Civilittee

SOIL

Notebook

Eng.
Hussien Aldeeky

BY: Madleen Yaser

28/1/2016

(1)

Soil....

*Soil Definition :-

civil engineering define soil as any un-cemented accumulation of minerals particles formed by the weathering of rocks, the voids spaces between the particles contain water and or air.

*Weathering of Rocks :-

1- physical weathering → Sand
→ Gravel > differ in size but the same properities.

ممكن تحويل Graval الى Sand حيث كمية الحجر لا يتغير عن خواص الحجر الأصلي .

2- chemical weathering → clay
→ silt > - differ in properities
- differ from the origin rock

لا يمكن تحويل نوع لآخر ، لا يتحدد من الحجم .

*origin of clay minerals [chemical weathering كيفية تشكل]

(تحلل)

" The contact between rocks & water produces clays, either at or near the surface of earth.

يحدث تفاعل بين الماء و الصخر لاعطاء clay .

. Water + Rock → clay (طاقة كبيرة و جزيئات كبيرة)

minirals ← تختلف في تركيبة و behaviour

ex CO_2 gas can dissolve in water & from carbonic acid H_2CO_3, which will become hydrogen ions H^+ & bicarbonate ions, and make water slightly acidic.

The acid water will react with the rock surface & tend to dissolve the K^+ ions & si ions from the Feldspar.

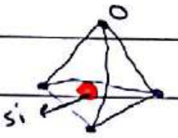
يسقط المطر الحمضي ويذوب ذرة البوتاسيوم والسليكون في معدن الفلسبار

← معدن أساسي في القشرة الأرضية . ٧٠٪ من تركيبة القشرة الأرضية

* Clay minerals توجد على شكل أزواج ثنائية (couples)، عائلة كبيرة وألوان مختلفة حسب ألوان البشر

* Basic unit of clay mineral:-

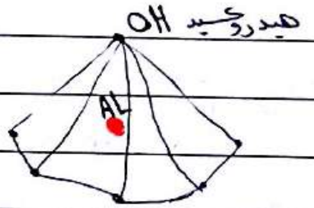
1- silica tetrahedra



ذرة سيليكون محاطة بـ ٤ ذرات أكسجين

2- AL- octahedra

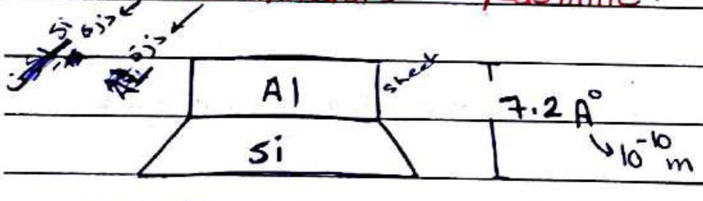
→ hexagonal hole.



clay mineral. يتشكلوا من هدول الاثنين معاً بيرتبطوا تركيبة أسهل

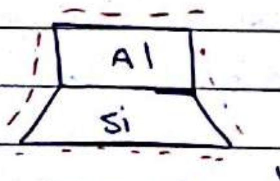
- particle of clay not visible.

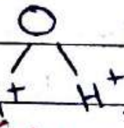
- 1:2 minerals - Kaolinite.



Basal ~~Spring~~ spacing

[تشكلة] platey shap (شرائح)





- الماء له خاصية القطبية polarity

- الـ clay بتمص ماء أكثر من sand والماء بين الفراغات.

لأن H_2O بيستاد بالسطح المشحون ليعادل الشحنات.

hydrogen bond

لا يستطيع الماء ابعادهم عن بعضها رابطة قوية

chemical weathering

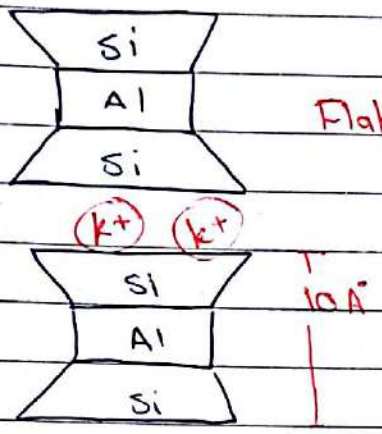
- هذا النوع مرغوب فيه لأنه لا يحدث swelling ← عدم حدوث cracks في الأبواب والشبابيك.

bonding between plates :-

1- von der waals force. — في مواد البناء يتعامل مع مواد معينة

2- hydrogen bonds. — أما في السويلز مع جميع المواد. selected

* 2:1 minerals - Illite. — طبقتين سيليكا بينهم طبقة Al



Flaky shap.

surface Area أكبر اذاً امتصاصه أكبر للماء غالباً

كلما كبرت Area يزداد الشحنات السالبة عليه

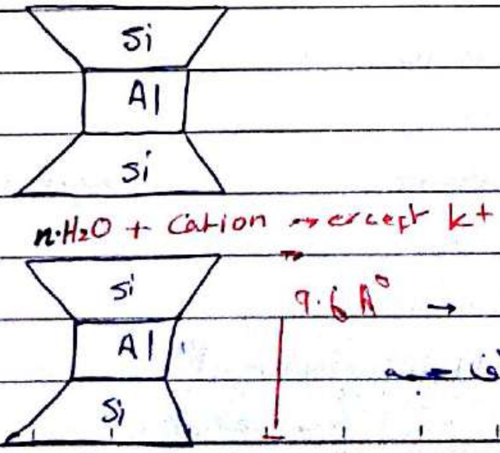
- Some of Si^{+4} in the tetrahedra are replaced by Al^{+3} & some of the Al^{+3} in octahedra sheet are replaced by the Mg^{+2}
- the change of deficiency is balanced by the potassium ion between layers [no swelling يعني غير ملحوظ]
- strong bonds →

* potassiom atom can exactly fit into the hexogon hole.

[يحجم الفراغ تماماً (لا يوجد ايون آخر بحجم هذا الفراغ]

فيستطيع الجلوس في الفراغ السداسي لان k نفسها حجم الفراغ أما باقي الذرات أكبر او أصغر

* 2:1 Minerals - Montmorillonite



→ بدون الماء

مع الماء ٣ أضعاف حجمه

31-1-2016

الأحد ⑦

* Soil texture :-

"التمييز بين الـ physical والـ chemical
كيف أعرف نوع السويل"

the texture soil is it appearance or feel & it depend on the relative size & shapes of the particles as well as the range or distribution of those sizes.

* sieve #200 (200 hole in every inch) D = 0.075mm

~~Sieve analysis~~ | hydrometer analysis

coarse-grained Soils → Gravel, Sand

Fine-grained soils → Silt, Clay

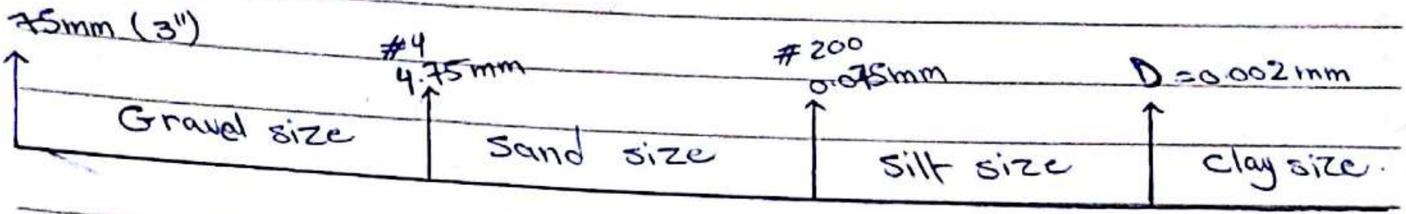
# sieve NO #200
D = 0.075 mm

← ما بنستخدم الهيدر لأنها
ما عليها
- physical weathering
negative charge.
(nonplastic ، particles ما بتنزل)

chemical weathering

	Gravels, sands	silts	clays
size	coarse grained ممكن نميزه بالنظر	Fine grained لا يمكن نميزه بالنظر	
characteristics	Non plastic لا تتماسك لأنه غير متماسك.	Nonplastic	plastic
water on behaviour	relative unimportant	Important	very important
effect of grain size	Important	Unimportant لأنه غير مرئي	

* Grain size



له حجم ذرة clay.

* حتى تكون clay لازم تكون من عائلة clay mineral

- الفرق بين silt و clay ← ممكن الذرة يكون الها plasticity بس ما تكون من clay minerals

اذا هي ليست clay.

- أصغر sieve ممكن أصنعه sieve # 200 (0.075mm)

- حتى يكون clay or silt يجب أن يتفاعل مع الماء، اذا مر من المنخل يصعب تمييزهما

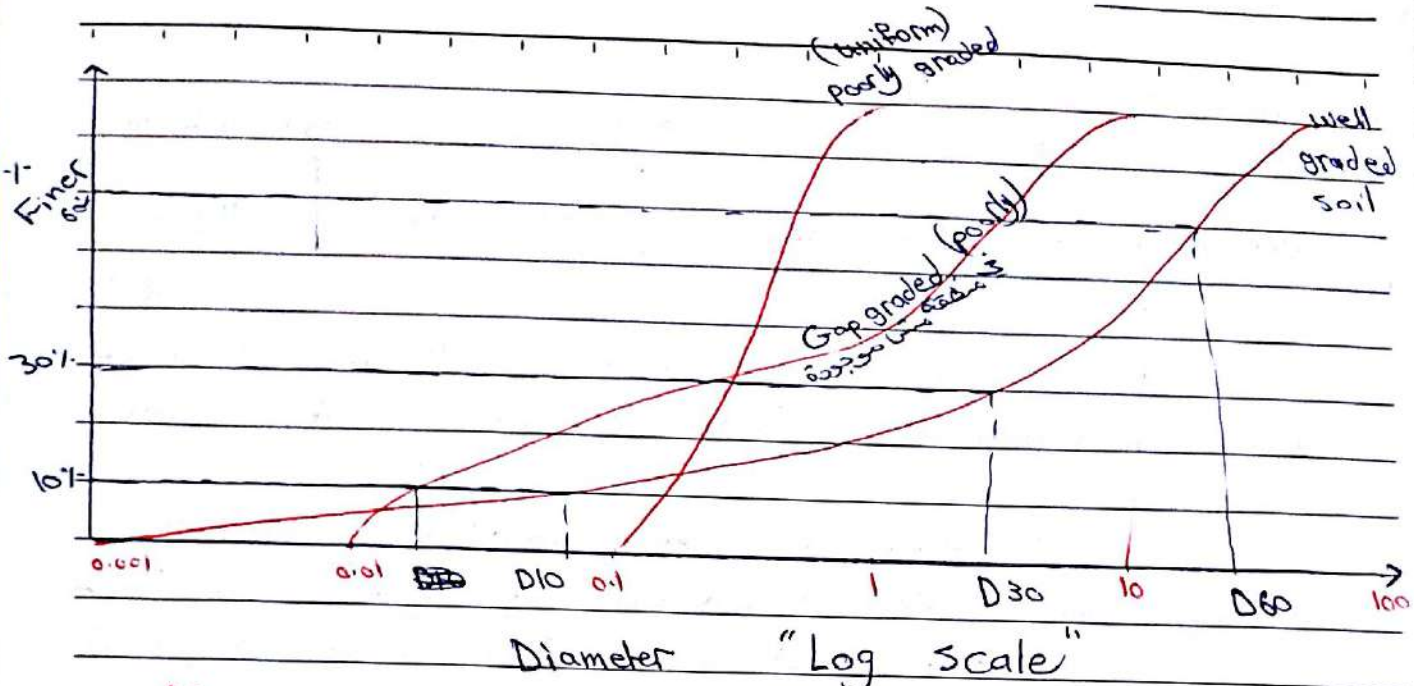
تفاعله مع الماء يكون نشاط ذراته بسبب حجم ال clay و silt

* Grain size distribution.

Diameter	wt of soil retained	Cumulative wt retained	% retained	% finer
D = 75 mm	x_1	x_1	$\frac{x_1}{1000} \times 100\%$	100% − % retain
63.5	x_2	$x_1 + x_2$	$\frac{x_1 + x_2}{1000} \times 100\%$	⋮
50	x_3	$x_1 + x_2 + x_3$		
37.5	x_4	⋮	⋮	⋮
25 mm	x_5			
3/4"	x_6			
# 4	x_7			
⋮	⋮			
# 200	x_8			
Pan	x_9			

* يجب أن أحدد التراكمي لانه ال soil الي بيمر على أي sieve بيرد عالاصغر منه

* نعمل shaking



effective size.

D10 : diameter crossponding 10% finer

D30 : diameter crossponding 30% finer

D60 : diameter crossponding 60% finer.

* coffecient of uniformity $C_u = \frac{D60}{D10}$

*

* coeffecient of curvature $C_c = \frac{(D30)^2}{(D10)(D60)}$

- Well Graded IF

1- $1 \leq C_c \leq 3$ and $C_u \geq 4$ ---- Gravel

2- $1 \leq C_c \leq 3$ and $C_u \geq 6$ --- Sand

- اذا واحد من هالشرطين ماتحقق بكون poorly.

*Particles shape :-

- Rounded.
- subrounded
- Angular
- sub angular

coarse -grained soil

Fine -grained soil.

sheet

2-2-2016

[3]. الثلاثاء

* Atterberg limits & consistency Indices.

The presence of water in fine-grained soil can significantly affect associated engineering behaviour, so we need a reference index to clarify the effects

liquid state

→ liquid limit (LL, WL)

plastic state

(اللي بتحويه ال soil من اي - WE)

الحد الفاصل بين المرحلتين . → plastic limit (PL), WP

} → measured.

semi solid state

upper level semi-solid state

Lower level plastic state

→ shrinkage limit (SL)

Soild state (ناشفة مافيها water)

$wc = \frac{M_w}{M_s}$

M_s → oven dry soil

* wc → reference كمية الماء مقاسا بالنسبة لكمية ال soild
↳ oven dry.

* بنعرف كمية الماء من خلال توزين ال solid وهو فيه ماء ثم بنجففه ونوزنه بالوزن فالفرق هو كمية الماء .

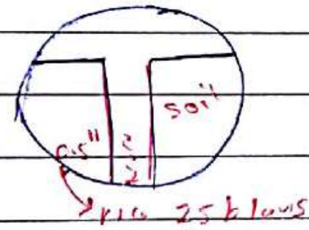
* Liquid limit (LL)

- Casagrande method (ASTM D4318-95a)
- The water content (Wc) percentage required to closure a distance of 0.5" (12.7mm). along the bottom of the groove after 25 blows

بنشقها

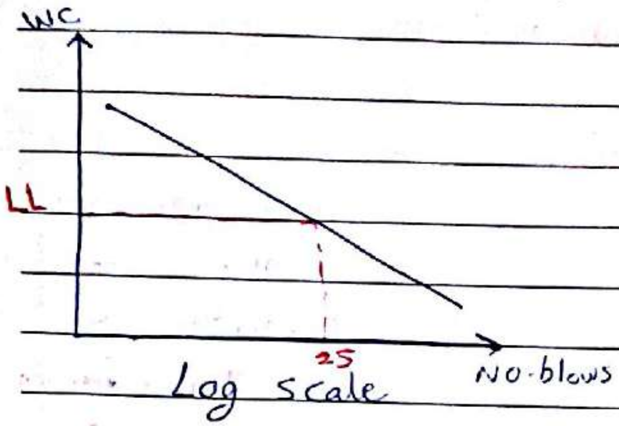
معناها الحبات

- كمية من السويل بنحطها في casagrande cup
ثم بعمل اخدود groove وبعمل فيصير عنا جزئين
بعدين بعمل dropping مسافة اسم وكل ما بعمل هيك
هالجزئين رح يتزلقوا ع بعض ولما يصير
groove = 0.5" عند عدد ضربات 25 blows
فـ wc عند هالحالة بتمثل (LL)
فبقيس وزنها ثم بنحطها بالفرن ليوم
واحد بعدها بوزنها ثم الفرق بينها
بكون كمية w ثم بقسمها ع وزنها (od)
فبطلع معي wc



No. of blows (N)	Wc
$N_1 \simeq 38$	Wc_1
$N_2 \simeq 28$	Wc_2
$N_3 \simeq 20$	Wc_3
$N_4 \simeq 15$	Wc_4

- وصلت ٢٨ ضربة وما صار 0.5" اذا ضيف ماء والعكس صحيح



* plastic limit, PL :- $LL > PL$

Water content, $\left(\frac{M_w}{M_s}\right)$ at which a soil thread with 3.2 mm (1/8)" just crumbles. تفتت

- لو صار السوك يتفتت قبل 1/8" ← بكون لسا في semi-solid.
- وصلت 1/8" وصار فيه شق بكون تجاوزنا plastic limit
- كلما ارتفع LL بكون عنده swelling

* plasticity index (PI) = LL − PL (plastic) مدى السوك بتشكل

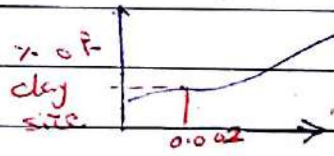
- Liquidity index, LI : $\frac{W_n - PL}{PI}$ → field (موقع)

W_n : Natural water content.

- $LI < 0.0$ → brittle fracture.

$0 < LI \leq 1.0$ → plastic.

$LI > 1.0$ → viscous liquid



* Soil Activity (A) = $\frac{PI \xrightarrow{} LL - PL}{\% \text{ of clay size}}$

لمؤشر لترتيب مدى التغير الحجمي للتربة الطينية.

Normal clay $0.75 < A < 1.25$

~~Enad~~ Inactive clay $A < 0.75$ ما فيه swelling

Active clay $A > 1.25$

في مشكلة expansive swelling

4-2-2016

1 large volume change when wetted (بالشتاء)
3 very reactive (chemically) [not stable]
2 large shrinkage when dried. (بالصيف)

- soil classification system. من ناحية ال behaviour تصنيف معين -
- PI مهم في viscosity [PI <12 اذا أكبر مخالف للمواصفة)
- تصميم خلائط السدود.

Soil Classification

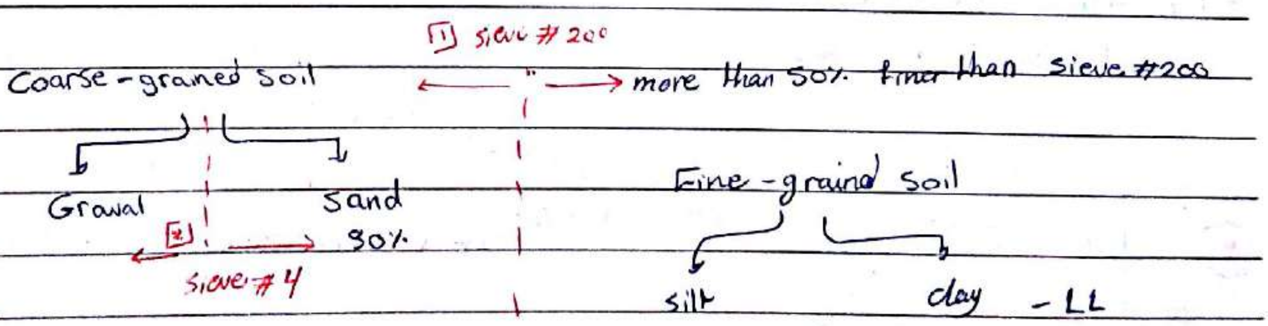
classifying soils into groups with similar behaviour, interms of simple indices, can provide geotechincal engineers a general guidance about engineering properties of soil.

simple indices → classification → Estimate engineering → Achieve engineering
GSD, LL, PL system (language) properities. purpose.

-unified soil classification system (USCS)

* Four major group :-
1- coarse-grained soil < Gravel, sand
2- Fine-grained soil < silt, clay
3- organic soils عضوية (مثل الفحم)
4- peat

} نظرياً مثل Fine



- LL
- PL

- Grain size distribution
- Cu (well graded or poorly graded)
- Cv

* soil symbols :-

G : Gravel
S : Sand
M : silt
C : clay
O : organic
Pt : peat

W: well graded
P: poorly graded
H: high plastic soil. $LL \geq 50\%$
L: low plastic soil $LL < 50\%$

- Sw: well graded sand.
- CH: high plastic clay
- ML : low plastic ~~m~~ silt
- SM : silty sand. ≠ MS

~~SH~~: ~~CW~~

Ⓘ IF more than 50% finer than sieve # 200

Fine grained soil :- Ⓐ $LL \geq 50\%$ — Inorganic soil — CH. IF PI soil on or above A-Line.
— MH IF PI soil below A line
— organic soil → $\frac{LL\ oven\ dry}{LL\ air\ dry} < 0.75$ → (OH) high plastic organic

- تطفو على الماء
- الها رائحة
- لونها اسود داكن.

- $PI - Aline = 0.73 (LL - 20)$

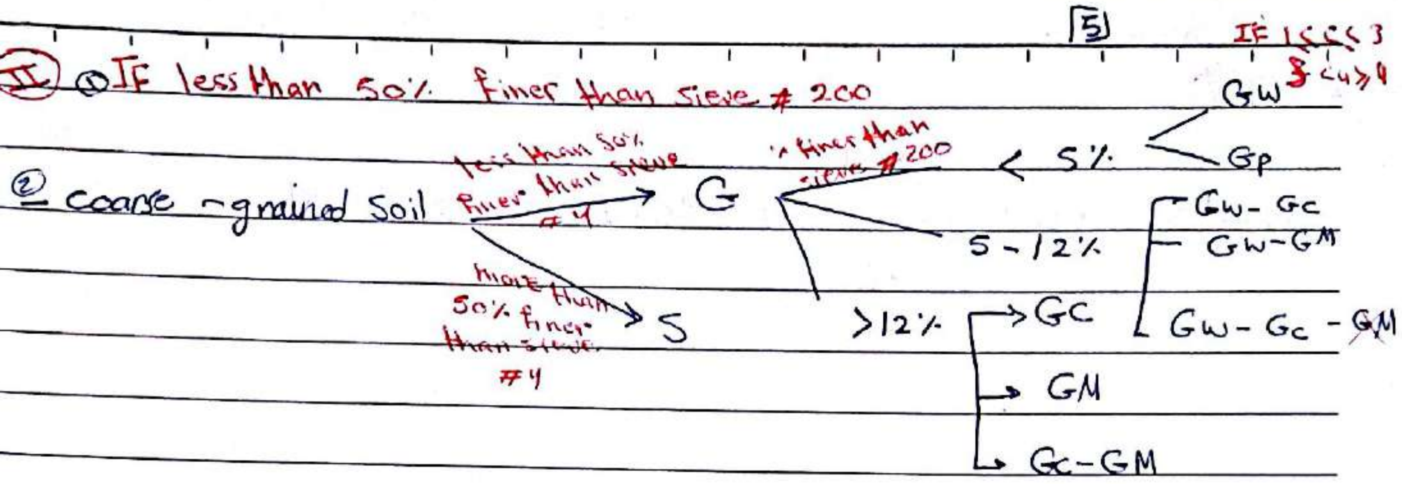
$PI\ soil > PI - Aline$

Ⓑ $LL < 50\%$ — Inorganic soil — CL IF PI soil ≥ A-line
→ CL - ML IF $4 \leq PI\ soil \leq 7$
— ML IF PI soil < Aline. 4-7
— organic soil → $\frac{LL\ ovendry}{LL\ air\ dry} < 0.75$ → (OL) low plastic organic

7-2-2016 الأحد

5

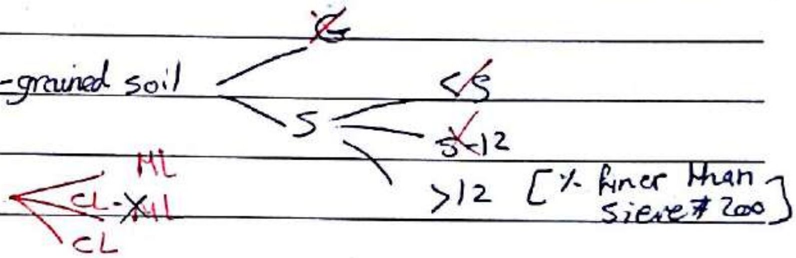
II ① IF less than 50% finer than sieve # 200



cx

sieve NO	% finer
#4	70%
#200	30%
LL	33%
PI	12%

⇒ coarse-grained soil



PI A-Line = 0.73 (33-20) = 9.49

(PI soil) 12% > 9.49 ∴ SC clay sandy

question :-

siev NO	soil ① % finer	soil ②	soil ③	soil ④	soil ⑤	soil ⑥
#4	99	99	97	100	100	100
#10	92	92	90	100	100	100
#40	86	86	40	100	100	100
#100	78	78	8	99	99	99
#200	60	60	5 →	97	97	50
LL	20	55	–	124	oven dry 124	124
PL	15	52	–	47	47	47
PI	5	3	N.P* Non plastic	77	77	77

LLair dry = [illegible] [illegible] 200

* Salution

* soil ①

Fine-grained soil
- cL
- cL-ML $4 \leq PI \leq 7 \rightarrow$ CL-ML
- ML

* soil ②

$PI - A line = 0.73(55-20)$

Fine-grained soil
- cH
- MH

$\therefore$ PI soil < PI-Aline.

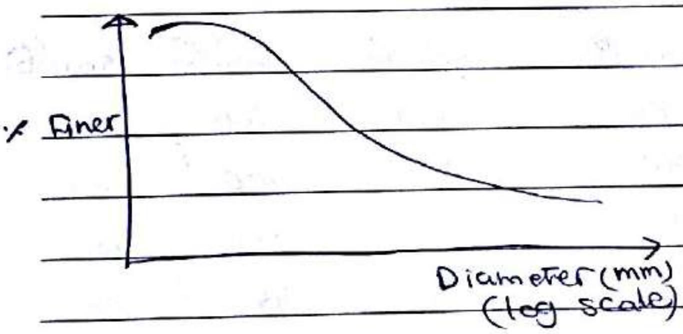
$\therefore$ MH : high plastic silt.

* Soil ③

coarse-grained soil % finer #200 < 5
- 5
  - < 5
  - 5-12
- > 12

$\therefore$ ~~SP~~ - SM

major, miner
↳ sand ↳ non plastic < clay / silt



D10 = 0.18 mm

D30 = 0.54 mm

D60 = 0.71 mm.

$$Cu = \frac{D60}{D10} = \frac{0.71}{0.18} = 3.9 \not> 6$$

* soil ④

Fine-grained soil — $\boxed{CH}$ / MH

PI-Aline = 0.73(124-20) = 75.9

PI-soil (87%) ≥ PI-Aline

* soil ⑤

$\frac{\text{LL oven dry}}{\text{LL air dry}} < 0.75$ [organic شرط يكون]

$\frac{124}{200} = 0.62 < 0.75$ --- OH

* soil ⑥ CH-SC

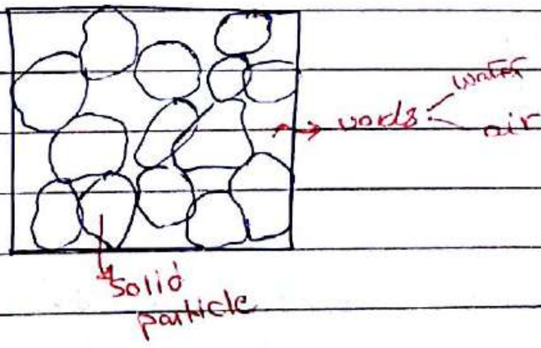
(I) As-aline → CH

(II) As acoarse. < ~~G~~ / S

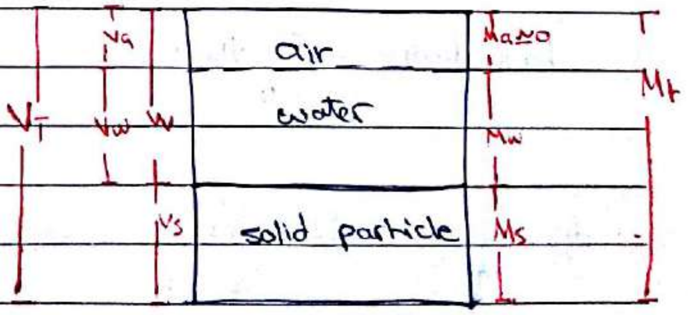
S — ~~<5~~ → SC
— ~~5-12%~~
— >12%.

solid part for soil ↑
voids ↓

"Phase Relationship"



* volume * mass



$M_t = M_w + M_s$

-Volume.

* The voids Ratio (e) = $\frac{V_v}{V_s}$ --- percent

indecimed (دائماً)

ex → Sands range 0.4 to 1.0 VT

clay range 0.3 to 1.5 [- حجم الفراغات أكبر من حجم solid بـ 1.5]

* porosity (n) $\frac{V_v}{V_T} * 100\%$ المسامية.

[0 - 1]

- for water n = 100%.

* Degree of saturation (sr) = $\frac{V_w}{V_v} * 100\%$ water

درجة الاشباع.

