الإمبراطور

في

الهندسة الجيوتقنية

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#1-Basic Characteristics of Soils :-

غير ملتصقة ببعضها

Soil: any un-cemented accumulation of Mineral particles formed by the weathering of rocks, the voids spaces between the particles contain water and or air.

امطار ، رياح ، - - -

weathering of rocks 1-Physical weathering (Sands , Gravel)

امطار ، حامضية ، - - -

2-Chemical Weathering (Clay)

*The contact of rocks and water produces clays.

*الية تكون الكالونايت :-

-ينتج حمض الكاربونيك نتيجة ذوبان غاز ثاني اكسيد الكربون في الماء حيث يصبح الماء له تاثير حامضي .

-عند ملامسة الماء الحامضي للصخور يتفاعل معها ويعمل على اذابة ايونات البوتاسيوم والسيليكا من معادن الفلسبار مما يحول الفلسبار الى كالونايت .

*كيف يصبح الماء حامضي ؟

$$CO_2+H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^-$$

تكوين التربة :

Cations -> sheet-> basic unit -> clay mineral

sheet : يتشكل هذا التركيب نتيجة تفاعل الايونات مع الاوكسجين او مجموعة الهايدروكسيل .

OH O

توجد على شكل :

Si

1-octahedral ()تتكون رئيسيا من الجبس 2- tetrahedral (تتكون من السيلكا

AL

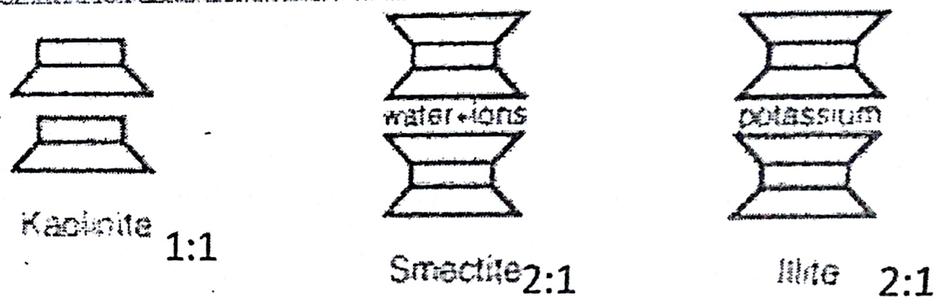
ارتباط ال sheets مع بعضها البعض بروابط تساهمية يؤدي الى انتاج ال* Basic unit

Basic unit : a) 1:1semi basic unit b) 2:1 semi basic unit .

Si AL

2.1 semi-basic unit

	Kaolinite	Montmorillonite	Illite



Shape	Platy shape	Film-like shape	Flaky shape
Basal spacing	7.2 A	9.6-INF (because of swelling	10 A
Bond	H-pond Vander waals force	Vander waals force	K- pond
swelling	No	Yes	No

Theisomorphouswill be occurs in smectite .

بعض المشاكل :

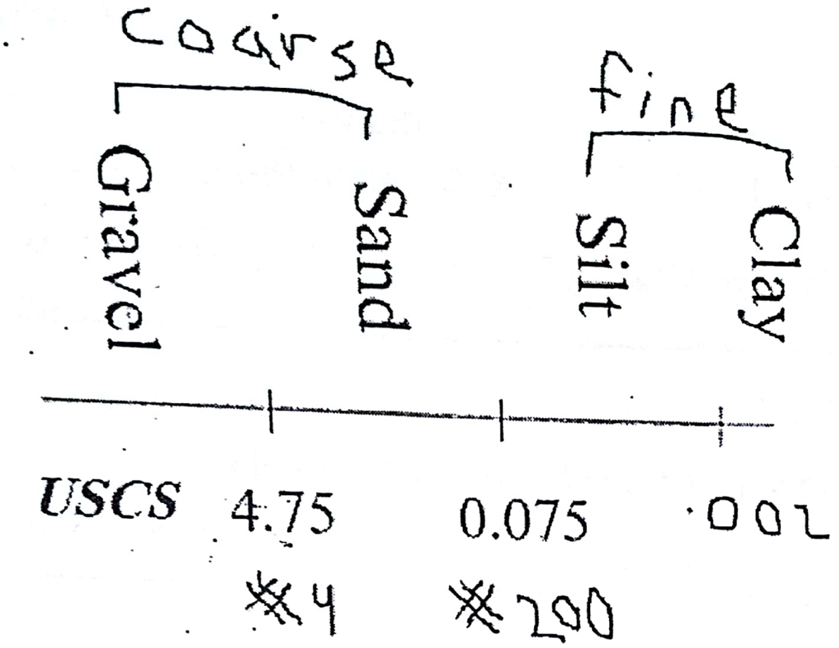
-تحل ايونات الالمنيوم (+٣) محل ايونات السيليكا (+٤) وكذلك قد يتم استبدال الالمنيوم(+٣) بالمغنيسيوم او الحديد (+٢) . ومن هنا ينشا (charge Deficiency) . تحدث في السمكتايل

-يتم التغلب على هذه المشكلة عن طريق ايونات البوتاسيوم اللتي تشكل روابط قوية .

هناك ايضا مشكلة الاحلال .. يجب قرائتها ..

#2 Soil Texture:

Soil Texture:its appearance or “feel” and it depends on the relative sizes and shapes .



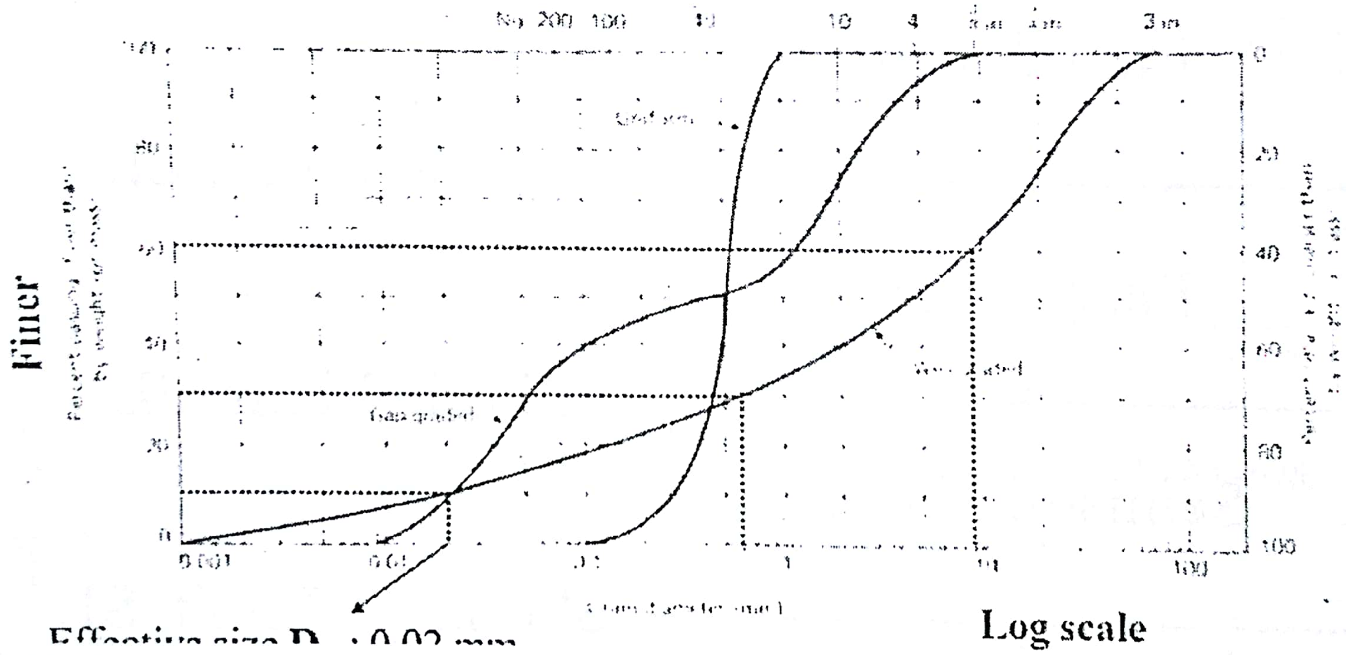
التركيز على الحدود وما تفصل بينه

To classify coarse soil we use sieve analysis

To classify fine soil we use Hydrometer analysis

Clay-size particles : : A small quartz particle may have the similar size of clay minerals.

For example: Kaolinite, Illite, etc.



D10: diameter corresponding to 10% passing. Effective size

D30: diameter corresponding to 30% passing.

D60: diameter corresponding to 60% passing.

Coefficient of uniformity

coefficient of curvature

$$C_u = \frac{D_{60}}{D_{10}} \quad C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$$

One size >>> Cu=1 >>> Cc=1

Particle shape.

Higher friction → angular ... ... lower friction → round soil particle

#3 atterberg limits and consistency index

Liquid State

Liquid Limit, LL

Plastic State

Plastic Limit, PL

Semisolid State

Shrinkage Limit, SL

Solid State

هي المحتوى المائي اللازم كي تتحول التربة من حالة الصلابة الى حالة شبه الصلابة : SL

هي المحتوى المائي اللازم كي تتحول التربة من حالة شبه الصلابة الى حالة plastic :PL

هي المحتوى المائي اللازم كي تتحول التربة من plastic الى الحالة السائلة :LL

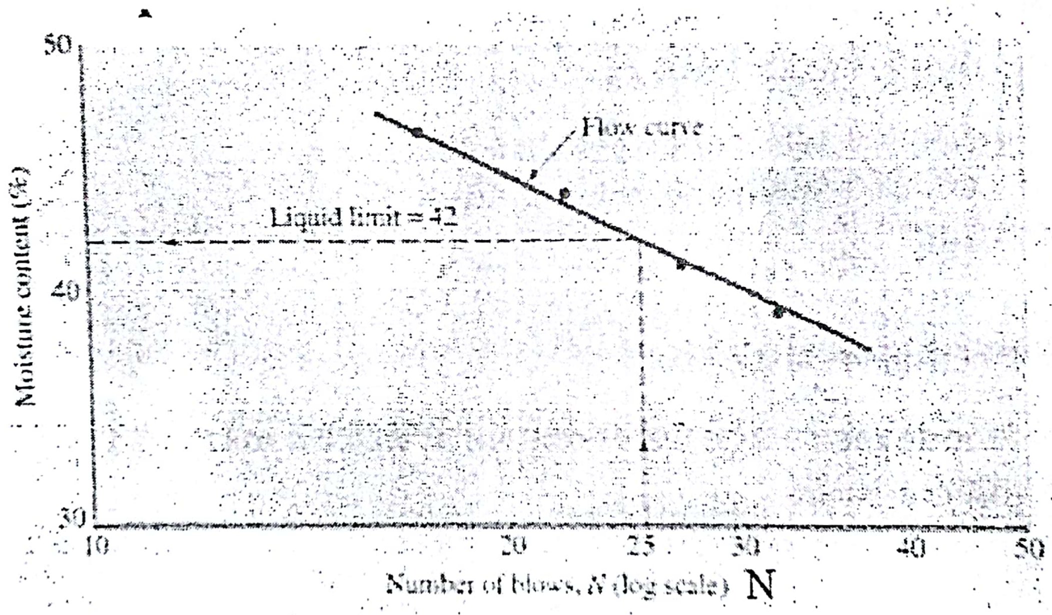
Determine LL :

a) Casagrande method
b) Cone Penetrometer Method

Particle sizes and water •Passing No.40 Sieve (0.425 mm). •Using deionized water.

-هي المحتوى المائي اللازم لاحداث منطقة مغلقة في العينة بطول نصف انش (١٢.٧ مم) بعد ضرب العينة ٢٥ ضربة .

Multi point :



$$Flow\ index, I_F = \frac{w_1 - w_2}{\log(N_2 / N_1)} (choose\ a\ positive\ value)$$

$$w = -I_F \log N + cont.$$

لا يشترط ان يكون عدد العينات اربعة .

•One-point Method

- Assume a constant slope of the flow curve.
- The slope is a statistical result of 767 liquid limit tests.

$$LL = w_n \left(\frac{N}{25}\right)^{\tan\beta}$$

$N = number\ of\ blows$

$w_n = corresponding\ moisture\ content$

$\tan\beta = 0.121$

Limitations:

- The β is an empirical coefficient, so it is not always 0.121.
- Good results can be obtained only for the blow number around 20 to 30.

Determine plastic limit :

PL is defined as the water content at which a soil thread with 3.2 mm diameter just crumbles.

LL,PL	Low	Med	High
SL	High	Med	Low
A	Low	Med	high

*see the width and depth

Plasticity index

PI For describing the range of water content over which a soil was plastic PI = LL – PL

•Liquidity index LI For scaling the natural water content of a soil sample to the Limits.

$$LI = \frac{w - PL}{PI} = \frac{w - PL}{LL - PL}$$

$w\ is\ the\ water\ content$

LI <0 (A). brittle fracture if sheared
0<LI<1 (B). plastic solid if sheared
LI >1 (C). viscous liquid if sheared

•Activity A

(Skempton, 1953)

$$A = \frac{PI}{\%\,\text{clay fraction(weight)}}$$

clay fraction: <0.002 mm

Normal clays: $0.75<A<1.25$

Inactive clays: $A<0.75$

Active clays: $A>1.25$

High activity:

- •large volume change when wetted
- •Large shrinkage when dried
- •Very reactive (chemically)

Mitchell, 1993

Soil Classification :

According to Unified Soil Classification System ..Four major divisions:

(1) Coarse-grained (2) Fine-grained (3) Organic soils (4) Peat

50 %

Coarse-grained soils: Gravel | Sand

Fine-grained soils: Silt Clay

50%

NO. 4 4.75 mm

NO.200 0.075 mm

•Grain size distribution

•C_u

•C_c

•PL, LL

•Plasticity chart

LL >50

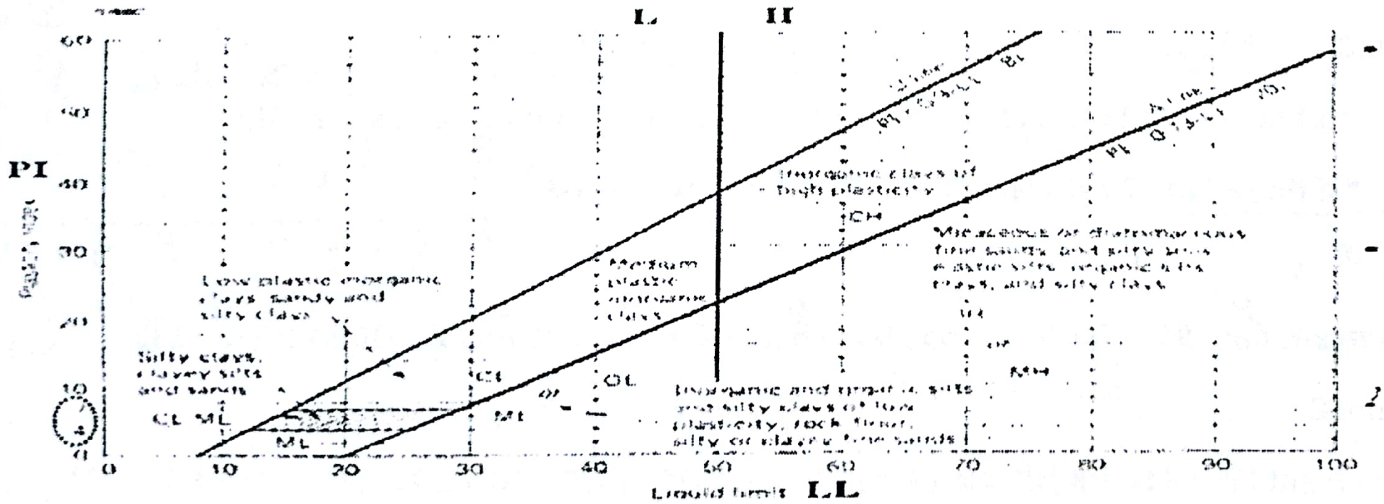
LL <50

Required tests: Sieve analysis

Atterberg limit

#

Corrse → { C_c , C_u }



u-line : upper pound of soil PI=0.9(LL-8) ... A-line upper pound of silt and organic soil .PI=.73(LL-20)

Soil symbols: G: Gravel S: Sand M: Silt C: Clay O: Organic Pt: Peat

Liquid limit symbols: H: High LL (LL≥50) L: Low LL (LL<50) Gradation symbols: W: Well-graded P: Poorly-graded

Example: SW, Well-graded sand SC, Clayey sand SM, Silty sand, MH, Elastic silt

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*Organic clay or silt(group symbol OL or OH):

"The soil's liquid limit (LL) after oven drying is less than 75 % of its liquid limit before oven drying." If the above statement is true, then the first symbol is O. The second symbol is obtained by locating the values of PI and LL <u>(not oven dried)</u> in the plasticity chart.

Example : if LL after oven dry = 72 & LL before oven drying = 100 clssify the soil

LL(after)/LL(before) = 72/100 =72% then the soil is organic

LL= ١٠٠>50 ... H ..... (OH)

**classify the soil :

1- See #200 .. a-IF percent passing > 50 the soil is fine.
.. b- IF percent passing < 50 the soil is coarse .

a- * see LL .. if>50 H .. IF < 50 L .
*calculate PI (LL-PL) & PI -A Line 0.73(LL-20)
*if PI = 4to 7 the soil is CL-ML
*if PI>PI-A the soil is C
*if PI<PI-A the soil is M

B- *if percent retained on #4 > 50 the soil is G

*if percent retained on #4 < 50 the soil is S

* see #200 .. if fine soil <5% calculate cc & cu .. the soil may be will or poor

*if fine soil > 12 calculate PI&PI-Aline the soil contain two type of soil ex SM

* if fine = 5to 12 calculate CC,CU,PI,PI-Aline ex GW-GM

Ex: a- percent finer #4= 100, #100= 65 ,#200=8 ,LL=30,PL=22,D10=0.085,D30=.12,D60=.135

SP-SC

b-- percent finer #4= 100, #100= 78 ,#200=58 ,LL=26,PL=20,D10=0.022,D30=.045,D60=.0933

CL-ML

5.3 Classify the following soils using the Unified soil classification system. Give group symbols and group names.

Soil No.	Sieve analysis (percent finer) No. 4	Sieve analysis (percent finer) No. 200	Liquid limit	Plasticity limit	Comments
1	94	3	—	NP (M)	$C_u = 4.48$ and $C_c = 1.22$
2	100	77	63	25	
3	100	86	55	28	
4	100	45	36	22	
5	92	48	30	8	
6	60	40	26	4	
7	99	76	60	32	

5.4 For an inorganic soil, the following grain-size analysis is given.

U.S. Sieve No.	Percent passing
4	100
10	90
20	64
40	38
80	18
200	13

For this soil, $LL = 23$ and $PL = 19$. Classify the soil by using:

1* Soil with 65% retained on #4, 32% on #200
LL = 10, PL = 2 classify the soil.

2* $e = 0.7$ $G_s = 2.72$ calculate:
• γ_d • γ_{sat} • γ' • unit weight & w at $S_r = 0.75$

3* $e = 0.62$, $w = 15\%$, $\rho_s = 2.65 g/cm^2$ calculate:
• ρ_t • ρ_d • n • S_r • w at $S_r = 100\%$

4* specimen weighs = 1.8 kN, $V = 0.1 m^2$, $w = 12.6\%$, $G_s = 2.71$, calculate
• moist unit weight. • void ratio. • Degree of saturation
• Dry unit weight. • porosity.

* moist unit weight = 19.2 kN/m³ , Gs = 2.69 , w = 9.8% compute

• n • γ_d • Sr

* $\gamma_{d\,min}$ = 14.2 , $\gamma_{d\,max}$ = 17.1 , Dr = 70% , w = 8%..
calculate the moist unit weight of the sand.

* γ = 16.98 kN/m³ , Dr = 82% , w = 8% , Gs = 2.65 , e_{min} = 0.44

• e_{max} = ??

• $\gamma_{d\,min} = \dfrac{G_s \gamma_w}{1+e_{max}}$

* V_t = 0.05 m³ , mass = 87.5 , w = 15% , Gs = 2.68

• e • γ_d • moist unit weight

• n • Sr

1* γ_{sat} = 20.1 kN/m² at w = 22% calculate

• Dry unit weight • Gs

0* moist unit weight = 18.79 kN/m³ , w = 12% , Gs = 2.65 calculate!

• void ratio • porosity

• Degree of saturation. • Dry unit weight.

* a saturated soil specimen has w = 36% and γ_d = 13.43 calculate.

• void ratio • porosity

• specific gravity. • saturated unit weight (in lb/ft³)

!* γ = 18.34 kN/m³ , Dr = 82% , w = 8% , Gs = 2.65

e_{min} = 0.49

• e_{max} = ?? • dry unit weight in the loosest state.

3*

#	A	B	C	D	E	F
4	100	100	95	95	100	100
10	95	80	80	90	94	94
40	82	61	54	79	76	86
200	65	55	8	64	33	76
LL	42	38	NP	35	38	52
PL	26	25	NP	26	25	28

10 * The end

2. Soil Texture

٩ / ٢٨

2. Soil Texture

2.1 Soil Texture

Sand, gravel

ما تقدر أشوفه
clay

تحسه

أقدر أشوفه
ملمس

The texture of a soil is its appearance or "feel" and it depends on the relative sizes and shapes of the particles as well as the range or distribution of those sizes.

sieve 200 (Diameter 0.075)

Coarse-grained soils: | Fine-grained soils:

Gravel Sand | Silt Clay

0.075 mm (USCS) -0.06 mm BS

Sieve No #200

Sieve analysis — Hydrometer analysis

Course ← | → Fine-grained Soils

Sieve analysis

محجوزة بالمنخل الطويل
Sieve #200
Diameter (0.075 mm)

hydrometer analysis

2.2 Characteristics

TABLE 2-2 Textural and Other Characteristics of Soils (Holtz and Kovacs, 1981)

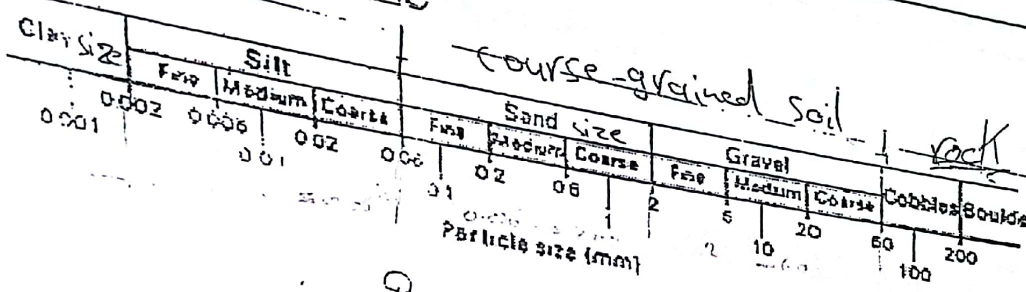
Soil name: نفس السلوك	Gravels, Sands	Silts	Clays
Grain size: حبوب	Coarse grained فردية Can see individual grains by eye	Fine grained Cannot see individual grains	Fine grained Cannot see individual grains
Characteristics:	Nonplastic Granular حبيبية	Nonplastic Granular	مرن Plastic
Effect of water on engineering behavior:	Relatively unimportant (exception: loose saturated granular materials and dynamic loadings)	Important	Very important
Effect of grain size distribution on engineering behavior:	Important	Relatively unimportant	Relatively unimportant

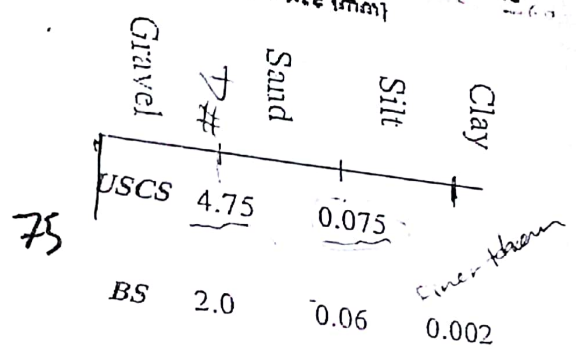
Course - grained Soil

3. Grain Size and Grain Size Distribution

clay
↓
Plastic

3.1 Grain Size





USCS: Unified Soil Classification

BS: British Standard

Note:

Clay-size particles < 0.002

For example:

A small quartz particle may have the similar size of clay minerals.

Clay minerals

For example:

Kaolinite, Illite, etc.

3.2 Grain Size Distribution

Sieve No | mass Retained
"3 | 0
#4

- **Sieve size**

▼ TABLE 1.5 U.S. Standard Sieve Sizes

Sieve no.	Opening (mm)
4	4.75
5	4.00
6	3.35
7	2.80
8	2.36
10	2.00
12	1.70
14	1.40
16	1.18
18	1.00
20	0.850
25	0.710
30	0.600
35	0.500
40	0.425
50	0.355
60	0.250
70	0.212
80	0.180
100	0.150
120	0.125
140	0.106
170	0.090
200	0.075
270	0.053

(Das, 1998)

Table 4.5(a). METRIC SIEVES (BS)

Construction	Aperture size: Full Set (A)	'Standard' set (B)	'Short' set (C)
Perforated steel plate (square hole)	75 mm	+	
	63	+	+
	50		
	37.5	+	
	28		
	20	+	+
	14		
	10	+	
	6.3	+	+
	5		
	3.35	+	
	2	+	+
	1.18	+	
	600 µm	+	+
	425		
	300	+	
	212		+
	150	+	
	63	+	+
Lid and receiver	+	+	+
	19 sieves	13 sieves	7 sieves

(Head, 1992)

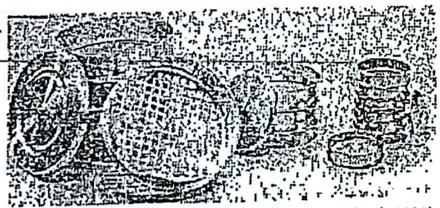
3.2 Grain Size Distribution (Cont.)

