30.9.15
Sunday

CE - 341

Lecture - 1

Muqtadir Sir

(*) Phase relationship

(*) Relative density

(*) Compaction of soil

4) Fluid flow through porous media

5) Hydraulic Properties of soil - এটার Ques (Craig এর করে দেখো problems)

6) weight volume of soil

** 7) Consolation characteristics of soil - soil mechanics এর প্রাণ ☺
1.5 or 1.75 question

8) Stress distribution of soil

Mechanics:
Action of force on bodies, bodies maybe rigid or deformable. and the body maybe at rest or in motion.

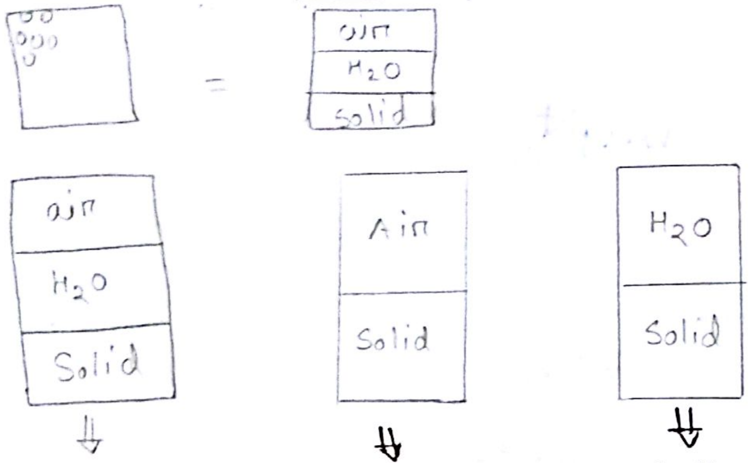
One of the oldest branches of physical science

Dynamic problem হতে হলে there has to be an inertia force. So there has to be an acceleration. If any body is at uniform motion then এটা dynamic problem না।

1.10.15

Tuesday

Lec - 2



3 phase diagram

dry soil sample

fully saturated

2 phase dia.

partially saturated

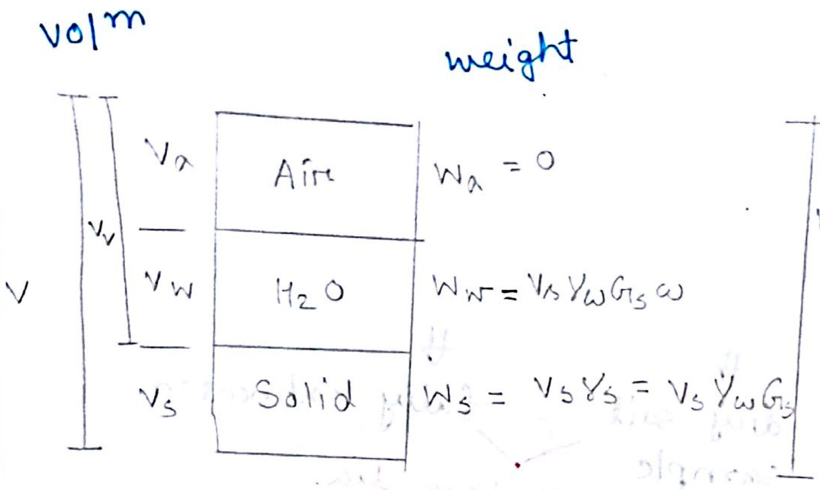
In context ~~of~~ of 3d partially saturated স্তর,

Two assumptions:

1. Solid sceleton or solid constituens are incompressible. Even H_2O is incompressible. incompressible হলে poisson ratio 0.5 & Bulk modulus ∝

2. The diagramic representation of real soil into its element soil sceleton, H_2O and air are called phase diagram or block diagram. It's extremely usful for studing terms like void ratio, porossity, ~~degrees~~ degrees of saturation, unit weights of various type etc & their interrelationship. The phase diagram provides a convinient means of developing

weight volume relationship of soil.



$$1\omega = \frac{W_w}{W_s} \times 100\%$$

* No vol^m reduction in water or air, only the voids decrease.

* Air + H_2O = volume of void

* Moisture content :

$$1\omega = \frac{\text{wt of water}}{\text{wt of solid}} \times 100\%$$ (percentage এ)

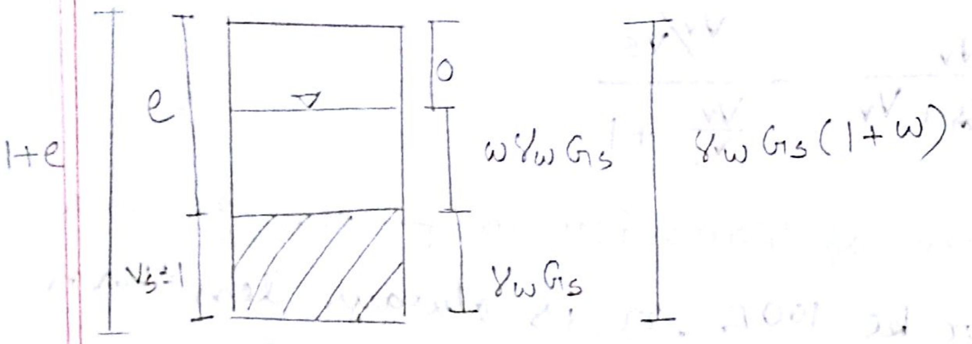
we want $W_w = W_s . w$

more than 100% for highly organic soil or gas

void ratio : e (expressed in decimel)

$\boxed{\frac{V_v}{V_{S=1}}}$, it can have a value of more than 1.

for calculating settlement of structure, permeability of soil এর দরকার। This parameter indecates compactness (loose / dense) of soil.



Dia-1

3. **Porosity** :/ percentage void

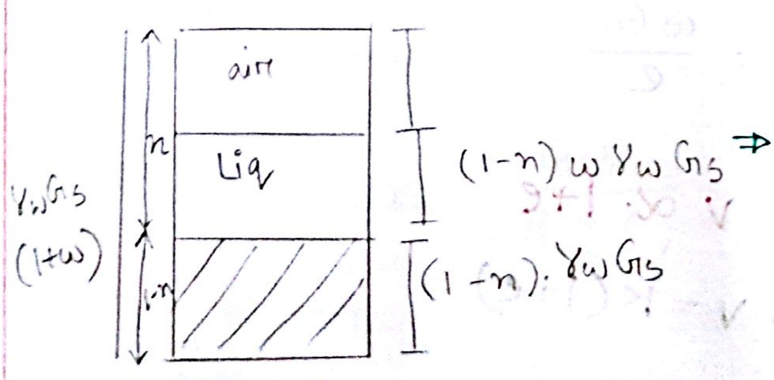
$n = \frac{V_v}{V}$, V = total volume

e only related to V_s, that's incompressible so V_s constant., so e কে ধরার variable., only V_v But v এ void আছে, so total volume V change হবে, So Here both V_v & V are variables. So two variables এ calculation করবি, so we use void ratio.

In structure for stability we need less variables.

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

$n = \frac{V_v}{V=1}$?



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

Dia-2

From basic ~~diagram~~ definition (not from block dia)

$$n = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{V_v/V_s}{\frac{V_v}{V_s} + 1}$$

The other name of porosity is percentage void. It can never be 100%. .n is always less than 100%.

4. Degrees of Saturation:

physically means to what extent void spaces are filled up with water and it's defined as

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

From void ratio diagram (Dia-1)

$$S_r = \frac{w G_s}{e} \qquad V_w = \frac{w \gamma_w G_s}{\gamma_w}$$

* Proof $S_r = \frac{w G_s}{e}$ from basic definition

From n diagram, (Dia-2)

$$S_r = \frac{(1-n) w G_s}{n} =$$

$$= \frac{w G_s}{\frac{n}{1-n}} = \frac{w G_s}{e}$$

Must remember: $V \propto 1+e$

$$\Rightarrow V = K(1+e)$$

* এখানে অব unit wt এ. Exam এ mass density দেয়া থাকে,
$\Downarrow$
(KN/m^3) (Kg/m^3 or ton/m^3)

⧉ determine the value of K and its physical meaning

5. Bulk Unit weight :

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

$$= \frac{(1+w)}{1+e} G_s \gamma_w \ \text{(Dia-1)}$$

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

Dia- ২ থেকে express করব

we know, $S_r = \frac{w G_s}{e}$ $\therefore w G_s = e S_r$

$$\therefore \gamma_{bulk} = \frac{(G_s + w G_s)}{1+e} \gamma_w$$

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

$$\frac{W_w + W_s}{V}$$

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

$$= (1-n) G_s \gamma_w (1+w)$$

6. Dry Unit weight :

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

এখানে W_s → weight of solid without moisture content

$$\gamma_d = \left(\frac{1+\overset{\text{cause m.c }(e)=0}{0}}{1+e}\right) G_s \gamma_w$$

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

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

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

6.9.15

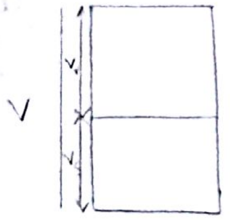
Sunday

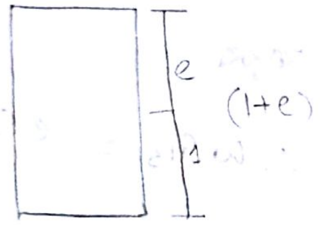
Lec - 3

Total volume $V \propto (1+e)$

$V = K(1+e)$

↳ value বের করতে হবে

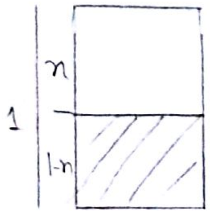




$$\frac{V_s}{V} = \frac{1}{1+e}$$

$$\Rightarrow V = \underbrace{V_s}_{\text{constant by assumption}}(1+e)$$

$V \propto (1+e)$

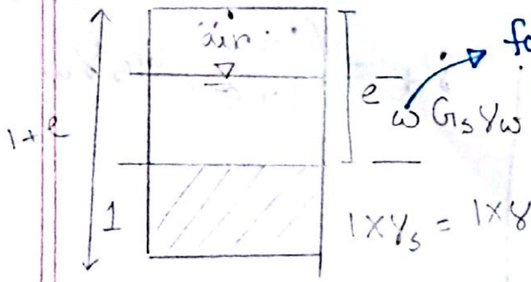


$$n = \frac{V_v}{V=1}$$

again, $\dfrac{V_v}{V_s} = \dfrac{n}{1-n}$

$$\Rightarrow V_v = \frac{V_s(n)}{1-n}$$

$$\therefore V_v \propto \frac{1}{1-n}$$



→ for partially saturated

$1 \times \gamma_s = 1 \times \gamma_w G_s = G_s \gamma_w$

$$\omega = \frac{S_r \times e}{G_s}$$

$$\boxed{\Rightarrow \omega G_s = S_r \times e = S_r e}$$

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

$$= \frac{G_s + G_s \omega}{1+e} \gamma_w = \frac{G_s + S_r e}{1+e} \gamma_w$$

$$\gamma_{bulk} = \gamma_d\cancel{ex} (1+\omega)$$

$\downarrow$ as dry → degrees of saturation $S_r = 0$

$$\gamma_d = \frac{\gamma_{bulk}}{1+w} \qquad \gamma_d = \frac{G_s \gamma_w}{1+e}$$

* wet থেকে dry এ গেলে assume no change in volume

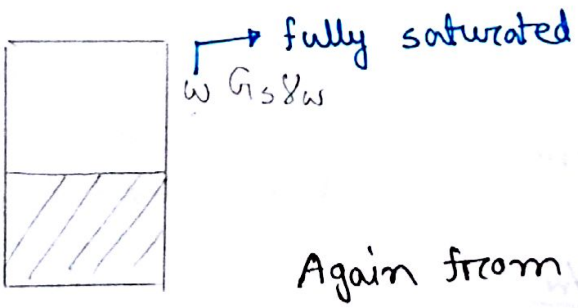
$$e = \frac{G_s \gamma_w}{\gamma_d} - 1$$

[এই expression থেকে void ratio in the field condition পাই,

Q. How to find void ratio in field?

Saturated Unit weight:

All void spaces are filled up with water



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

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

Again from γ_{bulk} $S_r = 1$ থেকে define করা যায়

max^m possible unit wt. a soil can have

Submerged Unit wt. of soil/effective unit wt:

Effective unit wt কমে যাবে cause পানিতে ডুবালে ঐ পরিমাণ পানি replaced হবে & ঐ পানি

∴ ওজনের সমান ওজন কমবে,

$$\gamma_{effective} = \gamma_{saturated} - \gamma_w$$

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

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

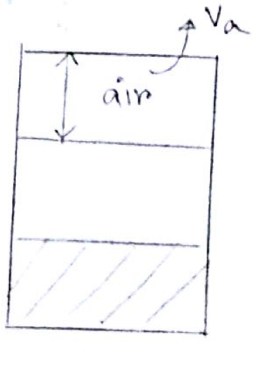
$$= \frac{\cancel{8}G_s - 1}{1+e}\gamma_w$$

Generally unit wt 120-125 gm/m^3

γ_{eff} generally almost half of normal value as unit weight of water is 62'4 gm/m^3

$$\gamma' = \frac{1}{2}\gamma_{eff}$$

Air Content, Av :



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

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

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

$$= \frac{1 - S_r}{1 + 1/e} = \frac{e}{1+e}(1 - S_r) = n(1 - S_r)$$

Density index / relative Density :

physical interpretation – it indicates the state of loosness or compactness of cohesion less soil.
অর্থ clay থাকতে পারবে না, sandy type soil.
very important parameter for design.

২টা soil ২ জায়গা থেকে collected. void ratio same. But দেখা যায় একটা dense অন্যটা less. So " "
can't uniquely define the state of compaction or lossness of soil. So density index use করতে হবে।

$$I_d = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 = \frac{(e_{max}+1) - (e+1)}{(e_{max}+1) - (e_{min}+1)} \times 100 \quad \text{---} ①$$

sand নিব laboratory তে from field. loosest form এ void ratio মাপব, just উপর থেকে ফেলার পর, ওটা e_{max}.
vibration এ vibrate করব with load, so that's e_{min}.
এ ২টা lab এ।
e বের করব field এ। volume বের করাই এখানে most difficult.
এগুলা বের করা difficult, so unit wt or unit mass এ বের করি।

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

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

$$\Rightarrow 1+e = \frac{\overbrace{G_s \gamma_w}^{\text{for a given soil const.}}}{\gamma_d}$$

$$\therefore 1+e \propto \frac{1}{\gamma_d}$$

$$1+e_{field} \propto \frac{1}{\gamma_{d\,field}}$$

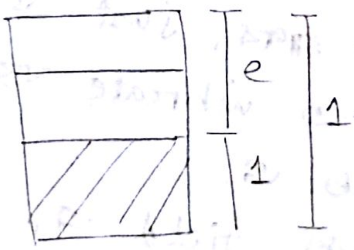
If e_{max} then $\gamma_{d\,min}$ & vice versa

putting this in eqn ①

$$= \frac{\frac{1}{\gamma_{dmin}} - \frac{1}{\gamma_{df}}}{\frac{1}{\gamma_{dmin}} - \frac{1}{\gamma_{dmax}}} \times 100$$

It can also be written as, eqn ① ⇒

$$I_d = \frac{V_{max} - V}{V_{max} - V_{min}}$$



so, $1+e = V$

Always remember

Based on I_d, State of compaction	I_d
Very loose	0-15
loose	15-35
Medium Dense (compact)	35-65
Dense	65-85
Very Dense	> 85

এটা মনে রাখব cause loose, very loose এ foundation বানান উচিত না, so we need to compact the soil.

The soil in Bd 90% of the soil is of medium dense.

যেখান থেকে soil আনা হয় ওটা borrow area. যেখানে আনার পর কাজে লাগান হয় ওটা finished or field area.

15.9.15

Tuesday

Lec - 4

(See lecture of other section)

Compaction:

Any ~~are structure~~ earth structure, (roadway, highway) বানাতে soil এতে compact করতে হয়।

compaction এর অনেক সুবিধা

(※) ~ is a process of densification of soil by giving mechanical energy into it and it's a fun. of

1) moisture content 2) mechanical energy given

3) type of soil

(#) Advantage: it reduces the settlement, increases the bearing capacity.

void ratio reduce করে, permeability কমায়।

⇒ Permeability is a property which dectates the ease of movement of water through soil.

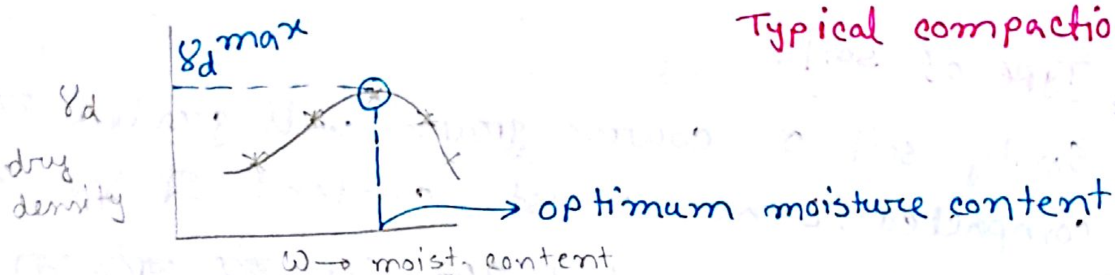
clay এর permeability incomparison to sand অনেক কম, so clay almost impermiable.

⇒ Soil compaction এর জন্য hammer থাকে, ওজন 5.5 lb. Height of fall এখনকার জন্য 12". Hammer is in a guide. [See figures from book]

Lab এর book pg - 43. 1st pg ভাল ভাবে পড়ব।

Dry sample কে moisture content বাড়াব। Then mould এ systematic manner এ dry density বের করব। 6, 9, 15, 18, 21% etc % of moisture content এ বের করব।

Typical compaction curve



Inverted V shape.

So moisture content এর সাথে γ_d related

(*) So moisture content corresponding to max^m dry density ~~water~~ content corresponding to max^m dry density
moisture
is called optimum moisture content

(*) পানি add করায় lubricating effect আসে, compaction by then soil skeliton slide with eachother.
If impact load ... void গুলো occupy করবে soil.
In the process of sliding

So void $\downarrow$ so unit wt $\uparrow$.

এটা continue করবে uptill water increase করতে থাকলে

ω_{opt}..

আরও পানি দিলে void spaces will be occupied by water.

So dry density কমতে থাকে।

Q. water add করলে $\gamma_d \uparrow$ then $\downarrow$ কেন?

Mechanical Energy :

2. 50টা blow দিলে বেশি energy. So γ_d তত বাড়বে। 1st এ বেশি বাড়ে. then কম বাড়ে।
Field এ roller এর ওজন & number of pass এর দ্বারা energy দেয়া হয়।

3. **Type of soil :**

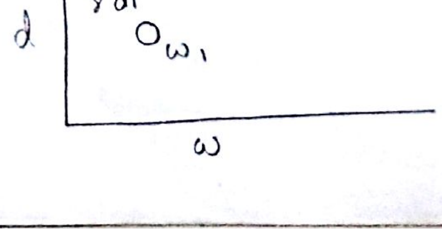
Sandy soil এ coarse grain, well graded হলে readily compacted. Lesser moist. content এ γ_d বেশি। gap, uniform graded হলে γ_d তত বাড়ে না incomparison to well graded, coarse grained.

Cohessionless soil : γ_d তত বাড়ে না। Sand এ 10-15% বাড়ে, clay এ 20-25% বাড়ে। Sand এ impact loading not effective. sand এ vibratory load দিয়ে compaction effective.

Test in lab :

$$\gamma_{Bulk} = \frac{\text{wt of soil sample}}{\text{internal vol}^m \ 1/30 \, ft^3}$$

$$\gamma_{d_1} = \frac{\gamma_{bulk}}{1+\omega_1}$$



wt 5'5 lb

$h = 12''$

⇒ blow = 25

layer = 3

vol^m = $1/30 \, ft^3$

for making 1 soil sample

∴ Energy used per unit volume

$$= \frac{5'5 \times 1 \times 25 \times 3}{1/30}$$

$= 12375$ J ?

Exactly same mold এ 5 টা layer. Height 18", wt = 10 lb

Blow in each layer 25.

∴ Energy per unit = 56250

So energy 4 গুণ বেশি।

But γ_d 4 গুণ বাড়ে না। 10-15% বাড়ে।

Q. $\boxed{\text{compaction করতে হবে field এ, why test in lab?}}$

Ans. An ideal condition to reach optimum moisture content to-reach max^m γ_d এর idea দেবে।

Q. Lab এ যে density field এ dry density diff কেন?

lab এ dry density 150 #/ft³. field এ γ_d এর থেকে বেশি হবে কেন? Field এ 120#/ft³ ও দিতে পারব। বেশি দিতে cost ↑.

$\gamma_d$max is not a unique value. It will depend on mechanical energy. But এটা costly বেশি energy দেয়,

Specification:

% compaction or Degrees of compaction:

$$\frac{\gamma_{df}}{\gamma_{dmax}} \times 100\%.$$

এটার 80-98%. এর মধ্যে যে কোন value হলেই হবে।

$\gamma_{df} = \gamma_{dmax}$ হলে 100%.

এটা performance type specification./end type specification

~~work type specification~~:

যদি %. compaction বলি, আর কিছু না বলি then বুঝতে হবে standard.

যদি 90%. modified বলি then lab এ যে γ_d max

পরে তার 90% field এ লাগবে.

γ_d & % compaction cohesive soil এর জন্য.

