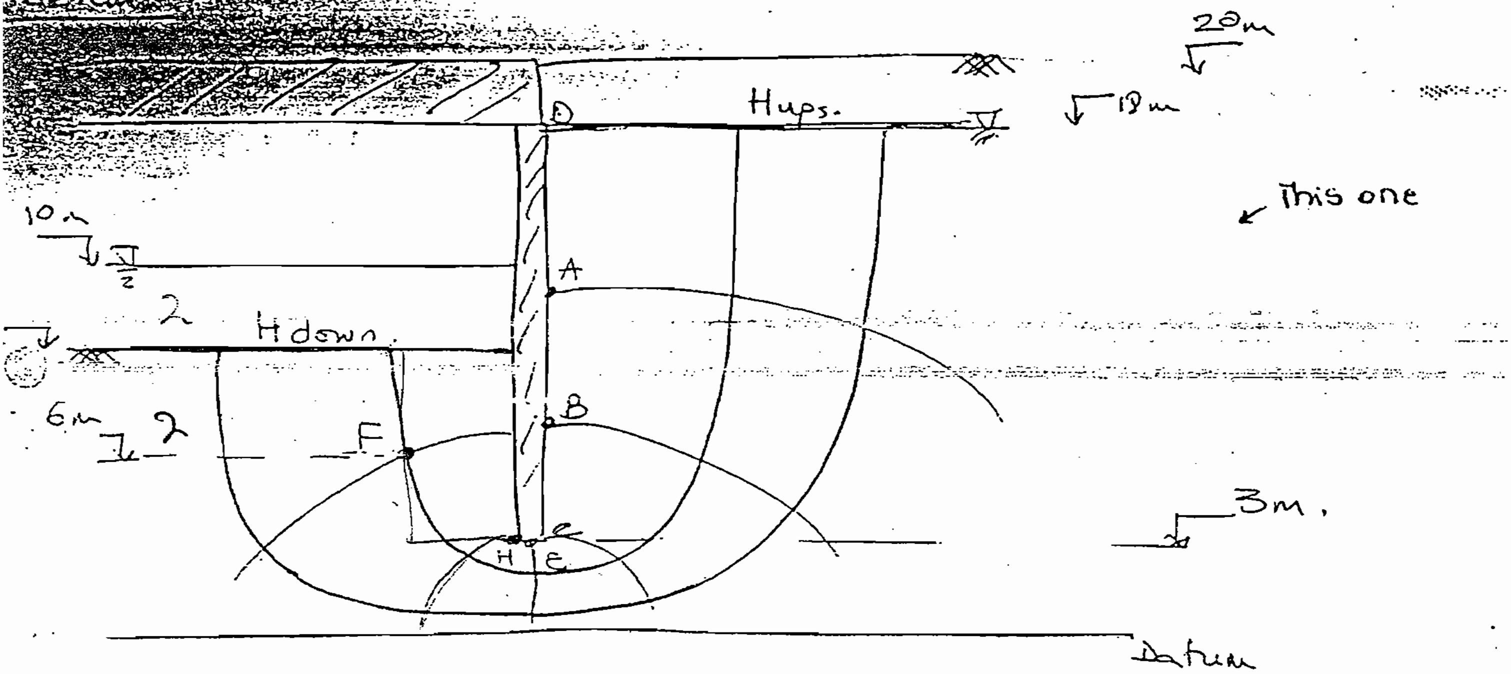
Quiz II
2000

- Pressure head is independent of the choice of datum.
- A soil element will respond to a change in total stress, (FALSE) It will respond to a change in effective stress by an appropriate change in volume.
- The construction of flow nets is useful that it allows us to solve 1-D, 2-D & 3-D problems (False). {No 3-D problems).

Problem 1:



The Flow net is only drawn for WET soil
⇒ It can't extend beyond the water table
not beyond places where there is soil.

: Find effective stress at point F. $(\sigma_F' = ?)$

$$H_F = \frac{P_F}{\gamma_W} + Z_F$$

$$\sigma_F' = \underbrace{\sigma_F}_{\text{total pressure}} - \underbrace{P_F}_{\text{pore water pressure}}$$

$$\sigma_F' = (2 \times \gamma_W + 2 \times \gamma_{SAT}) - P_F$$

But $P_F = ?$

$H_F = H_{upstream} - 6$ drops.

$Z_F = 6$ m.

$H_{upstream} = \frac{P_{ups}}{\gamma_W} + Z_{ups} = 0 + 18 \Rightarrow H_{ups} = 18$ m

~~$H_{ups} = 0 + 8 +$~~

$$\text{Head Drop} = \frac{\Delta H\,(\text{up} - \text{DOWN})}{\#\text{ of Drops}} = \frac{H_{ups} - H_{DOWN}}{\#\text{ of Drops}}$$

$$\therefore \text{Head Drop} = \frac{18 - (2+8)}{7} = \frac{8}{7}$$

$$\Rightarrow H_F = 18 - 6 \times \frac{8}{7}$$

Then $\frac{P_F}{\gamma_W} = H_F - Z_F \Rightarrow P_F = \gamma_W \left(18 - 6 \times \frac{8}{7} - 6\right)$

& then we find σ_F'.

σ_F = weight of everything above the point of interest.

Note: σ'_F can't be found as $\sigma_F + u$ where $u = 4\gamma_w$ & $\sigma_F = 2\gamma_w + 2 \times \gamma_{SAT}$

because there is flow from upstream to downstream, A piezometer at F will rise the water beyond $\underline{\underline{10\ m}}$.

Find the water pressure acting along CD.

$$H_A = \frac{P_A}{\gamma_w} + Z_A$$

what is P_A ??. Z_A = elevation from point A to datum.

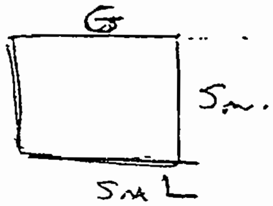
H_A = Hups − 1 × Drop

⇒ we can find P_A, P_B ; " same thing

Square element at toe of the column ??

toe / heel

$$F.S = \frac{\gamma_b}{i_{av} \cdot \gamma_w} = \frac{\gamma_{SAT} - \gamma_w}{i_{av}\ \gamma_w}$$



i_{av} = ?

$$i_{av} = \frac{H_L - H_G}{5}$$

$H_G = H_{down} = 10\ m$

H_L = ? = $(H_L)_{av}$ along the 5 m.

$H_a = H_{ups} - 5 \times Drops$

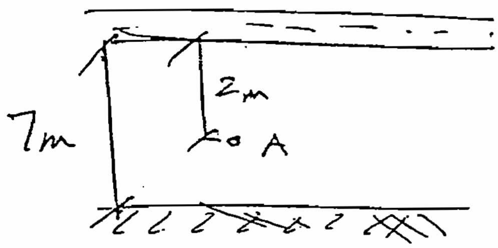
$H_b = H_{ups} - 5.5 \times Drops$

$$(H_L)_{av} = \frac{H_a + H_b}{2}$$

then $$F.S. = \frac{\gamma_{SAT} - \gamma_w}{i_{av} \cdot \gamma_w}$$

Problem - 2 -

This one



Single draining from top

Total Consolidation settlement for a layer of 7m is 30 cms

After 180 days since the consolidation process begins at point A below the drainage layer. pt A has U = 60%

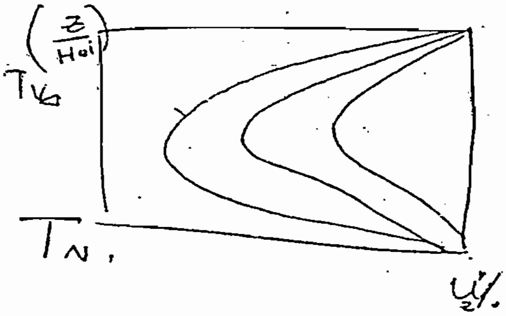
Compute coefficient of consolidation of the clay C_v

) $S_{TOT} = 30$ cm.

At point A, after 180 days $U = 60\%$.

$C_v = ?$

Not all points reach the same level of consolidation at the same time.



$\frac{z}{H_{dr}} = \frac{2}{7}$;, $U_z = 60\%$

using graph we find T_v.