- **Experiment**

Coarse-grained soils: | Fine-grained soils:

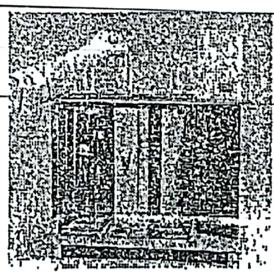
Gravel Sand relative | Exposed Silt Clay

0.075 mm (USCS)



(Head, 1992)

sieve analysis



Hydrometer analysis

sieve #40

تعرّض

0.5 : 6

حرارة عيارية معينة 1

cylinder 1 2 3

distilled water   soil mass

أزمان مختلفة

3.2 Grain Size Distribution (Cont.)

100% - % retained

Passed %

R

uniform = poor graded

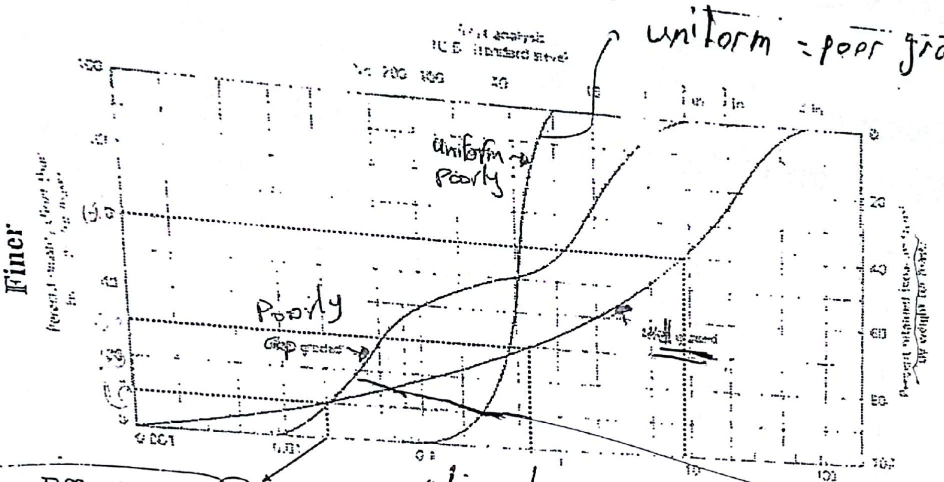


Fig 2.4 Typical grain size distributions

diameter corresponding to 10 % finer

Effective size D_{10}: 0.02 mm

D_{30}: D_{60}:

(Holtz and Kovacs, 1981)

D_{10}: Diameter Corresponding to 10% Finer

D_{30}, D_{60} same

3.2 Grain Size Distribution (Cont.)

- Describe the shape

Example: well graded

$D_{10} = 0.02\,\text{mm}$ (effective size)

$D_{30} = 0.6\,\text{mm}$

$D_{60} = 9\,\text{mm}$

انتظام

Coefficient of uniformity (C_u)

$$C_u = \frac{D_{60}}{D_{10}} = \frac{9}{0.02} = 450$$

$C_u = \frac{D60}{D10}$

انحناء

Coefficient of curvature (C_c)

$$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = \frac{(0.6)^2}{(0.02)(9)} = 2$$

$C_c =$

- Criteria

retained on seive # 200

Well-graded soil

$1 < C_c < 3$ and $C_u \geq 4$

(for gravels)

لازم يتحقق الشرطين

$1 < C_c < 3$ and $C_u \geq 6$

(for sands) → pass # 200

(Poorly) ← إذا لم يتحقق الشرطين

- Question

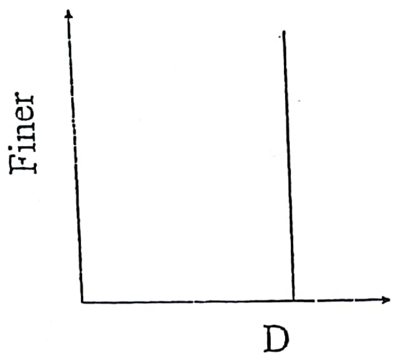
What is the C_u for a soil with only one grain size?

Poorly $C_u = 1$

Answer

• Question

What is the C_u for a soil with only one grain size?



Grain size distribution

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = 1$$

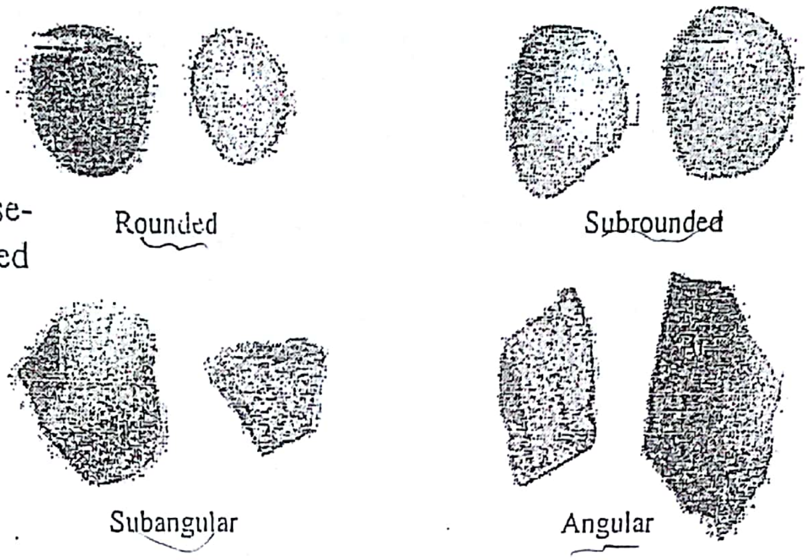
3.2 Grain Size Distribution (Cont.)

- **Engineering applications**
  - It will help us "feel" the soil texture (what the soil is) and it will also be used for the soil classification (next topic).
  - It can be used to define the grading specification of a drainage filter (clogging).
  - It can be a criterion for selecting fill materials of embankments D_{85} and earth dams, road sub-base materials, and concrete aggregates.
  - It can be used to estimate the results of grouting and chemical injection, and dynamic compaction.
  - Effective Size, D_{10}, can be correlated with the hydraulic conductivity (describing the permeability of soils). (Hazen's Equation).(Note: controlled by small particles)

The grain size distribution is more important to **coarse-grained** *soils.*

4. Particle Shape

Coarse-grained soils



(Holtz and Kovacs, 1981)

- Important for granular soils
- Angular soil particle → higher friction → shear strength, parameter
- Round soil particle → lower friction
- Note that clay particles are sheet-like.

fine (clay) →

sheet

شرائح

Voids ratio

$e = \frac{V_v}{V_s}$ as decimal
* for sand [0.4 - 1]
* for clay [0.3 - 1.5]

* Porosity

$n = \frac{V_v}{V_T} \times 100\%$

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

* degree of saturatain

$Sr = \frac{V_w}{V_v} \times 100\%$

* specific Volume
total Volume of soil that contains unit Volume of solid

$V = 1+e$

(Air contants (air voids)

$A = \frac{V_a}{V_T} \times 100\% \Rightarrow$

water content (moisture degree (w))

$w = \frac{M_{water}}{M_{solid}} \times 100\%$

① bulk density [total density (ρ or ρ_T)]

$\rho = \frac{M_T}{V_T}$

② dry density (ρ_d)

$\rho_d = \frac{M_s}{V_T}$

③ solid density

$\rho_s = \frac{M_s}{V_s}$

④ saturated density (ρ_{sat})

$\rho_{sat} = \frac{M_s + M_w}{V_T} = \frac{M_T}{V_T}$

* when Volume of Voids = Volume of water

$Sr = \frac{V_w}{V_v} \times 100\% = 1$

specific gravity of soil part.

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

Derived Relations

① $Sr \cdot e = \overset{\text{water content}}{w} G_s$

② $A = n(1 - S_r)$

$A = \frac{e - wG_s}{1+e}$

③ bulk density

$\rho = \frac{G_s + Sr \cdot e}{1+e}$

for fully saturated soil

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

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

* unit weight γ

$\gamma = \frac{W}{V}$

$\gamma = \frac{G_s + Sr\,e}{1+e} \gamma_w$

$\frac{G_s - 1}{1+e}$

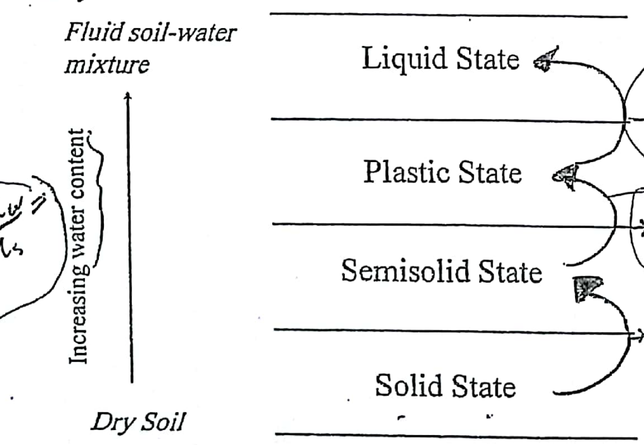
relative density (I_r)

$I_r = \frac{e_{max} - e_{field}}{e_{max} - e_{min}}$

5. Atterberg Limits
and
Consistency Indices

4.1 Atterberg Limits

The presence (وجود) of water in fine-grained soils can significantly affect associated engineering behavior, so we need a reference index to clarify the effects



I'm interested in

of Upper limit For

: water Content that Make the Soil be semisolid From Solid

$\frac{M_w}{M_s} \times 100\%$ =

كمية الماء / كمية التربة ovendry

هي المحتوى المائي اللازم لتحويل التربة من حالة " " إلى حالة " " .

the water content, In percentage, required to close a distance 0.5 in. along bottom of the groove 25 blows.

water content	No. of blows
W/C	N
WC_1	$N_1=34$
WC_2	$N_2=29$
WC_3	$N_3=21$
WC_4	$N_4=14$

هو المحتوى المائي لإحداث منطقة مغلقة من العينة بطول نصف انش (١٠٢٧ ملم) بعد ٢٥ ضربة

تعريف LL

4.2 Liquid Limit-LL

Casagrande Method 2 MPa (shear strength)

(ASTM D4318-95a)

- Professor Casagrande standardized the test and developed the liquid limit device.
- 1 Multipoint test
- 2 One-point test

Cone Penetrometer Method

(BS 1377: Part 2: 1990:4.3)

- This method is developed by the Transport and Road Research Laboratory, UK.
- Multipoint test
- One-point test

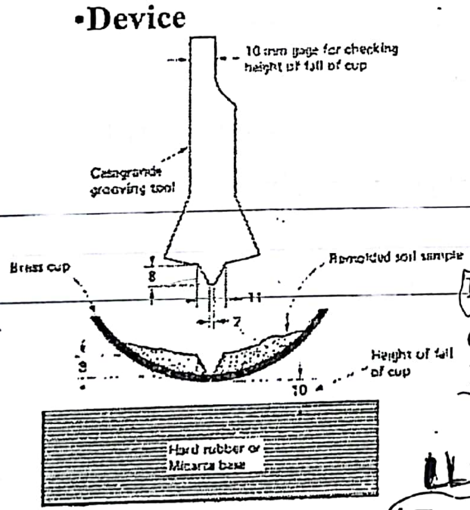
Particle sizes and water

- **Passing No.40 Sieve (0.425 mm).**
- **Using deionized water.** ماء منزوع الأيونات ← للحد من كمية الأيونات

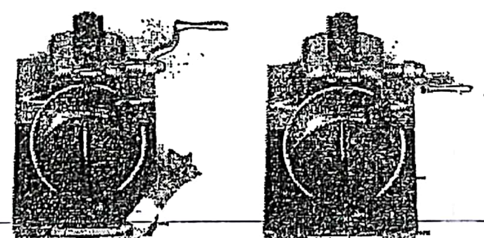
  The type and amount of cations can significantly affect the measured results. التركيز تؤثر على النتيجة

4.2.1 Casagrande Method

•Device

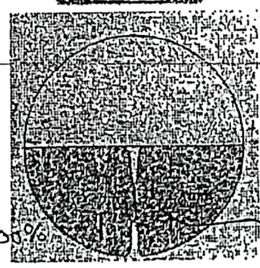


(Holtz and Kovacs, 1981)



N=25 blows

Closing distance = 12.7mm (0.5 in)



closed

LL $\frac{M_w}{M_m} \times 100\%$

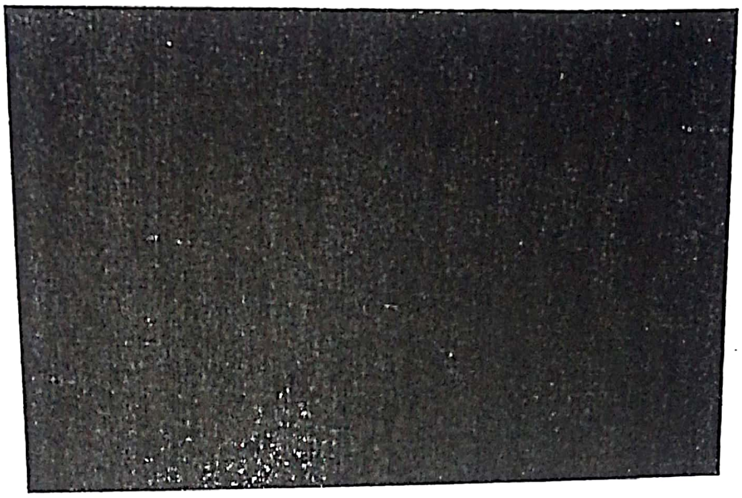
The water content, in percentage, required to close a distance of 0.5 in (12.7mm) along the bottom of the groove after 25 blows is defined as the liquid limit

عدد الضربات

بس ثابتين

متغيرين عدد الضربات closed

4.2.1 Casagrande Method (Cont.)

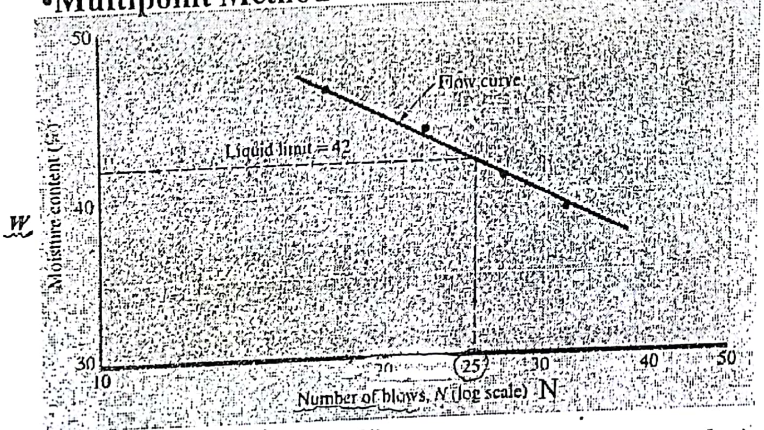


Reference: Budhu: Soil Mechanics and Foundation

4.2.1 Casagrande Method (Cont.)

* نحتاج لرقم ثابت لمعرفة LL

•Multipoint Method



Das, 1998

inverse linear

Flow index, $I_F = \dfrac{w_1 - w_2}{\log(N_2/N_1)}$ *(choose a positive value)*

$w = -I_F \log N + cont.$ لا يشترط أن يكون عدد العينات أربعة

trial	No. of blows	Wc
1	N_1	W_{c1}
2	N_2	W_{c2}
3	N_3	W_{c3}

* 21 ضربة ______ LL

* 33 ضربة ______ PL

4.2.1 Casagrande Method (Cont.)

•One-point Method

- Assume a constant slope of the flow curve.
- The slope is a statistical result of 767 liquid limit tests.

$$LL = w_n \left(\frac{N}{25} \right)^{\tan\beta}$$

$N = number\ of\ blows$

$w_n = corresponding\ moisture\ content$

$\tan\beta = 0.121$

Limitations:

- The β is an empirical coefficient, so it is not always 0.121.
- Good results can be obtained only for the blow number around 20 to 30.

4.3 Plastic Limit-PL

(Holtz and Kovacs, 1981)

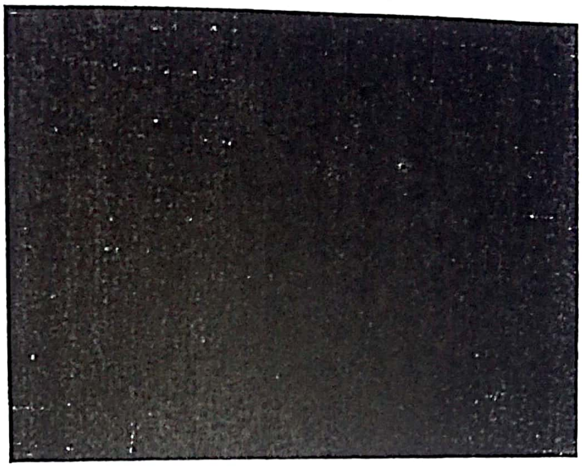
The plastic limit PL is defined as the water content at which a soil thread with *3.2 mm diameter just* crumbles. (1/8)"

ASTM D4318-95a, BS1377: Part 2:1990:5.3

— Montmorill أعلى LL PL

رطوبة طبيعية ، مساحة سطح عالية . Wc 100-900% 50-100%

4.3 Plastic Limit-PL (cont)



4.4 Typical Values of Atterberg Limits

Table 10.1 Atterberg Limit Values for the Clay Minerals.

Mineral[a]	Liquid Limit (%) (LL)	Plastic Limit (%) (PL)	Shrinkage Limit
Montmorillonite	100–900	50–100	8.5–15
Nontronite	37–72	19–27	
Illite	60–120	35–60	15–17
Kaolinite	30–110	25–40	25–29
Hydrated Halloysite	50–70	47–60	
Dehydrated Halloysite	35–55	30–45	
Attapulgite	160–230	100–120	
Chlorite	44–47	36–40	
Allophane (undried)	200–250	130–140	

(Mitchell, 1993)

عالي

	Koalinite	Illite	Montmorillonite (smectite)
LL, PL	Low	med.	High
SL	High	med.	Low
A	Low	Med.	High

$\frac{w-PL}{LL}$

LI سلوك المادة حسب قيمة

4.6 Indices

•Plasticity index PI

For describing the range of water content over which a soil was plastic

$\boxed{PI = LL - PL}$

Liquid State C

Liquid Limit, LL

PI *Plastic State* B

Plastic Limit, PL

Semisolid State A

Shrinkage Limit, SL

Solid State

•Liquidity index LI

For scaling the natural water content of a soil sample to the Limits. at site

$$LI = \frac{w_n - PL}{PI} = \frac{w_n - PL}{LL - PL}$$

w is the water content Natural

solid → متشقق

LI <0 (A), brittle fracture if sheared

0<LI<1 (B), plastic solid if sheared

LI >1 (C), viscous liquid if sheared

liquid

4.6 Indices

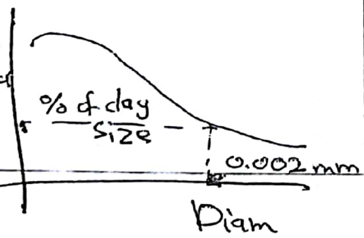
③ soil Activity

•Activity A قابلية للانتفاخ

(Skempton, 1953)

$$A = \frac{PI \ (LL-PL)}{\%\,\text{clay fraction (weight)}}$$

clay fraction: <0.002 mm



Normal clays: 0.75<A<1.25

Inactive clays: A<0.75

Active clays: A> 1.25

High activity:

•large volume change when wetted ⇒ Volume increase

•Large shrinkage when dried → Volume decrease

•Very reactive (chemically) Mitchell, 1993

•Purpose الكمية النوع

Both the *type* and *amount* of clay in soils will affect the Atterberg limits. This index is aimed to separate them.

Table 10.4—Activities of Various Clay Minerals.

Mineral	Activity[a]
Smectites	1–7
Illite	0.5–1
Kaolinite	0.5
Halloysite (2H$_2$O)	0.5
Halloysite (4H$_2$O)	0.1
Attapulgite	0.5–1.2
Allophane	0.5–1.2

expansive ⇔ shrinkage
Swelling

• Smectite has the highest activity, why ??

↑ plasticity ⇒ permeability ↓ ⇒ Compressibility ↓

4.7 Engineering Applications

- Soil classification
  (the next topic)

– The Atterberg limit enable clay soils to be classified.

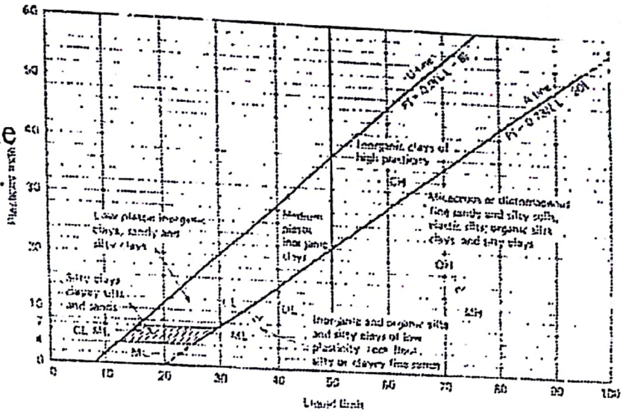


Fig. 3.2 Casagrande's plasticity chart, showing several representative soil types (developed from Casagrande, 1948, and Howard, 1977)

- The Atterberg limits are usually correlated with some engineering properties such as the permeability, compressibility, shear strength, and others.
  - In general, clays with high plasticity have lower permeability, and they are difficult to be compacted.
  - The values of SL can be used as a criterion to assess and prevent the excessive cracking of clay liners in the reservoir embankment or canal.

5. References

Main References:

Craig's Soil Mechanics 7th edition

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to GeotechnicalEngineering* Prentice Hall. (Chapter 1 and 2)

Others:

Head, K. H. (1992). *Manual of Soil Laboratory Testing, Volume 1: Soil Classification and Compaction Test*, 2nd edition, John Wiley and Sons.

Lambe, T.W. (1991). Soil Testing for Engineers, BiTech Publishers Ltd.

Mitchell, J.K. (1993). *Fundamentals of Soil Behavior*, 2nd edition, John Wiley & Sons.

Das, B.M. (1998). *Principles of Geotechnical Engineering*, 4th edition, PWS Publishing Company. (Chapter 2)

- Budhu M. (2007)"Soil Mechanics and Foundations" Wiley, New York

Rock + water → clay

$CO_2 + H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^- \rightarrow$ إذا لامس الصخور يتفاعل معها ويعمل
على إذابة أيونات البوتاسيوم والسيليكا من معادن الفلسبار مما يحول الفلسبار إلى الكالونايت ← Ex: Granite

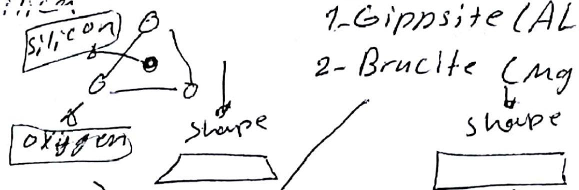
clay minerals ← Basic units ← sheet ← cations

1:1 semi basic unit

2:1 seme basic unit

Tetrahedral (Plural) — ~~Gibbsite~~ Silicon sheet)

Octahedral (~~silica~~ sheet)
1- Gippsite (Al^{+3})
2- Brucite (Mg^{2+})

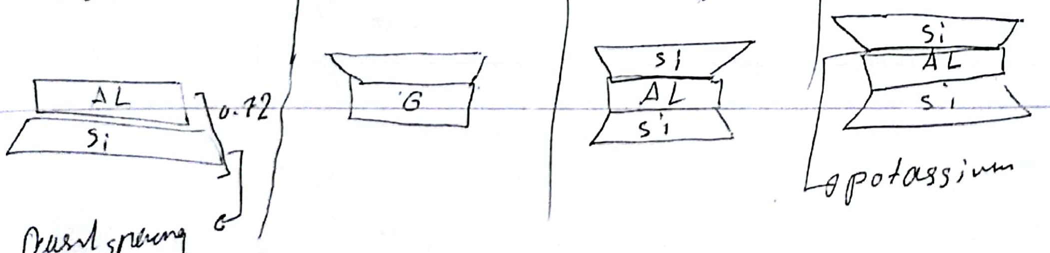


physical weathering → sand, Gravel

chemical weathering → silt, clay

seme basic units → clay minerals

Kaolinite ①	② Halloysite (water)	③ Montmorillonite	Illite (Al, Mg, Fe)
Basal spacing → 7.2 A°	hydrated: 10.1 dehydrated: 7.2	→ 9.6 A° → ∞	→ 10 A°
shape → platy	Tubular	→ Film-Like shape	→ Flaky
Bonding between layers → van der walls forces and Hydrogen bonds strong Bonding	irreversible	→ van der walls forces and cations (weak bonding)	→ strong Bonding ~~stacking~~
swelling → NO	→ No	→ yes important for engineering practice (expansive clay)	→ NO
width → 0.1 ~ 4 Mm		→ 1 or 2 Mm	→ 0.1 Mm
Thikness → 0.05 ~ 2 Mm		→ 10 ~ 1/100 A°	→ 30 A°



[redacted]

Best wishes
Ali Adel

Ex:- Soil sample has natural water content 20% , Liquid limit 50% & plastic limit = 15%

If the percent of clay fraction is 30%

1) what do you expect the sample will behave during shearing test ? & (draw the expect stress - strain curve)

2) Is the sample will undergo large volume change upon wetting ? → clay fraction غيرها → 22.5%

Sol:-

① $LI = \frac{w - PL}{LL - PL} = \frac{20 - 15}{50 - 15} = \frac{5}{35}$ → $0 < \frac{5}{35} < 1$ → soil will behave plastic

② $A = \frac{PI}{\%\ \text{clay fraction}} = \frac{50 - 15}{22.5} = \frac{35}{22.5} = 1.55 > 1.25$

∴ Soil will behave large volume chang upon wetting [illegible]

upon dry [illegible]

Q: which soil contains more clay particles ?-

clay [illegible] PI [illegible]

$I = LL - PL$

$LL > PL > SL$

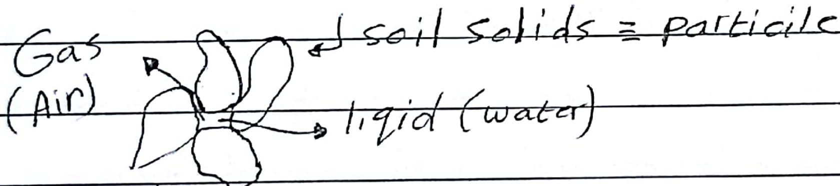
Subject soil | 8/3 12/4 | Date | No.

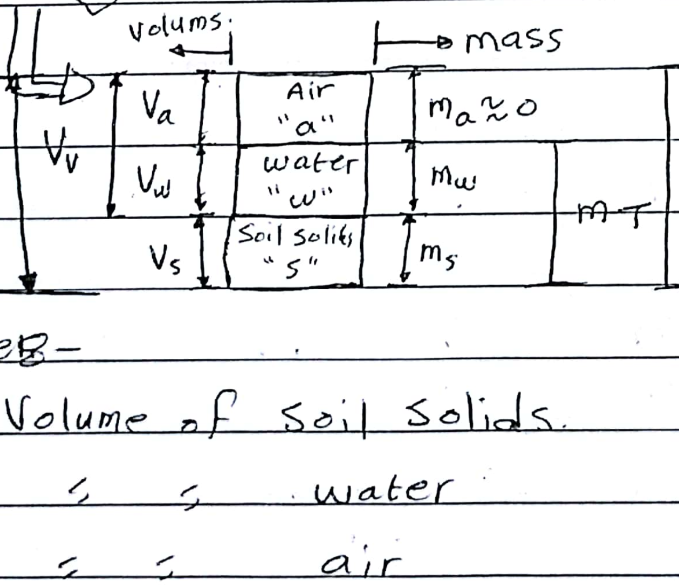
*phase Relationships

- Introduction
- Ratios :
  - volume/volume
  - mass/mass
  - mass/Volume
- Derivatives formula.