→ 0 - الفراغات كلها هواء dry

→ 1 الفراغات كلها ماء → saturated

→ sr>1 → wet

↳ علاقة خطأ مستحيل.

* air content or Air voids (A) = $\frac{V_a}{V_T} * 100\%$

→ 0 when sr = 1

→ 100% when sr = 0

IF sr = 0 → $V_v = \cancel{V_w}^{0} + V_a$

- Mass

* $W_c = \frac{M_w}{M_s} * 100\%$ wc > 1 → صح [في التربة marine soil بحرية في قاع البحار، ولما تكون عضوية.]

mass of oven dry ← M_s

- Mass & volume. Relation

natural, moist

1. bulk density = ρ, ρ_{wet}, ρ_t = $\frac{M_t}{V_t}$ t. total

2. dry density = $\rho_d = \frac{M_s}{V_T}$

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

9-2-2016 الثلاثاء

[3] solid density = $\rho_s = \frac{M_s}{V_s}$

[4] saturated density = $\rho_{sat} = \frac{M_t}{V_t}$ ، $S_r = 1.0$ شرط

$= \frac{M_s + M_w}{V_t}$

* specific Gravity = $G_s = \frac{\rho_s}{\rho_w} = \frac{M_s/V_s}{\rho_w} < \begin{matrix} 2.6 \\ - \\ 2.8 \end{matrix}$ } تقريباً measured value.

* unit weight (γ) = $\rho * g$ ($\rho \to 1 kg/m^3$, $g \to 9.806$) $\Rightarrow kN/m^3$ 9-2 *

* Relative density (Dr %) I D) :-

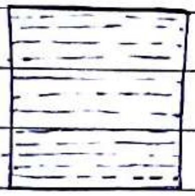
In case of sands & gravel, the density index (ID) or Relative density (Dr%)

$$Dr(\%) = \frac{e_{max} - e}{e_{max} - e_{min}}$$

e → field measured

$e_{max} - e_{min}$ → lab measured

* Dr تستخدم لتصنيف التربة الرملية sand فقط مش للطين.



Volume ⇒ 944 cm³

$V_T = V_v + V_s$

$e_{max} = \frac{V_v}{V_s} = \frac{V_T - V_s}{V_s}$ ($V_T \to 944 cm^3$, V_s عن G_s)

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

$G_s = \frac{M_s}{V_s/\rho_w}$

* after vibration. V_T تقل. → e_{min}.

V_s, M ثابتة

$V_t < 944 cm^3$

emin = Dense state [compacted state]

emax = loose state

e = Natural state, field state, in-situ state

* Derivel Relationship

$\boxed{1}$ $n = \frac{e}{1+e}$, $n = \frac{V_v}{V_T} = \frac{V_v}{V_s + V_v} = \frac{V_v/V_s}{(V_s+V_v)/V_s} = \frac{e}{1+e}$.

$\boxed{2}$ $\rho = \rho_d (1 + W_c)$, $\rho_{bulk} = \frac{M_t}{N_T} = \frac{M_s + M_w}{V_t} = \frac{M_s + M_s W_c}{V_t}$

$w_c = \frac{M_w}{M_s}$ // $\frac{M_s(1+w_c)}{V_T} = \rho_d (1 + W_c)$

$\boxed{3}$ $G_s W_c = sr\, e$

$\boxed{4}$ $\rho_d = \frac{G_s}{1+e} \rho_w$

$\boxed{5}$ $\rho_{sat} = \frac{G_s + e}{1+e} \rho_w$

$\boxed{6}$ air content $= A = n(1 - sr)$

Ex

$M_T = 2.290$ kg , $V_T = 1.15 \times 10^{-3}\, m^3$

$M_s = 2.035$ kg , $G_s = 2.68$ measured.

$\boxed{1}$ $\rho = \frac{M_t}{V_t} = \frac{2.29}{1.15 \times 10^{-3}} = 1990\ kg/m^3$

$\boxed{2}$ $\gamma = \rho g = \frac{1990 * 9.806}{1000} = 19.5\ kN/m^3$

$\boxed{3}$ water content $= \frac{M_w}{M_s}$ ~~*100%~~ $= \frac{M_t - M_s}{M_s} * 100\% = \frac{2.29 - 2.035}{2.035} * 100\%$

$W_c = 12.5\%$

[4] $e = \frac{V_v}{V_s} = \frac{V_T - V_s}{V_s} = \frac{1.15 * 10^{-3} - 7.59 * 10^{-4}}{7.59 * 10^{-4}} = 0.52$

$G_s = \frac{M_s / V_s}{\rho_w} = \frac{2.035 / 7.59 \times 10^{-4}}{1000} = 2.68$

[5] $n = \frac{V_v}{V_T} = \frac{1.15 * 10^{-3} - 7.59 * 10^{-4}}{1.15 * 10^{-3}} * 100\% = 34\%$

[6] $S_r = \frac{V_w}{V_v} * 100\% = \frac{V_w = 2.55 \times 10^{-4}}{1.15 * 10^{-3} - 7.59 * 10^{-4}} * 100\% = 65.2\%$

$\rho_w = \frac{M_w}{V_w} \rightarrow 1000 = \frac{2.29 - 2.035}{V_w} \rightarrow V_w = 2.55 * 10^{-4}\ m^3$

[7] $A = \frac{V_a}{V_t} = \frac{1.15 * 10^{-4} - 7.59 * 10^{-4} - 2.55 * 10^{-4})}{1.15 * 10^{-4}} * 100\% = 11.83\%$

$V_v = V_w + V_a = V_t - V_s$

[8] $\rho_{sat} = \frac{M_t}{V_t} = \frac{M_s + \overset{S_r=1}{M_w}}{V_T} = \frac{2.035 + 0.391}{1.15 * 10^{-3}\ m^3} = 2109.6\ kg/m^3$

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

$V_w = [1.15 * 10^{-3} - 7.59 * 10^{-4}] = 3.91 * 10^{-4}$

$M_w = V_w * \rho_w = 3.91 * 10^{-4} * 1000 = 0.391\ kg$

$\rho_w = \frac{M_w}{V_w}$

[9] $W_c = \frac{\overset{\text{at } S_r=1}{M_w}}{M_s} = \frac{0.391}{2.035} = 0.192 = 19.2\%$

at standard saturated density

ex 2 $= 1.76\,g/cm^3$ $\rightarrow 2.7\,g/cm^3$

$\rho = 1.76\,Mg/m^3$, $\rho_s = 2.7\,Mg/m^3$, $w_c = 10\%$

① dry density ρ_d

$\rho_t = \rho_d(1+w_c) \rightsquigarrow \rho_d = \frac{1.76}{1+0.1} = 1.600\,g/cm^3$

- assume $V_s = 1\,cm^3$

$\rho_s = \frac{M_s}{V_s} = 2.7 \Rightarrow M_s = 2.7\,g$

$w_c = \frac{M_w}{M_s} = 0.1 \Rightarrow M_w = 0.27\,g$

$M_t = 2.7 + 0.27 = 2.97$

$\rho = \frac{M_t = 2.97}{V_T} = 1.76$

$G_s = \frac{\rho_s}{\rho_w} = 2.7$ ~~g/cm³~~

$V_v = 1.6875 - 1 = 0.6875$

$\rho_d = \frac{M_s}{V_T} = \frac{2.7}{1.6875} = 1.6\,g/cm^3$

② void ratio (e)

$\rho_d = \frac{G_s}{1+e}\rho_w$, $= \frac{V_v}{V_s} = \frac{0.6875}{1} = 0.6875$

③ porosity (n)

$n = \frac{e}{1+e} = \frac{V_v}{V_T} = \frac{0.6875}{1.6875} \times 100\% = 40.7\%$

14-2-2015

الأحد.

[4] Degree of saturation Sr

$G_s w = S_r e$, $\frac{V_w}{V_v} = \frac{0.27}{0.6875} * 100\% = 39.36\%$

[5] Air content

$A = n(1-S_r) = \frac{V_a}{V_t} = \frac{0.6875 - 0.27}{1.6875} * 100\% = 24.7\%$

[6] $\rho_{sat} = \frac{G_s + e}{1+e} \rho_w = \frac{M_t}{V_t} = \frac{M_s + M_w}{V_t} = \frac{2.7 + 0.6875}{1.6875} = 2.007\ g/cm^3$

*14

Compaction

* Many types of earth construction such as dams, retaining walls highway, air port, require man-placed soil "fill" to compact soil that is to place it in a dense state. The dense state is achieved through reduction of the air voids in soil with little or No reduction in the water content.

$\rho = \frac{M}{V}$ ← بازيد M يا بقلل V

$- \rho_d \uparrow = \frac{M_s}{V_s + V_a}$

ما بدي اياها تتأثر من الي بتغير بالصيف والشتا.

*** objective :-** compaction السبب لعمل*

1- Decrese permeability * تقليل التغيرات الحجمية للتربة الانتفاخ والانكماش

2- Decrease future settlement (الهبوط) تقليل الهبوط * زيادة معامل الأمان للتربة.

3- Increase shear strength.

*** General compaction methods**

كيف أقلل air voids.
Vibrating
دك

- coarse-grained soil

- lab * vibrating hammer
- field ① Hand-operated vibration places.
  ② Motorized vibratory rollers.
  ③ Raber-tired equipment. "vibration"

- Fine-grained soil

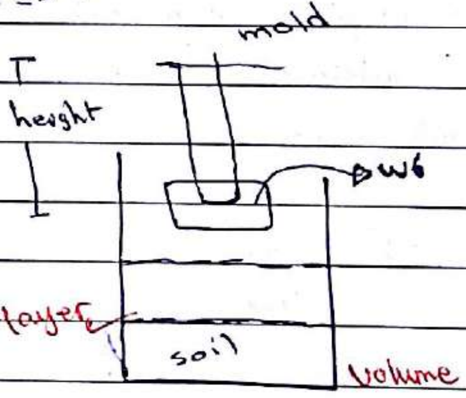
. Falling weight & hammer
. kneeding compactors
. static loading & press
① Hand-operated tampers.
② sheepsfoot rollers. مع أساسات الردم
③ Ruber-tired rollers (knaling)

*Laboratory compaction :- نوصّف comp لحتى نوصل ϱ اللي بدنا اياها

purpose
لنعرف كمية الماء as Wc اللي بدي استخدمها ع soil بال field

① To determine the proper amount of mixing water to be use in field.
② " " " resulting degree of denseness
↳ كم ϱ أوصل

- Standard proctor
غالباً - Modified proctor ⇒ ASTM-698



	Standard proctor	Modified proctor
height	12" = 30cm	18" = 45cm
wt. of hammer	5.5 Ib = 2.5kg	10 Ib = 4.5kg
- No of layer	3layer	5 layer
- No of blows per layer	25/layer	25/layer
volume of mold	1/30 ft^3 944 cm^3	1/30 ft^3 ↳ energy (بتعطي) أعلى
	الأحمال الخفيفة والطرق العادية	الأحمال الثقيلة وطرق المطارات

$\rho = \frac{M}{V}$

- Variables of compaction :- العوامل التي تؤثر بالدمك.

1- Dry density (ρ_d)   Modified بعطيني ρ_d أكبر و O.M.C أقل !!

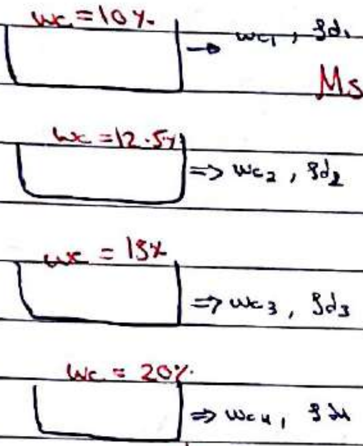
2- water content (w_c)   أعطيتها energy أكبر.

3- Compactive effort (E) جهد الدمك.

4- Soil type (gradation, plasticity)

graded ^ poorly   C ^ M

مثلاً Soil as CH



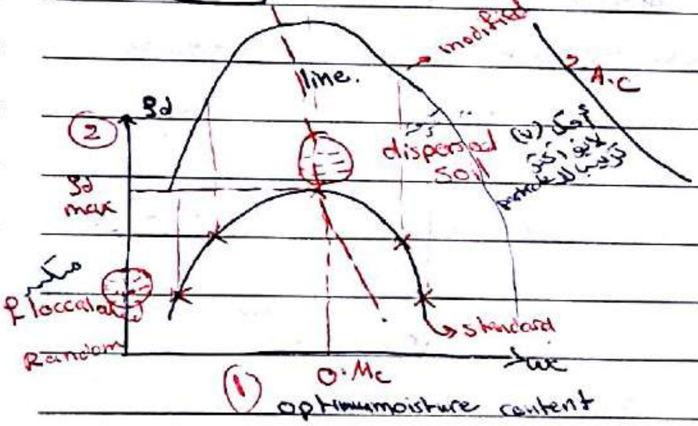
Ms for all = 5000g. , 1

$\rho_{bulk} = \frac{M_t}{944 cm^3}$

$v = 944 cm^3$

$\rho_d = \frac{\rho}{(1+wc)}$

- طبعاً الـ standar proctor.



- كل ما ضيف w بتزايد الـ ρ_d لحد معين بعدها بصير ρ_d يقل.

- التفسير الرسمي.

بدي اجيب ماتريال من برا لازم يكون صفيحا

plasticity ← clay ← عليه شحنات سالبة

يكون V_s particle ← Random لانو حاي بشكل flocculated

- peak point (ρ_{max} , O.M.C)

optimum water content W_{opt})

لمحتوى الرطوبة الأمثل بتعطي أقصى ρ_d

الشحنات المتشابهة تتنافر وكل ما كان الحجم Random بكون V كبير ف ρ قليلة

بلشت أضيف w فبعادل الشحنات ولما تتعادل فالتنافر رح يقرب فبصير يرتب حالهم فالـ V يقل ف ρ تزداد إلى أن نصل لحد معين ف soil بسميها dispersed فبكون كل الشحنات متعادلات (أكثر ترتيب للـ particle فأقل V فأكبر ρ) فأنا لما نضيف w فوق أوصل لمرحلة LS ف w صار كل محل soil فيصبح كالوحل و $\rho_{soil} > \rho_w$ فعندها الأمر بصير تقل معي. [اذا دخلت بحالة Liquid state معها حاولت تحمل ما energy حتأثر باشيا] water مهم بـ الدمك ولكنا منا amounts و ترتيب

16-2-2016

الثلاثاء . [8]

وزنا المطرقة

ارتفاع السقوط ↑ عدد الطبقات

$O.M.C_{modified} < O.M.C_{standard}$ * $E \uparrow \Rightarrow \gamma_{d\,max} \uparrow + O.M.C \downarrow$

$\rho_{d\,max}$ // $>$ $\rho_{d\,max}$ //

مهما عملت comp لا يمكن ان نصل sr=1

③ Zero air Voids curve (sr=100%) : The curve represends the fully ~~stand~~ saturated condition (sr=1) (It can't be reached by compacting) [theorytical curve)

- لا يمكن ان نستطيع مع compaction

- لا يمثل منحنى الدمك. - يستخدم للحكم على صحة منحنى الدمك.

- لا يوجد في الطبيعة وذلك لوجود هواء دائماً داخل الـ...

- للرسمة

$$\underset{calculated}{\rho_{d\,Z.A.V}} = \frac{G_s}{1+e}\rho_w \Rightarrow \rho_d = \frac{G_s}{1+\frac{G_s w}{S_r}}\rho_w$$

$S_r \rightarrow$ assumed

$G_s\, w = S_r e$

④ Line of optimum : A-line drawn through the peak points of several compaction curves of different compactive effects for the same soil.

نظرياً يكون موازي Z.A.C

- كلما زدنا بـ wc يزداد ρ_d لحد معين وبعدها يتناقص

- تأثير Compacted effort فكلما زاد يكون زيادة في No of hammer & No of blows

$\downarrow O.M.C$ و $\uparrow \rho_{d\,max}$

ex $\rho_d = 1.85\, g/cm^3$

$w_c = 17\%$

$G_s = 2.7$

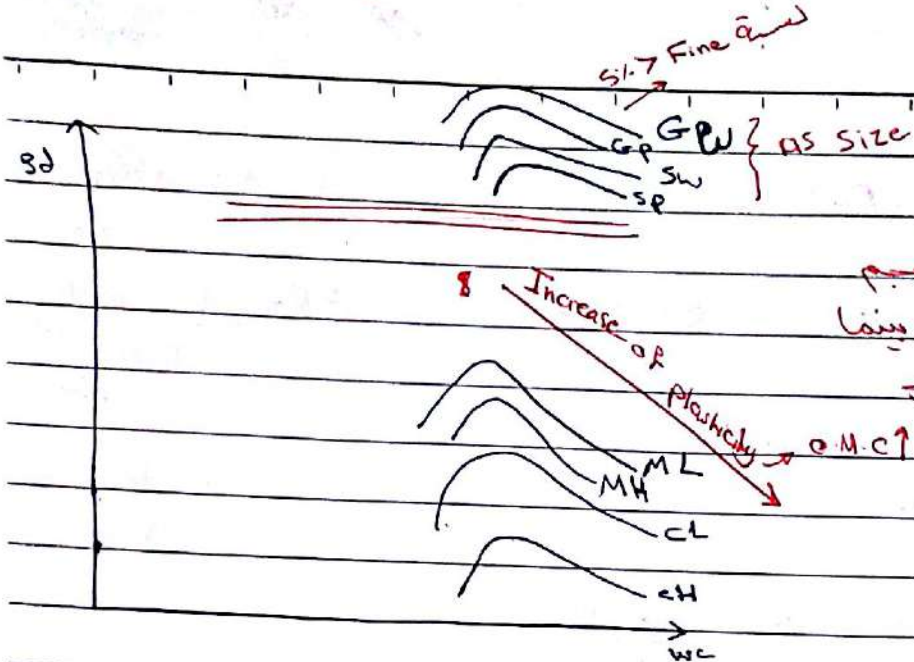
$$\overset{1.85}{\rho_d} = \frac{\overset{2.7}{G_s}}{1+\frac{G_s w^{0.17}}{S_r}} \rightarrow S_r = 105\%.$$

* effect of soil type on compaction :-

The soil that is, grain-sized distribution shapeof the soil grain, Gs, plasticity

size ↑ ⇒ $\gamma_{d\,max} \uparrow$ + O.M.C ↓

* التربة لما تكون well graded متجانسة أفضل كثافة جافة



- كلما زاد الحجم كلما كان تأثير wc مهمل.

- poorly بما انهم نفس الحجم رح يكونوا عندي فراغات أكبر بينما (w) رح يكونوا حشوين كل size/ فالفراغات أقل

poorly ← واحد size

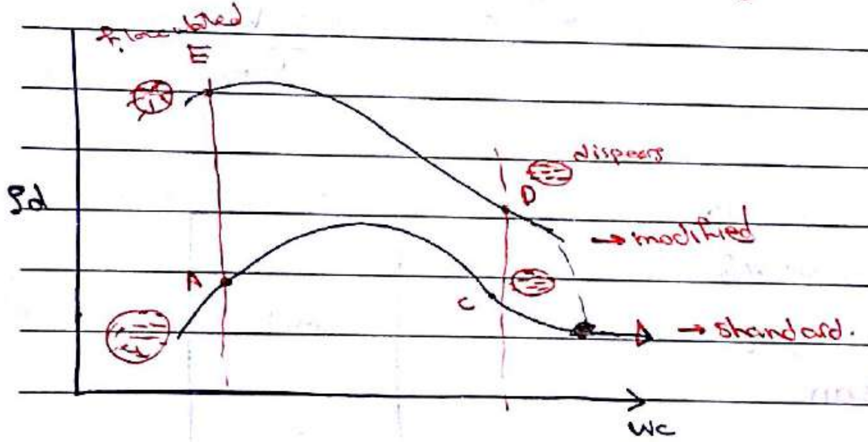
Gap Graded ← two size.

- كلما كانت نسبة plastisity كلما كان في تركيز أكثر عال clay minerals يعني ability to absorp water أكثر. [Sd بيقل]

* clay ال specific gravity أعلى من silt بسبب material وبدي wc أكبر.

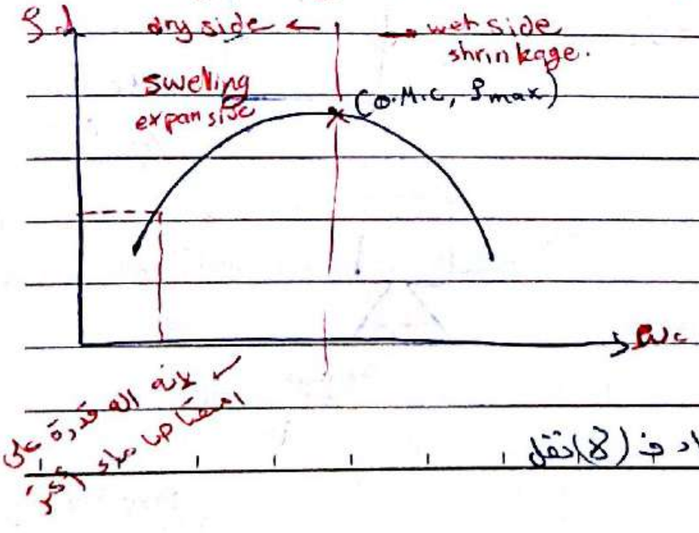
لهذا clay فيه فراغات أكثر من silt

* structure of compacted clays:-



❀ تأثير compacted effort اذا دخلت منطقة 11 هو رح يصير للماء ما في تأثير. وما عاد في airvoids عشان أعلى compaction

* Engineering properities - swelling.



Dry ← العلاقة طردية لان الماء بتساعد على انزلاق الحبيبات على بعض

wet ← و هلا لأعلى لما بجف أقل v ممكن ولان الحبيبات قريبة والفراغات مش موجودة ولو زودت نسبة الماء بعد O.M.C فالماء رح يشغل حجم كبير من الفراغات فيأخذ حجم من التربة فالحجم يزداد ف (لا) نقل

✓ لانه اله قدرة على امتصاص ماء أكثر

Relative compaction (Rc)

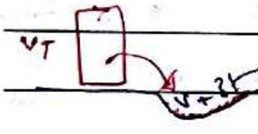
لازم اتأكد من الوصول $\rho_{d_{max}}$ OMC

$RC = \frac{\rho_d \text{ field}}{\rho_d \max} * 100\%$ R.C $\geq 90\%$ $\rho_d = \frac{M_s}{V_T} = \frac{M_s}{V_s + V_a + V_w}$

ثابتة

$w_c \text{ field} = O.M.C \mp 3\%$

بدنا ما أعلا comp ومنعرف w مارح تتأثر ρd.

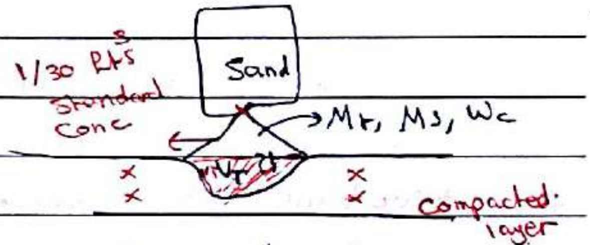


$\rightarrow M_t, M_s \Longrightarrow w_c = \frac{M_t - M_s}{M_s}$

* Relative Compaction R.C

بنقدر الحكم على الدحل مقبول أم لا في الموقع .

$R.c = \frac{\rho_d \text{ field}}{\rho_d max} \times 100\%$



$\rho_{sand} = \frac{M_s \text{ in hole}}{V_t ??}$

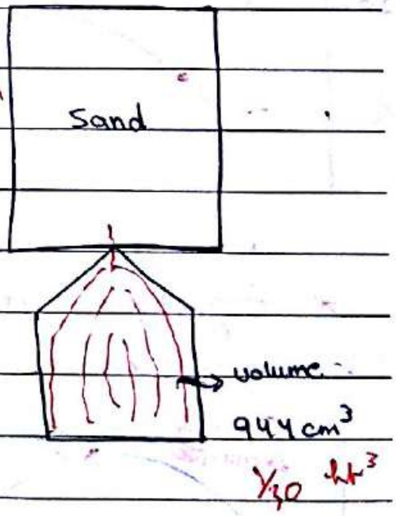
* distructive test

1- sand cone method.

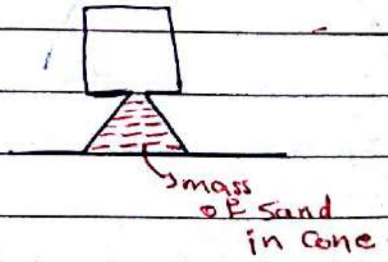
(i) fine-Sand pass sieve NO #10 retained on sieve NO #60

(ii) uniform sand (poorly graded) → الحجرة ما راح تأثر على ρd

(iii) rounded, dry, clean



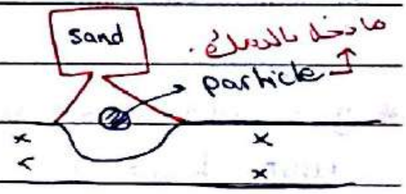
* ما لازم يكون vibration عشان ما تزيد M_{sand} فتزداد ρd.



18-2-2016

- The measuring error :-

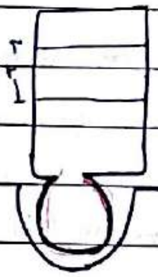
the vibration from nearby working equipment ~~with~~ will increase the density of sand in the hole which gives ~~layer~~ larger hole volume lower field density.



as amass X و as avolume & تدخل خارج particle
لأن حجم الحفرة رح تكون ثابتة ولما يوزنها Ms بوزنة
دون particle

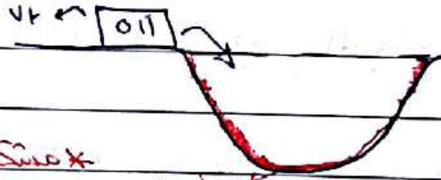
وقد تكون دقة الـ Accuracy قليلة وممكن يكون أثناء تفريغ البالون
*خارج المواصفات لذلك مرفوضة

② Ballon method.



IF the compacted fill is gravel or contain large gravel particle, Any kind of unevenness in the wall hole cause asingnificant error in the ballon method.

3) oil method.
[large hole]

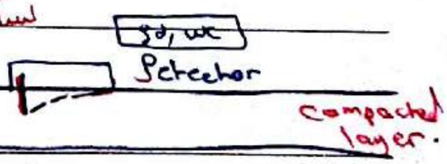


*مشكلة هذه الطريقة (٣) بدها نفرغ Wc وهذا يحتاج
٢٤ ساعة ومستهلك وقت وتكاليفها
يعطل المنطقة بلمترين

- lost for time & cost

* Non dustructive method.
أكثر شيوعاً ودقة

① Nuclear density meter يستخدم في المشاريع الحيوية والمهمة ، التي لا تحتاج للتأخير
- اذا اعطيته قيمة γ_d max رح يعطيني R.C
يطلع أشعة غاما
- لازم يكون مراقبة لاستخدامه
عيوبه: الإشعاعات والـ safty



(10)

[redacted]

الفيرست ↑

↓ second

21-2-2016 [redacted]

Water in Soil (كيف بتحرك water في soil)

* All soils are permeable materials water being free to flow through the interconnected pores between the solid particle".

- كل أنواع ال Soil بتكون permeable لأنه فيهم voids

* you must know how much water is flowing through a soil per unit time??.

- Design earth Dam.
- Determine the quanitity of seepage. (التسرب)
- dewater foundations before & during their construction.
  ← ما بقدر أشتغل تحت مستوى الماء.

* The pressure of the pore water is measured relative to atmospheric pressure & the level at which
* the pressure is atom atmospheric (i:e zero) is defined as the water table (▽)

أنو الضغط هو الي بخلي water يتحرك (flow) .
نتيجة الفرق في elevation

- below the water table the soil is assumed to be fully saturated

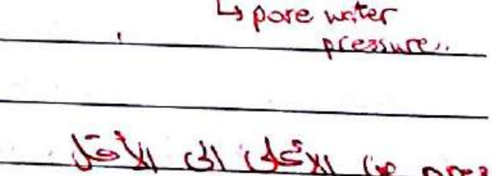
- متى يكون عندي Pressure لازم السويل يكون Full saturated

- Below the water table the pore water may be static or seeping through the soil.

static pore water ثابتة (A)

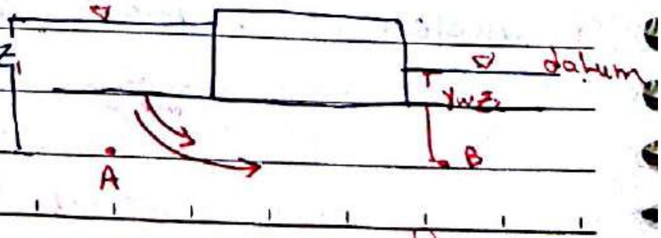
pore water pressure $= \gamma_w \times Z$

$10\,kN/m^3 \times m = 10\,N/m^2$



- reference نعتبر ← lower water table level

pressure من الأعلى إلى الأقل



Five Apple

21/2/2016

* Barnoullion theorm :-

Applie to the pore water but seepage velocities in soil are normally so small the velocity head can be neglected.