T_v = time factor at which point A reaches 60% cons

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

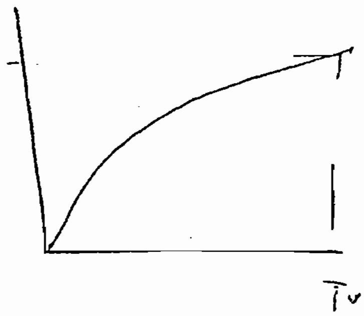
$t = 180$ days $\quad H_{dr} = 7$m

$T_v = \checkmark$

then C_v is found.

The C_v obtained is a property of the soil. (time has nothing to do with C_v.)

U%



for
...ole
...yer.

It gives an average degree of consolidation.

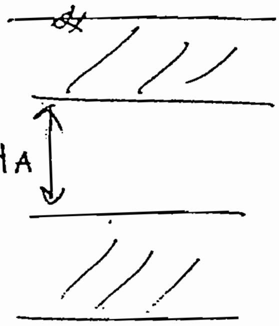
After 180 days, settlement of layer = ?

For C_v known get T_v & go to graph ($U\%$) to find

$S_{180} = 30 \times U\%$.

Problem-3-

* Settlement = 6 cm $\longrightarrow t = 4$ years
Assumption Double Drainage.
$S_{TOT} = 25$ cm.



Further investigation showed that clay could only drain from bottom side only.
$\Rightarrow$ SINGLE DRAINAGE

1) For SINGLE DRAINAGE, Estimate ultimate total settlement.
$S_{TOT} = 25$ cm. (time will be longer ONLY)

2) ~~Estimate time required~~ for 6 cm settlement to take place.

$\Rightarrow$ For 6 cm $t_2 = 4 \times t_1 = 16$ years.

) In reality, the thickness of layer is $1.2 \times H_A$.
What is the total settlement for this case.
$S_{TOT_{NEW}} = 1.2 \times S_{TOT_{OLD}} = 1.2 \times 25 = 30$ cm.

since S is proportional to thickness of layer

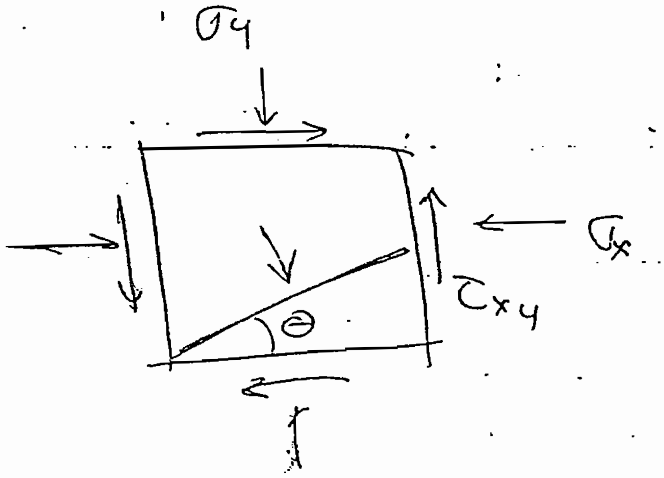
$$\frac{De \times H_0}{1+e_0} = S_{TOT}.$$

Estimate settlement after 4 years.

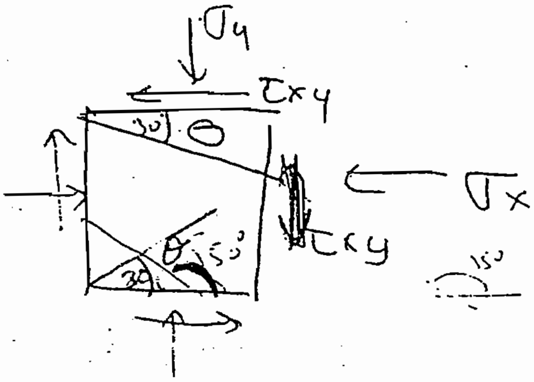
C_v is known
t is known
H_{new} is $1.2 H_A$.

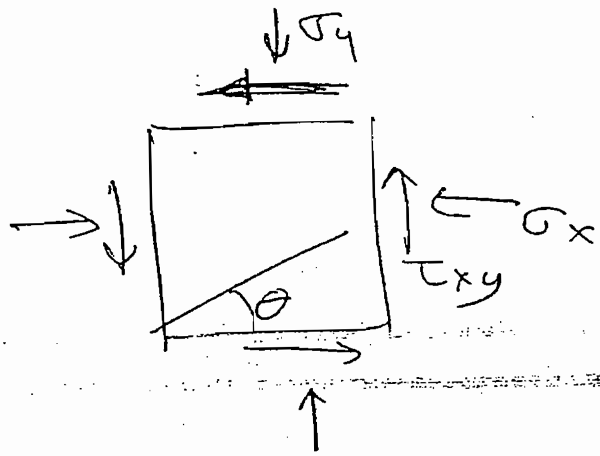
$$T_v = \frac{C_v t}{(1.2 H_A)^2}$$

⇒ T_v for new case is known
get U ≈
then multiply by S to get
settlement after 4 years



$$\sigma_N = \frac{\sigma_y + \sigma_x}{2} + \frac{\sigma_y - \sigma_x}{2}\cos 2\theta + \tau_{xy}\sin 2\theta$$





$$\tau_N = \frac{\sigma_y - \sigma_x}{2}\sin 2\theta - \tau_{xy}\cos 2\theta$$

QUIZ - II

CE 084
Soil Mechanics

SPRING 2000
Professor: Salah Sadek

CLOSED BOOK/NOTES, 1 1/2 HOURS

Name: Karim Nasser **ID #:** 97-02675

IMPORTANT NOTES:

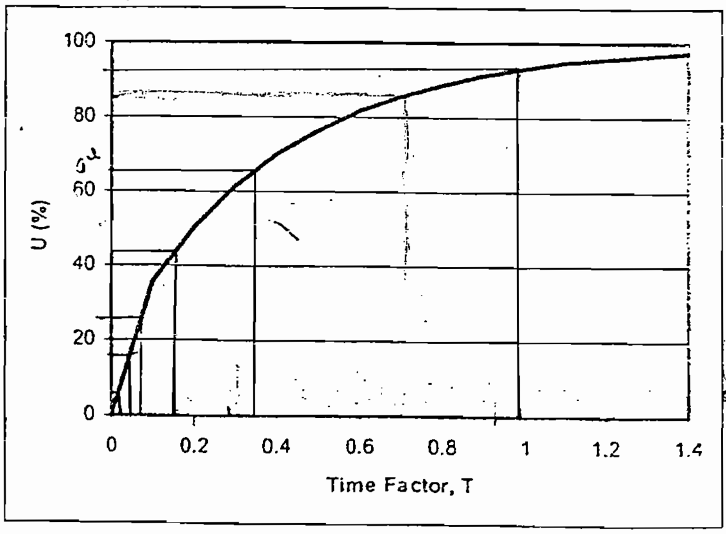
- Manage your time carefully (quickly read through all the problems before starting the exam, & do not spend an hour on one problem!).
- Whenever, you are "stuck" or feel that you need some information not provided, just assume, justify your assumption and proceed.
- Be NEAT!!!! Write Clearly and clearly highlight your answers.

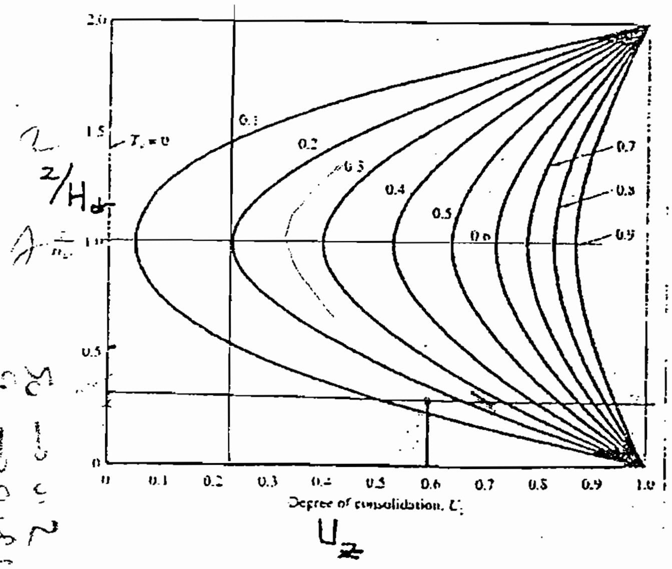
Good Luck!