Introduction :-

Soil :- Any accumulation of mineral. The voids bettween the particles contain either gas or liquid (both)





$M_T = m_s + m_w$

$V_T = V_v + V_s$

$= V_w + V_a + V_s$

$\therefore V_v = V_w + V_a$

where:-

V_s = Volume of soil solids.

V_w = 〃 〃 water

V_a = 〃 〃 air

V_v = Volume of voids.

V = Volume Total

m_s = mass of soil solids.

m_w = 〃 〃 water

m_T = Total mass.

NOTEBOOK

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Volume - Volume Ratios :-

- Void Ratio (e)

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

~~ناقصية~~

- Porosity (n)

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

درجة التشبع

- Degree of Saturation ($S = S_r$)

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

جاف

$S = 0 \rightarrow$ soil is totally dried

رطب

$S = 1 \rightarrow$ soil is fully saturated

- Air Content (A)

$A = \frac{V_a}{V_T}$

- specific Volume (ν)

if $V_s = 1$

$\nu = 1 + e \Rightarrow e = \frac{V_v}{V_s} \Rightarrow \frac{V_v}{1} = V_v \therefore \boxed{e = V_v}$

$\boxed{Se = G_s w} \Rightarrow$ العلاقة الذهبية

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mass _ mass Ratios.

- water content :-
- moisture contet (ω)

$$\omega = \frac{m_\omega}{m_s}$$

- specific Gravity (G_s). قاعدة ارخميدس

$$G_s = \frac{\text{mass of soil solids}}{\text{mass of displaced water}} = \frac{\rho_s}{\rho_\omega}$$

- Densities :-
- Density of water (ρ_ω)

$$\rho_\omega = \frac{m_\omega}{V_\omega}$$

⇒ if ρ_ω is not given use $\rho_\omega = 1000\ kg/m^3$
$= 1\ g/cm^3$

- (Bulk = Total), wet, moist) density $(\rho = \rho_T)$

$$\rho = \frac{m_T}{V_T}$$

- Density of soil Solids (ρ_s)

$$\rho_s = \frac{m_s}{V_s}$$

- Dry Density $(\rho_d = \rho_{dry})$ ($S = 0$ يعني نسبة الماء بالتربة = صفر)

$m_s = m_{dray}$

$$\rho_{dry} = \frac{m_s}{V_T}$$

NOTEBOOK

Subject

- Saturated density : (ρ_{SAT})

$$\rho_{SAT} = \frac{m_T}{V_T} \quad m_T \rightarrow m_s + m_w \quad (S = 100\%)$$

هون الفراغات كلها
ماء عشان هيك
حكينا انو
$V_v = V_w$

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

$$m_w = \rho_w V_v \quad \boxed{V_v = V_w}$$

فعالية الكثافة

- Bouyant Density - Effective Density

$$\rho' = \rho_{sat} - \rho_w$$

EX 1 :- Given :- $m_T = 2290$ g $\quad m_{dry} = m_s = 2035$

$V_T = 1.105 \times 10^{-3}$ m^3 $\quad G_s = 2.68$

Determain :- $\rho, \gamma, \omega, e, n, S, A$

V_A	A	?	m_T
V_w	ω	m_w	
V_s	S	m_s	

$m_w = m_T - m_s = 2290 - 2035 = 255$ g

① $\rho_w = \frac{m_w}{V_w} \Rightarrow V_w = \frac{m_w}{\rho_w} \quad \rho_w = 1000$ kg/m^3

$\Rightarrow V_w = \frac{255/1000}{1000}$ kg لتحويل لل $\Rightarrow V_w = 255 \times 10^{-6}$ m^3

$G_s = \frac{\rho_s}{\rho_w} \Rightarrow \rho_s = G_s \rho_w = 2.68 \times 1000$ kg/m^3

but $\rho_s = \frac{m_s}{V_s} \Rightarrow V_s = \frac{m_s}{\rho_s} = \frac{2035/1000}{2.68 \times 1000} = 7.59 \times 10^{-4}$ m^3

N O T E B O O K

Subject Date No. 29

$V_a = V_T - V_s - V_w = 1.15 \times 10^{-3} - 7.59 \times 10^{-3} - 0.285 \times 10^{-3}$

$= 1.36 \times 10^{-4}\ m^3$

$V_v = V_A + V_w = 1.36 \times 10^{-4} + 255 \times 10^{-6} = 3.91 \times 10^{-4}\ m^3$

- $\rho = \dfrac{m_T}{V_T} = \dfrac{2.290}{1.15 \times 10^{-3}} = 1.991\ kg/m^3$

- $\gamma = \rho . g = \dfrac{w}{\cancel{V_T}} = \dfrac{1991 \times 9.81}{1000} = 19.53\ kN/m^3$

- $w = \dfrac{m_w}{m_s} = \dfrac{225}{2035} = 0.125$

- $e = \dfrac{V_v}{V_s} = \dfrac{3.91 \times 10^{-4}}{7.59 \times 10^{-4}} = 0.515$

- $n = \dfrac{V_v}{V_T} = \dfrac{3.91 \times 10^{-4}}{1.15 \times 10^{-3}} = 0.34$

- $S = \dfrac{V_w}{V_v} = \dfrac{225 \times 10^{-6}}{3.91 \times 10^{-4}} = 0.65$

- $A = \dfrac{V_a}{V_T} = \dfrac{1.36 \times 10^{-4}}{1.15 \times 10^{-3}} = 0.118$

Subject

فرض مهم جداً للاشتقاقات حبا

$\boxed{\text{Let } V_s = 1} \Rightarrow e = \frac{V_v}{V_s} = V_v \quad \therefore V_T = 1 + e$

↳ specific volume

m_s

$G_s = \frac{\rho_s}{\rho_w}$

but ; $\rho_s = \frac{m_s}{V_s} = \frac{m_s}{1} \quad \therefore \rho_s = m_s$

$G_s = \frac{m_s}{\rho_w} \longrightarrow m_s = \rho_w G_s$

$w = \frac{m_w}{m_s} \longrightarrow m_w = w\, m_s = w \rho_w G_s$

$\therefore M_T = m_s + m_w = G_s \rho_w + w G_s \rho_w$

$= G_s \rho_w (1 + w)$

	A		
wG_s	W	$wG_s\rho_w$	
	S	$G_s\rho_w$	

اشتقاق بطريقة اسهل

$\rho = \frac{m_T}{V_T} = \frac{G_s \rho_w (1+w)}{1+e} \longrightarrow \frac{\rho}{1+w} = \frac{G_s \rho_w}{1+e}\frac{(1+w)}{1+w}$

$\rho_{dry} = \frac{m_s}{V_T} = \frac{G_s \rho_w}{1+e} = \frac{\rho}{1+w}$

$\frac{\rho}{1+w} = \frac{G_s \rho_w}{1+e} \Rightarrow \rho = \rho_{dry}(1+w)$

NOTEBOOK

$$\rho_s = \frac{m_s}{V_s} = \frac{G_s \rho_w}{1} = G_s \rho_w$$

$$n = \frac{V_v}{V_T} = \frac{e}{1+e} \qquad \rho_w = \frac{m_w}{V_w} \Rightarrow V_w = \frac{w G_s \cancel{\rho_w}}{\cancel{\rho_w}}$$

$$S = \frac{V_w}{V_v} = \frac{w G_s}{e} \longrightarrow Se = w G_s$$

$$A = \frac{V_a}{V_T} = \frac{V_v - V_w}{V_T} = \frac{e - wG_s}{1+e} = \frac{e}{1+e} - \frac{wG_s}{1+e}$$

$$\Rightarrow n - \frac{Se}{1+e} \Rightarrow n - S\left(\frac{e}{1+e}\right) = n - Sn = n(1-S)$$

$$* \quad \rho = \frac{G_s \rho_w (1+w)}{1+e}$$

$$\Rightarrow \frac{m_T}{V_T} = \frac{m_s + m_w}{V_v + V_s} = \frac{m_s\left(1 + \frac{m_w}{m_s}\right)}{V_s\left(\frac{V_v}{V_s} + 1\right)}$$

$$\Rightarrow \frac{m_s}{V_s}\left(\frac{1+w}{1+e}\right) \Rightarrow \rho_s\left(\frac{1+w}{1+e}\right)$$

$$\Rightarrow G_s \rho_w \left(\frac{1+w}{1+e}\right)$$

$$\rho = \frac{\rho_w (G_s + Se)}{1+e}$$

$S = 0$ $\qquad\qquad$ $S = 1$

$$\rho_{dry} = \frac{\rho_w G_s}{1+e} \qquad \rho_{SAT} = \frac{\rho_w (G_s + e)}{1+e}$$

Subject صفوات: Date No.

EX :- $\rho = 1.76\ Mg/m^3 = 1760\ kg/m^3$

$\rho_s = 2.7\ Mg/m^3 = 2700\ kg/m^3 \qquad w = 10\%$

Sol :-

1) $\rho_{dry} = \rho = \dfrac{\rho}{1+w} = \dfrac{1760}{1+\frac{10}{100}} = 1600\ kg/m^3$

2) Void ratio :-

$\rho_{dry} = \dfrac{\rho_s}{1+e} \Rightarrow e = \dfrac{\rho_s}{\rho_{dry}} - 1 \Rightarrow \dfrac{2700}{1600} - 1 = 0.68$

3) Porosity :-

$n = \dfrac{e}{1+e} = \dfrac{0.68}{1+0.68} = 0.407$

4) degree of saturation S :-

$Se = G_s w \Rightarrow G_s = \dfrac{\rho_s}{\rho_w} = \dfrac{2700}{1000} = 2.7$

$S = \dfrac{2.7 * 0.1}{0.68} = 0.39$

5) Air content :-

$A = n(1-S) \Rightarrow 0.407(1-0.39) = 0.248$

6) Saturated density :-

$\rho_{SAT} = \dfrac{G_s \rho_w (1+w)}{1+e} \Rightarrow \dfrac{\rho_w (G_s + w G_s)}{1+e}$

but $se = G_s w$

حلص للتدريب

2.3 SOLUTION OF PHASE PROBLEMS

Phase problems are very important in soils engineering, and in this section, with the help of some numerical examples, we illustrate how most phase problems can be solved. As is true for many disciplines; practice helps; the more problems you solve, the simpler they become and the more proficient you will become. Also, with practice you soon memorize most of the important definitions and relationships, thus saving the time of looking up formulas later on.

Probably the single most important thing you can do in solving phase problems is *to draw a phase diagram.* This is especially true for the beginner. Don't spend time searching for the right formula to plug into. Instead, always draw a phase diagram and show both the given values and the unknowns of the problem. For some problems, simply doing this leads almost immediately to the solution; at least the correct approach to the problem is usually indicated. Also, you should note that there often are alternative approaches to the solution of the same problem as illustrated in Example 2.2.

EXAMPLE 2.2

Given:

$\rho = 1.76\ \text{Mg/m}^3$ (total density)

$w = 10\%$ (water content)

Required:

Compute ρ_d (dry density), e (void ratio), n (porosity), S (degree of saturation), and ρ_{sat} (saturated density).

Solution:

Draw the phase diagram (Fig. Ex. 2.2a). Assume that $V_t = 1\ \text{m}^3$.

From the definition of water content (Eq. 2-5) and total density (Eq. 2-6) we can solve for M_s and M_w. Note that in the computations water content is expressed as a decimal.

$$w = 0.10 = \frac{M_w}{M_s}$$

$$\rho = 1.76\ \text{Mg/m}^3 = \frac{M_t}{V_t} = \frac{M_w + M_s}{1.0\ \text{m}^3}$$

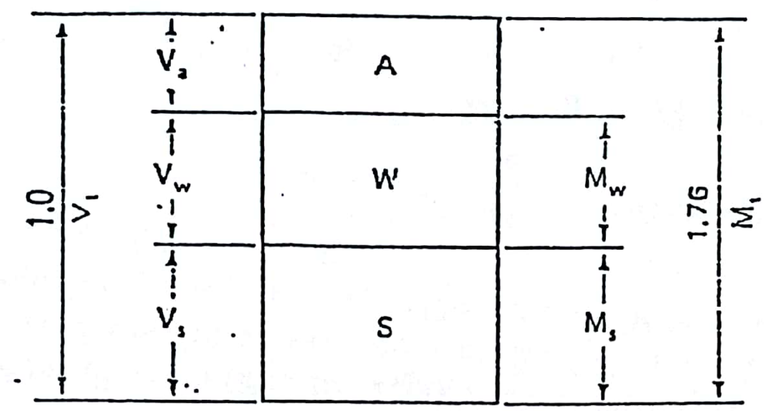


Fig. Ex. 2.2a

Substituting $M_w = 0.10M_s$ we get

$$1.76\ \text{Mg/m}^3 = \frac{0.10M_s + M_s}{1.0\ \text{m}^3}$$

$$M_s = 1.60\ \text{Mg} \quad \text{and} \quad M_w = 0.16\ \text{Mg}$$

These values are now placed on the mass side of the phase diagram (Fig. Ex. 2.2b), and the rest of the desired properties are calculated.

From the definition of ρ_w (Eq. 2-8) we can solve for V_w.

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

or

$$V_w = \frac{M_w}{\rho_w} = \frac{0.16\ \text{Mg}}{1\ \text{Mg/m}^3} = 0.160\ \text{m}^3$$

Place this numerical value on phase diagram, Fig. Ex. 2.2b.

To calculate V_s, we must assume a value of the density of the solids ρ_s. Here assume $\rho_s = 2.70\ \text{Mg/m}^3$. From the definition of ρ_s (Eq. 2-7) we

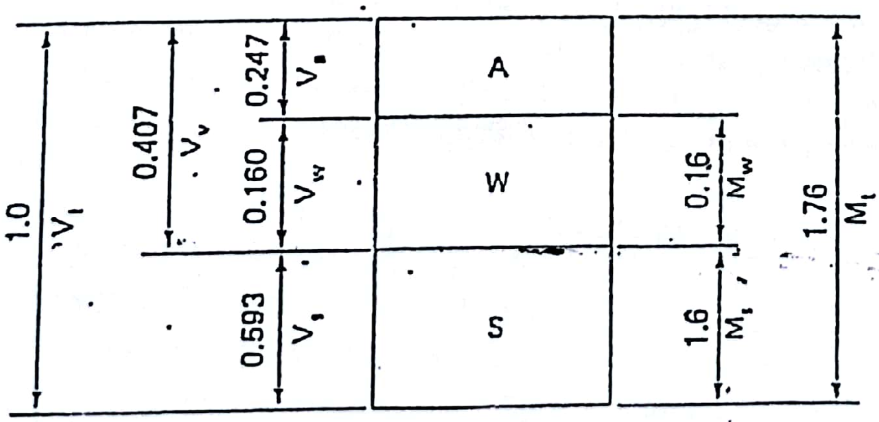


Fig. Ex. 2.2b

can solve for V_s directly, or

$$V_s = \frac{M_s}{\rho_s} = \frac{1.6 \text{ Mg}}{2.70 \text{ Mg/m}^3} = 0.593 \text{ m}^3$$

Since $V_t = V_a + V_w + V_s$, we can solve for V_a, since we know the other terms.

$$V_a = V_t - V_w - V_s = 1.0 - 0.593 - 0.160 = 0.247 \text{ m}^3$$

Once the phase diagram has been filled in, solution of the rest of the problem involves just plugging in the respective numbers into the appropriate definition equations. We recommend that when you make the computations, you write out the equations in symbol form and then insert the numbers in the same order as written in the equation. Also, it is a good idea to have the units accompany the calculations.

Solving for the remainder of the required items is easy.

From Eq. 2-9,

$$\rho_d = \frac{M_s}{V_t} = \frac{1.6 \text{ Mg}}{1 \text{ m}^3} = 1.6 \text{ Mg/m}^3$$

From Eq. 2-1,

$$e = \frac{V_v}{V_s} = \frac{V_a + V_w}{V_s} = \frac{0.247 + 0.160}{0.593} = 0.686$$

From Eq. 2-2,

$$n = \frac{V_v}{V_t} = \frac{V_a + V_w}{V_t} 100 = \frac{0.247 + 0.160}{1.0} 100 = 40.7\%$$

From Eq. 2-4,

$$S = \frac{V_w}{V_v} = \frac{V_w}{V_a + V_w} 100 = \frac{1.160}{0.247 + 0.160} 100 = 39.3\%$$

The saturated density ρ_{sat} is the density when all the voids are filled with water, that is, when $S = 100\%$ (Eq. 2-10). Therefore, if the volume of air V_a were filled with water, it would weigh $0.247 \text{ m}^3 \times 1 \text{ Mg/m}^3$ or 0.247 Mg. Then

$$\rho_{sat} = \frac{M_w + M_s}{V_t} = \frac{(0.247 \text{ Mg} + 0.16 \text{ Mg}) + 1.6 \text{ Mg}}{1 \text{ m}^3} = 2.01 \text{ Mg/m}^3$$

Another, and perhaps even easier way to solve this example problem, is to assume V_s is a unit volume, 1 m^3. Then, by definition, $M_s = \rho_s = 2.7$ (when ρ_s is assumed to be equal to 2.70 Mg/m^3). The completed phase diagram is shown in Fig. Ex. 2.2c.

Since $w = M_w/M_s = 0.10$, $M_w = 0.27$ Mg and $M_t = M_w + M_s = 2.97$ Mg. Also $V_w = M_w$ since $\rho_w = 1 \text{ Mg/m}^3$; that is, 0.27 Mg of water occupies

-ig/-
1000 kg/m^3

a volume of 0.27 m³. Two unknowns remain to be solved before we can proceed: they are V_a and V_t. To obtain these values, we must use the given information that $\rho = 1.76\ \mathrm{Mg/m^3}$. From the definition of total density (Eq. 2-6),

$$\rho = 1.76\ \mathrm{Mg/m^3} = \frac{M_t}{V_t} = \frac{2.97\ \mathrm{Mg}}{V_t}$$

Solving for V_t:

$$V_t = \frac{M_t}{\rho} = \frac{2.97\ \mathrm{Mg}}{1.76\ \mathrm{Mg/m^3}} = 1.688\ \mathrm{m^3}$$

Therefore

$$V_a = V_t - V_w - V_s = 1.688 - 0.27 - 1.0 = 0.418\ \mathrm{m^3}$$

You can use Fig. Ex. 2.2c to verify that the remainder of the solution is identical to the one using the data of Fig. Ex. 2.2b.

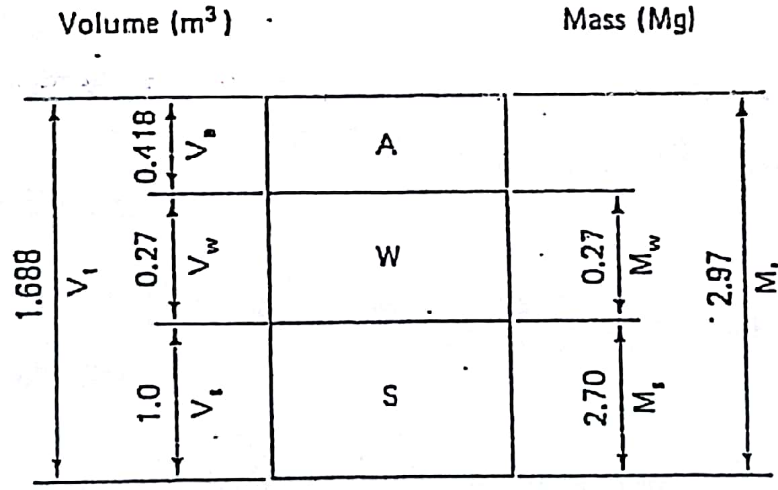


Fig. Ex. 2.2c

EXAMPLE 2.3

Required:

Express the porosity n in terms of the void ratio e (Eq. 2-3a) and the void ratio in terms of the porosity (Eq. 2-3b).

Solution:

Draw a phase diagram (Fig. Ex. 2.3a).

For this problem, assume $V_s = 1$ (units arbitrary). From Eq. 2-1, $V_v = e$ since $V_s = 1$. Therefore $V_t = 1 + e$. From Eq. 2-2, the definition of

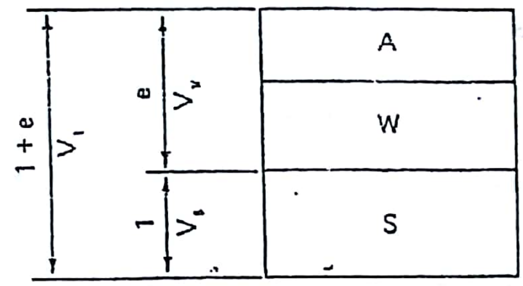


Fig. Ex. 2.3a

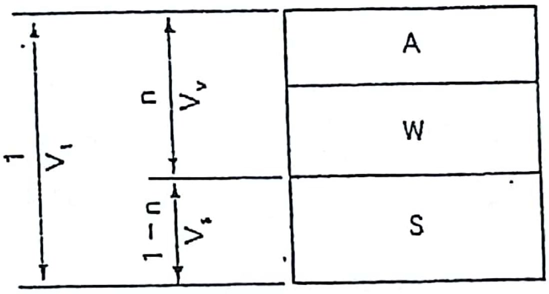


Fig. Ex. 2.3b

n is V_v/V_t, or

$$n = \frac{e}{1+e} \tag{2-3a}$$

Equation 2-3b can be derived algebraically or from the phase diagram (Fig. Ex. 2.3b). For this case, assume $V_t = 1$.

From Eq. 2-2, $V_v = n$ since $V_t = 1$. Therefore $V_s = 1 - n$. From Eq. 2-1, the definition of $e = V_v/V_s$. So

$$e = \frac{n}{1-n} \tag{2-3b}$$

EXAMPLE 2.4

Given:

$$e = 0.62, \quad w = 15\%, \quad \rho_s = 2.65\ Mg/m^3.$$

Required:

a. ρ_d
b. ρ
c. w for $S = 100\%$
d. ρ_{sat} for $S = 100\%$

Solution:

Draw phase diagram (Fig. Ex. 2.4).

a. Since no volumes are specified, assume $V_s = 1\ m^3$. Just as in

Example 2.3, this makes the $V_v = e = 0.62\ \mathrm{m}^3$ and $V_t = 1 + e = 1.62\ \mathrm{m}^3$. From Eq. 2-9,

$$\rho_d = \frac{M_s}{V_t}$$

and $M_s = \rho_s V_s$ (from Eq. 2-7). So

$$\rho_d = \frac{\rho_s V_s}{V_t} = \frac{\rho_s}{1+e} \quad \text{since } V_s = 1\ \mathrm{m}^3 \text{ in Fig. Ex.2-4}$$

$$= \frac{2.65}{1+0.62} = 1.636\ \mathrm{Mg/m^3}$$

Note: The relationship

$$\rho_d = \frac{\rho_s}{1+e} \tag{2-12}$$

is often very useful in phase problems.

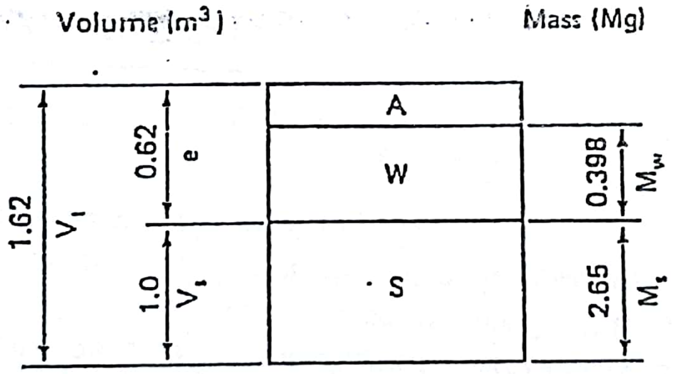


Fig. Ex. 2.4

b. Now for ρ:

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

We know that

$$M_w = wM_s \text{ (from Eq. 2-5)} \quad \text{and} \quad M_s = \rho_s V_s$$

$$\rho = \frac{\rho_s V_s + w\rho_s V_s}{V_t} = \frac{\rho_s(1+w)}{1+e} \quad \text{since } V_s = 1\ \mathrm{m}^3$$

Plug in the numbers.

$$\rho = \frac{2.65(1+0.15)}{1+0.62} = 1.88\ \mathrm{Mg/m^3}$$ 18.45

This relationship is often useful to know.

$$\rho = \frac{\rho_s(1+w)}{(1+e)} \tag{2-13}$$

Check:

$$\rho_d = \frac{\rho}{1 + w} \tag{2-14}$$

$$= \frac{1.88}{1.15} = 1.636 \text{ Mg/m}^3$$

You should verify that $\rho_d = \rho/(1 + w)$, which is another very useful relationship to remember.

c. Water content for $S = 100\%$. From Eq. 2-4, we know that $V_w = V_v = 0.62 \text{ m}^3$. From Eq. 2-8, $M_w = V_w\rho_w = 0.62 \text{ m}^3 (1 \text{ Mg/m}^3) = 0.62$ Mg. Therefore w for $S = 100\%$ must be

$$w_{(S=100\%)} = \frac{M_w}{M_s} = \frac{0.62}{2.65} = 0.234 \text{ or } 23.4\%$$

d. ρ_{sat}. From Eq. 2-10, we know $\rho_{sat} = (M_s + M_w)/V_t$, or

$$\rho_{sat} = \frac{2.65 + 0.62}{1.62} = 2.019 \text{ or } 2.02 \text{ Mg/m}^3$$

Check, by Eq. 2-13:

$$\rho_{sat} = \frac{\rho_s(1 + w)}{1 + e} = \frac{2.65(1 + 0.234)}{1.62} = 2.02 \text{ Mg/m}^3$$

EXAMPLE 2.5

Required:

Derive a relationship between S, e, w, and ρ_s.

Solution:

Look at the phase diagram with $V_s = 1$ (Fig. Ex. 2.5).

