[illegible] soil [illegible]

Soil mechanics

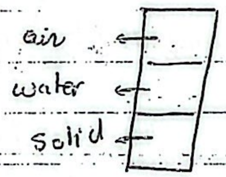
* Soil mechanics: is a branch of science which deals with the study of the physical properties of soil and the behavior of soil ~~masses when subjected to forces~~. It is part of geotechnical engin
under various types of forces

- grain size dis.
- permeability,
- volume change
- tensile strength

* geotechnical engineering: the science and practice of part of civil engineering that involves natural materials found close to the surface of the earth surface.

In general it includes the applications of the fundemental principles of soil mechanics and rock mechanics to foundation design.

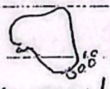
* In general sence of engineering, soil is defined as: the uncemented aggregates of mineral grains and decay organic material (solid part) along with liquid and gas which occupy the empty space between the solid particles ⟹ three phase material.



block diagram: three-phase material.

Origin of Soil:
Bed Rock (residual soil) — Weathered Rock (Transported soil)

* The product of weathering — fine grained in the surface, coarser at deeper area.


⟶ transported soil.

[illegible] residual soil

1) Glacial soil: drift → snow [illegible]
2) Allavial Soil: running water is the transporting agent.
3) lacustrine soil: deposit in a quite lake → very fine: silt & clay
4) marine soil: deposit in seas. shores.
5) Aeoline soil: wind is the transporting agent → sand [illegible].
6) Colluvial soil: transported by gravity force → slope.

* Particle size distribution :- [Gravel "G" ; Sand "S" silt "M" clay "C

① American association of State Highway and transportation officia
[AASHTO] American Associate of State and Highway Trans, offi

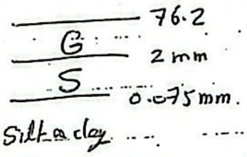
Grain size in mm

	Gravel "G"	granular Sand "S"	Silt "M"	Clay "C"
AASHTO :	3" - #10 (76.2 - 2) mm	#10 - #200 (2 - 0.075) mm	#200 0.075 - 0.002	< 0.002
USCS	(76.2 - 4.75) #4	4.75 - 0.075	#200 < 0.075	< 0.075

3" = 76.2 mm

#10 = 2 mm

#200 = 0.075 mm



silt & clay

wet analysis [hydrometer analysis]

2) Unified soil classificationsystem (USCS)

Gravel	Sand	silt + clay [fines]
76.2 - 4.75 #4	4 - 0.075	< 0.075

* Clay minerals → weathering of granite / feldspars.
- Kaolinite
- Illite
- Mantmorllinite.

↓ particle size decrease → S.A ↑ : — My Notebook

→ engineering property worse [expansion,

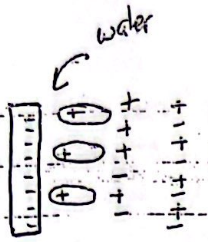
Clay min.  flatty (negatively charged).

dry : Ca, Na, k

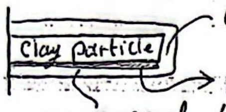
wet [water is added] → the positive ions float

Clay min attract water particles

H_2O (+ -) dipole effect.



* Types of bonding:
① Hydrogen Bonding,
② Dipole Effect.



diffuse double layer water (film).

adsorb water الجزء القريب من الحبيبات ديكون
ملتصق جيد أخصائصه اقرب لل
الصلب منه الى السائل زائد اللزوجة.
لماء من الماء العادي في منطقة أبعد

normal water.

The water around clay particles, gives the claysoil its plastic properties.

* plasticity of clay (امكانية التشكيل دون ان يتكسر) is due to the properties of the diffu. double layer of water so it can be shaped without crumbeling

* Mechanical analyses of Soil: Sieveand التوزيع الحبيبي / الحجمي للتربة

[1] ① sieve Analysis: Valid for 0.075mm → sieve #200, \

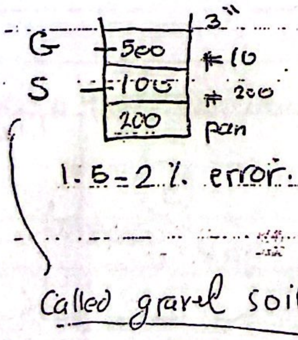
" oven dry " → same refereNce. تثبيت للمرجع

#200 = 0.075 mm

rubber mullet: لتفكيك الحبيبات → اذا ترابط

wash sample on sieve #200 then oven dry sample again.
∴ oven dry 1000 g → reference weight
oven dry after rinse: 800 g.

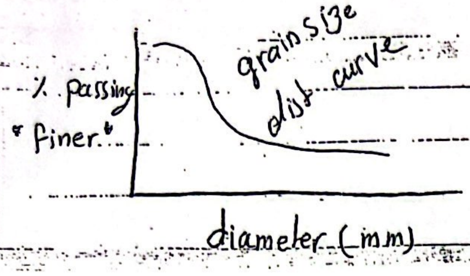
AASHTo classification



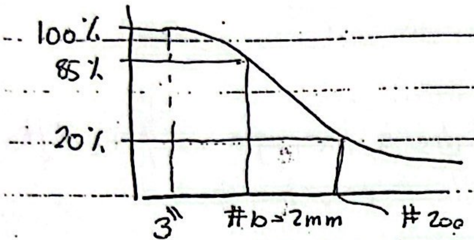
Sieve size	finer % passing
3"	100%
#10	50%
⋮	⋮

1.5-2% error.

Called gravel soil.



component for each type of soil particle (G, S, M, C).



→ % gravel = 15 %

→ % Sand = 65 %

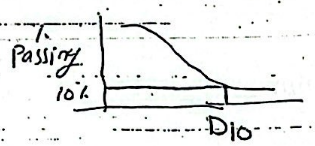
→ % fines = 20 % (silt + clay).

why the curve is used:

1) Components of each particle size. الخشن

2) Some parameters can be calculated → for classification of granular soil.
fine/cohesive soil : ناعم

a) effective size (D_{10}) = diameter corresponding to 10% fin



b) uniformity coefficient (C_u) = $\frac{D_{60}}{D_{10}}$

if = 1 → all particles have same size range.

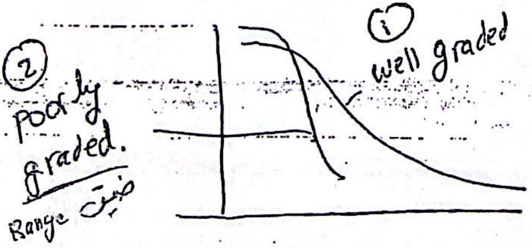
c) coefficient of gradation (concavity) = $C_c = \frac{D_{30}^2}{D_{60} * D_{10}}$

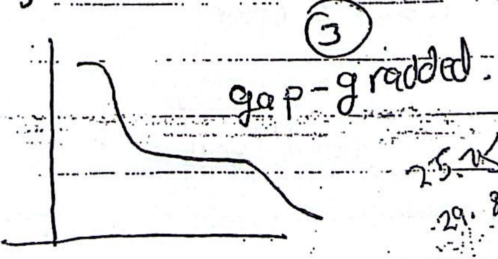
For the soil to be well-gradded:

1) C_u > 4 for gravel
> 6 for sand.

2) C_c between 1 & 3.

both conditions must apply





25.75
29.87

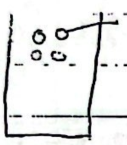
road → well graded.
allow water to pass → poorly graded } according to engineering use.

2 Hydrometer analysis "wet analysis" : Sedimentation principle.

% passing #200 ≥ 90%. → for agricultural uses.
↳ expansive potential of the soil / clay

قانون الترسيب

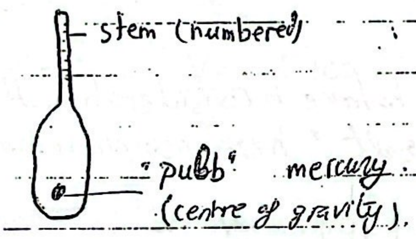
"Stocke's law" = "Sedimentation"

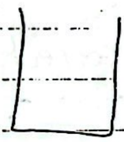


* سرعة الترسيب تعتمد على:
1) حجم الحبيبات 2) وزن الحبيبات

relationship bet. Sedimentation speed vs. density of the soil.

Hydrometer



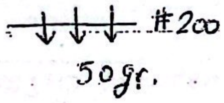


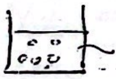
الجزء الأول على تركيزه
مع الوقت
ينزل أكثر.

قياس مقدار التربة المتبقية في كل وحدة زمنية بشكل تقريبي.

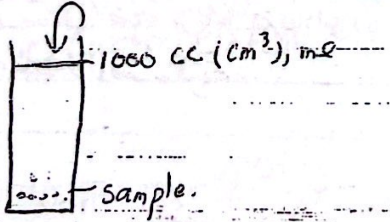
- Hydrometer conditions a procedure.

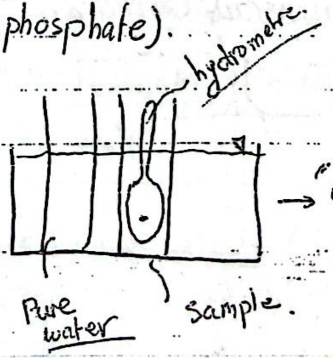
oven dry





Soak in a dispersing agent عامل مفكك for 8 hours
"hexametaphosphate).

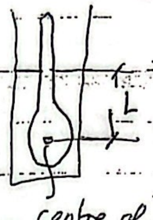




→ "water bath"
حرارة متجانسة

entro end shaking للتأكد من أن كل المواد معلقة غير مترسبة

"t" time	reading "L"	Corrections
15 sec	30	
30 sec	--	
1 min	--	
2 min	--	
4 min	--	
⋮		
1 hr	-- [in the 1st day].	
⋮		
24 hr	→ final reading	



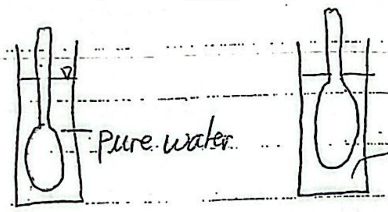
centre of gravity.

*hydrometer: gives density the solution around the of gravity.

* L/t = speed of particles

* Corrections: 1) zero correction → to take in consideration the density of salt " hexametaphosphate".

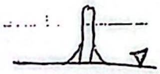
how?



zero correction = الفرق بين القرائتين

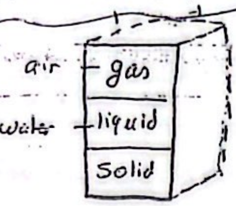
2) Temperature correction:
Temp. affects viscosity

3) miniscus correction



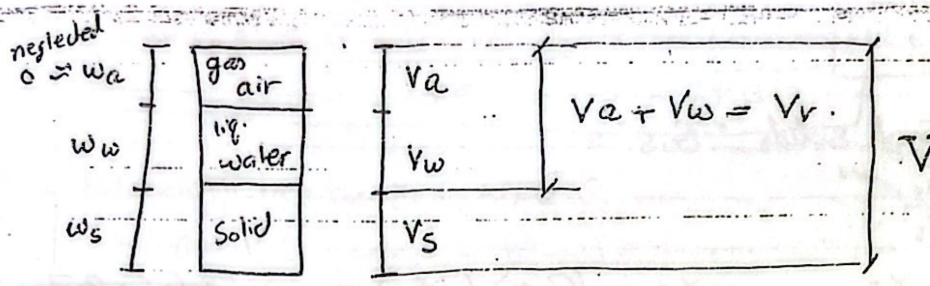
لأنه محلول فيه تربة تخفي القراءة العلوية
وتظهر السفلية.

* weight - volume relationships:



1 unit. الارتفاع عمل الحجم

block diagram.



$\boxed{W = W_s + W_w}$

1 * Void ratio "e" = $\frac{V_v}{V_s}$ in <u>decimal</u> 0.8, 1.2, ...

2 porosity "n" = $\frac{V_v}{V} \times 100\%$ [always < 1] in <u>percent</u> n = 60.5

3 air content "a" = $\frac{V_a}{V} \times 100\%$ [always < 1] in <u>percent</u> a = 5%, 10

درجة الاشباع

4 degree of saturation "S_r" = $\frac{V_w}{V_v} \times 100$ [always < 1] in <u>percent</u>

; $V_v = V_w + V_a$,

* Relationship between "e" and "n"

5

(5) $e = \frac{V_v}{V_s} = \frac{V_v}{V - V_v} = \frac{V_v/V}{V/V - V_v/V} = \frac{n}{1-n}$. $\Rightarrow \boxed{e = \frac{n}{1-n}}$

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

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

6 moisture (water) Content "w%" = $\frac{W_{water}}{W_s} \times 100\%$

7 unit weight "γ" $= \rho g$, $= \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{W_s}{V}\left(1 + \frac{W_w}{W_s}\right)$

total
bulk
moist.

* dry weight = Ws. ⇒ $W_s / V = \gamma_d$

* Note:
$\frac{m_w}{m_s} = \frac{W_w}{W_s}$

$$\Rightarrow \gamma = \frac{W_s}{V}\left[1 + \frac{W_w}{W_s}\right] = \gamma_d\left[1 + w\right].$$

$$\Rightarrow \boxed{\gamma = \gamma_d\,[1 + w]}$$

8 specific gravity of solids "G.s":

$$\gamma_s = \frac{W_s}{V_s}$$

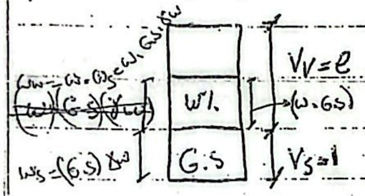
$$G.S = \frac{\gamma_s}{\gamma_w} \Rightarrow \gamma_s = (G.S)(\gamma_w)$$

ex. G.s = 2.7

Note: $\gamma_w = 9.81\ kN/m^3 = 62.4\ lb/ft^3$

$\rho_w = 1.0\ g/cm^3 = 1000\ kg/m^3 = 1\ ton/m^3.$

9 Relationship between γ, e, w, G.s *



$* \; e = \frac{V_v}{V_s} = \frac{V_v}{1} = V_v = e$

$* \; \boxed{W_s = \gamma_s * V_s = \gamma_s(1) = (G.S) * \gamma_w.}$

$* \; w = \frac{W_w}{W_s}$

$$\Rightarrow \boxed{W_w = w W_s = w * G.S * \gamma_w}$$

$$\boxed{V_w = w\,G.S.} \rightarrow \frac{W_w}{\gamma_w}$$

$$\boxed{\gamma = \frac{G.S\ \gamma_w\,[1 + w]}{1 + e}}$$

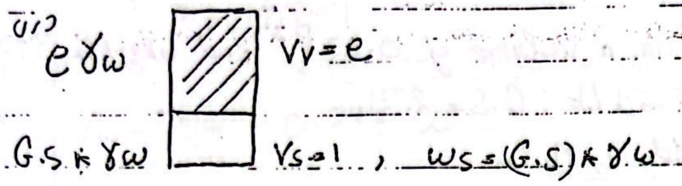
$$\boxed{\gamma_d = \frac{(G.S)\ \gamma_w}{1 + e}}$$

? // $S = \frac{w \cdot (G.S)}{e}$ $\Rightarrow \boxed{S * e = w * G.S.}$

(S) ↙ degree of Satu.

$S = \frac{V_w}{V_V} = \frac{G.S * w}{e} \Rightarrow S * e$

* If the soil is saturated with water:

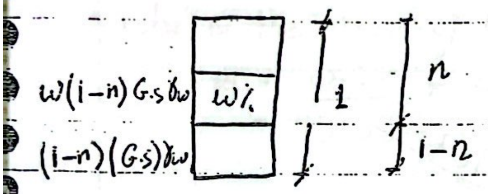


$$\boxed{\gamma_{saturated} = \frac{G.S\,\gamma_w + e\,\gamma_w}{1+e} = \frac{\gamma_w\,[G.S+e]}{1+e}}$$

[10] * Relationship between $\gamma, n, w, G.S.$

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

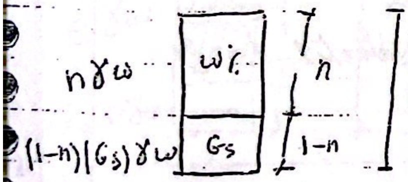
now let $V=1 \Rightarrow n = V_v.$



$$\gamma = \frac{(1-n)\,G.s\,\gamma_w + w(1-n)(G.S)\gamma_w}{1} = \frac{(1-n)(G.s\,\gamma_w)\,[1+w]}{1} = \gamma_d\,[1+w$$

$$\Rightarrow \gamma = \gamma_d\,[1+w]$$

* if the soil is saturated soil:-



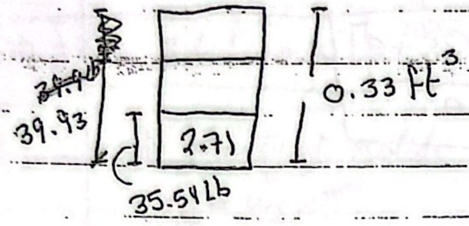
$$\gamma_{sat} = \frac{(1-n)\,(G.S)\gamma_w + n\,\gamma_w}{1}$$

$$\Rightarrow \gamma_{sat} = \boxed{\gamma_d + n\gamma_w}$$

Example 1: a moist soil sample has a volume of $0.33\ ft^3$ and weight 39

oven dry weight = 35.54 Lb , G.S = 2.71.

wanted: 1) w% , γ; γd, e, n, S.



$* W_w = 39.93 - 35.54 = 5.39\ Lb.$

$* V_w = \frac{5.39}{62.4} = 0.086\ ft^3.$

$* V_s = \frac{W_s}{\gamma_s} = \frac{35.54}{(2.71)(62.4)} = 0.204.$

1) $W\% = \frac{W_w}{W_s} = \frac{5.39}{35.54} * 100\%$

2) $\gamma = \frac{W_{total}}{V_{total}} = \frac{39.93}{0.33}\ Lb/ft^3.$

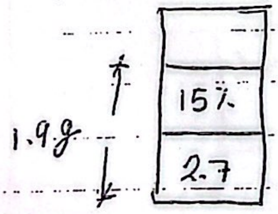
3) $\gamma d = \frac{\gamma}{1+w}$ or $\gamma d = \frac{35.54}{0.33} = \frac{V_s}{V}$ 107.697.

4) $e = \frac{V_v}{V_s} = \frac{0.33-0.204}{0.204} = 0.62$

5) $n = \frac{V_v}{V} =$ - — 6) $S = \frac{V_w}{V_v} = \frac{0.086}{0.33-0.204} * 100\% = 68.3\%$

Example #2: w = 15%, $\rho = 1.9\ g/cc$, G.S = 2.7

wanted: find S_r = ?, ρ_s = ? → saturated density



$\rho = \frac{mass}{V} = 1.9$

assume V = 1 ⇒ m = 1.9 g

$1.9 = m_s + m_w = m_s + 0.15 m_s \Rightarrow m_s$

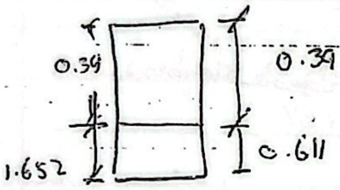
$\Rightarrow m_s = 1.652\ gr.$ $\Rightarrow m_w = 1.9 - 1.652 = 0.2$

$V_s = \frac{1.652}{2.7 (1g/cm^3)} = 0.611$ cc.

$V_w = \frac{m_w}{\rho_w} = 0.248$ cc.

$\Rightarrow$ 1) $S_r = \frac{V_w}{V_v} = \frac{0.248}{1-0.611} * 100\% = 63.8\%$

$\Rightarrow$ 2) $\rho_s = \frac{1.652 + 0.39}{1} = 2.042$



3). what is the amount of water required to saturate $1 m^3$.

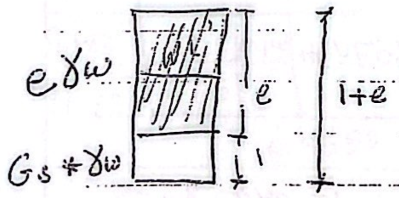
$w = \frac{0.39}{1.652} * 100 = 23.6\%$

$w \rightarrow 15\% \rightarrow 0.248$ g.

$\therefore$ for $1 cm^3$ add $(0.39 - 0.248)$ g.m of water

$\Rightarrow$ for $1 m^3$ add $(0.39 - 0.248) * 10^6$ g $= \approx$ 140 kg/m³.

*✓Example #3: show that $\gamma_{sat} = \frac{e}{w}\left(\frac{1+w}{1+e}\right)\gamma_w$.



$\gamma sat = \frac{(G.s)\gamma_w + e\,\gamma_w}{1+e} = \frac{\text{Wt weight}}{\text{Total Volume}}$

$= \gamma_w \frac{(G.s + e)}{1+e}$

Use This.
$* \boxed{S.e = w(G.s)}$ Sat $\Rightarrow S = 1 \Rightarrow G.s = e/w$

$\Rightarrow \gamma sat = \gamma_w \left(\frac{e/w + e}{1+e}\right) = \gamma_w \left(\frac{\frac{e + we}{w}}{1+e}\right)$

$= (\gamma_w)\frac{e}{w}\left(\frac{1+w}{1+e}\right)$ ✓

Embankment: طم

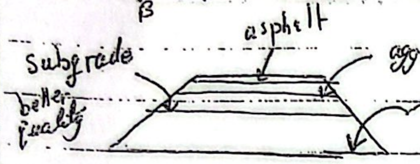
***. Example #4:** ✓

Borrosite: Construction site:

$e = 0.6$ $\rho_d = 1.76\ ton/m^3$ $G.S = 2.7$

$w = 15\%$ $w = 18\%$

$V_B = ?$ $V = 40{,}000\ m^3$



① embankment: الطم ونوعيتها اقل من التي على السطح ، اضعف

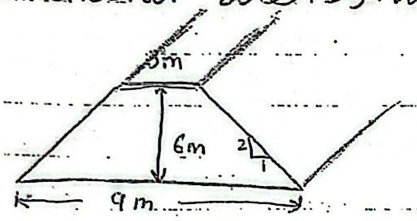
② subgrade: اساسي جيب الطريق ونوعيتها افضل

③ aggregate basecoarse:

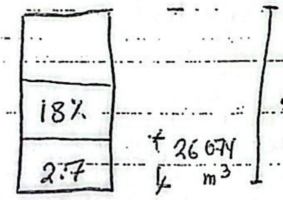
④ asphalt

توضيح

* Borrosite: ممكن يكون جانب الطريق / المكان الذي نجلب منه المواد الطافة



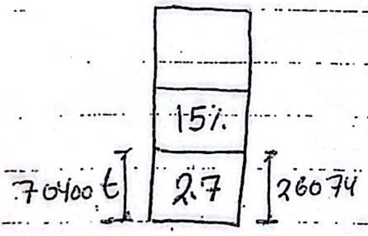
$V = \frac{1}{2}(3+9)(6)(1) = 36 m^3$ at Const. sit



$e_{cons. site} = \frac{40{,}000 - 260}{26074}$

$= 0.54$

$\rho_d = 1.76 = \frac{m_s}{V_s} = 70400\ ton.$ $\Rightarrow V_s = 26074\ m^3.$



$* e = \frac{V_V}{V_s} \Rightarrow V_V = 15644\ m^3$

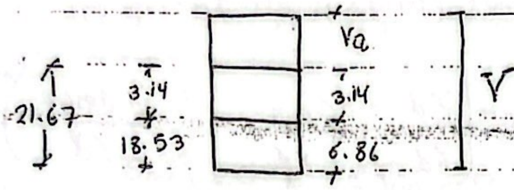
$\Rightarrow V_B = 26074 + 15644$

$= 41{,}718\ m^3.$

*b) Calculate the amount of water that should be added.

* Example # 5: ✓

moist sample weight = 21.67 gm, overdry weight = 18.53 gm, air content = 5 % , G.S = 2.7, e = ??



$e = \frac{V_v}{V_s} = \frac{0.53 + 3.14}{6.86} =$ ✓

air content = $\frac{V_a}{V_a + 3.14 + 6.86}$ = 0.53 cm^3

* Example #6 , H.w:-

- Two site for Borrpit (حفرة الاستعارة):

G.s = 2.65

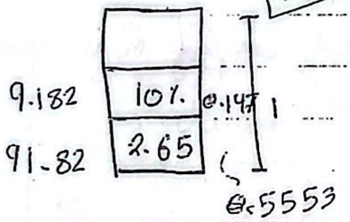
Site #1	site #2	Cons. site
γ = 101 Lb/ft^3	γ = 96	V = 35,000 ft^3
w = 10 %	w = 14 %	γ = 126
		w = 14 %

unit price = 3 $ for mat. & 4 $ for trans and 0.5 $ for water.

ques: which site will be more economic??

my sol. * site # 1

42,130.5 ft^3

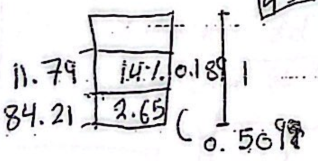


$\gamma = \frac{W_w + W_s}{V} = \frac{\gamma_w * G.S}{--}$ - - = γ_d [1 + w]

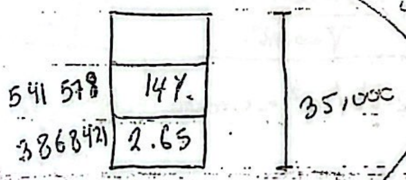
$\Rightarrow \gamma_d$ = 91.82 Lb/ft^3 = $\frac{W_s}{V}$ $\Rightarrow$ Ws = 91.82 Lb.

* site #2:

45937.8 ft^3



* Cons. site



* Cost (site #1) = $ ~~372,281.0~~ 296153.4

Cost (site #2) = $ 321,564.6 $

∴ Choose site #2.

* **Relative density (Dr)**:- it is commonly used to indicate the in sit denseness or loseness of granular soil (index property).

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \text{OR} \quad D_r = \left[\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right]\left(\frac{\gamma_{d\,max}}{\gamma_d}\right)$$

where:

$e \equiv$ void ratio at site $\rightarrow \gamma_d$ (in site)

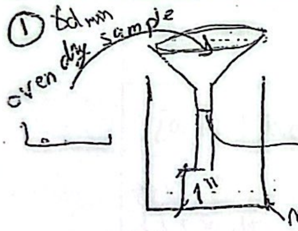
$e_{max} \equiv$ maximum void ratio $\rightarrow$ losest state $\rightarrow$ minimum dry density (

$e_{min} \equiv$ minimum " " $\rightarrow$ densest state $\rightarrow \gamma_{d\,max}$

* Dr (%) : d	Description of soil
0 - 15	very lose.
15 - 50	lose
50 - 70	medium
70 - 85	dense → for construction purposes (70-85 %)
85 - 100	very dense

---

① $\gamma_{d\,min}$



*ارتفاع ثابت 1" تحت الـ funnel

* **losest case** (fill the Mold with sand in its losest case using a funnel of (d = 1/2"))

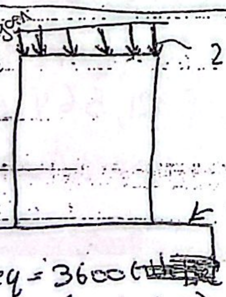
empty weight = W_1 , filled weight = W_2

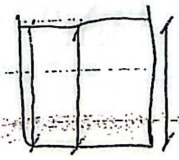
$$\Rightarrow \gamma_{d\,min} = \frac{W_2 - W_1}{V = 0.1\,ft^3}$$

② $\gamma_{d\,max}$

* fill the mold again

* Add surcharge we

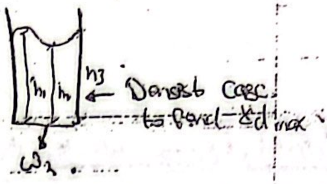




freq = 3600 (cycle/min) for 8 min {dynamic compaction}

* avg. height of 3 positions:

$$\gamma_{d\,max} = \frac{W}{V_{total}} = \frac{W_2 - W_1}{(A_{mold} \times (\text{avg height}))}$$



$V = A\left(\frac{h_1+h_2+h_3}{3}\right)$

* Chapter 3, Q 3, 7, 15, 24 → seventh eddition.

