n}$

$\therefore \underline{V_s = 1 - n}$

$W_s = V_s G_s \gamma_w$

or, $\underline{W = (1-n) G_s \gamma_w}$

where, G_s = specific gravity of soil



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

or, $W_w = wW_s$

or, $W_w = (1-n) G_s \gamma_w \omega$

Imp. • From this we can write,

void ratio, $\boxed{e = \frac{n}{1-n}}$ ⇒ Void ratio in terms of porosity

or, $e - ne = n$

or, $n(1+e) = e$

$\therefore \boxed{n = \frac{e}{1+e}}$ ⇒ porosity in terms of void ratio

Imp. Basic different between porosity and void ratio :—

\# We know, $e = \frac{V_v}{V_s}$

So in case of void ratio, with application of load only numerator changes i.e. only the volume of voids changes. But the volume of solid skeleton does not change.

So, in void ratio calculation, there is only one variable.

\# We know, $n = \frac{V_v}{V}$

In this case, with application of load, both the volume of total soil sample and the volume of voids change.

So, in porosity, there are two variables.

So for engineering purpose, void ratio is more useful than calculating porosity.

Q. Express phase diagram in porosity or void ratio.

(exam-এ কিছু বলা না থাকলে যেকোন একটায় express করলেই হবে।)

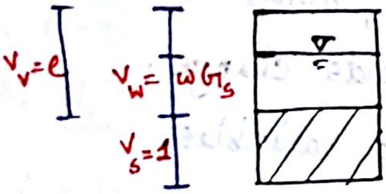
- In this phase diagram, $V \propto \frac{1}{1-n}$

4. Degrees of Saturation :– This indicates to what extent void spaces are filled up with water and it is expressed usually in %.

So the definition of degrees of saturation (S_r) is–

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

- From phase-diagram expressed in void ratio–

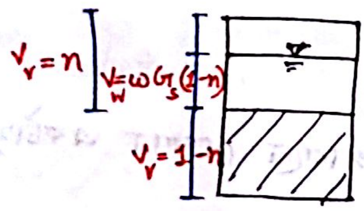


From this figure, we can write–

$$S_r = \frac{V_w}{V_v}$$

or, $\boxed{S_r = \frac{wG_s}{e}}$ ⇒ expressed in decimal

- From phase diagram expressed in porosity–



From this figure, we can write–

$$S_r = \frac{V_w}{V_v}$$

or, $\boxed{S_r = \frac{wG_s(1-n)}{n}}$ ⇒ expressed in decimal

** S_r = 100% when the soil is fully saturated

** S_r = 0% when the soil is completely dry.

Q. Prove, $S_r = \frac{wG_s}{e}$ from the first definitⁿ without using the phase diagram.

• From the first definition without phase diagram —

Void ratio is defined by the ratio of volume of voids to the volume of solid skeleton. So, void ratio, $e = \frac{V_v}{V_s}$

or, $V_v = eV_s$ ----(i)

Moisture content is defined by the ratio of weight of water content to the weight of solid skeleton. So moisture content, $w = \frac{W_w}{W_s}$

or, $W_w = wW_s$

or, $W_w = wG_s\gamma_w V_s$

where, γ_w = unit weight of water

and G_s = specific gravity of soil.

So, $V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w V_s}{\gamma_w}$

or, $V_w = wG_sV_s$ ----(ii)

Now, the definition of degree of saturation expressed in decimal is —

$S_r = \frac{V_w}{V_v}$

or, $S_r = \frac{wG_sV_s}{eV_s}$; [putting the values from (i) & (ii)]

$\therefore \boxed{S_r = \frac{wG_s}{e}}$

[proved].

5. Unit Weight (Weight per unit volume) :-

i. Bulk unit weight –

Definitⁿ of Bulk unit weight, γ_{bulk} is –

$$\gamma_{bulk} = \frac{\text{Total weight}}{\text{Total Volume}}$$

$$\text{or, } \gamma_{bulk} = \frac{W}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{W_s+W_w}{V}$$

$$\text{or, } \gamma_{bulk} = \frac{W_s + W_w}{V}$$

$$\text{or, } \gamma_{bulk} = \frac{1+\frac{W_w}{W_s}}{\frac{V}{W_s}}$$

$$\text{or, } \gamma_{bulk} = \frac{1+w}{\frac{1}{\gamma_d}}$$

$$\therefore \boxed{\gamma_{bulk} = \gamma_d(1+w)}$$

Memorise V.V. Imp.

and $\gamma_{bulk} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e}$

$$= \frac{G_s + wG_s}{1+e}\gamma_w$$

$$\therefore \boxed{\gamma_{bulk} = \frac{G_s + S_r e}{1+e}\gamma_w}$$

ii. Dry unit weight –

Definitⁿ of dry unit weight, γ_d is –

$$\gamma_d = \frac{\text{weight of solid}}{\text{Total Volume}}$$

$$\text{or, } \gamma_d = \frac{W_s}{V}$$

$$\therefore \boxed{\gamma_d = \frac{G_s\gamma_w}{1+e}}$$

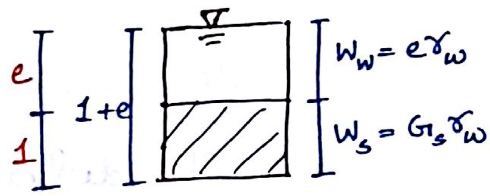
Memorise V.V. Imp.

** This is the minimum unit weight of a soil.

iii. Saturated Unit weight — In this case, voids are completely filled with water.

Definition of saturated unit weight, γ_{sat} is —

$$\gamma_{sat} = \frac{\text{Total Weight}}{\text{Total Vol}^m} = \frac{W}{V}$$



or, $$\gamma_{sat} = \frac{W_w + W_s}{V}$$

Here, the voids are filled with only H_2O, so,

$$W_w = e\gamma_w$$

$$\therefore \gamma_{sat} = \frac{e\gamma_w + G_s\gamma_w}{1+e}$$

or, $$\boxed{\gamma_{sat} = \frac{e+G_s}{1+e}\gamma_w}$$

** This is the maximum unit weight that a soil can have.

iv. Submerged Unit Weight (Effective Unit Weight / Buoyant Unit Weight) — If soil is submerged under water, then with the increase of depth, the unit weight of soil will decrease.

∴ Buoyant unit weight ($\gamma_b / \gamma' / \gamma_{eff}$) is —

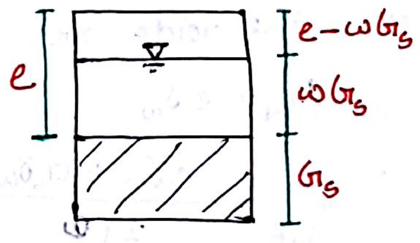
$$\gamma' = \gamma_{sat} - \gamma_w = \frac{G_s+e}{1+e}\gamma_w - \gamma_w$$

$$\therefore \boxed{\gamma' = \frac{G_s - 1}{1+e}\gamma_w}$$

- This eq^n of γ' is always used for design point of view. This is because, bearing capacity of cohesionless soil under submerged condition significantly decreases.

6. Air Content :–

The definition of air content is –



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

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

$$= \frac{1 - \frac{V_w}{V_v}}{1 + \frac{V_s}{V_v}}$$

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

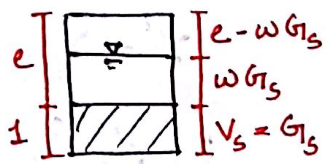
$$= \frac{e(1 - S_r)}{1 + e} \; ; \; \left[\text{as porosity, } n = \frac{e}{1+e}\right]$$

$$\therefore \boxed{A_v = n(1 - S_r)}$$

- Air content as a function of void ratio –

Here, $A_v = \frac{V_a}{V} = \frac{e - wG_s}{1 + e}$

or, $A_v = \frac{1 - \frac{wG_s}{e}}{1 + \frac{1}{e}}$



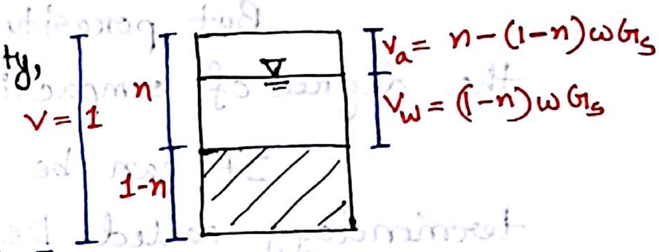
or, $A_v = \dfrac{1-S_r}{1+\frac{1}{e}}$; $\left[\text{as } S_r = \dfrac{wG_s}{e}\right]$

$\therefore$ $\boxed{A_v = \dfrac{e(1-S_r)}{1+e}}$

• Air content as a function of porosity –

Here, $A_v = \dfrac{V_a}{V}$

or, $A_v = \dfrac{n-(1-n)wG_s}{1}$; [for porosity, $V = 1$]



or, $A_v = n-(1-n)eS_r$

or, $A_v = n-(1-n)\dfrac{n}{1-n}S_r$; $\left[\text{as } e = \dfrac{n}{1-n}\right]$

or, $A_v = n - nS_r$

$\therefore$ $\boxed{A_v = n(1-S_r)}$

** For a particular project –

যেখান থেকে Soil আনা হয় ⇒ Borrow Area

যেখানে নিয়ে আসা হয় Soil ⇒ Fill Area

Here, V_f = Volume of fill area and V_b = Volume of borrow area.

So, কতটুকু বেশি Soil আনব ওটার পরিমাণ হবে —

$$\boxed{\frac{V_f}{V_b} = \frac{1+e_f}{1+e_b}}$$

7. Density Index (I_D) :-

• Normally void ratio and porosity describes the compactness / looseness of Cohessionless soil (for example - sand). But in case of clay, it is not applicable because clay is cohesive soil.

But porosity and void ratio can not describe the degree of compaction of cohesionless soil precisely.

It can be described precisely using a terminology called Density Index (I_D).

$\Downarrow$ a ratio expressed in %.

Density Index, $\boxed{I_D = \dfrac{e_{max} - e_f}{e_{max} - e_{min}} \times 100}$

Here, e_f = void ratio in field condition

e_{max} = void ratio in the loosest form

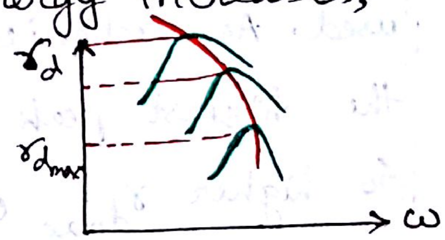
e_{min} = void ratio after vibrating for 15 mins.

Now, $I_D = \dfrac{(1+e_{max}) - (1+e_f)}{(1+e_{max}) - (1+e_{min})}$

or, $\boxed{I_D = \dfrac{\dfrac{1}{\gamma_{dmin}} - \dfrac{1}{\gamma_{df}}}{\dfrac{1}{\gamma_{dmin}} - \dfrac{1}{\gamma_{dmax}}} \times 100}$; [Here for e_{max}, γ_d will be min^m, because $e \propto \dfrac{1}{\gamma_d}$]

essentially constant. There is, therefore, an optimum amount of mixing water for a given soil and compaction process which will give a maximum weight of soil per volume.

- Mechanical energy — For the same soil, if blow number increases i.e. mechanical energy increases, the curve shifts to the left.



For Standard Proctor —

$$\text{Mechanical energy} = \frac{\text{weight} \times \text{no. of layer} \times \text{no. of blows} \times \text{tamping height}}{\text{volume}}$$

$$= \frac{5{\cdot}5 \times 3 \times 25 \times 1'}{\frac{1}{30}} = 12375 \#'$$

and for Modified Standard Proctor —

$$\text{Mechanical energy} = \frac{10 \times 5 \times 25 \times 1{\cdot}5}{\frac{1}{30}} = 56250 \#'$$

Thus mechanical energy increases by $\frac{56250}{12375}$ or $4{\cdot}54$ times which will increase γ_d about 6–7% and thus the shear strength considerably.

The Darcy's Law is —

$$\boxed{v = ki}$$

where, v = discharge velocity / Approach vel. / Apparant vel. / Fictitious velocity

Imp. k = coefficient of permeability

i = hydraulic gradient $= \frac{h}{L}$

here, h = head difference between the two end points of the length L.

• Definition of discharge velocity — The discharge velocity can be defined as the quantity of water that percolates in unit time across a unit area of cross-section oriented at right angle to the direction of flow.

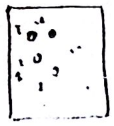
Now, flow rate, $q = \frac{Q}{t} = Av = KiA$

$\therefore K = \frac{Q}{iAt}$

or, $K = \frac{QL}{hAt}$

where, Q = discharge

and A = gross area.

** 

A

⇒ সেই gross area তে soil skeleton আছে যার অংশ দিয়ে water flow করে না।

$\therefore$ v in this case $\neq$ true v.

** Only velocity বললে it means <u>discharge velocity</u>

** Seepage velocity – for this velocity, the area used for determination is only the <u>void area</u>, not the gross area

Here, $\dfrac{Q}{t} = \underbrace{v}_{\text{discharge velocity}} \underbrace{A_g}_{\text{gross area}} = \underbrace{V_{sp}}_{\text{Seepage velocity}} \underbrace{A_v}_{\text{void area}}$

or, $v = V_{sp} \cdot \dfrac{A_v}{A_g}$

or, $\boxed{v = n V_{sp}}$ where, $n = \dfrac{A_v}{A_g}$

if But $n < 1$, $v < V_{sp}$.

- Seepage is the percolation of water through the continuous voids of the soil sample.

- Permeability of soil is a measure of easeness

Now, $i_c = \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}$

or, $i_c = \frac{(G_s - 1)\gamma_w}{(1+e)\gamma_w}$

or, $\boxed{i_c = \frac{G_s - 1}{1+e}}$

- Quick sand = চোরাবালি

- Usually $i_c = 0.85 - 1.15$

But for safety $i_c = 1$ can be assumed.

and for natural granular soil, $e = 0.5$ to 1.0.

- Quick condition generally occurs in fine sand and hardly occurs in coarse sand. This is because, larger the particle size, greater will be the porosity. To maintain critical hydraulic gradient for this type of soil (coarse sand/gravel), larger velocity of flow required which usually can not be found in natural flow condition.

- Under quick condition, $k \simeq \infty$.

Flow Line and the another set is called Equipotential line. And the two sets are mutually perpendicular.

- Flow line – Flow line is a line along which water particles move from the upstream to downstream.

Assumption :– Normally there are infinite number of flow lines from upstream to downstream. No two flow lines touch each other. Otherwise, it will become turbulent flow.

- Equipotential line – Equipotential line is the line along which all Peizometric reading will have same value.

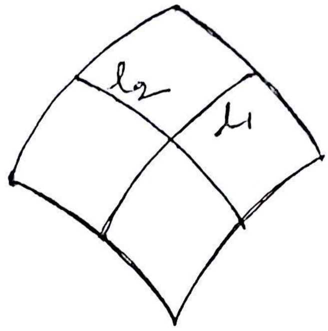
This line is perpendicular to flow line.

- Flow Channel – The region bounded between two successive flow lines is known as flow channel.

- Flow Element – The element bounded by two successive flow line and equipotential line is called flow element.

• Flow net diagram – The combination of flow channel, equipotential line and flow element is called flow net diagram.

Each flow element is a curvilinear square.



$\therefore \underline{\underline{l_1 = l_2}}$

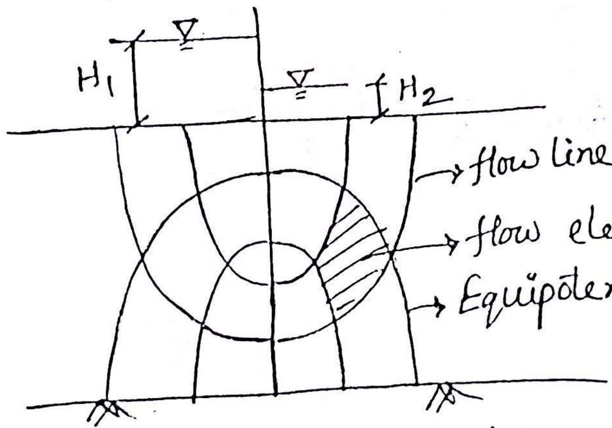
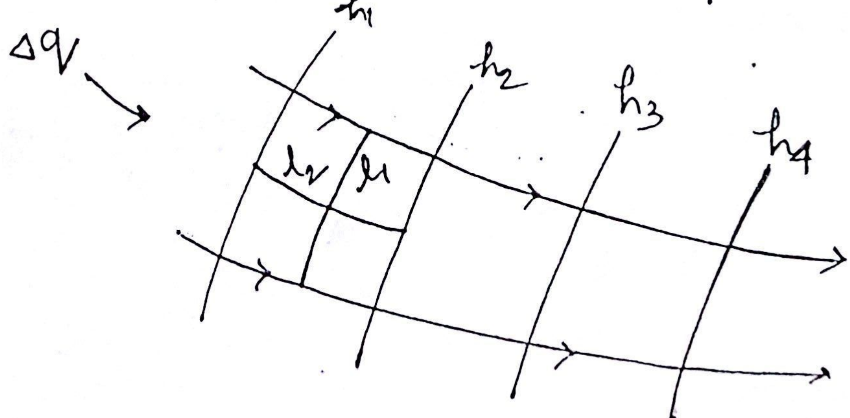


Fig: Flow net diagram

• Flow element in a larger scale –



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Muqtadir Sir

Consolidation Characteristics of Soil

- Karl Tarzaghi ⇒

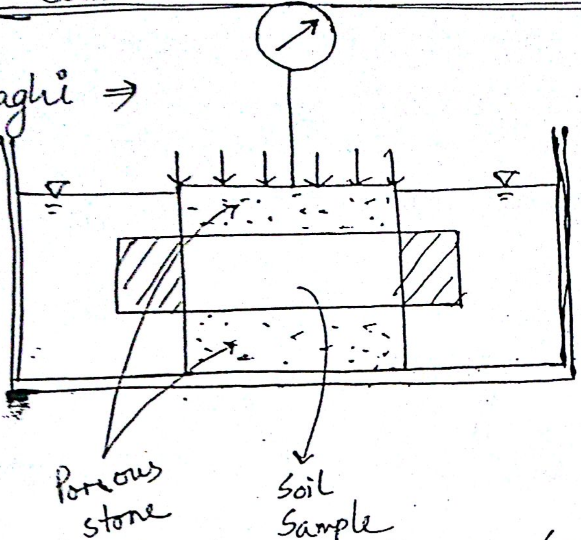
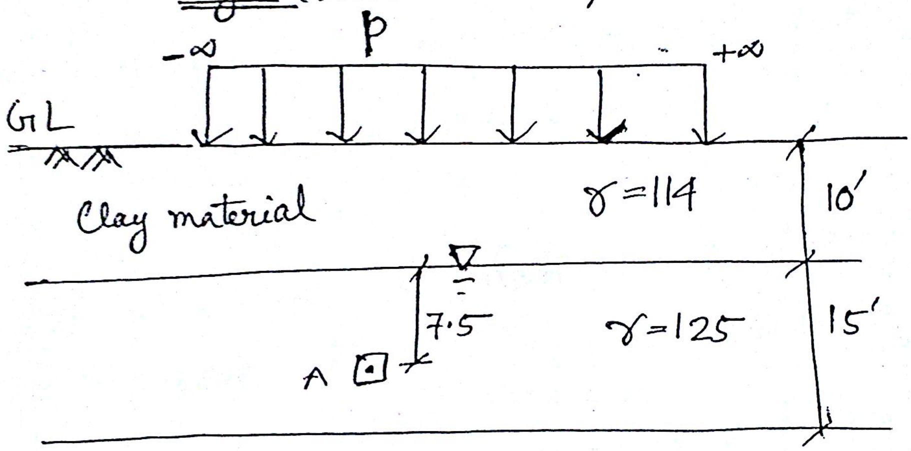


Fig: Consolidometer/Oedometer

* mainly settlement decrease করার হচ্ছে Engg. purpose



p load concentrated হলে slowly নিচে disperse করতো।
but এখানে infinity পর্যন্ত load আছে। So কমবে না load.

before load giving —

effective stress soil skeleton এর $\bar{p}_A = 114 \times 10 + (125 - 62.5)\,7.5$

$u_A = 7.5 \times 62.5$

• Diff. betⁿ Compaction & Consolidation —

Compaction ⇓ particially saturated so both air & water will dissipate

Consolidation ⇓ fully saturated so only H_2O dissipate হয়

• Defⁿ of Consolidation —

Consolidation is a time dependent phenomena associated with the dissipation of excess pore H_2O pressure.

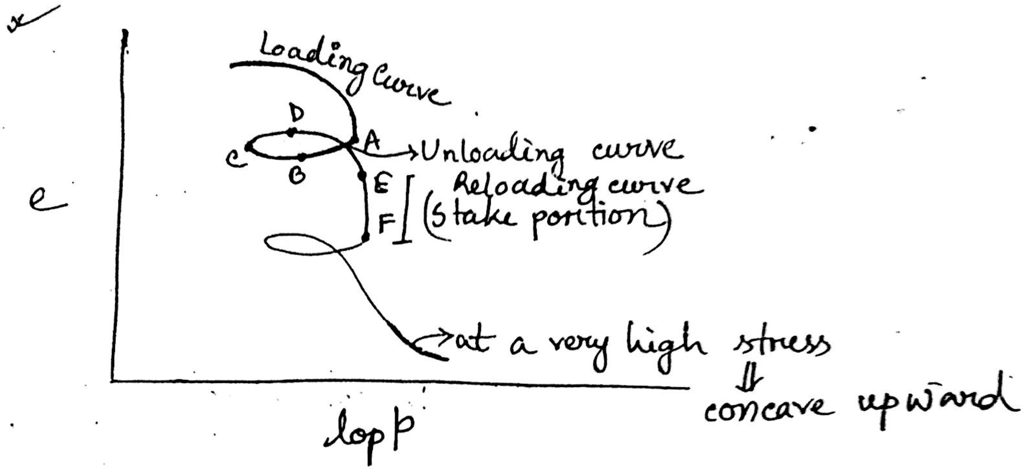
The process of gradual load transfer from pore water to soil skeleton & the corresponding gradual compression is called Consolidation.

⇓

This compression results in the settlement of structure

Tarzagi Defⁿ — A decrease in H_2O content of a saturated soil mass without the replacement of H_2O by air is called Process of Consolidation.

9.11.15



- Types — actually represents the condition of the soil.

1) Normally Consolidated Soil —

— if the existing pressure (overburden effective pressure) on the soil is the max^m pressure that the soil has been subjected to in its lifetime/stress history, then the soil is called <u>Normally Consolidated Soil</u> (Clay) ⇒ NC (mostly used.)

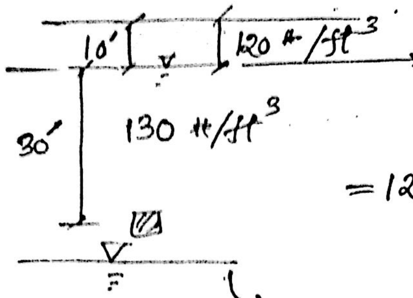
80% of our soil.

2) Over Consolidated Soil —

— If the soil is subjected to a pressure which is more than what is existing now, then the soil is called <u>Over Consolidated Clay (OC)</u>.

Scanned by CamScanner

• Due to fluctuation of water table —



previous

• about 100 years before —

$= 120 \times 10 + (130 - 62.5) \times 30$

for this, $120 \times 40 \Rightarrow$ more

V. imp. $\begin{cases} \text{Water table} \downarrow \text{(নিচে নামলে)} \Rightarrow OC \\ \quad " \quad " \quad \uparrow \text{(উপরে উঠলে)}. \Rightarrow NC \end{cases}$

1. Very stiff in nature
2. Magnitude of settlement is less.

• Over Consolidation Pressure (P_c) —

past max^m pressure to which the soil is subjected to is called Over Consolidation Pressure.

Current pressure $= P_o$

$\therefore$ Over Consolidation ration (OCR) $= \dfrac{P_c}{P_o}$

$< 1 \Rightarrow NC$

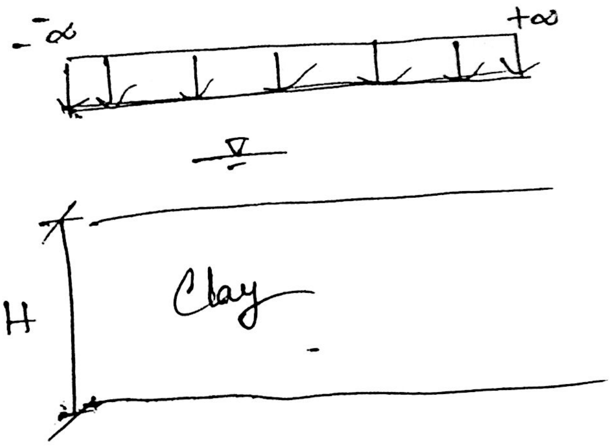
$> 1 \Rightarrow OC$

(*) $\therefore$ higher the degrees of consolidation, OCR $\uparrow$.

- Computation of the settlement—

Basic assumption

Clay layer subjected to uniform strain across its thickness for the load



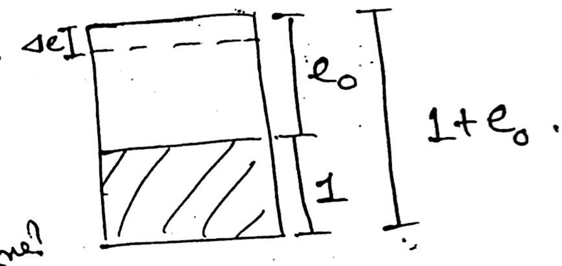
Settlement,

$$S = H\varepsilon$$

uniform strain

or, $\varepsilon = \dfrac{S}{H}$

$\therefore \dfrac{\Delta e}{1+e_0} = \dfrac{S}{H}$



$\Rightarrow \boxed{S = H \dfrac{\Delta e}{1+e_0}}$

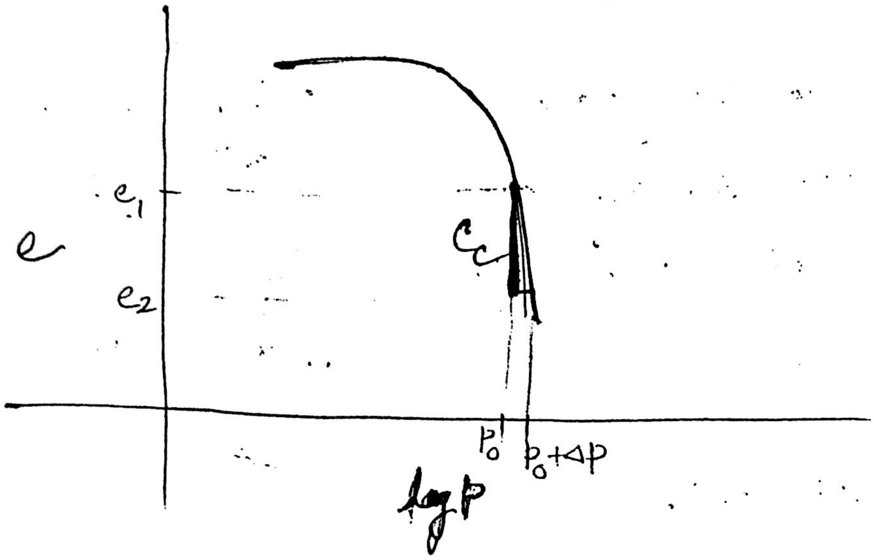
Δe → change in void ratio after giving load

how to determine? $\therefore \varepsilon = \dfrac{\Delta e}{1+e_0}$

S → settlement

H → thickness of the clay layer

e_0 → load দেয়ার আগমুহূর্তে void ratio



.15 – .35 ⇒ হয় plasticity ↑, C_c ↑.