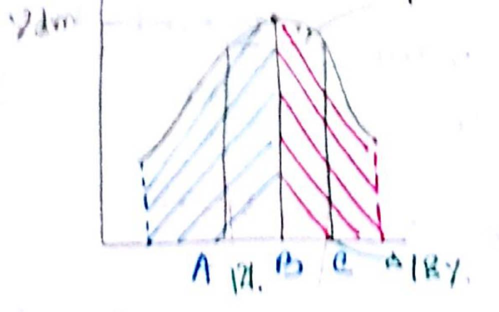
cohesionless এর জন্য relative density.
So only % compaction & modified % com. diff.

work type specification:

*chapter8

1) Type of equipment
2) The Lift thickness
3) The moisture content
4) The amount of work required to obtain necessary γ_d.

characteristics of compaction curve: Imp



95% of γ_{dm} বললে যেকোনো range of moisture content (AC)
left হল dry side [illegible]
right হল wet side

So কাজটা easy হবে.

12% & 18% এর মধ্যে রাখলেই ঠিক ভাবে full compaction characteristic পাওয়া যাবে।

more mechanical energy for same soil sample shifts the curve to left with higher γ_d and lesser w.

$\gamma_{d2} > \gamma_{d1}$
$w_2 < w_1$

সব তার 90% field এ লাগবে,

γ_d & % compaction cohesive soil এর জন্য,

cohesionless এর জন্য relative density.

So only % compaction & modified % com. diff.

work type specification:

1) Type of equipment

2) The Lift thickness

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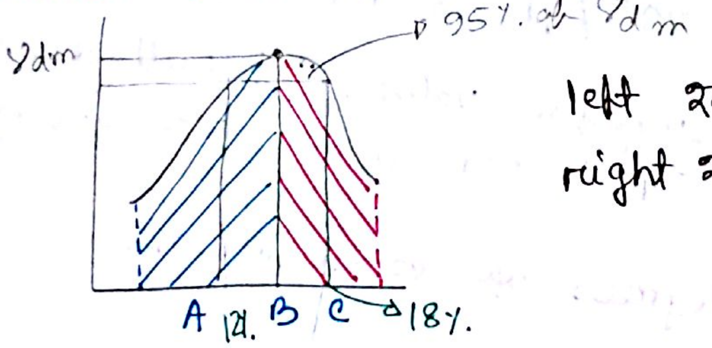
4) The amount of work required to obtain necessary γ_d.

Characteristics of compaction curve: Imp



95% of γ_{dm} বললে একটা range of moisture content (AC)

left হল dry side

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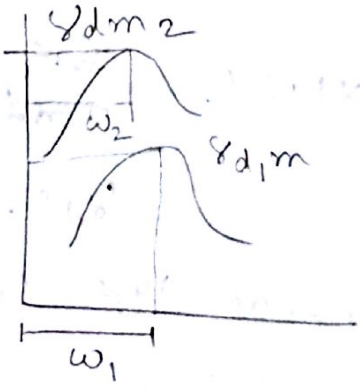
so কাজটা easy হচ্ছে,

12% & 18% এর মধ্যে কোনটা নিব সেটা জানতে full compaction characteristic জানতে হবে।

বেশি mechanical energy for same soil sample shifts the curve to left with higher γ_d and lesser ω.



$\gamma_{d_2} > \gamma_{d1}$

$\omega_2 < \omega_1$

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Lec-5

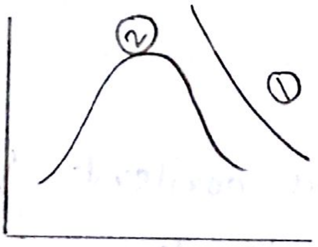
performance type এর সুবিধা contractor any tool use করতে পারবে, but performance same হতে হবে. রাস্তা, building এ এটা।

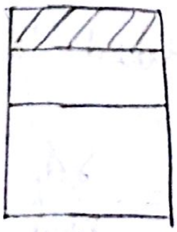
work type specification এ কি ধরনের compacting equipment লাগবে, number of pass, layer এর thickness, moisture content সব বলে দেয়া হয়. so specific degree of compaction বলে দেয়। এটা bulk construction - like dam এ এটা

air void ratio/ air content

$$\boxed{\gamma_d} = \frac{1 - \overset{\uparrow}{A_v}}{1 + w G_s} \gamma_w \qquad n_a = A_v = \frac{V_v}{V}$$

100% saturation line ① never touches the curve ②





$$V = V_a + V_w + V_s$$

$$1 = \frac{V_a}{V} + \frac{V_w}{V} + \frac{V_s}{V}$$

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

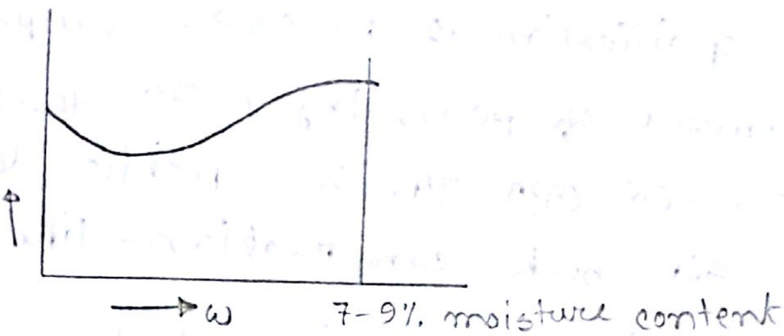
$$\Rightarrow A_v + \frac{w W_s}{\gamma_w V} + \frac{\gamma_d}{G_s \gamma_w}$$

$$\Rightarrow A_v + \frac{\gamma_d w}{\gamma_w} + \frac{\gamma_d}{G_s \gamma_w}$$

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

sand compaction এ moisture content 7-9% হলেই হয়। অন্যের method effective না। vibration or dynamic type effective

Sand compaction:



রাস্তায় roller type compaction. এগুলোর tyre type.

Japan, germany-তে 10-40' উঁচু থেকে cylindrical hammer দিয়ে compaction. Dynamic compaction.

Math:

Porosity $n = 40\%$.

unit wt at 12% moisture content $\gamma_{wt_{w=12\%}} = 17\ KN/m^3$

we have $100\ m^3$ soil sample. How much volume of H_2O would be added to make it fully saturated.

কিছু বলা না থাকলে wt/volume যেকোনোটা দিব

$$\gamma_{bulk} = 17\ KN/m^3 = \frac{\gamma_d}{1+w} = \frac{\gamma_d}{1.12}$$

$$\Rightarrow \gamma_d = \frac{\gamma_{bulk}}{(1+w)} = \frac{17}{1.12} = 15.17\ KN/m^3$$

$$15.17 = \gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.67 \times 9.81}{1+e}$$

$\therefore e = 0.73 \rightarrow$ void ratio

$$S_r = \frac{w G_s}{e} = 1$$

$$\Rightarrow w = \frac{e \times \overset{1}{S_r}}{G_s} = \frac{0.73}{2.67} = 0.27 = 27\%$$

In 27% moisture content bulk density

~~γ_{bulk} = ++~~ $\gamma_{sat} = \gamma_d (1+w_s) = 15.17(1+0.27)$

$= 19.27 \ KN/m^3$

or, $\gamma_{bulk} = \frac{(1+w) G_s \gamma_w}{1+e}$

For each meter^3 we need to add $(19.27 - 17) = 2.27$ KN of water

$\therefore$ 100 m^3 এর জন্য $= 227$ KN

Undisturbed soil in a borrowpit as a water content of 15% with void ratio 0.6 & $G_s = 2.7$. The soil from the borrow pit is to be used for the construction of embankment of a finished volume of 40,000 m^3. The specification for the embankment require a M. content of 18% and dry density of $\underbrace{1.76 g/cc}_{\text{mass density}}$

KN এ দিলে unit wt

Calculate the quantity of soil required to be excavated for the embankment of soil.

Sol^n:

G_s same for field & borrow area

Borrow Area

$w = 15\%$

$e = 0.60$

$G_s = 2.7$

volume = ?

Finished Area

$\gamma_d = 1.76$ g/cc (1.76 ton/m^3)

$G_s = 2.7$

$V_f = 40{,}000$ m^3

$$\gamma_d = \frac{G_s \times \text{mass density of water}}{1+e_f}$$

$$\Rightarrow 1.76 = \frac{2.7 \times 1}{1+e_f}$$

$$\therefore e_f = 0.53$$

$$V_f \propto (1+e_f)$$

$$V_b \propto (1+e_b)$$

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

Now,

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

$$\Rightarrow V_b = \frac{40{,}000(1+0.6)}{1+0.53} = \underbrace{41{,}830 \text{ m}^3}_{\text{(Ans)}}$$

$$V_b \propto (1+e_b)$$

$$\Rightarrow V_b = K(1+e_b) \qquad \therefore K = \frac{V_b}{1+e_b} = \frac{41{,}830}{1+0.6}$$

$$V_f \propto (1+e_f) \qquad \therefore K = \frac{V_f}{1+e_f} = \frac{40{,}000}{1+0.53}$$

New Chapter

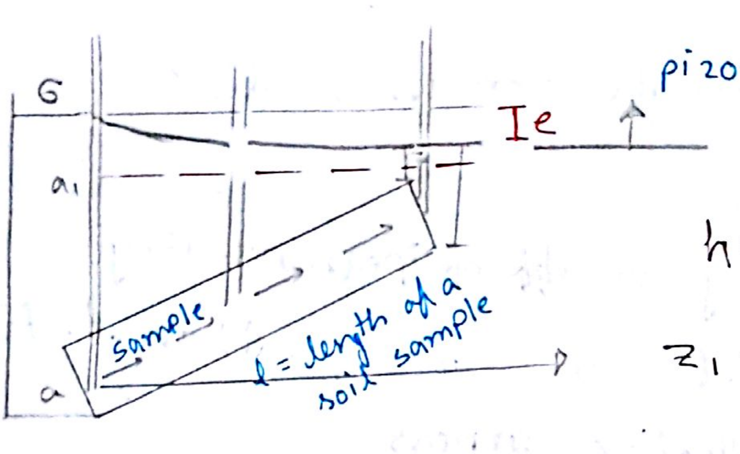
◫ Fluid flow Through Porous media:

1. Darcy's Principal
2. factors effecting co-eff of permiability
3. Determination of " " " (Lab & field)
*4. Principal of effective stress
5. Seepage pressure
6. Quick Sand
7. Governing diff eqn of fluid flowing through soil
8. flow net diagram and its application

6.10.15

Tuesday

Lec – 6



pizometric line at a,s,

h = head loss

$$z_1 + \frac{v^2}{2g} + \frac{p}{\gamma}$$

The percolation of water through the contineous void of soil sample is called seepage. ⊛ ⊕

soil এ যদি flow করতে পারে cause of permiability.
না করতে পারলে impermiable mat.

⊕ ⊛ which is measure of easiness of the flowing water through the contineous void of soil sample

The magnitude of velocity in field consider করলে এত কমে যে velocity head can be neglected.
The flow needs to be laminar. Streamlines are parallel

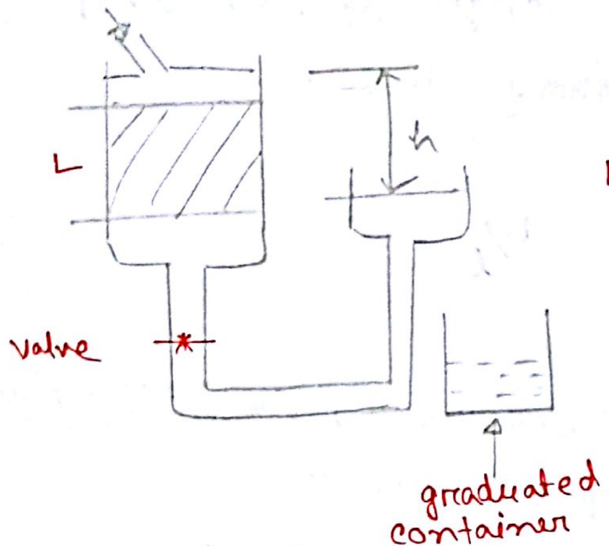
Darcy's Principal :

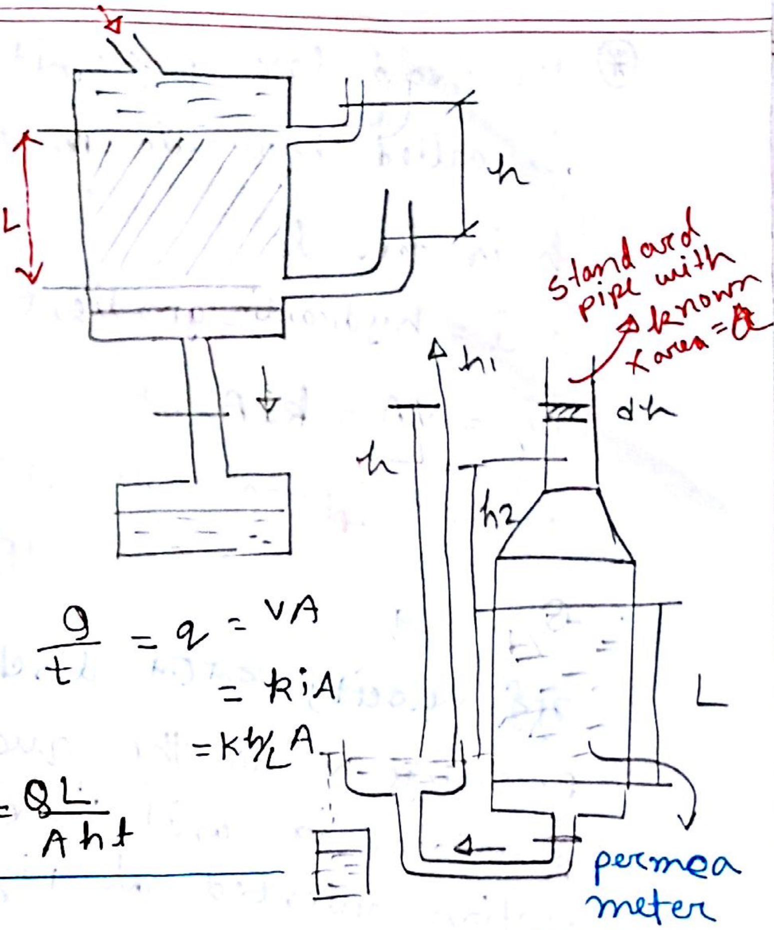
v. is directly proportional to hydraulic radius

$$V = K \underset{\downarrow}{i}$$

permeability of soil = co-eff of permeability

Next page





L = length of soil sample

(a) constant head method →

$$\frac{Q}{t} = q = VA$$
$$= kiA$$
$$= k\frac{h}{L}A$$

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

(B) variable head: (পতন মাথা)

$$h_L \quad -\int \frac{dh}{h} = \frac{KA}{La}\int_0^t dt$$

$$\ln \frac{h_1}{h_L} = \frac{KAt}{aL}$$

$$\textcircled{-}\, a\,dh = q \times dt$$
$$= kiA\,dt$$
$$= k\,\frac{h}{L}\,A\,dt$$

dh = change in head for dt
loss associated with increasing time, তাই (−)

⊛ The head loss per unit length of soil sample is called hydraulic gradient.

h is for l

$\therefore i$ = hydraulic gradient = h/l

$\boxed{q} = VA = kiA$

flow rate

Darcy calls it discharge / apparent / fictitious / approch velocity

$Q/t = q$

শুধু velocity বললে discharge velocity

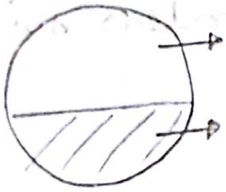
Discharge v. is the quantity of water that percolates in unit time across unit area of x-section oriented at right angle to the direction of flow.

⊞ factors Effecting K value : (Lambe)

1. size of shape of soil grain
2. Properties of water (viscosity & unit wt ⇒ function of temp)
3. Porosity & void ratio
4. Arrangement of the soil particle (floculated হলে বেশি dispersed হলে কম)
5. Degrees of saturation

Typical value of K:

Type		mm/s
Gravel	——	10 to 10^3
Coarse sand	——	10 to 0·1
fine sand	——	0·1 to 0·01
silt	——	0·01 to 0·0001
clay	——	less than 0·00001



→ passes → শুধু ফাঁকের মধ্যে যে velocity true vel.

→ এর মধ্যে doesn't pass

} সবের মধ্যে যে vel. of H_2O ওটা apparent

⊛ Diff betⁿ seepage & discharge v?

seepage ↓ x-section of ~~[illegible]~~ void

discharge ↓ x-section of whole (gross)

Relⁿ betⁿ true & app. v:

Flow Rate $= VA_g = V_{sp} A_v$

$$V = \frac{A_v}{A_g} V_{sp} = \underbrace{\boxed{\frac{V_v}{V_g}}}_{\to n} V_{sp}$$

$V = nV_{sp}$

n is less than 1. so, Vsp বেশি হবে seepage velocity ↑

Determination of K value :

1) Emphirical
2) Lab det^n (1) const head
2) variable head)
3) field determination

} for cohesionless soil (sand)

*) No direct method for cohesive soil. Indirectly from consolidation data

1) Emphirical :

clean, coarse, round, cohesionless soil এর জন্য. BD তে sylhet soil.

$$1K = C D_{10}^2$$

↓ ↳ in cm

Based on statistical approch const = $\frac{100}{cm.sec}$

very rough estimation

2) Lab :

(a) const. head method :

field sample এর density এর same density রেখে permeameter এ pressure দেয়

applicable for $K > 10^{-2}$ mm/s

for finer material variable head method (versatile & reliable method.

Draw Backs :

(1) Field এর soil arrangement copy করতে পারব না। Density same হবে না। So field condition can't be slimulated.

(2) Fully saturated হবে না। Air entrapped in voids. So in the process of making fully saturated -. If air entraped then creates obstruction for flow of water. So K ঠিক না। এজন্য suction করে air then density আবার ভরাই, But if suction in high then density কমে যায়।

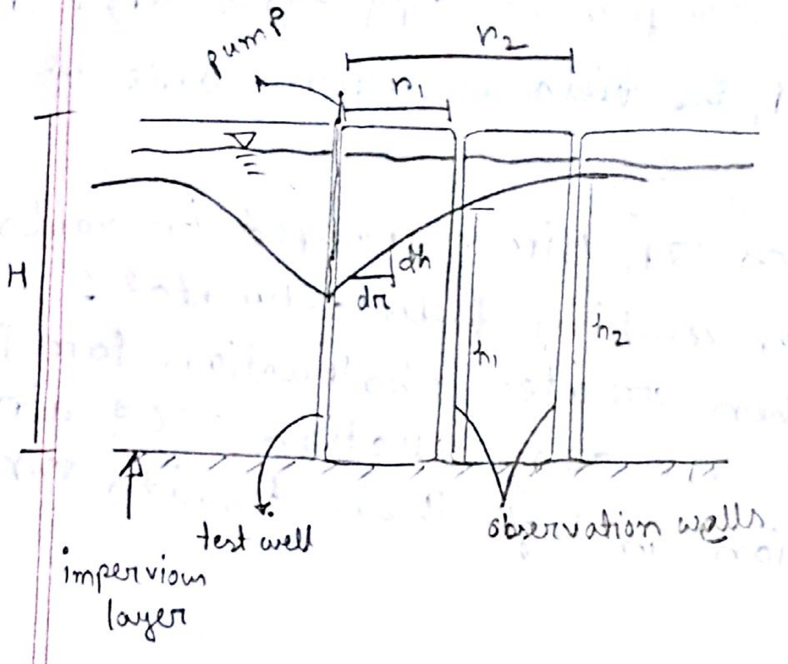
(Pg - 42)

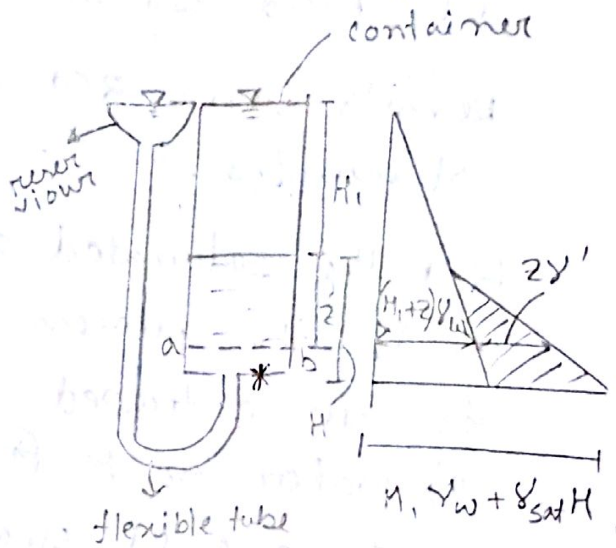
11.10.15

Sunday

Lec - 7

①



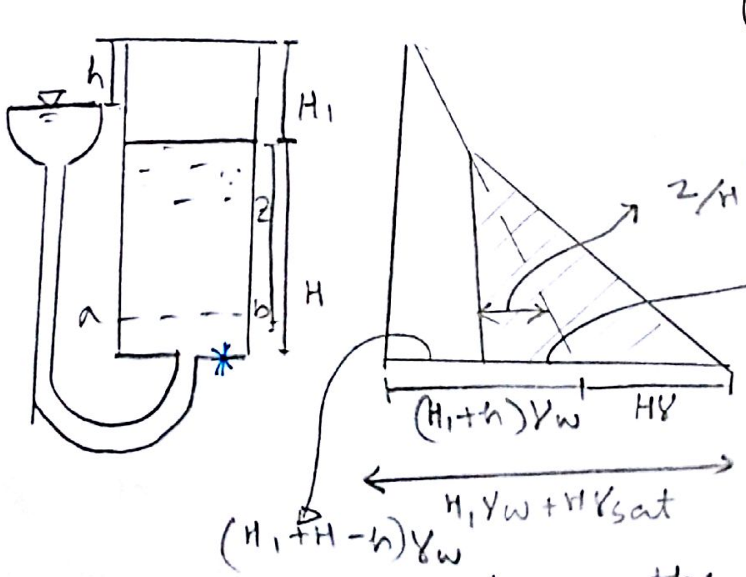


(a) at no flow conduit.

$$q = KiA = K\frac{dh}{dr}2\pi rh$$

㊀ $U_{wA} = (H_1 + h)\gamma_w$

③

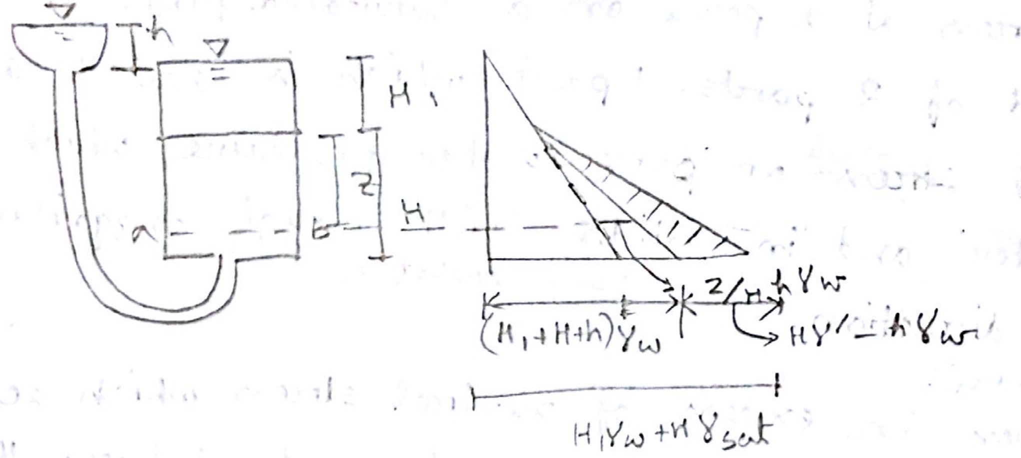


(b) Flow from the container to reservior

㊀ neutral stress = $(H_1 + H - h)\gamma_w$

water flows because of fric. resistance of soil sample.