From Eq. 2-4 and Fig. 2.5, we know that $V_w = SV_v = Se$. From the definitions of water content (Eq. 2-5) and ρ_s (Eq. 2-7), we can place the

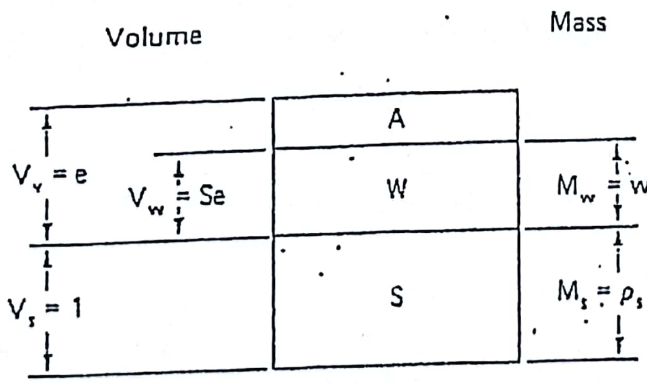


Fig. Ex. 2.5

equivalents for M_s and M_w on the phase diagram. Since from Eq. 2-8, $M_w = \rho_w V_w$, we now can write the following equation:

$$M_w = \rho_w V_w = w M_s = w \rho_s V_s$$

or

$$\rho_w S e = w \rho_s V_s$$

Since $V_s = 1\ \mathrm{m}^3$,

$$\rho_w S e = w \rho_s \tag{2-15}$$

Equation 2-15 is one of the most useful of all equations for phase problems. You can also verify its validity from the fundamental definitions of ρ_w, S, e, w, and ρ_s.

Note that using Eq. 2-15 we can write Eq. 2-13 another way:

$$\rho = \frac{\rho_s\left(1 + \dfrac{\rho_w S e}{\rho_s}\right)}{1 + e} = \frac{\rho_s + \rho_w S e}{1 + e}. \tag{2-16}$$

When $S = 100\%$, Eq. 2-16 becomes

$$\rho_{sat} = \frac{\rho_s + \rho_w e}{1 + e} \tag{2-17}$$

EXAMPLE 2.6

Given:

A silty clay soil with $\rho_s = 2700\ \mathrm{kg/m^3}$, $S = 100\%$, and the water content = 46%.

Required:

Compute the void ratio e, the saturated density, and the buoyant or submerged density in $\mathrm{kg/m^3}$.

Solution:

Place given information on a phase diagram (Fig. Ex. 2.6).

Assume $V_s = 1\ \mathrm{m}^3$; therefore $M_s = V_s \rho_s = 2700$ kg. From Eq. 2-15, we can solve for e directly:

$$e = \frac{w\rho_s}{\rho_w S} = \frac{0.46 \times 2700}{1000 \times 1.0} = 1.242$$

But e also equals V_v since $V_s = 1.0$; likewise $M_w = 1242$ kg since M_w is

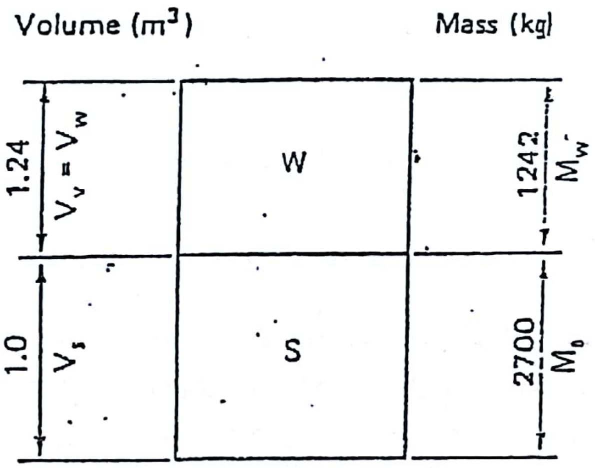


Fig. Ex. 2.6

numerically equal to V_w because $\rho_w = 1000$ kg/m³. Now that all the unknowns have been found, we may readily calculate the saturated density (Eq. 2-10).

$$\rho_{sat} = \frac{M_t}{V_t} = \frac{M_w + M_s}{1 + e} = \frac{(1242 + 2700)\text{ kg}}{(1 + 1.24)\text{ m}^3} = 1758\text{ kg/m}^3$$

We could also use Eq. 2-17 directly.

$$\rho_{sat} = \frac{\rho_s + \rho_w e}{1 + e} = \frac{2700 + 1000\,(1.242)}{1 + 1.242} = 1758\text{ kg/m}^3$$

When a soil is submerged, the actual unit weight is reduced by the buoyant effect of the water. The buoyancy effect is equal to the weight of the water displaced. Thus, in terms of densities, (Eqs. 2-11 and 2-17):

$$\rho' = \rho_{sat} - \rho_w = 1758\text{ kg/m}^3 - 1000\text{ kg/m}^3 = 758\text{ kg/m}^3$$

or

$$\rho' = \frac{\rho_s + \rho_w e}{1 + e} - \rho_w$$

$$= \frac{\rho_s - \rho_w}{1 + e} \tag{2-18}$$

$$\rho' = 758\text{ kg/m}^3$$

In this example, ρ' is less than the density of water. Go back and look at Table 2-1 for typical values of ρ'. The submerged or buoyant density of soil will be found to be very important later on in our discussion of consolidation, settlement, and strength properties of soils.

In summary, for the easy solution of phase problems, you don't have to memorize lots of complicated formulas. Most of them can easily be derived from the phase diagram as was illustrated in the preceding examples. Just remember the following simple rules:

1. Remember the basic definitions of w, e, ρ_s, S, etc.
2. Draw a phase diagram.
3. Assume either $V_s = 1$ or $V_t = 1$, if not given.
4. Often use $\rho_w Se = w\rho_s$. , $\rho_d = \frac{\rho_s}{1+e}$

2.4 SOIL TEXTURE

$\rho_d = \frac{\rho_t}{1+w\%}$

So far we haven't said very much about what makes up the "solids" part of the soil mass. In Chapter 1 we gave the usual definition of soil from an engineering point of view: the relatively loose agglomeration of mineral and organic materials found above the bedrock. We briefly described how weathering and other geologic processes act on the rocks at or near the earth's surface to form soil. Thus the solid part of the soil mass consists primarily of particles of mineral and organic matter in various sizes and amounts.

The *texture* of a soil is its appearance or "feel," and it depends on the relative sizes and shapes of the particles as well as the range or distribution of those sizes. Thus coarse-grained soils such as *sands* or *gravels* obviously appear coarse textured, while a fine-textured soil might be composed of predominantly very tiny mineral grains which are invisible to the naked eye. *Silts* and *clay* soils are good examples of fine-textured soils.

The soil texture, especially of coarse-grained soils, has some relation to their engineering ~~behavior. In fact, soil texture has been the basis~~ for certain soil classification schemes which are, however, more common in agronomy than in soils engineering. Still, textural classification terms (gravels, sands, silts, and clays) are useful in a general sense in geotechnical engineering practice. For fine-grained soils, the presence of water greatly affects their engineering response—much more so than grain size or texture ~~alone. Water affects the~~ interaction between the mineral grains, and this may affect their *plasticity* and their *cohesiveness*.

Texturally, soils may be divided into coarse-grained versus fine-grained soils. A convenient dividing line is the smallest grain that is visible to the naked eye. Soils with particles larger than this size (about 0.05 mm) are called coarse-grained, while soils finer than the size are (obviously) called fine-grained. Sands and gravels are coarse grained while silts and clays are fine grained. Another convenient way to separate or classify soils is according to their plasticity and cohesion (physics: cohesion—sticking

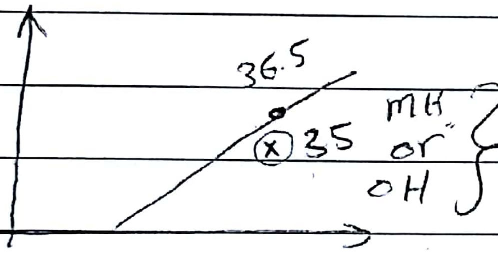
Sol: pass sieve # 200 = 70% > 50%

→ fine grained soil.

$LL_{AD} = 70\%$, $PL = 35\%$

$PI = LL - PL = 70 - 35 = \boxed{35\%}$

$\rightarrow PI = 0.73(LL - 20) = 0.73(70 - 20) = \boxed{36.5\%}$



$\rightarrow LL_{OD} \leq 0.75\, LL_{AD}$

$\leq 0.75 * (70)$

$= 52.5$

$\therefore 50 < 52.5 \rightarrow (OH)$

→ Soil classify according to USCS as OH.

Chapter 4 compaction

EX:- [illegible] بالسلايدات 3.3 انظر الرجعة *

Soil Compation curves shown in fig in my Right hand → 3.3 عج , In the field the same soil used for fill If soil total density in the field is $1800 kg/m^3$ and water content is 13%

1) Determine Relative Compaction with respect to modified proctor test ?

2) If project Required R.C = 90% and water content at the field ±2 of optimums Does the compaction satisfy project Requirement.

$LL_{AD} = 22\%$

$PL = 18\%$

GP - GC

GP - GM

Subject Date No.

$$\Rightarrow \frac{\rho_w (G_s + S.e)}{1+e} = \frac{\rho_w (G_s + e)}{1+e}$$

$$\Rightarrow \frac{1000(2.7+0.68)}{1+0.68} = 2011.9 \ kg/m^3$$

$LL_{OD} < \left(\frac{75}{100}\right) LL_{AD} \rightarrow$ organic

$OD \rightarrow$ oven dried

$AD \rightarrow$ Air dried

Ex:- passing No. 200 sieve 30% (coarse) & passing No. 4 sieve 70% (sand) & LL = 33 PI = 12

SC

Ex:- classify the following soil according to Unified soil classification system (USCS)?

Given:- $D_{10} = 0.01$ mm, $D_{30} = 0.1$ mm, $D_{60} = 1.1$ mm

% of soil pass Sieve # 200 = 8%

% of soil pass Sieve # 4 = 30% كلها الباقي

$LL_{AD} = 40$, $PL = 15\%$, $LL_{OD} = 28\%$

Ex:- % of pass sieve # 200 = 70%

% of pass sieve # 4 = 100%

$LL_{AD} = 70$ $PL = 35\%$

$LL_{OD} = 50\%$

classifiy this soil according to USCS? ~~باقي الأسئلة~~

Subject Date No.

3) Determine Dr & Gs ?

① من الرجعة (~~[illegible]~~)

Sol:- RC = ? ✓

$\rho_{d\,max\,Lab} = 1.878\ Mg/m^3 = 1878\ \frac{kg}{m^3}$

$W_{opt} = 12\ \%$ → Lab

$\rho_{d\,field} = \frac{\rho_T}{1+w} = \frac{1800}{1+0.13} = 1592.9\ kg/m^3$

$R.C = \frac{\rho_{d\,field}}{\rho_{max\,Lab}} * 100\ \% = \frac{1592.9}{1878} * 100\% = 84.8\ \%$

② $R.C = 84.4\ \% < 90\%$ ∴ Not ok.

$W_{opt} - 2 < W_{field} < W_{opt} + 2$

$10 < W_{field} < 14$

→ ok, compaction not satisfy project Requirement.

③ Dr = ?

$R.C = 80\% + 0.2\,Dr$ ∴ $Dr = \frac{84.8 - 80}{0.2} = 24\%$

Gs? $\rho_{Z.A.V} = \frac{\rho_w\, G_s}{1 + \frac{w\, G_s}{S}}$ $\rho_d = 1960\ kg/m^3$

$w = 15\%$

$\Rightarrow 1960 = \frac{1000 * G_s}{1 + \frac{0.15 * G_s}{1}}$ ∴ $\boxed{G_s = 2.77}$

Subject Date No.

Effective stress principle :-

Soil profile

G.S = Ground surface ~~Bulk~~ partialy saturated or dry

H_1 SP "sand" GWT — GWT: Ground water Table

H_2 Cl "clay" — fully saturated

H_3 SC : clayey sand

Bed Rock

Gs ▽ GWT ①

$S = 100\%$

γ_{SAT}

② A Z

σ_T = weight of soil column above the point

$= (\gamma_{SAT} * Z)$ KN/m² → KPa

$= \int_0^Z \gamma \, dZ$

Remember :-

$$h = \frac{P_w}{\gamma_w} + \frac{V^2}{2g} + Z$$

$\frac{P_w}{\gamma_w} \rightarrow \frac{u}{\gamma_w}$

Apply Bernoulis eq Between ① & ②

$$Z_1 + \frac{u_1}{\gamma_w} = Z_2 + \frac{u_2}{\gamma_w}$$

$$0 + 0 = -Z + \frac{u_2}{\gamma_w} \quad \therefore \boxed{u_2 = \gamma_w * Z}$$

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

$$\gamma_{SAT} * Z = \sigma' + \gamma_w * Z$$

$$\sigma' = (\gamma_{SAT} * Z - \gamma_w Z)$$

$$= (\gamma_{SAT} - \gamma_w) * Z$$

$$= \gamma' Z$$

Effective stress ①

* Total stress carraid by soil = (stress carraid by solid particles) + (stress carried by pore water)

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

* overburden pressure = Effective stress due to self-weight of soil

---

* total stress = $\sigma_v = \gamma_{sat} * z$
* pore water pressure = $u = \gamma_w z$

$\therefore$ Effective stress = $\sigma_v' = \sigma_v - u$

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

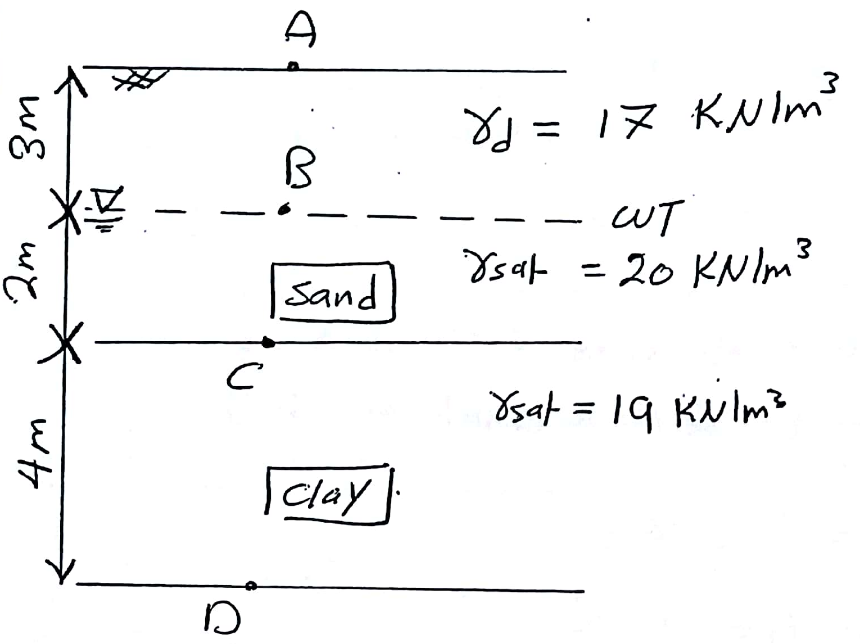
$$= (\gamma_{sat} - \gamma_w) z = \gamma' z$$

* u = pore water pressure = static pressure + exces pore pressure

$$= u_s + u_e$$

* after dissipation = drained condition $\Rightarrow u_e = 0.0$
* befor dissipation = Un-drained cond. $\Rightarrow u_e \neq 0.0$
* dissipation in sand is very Fast to occur
* dissipation in Fine soil (clay) is very slow to occur.

Example 1



plot the values of total vertical stress and effective vertical stress against depth.

Solution: AT "A" ⇒ total stress and effective = 0.0

AT "B" ⇒ σ_v = total = $17 \times 3 = 51$

pore pressure = $u = \gamma_w z = 9.81 \times 0 = 0.0$

∴ effective stress = $\sigma'_v = 51 - 0.0 = 0.0$

$z = 0.0$ (No water)

AT "C" ⇒ $\sigma_v = (17 \times 3) + (20 \times 2) = 91$ Kpa

$u = 9.81 \times 2 = 19.62$

$\sigma'_v = 91 - 19.6 = 71.4$ Kpa

AT "D" ⇒ $\sigma_v = (17 \times 3) + (20 \times 2) + (19 \times 4)$

$= 167$ Kpa

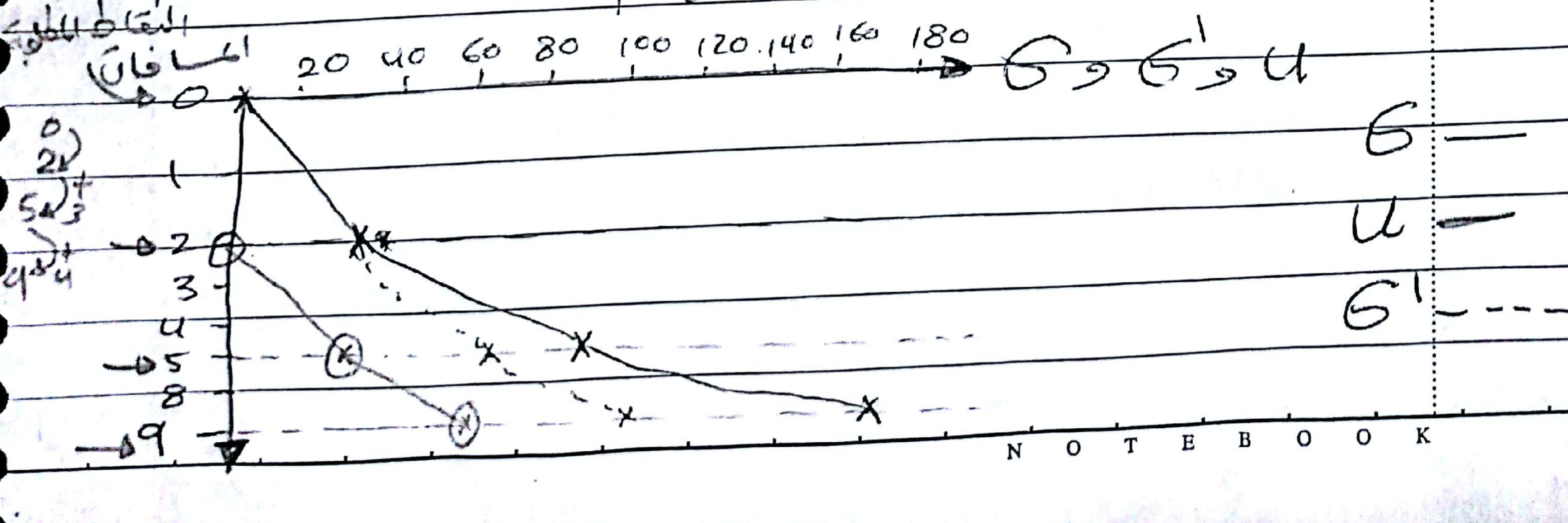
$u = 9.81(2+4) = 58.86$

$\sigma'_v = 108$ Kpa

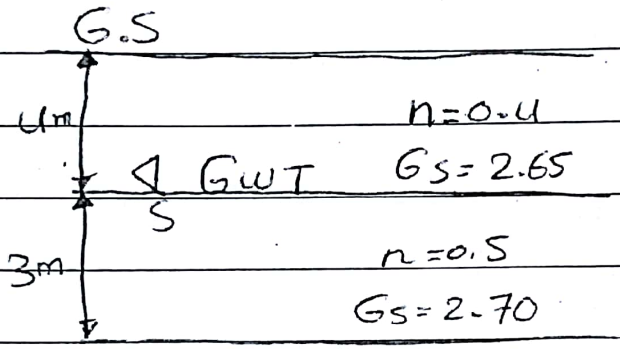
EX 8: Draw with depth the total Effective and pore water pressur for the given soil profile ?

(A) ——— $\gamma = 17$
2 — GWT ——
(B) sand $\gamma_{SAT} = 19.81$
3
(C) ——— اجباري ↳
لان تحت GWT
4m clay $\gamma_{SAT} = 20.81$
(D) ———

	(kPa)	(kPa)	(kPa)
(A)	$\sigma_T = \sum \gamma_i Z_i = 0$	$u = \gamma_w Z_w = 0$	$\sigma' = \sigma_T - u = 0$
(B)	$\sigma_T = 17 * 2 = 34$	$u = 0$ لانو هاي فاي فوق الماء تحت GWT	$\sigma' = 34 - 0 = 34$
(C)	$\sigma_T = 34 + \gamma_{SAT} Z = 34 + 19.81 * 3 = 93.43$	$u = \gamma_w Z_w = 9.81 * 3 = 29.43$	$\sigma' = 64$
(D)	$\sigma_T = 93.43 + 20.81 * 4 = 176.67$	$u = \gamma_w Z_w = 29.43 + 9.81 * 4 = 68.67$	$\sigma' = 108$



Ex: Determine the total over burden pressure, pore water pressure and effective stress at point X only!



Sol:-

الطبقة الاولى

$$\gamma_{dry} = \frac{G_s \gamma_w}{1+e} \qquad e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$$

$$\Rightarrow = \frac{2.65 * 9.81}{1+0.667} = 15.6 \ kN/m^3$$

الطبقة الثانية

$$\gamma_{SAT} = \frac{\gamma_w (G_s + e)}{1+e} \qquad e = \frac{0.5}{1-0.5} = 1$$

$$\Rightarrow = \frac{9.81(2.7+1)}{1+1} = 18.14 \ kN/m^3$$

$$\sigma_{TX} = \Sigma \sigma Z$$
$$= 15.6 * 4 + 18.14 * 3$$
$$= 116.82 \ kPa$$

$$U_X = \Sigma \gamma_w Z$$
$$= 9.81 * 3 = 29.43 \ kPa$$

$$\sigma'_X = \sigma_{TX} - U_X$$
$$= 116.82 - 29.43$$
$$= 87.39 \ kPa$$

Soil Classification

* الهدف : معرفة خصائص التربة

* we use simple indices :-

- GSD ⇒ Grain Size Distribution
- LL ⇒ Liquid Limit
- PI ⇒ plastic Index

→ سيتم شرحهم "لاحقاً"

---

* Class. systems :
  - USCS ⇒ مطلوب
  - AASHTO ⇒ مش مطلوبات
  - BS ⇒

* "USCS" includes Four major divisions :
  1- Coarse - grained
  2- Fine - grained
  3- organic soil
  4- peat (تربة زراعية)

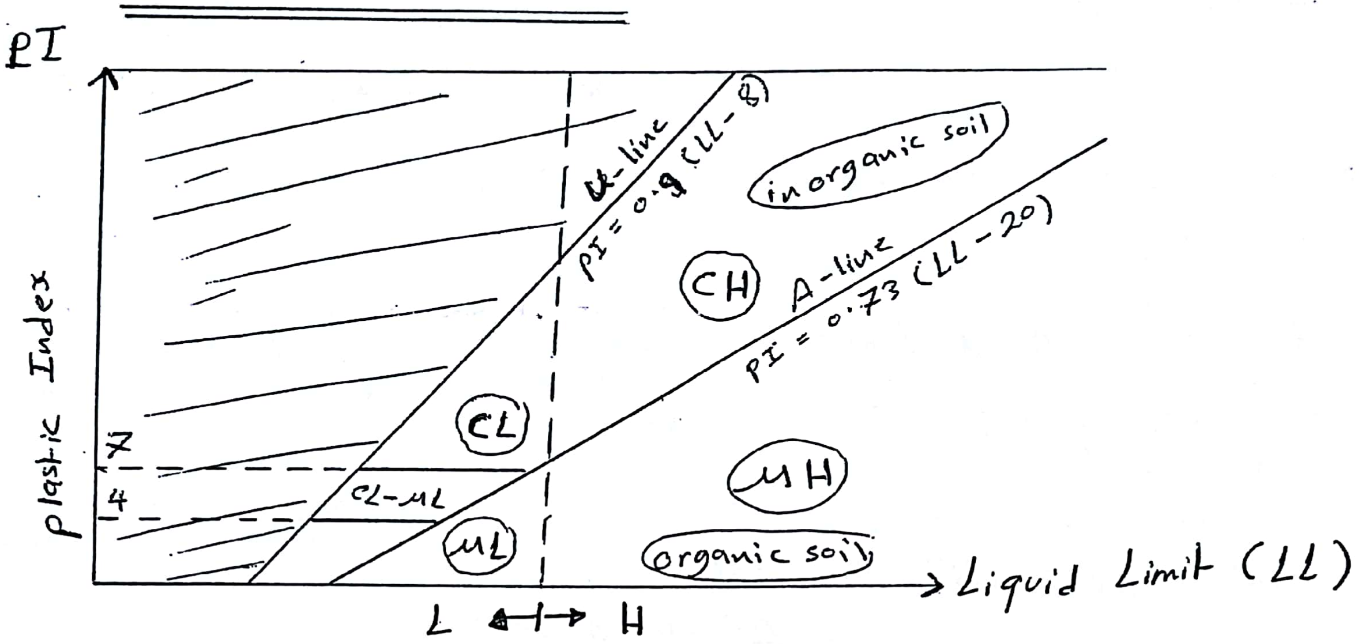
* General Guidance ⇒ Later on will be discussed.

* (Soil Symbols) * ⇒ From slides ("مهم جداً")

* يجب حفظ الرموز "جيداً" وفهم المثال المذكور في أسفل الصفحة.

* بعض الرموز قد يستحيل إجتماعها ⇐ ~~GG~~

"plasticity Chart"



⇐ * المنطقة المظللة لا يوجد فيها أي نوع من انواع التربة
لذلك يمكن تعريف :

U-line: upper bound For soil

⇒ * A-line: separates clay From silt, and organics From In organics.

$$\left.\begin{aligned} \text{Aline}: PI &= 0.73\,(LL-20) \\ \text{U-line}: PI &= 0.9\,(LL-8) \end{aligned}\right]$$ * حفظ

* $7 > PI > 4 \Rightarrow$ CL-ML :

*** لاحظ انه ال silt يوجد تحت A-line وال clay فوقه
*** لاحظ انه ال organic يوجد تحت A-line وال inorganic فوقه

Producere ⟶
طريقة التصنيف

*** ننظر الى sieve #200 ⇐ اذا كان المرتد على sieve #200 أكبر من 50%

المرتد أكبر من 50%

فيجب انه نعلم انه التربة هي Coarse واذا كان المرتد أصغر منه 50%

فالتربة هي Fine .

*** اذا كان المرتد على #200 أكبر منه 50% (Coarse) ننظر الى

sieve #4 ⇐ اذا كان المرتد على #4 أكبر منه 50% تكونه

التربة "Gravel" واذا كان المرتد اقل منه 50% تكون التربة

"Sand" .