✓ Examples: loose uncompacted sand has a depth of 6' ; Dr = 40% → loose material

$e_{max} = 0.9$, $e_{min} = 0.46$, G.S = 2.65

(a) what is γd.

(b) If the sand is compacted to Dr = 75% → Dense material

1) what is the decrease in the thickness.

2) γ moist = 115 Lb/ft³ , what is w = ??

$\boxed{a}$ $0.4 = \frac{0.9 - e}{0.9 - 0.46}$ ⇒ e = 0.724

① find e.
② find γd.

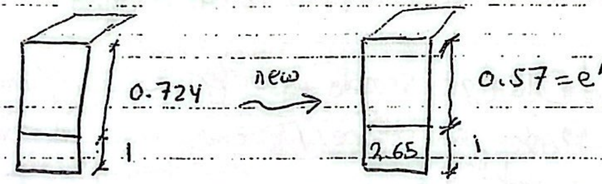
ws = 165.36 Lb = 2.65 (9.81) = G.S . γw

⇒ $\gamma_d = \frac{W_s}{V_{Total}} = \frac{165.36}{1.724} = 95.92$ [lb/ft³]

$\boxed{b}$ $0.75 = \frac{0.9 - e'}{0.9 - 0.46}$ ⇒ e' = 0.57

① find e'
② find V'
③ find d'

$\boxed{1}$



V = 1.724 → 1.57

d = 6 ft → y ⇒ y' = 5.464'

∴ decrease in thickness = 6 − 5.464 = # = 0.536'

$\boxed{2}$ $\gamma_d = \frac{165.36}{1.57} = 105.32$ Lb/ft²

γ moist = γd [1 + w]

⇒ w = # 9.186%

Quiz # 1

(A, W) ch. 3 − 4-5, 6, 17,

* Consistency of soil: clay soil

Alterberg limits:

water added. →

Solid | semi-solid | Plastic | liquid [4 stater]

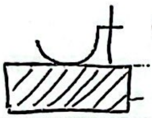
استخدم ← زراعي ← shrinkage limit (S.L.) — plastic limit (P.L) — liquid limit (L.L.) استخدم في تصنيف التربة

increasing w. → + w%

*

(* liquid limit:) water content in percent required to close $\frac{1}{2}$" along bottom of the groove after 25 blows.

(apparatus used: Casagrande device



)

* Sample preparation.

① air dry. OR 60 °C oven a

① حتى لا تطير الماء من الشكل

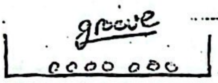
② حتى ما تحترق معادن مثل الـ Claite المواد العضوية

| passing.

② Sieve sample on sieve # 40.

حتى يدخل في التجربة الجزء الأكبر من الـ clay

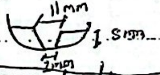
and take part of the passing sample.



③ 150 gr sample + water ⤳ uniform mixing until we get a paste

④ open groove, and 2 rev / second until it closes.

* if rev = 25 then it's the LL.



(* Cone method:

5 mm penetration → liquid limit

more accurate but less frequent.)

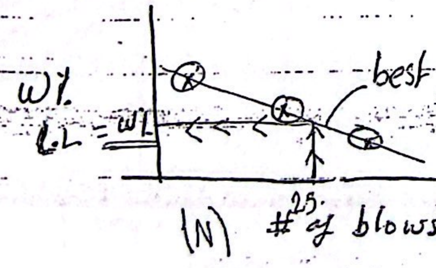
⑤ find w% By drying sample

* Make 3 Trials to Draw the curves

Range	# of blows (N)	water content %
(30-40)	37	w_1
(20-30)	28	w_2
(10-20)	17	w_3

40-30
30-20
20-10 } values for accuracy.

> 40 — ignore trial exclude it



liquid limit.

Flow curve → slope = Flow in

* One point liquid limit: Do only one trial ; such that $20 \leq N \leq 30$

$L.L. = w_n \left(\frac{N}{25}\right)^{0.121}$ $w \equiv$ water content for the trail

Condition: $20 \leq N \leq 30$

* Casagrandi concluded that each blow in standard liquid limit device corresponds to a shear strength of about $1 g/cm^2 = 0.1 kN/m^3$

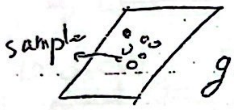
for $N=25 \Rightarrow 25 g/cm^2 = 250 kg/m^2 = 0.25 t/m^2$

$= 2.5 kN/m^2$ → very weak shear strength

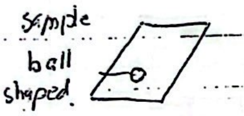
---

* plastic limit: w% at which the soil when rolled into threads of $\frac{1}{8}$" (3 mm) diameter ~~crumbles~~ crumbles.

↓↓↓↓ Passing # 40. air dry.



glass plate: Sample + little water.



rolling (90 strike/min) → diameter 3mm + crumbles تتفتت

By hands in surface

+Both conditions

→ Plasticity index = L.L − P.L.

المنطقة الي تكون التربة خلالها في الـ plastic state

plastic | liquid

P.L   L.L

وكل ما كان اكبر كانت التربة اسوء.

Ex.

soil 1	soil 2
L.L = 50	L.L = 30 → or أقل L.L → soil #2 is better
P.L = 40	P.L = 10
⇒ P.I. = 10%	PI = 20% → Soil #1 is better ✓

as P.I↑ Soil worse

PI	Description of Soil
0	non-plastic
1−5	slightly plastic
5−10	low plasticity
10−20	medium
20−40	high
>40	V. high

✓ Ex. PI = 35%, very plastic (High) ⇒ تربة بسيطة

- In subgrades, PI < 10.
- PI ↑ → amount of clay is ↑ or clay type is bad.
- PI = 0. رمل لايتماسك ، مثل الحجارة مفككة

- L.L, P.L, P.I → consistency of soil in remolded state. غير عن حالتها الطبيعية
- Consistency of soil in situe : water content in nature.

① Liquidity index $(L.I) = \frac{\acute{w}_n - P.L}{P.I.}$; w_n: water % in nature

$L.I = 0 \rightarrow$ soil is in plastic state. $w_n = P.L$

$L.I = 1 \rightarrow$ soil is in liquid state $w_n = L.L$ exercise

$w_n = L.L \Rightarrow \frac{L.L - P.L}{PI} = 1$

② Consistency index $(C.I.) = \frac{L.L. - w_n}{L.L - P.I}$

$C.I = 0 \rightarrow$ soil is in liquid state. ? ⇒ Check p.21 Activity

$C.I = 1 \rightarrow$ soil in plastic state

$w_n = L.L$, $\frac{L.L - L.L}{L.L - P.I} = 0$

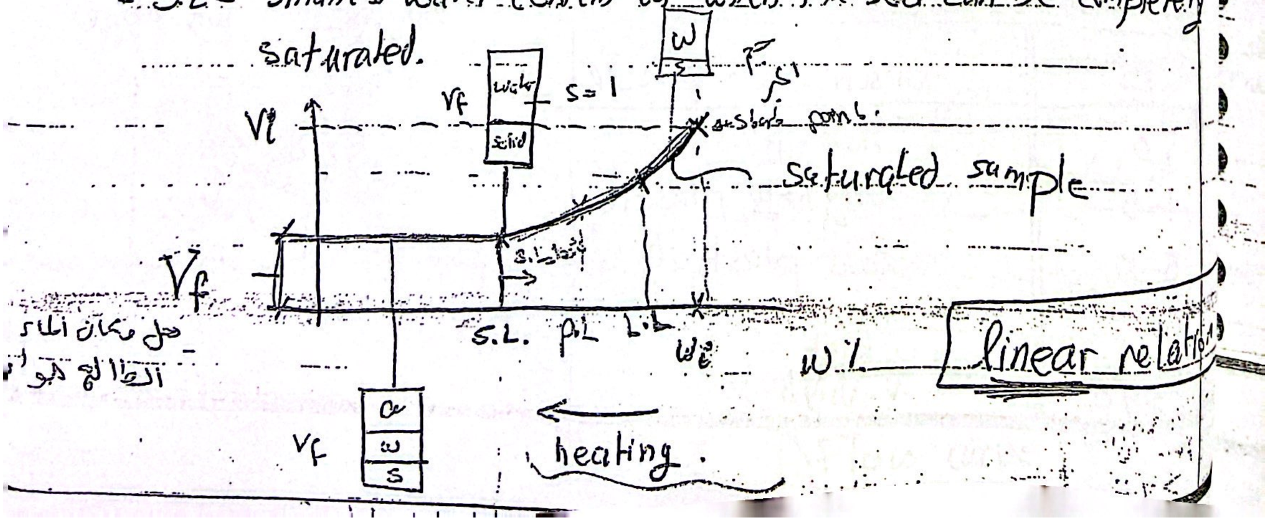
0 الى 1

- $G.S = \gamma s / \gamma w$.
- mass specific gravity : unit weight of all material present. $\gamma = \frac{W}{V}$

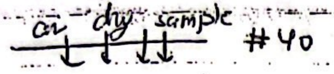
$G.m = \gamma / \gamma w$

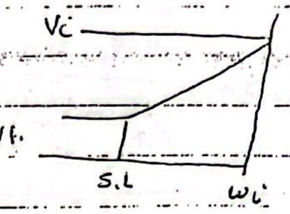
Shrinkage limit

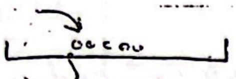
- S.L. = w% at which no volume change happened even the moisture content is decreased.
- S.L. = w% just sufficient to fill the voids when the soil is at the minimum volume.
- SL = Smallest water content at which the soil can be completely saturated.



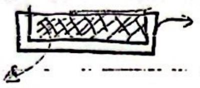
$* SL = w_i - \Delta w$







sample passing #40 + added water to form paste.

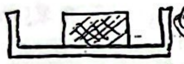


porcelan dish with known volume, and mass
$\overset{V_m}{}$ $\overset{m_m}{}$

① cover surface with oil then fill container with soil.

② * m_1 and V_1 are known ; m_1 = (weight after − m_m)

V_1 = volume of mold = V_m



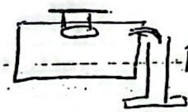
③ after 24 hrs in oven

④ m_2 is known, ~~V_2 is known~~ U_2 is unknown ,

$$\Rightarrow w_i = \left(\frac{m_1 - m_2}{m_2}\right) * 100\%$$

Note: $m_2 = m_s$

to determine V_2



paraffine or mercury

V_2 حجم العينة = حجم الزئبق الزائد

① cover with candles ;
unknown mass ~~&&~~ Vm

→ Bring Cylinder with water,
Seal volume increase

volume increase
Volume of Candle = V_2

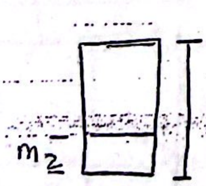
$$\Delta w = \frac{(\Delta V)\,\rho_w}{m_2} = \frac{(V_i - V_f)(\rho_w)}{m_2} * 100\%$$

Δ weight (gone) w% = w% current

$$\Rightarrow S.L. = \overset{w\% \text{ Before}}{\left(\frac{m_1 - m_2}{m_2} * 100\right)} - \overset{\Delta \text{ weight (gone)}}{\left(\frac{(V_i - V_f) * \rho_w}{m_2}\right)} * 100\%.$$

*** shrinkage ratio:** نسبة التغير في الحجم منسوبا لما بقي من ال ms

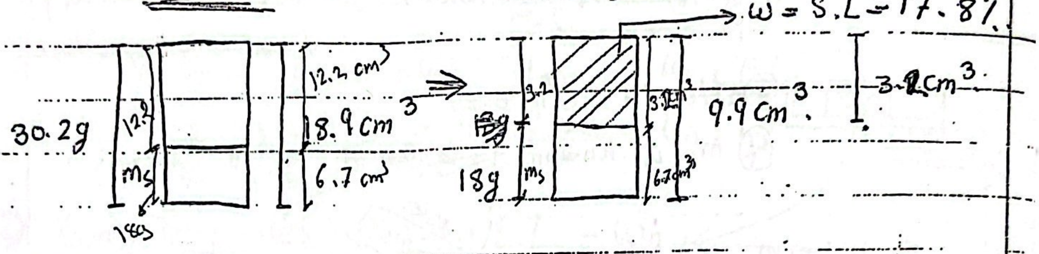
$$\Rightarrow \text{shrinkage ratio} = S.R. = \frac{\left(\frac{\Delta V}{V_f}\right)}{\left(\frac{\Delta m}{m_2}\right)} = \frac{m_2}{V_f * \rho_w} = \frac{\rho_d}{\rho_w}$$



$$\Rightarrow G.S = \frac{1}{\frac{1}{S.R} - \frac{S.L}{100}}$$ ← in % a wt. , S.L is in %.

* **Example #1:** saturated clay sample has a volume of 18.9 cm^3 and a m of 30.2 gm. On oven drying, the mass reduced to 18 gm and volume of dry sample is 9.9 cm^3. Wanted: Calculate S.L, G. and S.R.

Initial After drying



$* \Delta V = 18.9 - 9.9 = 9 \ cm^3$.

$* \Delta W = 30.2 - 18 = 12.2 g$. (12.2g).

$$* S.L = \left[\left(\frac{M_1 - M_2}{M_2} \right) - \left(\frac{V_1 - V_2}{M_2} \right) * \rho_w \right] * 100 = 17.8\%$$

$18g = m_s$

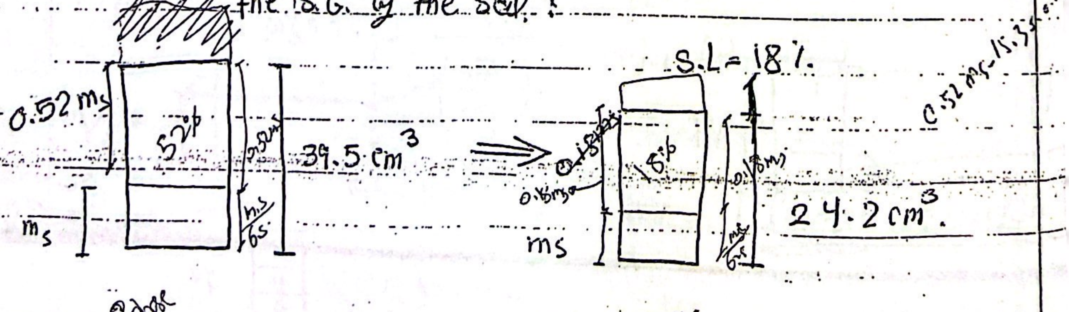
$\Rightarrow m_w = 0.178 * m_s = 18g * 0.178 = 3.2 g. \Rightarrow V_w = 3.2 \ cm^3$.

$\Rightarrow V_s = 9.9 - 3.2 = 6.7$

$$* G.S = \frac{18}{(6.7)(1)} = 2.69 = \frac{\rho_s}{\rho_w} = \frac{m_s/V_s}{\rho_w = 1}$$

$$* S.R = \frac{\rho_d}{\rho_w} = \frac{(18/9.9)}{1} = 1.82$$

* **Example #2:** A Soil has L.L = 52%, S.L = 18%, P.L = 30%. If the sample shrinks from a volume of 39.5 cm^3 at liquid limit to a volume of 24.2 cm^3 at S.L. What i the S.G. of the soil?



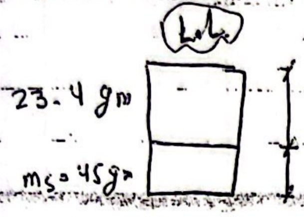
Gs

$0.18m_s + \frac{m_s}{G_s} = 24.2$

↳ m.s and G.s

→ $\Delta W = (0.52\,m_s - 0.18\,m_s) = 0.34\,m_s$

→ $0.34\,m_s = (39.5 - 24.2) \Rightarrow m_s = 45\,g$

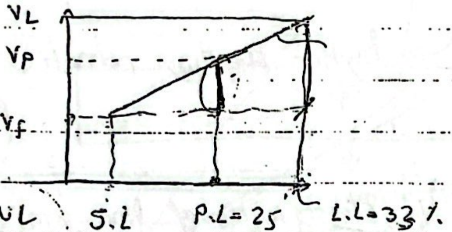


$\Rightarrow S.G = \frac{45}{16.1} = 2.80$

* Example #3: H.W: plastic limit of soil is 25% and PI = 8%, when the soil is dried from its state at P.L., the volume change is 25% of its volume at plastic limit.
When dried from L.L. state, the volume change is 34% of its volume at L.L.
wanted: Calculate S.L. and S.R.

$$\boxed{\begin{matrix} S.L = 16.4\,\% \\ S.R = 2.27 \end{matrix}} \rightarrow \text{final Ans.}$$

$0.66V_L = 0.75\,V_p = V_f$

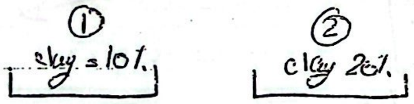


$\Rightarrow V_f = 0.66\,V_L = 0.75\,V_p \Leftarrow$

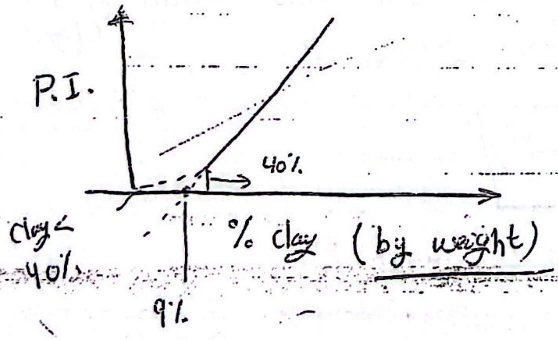
$V_f = 0.66\,V_L$
$V_f = 0.75\,V_p$

* Activity: Before Shrink

clay minerals + adsorb water ⟹ plasticity.



Skempton:



clay < 40% → passes through origin
clay > 40% → " at 9%.

* Activity: $\frac{P.I.}{\%\ clay}$ → % clay < 40%
"A"

$\frac{P.I.}{\%\ clay - 9}$ → % clay ≥ 40%

% clay ⓒ = 9.
= 0 → 2.5%

If A < 0.7 → inative soil.
0.7 < A < 1.2 → normal.
A ≥ 1.2 → active.

A	description
≤ 0.7	inactive
0.7 < A < 1.2	normal
≥ 1.2	active.

* swelling potential: depends on clay % an type.

kaolinite
clay 10%

Mantmorinite
clay 10% → higher swelling potential : more problematic

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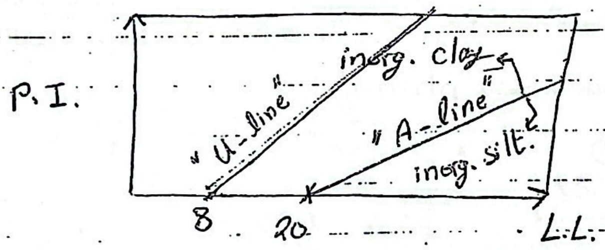
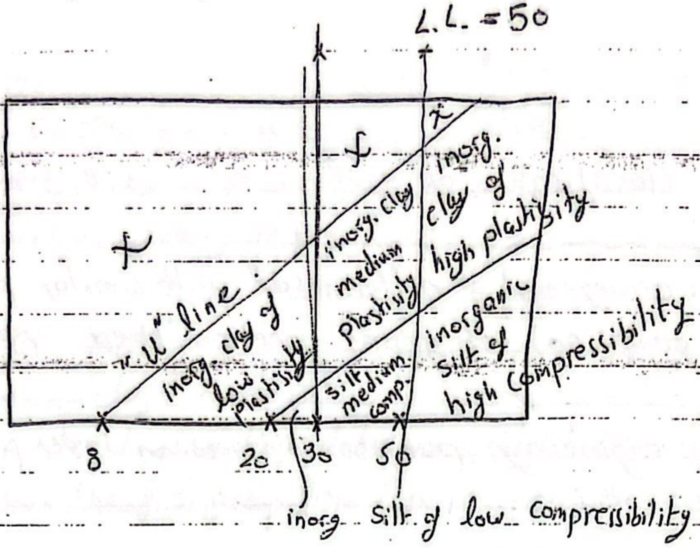
* plasticity charts :- "used in the classification of fine-grained soil.
Casagrandi.



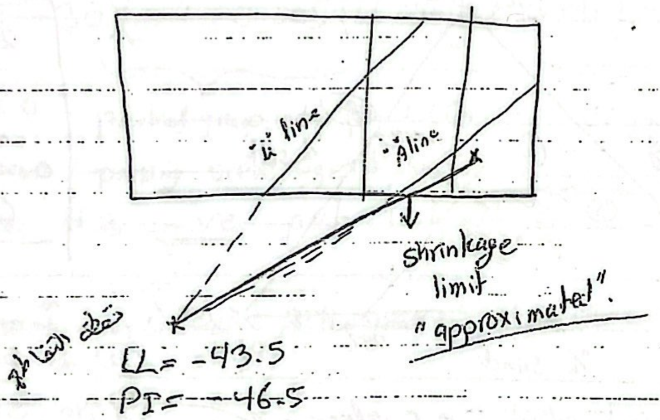
"A – line".

$\boxed{\text{"A-line" : } P.I. = 0.73(L.L - 20)}$ → on line: mixture of silt clay
∴ Ex. L.L = 60, P.I = 20 ⇒ inorg. silt

$\boxed{\text{"U-line" : } P.I = 0.9 (L.L. - 8)}$
↳ the upper limit between L.L. and P.I.



* shrinkage limit: can be approximated using plasticity chart:



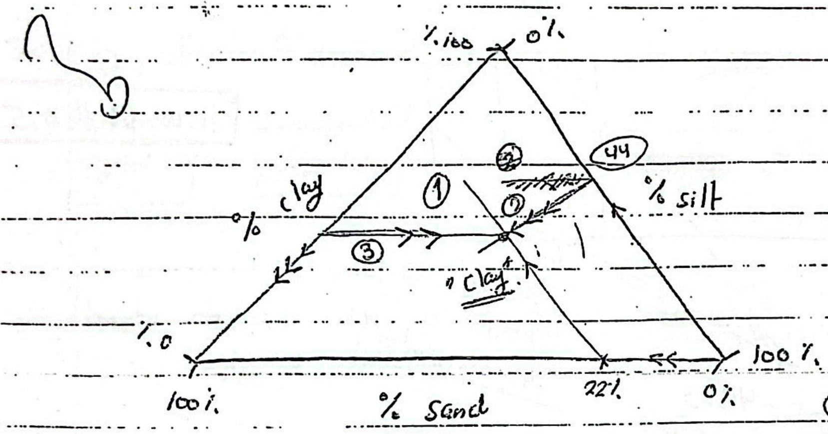
H.w @ #3,5

* Chapter #5: classification of soil: 1,2,3 ⇒ H.w.

+ classification: arrangement of different soil with similar properties into groups and sub groups based on their applications:-

- Class. system for engineering purposes is based on index properties & particle size.

[1] Textural classification: agricul. uses. التصنيف الزراعي حسب الملمس الخارجي



2 mm
S
0.05 mm
m
0.002 mm
C

to ____ 2m

(22) 20 Sand ____ 0.05

(44) 40 silt ____ 0.002

(33) 30 Clay

If gravel is present, adjust S, m, C values

$S = \frac{20}{100\% - G\%}$

∴ If gravel exists, we write: "gravelly clay".

[2] AASHTO classification system:

- soil is classified into seven major groups [A-1, --, A-7

Granular material (soil)	fine (silt or clay) materials.
% passing # 200 ≤ 35%	% passing # 200 ≥ 35%.
A1, A2, A3: sand dunes	A4, A5, A6, A7.

- this classification system is based on the following criteria:

⇒

* Grain size:

Gravel: 76.2mm – #10

Sand: #10 – #200

Silt & clay: passing #200.

"boulders" → retained on sieve si

\# 76.2 mm

"not considered s

* Plasticity:-

P.I ≤ 10 "silt"

P.I > 10 "clay"

- To classify a soil, the test data are applied from left to right by proce of elimination the first group from left into which the data fit is the corre classification.

A-2-4 (G.I.) G.I = group index.

- G.I. =

$$(F-35)\underbrace{[0.2+0.005(LL-40)]}_{\text{partial group index from L.L}} + \underbrace{0.01(F-15)(P.I-10)}_{\text{from P.I.}}$$

* imp

F = % passing sieve size #200.

If the # is -ve, put G.I as 0

The number is rounded to the nearest whole # G.I 3.6 ⟹ 4

3.4 ⟹ 3

No upper limit 0 – ∞

* A-2-4 (10) → better soil for engineering uses b/c passing #200↓

A-2-4 (20) LL↓ & PI↓.

* G.I. for A-1-a, A-1-b, A-2-4, A-2-5, A-3 ⟹ zero.

G.I. for A-2-6, A-2-7 → G.I is the partial group index of P.I.
الباقي جمع الاثنين .

A-7-5 → PI ≤ LL-30

A-7-6 → PI > LL-30

Example 1

* a Soil sample

% passing #10 = 100%, % passing #40 = 80%, % passing #200 = 5

L.L = 30, P.I = 10.

⇒ A-4 soil. $3.45 \rightarrow 3$

* G.I = 3.

∴ The soil classification is: A-4 (3) silty Soil.

---

* Example #2:

% passing #200 = 95%, LL = 60, PI = 40.

∴ Soil classification is: A-7-6 (42) Clayey soil.

---

* Example #3:

% passing #10 = 83, #40 = 48, #200 = 20, LL = 20, PL = 15

∴ Soil classification A-1-b (0).

$\boxed{3}$ unified soil classification system (USCS)

G: 76.2 – 4.75

S: 4.75 – #200

fines (M & C): < #200

USCS

coarse grained soil (granular)

% passing #200 ≤ 50%

∴ Gravel or Sand

fine grained soil (M & C)

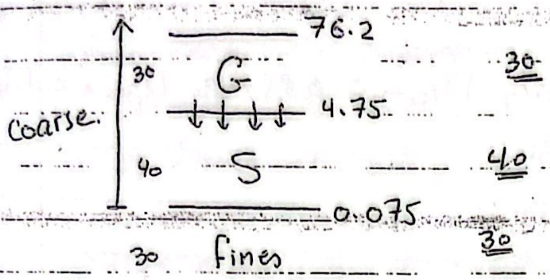
% passing #200 > 50%

∴ classification using plasticity chart

* fine grained Soil

1) inorganic silt (M)
2) inorganic clay (C)
3) organic silt & clay (O)
4) Highly organic soil (PT) - peat

* Coarse grained soil :- (G or S).



$\therefore$ % passing sieve #4 > 50% of coarse fraction → S.

Example: let classification be "S".

1 { + If % passing # 200 < 5% , it is well or poorly gradded.

$\therefore S_{\frac{w}{p}}$

\+ Calculate Cu and Cc.

if $Cu > 6$, $1 < Cc < 3$ [Note for gravel $Cu > 4$]

⇒ well graded.

$\therefore$ S, its well gradded sand with gravel [if % gravel ≥ 15% of total sample.

\+

2 { - If % passing # 200 > 12%

SC : clayey sand with gravel.

fines [clay or silt from chart] + gravel % ≥ 15 %.

3 { - If % passing # 200 between 5 & 12 [5 < % passing < 12].

dowel symbol is used.

$\underline{\underline{S}}_{\overset{w}{\cancel{P}}} - \underline{\underline{S}}_{\overset{C}{\cancel{M}}}$

if % > 15 %

well graded sand, clayey sand with gravel

* Example$_1$: % passing #10 = 100 %, #40 = 80%, #200 = 58 %,

LL = 30, PI = 10.

$\therefore$ # 200 > 50% ⇒ fine soil.

CL or OL

inorganic clay with low plasticity or organic clay w/ low plast.