* C_c = Compression index = slope of the verticle portion of the curve.

$$\therefore C_c = \frac{e_1 - e_2}{\log(p_0 + \Delta p) - \log p_0} = \frac{\Delta e}{\log \frac{p_0 + \Delta p}{p_0}}$$

$$\Rightarrow \Delta e = C_c \log \frac{p_0 + \Delta p}{p_0}$$

$\therefore$ Settlement ⇒ $S = H \frac{C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_0}$ ⇒ valid for <u>only NC Clay.</u>

Suppose, for the above fig.
$e_0 = e_1$
যে p_0 দিয়া হচ্ছে ঐখানে

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P_o = যে layer-এর settlement compute করব ঐ layer-এর middle-এ existing effective overburden pressure before applying load/construction

ΔP = after construction/giving load ঐ middle-এর ঐ point-এ কতটুকু stress increase হবে।

Example

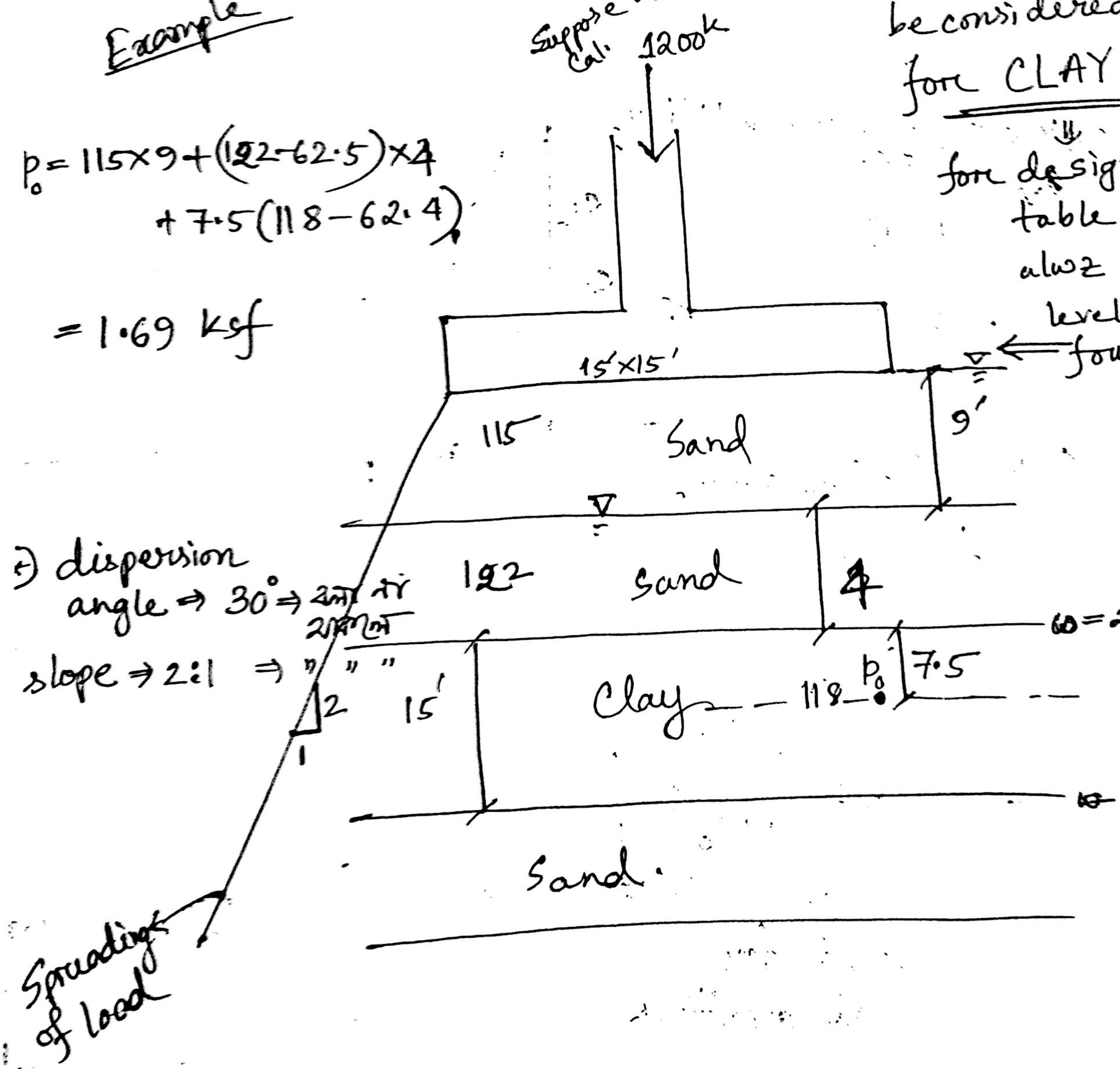
* Settlement should be considered only for CLAY

⇓

for design water table will be alwz at the level of foundation bottom.

$p_o = 115\times 9 + (122-62.5)\times 4 + 7.5(118-62.4)$

$= 1.69$ ksf



Clay middle layer footing থেকে 20.5' নিচে আছে

∴ 2:1 slope হলে hor. distance হবে 10.25.

$15 + 10.25 \times 2 = 35.5'$

⇓

So only the 15' layer ver. distance হতে 35.5' এর প্রত্যেকে 2 side 10.25 করে বেড়ে যাবে।

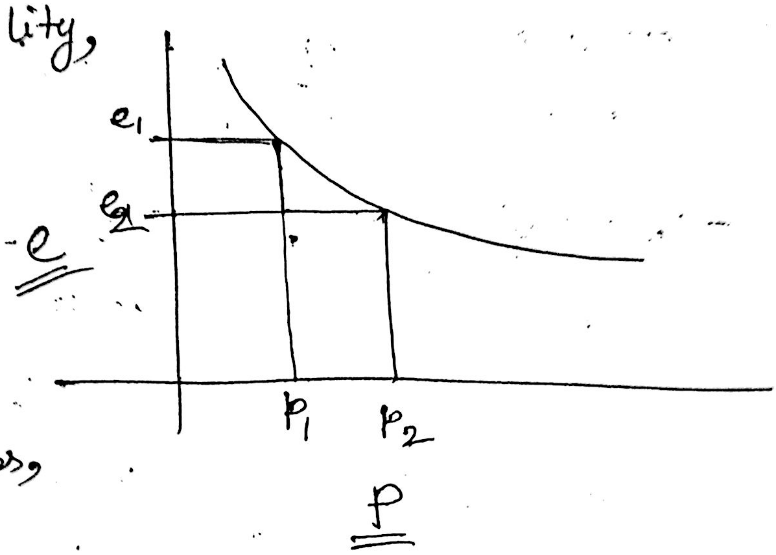
$$\Delta p = \frac{1200}{35.5 \times 35.5} =$$

$$\therefore e_0 = w G_s = .29 \times 2.7 =$$

⊛ Coefficient of permeability,

$$a_v = \frac{e_1 - e_2}{p_2 - p_1}$$

$$= -\frac{\Delta e}{\Delta p}$$



-ve becz একটির অন্যটির reverse with ↑ in effective stress, $a_v \downarrow$

⇓

তাই কখনও কখনও থাকে - sign.

$$\therefore S = H \frac{\Delta e}{1+e_0}$$

$$\Rightarrow \boxed{S = H \frac{a_v}{1+e_0} \Delta p}$$

⇓

valid for all conditions

$p_0 = 1.69$

$\Delta p = 0.95$

⇓

curve Graph - এ $p_1 = 1.69$ ধরব then 0.95 increase করব & p_2 পাব।

⇓

e_0 হবে 1.69 এর corresponding e.

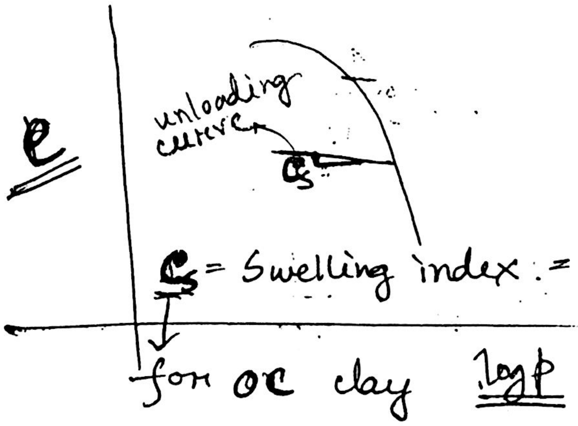
- Empirical formula:

$C_c = 0.009(LL - 10) \Rightarrow$ undisturbed soil sample

$C_c = 0.007(LL - 10) \Rightarrow$ ~~disturbed~~ " "
remolded

C_c
বের হবে
e vs log p curve
থেকে normally
এই formula
থেকেও করা যায়

- Swelling Index



C_s = Swelling index = 10 - 20% of C_c
for OC clay

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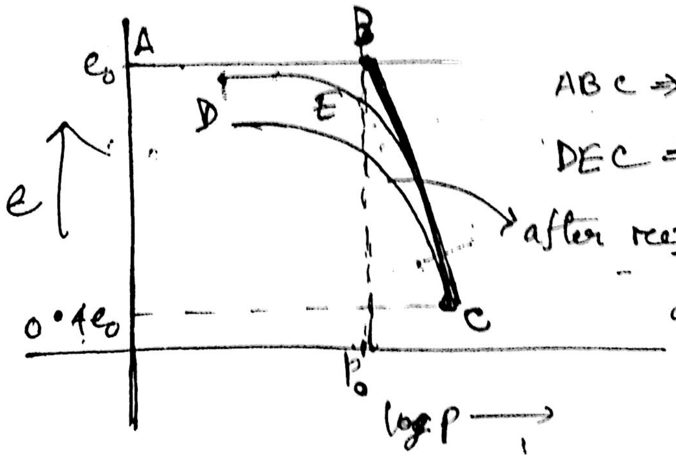
Max Settlement due to Primary Consolidation — 16.11.15

V. Imp.

• Laboratory e-logp curve — (undisturbed) & NC Clay



ABC ⇒ field e-logp curve

DEC ⇒ laboratory e-logp "

after remolding

↓

degrees of disturbation ↑.

Impirical relatⁿ ship ⇒ SHartMan

⇓

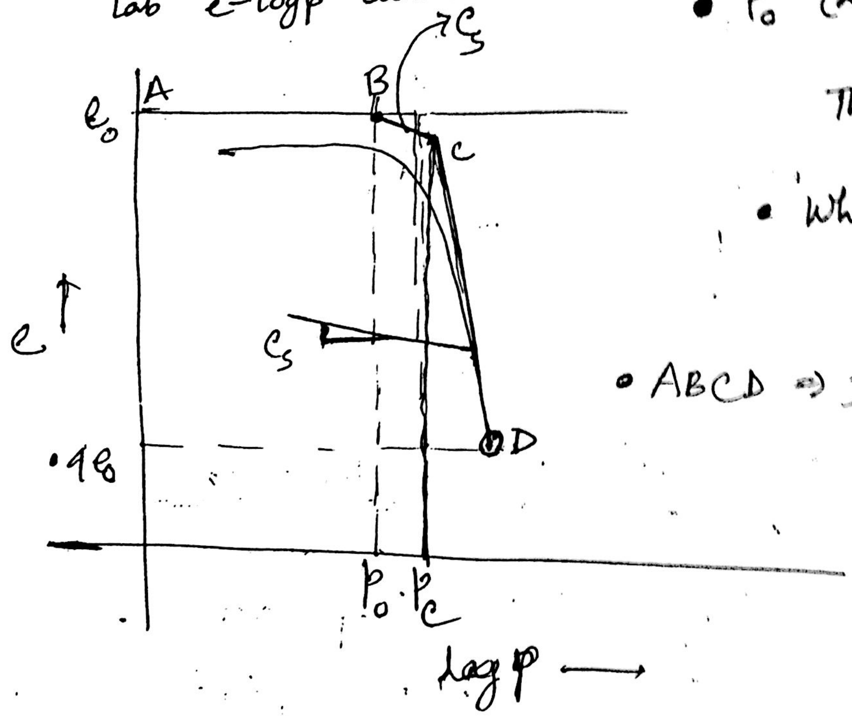
laboratory & field e-logp curve → straight ~~final~~ portion passing through a one point ⇒ corresponds to 40% of e_0.

$$S = H \frac{C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_0}$$

⊛ for OC Clay

p_o, p_c, e_0.

Lab e-logp curve.



- p_o থেকে p_c আগে
  ⇓
  Then OC
- when crosses p_c
  ⇓
  NC
- ABCD ⇒ field e-logp curve for OC Clay

Smart Men ⇒

$$\overrightarrow{BC}, \quad S_1 = H \frac{C_s}{1+e_0} \log \frac{p_o + \Delta p}{p_o}$$

p_o ↳ current/existing pressure

where, $p_o + \Delta p < p_c$

⇓

এখানে BC portion-এর জন্য total S only 5% or <5% হয়। So ছোট project-এ হিসাব করা হয় না।

⊛ OC
NC

* OC clay ⇒ যত load ↑ তত ভাল
NC " ⇒ যত " ↓ " "

$\overrightarrow{CD}$, $S_2 = H \frac{C_c}{1+e_0} \log \frac{p_c + \Delta p}{p_c}$ or $S_2 = H \frac{C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_c}$

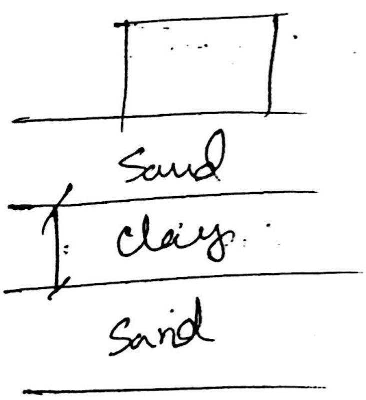
where $p_0 + \Delta p > p_c$

$\downarrow$

cz its starts from C and so here initial pressure is p_c

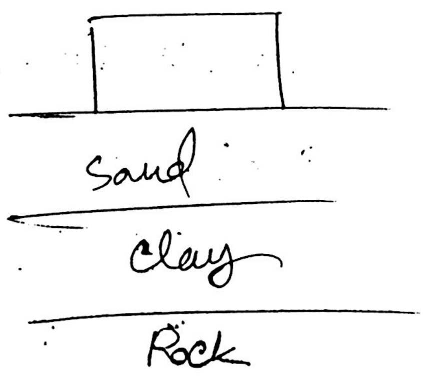
$$S = S_1 + S_2 = \frac{C_s H}{1+e_0} \log \frac{p_c}{p_0} + \frac{C_c H}{1+e_0} \log \frac{p_0 + \Delta p}{p_c}$$

$p_0 + \Delta p > p_c$



$\Downarrow$

drainge in both way

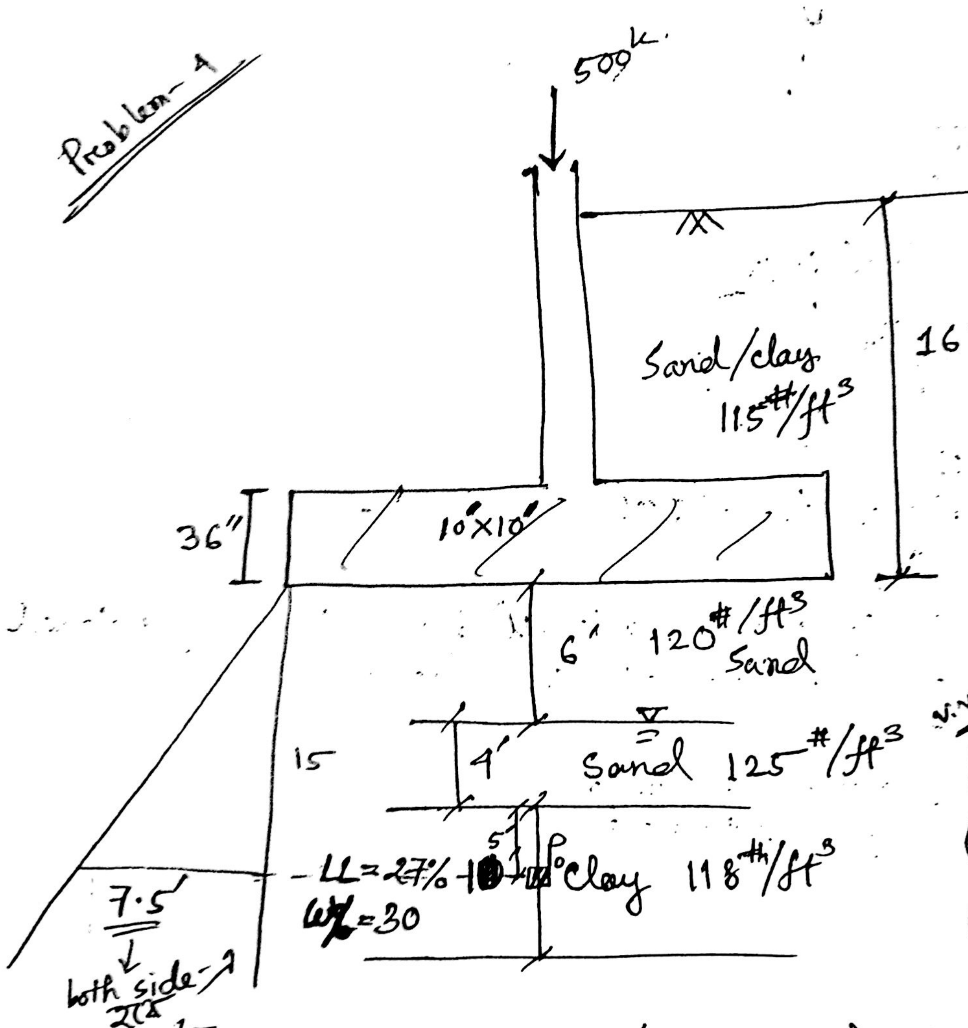


$\Downarrow$

One way drainage ⇒ needs more time

* Magnitude of settlement will be same irrespective of drainage condition ⇒ only depends on $\boxed{\text{time}}$

Problem-1



v.v.imp.

মাটি কাটার পর footing এবং নিচের সব soil will OC.

⇓

Thereafter construction, যদি OC-এর load/আগের soil-এর জন্য যে load ছিল তা exceed করলে then lower soil NC হবে

$$\bar{p}_c/\bar{p}_0 = 16\times115 + 120\times6 + (125-62.5)\times4 + (118-62.5)\times5$$

$\bar{p}_0$ → before doing any excavation

$$= 3.08$$

$$\bar{p} = \frac{500}{\underbrace{(10+15)^2}_{\text{area of the base}}} = 0.8\ \text{ksf}.$$

500 ← load

$\bar{p}$ → pressure for footing

$$\overline{p}_0 = 6\times120 + 4(125-62.5) + 5(118-62.4)\,7.5$$

$$= 152 \text{ ksf.}$$

ter
cavation

Settlement expression —

$\underbrace{1.52 \text{ to } 3.08}_{\text{OC}}$ এ আর

Initial void ratio, $e_0 = \omega G_s$

$$= 0.3\times2.7$$

$$= 0.81$$

$$C_c = .009(27-10)$$

$$= 0.15$$

$$C_s = 15\%\, C_c = 0.0225$$

stress for footing

$\overline{p}$

$$\therefore S = \frac{.0225\times120}{1+.8}\log\frac{3.08}{1.52} + \frac{.15\times120}{1+.8}\log\frac{3.08+.8}{3.08}$$

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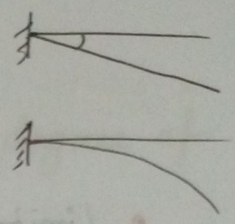
for rigid beam ⇒ no length change

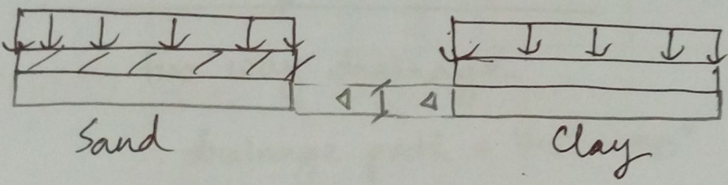
" flexible " ⇒ length changes



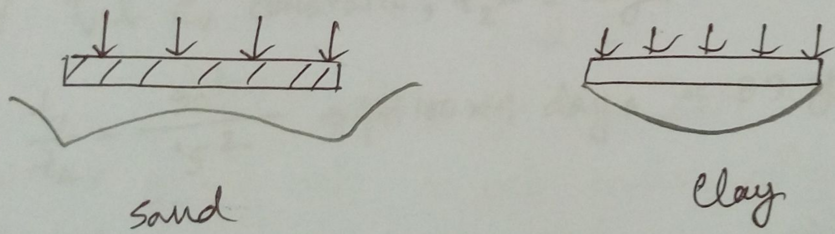
- Displacement & stress distribut^n will be considered.

Settlement –

- Rigid body resting on sand and clay subjected to uniform pressure ⇒ settlement will be equal.



- for flexible body —

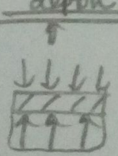


Stress Dist^n —

flexible

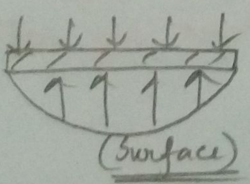
pressure dist^n will be equal & opposite in magnitude.

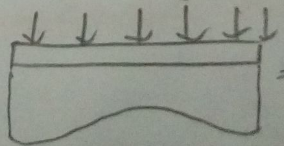
At very high depth



Rigid

⇐ with depth it becomes uniform





Singularity Problem

⇒ Theoretically it goes to ∞. But before that soil fails so the figure is like this

(4) Present effective overburden pressure is the max^m pressure that the soil has been subjected to in the past.

(4) Present effective overburden pressure is less than the soil has experienced in the past.

NCC

Settlement,

$$S_c = \frac{C_c H}{1+e_0} \log \frac{P_0' + \Delta P'}{P_0'}$$

C_c = compression index
= 0.009 (LL − 10) [undisturbed]
= 0.007 (LL − 10) [disturbed]

H = clay layer thickness
e_0 = initial void ratio
P_0' = effective overburden pressure
$\Delta P'$ = Increase of effective pressure.

OCC

$\boxed{\text{Case-1}}$: $P_c' \geq (P_0' + \Delta P')$

$$S_c = \frac{C_s H}{1+e_0} \log \left(\frac{P_0' + \Delta P'}{P_0'}\right)$$

C_s = swell index $\cong \frac{1}{5}$ to $\frac{1}{10}$ of C_c
P_c' = pre consolidation pressure.

$\boxed{\text{Case-2}}$: $P_0' < P_c' < P_0' + \Delta P'$

$$S_c = \frac{C_s H}{1+e_0} \log \frac{P_c'}{P_0'} + \frac{C_c H}{1+e_0} \log \frac{P_0' + \Delta P'}{P_c'}$$

When SPT value needed correction?

~~(iv) Inadequate cleaning of bore hole~~
(1) If drill rod length is more.
(2) for borehole dia.
(3) liner in sampler measurement
(4) Inappropriate release mechanism

N value depends on :-
(i) Energy
(ii) Hammer
(iii) length of drill rod
(iv) dia of borehole

Specification of Compaction Test

#	Standard Proctor ASTM D698-70	Modified Proctor ASTM 1557-70
weight of Hammer	5.5 lb	10 lb
Num of layers	3	5
Height of hammer drop	12 in	18 in

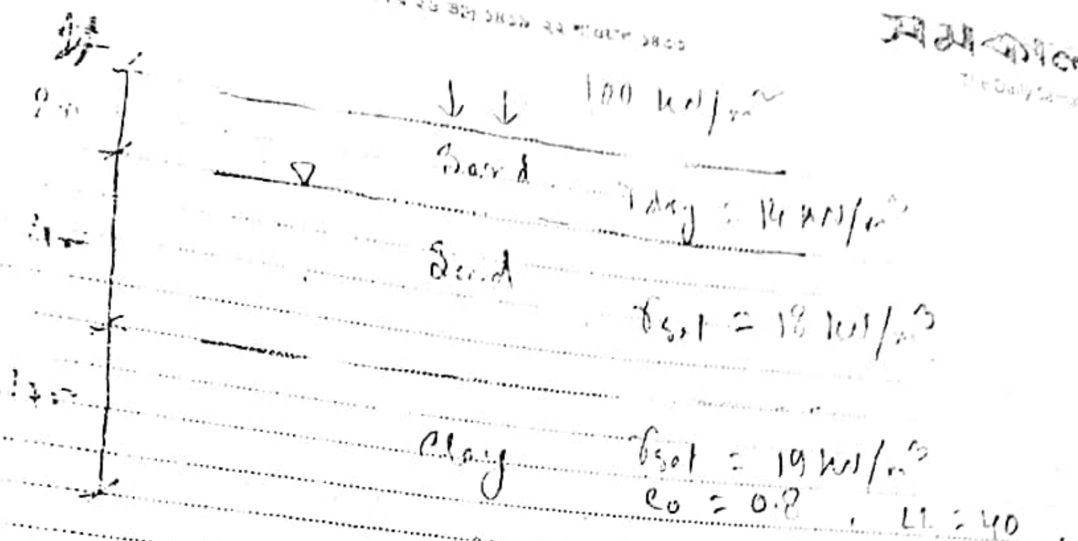
Two types of compaction molds: —

(i) dia = 4" ; ~~weight~~ volume = $1/30\ ft^3$

(ii) dia = 6" ; volume = $1/13.33\ ft^3$,

→ num of blow should be 56 instead of 25.

dia of mold	4"	6"
volume of mold	$1/30\ ft^3$	$1/13.33\ ft^3$
height of mold	4.58"	4.58"
Num of blows per layer	25	25
Tested on soil finer than sieve	No. 4	No. 4



find settlement of the clay layer if

(a) The clay is NCC, $C_c = 0.27$

(b) Pre consolidation pressure, $P_c' = 190\ kN/m^2$

(c) $P_c' = 170\ kN/m^2$ use, $C_s = \frac{1}{6} C_c$

(a) For NCC

$$S = \frac{C_c H}{1+e_0} \log \frac{P_0' + \Delta P'}{P_0'}$$

$$P_0' = 14 \times 2 + 4 \times (18 - 9.81) + 2 \times (19 - 9.81)$$

$$= 79.14\ kN/m^2$$

$$C_c = 0.009 \times (40 - 10) = 0.27$$

$$\therefore S = \frac{0.27 \times 4}{1 + 0.8} \log \frac{79.14 + 100}{79.14}$$

$$= 0.213\ m = 213\ mm.$$

(b) $P_0' + \Delta P' = 170.14$

so. $P_c' = 190\ kN/m^2 > P_0' + \Delta P'$

$$S = \frac{C_s H}{1+e_0} \log \frac{P_0' + \Delta P'}{P_0'}$$

$$= \frac{\frac{1}{6} \times .27 \times 4}{1+0.8} \log \frac{170.14}{70.14}$$

$$= 0.035\ m$$

(c) $P_c' < P_0' + \Delta P'$

$$S = \frac{C_s H}{1+e_0} \log \frac{P_c'}{P_0'} + \frac{C_c H}{1+e_0} \log \frac{P_0' + \Delta P'}{P_c'}$$

$$= \frac{\frac{1}{6} \cdot .27 \times 4}{1+0.8} \log \frac{170}{70.14} + \frac{0.27 \times 4}{1+0.8} \log \frac{170.14}{170}$$

$$= 0.033 + 0.0136$$

$$= 0.047\ m$$

"PERMEABILITY"

Permeability — is a soil property which indicates the ease with which ~~soil~~ water will flow through the soil.

