C.W.

14/07/19

Consolidation – Muqtadir Sir

NC clay

OC clay

It may emphasize that normally consolidated soil and over consolidated soil are not different types of soil but these are conditions in which the same type of soil can behave as normally consolidated soil in a certain pressure and over consolidated in some other pressure head.

Over consolidation (causes) :

1) Remove surface load.

If Natural moisture content is significantly less than the Liquid limit, the sample is over consolidated.

For OC clay, LI = (0 – 0.6)

For NC clay = LI = (0.6 – 1)

Red clay → 99% Sure → OC clay.

Settlement কম হয় OC clay তে
পানি দিলে ফুলে যায়।

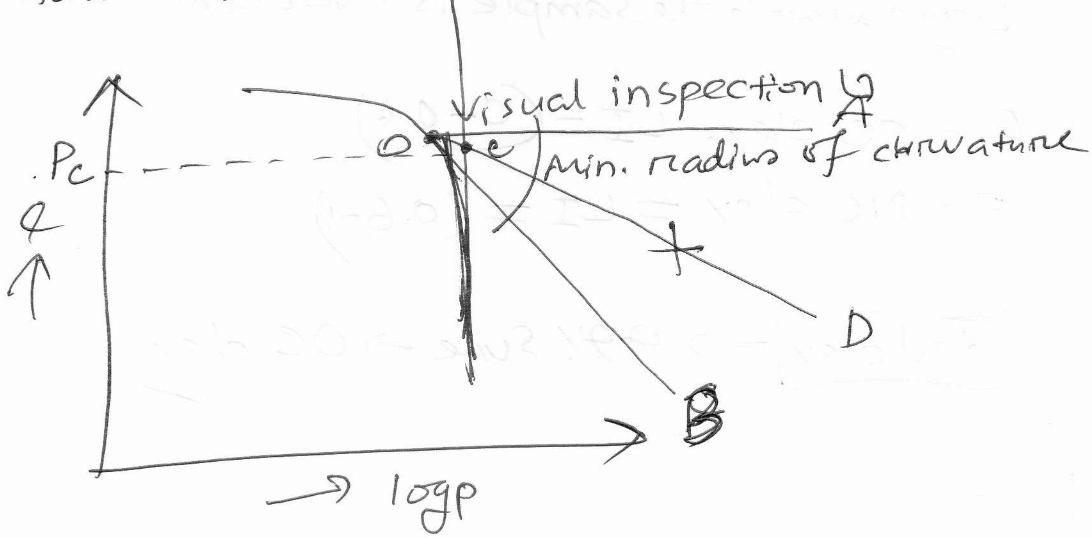
1. Compression Index (C_c) : Unitless
2. Co-efficient of compressibility (a_v) : Inverse of Pressure
3. Co-efficient of volume compressibility : Inverse of Pressure

এই ৩টি parameter lab এ measure করে settlement বের করা।

Chittagong, মধুপুর → Over consolidated clay
ঢাকার clay → NC clay হতে পারে।

e   P

বিভিন্ন Pressure এ void ratio lab এ পাব।

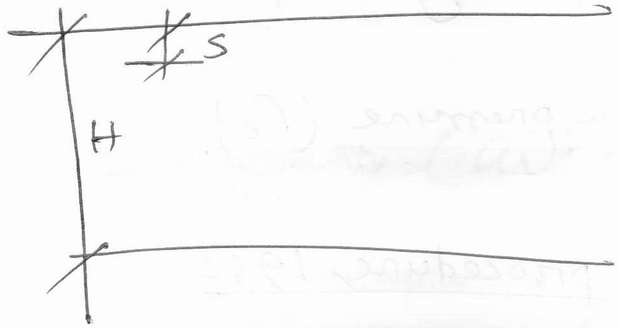


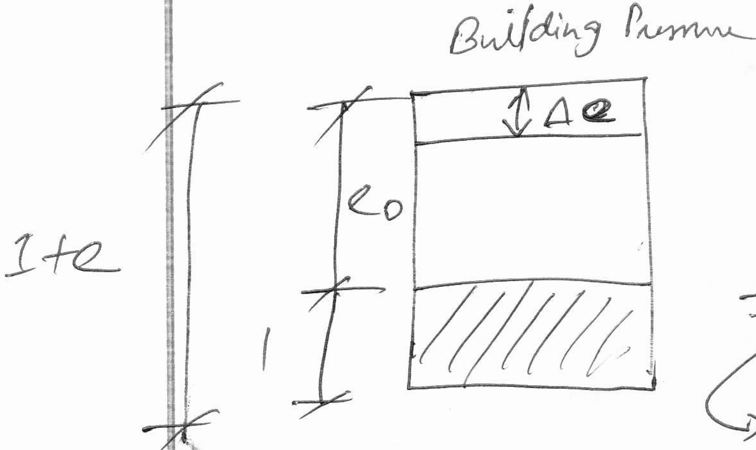
P_c: Past maximum pressure to which soil was subjected

Pressure corresponding to point c is called Pre-consolidation pressure (P_c).