④



Flow from reserviour to container

① Test well এর radial direction এ ২টা observation well. pump এর capacity লেখা থাকে, Q rate এ water extraction so H_2O table নিচে নামবে, pump এর আয়ত্বে max draw down.

 এটা draw down curve.

$$A = 2\pi r h$$

$$q \int_{r_1}^{r_2} \frac{dr}{r} = 2\pi K \int_{h_1}^{h_2} h\,dh \qquad [\text{eqn } 2.7]$$

$$K = \frac{2.3\,q}{\pi (h_2^2 - h_1^2)} \log_{10}\left(\frac{r_2}{r_1}\right)$$

☐ Stress in soil:

stress বললে stress at a point & its orientation বলতে হবে, stress at a point on a plane বলতে হবে,

The total stress at a point on a saturated plane is composed of 2 parts. 1 part which is called as neutral stress or pore water pressure which acts in water and in solids with equal magnitude
↳ soil skeleton
in all direction.

Other part:
The stress in excess of neutral stress which acts exclusively between the contact points between the soil skeleton is called effective stress.

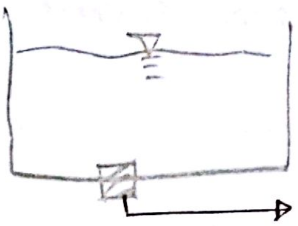
$$\underset{\text{total}}{p} = \underset{\text{neutral}}{u_w} + \underset{\text{effective (in excess of neutral)}}{\bar{p}}$$

only effective stress is responsible for inducing volume change behaviour in soil. It also provide the frictional resistance in soil.

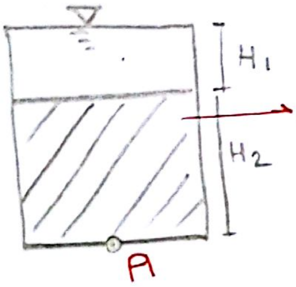
On the other hand pore water pressure has no role in inducing vol change behaviour nor does it have any role in providing frictional resistance.

These are called principal of effective stress proposed by Karl Kerzaki.

⊛ The total stress at a point on a plane is the stress contributed by the overline material.



এখানে total stress due to H_2O and উপরের soil sample.



saturated soil sample γ_{sat}

$\therefore$ stress total $\Rightarrow$ $p_A = H_1\gamma_w + H_2\gamma_{sat}$

soil এর basic assumption soil contains contineous voids. So এখানে according to principal of hydraulics neutral stress $u_{w_A} = (\cancel{A} H_1 + H_2)\gamma_w$

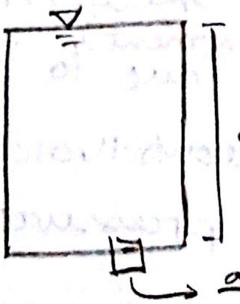
$\therefore$ Effective stress $\bar{p} = H_1\gamma_w + H_2\gamma_{sat} - H_1\gamma_w - H_2\gamma_w$

$\therefore \bar{p} = H_2\gamma_{sat} - H_2\gamma_w$

$\gamma_{sat} - \gamma_w$ = effective/submerged unit wt

$\therefore \bar{p} = H_2(\gamma_{sat} - \gamma_w) = H_2\gamma'$

effective ~~pressure~~ stress is independent of the height of water above the soil sample.



এখানে γ' লাগব

soil uptill z. H_2O আছে, But h of water doesn't matter.

So $\bar{p} = z\gamma'$

Down ward movement of H_2O

③ এ u_{wA} stress ↓ , but total stress same.

So $\bar{p}$ ↑.

$$\bar{p} \text{ increased by } = H\gamma' + \underbrace{\boxed{h}\,\gamma_w}$$

→ head loss

downward movement এর কারণে $\bar{p}$ ↑ by $h\gamma_w$ because of reduction of u_{wA}.

H height এর জন্য head loss h

∴ z depth " " " " $\frac{hz}{H}$

Along a section ab the stress

if no flow $\bar{p} = \gamma' z$

if there is flow (in 3) $\bar{p} = \gamma' z + \frac{hz}{H}\gamma_w$

head loss per unit length of soil sample = hydraulic gradient

$$\therefore \bar{p} = \gamma' z + \underbrace{iz\gamma_w}$$

→ seepage pressure

it occurs due to movement of water from higher to lower engery.

seepage pressure is the result of frictional drag of the flowing water onto the soil skeleton and the increase (in →) or decrease (upward movement) magnitude of effective stress due to the flow of water through the contineous void of the soil sample is called seepage pressure.

downward movement

④ upward movement of water

porce H_2O pressure ↑, so $\bar{p}$ ↓. So কমবে একই মাত্রায় $i z \gamma_w$. this would be -ve.

$$\therefore \bar{p} = \gamma z \enclose{circle}{\pm} i z \gamma_w$$

13.10.15

Tuesday

Lec - 8

$\bar{p} = \gamma' z - i z \gamma_w \qquad i = h/L$.

একটা stage এ $\bar{p} = 0$ if increase i,

This condition reffers to quick sand. It is not a particular sand but it reffers to a condition when $\bar{p}$ becomes 0 and the corresponding hydraulic gradient is called critical hydraulic gradient denoted by i_c.

$\bar{p} = \gamma' z - i z \gamma_w$

as $z \neq 0 \qquad \therefore i_c = \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} = \frac{G_s - 1}{1+e}$

under this condition completely looses its strength, it behaves like a fluid an co-eff of permiability approches very high value.

This is also called boiling sand / boiling condition.

It exists only in cohesionless soil.

Typical void ~~r~~ ration for granular soil 0.5 - 1

$G_s = 2.7 \qquad \therefore i_c = .83$ to 1.1

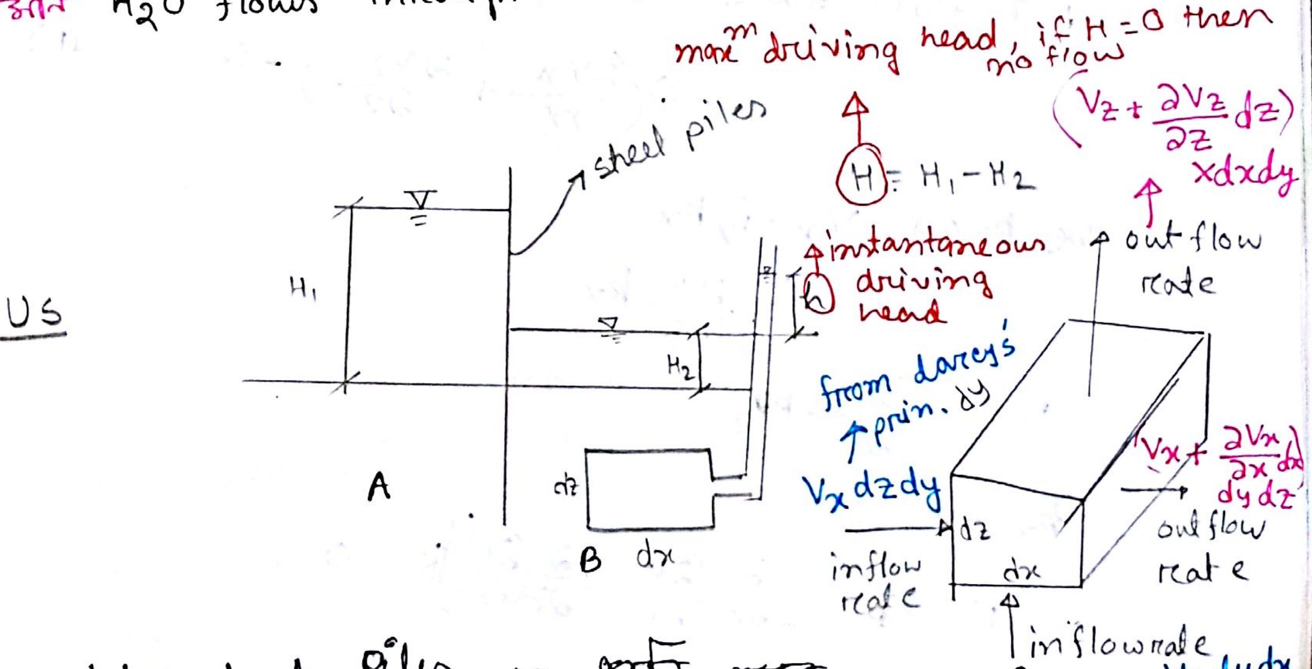
→ if value not given then assume $i_c = 1$

It occurs mainly in fine sand or silt, cannot occur in coarse sand. Larger the particle size, greater will be the porosity. To maintain critical hydraulic gradient larger velocity of flow is required which is not seen in nature flow condition. This condition also doesn't exist in cohesive soil. (ঐসব soil এ হবে where strength is directly $\propto$ to the normal stress)

[Short note আসলে এতটুকু]

(*) downward movement problematic না, upward movement is dangerous.

⊕ কি পরিমাণ H_2O flows through soil তার governing eqn form



impermeable sheet piles, so পানি ঘুরে আসবে from A to B. Then B থেকে H_2O again pump করে A তে নেয়া হয়।

Basic assumption for derivation

1) flow is two dimensional

2) soil skeleton, water particle are incompressible

3) soil is fully saturated

4) Homogeneous, isotropic

5) Darcy's principle applicable

6) flow condition doesn't change with time (steady state flow in = out ∴ net flow = 0)

ⓧ Along y axis no flow.

Net flow rate = inflow - outflow

$$\Rightarrow \left(\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} \right) = 0 \quad \text{---(1)}$$

we know, $V_x = i_x k_x$

$$= \frac{\partial h}{\partial x} k_x$$

$$V_z = \frac{\partial h}{\partial z} k_z$$

$k_x = k_z$

$$\therefore (1) \Rightarrow \left[\frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z}\right) \right] K = 0$$

$$K \neq 0$$

$$\therefore \frac{\partial}{\partial x}(\partial h/\partial x) + \frac{\partial}{\partial z}(\partial h/\partial z) = 0$$

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

$$\Rightarrow \nabla^2 h = 0$$

the algebric summation of the hydraulic gradient in x & z direction is zero. (Mathematical interpritation)

Physical interpritation:

physically this eqn represents 2 sets of curve. One set is called flow line and another set is called equipotential line. and both the lines are orthogonal to each other

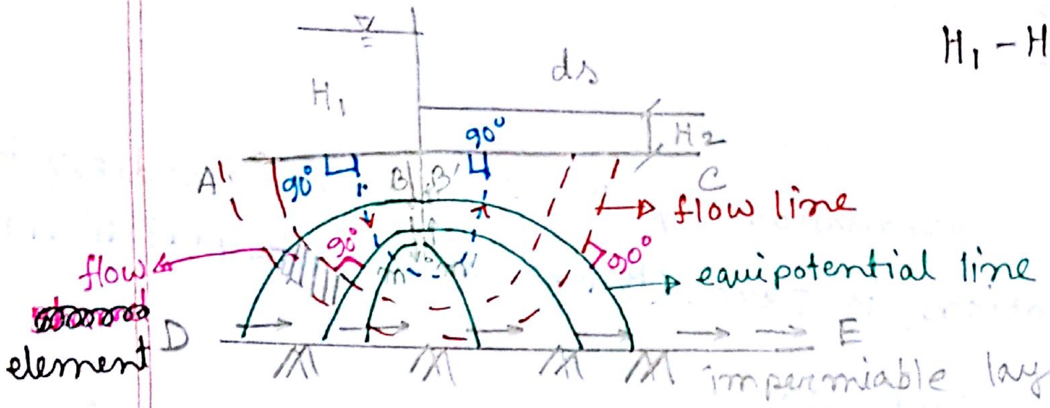
(*) Soln of a problem is done by assuming a function, that function has to satisfy the problem and satisfy the boundary condition. Only that will satisfy the problem.

⇒ Flow line is a line along which H_2O particles move from upstream to downstream (No flow line intersect each other, cause 1st এই laminar flow assumed)

⇒ Equipontential lines is the line along which all pizometric head will have a same value.

(*) Graphically solve করলে হবে boundary condition মানবে।

$H_1 - H_2 = H =$ driving head

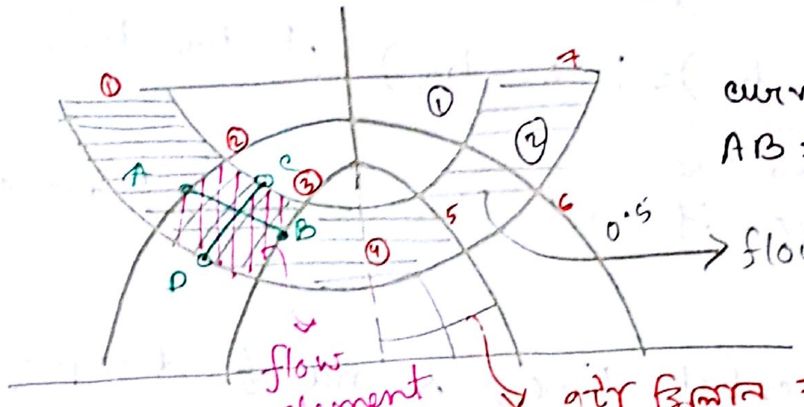


In upstream A to B all the points এ piezometer
shows H_1, so equipotential line

B' to C in dowstream ,, . shows head H_2

Bm = flow line m'B' = flow line
mm' = ,,
DE = ,,
domain, so অসংখ্য flow line.
এই flow line পুরো
flow line & eq. line ⊥
but we take specific
অনেক lines,
Region bound by two sussesive flow lines is
called flow channel. and the region bounded
betn flow lines (two successive) & 2 successive
eq. line is called flow element. And the combination
of flow line, eq. line is called flow net
diagram.

এমনভাবে flow line & eq line design করব যেন each flow element square ~~হয়~~ in shape হয় (in curvilinear sense)

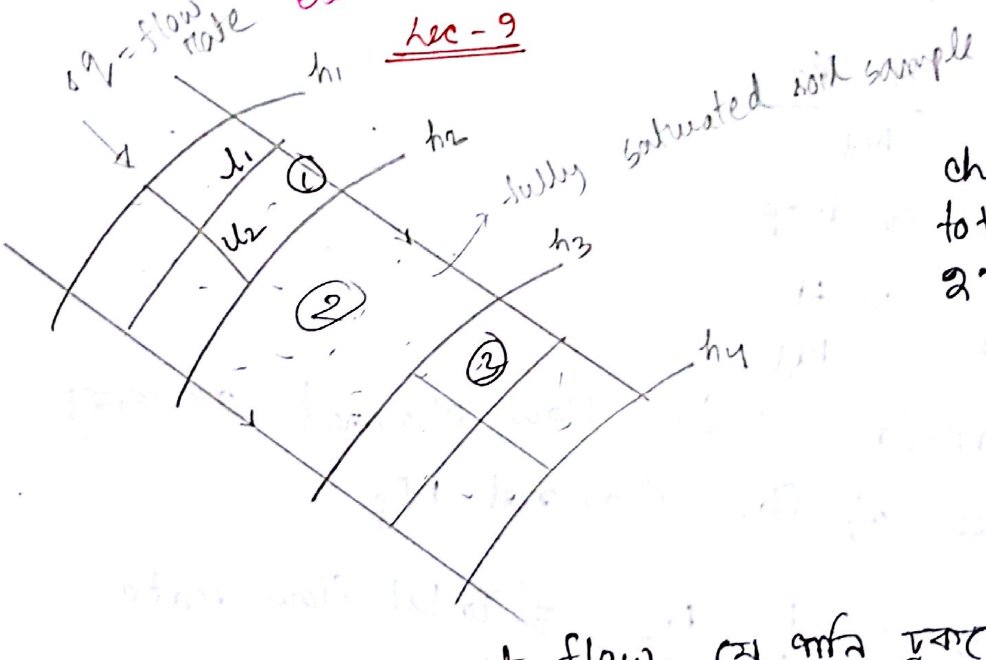


curve linear sense হলে AB = CD হলে square.

এটা মিলানো যায় না, then ratio $\frac{l_1}{l_2} = 0.5$ (suppose)

so এটা 0.5 channel, so total channel
2 + 0.5 = 2.5

Lec - 9



Basic assumption no net flow. যে পানি ঢুকবে ওটা বের হবে,

① $\Delta q = k i A$ $i = \frac{h_1 - h_2}{l_1}$ $l_1 = l_2$

$= k . \frac{h_2 - h_3}{l_1} \times l_1 \times 1$

for ② $\Rightarrow \Delta q = k \frac{h_2 - h_3}{l_2} \times l_2 \times 1$

③ $\Rightarrow \Delta q = k \frac{h_3 - h_4}{l_3} \times l_3 \times 1$

$$\therefore \Delta q = k(\underset{\substack{\downarrow \\ \text{head lose}}}{h_1 - h_2}) = k(h_2 - h_3) = k(h_3 - h_4)$$

এরা equal হবে if all flow elements are curvilinearly square

7 টা eq. potential line, so nu of drop = 7−1=6

$$\frac{H}{\underset{\substack{\Downarrow \\ \text{number of drop}}}{N_d}} = \frac{H_1 - H_2}{N_d}$$

$$\therefore \Delta q = K \frac{H}{N_d}$$

এই কারণে 1 টার flow channel এর জন্য

Number of flow channel N_f

$$\therefore \sum_{1}^{N} \Delta q = K \frac{H}{N_d} N_f = \text{Total flow rate}$$

⊛ যদি flow channel এঁকে দেয় then N_f fixed. অন্যরকম আঁকলে N_f change হবে,
এ যে H_2O যেটার seepage upstream → downstream এ lose.

sheet pile এর value unit ফিলাইড। ft, gauge এ দেয়া থাকতে পারে।

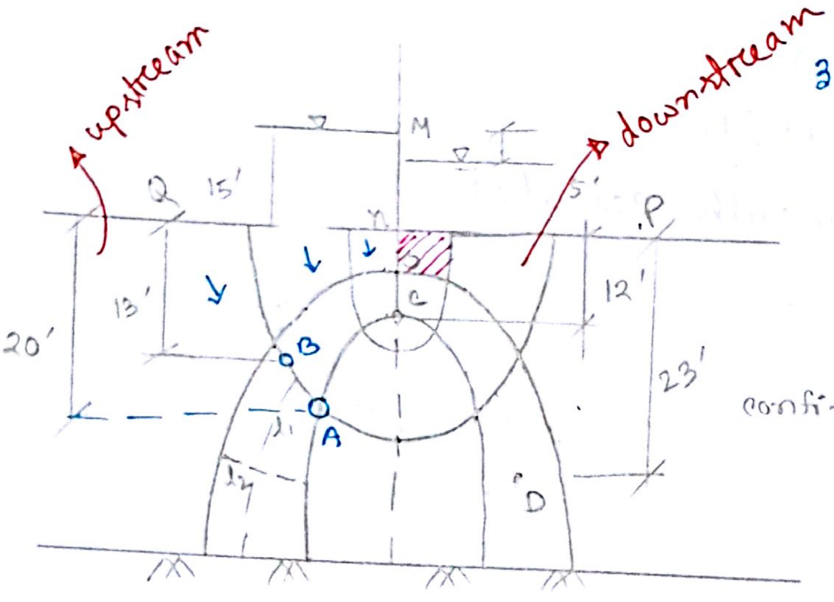
K দেয়া থাকবে।

q in 6 months calculate.