TECHNICAL NOTES:

The following might be of help:

Unit weight of water, $\gamma_w = 9.81\ kN/m^3 = 62.4\ lb/ft^3$
Consolidation:
Time factor $T=c_v.t/H^2$
Relation between Time Factor and U (% consolidation)





PROBLEM -1 (20 Pts)

Indicate where appropriate whether the following statements are true or false by circling the T or F respectively on the line to the right. (No penalties will be assessed for wrong answers). Other problems may require a short answer in the space provided.

a. (2pts) Which of the following is independent of the choice of datum. Check the correct answer(s):

(—)

- [x] Pressure head
- [x] ~~Total~~ head
- [ ] Elevation head

b. (2pts) A soil element will respond to a change in eff total stress by an appropriate change in volume (densification if the total stress increases)

Effect

T F

c. (2pts) Tarzan was right in being terrified of sinking in *Quick Sand.*

T F

d. (2pts) The construction of *flow nets* is useful because it allows us to solve graphically 1-D, 2-D and 3-D flow problems.

No (3-D)

T F

e. (2pts) For any given problem geometry, the flow net drawn is independent of the value of the hydraulic conductivity of the soil, provided that the soil is homogeneous and isotropic.

yes

T F

f. (3pts) The maximum past effective pressure of a clay sample was determined to be 4000 lb/ft^2. The sample was obtained from a depth of 25 ft below the ground surface. The unit weight of the soil is 110 lb/ft^3. The sample, as it was in the field, was *Normally Consolidated.*

~~$\sigma'_c = 4.4$ lb/ft~~ $\sigma' = 2750$ $\frac{\sigma'_p}{\sigma'} = 1.45 > 1$

~~$\frac{\sigma'_p}{\sigma'_o} = 909.1 > 1$~~ → Not Normally consolidated

T F

g. (4pts) It takes one hour for a sample in a consolidation test to reach a degree of consolidation of 85%. The sample is free draining at both ends and is *one inch thick.* A 30 ft thick layer of this clay in the field, also drained at top and bottom, will reach 85% consolidation in about 60 years.

by curve $T = 0.72 \Rightarrow \frac{C_v \times 1}{(\frac{1}{24})^2} \Rightarrow C_v = 1.25\times10^{-3}$ → H_v

T F

2nd way $T = \frac{1.25\times10^{-3} \times 60\times365\times24}{(15)^2} = 2.92$ rejected

h. (3pts) The soil below the dam shown in Figure-A has anisotropic hydraulic conductivities: $k_z > k_x$. In order to solve the flow problem using a flow net, Engineer-1 drew the transformed section of the dam by modifying the scales; his attempt is shown in Figure-B. Engineer-2 arrived at a completely different section, shown in Figure-C. Which Engineer drew the transformed section correctly?

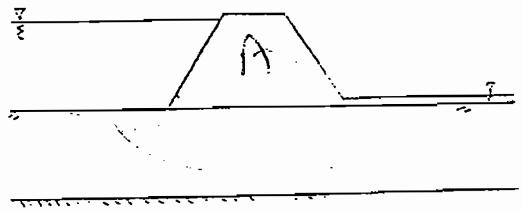


Figure -A

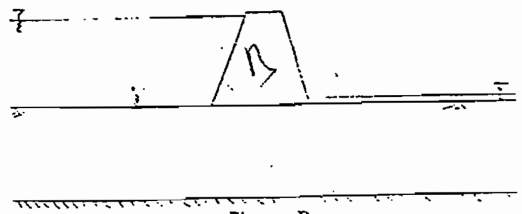


Figure -B

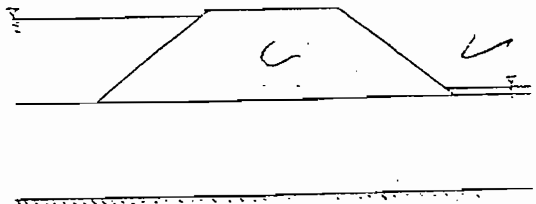


Figure -C

C.

Problem-2 (30 Points)

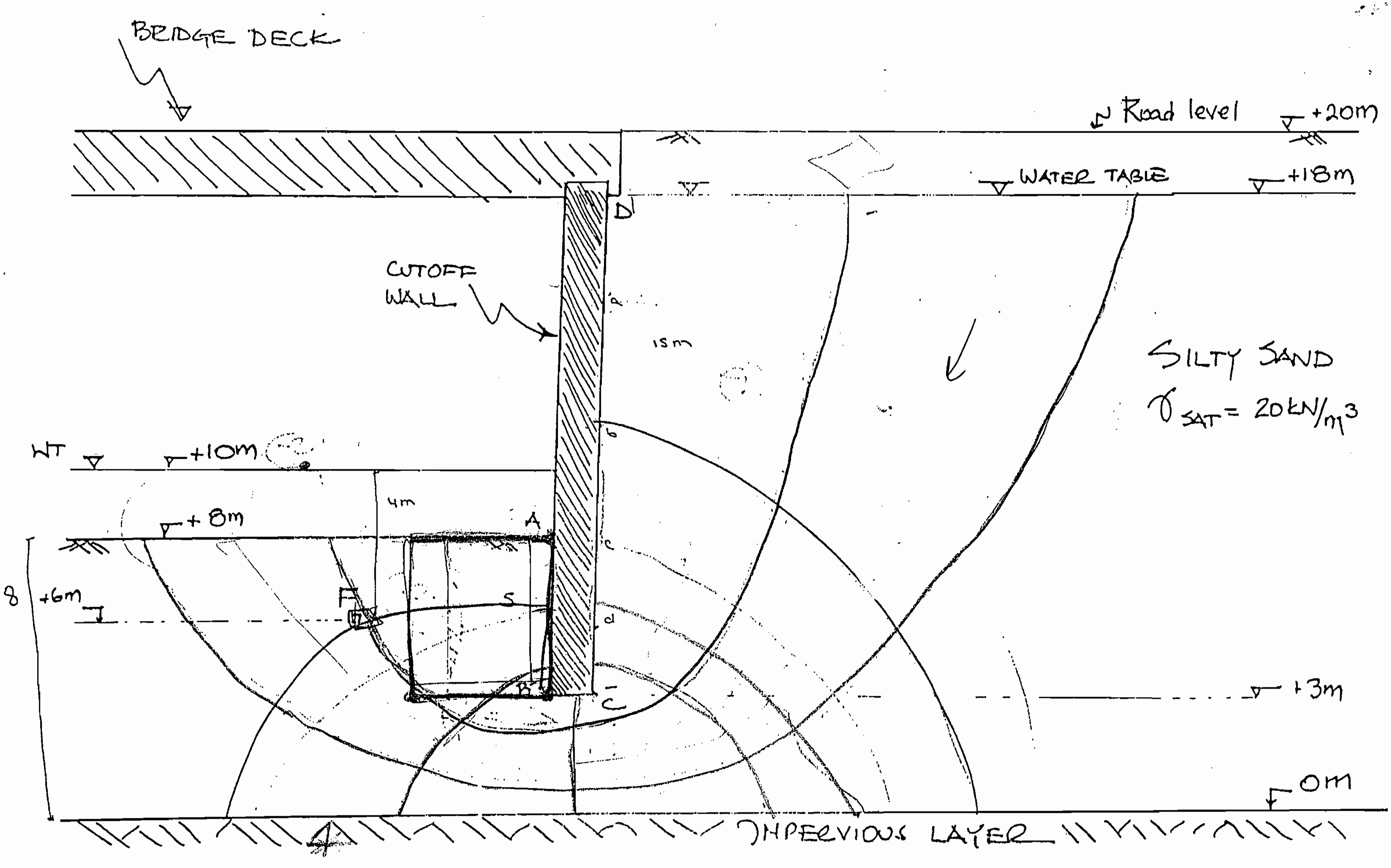
A section of a Bridge abutment is shown in the attached Figure (Page-3a). Given the water levels as shown on either side of the Sheet-Pile cutoff wall answer the following questions:

Note: Dimensions and levels are indicated on the Figure. Heights and lengths can be scaled from the figure. You need to draw the flow net in order to answer the questions below. (Try 3 flow channels)

(a) 10pts - Calculate the vertical effective stress at point F (shown on the figure).

(b) 10pts - Evaluate the variation of the water pressure along length of the sheet pile wall C-D. You may obtain an acceptable answer by calculating the uplift pressures at a number of points along C-D.