<u>Casagrande's procedure, 1936</u>:

Basic equation of settlement:





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

$$\Rightarrow \boxed{H \cdot \Delta e = (1+e_0)S}$$

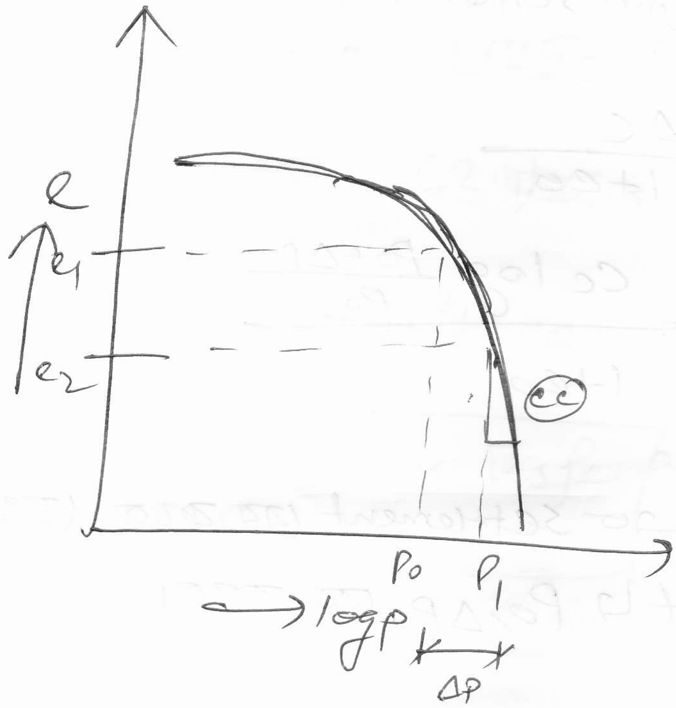
→ Basic equation of settlement

H → Height of clay layer

Δe →

e → lab → oppo

$e = wG$ At 100% degree of sat.