18.10.15

Sunday

Lecture – 10



3 flow channels

confined flow

flow net diagram

Flow rate through one channel $\Delta q = K\, H/N_d$

N_f number of flow channel, N_d number of equipotential block.

$$\sum_{1}^{N_f} \Delta q = KH/N_d$$

$$\therefore q = KH \frac{N_f}{N_d} \quad = \text{seepage loss}$$

(*) One imp assumption elements curvilinear square last square তার হবে l_1/l_2 ratio, suppose x. so

$N_f = 2+x$

$\gamma_{sat} = 120 \#/ft^3$

→ If there was no flow then the effective stress at A বের করব।

stress contributed by overlying material

~~total stress~~ $15\gamma_w + 20\gamma_{sat} - (20+15)\gamma_w$ = effective stress with no flow condition

$= 20(\gamma_{sat} - \gamma_w) = 1150$ psf

Again from formula $\bar{p} = H_2\gamma' = 20(\gamma_{sat} - \gamma_w)$

Here driving head $15 - 5 = 10'$

Here are total 6 drops to go from O to P

$\therefore$ Magnitude per drop $= 10/6 = 1.67'$

$\therefore$ stress " " $= 1.67\gamma_w$ (pore water pressure) most probably

In no flow condition at A pore H_2O pressure $= 35\gamma_w$

2 drops. So A এ pore H_2O pressure $= 35\gamma_w - 2 \times 1.67\gamma_w$

$= 31.6\gamma_w$

In point A effective stress with flow condition

$= 15\gamma_w + 20\gamma_{sat} - 31.6\gamma_w$

$= 1362.5$ psf

⊛ এখানে flow এর জন্য effective stress বাড়ে (due to downward movement of flow)

⊛ flow upward হলে effective stress বাড়তো, so দেখতে হবে flow upward না downward

At B, Total stress = $15\gamma_w + 13\gamma_{sat}$

Pore H_2O pressure$_B$ = ~~$13(\gamma_{sat} - \gamma_w)$~~
$(15+13)\gamma_w$ (in no flow)

Pore H_2O pressure at B (in flow) $= (15+13)\gamma_w - 1{\cdot}5 \times 1{\cdot}67\,\gamma_w$

$= 25{\cdot}495\,\gamma_w$

Effective stress at B $= 15\gamma_w + 13\gamma_{sat} - 25{\cdot}495\,\gamma_w$

$= 904{\cdot}0625$ psf

At D:

with flow (going from right to left) pore H_2O pressure $= 28\gamma_w + 1{\cdot}5 \times 1{\cdot}67\,\gamma_w$

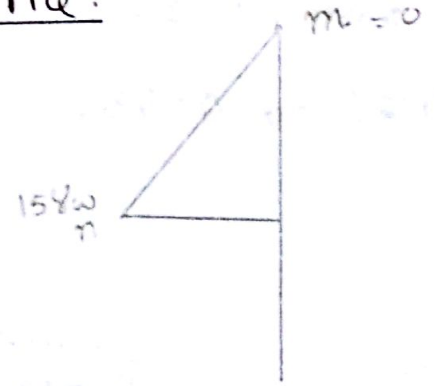
(+) হল cause D এর pressure total এর জন্য 28 হয়।

$= 1906{\cdot}5625$

(*) Effective stress কমে গেলে অত্যধিক, it arises the cond^n for quick sand.

Q. Draw the pore H_2O pressure in upstream side, then downstream side.

pore H_2O pre.



suppose NO = 6′

OC = 7′

at O = $(15+6)\gamma_w$ at no flow

with flow O = $(15+6)\gamma_w - 1.67\gamma_w$

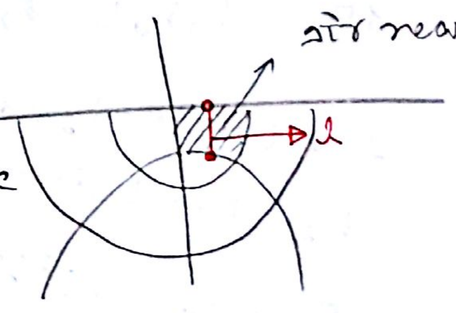
similarly at C = $\overbrace{(15+6+7)\gamma_w}^{\uparrow \text{ no flow}}$

with flow $- 1.67 \times 2\gamma_w$

At point C there are also 3, 4 drops, at O 5 drops, এখানে n এ আবার 5γw হয়।

Downstream এ nearest to the sheet pile যে element তার hydraulic gradient এর উপর depend করে quick sand হবে নাকি না।

▨ $i_{exit} = \frac{1.67}{l} =$ $l \rightarrow$ distance bet^n mid points



check it with critical hydraulic gradient

$i_c = \frac{G_s - 1}{1+e}$ or $i_c = \frac{\gamma'}{\gamma_w}$

In H_2O to make your structure i_{exit} should be less than i_c. Usually 3 or 4 F.S. provided.

cause $\bar{p} = \underbrace{\gamma' z - iz\gamma_w}_{\text{constant.}}$

If ic not given, we assume 1.

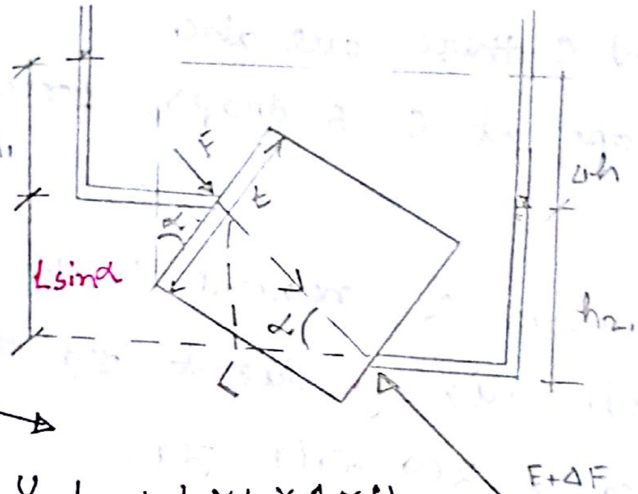
So exit ~~gra~~ gradient 0'25 to 0'33.

Lec - 11

1.11.15
Sunday

Seepage force :

It is a particular element



flow direction →

$$\Delta F = h_1\gamma_w L - h_2\gamma_w L + L \times L \times 1 \times \gamma_{sat}\sin\alpha$$

$$h_1 + L\sin\alpha = h_2 + \Delta h$$

$$h_2 = h_1 + L\sin\alpha - \Delta h$$

$$\therefore \Delta F = h_1\gamma_w L + L^2\gamma_{sat}\sin\alpha - \underbrace{(h_1 + L\sin\alpha - \Delta h)}_{h_2}\gamma_w L$$

$$= L^2(\gamma_{sat} - \gamma_w)\sin\alpha + \Delta h\gamma_w L$$

$$= L^2\gamma'\sin\alpha + \underbrace{\Delta h\gamma_w L}_{\text{seepage force}}$$ ⇒ for any arbitrary direction

seepage force → direction along the flows

$\bar{p} = \gamma' z \pm iz\gamma_w$ it's for up or downward

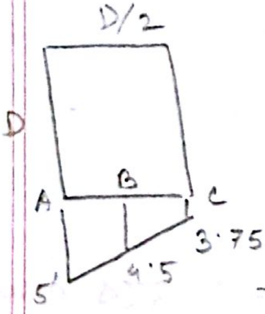
α = 90° হলে vertically down
α = 0° " horizontal

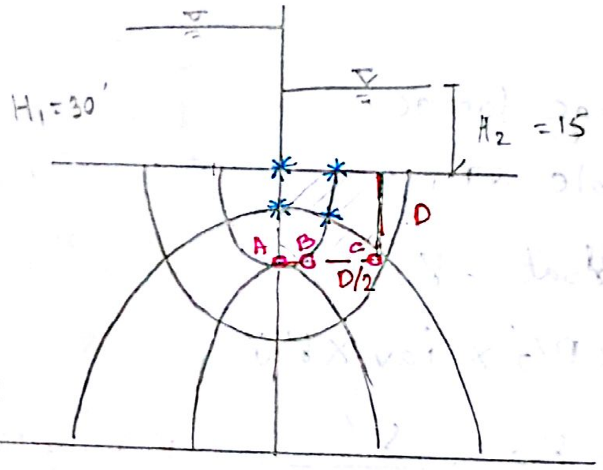
seepage force per unit vol^m $= \frac{\Delta h \gamma_w L}{L \times L \times 1}$

$= \frac{\Delta h \gamma_w}{L}$ → hydraulic gradient

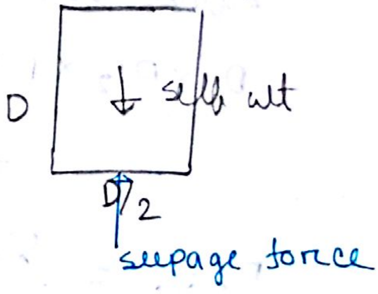
$= i\gamma_w$ (direction will coincide with dir. of flow.

Q. Prove S.F per unit vol^m $i\gamma_w$.





Sheet pile এর stability check by this method



depth of embankment on downstream side = D

D/2 এর width এর soil retain করতে পারলে stable.

এই block এর self wt downward, seepage force works upward

wt $= D \times D/2 \times 1 \; (\gamma_{sat} - \gamma_w)$ [↓]

If its volume is $D \times D/2 \times 1$

Then seepage force $= D \times D/2 \times 1 \times i\gamma_w$ [↑, block কে তুলে ফেলতে চাবে]

Total drops 6.

Per drop $15/6 =$

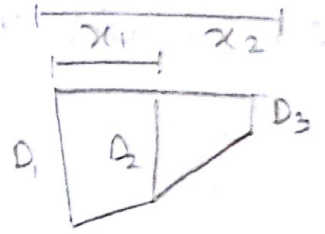
A তে 4 টার drop, so driving head 2 টার, so $2 \times \underbrace{15/6}_{2.5}$

$= 5$

B তে D.H $= ~~4~~ 1.8 \times 2.5$

c " D.H $= 1.5 \times 2.5$

Take weighted avg of 5, 4.5, 3.75 = h_{av}



$$\frac{D_1 \times 0 + x_1 \times D_2 + x_2 \times D_3}{x_2}$$

= wt. avg

$i_{av} = \frac{h_{av}}{D}$

If wt & seepage force equal then stable না,

$D \times D/2 \times 1 \times (\gamma_{sat} - \gamma_w)$

$= D \times D/2 \times i_{av} \times \gamma_w$

$$F.S. = \frac{\gamma_{sat} - \gamma_w}{i_{av}\gamma_w} = \frac{\gamma'}{\gamma_w i_{av}}$$

$\Downarrow$

4 ~ 5 (cause risk বেশি)

[F.S. রাখতে হবে cause stable রাখতে হলে downward force বেশি হতে হবে]

[আবার exit gradient দিয়েও বের করা যাবে of ⊛ element then compare it with critical hydraulic gradient.] It has to be 3 ~ 4

We know $\frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1 + e}$

[self wt. বাড়ালে F.S বাড়বে]

Filter material বেশি use করলে in down stream seepage force কমে যাবে cause H_2O passes easily. কারণ seepage force হয় for friction betn soil & H_2O. H_2O passing easy হলে friction কমে।

㊕ যে সব material → বাঁধের ভেতর যেটা base material.

Extra যে material used that is filter material.

⊗ filter media এমন হবে যেন H_2O easily pass করতে পারে, but base mat. যেন washout না হয়

২ criteria of filter media:

1. A filter material should be coarse enough so the parcolationg ~~[illegible]~~ H_2O moves through easily without any build up of pore H_2O pressure /seepage pre. in the filter. This criteria will be satisfied if

$D_{15}(f) > 5 D_{15}(b)$, if this is satisfied.

তবে D_{15} of filter mat. is greater than 5 time of D_{15} of base material.

২. ~ should be fine enough that the ~~base mat.~~ soil particles of the base mat. are not washed out through the filter. it is satisfied if

$D_{15}(f) < 5 D_{85}(b)$ is satisfied

combining both the criteria

$$\frac{D_{15}(f)}{D_{85}(b)} < 5 < \frac{D_{15}(f)}{D_{15}(b)}$$

filter mat. এই criteria fulfil করলে subsurface errosion, seepage force won't develop.

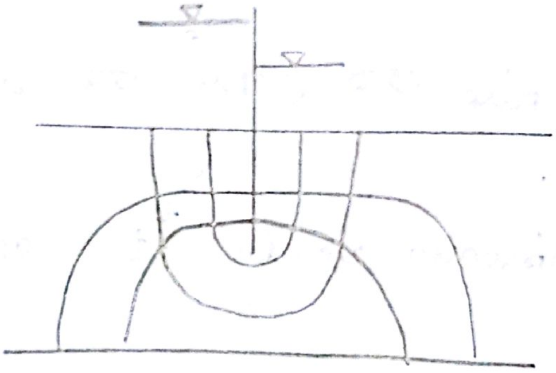
filter & bed material এর variation যদি significant হয়, তবে filter এর তুলনায় অনেক coarse হয় base material

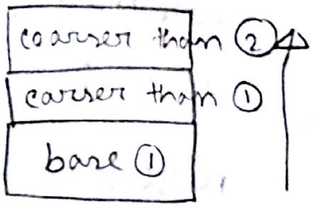
Then (next lecture)

8.11.15

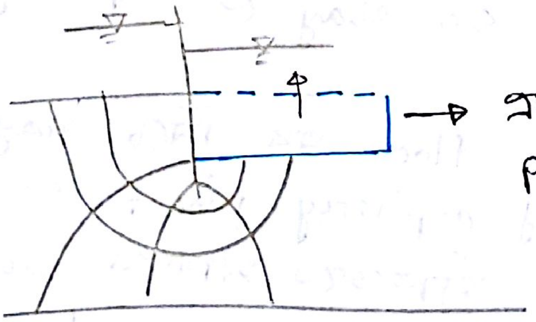
Sunday

Lec-12

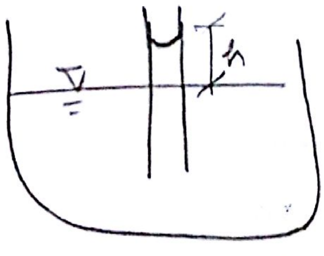


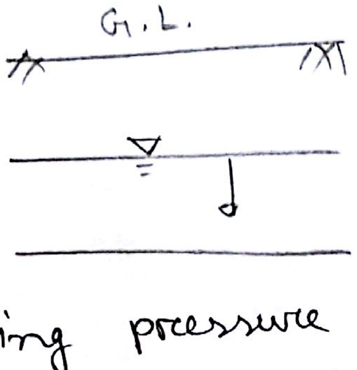


This is called reverse/graded filter. প্রতিটার 12'' এর মত thick. Filter mat base এর নিচে, gradually base coarser করা হয়।



→ এই portion এ filter paper দিলে flow lines change হবে। H_2O obstruction দ্বারাই উঠবে ↑ উঠবে





উঠবে degrees of saturation কমবে।

rises. cause existing pressure decreases than Atm. pressure. কতটা উঠবে depends on radius of tube

& properties of fluid.

$$h = \frac{4T\boxed{\cos\alpha}}{\gamma_w d}$$ → we can assume 1

যত dia ছোট rise তত বেশি, এই analogy is applied on soil.

Soil is of contineous voids. its called capilary rise.

coarse sand এ void বেশি.

capilary rise of soil এর emphrical eqn

$$h = \frac{C}{e D_{10}}$$

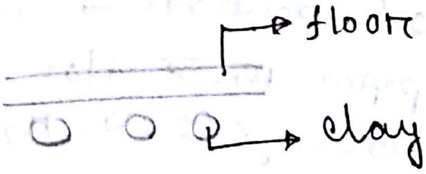
C → constant (10-15 m^2)

e ↓ void ratio

D_{10} ⇓ m

$\boxed{\text{clay তে } D_{10} < \text{sand}}$

so clay তে capilary rise অনেক বেশি



floor এর নিচে clay দিয়ে fill করলে capilary rise ↑. so floor Always স্যাঁতস্যাঁতে থাকবে, so always fill with coarse sand.

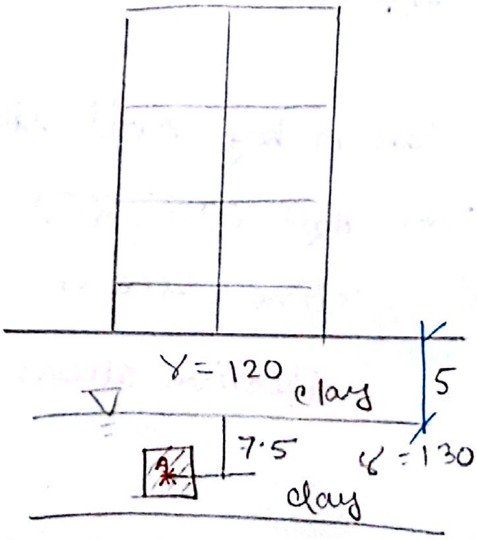
Soil Type	Size	Capilary rise h_c = cm
sand coarse	·2-·6	1·5 - 5
sand medium	·6-·2	5 - 15
sand fine	·2-·06	15 - 50
silt	0·06-0·002	50 - 150
clay	<·002	150 - 15000

* Craig & B.M. Das এর বই এ flow net diagram দেয়া আছে।

Consolidation of Soil

Consolidation Characteristics of soil :

From basic mechanics point of view



suppose suddenly building হঠাৎ place করলাম, এই load টা soil এ transfer হবে। নিচে আছে clay.

For this an element ▨

Building হওয়ার আগে overburden pressure ছিল।

" " পর imposed load আসলো, so then pressure Δp. ↳ excess pore H_2O pressure. (at time t)

এই additional load নিতে হবে, clay এর permiability অনেক কম, so H_2O সরতে পারবে না & additional load টা H_2O নিবে, it will produce excess pore H_2O pressure. ↳(if the H_2O is fully saturated)

$\bar{p}_A = 5 \times 120 + (130 - 62.5) \times 7.5$ = Existing overburden pressure

point A তে pore H_2O pressure (when there is no building) $= 7.5 \times \gamma_w$

in excess of $7.5 \times \gamma_w$ Δp will develop.

Though permiability কম,

আস্তে আস্তে void space থেকে H_2O বের হয়ে যাবে, Δp decipit করবে & তখন excess Δp will convert to effective stress.

যে পরিমাণ p. decipit করবে ততটা

effective stress is taken by soil skeleton.

if t = ∞ then সব H_2O বের হয়ে যাবে & তবে Δp decipit করবে & effective stress তত বাড়বে।
(so its time dependent)

100 years পর (suppose) effective stress $= \bar{p}_A + \Delta p$

H_2O বের হলে void কমে যাবে so soil squeeze হবে
& this will induce settlement of structure.

⊛ Sand হলে permiability বেশি, so H_2O আস্তে আস্তে বের হবে and the settlement will be instantaneous.

(so for clay settlement imp cause instantly বোঝা যায় না)

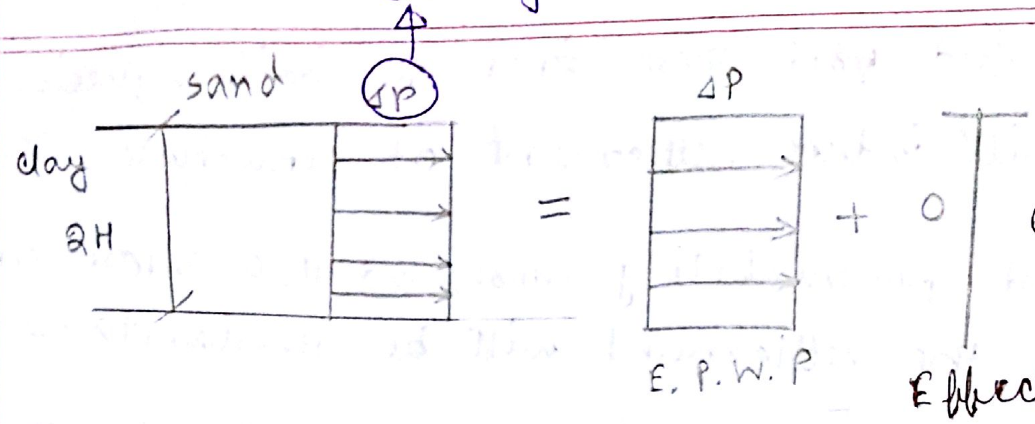
by mechanical force
→ compaction এ air reduce করি → partially saturated হলেও only air ejected
consolidation এ H_2O eject ”
natural process — partially saturated হলে H_2O + air both ejected

⊛ we will learn 1) কতটুকু settlement
2) settlement টা হতে কি পরিমাণ time লাগবে

The process of consolidation is a time dependent phenomena. It is associated with <u>excess pore</u> water pressure. The process of gradual load transfer from pore water to soil skeleton and corresponding gradual compression is called consolidation.