$h = \frac{u}{\gamma_w} + Z$ (elevation) $\Rightarrow \frac{\gamma_w \times Z_1}{\gamma_w} - Z_2 = Z_1 - Z_2$

h: total head, u: pore water pressure

γ_w : unit weight of water 9.806 kN/m^3

Z: elevation head above chosen Datum

water table لحد الـ ج.ز

*Darcy's law (1856)

$v_{20} = k i_{20}$

V = Darcy velocity (cm/sec)

k: hydralic conductivity of soil (cm/sec) coefficient of permeability

i : hydraulic gradient $= \frac{\Delta h}{L}$ فرق منسوب المياه بين النقطتين — L → المسافة العمودية بينهم.

الحرارة تقلل

سرعة المياه خلال فراغات التربة فقط ← seepage velocity $= \frac{V}{n} \rightarrow$ porosity $= \frac{e}{1+e}$ or $\frac{V_v}{V_T}$ — تقلل ودا ↑ اللزوجة

المساحة العمودية على اتجاه السريان

$- q = VA = k i A = \frac{V}{t}$ حجم المياه خلال زمن معين

↳ cm^3/sec معدل التصرف — له الزمن — connected voids بسبب ↑ permeability ← coarse-grain (أكبر)

متغيرة — Discharge velocity — بالرغم من أنه حجم الفراغات أقل

K → coarse grained soil $*10^{-4}$ m/sec [Fine السرعة في < coarse السرعة في لأن

↳ Fine grained soil $*10^{-9}$ m/see.

K → measured value.

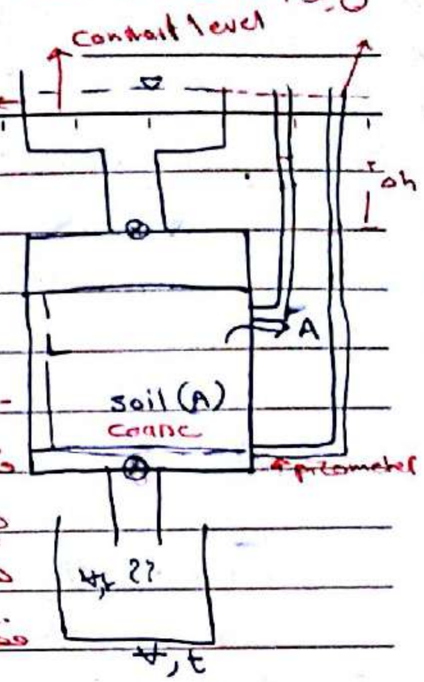
* Measuring k in laboratory :-

هناك طرق لقياس k — lab ©, fields ©, empirical ©

1- coeff of permeability for coarse grained soil → constant-head test

2- " " " " fine-grained soil → Falling-head test.

ح يوصلوا لهاد ال level لأنو هاسنين حزقة في head pressure و مسكرين



- constant head

الاسطوانة الها A مقدر

$$\frac{\forall}{t} = k * \frac{\Delta h}{L} * A$$

اعرفنا V ثم G ثم e ثم n.

$$\Rightarrow k_{20} = \frac{\forall * L}{\Delta h * t * A}$$

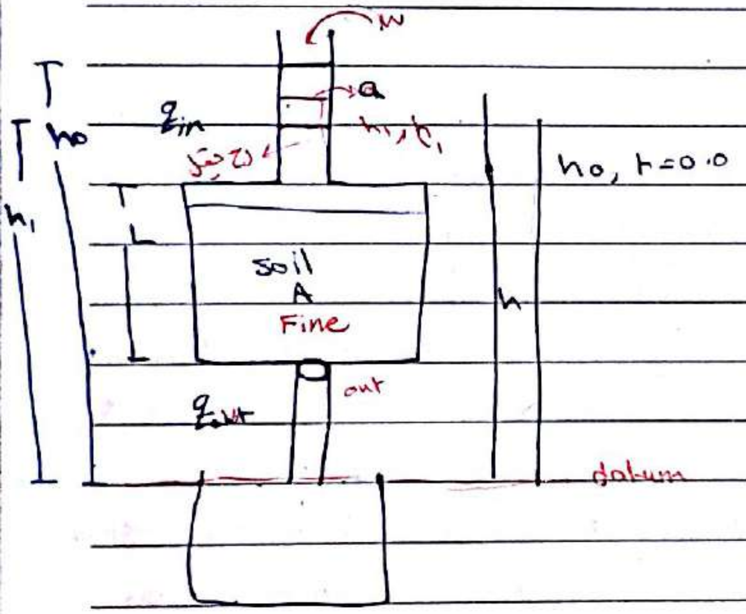
cross section

- بعد فتح Twopin رح تنزل الماء فالسويل Full saturated بعد هاد ح يطلع من الحنفية ما دنه صار two level ما بينها movement & lost نفس الشي. [Δh] ثابت.

فقط علي أجمع volume/unit time & water/unit time

- Falling head test

لأ Fine بيكون head pressure مش ثابت



K خاصية لل soil اللي ما رح تتغير مادام السويل ضلت نفسها

$$q_{in} = q_{out}$$

$$\frac{a*dh}{dt} = k i A = k \frac{h}{L} * A$$

$$\int_{h_1}^{h_0} a \frac{dh}{h} = \int_{t=0}^{t_1} \frac{k}{L} * A \, dt \Longrightarrow k_{20} = \frac{aL}{A*t} Ln \frac{h_0}{h_1}$$

$$a \, Ln \frac{h_0}{h_1} = \frac{k}{L} * A*t$$

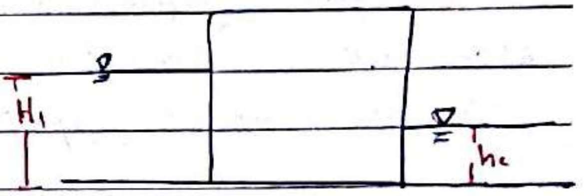
23/2/2016

- Hanzen ~~[illegible]~~ showed © empirical equation.

$K = 10^{-2} (D10)^2$ (m/sec) ← mm

For Fine-uniform sand (SP)

sand ← يمر من sieve #10 ويرسب ع sieve #60



هنا في فرق في head pressure اذا في حركة و lost of water يعني بمقدار q ؟!

"seepage theory"

The general case of seepage in two dimension will how be considered "Laplace's equation of Continuity"

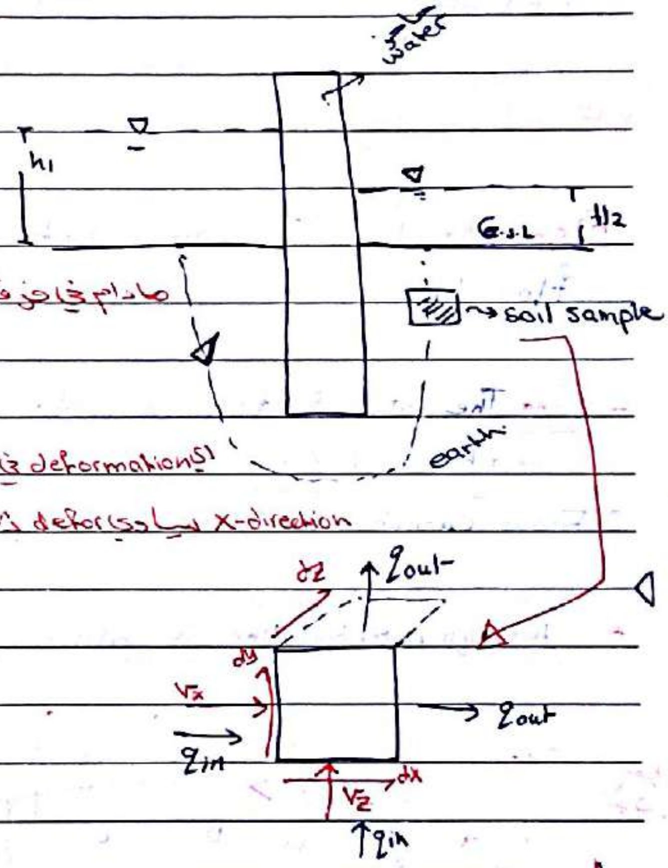
طالما في فرق في head pressure في حركة

* Assumption (فرضيات)

1) soil is hamogeneous & isotropic → في deformation اذا

hetrogeneous ↙ → $Kx = Kz$

2) Generlized Darcy law   y-direction في deform بيساوي x-direction



المدفوع

- $q_{in} = q_{out}$ (full saturated)

طالما فش تسريب.

$V_x A =$

$v_x = k_x i_x = k \frac{dh}{dx}$

$v_z = k_z i_z = k \frac{dh}{dz}$

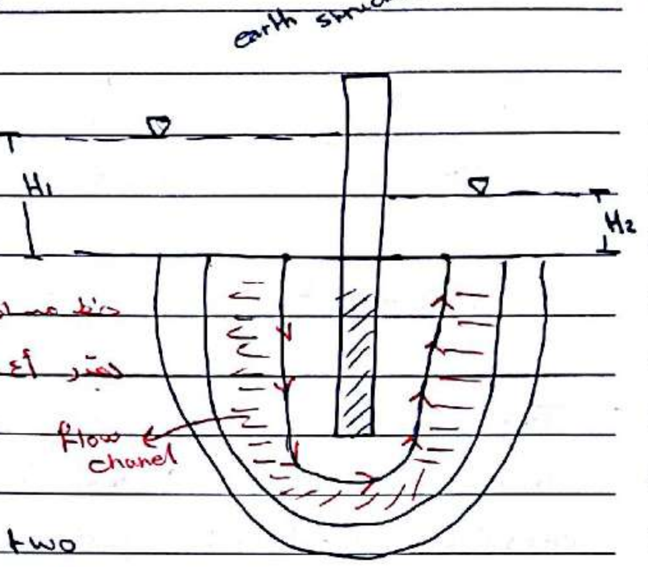
Flow نفس الشي ولكن velocity مختلفة ↵

$$v_x\, dy\, dz + v_z\, dx\, dy = \left(v_x + \frac{dv_x}{dx} dx \right) dy\, dz + \left(v_z + \frac{dv_z}{dz} dz \right) dx\, dy$$

$$\Rightarrow \frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0.0$$

"Flow net" - حل معادلة laplace

A Fllow net is graphical representation of a flow field & comprises a family of flow lines & equipotential lines.



Flow lines :- or stream lines
represent flow paths of particles of water بتمشي من ال water particle ال upper level لل lower level
بتقدر أعلا من Flow line

*Fluid is easily flow.

Flow chanel: The area between two flow line (بتقدر أعلا من منها)

-. ميزة The rate of flow in a flow channel is const but the velocity isn't const. (cross section Area ثابت لا يلزم)

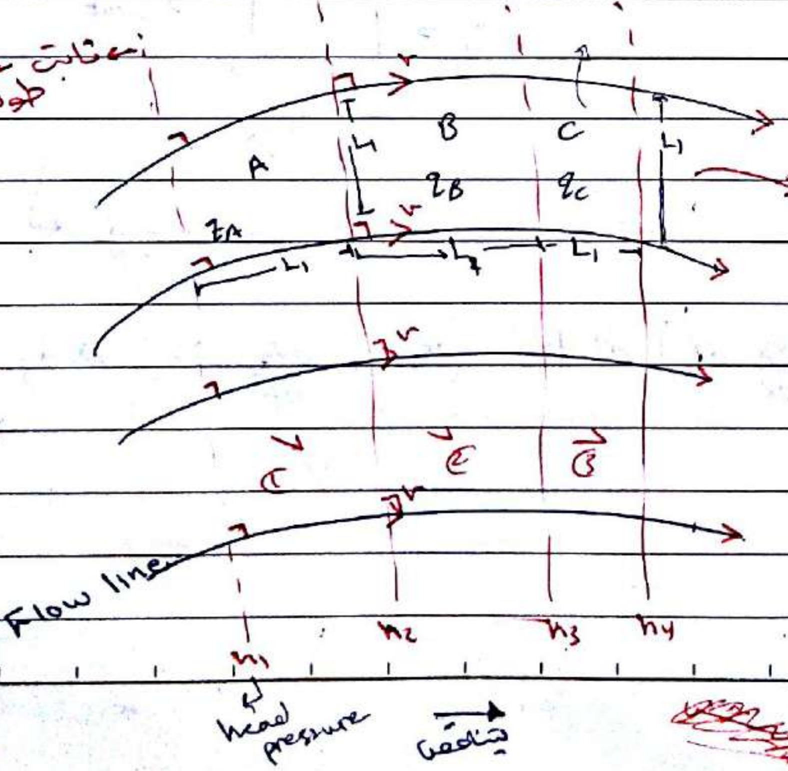
- Flow cannot occur across flow lines. Flow line ما بتتقاطع ← السبب

خطوط تساوي الضغط

* An-equipotential line is a line joining points with the same head

نقاط ثابتة على طول line.

Flow ↑ velocity تكون عكسي على head pressure



→ Flow chanel

Ground water surface تتعرض لل head pressure
↳ atmospheric pressure

↘ head drop.

٥٢   $\frac{٥١}{ND}$

- The difference $\overset{\text{in head}}{\uparrow}$ between two equipotential lines is called potential drop or head loss

- $q_A = q_B = q_c \ldots\ldots q_n$ * flow rate const

$[KAi]_A = [kiA]_B = [KiA]_c = \ldots\ldots$

$$\cancel{K}\frac{(h_2-h_1)}{\cancel{L_1}} * \overbrace{\cancel{L_1} * 1.0}^{A} = \cancel{K}\frac{[h_3-h_2]}{\cancel{L_1}} * \cancel{L_1} = \frac{[h_4-h_3]K L_1}{L_1}$$

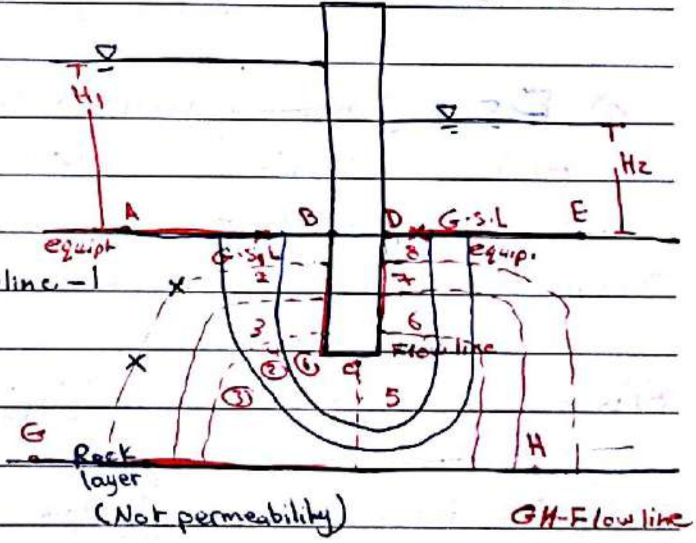
[illegible]

$\therefore \frac{\Delta H}{ND}$ → Number of drop or head loss $= \frac{H_1 - H_2}{ND}$ = head pressure [ΔH : head pressure الفرق في بين البداية والنهاية.]

- $q_{total} = kiA = \frac{k\,\Delta H}{ND} * N_f$ → Number of flow chanel. $q = m^3/s/m$

G.S.L   ground surface level.

* q ح يولد مشاكل على earth structure



GH-Flow line

equipotential line ⇒ AB , DE

Flow line ⇒ GH , BCD

Number of drop NP = 8 = No of Flowline −1

Flow chanel ③ NF

*NP = No of equip −1

Scanned by CamScanner

25/2/2016

* Interpretation of Flow net $\gamma' = \gamma_{sat} - \gamma_w$

pressure ← طاقة بين ال متسويين

1) Head loss $(\Delta h) = \frac{\Delta H}{ND}$ = head pressure

2) Flow rate total $(q) = k \frac{\Delta H}{ND} * N_f$ ← عدد مجرى $\Delta q = k \frac{\Delta H}{ND}$ ↳ flow through each flow channel

3) hydralic gradiant $(i) = \frac{\Delta h}{L} = \frac{\Delta H}{ND * L}$

4 $\text{Max}(i) = i_{max} = \frac{\Delta h}{L_{min}} = \frac{\Delta H}{ND * L_{min}}$

5) critical hydralic gradiant $(I_{cr}) = \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} = \frac{G_s - 1}{1 + e}$

that brings a soil ~~that~~ mass to static liquefaction ↳ $i = \frac{\Delta h}{L_{min}}$ → مقدار المفترض الحد
hearing boiling & piping $\Delta h = \frac{H}{ND}$

$i > i_{cr}$ → piping

6) F.S $= \frac{I_{critical}}{I_{max}}$ max ≥ 1.0 $i < i_{cr}$ → No piping
F.O.S.   ↳ safe

عند نقطة معينة

7) pressure head $h_{pj} = \Delta H - ND_j \times \frac{\Delta H}{ND} - h_z$

8) pore water pressure $u_j = [h_{pj} - h_z] * \gamma_w$ → from reference ~~lower level of~~ water

9) Uplift force $(p_w) = \sum_{j=1}^{n} u_j * \Delta x_j$

طول السد

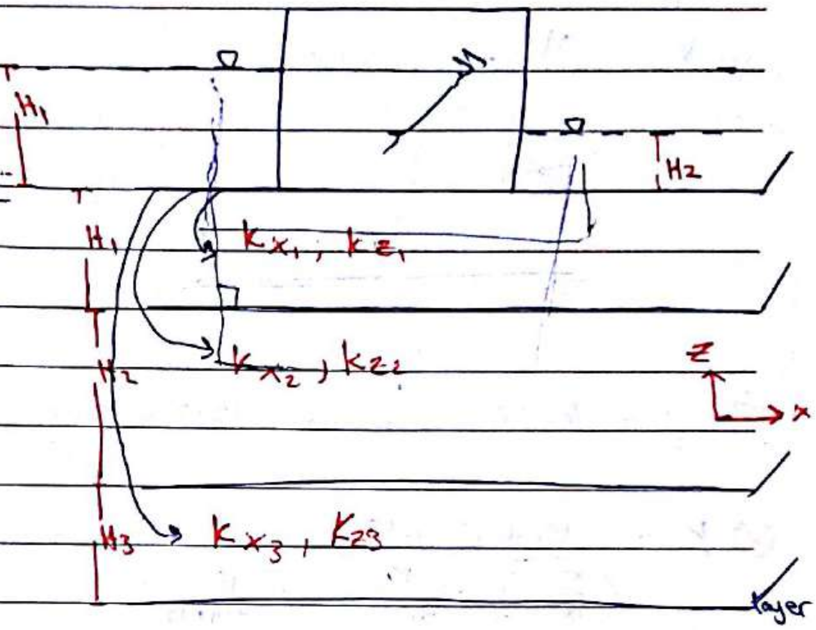
10) $p_w = \frac{\Delta x}{3} \left[u_1 + u_n + 2 \sum_{odd} u_i + 4 \sum_{even} u_i \right]$ ≤ wt. of dam

*** Anizotropic Soil Condition** $K_x \neq K_z$ تربة تختلف خواصها في جميع الاتجاهات

- $K_{avg} = \sqrt{K_z \times K_x}$

- $\bar{K}_x = \dfrac{H_1 K_{x_1} + H_2 K_{x_2} + H_3 K_{x_3} + \cdots}{H_1 + H_2 + H_3}$

- $\bar{K}_z = \dfrac{H_1 + H_2 + H_3 + \cdots}{\left(\frac{H_1}{K_{z_1}}\right) + \left(\frac{H_2}{K_{z_2}}\right) + \left(\frac{H_3}{K_{z_3}}\right)}$



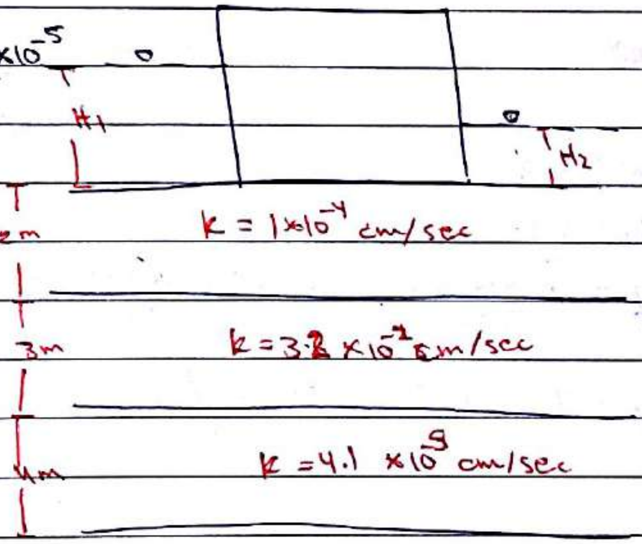
$\bar{K}_x = \dfrac{2 \times 1 \times 10^{-4} + 3 \times 3.2 \times 10^{-2} + 4 \times 4.1 \times 10^{-5}}{2 + 3 + 4}$

$\bar{K}_x = 7.07 \times 10^{-4}$ cm/sec.

$\bar{K}_z = \dfrac{2 + 3 + 4 \cdots}{\left(\frac{2}{1 \times 10^{-4}}\right) + \left(\frac{3}{3.2 \times 10^{-2}}\right) + \left(\frac{4}{4.1 \times 10^{-5}}\right)}$

$\bar{K}_z = 0.764 \times 10^{-4}$ cm/see

$K_{avg} = \sqrt{7.07 \times 10^{-4} + 0.764 \times 10^{-4}} = 9.0497 \times 10^{-4}$ cm/see

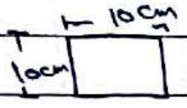


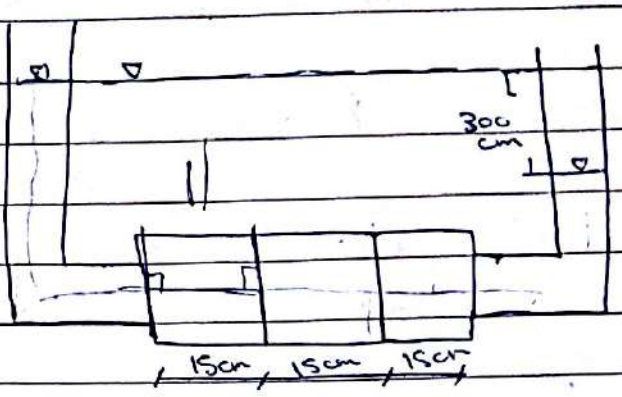
ex

$K_A = 1 \times 10^{-2}$ cm/sec

$k_B = 3 \times 10^{-3}$ cm/sec

$k_c = 4.9 \times 10^{-4}$ cm/sec





① $q = k i A \quad = 0.001213 \times \frac{300}{45} \times (10 \times 10) = 0.0809 \text{ cm}^3/\text{sec}$

② $k = \frac{15 + 15 + 15}{\left(\frac{15}{1 \times 10^{-2}}\right) + \left(\frac{15}{3 \times 10^{-3}}\right) + \left(\frac{15}{4.9 \times 10^{-4}}\right)} = 0.001213$ cm/sec

كيف عرفنا اتجاه الدفع

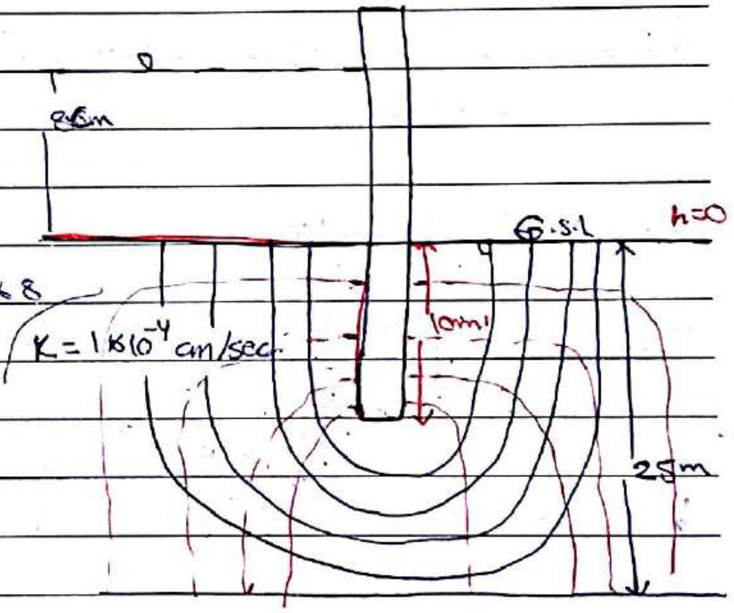
ex

$N_f = 8$

$N_D = 18$

$q = k \frac{\Delta H}{N_D} \times N_F = 1 \times 10^{-4} \times \frac{800\text{cm}}{18} \times 8$

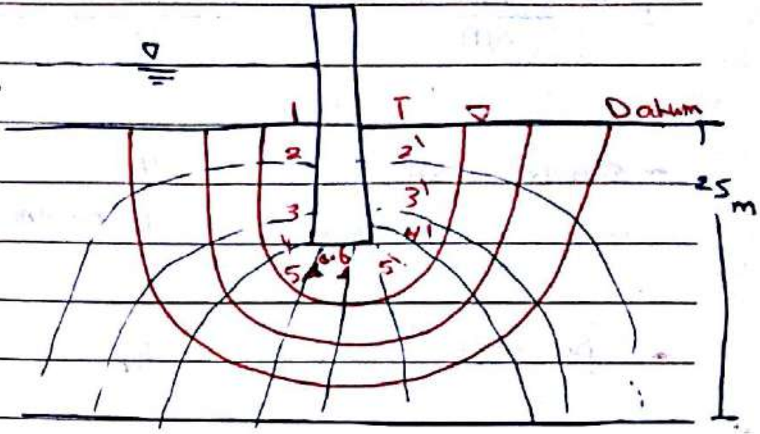
$q = 3.5 \times 10^{-4} \text{ cm}^3/\text{sec}/\text{m}$



28/2/201
الأحد

ex 1

$N_f = 8$
$N_D = 18$



$- i_{max} = \frac{\Delta h}{L_{min}} = \frac{\Delta H}{N_D L_{min}} = \frac{8}{18 \times 0.6}$

$= 0.74 < I_{cr}$

أصغر مربع [illegible] 0.5

$- P_w = \frac{\Delta x}{3} \left[u_1 + u_n + 2 \sum_{odd} + 4 \sum_{even} \right]$

$u = [h_p - h_z] \gamma_w$

$h_p = \Delta H - N_{Dj} \times \frac{\Delta H}{N_D} \boxed{- h_z}$

$u_1 = [8 - 0 - 0] \gamma_w = 8 \times 9.806 = 78.9$

$u_2 = [8 - \frac{1 \times 8}{18} - 2] \times 9.806 = 93.6$

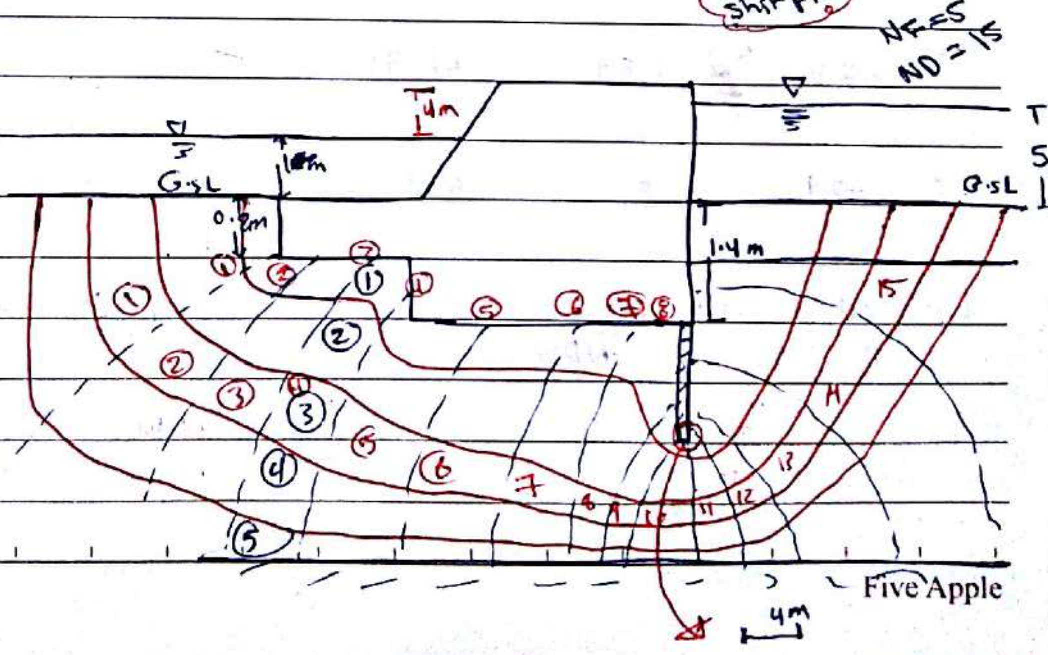
[illegible]