$$\Delta e = e_1 - e_2$$

$$C_c = \frac{\Delta e}{\log P_1 - \log P_0}$$

$$= \frac{\Delta e}{\log P_1/P_0}$$

$$= \frac{\Delta e}{\log \frac{(P_0 + \Delta P)}{P_0}}$$

$$\boxed{\Delta e = C_c \times \log \frac{P_0 + \Delta P}{P_0}}$$

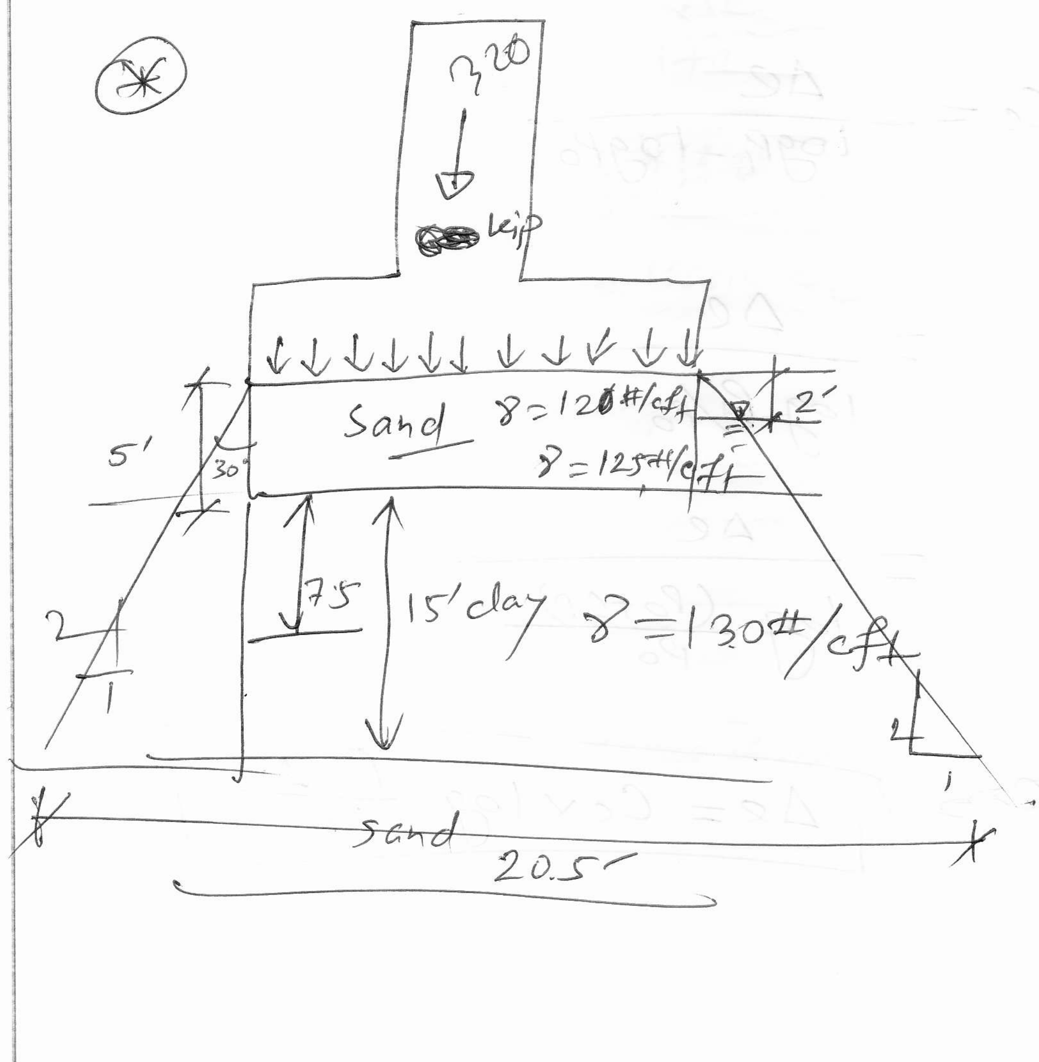
Final expression for settlement,

$$S = H \times \frac{\Delta e}{1+e_o}$$

$$= H \times \frac{C_c \log \frac{P_o + \Delta P}{P_o}}{1+e_o}$$

যে clay layer এর settlement বের করতে সেই clay layer এর middle point এ $P_o, \Delta P$ বের করব।

(*)



$P_o = 120 \times 2 + (125 - 62.4) \times 3 +$

$(130 - 62.4) \times 7.5$

$= 0.935$ kft

$$\frac{320}{20.5 \times 20.5} = 0.75 \text{ ksf}$$

$C_c = 1.8 - 1.5$ → very soft clay

$$S = \frac{15 \times 12 \times 0.2}{1 + e_o} \log \frac{0.935 + 0.75}{0.935}$$

If $s > 1$ inch,

Not acepted.

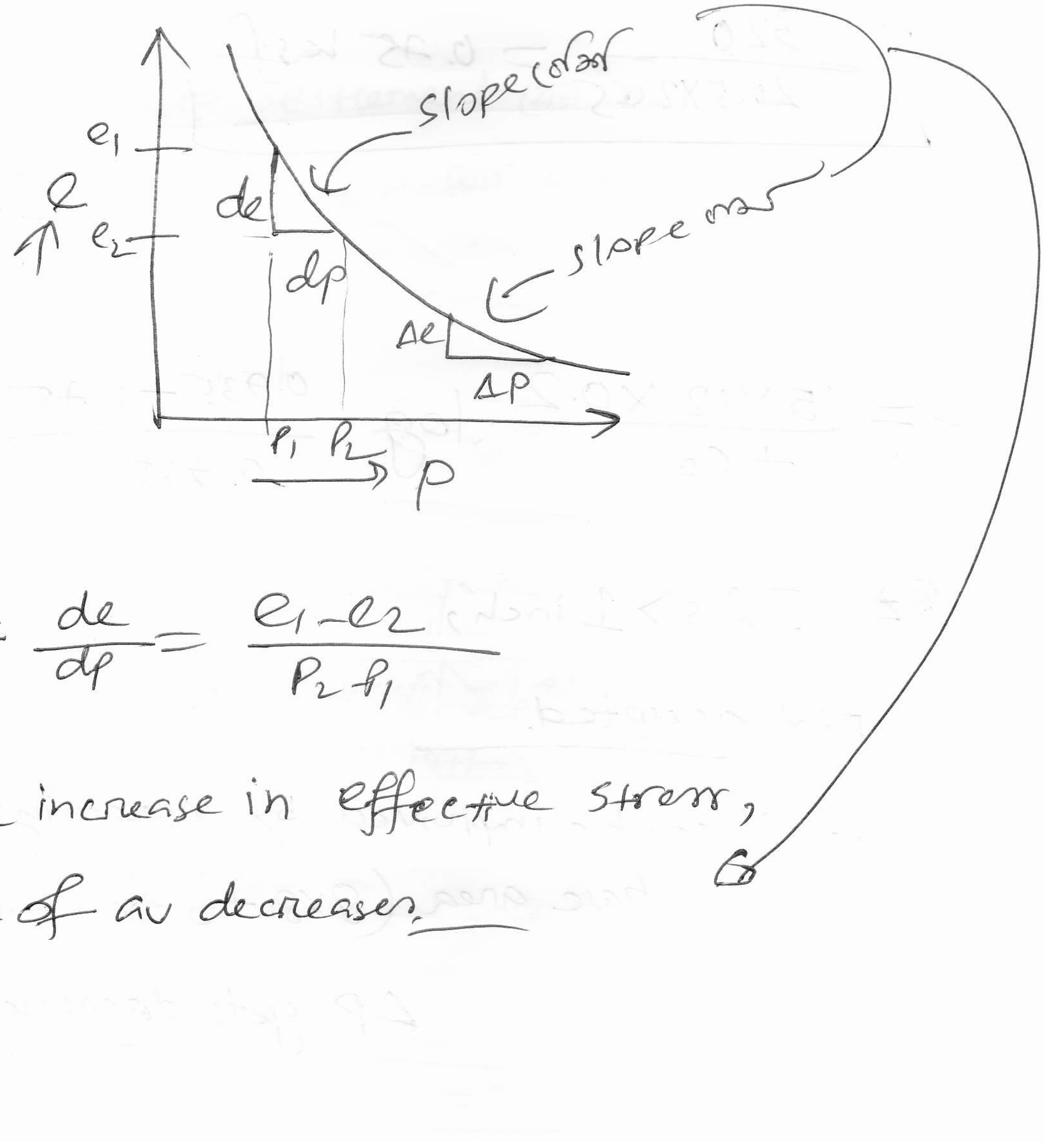
So, it can be improved by increasing faundation base area (8×8 to 9×9 or 10×10), so ΔP gets decreased.