(c) 10pts - The zone of concern in terms of stability with respect to "boiling" is at the toe of the wall AB. It is assumed that this zone is represented by a square soil element of side AB. Calculate the Factor of safety against "boiling" conditions at the toe of the wall.

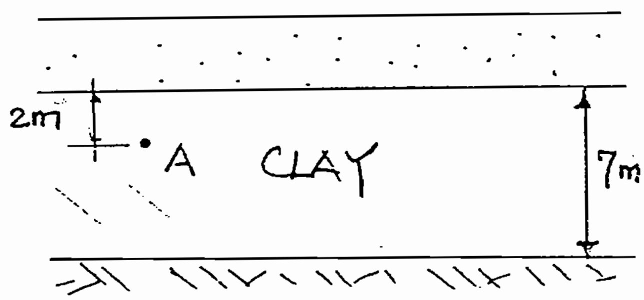

BRIDGE DECK
Road level
+20m
WATER TABLE
+18m
D
CUTOFF WALL
a
15m
b
SILTY SAND
$\gamma_{SAT} = 20 kN/m^3$
WT
+10m
4m
A
+8m
c
8
+6m
F
S
d
B
C
+3m
0m
IMPERVIOUS LAYER

PROBLEM 3 (25Pts)

The total consolidation settlement for a compressible layer 7m thick is estimated to be about 30cms. After about 6 months (180 days), a point 2m below the top of the singly drained layer (point A), had a degree of consolidation of 60%.

(a) – Compute the coefficient of consolidation of the clay, c_v, in m^2/day.

(b) – Compute the settlement of the clay layer at 180 days.



PROBLEM 4 (25Pts)

The settlement analysis for a proposed structure indicates that 6cm of settlement will occur in 4 years and that the ultimate total settlement will be about 25cm. The analysis is based on the assumption that the compressible clay layer is drained at both its top and bottom surfaces. Further investigations showed that the clay could only drain at the bottom surface.
For the new case of single drainage:

(a) - 5Pts Estimate the ultimate total settlement

(b) - 10Pts Estimate the time required for 6cm settlement to take place.

(c) – 10Pts Further investigation revealed that in reality, the thickness of the compressible clay layer is 20% greater than assumed in the settlement analysis. For this new case, estimate the new ultimate total settlement, **and**, the settlement after 4 years.

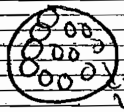
Q. $\frac{q}{A} = v = K \cdot i$

Darcy velocity.

$V \neq V_s$ seepage vel because there are pores ~~and~~ in the area A.

assumptions

$V_s = \frac{q}{A_v}$ (area of voids) $\quad \boxed{\frac{V_s}{v} = \frac{A}{A_v}} \quad \boxed{\frac{A_v}{A} = \frac{V_v}{V}} = n$



voids

$\boxed{V_s = \frac{V}{n}}$ ⇒ velocity of seepage = $\frac{\text{discharge vel}}{n}$.

Parameters affecting k?

Internal structure and Pore space: grain size dist

e

S

part shapes.

Fluid related: Density

viscosity

K = hydraulic conductivity

k = absolute permeability. $\quad k = \frac{K \times \text{viscosity}_f}{\gamma_f} \; [L^2]$

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cm/sec.

clean gravel
coarse sand
Fine sand
Silty clay
clay

Obtaining K (conductivity)

bility ↓

A. Predictions / Correlation / Estimates
B. Lab. ← Constant Head, Falling Head, Cte flow rate test.
C. Field

Cte flow rate test: go to lab, $q = k\, i\, A$

$$q = K \frac{\Delta H}{\Delta L} A$$

A- 1- Hazen Formula.

Purely ~~imper~~ empirical. $\boxed{K = Cte \times D_{10}^2}$ (cm/s) ↓ (mm)

$1 \to 1.5$

D_{10} ~~D~~ is used because small particles controls the passage.

2- Semi-theoretical

Kozeny-Carman EQN (Capillary Model)

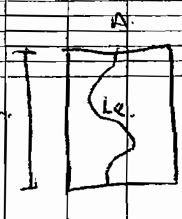
real represented by tubes.

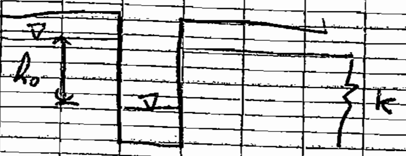
$$K = \underbrace{\boxed{\frac{\rho g}{\mu}}}_{\text{Fluid properties}} \cdot \frac{1}{K_0} \cdot \underbrace{\frac{1}{S_0^2 T^2}}_{} \cdot \underbrace{\frac{e^3}{1+e}}_{\text{void ratio}}$$

T = tortuosity (path ≠ straight) $= \frac{L_e}{L}$

specific surface area
i.e. (size of capillaries)

related to shape of cross-section capillary.
$K_0 \to$ circle $= 2$.
Ellipse
[illegible] 1.78



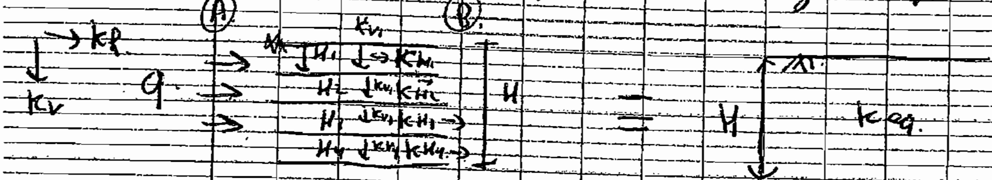


NAVAC Manual.

Equivalent k.

≠ Homogenous

A. Horizontal stratification – Horizontal flow



$$q = q_1 + q_2 + q_3 + \dots + q_n$$

$$q = V \cdot A = V \cdot 1 \cdot H$$

$$V_i = k_i \, i_i$$

$$K_{Heq} \cdot i_{eq} = V_1 \cdot 1 \cdot H_1 + V_2 \cdot 1 \cdot H_2 + \dots \; V_n \cdot 1 \cdot H_n$$

$$= K_1 i_1 H_1 + K_2 i_2 H_2 + \dots + K_{Hn} i_n H_n$$

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

$$K_{Heq} = \frac{K_{H1} H_1 + \dots + K_{Hn} H_n}{H}$$

$$\boxed{K_{Heq} = \frac{\sum K_{Hi} H_i}{\sum H_i}}$$

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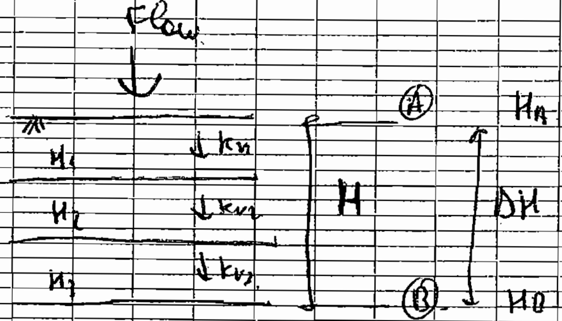


$$\Delta H = \Delta H_1 + \Delta H_2 + \Delta H_3 + \dots$$

A=1.

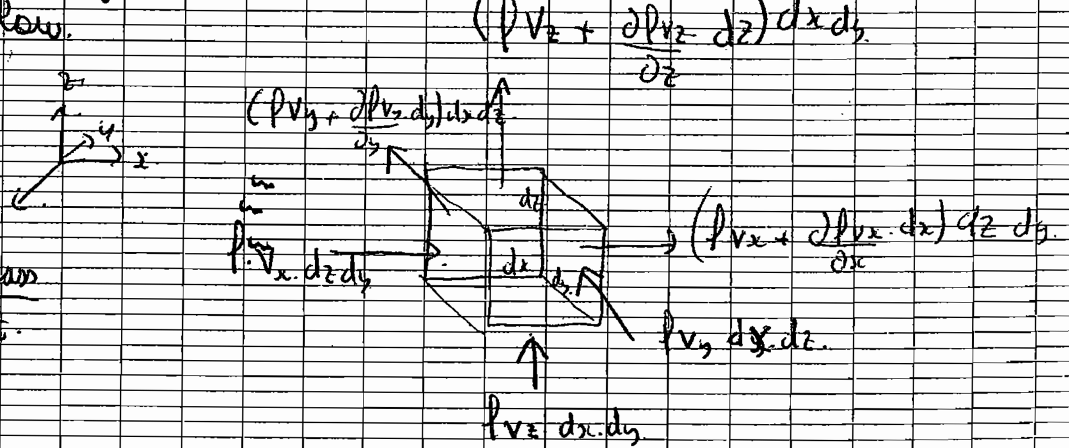
$$A \cdot \frac{\Delta H}{H} \cdot k_{veq} = q$$

$$\frac{\Delta H_1}{H_1} \cdot k_1 \cdot 1 = q_1$$

$$\frac{H q}{k_{veq}} = \frac{H_1 q}{k_{v1}} + \frac{H_2 q}{k_{v2}} + \dots$$

$$\boxed{k_{veq} = \frac{\sum H_i}{\sum \frac{H_i}{k_{vi}}}}$$

Seepage Flow.



