lecture 8
12/11/18

CHAPTER-6

compressibility and consolidation

1) compressibility

1) It is the gradual ↓ in volume of soil due to application of load on soil mass ~~cond~~ under consideration.

compressibility

Initial compression or compaction

— Due to compression or expulsion of pore Air

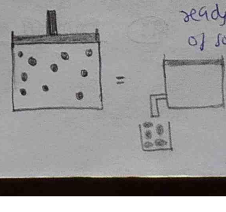
~~consideration~~ consolidation

Primary consolidation

— Due to expulsion of pore ~~pressure~~ water

Secondary consolidation

— Due to Plastic readjustment of soil solids



① Initial compression or compaction :-

1) It is an instaneous process which occurs immediately after loading and is due to compression and expulsion of Pore air

② Primary consolidation

1) It begains when soil reaches to full saturation (after the completion of Pore air) and remains saturated during entire Process of Primary consolidation.
2) It takes place due to expulsion of pore water and It complete's when expulsion of pore water stop
3) It is time taking Phenomenon. Rate of consolidation depend upon on
   ① permeability of soil
   ② Length of drainage path
   ③ Loading cond

③ Secondary consolidation (2°)

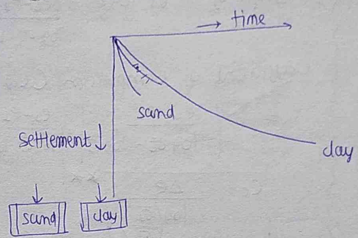
It start after completion of Primary consolidation.

If load is constant and volm change is recorded with passes of time, without expulsion of pore water then volm change is recorded due to plastic readjustment of soil solids. It is also termed as Secondary consolidation settlement

. In coarse grain soil and stiff clays secondary consolidation is insignificant but in highly plastic clay secondary consolidation can be 10% to 20% of total settlement.

2.1 Primary consolidation characteristic

2.2 Settlement V/s time



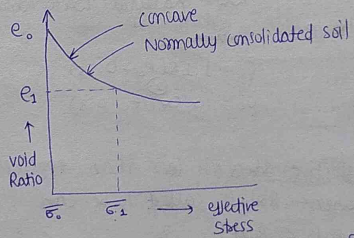
Device : oedometer use

- The rate of consolidation in sand is more due to it's high permeability but total settlement of clay is much greater than that of sand, at same loading rate & having same thickness of samples of both the soil
- This settlement of clay is higher becoz of it's high void Ratio and porosity thus for Practical Purposes settlement of sand is neglected in comparision of clay.

2.3 Void Ratio V/s effective stress

(Normally consolidated soils and normal compression curve)

- Normally consolidated soils are those which r subjected to 1st time in the History to the present applied effective stress (Present > Past)



slope of e Vs σ $= \frac{e_0 - e_1}{\sigma_0 - \sigma_1}$

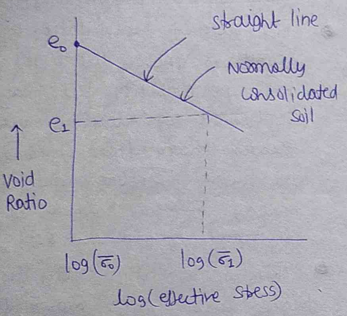
① Coefficient of compressibility $(a_v) = \left|\frac{e_0 - e_1}{\sigma_0 - \sigma_1}\right|$

$\boxed{a_v = \frac{\Delta e}{\Delta \sigma}}$ ---- ① formula

Since a_v is not constant for whole curve .it is not widely use. but when we draw this curve in semi-log graph paper straight line is obtained

③ Void Ratio Vs log(effective stress)

Important



Slope of e vs $\log(\bar\sigma) = \dfrac{e_0 - e_1}{\log\bar\sigma_0 - \log\bar\sigma_1}$

Coefficient of compression (C_c)

$$= \left|\frac{e_0 - e_1}{\log\bar\sigma_0 - \log\bar\sigma_1}\right| = \frac{e_0 - e_1}{\log\bar\sigma_1 - \log\bar\sigma_0}$$

$$\boxed{C_c = \frac{\Delta e}{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)}} \quad ** \quad \text{------ (2)}$$