Scampton / Tanizaki

$C_c = 0.009(LL-10)$ (Completely Undisturbed)

$= 0.007(LL-10)$ (Disturbed, remolded)

Co-efficient of compressibility:



$$a_v = \frac{de}{dp} = \frac{e_1 - e_2}{P_2 - P_1}$$

With the increase in effective stress, the value of av decreases.

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

$$= \frac{H a_v}{1+e_0} \Delta p = H (m_v) \Delta p$$

m_v → co-efficient of volume compressibility

যেহেতু av vary করে, এই expression settlement বের করার জন্য কম use করি।

Effective overburden Pressure যদি থাকে হয় OC clay

$\frac{C}{P'} = 0.10 + 0.004 I_p$ → plasticity

C, ϕ → Angle of internal friction

Shear strength of soil

Soil এর strength characteristics

Onconfined compaction test

Shearstrength theory

Volume Compressibility (m_v)

m_v is the volumetric strain per unit increase in effective stress

$$\text{Volumetric strain} = \frac{\Delta V}{V_0}$$

$$m_v = \frac{\Delta V}{V_0} \times \frac{1}{\Delta p}$$

$$\frac{\Delta V}{V} = \frac{\Delta e}{1+e_0}$$

for 1d consolidation,

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

$$m_v = \frac{\Delta H}{H} \times \frac{1}{\Delta p}$$

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

$$m_v = \frac{a_v}{1+e_0}$$

$$\Delta e = a_v \cdot \Delta p$$

a_v, m_v

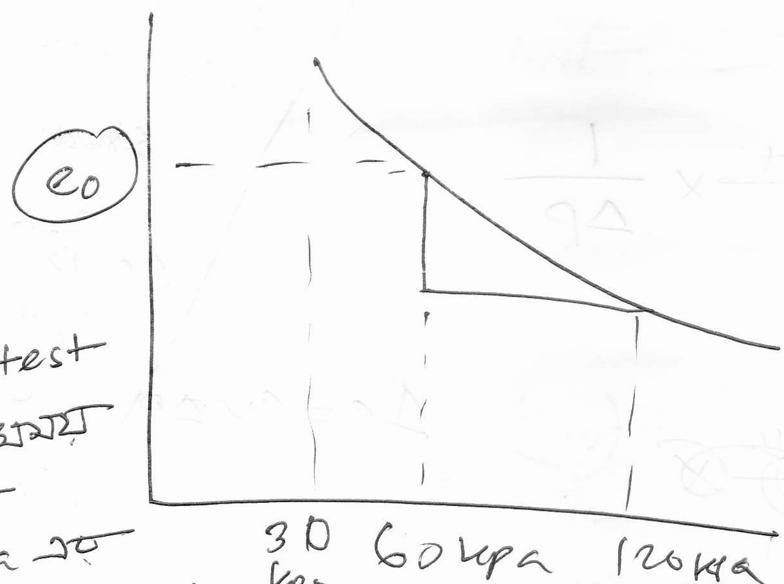
They are the function of the current stage of stress