* Example #2:

% passing #40 = 100%, #200 = 8%, D_{10} = 0.085 mm, D_{30} = 0.12

D_{60} = 0.135, LL = 30, P.L = 22

⇒ passing #200 = 8% ⇒ G or S

" #4 = 100% ⇒ Sand

" #200 = 8% [bet 5 ~ 12] ⇒ dual symbol $S_{\substack{w\\p}}$ -

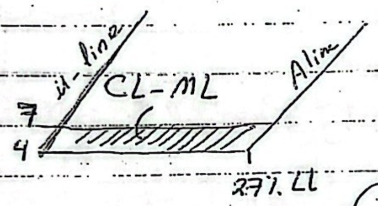
Cu = D60 / D10 = 1.59 ⇒ poorly gradded

from LL and P.L ⇒ clay

⇒ classification is Sp - Sc

* Example #3:-

% passing #200 = 61%, LL = ~~30~~ 26%, PL = 20%



⇒ this soil is CL-ML

and

not or

combination

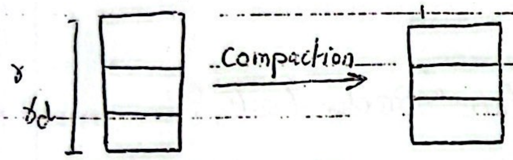
PI = 26-20 = 6

* Soil compaction and soil stabilization " chapter 6 "

- when voids are reduced:
  1) settlement is reduced
  2) higher strength is achieved.
  3) bearing capacity is increased.
  4) permeability is decreased

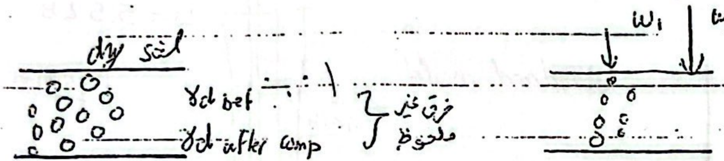
} The Engineering properties of the soil are improved.

- Reason:



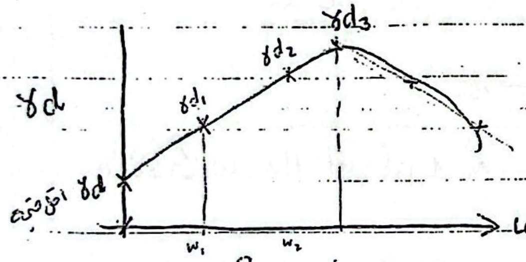
$\gamma_d \uparrow$: soil becomes denser.

*



$\gamma_d \backsim \gamma_{d1} \sim \gamma_{d2}$

صعب ترتيب حبيباتها تنزل

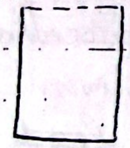


Compaction curve.

Compaction Curve

Optimum water (moisture content) ≡ OMC = γd₃ % — كمية الماء الكافية لتغليف كل حبيبات التربة وشرط انه لا تحل محل بعضها البعض

maximum dry density ≡ $\gamma_{dmax} = \gamma_{d3}$



V = constant.

γd. ثابتة

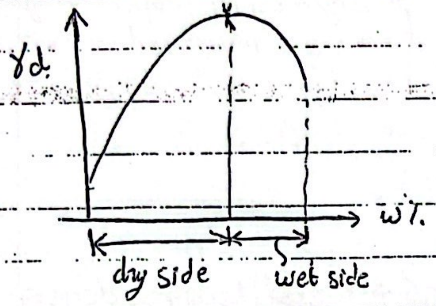
الحصول على طبقات

$\xrightarrow{\text{add water}} \gamma_{d3} \xrightarrow{\text{add water}} w_4$ [excess water)

الزيادة water act as a softening agent [lubricating agent] $\gamma_d \uparrow$

عند الزيادة فوق OMC الماء يشغل حيز من الفراغ فتقل كمية الصلب وبالتالي تقل γd

* Best case for comption is being around the optimum → minimum amoun energy will be required

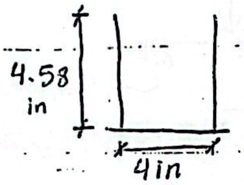


Compaction curve.

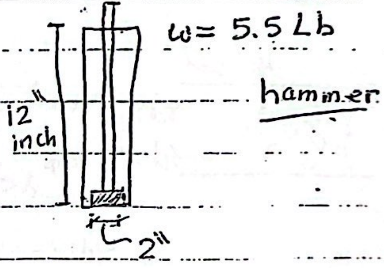
It is better to work at the d side.

* OMC & γ_{dmax} are determined in the lab " Proctor test ".

" Standard proctor test "

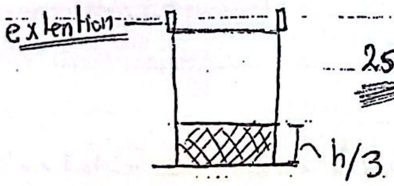


standard mold



- material used: passing # 4.

at least: 3 pts on the dry side & 1 pt at the wet side.



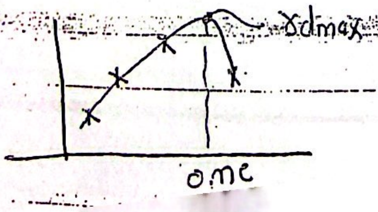
25 blows

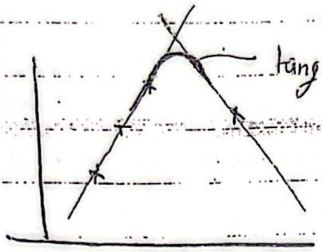
$$\boxed{V = \frac{1}{30}\ ft^3}$$

w_1	γ_{b1} = (bulk)	γ_{d1} = ✓	w_1 % ✓
⋮	⋮	⋮	⋮
w_5	γ_{b5}	γ_{d5}	w_5 %

} repeat procedure 4 times. at least

draw compaction curve

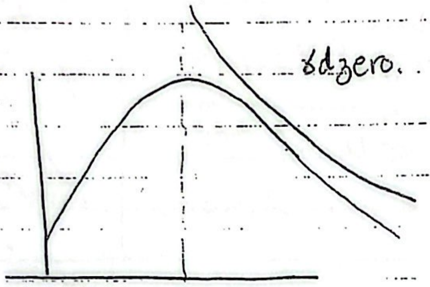




عكسية {
silt on Clay OMC ↑ , γd ↓
Courie soil OMC ↓ , γd ↑

* zero air-void curve:

theoretical maximum dry density γd_0



2 - 92%

always on the wet side.

it never intersects the Compaction curve.

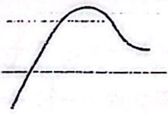
$$\gamma zero = \frac{\gamma w}{w + \frac{1}{G.S}} \quad ex. \; \gamma zero = \frac{9.81}{3 + \frac{1}{2.7}} \rightarrow \gamma zero \sim 3\%$$

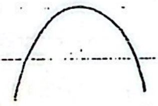
$$\gamma d zero = \frac{G.S \; \gamma w}{1 + \frac{w G.S}{S}} \quad ex. \; \gamma zero = \frac{2.7 * 9.81}{1 + \frac{3 * 2.7}{1}}$$

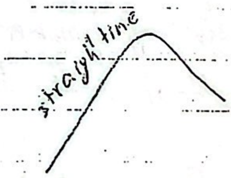
* Factors affecting compaction:

* 1) moisture content.

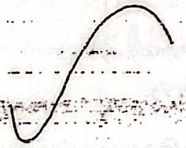
2) type of soil.


a Sandy-silt


b Silty-clay.



c heavy clay.


d SP: poorly graded sand

∨. due to bulking of sand

عند ما تكون خفيفة الماء قليلة تؤدي هذا ال
زيادة الحجم بشكل كبير.

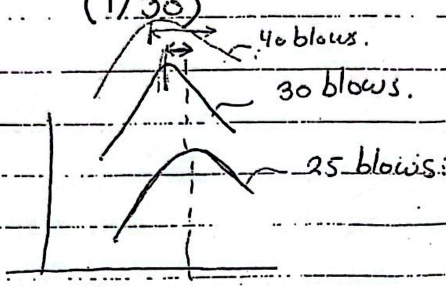
3) Effect of energy:-

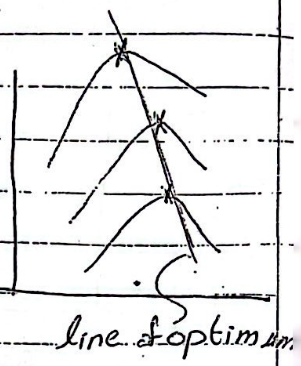
- for standard proctor test:

Energy = # of blows per layer * # of layer * weight of h

* h / V

$$\Rightarrow E = \frac{25 * 3 * 5.5 * 12}{(1/30)} = 12375 \text{ ft.Lb/ft}^3.$$





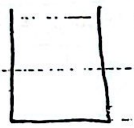
$\therefore$ as E ↑, OMC ↓, γd_{max} ↑.

* Modified proctor test:- development in rollers $\gamma d_{max} = 20 \text{ kN/m}^3$ instead of 18

المستخدم حاليا

→ material used: passing $\frac{3}{4}$ in [19 mm].

→ mold size



$V = \frac{1}{13.33} \text{ ft}^3.$

→ # of layers: 5

→ # of blows: 56 blows.

→ weight of hammer: 10 Lb.

→ height of fall: 18".

$$\therefore E = \frac{56 * 5 * 10 * 18}{(1/13.33)} = 55986 \text{ ft.Lb/ft}^3.$$

متماسكة

* Effect of compaction on cohesive soil properties — fine soil

Clay

cohesionless soil: عدم التماسك

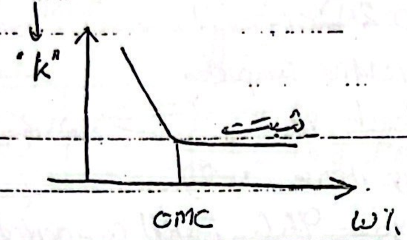
* Effect of Compaction on Cohesive soil properties:

1) Structure:

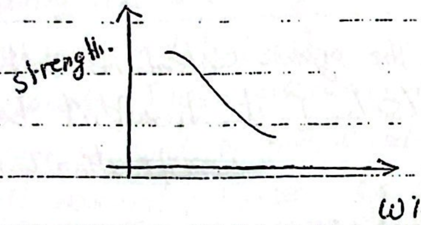
flocculent structure $\xrightarrow{\text{Compaction}}$ dispersed structure.

2) permeability: decreases:

perm. = "k"



3) Strength: compaction increases strength.



* Field compaction:

1 + Smooth wheel roller: coverage for the whole area

2 + penumatic rubber tired roller. [pneumatic]: coverage area 80%

3 + Sheep foot roller:

for cohesive soil تنزل عامات في الأرض

3 + Vibrating rollers.

* manual → Hand held plates.

* specification for field compaction:

1- Degree of compaction [relative compaction].

generally for earth work (90-100%) of proctor value.

from modified proctor test → γd_{max}.

$$\text{relative Compaction} = \frac{\gamma d \text{ (field)}}{\gamma d \text{ (max)}} * 100\%$$

Ex. SP 98% $\gamma_{dmax} = 20\ kN/m^3$

$\gamma_{field} = 19\ kN/m^3$ ~ 95% ✗ rejected.

2) Sand, granular soil R (بدلالة Dr.) يعبر عنه هكذا بـ

~~[illegible]~~ $R = 80 + 0.2 * Dr$

R = degree of compaction / relative compaction.

relative compaction: at least 85%. حط مفروده من القلاب

for Dr = 80% [very dense soil).

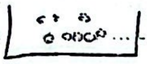
$R = 80 + 0.2 * 80 = 96\%$ [well compacted)

3) organic soil:-

$\omega\% \uparrow$, $\gamma_{field} \downarrow$ as the organic content increases.

If organic content $\geq 10\%$ [it shouldn't be used in construction work].

*

 soil sample $\xrightarrow{\text{oven dry}}$ 105 °C $\longrightarrow$ ω_1 وزن
100g

105°C $\xrightarrow{\text{temp}}$ 400°C $\longrightarrow$ ω_2 توزن
90gr.

$$\Rightarrow O.C. = \frac{\text{loss in weight}}{\text{dry weight at 105}} = \frac{\omega_1 - \omega_2}{\omega_1} * 100\%$$

* Determination of field density:-

$$R = \frac{\gamma_{d\,field}}{\gamma_{d\,max}} * 100\% \geq \text{specs}$$

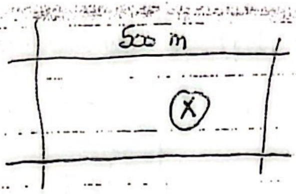
~ Lab

- methods used:

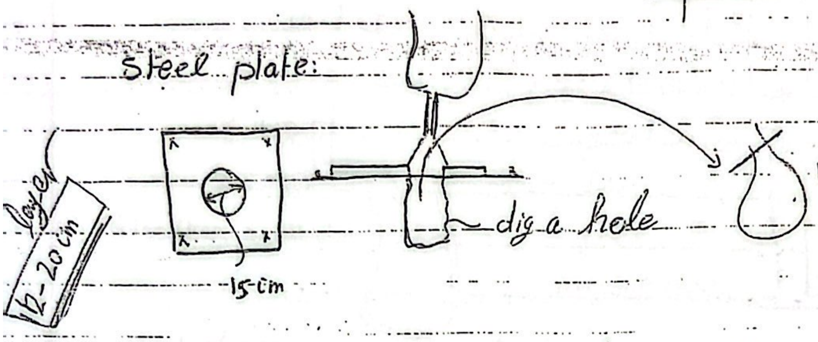
$\boxed{1}$ sand cone method: oldest, most common & accurate.

$\gamma = \frac{W}{V}$ $\gamma_d = \frac{\gamma}{1+\omega}$

500 - 1000 m^2 → sample.



Steel plate:



plastic bag.
Close to control "ω"
& keep it unchanged

+ Depth of hole = depth of tested layer. [layer 20 cm, dig 20 cm].

+ weight sample in Lab [w_1]

dry sample [w_2 = dry mass + ω%]

fill cone w/ sand & find ω [set + sand]. e.g. 5000 gm.

weight remaining sand + set e.g. 2000 gm.

∴ weight of sand to fill the hole + cone = 3000 gm.

In Lab, fill plastic jar w 1000 gm & open to fill the cone.

weight remaining sample e.g. 700 gm.

∴ weight to fill cone = 300 gm [calibration of the cone]

⇒ weight to fill the hole = 2700 gm

now find density of sand

15 * 15

, known volume.

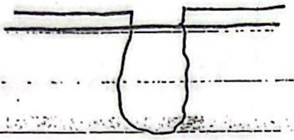
cylinder, w_2 to fill cone & cylinder

⇒ w to fill cylinder = $w_2 - 300$.

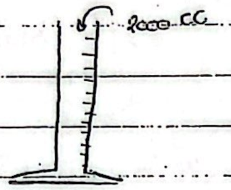
Calibration of sand

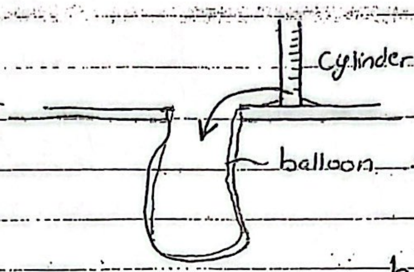
chap6: 1, 3, 5, 9, 13, 15

[2] Rubber balloon method:-



الحفرة بنفس الطريقة السابقة..





نص (ضاغط الماء

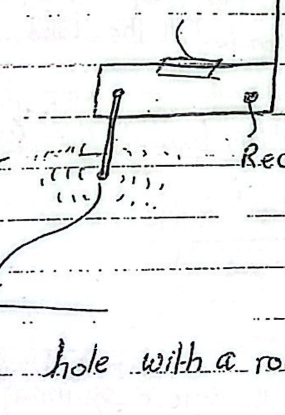
read the volume = volume of ~~cylinder~~ hole.

[3] Nuclear guage method:- [γ-rays]

قياس مقدار الاشعاع المرتد.

γ bulk & w

test time:
2-3 min



w. content
& moist unit weight
→ find $\gamma_d = ?$

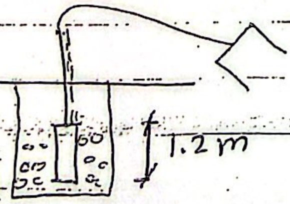
hole with a rod. مقدار الاشعاع بيحدد الكثافة و درجة ال compaction

---

* Special compaction techniques: Stabilization

1) Vibro flotation:- تحويم التربة

granular soil: sand or gravel.



بيضخ ماء: يطلع ماء مضغوط.

liquifaction: تحلل للطبقة ⇒ lose case

* grainsize distribution:-

suitability # $[S_N]$:-

$$S_N = 1.7 * \sqrt{\frac{3}{(D50)^2} + \frac{1}{(D20)^2} + \frac{1}{(D10)^2}}$$

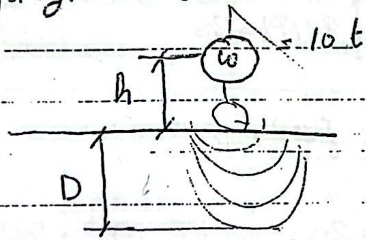
S_N	Rating as back fill
upto 10	excellent.
10-20	good.
20-30	fair
30-50	poor.
>50	unsuitable. A6 & A7 soils

* Example: $D_{10} = 0.11$ mm, $D_{20} = 0.19$ mm, $D_{50} = 1.3$ mm.

$S_N = 18$ ⇒ good mat. as a back fill.

$\boxed{2}$ Dynamic compaction

for granular soil.



10 t, 15 t [steel spheres]

efficient for large projects & large depth of compaction.

depends on: weight, height of fall, # of repetitions, spacing between drops.

$$D = \frac{1}{2}\sqrt{w_h * h} \qquad w_h\ [t],\ h\ [m].$$

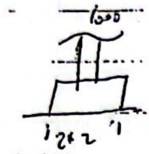
e.g. $w_h = 15$ t, $h = 12$ m

$$\Rightarrow D = \frac{1}{2}\sqrt{15*12} = 6.7 \text{ m} = \underline{\text{Significant depth.}}$$

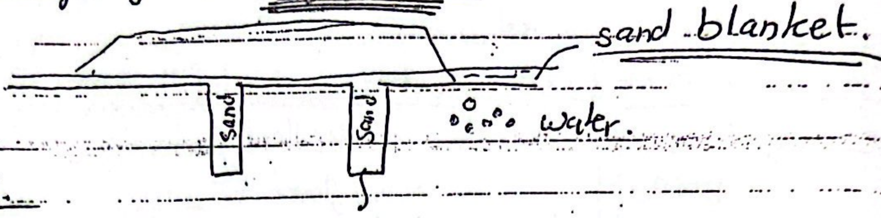
⇛

3 drainage using sand blanket and drains:-

for fine soil / clayly soil



load ≥ stren in found.



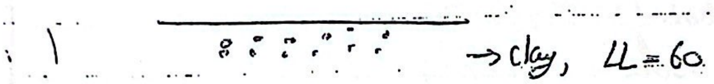
sand drains.

الماء يتجمع داخل الحفر لأن الفراغات بين الرمل أكبر

this method is accompanied by pre loading method.

4 Grouting method: حقن

- used for fine-grained soil.



* lime

cement + water → slurry [under pressure].

* function: 1) LL = 40 PL = 20 → PI red ; soil prop ↑

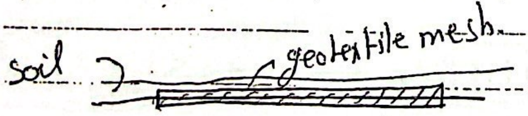
2) bearing capacity ↑

3) shear strength ↑

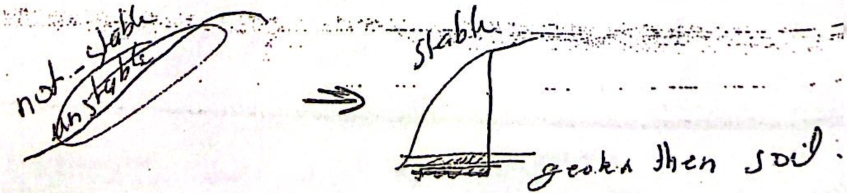
5 the use of Geotextile

polyster, nylon, and poly ethelene

- used in road projects.

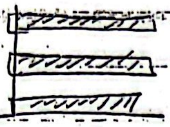


- fric bet. upper & lower soil layer ↑.



Sunday
7-3-2010
@ 2:00

$\boxed{6}$ Earth reinforcement: plates of steel . [illegible]



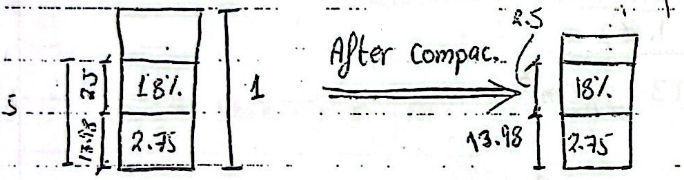
* Example #1:

Borrow site:
$w = 18\%$
$\gamma = 16.5$ kN/m^3
$G.s = 2.75$

Construction site:
$\gamma_d = 16.27$ kN/m^3.
$w = 18\%$
$V_{comp.} = 7651$ m^3. [7651]:

a) $V_{B.S} = ?$

b) truck carry 178 KN, find # of trucks/lifts.



$$\gamma_d = \frac{13.98}{V} \Rightarrow V = 0.859 \text{ m}^3.$$

$1 \longrightarrow 0.859$ m^3
$x \longleftarrow 7651$ m^3 $\Rightarrow x = \underline{8906.9}$ m^3. [a]
8902.3

of lift(W) $= \frac{8906.9 * 16.5}{178} = \#$ of lifts
825.2 ≈ 826

c) what is $\gamma_o = \frac{13.98}{V_s + V_w}$ = maximum dry density

⇒

+ **Example #2: Ques 13 in book:**

Field density:

$\gamma_{sand} = 1667\ kg/m^2$

mass of cone + sand (before testing) = 5.99 kg

mass of cone + sand (after test) = 2.81 kg

mass of sand required to fill the cone = 0.117 kg

mass of soil excavated from hole = 3.331 kg

Water content = 11.6%

$\gamma\ dmax = 1.75\ g/cm^3$

R = 95%

Sol: mass (sand) to fill hole = 5.99 − 2.81 − 0.117 = 3063 g.

$$V(hole) = V(sand) = \frac{3063}{1.667} = 1837.43\ cm^3$$

$$\gamma = \frac{3331}{1837.43} = 1.813\ g/cm^3$$

$$\gamma d = \frac{1.813}{1+0.116} = 1.624\ g/cm^3$$

$$R = \frac{\gamma\ field}{\gamma\ max} = \frac{1.624}{1.75} = 92.8\% < 95\%$$

∴ Soil does not satisfy requirement

3.063

3067

11.6%

3331

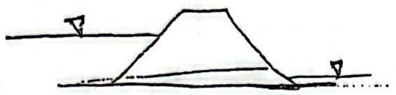
H.w. 3,5,8,15.

Lecture Chapter 7: Flow of water in soil: permeability an seepage:

* Permeability : is the property of soil which permits water to flow through its voids.

- Some engineering properties:-

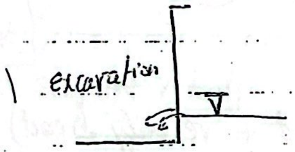
1) Dams:



to calculate leekage %.

2) Rate of consolidation a settlement:-

3) dewatering of deep excavations

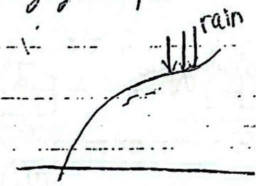


choosing in pumps

$D_{15(F)} < 4D_{85(B)}$

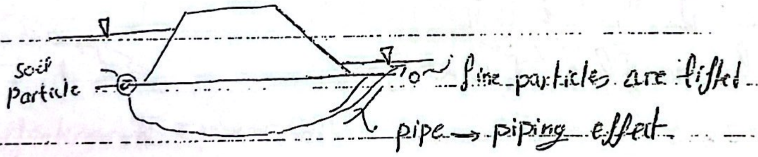
$D_{15(F)} > 4D_{15(B)}$

4) Stability of slope:



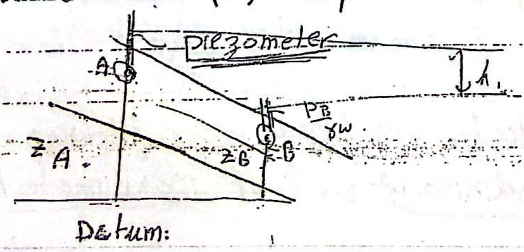
voids

5) ~~Corresion~~ Erosion and transportation of fine-grained soil "piping effect".



* Terms:

a) Hydrolic gradient: Total head (h) at a point in water under motion is



0599929302

$h_A = Z_A$ (elevation head)

$h_B = Z_B$ (" ")

$h_A = \underset{\text{Total head}}{} \; \underset{\text{elevation head}}{Z_A} + \underbrace{\frac{P_A}{\gamma_w}}_{\text{pressure head}} + \frac{V^2}{2g}$ ~ velocity head ≈ 0 [neglected]

$$\Rightarrow \boxed{h_A = Z_A + \frac{P_A}{\gamma_w}}$$

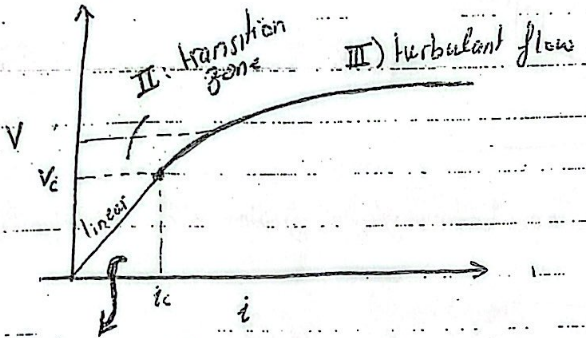
$$* \boxed{h = h_A - h_B}$$

$* \boxed{i = \frac{h}{l} = \text{hydrolic gradient}}$ ؟ مقدار الطاقة المفقودة في كل وحدة طول

---

energy (elevation head + pressure head + velocity head).

* Relationship between i and v



I) laminar flow

II) transition zone

III) turbulant flo

قاعدة دارسي

I) laminar flow [$i \propto V$].

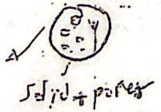
كل جزئ يمشي في خط لا يتداخل مع الآخر

$V \propto i$ ⇒ Dary law

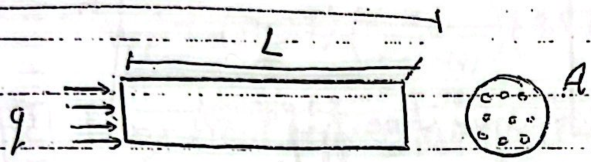
$\Rightarrow \boxed{V = k\, i}$, where k = coefficient of permeability [m/s]

V = discharge velocity

V = discharge velocity, quantity of water flowing in $\boxed{\text{unit}}$ time through gross-sectional area of soil at right angle to the direction o



Soil type	k [cm/S]
gravel	1 - 100 ↑ high
Coarse sand	0.01 - 1 ↑ high
fine sand	0.001 - 0.01 ↑ high
Silty clay	0.0001 - 0.001
Clay	< 0.000001



$Av + As = A$

$$\boxed{\begin{array}{l} q = V * A \\ q = V_s * A_v \end{array}}$$

where V_s = seepage velocity.

$$\Rightarrow q = V * A = [A_v + A_s] * V = V_s * A_v.$$

$$\Rightarrow V_s = \frac{V[A_v + A_s]}{A_v} = \frac{V[A_v + A_s]}{A_v} * \frac{L}{L} = \frac{V[V_v + V_s]}{V_v}$$ divide by V_s

$$\Rightarrow \boxed{V_s = V\frac{[e+1]}{e}} \Rightarrow \boxed{V_s = \frac{V}{n}}$$

seepage V > V discharge.