Emphrical Relation of determination of C_c

Remember

1) For undisturbed soil (clay) $\boxed{C_c = 0.009(W_L - 10)}$ *

2) For Remoulded clay $C_c = 0.007(W_L - 7)$
W_L = liquid limit in %

3) for organic clay $C_c = 0.0115(W_n)$
W_n = natural water content

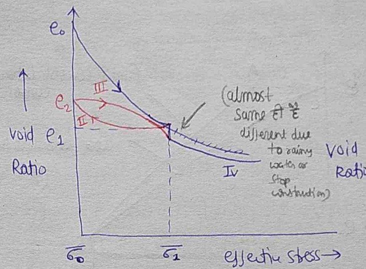
4) When initial void ratio (e_0) is known
$C_c = 1.15(e_0 - 0.35)$
or
$C_c = 0.54(e_0 - 0.3)$

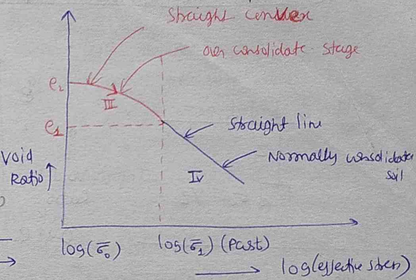
(4) over consolidation stage / Pre consolidation stage

1) If Presently applied effective stress is less than applied effective stress in the past when soil is called to be an over consolidation stage (Present < Past)

2) over consolidation soils show less volume change and highly over consolidated clay behave like dense sand

I) Normally consolidated stage (concave) or virgin compression curve
II) swelling / Rebound curve
III) over consolidation curve (convex)
IV) Normally consolidation curve (concave)





Coefficient of Recompression ← {Slope of e Vs $\log\bar\sigma$ in overconsolidated soil stage} $= \left\{\dfrac{e_2 - e_1}{\log\bar\sigma_0 - \log\bar\sigma_1}\right\}$

$$= \left|\frac{e_2 - e_1}{\log\bar\sigma_0 - \log\bar\sigma_1}\right| = \frac{e_2 - e_1}{\log\bar\sigma_1 - \log\bar\sigma_0}$$

$$\boxed{C_R = \frac{\Delta e}{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)}} \quad *** \quad \text{------ (3)}$$

Generally, $\boxed{C_R = \frac{1}{5} \text{ to } \frac{1}{10} \text{ of } C_c}$

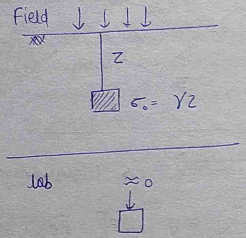
* over consolidated Ratio

$$\boxed{O.CR = \frac{\text{Max}^m \text{ applied eff. stress in Past}}{\text{Max}^m \text{ applied eff. stress in Present}}}$$

for O.C stage $\boxed{O.CR > 1}$
for N.C stage $\boxed{O.CR = 1}$
for under consolidated stage $\boxed{O.CR < 1}$

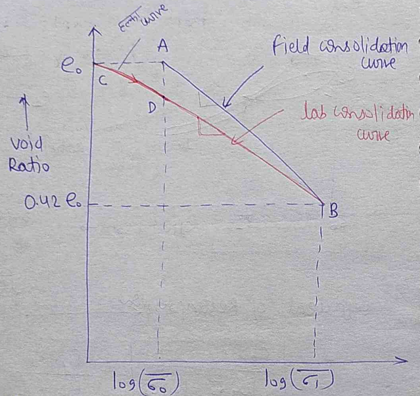
Note: Melting of glaciers, Removal of construction load, Rising of water table, Dissication of clay are some eg of over consolidation

V) ⑤ Field consolidation curve



CD → O.C stage

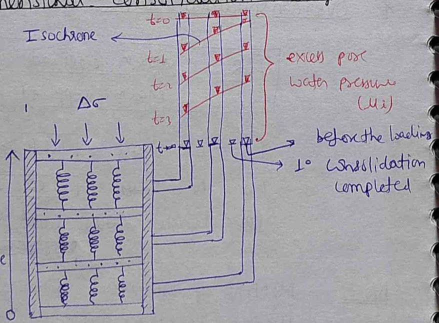
DB → N.C stage



Note : sample are slightly disturbed when these are taken out from the ground. the disturbance cause a slight decrease in the slope of laboratory consolidation curve

* Terzaghi's 1 dimensional consolidation theory

Spring Analogy



1) Application of total stress

2) At t=0
(Excess Pore water Pressure developed)