$P_w = \frac{10}{3} [78.4 +$

$u'_1 = [8 - \frac{18 \times 8}{18} - 0] \gamma_w = 0$

ex 2

$k = 2.5 \times 10^{-5}$ m/sec



Five Apple

$$q = K\frac{\Delta H}{ND} \cdot NF$$

$$- q = \frac{K\,\Delta H}{ND} \times NF = \frac{2.5\times10^{-5}\times 4\times 5\times 1}{15} = 3.1\times10^{-5}\ m^3/sec/m$$

$$- i_{max} = \frac{\Delta H}{ND\times L_{min}} = \frac{4}{15\times 0.4m} =$$

- pw = ?! hp

Point	hz	hp	$(hp-hz)\gamma_w$
1	-1.8	0.27	20.3
2	-1.8	0.53	$\boxed{22.9}$
3	-1.8	0.8	25.5
4	بالمسطرة -2.1	1.07	$\boxed{31.1}$
5	-2.4	1.33	36.6
6	-2.4	1.6	$\boxed{39.2}$
7	-2.4	1.87	41.9
8	-2.4	2	43.1

$$* \; pw = \frac{20}{3}\left[20.3 + 43.1 + 2(25.5+36.6+41.9) + 4(22.9+31.1+39.2)\right]$$

$$pw = 4292.3\ N$$

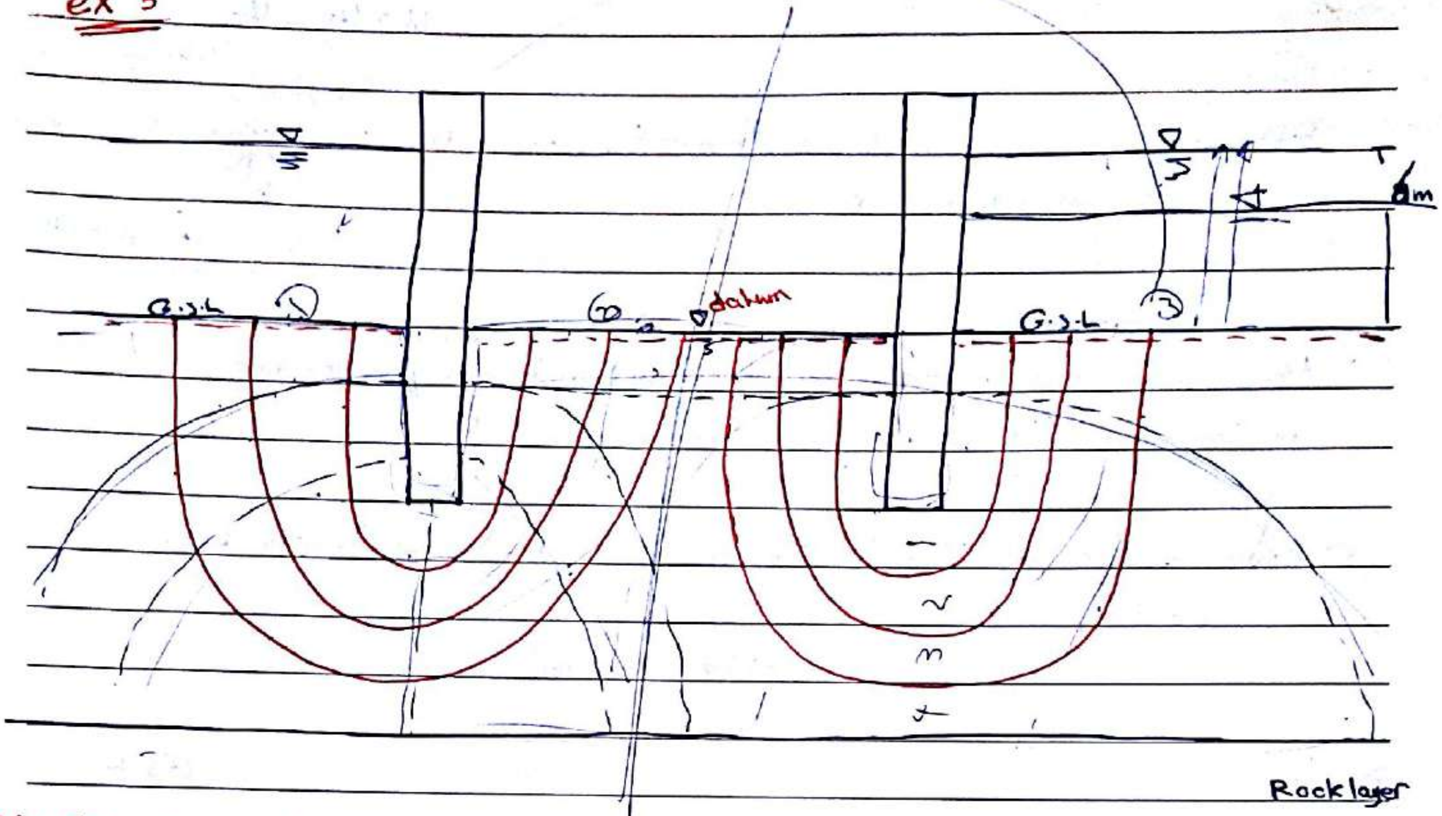
$$hp = \frac{\Delta H}{4} - ND_j \times \frac{\Delta H^{4}}{ND_{15}}$$

~~15~~
~~14~~
~~13~~
~~12~~
~~11~~
~~10~~
~~9~~
~~8~~
7.5

$q = q_1 + q_2$

$q = 2 \times q_1$

ex 3



No Flowline = 4

- $q_t = q_{t1} + q_{t2}$

$= \left(k \frac{\overset{8}{\Delta H}}{ND} \times NF\right)_1 + \left(k \frac{\overset{6}{\Delta H}}{ND} \times NF\right)_2$

- اختلاف level على الجهتين يؤدي الى اختلاف head pressure

2:45 – 3:45

(٢) c.١٤ c.١٩

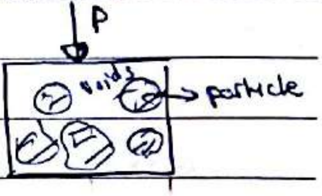
(١) c.٩ c.١٢

1/3/2016

$u = \underset{\gamma_w h}{\underline{u_s}} + u_e$ ↳ excess

يسببها ال load

① $\bar{\sigma}$ Effective stress: the force transmitted through the soil skeleton from particle to particle

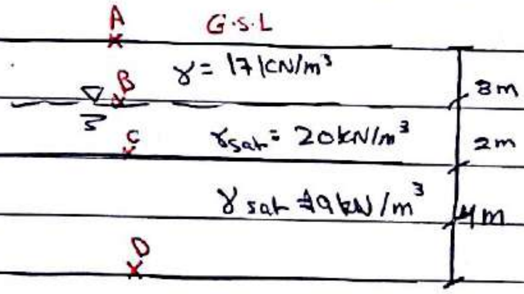


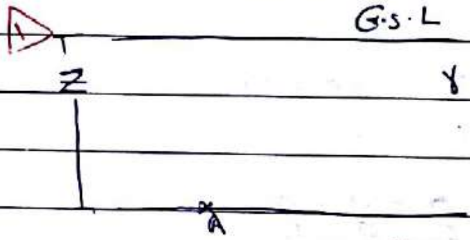
② the total normal stress :- Force per unit area transmitted in anormal direction across the plane (σ)

③ pore water press (u) : The pressure of the water filling the void space $\bar{\sigma} = \sigma - u$

↳ $\gamma * z$ = kN/m²

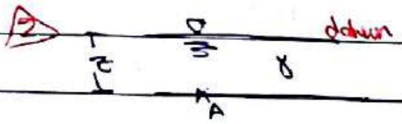
③ ex





$\sigma = \gamma * Z$

$u =$



Point	level	σ kN/m³	u kN/m³	$\bar{\sigma}$ kN/m²
A	0.0	0.0	0.0	0.0
B	3m	51	0	51
C	5m	91	19.6	71.4
D	9m	17*3 + 20*2 + 4*19 = 167	9.806*6 = 58.8	108.2

17*3 + 2*20

(γ_w * 2) 9.806

→ (17*3 + 2*(20-9.806)

↳ ((19-9.806)*4 + 2(20-9.806) + 17*3

$\sigma = \gamma * z$

$u = [h_p - h_z]\gamma_w$ 0

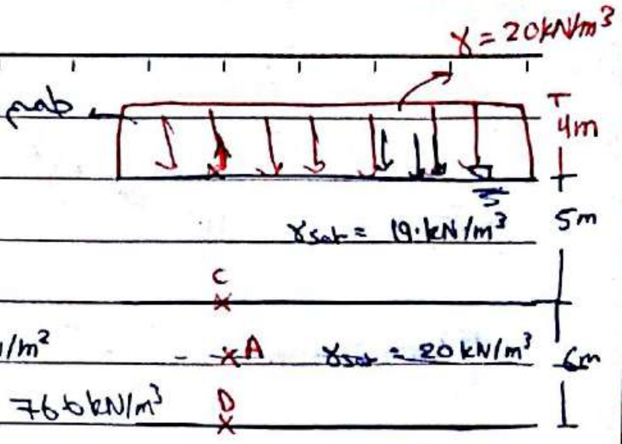
$= Z\gamma_w$

$\bar{\sigma} = [\gamma_{sat} Z - \gamma_w Z]$

$= Z \underbrace{[\gamma_{sat} - \gamma_w]}_{\gamma'}$

lower level في اذا تعمقنا pore water pressure ↑

ex



- $\bar{\sigma}_{A\ \text{before Backfill}} = \sigma_t - u = [$

$= [19*5 + \frac{6}{2}*20] - [8*9.806] = 76.6\ kN/m^2$

$= [(20-9.806)*3 + 5*(19-9.806)] = 76.6\ kN/m^3$

- $\bar{\sigma}_{A\ \text{after Backfill}} = [20*4 + 19*5 + 3*20] - [8*9.806] = 156.6\ kN/m^3$

$= \bar{\sigma}_{before} + 4*20 = 156.6\ kN/m^3.$

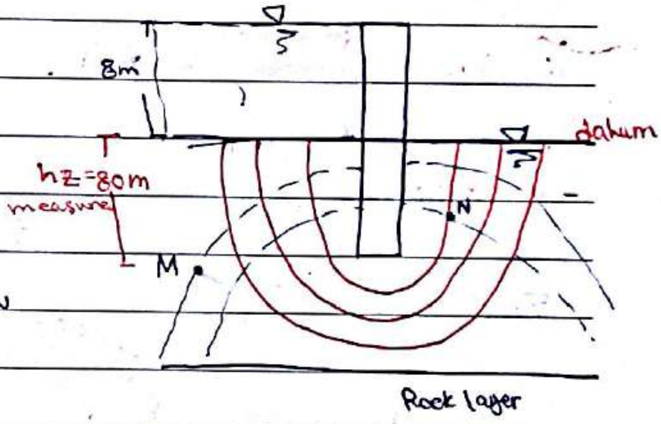
ex

* مثال سابق

$NF = 8 \qquad ND = 18$

$G_s = 2.7 \qquad e = 0.8$

- Find effective stress at point M, N



* $\bar{\sigma}_M = \sigma_t - u$

$= [8*9.806 + 8*\gamma_{sat}] - [h_p - h_z]\gamma_w$

$= 78.34\ kN/m^3$

$u = [h_p - h_z]\gamma_w = [\overset{\times 0}{\Delta H} + ND_M \frac{\overset{\times 0}{*\Delta H}}{ND} - \overset{16}{h_z}]\gamma_w$ ↳ lower level water

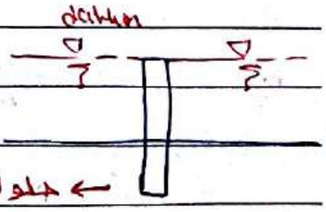
$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w$

$= [8 - \frac{1*8}{18} - {}^{-}8] \times 9.806$

$= \frac{2.7 + 0.8}{1.8} * 9.806$

$\gamma_{sat} = 19.06$

* $\bar{\sigma}_N = [\underset{\hookrightarrow + 8*9.806}{\gamma_{sat} * 7}] - [\overset{0}{8} - \frac{16*\overset{0}{8}}{18} - \underset{15}{{}^{-}7}] * 9.806$



حلوا فوقه الشطب ←.

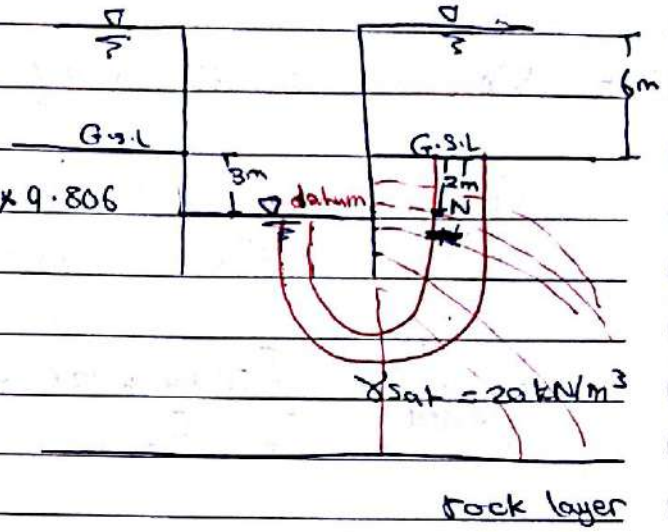
ex

$\bar{\sigma}_N = \sigma_t - u \sim (\Delta H - \frac{N_D \times \Delta H}{N_D} - h_z) * \gamma_w$

$= [6 \times 9.806 + 2 \times 20] - [9 - \frac{2 \times 9}{6} - 1] \times 9.806$

lower [illegible]

$\bar{\sigma}_n = 49.806$



3/3/2016

* stress ~~increment~~ increment from elastic solution.

ex

- point ①

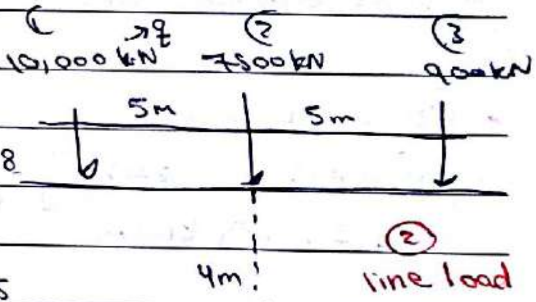
$\sigma_z = \frac{Q}{z^2} * I_p$

$f(\frac{0}{4}) = 0 \rightarrow I_p = 0.478$

r* [illegible] radius [illegible] horizontal

$I_p \rightarrow$ tables 5.1 , $f(\frac{r}{z}) \Rightarrow f(\frac{5}{4}) = 1.25$

$\Rightarrow I_p = 0.478$



② line load

Q/m

$\sigma_z = \sigma_{z_1} + \sigma_{z_2} + \sigma_{z_3} = \frac{10000}{(4)^2} \times 0.0450 + \frac{7500}{(4)^2} \times 0.478 + \frac{900}{(4)^2} \times 0.045$

العمق

$f(\frac{0}{4})$ $f(\frac{5}{4})$

$\sigma_z = 277\ kN/m^2$

- د Distribution لازم يكون Area

- نفس المثال بس بصير line load.

② $\sigma_z = \frac{2Q}{\pi} \left[\frac{z^3}{[x^2 + z^2]^2} \right]$

↳ load per meter

(لازم يكون [illegible] على Area)

- ~~قيمة load [illegible]~~ ⟵

$\sigma_z = \frac{2 \times 10000}{\pi} \left[\frac{4^3}{[5^2 + 4^2]^2} \right] + \frac{2 \times 7500}{\pi} \left[\frac{4^3}{[0^2 + 4^2]^2} \right] + \frac{2 \times 900}{\pi} \left[\frac{4^3}{(5^2 + 4^2)^2} \right]$

① & ② case, line

-كيف ينتقل load إلى السويل .

3-3-2016

③ strip area, carrying ~~unit form~~ uniform pressure.

* حسبة مساحات footing

* σ_t من G.S

* pore water pressure من lower level

يمثل line على قاعدة الجدار الاستنادي

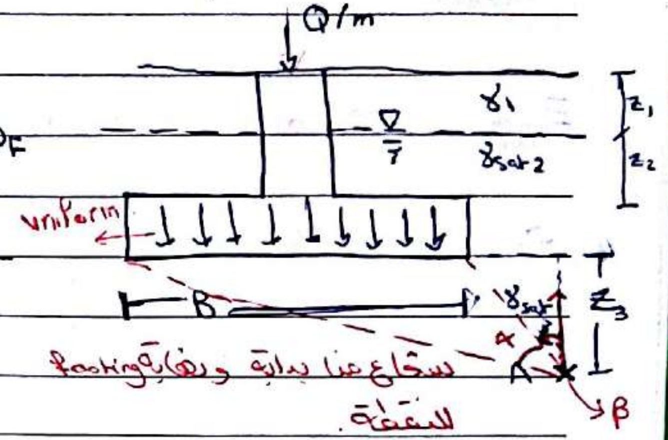
$$-\ \bar{\sigma}_A = \sigma_t - u = ,$$

$$[\gamma_1 Z_1 + \gamma_{2sat} Z_2 + \gamma_{3sat} Z_3] - [9.806 \times (Z_3 + Z_2)]$$

$$-\ \bar{\sigma}_z = \frac{q}{\pi}\left[\alpha + \sin\alpha \cos(\alpha + 2\beta)\right]$$

$\alpha \to$ rad $\quad \frac{180 \to \pi}{? \to ?} \quad \times \pi/180$

$$q = \frac{Q}{B \times 1} = \text{Applied pressure}$$



From + vertical direction toward end array counter Clockwise.

strip:- القاعدة تحت الجدران وعادة طولها كبيرة

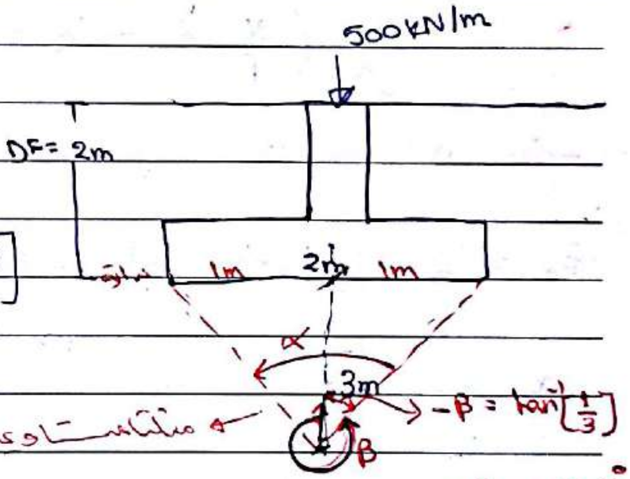
(أكبر من 10 أضعاف العرض load يمثل concentrated مركز)

ex

$$36.8 \to \frac{\pi\, 36.8}{180} = 0.64$$

$$-\ \bar{\sigma}_z = \frac{500}{\frac{2\times1}{\pi}}\left[0.64 + \sin 36.8 \cos(36 - 2\times18.4)\right]$$

$$\bar{\sigma}_z = 99.0\ kN/m^2$$



$-\beta = \tan^{-1}\left[\frac{1}{3}\right]$

$\Rightarrow \beta = -18.4°$

$\therefore\ \alpha = |2\beta| = 2 \times 18.4°$

$= 36.8$

* α , β بعد الشعاع (array) من بداية footing لعند نقطة A وشعاع آخر من نهاية footing لعند نقطة A . (A) نقطة اللي رح احسب عندها stress فتكونوا الزاوية α بالراديان أما β من +ve vertical direction باتجاه ال end array ← C.C.W

8/3/2016

ex $\sigma_{z_A} = \frac{250}{\pi}\left[0.59 + \sin(33.7)\cos(33.7 \pm 0)\right]$

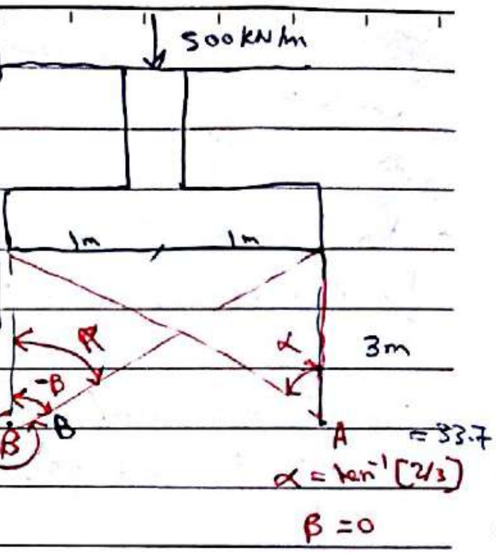
$\sigma_{z Ⓐ} = 83.68 \, kN/m^2$

$\bar{\sigma}_{z Ⓑ} = \frac{250}{\pi}\left[0.59 + \sin(33.7)\cos(33.7 - 2 \times 33.7)\right]$

$\alpha = |\beta|$

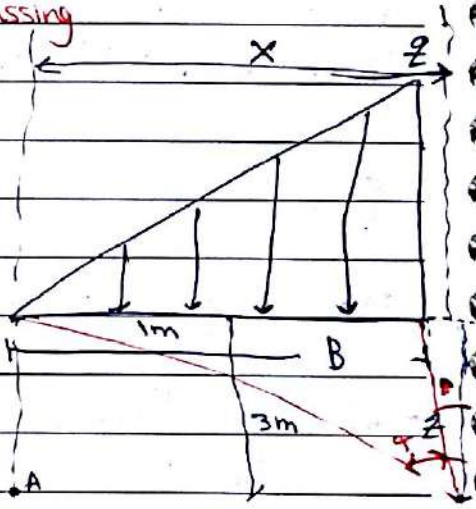
$\sigma_{z Ⓑ} = 83.68 \, kN/m^2$

$B = \tan^{-1}(2/3)$



Strip area carrying linearly increasing pressure

لو بدي رأس قاعدة شكلها لساحة مشكلة slope .



* $\bar{\sigma}_z = \frac{q}{\pi}\left(\frac{X}{B}\overset{rad}{\alpha} - \frac{1}{2}\sin 2\beta\right)$

- نفس المثال السابق -

$q = 250.$

$\bar{\sigma}_z = \frac{250}{\pi}\left[\frac{1}{2} \times 0.64 , - \frac{1}{2}\sin(2 \times -18.9)\right)$

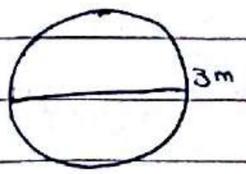
x: بعد النقطة اللي بدي احسب عليها stress من البداية أفقياً. - البداية من الشمال.

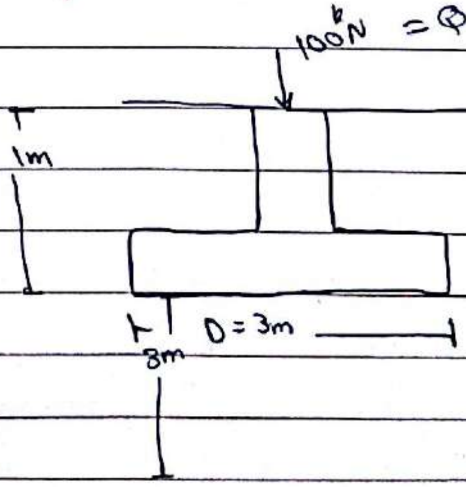
* circular footing carrying uniform pressure.

$\sigma_z = q \times I_c$

$I_c \rightarrow$ figure 5.9, $f\left(\frac{\overset{\text{Diameter}}{D}}{z}\right)$

$q = \frac{Q}{\pi/4 \, D^2}$





6/3/2016

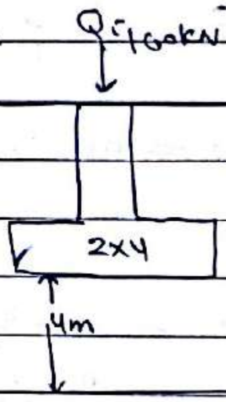
- $\sigma_z = \frac{100}{\frac{\pi}{4}(3)^2} * 0.26 = 3.68$

(هالشكل مستطيل ، نقطة ↑ corner الـ تحت).

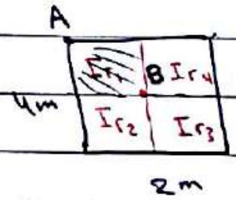
* Rectangular area carrying uniform pressure:-

$q = \frac{load}{Area} = \frac{Q}{A}$

$\sigma_z = q * I_r$



- $\sigma_{z_A} = q * I_r = \left[\frac{100}{4*2}\right] I_r$



$= \frac{100}{4*2} * 0.12$

Profile.

- $I_r \Rightarrow m = \frac{2}{4} = 0.5$

$n = \frac{4}{4} = 1 \Rightarrow$ fig 5.1 $= 0.12$

$m = \frac{B}{Z}$ → width (smallest Dimension)

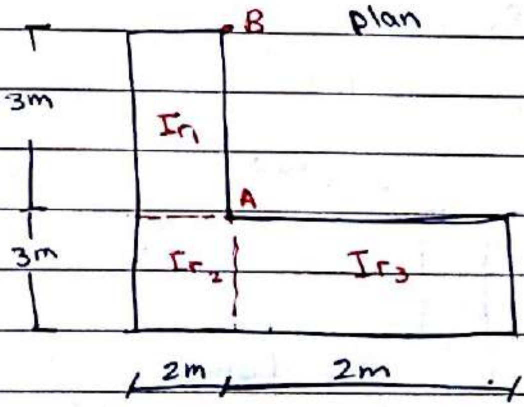
$n = \frac{L}{Z}$ → length → (large Dimension)

* $\sigma_{z_B} = \left[\frac{100}{4*2}\right]\left[I_{r_1} + I_{r_2} + I_{r_3} + I_{r_4}\right] = 4 * 0.06 * \left[\frac{100}{2*4}\right]$

$I_{r_1} \Rightarrow m = \frac{1}{4} = 0.25$

$n = \frac{2}{4} = 0.5 \Rightarrow 0.06$

eX2



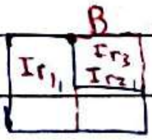
$\sigma_{zA} = \frac{100}{(3\times4)+(3*2)} * \left[I_{r_1} + I_{r_2} + I_{r_3}\right]$

$I_{r_1} = m = \frac{2}{4}$

$n = \frac{3}{4}$

$I_{r_2} \Rightarrow m \frac{2}{4}$, $I_{r_3} \Rightarrow m = \frac{2}{4}$

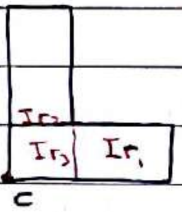
$n_1 = \frac{3}{4}$ $n = \frac{3}{4}$



* $\sigma_{zB} = \frac{100}{3*4 + 3*2} * [I_{r_1} + I_{r_2} +^{-} I_{r_3}]$

$I_{r_1} \Rightarrow m = \frac{2}{4}$, $n = \frac{6}{4}$, $I_{r_2} \Rightarrow m = \frac{2}{4}$, $n = \frac{6}{4}$, $I_{r_3} \Rightarrow m = \frac{2}{4}$, $n = \frac{3}{4}$

* $\sigma_{zc} = q\ [I_{r_1} + I_{r_2} - I_{r_3}]$



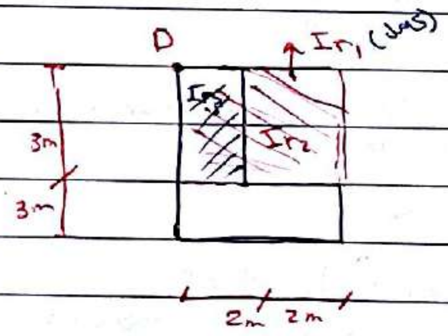
$I_{r_1} \Rightarrow m = \frac{4}{4}$

$n = \frac{3}{4}$ → figure 5.1

$I_{r_2} \to m = \frac{2}{4}$, $n = \frac{6}{4}$, $I_{r_3} \to m = \frac{2}{4}$, $n = \frac{3}{4}$

* كل Ir لازم يكون corner للنقطة.