Seepage — is the slow movement of water through the continuous void sample.

Flow net — equipotential line + flow line.

Equipotential line — is a line along which piezometric head reading is same at all points on it. It is orthogonal to flow line

Flow line — is the line along which water particles ~~flow~~ ~~through~~ travel from u/s to d/s.

Factors affecting the permeability of soil

(1) The size of soil grains.
(2) The properties of pore fluid.
(3) The void ratio of soil.
(4) The shape & arrangements of pores.
(5) The degree of saturation.

Constant head method | Falling head method

(i) $k = QL/hAt$ (i) $k = 2.3 \frac{al}{At_1} \log h_0/h_1$

(ii) for highly permeable materials (ii) for low permeable materials.

(iii) not precise method (iii) precise method

In a constant head permeability test a sample 8cm long was tested in a permeameter with an inside dia of 5cm. After a state of steady flow was established under a head of 50cm, discharge of 120 cucm was collected in 30 sec. Compute the value of k.

Ans:

$$k = \frac{QL}{hAt} = \frac{120 \times 8}{50 \times \frac{\pi}{4}(5)^2 \times 30}$$

$$= 3.3 \times 10^{-2} \text{ cm/sec.}$$

A falling head permeability was performed in a permeameter with an inside dia of 5 cm. The inside dia of the standpipe was 2 mm. The sample had a length of 8 cm. During a period of 6 min the head on the sample decreased from 100 to 50 cm. Compute the value of k.

Ans.

$$k = 2.3 \frac{aL}{At} \log h_2/h_1$$

$$= 2.3 \times \frac{\frac{\pi}{4}\times(0.2)^2 \times 8}{\frac{\pi}{4}\times 5^2 \, 6\times 60} \log \frac{100}{50}$$

$$= 2.5 \times 10^{-5} \text{ cm/sec.}$$

Seepage velocity, $V_s = \frac{V}{n} = \frac{V}{\frac{e}{1+e}} = V \times \left(\frac{1+e}{e}\right)$.

Hydraulic conductivity of clay soil is 3×10^{-7} cm/s. The viscosity of water at 25°C is 0.0911×10^{-4} g.sec/cm². Calculate the absolute permeability of soil

$$k = \bar{k} \frac{\gamma_w}{\eta}$$

$$\Rightarrow \bar{k} = \frac{k\eta}{\gamma_w} = \frac{3\times10^{-7}\times 0.0911\times10^{-4}}{1 \text{ gm/cm}^3} = 0.2733\times10^{-11} \text{ cm}^2$$

(ঝ) In natural state, a moist soil has a volume of $0.33\ ft^3$ and weights 39.93 lb. The oven dry weight of the soil is 34.54 lb. If $G_s = 2.71$. Calculate, w, γ_w, γ_d, e, n, S.

Ans:- $w = \frac{W_w}{W_s} = \frac{W - W_s}{W_s} = \frac{39.93 - 34.54}{34.54} = 15.6\%$

$\gamma_w = \frac{W_w}{V} = \frac{39.93}{0.33} = 121\ lb/ft^3$

$\gamma_d = \frac{W_s}{V} = \frac{34.54}{0.33} = 104.67\ lb/ft^3$

$e = V_v / V_s$; $V_s = \frac{W_s}{G_s \gamma_w} = \frac{34.54}{2.71 \times 62.4} = 0.2043\ ft^3$

$V_v = V - V_s = 0.33 - 0.2043 = 0.1257\ ft^3$

$= \frac{0.1257}{0.2043}$

$= 0.615$

$n = \frac{e}{1+e} = \frac{0.615}{1+0.615} = 0.381$

$S = \frac{V_w}{V_v}$; $V_w = \frac{W_w}{\gamma_w} = \frac{39.93 - 34.54}{62.4} = 0.0864\ ft^3$

$= \frac{0.0864}{0.1257}$

$= 68.74\%$

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(*) for a soil, $e = 0.75$, $w = 22\%$, $G_s = 2.66$
Calculate, n, γ_w, γ_d, S.

Ans.

$$n = \frac{e}{1+e} = \frac{0.75}{1+0.75} = 0.429$$

$$\gamma_w = \frac{(1+w)\,G_s\gamma_w}{1+e} = \frac{(1+0.22)\times 2.66\times 62.4}{1+0.75} = 115.7\ lb/ft^3$$

$$\gamma_d = \frac{G_s\gamma_w}{1+e} = \frac{2.66\times 62.4}{1+0.75} = 94.85\ lb/ft^3$$

$$S = \frac{wG_s}{e} = \frac{0.22\times 2.66}{0.75} = 78.02\%$$

(*) $W = 15\%$, unit wt $120\ lb/ft^3$, $e_{min} = 0.50$ and $e_{max} = 0.85$ for densest & loosest state. Compute S, I_d, $G = 2.65$.

Ans.

$$I_d = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.85 - e}{0.85 - 0.5}$$

$$= \frac{0.85 - 0.59}{0.85 - 0.5} = 0.74$$

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

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

Now,

$$W = \frac{W_w}{W_s} = 0.15 \Rightarrow W_w = 0.15\,W_s$$

Again. $W_w + W_s = 120 \Rightarrow 0.15\,W_s + W_s = 120$ $\therefore W_s = 104.3\ lb$.
$W_w = 15.7\ lb$

$V_s = \frac{W_s}{G_s \cdot \gamma_w} = \frac{104.3}{2.65 \times 62.4} = 0.63\ ft^3$

Let, Total volume, $V = 1$

$\therefore V_w = \frac{W_w}{\gamma_w} = \frac{15.7}{62.4} = 0.25\ ft^3$

$\therefore V_a = V - (V_w + V_s) = 1 - (0.25 + 0.63) = 0.12\ ft^3$

$S = \frac{V_w}{V_v} = \frac{0.25}{0.25 + 0.12} = \frac{0.25}{0.37} = 67.57\%$

$e = \frac{V_v}{V_s} = \frac{0.37}{0.63} = 0.59$.

(F) Unit wt = 131 lb/ft³, γ_d, γ_d at zero air. γ_{sat} if the voids are filled with water. $G_s = 2.67$, $w = 14\%$

Ans: $w = W_w / W_s = 0.14 \Rightarrow W_w = 0.14 W_s$

$\therefore W_w + W_s = 131 \Rightarrow 0.14 W_s + W_s = 131 \therefore W_s = 115$ lb

$\therefore W_w = 16.1$ lb

$\gamma_d = W_s / V = \frac{115}{1} = 115\ lb/ft^3$

$V_s = \frac{W_s}{G_s \gamma_w} = 115 / 2.67 \times 62.4 = 0.69\ ft^3$

$V_w = W_w / \gamma_w = \frac{16.1}{62.4} = 0.26\ ft^3$

$V_v = V - V_s = 1 - 0.69 = 0.31\ ft^3$

$V_a = V_v - V_w = 0.31 - 0.26 = 0.06\ ft^3$

$\gamma_z = \frac{W_s}{V_s + V_w} = \frac{115}{0.69 + 0.26} = 121.1\ lb/ft^3$

$\gamma_{sat} = 115 + 0.31 \times 62.4 = 134.4\ lb/ft^3$

$V_s = \frac{W_s}{G_s \gamma_w} = \frac{104.3}{2.\ldots \times 62.4} = 0.63\ ft^3$

Let, total volume, $V = 1$

$V_w = \frac{W_w}{\gamma_w} = \frac{15.7}{62.4} = 0.25\ ft^3$

$\therefore V_a = V - (V_w + V_s) = 1 - (0.25 + 0.63) = 0.12\ ft^3$

$S = \frac{V_w}{V_v} = \frac{0.25}{0.25 + 0.12} = \frac{0.25}{0.37} = 67.57\%$

$e = \frac{V_v}{V_s} = \frac{0.37}{0.63} = 0.59$.

(7) Unit wt = 131 lb/ft³, γ_d, γ_d ~~[illegible]~~ at zero air voids, γ_{sat} if the voids are filled with water. $G_s = 2.\ldots$, $w = 14\%$

Ans: $w = W_w / W_s = 0.14 \Rightarrow W_w = 0.14\ W_s$

$\therefore W_w + W_s = 131 \Rightarrow 0.14\ W_s + W_s = 131 \therefore W_s = 115$ lb

$\therefore W_w = 16.1$ lb

$\gamma_d = W_s / V = \frac{115}{1} = 115$ lb/ft³

$V_s = \frac{W_s}{G_s \gamma_w} = 115 / 2.69 \times 62.4 = 0.69\ ft^3$

$V_w = W_w / \gamma_w = \frac{16.1}{62.4} = 0.26\ ft^3$

$V_v = V - V_s = 1 - 0.69 = 0.31\ ft^3$

$V_a = V_v - V_w = 0.31 - 0.25 = 0.06\ ft^3$

$\gamma_z = \frac{W_s}{V_s + V_w} = \frac{115}{0.69 + 0.26} = 121.1$ lb/ft³

$\gamma_{sat} = \frac{115 + 0.31 \times 62.4}{1} = 134.4$ lb/ft³

$\boxed{* \text{BCS}}$ A 27.50 lb soil sample has a volume of 0.320 cft moisture content of 15.20% and specific gravity of soil solids 2.67. Compute bulk density, dry density, Sw, e,

$W = 27.50$ lb, $V = 0.320$ cft, $w = 0.152$, $G_s = 2.67$

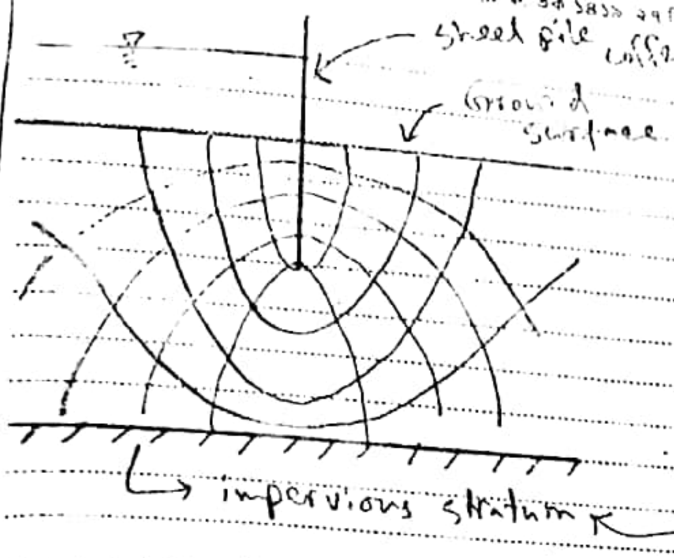
$$\gamma_{bulk} = \frac{(1+w)G_s\gamma_w}{1+e} = \frac{(1+0.152)\times 2.67\times 62.5}{1+e} = 85.94$$

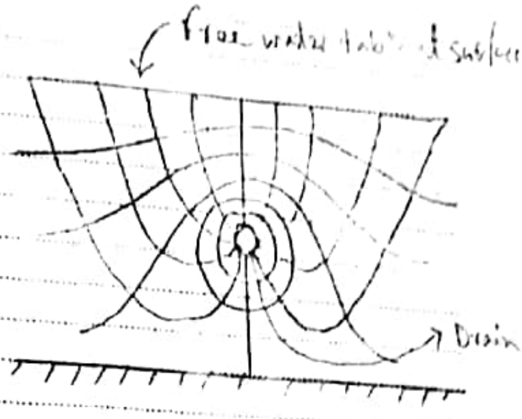
$\therefore e = 1.24$

$$\gamma_{bulk} = W/V = 27.50/0.320 = 85.94 \text{ lb/ft}^3$$

$$\gamma_{dry} = \frac{\gamma_{bulk}}{1+w} = \frac{85.94}{1+0.152} = 74.6 \text{ lb/ft}^3$$

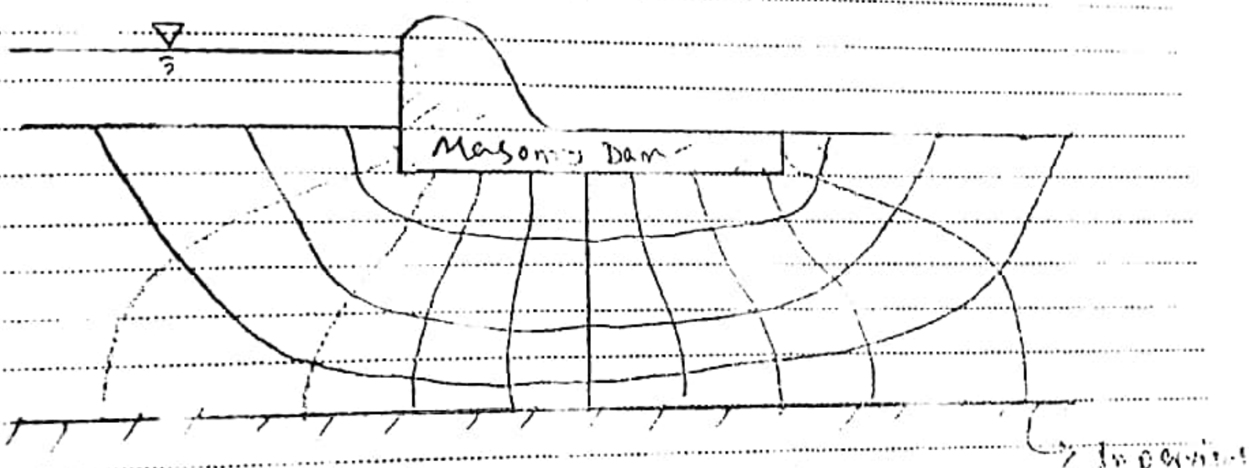
$$S_w = \frac{wG_s}{e} = \frac{0.152\times 2.67}{1.24} = 32.73\%$$



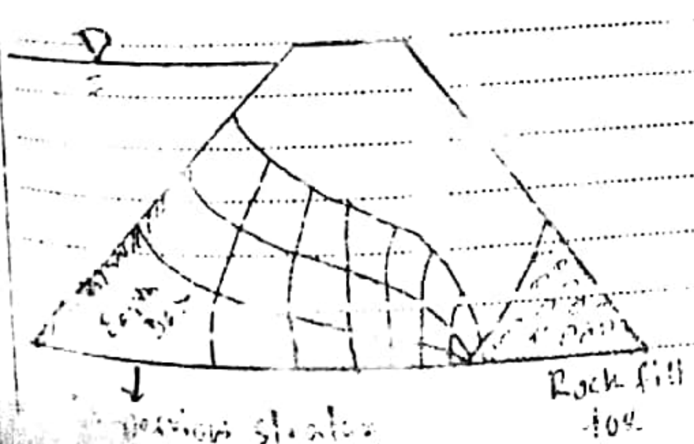


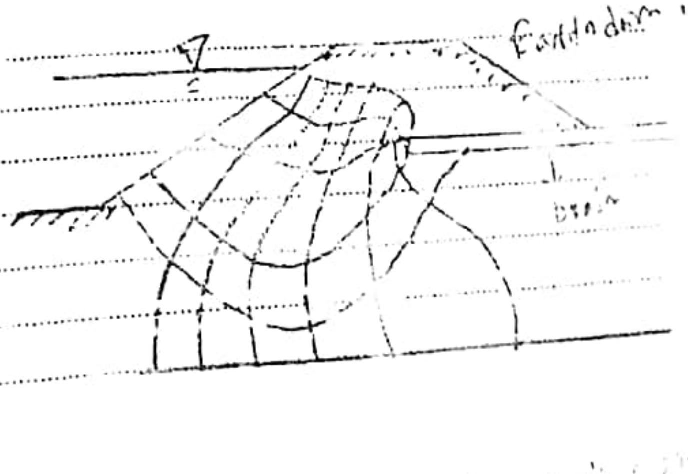
(a) flow net for sheet pile cofferdam

(b) flow net for drain.



(c) flow net for masonry dam

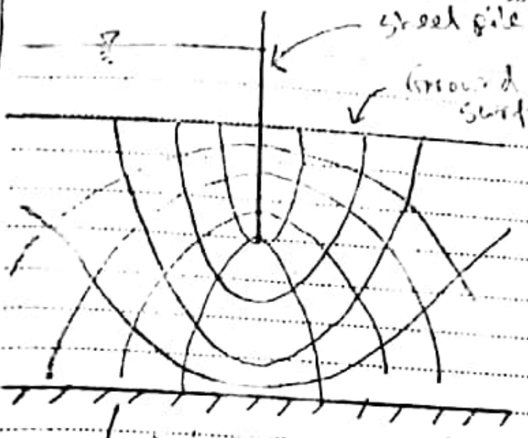


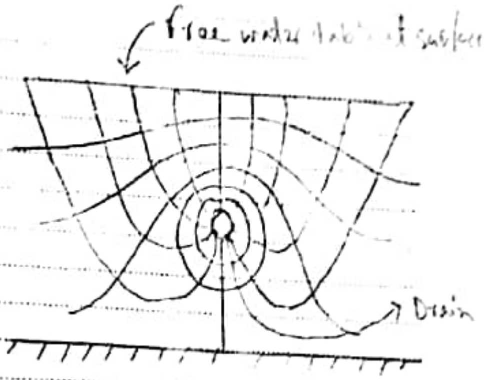


(d) flow net for earth dam

(e) flow net for earth dam ...

Flow Net

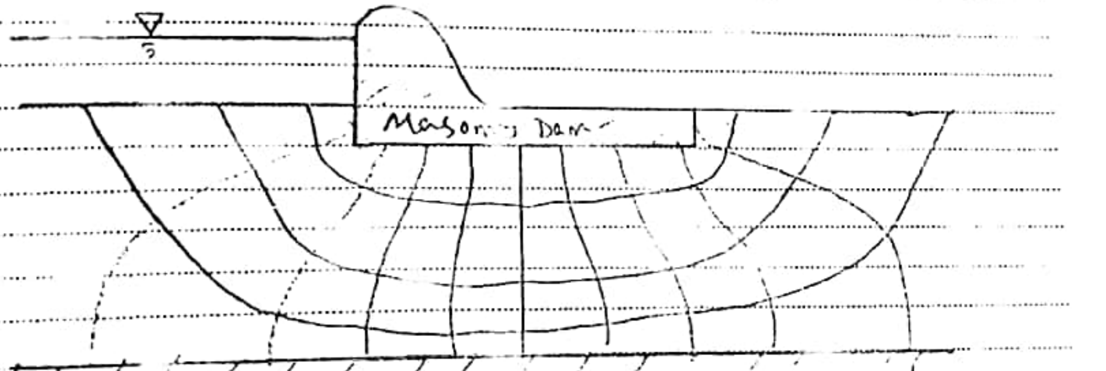




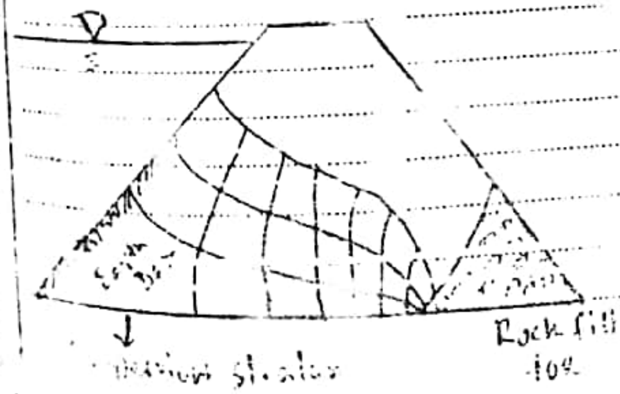
impervious stratum

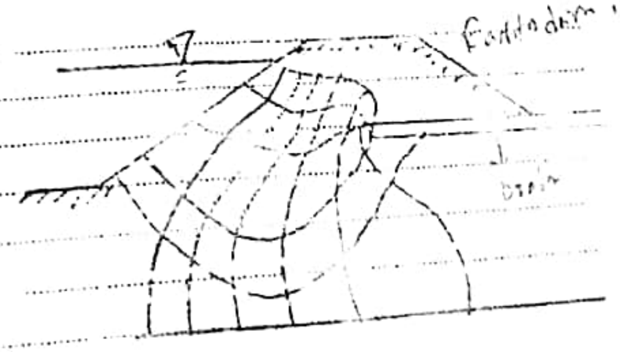
(a) flow net for sheet pile cofferdam

(b) flow net for drain.



(c) flow net for masonry dam





(d) flow net for earth dam

(e) flow net for earth dam with ... drain

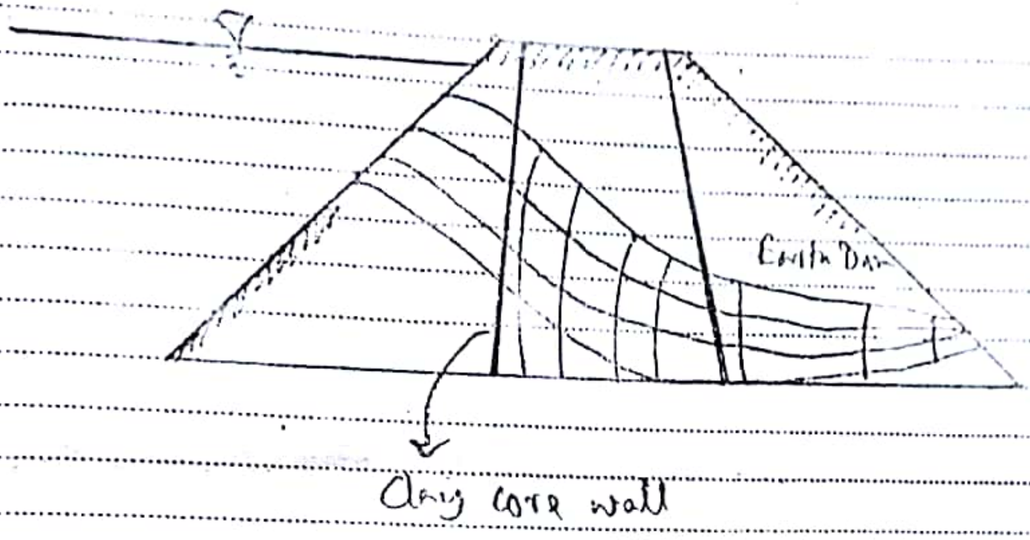


Fig. Flow net for earth dam with clay core wall

Quick Sand :

Quick Sand is a condition in which cohesionless soil loses all its strength because of upward flow of water.

# Effective stress	Pore water pressure (neutral stress)	Total stress
(1) It represents an excess over neutral stress and acts exclusively between the point of contact of solid constituents.	(1) acts in the water and in the solid in every direction with equal ~~to~~ intensity.	(i) it acts at any point on a section through a mass of saturated soil or rock.
(2) It can induce changes in volume of soil mass.	(2) It can not cause volume change.	
(3) It can produce frictional resistance	(3) It can not produce frictional resistance	

Quick Sand: It may develop in excavations for footings of buildings or bridges when water tends to flow upward into unwatered areas / earthquake, pile driving → due to vibration

Solution — to build an enclosing wall of sandbags around the area and to allow the water to pond within the enclosure. This stored water builds up a back pressure and reduces hydraulic gradient and quiets the soil in the boil area.

OCTOBER 15 Monday

সোমবার ৩০ আশ্বিন ১৪২৫ ৫ সফর ১৪৪০

সমকাল
The Daily Samakal

Immediate Settlement	Secondary Settlement
(1) It is the result of elastic deformation of dry soil and of moist & saturated soil.	(1) It is the result of plastic adjustment of soil fabric.
(2) It requires considerably a short time.	(2) It occurs in a very slow rate.
(3) Excess hydrostatic pressure is present.	(3) Excess hydrostatic pressure becomes low.

Scanned by CamScanner

(*) Uniformity Co-eff : Cu

$$C_u = \frac{D_{60}}{D_{10}}$$, যত বেশি বুঝব soil particle এর size এর ভেতর variation বেশি,

(*) Co-eff of curvature : $\boxed{\text{Spread}}$ of the curve বুঝি,

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

যদি একই size এর particle থাকলে $C_u = 1$, then curve verticle.

Cu is a measure of spread of the curve.

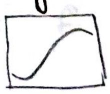
S shape curve হলে well graded soil.

Cz থেকে

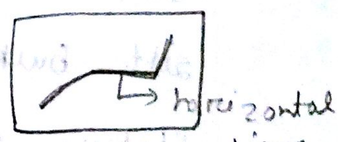
co-eff of curvature থেকে $\boxed{\text{shape}}$ of the curve.

if $1 < C_z < 3$ then S shape.

From Cu & Cz & curve থেকে gradation জানতে পারি,



Gradation basically - well graded
- poorly graded
  - uniformly
  - Gap graded



So কখনও plastic চাই না, so from limits we can conclude soil ভাল না খারাপ,

Definition of limits:

* liquid limit:
ω_L = water content W_L = weight

Small shear strength = 1.7 KPa

* Plastic Limit:

3. Shrinkage Limit:

Smallest water content, saturated water content below this volume change হবে না even if we change H_2O content.

Small shear usually machine দিয়ে করা যায় না,

Liquid Limit Determination: for fine grain soil

- Casagrande's Method
- Cone Penetration method

→ demarkation line of fine & medium sand

Prep of soil: Dry in air → powder → seive (#40)
0.425 mm

↓

mix with water → keep for 24 hrs

↓

throughly mix

7.10.15
Wednesday

Lec - 7

Determination of liquid Limit :

1) Casagrande's Method : (Popular in USA)

25 blow এর পর $\frac{1}{2}$" close হলে it's in liquid limit. আগের soil নিয়ে water content নিব then আবার water add

No. of blow	w. content
34	51
29	59
24	63
19	71
16	73

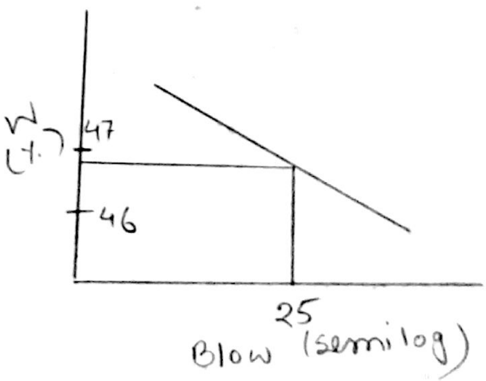
max 35 এর কম min 15 blows.