$\Delta\sigma = \cancel{\Delta\bar{\sigma}}^{0} + u_i$

3) At any time (t) (Excess pore pressure decreases , eff stress ↑)

$\Delta\sigma = \Delta\bar{\sigma} \uparrow + u_i \downarrow$

4) At t=∞ (Excess pore water Pressure reduces to zero , 1° consolidation completed

$\Delta\sigma = \Delta\bar{\sigma} + \cancel{u_i}^{0}$

- (Consolidation depends on effective stress not total stress)
- (Isochrone → different level of Piezometric at same time)

Assumptions

1) Soil is Homogeneous and isotropic.
2) Darcy's law is valid
3) The consolidation is 1 dimensional it means flow of spelling water is uni-directional (In vertical) and there is no change in areal volume change is due to change in depth only)
4) Soil is fully saturated and remains saturated throughout the process of consolidation it means initial compression is not considered ,
5) Strain coming on soil solids is small and neglisible it means solids are incompressible Hence 2° secondary consolidation is not considered
6) The hydrodynamic lag is considered whereas plastic lag is ignore However plastic lag is found to be exist it means time required for plastic readjustment of soil solids is neglected Hence secondary consolidation is not considered
7) Coefficient of consolidation and compressibility are constant

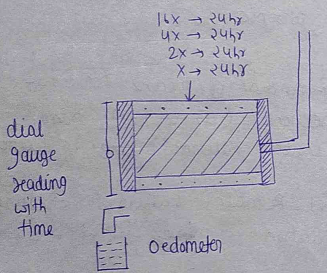
- In ordered to developed field cond in the lab consolidation test is being performed on soil mass in consolidometer or Oedometer there are two type of consolidated consolidometer

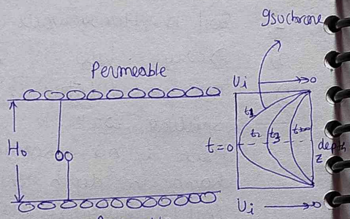
① Fixed Ring cell (Only top Plate is allowed to move)

② Floating Ring cell (both top and bottom plate are allow to move .)

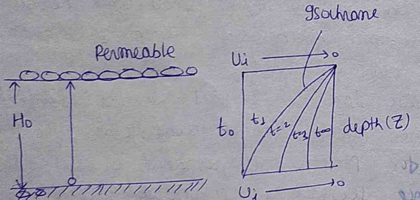
* In consolidation test pressure on the soil specimen is doubled in each stage so that, significant change in void ratio can be obtained in small time.

* In oedometer load increment ratio is 2





2-way drainage
length of drainage path $\boxed{d = \frac{H_0}{2}}$



1-way drainage
length of drainage path
$\boxed{d = H_0}$

Note Isochrones are the curve which represents the excess pore water pressure at same time and at different depth.

Terzaghi's 1D Consolidation Eqn

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

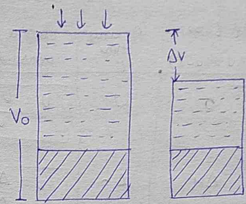
[$\frac{\partial u}{\partial t}$ = Rate of change excess pore water pressure. It also represent the Rate of consolidation

C_v = coefficient of consolidation

$$\boxed{C_v = \frac{K}{m_v \gamma_w}} \text{ ** } \quad \text{---- (4)}$$

coefficients

K = coefficient of permeability
m_v = coefficient of volm compressibility of modulus of volm change



$$m_v = \frac{-\frac{\Delta V}{V_0}}{\Delta \sigma} = \frac{\frac{-\Delta e}{1+e_0}}{\Delta \sigma} = \frac{-\Delta e}{\Delta \sigma} \cdot \frac{1}{(1+e_0)}$$

a_v

$$\boxed{m_v = \frac{a_v}{1+e_0}} \text{ *** } \quad \text{---- (5)}$$

C_v = Coeff. of compressibility
e_0 = Initial void ratio

	Coefficient	Symbol	formula	unit
(1)	Coefficient of ~~Permeability~~ compressibility	a_v	$\boxed{a_v = \frac{\Delta e}{\Delta \sigma}}$ *	$\frac{m^2}{KN}$
(2)	Coefficient of compression	C_c	$\frac{\Delta e}{\log\left(\frac{\sigma_1}{\sigma_0}\right)}$	—
(3)	~~Volume compressibility~~ Recompression	C_R	$\frac{\Delta e}{\log\left(\frac{\sigma_1}{\sigma_0}\right)}$	—
(4)	Volm compressibility	m_v	$\frac{a_v}{1+e_0}$	$\frac{m^2}{KN}$
(5)	Consolidation	C_v	$\frac{K}{m_v \gamma_w}$	$\frac{m/sec}{\frac{m^2}{KN} \cdot \frac{KN}{m^3}} = \frac{m^2}{sec}$

Solution of terzaghi eqn

(1) Degree of consolidation (% U)

1) It is the fraction of ultimate consolidation which is completed at any time (t) during the consolidation.
2) It is defined only for primary consolidation.