* $I_{r_1} \to m = \frac{4}{4}$, $n = \frac{6}{4}$

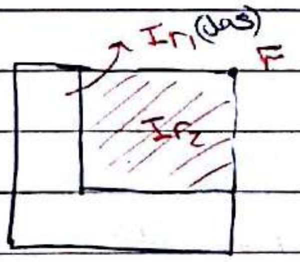


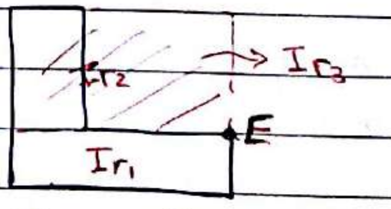
$I_{r_2} \to m = \frac{4}{4}$, $n = \frac{3}{4}$ → figure 5.1

$\sigma_D = \frac{100}{3*4 + 3*2} * [I_{r_1} - I_{r_2} + I_{r_3}]$

$I_{r_3} \to m = \frac{2}{4}$

$n = 3/4$

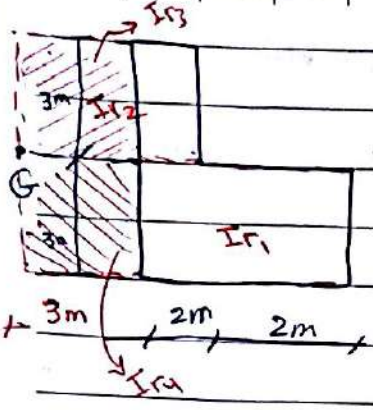




$I_{r_1} - I_{r_2}$

$I_{r_1} + I_{r_2} - I_{r_3}$

*ملاحظة stress لا تأخذ مساحات.



$Ir_1 \longrightarrow m = \frac{7}{4}$

$n = \frac{3}{4}$

$Ir_2 \Rightarrow m = 5/4$

$n = 3/4$

$$\sigma_{z_G} = \frac{100}{3*4 + 3*2} * \left[Ir_1 + Ir_2 - Ir_3 - Ir_4 \right]$$

$Ir_3 \Rightarrow m = \frac{3}{4} \Rightarrow 1$

$n = \frac{3}{4} \Rightarrow 1$

$Ir_4 \Rightarrow m = 3/3 \Rightarrow 1$

$n = 3/4 \Rightarrow 1$

* Approximate method 2:1 method.

Just for :-

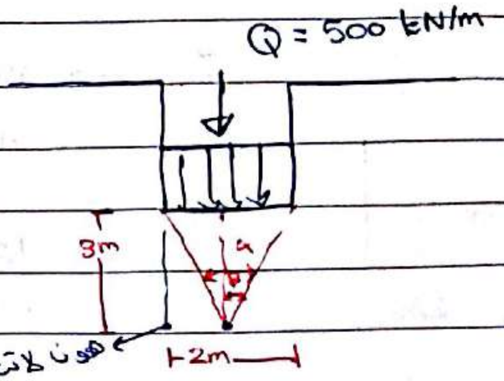
① strip footing , Rectangular footing
② under center of footings

$Z \leftarrow$ عمق من تحت footing

$$\sigma_z = q * A = \frac{q * [B * 1]}{[B + Z]} \text{ ---- strip}$$

$$\sigma_z = q * A = \frac{q * [B * L]}{[B + Z][L + Z]}$$

ex

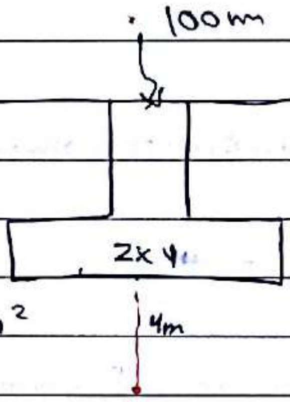


$\sigma_z = 99\ kN/m^3$ → more accurate

σ_z Approximate method =

$$\frac{[500/2\times1]\times[2\times1]}{[2+3]} = \frac{500}{5} = 100\ kN.$$

ex



$$\sigma_z = \frac{100}{[2\times4]} * 4 * 0.06 = 3\ kN/m^2$$

$$\sigma_z = \frac{\frac{100}{2*4} * [2*4]}{[2+4][4+4]} = \frac{100}{6*8} = 2.1\ kN/m^2$$

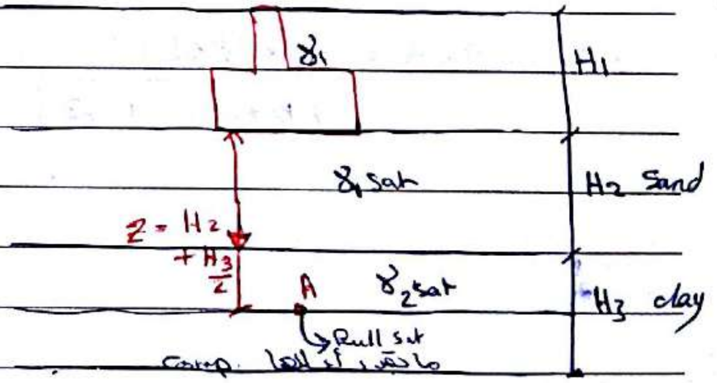
→ ground surface

* σ نحسبه من G.S

$\bar{\sigma}_z$ نحسبه من تحت footing (σ ناتجة من منشأتي].

u نحسبه من w.t ← water table

ex



$\bar{\sigma}_A = \sigma_t - u$

$$= \left[\gamma_1 H_1 + \gamma_{1sat} H_2 + \gamma_{2sat} \frac{H_3}{2}\right]$$

$$- \left[\gamma_w \times \left[H_2 + \frac{H_3}{2}\right]\right]$$

$\sigma_z = q * I_r \rightarrow 4 I_{r_1}$

z من تحت footing

A* لاقية σ_z & $\bar{\sigma}$

8/3/2016

ex sand الها permeability أعلى.

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

w
Solid

Consolidation Theory

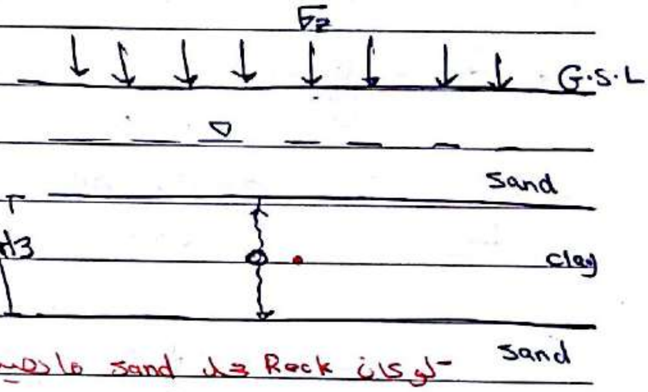
① نقصان بـ V لل voids $V_v = V_w$

is the grandual reduction in avolume of afully saturated soil of low permeability due to drainge of some of the pore water
(fine grained soil) تصريف ③

لـ تحرك نتيجة σ_z

* drainge يعني وجود طبقة الها أعلى permeability تسمح بتحرك الـ water

① at time t = 0 (immediately after stress is applied). the excess pore water pressure. تجاوز



total — water — ← كل الـ stress توزع على

$\bar{\sigma} = \sigma_z - \Delta u_e \Rightarrow \bar{\sigma} = 0$

- لو كانت موجودة clay وتحتها مافي drainge لا توجد طبقة أعلى permeability.

- لو كان Rock بدل sand ما بصير فيه reduction.

② ~~at time~~ Hence, increase in effective stress at any time, t

$\uparrow \bar{\sigma} = \sigma_z - \Delta u_e \downarrow$

ثابت.

③ theoritical, at time $t = \infty$

$\bar{\sigma} = \sigma_z$, $\overset{excess}{pore} = 0.0$ * هون مافي فائدة الهبوط settelment.

لـ لما كل الـ stress صار على الـ soil skelton مارح يكون في reduction.

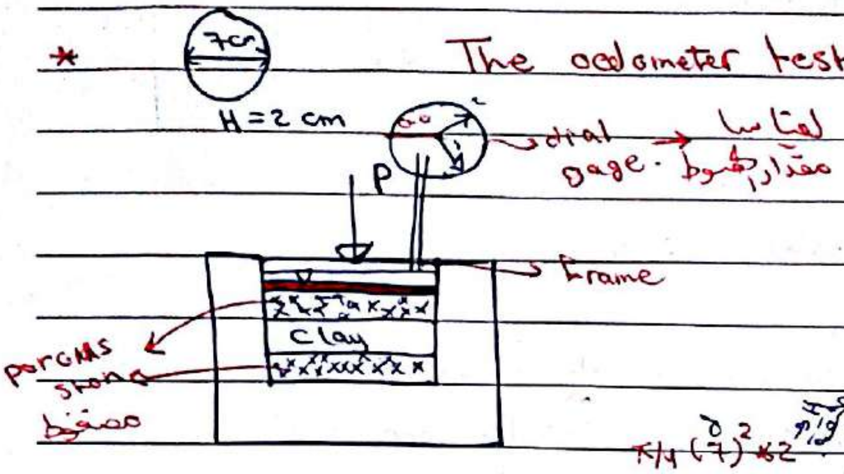
- This gradual increase in the effective stress in the clay layer will cause ① settlement over aperiod of ② time, and is referred to aconsolidation.

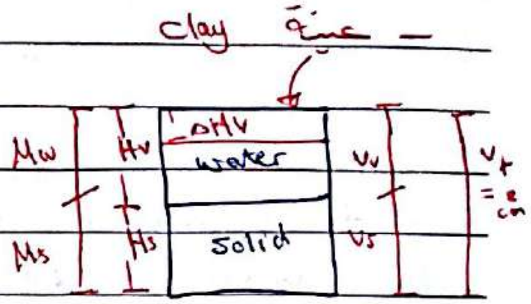
- الماء اجا عليها stress وزع بحملها pressure ، هون particle اذا توزعها pressure رح تتحرك . [excess pore water pressure] بسبب هاي نتيجة load

- اللي همنا هون الشابق كم مقدار هبوط؟ وفتى بتوقف الـ settelment ؟؟
لـ عندما $\bar{\sigma} = \sigma_z$

theory → lab → Field.

* The oedometer test :-





$\pi/4 (7)^2 \times 2$ الحجم الكلي

$$e_0 = \frac{V_v}{\underset{\text{ثابت}}{V_s}} = \frac{V_v}{V_s} = \frac{V_T - V_s}{V_s}$$

initial ← اذا وضعت load لا يقل

V_s ← من Gs

$$e = \frac{V_v}{\boxed{V_s}} \to \text{Const.}$$

- $G_s = \frac{M_s \to \text{const.}}{\boxed{V_s}\, \rho_w}$
- $G_s w = \overset{1}{\cancel{S_r}}\, e$ ↳ full saturated

- $\dfrac{\bar{\sigma}}{\dfrac{A}{\frac{\pi}{4}(7)^2}} = \bar{\sigma}_1$
- $e = e_0 - \Delta e$ ← مقدار التغير منه load

$$e_0 - \frac{\Delta V_v}{V_s} = e_0 - \frac{\cancel{A} \times \Delta H_v}{\cancel{A} \times H_s}$$

ΔH_v → الارتفاع الوحيد الي بتغير

H_s ↳ height for solid particle

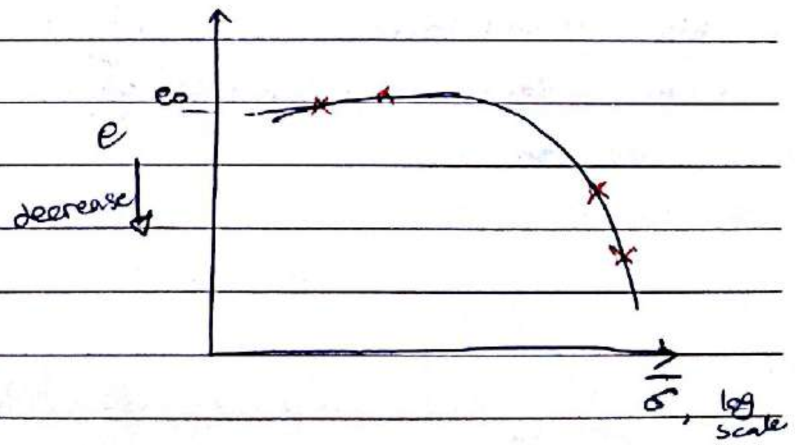
$$e_1 = e_0 - \frac{\Delta H_v}{H_s}$$

ΔH_v ~ dial gage. قراءة

$$\boxed{V_s = A \times H_s}$$

- $\frac{2P_1}{A} = \frac{P_2}{\frac{\pi}{4}(7)^2} = \bar{\sigma}_2$ $\qquad e_2 = e_0 - \frac{\Delta H_{v_2}}{H_s}$

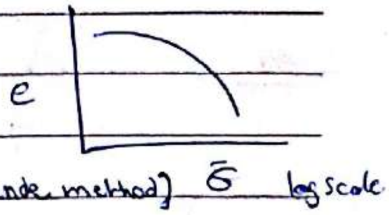
- $\frac{P_3}{A} = \frac{2P_2}{A} = \bar{\sigma}_3$ $\qquad e_3 = e_0 - \frac{\Delta H_{v3}}{H_s}$



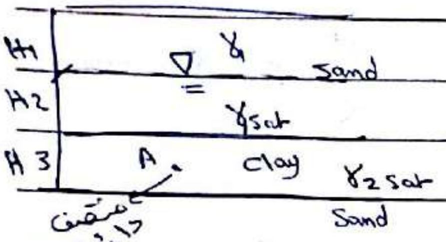
13/3/2016

① coefficient of volume compressibility (mv)

The volume change per unit volume per unit in increase in effective stress (m^2/MN) $m = \frac{1}{1+e_0}\left[\frac{e_0 - e_1}{\bar{\sigma}_1 - \bar{\sigma}_0}\right]$



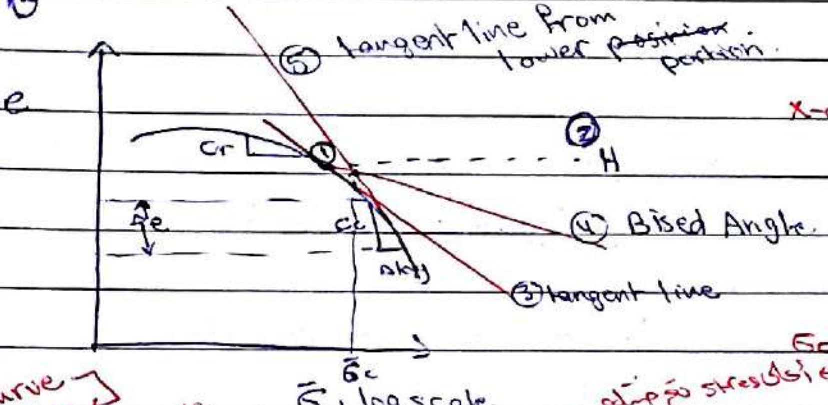
② pre consolidation pressure ($\bar{\sigma}_c$, $\bar{\sigma}_p$) The Maximum pressure acted on the clay layer in Past [from casagrande method]



$\bar{\sigma}_A = \left[\gamma H_1 + \gamma_1 H_2 + \gamma_{2sat}\frac{H_3}{2}\right] - \left[\gamma_w \times \left(\frac{H_3}{2} + H_2\right)\right]$

منتصف دائماً منأخذ الأوفق

* Compresability هو التغير في معدل voids ratio



① باجي آخذ نقطة على curve

② برسم خط أفقي يوازي x-axis

③ من point بعمل tangent line.

④ صار عندي زاوية نقوم بتنصيفها

⑤ من lower portion بعمل extend

⑥ رح يتقاطع مع نقطة بكون σc

curve بكون حوز

linear بيجد curve

ل اذا soil تعرضلها stress

- أقل قيمة لل $\bar{\sigma}$ هو $\bar{\sigma}_{vo}$

③ over consolidation ratio (OCR)

أقل قيمة ال

$OCR = \frac{\bar{\sigma}_c}{\bar{\sigma}_{vo}}$ ⟹ OCR = 1 ⟹ $\bar{\sigma}_{vo} = \bar{\sigma}_c$ Normally consolidation clay (N.C)

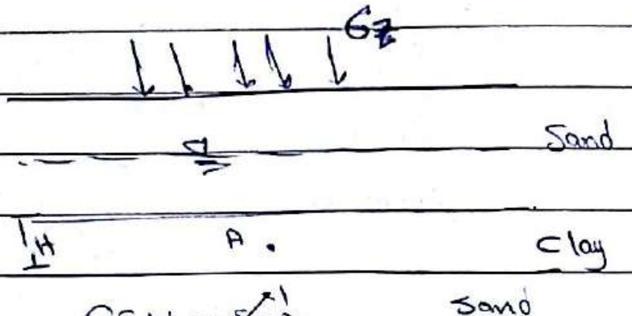
O.C > 1 ⟹ $\bar{\sigma}_c > \bar{\sigma}_{vo}$ -- over consolidation clay (O.C)

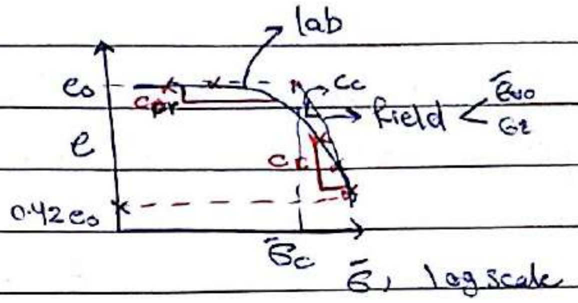
④ compression index $C_c = \left|\frac{\Delta e}{\log \bar{\sigma}_1 / \bar{\sigma}_2}\right|$ * Slope of lower portion

↱ slope for upper portion.

⑤ recomposition index or swelling index $C_{cr} = C_s = \left| \frac{\Delta e}{\log \frac{\bar\sigma_2}{\bar\sigma_1}} \right|$

ex

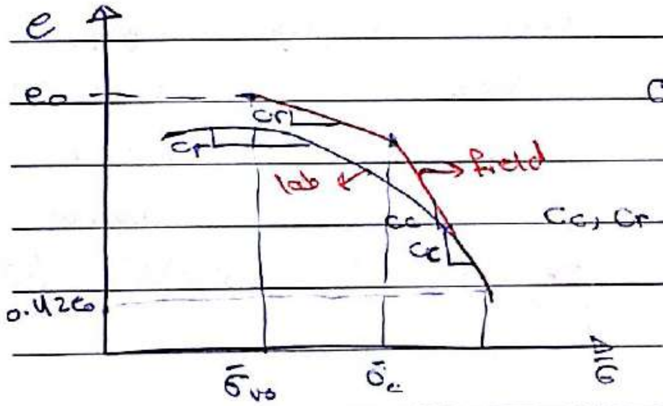




$G_s w = \overset{1}{\cancel{S_r}} e_o$
measured

- IF soil N.C clay OCR=1.0 ⟶ $\bar\sigma_c = \bar\sigma_{vo}$

$\bar\sigma_{vo}, e_o, C_c$ مشتركة بين اللاب والموقع ×.   مثلاً :- $e_o = 0.35$ ← أول عينة.
$0.42e_o$

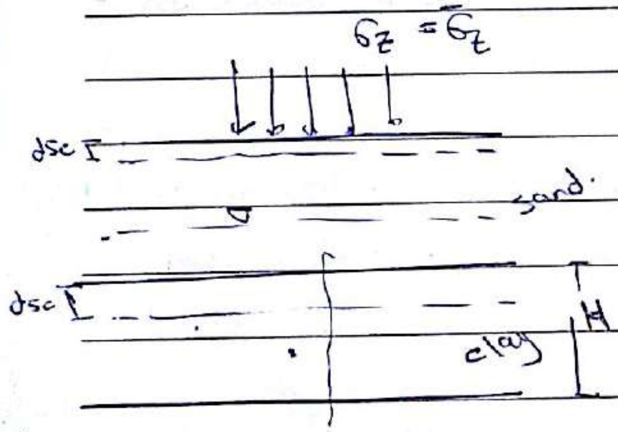


O.C clay

OCR > 1 ⟹ $\bar\sigma_c \geq \bar\sigma_{vo}$

Cc, Cr

Cr , Cc → مشتركة بين اللاب والموقع

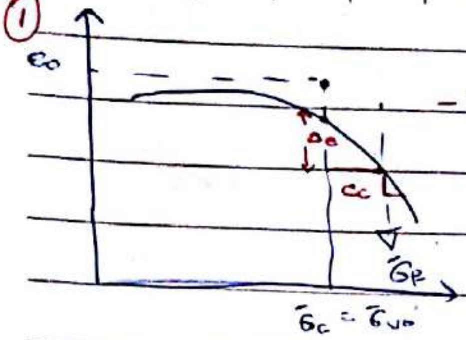


$\int dsc = \frac{\Delta V}{V_o} = \int_0^H \frac{\Delta e}{1+e_o} dz$

$Sc = \frac{\overset{؟؟}{\Delta e}}{1+e_o} H$ → سماكة clay layer

final ← $\bar\sigma_f = \bar\sigma_{vo} + \bar\sigma_z$

~~intiate~~ $\bar\sigma_{vo}$
initial

① 

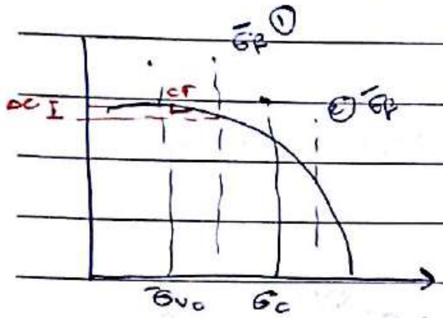
- NC clay (OCR = 1)

Δ ج3. $\bar{\sigma}_{vo}$ $\bar{\sigma}_p$?

$$C_c = \frac{\Delta e}{\log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}}}$$

$$\Rightarrow S_c = \frac{C_c H}{1+e_0} \log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}} \quad \text{--- NC clay.}$$

② OCR > 1 … over consolidation



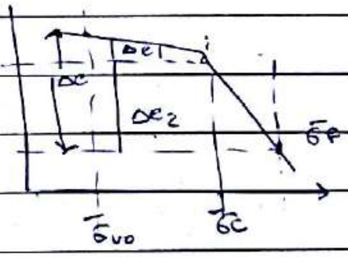
$$\bar{\sigma}_p = \bar{\sigma}_{vo} + \bar{\sigma}_z$$

$$C_r = \frac{\Delta e}{\log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}}}$$

(i) $\bar{\sigma}_{vo} < \bar{\sigma}_p < \bar{\sigma}_c$, $S_c = \frac{\Delta e\, H}{1+e_0}$ ~~$\log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}}$~~ ~~NC~~ clay

$$S \quad \frac{C_r H}{1+e_0} \log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}}$$

(ii) $\bar{\sigma}_{vo} < \bar{\sigma}_c < \bar{\sigma}_p$



$$S_c = \frac{\Delta e\, H}{1+e_0} = \left[\frac{C_r H}{1+e_0} \log \frac{\bar{\sigma}_c}{\bar{\sigma}_{vo}} + \frac{C_c H}{1+e_0} \log \frac{\bar{\sigma}_p}{\bar{\sigma}_c}\right]$$

انتبه ← (σc/σvo)

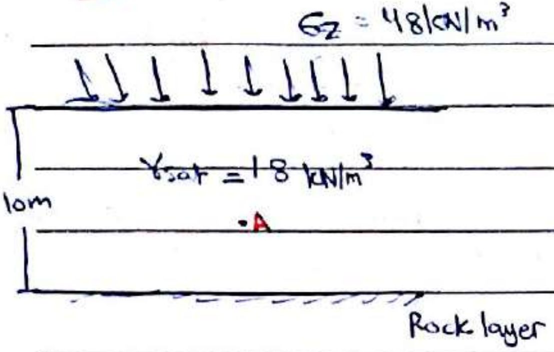
انتبه ← (σc)

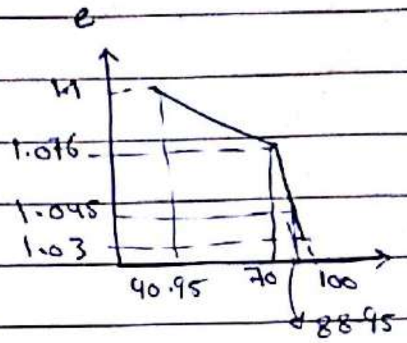
يكون هبوط أكبر لأن تعرض لـ stress أعلى من σc

$$\Delta e = \Delta e_1 + \Delta e_2$$

- في حالة ~~OC~~ N.C بكونا الهبوط أكبر. ~~([illegible])~~.
- مادام في زيادة ب ال stress مادام في تغير ب volume وكلما زاد stress كلما زاد التغير في voids ratio يعني صار voids ratio قل.

ex





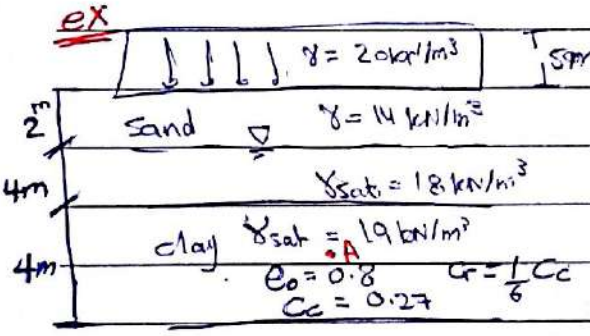
- Find primary consolidation Settlement.

$$S_c = \frac{\Delta e}{1+e_0} H = \frac{1.1 - 1.045}{1+1.1} \times 10 m = 0.262 m = 262 mm$$

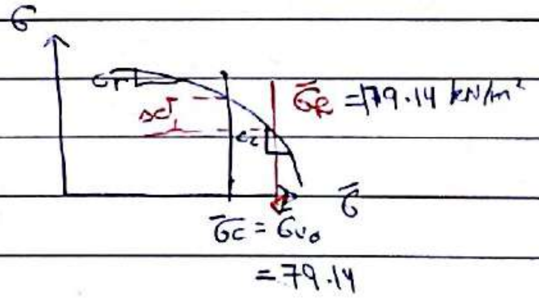
$$\bar{\sigma}_{v_0} = \sigma_T - u = \frac{10}{2}[18] - \frac{10}{2}[9.806] = 40.95 \overset{kPa}{\longrightarrow} e_0 = 1.10$$

$$\bar{\sigma}_p = \bar{\sigma}_{v_0} + \bar{\sigma}_z = 40.95 + 48 = 88.95 \Rightarrow e = 1.045$$

ex



* Find primary consolidation settlement



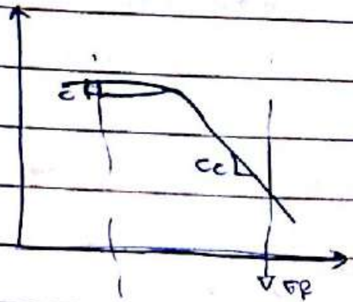
① IF OCR = 1

$$\bar{\sigma}_{v_0} = \sigma_T - u = [14 \times 2 + 18 \times 4 + \frac{4}{2} \times 19] - [6 \times 9.806] = 79.14 kN/m^2$$

$$\bar{\sigma}_p = \bar{\sigma}_{v_0} + \bar{\sigma}_z = 79.14 + 5 \times 20 = 179.14 kN/m^2$$

$$\therefore S_c = \frac{\Delta e}{1+e_0} H = \frac{0.27 \times 4}{1+0.8} \log \frac{179.14}{79.14} = 0.213 m = 213 mm$$

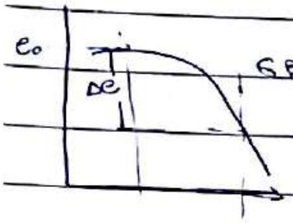
② IF preconsollidation pressure = 190 kN / m² $= \bar{\sigma}_c$



$$S_c = \frac{\Delta e}{1+e_0} H = \frac{(1/6) * 0.27 * 4}{1+0.8} \log \frac{179.14}{79.14}$$

$$S_c = 0.036\ m = 36mm$$

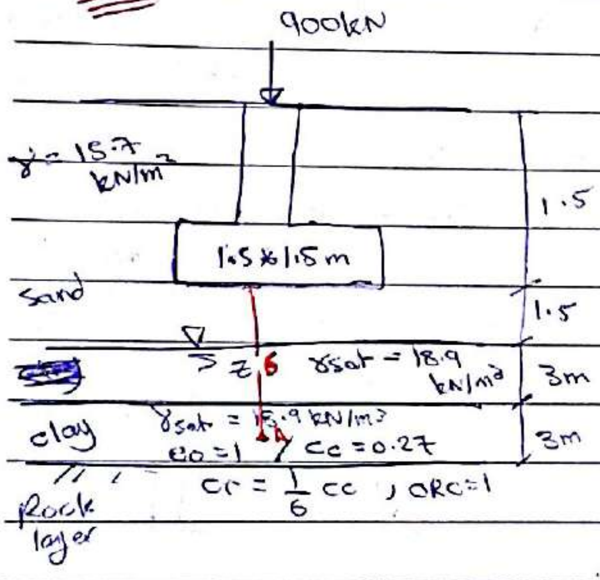
③ IF preconsolidation pressure = 170 kN/m²



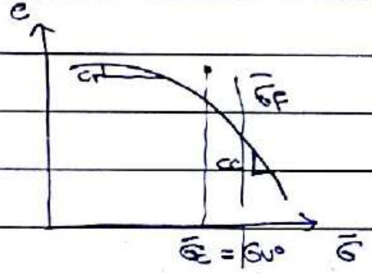
$$S_c = \frac{\Delta e}{1+e} H = \frac{4 \times (1/6) * 0.27}{1+0.8} \log \frac{170}{79.14}$$

$$+ \frac{4.0 \times 0.27}{1+0.8} \log \frac{179.14}{170} = 0.0486 = 48.6mm$$

④ ex



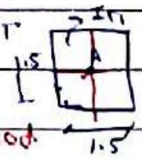
$$S_c = \frac{\Delta e}{1+e_0} H$$



ناخذ منتصف σ' avg.

$$\sigma_{v0} = \sigma_t - u = [15.7*3 + 3*18.9 + 1.5*18.9] - [4.5*9.806] = 88.01\ kN/m^2$$

$$\bar{\sigma}_p = \bar{\sigma}_{v0} + \bar{\sigma}_z = 88.01 + 12 = 100.01 \qquad \bar{\sigma}_z = q * I_r$$



$$\bar{\sigma}_z = q * 4 I_{r_1} = \frac{900}{1.5*1.5} * 4 *$$

Approxomate method

$$\bar{\sigma}_z = \frac{900}{(1.5+6)(1.5+6)}$$

$$I_{r_1} \Rightarrow m = \frac{B}{Z} = \frac{0.75}{6} \longrightarrow I_r = 0.0075$$

$$n = \frac{L}{Z} = \frac{0.75}{6}$$

Figure 5.1

①

17/3/2016

$\sigma_p = \bar{\sigma}_{vo} + \bar{\sigma}_z$

$$S_c = \frac{\Delta e}{1+e_o} H = \frac{\overset{\overset{\times}{H}}{3} \times \overset{\overset{\times}{C_c}}{0.27}}{1+\underset{\times}{1_{e_o}}} \log \frac{100.01}{\underset{\underset{\times}{\sigma_{vo}}}{88.01}} = 22.5\ mm$$

كل الي عليهم اشارة (x) ما بتقدر تغيرهم .