* factors affecting 'k' ; permeability:

1) viscosity of fluid
2) pore-size distribution   clay has high void ratio but dif in —
3) grain-size distribution
4) Degree of saturation. [direct relationship].

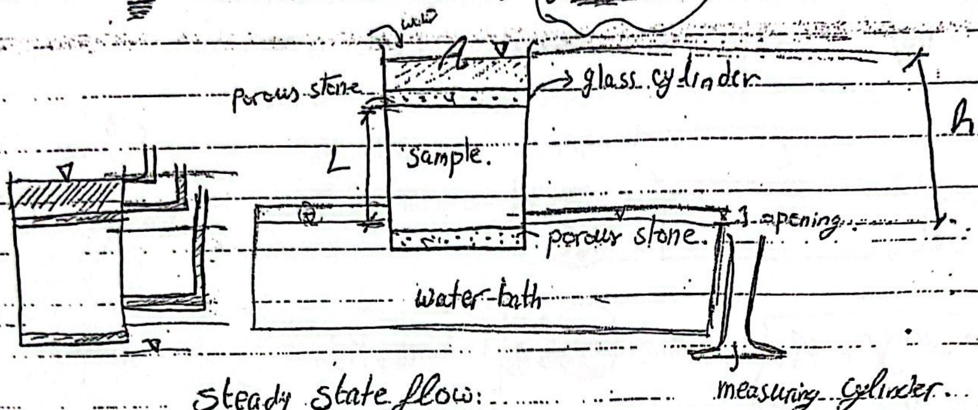
packing

Vice president. J

* Determination of k :-

A in the laboratory:

1) Constant head test: for granular soil



steady state flow:

Q = #.

t = ?

$$Q = \overset{\text{unit time}}{[VA]} * t = [\overset{\text{darcy}}{\underline{k * i}} * A * t]$$

$$= k \left[\frac{h}{L}\right] * A * t$$

$$\boxed{k_{T_i} = \frac{Q * L}{hAt}} \quad \frac{m^3}{m^2 * s}$$

Q = volume

$$\boxed{k_{20°C} = \left(\frac{\eta_{T°C}}{\eta_{20°C}}\right) * k_{T°C}}$$ given in table 7.2 in book

Example: for a Constant head test,

diameter of sample = 150 mm, L = 300 mm, h = 500 mm

Time of collection = 5 min , q = 350 cm^3 , T = 24°C.

wated: find $k_{20°C}$.

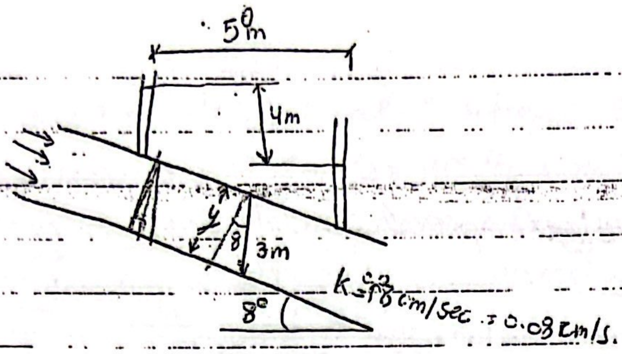
$\Rightarrow k_{24°C} = 3.96 * 10^{-2}$ mm/sec = $3.96 * 10^{-3}$

$\Rightarrow k_{20°C} = 3.96 * 10^{-3}$ cm/sec * 0.9097

$= 3.6 * 10^{-3}$ cm/sec.

∴ fine sand soil.

* Example



Find $q = ?$ [in m^3/s]

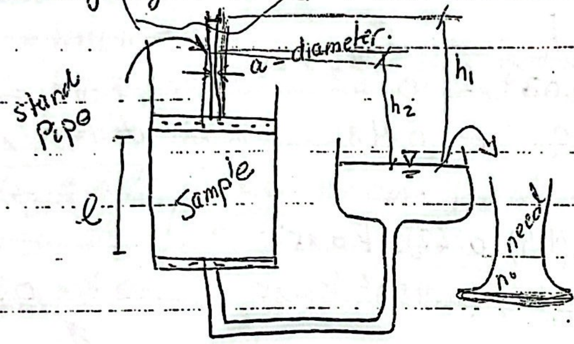
$\boxed{q = (k\,i) * A}$

$L = 50/\cos 8° = 50.491$ m

$\Rightarrow q = \left(\frac{0.08}{100}\right)\left(\frac{4}{50.491}\right)(2.971)(1) = 0.188 * 10^{-3}$ m^3/sec/m width.

$\boxed{2}$ falling head test / variable head test

+ suitable for fine soils



$q = a\left(\frac{dh}{dt}\right)$.

$$\Rightarrow \boxed{k = 2.303 \frac{a\,l}{A\,t} * \log \frac{h_1}{h_2}}$$

للحفظ

Example: result of falling (variable) head test

$A = 1200$ mm², $L = 150$ mm, $a = 50$ mm², $t = 5$ min,

$h_1 = 400$ at $t = 0$, $h_2 = 200$mm at $t = 5$ min.

$\Rightarrow k = 1.44 * 10^{-3}$ cm/sec.

* Determination of "k" using empirical correlations:

k كتوبر

a. for uniform sand: ≈ same size

$$\Rightarrow k\,(cm/s) = \underset{(1-1.5)}{C} * \underset{(mm)}{D_{10}^{2}}$$

→ Coarse sand 1.5
find sand 1

b. for fine to medium clean sand → no silt or clay ; fine materi

$$\Rightarrow k_e = 1.4(e^2)\,\underline{k_{0.85}}$$

↳ coefficient of permeability for e = 0.85

example

ex	k	e
	0.03	0.62
not linear interpolation	? ←	0.48

k will decrease, less voids

$\Rightarrow 0.03 = (1.4)(0.62)^2\, k_{0.85}$

$k = (1.4)(0.48)^2\, k_{0.85} \rightarrow k = 0.018.$

average value in calculations.

c. For fine sand to medium, clean.

$$\Rightarrow k = C_1 \frac{e^3}{1+e}$$

for previous example, k = 0.015.

d. for normally consolidated clay:

+ normally consolidated: current pressure is the highest in soil his
+ over consolidated: current " " not the " " "

$$\Rightarrow k = C_3 \frac{e^n}{1+e}$$

∴ solve equations for both "n" and "C"

e	k
1.1	$0.302 * 10^{-7}$
0.9	$0.12 * 10^{-7}$
0.75 →	??

solve for $k_{0.75}$

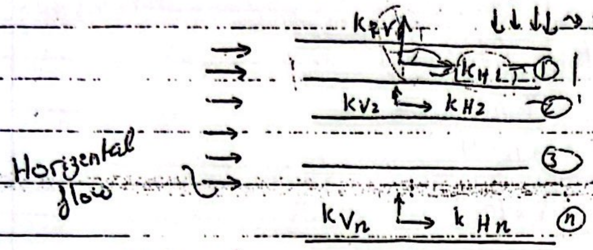
$n = 5.1$

$C_3 = 0.39 * 10^{-7}$

$k = 0.514 * 10^{-8}.$

* Equivalent permeability in stratified soil: "isotropic" soil.

isotropic properties in all dimentions are the same



assume: "$k_{H1} = k_{V1}$" in the same layer

→ Horizental flow

① $k_{H(eq)} = \frac{1}{H} \left(k_{H_1} * H_1 + k_{H_2} * H_2 + \cdots + k_{H_n} * H_n \right)$

where H = total thicknes of all layers.

$q = vA$

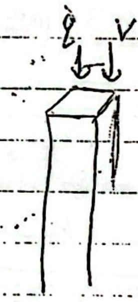
$V_1 = k_{H1} * i_1$

$V_2 = k_{H_2} * i_2$ } $(i_1 = i_2) \Rightarrow$ same hydrolic gradient for all layers.

② $\boxed{q_{eq} = k_{Heq} * i}$ $\quad$ i is the same for all layers.

→ Vertical flow

$$k_{veq} = \frac{H}{\frac{H_1}{k_{V_1}} + \frac{H_2}{k_{V_2}} + \cdots + \frac{H_n}{k_{V_n}}}$$



A: same q, diff V, ~~[illegible]~~ [illegible]

Vertical: Same V, diff i

same q

$V_1 = V_2$, $k_1 i_1 = k_2 i_2$

$~~q_{eq}~~ = q_1 = q_2 = \cdots = q_n = q$

$V_1 = k_{V_1} * i_1$

$V_2 = k_{V_2} * i_2$

$\Rightarrow V_1 = V_2$ [is applied]

but $i_1 \neq i_2 \neq i_3 \neq \cdots \neq i_n$ ⊛

$i_1 = \frac{\Delta h_1}{H_1}$ $\quad$ $i \propto \frac{1}{k}$

$V_{eq} = V_1 = V_2 = V_3$

$k_{veq} \cdot i = k_1 \cdot i_1 = k_2 \cdot i_2 = k_3 \cdot i_3$

* Example # 3:

Layer	Thickness (ft)	k_H (ft/min)
1	20	$1*10^{-1}$
2	5	$1*10^{-4}$
3	10	$1.5*10^{-1}$

* for this ~~lay~~ soil, find $k_{H\,eq}$

$\Rightarrow k_{H\,eq} = 0.1$ ft/min.

* assume isotropic flow, and calculate $k_{V\,eq}$

$\Rightarrow k_{V\,eq} = 6.96 * 10^{-4}$ ft/min.

* ratio $\dfrac{k_{H\,eq}}{k_{V\,eq}} = \underline{\underline{144}}$.

for most soil types $\dfrac{k_{H\,eq}}{k_{V\,eq}} > 1$ [flow in horizental direction is easi

* Example # 3: imp.

2.0 m $k_1 = 5*10^{-4}$ cm/sec

5.0 m $k_2 = 2*10^{-2}$ cm/sec

3.0 m $k_3 = 3*10^{-3}$ cm/sec

given: $i = 0.3$ for vertical and horizental.

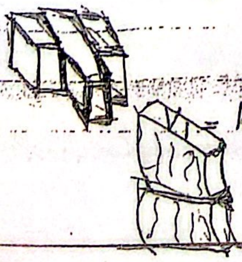
a) Calculate k_H / k_V [equivallent]

$\Rightarrow \dfrac{k_{H\,eq}}{k_{V\,eq}} = \dfrac{0.011}{1.9*10^{-3}} = 5.78$

b) For horizental flow, find the discharge and discharge velocity.

$q = k_{H\,eq} * i = (0.011 * 0.3) * [1\,cm * 1000\,cm]$

$= 3.3\ cm^3/\ sec/\ cm$ width

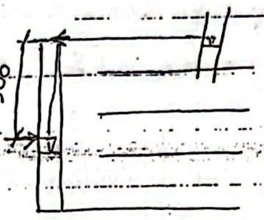


$V_1 = k_{H_1} * i_1 = (5*10^{-4})(0.3) = 1.5 * 10^{-4}$ cm/sec

$\Rightarrow V_2 = H$, $V_3 = H$.

(c) for the vertical flow, find the hydrolic gradient and head loss in each layer

$i_{eq} = \frac{\Delta H}{L}$



$0.3 = \frac{\Delta H}{1000} \Rightarrow \Delta H = 300$ cm.

$V_{eq} = V_1 = V_2 = \ldots = V_n$

$V_{eq} = k_{v\,eq} * i_{eq}$

$\Rightarrow V_{eq} = (1.9 * 10^{-3}) * (0.3) = 5.7 * 10^{-4}$ cm/sec

$\Rightarrow V_1 = 5.7 * 10^{-4} = (5 * 10^{-4}) * i_1 \Rightarrow i_1 = 1.14$

$i_1 = \frac{\Delta H}{200}$

$\Rightarrow \Delta H = \underline{228 \text{ cm}}$

Similarly $V_2 = 2 * 10^{-2} * i_2 \Rightarrow i_2 = 0.0285$

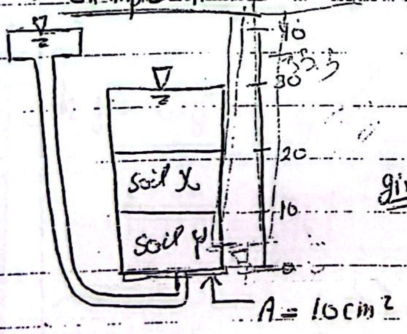
$\Rightarrow \underline{\Delta H} = 14.25$ cm.

$V_3 = 3 * 10^{-3} * i_3 \Rightarrow i_3 = 0.19$

$\Delta H = \underline{57 \text{ cm}}$.

~~Afnan~~ Battout.

* Example # 4:



$* \Delta H = 10$ cm.

given $\begin{cases} * k \text{ for } x = 4 * 10^{-3} \text{ cm/sec.} \\ * \text{head loss in } Y \text{ is } 9 \text{ times head loss in } X. \end{cases}$

$\therefore \underline{k_Y << k_X}$.

* a) what is ky?

$* V_x = V_y$

$* 9h_x + h_x = 10$ cm.

$\Rightarrow h_x = 1$ cm & $h_y = 9$ cm

$V_x = k_x * i_x = (4 * 10^{-3})\left(\frac{1}{10}\right) = k_y * \left(\frac{9}{10}\right) \Rightarrow \underline{k_y = 4.44 * 10} \text{ cm/sec}$

(b) what is q/hr [disharge in 1 hr]

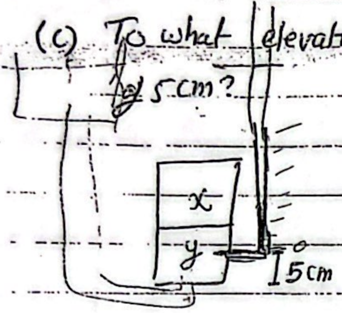
$q_{eq} = q_1 = q_2$ $s \rightarrow hr$

$q_{eq} = k_x * i_x * Area = (4*10^{-3})(\frac{1}{10})(10\ cm^2) * (60*60)$

$= 14.4\ cm^3/hr.$

(c) To what elevation water will rise in piezometer inserted in y @ an el

by 5 cm?

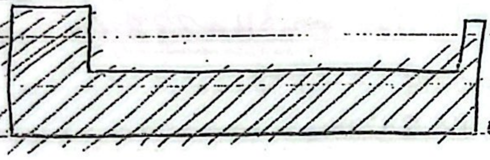


$i = \frac{\Delta H}{L} = \frac{9}{10} = \frac{\Delta h}{5} \Rightarrow \Delta h = 4.5\ cm.$

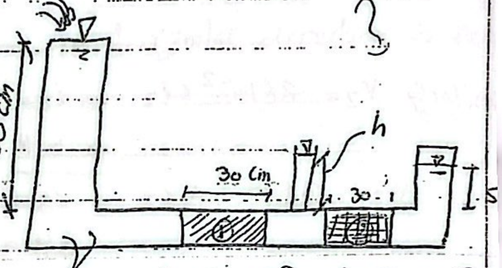
$\Rightarrow$ pilometer reaches elevation 35.5.

---

* Example 5:



50 is constant.



* Wanted: distribution of the 45 cm head loss.

$A = 25\ cm^2$

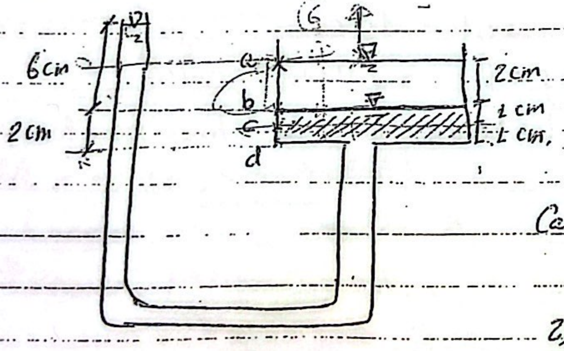
$k = 2*10^{-2}$ cm/s

$k = 7*10^{-3}$

Note: this is a vertical flow.

---

* Example 6:



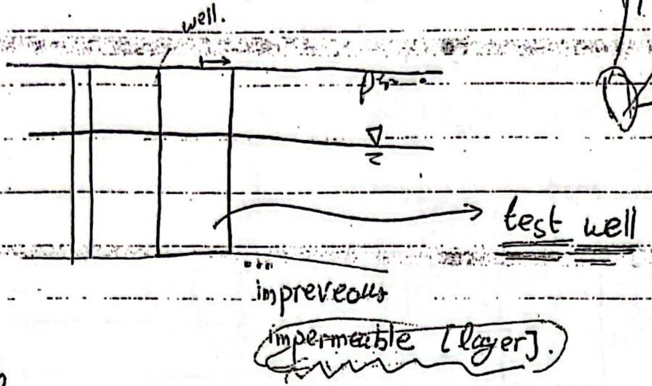
Calculate:

1) hp → pressure head

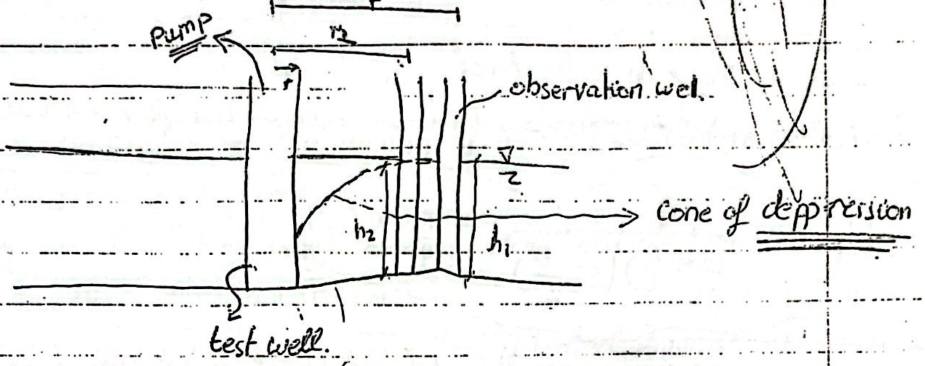
2) he → elevation head

total head 3) h_T at points a, b, c, d.

$\boxed{1}$ * Permeability test in the field by pumping from wells:-



radies = R.

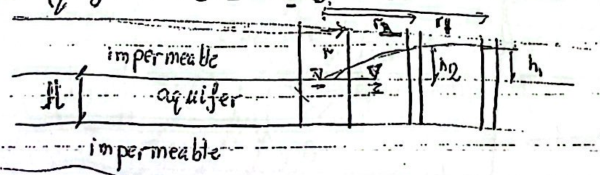


pump water until steady state flow is reached.

$$K = \frac{2.303 * \left(\log \frac{r_1}{r_2}\right) * q}{\pi (h_1{}^2 - h_2{}^2)}$$

$\left(\log \frac{r_1}{r_2}\right)$

$\boxed{2}$ k for confined aquifer الطبقة الحاملة للمياه الجوفية مضغوطة

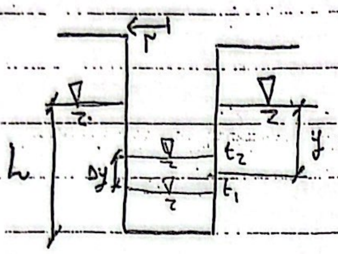


h_1, h_2 الفرق في المناسيب

$$k = \frac{q * \log\left(\frac{r_1}{r_2}\right)}{2.727 * H * (h_1 - h_2)}$$

$\boxed{3}$ K - from auger holes
حفارة

single hole is made, "slug test".



$\Delta y \equiv$ Change in water level.

$y \equiv$ المسافة من سطح الماء / منسوب المياه الجوفية إلى منتصف التغير الحاصل.

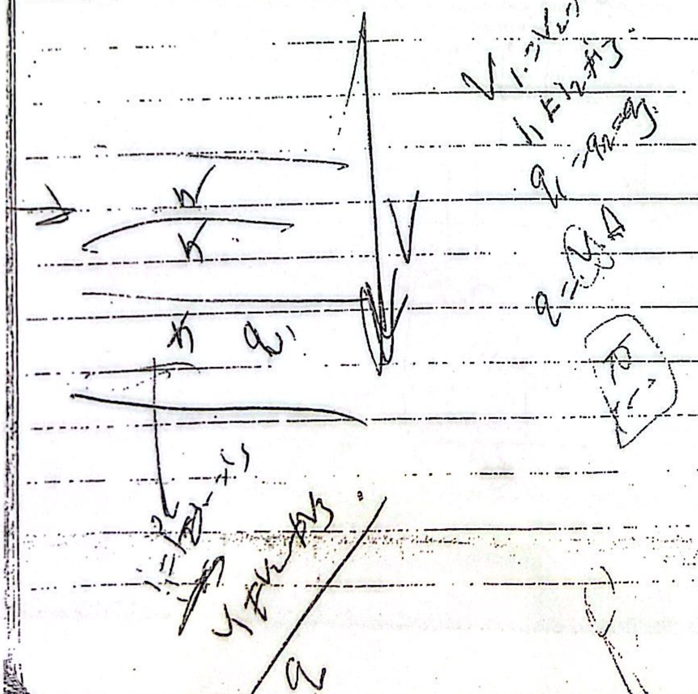
$$k = \frac{40}{\left(20 + \frac{L}{r}\right)\left(2 - \frac{y}{L}\right)} * \frac{r}{y} * \frac{\Delta y}{\Delta t}$$

* **Example #7:** An Auger hole test was performed & the following dat
collected:

$r = 0.15 m$, $L = 3.5 m$, $\Delta y = 0.45$, $\Delta t = 8 min$, $y = 3.2$

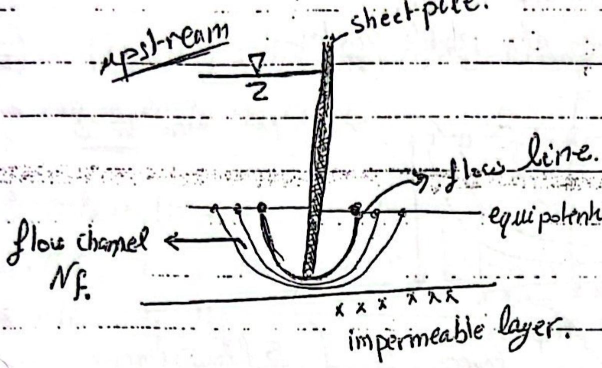
$\Rightarrow k = 2.24 * 10^{-3}$ m/min

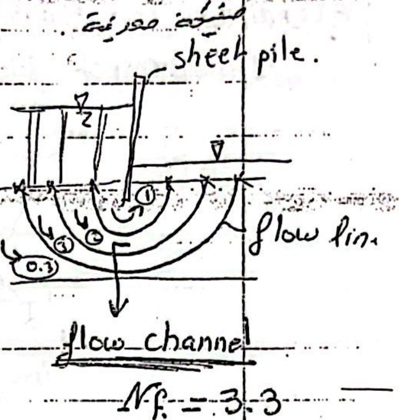
Coarse sand.



* Flownet: شبكة الجريان

1- for isotropic soil:- $k_x = k_z$



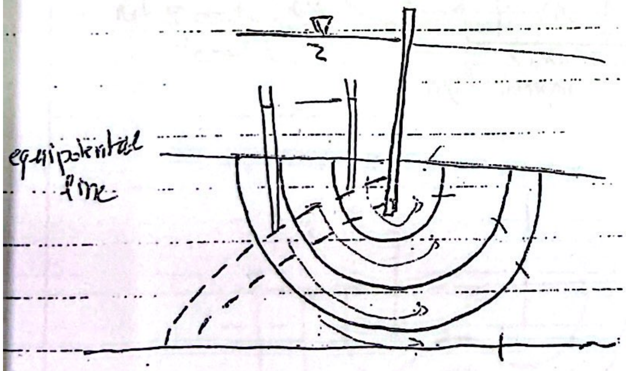


* Flow net: is a combination of a number of lines. It is constructed to calculate the ground water flow.

* Flow lines: a line along which a water particles travels from the upstream site to the downstream side.

* equipotential lines: a line with which the potential head at all points is the same. خط تساوي الطاقة total head.

impreveous sheet pile = flow line



Nf = 3.5
Nd = 5

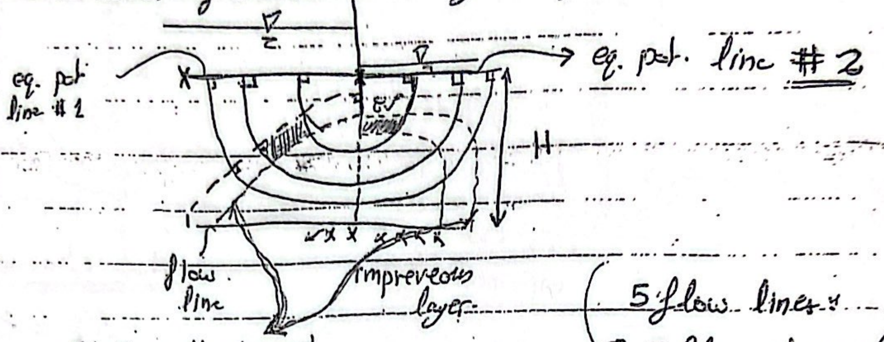
* Conditions of equipotential line & flow net:

1- flow net is constructed graphically to scale.

2-

* Conditions for equipotential lines:

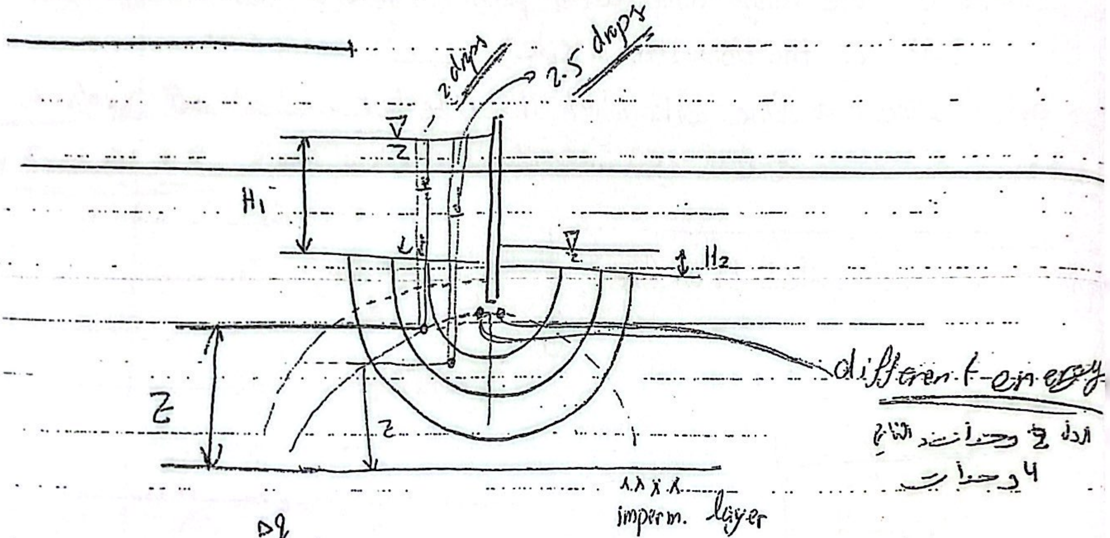
1- intersect flow lines at right angles.

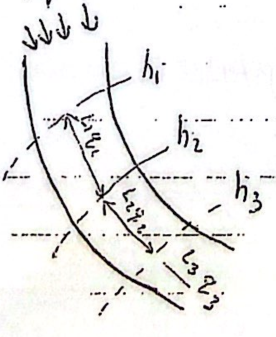


ليس لم نفس الطاقة

5 flow lines :
3.7 flow channels
5 drawn equipotential li

2- The flow element formed are approximatelly [square]
"field elements"





$$\Delta q = q_1 \neq q_2 = q_3 \cdots$$

$$\Rightarrow \Delta q = k i_1 A_1 = k i_2 A_2 = \cdots$$

$\Delta q = K_1 \left(\frac{h_1 - h_2}{L_1}\right) * (L_1 * 1) = K \left(\frac{h_2 - h_3}{L_2}\right) * (L_2 * 1) =$ ——

$(h_1 - h_2) = (h_2 - h_3) = (h_3 - h_4)$ مقدار الطاقة للسائل عند الانتقال من equip لآخر متساوي

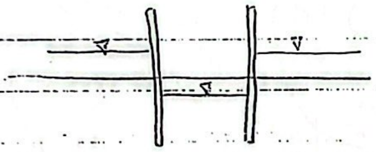
→ loss in energy is equal.