	At $t=0$ ($\%U=0$)	At $t=\infty$ ($\%U=100\%$)	At any time 't'	degree of consolidation
① When excess pore pressure is known	U_i	0	U	$\boxed{\%U = \frac{U_i - U}{U_i}\times 100}$ *
② When settlement is known	0	ΔH	Δh	$\boxed{\%U = \frac{\Delta h}{\Delta H}\times 100}$ **
③ When void Ratio is known	e_0	e_{100}	e	$\boxed{\%U = \frac{e_0 - e}{e_0 - e_{100}}\times 100}$

② time factor

It is the Parameter which relates to the degree of consolidation and time required for that consolidation

$\boxed{T_v = \frac{C_v t}{d^2}}$ * C_v = coefficient of consolidation

$$C_v = \frac{K}{m_v \gamma_w} = \frac{K}{\frac{a_v}{1+e_0}\gamma_w} = \frac{K}{\frac{\Delta e}{\Delta\sigma (1+e_0)}\gamma_w} = \frac{K\Delta\sigma}{\frac{\Delta e}{1+e_0}\gamma_w}$$

\# $d \rightarrow$ length of drainage Path

for 2way drainage $\boxed{d = \frac{H_0}{2}}$

for 1 way drainage $\boxed{d = H_0}$

\# $t \rightarrow$ time at which time factor is required

\# T_v = time factor

$$\boxed{\begin{aligned} T_v &= \frac{\pi}{4}U^2 = \frac{\pi}{4}\left(\frac{\%U}{100}\right)^2 \quad \%U \le 60\% \\ T_v &= 1.781 - 0.933\log_{10}(100 - \%U) \quad \%U > 60\% \end{aligned}}$$

Rule

$\boxed{\Delta h \rightleftharpoons t}$

$\%U = \frac{\Delta h}{\Delta H}\times 100 \longrightarrow T_v \longleftarrow T_v = \frac{C_v t}{d^2}$

50% $U \rightarrow$ 1 year

$\%U$	50%	90%	100%	60%
T_v	0.196	0.848	∞	0.283

Note theortically 100% consolidation takes ∞ time but consolidation is assumed to be completed when $\%U \ge 90\%$.

Determination of C_v

1) C_v depends upon type of soil and loading consolidation generally, it has been observe that liquid limit ↑ C_v↓

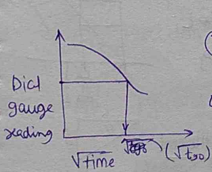
$w_L \uparrow \longrightarrow$ compressibility $\uparrow \longrightarrow a_v\uparrow \longrightarrow m_v\uparrow \longrightarrow C_v\downarrow$

$C_v = \frac{K}{m_v \gamma_w}$ $C_c \uparrow = 0.009(w_L - 10)$

There are two method to find C_v

Ⓐ Square Root of the fitting Method

Given by taylor $\boxed{\%U = 90\%}$



$(T_v)_{90} = \frac{C_v t}{d^2}$

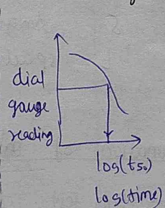
$0.848 = \frac{C_v t_{90}}{d^2}$

$C_v = \frac{0.848 \times d^2}{t_{90}}$

** Square root of time fitting Method is better

Ⓑ logrithmic of time fitting method

Given by Casagrande $\boxed{\%U = 50\%}$



$(T_v)_{50} = \frac{C_v t}{d^2}$

$0.196 = \frac{C_v t_{50}}{d^2}$

$C_v = \frac{0.196\, d^2}{t_{50}}$

Settlement analysis

① total settlement is the sum of initial / immediate settlement, Primary consolidation settlement and secondary consolidation settlement. $\boxed{S = S_i + S_c + S_s}$