** اذا كان المرتد على #4 أكبر منه 50% سيكون تصنيف التربة

تابع لإحدى المجموعتين :

$\boxed{1}$ GW or GP $\boxed{2}$ GM or GC

وحتى نختار إحدى المجموعتين نعود الى #200

** اذا كان نسبة المار (pass) على #200 اقل منه 5%

نختار المجموعة الأولى واذا كان نسبة المار

اكثر منه 12% نختار المجموعة الثانية

* اذا كان المار على #200 اقل منه 5% ⇐ المجموعة الاولى ،

من المجموعة الاولى يوجد خيارين :

A) GW B) GP

وحتى نختار احدهما :

~~IF Cu~~

IF $C_u > 4$ and $1 \leq C_c \leq 3 \Rightarrow$ GW

otherwise $\Rightarrow$ GP

Example

* passing No. 200 = 30%
* passing No. 4 = 70%
* LL = 33 * PI = 12
* PI = 12

Solution

passing # 200 = 30% ⇒ less than 50% ⇒ "~~G~~"

∴ Coarse soil

passing No. 4 = 70% ⇒ more than 50% ⇒ (Sand)

passing # = 200 = 30% ⇒ more than 12%.

∴ → SC or SM

But $PI_{A\text{-line}} = 0.73(33-20) = 9.5$

$(PI = 12) > (PI_{A\text{-line}} = 9.5)$

→ sample is above A-line

∴ (SC) = clayey sand

* You should understand Example 3.1 well

3

III.
Soil Classification

$[(20 \times 4) + (19 \times 5) + (20 \times 3)] - 9.81(5+3) = 156.52$

$R_1 = 1.005$

$R_2 = 0.199$

7 ≡ 21

Outline

1. Purpose
2. Classification Systems
3. The Unified Soil Classification System (USCS)
4. American Association of State Highway and Transportation Officials System (AASHTO)
5. Suggested Homework

1. Purpose

Classifying soils into groups with similar behavior, in terms of *simple* indices, can provide geotechnical engineers a general guidance about engineering properties of the soils through the *accumulated experience*.

Communicate between engineers

Simple indices GSD, LL, PI → $\boxed{\text{Classification system (Language)}}$ → Estimate engineering properties → Achieve engineering purposes

Use the accumulated experience

2. Classification Systems

Two commonly used systems:

- Unified Soil Classification System (USCS).

AASHTO

- American Association of State Highway and Transportation Officials (AASHTO) System
- Craig's Soil Mechanics use **BS**

3. Unified Soil Classification System (USCS)

Origin of USCS:

This system was first developed by Professor A. Casagrande (1948) for the purpose of airfield construction during World War II. Afterwards, it was modified by Professor Casagrande, the U.S. Bureau of Reclamation, and the U.S. Army Corps of Engineers to enable the system to be applicable to dams, foundations, and other construction (Holtz and Kovacs, 1981).

Four major divisions:

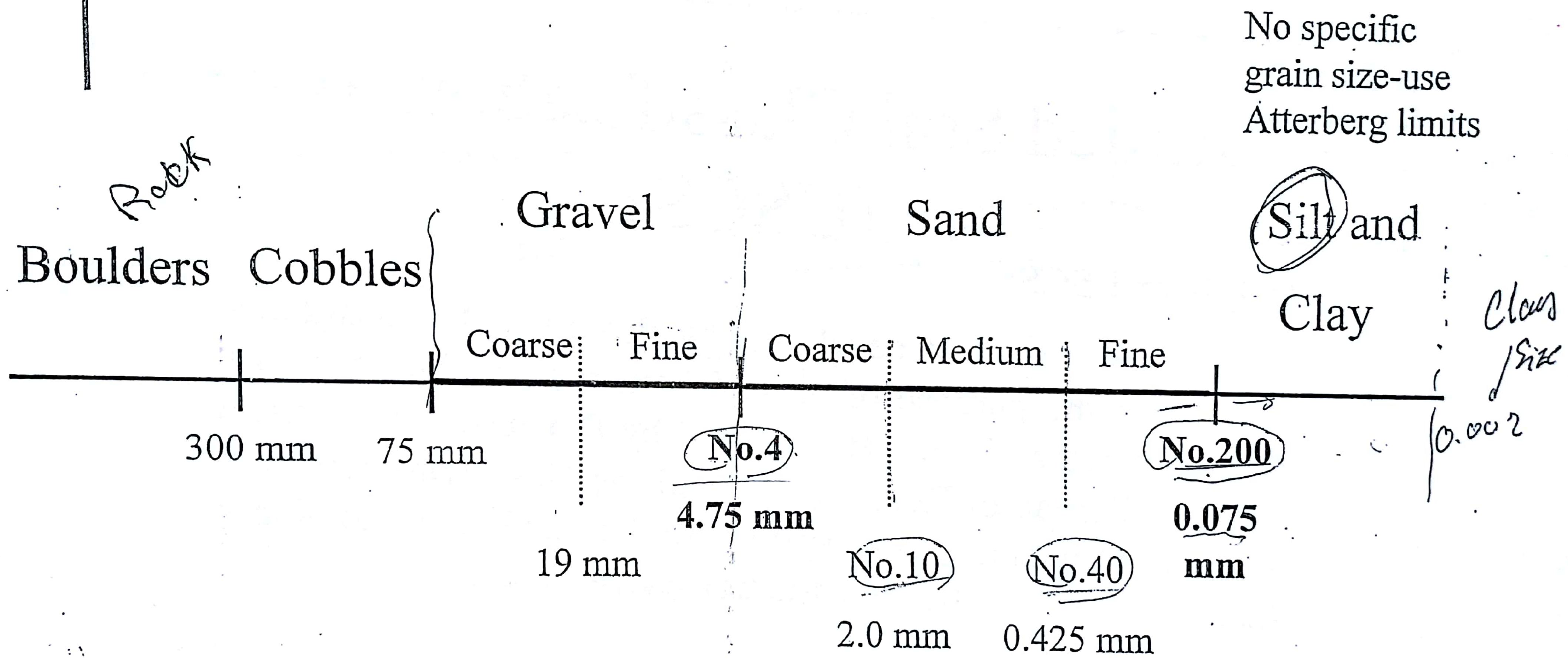
civil (1) Coarse-grained — Retarned on seive #200 → Gravel, Sand — (<%50)

(2) Fine-grained — Passed seive #200 → Silt, Clay — (>%50)

(3) Organic soils

(4) Peat تربة زراعية

3.1 Definition of Grain Size



3.2 General Guidance

50 %

Coarse-grained soils:

Gravel
retained

50% finer → sand

Sand
Passed

50%

NO. 4

4.75 mm

Fine-grained soils:

organic or
Inorganic

Silt

Clay

NO.200

0.075 mm

اللي نسبة اكثر من 50%
يكون هو "Fine or Coars"

•Grain size distribution

•PL, LL

$LL \geq 50$

$LL < 50$

•Plasticity chart

	% finer
#3"	100%
	100 %
#4	70%
#10	65 %
#20	53 %
#40	
#200	

(1) → Gravel, Sand

•C_u

•C_c

#200 → (1) Courses (2) fine

Required tests: Sieve analysis

Atterberg limit

3.3 Symbols

Soil symbols:

G: Gravel

S: Sand

M: Silt

C: Clay

O: Organic

Pt: Peat

Example: SW, Well-graded sand

SC, Clayey sand

SM, Silty sand,

MH, Elastic silt

CL - GC : mixture of clayed Gravel and low plastic clay.

Liquid limit symbols:

H: High LL (LL>50)

L: Low LL (LL<50)

Gradation symbols:

W: Well-graded

P: Poorly-graded

→ course

Well – graded soil

$1 < C_c < 3$ and $C_u \geq 4$ (for gravels)

$1 < C_c < 3$ and $C_u \geq 6$ (for sands)

3.4 Plasticity Chart

PI = 0.73(LL − 20)

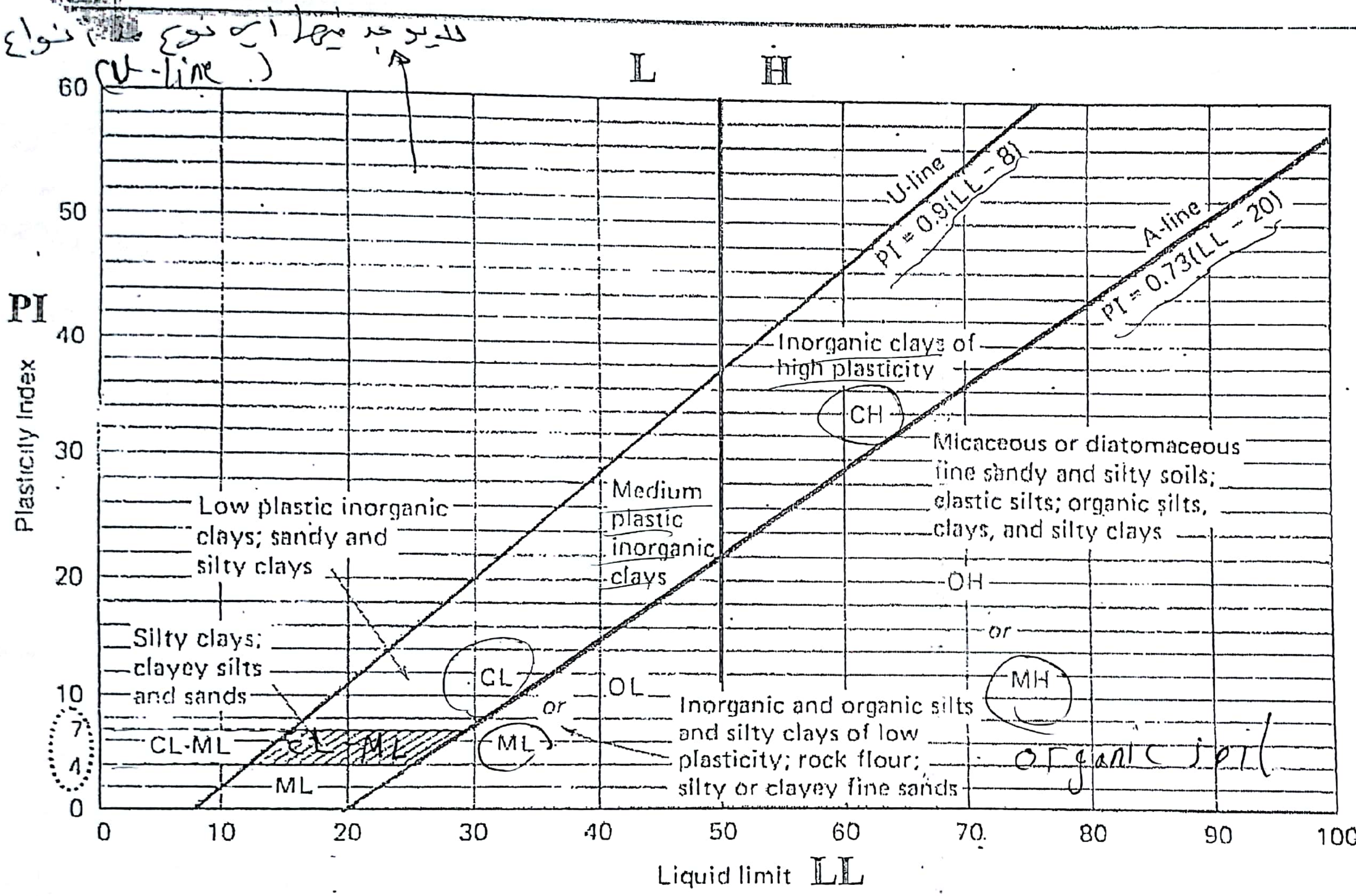


Fig. 3.2 Casagrande's plasticity chart, showing several representative soil types (developed from Casagrande, 1948, and Howard, 1977).

(Holtz and Kovacs, 1981)

- The A-line generally separates the more claylike materials from silty materials, and the organics from the inorganics.
- The U-line indicates the upper bound for general soils.

Note: If the measured limits of soils are on the left of U-line, they should be rechecked.

M: - تحت ال A-line
C: فوق ال A-line

O: - تحت ال A-line
ino فوق ال A-line

3.5 Procedures for Classification

GW - GM

Coarse-grained material	COARSE More than 50% retained sieve #200	Gravel: more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4,\ 1 \le C_c \le 3$	— GW
			5–12 %	Not satisfying GW	— GP
			More than 12% fines	Below 'A' line	— GM
				Above 'A' line	— GC
Grain size distribution		Sand: less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6,\ 1 \le C_c \le 3$	— SW (well)
			5–12 %	Not satisfying SW	— SP (poor)
			More than 12% fines	Below 'A' line	— SM
				Above 'A' line	— SC

LL & PI
PI = 0.73(LL − 20)

−(SC − SM) 5% − 12%

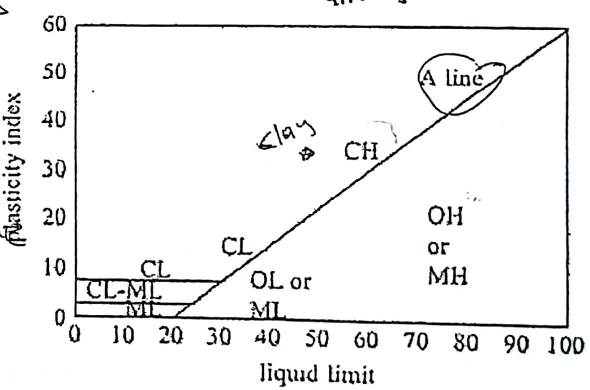
#200

(CH, MH
(CL, ML, CL-MH

Fine-grained material — FINE: Less than 50% retained sieve #200

LL < 50 — organic soil if (LL oven dry / LL air dry < 0.75) → OH

LL > 50



ML
CL
OL
MH
CH
OH

Highly ORGANIC SOILS — PI

LL, PI

L ≥ 50 → organic; inorganic → silt (M) → if PI soil below A line → MH; clay (C) → if on or above → CH

L < 50 → inorganic → ML; (CL − ML) → (if $4 \le PI_{soil} \le 7.0$); CL

Fine-grained soil
$LL \geq 50$
$LL_{soil} < 50$ → Inorganic soil → ML → If $PI_{soil} < PI$-Aline
CL-ML → If $4 < PI_{soil} < 7$
CL → If $PI_{soil} \geq PI$-Aline $= 0.73(LL-20)$
→ organic soil → $\frac{LL\ ovendry}{LL\ airdry} < 0.75$ → OL

Table 1.6 Unified Soil Classification Chart (after ASTM, 2005)

Criteria for assigning group symbols and group names using laboratory tests[a]				Group symbol	Group name[b]
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[c]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[e]	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[e]	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly graded sand[i]
		Sand with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[k,l,m,n] / Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[k,l,m,p] / Organic silt[k,l,m,q]
Highly organic soils		Primarily organic matter, dark in color, and organic odor		PT	Peat

[a] Based on the material passing the 75-mm (3-in.) sieve.
[b] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[c] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.
[d] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.
[e] $C_u = D_{60}/D_{10}$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[f] If soil contains ≥15% sand, add "with sand" to group name.
[g] If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[h] If fines are organic, add "with organic fines" to group name.
[i] If soil contains ≥15% gravel, add "with gravel" to group name.
[j] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[k] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[l] If soil contains ≥30% plus No. 200, predominantly sand, add "sandy" to group name.
[m] If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[n] PI ≥ 4 and plots on or above "A" line.
[o] PI < 4 or plots below "A" line.
[p] PI plots on or above "A" line.
[q] PI plots below "A" line.

If Soil
$LL = 60$ (H)
$PL = 30$
$PI_{soil} = 60 - 30 = 30$
$PI\ Aline = 0.73(60-20)$
$= 29.3$
$PI_{soil} \geq PI\ Aline$
clay
CH

S → 5-12% →
SW-SC
SW-SM
SP-SC
SP-SM

Soil % finer
#4 90%
#10 80%
#200 50%
LL = 60%
PL = 30%
SC − CH → fine

Table 1.6 Unified Soil Classification Chart (after ASTM, 2005)

Criteria for assigning group symbols and group names using laboratory tests[a]				Soil classification: Group symbol	Soil classification: Group name[b]
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[c]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[e]	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[e]	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly graded sand[i]
		Sand with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[k,l,m,n] / Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[k,l,m,p] / Organic silt[k,l,m,q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[a] Based on the material passing the 75-mm. (3-in) sieve.
[b] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[c] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.
[d] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.
[e] $C_u = D_{60}/D_{10}$ $\quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[f] If soil contains ≥15% sand, add "with sand" to group name.
[g] If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[h] If fines are organic, add "with organic fines" to group name.
[i] If soil contains ≥15% gravel, add "with gravel" to group name.
[j] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[k] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[l] If soil contains ≥30% plus No. 200, predominantly sand, add "sandy" to group name.
[m] If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[n] PI ≥ 4 and plots on or above "A" line.
[o] PI < 4 or plots below "A" line.
[p] PI plots on or above "A" line.
[q] PI plots below "A" line.

> 50% passed Sieve #200

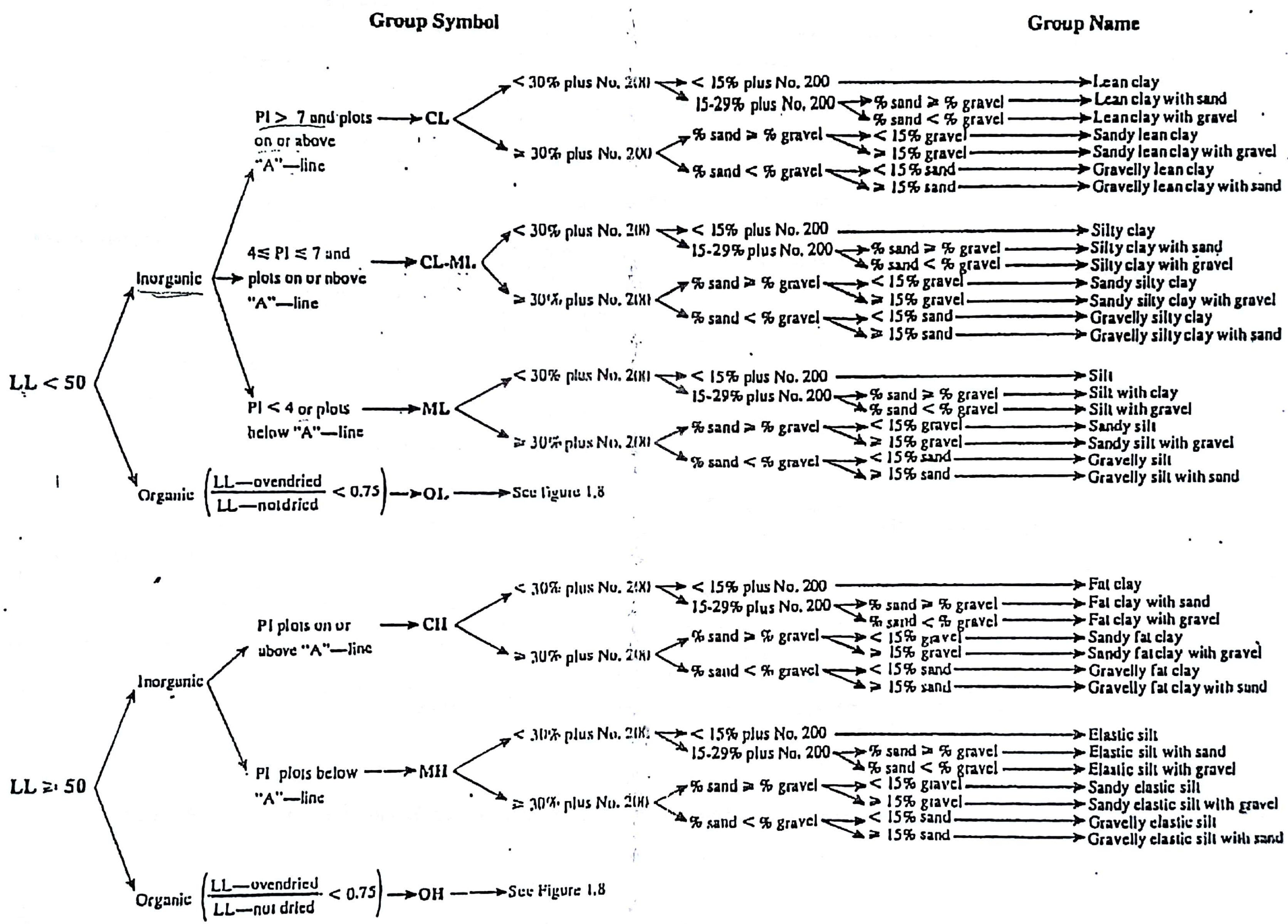


Figure 1.7 Flowchart for classifying fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2005)

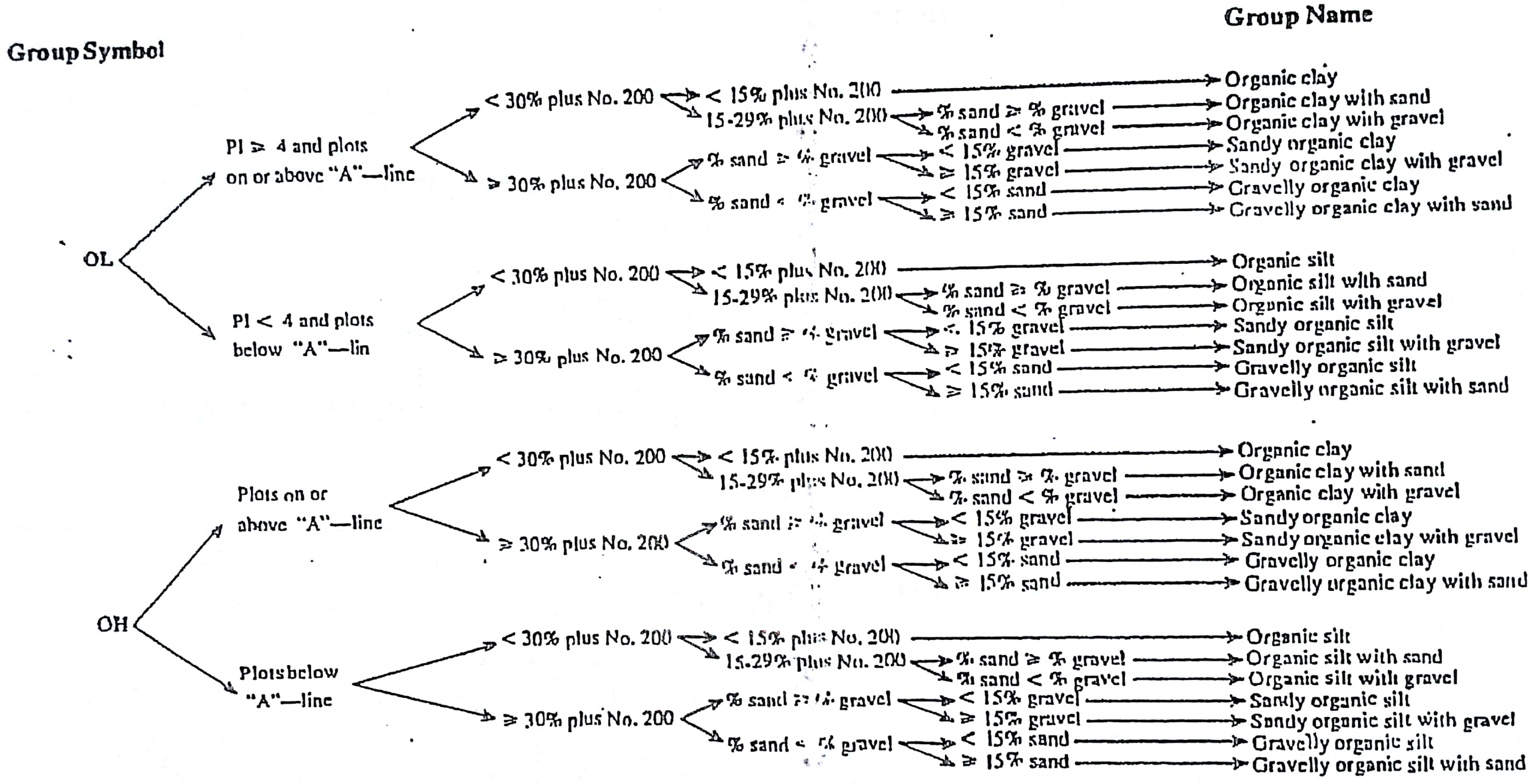


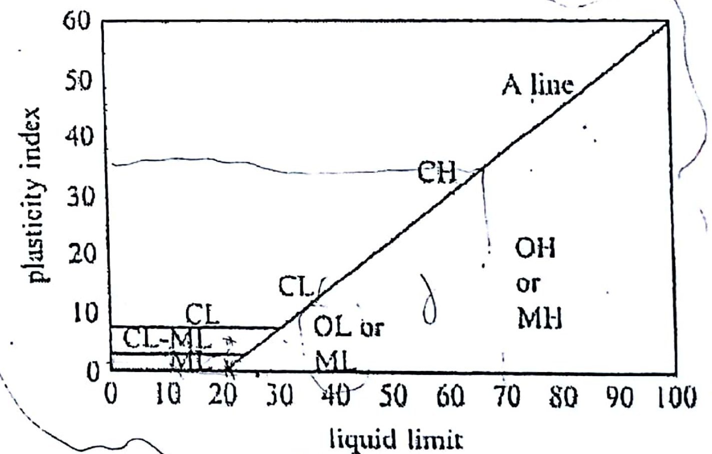
Figure 1.8 Flowchart for classifying organic fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2005)

3.6 Example

Passing No.200 sieve 30 % LL= 33

Passing No.4 sieve 70 % PI= 12

COARSE More than 50% retained sieve #200	Gravel: more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4$, $1 \le C_c \le 3$	→ GW
			Not satisfying GW	→ GP
		More than 12% fines	Below 'A' line	→ GM
			Above 'A' line	→ GC
	Sand: less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6$, $1 \le C_c \le 3$	→ SW
			Not satisfying SW	→ SP
		More than 12% fines	Below 'A' line	→ SM
			Above 'A' line	→ SC
FINE Less than 50% retained sieve #200	$LL < 50$			ML
				CL
				OL
	$LL > 50$			MH
				CH
				OH
Highly ORGANIC SOILS				— Pt



Gp - Gc

Passing No.200 sieve 30 %

Passing No.4 sieve 70 %

LL= 33

PI= 12

PI= 0.73(LL-20), A-line

PI=0.73(33-20)=9.49

SC

(≥15% gravel)

Clayey sand with gravel

*LL

> 50% retained on sieve #200

Sieve	% finer
#4	70%
#200	30%

LL = 35%

LI = 12%

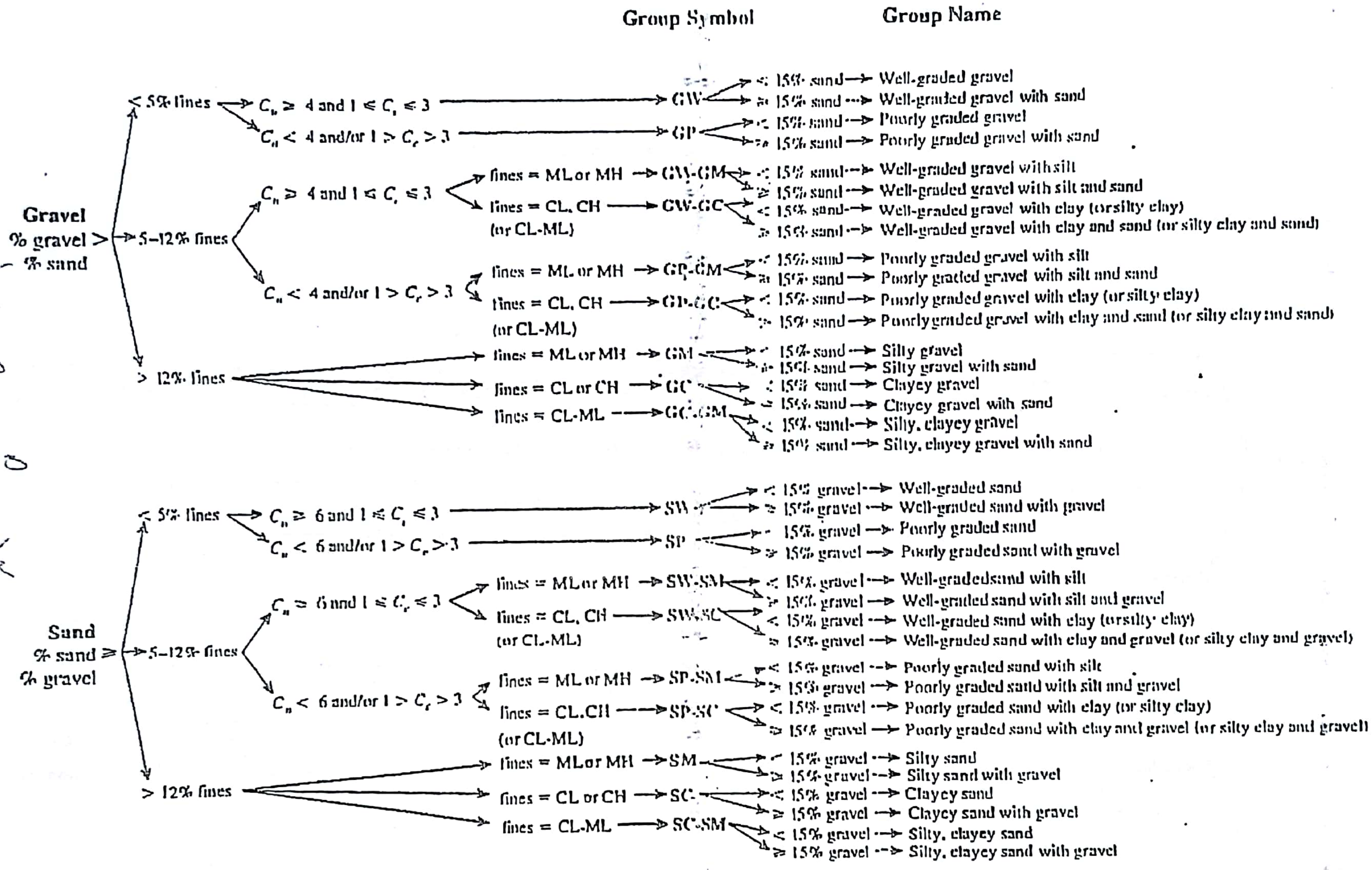


Figure 1.6 Flowchart for classifying coarse-grained soils (more than 50% retained on No. 200 Sieve) (After ASTM, 2005)

→ Cours-grained soil → S → >12 → SC

3.7 Organic Soils

- **Highly organic soils- Peat (Group symbol PT)**
  - A sample composed primarily of vegetable tissue in various stages of decomposition and has a fibrous to amorphous texture, a dark-brown to black color, and an organic odor should be designated as a highly organic soil and shall be classified as peat, PT.