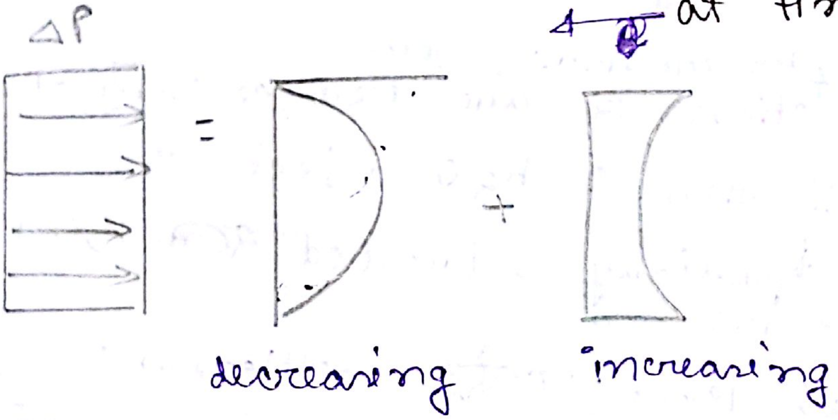
consolidation characteristics বের করার জন্য Lab এ used apparatus Oedometer / consolidometer.

coming from superstructure

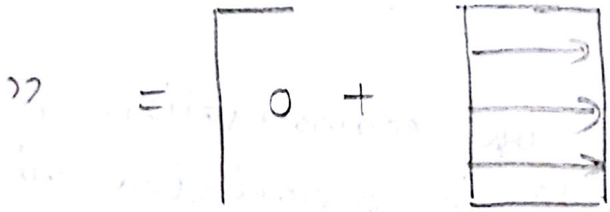


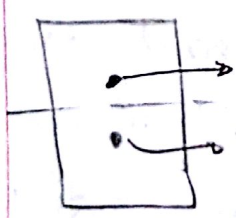
at starting time

at time $t = t_1$



At $t = \infty$





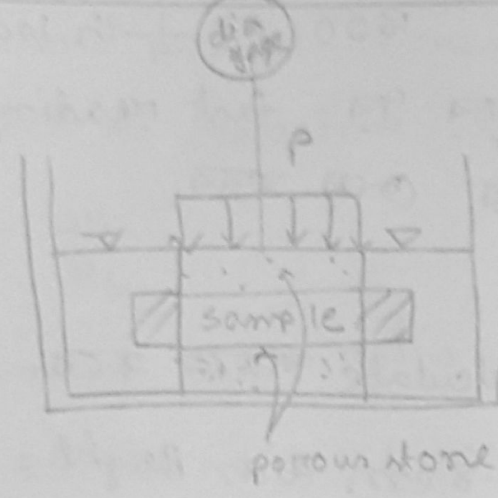
এই point এ উপরে যাবে

এই point এ void থেকে H_2O নিচে যাবে

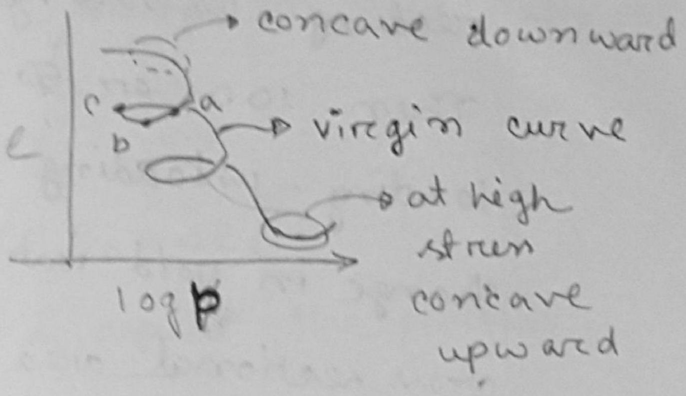
cause least resistance এর path এ travel করবে, so shape is D cause * এ point এ H_2O easily বের হবে & R.p.w.p easily কমবে.

10.11.15
Tuesday

Lec - 13



adometer



consolidation হয় expersion of water from void space.

Assumption:

1) soil is saturated fully
2) volm change in neglected for H_2O & void

যতক্ষন পর্যন্ত p load effective stress create না করবে ততক্ষন dia reading নেব।

25, 50, 100, 200, 400, 800, 1600 kPa

Foundation এর উপর সে over

15s, 30s, 1m, 2m, 4m, 8m, 15m, 30m, 1hr, 2hr, 4, 8 24 hr

এভাবে প্রত্যেক load দিয়ে 24hr reading নিব।
যখন dial reading change হবে না তখন all 25 kPa converted to Effective stress.

Then void ratio calculate করতে হবে।

So 7 days লাগল. Then 1600 এর $\frac{1}{4}$th load দিয়ে reading. unloading করার পর fast reading দেয়. Then 100 or 50 দিয়ে শেষ করব.

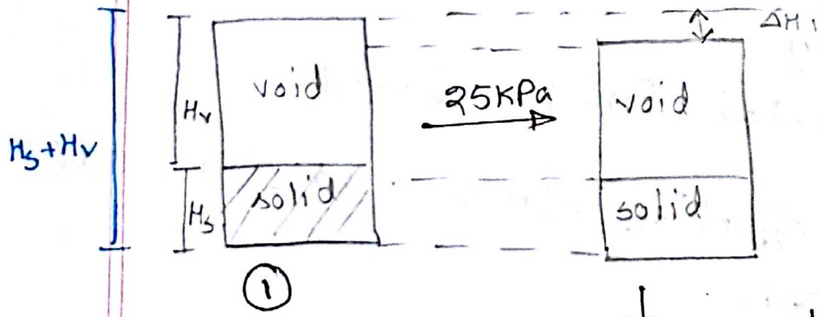
loading - unloading

change in void ratio calculate করতে হবে.

cross sectional area 6-8 cm dia, Height = $\frac{3}{4}$" to 1"

so area change হবে না, just height of soil sample changes.

24 hr এ যত কমল height (যদি ASTM এর negligible limit এ পড়লে stop reading)



Over dry → dry soil এর weight এর

⊛ 1st এ soil এর ① এর G_s নির্ণয় করে নিতে হয়

⊛ Laboratory তে 1st এই moisture content test করা হয়.

①⊛ $H_s = \frac{W_s}{A G_s \gamma_w}$ → out of dry soil

② $H_v = H - H_s$

③ $e_0 = \frac{H_v}{H_s}$ / $\underbrace{e_0 = wG_s}_{\text{less precise}}$

load apply করার আগে initial void ratio e_0

if I know moisture content then

④ At the end of 1st incremental load, when 25 is converted to effective stress change in void ratio,

$$\Delta e_1 = \frac{\Delta H_1}{H_s}$$

⑤ The current void ratio $e_1 = e_0 - \Delta e_1$

⑥ Again 25 দিব so total 50KPa & repeat the process

and $e_2 = e_1 - \Delta e_2$

Then plot the void ratios in graph e-logp curve.

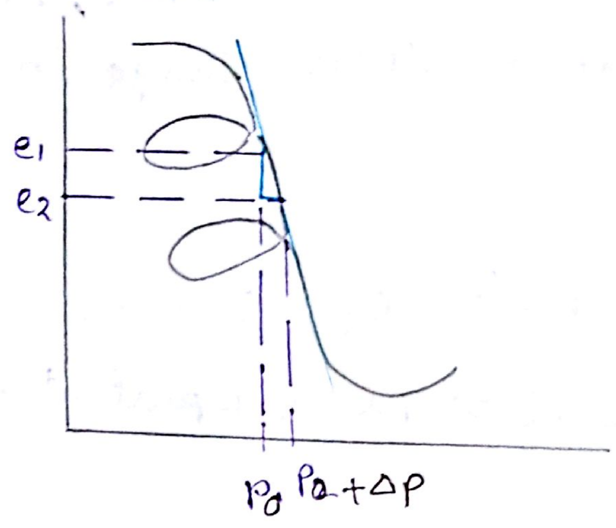
curve shape হয় concave downward

abc কে expansion /swelling / unloading curve

c থেকে again reloading or recompression. As soon as it exceeds the <u>past max^m pressure</u> (a) the curve will follow the same straight line.

এই st. line কে বলে virgin portion, slope of this curve is called <u>compression index</u> (C_c)

At a very high stress level the curve becomes concave upward



$$C_c = \frac{e_1 - e_2}{\log(P_o + \Delta P) - (\log P_o)}$$

$$= \frac{\Delta e}{\log \frac{P_o + \Delta P}{P_o}}$$

Depending on the state of stress soil can be classified :

1) Normally consolidate (N.C) clay

If the current effective overburden pressure is the max^m. pressure to which the soil is subjected during its lifetime (stress history) is called ~.

2) Over consolidated clay :

If the current overburden pressure is less than what the soil has been subjected to. in the past the soil is called ~.

And the past max^m pressure is called pre-consolidation pressure denoted by p_c.

current pressure p_o

The ratio of p_c/p_o is called over-consolidation ratio.

$p_c/p_o > 1$, OC clay

$p_c/p_o < 1$, N.C "

The greater the value of OCR the degrees of over consolidation would be higher.

At point ⓑ its OC এর past max^m pressure a.

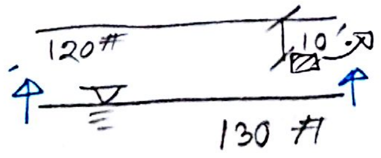
Straight line বরাবর থাকলে N.C.

N.C, O.C types of clay না, এরা stress state বোঝায়।

Recent earthfilling এ OCR = 1, মোট এখনও consolidation হয় নাই,

O.C : building ভেঙ্গে ফেললে এর নিচের clay, পাহাড়ি এলাকায় they are very stiff in nature, settlement very low.

Delta region এ N.C, Dhaka এও N.C.



এখানে stress = 120×10

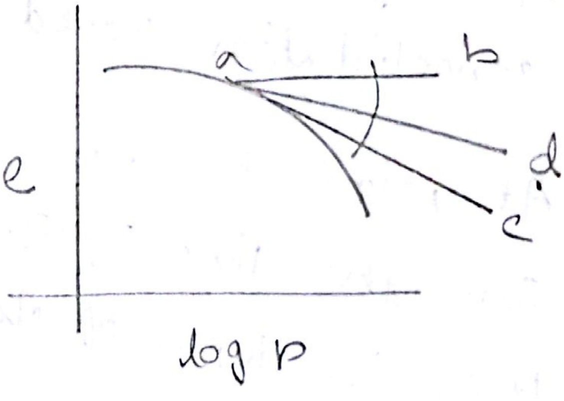
water table rise করলে stress = (130 - 62.5) ×10

Downward movement হতে থাকলে seepage pressure
$\gamma z' + i z \gamma_w$.

Earthquake হয়ে গেলে crack of ground, so stress released, so O.C.

downward movement বন্ধ হলে seepage pressure কমে যাবে, so $\gamma z'$ হবে, so O.C হবে।

Minimum radius of curvature is the max^m radius

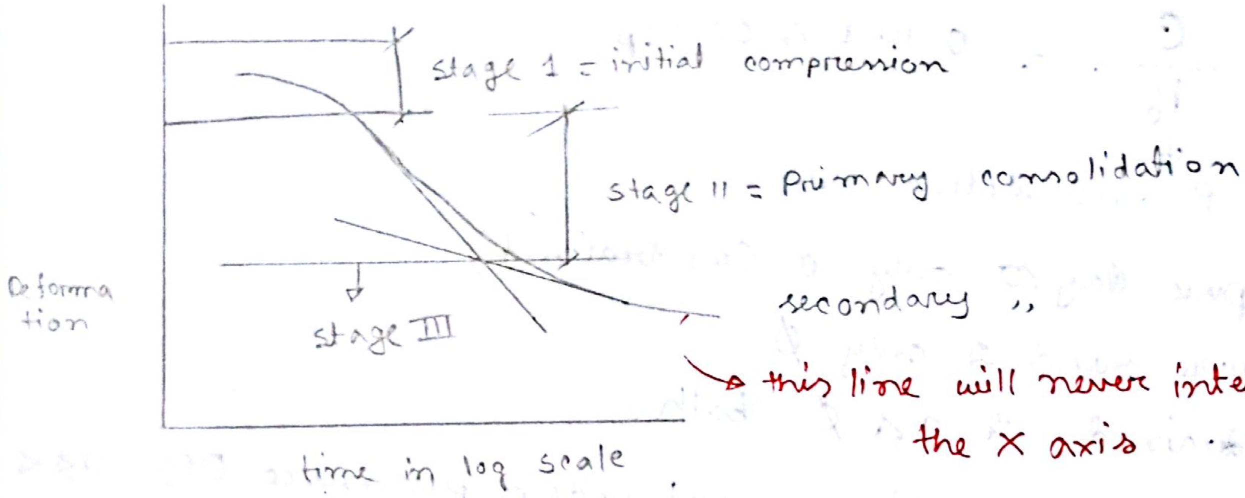


① By visual inspection slect a point on the curve which gives min radius of curvature
② Draw a line parallel by $\bar{p}$

③ Draw a tangent at ac
Bisect the angle dac
Draw a tanget at st. portion and extend it
e এর corresponding pressure হল p consolidation pressure.

15.11.15
Sunday

Lec-14



this line will never intersect the X axis

stage I এ compression হয় for elastic soil particle and water. Load দেয়া হয়।

stage-II এর contribution max^m. It is associated with decipition of access porce water pressure. This decipition continue if $t \to \infty$

যদি excess PH_2O Pre. completely decipite করে, এখানে 15, 10m এমন thickness এর জন্য হতে হলে 100s of year লাগবে।
এর পরেও দেখা যায় settlement হচ্ছে, যার due to plastic deorientation to the particle (~~stag~~ secondary consoli)

⊞ How to determine P consolidation Pressure :

liquidity Index

OC → 0.6 — 1.6

NC → 0.0 — 0.6

Lab test থেকেও বলা যায় OC and NC কোনটা

→ expressed in %

$$\frac{c}{p_c} = 0.10 + 0.004 I_p$$

↓

p consolidation Pressure

pure clay তে only c (undrained)

pure sand এ only ø

drained এ c & ø both

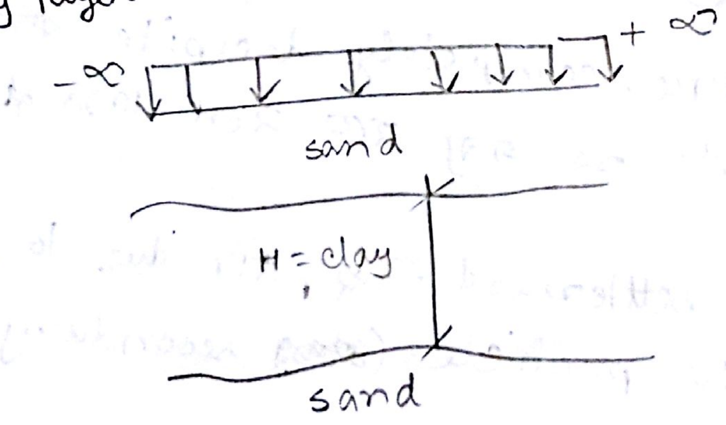
soil এর existing overburden pressure বের করব,

P_c এর value বের করে,

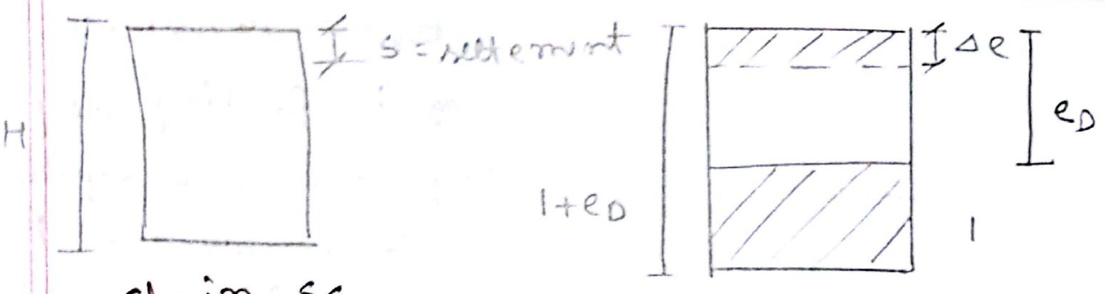
Then বলব NC / OC দেখব (see book → confused)

How to determine settlement : (magnitude of settlement)

clay layer এ settlement computation



Basic assumption clay layer is subjected to uniformly distributed strain across its thickness



strain $\frac{\Delta e}{1+e_D}$

strain S/H

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

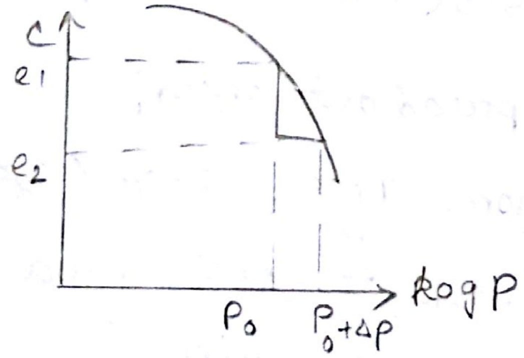
e_o = load application এর আগে void ratio

$$\therefore S = \frac{\Delta e}{1+e_o} H \quad \text{---(1)}$$

5 year আগে building বানালাম. Then আবার একটা building. so তখন e_0 হবে e_5.

$$e_5 = e_o - \Delta e$$

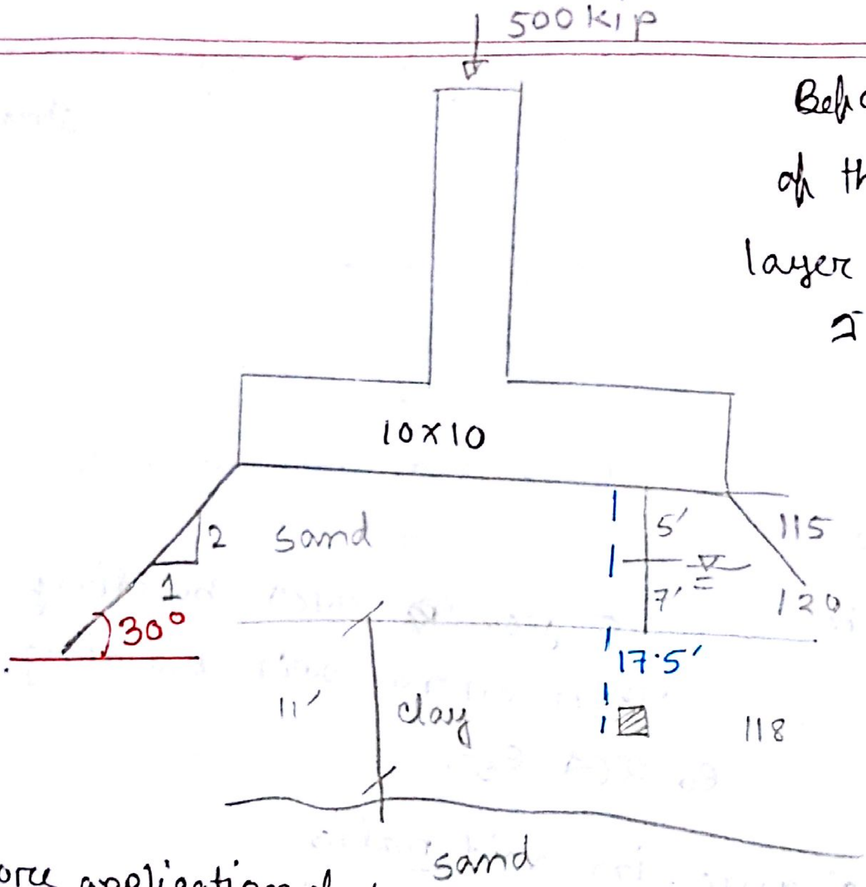
To measure change in void ratio



$$C_c = \frac{e_1 - e_2}{\log(P_o + \Delta P) - \log P_o}$$

$$= \frac{\Delta e}{\log\left(\frac{P_o + \Delta P}{P_o}\right)}$$

$$\therefore \Delta e = C_c \log\left(\frac{P_o + \Delta P}{P_o}\right)$$ putting it in ① we get settlement



Before construction of the footing clay layer এর mid depth এ overburden P কত?