IS CODE GUIDELINES :-

1) Total Permissible settlement

	For steel Structure	for R.C.C Structure (mm)
for isolated footing on sand	50	50
for isolated footing on clay	50	75
for Raft footing on sand	75	75
for Raft footing on clay	100	100

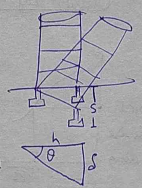
2) Permissible Angular settlement

	For steel Structure	For R.C.C Structure
for isolated footing	1/300	1/500
for Raft footing	1/300	1/666

3) Permissible differential settlement(s)

δ = Angular settlement * L

L = spacing b/w the footing



angular settlement $= \frac{\delta}{L}$

Not imp

Determination of initial / immediate settlement

cone penetration test data can be used to determine the initial settlement due to expulsion of pore air.

$$S_i = \frac{H_0 \bar{\sigma}_0}{1.5\, q_c} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

q_c = static cone penetration resistance

H_0 = thickness of compressible layer

$\bar{\sigma}_0$ = Initial of stress at c/c of compressible layer's

$\Delta\bar{\sigma}$ = change in " " " " "

B) Elastic settlement Important

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Small elastic settlement can occurs due to squeezing of soil particles

$$\boxed{S_i = \frac{qB(1-\mu^2)I_t}{E_s}} \; **$$

q = uniform pressure on the footing

B = Width of footing

μ = Poisson's ratio of soil (0.35 to 0.5)

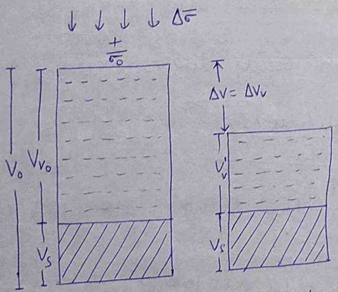
E_s = young's modulus of soil.

I_t = influence factor or shape factor which depends upon type of soil

	Flexible footing centre	corner	Avg	Rigid footing
① circular	1.00	0.64	0.85	0.8
② square	1.12	0.56	0.95	0.82
③ Rectangular				
$\frac{L}{B} = 1.5$	1.36	0.68	1.20	1.09
$\frac{L}{B} = 2$	1.52	0.76	1.31	1.22

Determination of Primary consolidation settlement

A) When change in void ratio is known



$$\Rightarrow \frac{\Delta V}{V_0} = \frac{\Delta H \cdot \cancel{A}}{H_0 \cancel{A}} = \frac{\Delta H}{H_0} \quad ---- (A)$$

{terzaghi's 1-D consolidation}

$$\Rightarrow \frac{\Delta V}{V_0} = \frac{\Delta V_v}{V_s + V_{v_0}} = \frac{\Delta V_v}{V_s\left[1 + \frac{V_{v_0}}{V_s}\right]}$$

$$= \frac{\frac{\Delta V_v}{V_s}}{1 + \frac{V_{v_0}}{V_s}}$$

$e_0 = \frac{V_{v_0}}{V_s}$ $\quad e_{100} = \frac{V_v'}{V_s}$

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0} \quad --- (B)$$

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

$\because \frac{V_{v_0}}{V_s} = e_0$

from (A) and (B)

$$\boxed{\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}} \; * \quad --- (1)$$

ΔH = Ultimate consolidation settlement

H_0 = Initial thickness of compressible layer

$\Delta e = e_0 - e_{100}$ = change in void ratio

e_0 = initial void ratio

(B) When coefficient of vol^m compressibility (m_v) is known

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{1+e_0} \times \frac{\Delta\bar{\sigma}}{\Delta\bar{\sigma}} = \underbrace{\frac{\Delta e}{\Delta\bar{\sigma}}}_{= a_v} \times \frac{\Delta\bar{\sigma}}{1+e_0} = \left(\frac{a_v}{1+e_0}\right)\Delta\bar{\sigma}$$

$$= m_v \bar{\sigma}$$

$$\boxed{\Delta H = H_0 m_v \Delta\bar{\sigma}} \; ***$$

(C) When coefficient of compression (C_c) is known (compression index)

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{1+e_0} \times \frac{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)} = \underbrace{\frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}}_{C_c} \times \frac{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}{1+e_0}$$

$$= \frac{C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)$$

$$\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)$$

****

$$\boxed{\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

Note (1)