- **Organic clay or silt(group symbol OL or OH):**
  - "The soil's liquid limit (LL) after oven drying is less than 75 % of its liquid limit before oven drying." If the above statement is true, then the first symbol is O.
  - The second symbol is obtained by locating the values of PI and LL (not oven dried) in the plasticity chart.

3.8 Borderline Cases (Dual Symbols)

For the following three conditions, a dual symbol should be used.

- Coarse-grained soils with 5% - 12% fines.
  - **About 7 % fines can change the hydraulic conductivity of the coarse-grained media by orders of magnitude.**
  - The first symbol indicates whether the coarse fraction is well or poorly graded. The second symbol describe the contained fines. For example: SP-SM, poorly graded sand with silt.
- Fine-grained soils with limits within the shaded zone. (PI between 4 and 7 and LL between about 12 and 25).
  - It is hard to distinguish between the silty and more claylike materials.
  - CL-ML: Silty clay, SC-SM: Silty, clayed sand.
- Soil contain similar fines and coarse-grained fractions.
  - possible dual symbols GM-ML

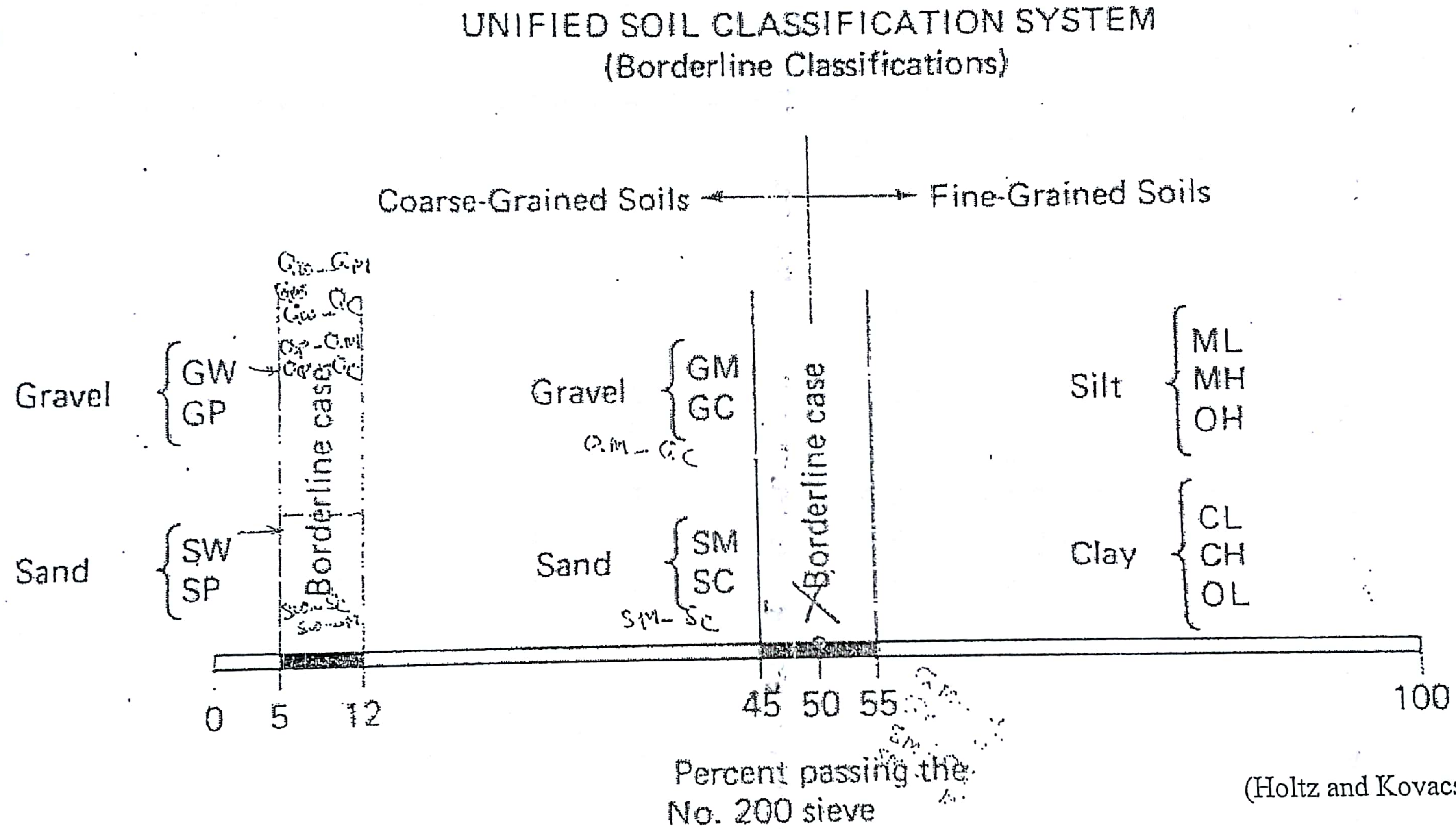
GC – CL

SC – SM

CL – ML

SC – SM

3.8 Borderline Cases (Summary)



(Holtz and Kovacs, 1981)

Note: Only two group symbols may be used to describe a soil.
Borderline classifications can exist within each of the above groups.

6. References

Main References:

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall. (Chapter 3)

Das, B.M. (1998). *Principles of Geotechnical Engineering*, 4th edition, PWS Publishing Company. (Chapter 3)

EXAMPLE 3.1

Given:

Sieve analysis and plasticity data for the following three soils.

Sieve Size	Soil 1, % Finer	Soil 2, % Finer	Soil 3, % Finer
No. 4	99	97	100
No. 10	92	90	100
No. 40	86	40	100
No. 100	78	8	99
No. 200	60	5	97
LL	20	—	124
PL	15	—	47
PI	5	NP[a]	77

[a]Nonplastic.

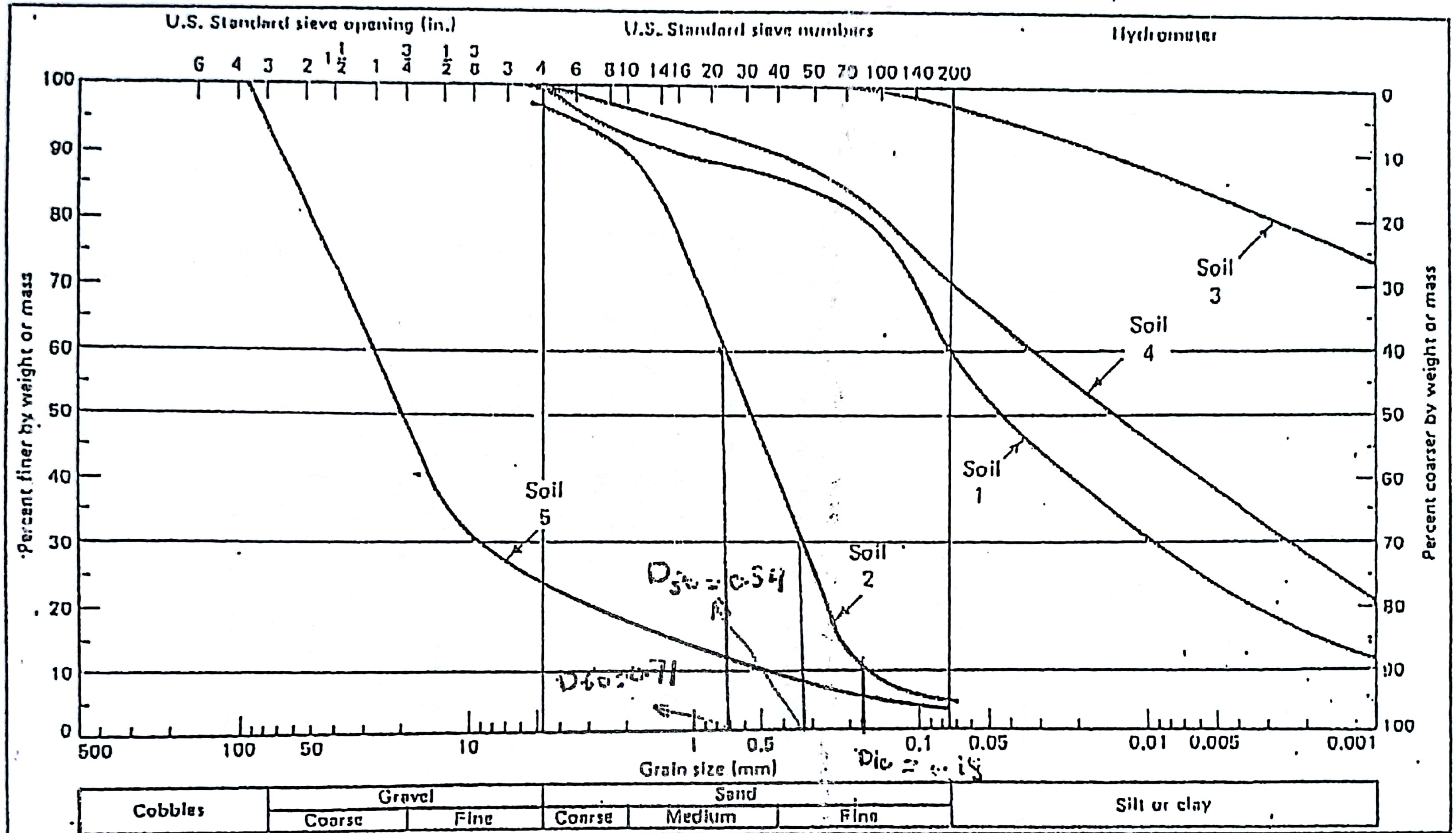
Required:

Classify the three soils according to the Unified Soil Classification System.

Solution:

Use Table 3-2 and Fig. 3.4.

1. Plot the grain size distribution curves for the three soils (shown in Fig. Ex. 3.1).
2. For soil 1, we see from the curve that more than 50% passes the No. 200 sieve (60%); thus the soil is a fine-grained soil and the Atterberg limits are required to further classify the soil. With LL = 20 and PI = 5, the soil plots in the hatched zone on the plasticity chart. Therefore the soil is a CL-ML.
3. Soil 2 is immediately seen to be a coarse-grained soil since only 5% passes the No. 200 sieve. Since 97% passes the No. 4 sieve, the soil is a sand rather than a gravel. Next, note the amount of material passing the No. 200 sieve (5%). From Table 3-2 and Fig. 3.4, the soil is "borderline" and therefore has a dual symbol such as: SP-SM or SW-SM depending on the values of C_u and C_c. From the grain size distribution curve, Fig. Ex. 3.1, we find that $D_{60} =$ 0.71 mm, $D_{30} = 0.34$ mm, and $D_{10} = 0.18$ mm. The coefficient of



Sample No.	Elev. or depth	Classification	Nat w%	LL	PL	PI	Project Ex. 3.1
1				20	15	5	
2						NP	Area
3				124	47	77	
4				49	24	25	Boring no.
5						NP	Date
Gradation Curves							

Fig. Ex. 3.1

uniformity C_u is

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.71}{0.18} = 3.9 < 6$$

and the coefficient of curvature C_c is

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(0.34)^2}{0.18 \times 0.71} = 0.91 \simeq 1$$

For a soil to be considered well graded, it must meet the criteria shown in column 6 of Table 3-2; it does not, so the soil is considered poorly graded and its classification is SP-SM. The soil is SM because the fines are silty (nonplastic).

4. A quick glance at the characteristics for soil 3 indicates the soil is fine grained (97% passes the No. 200 sieve). Since the LL is greater than 100 we cannot directly use the plasticity chart (Fig. 3.2). Use instead the equation for the A-line on Fig. 3.2 to determine if the soil is a CH or MH.

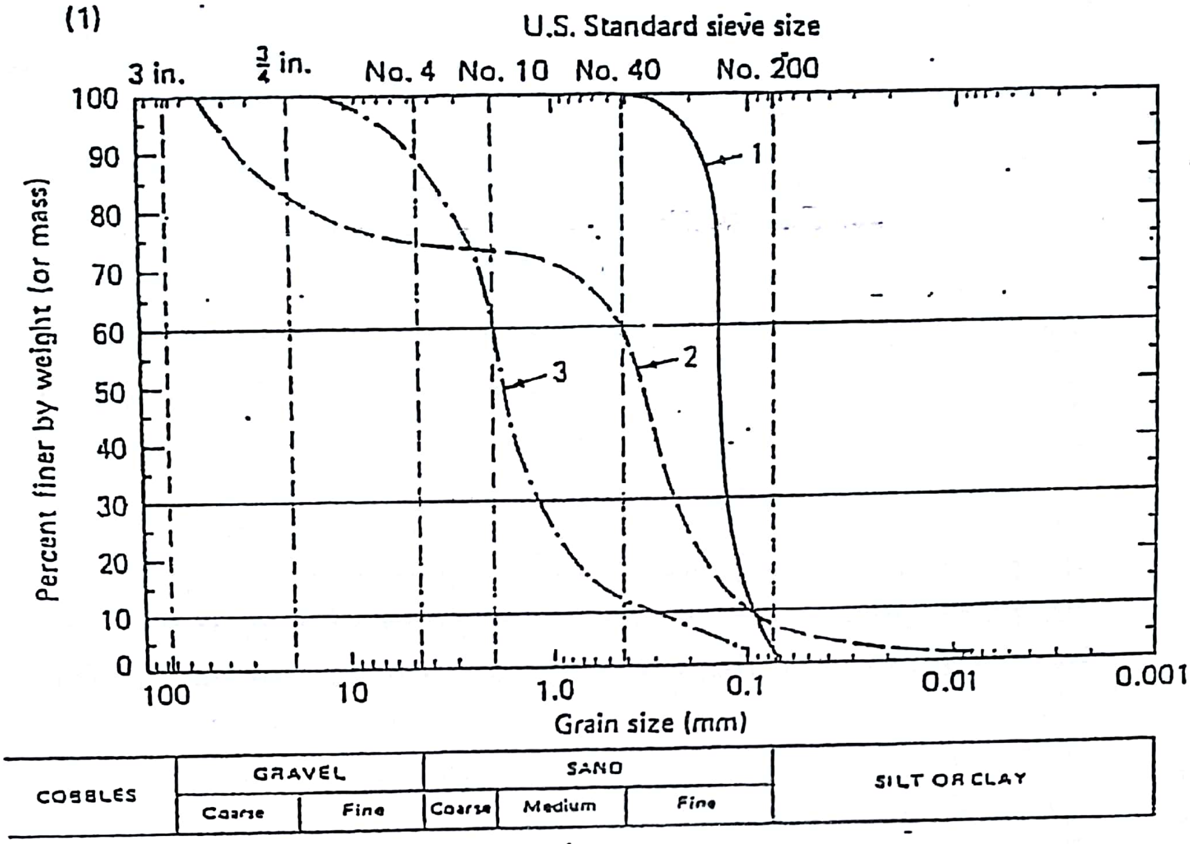
$$PI = 0.73(LL - 20) = 0.73(124 - 20) = 75.9$$

Since the PI is 78 for soil 3, it lies *above* the A-line and thus the soil is classified as a CH.

3.3 THE AASHTO SOIL CLASSIFICATION SYSTEM

In the late 1920's the U.S. Bureau of Public Roads (now the Federal Highway Administration) conducted extensive research on the use of soils especially in local or secondary road construction, the so-called "farm-to-market" roads. From that research the Public Roads Classification System was developed by Hogentogler and Terzaghi (1929). The original system was based on the stability characteristics of soils when used as a road surface or with a thin asphalt pavement. There were several revisions since 1929, and the latest in 1945 is essentially the present AASHTO (1978) system. The applicability of the system has been extended considerably; AASHTO states that the system should be useful for determining the relative quality of soils for use in embankments, subgrades, subbases, and bases. But you might keep in mind its original purpose when using the system in your engineering practice. (See Casagrande, 1948, for some comments on this point.)

(1)

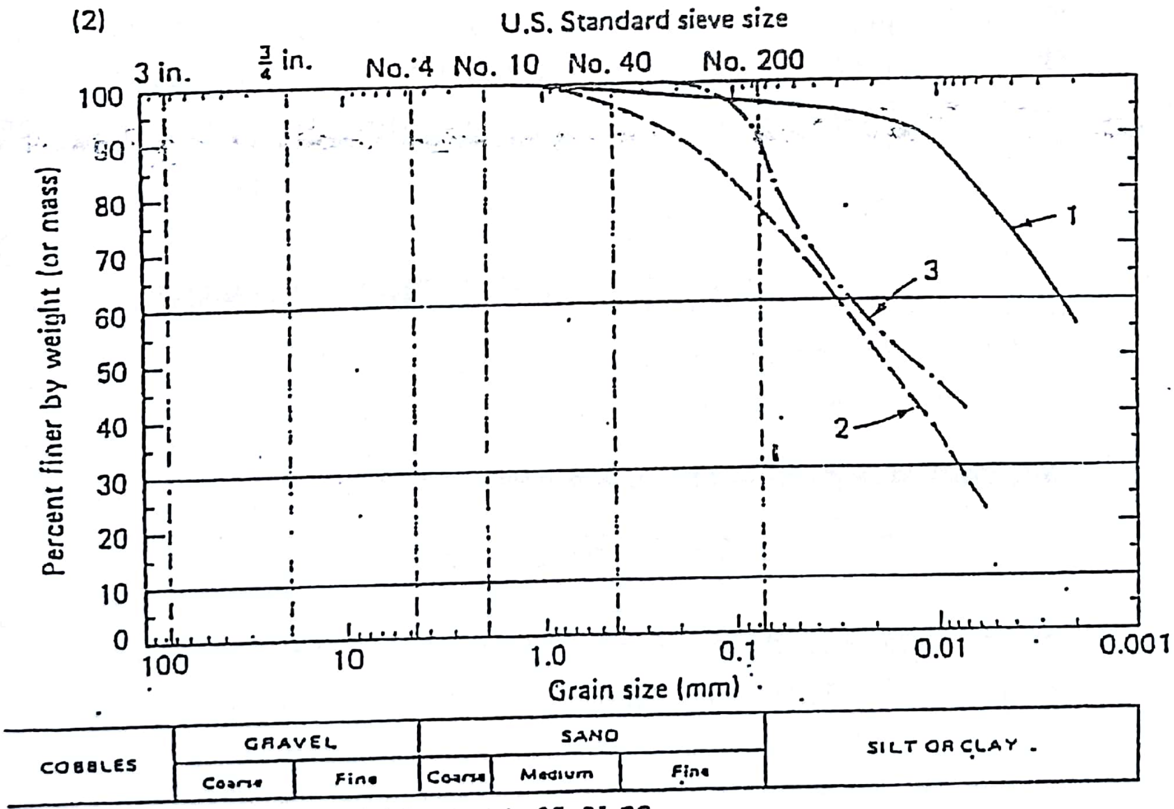


Curve 1: Uniform fine sand; nonplastic.

Curve 2: Poorly graded gravelly sand mixture; nonplastic. Approximately 7% fines makes this a borderline soil, symbol SP-SM.

Curve 3: Coarse to medium sand; nonplastic. Approaching uniform gradation.

(2)



Curve 1: Organic clay (tidal flats); LL=95, PI=39.

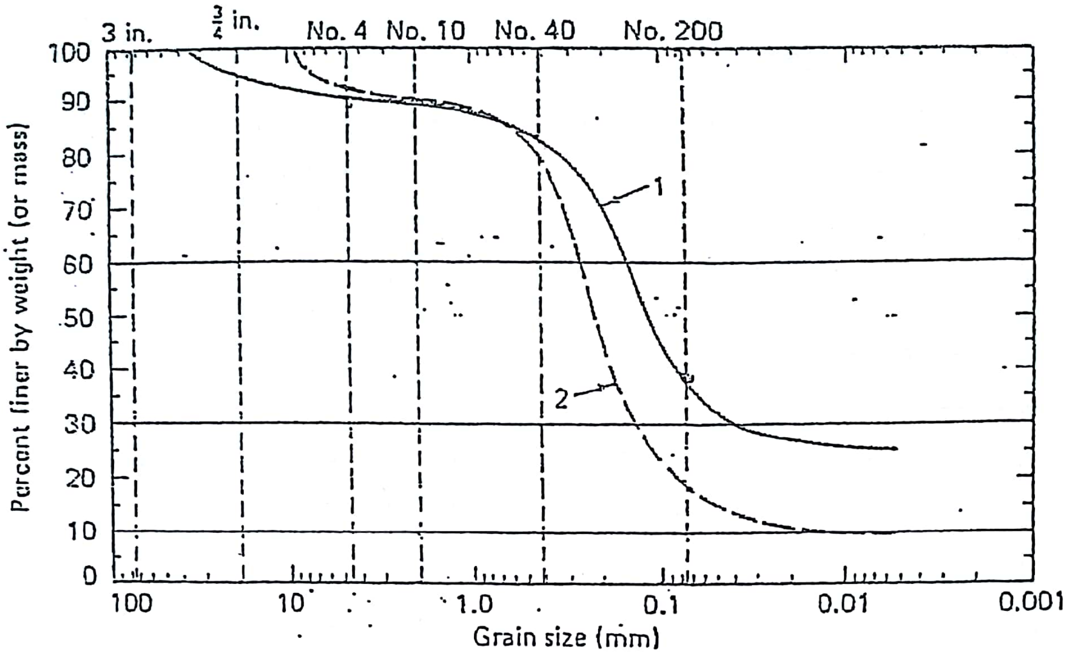
Curve 2: Alkali clay with organic matter; LL=66, PI=27.

Curve 3: Organic silt; LL=70, PI=33 (natural water content); LL=53, PI=19 (oven dried).