Try and error method.

25 blows এ $\frac{1}{2}$" closing এর জন্য w.content liquid limit.

liquid limit whole number unless it's less than 10.

water 47 এর কাছাকাছি,
so it's 47.
Flow curve



2) Cone Penitration :

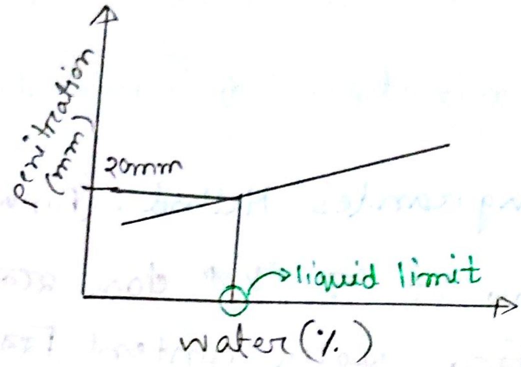
penitration হল ▽→ এর surface এর shear strength.
20 mm penitrate করলে সেটা liquid limit. suppose 12.5 mm হল, so we need to add more water.

In scale এক ঘর = 0.1 mm

previous এ X এ blow

Y এ w(%)

এখানে উল্টা।



Q. What is flow index?

Flow curve এর slope flow index. এটা দিয়ে strength measured

we can use one point to determine liquid limit.

1st method এ যত 25 এর কাছে আসবে তত ভালো।

Then for single point method in (1) :

$$\omega_L = \omega\left(\frac{n}{25}\right)^{e \to 0.1}$$

$$= 63\left(24/25\right)^{0.1} = 62.74 \approx 63$$

25 এর কাছাকাছি থাকলে better result

$$\omega_L = 71\left(19/25\right)^{0.1} = 69$$

(*) cone penitration এ single point method can be used

cone penitration for " " should be ~~26-60 16~~.

26 - 16 (water content)

Plastic limit holds almost 100% more strength than liquid limit.

(*) Plasticity index = diff bet^n liquid & plastic limit

(PI or I_P) = LL − PL

(*) Liquidity Index (LI or I_L) = $\frac{w - PL}{LL - PL} = \frac{w - PL}{I_P}$ (w = natural water content)

⇒ If the natural water content যদি LL এর কাছে then LI = 1. (soil soft)

⇒ if " " " " PL " " " LI = 0 (soil stiff)

⇒ LL & PL of ~~water~~ a soil is fixed. only determiner is w. So LI can be negetive. If a soil is dry then w = 0.

⇒ LI > 1 is unlikely. Cause LI এর বেশি w থাকলে flow করবে. But soil slurry হিসেবে থাকলে it can be > 1.

⇒ consistensy Index CI or $I_C = \frac{LL - w}{LL - PL}$

just reverse of LI.

⇒ Flow Index

⇒ Toughness Index: Measure of strength of soil = $\frac{I_P}{I_F}$
I_T

Determination of shrinkage limit:

swelling soil কে classify এর জন্য.

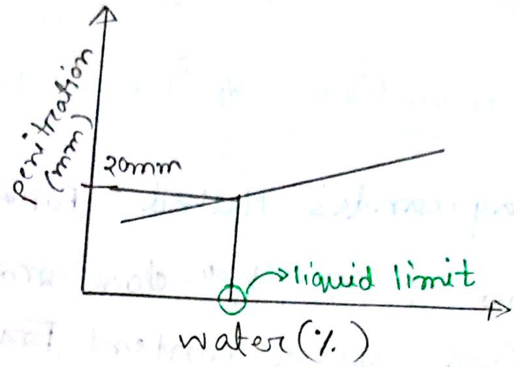
Bet^n solid and semisolid w content.

Sh. Limit is the saturated water content of the soil at its minimum volume during the process of drying.

previous এ X এ blow

Y এ w(%)

এখানে উল্টা.



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↑ natural water content

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Determination of shrinkage limit:

swelling soil কে classify এর জন্য।

Betn solid and semisolid ω-content.

Sh. Limit is the saturated water content of the soil at its minimum volume during the process of drying.

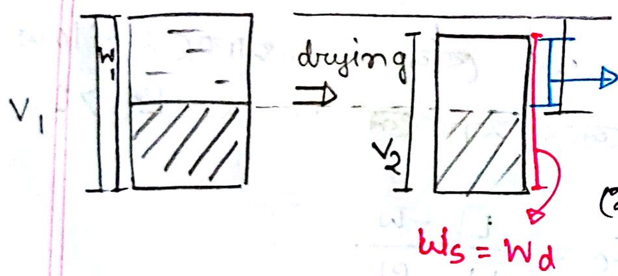
Oven এ dry করলে irregular shape হবে, এই অবস্থায় (min volume এ) যে w content.

i) S.g of soil is unknown

ii) S.g " " " known

i) SG unknown:

When soil is made slurry then phase dia soil + water cause voids are filled with H_2O



dry করলে এখানে air void. এটা min volume. এখানে air না থেকে H_2O থাকলে Shrinkage limit

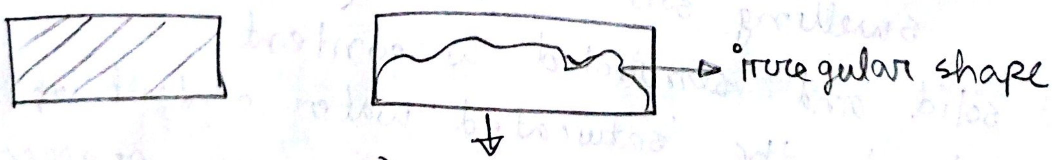
$W_1 - W_d$ করলে initial water weight.

$V_1 - V_2$ এ যে water content হল কতটা water কমল,

This is $(V_1 - V_2)\gamma_w$

$$\text{Shrinkage limit} = \frac{(W_1 - W_d) - (V_1 - V_2)\gamma_w}{W_d}$$

* solid soil never changes volume.



এটার vol বের করতে mercury used.

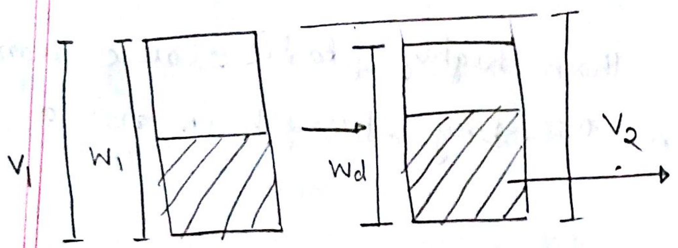
এই পাত্রে mercury নেব, whole soil shape immerse করব, যে mercury replaced হবে তার $W_{Hg}/13.6$ হল vol m.

10.10.15
Saturday

Lec-8

Determination of shrinkage limit:

⊛ SG is known:



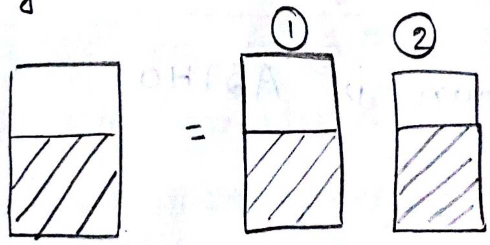
$V_s = \frac{W_d}{SG}$

$$\text{Volume of void} = \left(V_2 - \frac{W_d}{G_s}\right) \times \gamma_w$$

$$\text{shrinkage limit} = \frac{\left(V_d - W_d/G_s\right) \times \gamma_w}{W_d}$$

Q. write derivation of shrinkage limit if Gs is known and unknown.

* S.L is 5, S.L is 10
This is a better soil. We want the volume of the soil not to change. If volume change বেশি হয় then void এর volume change হবে।



① এর deformation কম, so shrinkage limit বেশি,
② ,, ,, ,, , so SL কম, so SL যত বেশি তত ভাল soil.

Determination of Linear Shrinkage :

Shrinkge বেশি হলে length shrinkage measured. Linear shrinkage যত ↑ soil টা poor. L.shrinkage বেশি হলে deformation ↑.

Non-plastic soil better than highly plastic. Cause non plastic এ deformation কম, Dry strength is not a criteria.

Relation LS, Plasticity Index ⇒ $I_p = 2.13\,LS$

Soil H_2O এর contact এ আসলে expansive হবে নাকি see from shrinkage limit. SL ↑ হলে linear shrinkge ↓.

Shrinkage potential low থাকে when it comes in contact with H_2O it will expand less.

Singificance of Indices :

Q. কোন index → like flow, plasticity index ↓ / ↑ হওয়া ভাল?

Unified Soil classification system :

Another clossification is ASTHO.

construction এ USCS

Road, airport ASTHO

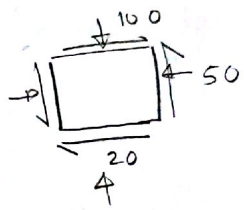
Mohr Circle Diagram:

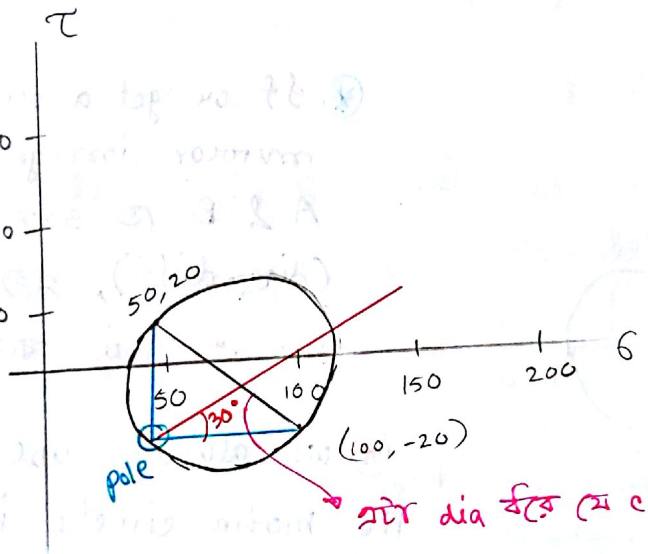
soil এ most of the load compressive. This is (+ve).

counter clock wise = (+ve)

Shear

σ, τ plane. in same scale





এটার dia ধরে যে circle

consider two orthogonal plane

in horizontal plane (100, -20)

counter clockwise

In verticle (50, 20)

pole of the Mohr Circle / ~~centre of~~ controling point

100, -20 acts on a horizontal plane, এর parallel এ line

50, 20 " " a verticle plane, এর " এ line.

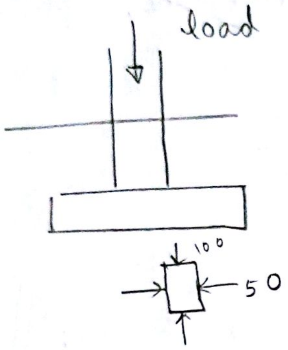
Intersection হল pole of Mohr circle

30° বাঁকে horizontal এর সাথে plane

4.11.15

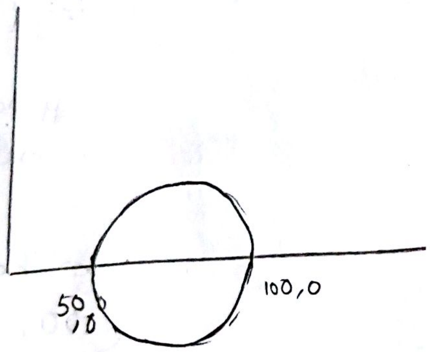
Wednesday

Lec - 11

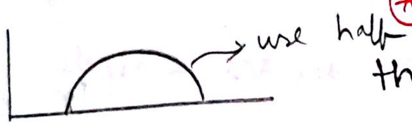


soil granular হলে lateral force থাকবে on stress element

if no shear then we call it principal stress



* If we get a circle which is mirror image of x axis A & B তে same shear (dir. diff), এজন্য কোন dir. এ fail করবে বলা diff.



* we always use half of the mohr circle in soil mecha. cause দুই point এর opposite shear ও থাকবে.

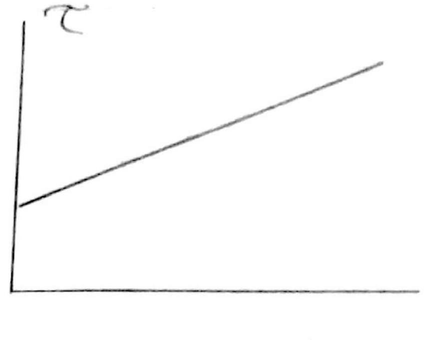
Mohr Coulumb Failure theory :

$\tau = c + \sigma \tan\phi$ ↳ slope angle

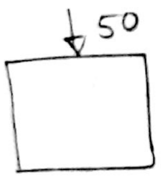
c ↓ intersection of Y

↓ shear strength parameter

Mohr বলেছে failure stresses on the circumfarance, coulumb বলেন failure stress particular plane এ shown by st. line.



so ২টো satisfy করতে the st. line must be a tangent on the mohr circle.



$$\tau_f = c + \sigma_f \tan\phi$$

50 জানা, so σ_f জানা, to determine τ_f we need to known → c & ϕ

c ⇓ apparent cohesion

ϕ ⇓ apparent angle of internal friction.

Dete. of shear strength parameter:

অনেক method, we use 4 in this course.

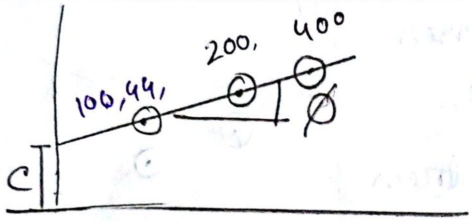
*) Direct shear:

Suppose normal 100, shear 44. এ particular plane এ failure

Again $N = 200, \tau = 50$

$N = 300, \tau = 55$

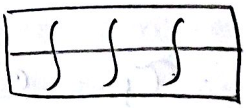
Scanned by CamScanner



Adv → simple & quick

Disadv → particular predetermined plane এ failure, weakest plane এ না।

In case of plastic soil deformation of fiber

But shear হতে হলে fibers  হবে, so shear failure

আলাদা হতে হয় like  এর false impression দেয়।

11.11.15
Wednesday

Lec - 13

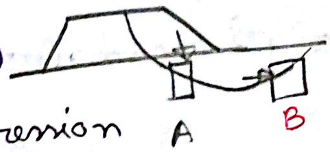
Direct Shear Test (continued):

২ ভাবে fail করান যায় by verticle & by horizontal load.

⇓ Triaxial ~~direct shear~~ compression

⇓ ~~direct shear~~ Extension

Triaxial — compression
Triaxial — Extension (not very common)



A fails in verticle load, so compression

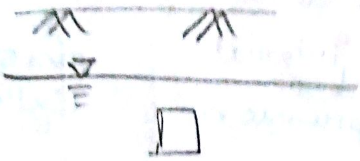
B তে horizontally fail করান হচ্ছে though the verticle load is constant.

cell কে ভরাব, cell এ যে pressure দিব তা equal. এটা (cell pressure)
loading ram দিয়ে load দেব, soil sample এ যাবে।

Here we can make the soil sample saturated.
Bottom & top এর ২টা connection কে water sample এর সাথে connect করব।
এটা হল back pressure.
যখন অনেক pressure দিয়ে saturated করার try করলে crack form করবে, so balance by back pressure.

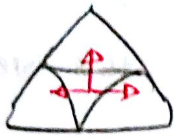
Saturated হলেই triaxial test.

construction stage } both case এ excess pore H_2O pressure হবে।
loading ,,



A তে H_2O pressure হবে static pore ,, ,, .

এর উপর extra load দিলে excess pore H_2O pressure হবেই concern.



এখানে extra pore H_2O p ঢুকবে দেবে, so উপরে উঠবে।

Depending on two stages

1) consolidation (in contruction stage) – Consolidated

2) Drainage (in loading ,,) – Drained

H_2O এ যেকোন pressure develop করলে leakage হতে চাইবে, interparticular gap দিয়ে পানি ~~[illegible]~~ বের হবে so expulsion of H_2O & E.P H_2O p. decrease করবে।

Drainage ও একই as consolidation, just to diff the stages এই name.

* Triaxial test 3 divisions:

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1) Unconsolidated Undrained — UU Test - Quick Test — Q Test

2) Consolidated Undrained — CU Test — R Test

3) Consolidated drained —— CD Test - Slow Test - S test

We don't want the P. H_2O pressure to decipate.
so fast load apply করতে হবে, তখন UU present
so quick test

CD → Slow load apply করি যেন H_2O decipit করে

Soil sample saturation এর পর construction এর load
দেব cell pressure দিয়ে,
Extra load টা দিব উপর দিয়ে।
If we want Q Test then top & bottom connection
close করব, so H_2O বের হতে পারবে না।

R-Test : construction অনেক slow হলে consolidated
loading fast দিলে undrained.
Undrained হলে connection of H_2O tube closed.
Drained " " " " open.

(*) cell pressure apply করে construction stage stimulate
করব and তখন H_2O tube খোলা, তাহলে consolidated
হবে।

(*) water expell হচ্ছে নাকি বুঝতে drainage tube এ
burette লাগায়, তাহলে H_2O এর পরিমাণ determine
করতে পারি, usually takes 2 days for total drainage
according to cell pressure.

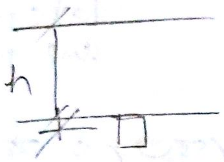
21.11.15
saturday

Lec - 16

LATERAL EARTH PRESSURE

Earth pressure 2 types

1) V. pressure — for over burden, ~~eggily~~ easy to determine



$h \times \gamma_{soil}$

2) Lateral E.P : $K \times \gamma h$
↳ co-eff of lateral earth pressure.

Automatically induced in soil.

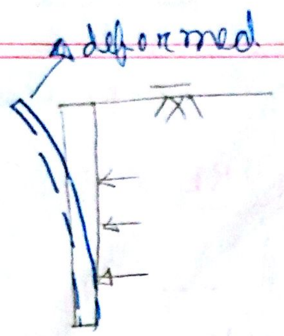
Depends on : 1) Physical characteristic of soil
2) Deformation " of structure.

Soil 2 types : cohesive, cohesionless

1) So from phy. chharacteristic of soil — cohesionless soil / cohesive soil

if 2 ~~poris~~ parameters, of friction exist then $\boxed{c - \phi \text{ soil}}$.

2) Def. of str — i) No deformation
ii) lateral expansion (Active Earth pressure)
↓ lateral stress max^m due to shear
so failure condition taken
iii)

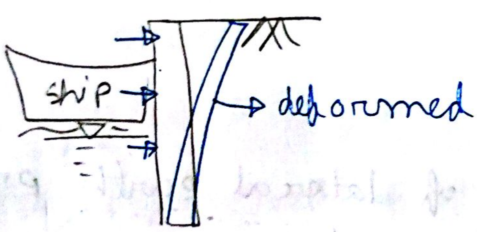


so soil side এ ধাক্কা দিবে and ফিরেও যাবে।

This is lateral expansion, movement of soil outward & down

iii) Lateral contraction: (Passive earth Pressure) (Min^m lateral EP) due to shear

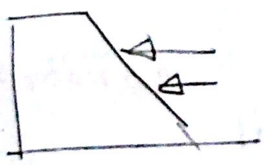
movement of soil inward & upward



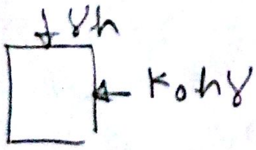
ship ভিড়লে ভেতরে ধাক্কা দিবে on retaining structure.

(*) In case of no deformation of structure, we call it earth p. at rest.

no deformation when the structure is massive



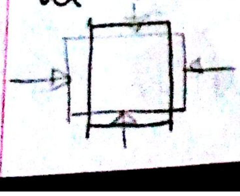
1) No Deformation:



K_0 = co-eff of earth p. at rest

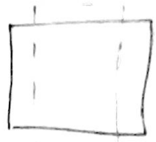
2) if $K\gamma h$ reduced so the soil expands and fails at max^m shear.

K_a = co-eff of act. E.P



3) fails due to lateral contraction. ↑ $K\gamma h$, so contract করে fail করবে,

so K_p = co-eff of passive E.P

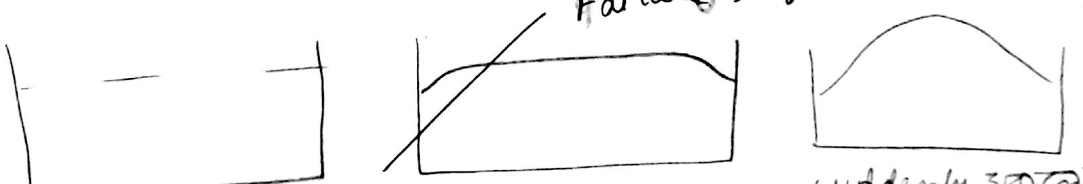


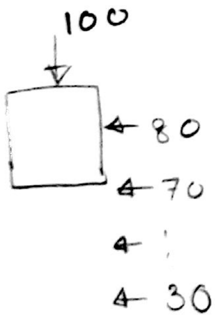
Q. Def^n K_o, K_p, K_a.

The ratio of Lateral to V E.P at this condition is co-eff of ~ . $K_o = \frac{K_o \gamma h}{\gamma h}$

A.E.P:

* To reduce L.E.P we can move the support apart



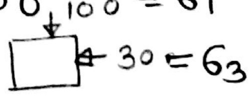


→ suppose 30 করলে failure surface develop করবে, 20, 10, 0 তেও " " থাকবে.

so max^m shear at which failure 30

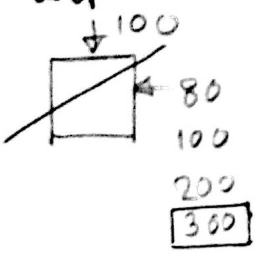
$\therefore$ A.E.P = 30 $\qquad \therefore K_a = \frac{30}{100} = \boxed{\sigma_3/\sigma_1}$

$100 = \sigma_1$

At failure condition 

P.E.P:

lateral contraction এ fail করাতে ↑ L.P.



300 এ L.P দিলে failure surface.

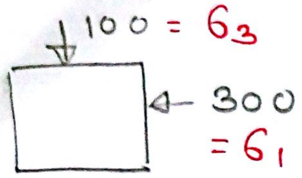
400, 500, 600 দিলেও " " থাকবে,

so the min^m shear = 300

Q. 100 কি always 300 থেকে কম দিতে হবে?

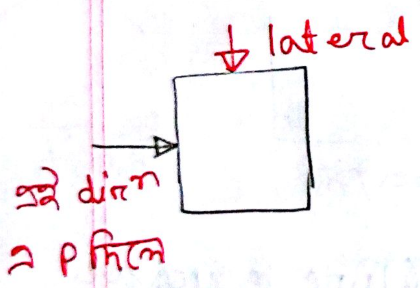
$\therefore$ P.E.P = 300

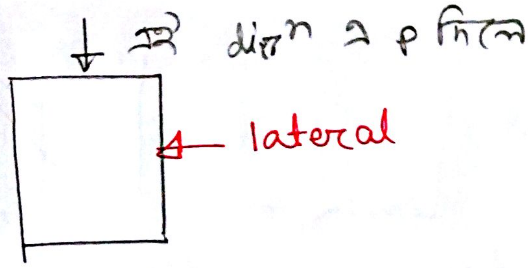
$\therefore K_p = \frac{300}{100} = 3.00 = \boxed{\frac{\sigma_1}{\sigma_3}}$



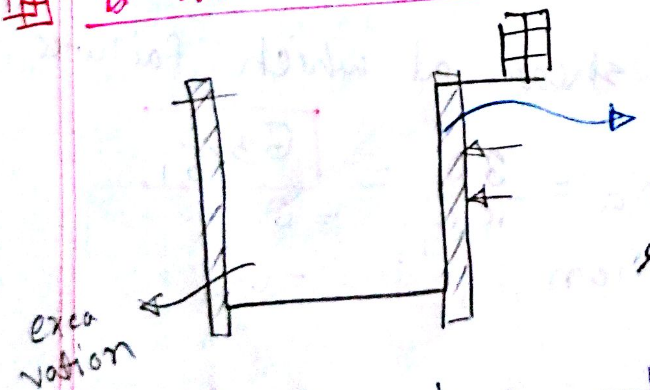
(*) Practically V. load doesn't change that much. so we are only considering lateral load.

$K_a < K_0 < K_p$



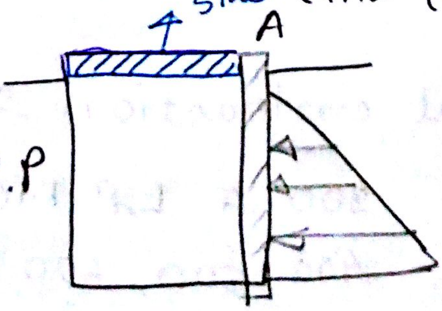


E.P at rest :



stiffness কম হলে moves away, so A.E.P

still থাকলে E.P @ R.



(2nd অংশের)

so if the p is so this will be A.E.P

so this will be A.E.P cause A point free.

যে moment এ slab বানালাম, A আটকে গেল, so E.P @ rest.

যেন A.E.P এ fail না করে সেজন্য strutting/bracing দেয় হয়, Then consider E.P @ Rest.

Bracing না দিলে consider A.E.P.

we don't want deformation, so want E.P@R.

E.P@ Rest :

$\sigma_h = \sigma_3 = K_0 \sigma_v = K_0 \gamma h$ $\quad\because \sigma_v = \gamma h$

↳ verticle p.

$K_0 = \frac{\sigma_h}{\sigma_v}$

Lateral strain $\epsilon_h = \frac{1}{E[\sigma_h - \underset{\Downarrow\ \text{poisons ratio}}{\mu}(\sigma_v + \sigma_h)]}$ $\Rightarrow \sigma_h =$

we won't allow deformation $\therefore \epsilon_h = 0$

$\sigma_h - \mu\sigma_v - \sigma_h\mu = 0$

$\Rightarrow \sigma_h(1-\mu) = \mu\sigma_v$

$\therefore \sigma_h = \frac{\mu}{1+\mu}\sigma_v$ and $\sigma_h = K_0 \sigma_v$

$\therefore$ from there two $K_0 = \frac{\mu}{1-\mu}$

determination of μ is very difficult, so try to avoid it.

So we use emphirical relations.

1) ~~cohesionless (sandy)~~ ~~normally consolidated~~ soil এ ϕ জানলে

for flat Ground surface

Jaky's formula: $K_0 = 1 - \sin\phi'$

∴ ϕ determined in drained condition. (for sandy and NC)

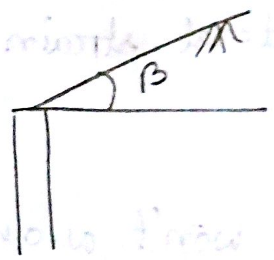
K_0 for over·C $= (1-\sin\phi) . \underbrace{OCR}_{\text{over consolidation ratio}}$

(*) If we determine LL, PL and PI : For clayey

$K_0 = 0.19 + 0.233 \log(I_P)$ (NC)

$K_{0(OC)} = K_0 \sqrt{OCR}$ (OC)

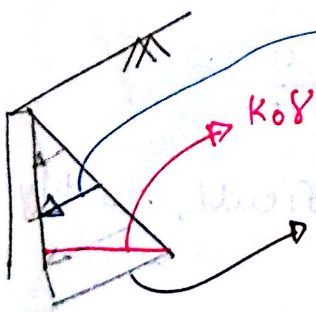
(*) If inclined soil then P ↑ फिर



?? $(1+0.5\tan\beta)^2 K_{0(OC)} = \sqrt{OCR} . K_0$

(*) L.E.P. always works parallel to earth surface

??