If soil is in N.C stage ⟶ use C_c

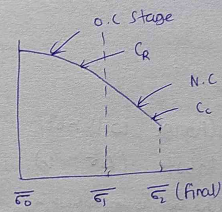
If soil is in O.C stage ⟶ use C_R

1) past ($\bar{\sigma}_1$)

2) Present ($\bar{\sigma}_0$)

3) change ($\Delta\bar{\sigma}$)

4) final ($\bar{\sigma}_2$)



$$\boxed{\Delta H = \frac{H_0 C_R}{1+e_0} \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right) + \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)}$$

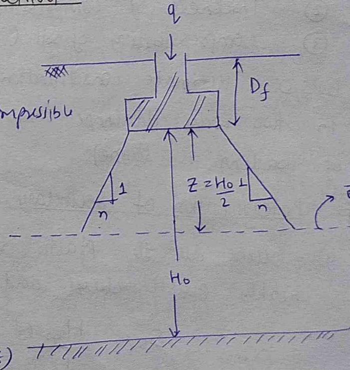
Very important

Note (2) Four step Method

Step (1) H_0

Step (2) $\bar{\sigma}_0$ at c/c of compressible layer

Step (3) load distribution 1 : n



$$\Delta\bar{\sigma} = \frac{q(B*L)}{(B+2nZ)(L+2nZ)}$$

$$= \frac{Q}{(B+2nZ)(L+2nZ)}$$

load = Force / Area

Force = load × Area = $q(B \times L)$ = Q

Not if load distribution is not given then Assume 2V : 1H

Step ④ Ultimate consolidation settlement

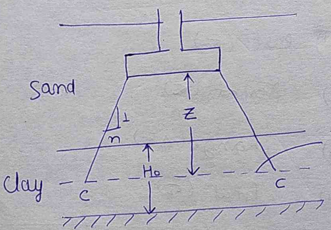
$$\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

Note ③ Determine settlement of clay layer

Note ④

surcharge will not distribute ($\Delta\bar{\sigma} = q$)

(surcharge will distribute in large area)



Note ⑤ ultimate consolidation settlement depends upon

① change in effective stress
② Thickness of compressible layer
③ compressibility of soil (a_v, m_v, C_c)

Note ⑥ ultimate consolidation settlement will be same in two way drainage and one way drainage (if soil and loading is same)

Determination of secondary consolidation

① It occurs due to plastic rearrangement of soil solid in highly plastic clays and in organic clays

(Not too much imp)

$$S_s = \frac{H_{100} C_s}{1+e_{100}} \log\left(\frac{t}{t_{100}}\right)$$

C_s = secondary compressive index it is the slope of e vs log(time) curve after primary consolidation obtained from laboratory

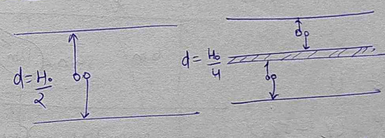
H_{100} = thickness after primary consolidation

e_{100} = Void ratio after primary consolidation

t_{100} = time required for primary consolidation approximately (90%) (t_{90})

t = time at which secondary consolidation is required.

⑥ Solution Page 37



$$T_v = \left(C_v \cdot \frac{t}{d^2}\right) \qquad \frac{T_v}{C_v} = \frac{t}{d^2} = \text{constant}$$

$$\frac{9}{\left(\frac{H_0}{2}\right)^2} = \frac{t_2}{\left(\frac{H_0}{4}\right)^2}, \quad t_2 = \frac{9}{4}\text{ year} +$$

2 years 3 month

W.B. 47 Page 43

I^{st} 60% ⟶ 1 year ($d = H_0$) 1 way

II^{nd} $3a_v$, $6K$, $3H_0$ ($d = \frac{3}{2}H_0$) ⟶ imp 2 way

I^{st} $$T_v = \frac{C_v t}{d^2} = \frac{K}{m_v \gamma_w d^2} \cdot t = \frac{K}{\frac{a_v \cdot \gamma_w}{1+e_0}} \cancel{\times} \frac{12 \text{ years}}{(H_0)^2}$$

II^{nd} $$T_v = \frac{6K}{\frac{3a_v}{1+e_0} \gamma_w \cdot \left(\frac{3H_0}{2}\right)^2} \cdot t$$