Fig. P3-6

D 2487

(3)

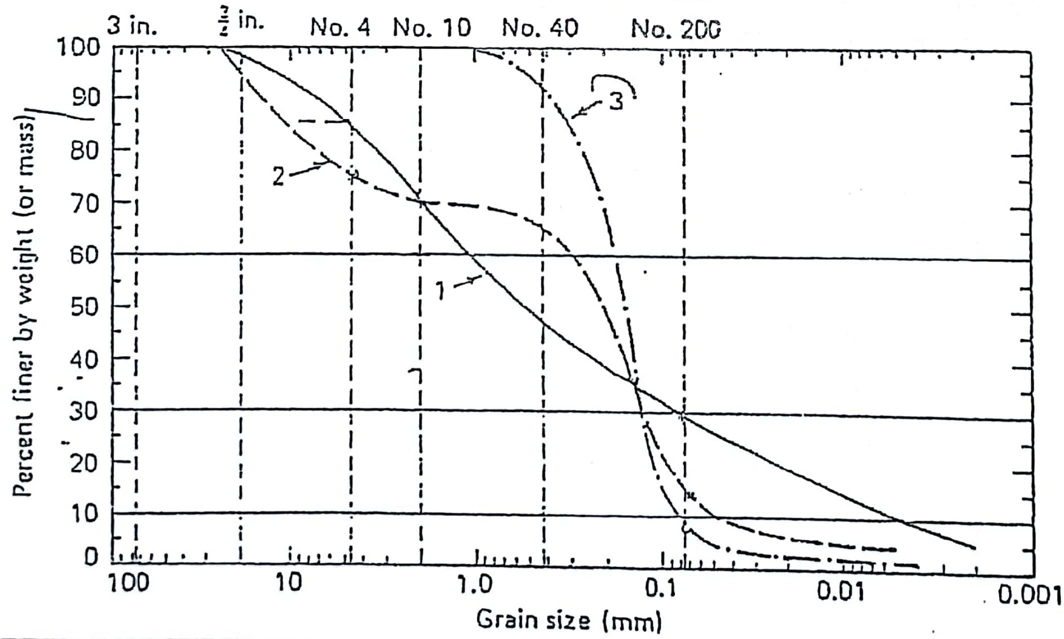


ASHTO

COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Curve 1: Clayey sand; LL=23, PI=10. Poorly graded mixture of sand-clay and fine silty sand.
Curve 2: Limerock and sand mixture; LL=23, PI=8. Poorly graded.

(4)



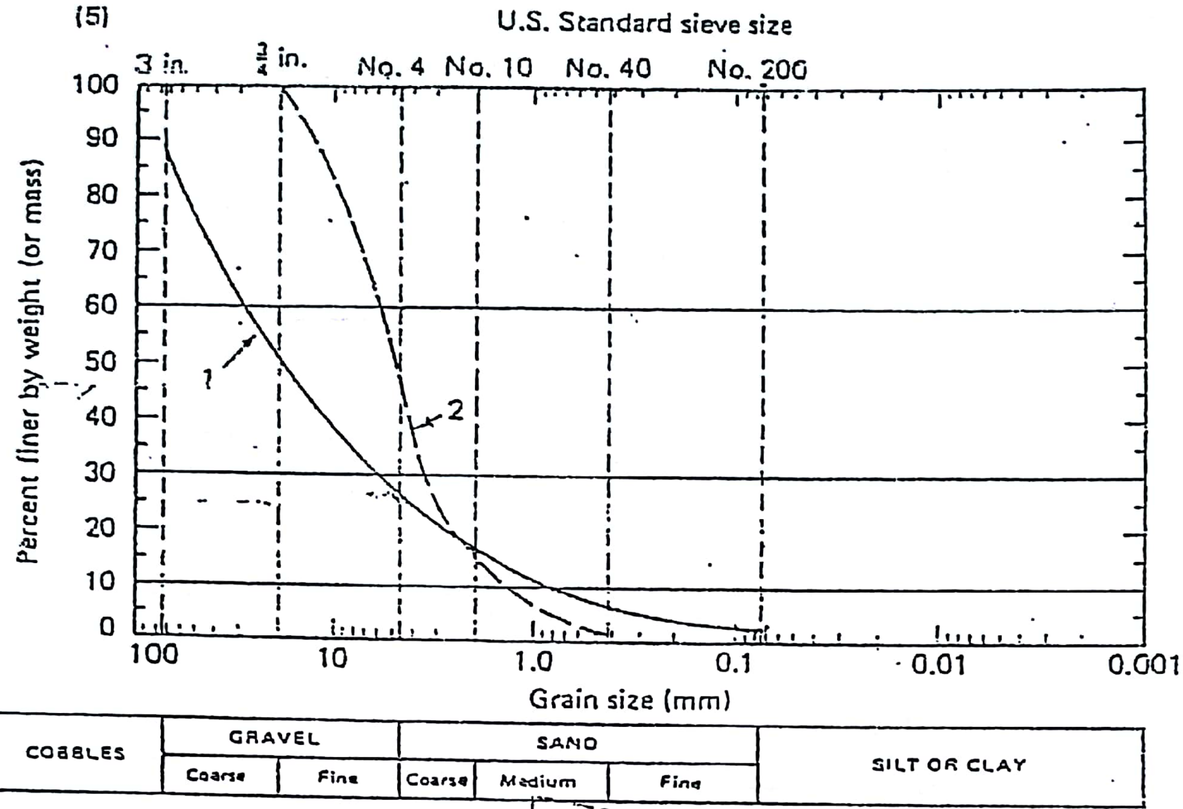
USCS

COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Curve 1: Silty gravelly sand; nonplastic. Micaceous silt stabilized with sandy chert gravel.
Curve 2: Mixture of gravel-sand and fine silty sand; nonplastic. Poorly graded mixture; note absence of coarse and medium sand.
Curve 3: Silty fine sand; LL=22, PI=5.

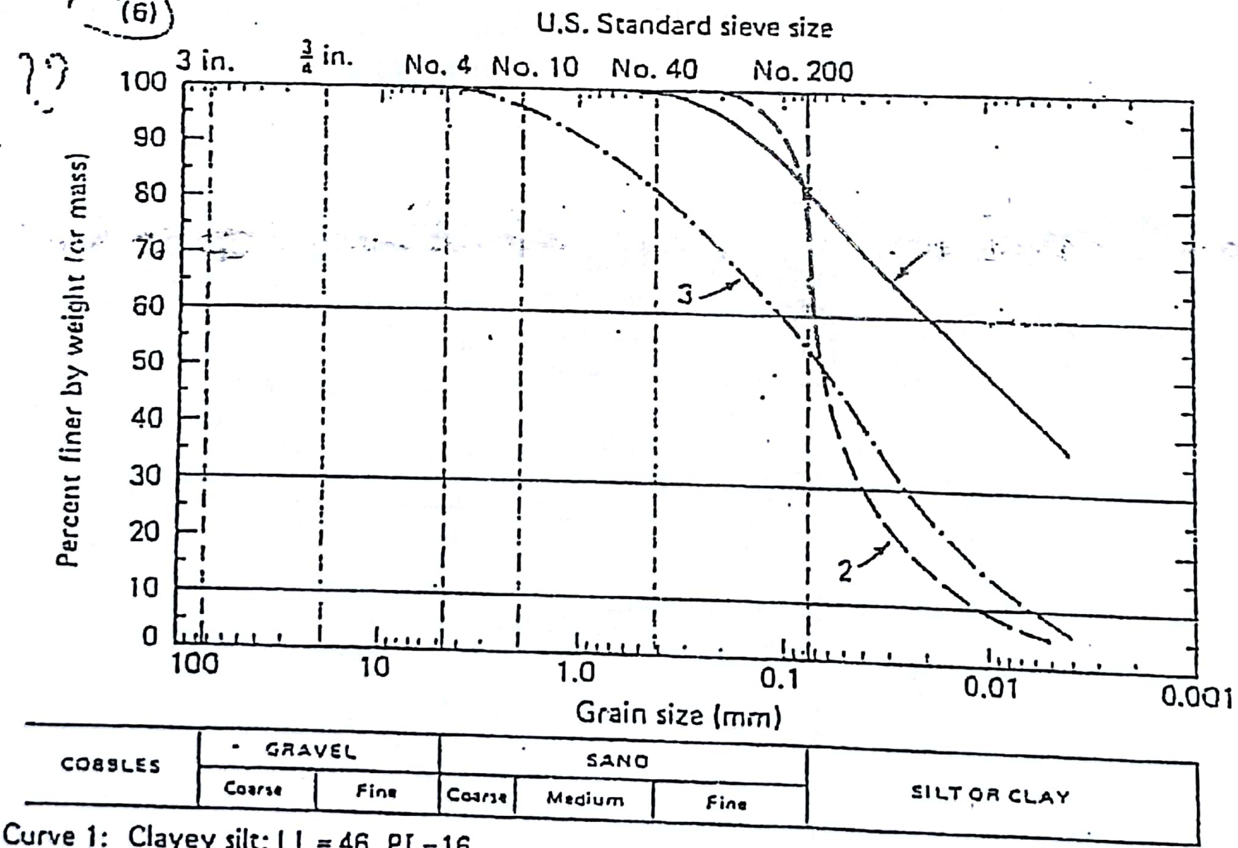
Fig. P3-6 Continued.

(5)



Curve 1: Pit run gravel; nonplastic; well graded; small percentage of fines.
Curve 2: Sandy gravel; nonplastic; no fines.

(6)



Curve 1: Clayey silt; LL = 46, PI = 16.
Curve 2: Uniform sandy silt; LL = 30, PI = 3.
Curve 3: Sandy silt; LL = 34, PI = 3
General: Note curves 2 and 3 have about the same plasticity but vary in grain size distribution.

Fig. P3-6 Continued.

Compaction

عملية دمج / دمك التربة
و الهدف الرئيسي هو زيادة كثافة التربة من خلال تقليل الحجم عن طريق إخراج الهواء → dense state

$$* \; V_T = V_s + V_w + \underline{V_a}$$

reduce this term

* يجب التفريق بين عملية الـ "Consolidation" وعملية الـ "Compaction".

Consolidation ⇒ $V_w \downarrow$
compaction ⇒ $V_a \downarrow$

* اهداف عملية الـ compaction :

1) تقليل الهبوط في التربة عند تعرضها للحمل (Load)
2) تقليل النفاذية
3) زيادة القوة القصية
4) الطمم المستخدم في المطارات والطرق بحاجة إلى دمك كي يتم الاستفادة منه على أكمل وجه

General compaction Methods

* هنالك طرق مخبرية وأخرى تستخدم في الموقع ولكل نوع من انواع التربة طرق خاصة

* يجب حفظها (على الأقل طريقة لكل حالة)

* تأتي على شكل املأ الفراغ / إذكر

Laboratory Compaction

* أشهر الاختبارات المتعلقة بعملية ال compaction
يسمى : "proctor test"

⇐ * الهدف من هذا الاختبار : لتحديد كمية الماء التي يجب خلطها مع التربة حتى تصل التربة الى أكبر قيمة كثافة (dense state)

Comparison

proctor test → standared / modified

	standard Test	Modified Test
"height of drop"	12 in = 1 ft	18 in = 1.5 ft
"hammer wieght"	5.5 lb	10 lb
"blows / layer"	25	25
"mold size"	$1/30\ ft^3$	$1/30\ ft^3$
"Energy"	$12375\ ft \cdot lb/ft^3$	56250
"layers"	3	5

⇐ * لاحظ انه الطاقة في ال modified أكبر ، فسر ذلك ؟؟

What are the variables of Compaction? ~~impact compaction~~
(Four points) ⇒ From slide

* compactive effort depends on :- + impact compaction in proctor
1 - weight of hammer 5 - height of drop
2 - Number of blows per layer
3 - Number of layers
4 - Volume of mold

Results

* $\rho = \frac{M_t}{V_t}$
* $\rho_d = \frac{\rho}{1+w}$

w: water content
ρ_d: dry density
ρ: wet / total density

* عند تعرض التربة للرص تصبح الجزيئات = الحجر تقارباً و انتظاماً
* وبالتالي يقل الحجم الكلي

* وبما أن $\rho = \frac{m}{v}$ ⇐ الكثافة تزداد

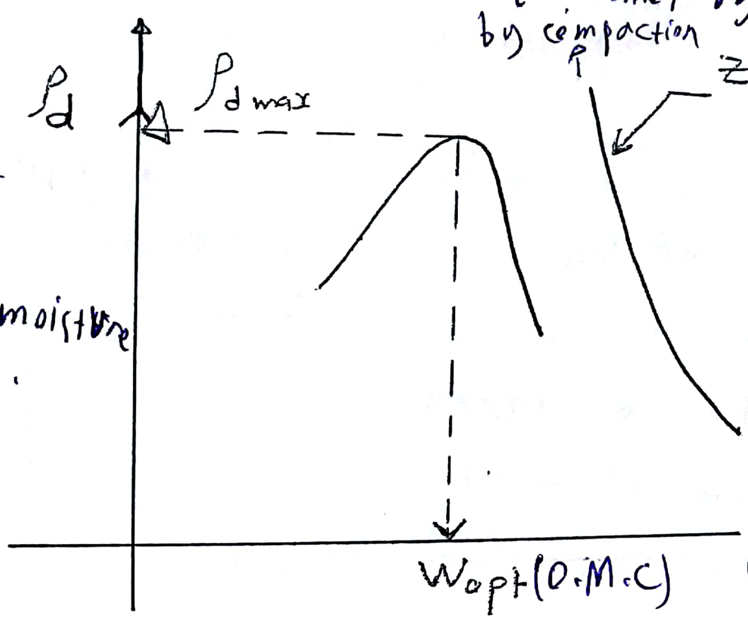
* قياس كثافة التربة وكمية الماء (w) الموجودة بعد كل عملية دمك / رص يمكننا من حساب (ρ_d)
حيث أنها تساوي $\frac{\rho}{1+w}$

* بالاعتماد على العلاقة $\rho_d = \frac{\rho}{1+w}$ يمكننا رسم منحنى العلاقة الذي يربط (ρ_d) و (w) و يسمى هذا المنحنى ب (Compaction curve)

① Peak point :-

O.M.C : optimum moisture content.

opt :- optimum

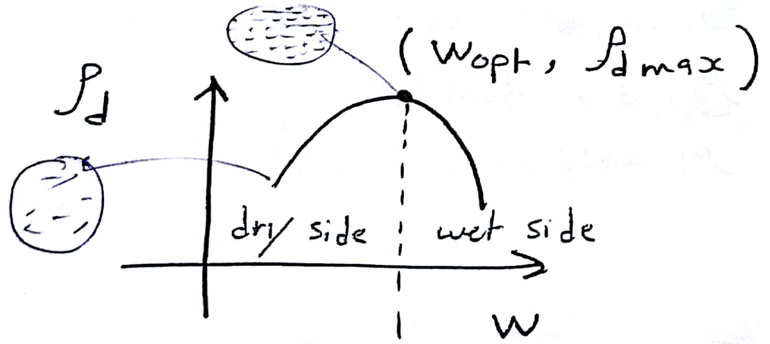


$\rho_{d\,modified} > \rho_{d\,stan}$

O.M.C modified < O.M.C stan

$$* \quad \rho_d = \frac{\rho}{1+w} = \frac{\rho_w \, S_r}{w + \frac{S_r}{G_s}} = \frac{\rho_s}{1+e}$$

W_{opt} : كمية الماء التي يحدث عندها أكبر قيمة للكثافة



مناقشة

* قبل وصول الـ W_{opt}

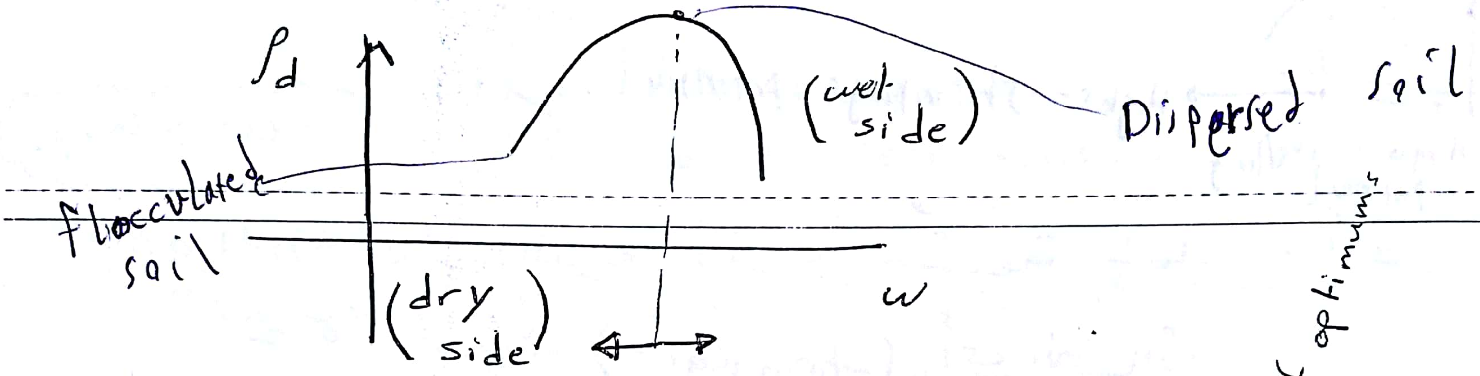
زيادة الماء في هذه المرحلة ليساعد في تشكيل غلاف ليسهل حركة جزيئات التربة ويجعلها في وضع أكثر إنتظاما ليعطي (denser configuration)

* وصول الـ W_{opt}

عند وصول كمية الماء الى حد لتصبح الكثافة أقصى ما يمكنه ولنه تزداد الكثافة بعد ذلك "ابدا" -

ما بعد الـ W_{opt}

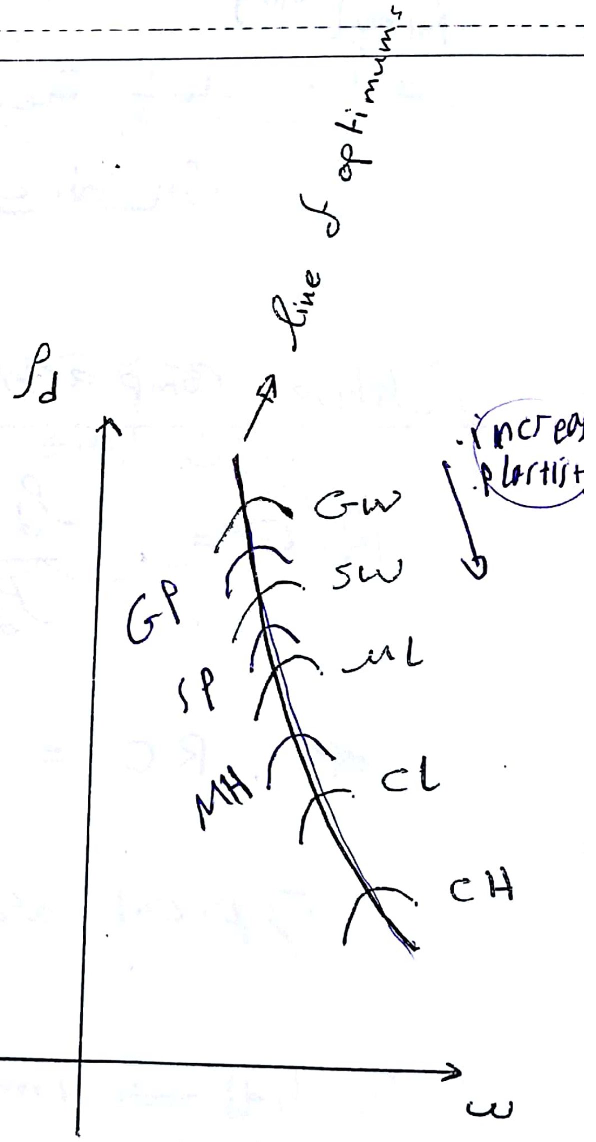
يبدأ الماء بالتفوق من حيث الكمية على كمية التربة ، اي انه كمية الماء تصبح اكثر من التربة ، تصبح التربة وكأنها مذابة في الماء ⇐ تصبح التربة "محلول" ⇐ وهذا يؤدي الى انخفاض الكثافة



Effect of soil Types

* هذه الرسمة يجب فهمها وحفظها

* عادة ما تأتي على شكل ارسم



⇒ ρ_d For Gravel > sand

For silt > Clay

For low limit > high limit

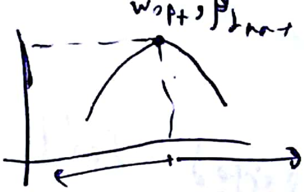
← لكن لو تم ترتيبهم حسب W_{opt} سيتم عكس كل الترتيب حيث انه الأعلى في ρ_{dmax} هو الأقل W_{opt}

كلما زادت Plasticity يقل ρ_d

كلما زاد size زاد ρ_d وقل O.M.C

In case $w < w_{opt}$

في هذه الحالة تكون التربة شبه جافة ومتعطشة للماء ولذلك فهي أكثر ميولاً لحدوث (swelling) أي إنتفاخ عند تعرضها للماء.



In case $w > w_{opt}$

في هذه الحالة تكون التربة تقريباً مشبعة بالماء وأكثر ميولاً لحدوث (shrinkage) أي انكماش.

Relative Compaction (RC)

$$R.C^{*} = \frac{\rho_{d\text{-}field}}{\rho_{d\,max\text{-}lab}} * 100\%$$

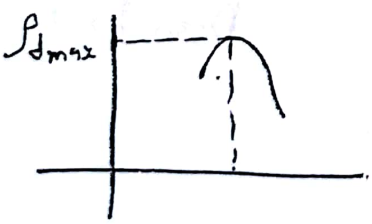
or $RC = 80 + 0.2\,D_r$ (for gravel and sand) $\rightarrow$ relative density

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

Typical value (90 − 95)%.

$\rho_{d\text{-}field} \Rightarrow$ From field $\Rightarrow$ Given in exam.

$\rho_{d\,max\text{-}lab} \Rightarrow$ From compaction curve



or $$\rho_{d\,max\text{-}lab} = \frac{\rho_w \times S_r}{W_{opt} + \frac{S_r}{G_s}}$$

$$W_c = \frac{M_w}{M_s} = \frac{M_t - M_s}{M_s} * 100\% = O.M.C \pm 3\%$$

…ermine "RC" in Field
OR Determine ρ_d - field

⇒ Two Types of methods
↳ 1) Destructive : sand cone / Balloon / oil
2) Non destructive method :
↳ Nuclear density

⇐ * حفظ أسماء الطرق + فهم آلية كل طريقة
+ سلبيات كل طريقة

Do your Best ⇒ Easy come Easy go

Water in Soil
Sections 2.1-2.6 in Craig

Outlines

- ☐ Introduction
- ☐ Darcy's Law
- ☐ Volume of water flowing per unit time
- ☐ Measuring K in laboratory
- ☐ Seepage Theory
- ☐ Flow Net

Introduction

- All soils are permeable materials, water being free to flow through the interconnected pores between the solid particles.
- You must know how much water is flowing through a soil per unit time.
- This knowledge is required to
  - Design earth dams.
  - Determine the quantity of seepage under hydraulic structures.
  - and dewater foundations before and during their construction.
- The pressure of the pore water is measured relative to atmospheric pressure and the level at which the pressure is atmospheric (i.e. zero) is defined as the water table (WT) or the phreatic surface.
- Below the water table the soil is assumed to be fully saturated,
- Below the water table the pore water may be static, the hydrostatic pressure depending on the depth below the water table, or may be seeping through the soil under hydraulic gradient: this chapter is concerned with the second case.

Introduction

- Bernoulli's theorem applies to the pore water but seepage velocities in soils are normally so small that velocity head can be neglected

$$\boxed{h = \frac{u}{\gamma_w} + z} \Rightarrow \text{pressure head}$$

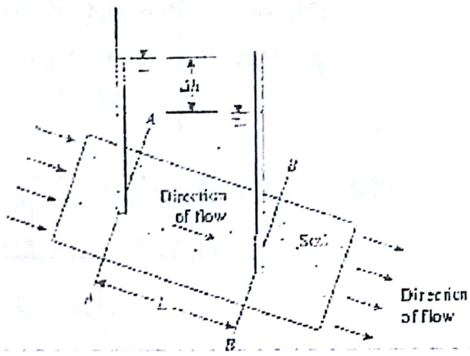
where h is the total head, u the pore water pressure, γ_w the unit weight of water (9.8 kN/m^3) and z the elevation head above a chosen datum.

#احفظ فقط الي معملك عليه#

Darcy's law

- Darcy (1856) proposed the following equation for calculating the velocity of flow of water through a soil:

$$\boxed{v = ki}$$



In this equation,
v = Darcy velocity (unit: cm/sec)
k = hydraulic conductivity of soil (unit: cm/sec)
i = hydraulic gradient

The hydraulic gradient is defined as

$$\boxed{i = \frac{\Delta h}{L}}$$

where
Δh = piezometric head difference between the sections at AA and B B
L = distance between the sections at AA and BB
(Note: Sections AA and BB are perpendicular to the direction of flow.)

Volume of water flowing per unit time

$$q = Aki$$

where q is the volume of water flowing per unit time, A the cross-sectional area of soil corresponding to the flow,

The K also varies with temperature, upon which the viscosity of the water depends. If the value of k measured at 20 C is taken as 100% then the values at 10 and 0 C are 77 and 56%, respectively. The coefficient of permeability can also be represented by the equation:

$$k = \frac{\gamma_w}{\eta} K$$

where γ_w is the unit weight of water, the viscosity of water η and K (units m2) an absolute coefficient depending only on the characteristics of the soil skeleton.

The values of k for different types of soil are typically within the ranges shown in Table

Table 2.1 Coefficient of permeability (m/s) (BS 8004: 1986)

1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
Clean gravels		Clean sands and sand–gravel mixtures			Very fine sands, silts and clay-silt laminate			Unfissured clays and clay-silts (>20% clay)		
				Desiccated and fissured clays						

seepage velocity

On the microscopic scale the water seeping through a soil follows a very tortuous path between the solid particles but macroscopically the flow path

The seepage velocity

$$v = \frac{q}{A_v} \qquad v = \frac{k i}{n}$$

seepage velocity

$$\boxed{v = \frac{k i}{n}}$$

Av: the average area of voids

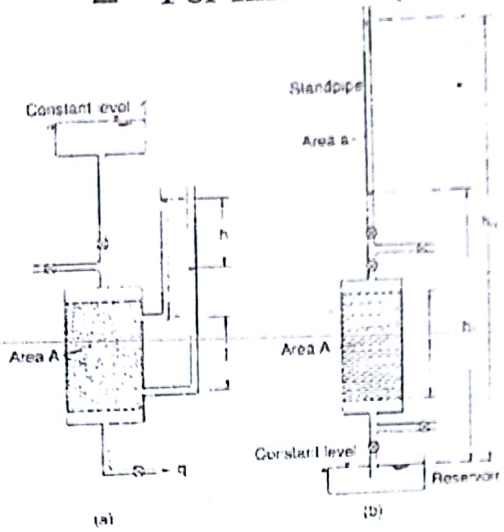
The porosity, n, can also be expressed as

$$n = \frac{A_v}{A} = \frac{V_v}{V_T}$$

Measuring K in laboratory

☐ Two Main Method

- The coefficient of permeability for coarse soils can be determined by means of the constant-head permeability test
- For fine soils (clays and silt) the falling-head test should be used



Laboratory permeability tests:
(a) constant head and
(b) falling head.

Measuring K in laboratory

(a) constant head and

$$k = \frac{ql}{Ah}$$

مش مهم

(b) falling head.