Before application of load

$P_0 = 5\times 115 + 7(120-62.4) + 5.5(118-62.4) = 1{,}284$ Ksf

Load vertically আসবে না, spread out করবে।

load vertically 17.5' & hor. $\frac{17.5}{2} = 8.75'$ ছড়াবে।

এটা একটা dimension এ, 3D তে একই area

∴ ~~17.5+8.75 ≈ 27'×27' নিবে 500 kip~~ $(17.5+10)^2$ 8.75×2

$\Delta p = \frac{500}{(27\cancel{\times 27}.5)} \approx .66$ Ksf

11×12''

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

$\Delta P \rightarrow 0.66$

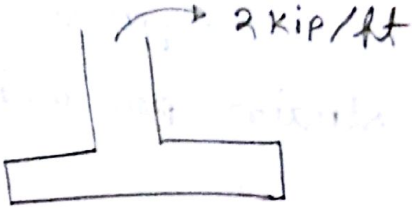
$P_0 = 1.284$

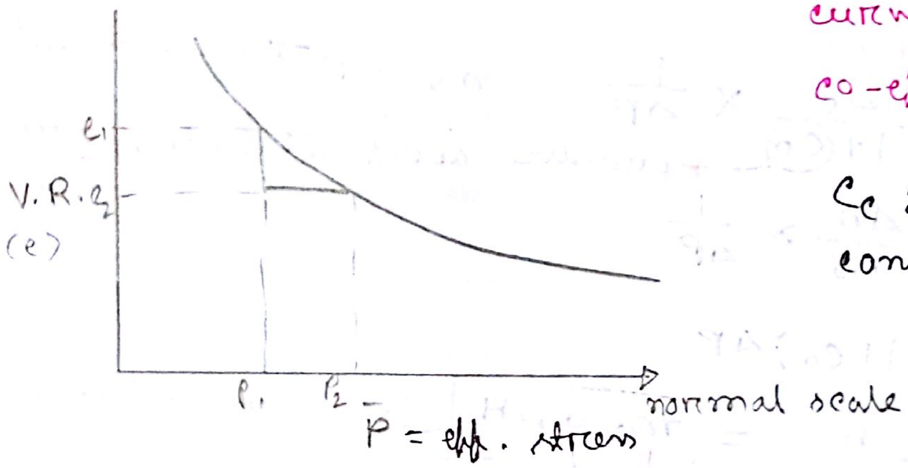
s max^m হতে পারবে 1", যদি s > 1 হয় then ΔP কমাতে হবে।

⊞ contineous footing:

10x10 এ 2kip ft

27x27 এ কত





curve এর slope হল co-eff of compressibility

C_c এর value remains constant in the curve

$\Delta e = e_1 - e_2$

$\Delta p = P_2 - P_1$

$a_v = \frac{\Delta e}{\Delta p}$ = it changes $\bar{p}\uparrow$ $a_v\downarrow$. নিচে $p\uparrow$ হলে a_v rapidly decreases,

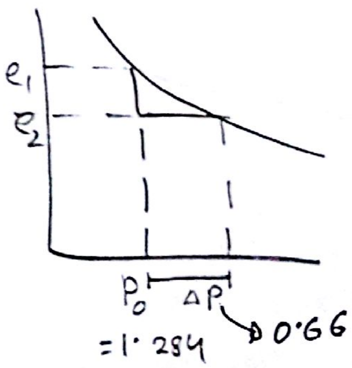
(fun of state of stress, unit হল inverse of p)

$\therefore \Delta e = a_v \Delta p$

using this curve $P_0 = 1.284$

$$\therefore S = \boxed{\frac{a_v \Delta p}{1+e_1} H}$$ → in terms of a_v

↳ এখানে e_0 নিয়ে e_1



17 **Co-eff of vol^m compressibility :**

Volmetric strain per unit increase in effective stress.

$$m_v = \underbrace{\frac{\Delta V}{V_0}}_{\text{vol}^m \text{ strain}} \times \underbrace{\frac{1}{\Delta p}}_{\text{per unit increase in eff stress}}$$

$$= \frac{\Delta e}{1+e_0} \times \frac{1}{\Delta p}$$

$m_v = 10^{-4}$ to $10^{-3}/m^2$

e_0 → pressure apply এর আগ মুহূর্তে যে void ratio

$$= \frac{\Delta H}{H_0} \times \frac{1}{\Delta P}$$

$$\Delta e = m_v (1+e_0) \Delta P$$

$$\therefore S = \frac{\Delta e}{1+e_0} H = \boxed{m_v \Delta p H}$$ in terms of m_v

⇓

function of current effective stress

17.11.15
Tuesday

Lec-15

ক] Rate of settlement :

⊛ One-D consolidation Theory :

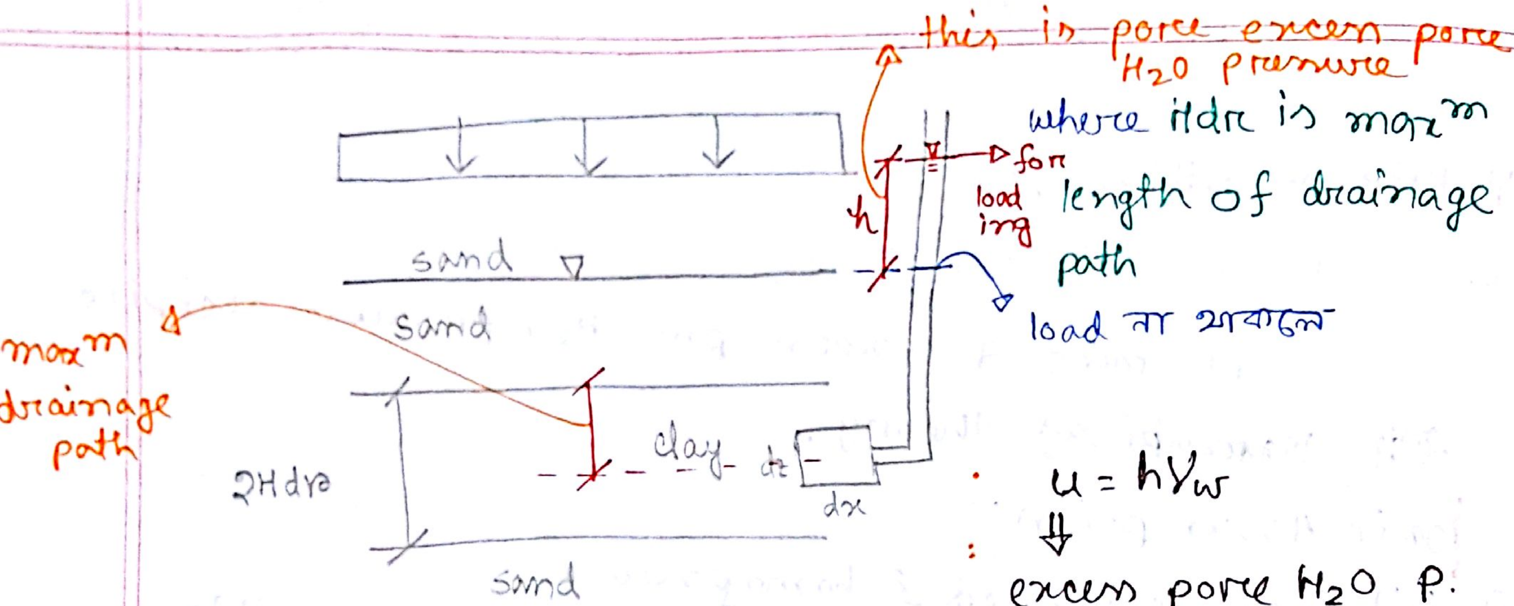
কি rate এ excess pore H_2O pressure decipite হচ্ছে, Kanzaki এর theory.

Basic Assumption :

1) Soil fully saturated & homoganeous
2) H_2O particle & soil skeleton are incomprissible
3) Flow is one directional (lateral flow হবে না)
4) Darcy's principal applicable + k remains constant accross the thickness of clay layer.
5) Thickness of clay layer is small in comparison with loaded area.
6) There is a uniqu relation that exist bet^n void ratio and effective stress. & the relationship remains constant during the load impliment.

Means a_v and m_v এর relation constant

$\frac{K}{\gamma_w m_v}$ constant থাকলে বাকিগুলোও constant থাকবে

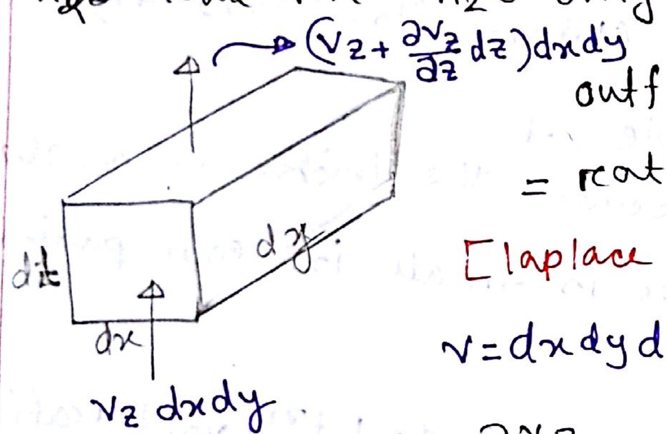


$u = h\gamma_w$

$\Downarrow$

excess pore H_2O P.

load দিলে H_2O only উপরে যাবে as 1-D flow.



outflow rate − inflow rate

= rate of vol^m change

[laplace এ out − in = 0 ছিল]

$v = dxdydz$

$$\therefore \frac{\partial v_z}{\partial z} dxdydz = \frac{\partial V}{\partial t}$$

$$-\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} dxdydz = \frac{\partial V}{\partial t}$$

$$V_t = -ki$$

$$= -k\frac{\partial h}{\partial z}$$

$$= -\frac{k}{\gamma_w}\frac{\partial u}{\partial t}$$

$$\Rightarrow -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} dxdydz = \frac{dxdydz}{1+e_0} \times \frac{\partial e}{\partial t}$$

$\Rightarrow$ since $dxdydz \neq 0$

$$\therefore -k/\gamma_w \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e_0}\frac{\partial e}{\partial t}$$

$$V = V_s + V_v = V_s + eV_s$$

$$\frac{\partial V}{\partial t} = V_s \frac{\partial e}{\partial t}$$

$$\frac{V_s}{V} = \frac{1}{1+e_0}$$

$$\frac{\partial V}{\partial t} = \frac{(V)}{1+e_0}\frac{\partial e}{\partial t}$$

$$\frac{\partial e}{\partial \bar{p}} = a_v$$

$$\therefore \partial e = \partial \bar{p} \, a_v$$

$$\tilde{p} = \bar{p} + u \qquad \partial e = -\partial u \, a_v$$

$$0 = \partial \bar{p} + \partial u$$

$$\therefore \partial \bar{p} = -\partial u$$

$$\therefore -\left(\frac{k}{\gamma_w}\right)\frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e_0}\frac{\partial e}{\partial t}$$

$$= -\frac{\partial u}{\partial t} \times \frac{a_v}{1+e_0}$$

$$\therefore \frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = \frac{a_v \, \partial u}{(1+e_0)\,\partial t}$$

$$\therefore \frac{\partial u}{\partial t} = \underbrace{\frac{k}{\gamma_w m_v}}_{\Downarrow} \frac{\partial^2 u}{\partial z^2} \text{ —— (iii)}$$

c_v = coeff of consolidation (This will be determined in the lab).

$$\therefore \frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

$$\boxed{k = \underset{\Downarrow}{c_v}\, \underset{\Downarrow}{m_v}\, \gamma_w}$$ —— from (iii)

determine in lab

boundary condition/ boundary displacement type of boundary condition based on geometry. দুই parameter দুটোকে boundary condiⁿ satisfy করতে হবে (at time t = something)

prob: Previous Fig:

BC geometric

$z = 0$ $u = 0$ $t = 0$ $u = u_0$

$z = 2H_{dr}$ $u = 0$ suppose $t = 0$ তে 10 kip load দিলাম

$\therefore t = 0$ $u = 10$ kip

Based on this BC:

$$u = \sum_{m=0}^{\infty} \left[\frac{2u_0}{M} \sin\left(\frac{Mz}{H_{dr}}\right) \right] e^{-M^2 T_V}$$

$M = \frac{\pi}{2}(2m+1)$

⊛ For both way drainage H_{dr} will be ½ of the thickness of clay layer (উপরে & নিচে sand)

⊛ For one way drainage total length of the drainage path will be thickness of clay layer. (suppose উপরে sand নিচে rock)

T_V = time factor

= non dimensional parameter

$= \frac{C_v \times t}{H_{dp}^2}$ = time factor

u_0 = Initial E.P.W.P

⊛ Degrees of consolidation is equal to any point

capital U → $U_z = \frac{u_0 - \boxed{u_z}}{u_0}$ → at any time E.P.W.P

$u_0 - u_z$ = decipited E.P.W.P

$$U_z = \frac{\text{desipited E.P.W.P}}{\text{Initial P.W.P}}$$

অনেকদিন পর $u_z = 0$ হয় then $U_z = \frac{u_0 - 0}{u_0} = 1$
↑ at infinity time

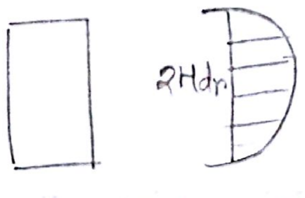
∴ 1 হলো 100% consolidation.

$u_0 = 5$ Ksf 24hr পর $u_z = 3$ Ksf

∴ $u_0 - u_z = 5 - 3 = 2$ Ksf

∴ consolidation $= 2/5 = 40\%$.

100% consolidation হলে যা settlement হওয়া বাকি হয়ে গেছে. This is at a particular point. But we want avg value so layer by layer (E.P.W.P) integreate করে layer thickness দিয়ে ভাগ করলে avg value.



Degrees of consolidation at point z, $U_z = 1 - \frac{u_0}{u_z}$

Avg degrees of consolidation at any point,

$$U = 1 - \cancel{\int_0^{2H_{dr}} u_z \, dz \left(\frac{1}{2H_{dr}}\right) / u_0} \; \frac{\left(\frac{1}{2H_{dr}}\right)\int_0^{2H_{dr}} u_z \, dz}{u_0}$$

U এর সাথে settlement directly proportional

$$\therefore U = \frac{S_c}{S_f}$$
S_c → current settlement
S_f → final " (using $a_v/c_v/m_v$)

24.11.15
Tuesday

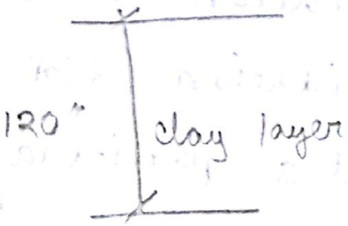
Lec-17

For a given soil ~~in~~ under similar loading condition C_v remains constant.

C_v is a func. of LL. Its value decreases with the increase in LL value.

$$T_v = \frac{C_v \times t}{H_{dr}^2}$$

constant



so t varies and T_v varies

$$\therefore T_{v_1} = \frac{C_v \times t_1}{H_{dr}^2}$$

$$\therefore \frac{T_{v_1}}{T_{v_2}} = \frac{t_1}{t_2}$$

Problem An undisturbed sample of clay statum with 2m thickness was tested in the lab and C_v was determined as 2×10^{-4} cm²/s. If a structure is built on clay how long will it take to attain half of its ultimate settlement under the load of structure.

Assume double drainager cond.

↓

ওপরে 4 নিচে sand

2 way drainage $\therefore H = 2/2 = 1 m = 100 cm$

50% consolidation $T_v = \cdot 197$ $\therefore t = \frac{H_{dn}^2 \times T_v}{C_v}$

$$= \frac{(100)^2 \times 0 \cdot 197}{2 \times 10^{-4}}$$

$$=$$

Q 3m thick clay layer beneath a building is overlaying by a permeable statum and is underlaine by a impervious rock. The co-eff of consolidation of clay layer was 0'025 cm^2/min, The final expected settlement for the layer is 8 cm. How much time will take for 80% of the total settlement to take place. Determine t for a settlement of 2·5 cm to occur. compute the settlement to occur at the end of 1 year?

Ans./ $H = 3 m$ $T_v = 0 \cdot 025 \text{ cm}^2/\text{min}$

$U_{ult} = 8 cm$ $t_{80\%} = ?$

for $U > 60\%$ $T_v = 1 \cdot 781 - 0 \cdot 933 \log_{10} (100 - 80)$

$= 0 \cdot 567$

$$\therefore 0 \cdot 567 = \frac{0 \cdot 025 \text{ cm}^2/\text{min} \times t}{(300)^2}$$

$\Rightarrow t = 2041200$ min $= 3 \cdot 88$ year

2) Here $U_{ult} = 8$ cm

$U_{present} = 2.5$ cm

∴ Degrees of consolidation $= \frac{2.5}{8} \times 100 = 31.25\% < 60\%$

∴ $T_v = \pi/4 \left(\frac{31.25}{100}\right)^2 = 0.075$

∴ $\underset{2.5\text{ cm}}{t} = \frac{(300)^2 \times 0.075}{0.025} = 187.5$ day

$t_2 = \frac{t_1 \times T_{v2}}{T_{v1}}$

$= \frac{3.88 \times 0.075}{0.567}$

↑ in this

see the use of

$\frac{T_{v1}}{T_{v2}} = \frac{t_1}{t_2}$

3) compute the settlement in 1 year

$T_v = \frac{0.025 \times 1 \text{ year}}{(300)^2}$

$= \frac{0.025 \times 525600 \text{ min}}{(300)^2}$

$= 0.146 < T_{60}$

$\hookrightarrow \pi/4 \left(\frac{60}{100}\right)^2 = 0.286$

$=$ [illegible]

∴ ~~settlement in 1 year~~

∴ ~~$T_v = \pi/4 \left(\frac{14.6}{100}\right)^2 = 0.0167$~~

[if $T_v > 0.286$ then we had to use ← Here $T_v = 1.781 - - - -$]

Now use $T_v = \pi/4 \left(U/100\right)^2$

∴ $0.146 = \pi/4 \left(x/100\right)^2$

∴ $x = 43\%$

∴ $0.4311 \times 8 = 3.45$ cm

∴

Prob A building cons. on a compressible layer with double drainage settles by 80mm in 4 years time. What will be the settlement in 9 years in the final settlement is expected to be 300 mm. What will be the time required to settle 210 mm and what will be the settlement in 25 years.

Ans. 120mm, 28·8 yr, 198mm

a) $t = 4$ years $\quad U = 80\text{mm} \quad U_{ult} = 300\text{mm}$

২৭. 11.15

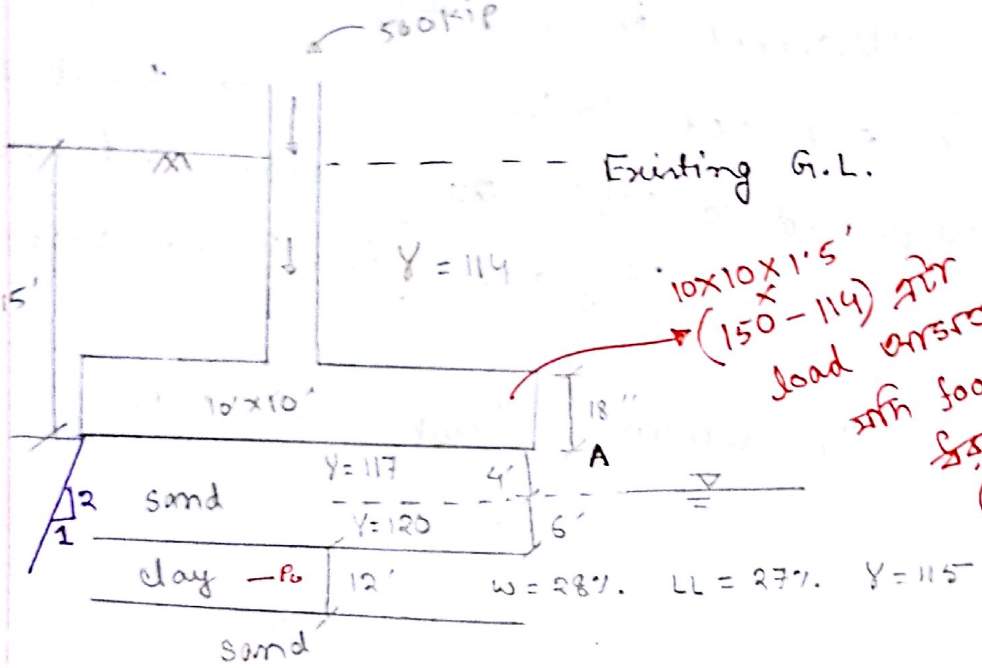
Saturday

Lec - 18

Art 3.3, 3.4, 3.5 ভালভাবে পড়ব।

prob

A building contructed on a compressible layer with



$10 \times 10 \times 1.5' \times (150-114)$ এর additional load আসত with 500 যদি footing এর weight ধরতাম। (This is more precise)

Excavation করলে soil turns into OC।

Past maxm pressure যতক্ষণ current maxm pressure exceed না করলে ~~[illegible]~~ ~~[illegible]~~ OC থাকবে।

⊛ C_s এর value C_c এর তুলনায় 10-15%.

$$P_o = 15 \times 114 + 117 \times 4 + (120-62.4)6 + \underbrace{6}_{\text{mid layer}}(115-62.4)$$

↓

mid layer এ Pressure (Pre consolidation pressure P_c)

↳ Before load application (Past maxm pressure)

$$= 2839.2\ psf = 2.84\ ksf$$

from A to mid layer 16'

$$\therefore \frac{500}{(10+16)^2} = 0.74 \text{ Ksf} = \Delta p$$

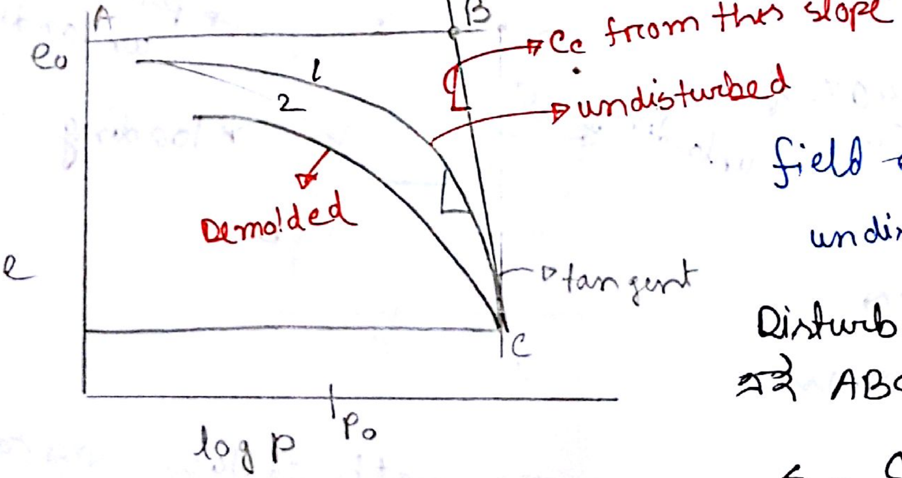
Excavate করলে 15' মাটি চলে যাবে

$\therefore 15 \times 114 = 1710$ psf $= 1.71$ Ksf (Excavate করলে এটা released)

so after excavation pressure $4 \times 117 + (120-62.4) \times 6$
(current pressure) $+ 6 \times (115-62.4)$
$= 1129.2$ psf $= 1.129$ Ksf

Now it is OC soil. (continued later)

⊞ Field e-log p curve: Smartmen method: (Art. 3.3)



field e-log p

field e-log p curve for undisturbed soil sample

Disturbance ছাড়া soil এ এই ABC curve থেকে C_c

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

settlement for NC soil

~~[illegible]~~ $e_0 = wG_s$, p_0 = existing over burden pressure.

select 40% void ratio of initial v. ratio

Cc বের করার emphirical formula

→ see book

$C_c = 0.007\,(LL-10)$ undisturbed soil sample .