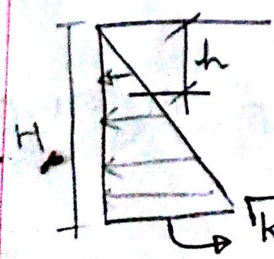
total thrust → ∴ $P_0 = \frac{1}{2} K_0 \gamma H^2$

$K_0 \gamma H \cos\beta (1+0.5\tan\beta)^2$

$K_0 \gamma H (1+0.5\tan\beta)^2$ $\quad \cos\beta(1+0.5\tan\beta)^2$

[Find hor. component of thrust $\times \cos\beta$]

$\gamma = 16 \, KN/m^3 \quad h = 5m$



$K_0 = \gamma h$ ∴ Total thrust कुल force

force ?? pressure maybe so unit $\boxed{KN}$

$\boxed{K_0 \gamma H} = KN/m^2$

$\therefore$ Total thrust per unit width $= \overbrace{\tfrac{1}{2} K_0 \gamma H}^{KN/m^2} \times \overbrace{\boxed{H}}^{m}$

$= \underbrace{\tfrac{1}{2} K_0 \gamma H^2}_{\boxed{KN/m}}$.

Lec - 17

25.11.15
Wednesday

Earth P. Theory:

1) Rankine's theory
2) Coulombs theory

} for A & P E.P } for cohesionless and cohesive soil

And under the heading of horizontal and inclined ground surface

water Table
soil stratification
load

} Graphical method
1) Culmann method
2)

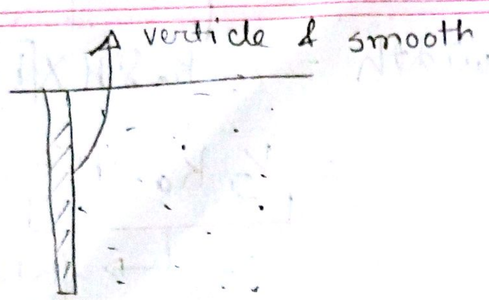
A.E.P

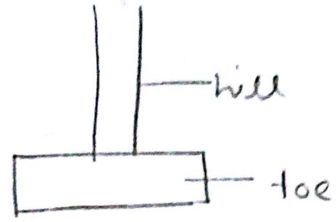
Cohesionless soil:

Rankein's theory:

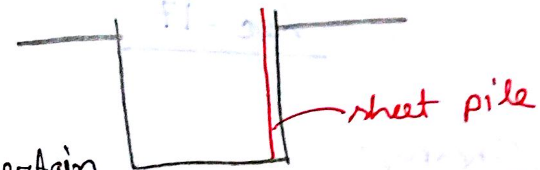
→ infinite soil
homogeneous
Ground surface horizontal/inclined

most imp assum: soil and retaining wall 's contact surface verticle & smooth



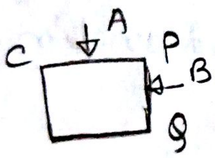


basement করতে retaining wall design করা যায় না, তখন sheet pile / pile.



wall smooth করতে certain কাজ করতে হয়, plastic দিয়ে casting করলে smooth.

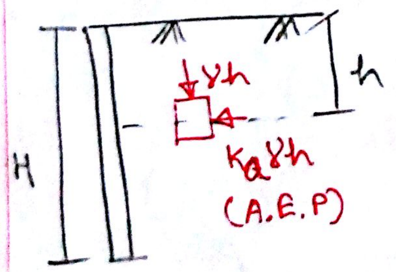
4.



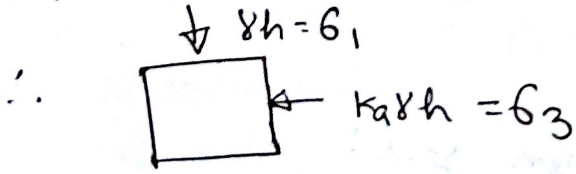
stress element এর stress A parallel to PQ

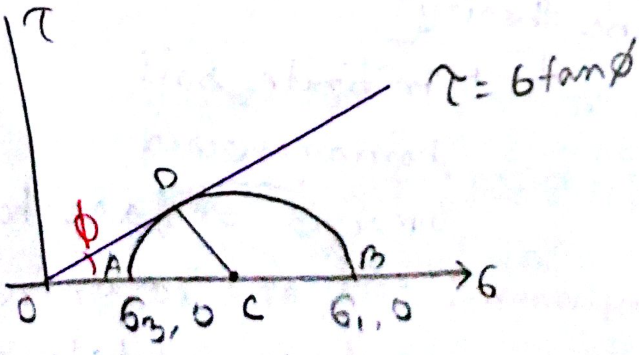
stress at B parallel to PC

Let's assume that active earth pressure prevails



$K_a\gamma h < \gamma h$





cohesionless soil $\therefore \tau = \cancel{c}^{0} + \sigma \tan \phi$

$\therefore \tau = \sigma \tan \phi$ (failure envelop) passing through origin

$$K_a = \frac{\sigma_3}{\sigma_1} = \frac{OA}{OB} = \frac{OC - AC}{OC + BC}$$

$$= \frac{OC - CD}{OC + CD}$$

$$= \frac{OC/OC - CD/OC}{OC/OC + CD/OC} = \frac{1 - CD/OC}{1 + CD/OC}$$

$CD/OC \rightarrow \sin \phi$

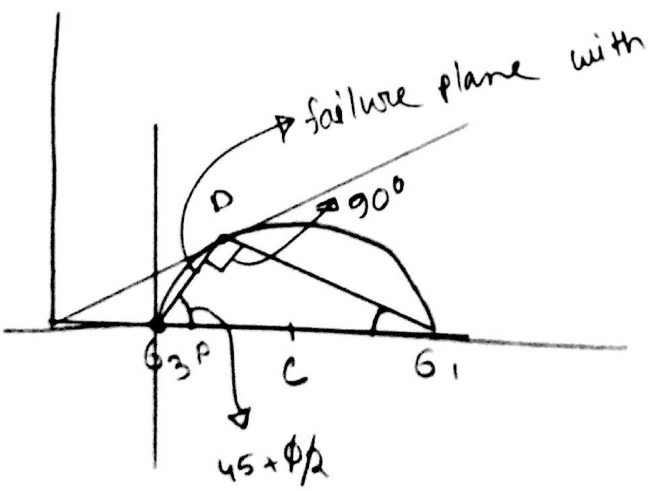
$$\boxed{\therefore K_a = \frac{1 - \sin\phi}{1 + \sin\phi}} = \tan^2 (45^\circ - \phi/2)$$

* Angles must be in degrees.

* Orientation of failure plane:

σ_1 working horizontal so —

σ_3 " vertical plane so |



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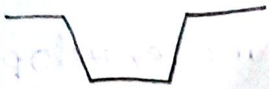
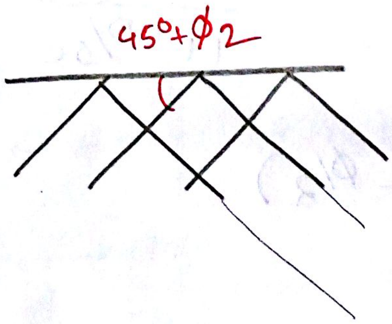
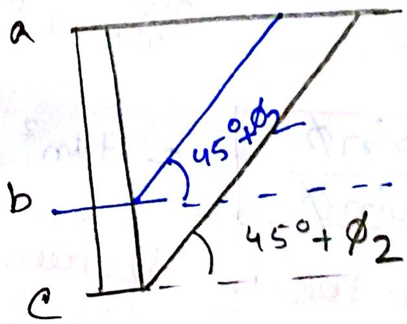
 slope maintain করি cause always fails at $45° + \phi/2$

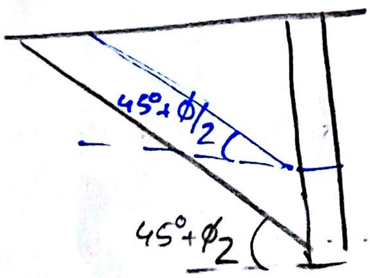
Depending on height of retaining wall failure ori.

$\boxed{45° + \phi/2}$

Q. Draw or. of failure plane for A.E.P







ab height হলে blue failure orientation.

ac height ,, নিচেরটা

so 2 দিকেরই হতে পারে failure depending position of soil.

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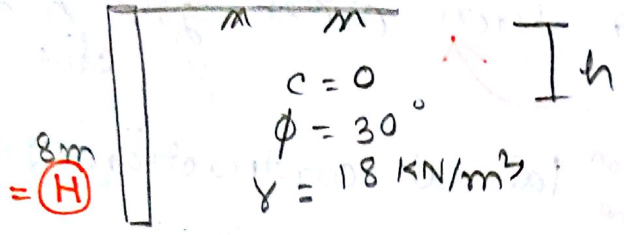
Problem:



$c = 0$

$\phi = 30^\circ$

$\gamma = 18\ KN/m^3$

Ac. Thrust = ?
↓
force

Active E. Pressure এর জন্য করার

St-1:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$

St-2:

Active E.P P_a at any depth h $\quad P_{a(h)} = K_a \gamma h$

$$= 0.333 \times 18 \times h$$

$$= 6h$$

$\therefore P_{a(h)} = 6h$ (st. line eqn)

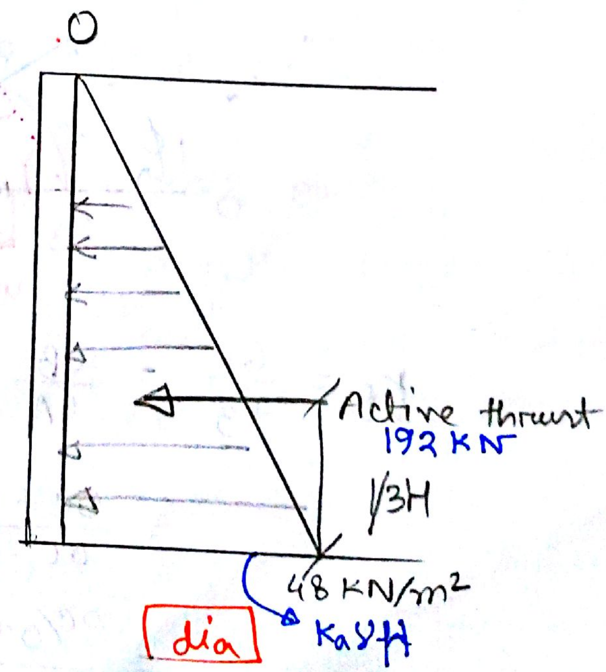
Step-3:

$P_{a(0)} = 6 \times 0 = 0$

$P_{a(8)} = 6 \times 8 = 48\ KN/m^2$

⇓

it is pressure



* Area র জন্য unit length consider

$P_a = 1/2 \times 48 \times 8 = 192\ KN$

(Thrust)

$P_a = 1/2\ K_a \gamma H \times H$

$= 1/2\ K_a \gamma H^2$

} dir. এটাও use করা যায় if diagram not wanted

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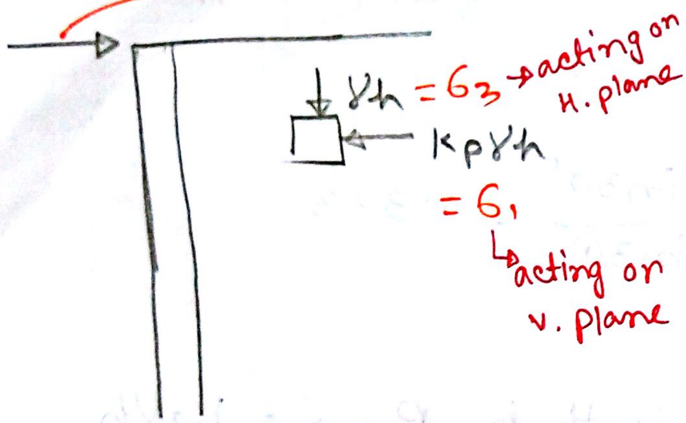
Pr - 2 : Passive

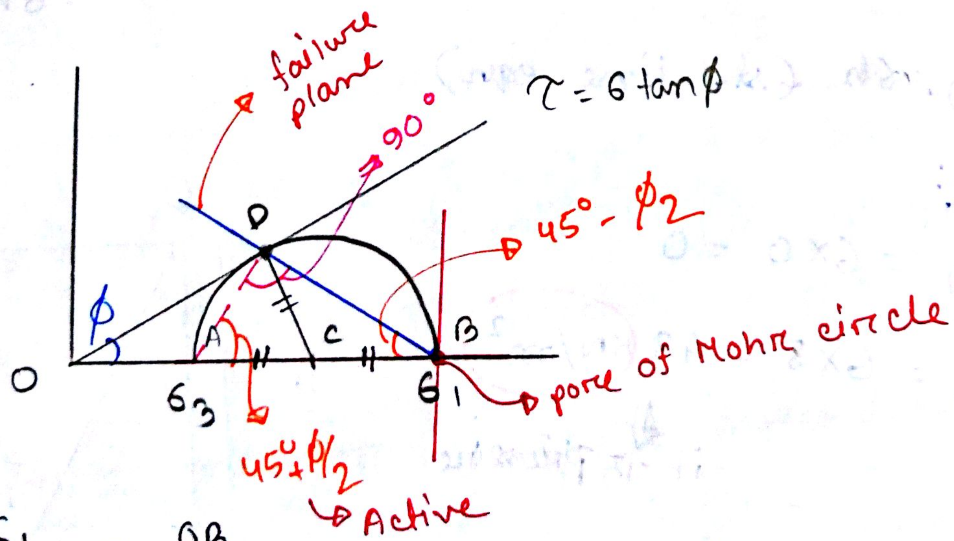
(Exm এ বলা না থাকলে dir^n দেখে বুঝব A না P]
ctive assive



lateral contraction এ fail করতে হবে.

so $K_p\gamma h > \gamma h$

soil element very close to retaining wall, wall টা smooth, so downward movement এবং no friction

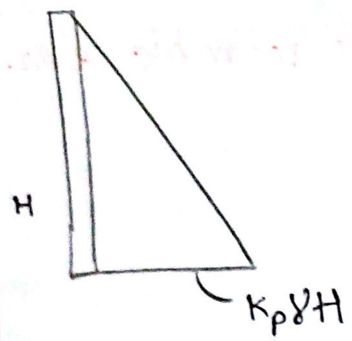


$$K_p = \frac{\sigma_1}{\sigma_3} = \frac{OB}{OA}$$

$$= \frac{OC + BC}{OC - AC} = \frac{OC + CD}{OC - CD}$$

$$= \frac{OC/OC + CD/OC}{OC/OC - CD/OC} = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1}{K_a}$$

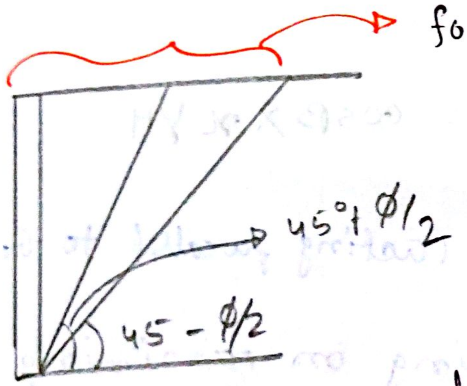
Total Thrust for P.E.P



$$P_p = \frac{1}{2} K_p \gamma H^2$$

Draw orientation of failure plane for P.E.P?

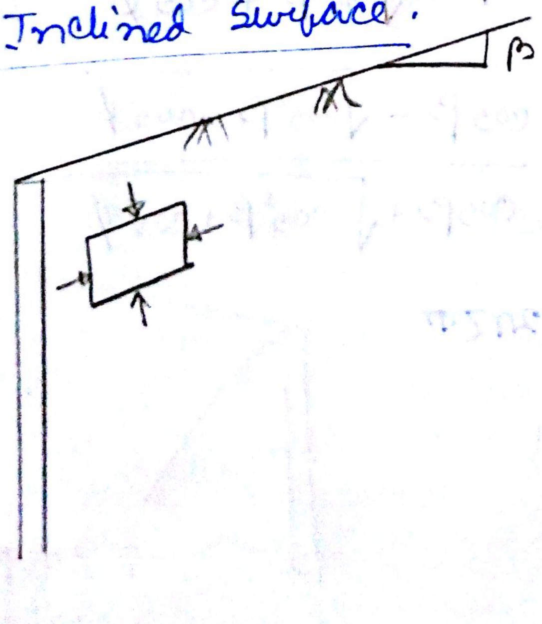
passive এ fail করাতে movement ↑ লাগবে।



for passive, so passive এ fail করতে distance বেশি cause failure in passive এ 45° - φ/2 এ fail করবে। এটা achieve করতে অনেক deformation লাগবে।

Q. কোনটায় ↑ movement & why?

A.E.P → Inclined Surface:

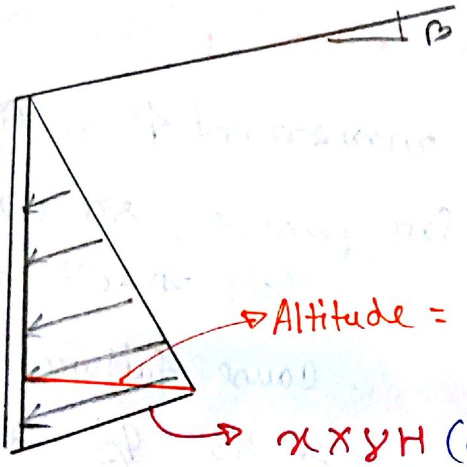


যেহেতু earth p. parallel to ground surface, so rombick element চিন্তা করব।

x

$$k_a = \left(\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta + \cos^2\phi}}\right)$$ [For rombic element]

Pressure Distribution Dia:



→ Altitude = $\cos\beta \times x\gamma H$

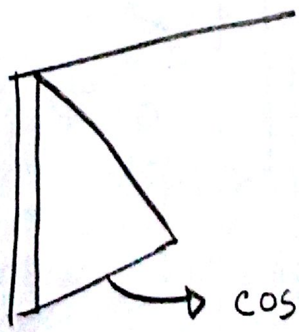
→ $x \times \gamma H$ (acting parallel to G.S)

Total active thrust coming on retaining wall

$$P_a = \frac{1}{2}\cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta + \cos^2\phi}}\gamma H . H$$

$$= \frac{1}{2}\gamma H^2 \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta + \cos^2\phi}}$$

(*) in most cases বলা থাকে



→ $\cos\beta . x$

$P_a = \frac{1}{2} K_a \gamma H^2$

Pressure dis. diagram এ must use $\boxed{x \text{ as } K_a.}$

Thrust এ যেকোনটাই can be used.

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28.11.15

Saturday

Lec - 18

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

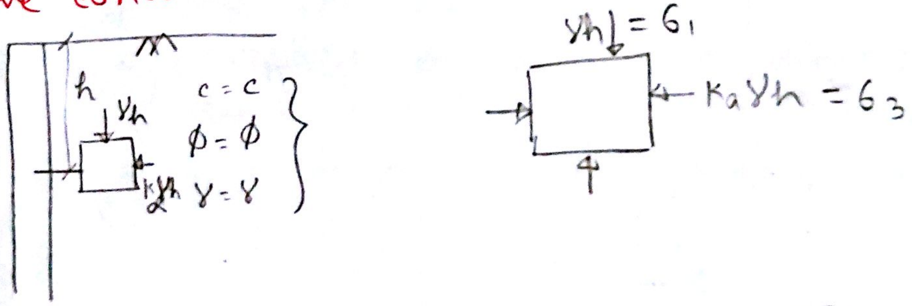
$$K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

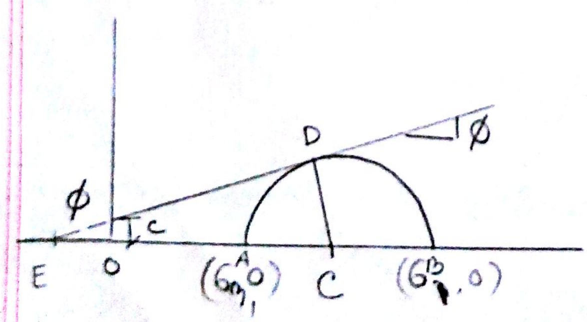
$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

If a soil has cohesion & friction both:– $c\phi$ soil

Active condition:





$$\sin\phi = \frac{CD}{EC}$$

$$= \frac{CD}{EO + OC}$$

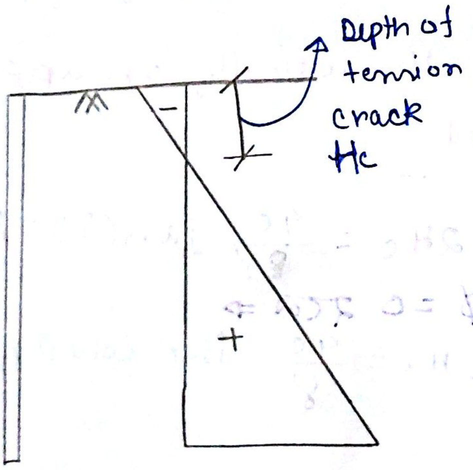
$$= \frac{\frac{\sigma_1 - \sigma_3}{2}}{c\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$ → diameter

$$= \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 + 2c\cot\phi}$$

$$\therefore \sigma_3 = \left(\frac{1-\sin\phi}{1+\sin\phi}\right)\sigma_1 - \boxed{\frac{2c\cos\phi}{1+\sin\phi}} \rightarrow \text{simplify} = \sqrt{K_a}$$

$$= \left(\frac{1-\sin\phi}{1+\sin\phi}\right)\sigma_1 - 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

$$\boxed{\sigma_3 = K_a \gamma h - 2c\sqrt{K_a}}$$ Active E.P for $c\phi$ soil



$h = 0$ হলে $\sigma_3 = -2c\sqrt{K_a}$

Pressure (−) মানে tension, soil can't take much tension

Tension নিতে না পারলে crack.

At H_c lateral pressure zero.

$$\therefore \sigma_3 = 0$$

$$\therefore 0 = K_a \gamma H_c - 2c\sqrt{K_a}$$

$$\Rightarrow H_c = \frac{2c\sqrt{K_a}}{\gamma K_a}$$

$$= \frac{2c}{\gamma\sqrt{K_a}}$$

$$= \frac{2c}{\gamma}\frac{1}{\sqrt{\tan^2(45-\phi/2)}}$$

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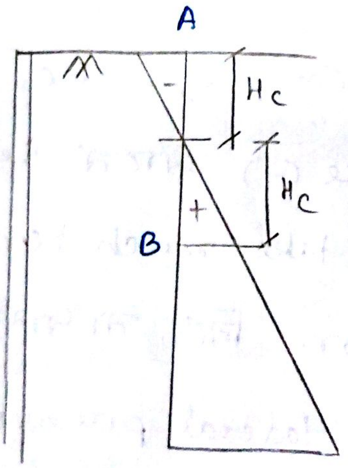
$$= \frac{2c}{\gamma} \cot(45 - \phi/2)$$

$$= \frac{2c}{\gamma} \tan(45 + \phi/2)$$ [Prove for a cϕ soil the depth of tension crack . . .]

for purely cohesive soil $\phi = 0$

$$\therefore H_c = \frac{2c}{\gamma}$$ [Prove for cohesive soil]



AB তে total thrust zero.

Called theoretically unsupported height

$$H_u = 2H_c = \frac{4c}{\gamma} \tan(45 + \phi/2)$$

$\phi = 0$ হলে ⇒

$$\therefore H_u = \frac{4c}{\gamma}$$ (for cohesive soil)

$c = 20 \ KN/m^2$

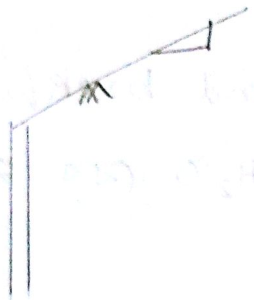
$\gamma = 16 \ KN/m^3$

$\therefore H_u = \frac{4 \times 20}{16} = 5m$, so theoretically এতটুকুতে support লাগবে না, cohesion থাকে & excavation এ

□ For Passive E.P:

$$\boxed{P_p = K_p \gamma h + 2c\sqrt{K_p}}$$ = G,

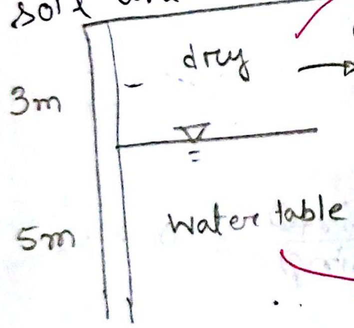
C-Φ soil with sloping surface:



graphical method used,
But formula আছে, don't need to memorize.

Effect of water table on A. & P earth pressure:

Active E.P is only for soil.
soil uniform



→ Here $\gamma = 18\ KN/m^2$

$\gamma = 18\ KN/m^3$
$\phi = 30°$
$c = 0$

or, $c = 0$
$\phi = 30°$
$\gamma = 18\ KN/m^3$

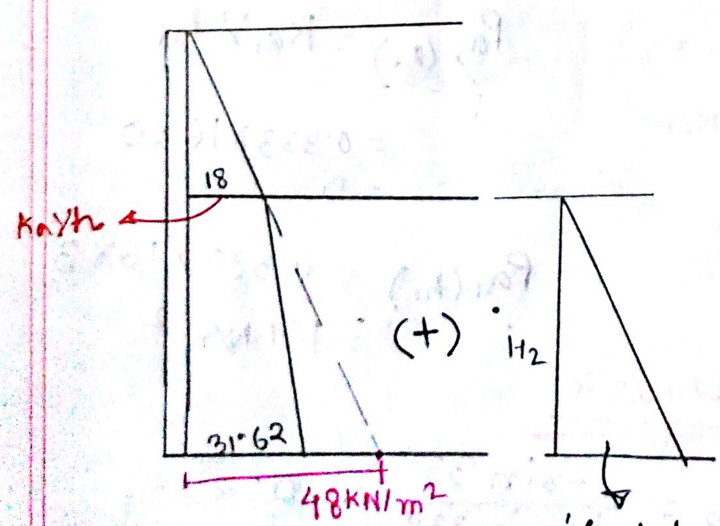
* অনেক সময় γ_{sat} ও বলতে পারে।

→ Here $\gamma' = \gamma_{sat} - \gamma_w = 18 - 9.8$

$K_a\gamma h = 0.333 \times 18 \times 3 = 18\ KN/m^2$

$0.333 \times 18 \times 3 + \overbrace{0.333 \times \underbrace{(18 - 9.81)}_{\text{submerged unit wt}}}^{\text{for } H_2O} \times$

$= 31.62\ KN/m^2$



H_2O না থাকলে, $K_a\gamma H$

$= 0.333 \times 18 \times 8$

$= 48\ KN/m^2$

if whole was filled with H_2O then $P = \gamma_w H_2$

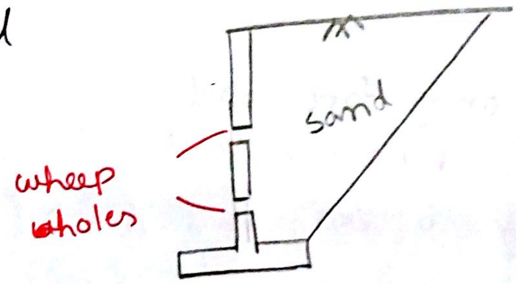
$= 9.81 \times 5 = 49.2$

∴ water থাকলে pressure develop করবে, it's not good.