$\frac{Mass}{t}$

inflow - outflow = 0 No sinks or sources.

$$\left(\frac{\partial}{\partial x}\rho v_x + \frac{\partial}{\partial y}\rho v_y + \frac{\partial}{\partial z}\rho v_z\right) dx\, dy\, dz = 0.$$

If fluid is Water

1). incompressible $\Rightarrow \rho = cte$

$$\rho\left[\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z}\right] dx\, dy\, dz = 0.$$

2). Darcy's law applies

$v_x = k_x i_x = k_x \cdot \frac{\partial h}{\partial x}$ $\boxed{h = z + \frac{P}{\gamma_w}}$

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial h}{\partial z}\right) = 0.$$

3). Homogeneous

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0.$$

4). Isotropic $\Rightarrow k_x = k_y = k_z$

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0. \Leftrightarrow \boxed{\nabla^2 h = 0.}$$

Laplace

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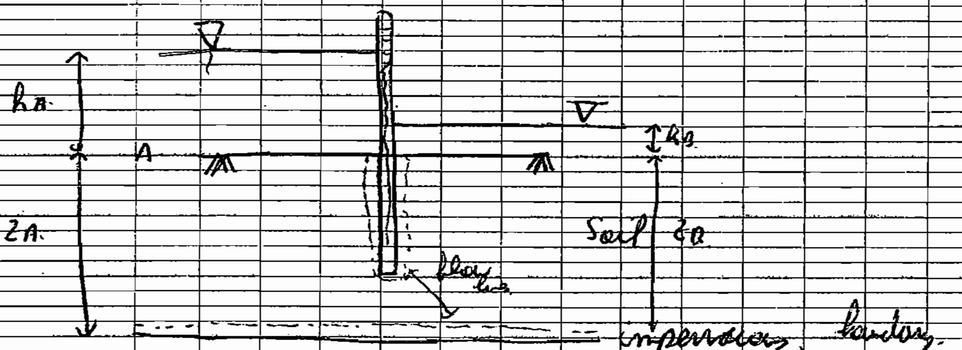
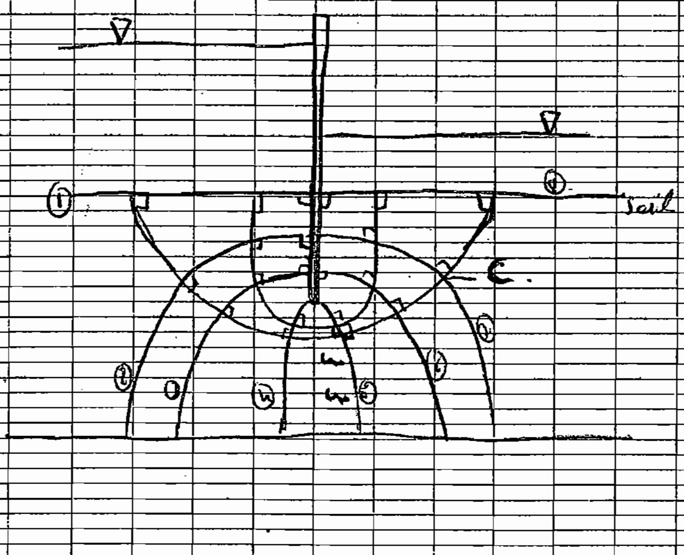
Flownets: graphical solution of $\nabla^2 H = 0$. 2D

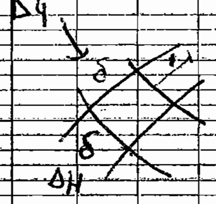


To draw flow lines and Equipotential lines.

Cond: I) anything intersection of FL and EL is $\perp$

II) Elements defined by 2 FL and 2 EL is a square





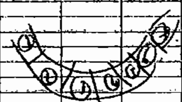
2, 4, 7, 12, 14, 15, 16, 17

$$\Delta q = k \cdot i \cdot A$$
$$= k \frac{\Delta H}{\delta} \delta \cdot 1$$

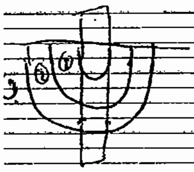
$$\Delta q = k \, \Delta H$$

$$\Delta H = \frac{(H_A - H_B)}{N_d}$$

N_d → no. of drop ⇒ 7 in pic.



$$\Delta q = k \cdot \frac{(H_A - H_B)}{N_d}$$



no. of flow

$$q = N_f \cdot \Delta q$$

$$\boxed{q = \frac{N_f}{N_d} k (H_A - H_B)}$$

What is the Pressure at c: U_c or P_c

$$H_C = H_A - \frac{(H_A - H_B)}{7} \times 6 = H_B + \left(\frac{H_A - H_B}{7}\right) \times 1$$

$$H_C = z_c + \frac{P_c}{\gamma_w}$$

If $k_x \neq k_z$

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

length $L = 50$

$$L' = \sqrt{\frac{k_z}{k_x}} \times L$$

divide by k_z

$$\frac{\partial^2 h}{\left(\frac{k_z}{k_x}\right) \partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad x' = \sqrt{\frac{k_z}{k_x}} \, x \Rightarrow \boxed{\frac{\partial^2 h}{\partial x'^2} + \frac{\partial^2 h}{\partial z^2} = 0}$$

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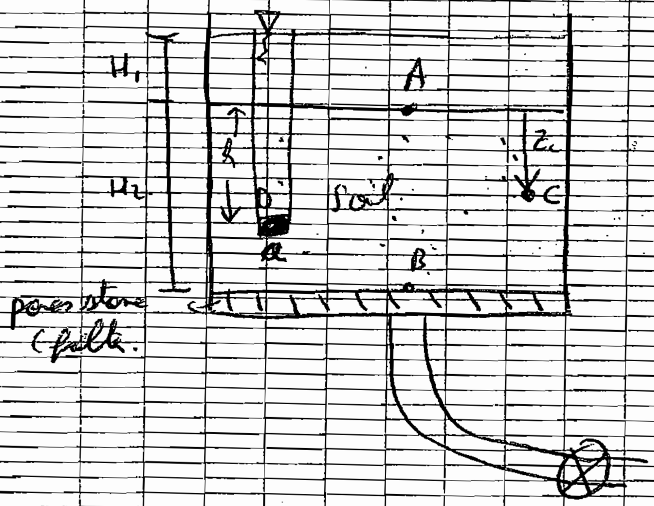
Chap 5 Stress in a soil mass

Effective Stress Concept.



When the valve is closed, the head at A = head at B = head at C because there is no flow.

~~Stress~~ Total Stress: stress that is resulting of whatever is in a column ~~res~~ extending from a pt upward

Vertical stresses: total stress at D = σ_D = ~~$H_1 \gamma$~~ $\dfrac{W_{water} + W_{soil+water}}{Area}$

$$\sigma_D = \frac{\gamma_w (H_1 \cdot a) + \gamma_{sat}(h \cdot a)}{a}$$

$$\boxed{\sigma_D = \gamma_w H_1 + \gamma_{sat} h}$$

Total stress is resisted by: 1) Water Pressure $\boxed{u}$.
2) Soil ~~Skeleton~~ Skeleton.
Effective stress

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

σ: total pres. σ': Effective stress. u: water pres.

Vertical $\boxed{\sigma' = \sigma - u}$

σ': reflects level of stress caused by soil skeleton.