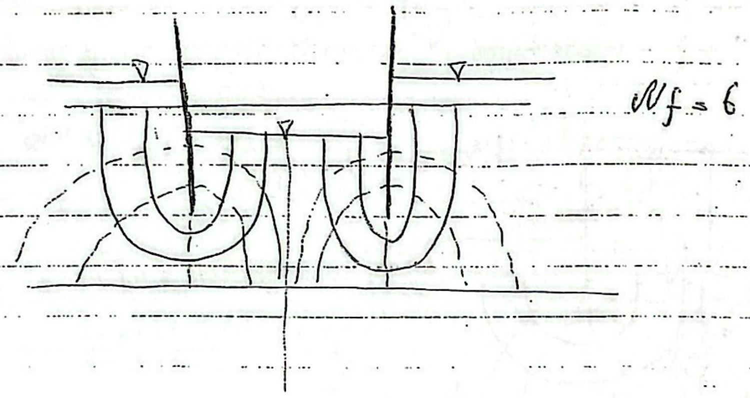
$* \Delta h = H_1 - H_2$

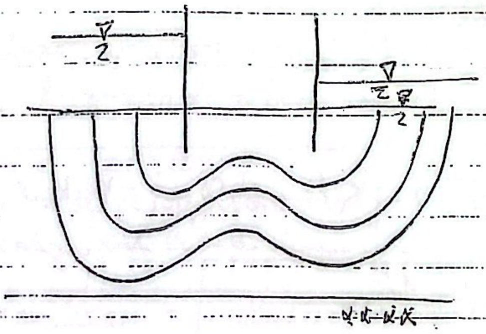
$* N_d = 6$ in this case [# of drops].

$\Rightarrow \Delta h = \frac{H_1 - H_2}{6}$

$\Rightarrow q_T = k \left(\frac{\Delta H}{N_d}\right) * N_f$ [N_f = # of flow channels].



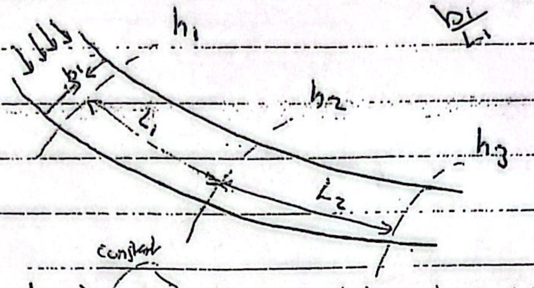




* Condition of rectangular intersections and square intersections:

⇒ width = constant for all elements.

length , $\frac{b_i}{L_i} = n$.



constant

$$\Delta q \Rightarrow k * \left(\frac{h_1 - h_2}{L_1}\right)(b_1 * 1) = k\left(\frac{h_2 - h_3}{L_2}\right)(b_2 * 1).$$

$$\Rightarrow \Delta q = k\,(h_1 - h_2) * n = k\,(h_2 - h_3) * n$$

$$\Rightarrow \Delta q_{total} = k * \left(\frac{\Delta H}{Nd}\right) n * N_f$$

$V \propto h$

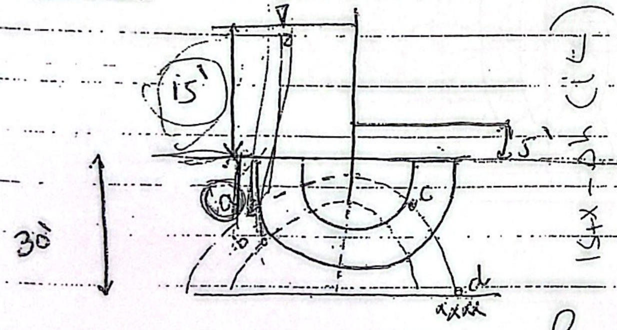
for square $n = 1$

for rect $n = \frac{b}{L}$

* Example:

isotropic soil:

$k_x = k_z = 5 * 10^{-3}$ cm/sec



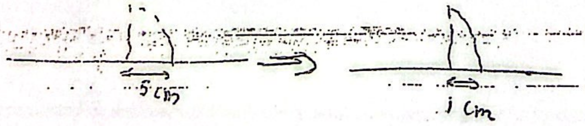
Note If Non-isotropic, $k_x \neq k_z$.

Tran → horizental: $\sqrt{\frac{k_z}{k_x}}$? < 1 نغير b , vertical نثبت الـ

scale الافقي بنفس درجة

$$q = \sqrt{k_x * k_z}\left(\frac{\Delta H}{Nd}\right) * N_f.$$

ex. $\sqrt{\frac{k_z}{k_x}} = 5$ cm



a) what is the height above ground surface the water will rise in piro meter at a, b, c, d.

- $N_f = 2.8$ approx ≈ 3
- $N_d = 6$
- $\Delta H = 10$ ft.

$\Big\}\ \frac{10}{6} = 1.67$ ft each drop

$\Rightarrow$ @ a, 15 - 1 drop = 15 - 1.67 = 13.33

$\Rightarrow$ pressure of water = $13.33 * \gamma_w = 13.33\,[62.4]$.

@ b, 15 - 2(1.67) = 11.67. pressure = 11.67 * 13.33.

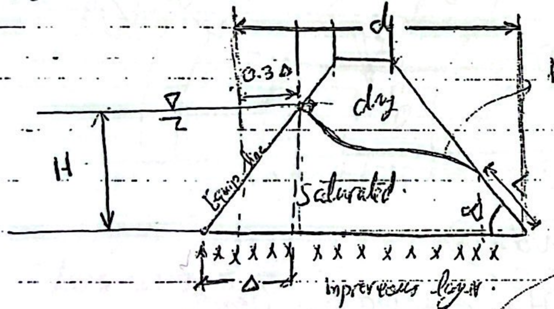
@ C = d, 15 - 5 * 1.67 = 6.67'.

b) Calculate $\Delta q = ?$

$$\Delta q_f = \underset{\nearrow \atop k\left(\frac{\Delta H}{N_d}\right)(N_f)}{k} \left(\frac{10}{6}\right)(3) = 8.2 * 10^{-4}\ \text{ft}^3/\text{sec}/\text{ft width}.$$

---

* Seepage through an earth dam on an impreveous layer.



top flow line
" phreatic line

L: مسافة من اول flow line خروج اول flow line من الجهة الثانية

1) By equations; analytical method.

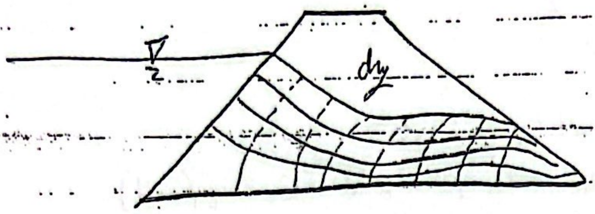
$$q = k * \tan\alpha * L * \sin\alpha.$$

*phreatic line [top flow line]

$$L = \frac{d}{\cos\alpha} - \sqrt{\frac{d^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}$$

Anas Thaher Ballout.

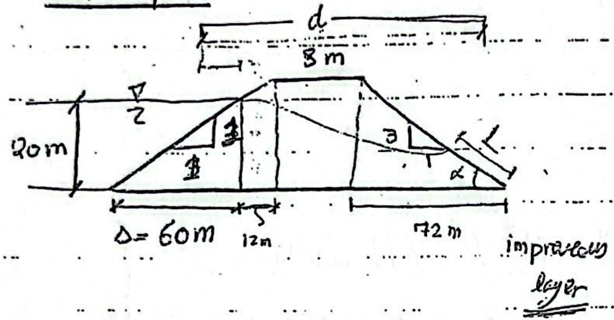
2) By drawing the flow net.



N_f } من التوليج

N_d } الناتج

Example:



same slope both sides

$k_x = 4 * 10^{-4}$ cm/sec.

$k_z = 2 * 10^{-4}$

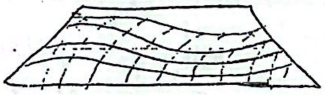
$L = 0.634$ m (0.6394)

$\Rightarrow q = \sqrt{k_x * k_z}$ ------ $= 0.0512$ cm^3/sec/cm width.

If a flow net is drawn,

$q = 0.062$

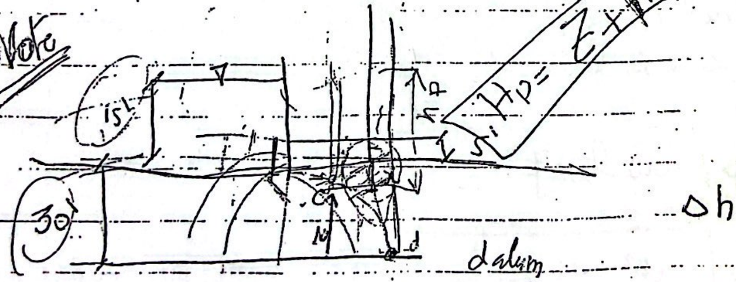
0.0514



$N_d = 20$

H = Z + pressure head.

* Note



$\Delta h = 1.67'$

for c: $Z = 30 - x$

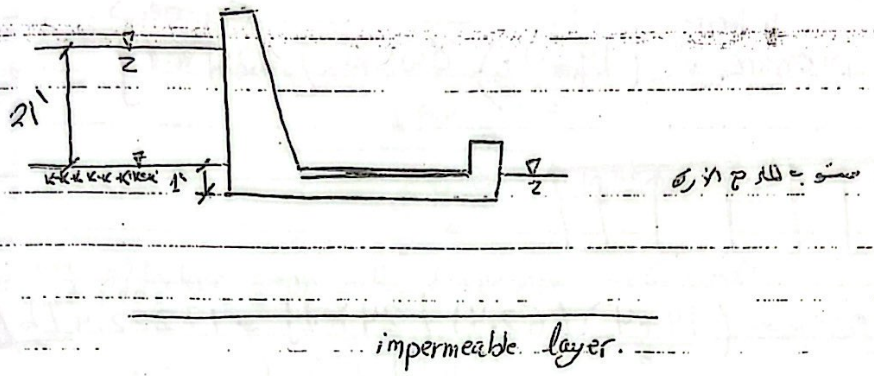
$h_p = ((x + 15) - \underbrace{5 * 1.67}_{\text{all down drop}}) = x + 6.65$

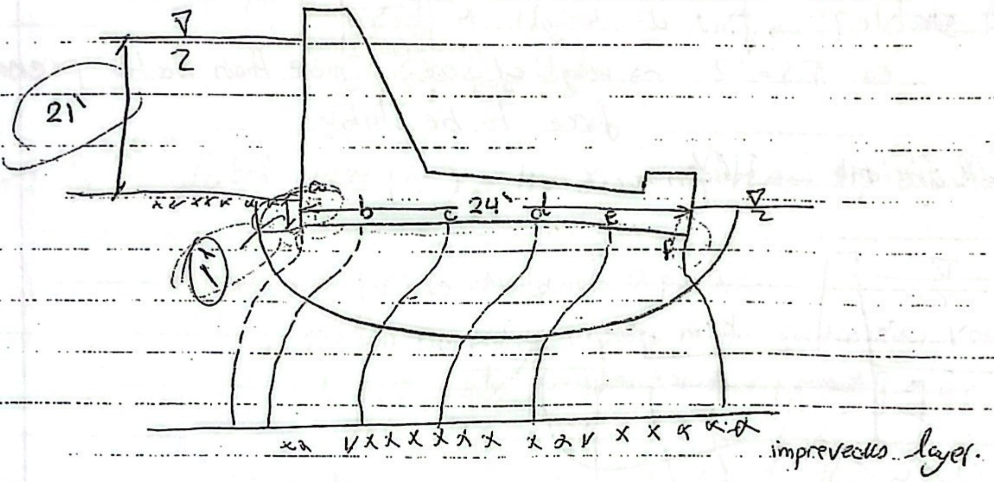
$\Rightarrow H_c = 36.65$ m.

for point d:- Z = zero

hp = 45 = 5 + 1.67 = 36.65.

* Pressure affecting marine structurer:





$L_{ess} = \frac{21}{7} = 3`$

wanted: uplift force affecting the dam.

@ a: hp = (21 + 1) - 3 = 19` ⟶ 19 γ_w = pressure.

@ b: hp = (22) - 6 = 16` ⟶ 16 γ_w

@ c hp = 22 - 9 = 13` ⟶ 13 γ_w

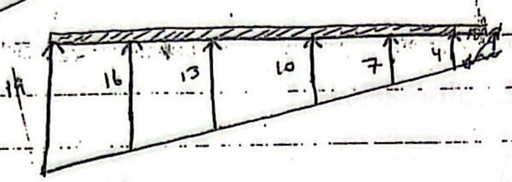
@ d hp = 22 - 12 = 10` ⟶ 16 γ_w

@ e hp = 22 - 15 = 7` ⟶ 7 γ_w

@ f hp = 22 - 18 = 4` ⟶ 4 γ_w

Σ

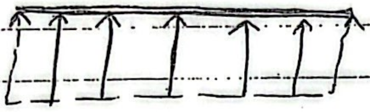
Pressure head is taken into consideration



almost linear

↑ force

average pressure = $\left(\frac{19+4}{2}\right) \gamma_w * [24 * 1]$ area



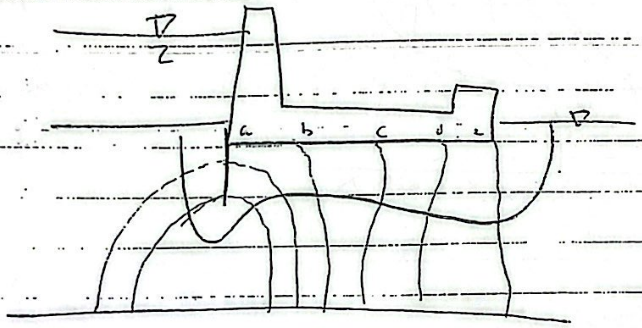
average force = $\left(\frac{19+4}{2}\right)(62.4)[24 * 1] = 17222.4$ lb

Is it stable? → find its weight * F.S.

ex. F.S = 2 → weight of section more than double p

force to be stable

If unstable → solution :-



assume 10 equipotential line:

drop = $\frac{21}{10} = 2.1$

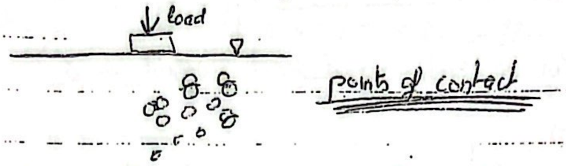
for a, 4.5 drops $22 - (4.5)(2.1) = 12 \gamma_w$ pressure

* Effective stress:- (σ'):-

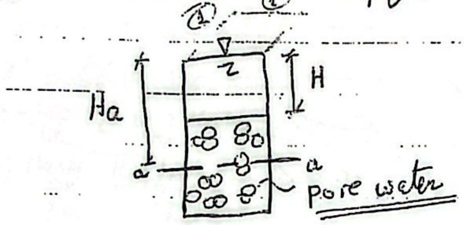
- Effective stress: force per unit area carried by the soil skeleton. it controls the volume change and strength in the soil.

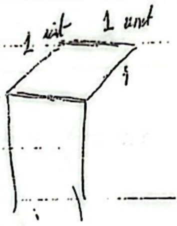
higher effective stress will induce denser soils.

soil skeleton at its contact points.



1) effective stress without seepage: static condition :





$$\sigma = \frac{(1 \times 1 \times H)(\gamma_w)}{1 \times 1}$$

كل الأوزان فوق مستوى معين

Total stress $(\sigma) = H * \gamma_w + (H_A - H)\,\gamma_{sat}$.

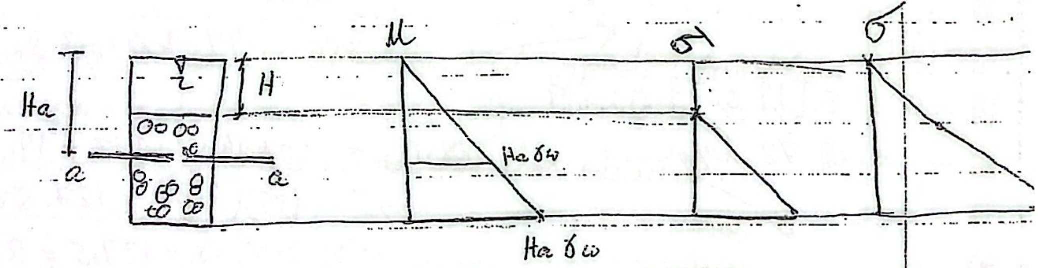
⇒ Total stress (σ) is divided into 2 parts:

1) portion carried by water in the continuous void space.
[pore water pressure, neutral pressure] ≡ u
$u = H_a * \gamma_w$

2). portion carried by the soil solid at their points of contact
[effective stress, σ']

على مستوى معين ، هذا المجموع ثابت

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



$\sigma = \sigma' + u$

$\Rightarrow \sigma' = \sigma - u$

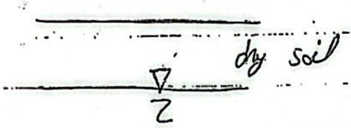
$= [H \gamma_w + (H_a - H)(\gamma_{sat})] - [H_a \gamma_w]$

$$\Rightarrow \boxed{\sigma' = (H_A - H)(\gamma_{sat} - \gamma_w)}$$

$\gamma_{sat} - \gamma_w$ = submerged unit weight = γ'

---

* Example # 1:

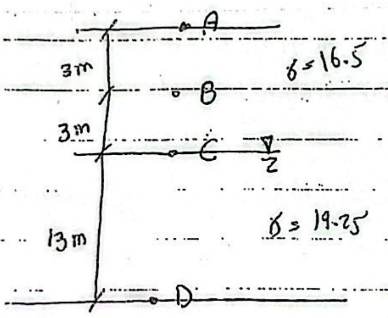


لو تحت مستوى الماء أي الغاطس ونضيف وزن التربة الجافة فوق مستوى الماء

$\times \sigma'$

---

* Example # 2:



@ A, $\sigma = 0$, $u = 0$, $\sigma' = 0$.

@ B, $\sigma = 3 * 16.5 = 49.5$ kN/m²
$u = 0$
$\sigma' = 49.5$ kN/m²

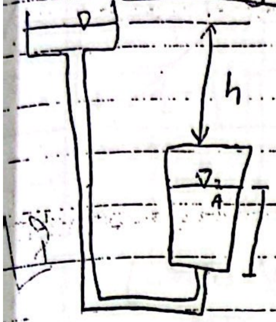
@ C, $\sigma = 6 * 16.5 = 99$ kN/m²
$u = 0$
$\sigma' = 99$ kN/m².

or @ D:
$\sigma' = 13[19.25 - 9.25) + 99$
$= 221.72$ kN/m²

✓ same answer

@ D, $\sigma = (6 * 16.5) + (13 * 19.25) = 3$
$\rightarrow u = (13)(9.81) = 127.52$ kN/m²
$\sigma' = 349.5 - 127.5 = 221.72$

Upward flow:



$\sigma' = Z\gamma'$

Seepage

$\sigma' = Z\gamma' - iZ\gamma_w$

↖ seepage pressure.

? $\cancel{H} = H_1 + Z + iZ\gamma_w = u$

$\sigma' = Z\gamma' - i_{cr} Z\gamma_w = 0$

$\Rightarrow Z\gamma' = i_{cr} Z\gamma_w$

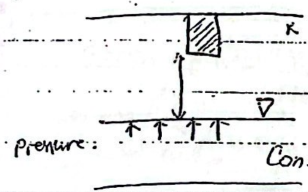
$$\boxed{i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1 + e}}$$

for most soil types

$i_{cr} = (0.9 - 1.1) \approx 1$

$\sigma' = 0 \rightarrow$ boiling or quick condition

meaning:



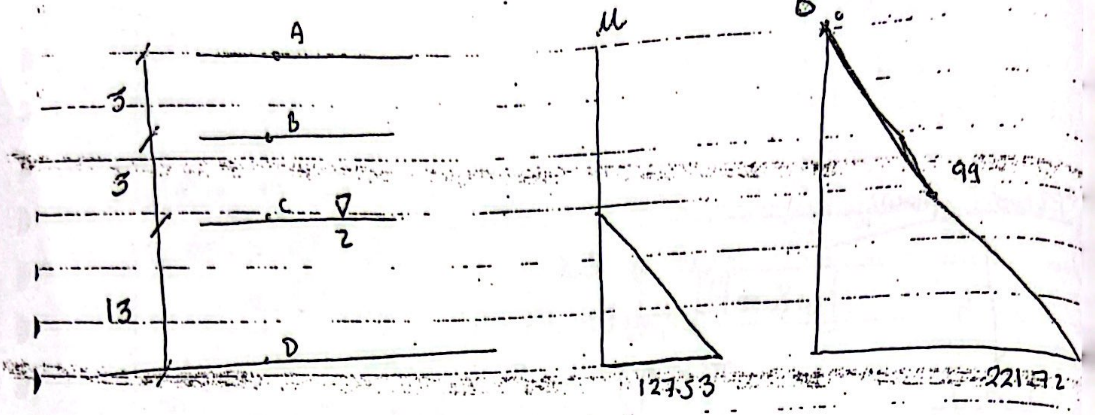
Stress = weight.

Confined soil. more hauling → water in soil will come out

effective stress = 0 → ~~yielding~~ bearing, shear strength of the soil = 0.

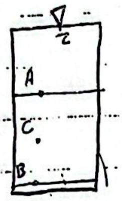
* occur only in fine sand or silt → due to its deposition conditions for which pore space is high.

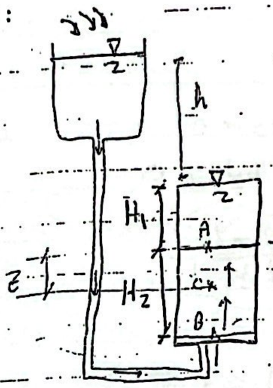
- In clay → Cohesion so the quick condition does not occur.



2 Stress in saturated soil with seepage :- [movement of water]

(a) Case a: upward seepage:





upward
effectiv
red

@ A:- $\sigma = H_1 * \gamma_w$

$u = H_1 * \gamma_w$

$\Rightarrow \sigma' = zero$

@ B:- $\sigma = H_1 * \gamma_w + H_2 * \gamma_{sat}$

$u = (H_1 + H_2 + h)\, \gamma_w \longrightarrow$ still no low.

$\sigma' = H_2 (\gamma_{sat} - \gamma_w) - h\gamma_w = H_2\, \gamma' - h\gamma_w$

@ C: $\sigma = H_1 * \gamma_w + Z * \gamma_{sat}$ $\quad i = \frac{h}{H_2} \rightarrow$ الفاقد المفقود للمسافة الي بيمشيها

$u = (Z + H_1 + h)\gamma - \underbrace{\left(\frac{h}{H_2}\right)}_{i} * \underline{(H_2 - Z)}$

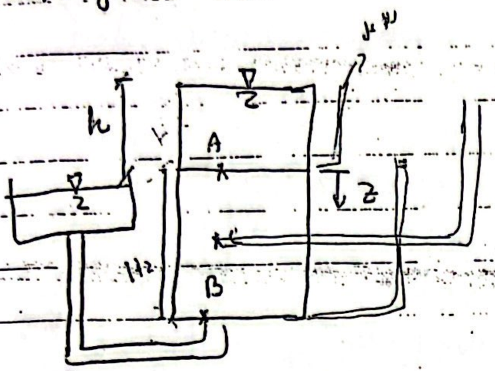
الطاقة الي متبقية

مقدار الطاقة المفقودة

$u = (Z + H_1)\gamma + \underbrace{\left(\frac{h}{H_2}\right)}_{i} * Z * \gamma_w$

$\rightarrow \boxed{\sigma' = Z\gamma' - i * Z * \gamma_w} \rightarrow$ general case.

- downward flow:

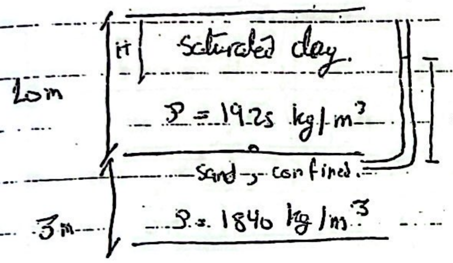


at B: loss $= \frac{h}{H_2} * H_2 = h$.

at C: $(H_1 + Z - i * Z)\,\gamma_w$

$$\Rightarrow \sigma' = Z\gamma' + iZ\gamma_w$$

* Example:



If sat. clay layer is excavated find the max. depth for which no problems occur "h".

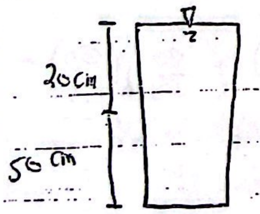
Note $\frac{\nabla}{z}$ saturated + submerged.

stress of sand = stress of upper layer

$\Rightarrow 6\gamma_w = (10 - H)\,\gamma_s$

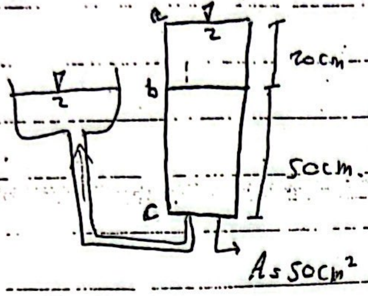
$\Rightarrow H = 6.88$ m

* Example # 2:



→ see next page

* Example #2:- find $\underline{k}$



$q = 0.03\ cm^3/sec.$

$\gamma_s = 2\ g/cm^3$

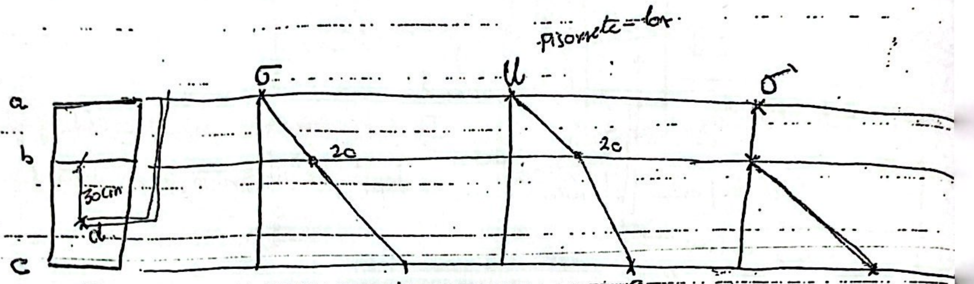
$\gamma_w = 1\ g./cm^3$

$q = i * k *$

$q = k * i * area.$

$\rightarrow 0.03 = k * \left(\frac{20}{50}\right)(50) \rightarrow k = \# = 1.5 * 10^{-3}\ cm./sec.$

2) plot the variation of σ, σ', u along the sides a, b, c.



$20+50-loss$

$= 20+50-20$

$= 50$

Note:- 1) $u = (Z + H_1 - iZ)\ \gamma_w = \left[(50+20) - \frac{20}{50} * 50\right] \gamma_w.$

2) $\sigma' = Z\gamma' + iZ\gamma_w = (50)(1) + \left(\frac{20}{50}\right)(50)(1) = 50 + 20 =$

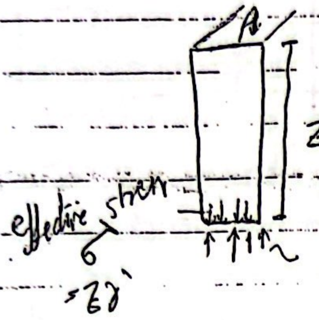
3) find h @ point d

$\Rightarrow$ at point d, $h = (30+20) - i * Z = (30+20) - \left(\frac{20}{50}\right)(30) =$

$\Rightarrow$ elevation above the sand layer = $\underline{8\ cm.}$

* **Seepage force:**

direction of seepage pressure = direction of flow.