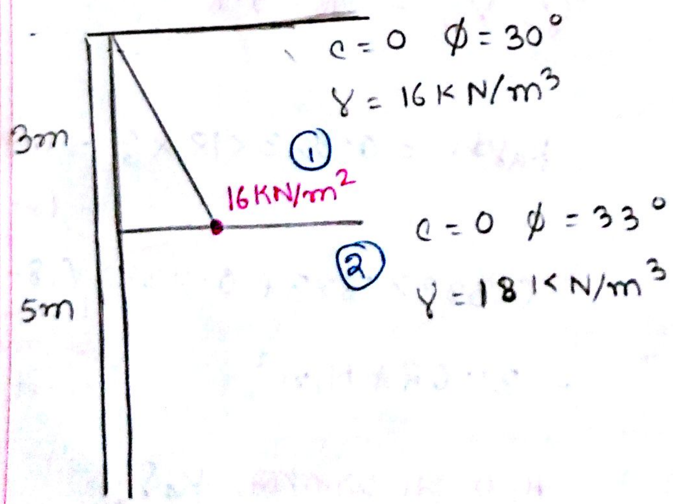
এজন্য retaining wall এ holes, and backfill material sand / coarse soil. যেন H_2O বের হতে পারে

clay থাকলে হবে না।

~~যতটা পর্যন্ত~~ earth pre. থাকবে $45+\phi/2$ or $45-\phi/2$ ততটা backfill



Effect of soil stratification:



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.333$$

Layer ①:

$$P_{a_1}(h_1) = K_{a_1}\gamma_1 h_1$$

$$= 0.333 \times 16 \times 0$$

$$= 0$$

$$P_{a_1}(h_1) = 0.333 \times 16 \times 3$$

$$= 16\ KN/m^2$$

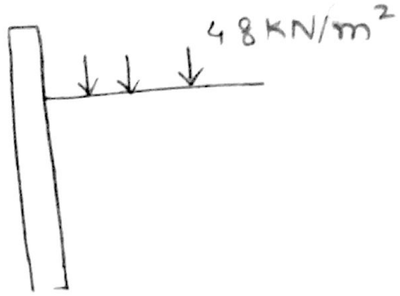
2) Layer 2

$$K_{a_2} = \frac{1-\sin 33^\circ}{1+\sin 33^\circ} = 0.295$$

এখানে soil uniform না

3m এ যে soil আছে তার জন্য verticle pressure

$= \underset{16}{\gamma} \times 3 = \underbrace{48}_{q}$ KN/m², এটাকে further replace করতে চাই.

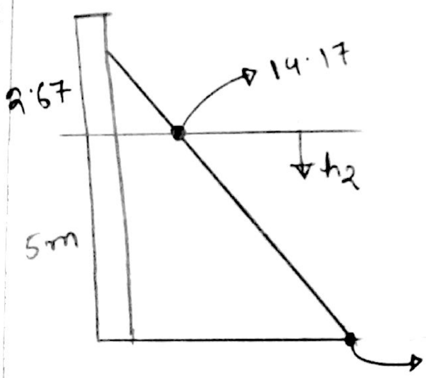


যদি 48 KN/m² এর জায়গায় layer 2 এর soil থাকত then কতটা height লাগবে of soil of layer 2.

$\therefore 48 = \gamma_2 H \Rightarrow \frac{48}{18} = 2.67$

∴ Equivalent height,

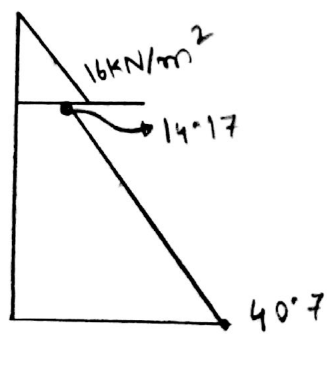
$H_e = q/\gamma_2$



$Ka_2 \times \gamma_2 \times (5 + 2.67) = 0.295 \times 18 \times 7.67$
$= 40.7$

$Pa_2(h_2) = Ka_2 \gamma_2 (H_e + h_2)$

If $h_2 = 0 \quad \therefore Pa_2(0) = Ka_2 \times \gamma_2 \times H_e$
$= 0.295 \times 18 \times 2.67$
$= 14.17$ KN/m² → for layer 2



এখানে break হয় because of change of angle of friction.

ϕ 2 তে বাড়ে so ভেতরে break.

ϕ 2 তে কমলে break বাইরে

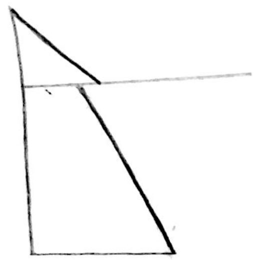
slope of the lines depend on γ↑ হলে slope flat.

γ↓ " " stiff.

Q. Exam এ বললে $\gamma_1 > \gamma_2$ and $\phi_1 < \phi_2$ then pressure distribution আঁক.

① slope flat

① break inside



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The **angularity of a bulky particle**, A is defined as

$$A = \frac{Average\ radius\ of\ corners\ and\ edges}{Radius\ of\ the\ maximum\ inscribed\ sphere} \qquad (3.1a)$$

The **roundness or shericity** of a particle is defined as the ratio of minimum radius of the particle edges to the inscribed radius of the entire particle.

$$S = \frac{D_e}{L_p} \qquad (3.1b)$$

Where,

D_e = equivalent diameter of the particle = $\sqrt[3]{\frac{6V}{\pi}}$

V = volume of particle

L_p = length of particle

Flaky particles have very low sphericity, usually 0.01 or less. These particles are predominantly clay minerals.

Needle shaped particles are much less common than the other two types of particles. Coral deposits and attapulgate clays are the examples of needle shaped particles.

(ii) **Effective size:** is the grain size corresponding to 10 percent of the passing by weight that is 10 percent of the materials are smaller than effective size. Effective size is termed by D_{10}.

(iii) **Coefficient of curvature, C_z:** The spread of the grain size distribution curve is expressed by this parameter as

$$C_Z = \frac{(D_{30})^2}{D_{60} \times D_{10}} \tag{3.4}$$

Where, D_{30} is the diameter corresponding to 30 percent finer. A coarse grained soil is considered well graded if the coefficient of curvature, C_Z is between 1 and 3 and C_u is greater than 4 for gravels and 6 for sands.

(iv) **Sorting coefficient of curvature, S_o:** This is another parameter to measure uniformity and is generally used by the geologist. The use of sorting coefficient is not frequent in geotechnical engineering.

$$S_o = \sqrt{\frac{D_{75}}{D_{25}}} \tag{3.5}$$

The definition of the sizes of the particles is illustrated in Fig. 3.16.

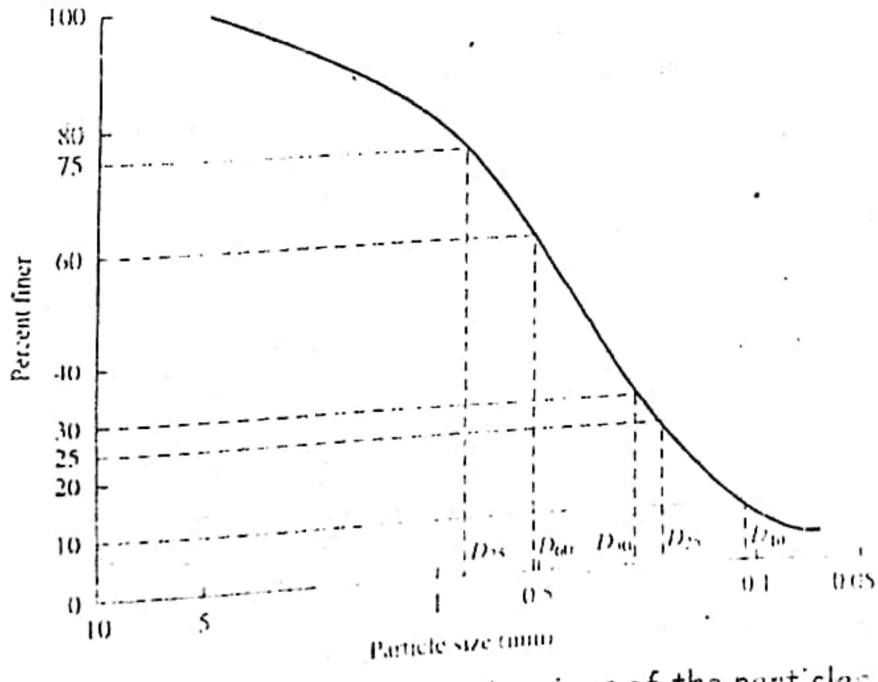

Fig. 3.16 Definitions of the sizes of the particles

Illustrative Examples

Example 1: From the data of a grain size analysis of a sandy soil, grain size distribution curve was drawn and the following results were obtained; D_{10} = 0.06mm, D_{30} = 0.20 mm and D_{60} = 0.40 mm. Comment on the gradation of the soil.

Solution:

$$Uniformity\ coefficient, C_U = \frac{D_{60}}{D_{10}} = \frac{0.40}{0.06} = 6.67$$

$$Coefficient\ of\ curvature, C_Z = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{0.20^2}{0.06 \times 0.40} = 1.67$$

Uniformity coefficient is greater than 6 and coefficient of curvature is in between 1 and 3. Both of them satisfy the criteria of well graded soil. So, the soil is well graded.

Example 2: A sample of soil was tested for liquid limit in a cone penetrometer and the following results were obtained.

Cone penetration (mm)	16	18	22	25	30
Water content (%)	33	40	50	63	83

Determine liquid limit and check the results for single point method using any of the data.

Solution: For the data, the plot of water content vs. cone penetration is drawn in Fig. E.3.1.

From Fig. E.3.1, Liquid limit, w_L of the soil is 46.

<u>Check for one point method</u>

The point considered is (w=40 [illegible]), [illegible] penetration and w the water content. Now, using Eq. (3.7)

$$w_L = \frac{w}{0.65 + 0.0175D}$$

$$= \frac{40}{0.65 + 0.0175 \times 18} = 42$$

Therefore, one point method gives a lower value of liquid limit (42) as compared to actual value of 46.

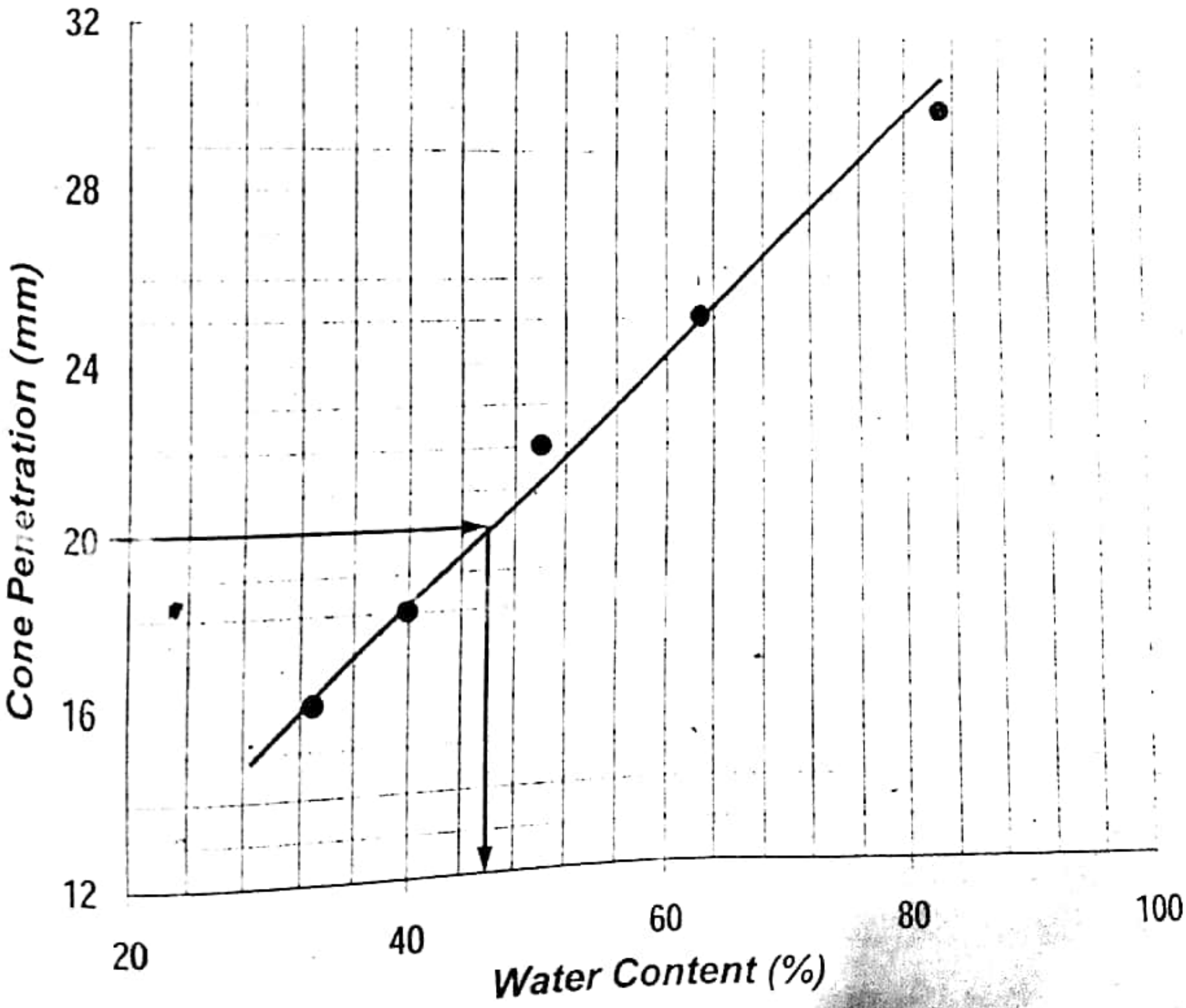

Fig. E3.1 Moisture content vs cone penetration curve

Example 3: A saturated clay sample has a volume of 62.5 cm^3 and a weight of 117.3g. On drying, sample attains a volume of 47.8 cm^3 and a weight of 85.6g. Determine the shrinkage limit of the soil.

Solution: Weight of water at the initial (saturated) condition = 117.3 - 85.6 = 31.7g

Volume of initial water = 31.7 cm^3.

Volume of soil solid = 62.5 - 31.7 = 30.8 cm^3.

Volume of water required to saturate the dry sample = 47.8 - 30.8 = 17 cm^3.

Weight of water to saturate the dry sample = 17g.

That is, weight of water at shrinkage limit = 17 g.

Therefore, shrinkage limit, $w_L = \dfrac{W_w}{W_s} \times 100 = \dfrac{17}{85.6} \times 100 = 19.9\ (\%)$

Table 3.3 Typical values of liquid and plastic limits(after Atkins, 1997)

Soil	Liquid limit, w_L	Plastic limit, w_P
Silt clay mixture	25-40	20-30
Kaolinite clay	40-70	20-40
Montmorillonite clay	300-600	100-200

Plasticity index is sometimes used to describe the soils. Table 3.4 gives a description of soils in terms of plasticity shows its relation with dry strength. The relevant field tests for the soils are also mentioned in the Table.

Table 3.4 Plasticity of soils (after Atkins, 1997)

Plasticity index, I_P	Term used for the soil	Dry strength	Field test
0 - 3	Nonplastic	Very low	Grains fall apart easily
4 - 6	Slightly plastic	Low	Easily crushed by fingers
7 - 15	Moderately plastic	Low to medium	Slight pressure required to crush
16 - 35	Plastic	Medium to high	Difficult to crush
Over 35	Highly plastic	High	Impossible to crush with fingers

The current state of a soil, in terms of Atterberg limits, is defined by the liquidity index (I_L or LI) which can be expressed as

$$I_L = \frac{w_n - w_P}{w_L - w_P} = \frac{w_n - w_P}{I_P} \qquad (3.9)$$

Soft soils have liquidity index near to unity, whereas, stiff clays may have values near to zero. Quick clays have a liquidity index greater than 1.0.

Toughness index, I_T is defined as the ratio of plasticity index (I_P) to the flow index (I_F). That is,

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The shrinkage limit of the soil can be calculated in two ways considering whether the specific gravity of the soil solids is known or unknown. Referring to Fig. 3.25, let,

W_1 = weight of wet soil in the shrinkage dish
V_1 = volume of wet soil (or dish)
W_d = weight of dry soil pat (W_s)
V_d = volume of dry soil pat (V_s)
G_s = specific gravity of soil solids

Shrinkage limit expression for unknown specific gravity

The determinations required in this case are the initial and final weights and volumes of the soil samples in the shrinkage dish. Thus,

Initial weight of water $= W_1 - W_d$
Volume decreased (Shrinked) $= V_1 - V_d$
Weight of water occupying shrinked volume $= (V_1 - V_d)\gamma_w$
Weight of water at Shrinkage limit $= (W_1 - W_d) - (V_1 - V_d)\gamma_w$

Therefore,

$$Shrinkage Limit, W_L \text{ or } SL = \frac{Weight\ of\ Water\ at\ Shrinkage\ Limit}{Weight\ of\ Soil\ Solid} = \frac{(W_1 - W_d) - (V_1 - V_d)\gamma_w}{W_d} \tag{3.12}$$

Shrinkage limit expression for known specific gravity

If the specific gravity, G_s, of the soil solid is known (predetermined), the only data required to be determined during this test is the weight and volume of the soil pat in dry state. Let W_d and V_d be the dry weight and volume of the soil pat respectively. As such

Volume of void at shrinkage limit $= (V_d - W_d/G_s)$
Weight of water required to fill the void $= (V_d - W_d/G_s)\gamma_w$

Therefore,

$$Shrinkage Limit, W_L \ or \ SL = \frac{Weight\ of\ Water\ at\ Shrinkage\ Limit}{Weight\ of\ Soil\ Solid}$$

$$= \frac{\left(V_d - \frac{W_d}{G_s}\right)\gamma_w}{W_d} = \left(\frac{V_d}{W_d} - \frac{1}{G_s}\right)\gamma_w \qquad (3.13)$$

Linear shrinkage test

The linear shrinkage is a measure of degree of decrease of a soil sample in one dimension. The apparatus, Fig. 3.26, mainly consists of a brass mould having a length, L_m, and semi-circular cross section, where the soil paste (slurry) is placed taking care not to entrap air. The surface of the soil paste struck off level. Similar to that of shrinkage limit test, the sample is initially dried in air until it shrunk clear of the mould and then placed it in an oven to complete the drying. After cooling, the length of the sample, L_d is measured and the linear shrinkage is obtained as follows:

$$Linear\ Shrinkage,\ LS(\%) = \left(1 - \frac{Final\ length}{Initial\ length}\right) \times 100$$

$$= \left(\frac{L_m - L_d}{L_m}\right) \times 100 \qquad (3.14)$$

For soils with very small clay content the liquid and plastic limits may not produce reliable results. An approximation of the plasticity index may be obtained by measuring the linear shrinkage and using the following expression (Whitlow, 1996).

$$I_p = 2.13\,LS \qquad (3.15)$$

Shrinkage limit and linear shrinkage, along with the other soil test results, can provide indication of the swelling properties of the soil. However, shrinkage test results are not directly used in any classification scheme. Table 3.5 presents qualitative swelling potential of soils against shrinkage limit, linear shrinkage and other soil properties.

Plasticity Index: Plasticity index (I_P or PI) is the range of water content within which the soil remains in the plastic state. It is equal to the difference between the liquid (w_L) and the plastic limit (w_P).

Liquidity Index: Liquidity index (I_L or LI) is the ratio of difference between the natural water content the plastic limit to the plasticity index as given by Eq. (3.9).

$$I_L = \frac{w_n - w_P}{w_L - w_P} = \frac{w_n - w_P}{I_P}$$

The liquidity index of a soil indicates the nearness of its water content to its liquid limit. When the soil is at its liquid limit, its liquidity index is 100% and it behaves as a liquid. When the soil is at the plastic limit, its liquidity index is zero. Negative values of the liquidity index indicate water content smaller than the plastic limit. The soil is then in a hard (desiccated) state.

Consistency Index: Consistency index (I_C, CI) or relative consistency is defined as the ratio of the difference between the liquid limit and the natural water content to the plasticity index of soil.

$$I_C = \frac{w_L - w_n}{w_L - w_P} = \frac{w_L - w_n}{I_P}$$

Where, w_n = water contents of the soil in natural condition. The consistency index indicates the consistency (firmness) of a soil. It shows the nearness of the water content of the soil to its plastic limit. A soil with a consistency index of zero is at the liquid limit. It is extremely soft and has negligible shear strength. On the other hand a soil at a water content equal to the plastic limit and has a consistency index of 100%, indicating that the soil is relatively firm. A consistency index of greater than 100% shows that the soil is relatively strong, as it is in the semi-solid state. A negative value of consistency index is also possible, which indicates that the water content is greater than the liquid limit. The consistency index is also known as relative consistency. It is worth noting that the sum total of the liquidity index and the consistency index is always equal to 100%, indicating that a soil having a high value of liquidity index has a low value of consistency index and vice-versa.

Flow Index: Flow index (I_f) is the slope of the flow curve (a straight line) obtained between the number of blows and the water content in Casagrande's method of determination of the liquid limit.

$$I_F = \frac{w_l - w_2}{\log \frac{n_2}{n_l}}$$

The flow index is the rate at which a soil mass looses its shear strength with an increase in water content. Fig. 3.28 shows the flow curves of two soils (1) and (2). The soil (2) with a greater value of flow index indeed has a steeper slope and possesses lower shear strength as compared to soil (1) with a flatter slope. In order to decrease the water content by the same amount, the soil with a steeper slope takes a smaller number of blows, and, therefore, has lower shear strength.

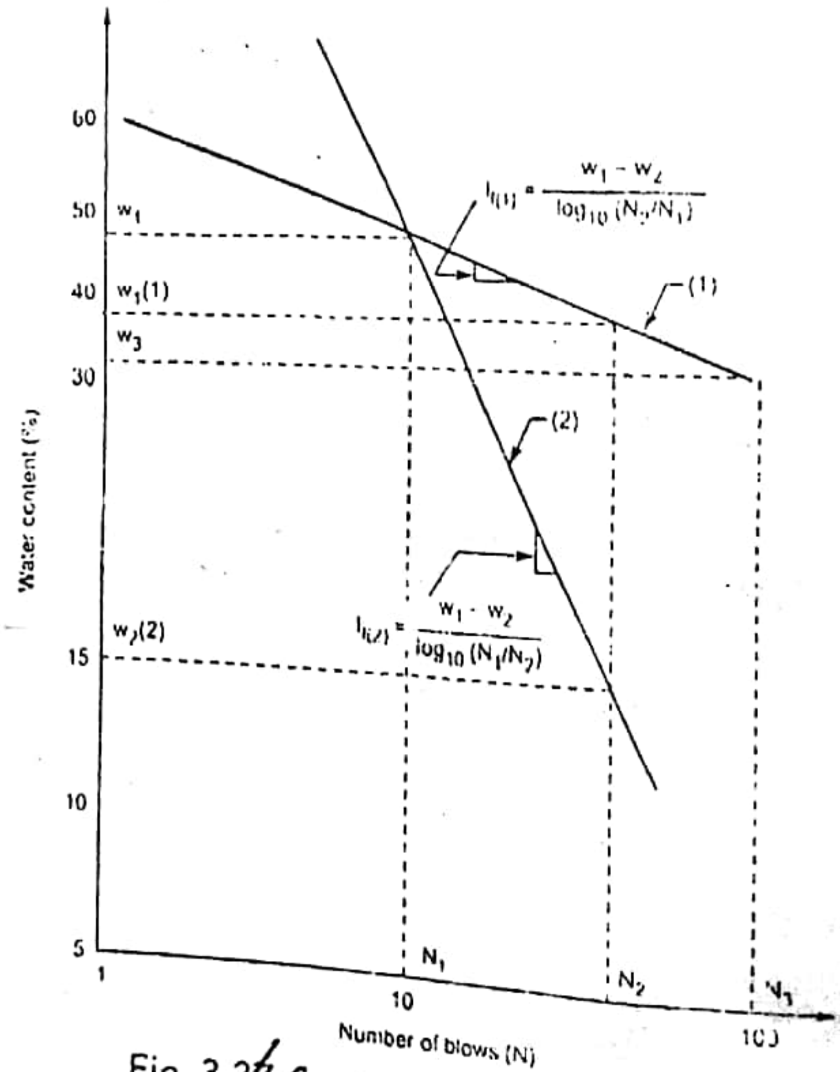


Fig. 3.27 Comparison of Flow Index

Toughness Index: Toughness index of a soil is defined as the of the plasticity index (I_P) to the flow index (I_F).

$$I_T = \frac{I_P}{I_F} = \frac{w_L - w_P}{\frac{w_2 - w_1}{\log \frac{n_1}{n_2}}} = \frac{(w_L - w_P)}{(w_2 - w_1)} \times \log \frac{n_1}{n_2}$$

The toughness index of a soil is a measure of shearing strength of the soil at the plastic limit. Let us assume that the flow curve is a straight line between the liquid limit and the plastic limit. As the shearing resistance of the soil is directly proportional to number of blows in Cassgrande's device

$$kS_L = n_L \quad and, \quad kS_P = n_P$$

Where,

n_L = number of blows at the liquid limit when the shear strength is S_L

n_P = number of blows at the plastic limit when the shear strength is S_P

If $w_2 = w_L$, $n_2 = n_L$ and $n_1 = 1$, then

$$I_T = \frac{I_P}{I_F} = \frac{w_L - w_P}{\frac{w_2 - w_1}{\log \frac{n_1}{n_2}}} = \frac{(w_L - w_P)}{(w_2 - w_1)} \times \log \frac{n_1}{n_2}$$

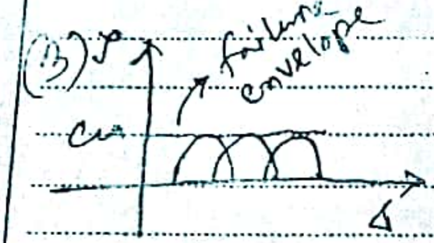
Factors affecting shear strength of soil.

(1) Resistance due to interlocking of the particles → cohesion
(2) Frictional resistance betwn the individual soil grains which may be either of the sliding or rolling frictions or both.
(3) Adhesion betwn soil particles called cohesion.

UU

(1) Sample is unconsolidate & drainge is not allowed.