$\cancel{\times}\ q \times I_r = \sigma_z$

$q \to \frac{Q}{A}$ x

A زادت ← q بتقل ← فال σ_z بتقل و Sc بتقل .

ما لازم يزيد 25mm = 1in

Secondary Compression settlement.

بيلش بعد انتهاء primary

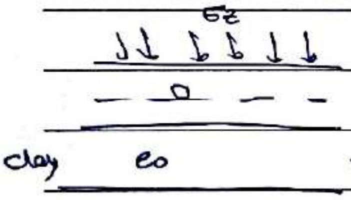
x تغير خلال effective stress بعد primary

$$\overset{primary}{S_c} = \frac{\overset{\bar{\sigma}_{vo} \to \bar{\sigma}_p}{\Delta e}}{1+e_o} H \quad , \quad S_s = \frac{\overset{f(t)}{\Delta e}\ H}{1+e_o}$$

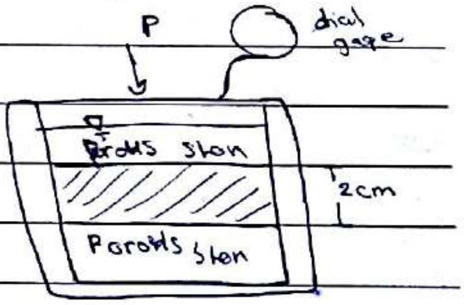
t_p : end of primary consolidation

→ t ??

e_p : void ratio at end of primary $= \underset{\text{initial void ratio}}{e_o} - \underset{\text{primary consolidation.}}{\Delta e}$



$e_p = e_o - \Delta e$
$= e_o - \frac{\Delta H_v}{H_s}$



time	dial gage reading	e
1/2	d_1	$e_o - \frac{\boxed{H_v}}{H_s}$ → dial gage

$\bar{\sigma} = \frac{P}{A} = e$

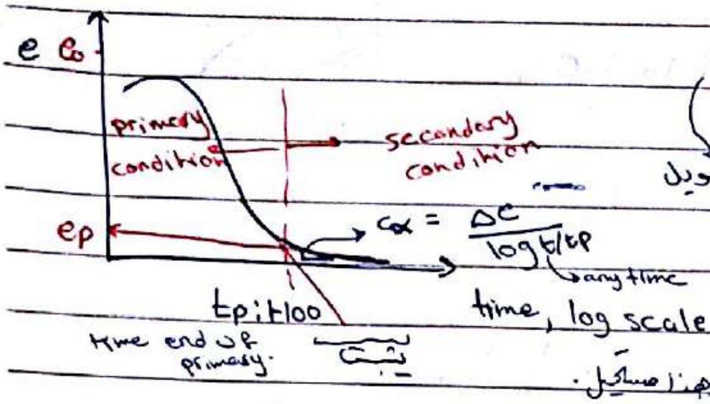
$e_o - \frac{\boxed{H_v}}{H_s}$ → dial gage reading.

x e بنحسب نقل كل ما يتغير primary $\sigma = \bar{\sigma}$

②

17/3/2016

* شت لما excess pore water = صفر، لما كل total6 طاقة $\bar{\sigma}$



$C_\alpha = \dfrac{\Delta e}{\log t/t_p}$

t100 : time of end of primary

tp

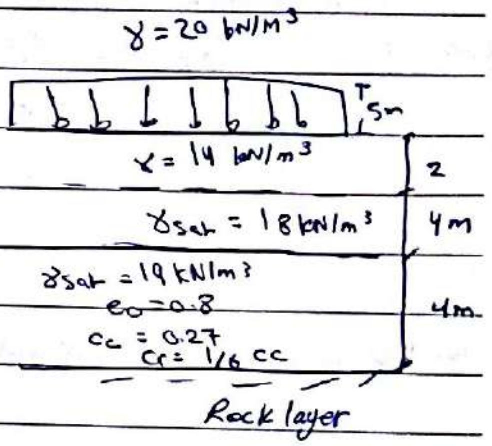
خاصية لسويل

Cα : secondary compression index

- كل ما زاد الزمن تقل e

- مستحيل يوصل صفر انو ما يكون voids بالمرة وهذا مستحيل.

$\therefore S_s = \dfrac{C_\alpha H}{1+e_p} \log \dfrac{t}{t_p}$



ex

Assume that primary consolidation will be completly in 3.5 years. Estimate secondary compression settlement after 10 years. → Cα → from lab = 0.022

① IF OCR = 1 → Sc = 213 mm

② IF $\bar{\sigma}_c$ = 190 kN/m³ → Sc = 36 mm

③ IF $\bar{\sigma}_c$ = 170 kN/m³ → Sc = 48.6 mm

$\bar{\sigma}_{v0}$ = 79.14 kN/m³ , $\bar{\sigma}_F$ = 179.14 kN/m³

* ما بقدر أهمل أي نوع سترسش

اذا كان organic soil لانو settle بتغير

ومرتبطة بـ t

① $S_s = \dfrac{\Delta e\, H}{1+e_0} = \dfrac{C_\alpha H}{1+e_p} \log \dfrac{t}{t_p} = \dfrac{0.022 \times 4}{1+e_p} \log \dfrac{10}{3.5} = 23.5\,mm$

e_p ??

$e_p = e_0 - \underset{primary}{\Delta e} = 0.8 - \Delta e = 0.704$

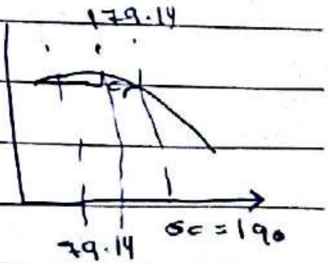
$0.27 = C_c = \dfrac{\Delta e}{\log \dfrac{\sigma_F}{\bar{\sigma}_{v0}}} \quad \dfrac{179.14}{79.14} \Rightarrow \Delta e = 0.27 \log \dfrac{179.14}{79.14}$

② ep فقط الي بتغير → $\Delta e = 1/6 \times \overset{0.27}{\cancel{C_c}}$

$S_s = 0.0225\ m$

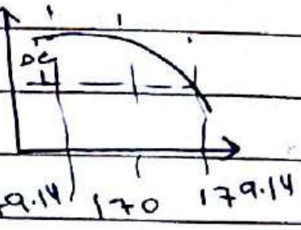
$e_p = e_0 - \Delta e = 0.8 - 0.0159 = 0.7841$

$\Delta e = C_r \log \dfrac{\bar{\sigma}_p}{\bar{\sigma}_{v0}} = \dfrac{0.27}{6} \log \dfrac{179.14}{79.14} = 0.0159$



③

$\underset{primary}{\Delta e} = \Delta e_1 + \Delta e_2 = \frac{1}{6} \times 0.27 \log \frac{170}{79.14} +$



$0.27 \log \frac{179.14}{170} = 0.021$

$e_p = 0.8 - 0.021 = 0.779$

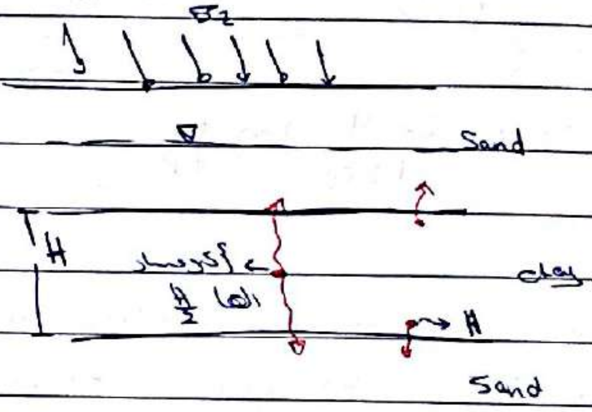
<u>ex</u>

σ_z

$\bar{\sigma} = \sigma_t - u$

$t=0 \leftarrow 0 = \bar{\sigma} = \sigma_z - u_e$

$\sigma_z \downarrow 0$



- Degree of consolidation. * السكر

$U_z = \frac{e_0 - e}{e_0 - e_p}$ or $U_z = \frac{\bar{\sigma} - \bar{\sigma}_{v0}}{\bar{\sigma}_f - \bar{\sigma}_{v0}}$

at end of primary

U_z : degree of consolidation

e_0 : initial void ratio $\rightarrow \bar{\sigma}_{v0}$

e_p : void ratio at end of primary $\Rightarrow \bar{\sigma}_p$

e : void ratio at any time.

$0.0 \leq U_z \leq 100\%$

at $t=0$

يعتمد على t, Ue, سماكة clay layer

الوقت الي بدي أحسبة

$u_e = [1 - U_z] \times \bar{\sigma}_z$

excess pore water pressure

degree of consolidation

when, $t=0 \rightarrow u_e = \sigma_z$

$t = t_p \rightarrow u_e = 0$

20/3/2016

Terzaghi's Theory of one-dimensional consolidation.

The theory relate the following three quantities

① The excess pore water pressure (U_e) $U_e = [1 - U_z]\bar{\sigma}_z$

② The depth (z) below the top of clay layer.

③ The time (t) from application of the total stress.

$$\frac{\partial u_e}{\partial t} = C_v \frac{\partial^2 u_e}{\partial z^2} \Longrightarrow \boxed{T_v = \frac{C_v t}{(d)^2}} \Longrightarrow \frac{\pi}{4}(0.5)^2 = \frac{C_v t_{50}}{\left(\frac{0.02}{2}\right)^2}$$

path الي تقطعه المية

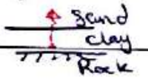
T_v : ~~time~~ time factors (unitless)

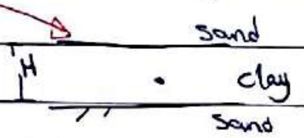
$T_v = \frac{\pi}{4} U_z^2$ --- $U_z < 0.6$

$T_v = -0.933 \log(1 - U_z) - 0.085$ $U_z \geq 0.6$ (مشابهة للمعادلة الأولى)

$d = \frac{H}{2}$ --- double drainge. (two path)

$d = H$ single drainge (one path)

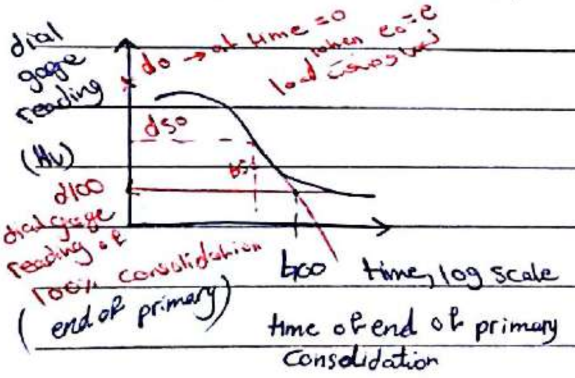




C_v : coeff of consolidation (m^2/time)

يعطي مقدار U_z مع t

① Casagrande method (t_{50})



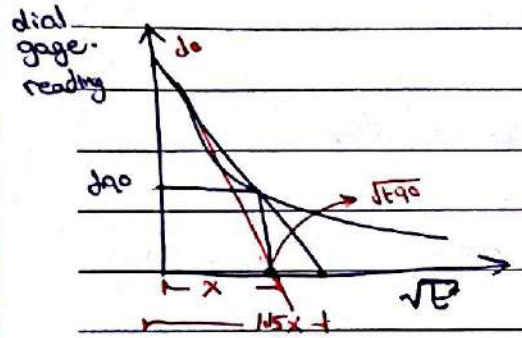
$$U_z = \frac{e_0 - e}{e_0 - e_p}$$

$$T_v = -0.933 \log(1 - U_z) - 0.085$$

$$e_0 = \frac{V_v}{V_s} = \frac{A\,H_v}{A\,H_s}$$ → d0

$d_{50} = \frac{d_0 + d_{100}}{2}$ Degree in void ال height يعني of consolidation . 50

② Taylor construction method (square-root method)



$$0.898 = \frac{C_v * t_{90}}{\left(\frac{0.02}{2}\right)^2} \Rightarrow C_v = \frac{0.898\, d^2}{t_{90}}$$

$$U_z = \frac{S_t}{S_c} \qquad , U_z = \frac{\overset{\Delta e, t}{e_0 - e}}{\underset{S_c \text{ end of primary.}}{e_0 - e_p}}$$

St : settlement at any time.

Sc : settelement at end of primary. Consolidation.

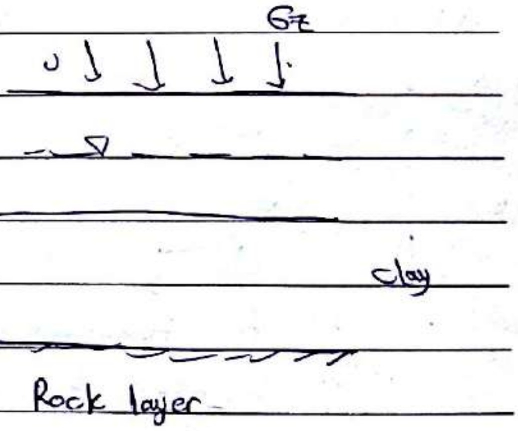
$K = C_v * M_v * \gamma_w$

M_v : coeff of compressability

$$M_v = \frac{\Delta e}{[1+e_0]\Delta\bar{\sigma}} = \frac{1}{1+e_0}\left[\frac{e_0 - e_p}{\sigma_f - \bar{\sigma}_v}\right]$$

ex

Find the required to reach 50%
degree of consolidation.



Field مو مشكلة

$$\frac{\pi}{4}(0.5)^2 = T_v = \frac{C_v * t_{50}}{(3)^2} \Rightarrow t_{50} = 8$$

↳ singale drainge

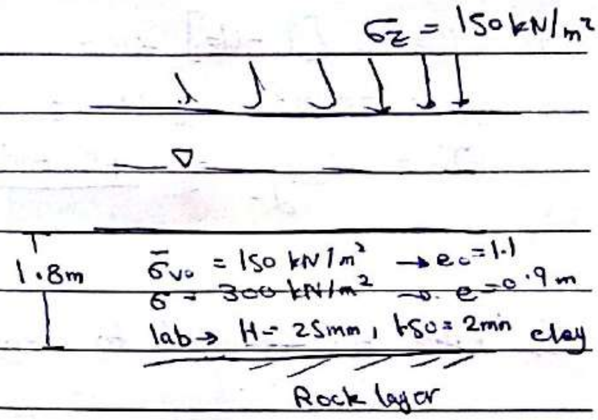
$$\frac{8064,000 \text{ sec}}{60 * 60 * 24} = 93.33 \text{day}$$

lab -- H = 25mm , t50 = 2min . 20 sec . 3m.

$$\frac{\pi}{4}(0.5)^2 = \frac{C_v * 140 \text{ sec}}{\left(\frac{0.025}{2}\right)^2} \Rightarrow C_v =$$

$$0.898 = T_{v90} = \frac{C_v * t_{90}}{(3)^2} \Rightarrow t_{90} =$$

ex



① Find primary consolidation settelement

$$S_c = \frac{\Delta e\ H}{1+e_0} = \frac{1.1 - 0.9}{1+1.1} * 1.8 = 17.14\ cm$$

② Find time required to reach 60% degree of consolidation

$$\frac{\pi}{4}(0.6)^2 = T_v = \frac{C_v * t_{60}}{(1.8)^2}$$

$$t_{60} = \frac{\pi/4\,(0.6)^2 * (1.8)^2}{1.53*10^{-5}} = \frac{60565\ min}{60*24}$$

$$= 42.06\ day$$

lab

$$\frac{\pi}{4}(0.5)^2 = \frac{C_v * 2min}{\left(\frac{0.025}{2}\right)^2}$$

$$\Rightarrow C_v = 1.53*10^{-5}\ m^2/min$$

③ Find the excess pore water pressure at 42.06 day

$$U_e = [1 - U_z]\,\bar{\sigma}_z = [1 - 0.6] * 150$$

④ Find " " " at 80 days.

$$T_v = \frac{1.53 * 10^{-5}\ m^2/min * (80 * 24 * 60)}{(1.8)^2} =$$

$$T_v = -0.933 \log(1 - U_z) - 0.085 \Rightarrow U_e.$$

22/3/2016

ex . نفس المثال السابق

4) $U_e = [1 - U_z] * 150$

$T_v = -0.933 \log(1 - U_z) - 0.085$

$$T_v = \frac{C_v * t}{d^2} = \frac{1.53 * 10^{-8}\ m^2/min * 80 * 24 * 60}{(1.8)^2}$$

5) Find the required to reach asettlement 10cm.

$$U_z = \frac{S_t}{S_c} \quad , \quad U_z = \frac{e_o - e}{e_o - e_p} \quad , \quad S_c = \frac{\Delta e}{1 + e_o} H$$

$$\rightarrow \frac{\frac{\Delta e\, H}{\cancel{1 + e_o}}}{\frac{e_o - e_p}{\cancel{1 + e_o}} H} = \frac{\Delta e}{e_o - e_p} = \frac{e_o - e}{e_o - e_p}$$

$$U_z = \frac{10}{17} = 0.58 \quad , \quad T_v = \frac{1.53 \times 10^{-5} \times t'}{(1.8)^2}$$

⑥ Estimate time required to end of primary.

$$0.898 = T_v = \frac{C_v * t_{90}}{(d)^2} = \frac{1.53 * 10^{-5} * t_{90}}{(1.8)^2} \Rightarrow t_{90} = \frac{0.848\,[1.8]^2}{1.53 \times 10^{-5}}$$

$T_v = -0.933 \log(1 - 09) - 0.085$

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

⑦ Find the coefitient of permeability (k)

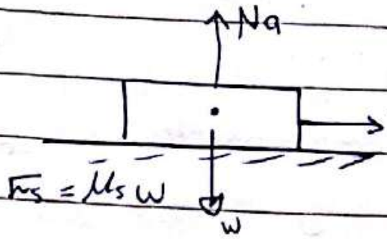
$$k = C_v * m_v * \gamma_w \Rightarrow C_v * \frac{1}{1 + e_o} \frac{\Delta e}{\Delta 6} * \gamma_w$$

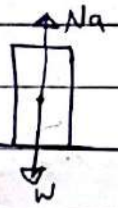
$$= 1.53 * 10^{-5}\ m^2/min * \frac{1}{1 + 1.1} * \frac{1.1 - 0.9}{150\ kN/m^2} * 9.806\ kN/m^3 = 95.3 * 10^{-9}\ m/min$$

24/3/2016

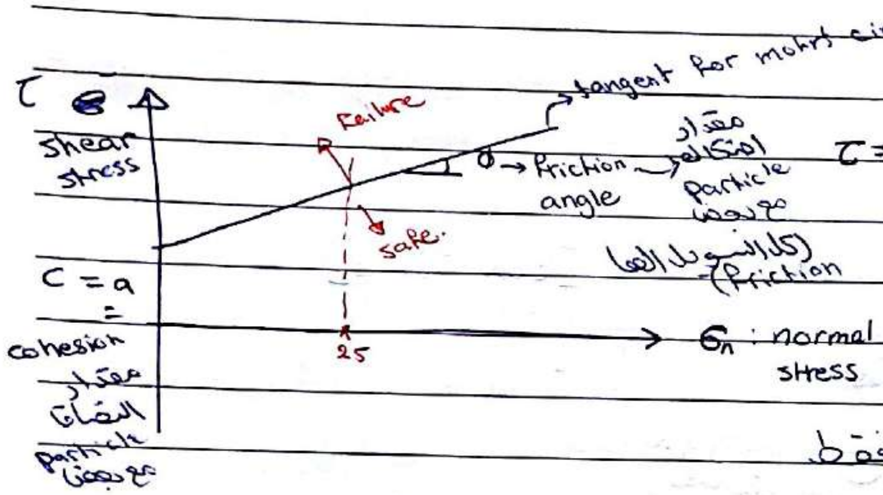
كل أنواع السويل failure منها تكون shear

shear strength parameter





$F_s, F(c,\phi)$



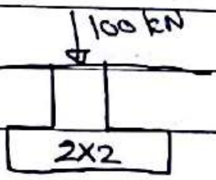
$\tau = a + \sigma_n \tan\phi$

- اذا shear تجاوز هنا ح يصير عنا failure. line

- الالتصاق خاصية لل clay فقط.

- تحت المساحة من يصح safe.

- c & φ من اللاب.



$\downarrow \sigma_n = \frac{100}{4\uparrow} = 25\ kN/m^2$

* laboratory strength test :- كيف أعرف قيمة c & φ من اللاب.

the shear strength parameter $\underline{c\ \&\ \phi}$ total stress

or $\bar{c}\ \&\ \bar{\phi}$ effective stress determind from lab :-

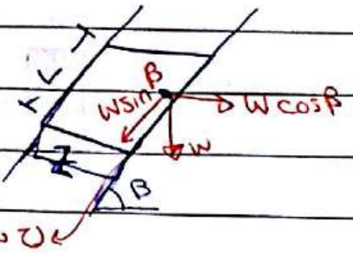
① Direct shear test

② triaxial shear test → UU-test, CU-test, CD-test

- كل واحد منهم يمثل simulation الموجودة ب site

Direct shear stress بعرضها لسويل فورس مباشر

$\gamma_{soil} = \frac{m}{V}$



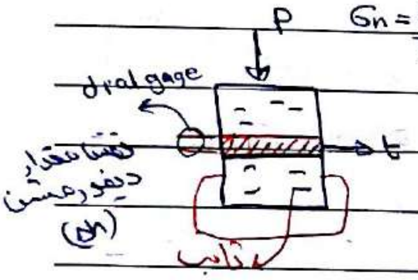
$\tau_f = c + \sigma_n \tan \phi$

Direct فقط في حالة slope stability

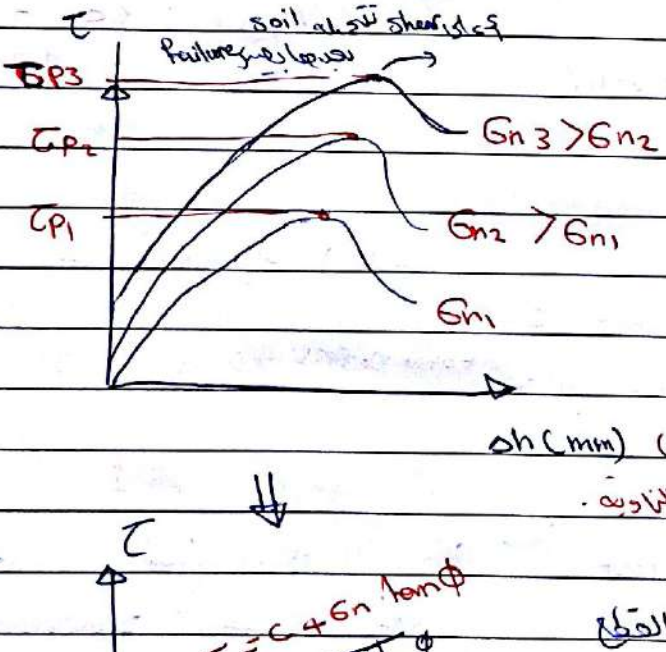
ال يقاوم هاد الاتجاه
ومقدارها friction يمنعوا من الانهيار

laboratory strength test 6cm X 6cm X 6cm [box مصنوعة]

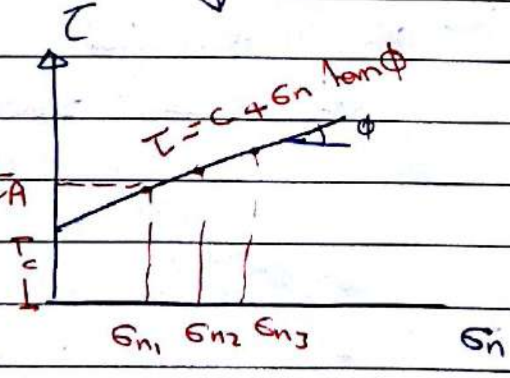
$\sigma_n = \frac{P}{A}$ نغير نورمال لود على كل تجربة



$\tau = \frac{t}{6 * 6}$ → dial gage نقيس مقدار اللود



⇓



* بدي الحكم بالزاوية β اي هي زاوية القطع
بحيث يصير عندي safe

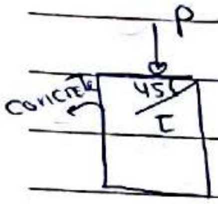
*** Advantages** مزايا Direct shear

① cheap, fast, simple ⇒ specially for sand

② Failure occurs along a single surface. $\theta = 0.0$

failure angle : 0.0

- يكون Failure نفس اتجاه shear



normal load
shear ' صار له
Failure

*** Disadvantages**

- ما تقدر أتحكم اذا كان $\bar{\sigma}$ أو لا. إذا لما يصير لل (excess pore water pressure = صفر)

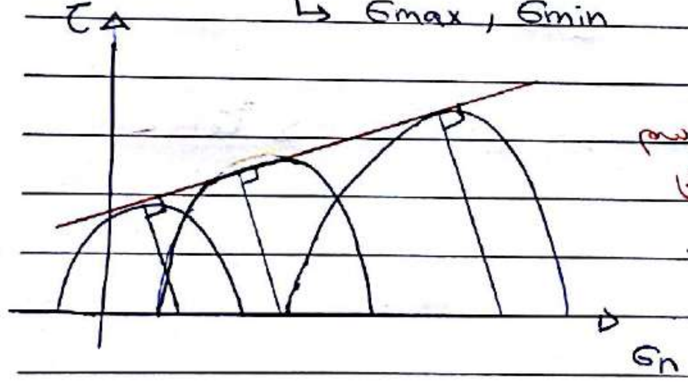
① Difficult or impossible to control drainge espeacially for fine soile

② Failure plane is forced — أجبرته يفشل بزاوية معينة

③ Non-uniform stress could addition. (المفروض بالعكس)

④ The principle stress rotates during shear

↳ σ_{max}, σ_{min}



ممكن أرسم
عدد لانهائي
من الدوائر

$\bar{\sigma}_{v0} \rightarrow \sigma_3 = min$

$\bar{\sigma}_p = \bar{\sigma}_{v0} + \sigma_z \Rightarrow \sigma_1 = max$

② triaxial test indirect [بعرضنا السويل لل σ_n ولكن نفشل as a shear]

① UU-test (Unconsolidation-undrained test) — لا يوجد consolid

→ two stages مرحلتين — excess pore w ≠ 0

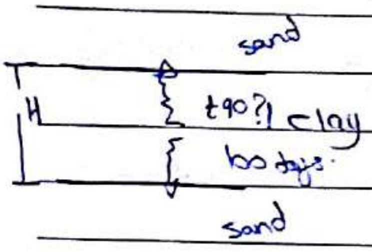
① confinement stress ② shearing — لذلك الستريس يكون total stress

ضغط محيط

σ_{v0}
σ_p

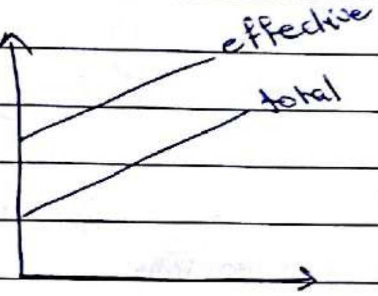
⇒ interm of total stress

⇒ لا يوجد consolidation يعني excess pore water pressure $\neq 0$

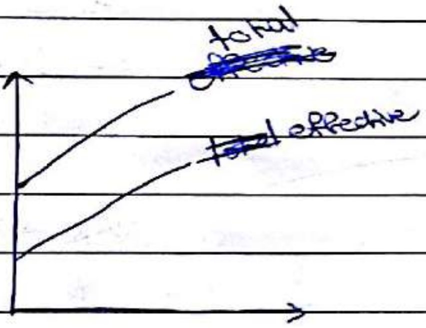


- بمنع الحبة من انو يصير لها consolidation عن طريق منع drainage .