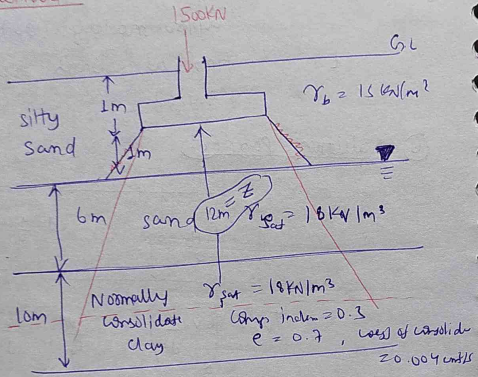
% U same ⟶ (T_v - same)

($t = 13.5$ year)

Solution (48) Page (43)

Four step method

Step (1)



$H_0 = 10m$

Step (2)

$\bar{\sigma_0} = \sigma - U$

$= (2\gamma_1 + 6\gamma_2 + 5\gamma_3) - 11\gamma_w$

$= 2\times(15) + 6\times(18) + 5\times 18 - 11(10) = 118$

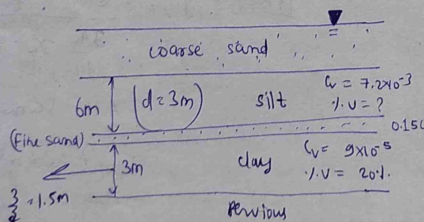
Step (3) $\Delta\bar{\sigma} = \frac{force}{Area} = \frac{Q}{\frac{\pi}{4}\times(D+2nZ)^2} = \frac{1500\ kN}{\frac{\pi}{4}\left(3+2\times\frac{1}{2}\times12\right)^2}$

$\Delta\bar{\sigma} = 8.48$

Step (4) $\Delta H = \frac{H_0 C_c}{1+e_0}\log\left(\frac{\bar{\sigma_0}+\Delta\bar{\sigma}}{\bar{\sigma_0}}\right) = \frac{10\times0.3}{1+0.7}\log\left(\frac{118+8.48}{118}\right)$

$= 0.0538m = 53.8mm$

Solution (31) Page (41)



$d = \frac{3}{2} = 1.5m$

(1) If % U ≤ 60%

For clay $(T_v)_1 = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 = \frac{\pi}{4}\left(\frac{60}{100}\right)^2 = 0.282$

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

$0.282\times(d)^2 = \frac{\cancel{7.2\times10^{-3}}\ 9\times10^{-3}\times t}{}$

$0.282(1.5)^2 = \cancel{7.2\times10^{-3}}\ 9\times10^{-3}\times t$

$t =$

For silt $T_v = \frac{C_v t}{d^2} = \frac{7.2\times10^{-3}}{(3m)^2}\times\left(\frac{\frac{\pi}{4}(0.2)^2\times(1.5)^2}{9\times10^{-5}}\right)$

$T_v = 0.628$ (> 60% $(T_v)_{60} = 0.283$)

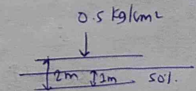
$T_v = 1.781 - 0.933\times\log_{10}(100 - \%U) = 0.68$

doubt $\boxed{\%\ U = 82.79\%}$

Solution (9) Page (38)

logic

Solution (40)



$K = 3mm/year$

$(T_v)_{50} = \frac{C_v t}{d^2} = \frac{K}{m_v\gamma_w}\cdot\frac{t}{d^2}$

$0.196 = \frac{\frac{3mm}{\cancel{10}}\times10^{-3}\ m\times 1\ yr}{m_v\times 9.81\frac{kN}{m^3}\times(1m)^2}$

$m_v = 1.56\times10^{-3}\ m^2/kN$

$\Delta H = H_0 m_v \Delta\bar{\sigma}$

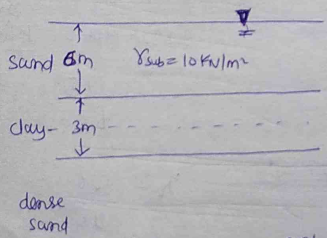
$= 2\times1.56\times10^{-3}\times49.05 = 0.1535$