Cc is not, independent of current stress

Slope of virgin portion of e-log p curve

In terms of mv, consolidation determination always use করি।

$\Delta H = H m_v \Delta P$



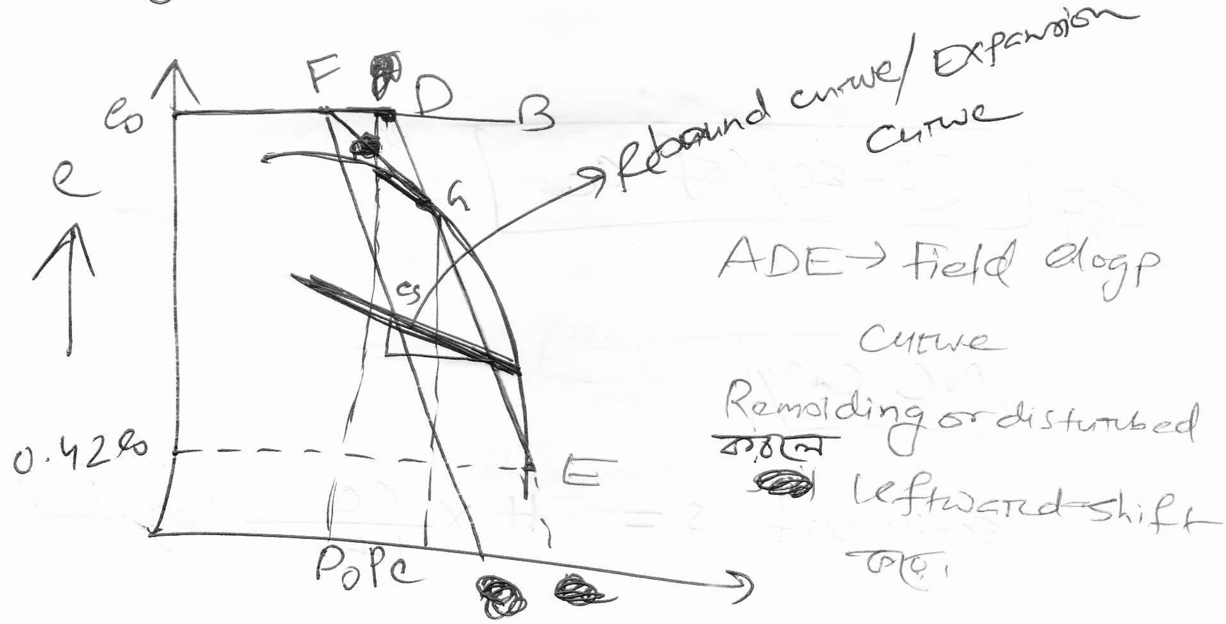
Mv test এর মাধ্যমে দুইটি data এর মধ্যে beginning data এর জন্য (for 60kpa)

Not initial value of 30kpa

Remolded soil sample এর Stiffness কমে যায়।

Schmertmann → (1955) procedure

For NC Clay,



Disturbed soil sample এর elogp curve and Undisturbed soil sample এর (field elogp) elogp curve pass through a common point corresponding 42% of initial void ratio.

Unloading & Reloading ~~path~~ curve same path follow করে।

It follows the slope of the rebound curve

AFGE

AFG → OC clay (P_c এর কম)

GE → NC clay (P_c exceed করে)

C_s = (10 − 20)% of C_c

NC clay

settlement, $S = H \times \frac{C_c}{1+e_o} \log \frac{P_o + \Delta P}{P_o}$

$S_{F \to G} = H \times \frac{C_s}{1+e_o} \log \frac{P_o + \Delta P}{P_o}$ $(P_o + \Delta P \le P_c)$

$S_G = H \times \frac{C_s}{1+e_o} \log \frac{P_c}{P_o}$

$S_{G \to E} = \frac{H C_s}{1+e_o} \log \frac{P_c}{P_o} + \frac{H \times C_c}{1+e_o} \log \frac{(P_o + \Delta P)}{P_c}$

$(P_o + \Delta P) > P_c$

C.W.