- باللاب ما بنحط فوقه وتحته porous stone بنحط steel بمنع drainage فلا يحدث consolidation.



- لماذا نلجأ لهذا الاختبار ؟!
لانو بتأخر الـ clay ما بتصير $\bar{\sigma}$ بسرعة
بدو سنين وايام . أول اشي σ بصير
بعد ١٠٠ ~~[illegible]~~ يوم بصير $\bar{\sigma}$ فأي واحد
نختار ؟! بندرس الشيئين ونختار الأقل
مش لانتظار البناء.



- دائماً لازم اختار بين σ وبين $\bar{\sigma}$
واختار الأقل

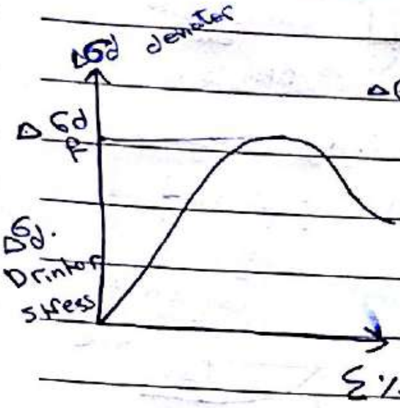
$$\overset{0.848}{\overset{\uparrow}{T_v}} = \frac{c_v t_{90}}{d^2}$$

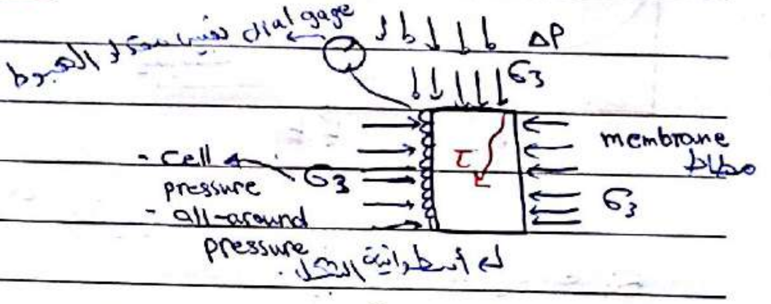
$$t_{90} = \frac{T_v d^2}{c_v}$$

27/3/2016

Triaxial test

① U-U test = Un consolidation - Undrained test (Q-test) :-





$\Sigma = \frac{\Delta L}{L_o}$

① Confinment

② Shearing

إزاحة المسافة G_n

$L = 36m$

$d_o = 38mm$

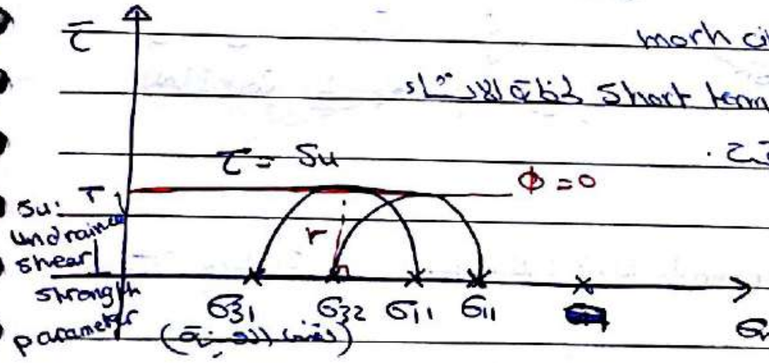
V: const (drainge لا يوجد)

$A_o L_o = A_c [L_o - \Delta L]$

$\Rightarrow A_c = \frac{A_o L_o}{L_o - \Delta L} = \frac{A_o}{1 - \boxed{\frac{\Delta L}{L_o}}} \Rightarrow \Sigma$

$\Delta G_d = \frac{\Delta P}{A_c}$

$G_1 = G_3 + (\Delta G_d)_f$



- بنرسم two stress نقدر أرسم morh circle

- هنا case بنربطها clay يعني كما short term لحظة الإنشاء

لما السويل ما صرلها وقت لأنو بحاجة لوقت.

* Φ=0 ← أنو water هو إلي مانع يصير

friction من particle لأنو full saturated

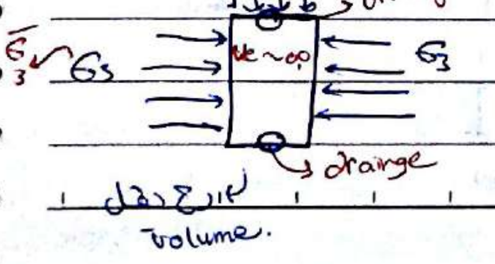
- هنا test منصلي في sand لأنه عندي load ومش ساعة يصير عندي

drainge و النفاذية للـ sand عالية وكمان رح يصير في load وما تطلع الماء لأنو حالاً مواجهة

اذا كان sand وموجود

② CU-test = conseildated - Undrained test (R-test) ⇒ total stress.

earthquake (زلازل).



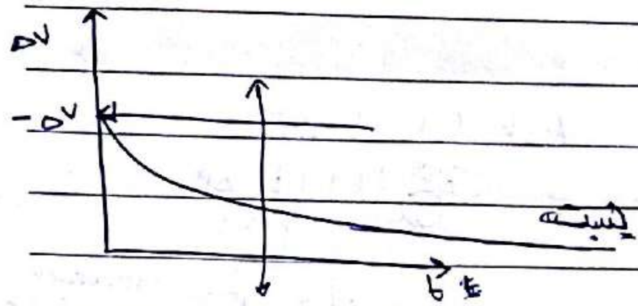
① confinment

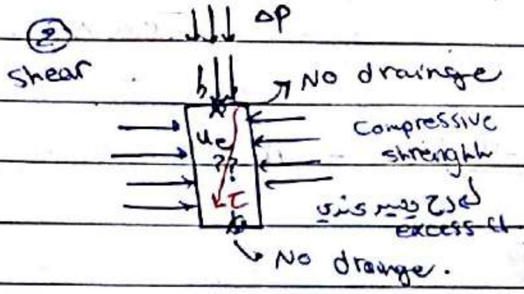
بس في مرحلة confinment رح أسمح يصير تصلد

أما في shear ما رح اسمح يصير تصلد

إزاحة حجم

volume.

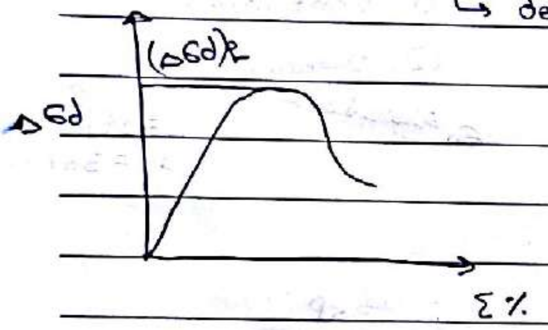




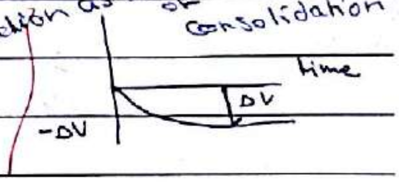
* عينة اسطوانية الشكل

- مادام في load انا في excess u حتى يصير الوصول failure

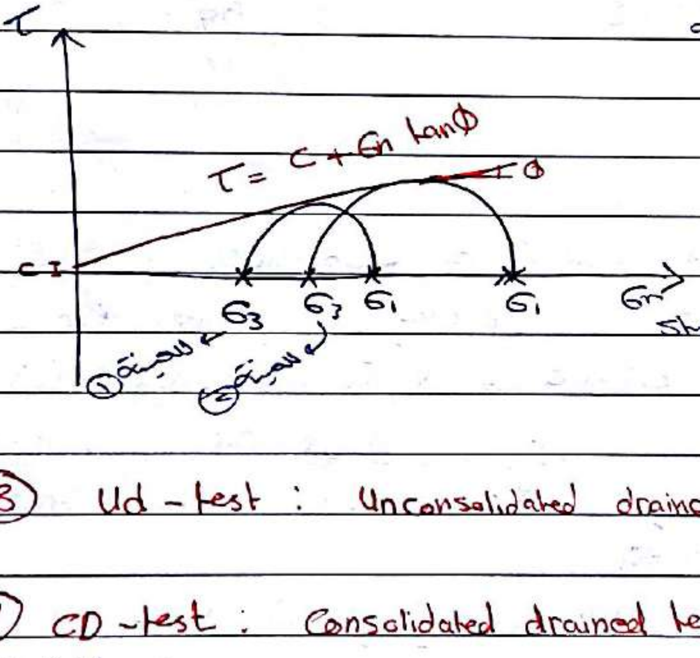
$\sigma_1 = \sigma_3 + (\Delta\sigma_d)_f$
↳ deviator stress at failure.



$V_o \rightarrow V_1 = V_o - \Delta V$ reduction as result of consolidation
بسبب ال confinement
↳ const
لمرحلة shear



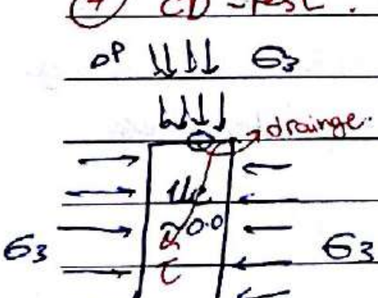
- CU test تعتبر أحسن total & effective stress
ما بقيس measured
نحسبهم



مش نفس ال لانو كل ما تزيد σ_3
وبساعد العينة بتحمل حتى
الماء أكثر فـ σ_3 أكبر فنج يزيد shear

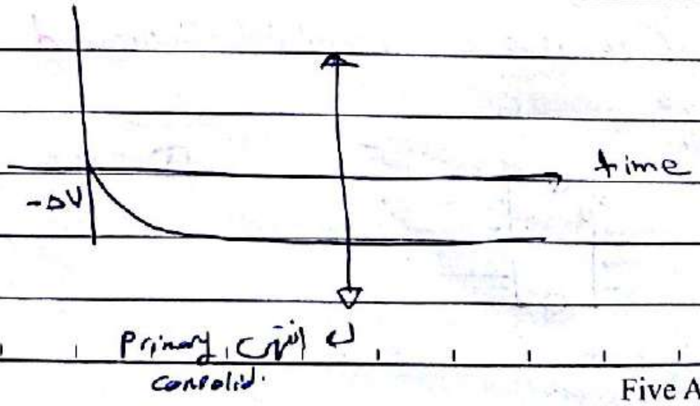
③ Ud - test : Unconsolidated drained test (R-test) : لا تحدث غالباً.

④ CD - test : Consolidated drained test (S-test) :- effective stress.



$\sigma_3 \sim \bar{\sigma}_3$

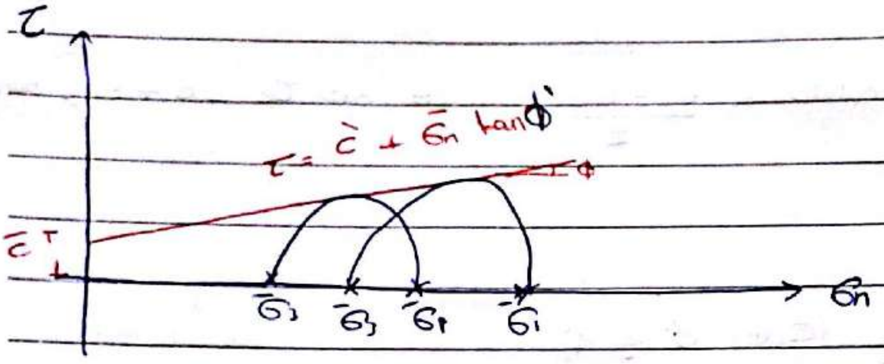
لما تثبت V

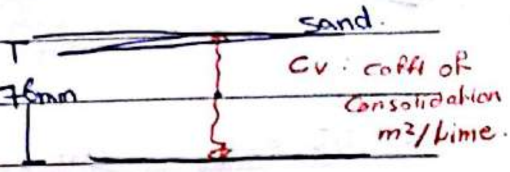


لـ انتهى Primary consolid.

29/3/2016

rate of loading < Cv : coefficient of consolidation.
mm/time.





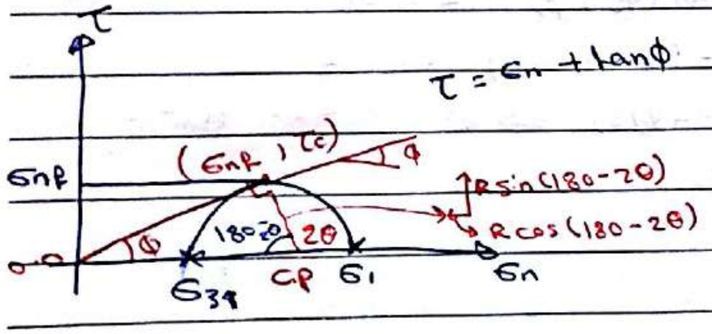
sand
double-drainge.

$\bar{\sigma}_1 = \bar{\sigma}_3 + (\Delta\sigma d)_f$

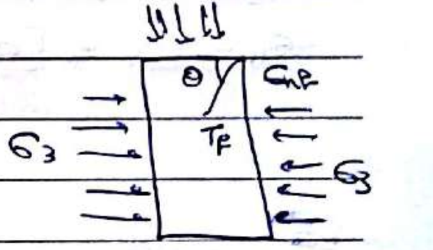
اذا كان rate < cv تمشي الحبيبة water particles لما توصل ال drainge path
- بيطلع ال sand لانو النفاذية عالية

Prediction of shear strength parameter (c , ϕ) or ($\bar{c}$, $\bar{\phi}$)

① for sand & N.c clay (C = 0.0)



$\sigma = \sigma_3 + (\Delta\sigma d)_f$



① $\sin\phi = \dfrac{\frac{\sigma_1 - \sigma_3}{2}}{\sigma_3 + \frac{\sigma_1 - \sigma_3}{2}} = \dfrac{\frac{\sigma_1 - \sigma_3}{\cancel{2}}}{\frac{2\sigma_3 + \sigma_1 - \sigma_3}{\cancel{2}}} = \dfrac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$

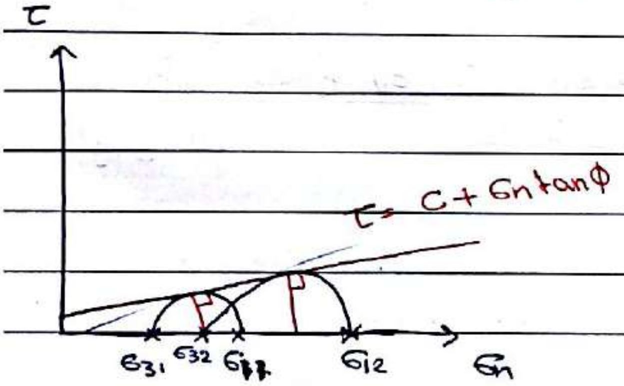
$\Rightarrow \phi = \sin^{-1}\left[\dfrac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right]$

② $\phi + 90 + 180 - 2\theta = 180 \Rightarrow \theta = 45 + \frac{\phi}{2}$, θ Failure Angle

③ σ_{nf} : Normal stress at failure $= \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta$

④ τ_f : shear stress at failure $= \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta$ or $\tau_f = 0 + \sigma_{nf} \tan\phi$

② over consolidation clay (C, ϕ) c' & ϕ'



$\sigma_{1_1} = \sigma_{3_1} \tan^2(45 + \phi/2) + 2c \tan(45 + \phi)$ ---- ①

$\sigma_{1_2} = \sigma_{32} \tan^2(45 + \phi/2) + 2c \tan(45 + \phi)$ ---- ②

$\sigma_{1_1} - \sigma_{1_2} = [\sigma_{3_1} - \sigma_{32}] \tan^2(45 + \phi/2) \Rightarrow \phi$ ---- ① - ②

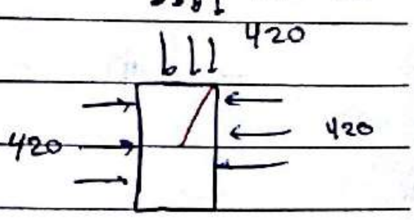
sub in ① or ② $\Longrightarrow$ C.

ex

cu

A speciman of saturated sand was consolidated under or around pressure of 420 kN/m². The axial stress was then incressed without permeability drainge.. speciman failed when the axial stress reached 350 kN/m² $(\Delta\sigma_d)_f$

pore water pressure at failure was 289.45 kN/m²

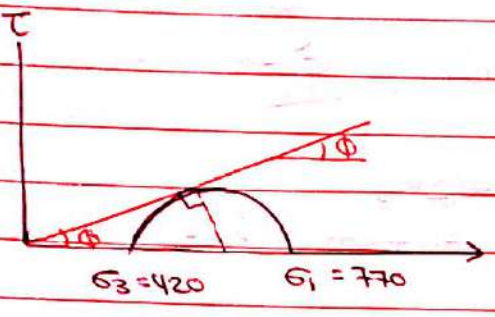


Find :-

① Consolidated - undrained shear strength parameter total stress $(c_T^{=0} , \phi_T)$

② effective shear strength parameter (drained) $(\underset{=0.0}{c'} , \phi')$

$\sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 420 + 350 = 770$

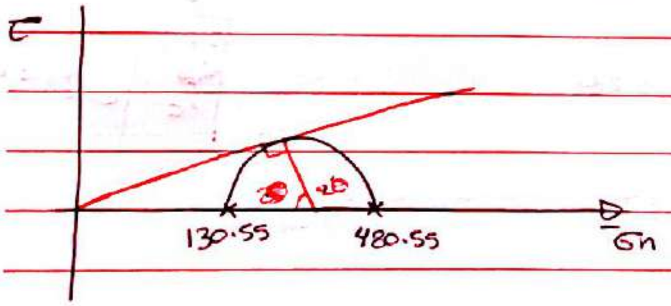


$$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \frac{770 - 420}{770 + 420} = 0.294 \Rightarrow \phi = \sin^{-1}[0.294] = 17.1^\circ$$

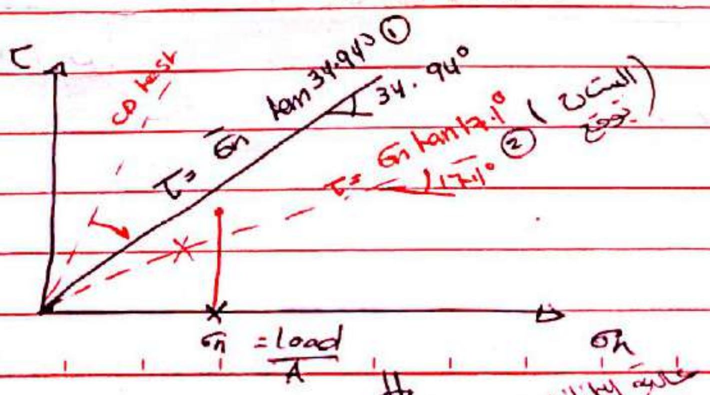
$\tau = \sigma_n \tan 17.1^\circ$

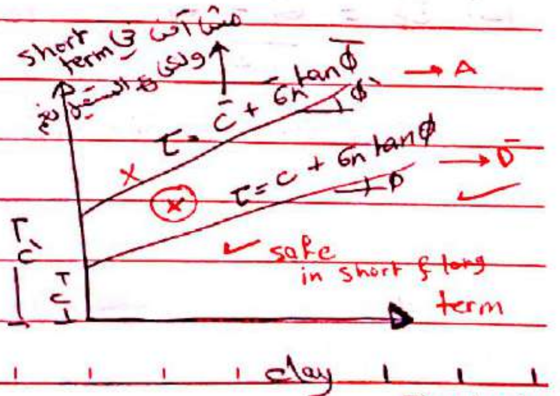
$\bar{\sigma}_3 = \sigma_3 - u = 420 - 289.45 = 130.55 \text{ kN/m}^2$

$\bar{\sigma}_1 = \sigma_1 - u = 770 - 289.45 = 480.55 \text{ kN/m}^2$



$$\sin\phi' = \frac{480.55 - 130.55}{480.55 + 130.55} = 0.5727 \Rightarrow \phi' = \sin^{-1}(0.5727) = 34.94^\circ$$





clay

sand (permeability عالية و drainage سريع ⇒ $\bar{u}$ فالشغل الصح هو ①

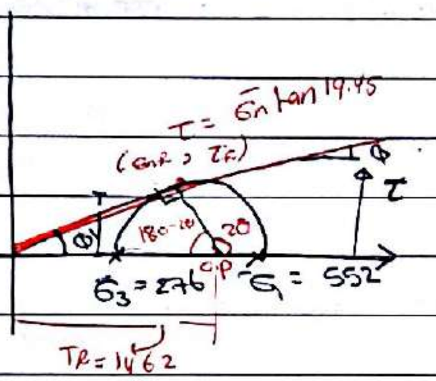
في clay قليلة permeability و drainage وقليل حتى أصل لـ $\bar{u}$ بدي سنين طويلة $(\bar{D})$

31/3/2016

ex.

A consolidated - drained triaxial test was Calculated on anormally (CD) clay [c = 0.0] $\bar{\sigma}_3 = 276$ kN/m² $(\Delta\sigma_d)_f = 276$ kN/m²

① effective shear strength parameters (c', ϕ') $c' = 0$

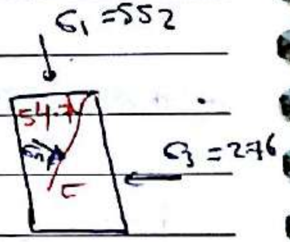


$\bar{\sigma}_1 = \bar{\sigma}_3 + (\Delta\sigma_d)_f$

$= 276 + 276 = 552$

$$\sin\phi' = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_1 + \bar{\sigma}_3} = \frac{552 - 276}{552 + 276} \Rightarrow \phi' = 19.45^\circ$$

② Angle of failure $(\theta) \Rightarrow \theta = 45' + \frac{\phi'}{2} = 45 + \frac{19.45}{2} = 54.73^\circ$



③ $$\sigma_{nf} = \frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2} + \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2} \cos 2\theta$$

$$= \frac{552 + 276}{2} + \frac{552 - 276}{2} \cos 2 * 54.73$$

$$= 368.03$$

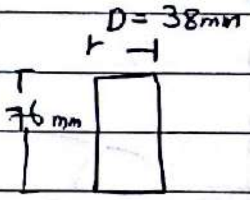
④ $$\tau_f = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2} \sin 2\theta = \frac{552 - 276}{2} * \sin 2(54.73^\circ) = \boxed{130.12 \text{ kN/m}^2}$$

$\tau_f = 0.0 + 368.03 \tan 19.45^\circ$

⑤ Find normal stress at maximum shear stress.

$$\sigma_n = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2} + \cancel{\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}} \xrightarrow{} 0 \; \cos 90^\circ = \frac{552 + 276}{2} = 414 \ kN/m^2$$

$$\tau_{max} = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2} = \frac{552 - 276}{2} = \boxed{138 \ kN/m^2}$$



$* \ \tau_f = 0 + 414 \tan 19.45 = \boxed{146.2 kN/m^2}$

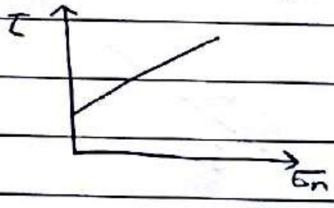
ex over conselidation clay ⇒ C-D test

① $\sigma_3 = 100 \ kN/m^2$, $\Delta\sigma_{df} = 410.6 \ kN/m^2$
② $\sigma_3 = 50 \ kN/m^2$, $(\Delta\sigma_d)_f = 384.37 \ kN/m^2$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan(45 + \frac{\phi'}{2}) + 2c' \tan(45 + \phi/2)$$

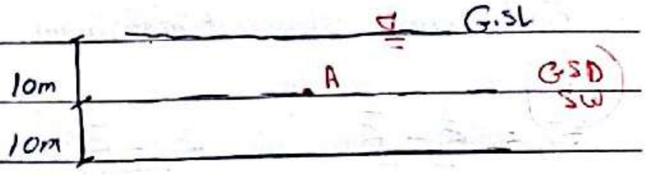
$510.6 = 100 \tan^2(45 + \phi'/2) + 2 \times c' \tan(45 + \phi'/2)$ ---- ①
~~403~~ $434.37 = 50 \tan^2(45 + \phi/2) \mp 2 \times c' (\tan 45 + \phi'/2)$ -- ②
$76.23 = 50 \tan^2(45 + \phi'/2) \Rightarrow \phi'/2 + 45 = \tan^{-1}\sqrt{\frac{76.23}{50}}$

$\Rightarrow \phi' = 12^\circ \Rightarrow c' = 145 \ kN/m^2$
$\tau = 145 + \bar{\sigma}_n \tan 12^\circ$



ex

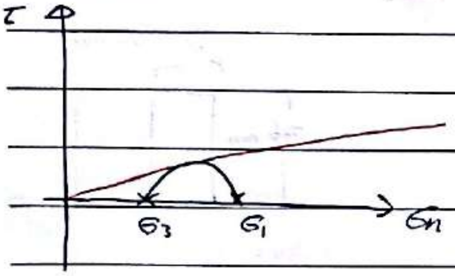
$$\tau = \overset{0}{\cancel{C}} + \overset{(\sigma_n - u)}{\sigma_n} \tan\phi \rightarrow 35^\circ$$



$\sigma_n = \gamma_{sat} * 10$

#4
#200
LL
PL
→ classification

C-D test



$$\sin\phi = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_1 + \bar{\sigma}_3} \Rightarrow \phi = 35^\circ$$

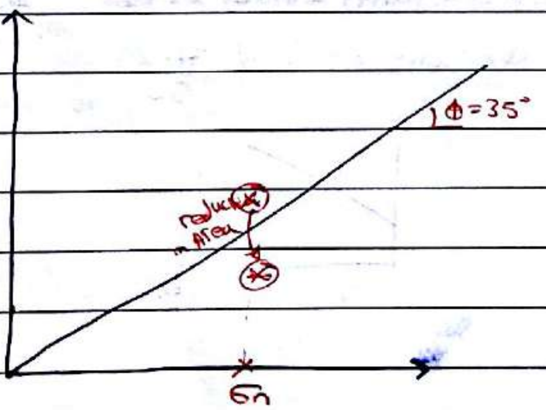
$$\bar{\sigma}_n = [\gamma_{sat} * 10m] - [10 * 9.806]$$

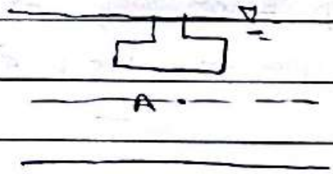
$$\gamma_{sat} = \frac{\overset{2.67}{G_s} + e}{1 + \underset{0.8}{e}} \gamma_w$$

measured in lab

$$\overset{2.67}{G_s}\, w = \overset{1.0}{S_r}\, e \Rightarrow e = 0.8$$

قيسه



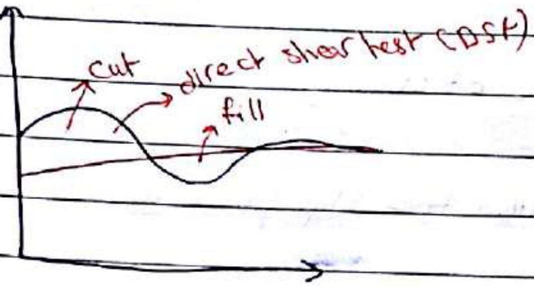


$$\sigma_{nf} = \bar{\sigma}_{vo} + \bar{\sigma}_z$$

$\rightarrow q * I_r$

$q \leftarrow \frac{Q}{A}$

3/4/2016



Trixial ← معاملها زي [illegible]

Slope stability

Slope in soils and rock are ubiqutions in nature & man-made structures.