$u_D = (H_1 + h)\gamma_w$

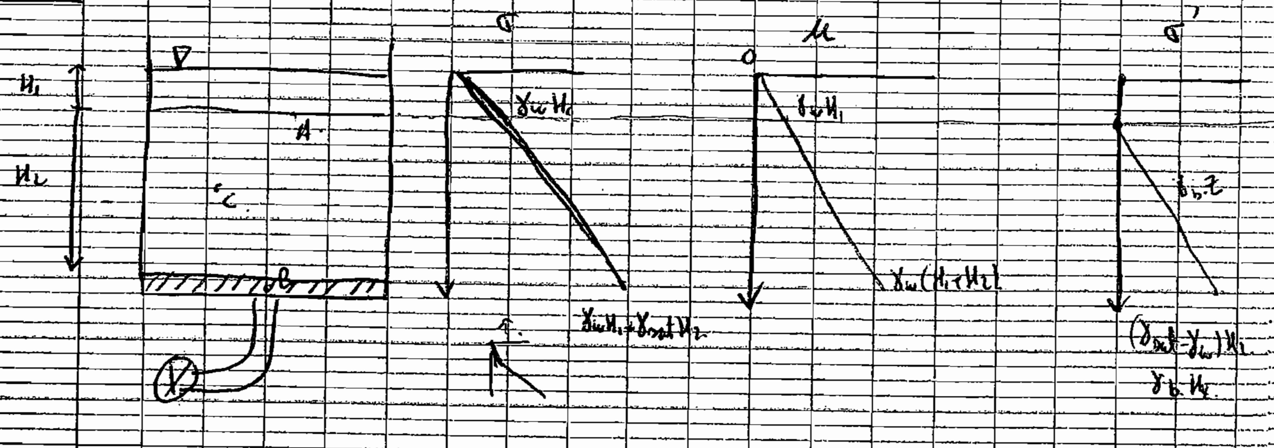
$\sigma'_D = \underbrace{(\gamma_{sat} - \gamma_w)}_{\gamma_b \text{ or } \gamma'} h$

If we change the height of water, it doesn't affect the soil skeleton.

contact stress can be higher than the eff stress.

effective stress is the avg value over the whole area.

No Flow



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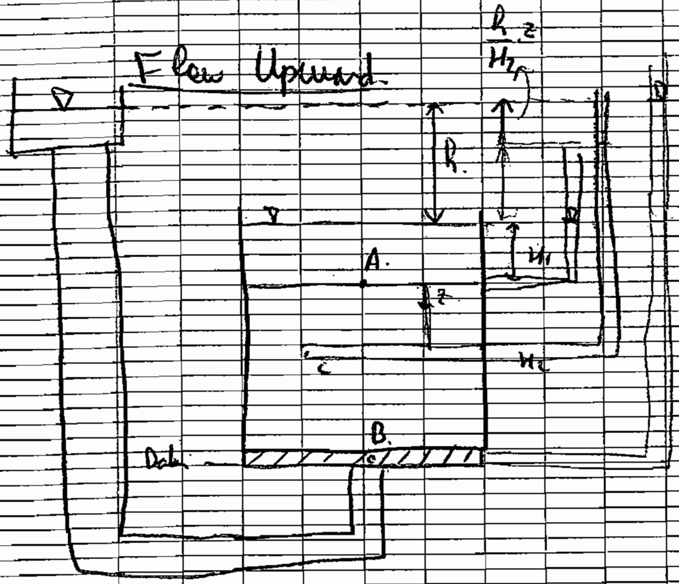
Flow Upward



$\sigma'_{da} < \sigma'_{hydrostatic}$ (no flow)

elev.

$H_A = H_2 + H_1$

$H_B = 0 + H_2 + H_1 + h$

$H_B - H_A = h > 0$ flow from B → A

At pt A.

Total stress = $H_1 \gamma_w$

Pore Water Pressure = $u_A = H_1 \gamma_w$

Eff stress = $\boxed{\sigma'_A = \sigma_A - u_A = 0}$

At pt B.

$\sigma_B = H_1 \gamma_w + H_2 \gamma_{sat}$

$u_B = (H_1 + H_2 + h)\gamma_w$

$\boxed{\sigma'_B = H_2 \gamma_b - h \gamma_w}$

At pt C.

$\sigma_C = H_1 \gamma_w + z \gamma_{sat}$

$u_C = (z + H_1 + \frac{h}{H_2} z)\gamma_w$

$\boxed{\sigma'_C = z\gamma_b - \frac{h}{H_2} z \cdot \gamma_w}$

Quick condition:

$\sigma = u \Rightarrow \sigma' = 0$

no contact btw particles

Water is controlling.

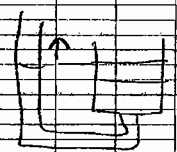
Shear strength of soil = 0.

$\sigma'_c = 0 \quad \cancel{z}\,\gamma_b = \frac{h}{H_2}\,\cancel{z}\,\gamma_w.$

$$\boxed{\gamma_b = \frac{h}{H_2}\gamma_w.} \qquad \frac{h}{H_2} = i_{critical}.$$

$$\boxed{i_{cr} = \frac{h}{H_2} = \frac{\gamma_b}{\gamma_w}} \Rightarrow \text{Quick condition.}$$

Have to raise the reservoir $\frac{\gamma_b}{\gamma_w} \times H_2 = h$



because $\gamma_{person} \approx \gamma_w$ and $\gamma_{sat} > \gamma_w$ ⇒ At some point a person will float in quicksand because of large buoyant force.

Downward seepage:

$\sigma' > \sigma'_{hydrostatic}$

$$\boxed{\sigma'_B = H_2\gamma_b + h\gamma_w.}$$

Seepage Force: $\boxed{\frac{J}{V} = i\gamma_w}$

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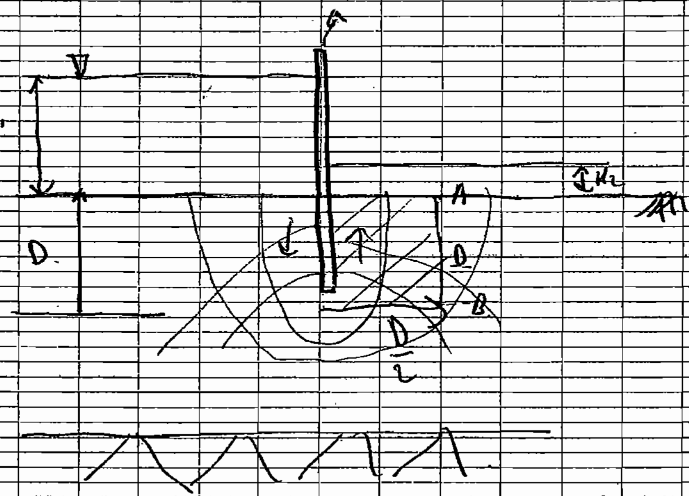
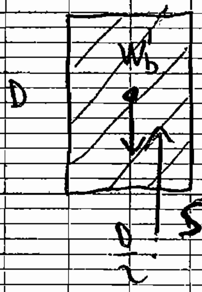
Heave.

Base stability. stability coming from soil (passive direction) →





Seepage force acts in the same dir of flow.

$S = i \cdot \gamma_w$

Factor of Safety $= \frac{W'}{S}$

FS > 1.

1.2 → 1.5.

2.0.

$\frac{W'}{S}$ $\quad W' = \gamma_b \times D \cdot \frac{D}{2} = \frac{1}{2} \gamma_b D^2$

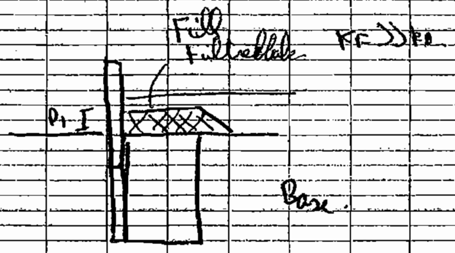
$S = i \gamma_w \cdot D \frac{D}{2}$ $\quad FS = \frac{\gamma_b}{i \gamma_w}$ $\quad \boxed{FS = \frac{\gamma_b}{i_{av} \gamma_w}}$

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

$i_{av} = \frac{H_{B} - H_A}{D}$ $\quad$ Draw flow line to get H_B.

Before const: If ${}^{\text{calculated}}\uparrow$ FS = 2.0 and we want FS = 2.5 ⇒ we increase D.