Consolidation → BM Das   Muqtadir Sir

কিছু mention করা না থাকলে normally settlement বলতে primary settlement বুঝায়।

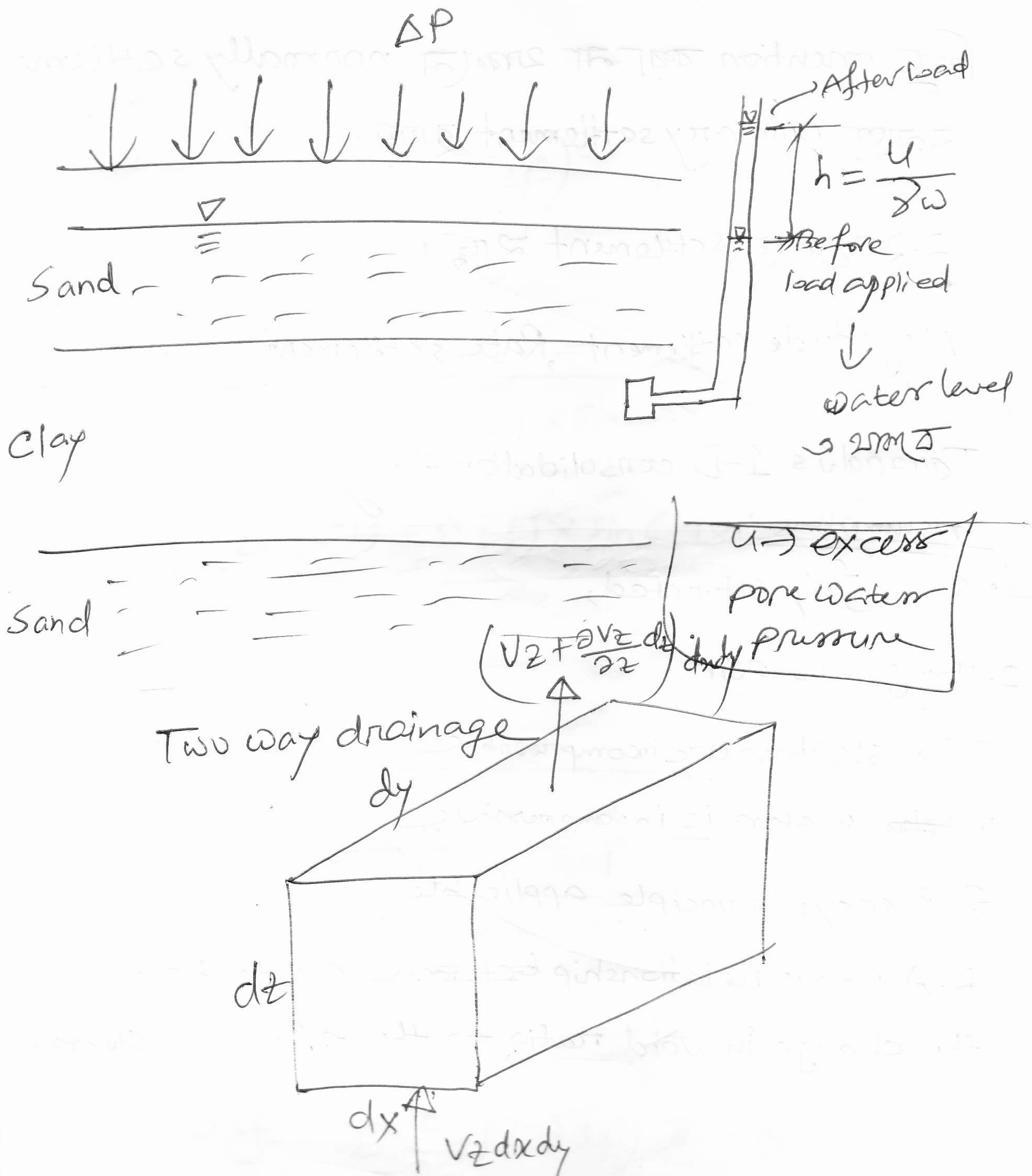
১-২ বছরে যে settlement হচ্ছে।

Magnitude settlement, Rate settlement

Terzaghi's 1-D consolidation theory

Assumptions:

1. Soil is fully saturated,
2. Homogenous soil
3. Soil skeletons are incompressible
4. ~~[illegible]~~ Water is incompressive
5. Darcy's principle applicable.
6. A unique relationship ~~between~~ exists between the change in void ratio to the effective stress.



Outflow rate − Inflow rate = Rate of volume change

$V = dx.dy.dz$

$$\left(V_z + \frac{\partial V_z}{\partial z} \cdot dz\right) \cdot dx\,dy - V_z\,dx \cdot dy = \frac{\partial V}{\partial t}$$

$$\Rightarrow \frac{\partial V_z}{\partial z} \cdot dx\,dy\,dz = \frac{\partial V}{\partial t}$$

$V_z = -k_z i_z$ ($\ominus$ is used to indicate loss)

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

$$= -\frac{k}{\gamma_w} \times \frac{\partial u}{\partial z}$$

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} dx\,dy\,dz = \frac{\partial u}{\partial t}$$

Rate of volume change:

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

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

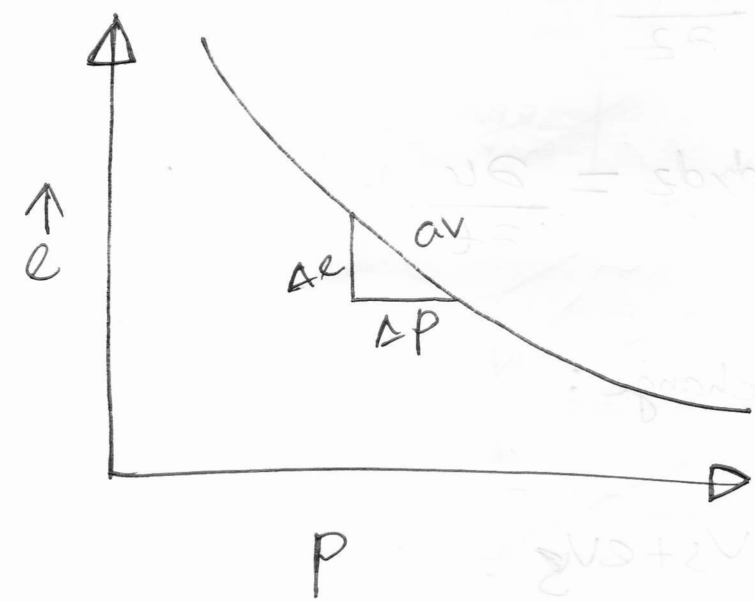
$$V = V_s(1+e)$$

$$\Rightarrow \boxed{V_s = \frac{V}{1+e}} = \frac{dx \cdot dy \cdot dz}{1+e}$$

$$\frac{\partial V}{\partial t} = \frac{dx.dydz}{1+e} \cdot \frac{\partial e}{\partial t}$$

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} (dx.dydz) = \frac{(dx.dy.dz)}{1+e} \cdot \frac{\partial e}{\partial t}$$

$$\Rightarrow -\frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} = \frac{\partial e}{(1+e)\partial t}$$