$C_c = 0.009\,(LL-10)$ Remolded " "

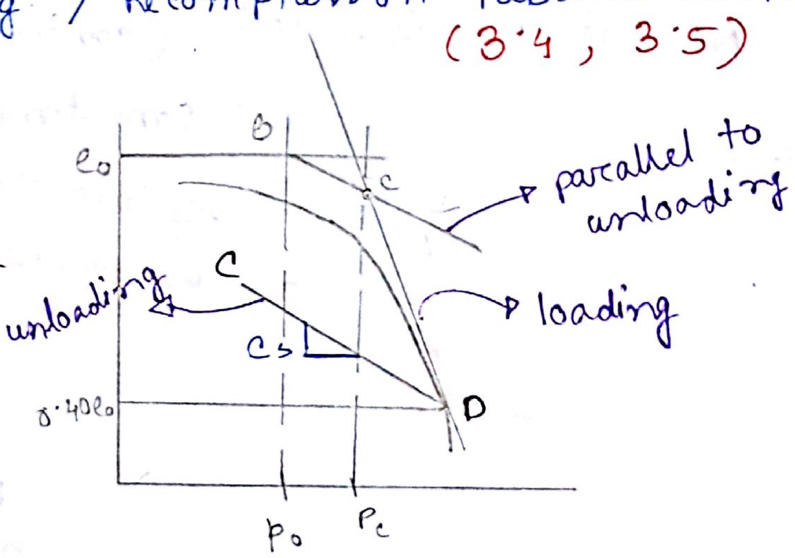
① এর slope বেশি, so $C_c \uparrow$, so $S\uparrow$.

Remolded করলে curve shifts to left, so slope↓,

$C_c\downarrow$, $S\downarrow$

☐ Loading - unloading / Recompression - rebound curve :
(3.4 , 3.5)

1) e_0 বের করব
2) P_0 বের করব
3) P_c " "
4) select point of 40% of e_0
5) Tangent at D
6) BC ∥ to DC



P_0 থেকে P_c গেলে soil OC, pc cross এর পর nc.

P_0 থেকে P_c তে যাবে কোন path follow করবে,

C_s = swell index = value না দিলে assume 10-20% of Cc. (15% is a good guess)

$$S_1 = \frac{HC_s}{1+e_0}\log\frac{p_0+\Delta p}{p_0} \qquad p_0+\Delta p \le p_c$$

within BC

$$S_2 = \frac{H\overset{\to 10\text{-}20\%\text{ of } C_c}{\boxed{C_s}}}{1+e_o} \log \frac{P_c}{P_o} + \frac{HC_c}{1+e_o} \log \frac{P_c+\Delta p}{P_c}$$

(for normally consolidated portion → after P_c)

continuation of math:

$C_c = 0·007\ (28-10) = 0·12$

$G_s = 2·67$

suppose $C_s = 10\%$ of $C_c = 0·012$

$e_o = wG_s$
$= 0·28 \times 2·67$

$$\therefore \text{settlement } S = \frac{C_sH}{1+e_o} \log \frac{P_c}{P_o} + \frac{HC_c}{1+e_o} \log \frac{P_c+\Delta P}{P_c}$$

[H use করব full height irrespective of drainage condition]
যদিও mid height এ pressure বের করছি.

$$= \frac{0·012 \times 144''}{1+e_o} \log \frac{2·84}{1·129} + \frac{0·12 \times 144}{1+e_o} \log \frac{2·84 + \overset{\overset{\Delta P}{\uparrow}}{\boxed{0·74}}}{2·84}$$

1.12.15
Tuesday

Lec - 19

Both way drainage হলে single way drainage এর 1/4 time লাগবে।

Rate analysis:

$$T_v = \frac{C_v \times t}{H_{dr}^2}$$: varies with drainage condition

For same soil under similar loading condition C_v remains constant.

$\therefore \frac{T_{v_1}}{T_{v_2}} = - - -$

⊞ 3-D consolidation

⊞ construction of time settlement curve during construction period.

Stress Distribution in Soil

why stress distn imp?

For sandy type soil 100% dominated by settlement criteria. So magnitude of settlement really imp.

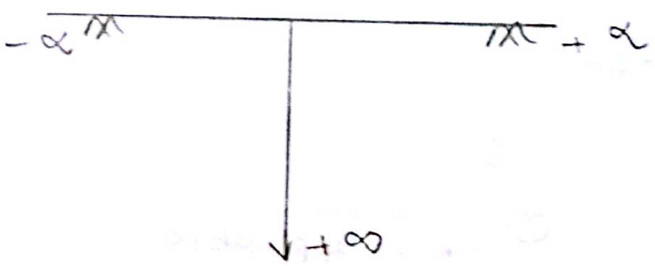
2:1 slope এ " বের করা হয়। But এটার precise না।
precise হতে হবে।

[learn no derivation]

☐ Basic Assum:

1) Soil homogeneous, isotropic
2) Its linearly elastic, obeys hook's law
3) Ignore self wt of soil

এ problem under consideration called Elastic half space "



Fully space problem at C.G of earth

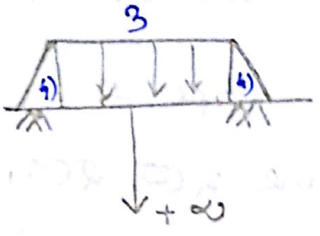
⊞ Loading under consideration:

1) concentrated / point load

2) line load : infitely long (+∞ — −∞)
(Brick walls load)

3) Finite width (infinitely long) (more precise than 2)

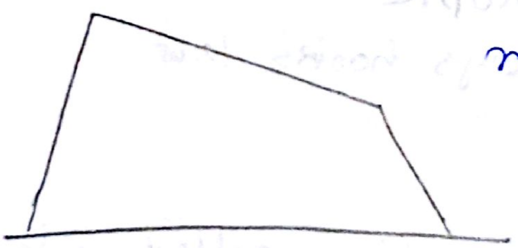
4) Triangular variation of load "



5) Rectangular (যত rectangular হবে
6) Circular efficiency তত কমবে)

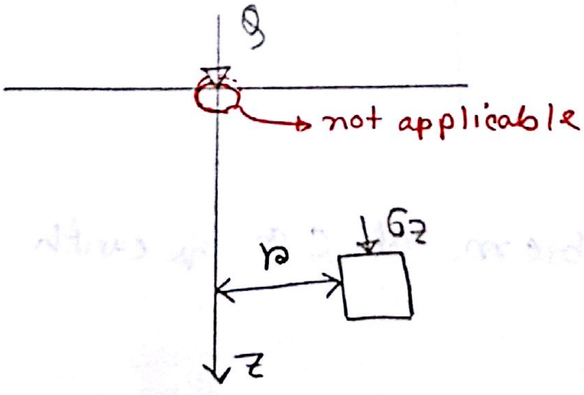
7) Any arbitary shape of loading (Graphical /approximate soln)

newmark chart method



⊛ সব soln based on Boussinesq's theory

1) Concentrated Load :



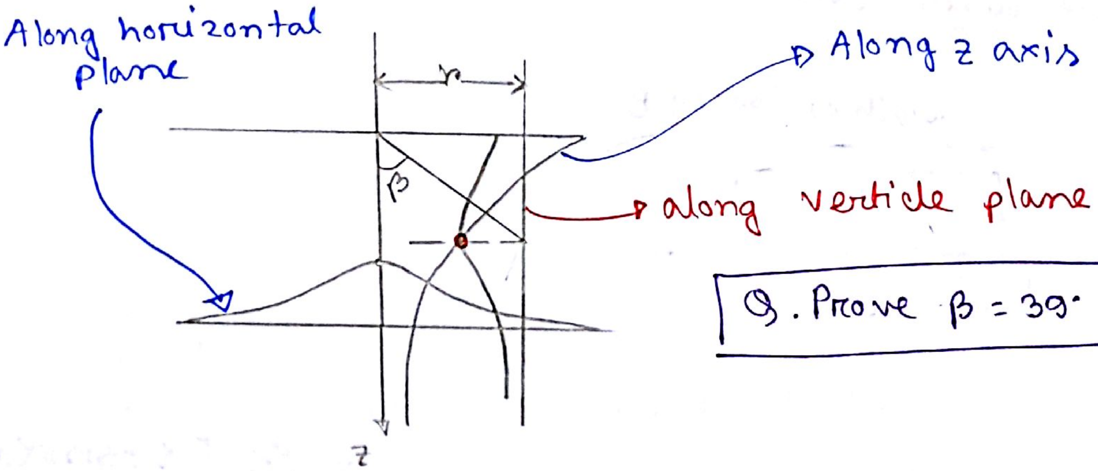
Q load, z নিচে যাবে, everything in 3-D space

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

(*) This soln is valid everywhere except to the neighbourhood of the point of application of applied load.

cause এ part এ stress goes to ~~zero~~ infinity, called singularity problem.

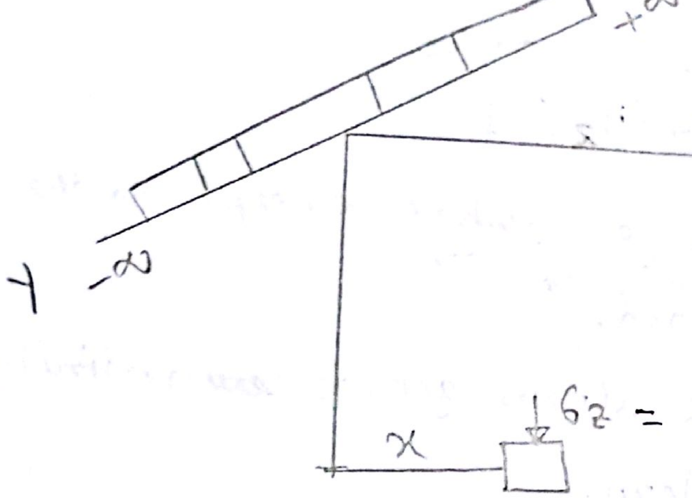
z যত উপরে peak তত বাড়বে।



Q. Prove β = 39° ...

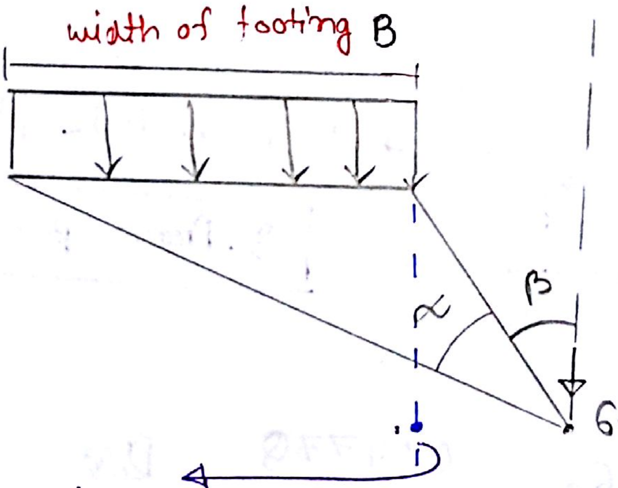
Along z axis $\sigma_z = \frac{0{\cdot}477Q}{z^2}$ =

2. Line Load:



$$\sigma_z = \frac{2q}{\pi} \frac{z^3}{z^2+x^2}$$

3. Finite width: (width finite, but length infinite)

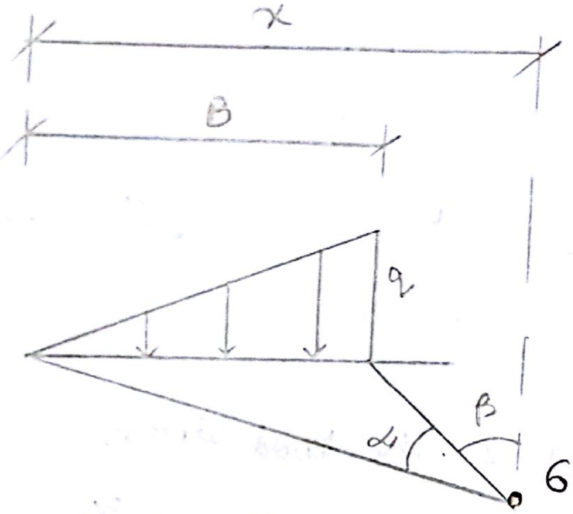


$$\sigma_z = \frac{q}{\pi} [\alpha + \sin\alpha \cos(\alpha+2\beta)]$$

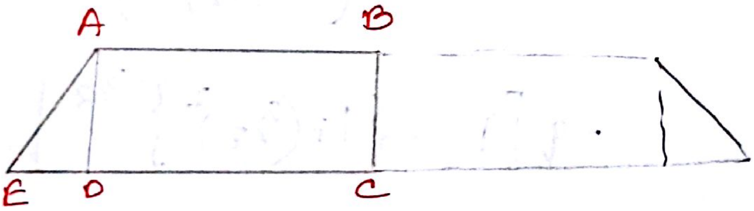
Along this line $\sigma_z = q/\pi \left[\alpha + \frac{\sin 2\alpha}{2} \cancel{\cos}\right]$

Triangular Loading :

Embankment, highway, any earth structure এর side slope loading



$$\sigma_z = \frac{q}{\pi}\left[\frac{x}{B}\alpha - \frac{1}{2}\sin 2\beta\right]$$



এমন embankment থাকলে take ABCE.

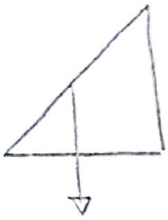
Then ΔAED & rectangle ABCD এ 2 types expression.

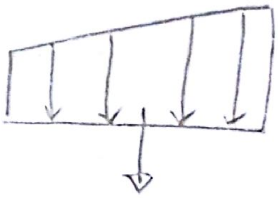
2টার result ~~multiply~~ (+) করব & multiply by 2 cause same loading right এও আছে।

5. 12. 15

Saturday

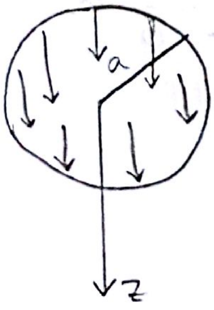
Lec - 20





previous class এর formulas use করে cg তে load

Circular Loading :



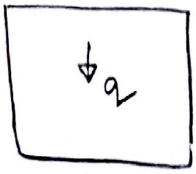
along z the ~~load~~ stress

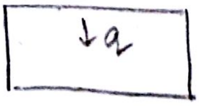
$$\sigma_z = q\left[1 - \left\{\frac{1}{1+(a/z)^2}\right\}^{3/2}\right]$$

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

Square loading / Rectangular

similar to footing. ~~q~~ uniformly loaded with magnitude q.

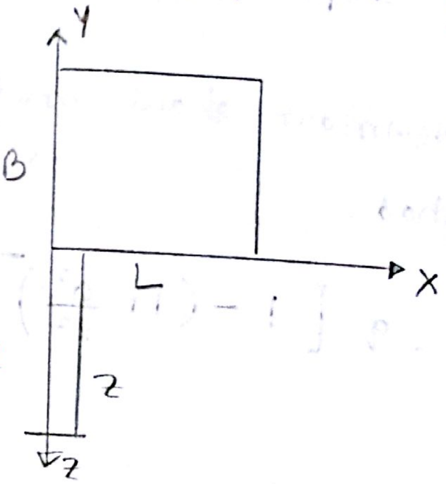




all the previous ones and this applicable to shallow foundation.

✱ Plan view :

এখানে any corner point এ increase in verticle stress calculate.



$B/z = m$
$L/z = n$
$\Big\}$ mutually interchange able

& assumping $m^2+n^2+1 > m^2n^2$

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1^2+m^2n^2}\right]$$

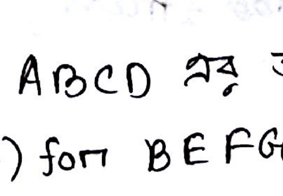
otherwise : if $m^2+n^2+1 < m^2n^2$

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \times \frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1^2+m^2n^2}\right]$$

$\downarrow$ Stress at a distance z below corner point

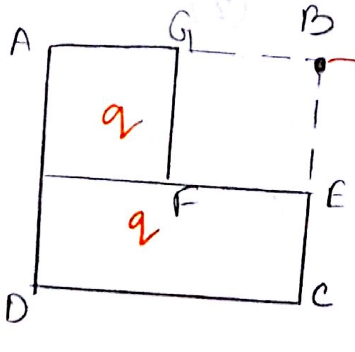
* Every thing is measured in radius

* Max^m stress at centre.



এভাবে 4টি square ধরে corner এ যোগ করব stress.

Or,



→ এই point এ stress

ABCD এর জন্য at B

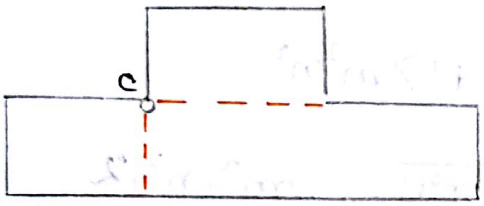
then (−) for BEFG at B.

so B তে পাব stress for AGFECD

⊞ Any shape subjected to any pressure :

Newmark chart method, graphical & approximate method.

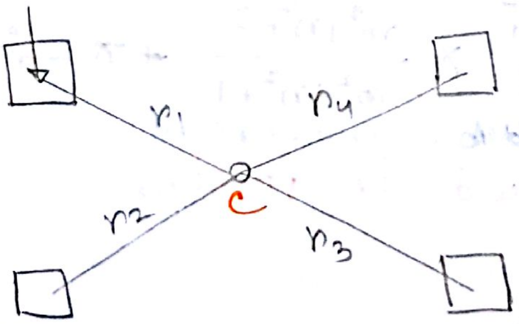
$$\sigma_z = q\left[1-\left(1+\frac{a^2}{z^2}\right)^{-3/2}\right]$$



এভাবে 3 sagments, add at point c

water tank এর 4টা leg, total verticle load জানি,

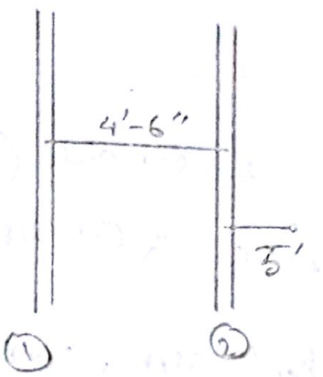
symmetric হলে $r_1 = r_2 =$



∴ এখানে concentrated load এর জন্য একটা block হিসাব করব, then r radius এ হিসাব করব, multiply it by 4, to find stress at c

\# Train, Excel এ 10 kip/ ft. এই rail (2) এর নিচে 5' depth এ too stress?

??

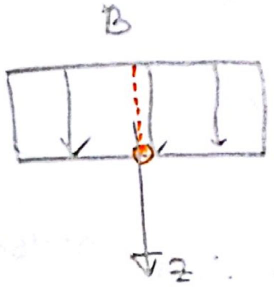


for rail ① $r = 4'6''$

" ② $r = 0$ as radius নাই।

(for intinitely long loading)

so ২টা (+) হবে



corner এ জন্য, center এ বের করতে হবে।
so ½ এ devide করে center এ আনব।

⊛ For any arbitary shape of loading :

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

q = surface load

σ_z = increase in v stress

$q > \sigma_z$ always

$$\Rightarrow \frac{\sigma_z}{q} = 1-\left(1+a^2/z^2\right)^{-3/2}$$

$$\Rightarrow -\frac{\sigma_z}{q}+1 = \left(1+a^2/z^2\right)^{-3/2}$$

$$\Rightarrow \frac{a^2}{z^2} = \sqrt[-2/3]{1-\sigma_z/q} - 1 \qquad \therefore a/z = \sqrt{\left(1-\sigma_z/q\right)^{-2/3}-1}$$

$\sigma_z/q = 0{\cdot}1,\ 0{\cdot}2,\ 0{\cdot}3,\ 0{\cdot}4,\ 0{\cdot}5,\ 0{\cdot}6,\ 0{\cdot}7,\ 0{\cdot}8,\ 0{\cdot}9$

এ সবগুলোর জন্য a/z এর value.

for 0.9 ⇒ $a/z = 1.9$

$a = 1.9\boxed{z}$ → depth under consideration.

এটা practically means radius of circle (a).

0.9 এর বেশি হলে radius infinite হয়ে যাবে,

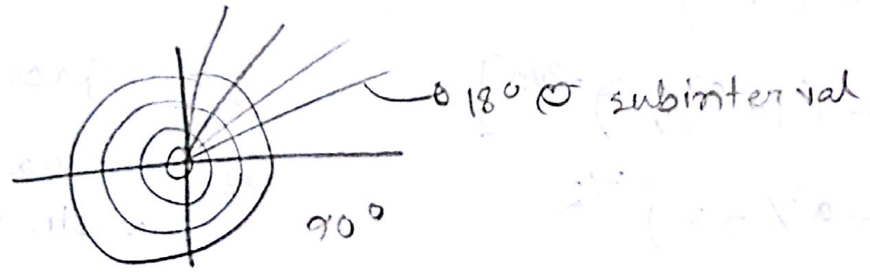
খাতায় left এ 1" বাদ, right এ 1/2" বাদ, খাতা 8".