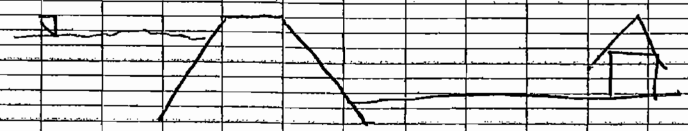
After const: If we want to ↑ FS,



$$FS = \frac{W' + W'_F}{S} = \frac{\gamma_b + \left(\frac{D_1}{D}\right)\gamma_F}{i_{av}\,\gamma_w}$$

$$W'_F = \frac{D_1 + D}{2} \times \gamma_{bF}.$$

Levees:



Filter Criteria.

Filter associated to a base material:

1. Coarse enough to have very little head loss through it.

   $k_F \gg k_{base}$. $\boxed{\frac{(D_{15})_{fil}}{(D_{15})_{base}} > 4\text{-}5.}$

2. Fine enough to trap the base material.

   $\boxed{\frac{(D_{15})_F}{(D_{85})_{base}} < 4\text{-}5.}$

Ex: $(D_{15})_F \approx 5\,(D_{15})_B$

$k_F = ?\ k_B$ Hazen Formula: $k = ct\,(D_{10})^2$.

$k_F = 25\ k_B$
(5^2)

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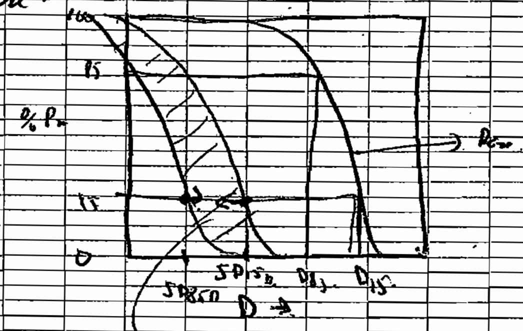
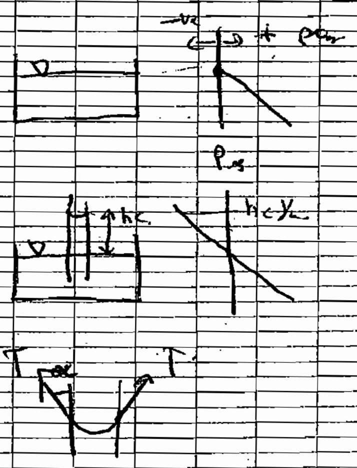


Range of acceptable.

Capillarity:

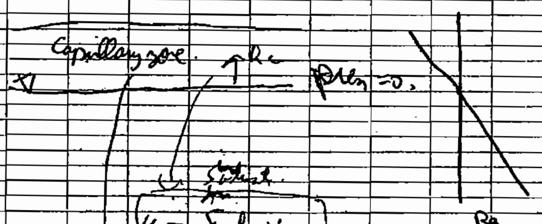
Capillary Rise.



$W = T\cos\alpha \cdot \pi d.$

$W = h_c \frac{\pi d^2}{4} \cdot \gamma_w.$

$$\boxed{h_c = \frac{4T\cos\alpha}{\gamma_w d}}$$ → diam



$u = \frac{S}{100} h_c \gamma_w$

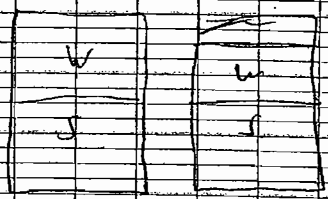
Take press = 0.

Consolidation.

6.1→6.12.

Compaction: densification resulting from reduction of air volume.

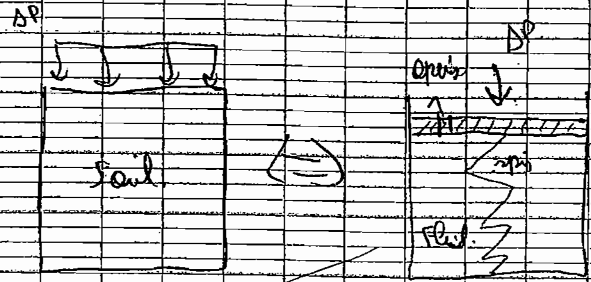
Def =



Push water out.

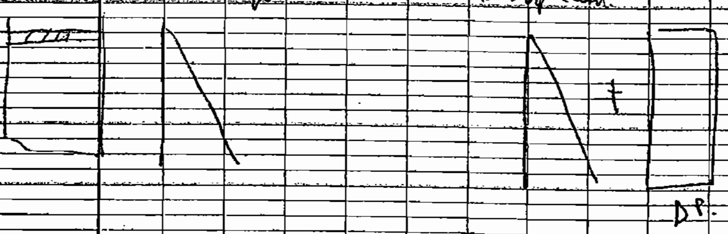
high permeability → ↓ time for consolidation

The opening ↑ of possibility is high



If fluid is incompressible

Press before load   Press after load



Immediately after adding the load the water will take this load.

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

$t=0 \quad = \Delta\sigma + 0$

$t=\infty \quad = 0 + \Delta\sigma$

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$\Delta\sigma$

2.

SAND

Clay

SAND

Infinite load → $\Delta\sigma$ is the same with depth.

In this case → drainage from both sides

$\Delta\sigma = \Delta\sigma' + \Delta u$.

at $t = 0$.

OEDOMETER Test

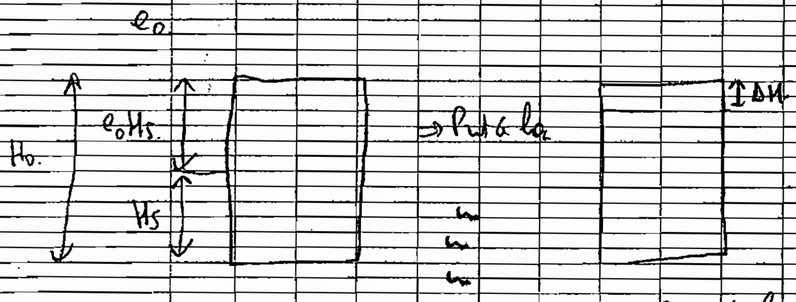
soil → soil in a ring.

very rigid.

(meniscus)

~~meniscus~~

Setup placed under water to break the meniscus (boundary between atmosphere and soil)



e_1 @ end of loading step

~~$e_1 = e_0 + \Delta e$.~~

$e_0 = e_1 + \Delta e$.

$\Delta H = \Delta e H_s$

$H_0 = (1 + e_0) H_s$.

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

Consolidation.

settlement

How much — How long

Does it take to develop.

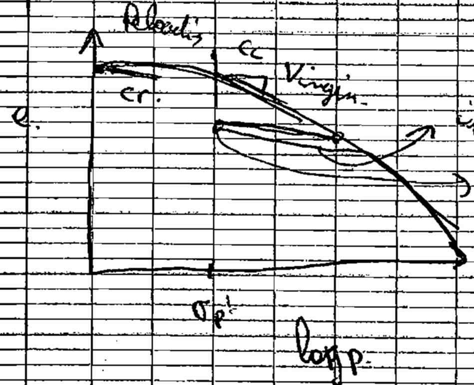
In test:

take a sample under 20m the surface: the sample is going to rebound a little

So e_0 is at the lab condition not in the field under 20m

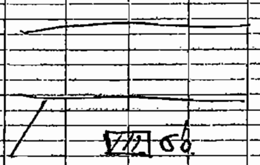
Reloading

After reloading (Virgin comp phase), the rate of deformation changes



if unloaded ⇒ same slope as Reloading

if we reload ⇒ " " " "



8. Compare σ_0' and σ_p'

If $\sigma_0' = \sigma_p'$ ⇒ n.c. (Normally Consolidated Clay) $\frac{\sigma_p'}{\sigma_0'}$ = OCR

$\sigma_p' > \sigma_0'$ ⇒ OCR > 1 ⇒ Good because settlement because it has seen it before.

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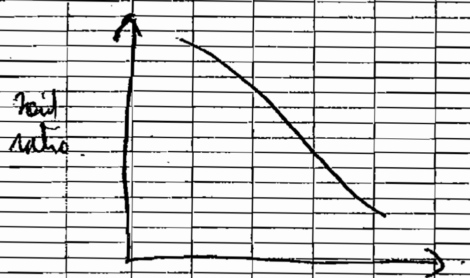
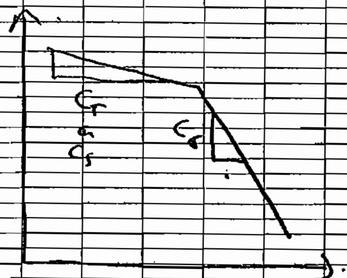
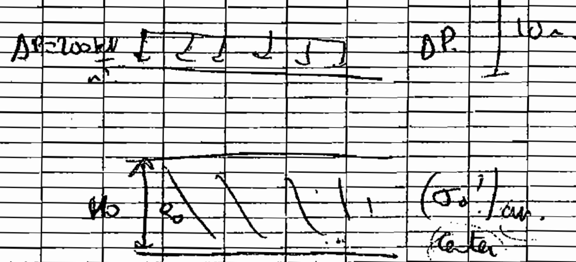
Sample taken ~~took~~ from 20m.