A = cross sectional area.
upward flow.

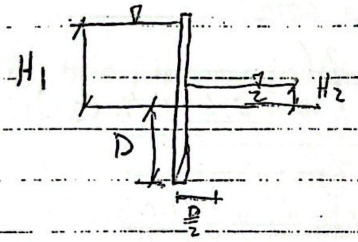
Seepage pressure = $i Z \gamma_w$

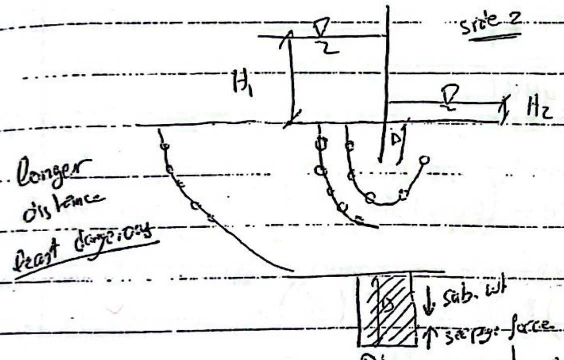
force = $i Z \gamma_w A$

+ Resultant force = $(Z\gamma' - i Z \gamma_w) A$

* Seepage force per unit volume = $\frac{i Z \gamma_w * A}{A Z} = i * \gamma_w$ → same direction of flow.

الارتفاع → swell
رفع التربة * Heave in soil due to flow around sheet piles:-





Heave at side 2 of the sheet pile.

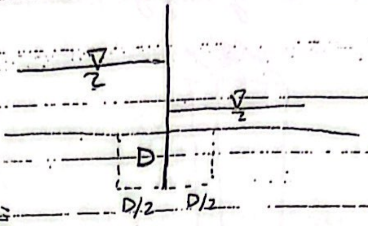
D/2 → critical region. نصف عمق الشيت هو منطقة الخطر

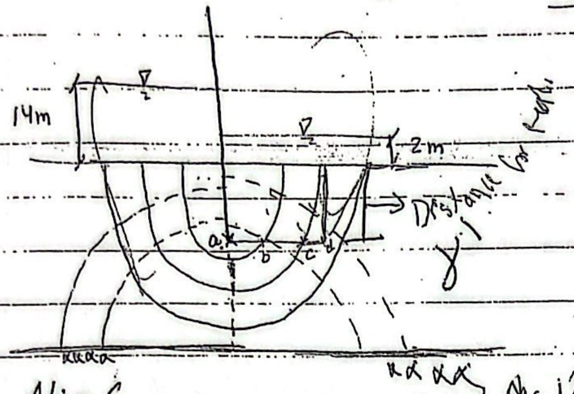
forces affecting this area:

1) submerged weight of soil ↓
2) seepage force ↑

+ Safety factor ≥ 4 [4 or 5]

* Seepage force / unit volume = $\underset{avg.}{i} * \gamma_w$





$N_j = 6$

$H = 12 m \Longrightarrow \Delta h = 2 m$

$\Delta h = 12m$
$l = 2m$

Driving head at a = 6 m = 14 - (3×2) =

at b = 2.2 * 2 = 4.4 m.

at C = 1.6 * 2 = 3.2m.

at d = 1.3 * 2 = 2.6.

$\Rightarrow$ average head = $\dfrac{6 + 4.4 + 3.2 + 2.6}{\underset{\text{عدد النقاط}}{4}} = 4.05 \ m$

$\Rightarrow i_{avg} = \dfrac{\text{average head}}{D} = \dfrac{4.05}{D}$.

w' = submerged weight.

U = uplift force = $(i_{avg} * \gamma_w)\left[\dfrac{D^2}{2} * 1\right] = \dfrac{i_{avg} * \gamma_w}{2}$

$w' = \left(\dfrac{D}{2} * D * 1\right) * \gamma' = \dfrac{D^2}{2}(\gamma')$.

Factor of safety against heave = $\dfrac{(D^2/2)\,\gamma'}{(i_{avg})\,\gamma_w\,(D^2/2)} = \dfrac{W'}{U}$

$\Rightarrow F.S. = \dfrac{\gamma'}{i_{avg} * \gamma_w} = \dfrac{i_{cr}}{i_{avg}}$

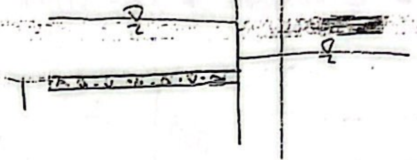
where F.S : 4-5 [minimum = 4] -

* Factor of safety against piping = $\frac{i_{cr}}{i_{exit}}$ → particle لآخر
(4-5)

من آخر equipotential line و الخط اللي على ال flow line
في المثال السابق بان وحدة طاقة والمسافة نفسها

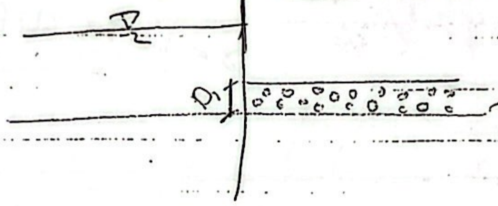
To increase factor of safety against piping:

1) Concrete pavement on the flow side
upstream.



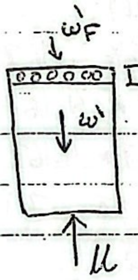
To increase factor of safety against heave:

1) add filter material at the down stream side.



1) small voids so that soil particles are not allow to pass it.
2) large enough so that it does not resist flow.

height of filter must be determined as follows:



≡ filter material.

resisting forces are $w' + w'F = \frac{D^2}{2}\gamma' + \left(\frac{D}{2} * D_1 * 1\right)\gamma'_F$.

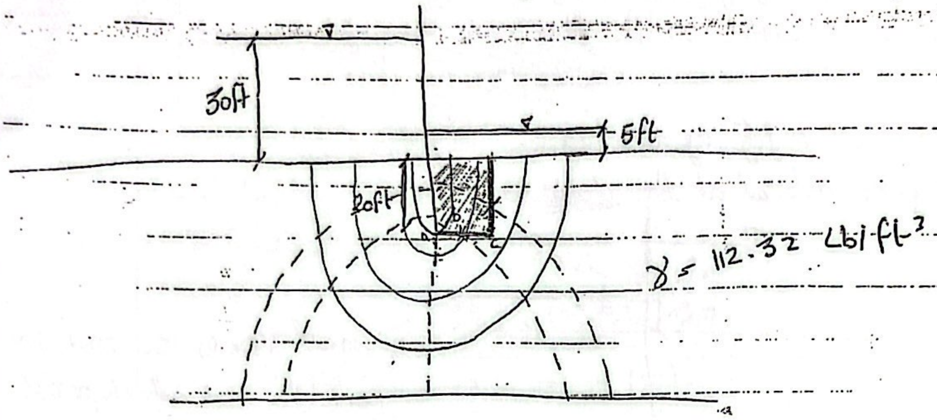
$U = i_{avg} * \gamma_w * D^2/2$.

$$\Rightarrow F.S = \frac{(D^2/2)\gamma' + (D/2 * D_1)\gamma'F}{D^2/2 * i_{avg} * \gamma_w} = \frac{\gamma' + \left(\frac{D_1}{D}\right)\gamma'F}{i_{avg} * \gamma_w}$$

If w/no filter F.S = 3
$(F.s)_{req} = 4.5$ ⇒ use law to find D_1 of filter

* **H.w.** → chapter 8 : 1, 3, 7, and 9.
chapter 9 : 1, 6, 7, 10 and 11.

* Example # x :- check the safety against heave & piping



head = 25 ft.

$N_d = 6$ ft.

⇒ each drop = $25/6 = 4.167$

⇒ driving head at "a" = $3 * 4.167 = 12.5$ ft.

at "b" = $2 * 4.167 = 8.33$ ft.

at "C" = $1.7 * 4.167 = 7.1$ ft.

⇒ average head = $\frac{12.5 + 8.33 + 7.1}{3} = 9.3$ ft

⇒ $i_{avg} = \frac{\text{average head}}{D} = \frac{9.3}{20} = 0.46$

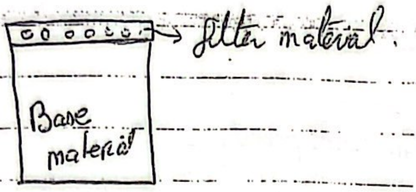
1) F.s against heave = $\frac{\gamma'}{i_{avg}\,\gamma_w} = \frac{(112.32 - 62.4)}{(0.46)(62.4)} = 1.74 < 4.$

Now find D_1 if $\gamma'_f = 60$ Lb/ft^3 & F.s = 4.

2) F.s against piping = $\frac{\gamma'}{(i_{exit})(\gamma_w)} = \frac{112.32 - 62.4}{(0.62)(62.4)} = 1.28$

$i_{exit} = \frac{4.16}{\approx 7} = 0.62$

* Suggestions for choosing filter materials:

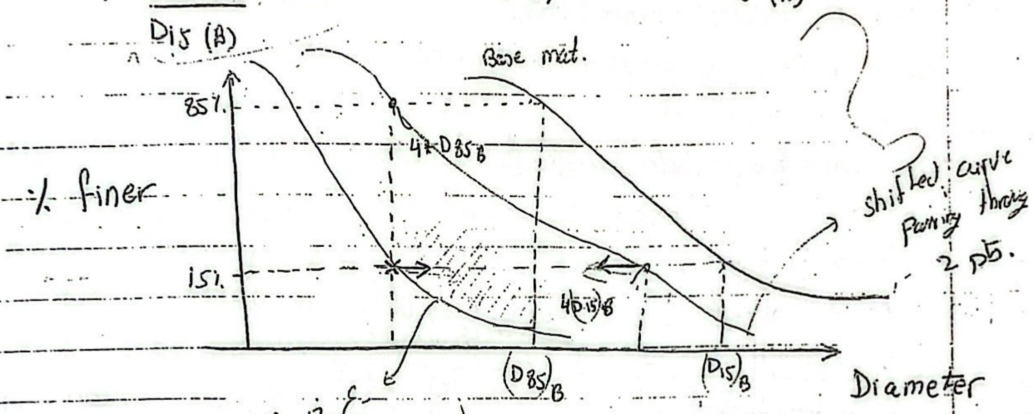


\+ Terzaghi and Peck suggested:

1) $\frac{D_{15}(F)}{D_{85}(B)} < 4 \Rightarrow (D_{15})_F < 4 (D_{85})_B$

reason: to prevent movement of soil particles from filter.

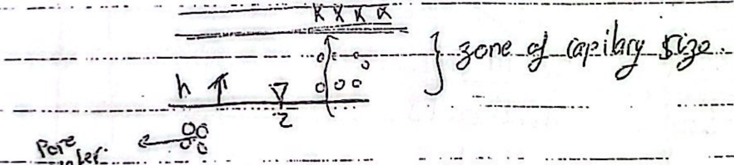
2) $\frac{D_{15}(F)}{D_{15}(B)} > 4 \Rightarrow D_{15}(F) > 4 * D_{15}(B)$:



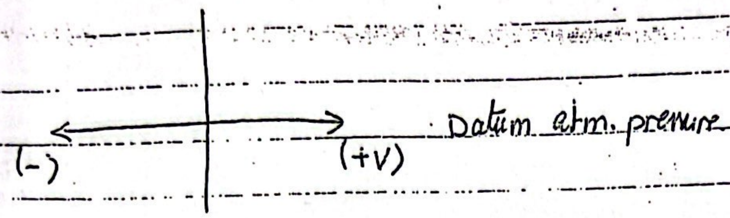
عن نقطة التقاطع حسبة المرشحة والصلبة Base mat.

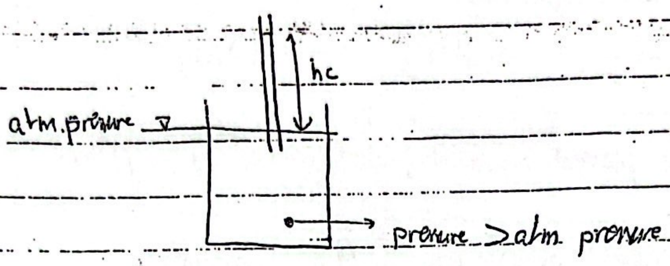
filter mat. grain size dist curve must all lie in the shaded region.

* Capillary rise in soil



finer soil ≡ higher rise.

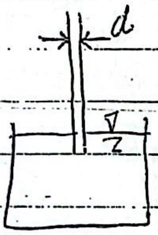




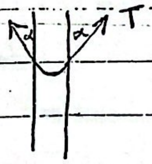
pore water reduces effective stress.

water in cappilary rise layer increases effective stress.

Sign convention: below water level : +ve pressure
above " " : -ve "



$W_w = \frac{\pi}{4} * d^2 * h_c \gamma_w .$



Surface tension.

$\Rightarrow \frac{\pi}{4} * d^2 * h_c \gamma_w = (T \cos\alpha) * \pi d$ [illegible]

$\Rightarrow \boxed{h_c = \frac{4 T \cos\alpha}{d * \gamma_w}}$ $\therefore h_c \propto \frac{1}{d}$.

Soil type	h_c in [m]
Coarse sand	0.12-0.18
fine sand.	0.3-1.2
silt	0.76-7.6
clay	7.6-23.

* Pore water pressure at a point in a layer of soil fully saturated by cappillary rize = $-h\gamma_w$. [S = 100% due to cappillary rize]

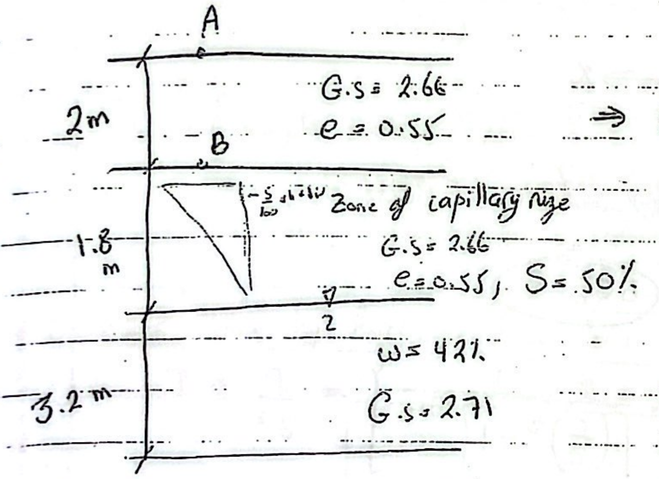
~~xxxxxxxxxx~~ γ_s ? → use γ bulk.

S = 100% ⇒ use γ saturated <u>not</u> γ submerged.

* For any degree of saturation.

$$\Rightarrow u = -\left(\frac{S}{100}\right) * h * \gamma_w.$$

Example # — :-

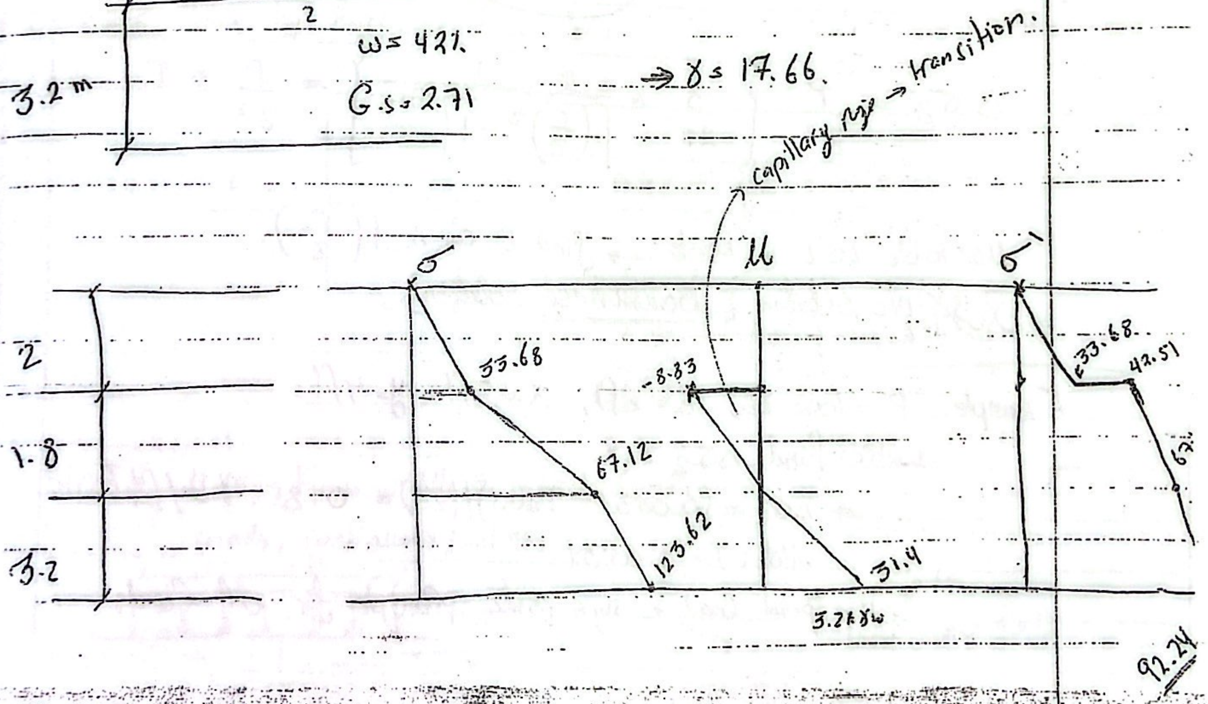


Calculated

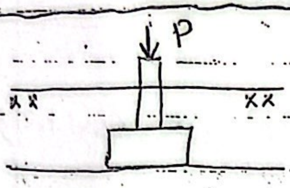
⇒ γ = 16.84 kN/m³

⇒ γ = 18.57

⇒ γ = 17.66.



* Chapter 10: stress in a soil mass.



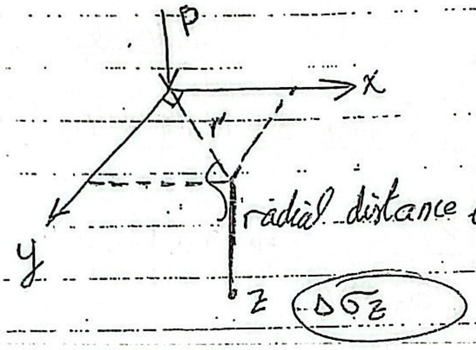
① . $\Delta\sigma_1 = ?$

② . $\Delta\sigma_2 = ?$

↓ increase in stress decreases

settlement → sho
with
l

* Case #1: Stress due to a point load; Concentrated :-



radial distance at depth z.

$$\Delta\sigma_z = \frac{P}{z^2}\left\{\frac{3}{2\pi} * \frac{1}{\left[\left(\frac{r}{z}\right)^2+1\right]^{5/2}}\right\} = \frac{P}{z^2} * I_1$$

Use table 10.1 in book → find I as a f($\frac{r}{z}$).

Gozanic solution [Boussinesq Solution].

Example: P = 1000 Lb, z = 2ft, x = 3ft, y = 4ft.

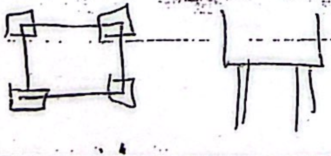
wated: find $\Delta\sigma_z = ?$

$\Rightarrow \Delta\sigma_z = (0.0034 * 1000)/(2^2) = 0.85 \; Lb/ft^2$.

Note: I = 0.0034

other point load → super position principle for all loads.

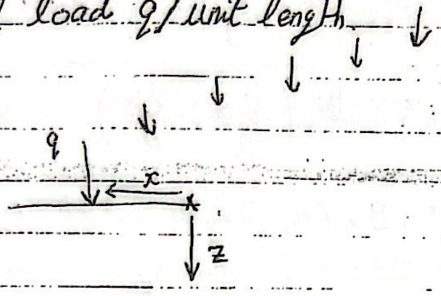
* Example: elevated water tanks



* Case #2: line load → vertical component.

Load of an infinite length.

intensity of load q/unit length



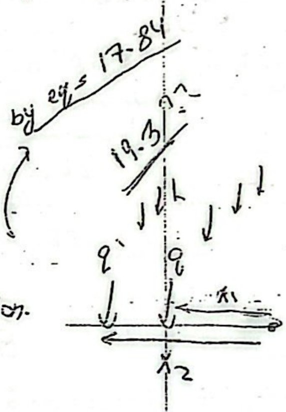
$$\Delta\sigma_z = \frac{2q * z^3}{\pi(x^2+z^2)^2}$$

Variation of $\frac{\Delta\sigma_z}{(q/z)}$ $f\left(\frac{x}{z}\right)$ table 10.2

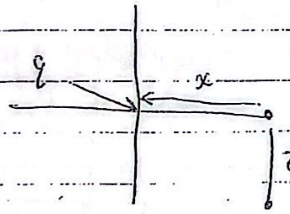
① ⑦ + Example: q = 1800 lb/ft, x = 5 ft, z = 3 ft.

⇒ from table or eq → σ_z = 17.4 lb/ft².

+ Note: two line loads → by principle of superposition.



* Case #3: Cause by horizontal line load.



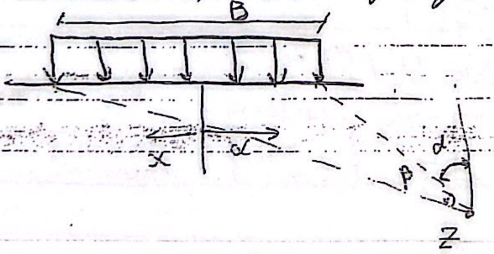
$$\Delta\sigma_z = \frac{2q}{\pi} * \frac{x z^2}{(x^2+z^2)^2}$$

value of

$\frac{\Delta\sigma_z}{(q/z)}$ $f\left(\frac{x}{z}\right)$ → table 10.3

* Strip load (q): Finite width & infinite length:

roads, continuous footings



ضغط ع ع

⇒ load نزل دآ خاز

* measured from the centre

$$\Delta\sigma_z = \frac{q}{\pi}\left[\left(\beta + \sin\beta \cos(\beta + 2\delta)\right)\right]$$

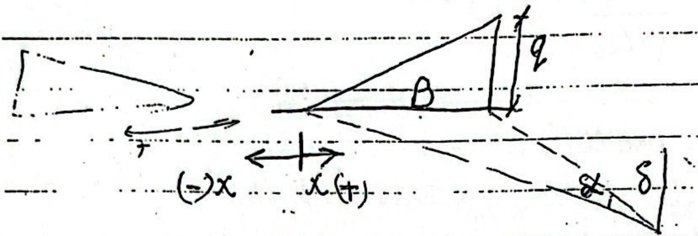
Variation of $\left(\frac{\Delta\sigma_z}{q}\right)$ with $\left(\frac{2z}{B}, \frac{2x}{B}\right)$.

* Example: $q = 200\ kN/m^2$, $B = 6m$, $z = 3m$, Det $\frac{\Delta\sigma_z}{q}$ @ $x = \pm 9$

$\Rightarrow \Delta\sigma_z = 3.42$

---

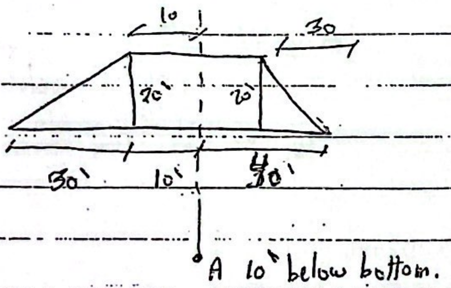
* Case #5: linearly increasing load
Finite width & infinite length

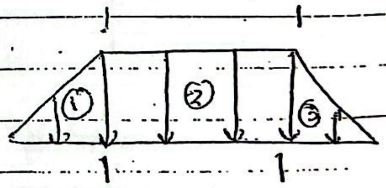


$$\Delta\sigma_z = \frac{q}{\pi}\left(\frac{x}{B}\alpha - \frac{\sin 2\delta}{2}\right)$$

Variation of $\left(\frac{\Delta\sigma_z}{q}\right)$ for $\frac{2x}{B}$, $\frac{2z}{B}$ → in table 7.4

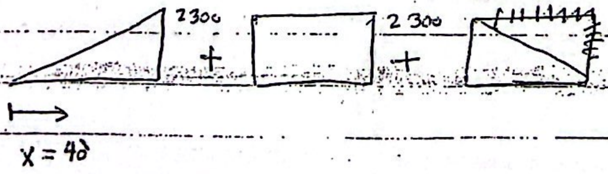
* Example: earth embankment of height 20ft and unit weight of
soil = 115 Lb/ft³.





$\Delta\sigma_A = \Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3$

$q = \gamma * H = 115 * 20 = 2300\ Lb/ft^2$



$\frac{2x}{B} = \frac{2*40}{30} = 2.67$

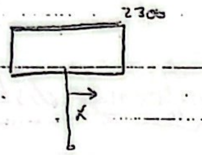
$\frac{2z}{B} = \frac{2*10}{30} = 0.67$

for 1 & 3

[illegible] ??

$\Rightarrow \frac{\Delta\sigma_z}{q} = 0.18 \qquad \Rightarrow \Delta\sigma_z = 0.18 * 2300 = 414\ lb/ft^2.$

for 2:



$\frac{2x}{B} = 0, \quad \frac{2z}{B} = \frac{2*10}{20} = 1$

1882.89

from table $\Rightarrow \frac{\Delta\sigma_z}{q} = 0.818 \rightarrow \Delta\sigma_z = 0.818 * 2300 = 1881.4\ lb/ft^2$

$\Rightarrow \Delta\sigma_{z\,total} = 1881.4 + 414 * 2 = 2709.4$

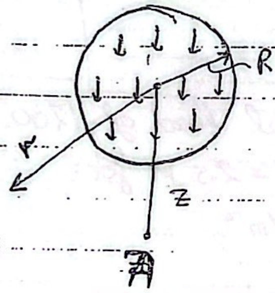
	1	1.5	2
1	0.927		0.859
1.67	0.751		0.783
2	0.653		0.735

$\rightarrow 0.3988 \rightarrow 0.17671$

1 0.030867

	0.5	1
2	0.4220	0.3524
3	0.0152	0.0622

6) $\Delta\sigma_z$ below the centre of a uniformly loaded circular area.



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

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

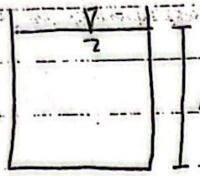
under the centre

$\frac{\Delta\sigma_z}{q} = f\left(\frac{R}{z}\right) \rightarrow \frac{z}{R}$ in table 10.5

لأي نقطة خارج ال centre

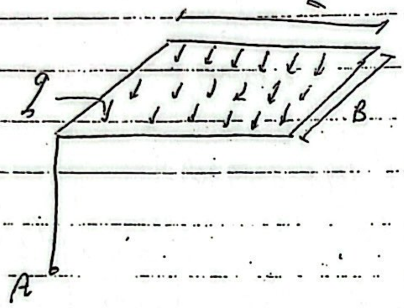
$\Delta\sigma_z = q\,(A' + B')$

where $A', B' = f\left(\frac{z}{R}, \frac{r}{R}\right)$ in table 10.6, 10.7



$10\gamma_w = q$

7) $\Delta\sigma_z$ due to a rectangular loaded area:-



1) uniformly distribu

kN/m^2.