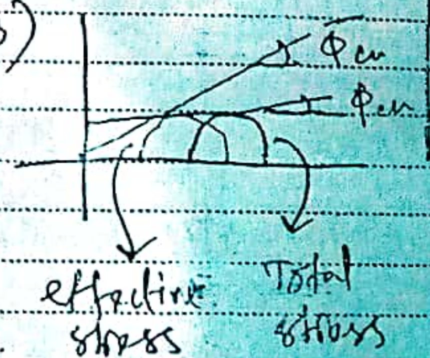
(2) $\tau_f = c_u$

(3) 

CU

(1) Sample is consolidated & drainge is not allowed.

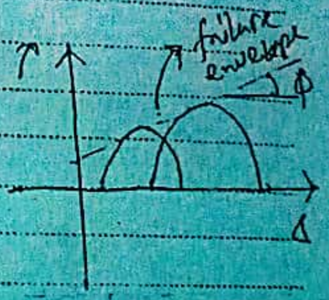
(2) $\tau_f = c_{cu} + \sigma \tan\phi_{cu}$

(3) 

CD

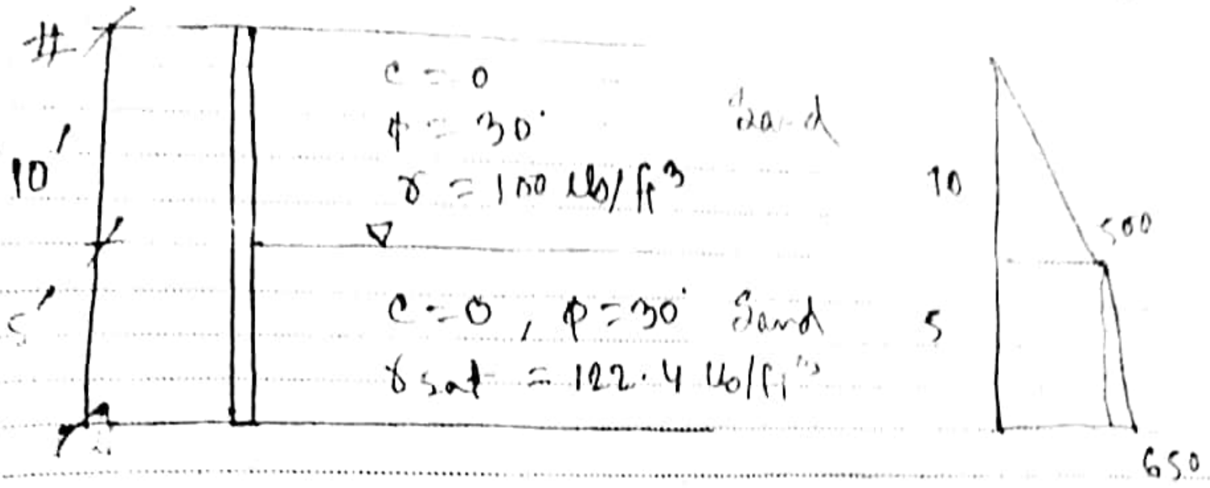
(1) Sample is consolidated & drainge is allowed.

(2) $\tau_f = c' + \sigma' \tan\phi'$

(3) 

Shear Strength of soil:

The sliding capability of soil, while to support loading from structure, or to support its own overburden pressure or to sustain a slope in equilibrium, is called shear strength of soil.



Layer - 1

$K_0 = (1-\sin\phi) = 1-\sin 30° = \frac{1}{2}$

$P_0(h) = K_0 \gamma H = \frac{1}{2} \times 100 \times h = 50h$

$\therefore P_0(10) = 50 \times 10 = 500 \text{ lb/ft}^2$

Layer - 2

surcharge

$q = \gamma H = 100 \times 10 = 1000$

$H_e = q/\gamma_{e2} = 1000/(122.4 - 62.4) = 16.67$

$\therefore P_0(h) = K_0 \gamma (H + H_e) = \frac{1}{2} \times (122.4 - 62.4) \times (h + 16.67)$

$= 30h + 500$

$\therefore P_0(5) = 30 \times 5 + 500 = 650 \text{ lb/ft}^2$

$\therefore$ E.P @ rest $= \frac{1}{2} \times 500 \times 10 + 500 \times 5 + \frac{1}{2} \times (650 - 500) \times 5$

$= 5375 \text{ lb/ft}^2$

$\therefore$ W.P $= \gamma_w h = 62.4 \times 5 = 312 \text{ lb/ft}^2$

$\therefore$ Total thrust $= 5375 + 312 \times 5 = 6155 \text{ lb/ft}^2$

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(4)

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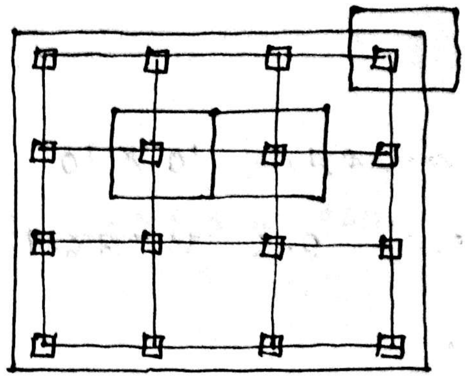
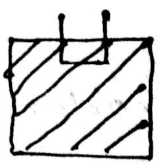
Raft Foundation :

* Raft is used :

- When the footings are too large
- When the columns are on boundary line
- To create space in basement



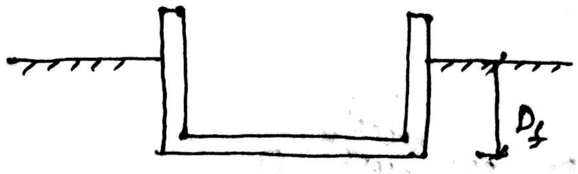


Article 18.3

* $F = \dfrac{cN_c}{q_b - \gamma D_f}$ eq^n 18.5

- F = Factor of safety
- q_b = gross contact pressure = $\dfrac{Q}{A}$
  - Q = Total wt. of the structure
  - A = Area of the raft

* When $q_b = \gamma D_f$, $F = \infty$ ⇒ Fully Compensated foundation / Floating foundation



* Vol^m of earth excavated $A \times D_f$

* Wt. of " " $\gamma A D_f$

$$q_b = \frac{Q}{A}$$

$$\Rightarrow Q = q_b A$$

$$\Rightarrow \underbrace{Q}_{\text{wt. of building}} = \underbrace{A\gamma D_f}_{\text{wt. of earth excavated}}$$

* যেখানে মাটি খারাপ, সেখানে fully compensated foundation করা হয়। (Pg. 277, 2nd Para)

- Whole structure is floating in soft soil.

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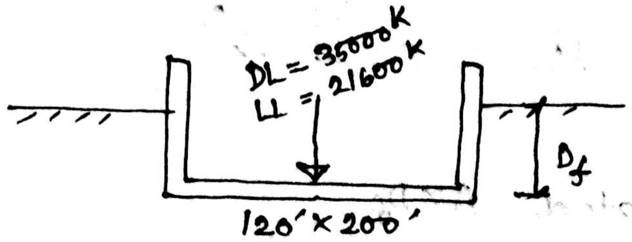
Problem (pg 278)



$\gamma = 115$ pcf

$q_u = 0.3$ tsf

(a) Find D_f for fully compensated foundation.

(b) Find D_f for $F = 3$

Soln:

$\boxed{(a)}$ For full compensation,

$$q_b = \gamma D_f$$

$$\Rightarrow \frac{Q}{A} = \gamma D_f = q_b = 2360 \text{ psf}$$

$$\Rightarrow \frac{35000 + 21600}{120 \times 200} = \frac{115}{1000} D_f$$

$$\therefore \boxed{D_f = 20.5'}$$

$\boxed{(b)}$

$$F = \frac{cN_c}{q_b - \gamma D_f}$$

$$\Rightarrow q_b - \gamma D_f = \frac{cN_c}{F}$$

Fig 18.3 → $cN_c = q_a \times 3$

assume, $\frac{D_f}{B} = 0.1$

$$2360 - 115 D_f = \frac{3 q_a}{3} \rightarrow \quad q_u = 0.3 \text{ tsf}$$

$$q_a = 0.26$$

for rectangular footing, $q_a = .26\left(1 + .2\frac{B}{L}\right)$

$$= .26\left(1 + .2 \times \frac{120}{200}\right)$$

$$= 0.29 \text{ tsf}$$

$$2360 - 115 D_f = 0.29 \text{ tsf} = 580 \text{ psf}$$

$$\Rightarrow \boxed{D_f = 15.5'}$$

$\boxed{(c)}$ If (i) q_b increases by 25%

(ii) q_b " " 50%

$D_f = 15.5'$

$F = ?$

(i) $q_b = 2360$

25%

$$q_b = 2360 \times 1.25$$

$$= 2950$$

$$2950 - 115 \times 15.5 = \frac{580 \times 3}{F}$$

$$\therefore \boxed{F = 1.49}$$

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(ii) $q_b = 2360$

50% increase

$= 2360 \times 1.5$

$= 3540$

$$3540 - 115 \times 15.5 = \frac{580 \times 3}{F}$$

$\therefore \boxed{F = 0.99}$

* F is not linear.

Assignment

* Pg 302 & 303 (Math 1 to 5)

(d) $F = 1,\ D_f = ?$

$$2360 - 115 D_f = \frac{580 \times 3}{1}$$

$\boxed{D_f = 5.4'}$

(e) $q_u = .25$ tsf instead of 0.3 tsf

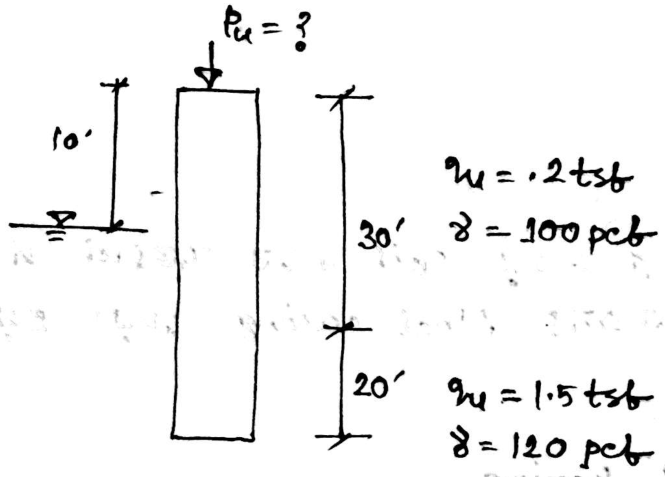
$D_f = 15.5$

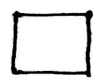
$$2360 - 115 \times 15.5 = \frac{(580 \times 3)(.25/.3)}{F}$$

$\therefore \boxed{F = 2.5}$

* F is primarily dependent on shear strength of soil.

Problem :





Pile 15"×15" (square)
length 50'

* Skin fraction = $\alpha_2 c$ (Pile) × [$A_{surface}$]

  = $\alpha_1 c$ (Pier) [$A_{surface}$]

- α = reduction factor

* End Bearing = $cN_c[A_{tip}]$

Sol^n :

0-30' :

$q_u = .2$ tsf

$\alpha_2 = 0.98$ (fig 18.7)

$c = \frac{q_u}{2} = 0.1$ tsf $= .2$ ksf

$$0-30' = .98 \times .2 \times \left[4 \times \frac{15''}{12} \times 30\right]$$

$$= 29.4^{k}$$

<u>30'-50'</u>:

$$q_u = 1.5\ tsf$$

$$\alpha_2 = .65$$

$$c = \frac{q_u}{2} = 1.5\ ksf$$

$$30'-50' = 0.65 \times 1.5 \times \left(4 \times \frac{15}{12} \times 20'\right)$$

$$= 97.5^{k}$$

$$\text{Total skin friction} = 29.4 + 97.5$$

$$= 127^{k}$$

$$\text{End bearing} = cN_c A_{tip}$$

[End bearing এ c 1.5 হবে as bearing হবে নিচে]

$$= 1.5 \times 9 \times \frac{15'' \times 15''}{144}$$

$$= 21^{k}$$

$$P_u = \text{skin friction} + \text{End bearing} = 127 + 21 = \boxed{148^{k}}$$

Load carrying mechanism অনুসারে pile

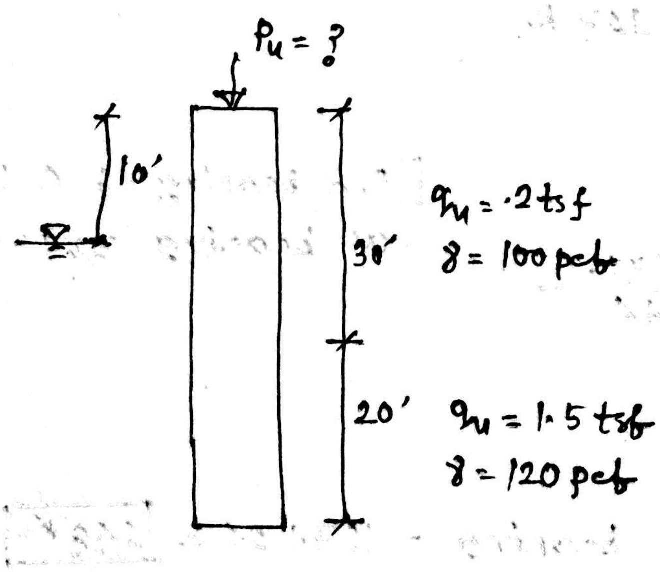
2 types :

(1) Friction pile (Skin friction > end bearing)

(2) End bearing " (end bearing > Skin friction)

* নিচে rock থাকলে end bearing pile হয়, clay soil এ থাকলে friction pile হয়।

Problem :



Drilled Pier

D = 24"

length = 50 ft

Sol^n :

skin fraction = $\alpha_1 \, c \, A_{surface}$

$\alpha_1 \rightarrow (.45)$

$0-30' = .45 \times .2 \, [\pi 2 \times 30]$

$= 17^k$

$30'-50' = .45 \times 1.5 \, [\pi \times 2 \times 20]$

$= 85^k$

Total skin friction $= 17 + 85 = 102^k$

End bearing $= 1.5 \times 9 \times \frac{\pi D^2}{4}$

$= 44^k$

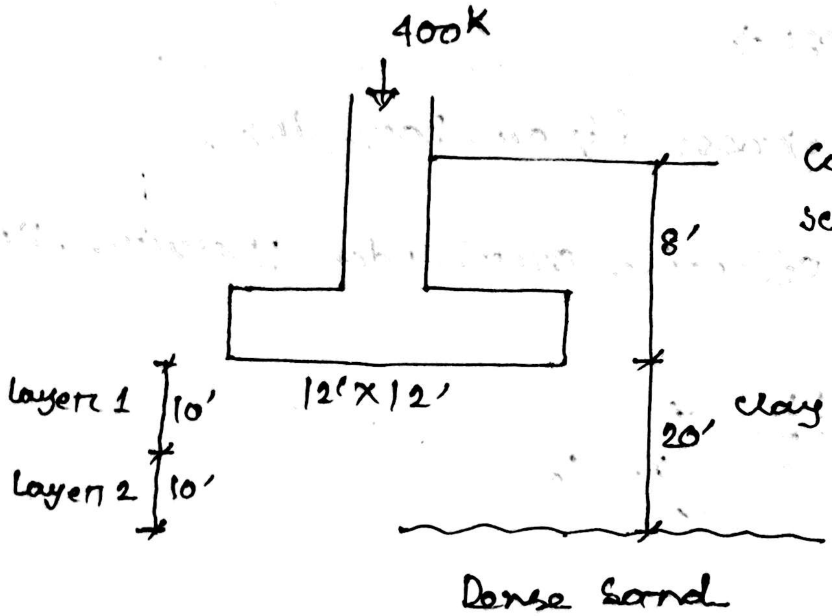
$\therefore P_u = 102 + 44 = \boxed{146^k}$

α_1 এর মান α_2 এর চেয়ে কম কেন? (Pg 279)

$\Rightarrow$ α_1 is due to oversized hole

AASHTO $\rightarrow$ $\alpha_1 = 0.55$

Settlement of footing on clay : (Problem)



Calculate the settlement of footing.

$\gamma = 100$ pcf
$C_c = 0.15$
$e_0 = 1.1$

[* C_c and C_r দেয়া না থাকলে, NC consider করব।]

Soln :

- Divide the 20 ft clay layer into 2 layer.
- Calculate the settlement for layer 1

$$\therefore S_1 = \Delta H_1 = \frac{C_c H}{1+e_0} \log\left(\frac{\Delta p + p_0}{p_0}\right) \quad [NC]$$

$$= \frac{15 \times 120}{1+1.1} \log\left(\frac{1380+1300}{1300}\right) = 2.7''$$

Layer 1 :

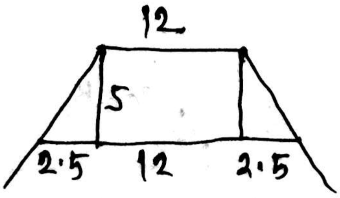
$C_c = 1.5$

$H = 10' = 120''$

$e_0 = 1.1$

* P_0 = effective overburden pressure at middepth of layer I = $(8+5)\,100 = 1300$ psf

* Δp = increase in effective stress due to foundation load at midspan depth of layer 1 using 2:1 method = 1380 psf



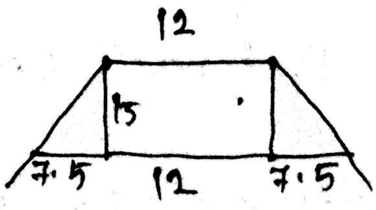
$$\Delta p = \frac{400}{(12+5)(12+5)} = 1380 \text{ psf}$$

Layer II :

$C_c = 1.5$

$H = 10'' = 120'$

$e_0 = 1.1$



$P_0 = (8+15)100 = 2300$ psf

$\Delta p = 548$ psf

$$\Delta p = \frac{400}{(12+15)(12+15)} = 548 \text{ psf}$$

$$\therefore S_2 = \Delta H_2 = \frac{C_c H}{1+e_0} \log\left(\frac{\Delta p + p_0}{p_0}\right)$$

$$= \frac{.15 \times 120}{1+1.1} \log\left(\frac{548+2300}{2300}\right)$$

$$= .8''$$

$\therefore$ Total settlement $= \Delta H_1 + \Delta H_2 = 2.7 + .8 = \boxed{3.5''}$

Allowable settlement for footing on clay for RCC building is 2".

$\therefore$ The settlement is more than the allowable limit.

Problem :

OC (Over consolidated)

$C_r = 0.04$

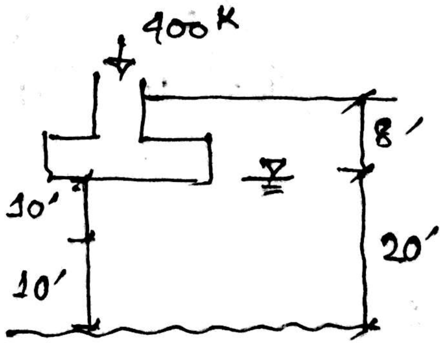
$\gamma = 120$ pcf

$e_0 = 0.9$

$P_0' = q_{vmax} = 6000$ psf

↓

Past max^m overburden pressure



Calculate the settlement of footing.

Solⁿ :

$P_o = 8\times120 + 5\times(120-62.5) = 1247$ psf

$\Delta p = 1380$ psf

} Layer 1

$\Delta p + P_o < 6000$

$\therefore S_1 = \Delta H_1 = \frac{C_r H}{1+e_o}\log\left(\frac{\Delta p + P_o}{P_o}\right)$ [OC]

$= \frac{.04\times120}{1+.9}\log\left(\frac{1380+1247}{1247}\right)$

$= .82''$

$P_o = \underbrace{1822}_{(8\times120)+15(120-62.5)}$ psf

$\Delta p = 548$ psf

} Layer 2

$\therefore S_2 = \Delta H_2 = \frac{.04\times120}{1+.9}\log\left(\frac{548+1822}{1822}\right) = .29''$

$\therefore \Delta H_1 + \Delta H_2 = .82 + .29 = \boxed{1.11''}$

$\therefore$ Allowable settlement 2″
Calculated " 1.11″

$\therefore$ Settlement is within allowable limit.

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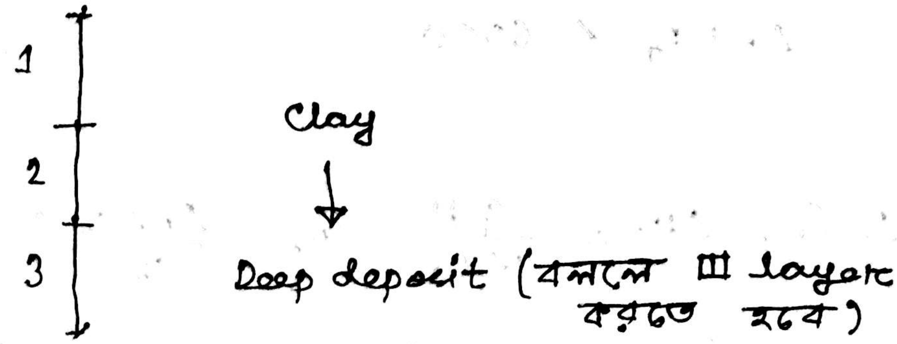
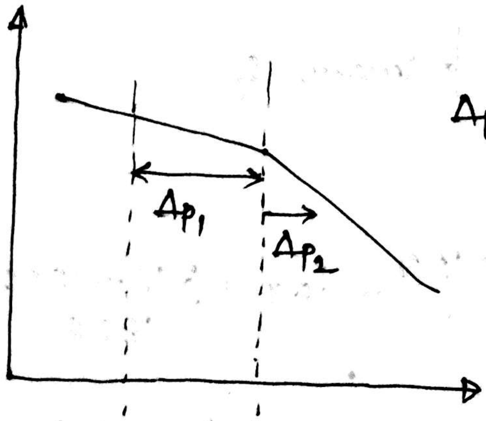
Problem :

* Dense sand এর জায়গায় layer III continue করলে (Deep deposit) settlement ·1" আসে, নিচের দিকে settlement কম আসলে stop করব।



When $\Delta p + P_0 > P_0'$,

$$S = \frac{C_r H}{1+e_0} \log\left(\frac{\Delta p + P_0'}{P_0}\right) + \frac{C_c H}{1+e_0} \log\left(\frac{\Delta p + P_0}{P_0}\right)$$



$\Delta p_1 + \Delta p_2 = \Delta p$

$$\therefore S = \frac{C_r H}{1+e_0} \log\left(\frac{\overset{P_0+\Delta p_1}{P_0'}}{P_0}\right) + \frac{C_c H}{1+e_0} \log\left(\frac{\overset{P_0'+\Delta p_2}{P_0'+\Delta p}}{P_0'}\right)$$

CE441

(10)

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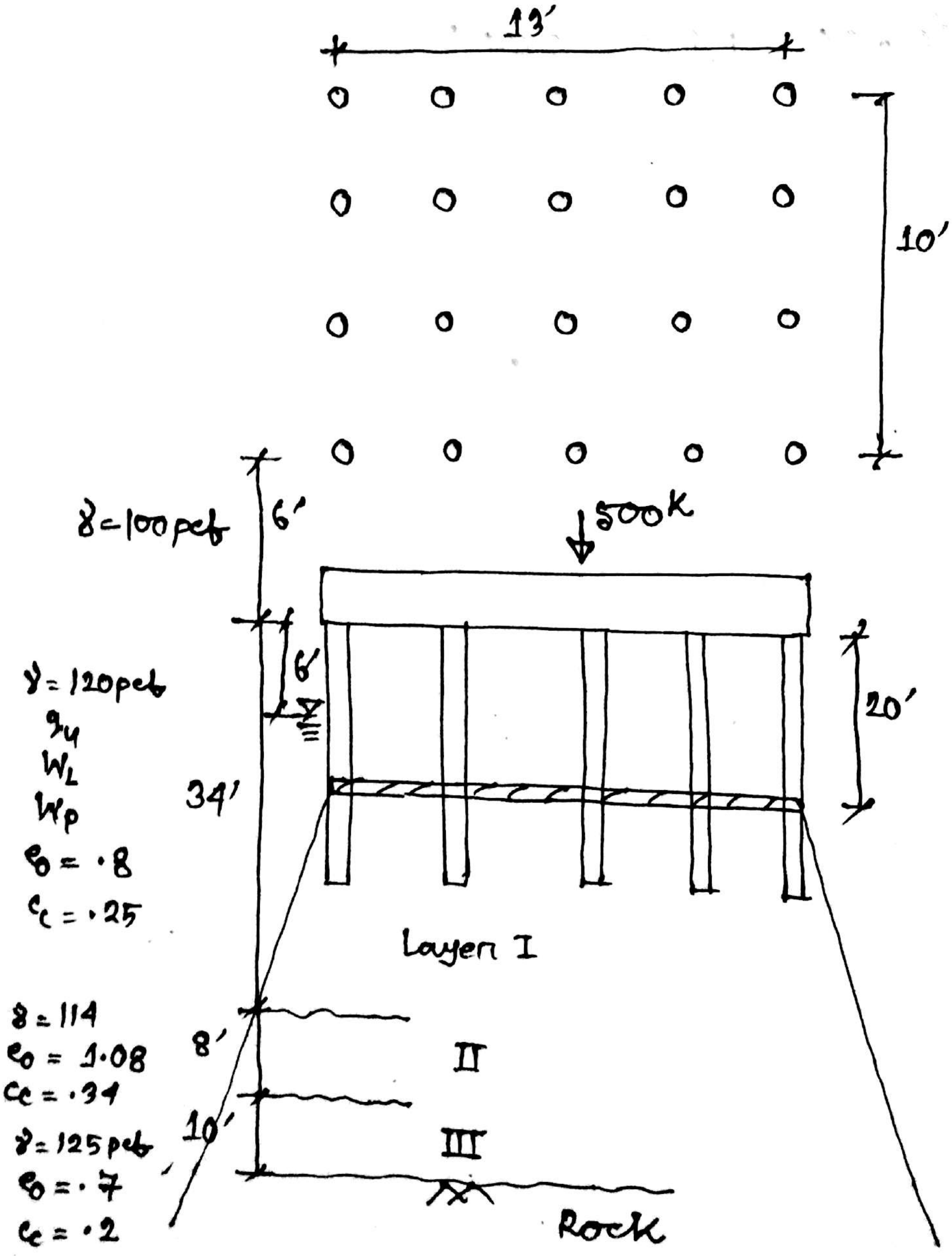
Pile foundation design:

Computation of settlement:



Pile group = 20 piles
Pile D = 12"
c/c spacing = 3 ft
L = 30'

* End bearing pile → Pile resting on dense sand or rock

  - Assume an equivalent at pile bottom

* Friction pile → Piles embedded in clay

  - Assume an equivalent footing at $2/3$ L

- Pile type → Friction or end bearing
- The pile is embedded in clay ∴ Friction pile
- Assume an equivalent at $\frac{2}{3}$ L, as shown in fig
- Equivalent footing size = Area of the group

  $= 10' \times 13'$

- Soil is N.C.