⇒ Sample has been disturbed.



$\gamma = 20\,kN/m^3$



Calculate ΔH.

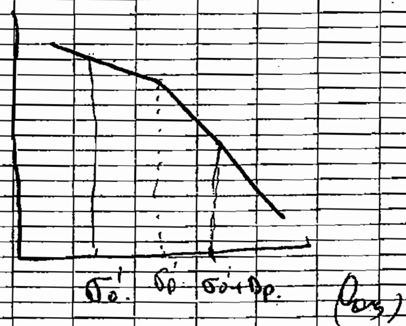
$$\Delta H = \frac{\Delta e}{1+e_0} \cdot H_0$$

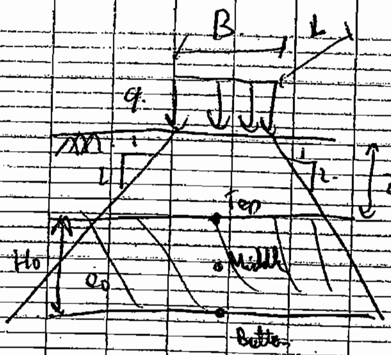
How to calculate Δe.

3 cond:

If $\sigma'_0 + \Delta p > \sigma'_p$

$$\boxed{\Delta H = \frac{C_r H_0}{1+e_0} \log \frac{\sigma'_p}{\sigma'_0} + \frac{C_c H_0}{1+e_0} \log \frac{\sigma'_0 + \Delta p}{\sigma'_p}}$$





$q \cdot B \cdot L = \text{load} = Q$

$(\sigma_o')_{av}$

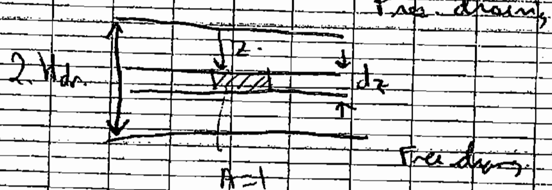
How to Calculate ΔP.

Use 2:1 Approximation

$\Delta\sigma$ $\boxed{\sigma_{(z)} = \dfrac{Q}{(B+z)(L+z)}}$

$$\boxed{\Delta\sigma = \frac{\sigma_{top} + 4\sigma_{middle} + \sigma_{bott}}{6}}$$

Terzaghi.



At $t=0 \rightarrow u = u_o$

$t>0$ $u=0$ @ $z=0$, $z=2H_{dr}$

inflow - outflow = ΔV.

u = pore press (z, t) $\boxed{\dfrac{k(1+e_o)}{a_v \cdot \gamma_w} \dfrac{\partial^2 u}{\partial z^2} = \dfrac{\partial u}{\partial t}}$

C_v = coeff of consolidation

$= \dfrac{k(1+e_o)}{a_v \cdot \gamma_w}$

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

1. @ $t=0$ $u = \Delta p$ at all z $0 \le z \le 2H_{dr}$
2. @ $t>0$ $u=0$ at $z=0$, $z=2H_{dr}$

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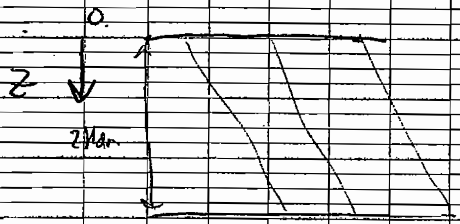
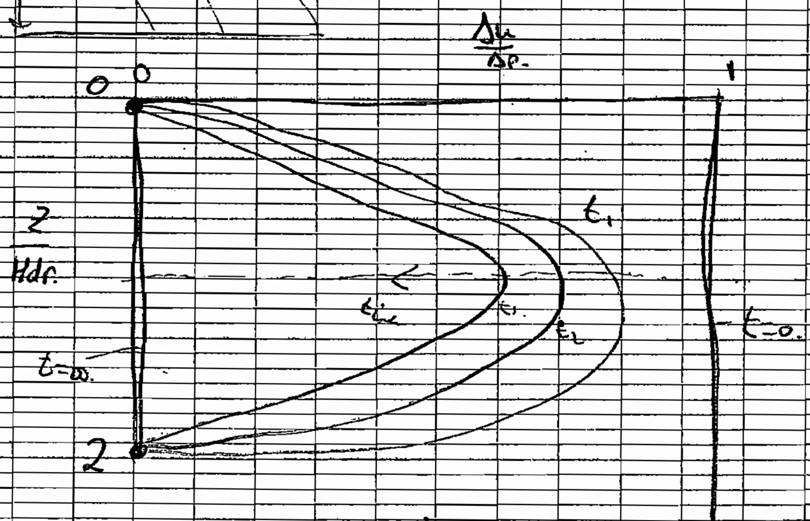
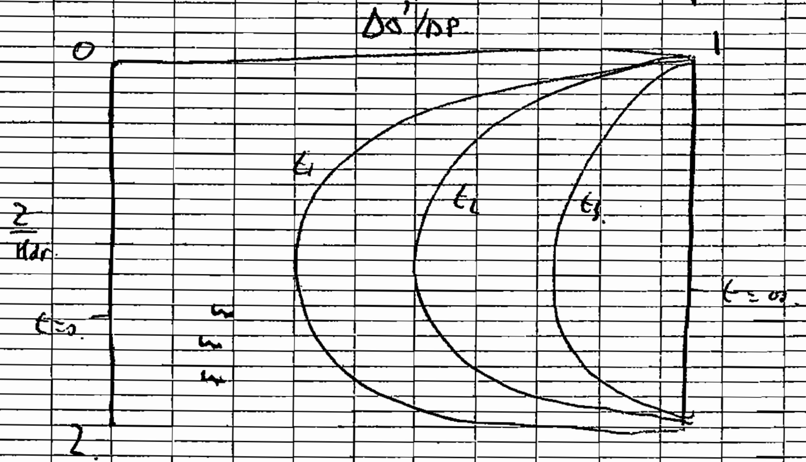
Dimensionless Solutions:

(a)

$$\frac{\Delta u}{\Delta p} = f\left(\frac{z}{H_{dr}}, T\right) \qquad \boxed{T = \text{time factor} = \frac{C_v \cdot t}{H_{dr}^2}}$$



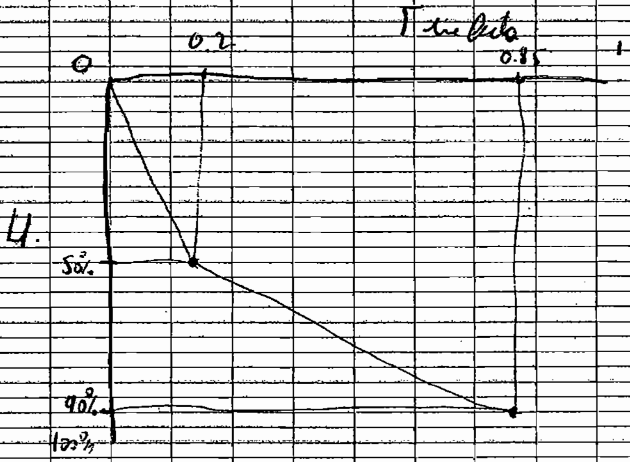




U = % consolidation

$U = 0$.   0 displacement $\Delta = 0$

$U = 100\%$. $\Delta = \Delta_{final}$



$U < 50$. $\frac{U}{100} = 2\sqrt{\frac{T}{\pi}}$

$U > 50$ $\frac{U}{100} = 1.$

$$T = \frac{C_v t}{H_{dr}^2} \qquad U = \frac{\Delta(t)}{\Delta_{final}}$$

How much:

1) σ'_0 at center compare σ'_p

2)

How long. For 90% def the k adiest.

$U = 90\% \Rightarrow T = 0.85 = \frac{C_v (t)}{H_{dr}^2} \rightarrow$ Ho log.

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