2) point should be at a

of the loaded area.

a) if not at a corner:

نكسر المساحات بحيث تقع على الزاوية

b) المستطيل: نمد نغطي المنطقة

ثم نطرح الزيادة

$\Delta\sigma_z = q\, I_3$

$I_3 = f(m, n)$

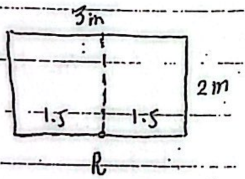
$m = \frac{B}{z}$, $n = \frac{L}{z}$

table 10.8.

* **Example:** A rectangular loaded area with a total load of 1200 kN, B = 2m, L = 3m, Calculate $\Delta\sigma_z$ at $z = 2.5$ m for:

$q = 1200/Area = 1200/6 = 200\ kN/m^2$

1)



$m = \frac{1.5}{2.5} = 0.6$

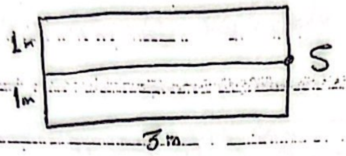
$n = \frac{2}{2.5} = 0.8$

table ⇒ $I_3 =$

$\Rightarrow \sigma_{\Delta R} = [200 * 0.1247] * 2 = 49.9\ kN/m^2$

2) next page ⇒

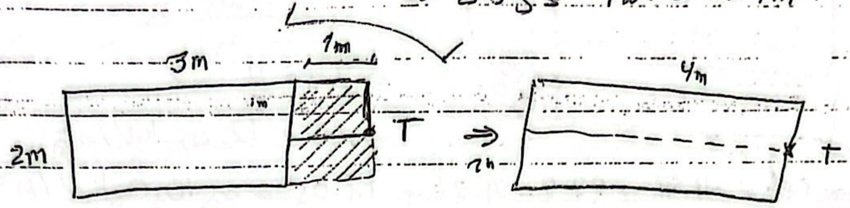
2)



$m = \frac{1}{2.5}$, $n = \frac{3}{2.5}$ $\Big\} \Rightarrow I_3 = \#$

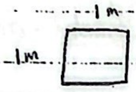
$\Rightarrow \Delta\sigma_S = 42.5\ kN/m^2.$

3)



$\Rightarrow m = \frac{1}{2.5} = 0.4$, $n = \frac{4}{2.5} = 1.6$ $\Rightarrow I_3 = 0.1114.$

$\Rightarrow \Delta\sigma_T = 200 * 2 * 0.1114 = 44.54\ kN/m^2$: (44.56).



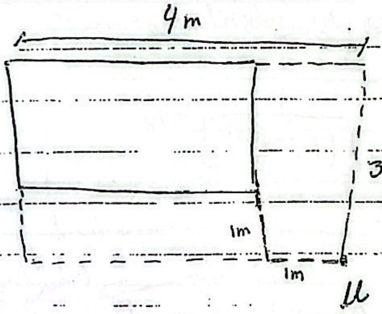
$m = n = \frac{1}{2.5} = 0.4 \Rightarrow I_3 = 0.06$

$\Rightarrow \Delta\sigma_T = 2 * \left[200 * 0.06\right] = 24.1\ kN/m^2.$

$\Rightarrow$

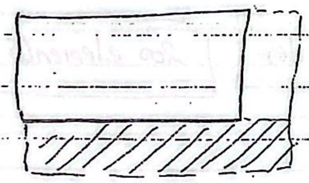
Net $\Delta\sigma_T = 44.54 - 24.1 = 20.44\ kN/m^2.$

4)



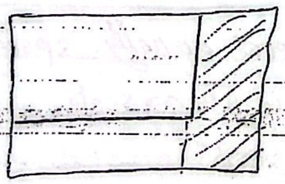
$m = \frac{3}{2.5} = \#$, $n = \frac{4}{2.5} = \#$ $\Big\} I_3 = 0.20731.$

$\Rightarrow \Delta\sigma = 200 * 0.20731 = 41.46\ kN/m^2$



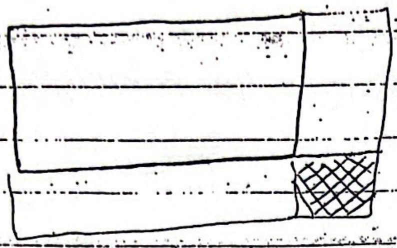
$m = \frac{1}{2.5}$, $n = \frac{4}{2.5} \Rightarrow I_3 = 0.111$

$\Delta\sigma = 200 * 0.111 = 22.2\ kN/m^2.$



$m = \frac{1}{2.5}$, $n = \frac{3}{2.5} \Rightarrow I_3 = 0.106$

$\Rightarrow \Delta\sigma = 0.106 * 200 = 21.26\ kN/m^2.$



$m = n = \frac{1}{2.5} = 0.4$

$\Rightarrow I_3 = H$

$\Rightarrow \Delta\sigma = 12.05\ kN/m^2$

$\Rightarrow net = 41.46 - 22.2 - 21.26 + 12.05 = \approx 10.0\ kN/m^2$

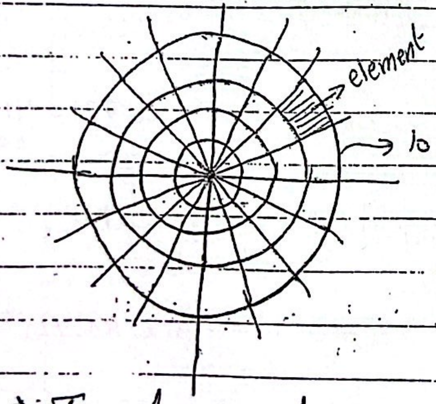
* **Case #8 : $\sigma_{\Delta z}$ using influence chart (Newmark's chart) :-**

* this chart can be used to determine the ~~vertical~~ increase In vertical stress at any point below a uniformly loaded area shape.

* The chart is constructed by drawing concentric circles, the ~~ratio~~ radius of circle are equal to $\frac{R}{z}$ value corresponding to $\frac{\Delta\sigma}{q} = 0, 0.1, \ldots$

for $\frac{R}{z} = 0$ for $\frac{\Delta\sigma}{q} = 0 \rightarrow$ by substitution

using relation : $\frac{\Delta\sigma}{q} = 1 - \frac{1}{\left[\left(1+\left(\frac{R}{z}\right)^2\right)\right]^{3/2}}$



→ 10 circles, 9 visible on graph.

Nine circles

200 elements ≡ 1 unit

* To get $\Delta\sigma_z$ at any point :

- nine circles are devided by several equally spaced radial

unit length for plotting the circles is $\overline{AB} \rightarrow$ scale

Influence value = $\frac{1}{N} = \frac{1}{200} = 0.005$

~~200 etc.~~

Sol

H.W: 7, 9, 11, 13, 17,
18.

Example: triangle, uniformly loaded.

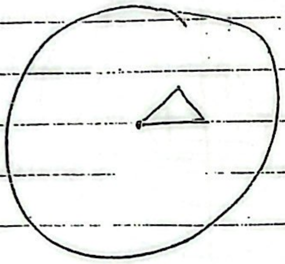
$\overline{AB} = 2$ cm.

$\overline{AB} = Z$

let $Z = 10$ m. $\Rightarrow$ 2cm $\rightarrow$ 10 m = 1000 cm.

$\Rightarrow$ 1 cm $\Rightarrow$ 500 cm

Point A $\rightarrow$ origion, concentric point.



$1 + 0.7 + 0.1 + 0.5 + 0.7 = \underline{\underline{M}}$.

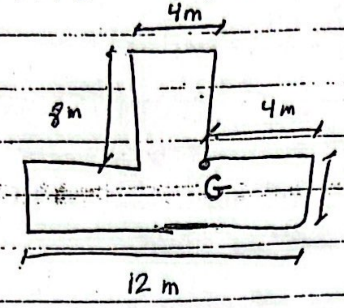
$$\Rightarrow \boxed{\Delta\sigma_z = M * \underbrace{0.005}_{\text{influence value}} * q}$$

$$\boxed{\Delta\sigma_z = q}$$ يغطي كل الشكل

بشغل بنفس المقدار

~~Second hour~~

* Example: a T-shaped footing carrying a load of 5600 kN.

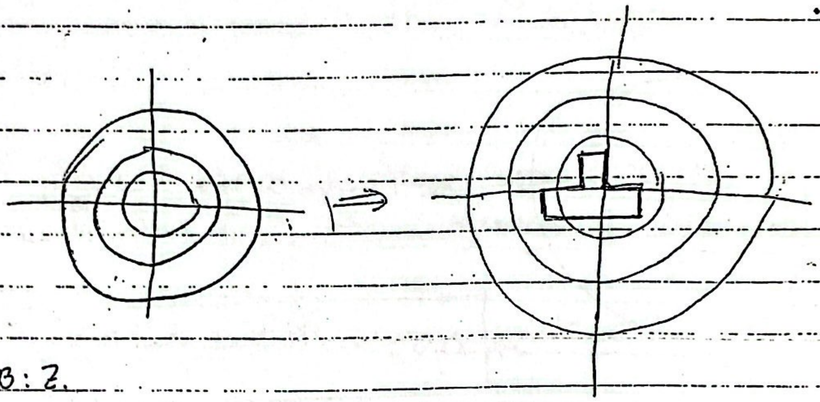


48
32
80

area = 80 m².

⇒ $q = 5600/80 = 70\ kN/m^2$.

wanted: $\Delta\sigma_z$ at $z = 6m$ below point G.



AB : z.

AB = 1.9 cm → 600 cm.

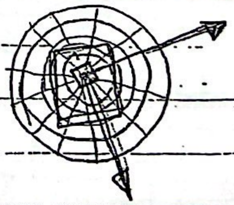
⇒ 100 cm = 1 m = (1.9/6) cm =

⇒ # of covered elements = 66.

⇒ $\Delta\sigma_z = 66 * 0.005 * q = 23.1\ kN/m^2$.

for rectangular loaded area ($q = 100\ kN/m^2$) → 34.9

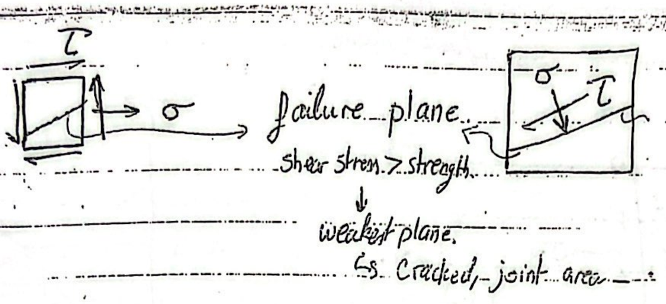
" this method " → 33



Till here → second hour exam:

Shear strength of soil:

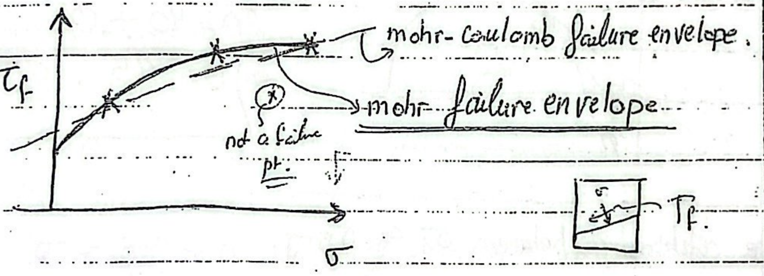
* Shear strength of soil: is the internal resistance the soil offers to resist failure and sliding along any plane inside it.
  - It is important in stability, bearing capacity, or lateral earth pressure analysis.



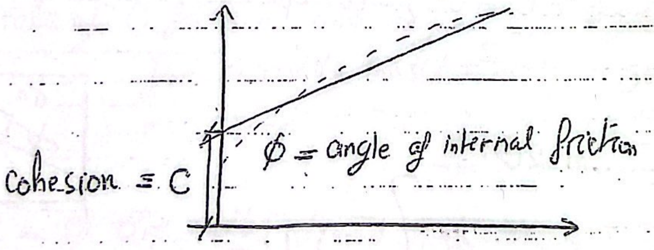
* Mohr presented a theory of failure that failure occur because of critical combination of normal stress "σ" and shear stress "τ" and not either maximum normal or shear stress alone.

$$\therefore \tau_f = f(\sigma)$$

The failure envelope is a curved line for most of the soil ~~types~~.



* The failure envelope is approximated as a linear function of normal stress.

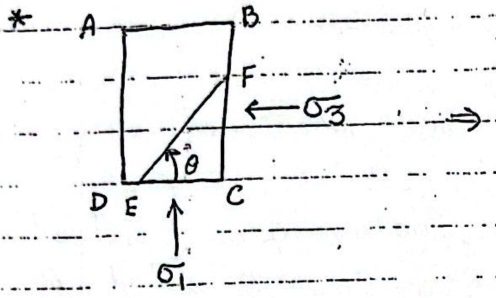


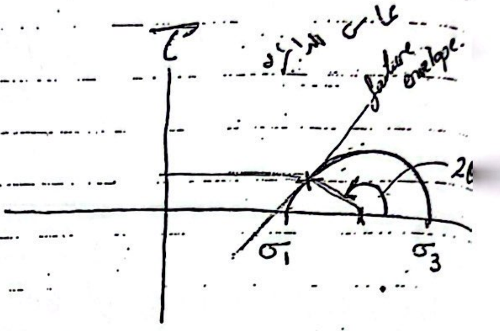
* shear strength parameters are C & ϕ.

$$\Rightarrow \tau_f = C + \underset{\tau}{\sigma} * \tan\phi . \qquad [\tau_f = f(\sigma)]$$

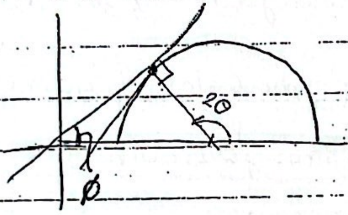
normal stress at a failure plane

*





failure envelope → tangent of the circle.



$$2\theta = 90 + \phi$$

$$\Rightarrow \boxed{\theta = 45 + \frac{\phi}{2}}$$

* The relationship between σ_1 & σ_3:

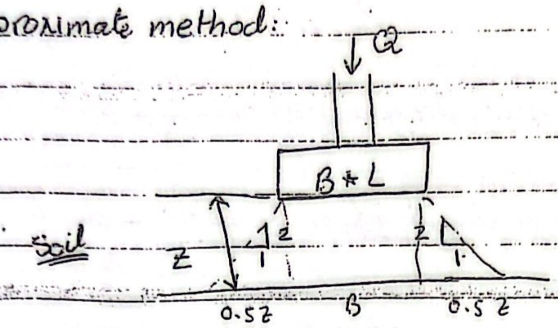
$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2C \tan\left(45 + \frac{\phi}{2}\right)$$

General
$$\begin{cases} \sigma_n = \dfrac{\sigma_1 + \sigma_3}{2} + \dfrac{(\sigma_1 - \sigma_3)}{2} \cos 2\theta \\ \tau = \dfrac{\sigma_1 - \sigma_3}{2} \sin 2\theta . \end{cases}$$

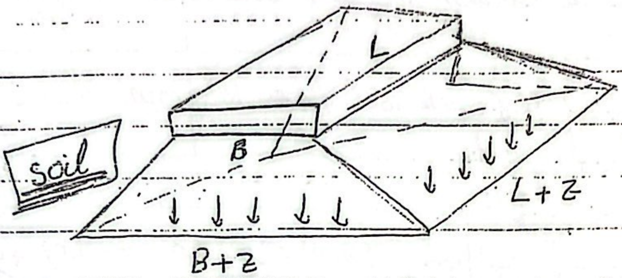
at failure, $\theta = 45° + \frac{\phi}{2}$ → متساويات

$\tau_f = C + \sigma \tan\phi$

* approximate method:



new area: $(B+Z)(L+Z)$



$$\Rightarrow \sigma_z = q / \left[(B+Z)(L+Z)\right]$$

* for Rock: [1:1]

new area: $[B+2Z]*(L+2Z)$ lower stren than soil at depth Z

strip footing: $B' = B+0.15Z$.

$\tau_f = C + \sigma \tan\emptyset$

Saturated Soil:

$\sigma = \underline{\sigma'} + u$ only effective is used.

$\Rightarrow \tau_f = C + \sigma' * \tan\emptyset \rightarrow$ shear strength is reduced

* Value of C for sand and inorganic silt = zero.

for normelly consolidated clay ≃ zero.

" over " " = $\underline{C}$

* C & Ø are called drained shear strength parameter.

dry or if water is present it is drained.

↳ long term analysis.

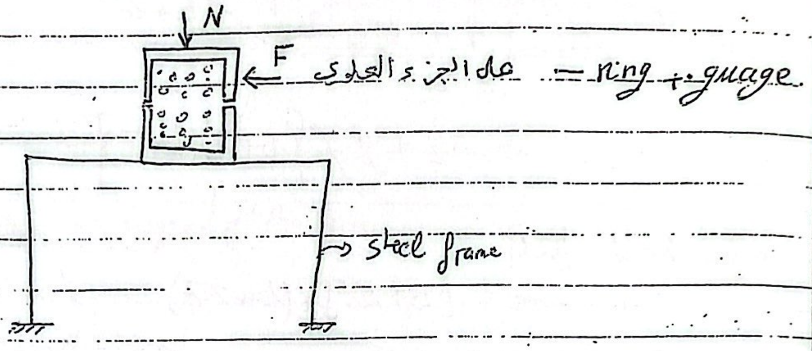
* C_u and ϕ_u are called undrained shear strength parameters → short term

+ shear strength parameter for drained or dry Sand is the same → due to the high permeab

* Shear strength parameters are determined in the lab.

1) direct shear test:-

oldest test, used for all soil types but most accurate result are for sand.



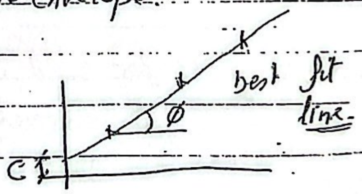
Time	Normal force	Shear force	horizontal displacement

failure: تثبت القراءة أو تنزل

σ_{N_1} & τ_{f_1} → a point on the failure envelope.

new value for N: σ_{N_2} & τ_{f_2} then σ_{N_3} & τ_3.

Now draw failure envelope:



Note: same horizental & vertical scale

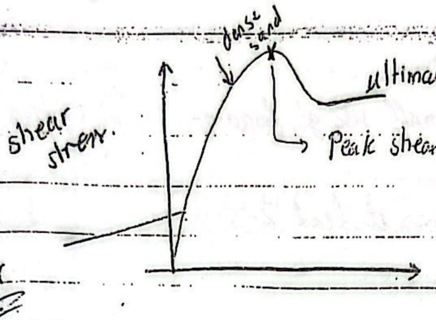
Defects: 1) failure plane حددناها in the middle which is wrong

2) Shear stress is assume uniformly dist هو أكبر في منطقة تأثيره

3) area is decreased as the box is moved.

- Two types of direct shear test :
  1) stress controlled. 2) Strain controlled: better method.

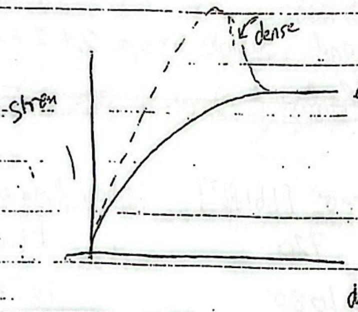
shear force added as equal increments & strain is measured.
∴ failure between two increments → approximation.



ultimate shear strength. [dense sand → loose sand].

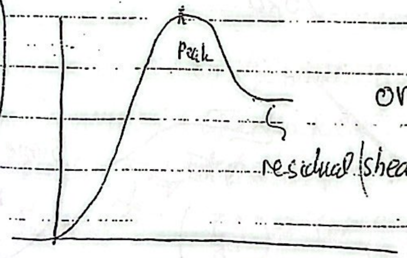
→ Peak shear strength for dense sand

linear



ultimate shear strength

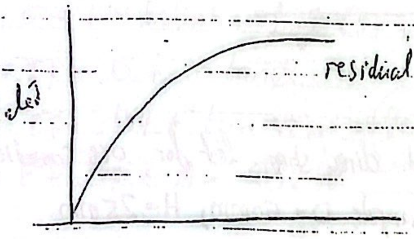
over: رحنا مرحلة stress أعلى من الأن وهي أقوى من الـ normal



over consolidated clay

residual (shear) strength. المتبقي

Normally consolidated



residual shear strength

≫

* For sand:

$\tau_f = \overset{0}{\cancel{C}} + \sigma \tan\phi = \sigma' \tan\phi$ [same result for sand $\sigma =$

$\phi_u = \phi$

* For clay:

Drained test on clay ⇒ very small rate of loading e.g. $\overset{2}{5}$ days for test

$\tau_f = C + \sigma \tan\phi$

5 $\tau_f = C + \sigma' \tan\phi$ ⇝ at least 2-5 days

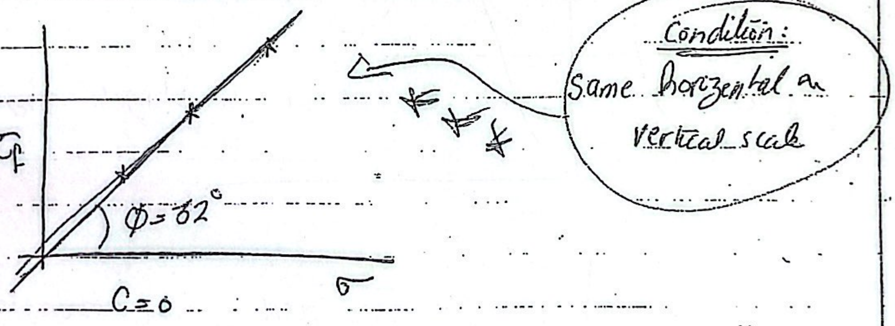
* Examples on direct shear test:

A. Direct shear test on ~~direct~~ dry sand. Sample size = 2 * 2 * 0.75 in

~~Test # Normal force (Lb) $\sigma = \sigma'$ [Lb/ft²]~~

Test #	Normal force [Lb]	$\sigma = \sigma'$ [Lb/ft²]	shear force at failure [lb]	τ_f
1	20	720	12.0	432
2	30	1080	18.3	658.
3	70	2520	42.1	1515.6

12*36
360
60
12
360+72
432

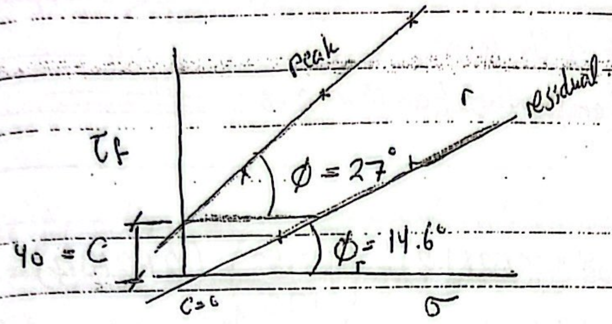


Condition:
Same horizontal and vertical scale

* Example #2: Results of drained direct shear test for over consolidated clay.
Sample, circular sample. D = 50mm, H = 25mm.

Test #	Normal force "N"	σ' [kN/m²]	peak shear force "N"	τ_f	Residual τ

Test #	Normal force 'N'	σ [kN/m²]	peak shear force N	τf at peak	Resid S.F.	τr at Resi
1	150	76.4	157.5	80.2	44.2	22.5
2	250	127.3	199.9	101.8	56.6	28.8
3	350	178.3	257.6	131.2	102.9	52.~~4~~



at peak

$\Rightarrow$ equation : $\tau_f = C + \sigma \tan\phi = 40 + \sigma * \tan 27°$.

$\tau_r = 0 + \sigma \tan 14.6 = \sigma * \tan 14.1$

*imp. // * Resided stren → transforming sample to normally consolidated.

ϕ (residual) is lower than peak ϕ due to sliding of particles over eachother

residual internal angle of friction

~~peak~~ internal angle of friction

* Example # 3: direct shear test on dry sand. Sample size = (50*8)*(50.8)*24.4 m

$\sigma = 192\ kN/m^2$, $\tau_f = 120\ kN/m^2$. find $\phi = ?$

* $\tau_f = C + \sigma * \tan\phi = 0 + \sigma * \tan\phi$

$\Rightarrow \phi = \tan^{-1}\left(\frac{\tau_f}{\sigma}\right) = \tan^{-1}\left(\frac{120}{192}\right) = 32°$.

- for $\sigma = 144\ kN/m^2$, find $\tau_f = ?$ and shear force at failure.

$\tau_f = 0 + \sigma \tan\phi$

$\Rightarrow \tau_f = 144 * \tan 32 = \boxed{90}$?

$\Rightarrow F = \tau_f * A = 232\ N$.

* Example #4: direct shear test; dry; sand; Sample size = 2 * 2 * 1.2 in.

$\tan\phi = \frac{0.6}{e}$, $G.s = 2.66$, $\sigma = 20\ Lb/in^2$, $\tau_f = 16\ Lb/in^2$

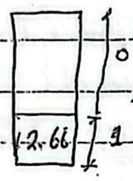
what is the weight of the sample.

$\tau_f = C + \sigma \tan\phi \Longrightarrow \tan\phi = 0.8$

$\Rightarrow 0.8 = \frac{0.6}{e}$

$\Rightarrow e = 0.75$

$* w = \gamma * V = \gamma * (2*2*1.2) = \left(\frac{165.98}{1.75}\right)(2*2*1.2) = 0.2$

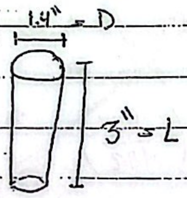


$\Rightarrow w_s = 165.98$

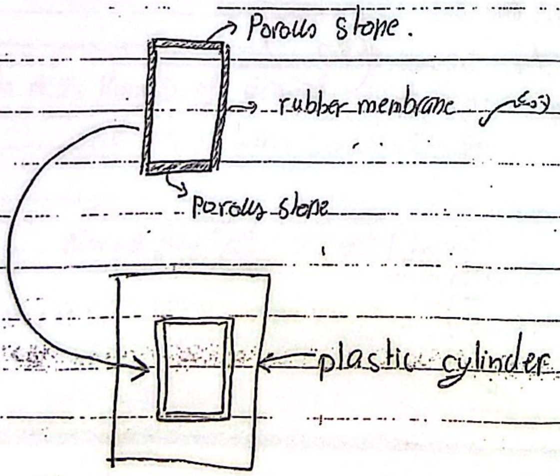
$\gamma = \frac{165.98}{1.75} = 94.85$

H.w. = 1, 3, 6, 8, 14, 19

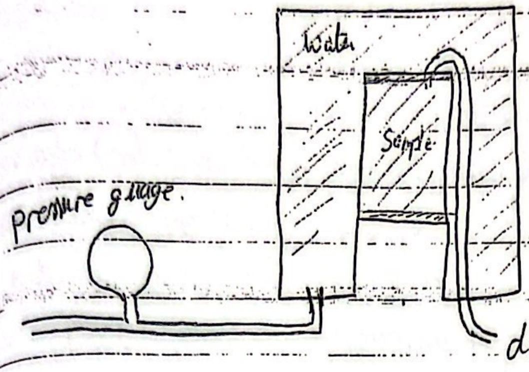
* Triaxial shear test: for cohesive soil [clay] specifically.



C & ϕ parameters.



لحماية العينة من ضغط الماء الجانبي من تكسر العينة.



* saturated sample

drainage ≡ drainage valve . هاد داخل العينة.

pe for suction of air a entry of water at low speed to ... : أنبوب
saturate the sample [entry of water only through porous stone].
Confining pressure (chamber pressure), σ_3 ضغط العينة / الماء في الوعاء ديجي الضغط

Step 3: Either one of the following tests:

1) Consolidated drain test "C.D."
2) Consolidated undrained test "C.U."
3) Unconsolidated undrained test "U.U." — also called "quick test"

* Case # 1: C.D test : مستعمل في الابحاث نتيجة للوقت الطويل المأخوذ

- In this test, the saturated sample is first subjected to all around confining pressure (σ_3) by compression of water → σ_3 ≡ confining / chamber pressure.