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

$$\partial e = a_v \, \partial(\Delta p) = -a_v \partial u$$

$$\partial e =$$

Increment দেয়ার আগে যে void ratio থাকে e_0 হলো।

$$\frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} = \frac{a_v}{1+e_0} \frac{\partial u}{\partial t}$$

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

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

$$\Rightarrow \frac{\partial^2 u}{\partial z^2} = C_v \cdot \frac{\partial^2 u}{\partial z^2}$$

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

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

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

C_v বের করব lab এ।

m_v পাব।

পরবর্তীতে k পাব।

Rate Analysis : Muqtadir Sir

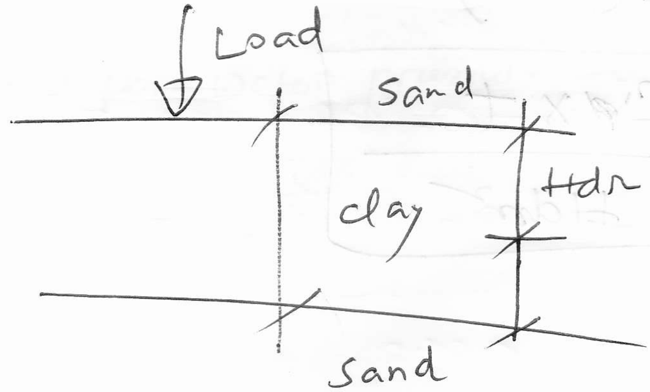
$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2}$$

$$C_v = \frac{K}{\gamma_w m_v}$$

(K/m_v) ratio is constant
γ_w constant ~~for~~ different forces.

* logarithm of time method / casagrande's procedure
* Sq. root of time method / Tailor's method.

Boundary condition:



$H_{dr} = H/2$

For both way drainage, length of the drainage path (H_{dr}) will be half of the clay layer thickness (H)

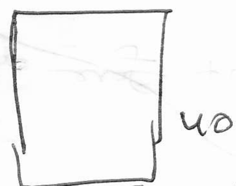
For one way drainage, length of the drainage path will be thickness of clay layer

B.C associated with geometry

$z = 0, \; u = 0$

$z = 2H_{dr}, \; u = 0$

Initial condition at $t = 0$ →



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

T_v = Dimensionless time factor

$m = 0, 1, 2, - - - - \infty$

$M = \pi/2 \, (2m+1)$

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

$$U_z = \frac{u_0 - u_z}{u_0} = 1 - \frac{u_z}{u_0}$$

↓

Degrees of consolidation

↓

Depends on the depth (Length of the drainage path)

$$U_{av} = 1 - \frac{\frac{1}{2H_{dr}} \int_0^{2H_{dr}} u_z \, dz}{u_0}$$

$$= 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} e^{-M^2 T_v}$$

Previously settlement measure করা হয়েছিল সেটা ছিল Ultimate settlement.

এখন time and pore water pressure এর উপর depend করে এমন settlement বের করবে।

T_v corresponding to 50%

T_v corresponding to 90%.

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

$$= \pi/2$$

$$U_{av} = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} [e]^{-M^2 T_v}$$

$$\Rightarrow 0.90 = 1 - \frac{2}{\frac{\pi^2}{4}} \times e^{-\pi^2/4 \times T_v}$$