$= 153.5mm$

After 1 years $\%U = \frac{\Delta h}{\Delta H} \times 100 = 50\%$

$$\Delta h = \frac{\Delta H}{2} = \frac{153.5}{2} = 76.6\,mm$$ Ans

Solution (29)



compression index $= 0.35$

$w = 40\%$

$G = 2.7$

Step ①

$H_0 = 3m$

$\Delta\bar{\sigma} = 120$

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

$10 = \gamma_{sat} - \gamma_w$

$\gamma_{sat} = 20$

Step② $\bar{\sigma} = \sigma - u$

$= \cancel{5\times} \; 1.5\times 20 + 6\times\gamma_{sat} - 7.5\times\gamma_w$

$= 30 + 6\times(10) - 7.5\times(10)$

$= 30 + 60 - 75$

$= 90 - 75 = 15\,KN/m^3$

$= \cancel{3\times(\gamma')} \; 1.5(\gamma') + 6\times\gamma'$ $(1.5\gamma_{sat} + 6\times\gamma_{sat} - 7.5\times 10)$

$\bar{\sigma}_0 = 1.5\times(10) + 6\times(7.01) = 75\,KN/m^2$

$$\Delta H = \frac{H_0 C_c}{1+e_0} \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= \frac{3\times 0.35}{1+1.08} \times \log\left(\frac{72.01 + 120}{72.01}\right)$$

$= 0.2149\,m$

$21.49\,cm \approx 22\,cm$

(30) In 2 months $\longrightarrow$ 4.4 cm

$$\%U = \frac{\Delta h}{\Delta H} \times 100 = \frac{4.4}{22} \times 100 = 20\%$$

$$(T_v)_{20} = \frac{\pi}{4} U^2 = \frac{C_v t}{d^2}$$

$$\frac{\pi}{4} \times (0.2)^2 = \left(\frac{C_v}{d^2}\right) \cdot 2\text{ month}$$

for 50% of consolidation

$$(T_v)_{50} = C_v \frac{t}{d^2}$$

$$0.196 = \frac{\frac{\pi}{4}(0.2)^2}{(2\text{month})} \times t$$

$t = 12.5$ month

$e = \frac{wG}{S}$

$= \frac{0.4\times 2.7}{1}$

$= 1.08$

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

$= 8.01\,KN/m^2$

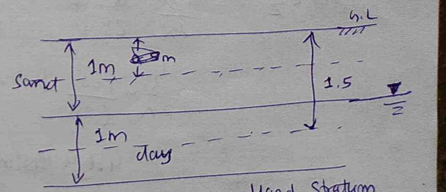
Solution. (51)

$C_R = 0.05$

$H_0 = 1m$

$\bar{\sigma}_1 = 60\,KPas$

$w = 25\%$



Present $(\bar{\sigma}_0)$

$= 0.5\times\gamma_{sat} + 1\times\gamma_{bulk} - 1.5\times\gamma_w$

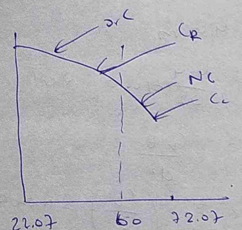
$= 0.5\times(20.15) + 17 - 0.5\times(10)$

$= 22.07\,KN/m^2$

③ change $(\Delta\bar{\sigma}) = 2.5\times 20 = 50\,KPa$

④ final $(\bar{\sigma}_2) = \bar{\sigma}_0 + \Delta\bar{\sigma} = 22.07 + 50$

$= 72.07$



$$\Delta H = \frac{H_0 C_R}{1+e_0} \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right) + \frac{H_0 C_c}{1+e_0} \cdot \log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)$$

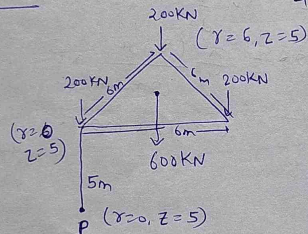
$$= \frac{1 \times 0.05}{1+0.675} \times \log\left(\frac{60}{28.07}\right) + \frac{1.05}{1+0.675} \times \log\left(\frac{72.07}{60}\right)$$

$$= 0.0367\,m$$

$$= 36.7\,mm$$

Page NO. 16

Ans - 16



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

$$= \frac{3}{2\pi} \times \frac{200}{(5)^2} + 2 \times \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{6}{5}\right)^2}\right)^{\frac{5}{2}} \cdot \left(\frac{200}{5^2}\right)$$

$$= 4.64\ KN/m^2$$

CHAPTER-8

Shear strength of soil

SSS of a resistance offered by the soil grain against shear deformation. Soil may deriv its shear strength from following parameter ① friction in b/w particle of sliding and rolling (sand, gravel, silt). ② Interlocking b/w particles (Gravel and dense sand) ③ Intermolecular attraction due to cohesion and adhesion (silt and clay)