- Settlement due to layer 1: (Stress distribution 2:1)

$$S_1 = \frac{C_c H}{1+e_0} \log\left(\frac{\Delta p + p_0}{p_0}\right)$$

$e_0 = .8$

$H = 14'$

$C_c = .23$

$$\Delta p_1 = \frac{500 \times 10^3}{(13+7)(10+7)} = 1470 \text{ psf}$$

$$P_0 = 6 \times 100 + 6 \times 120 + (14+7) * (120 - 62.5)$$

$$= 2527 \text{ psf}$$

$$\therefore S_1 = \frac{.23 \times 14 \times 12}{1 + .8} \log \left(\frac{1470 + 2527}{2527}\right)$$

$$= 4.3''$$

• layer - II :

$$\Delta p_2 = \frac{500 \times 1000}{(13+18)(10+18)} = 580 \text{ psf}$$

$$P_0 = 2527 + 7'(120 - 62.5) + 4(114 - 62.5)$$

$$= 3136 \text{ psf}$$

$$\therefore S_2 = \frac{.34 \times 8 \times 12}{1 + 1.08} \log \left(\frac{580 + 3136}{3136}\right)$$

$$= 1.2''$$

• layer - III :

$$P_0 = 3136 + 4'(114 - 62.5) + 5'(125 - 62.5)$$

$$= 3657 \text{ psf}$$

$$\Delta p_3 = \frac{500 \times 1000}{(13+27)(10+27)}$$

$$= 340 \text{ psf}$$

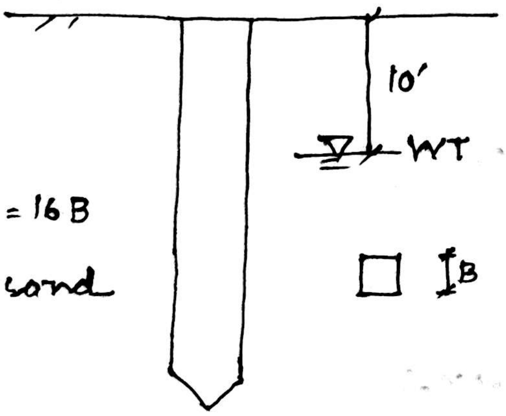
$$\therefore \; S_3 = \frac{.2 \times 10 \times 12}{1 + .7} \log\left(\frac{340 + 3657}{3657}\right)$$

$$= 0.5''$$

$$\therefore \; S = S_1 + S_2 + S_3 = \boxed{6''}$$

** Settlement বেশি হলে for pile foundation, we can go for longer piles and for footing we can increase size of footing or insert pile.

Pile foundation in sand: (Problem)



Length of pile = 40'
Size 15" x 15"

D_c = critical depth = 16 B

□ B

$\gamma = 120$ pcf

Medium to dense sand

$\delta = 25°$

$N_q = 60$

$K = 1.5$

Find the capacity of the pile.

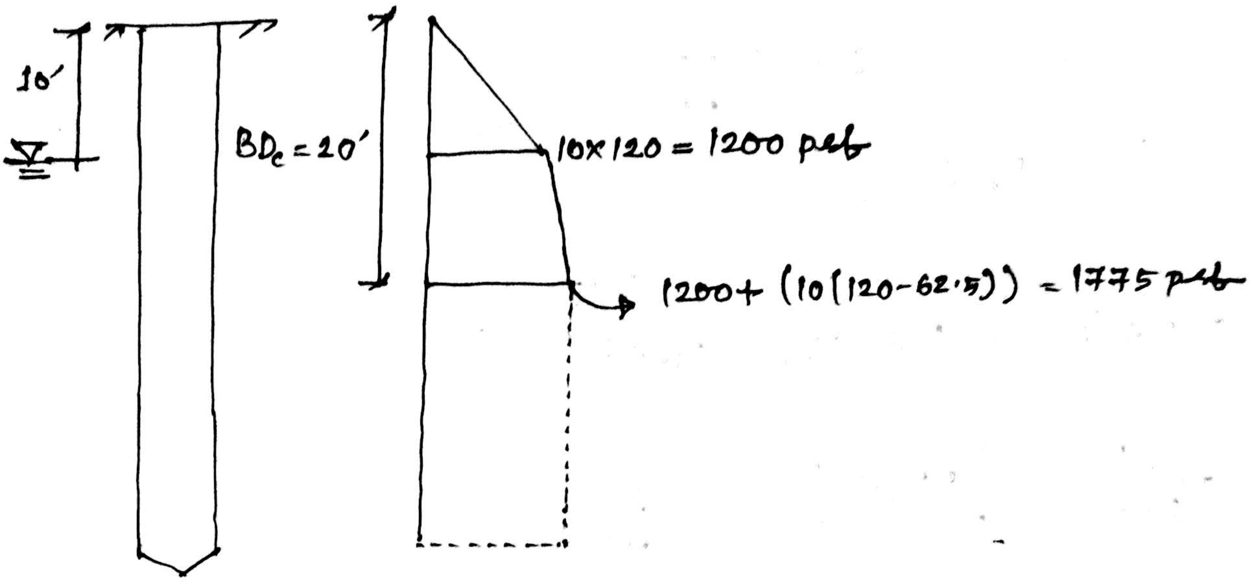
* Critical depth = $BD_c = \frac{15}{12} \times 16 = 20$ ft

Beyond this depth, σ_v remains constant

* $D_c = f(\phi)$ (10 ~ 20)

• N এর মান দেয়া থাকলে, δ, N_q, K বের করা যাবে।

sol^n :



• **Skin friction :**

$$0 \sim 10 = k \tan\delta \, \sigma_v \, A_{surface}$$

$$= 1.5 \times \tan 25^\circ \times \left[\frac{0+1200}{2}\right] \times \left[\frac{15 \times 4 \times 10}{12}\right]$$

$$= 1.5 \times .47 \times 600 \times 50$$

$$= 21^K$$

$$10 \sim 20' = 1.5 \times 0.47 \times \left(\frac{1200+1775}{2}\right) \times 50$$

$$= 52^K$$

$$20 \sim 40' = 1.5 \times 0.47 \times (1775) \times 5 \times 20$$

$$= 125^K$$

$\therefore$ Total friction $= 198^K$

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• End bearing $= q N_q A_{tip}$

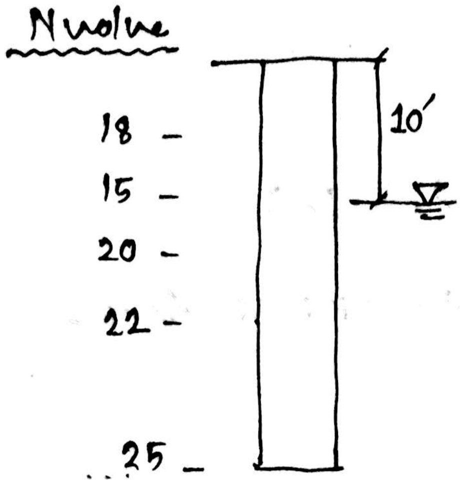
$$= 1775 \times 60 \times \frac{15 \times 15}{144}$$

$$= 166^K$$

Total capacity $= 198 + 166 = 364^K$

$$P_a = \frac{364}{2.5} = \boxed{146^K}$$

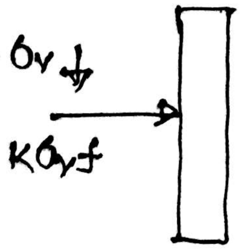
Problem : (Drilled Pier)



Drilled Pier
length = 40'

$D = 24''$

Find the capacity.



- $K = K_o \sim K_a$

* $f = \tan\delta$

depends on surface (smooth হলে কম
Rough " বেশি)

- $\delta = \frac{2}{3}\phi$ (concrete)
- $\tan\delta = 0.1$ (Steel)

* বড় pile হলে K_p বেশি হবে।

* For bored pile, $\delta \approx \phi$

• AASHTO formula use করবে।

* Skin friction

$\hookrightarrow$ $k \tan\delta \cdot \sigma_v \, [A_{surface}]$

* $\beta = 1.5 - 0.135\sqrt{Z}$ (include $\tan\delta$, K, σ_v)

(does not depend on N as excavate করলে dense sand loose হয়ে যায়)

* End bearing $= 1.2 \, N \, A_{tip}$

Soln :

• Skin friction :

$0 \sim 10' = \sigma_v \, \beta \, [A_{surface}]$

$\beta = 1.5 - .135\sqrt{5}$

$= 1.2$

$= [5 \times 120] \, 1.2 \, [2\pi \times 10]$

$= 45^k$

$10 \sim 20' = [10 \times 120 + 5(120 - 62.5)] \, (2\pi \times 10) \, (1.5 - .135\sqrt{15})$

$= 92^k$

$20 \sim 30' = [10 \times 120 + 15 \, (120 - 62.5)] \times (1.5 - .135\sqrt{25}) \times (2\pi \times 10)$

$= 106^k$

$$30-40' = [10\times120+25(120-62.5)]\times(1.5-.135\sqrt{35})\times 2\pi 10$$

$$= 116^K$$

$\therefore$ Total Skin friction $= 359^K$

- End bearing $= 1.2N\,(A_{tip})$

$$= 1.2\times 25\times\left(\pi\times\frac{2^2}{4}\right)$$

$$= 94^K$$

$$P_u = 359+94 = 453^K$$

$$P_a = \frac{453}{2.5} = \boxed{180^K}$$

Soil ***Investigation****!!* *Sub-soil* ***Exploration****!!*

Site Exploration and Characterization

The process of exploring to characterize or define small scale properties of substrata at construction sites is unique to geotechnical engineering. In other engineering disciplines, material properties are specified during design, or before construction or manufacture, and then controlled to meet the specification. Unfortunately, subsurface properties cannot be specified; they must be deduced through exploration.

Charles H. Dowding (1979)

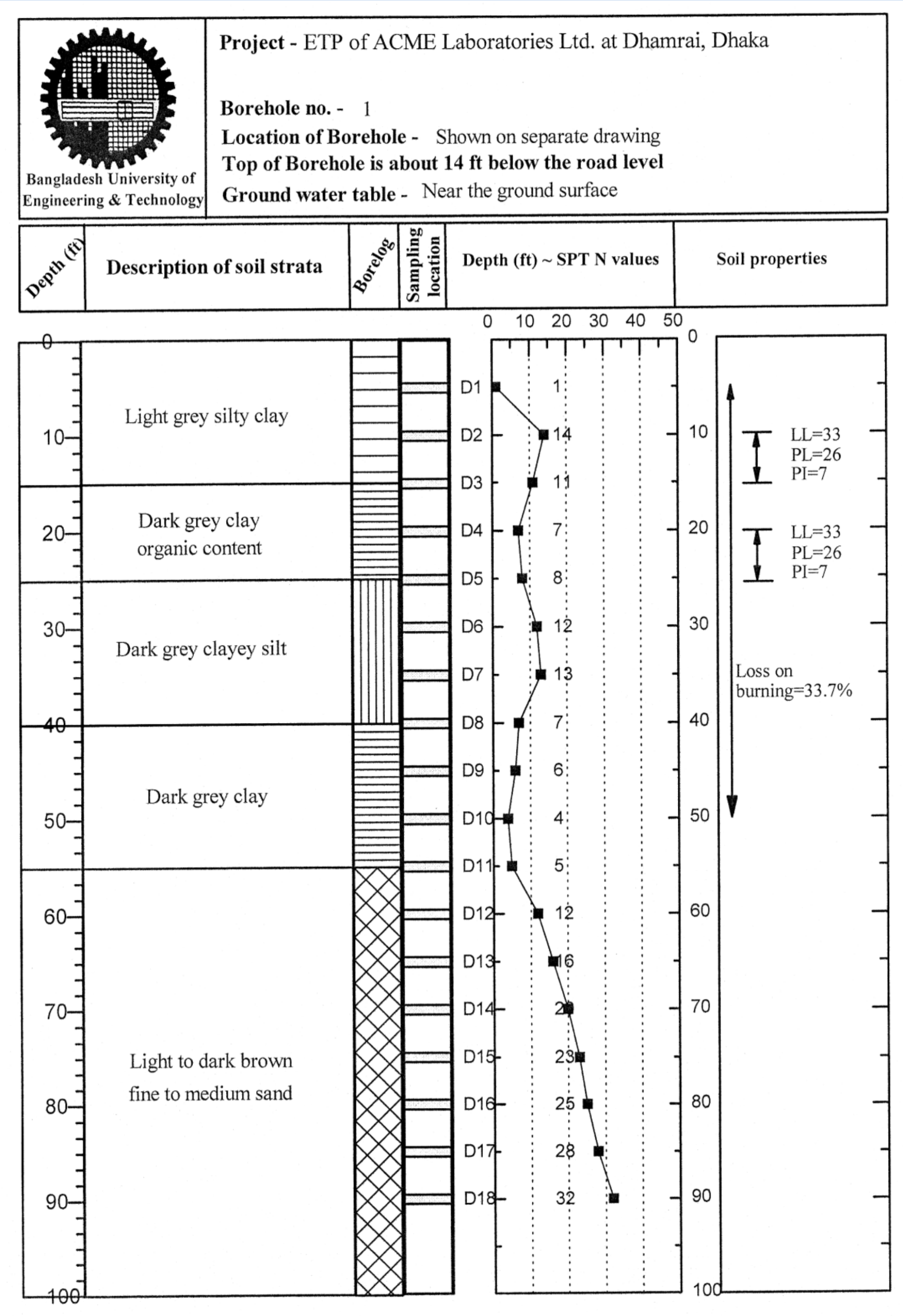
Coduto p.46

Bore-log

The detailed information gathered from each borehole is presented in a graphical form termed as 'boring log' or 'bore-log'

A bore-log usually contains:

1. Name and address of drilling company
2. Project name & Location of site
3. Date of boring
4. Bore-hole number and type of boring
5. Sub-surface stratification
6. RL of GL and elevation of water table
7. SPT-N value
8. Number, type and depth of soil sample collected



Soil sampling

Two types of samples may be obtained

Disturbed samples

Undisturbed samples

A disturbed sample is one obtained with no attempt to retain the in-place structure, however there is no change in the constituents/composition.

In a strict sense no sample is perfectly undisturbed, what we mean is that the disturbance is small

The samples must be representative (disturbed or undisturbed)

Both disturbed and undisturbed samples are collected as the bore-hole proceeds

The kind of samples that should be obtained from an exploratory drill hole depend on the purpose for which the exploration is made.

Disturbed samples can be used for
Visual observation and classification
Grain size analysis
Sp.Gr. determination
Atterberg limits determination
Organic content determination
Chemical tests (pH, chloride, Sulfate etc.)

Undisturbed samples are required for
Consolidation test
Shear strength (Direct shear, Unconfined compression, Triaxial test)
Permeability, Hydraulic conductivity

For proper identification and classification, representative samples are required. A representative sample is one that contain all the constituents in their proportions. Such samples are adequate for visual classification, mechanical analyses, determination of Atterberg limits, Unit weight/Sp.Gr of solid constituents, chemical/organic content etc.

Mechanical properties of the disturbed soil samples are significantly altered by the sampling process. For determination of stress~strain characteristics or density of the soil strata we need samples that have undergone negligible deformation during sampling. Such samples are called undisturbed, although a certain amount of disturbance is regarded as inevitable.

SPT and disturbed sampling

When a borehole is extended to a predetermined depth, the drill tools are removed and the sampler is lowered to the bottom of the borehole.
The sampler is driven into the soil by hammer blows to the top of the drill rod.
The standard weight of the hammer is 140 lb (622.7 N) and for each blow the hammer drops a distance of 30 in (0.762 m). The number of blows for three 6” (152.4 mm) intervals of the spoon are recorded. The number of blows required for the last two intervals are added and is termed as the N-value (or SPT-N or Standard Penetration Number) at that depth.
The sampler is withdrawn, the shoe and coupling are removed and the soil sample recovered from the tube is collected (disturbed sample).

Field Tests / In-situ Tests

Standard Penetration Test
Vane Shear Test
Cone Penetrometer Test
Pressuremeter Test
Dilatometer Test
Dynamic Penetration Tests (DPL, DPM, DPH, DPSH)

Standard Penetration Test (SPT)

Advantages

Simple equipments
Low cost
Do not require much expertise for field operations
Disturbed samples are collected

Disadvantages

Much variation in the results i.e. poor repeatability

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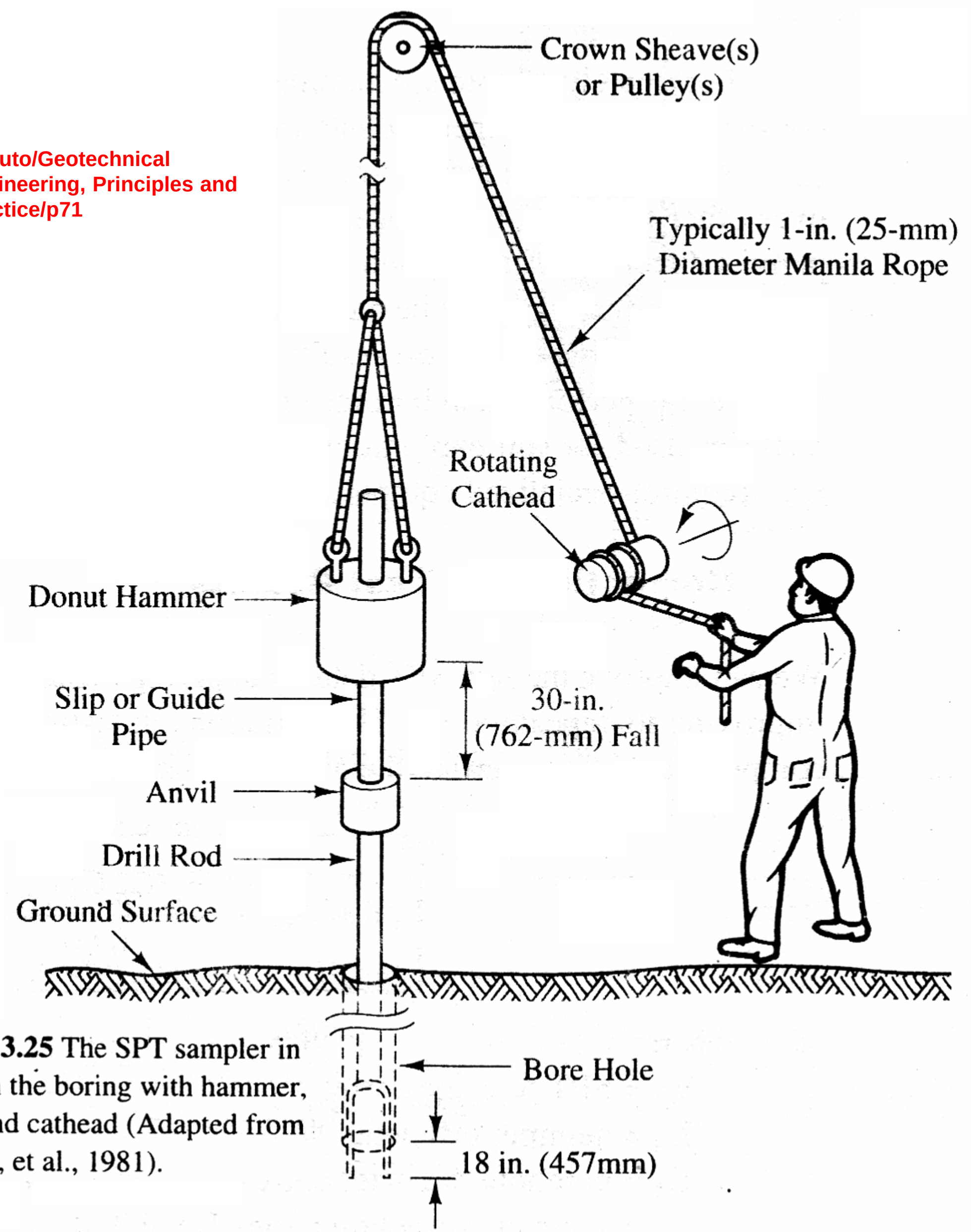


Figure 3.25 The SPT sampler in place in the boring with hammer, rope, and cathead (Adapted from Kovacs, et al., 1981).

2.15 CONE PENETRATION TEST

The cone penetration test (CPT), originally known as the Dutch cone penetration test, is a versatile sounding method that can be used to determine the materials in a soil profile and estimate their engineering properties. This test is also called the *static penetration test*, and no boreholes are necessary to perform it. In the original version, a 60° cone with a base area of 10 cm^2 was pushed into the ground at a steady rate of about 20 mm/sec, and the resistance to penetration (called the point resistance) was measured.

The cone penetrometers in use at present measure (a) the *cone resistance* (q_c) to penetration developed by the cone, which is equal to the vertical force applied to the cone divided by its horizontally projected area, and (b) the *frictional resistance* (f_c), which is the resistance measured by a sleeve located above the cone with the local soil surrounding it. The frictional resistance is equal to the vertical force applied to the sleeve divided by its surface area — actually, the sum of friction and adhesion.

Photograph of a cone penetrometer

Advantages of CPT

- CPT is a useful means to determine the soil profile. Since it retrieves data continuously with depth (with electric cone) or at very close interval (with mechanical cone), CPT can detect thin layers in stratigraphy. Sometimes use of CPT in the first phase facilitate better specification for boring and sampling in the second phase.
- It is also less prone to error due to automated operation of the equipment and electronic data recording.

Disadvantages of CPT

- No soil sample is recovered. So no opportunity to inspect the soils.
- The test is unreliable or unusable in soils with significant gravel content.
- Although the cost per foot of penetration is less than that for borings, it is necessary to mobilize a special rig to perform the CPT. CPT at a certain site may not be possible from the point of equipment mobilization.

Vane Shear Test

The *vane shear test* (ASTM D-2573) may be used during the drilling operation to determine the *in situ* undrained shear strength (c_u) of clay soils — particularly soft clays. The vane shear apparatus consists of four blades on the end of a rod, as shown in Figure 2.24. The height, H, of the vane is twice the diameter, D. The vane can be either rectangular or tapered (see Figure 2.24). The dimensions of vanes used in the field are given in Table 2.6. The vanes of the apparatus are pushed into the soil at the bottom of a borehole without disturbing the soil appreciably. Torque is applied at the top of the rod to rotate the vanes at a standard rate of 0.1°/sec. This rotation will induce failure in a soil of cylindrical shape surrounding the vanes. The maximum torque, T, applied to cause failure is measured. Note that

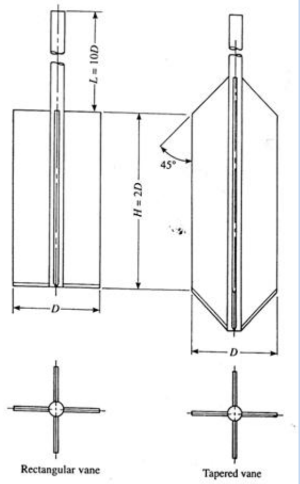
$$T = f(c_u, H, \text{and } D) \qquad (2.13)$$

or

$$c_u = \frac{T}{K} \qquad (2.14)$$

where T is in N·m, and c_u is in kN/m²

K = a constant with a magnitude depending on the dimension and shape of the vane



▼ FIGURE 2.24 Geometry of field vane (after ASTM, 1992)

Field Vane

Hand Vane

Advantages and Limitations of Field Vane Shear test

Field vane shear tests are moderately rapid and economical and are used extensively in field soil-exploration programs. The test gives good results in soft and medium-stiff clays, and it is also an excellent test to determine the properties of sensitive clays.

Sources of significant error in the field vane shear test are poor calibration of torque measurement and damaged vanes. Other errors may be introduced if the rate of vane rotation is not properly controlled.

For actual design purposes, the undrained shear strength values obtained from field vane shear tests [$c_{u(VST)}$] are too high and it is recommended that they be corrected, or

$$c_{u(corrected)} = \lambda c_{u(VST)} \tag{2.19}$$

where λ = correction factor

Several correlations have been previously given for the correction factor, λ, and some are given in Table 2.7.

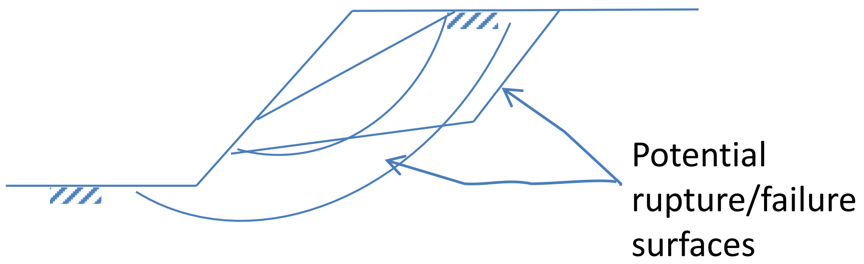
Undrained shear strength, pre-consolidation pressure and OCR can be determined using appropriate correlations.

Slope

An exposed ground surface that stands at an angle with the horizontal is called an *unrestrained slope*. The slope can be natural or man-made. If the ground surface is not horizontal, a component of gravity will tend to move the soil downward. If the component of gravity is large enough, slope failure can occur.

Slope Stability Analysis

In many cases, civil engineers are expected to make calculations to check the safety of natural slopes, slopes of excavations, and compacted embankments. This check involves determining and comparing the shear stress developed along the most likely rupture surface with the shear strength of the soil. This process is called *slope stability analysis*. The most likely rupture surface is the critical plane that has the minimum factor of safety.



Factor of Safety

As engineer we need to ensure the safety of a slope through determining the factor of safety. The factor of safety is defined as

$$F_s = \frac{\tau_f}{\tau_d}$$

Where

Fs = Factor of safety with respect to strength
τ_f = Average shear strength of the soil
τ_d = average shear stress developed along the potential failure surface

The shear strength of a soil consists of two components, cohesion and friction, and may be written as

$$\tau_f = c + \sigma \tan \phi \tag{13.2}$$

where c = cohesion
ϕ = angle of friction
σ = normal stress on the potential failure surface

In a similar manner, we can write

$$\tau_d = c_d + \sigma \tan \phi_d \tag{13.3}$$

where c_d and ϕ_d are, respectively, the cohesion and the angle of friction that develop along the potential failure surface. Substituting Eqs. (13.2) and (13.3) into Eq. (13.1), we get

$$\boxed{F_s = \frac{c + \sigma \tan \phi}{c_d + \sigma \tan \phi_d}} \tag{13.4}$$

Now we can introduce some other aspects of the factor of safety — that is, the factor of safety with respect to cohesion, F_c, and the factor of safety with respect to friction, F_ϕ. They are defined as follows:

$$\boxed{F_c = \frac{c}{c_d}} \qquad (13.5) \qquad \text{and} \qquad \boxed{F_\phi = \frac{\tan \phi}{\tan \phi_d}} \qquad (13.6)$$

When we compare Eqs. (13.4) through (13.6), we can see that when F_c becomes equal to F_ϕ, it gives the factor of safety with respect to strength. Or, if

$$\frac{c}{c_d} = \frac{\tan \phi}{\tan \phi_d} \qquad \text{we can write} \qquad F_s = F_c = F_\phi \qquad (13.7)$$

When F_s is equal to 1, the slope is in a state of impending failure. Generally, a value of 1.5 for the factor of safety with respect to strength is acceptable for the design of a stable slope.