$$\Rightarrow T_v = 0.848$$

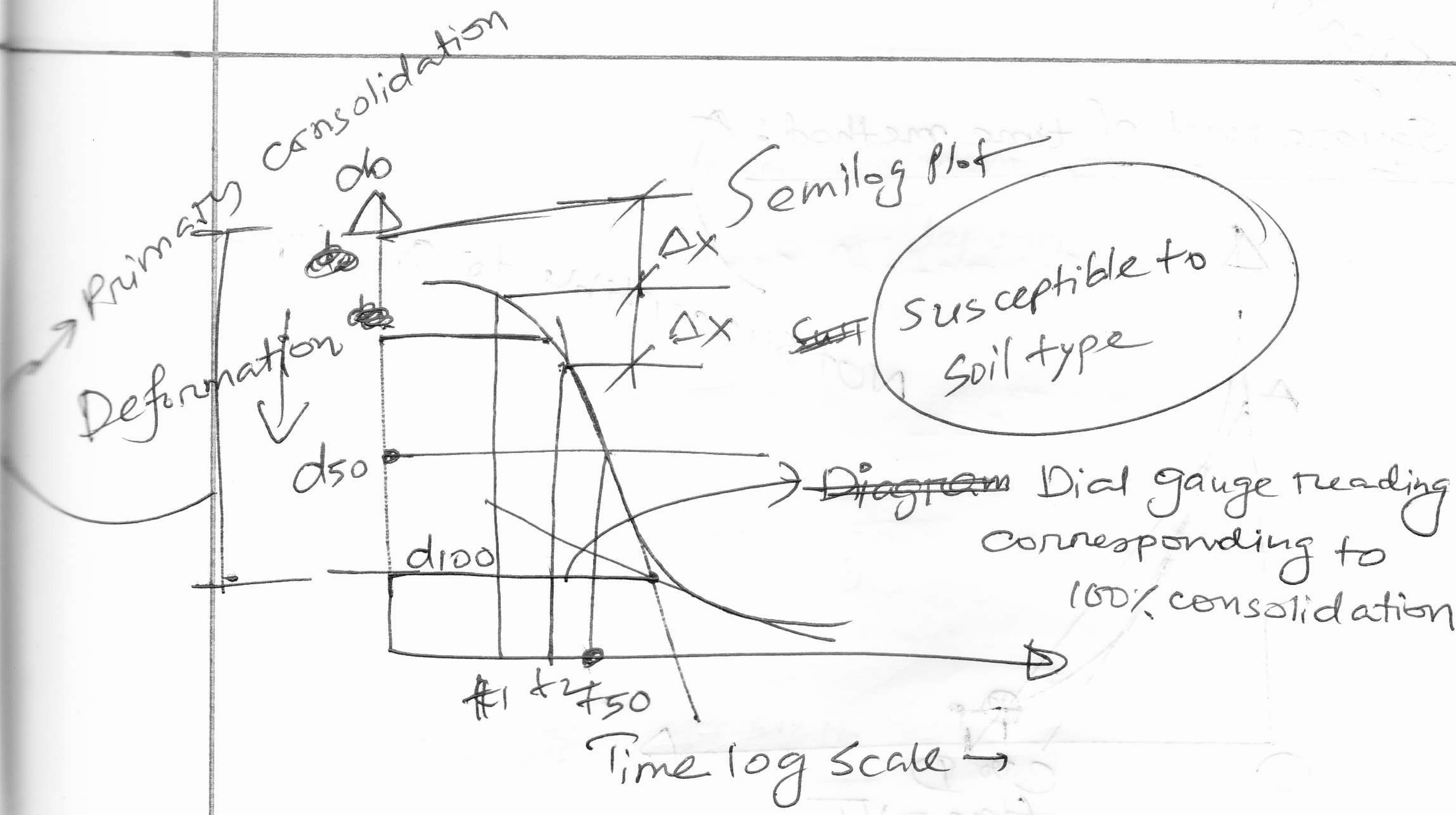
Curve fitting Method:

$$T_v = \pi/4 \times (U/100)^2 \qquad U \leq 60$$

$U = 50$, $T_v = 0.196$

$$T_v = 1.781 - 0.9331 \log_{10}(100-U) \qquad U > 60$$

$U = 90$, $T_v = 0.848$



Deformation associated with 0% consolidation.

" " " 100% "

~~$t_2 = 4t_1$~~ $t_2 = 4t_1$

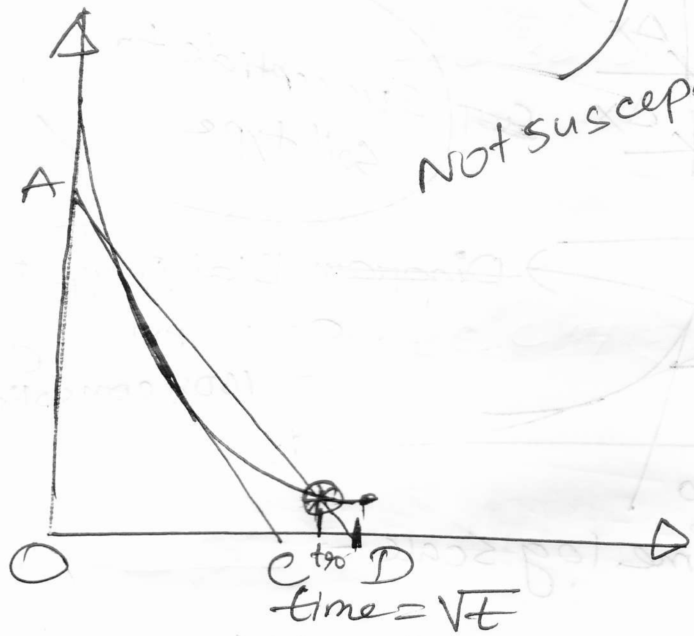
$$d_{50} = \frac{d_0 + d_{100}}{2}$$

$$T_{v_{50}} = \frac{c_v \times t_{50}}{H_{dr}}$$

Tailor's Method

Square root of time method:

Not susceptible to soil type



A → Dial gauge reading corresponding to 0% consolidation

$$OD = 1.15\, OC$$

AD line graph কে যে point এ ছেদ করবে সেই point 90% consolidation নির্দেশ করে।

$$\sqrt{\frac{0.848}{0.636}} = 1.15$$

$$U = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 = 0.636$$

Total settlement/Ultimate settlement সময়ের উপর/

Drainage condition এর উপর depend করে না।

$$u = \frac{u_0 - u_z}{u_0} = \frac{S_c}{S_f}$$

<u>C section থেকে এটা Math</u>