∴ বাকি থাকে 6.5", so radius 3.25".

∴ $a = 1.9z$ ∴ $z = \frac{3.25''}{1.9} = 1.7105$,

Let $z = 1.75$, so scale $\underset{z}{\overset{1.75}{\longmapsto}}$

again for 0.5, $a/z = 0.76$ $z = 1.75$ (ধরলাম) ∴ $a = 0.76 \times 1.75$
$= 1.33$
↓
এই radius দিয়ে 5th circle আঁকব,



\# graph এ করতে define 2 parameters

z (আগে থেকে decide করব)

Influence factor — 1/200 (as 18° তে segment করলে 5টা segment, 10টা circle, so 90° তে 50টা)

increase in mag. of verticle stress = count the no of division under loading X I.F X the intesity of loading

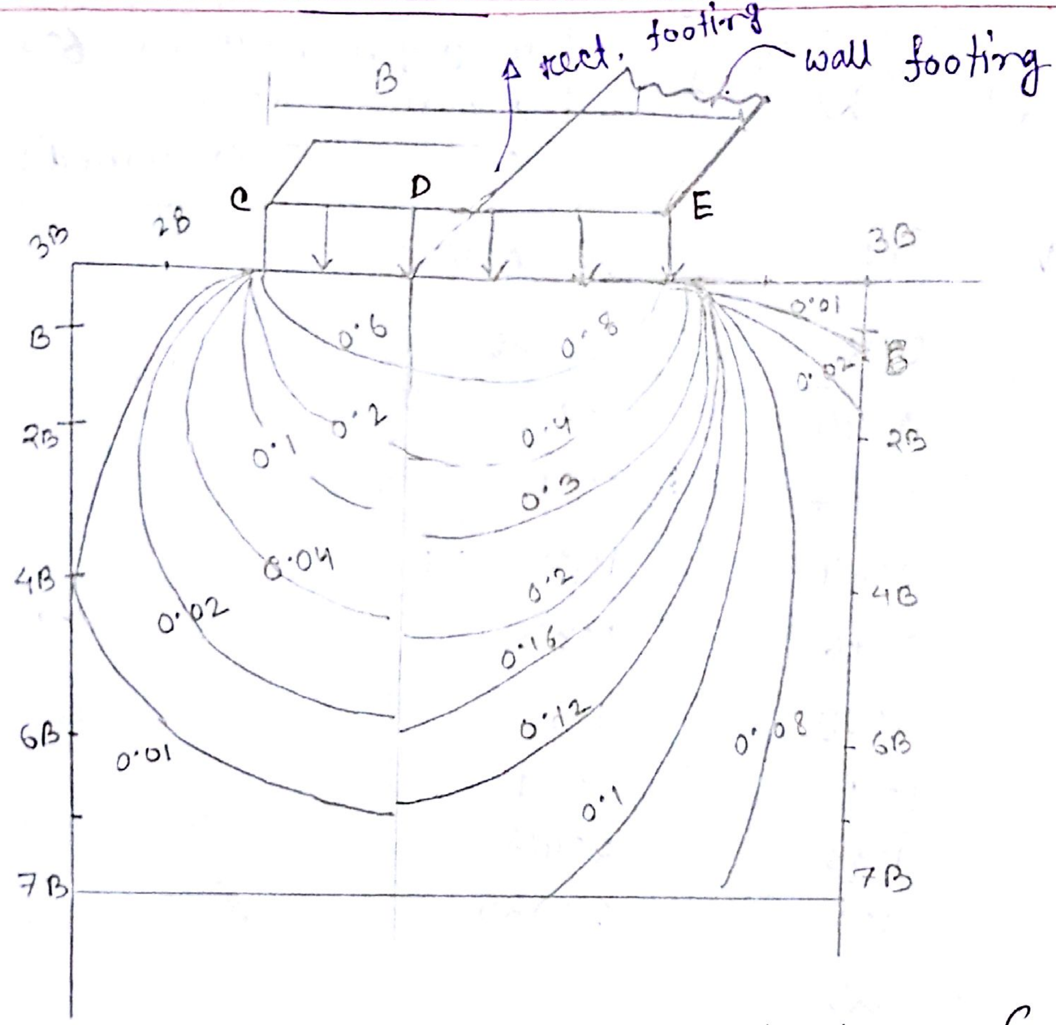
$a/z = \sqrt{(1-\sigma_z/q)^{-2/3} - 1}$

In graph max^m dia = 6·5" (assumed)
∴ radius = 3·25"
∴ z = 1·75" (assumed)

σ_z/q	a/z	a (radius)
0·1	0·27	0·4725
·2	0·40	0·7
·3	0·52	0·91
·4	0·64	1·12
·5	0·76	1·33
·6	0·92	1·61
·7	1·11	1·94
·8	1·38	2·415
·9	1·9	3·325
1	∞	

6.12.15
Sunday

Lec - ২1



rectangular footing এর c.g তে depth এ বিভাবেবে wall footing এর cg তে depth বরাবর বিভাবে stress vary করে।

DE হল wall footing, CD হল square, square এর জন্য B তে 60% & wall এর " B তে 80%.

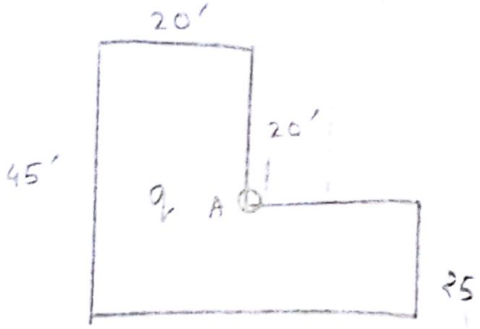
প্রতিটি line is called isobar, lines of equal pressure or equal stress.

so square এ load fast decipite করে।

Δp এর value যত বেশি settlement তত বেশি।

এখানে 60% & 80% হল Δp.

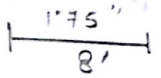
Z = depth under consideration



Z = 8'

∴ in the graph we took Z = 1.75"

∴ 1.75" = 8'



* ∵ 8' → 1.75'

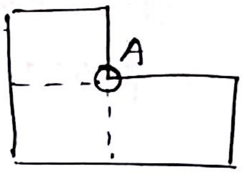
20' = 4.375"

25' = 5.468"

45' = 9.84375"

(*) $a = z \times 1.91$
↓
take such a value যেন a round হয়

যে plan আকালাম তার জন্য A point এ formula at corner point থেকে বের করতে হবে। Then check. (in terms of m & n)

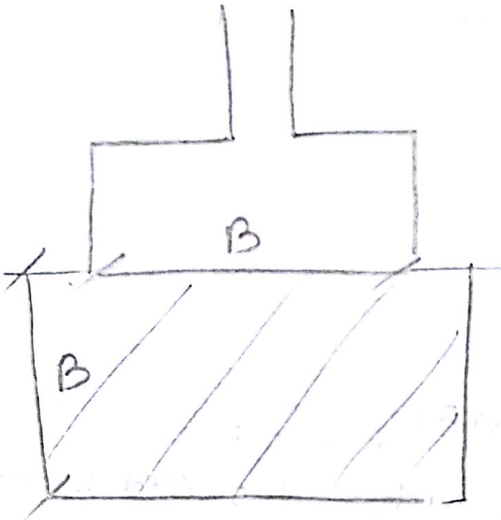


(*) plot these converted scale, count the number of divisions then $\frac{1}{200} \times \text{div} \times q = \sigma_z'$

ΔP calculate করব ততক্ষণ পর্যন্ত যতক্ষণ 80% decipite না করে।

suppose 2 Ksf load on footing, এর 20% remains মানে 0.4 Ksf হবে যতটা depth এ এটাকে বলে significant depth. এই depth পর্যন্তই আমরা settlement compute করি, or, significant stress zone.

Footing এর নিচে footing এর যে width তার depth নিয়ে concerned.



1) For square footing—
significant depth 2B

2) For long footing
S.D = 3B

⊛ or, go upto that depth until increase in stress for superimposed load is upto 20%. (Handout)

ক Contact Pressure Distribution:

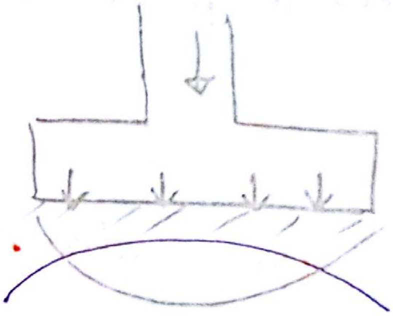
footing & soil interface এ superimposed load এর জন্য যে pressure.

~ is the pressure exerted by the foundation to the soil immidiately below it. It is the function of relative stiffness of soil & footing, soil properties, & soil type.

(rigid / flexible)

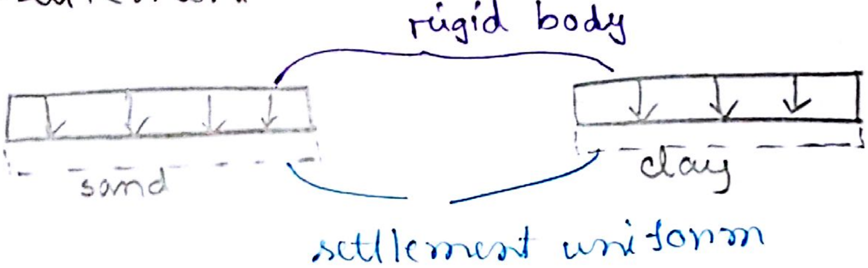
(soil এর e value, footing এর e & poisons ratio)

sandy / clayey

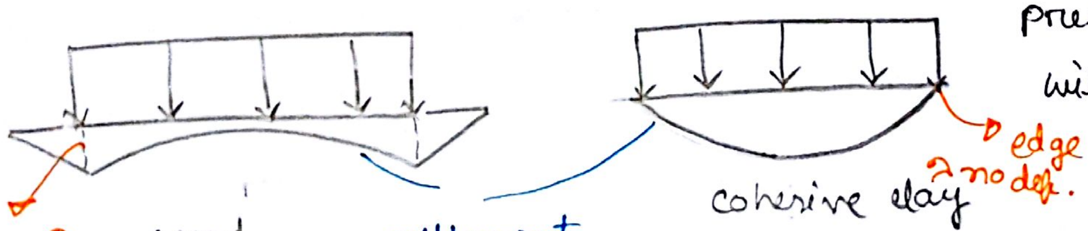


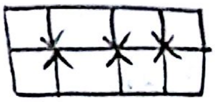
contact face এ stress distⁿ কেমন হবে জানতে চাই.

rigid body হলে, irrespective of soil type below footing the settlement will be uniform.



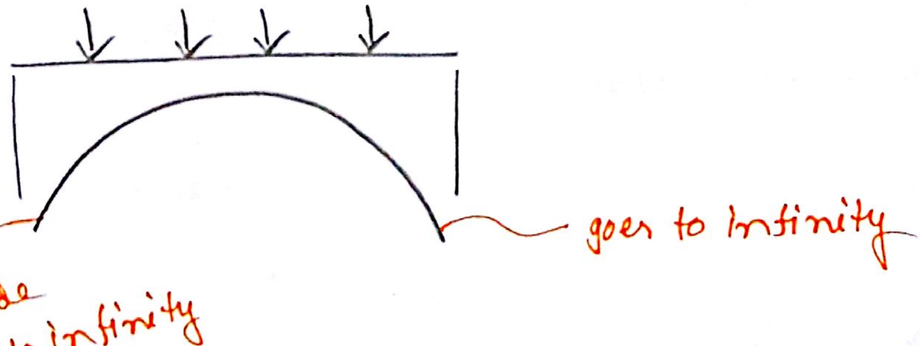
For flexible body resting on clay / sand the contact pressure distⁿ will be uniform



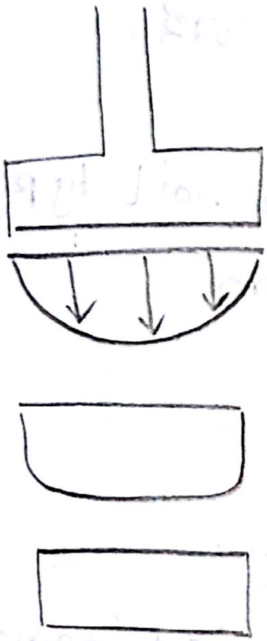


for both sand and clay

rigid body resting on clay ~ distⁿ



Rigid body on sand circular, যত deep এ যাব তত uniform হবে।



[see def^n from section C]

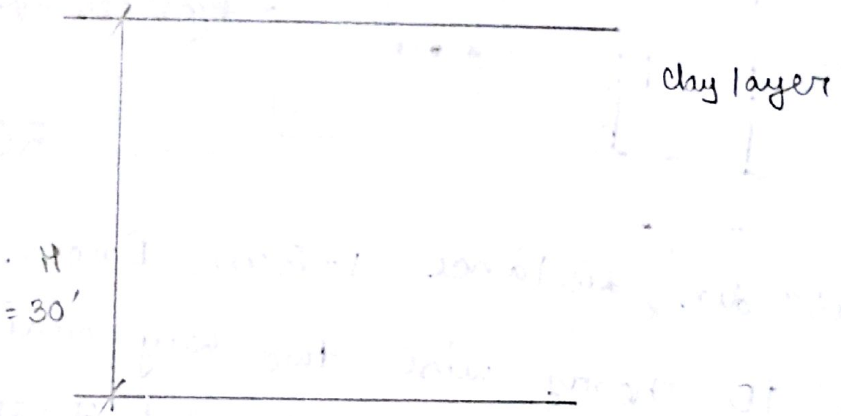
H_{dr}

8.12.15
Tuesday

Lec-22

⊞ Application of 3D consolidation theory:

settlement vertically & also P.W.P dissipate laterally.
so used co-ordinate system cylidrical (z, r)



suppose highway/embankment. এর 1Km highly compressible clay. Lab এ test করে দেখা গেল 30′ এর জন্য 30″ settle করল।

Highway তে piling করা যাবে না, So existing condition কে better করতে হবে।

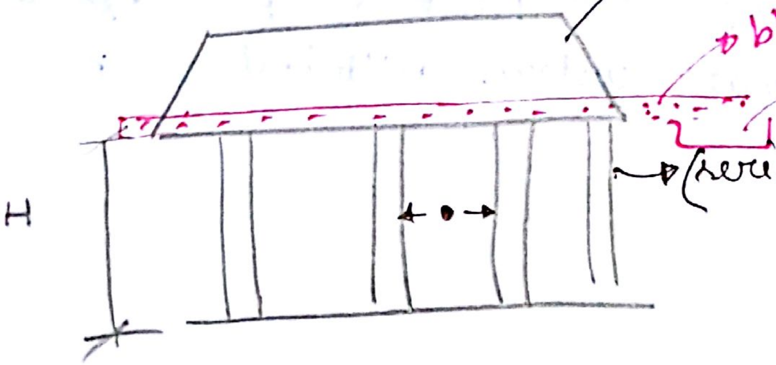
[illegible]

~~settlement~~ soil sample এর জন্য Lab এ 20mm এর হতে settlement 0.375″ settlement time (t_1), 30′ এর জন্য t_2

↓ 15′ = 180″ cause 2 way drainage

$t \propto H_{dr}^2$

$\therefore \frac{t_1}{t_2} = \frac{H_{dr_1}^2}{H_{dr_2}^2}$ $\Rightarrow \frac{t_1}{t_2} = \frac{0.\boxed{375^2}}{(180)^2}$?? $\Rightarrow t_2 = 631$ years so এই settlement acheive করতে এত time দেয়া হবে না।



(as clay এর so water যেতে পারত না, but blanket
(series of bore hole) দেয়াই
এটাই H_2O বের হয়ে যাবে।

12"–24" dia, distance 1–2 m, Excess P.W এর জন্য 1D Theory wise two way drainage এ দোদিকে least resistance যেদিকে যাবে। But এজন্য 180", but side এ চলে যাবে if bore holes.

load যত বেশি দিব settlement তত হবে। required settlement in required time acheive করতে load বেশি দিতে হবে।

⊛ Ch parameter:

$$T_v = \frac{C_v \times t}{H_{dr}} \quad \text{(vertically)}$$

$$T_R = \frac{C_h \times t}{2 \times (\underbrace{2R}_{\text{radius of influence}})^2} \quad \text{(Horizontally)}$$

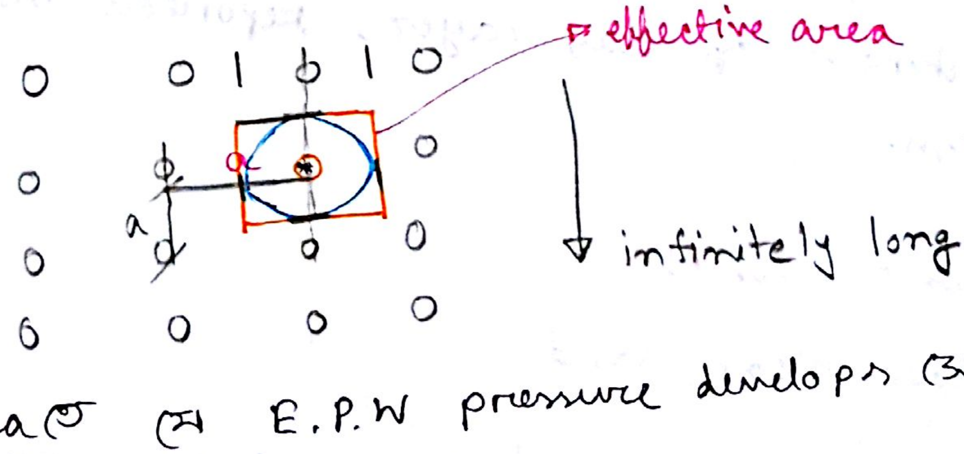
$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} + c_h \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)$$

⊛ ~~sand~~ steel casing of দিয়ে bore hole করি of diameter of hole. পরে stell casing বের করে ফেলে।

⊛ Diameters of the drain varies from 300-600mm. Dia less than 300 are difficult to install unless the surrounding soil is significantly remolded.

⊛ Spacing of sand drain (a) depending on soil in which they are placed normally varies from 1·5 -4'5 m. Sand drains are effective if the spacings are less than thickness of the consolidating layer.

⊛ Arrangement of sand drain:



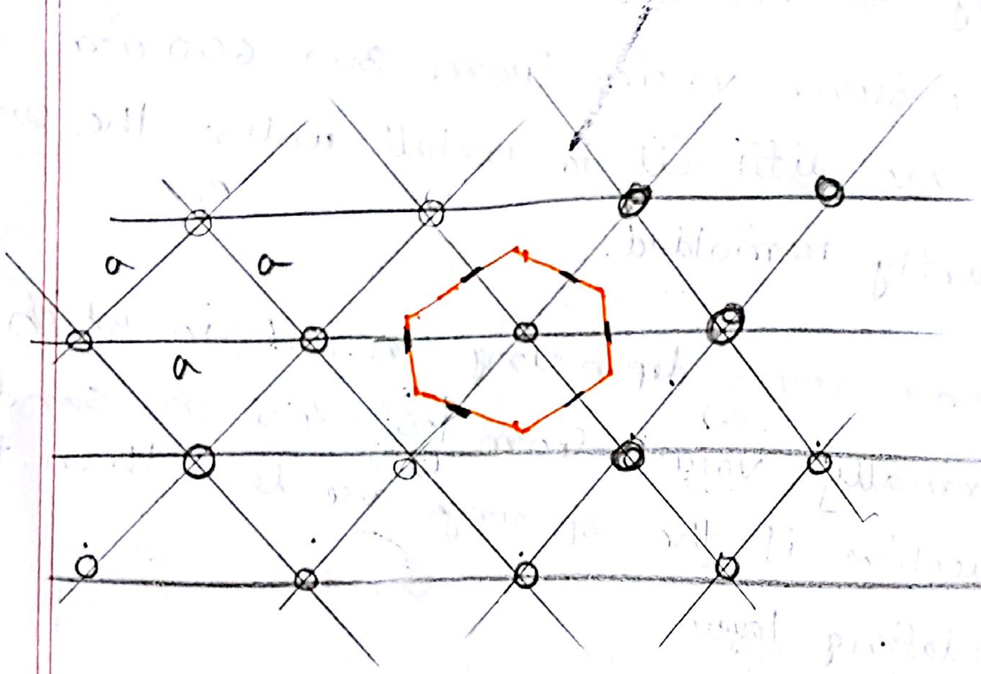
এই area তে যে E.P.W pressure develops ওটা ⊛ এই bore hole দিয়ে pass করবে।

এইটার area টাকে r এ convert করার জন্য circle করলাম।

area = πR^2 = $a \times a$.

R = 0·564 a

[mostly this used]



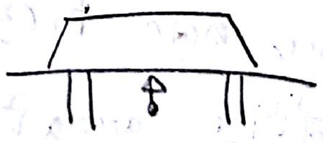
Hexagon এর
area = πR^2 (ধরব)
এটা থেকে
R = 0'525a

Depth of the sand drain it is dectated by the thickness of clay layer, reported max^m value 45m.

Types of Sand:

coarse clean sand

Drainage Blanket: thickness 0·33 – 1m.



consists of gravel & sand

↑ এভাবে water বের হবে না as clay,
so blanket দেয়া হয়, so H_2O বের হতে পারে।

Overfill of over charged load:

gradually load impose করতে হবে। radial strain negligible in comparison will verticle strain. So computer settlement by 1D formula.

Lec-23

13.12.15
Sunday

Limitation:

Sand drain are particularly suitable for soft clay but have little effect on the soils with small primary consolidation but significant secondary effect such as peat soil / organic soil.