- high ways, dams, leeves, bund-walls, and stock piles are constructed by sloping the ~~la~~ face of the soil

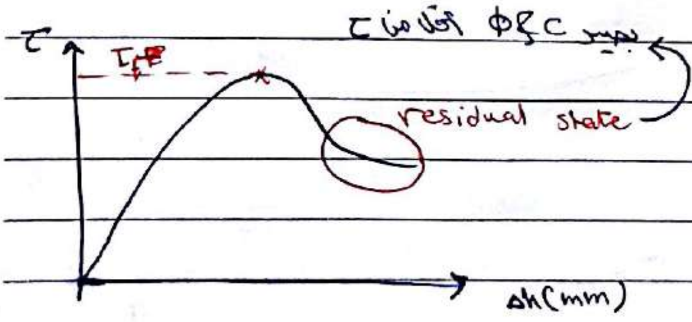
= Natural forces (wind, water, sho, --), change the tography on earth often creating unstable slopes.

- Failure of such slope, resulted in human loss & dustruction.

* Definition of key terms:-

① Slip or failure zone :- A thin zone of soil that reaches critical state or "residual state" and result in movement of the upper soil mass

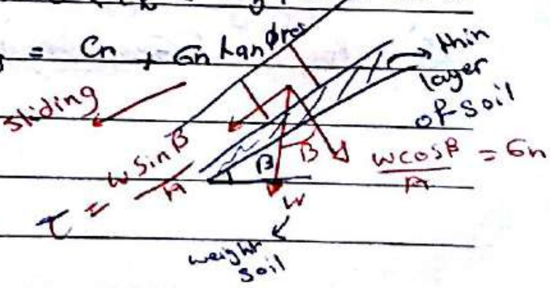
τ يثبت ϕ & C



stable ← $\tau < \tau_f = C_f + \sigma_n \tan\phi_f$

$\tau > \tau_{res} = C_r + \sigma_n \tan\phi_{res}$

sliding ←



$\tau = \frac{W \sin\beta}{A}$

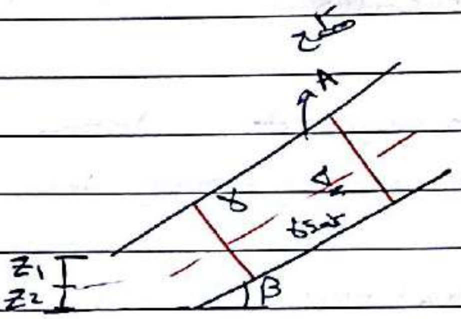
$\frac{W \cos\beta}{A} = \sigma_n$

② Slip plane :- failure plane :- Surface of sliding

③ sliding mass :- mass of soil within the slip plane and the ground surface

④ slope Angle : Angle of inclination of aslope to the horizontal

⑤ pore water pressure ratio (ru) : The ratio of pore water force on aslip surface to the total weight of the soil & any external loading



$$u = \frac{\boxed{\gamma_w \; Z_2} * \cancel{A}}{([\gamma + Z_1] + [\gamma_{sat} + Z_2]) \cancel{*A}}$$ as a force

$$r_u = \frac{\gamma_w \; Z_2}{(\gamma + Z_1) + (\gamma_{sat} + Z_2)}$$

* slope failure depend on :-

① Type of soil < fine-grained / Course-grained ⇒ C, ϕ

② soil straitification

③ Ground water table

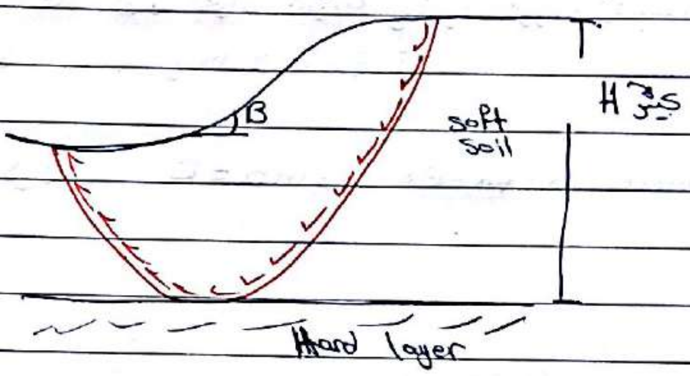
④ seepage (flow rate)

⑤ Angle (Geometry)

* common type of slope failure:-

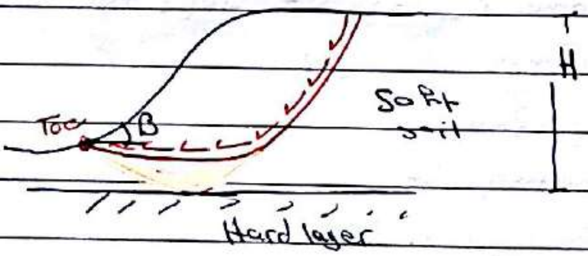
① movement of soil mass along a thin layer of weak soil

② Base slide.

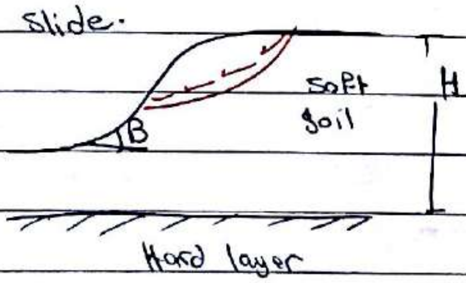


soft يعني ← C منخفضة

③ Toe slide:-

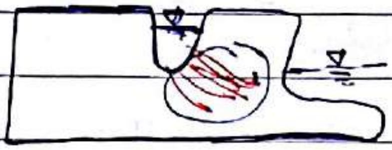


④ Slope slide.

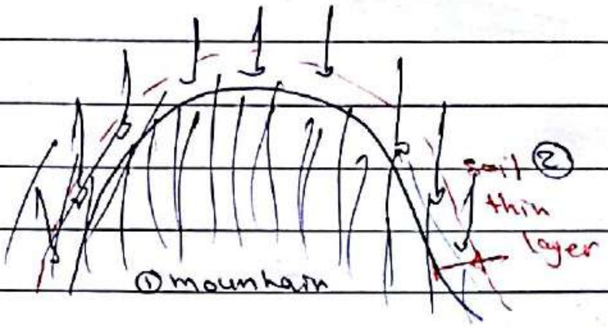


⑤ Block slide

بسبب seepage



⑥ slope أسوء أنواع → flow slide



③ Huge rainfall

- اجا water فارتفع pore water pressure فيزداد τ

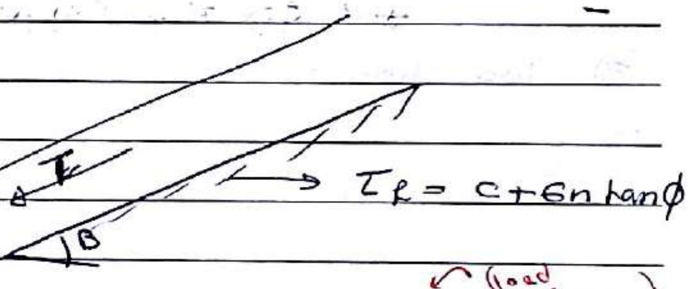
- الأمطار تعمل إخزان للتربة وتزيد الوزن (بسبب ارتفاع منسوب water pore) في shear يزداد
ف magnitude ال force تزداد.

- الزلزال shear wave → acceleration ~~[illegible]~~ → ma = F
له خطورة.

أما waves بتكون vertical
مش خطرة

* Some Causes of slope failure.

① Erosion الإزالة
② Rainfall
③ Earthquake
④ Geological features
⑤ Construction activity
⑥ External loading
⑦ Excavated Slope
⑧ Fill slope.
⑨ Rapid draw Down.



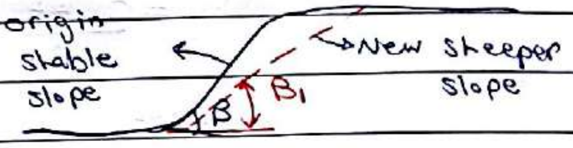
$\tau_f = c + \sigma_n \tan\phi$

① increase of shear (زيادة load)
② steeper Angle (Geomatric)
antenetion angle

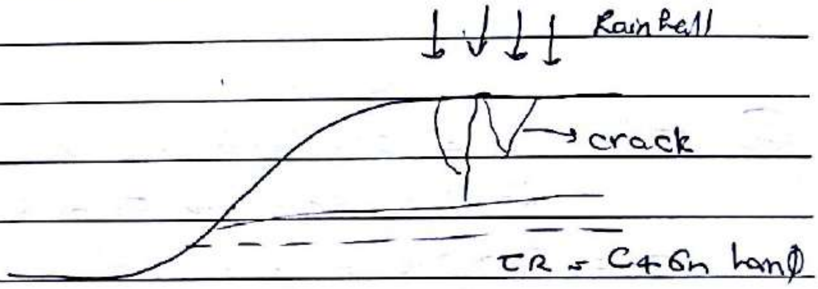
5/4/2016

* some causes of slope failure:-

① Erosion :- water and wind continuously erode natural and man made slopes.
it changes the geometry of the slope, ultimately resulting in slope failure or more aptly, landslide.

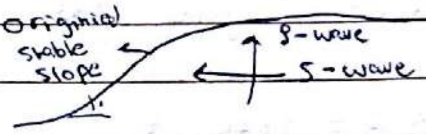


② Rain fall : long period of rainfall saturate, soften & erode soils.

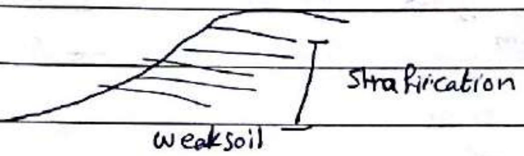


5/4/2016.

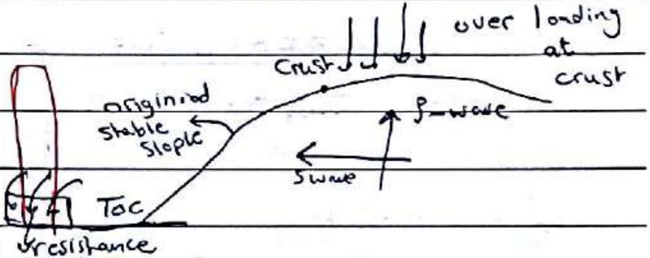
③ Earth quake : Introduce dynamic force, especially dynamic shear force



④ Geo logical factors:-
(الطبقات)



⑤ external loading: load placed on the crust of slope, add to the gravitional load, and may cause slope failure.



لإذا وضعنا Structure عند Toe رح يشكل resistance
ضد انهيار الانزلاقي رح يقلل slope failure أي stability

⑤ Construction Activity:-

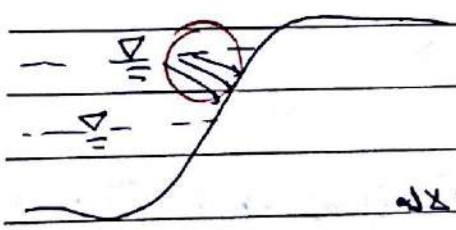
- Excavated slope : If the slope failure were to occur, they would take place after construction is completed.

- Fill slope : failure occur during construction.

⑥ Rapid Draw down:-

يحدث في السدود.

later force provided by water removed and excess p.w.p dones not have enough time to dissipated

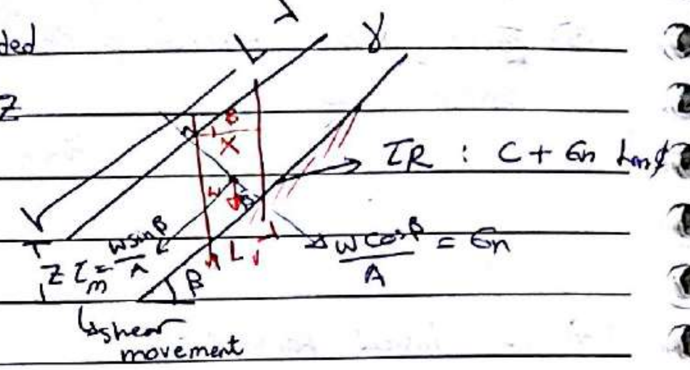


$\bar{\sigma} = \sigma_t - u$

- اذا زادت الأمطار وبلشت السد يعبي اذا تجاوز capacity تاعه
رح ينهار السد فافتح السد بوابات gates نفتحها هاي البوابات و نصرف excess water
و water level رح ينزل

Infinite slope :-

a slope that have dimension extended over agreted distance. $L \gg Z$



$$S.F = \text{Safty Factor}$$
$$= \frac{\tau_R}{\tau_m} \geq 1.5$$

τ_m : movement shear , unsaved & stable ← $1 < S.F < 1.5$ [illegible]

τ_R : resistance shear

$$\tau_m = \frac{W \sin\beta}{A} = \frac{\gamma V g \sin\beta}{A} = \frac{\gamma * [X * Z * 1] \sin\beta}{L_1 * 1}$$

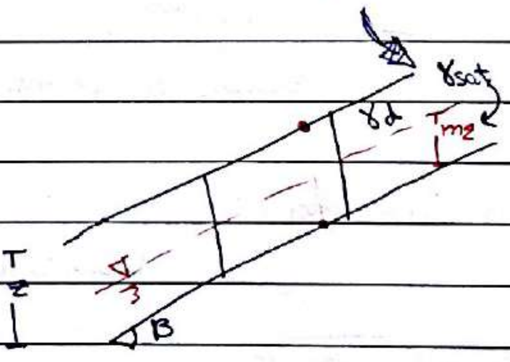
$$= \frac{\gamma [X * Z * 1] \sin\beta}{\frac{X}{\cos\beta} * 1} = \gamma * Z * \sin\beta \cos\beta$$

$$\tau_R = C + \sigma_n \tan\phi$$

$$\sigma_n = \frac{\gamma * [X * Z * 1] \cos\beta}{\underbrace{\frac{X}{\cos\beta}}_{L_1 * 1}} = \gamma * Z * \cos^2\beta$$

$$\therefore SF = \frac{C + [\gamma * Z * \cos^2\beta] \tan\phi}{\gamma * Z * \sin\beta \cos\beta}$$

$0 < m \leq 1$



* $\tau_R = C + \sigma_n \tan\phi$

~~$W = \gamma_d$~~ water * في حالة وجود

$$W = \gamma_d [[Z - mZ] * X * 1] + \gamma_{sat} * mZ * X * 1$$

$$\tau_m = \frac{W \sin\beta}{A} = \frac{\sin\beta}{\frac{X}{\cos\beta}} = Z[\gamma_d (1-m) + m \gamma_{sat}] \frac{\sin\beta}{\cos\beta}$$

$\gamma_{sat} > \gamma_d$

$$\ast\ S.F = \frac{C + [\sigma_n - \overset{\text{pore water pressure}}{U}] \tan\phi}{\tau_m}$$

$$\ast\ \sigma_n = z\,[\,\gamma_d(1-m) + m\,\gamma_{sat}\,]\cos^2\beta$$

$$\ast\ U = \gamma_w \frac{[\cancel{x} \times 1 \times mz]\cos\beta}{\cancel{x}/\cos\beta} = \gamma_w \times mz \times \cos^2\beta$$

$$\ast\ \tau_R = C + [\sigma_n - U]\tan\phi$$

$$S.F = \frac{\tau_R \uparrow}{\uparrow \tau_m} \rightarrow \text{saturated} \qquad S.F \uparrow \rightarrow \text{sat}$$

← زدنا المقام وقللنا البسط

* وجود water table ⇐ SF بقلل (تأكدوا من دكتور).

τ_R → max زادت لأن σ_n زادت.

Second 14/4/2016

2:45 - 3:45 7/4/2016

(٢) قاعة ٢:٥٠ ظ

١٠:٠١ ظ

٢:١٢ ظ (١) قاعة

٢:٧ ظ

$$S.F = \frac{\tau_R}{\tau_m} \geq 1.5$$

$$\tau_m = \left[(1-m)\gamma_d + m\,\gamma_{sat}\right] Z \cos\beta \sin\beta$$

$$\tau_R = C + [\sigma_n - U] \tan\phi$$

$$\sigma_n = \left[[1-m]\gamma_d + m\,\gamma_{sat}\right] Z \cos^2\beta$$

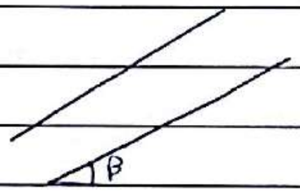
$$U = m Z \times \gamma_w \times \cos^2\beta$$

① If $m = 0.0$, $C = 0.0$

$\Rightarrow$ if $SF = 1 \Rightarrow \tau_R = \tau_m$

$$\tau_m = \gamma_d \times Z \cos\beta \sin\beta$$

$$\tau_R = [\gamma_d \times Z \cos^2\beta] \tan\phi$$



$$\frac{\cancel{\gamma_d} \times \cancel{Z} \cos^{\cancel{2}}\beta \tan\phi}{\cancel{\gamma_d} \times \cancel{Z} \times \cancel{\cos\beta} \sin\beta} = 1$$

$$\frac{\tan\phi}{\tan\beta} = 1 \Rightarrow \tan\beta = \tan\phi$$

$$\beta_{critical} = \Phi$$

$$\Rightarrow \frac{\tau_R}{\tau_m} = 1 \Rightarrow \frac{0.0 + [\gamma_{sat} \times Z \cos^2\beta - \gamma_w \times Z \times \cos^2\beta] \tan\Phi}{\gamma_{sat} \times \gamma Z \times \cos\beta \sin\beta}$$

$$= \frac{[\gamma_{sat} - \gamma_w] \cos\beta \tan\Phi}{\gamma_{sat} \times \sin\beta}$$

$$= \frac{\bar{\gamma}}{\gamma_{sat}} \times \frac{\tan\phi}{\tan\beta} - 1 \Rightarrow \tan\beta = \frac{\grave{\gamma} \tan\phi}{\gamma_{sat}}$$

$$\Rightarrow \beta \sim \frac{1}{2}\phi'$$

* IF m = 1   Cu = , $\phi = 0.0$

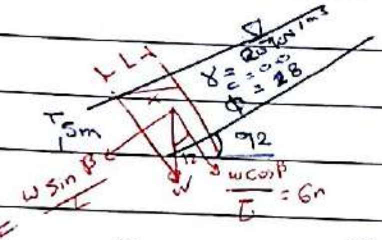
$$\frac{\tau_R}{\tau_m} = S.F = 1 \Rightarrow \frac{\tau_R = C + [\sigma_n - U]\tan\phi}{\tau_m} = \frac{Cu}{\gamma * z * \sin\beta \cos\beta}$$

$$\Rightarrow \sin\beta \cos\beta = \frac{Cu}{\gamma * z}$$

$$\tfrac{1}{2} \sin 2\beta = \frac{Cu}{\gamma * z} \longrightarrow \sin 2\beta = \boxed{\frac{2\,Cu}{\gamma \times z}} \rightarrow \text{stability NO}$$

$$\sin 2\beta = 1 \Rightarrow \beta_{critical} = 45^\circ$$

ex



$$S.F = \frac{\tau_R}{\tau_m}$$

$\tau_r =$

$$S.F = \frac{0.4\ [\gamma_{sat} * z \cos^2\beta - \gamma_w * z * \cos^2\beta]\tan\phi}{\gamma_{sat} * z * \sin\beta \cos\beta}$$

$$= \frac{[20 * 5 * \cos^2(12) - 9.806 * 5 * \cos^2(12)]\tan 28}{20 * 5 * \sin 12^\circ \cos 12^\circ}$$

~~12.06~~ 12

$$= \frac{[20 - 9.806]\cos(12)\tan(28)}{20 \sin(12)\beta} = 1.27 < 1.5$$

unsaved.
(stable)

$\rightarrow \tan\beta$ $= 1.5$

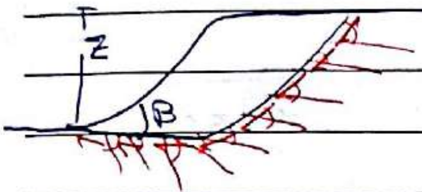
لو $\beta = 10^\circ$

لما تكون cut مقبول هنسيبها.

10/4/2016

ex

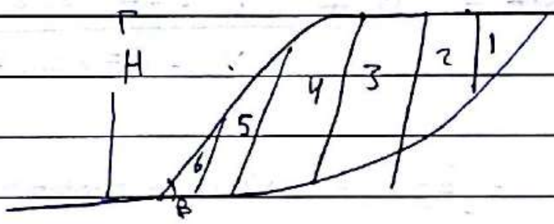
$\beta \rightarrow$ failure [illegible]

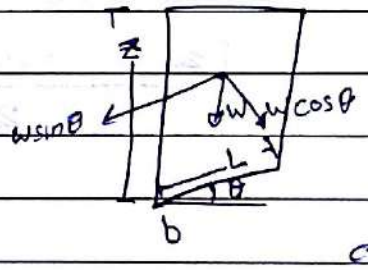


soil

* slope stability can be analyzad on different method.

- Limit equilibrium (most





$$\cos\theta = \frac{b}{L}$$

$$\Rightarrow L = \frac{b}{\cos\theta}$$

$$S.F = \frac{\tau_R}{\tau_m} = \frac{C + \sigma_n \tan\phi}{\tau_m}$$

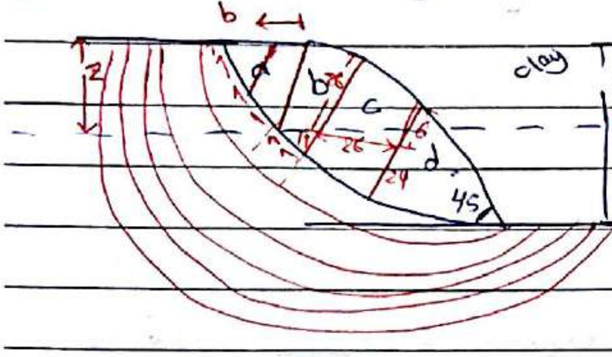
$$= \frac{C + \frac{W\cos\theta}{A}\tan\phi}{\frac{W\sin\theta}{A}}$$

$$= \frac{CA + W\cos\theta \tan\phi}{W\sin\theta} = \sum_{i=1}^{n} \frac{C_i * b_i * \sec\theta_i + W_i \cos\theta_i \tan\phi_i}{W_i \sin\theta_i} \geq 1.5$$

Fellenuis method

ex

two layer



γ = 110 pcf

Soft (A) φ = 18° C = 350 psf

soft

(B) γ = 120 pcf

φ = 34°

c = 0.0

sand

slices Number Base in layer ω tanφ c b θ secθ cosθ sinθ u×b $\frac{c \times b}{\sec\theta}$

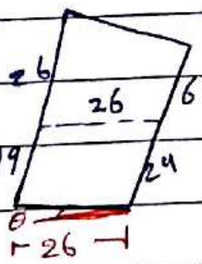
1×γ×المساحة →

Slice No.	Base in layer	W	tanφ	c	b	θ	secθ	cosθ	sinθ	u×b	c×b secθ	(ω-ub) cosθ tanφ	W sinθ
a	A	20.1	0.325	350	14	62°	2.13	0.469	0.883	0	10.44	3.06	17.75
b	B	108	0.675	0.0	27	33°	1.192	0.839	0.545	0.0	0	61.16	58.86
c	B	112.8	0.675	0.0	26	12°	1.022	0.978	0.208	0	0	74.46	21.54
d	B	40.3	0.675	0.0	26	-12	1.022	0.978	-0.208	0	0	26.6	-8.38
									Σ	0	10.44	165.79	89.77

Slice b

ω = [26 × 27 × 110] + [½ × 27 × 19 × 120] = 108k

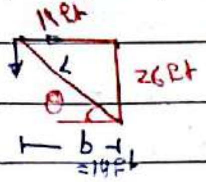
Slice c



$\omega = \frac{26+6}{2} * 110 \times 26] + [\frac{19+24}{2} \times 26 \times 120]$

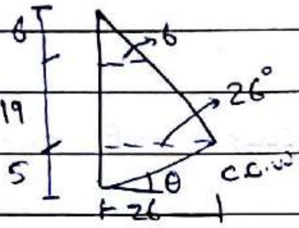
= 112.8

Slice a



$W = \frac{1}{2} \times 14 \times 26 \times 110 = 20.1k$

Slice d



$W = \left[\frac{1}{2} \times 6 \times 6 \times 110\right] + \cancel{6\times}\left[\frac{6+26}{2} \times 19 \times\right]120$

$= 40.3k$

$\therefore S.F = \frac{10.44 + 165.79}{89.77} = 1.96 > 1.5$ --- ok

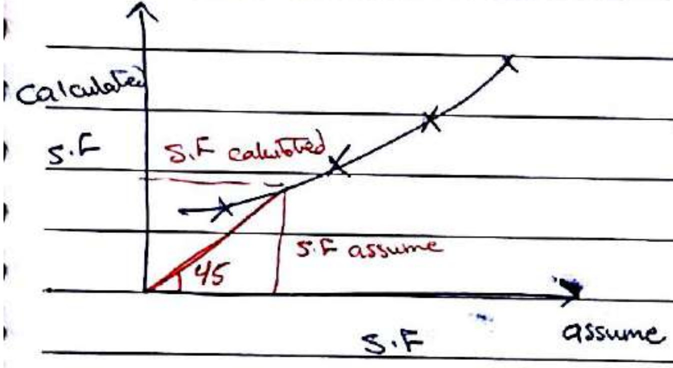
Scanned by CamScanner

12/4/2016

② Bishop method.

$$\underset{\downarrow\ \text{calculated}}{S.F} = \frac{\left[\sum_{i=1}^{n} c_i b_i + \sum_{i=1}^{n} (w - u \times b) \tan\phi\right] \frac{1}{ma}}{\sum_{i=1}^{n} w_i \sin\theta_i}$$

$$ma = \left[1 + \frac{\tan\phi \tan\theta}{\underset{\rightarrow\ \text{assumed}}{S.F}}\right] \cos\theta$$

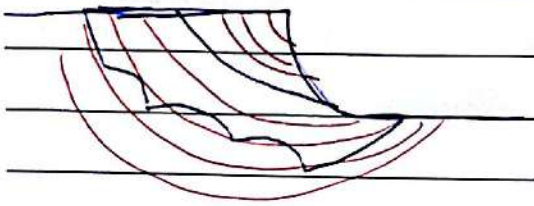


ex نفس المثال السابق.

slice No	Base in layer	ω	tanϕ	c	b	ub	θ	sinθ	ma	$\frac{1}{ma}[c_ib_i + w\tan\phi]$	$\omega\sin\theta$
a	A	20.1	0.325	3.50	14	0	62°	0.883	0.65	17.58	17.75
											-
b	B	108	0.675	0	27	0	33°	0.545	1.03	70.78	58.86
c	B	112.8	0.675	0	26	0	12°	0.208	1.03	73.92 ~~21.54~~	21.54
d	B	40.3	0.675	0	26	0	-12°	-0.208	1.03	29.2	-8.38
										Σ = 191.48	Σ = 89.77

assume S.F = 2.0

∴ $S.F = \frac{191.48}{89.77} = 2.13$ ← هاد المعتمد (دائماً أعلى)*



② Simplifid Janbu's method

Janbu assumed a non-circular slip surface

$$S.F = f_0 \left[\frac{\sum_{i=1}^{n} Gb_i + [\sum_{i=1}^{n} w_i - u_i b_i] \tan' \phi}{\sum_{i=1}^{n} w_i \tan \theta_i} \right] \frac{1}{m(\theta) \cos \theta_i}$$

✓

calculated

f_0 = corruction factor, 1.00 to 1.12

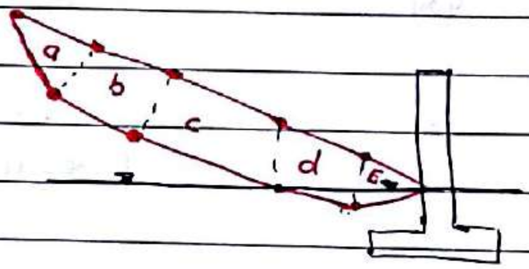
Example →

f_0 = 1.04

assume S.F = 1.40

Layer A

$\gamma = 18 kN/m^3$
$\phi = 18°$
$C = 12 kN/m^2$



Layer B

$\gamma_{sat} = 18 kN/m^3$
$\theta = 20°$
$C = 14.4 kN/m^2$

silica No.	Base in layer	W	C	tan Φ	b	θ	cosθ	tanθ	U	m(θ)
a	A	108	12	0.325	3	53°	0.602	1.327	0.0	0.80
b	A	1139	12	0.325	11.5	24°	0.914	0.445	0.0	1.9
c	A	957	12	0.325	8.5	18°	0.951	0.325	0.0	1.02
d	~~A~~ B	813	14.4	0.364	9.5	12°	0.978	0.364	7.35	1.05
e	B	~~[illegible]~~ 126	14.4	0.364	3.5	-13°	0.974	-0.231	9.806	0.92

$\frac{cb + (w - vb)\tan\phi}{\cos\phi \quad m(\phi)}$	$w\tan\theta$
152	143
551	507
414	366
404	191
94	-29
$\Sigma = 1615$	$\Sigma = 1178$

$S.F = 1.04 * \frac{1615}{1178} = 1.42$