⇒ pore water pressure increases in the same amount "u_c"

ارتفاع في ضغط الفراغات بين الحبيبات نتيجة الضغط المحاز

$$B = \frac{u_c}{\sigma_3} \equiv \text{Skimpton's pore-pressure coefficient}$$

$$B = \approx 1 \text{ for saturated sample.}$$

* decrease in sample size ≡ Volume of drain water

⇒ consolidation has occured to the soil sample

F ينضغط العينة تحت تأثير الوزن المضاف ينقص حجمها بنفس مقدار (التقلص) حجم الماء الخارج

at least 3 samples ⇒ required

now: gauge reading returned to zero. $\nearrow \sigma d$

start increasing axial load (axial load deviator) at a very slow rate $\underset{\sigma_D d}{}$ ويمكن حدوث ال Drainage :

keep the valve open ⇐ apply axial at a very slow rate ⇐

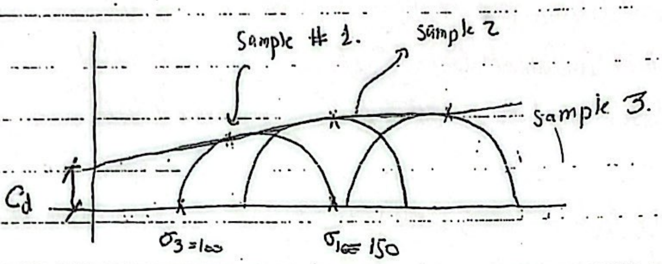
→ u_d = zero [increase of pore water pressure = zero]

→ $\sigma = \sigma'$

→ Drained parameters are calculated from this test.

$$\sigma_1 = \sigma_3 + \Delta\sigma d \quad [\text{major stress}]$$

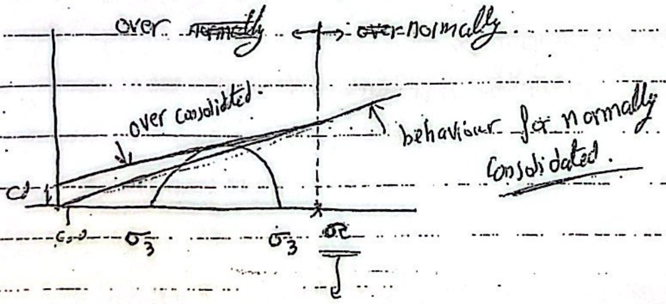
mohr circle for the sample:



failure envelope [tangent to the three circle] → find C_d an ϕ_d.

* for over consolidated soil [to over-consolidate the sample]:-
- Confine sample to pressure = 100
- decrease pressure to 50 ⇒ artificial over consolidation.
- Complete the test.

→ mohr circles for these samples:



الحد الفاصل بين ال normal وال over

if pressure > σc ⇒ normally consolidated.

Equation for over: $\tau_f = C + \sigma' \tan\phi_1$

" " normal: $\tau_f = \sigma' \tan\phi_2$

Example: Consolidated-drained triaxial test on normally Consolidated clay

$\sigma_3 = 276 \ kN/m^2$.

$(\Delta\sigma_d)_f = 276 \ kN/m^2$.

1) find ϕ', θ, σ', τ_f on the failure plane.

* $\sigma_3 = \sigma_3' = 276 \ kN/m^2$

* $\sigma_1 = \sigma_1' = \sigma_3 + (\Delta\sigma_d)_f = 276 + 276 = 552 \ kN/m^2$.

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + \cancelto{\text{zero}}{2C \tan(45 + \phi/2)}$$

$\Rightarrow \phi = 19.5°$

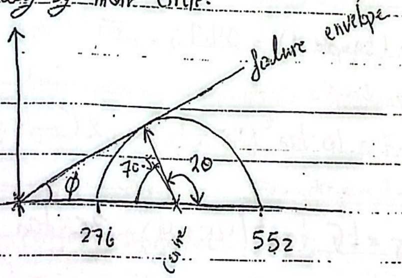
$$\theta = 45 + \frac{\phi}{2} = 45 + \frac{19.5}{2} = 54.75°$$

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta \quad \Rightarrow \sigma = 368 \ kN/m^2$$

① $\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta \quad \Rightarrow \tau = 130 \ kN/m^2$ same

② or at failure: $\tau = C + \sigma \tan\phi = 368 \tan 19.5° = 130 \ kN/m^2$

③ or graphically by mohr circle:



C=0 → draw on graph

$$R = \frac{552 - 276}{2} = 198 \ [138]$$

$$\text{Distance from origion} = \frac{\sigma_1 + \sigma_3}{2} = 414$$

367.347

$\sin\phi = \frac{\text{Radius}}{\text{الوتر}} = \frac{138}{414} = \# \Rightarrow \phi = 19.5°$

$\theta = 45 + \phi/2 \Rightarrow \theta = 54.7 \Rightarrow 2\theta = 109.5°$

$\Rightarrow \frac{\tau_f}{R} = \sin 70.5 \Rightarrow \tau_f = 130 \text{ kN/m}^2$

$\frac{x}{R} = \cos 70.5 \Rightarrow x = \#$

$\Rightarrow \sigma = 414 - x = 368 \text{ kN/m}^2$

drained Parameters

Example #2: Equation of effective stress failure for normally consolidated clay is

$\tau_f = \sigma' \tan 30°$

$\sigma_3 = 10 \text{ Lb/in}^2$, find $(\Delta\sigma d)_f = ??$

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + \cancelto{0}{C}$$

$$\Rightarrow \sigma_1 = 10 \tan^2\left(45 + \frac{30}{2}\right) = 30 \text{ Lb/in}^2$$

$$(\Delta\sigma d)_f = \sigma_1 - \sigma_3 = 30 - 10 = 20 \text{ Lb/in}^2$$; $\tau_f = ??$

Example #3: Result of drained triaxial test on sat. clay ??

Sample 1: $\sigma_3 = 10 \text{ Lb/in}^2$ $(\Delta\sigma d)_f = 24.7$ Lb/in^2

" 2: $\sigma_3 = 15$ " $(\Delta\sigma d)_f = 33.5$ "

$\Rightarrow \sigma_1 (\text{sample } 1) = 34.7$, $\sigma_1 (\text{sample } 2) = 48.5$

from equation

$$34.7 = 10 \tan^2\left(45 + \frac{\phi}{2}\right) + 2C \tan\left(45 + \frac{\phi}{2}\right) \text{ ①}$$

②

$$48.5 = 15 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$\phi = 27.91$
$C = 2.13$

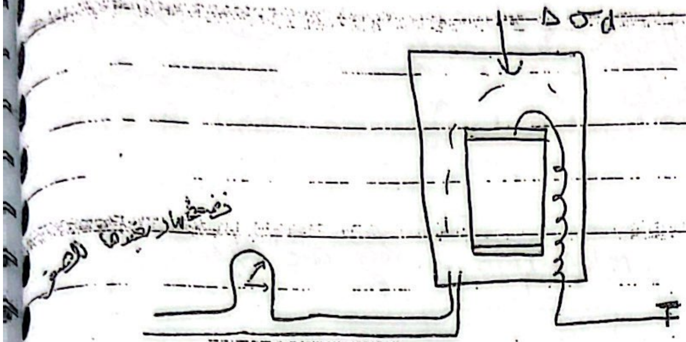
$\Rightarrow \phi = 28°$

$\Rightarrow C = 2.1 \text{ Lb/in}^2$

∴ The clay is over consolidated due to the presence of C value

→ more popular:

$\boxed{2}$ C-U test : Consolidated-undrained test.



1) consolidation → by pressure
2) valve is closed.
3) add axial load ($\Delta\sigma_d$)
4) at time of failure read $\Delta\sigma_{df}$ & Δu_{df}

. الزيادة في ضغط الماء بسبب الـ deviator stress

$* \; B = \frac{u_c}{\sigma_3}$

$$\bar{A} = \left[\frac{\Delta u_d}{\Delta\sigma_d} \right].$$

for normally consolidated clay , $\bar{A} = 0.5 - 1.0$
" over " " , $\bar{A} = \underline{-0.5} - 0.0$
↳ expansion for the test sample
يكبر حجم العينة ← "dilatancy".

Two types of stress equations can be written:

$\sigma_1 = \sigma_3 + \Delta\sigma_d$

$\sigma_1' = \sigma_1 - \Delta u_d$; نطرح الزيادة في ضغط pore-water pres

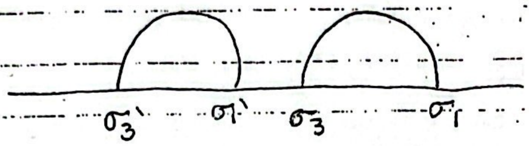
σ_3 = confining pressure

$\sigma_3' = \sigma_3 - \Delta u_{df}$

∴ by this test, we can find both the drained and the undrained paramet
(C_u & ϕ_u) undrained) & (C, ϕ) - drained

* for drained parameters → use effective stress analysis. معطا سيون الفضل
* undrained parameters → use total " " طريقة ما بحدث الرأي

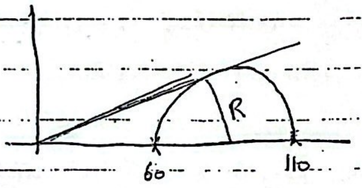
* Examples:
$\sigma_1 - \sigma_3$ & $\sigma_1' - \sigma_3'$ relationship → they are equal.



same circle diameters.

* Example: A sample of saturated sand was consolidated under $\sigma_3 = 60 Lb/in^2$
axial stress increased & drainage was prevented:
At failure: $\Delta\sigma_d = 50 Lb/in^2$
$\Delta U_{df} = 41.35 Lb/in^2$

1) find: $\phi_u = ??$
$\sigma_3 = 60$
$\sigma_1 = \sigma_3 + 50 = 110.$



or: $\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi_u}{2})$.
$\Rightarrow \phi_u = 17.1°$

2) find $\phi_u = ??$ drained angle = ?
$\Rightarrow \sigma_1' = \sigma_1 - 41.35 = 110 - 41.35 = 68.65$
$\Rightarrow \sigma_1 = (60 - 41.35) \tan^2(45 + \frac{\phi}{2})$ $\Rightarrow \phi = 34.9°$.

∴ drained angle > undrained angle.
لأن الماء بيقلل الاحتكاك وعند ازالتو بتصير الزاوية أعلى في القرار.

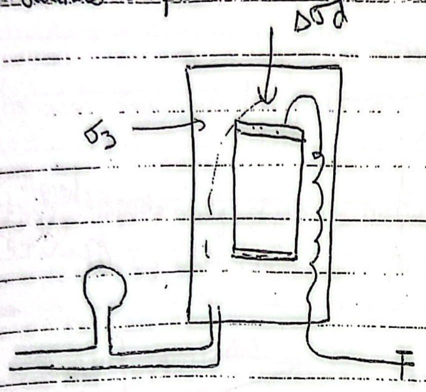
imp
3) what is $\Delta\sigma_{df}$ if C-d test is performed with $\sigma_3 = 60$?
$\sigma_1' = \sigma_3' \tan^2(45 + \phi/2) + ~~2C~~$
$\Rightarrow \sigma_1' = \sigma_3' \tan^2(45 + \frac{34.9}{2}) + 0$.

$\Rightarrow \sigma_1' = 60 * \tan^2 (45 + 34.9/2) = 220.85$ Lb/in².

$\Rightarrow \Delta\sigma d_f = \sigma_1' - \sigma_3' = 160.85$ Lb/in²

U-U test : unconfined, undrained test "<u>quick test</u>"

i) saturate sample:



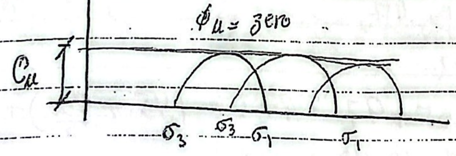
$U_c = B * \sigma_3$ الزيادة في الضغط الكلي

$U_d = \bar{A} * \Delta\sigma d$ الزيادة في الضغط الراسي

$\Rightarrow \Delta U = B\sigma_3 + \bar{A} (\sigma_1 - \sigma_3)$

∴ Total stress → undrained parameter

بغض النظر عن قيمة σ_3 المعطاة بتطلع $\Delta\sigma d$ ثابتة وبالتالي
يكون قطر الدوائر نفسه

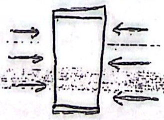


$\Rightarrow \phi_u$ = zero , C_u = radius of circle : $\frac{\sigma_1 - \sigma_3}{2}$

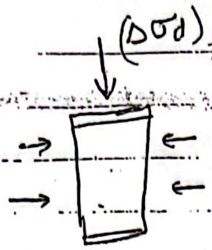
$\boxed{4}$ Unconfined - compression test on <u>saturated</u> clay.

حتى تكون التربة في أضعف حالاتها ← أسوأ حالة

- This is a special type of U-U test.

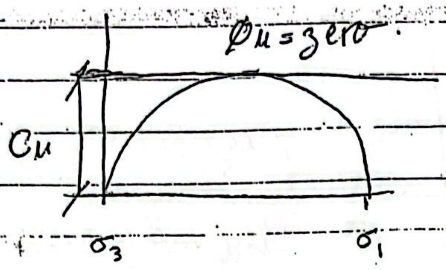


$\times \sigma_3 = 0$ → unconfined test.



$\sigma_1 = 0 + (\Delta\sigma_d)_f$

mohr circle.



shear strength < actual

$\tau_f = C + \sigma \tan\phi$ → zero

$\Rightarrow C_u = \frac{\sigma_1}{2} = \frac{q_u}{2}$

$\sigma_1 \equiv$ unconfined Compressive strength (q_u)

---

* Example: for normally consolidated clay.

$\tau_f = \sigma' * \tan 28.$ ⇒ 28 = drained angle.

CU test was performed on the sample resulting in:

$\sigma_3 = 105$

$(\Delta\sigma_d)_f = 97.$

Find: 1) $\phi_u = ?$

$\Rightarrow \sigma_1 = 105 + 97 = 97 \tan^2(45 + \frac{\phi_u}{2}) + \overset{0}{\cancel{2C}}$

$\phi_u = 18.4°$

2) What is $(\Delta u_d)_f$.

$(105 + 97) - \Delta u_d = (97 - \Delta u_d) \tan^2(45 + \frac{28}{2})$

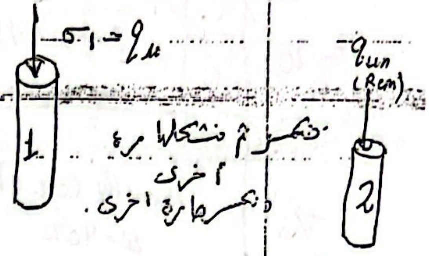
$\Rightarrow \Delta u_d = 50.2 \ kN/m^2$

$A = 50.2/97 \approx 0.5$ ⇒ normally consolidated clay.

Sensitivity & thixotrophy in clay:

+ Sensitivity: is the rate of unconfined compressive strength in undisturbed state to that in remolded state.

$$S_t = \frac{q_u(\text{undisturbed})}{q_{um}(\text{remolded})}$$

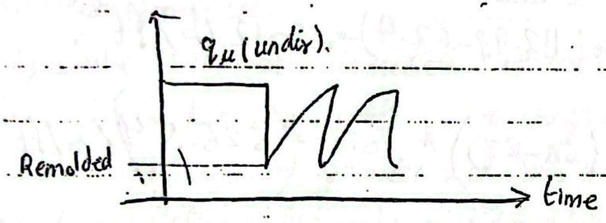


نكسر ثم نشكلها مرة أخرى

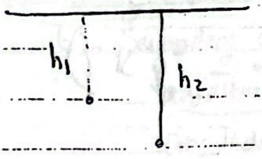
نكسرها مرة اخرى.

for most clay., Sensitivity (1 - 8).

+ Thixotrophy: the property of gaining the strength of remolded sample (no change in w%) with time.



+ Empirical Relations between (C_u) and effective overburden pressure (σ')



$$\tau = C + \underset{\text{تزداد مع العمق.}}{\underline{\underline{\sigma}}} * \tan\phi$$

1) for normally consolidated clay:

$$\frac{C_u}{\sigma'} = 0.11 + 0.0037 \text{ P.I.}$$

2) for over-consolidated clay

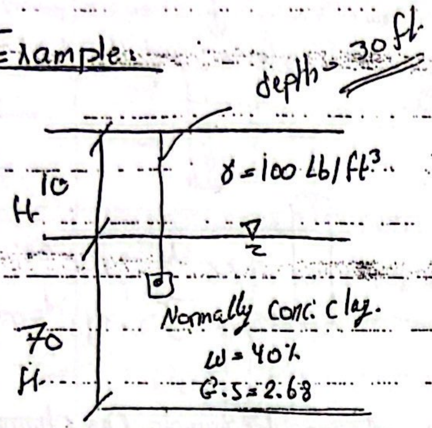
$$\left(\frac{C_u}{\sigma'}\right)_{over} \Big/ \left(\frac{C_u}{\sigma'}\right)_{normal} = (OCR)^{0.8}$$

where $OCR = \left(\frac{\sigma_c'}{\sigma'}\right)$ maximum pressure

*) C_u (design) $= \lambda C_u$.

where $\lambda = 1.7 - 0.54 * \log \text{P.I.} \leq 1$

* **Example:**



given:
$LL = 68\%$
$PL = 27\%$
Find q_u = ?? for this soil.

⇒ by calculation, $\gamma = 112.99\ Lb/ft^2$

* $\frac{C_u}{\sigma'} = 0.11 + 0.0037 * PI = 0.11 + (0.0037)(68-27).$

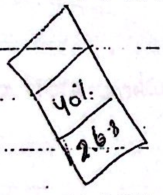
* $\sigma' = (10 * 100) + 20(112.99 - 62.4) = 2012\ Lb/ft^2.$

$\Rightarrow C_u = \Big(0.11 + (0.0037)(68-27)\Big) * 2012 = \Big(526.54\Big) Lb/ft^2$

$\lambda = 1.7 - 0.54 \log (68-27) = 0.829.$

$\Rightarrow C_u(design) = 0.829 * 526.54 = 436.55\ Lb/ft^2.$

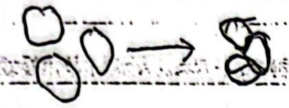
$\Rightarrow q = 2 * C_u$ (لا qu - يقف الكعليت ؟ 7).
873.11

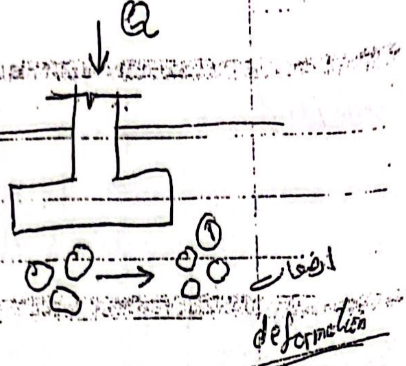


Compressibility of soil:-

1) * deformation of soil particles ◯ → o

2) relocation of soil particles





* 3) Expulsion of water or air from the voids.

Compressibility = settlement الهبوط

* Settlement forms are:
  1) elastic settlement "Immediate settlement" الها كل أنواع التربة بيصير
     - hours → weeks , no change in % due to deformation.
     - major in: granular soil such as sand.
  2) Consolidation settlement: is divided into two parts:
     a) primary consolidation settlement:
        it is a result of volume change in saturated cohesive soil because of expulsing the water from the voids (void space).
     b) Secondary consolidation: it occur as a result of plastic adjustment of soil fabrics in Cohesive soil after primary consolidation is completed.

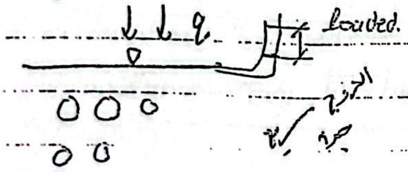
الأعلى باستثناء التربة العضوية organic soil

سنوات

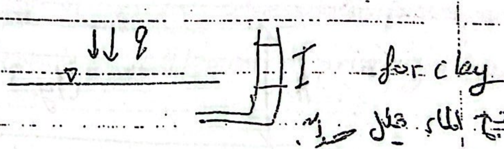
الأساسي دستعزن

S_T = elastic + primary + secondary

for sand

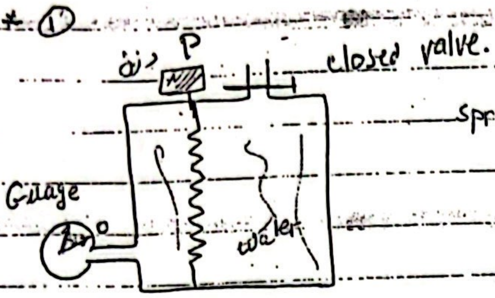


for clay:



for clay
خروج الماء قليل جداً
gradual.

* ①



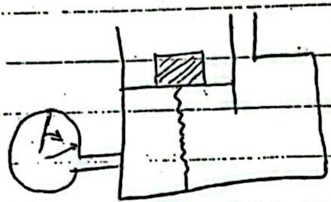
spring: represents soil skeleton.

$P_T = P_s + P_w = P_w$ (closed guage)

$\Rightarrow \Delta U = \frac{P}{A} = \frac{P_w}{A}$

② valve is opened

guage reading is reduced. the spring is compressed.



$\Delta u = P_w < \frac{P}{A}$

بعد مدة : guage → zero.

$\rightarrow \underline{\underline{\Delta u = zero.}}$

* the end of the premary consolidation state ↲

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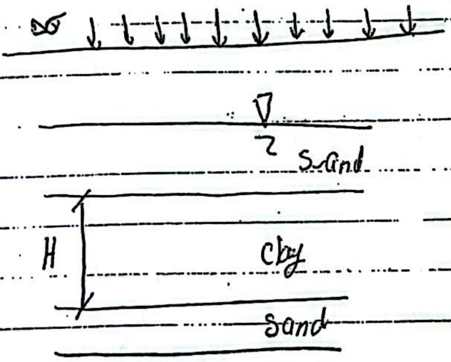
* If we have a saturated clay layer subjected to an increase in the stress $= \Delta\sigma$

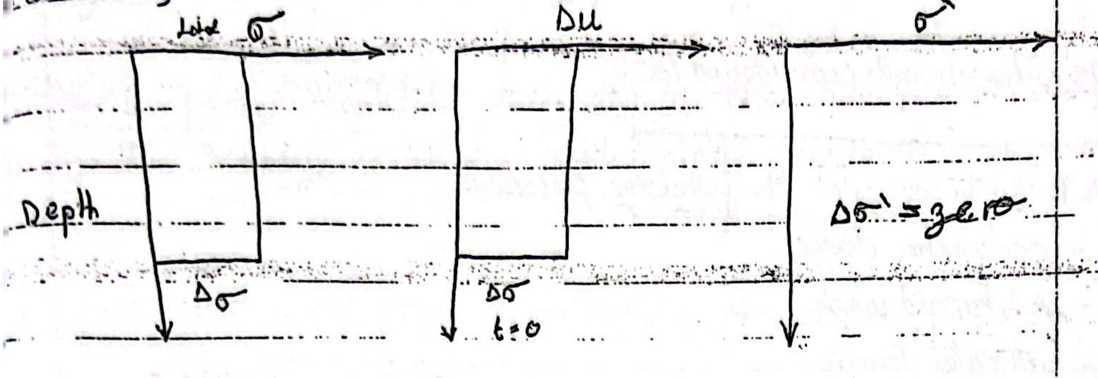


يتوزع على مساحة كبيرة
فينتقل تحت
بنفس المقدار

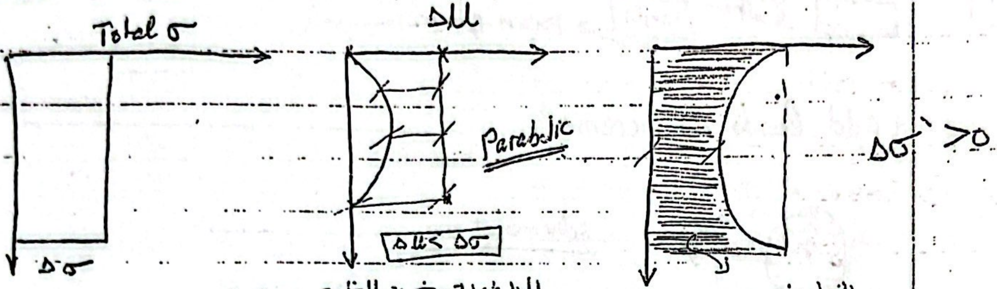
على أي عمق مقدار
الزيادة = $\Delta\sigma$.

⇒

at t=0,



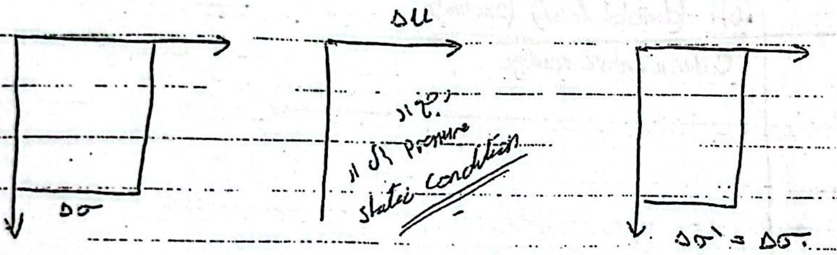
* at t = Δt



المناطق القريبة من المخارج سهلة الخروج
دائماً المنتصف بحاجة لوقت أكبر.

الزيادة في ال
Effective stress.

* at t = ∞



∴ primary consolidation under this load has ended → Δu = zero, Δσ` = Δσ

* Consolidation is defined as the gradual process of drainage under an additional load application and the associated transfer of excess pore-water pressure to the effective stress causing the time-dependent settlement (primary consolidation settlement).

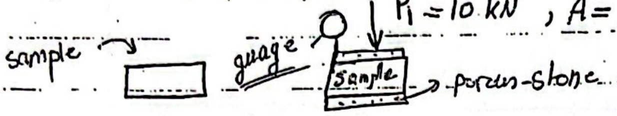
* One – dimentional consolidation test:

* Terzaghi suggested the following procedure:
- oedometer device.

1) - undisturbed sample

2) - Saturated Sample

$P_1 = 10\ kN\ ,\ A = 10\ cm^2 \Rightarrow \Delta\sigma_1 = 1\ kN/cm^2$.



+ add loads as increments.

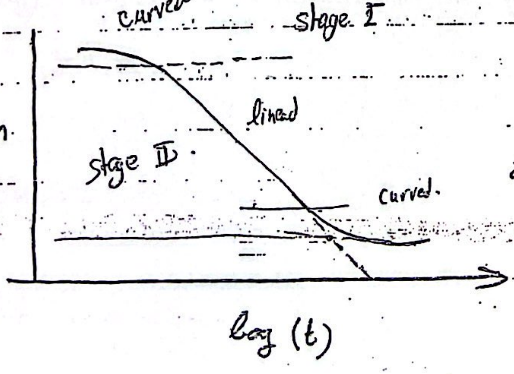
Table preparation:

$P_1 = 10\ kN \Rightarrow \sigma_1 = 1\ kN/cm^2$ static load, not dynamic loaded

time	SH	DH (doubled load) ($2kN/cm^2$)
0	0	Cumulative reading
15 sec	5	
30 sec	12	
60 sec	15	
⋮		
1 hr	40	
⋮		
24 hrs	final reading,	final reading.

يتم التسجيل حتى نصل ال Range للزيادة يعادل المطلوب في الطبيعة

1^{st} drawing:



بمد الجزء المستقيم على استقامة

Tangent for curve. ←

نقطة التقاطع : نهاية

Primary consolidation.

- Stage I: initial compression caused by pre-loading.

ترجيع العينة للوضع الذي كانت عليه في الطبيعة قبل استخراجها

- Stage II: primary consolidation during which the excess water pressure dissipated
- Stage III: Secondary consolidation starts { plastic adjustment ... تكسير الحبيبات الزاوية

  creep consolidation.