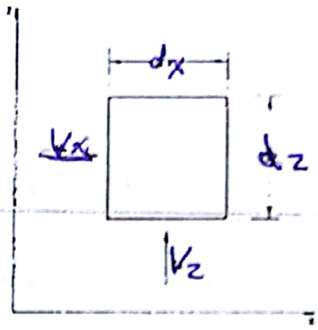
$$k = \frac{al}{At_1} \ln \frac{h_0}{h_1}$$
$$= 2.3 \frac{al}{At_1} \log \frac{h_0}{h_1}$$

For Clean Uniform sands, Hazen showed that the approximate value of k is given by $k = 10^{-2} D_{10}^2 \ (m/s)$

where D_{10} is the effective size in mm.

Seepage Theory

- ☐ The general case of seepage in two dimensions will now be considered
- ☐ Assumption
  - soil is homogeneous and isotropic
  - Generalized Darcy Law will be used



Derivation

$$v_x = k i_x = -k\frac{\partial h}{\partial x} \quad (1)$$

$$v_z = k i_z = -k\frac{\partial h}{\partial z}$$

total head h decreasing in the directions of v_x and v_z

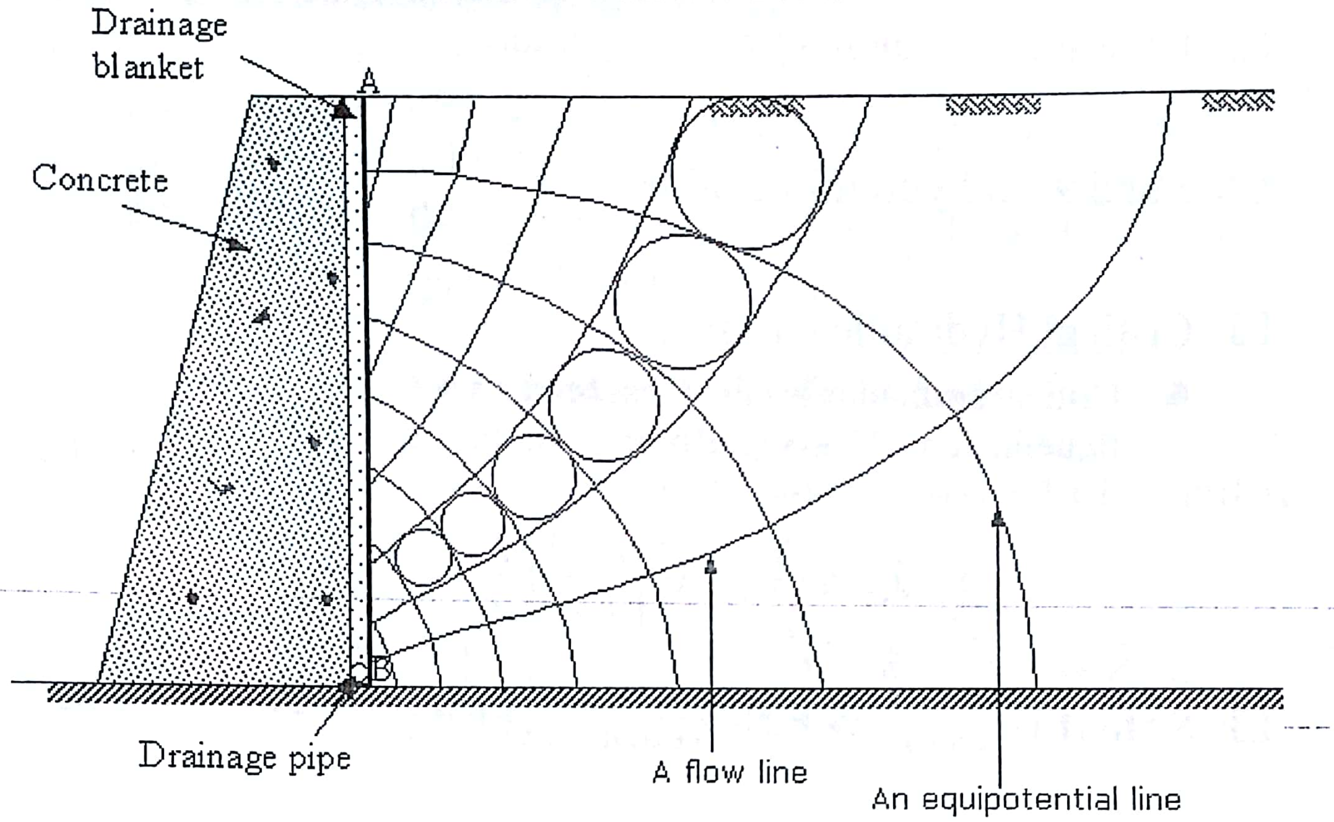
$q_{in}=q_{out}$ (2)

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

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0 \quad (3)$$

equation of continuity in two dimensions.

Flow Net in Backfill of Retaining Wall



INTERPRETATION OF FLOW NET

- Head loss between each consecutive pair of equipotential lines

$$\Delta h = \frac{\Delta H}{N_d}$$

- Flow through each flow channel for an isotropic soil from Darcy's Law

$$\Delta q = Aki = (b \times 1)\,k\frac{\Delta h}{L} = k\Delta h\frac{b}{L} = k\frac{\Delta H}{N_d}\frac{b}{L}$$

- Total flow

$$q = k\sum_{i=1}^{N_f}\left(\frac{\Delta H}{N_d}\right)_i = k\Delta H\frac{N_f}{N_d}$$

Hydraulic Gradient

- Hydraulic gradient over each square

$$i = \frac{\Delta h}{L}$$

- Maximum hydraulic gradient

$$i_{max} = \frac{\Delta h}{L_{min}}$$

- Critical Hydraulic Gradient
  - Critical hydraulic gradient that brings a soil mass to static liquefaction, , Heaving, Boiling, and Piping

$$i = i_{cr} = \frac{\gamma'}{\gamma_w} = \left(\frac{G_s - 1}{1+e}\right)\frac{\gamma_w}{\gamma_w} = \frac{G_s - 1}{1+e}$$

- Safe if $i < i_{critical} \Rightarrow F.S = i_{critical}/i_{exit} \geq 1.0$

Pore Water Pressure Distribution

- **Pressure head**

$$(h_p)_j = \Delta H - (N_d)_j \Delta h - h_z$$

- **Pore water pressure**

$$u_j = (h_p)_j \gamma_w$$

- **Uplift Forces**

$$P_w = \sum_{j=1}^{n} u_j \Delta x_j$$

- **Calculating the uplift force per unit length using Simpson's rule**

$$P_w = \frac{\Delta x}{3}\left(u_1 + u_n + 2\sum_{\substack{i=3\\ \text{odd}}}^{n} u_i + 4\sum_{\substack{i=2\\ \text{even}}}^{n} u_i\right)$$

ANISOTROPIC SOIL CONDITIONS

- Most natural soil deposits are anisotropic, with the coefficient of permeability having a maximum value in the direction of stratification and a minimum value in the direction normal to that of stratification; these directions are denoted by x and z, respectively, i.e.

$$k_x = k_{max} \quad \text{and} \quad k_z = k_{min}$$

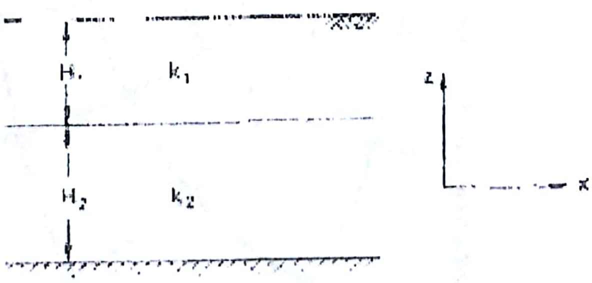
- Same solution but you need to have x_t instead of x and K' instead of K in flow equation.

$$x_t = x\sqrt{\frac{k_z}{k_x}} \qquad k' = k_x\sqrt{\frac{k_z}{k_x}} = \sqrt{k_x k_z}$$

Non-homogeneous Soil Conditions

- For horizontal flow, the head drop Dh over the same flow path length H1+H2 will be the same for each layer.

$$\bar{k}_x = \frac{H_1 k_1 + H_2 k_2}{H_1 + H_2}$$



- For vertical flow, the flow rate q through area A of each layer is the same.

$$\bar{k}_z = \frac{H_1 + H_2}{\left(\frac{H_1}{k_1}\right) + \left(\frac{H_2}{k_2}\right)}$$

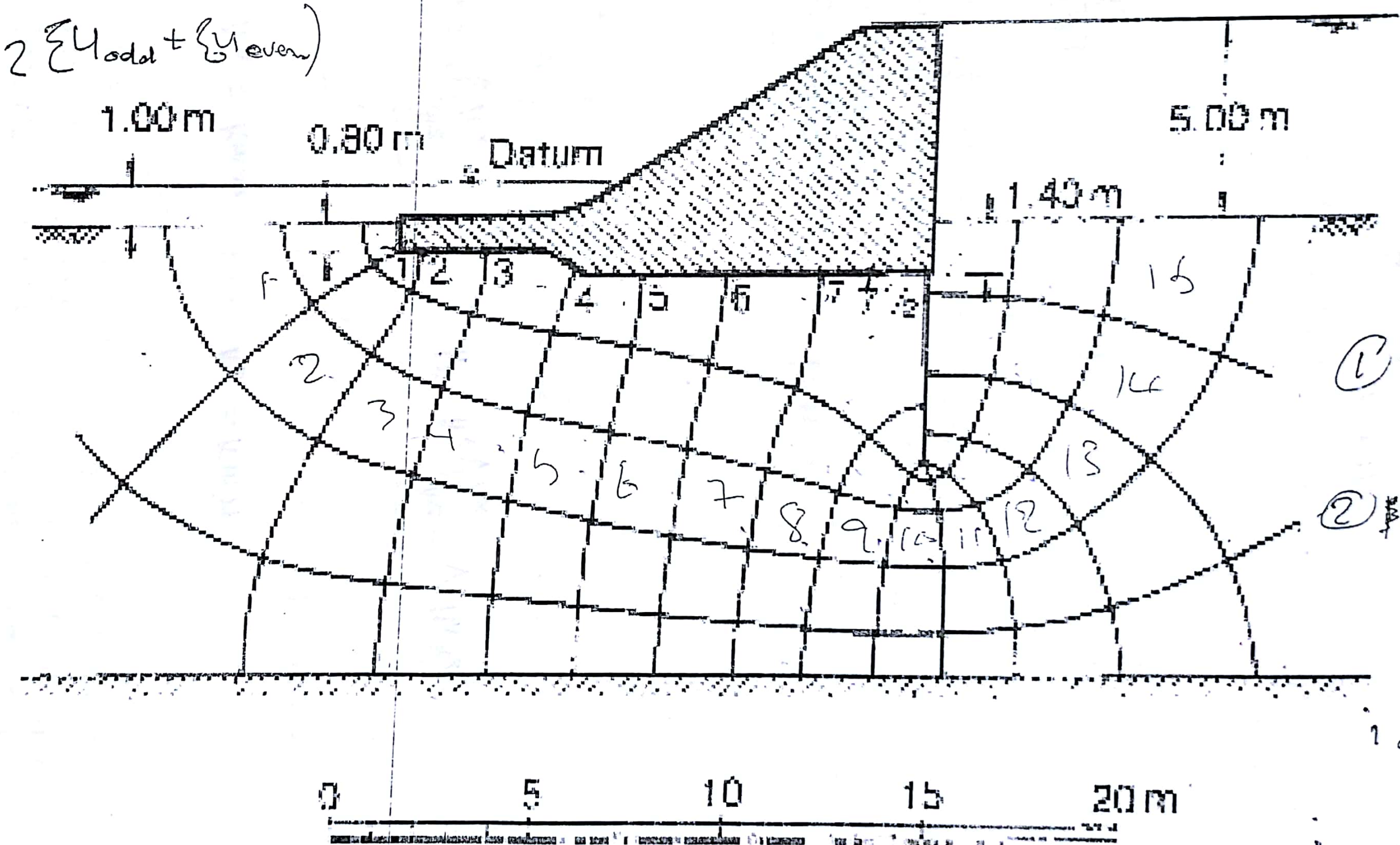
Example 2.2 (text)

- The section through a dam is shown in Figure. Determine the quantity of seepage under the dam and plot the distribution of uplift pressure on the base of the dam. The coefficient of permeability of the foundation soil is $2.5x10^{-5}$ m/s.

$\Delta H - \frac{N_d \Delta H}{N_d}$

$u = (h_p - h_z)\gamma_w$

$P = \frac{\Delta X}{3}(u_1 + u_n + 2\sum u_{odd} + \sum u_{even})$



$\Delta H = 4$

$N_D = 15$

$N_f = 5$

① $q = k \frac{\Delta H}{N_D} N_f$

② F.S. $= \frac{i_{cri}}{i_{max}}$

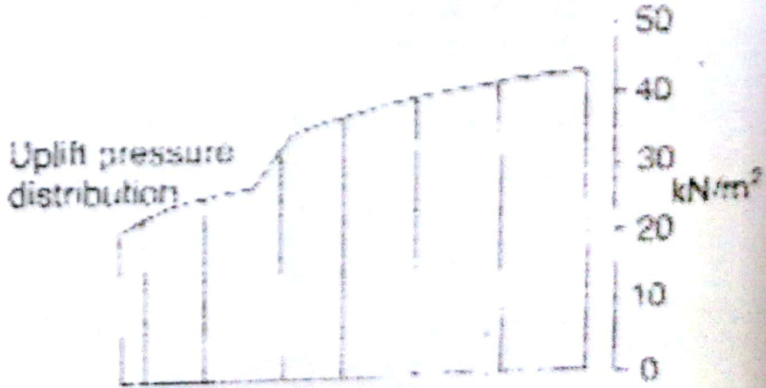
$i_{critical} = \frac{\gamma'}{\gamma_w}$

$i_{max} = \frac{\Delta H}{L \cdot N_D}$

$$q = kh\frac{N_f}{N_d} = 2.5 \times 10^{-5} \times 4.00 \times \frac{4.7}{15}$$

$$= 3.1 \times 10^{-5}\ \mathrm{m^3/s} \quad \text{(per m)}$$

Point	h (m) $(\Delta H - \frac{N_{d_i}}{N_d}\Delta H)$	z (m)	$h - z$ (m)	$u = \gamma_w(h_p - z)$ (kN/m^2)
1	0.27	1.80	2.07	20.3
2	0.53	1.80	2.33	22.9
3	0.80	1.80	2.60	25.5
4	1.07	2.10	3.17	31.1
5	1.33	2.40	3.73	36.6
6	1.60	2.40	4.00	39.2
7	1.87	2.40	4.27	41.9
$7\frac{1}{2}$	2.00	2.40	4.40	43.1



Constrained for Flow Net

CONSTRAINTS FOR SKETCHING FLOW NET

A flow net must satisfy the following criteria.

1. Flow conditions at entrances and exits.

2. Flow lines must intersect equipotential lines at right angles.

3. The area between flow lines and equipotential lines must be curvilinear squares. A curvilinear square has the property that an inscribed circle can be drawn to touch each side of the square and continuous bisection results, in the limit, to a point.

4. The quantity of flow through each flow channel is constant.

5. The head loss between each consecutive equipotential line is constant.

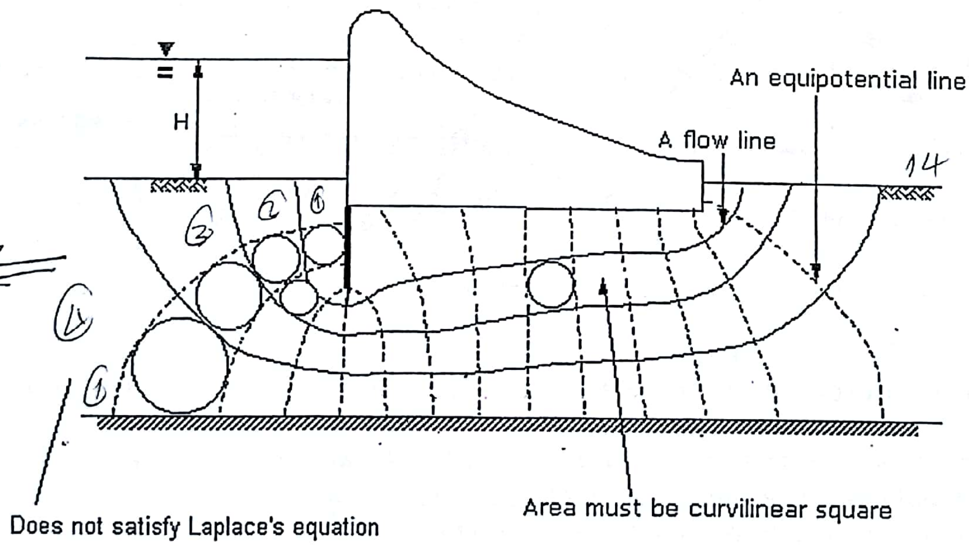
6. A flow line cannot intersect another flow line.

7. ~~An equipotential line cannot intersect another equipotential line.~~

An infinite number of flow lines and equipotential lines can be drawn to satisfy Laplace's equation. However, only a few are required to obtain an accurate solution. Thus, a very fine mesh may not result in a significant increase in accuracy.

Flow Net under Dam

Ex #2



Solution

- Two ⊥ Functions satisfy Laplace Equation
  - First function $\phi(x, z)$, called the potential function,
  - Second function $\psi(x, z)$, called the flow function,

مهم جدا

$$\frac{\partial \phi}{\partial x} = V_x = -k\frac{\partial h}{\partial x}$$
$$\frac{\partial \phi}{\partial z} = V_z = -k\frac{\partial h}{\partial z}$$

⟹ In Eq 3 $\quad \frac{\partial^2 \phi}{\partial x^2} - \frac{\partial^2 \phi}{\partial z^2} = 0$

تحقق المعادلة -

$$-\frac{\partial \psi}{\partial x} = V_z = -k\frac{\partial h}{\partial z}$$
$$\frac{\partial \psi}{\partial z} = V_x = -k\frac{\partial h}{\partial x}$$

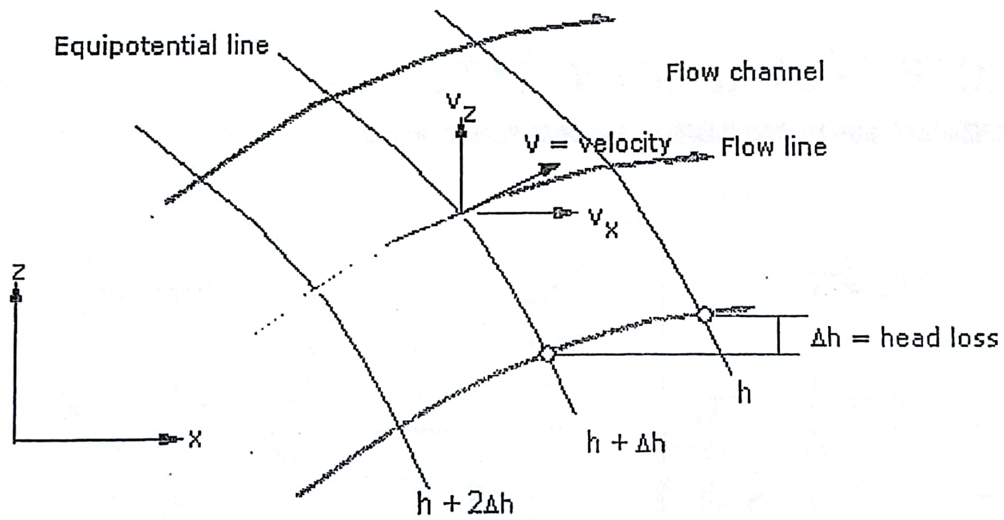
If the function $\psi(x, z)$ is given a constant value ψ_1 then $d\psi=0$ and

$$\frac{dz}{dx} = \frac{V_z}{V_x}$$

If $\phi(x, z)$ is constant then $d\phi = 0$ and

$$\frac{dz}{dx} = -\frac{V_x}{V_z}$$

Flow Net



A flow net is a graphical representation of a flow field and comprises a family of flow lines and equipotential lines. The flow terms are:

1. Flow lines or streamlines represent flow paths of particles of water.
2. The area between two flow lines is called a flow channel.
3. The rate of flow in a flow channel is constant.
4. Flow cannot occur across flow lines.
5. An equipotential line is a line joining points with the same head.
6. The velocity of flow is normal to the equipotential line.
7. The difference in head between two equipotential lines is called the potential drop or head loss.

Subject Water   Date   No.

#EX:- Given: $H = 10\ m$ $K = 4 * 10^{-4}\ cm/s$

$\gamma_{sat} = 20.8\ kN/m^3$

الكمية quantity

① Find ~~quatity~~ of seepage in $m^3/yr/m$

② Factory of safety against piping?

③ Effective ~~shere~~ stress at point A and B.
engnore self weight of dam

④ Factor of safety against uplift if the weight
dam cross section is 3000 kN/m.

Sol:-

① $\Delta H = 10\ m$

$q = K \Delta H \frac{N_F}{N_d}$ $N_F = 4$ $N_d = 14$

$q = (4 * 10^{-6})(10)(4/14) = 1.14 * 10^{-5}\ m^3/s/m$

m لازم احولها

ساعة دقائق ايام

$\Rightarrow 1.14 * 10^{-5} * 60 * 60 * 24 * 365$

$\Rightarrow 6\ m^3/yr/m$

year لكل

Subject Combaction ملاحظات على # Date No.

بعد ما عبيناه ٣ طبقات وكل طبقة ٢٥ ضربة
وهو فاضي

$$\rho_{bulk} = \frac{W_2 - W_1}{V_t \to 1000}$$

له حجم القالب من التجربة

* حساب الكثافة الكلية

* بعد ما أوزن القالب وهو مليان تراب وبعد الضرب 25 ضربة ما بلزمني
بس لوخذ عينة لحساب Water content هاي العينة بوزنها $\boxed{W_1}$
بعدين بنحط العينة بالفرن لمدة ٢٤ ساعة بعدين ← كمل الوضع في الفرن.
بنوزنها W_2 بخرج الماء هي

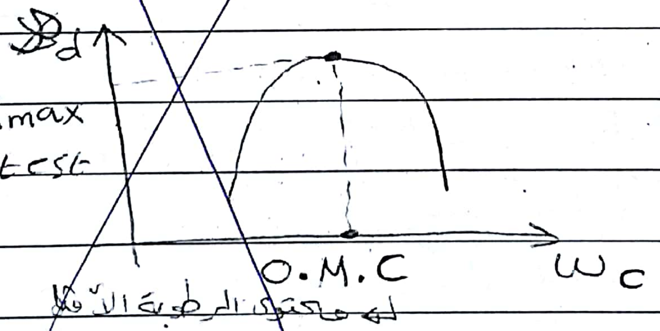
$$W_c = \frac{W_1 - W_2}{W_2} = \frac{W_w}{W_s}$$

* حساب الكثافة الجافة

$$\rho_{dry} = \frac{\rho_b}{(1 + W_c)}$$

* تكرار الخطوات مع تغير نسبة الماء

ρ_d	✓	✓	✓
W_c	✓	✓	✓



Standard proctor test (S.P.T)

في المختبر

Modified proctor Test (M.P.T)

	S.P.T	M.P.T
Wt. of ham	2.5 kg	4.5 kg
Drop height	30.5 cm	45 cm
Layers	3 طبقات	5 طبقات
No. of blows	25	25
uses	الاحمال الخفيفة والطرق العادية	الاحمال الثقيلة وطرق المطارات.

M.P.T > 4.5 S.P.T

NOTEBOOK

Subject Date No.

② $\Delta h = \dfrac{\Delta H}{N_d} = 10/14$

$i_{exit} = \dfrac{\Delta h}{L_{exit}} = \dfrac{10/14}{1.4} = 0.510$

$1.4 \rightarrow$ معطى بالرسمة

$i_{crit} = \dfrac{\gamma'}{\gamma_w} = \dfrac{\gamma_{SAT} - \gamma_w}{\gamma_w} = \dfrac{20.8 - 9.81}{9.81} = 1.12$

$F.S = \dfrac{i_{crit}}{i_{exit}} = \dfrac{1.12}{0.51} = 2.19 > 1 \rightarrow ok$

③ $\sigma'_A = \sigma_{TA} - u_A$

$\sigma_{TA} = \Sigma \gamma z = (9.81)(10) + (20.81)(7) = 243.77 \text{ kpa}$

$u_A = \gamma_w(\Delta H - N_d \Delta h - z]$

$= 9.81[10 - (3 * 10/14) - (-7)] = 145.7 \text{ kpa}$

$\sigma'_A = \sigma_T - u = 98.07 \text{ kpa.}$

④ $u_{c_1} = \gamma_w[\Delta H - N_{da} \Delta h - z_a]$

$= 9.81[10 - (5.5)(10/14) - (-1.5)] = 74.27 \text{ kpa.}$

$u_{c_2} = 9.81[10 - (12.5)(10/14) - (-1.5)] = 25.2 \text{ kpa.}$

Subject Date No.

$$P_w = \frac{(U_{c1} + U_{c2})}{2}\ \bar{x}$$

$$= \frac{(74.27 + 25.2)}{2} * 20 = 994.7 \text{ kN/m}$$

$$F.S_{uplift} = \frac{W}{P_w} = \frac{3000}{994.7} = 3.01 > 1 \text{ ok}$$

Ex:- problem من السلايد $G_s = 2.65$

$N_f = 8$ $\Delta H = 8$ m

$N_d = 18$

$$q = k \Delta H \frac{N_f}{N_d} \ (m^3/s/m)$$

$$q = (1*10^{-6})(8)\left(\frac{8}{18}\right) = 3.56*10^{-6} \ m^3/s/m$$

F.S against piping

$$F.S = \frac{i_{crit}}{i_{exit}}$$

$$i_{crit} = \frac{G_s - 1}{1 + e} = \frac{2.65 - 1}{1 + 0.8} = 0.92$$

$$i_{exit} = \frac{\Delta h}{\cancel{L_d}\ i_{exit}} = \frac{8/18}{2.42} = 0.18$$

L_o

$$\Delta h = \frac{\Delta H}{N_d} = \frac{8}{18}$$

$$i_{exit} = \left(\frac{3}{31}\right)\left(\frac{25}{L_o}\right)$$

اخذ نقطة خارجية

بعد ما نقيس المقطع بالمسطرة

العنصر كامل بالمسطرة

معطى L_o

الطول كامل

Subject　　　　Date　　　　No.

$\therefore F.S = \frac{0.42}{0.18} = 5.1 > 1 \rightarrow ok$

$\gamma_{SAT} = \left(\frac{G_s + e}{1+e}\right)(\gamma_w)$

$= \left(\frac{2.56+0.8}{1+0.8}\right) * (9.81) = 18.812\ kN/m^3$

At C_1

$\sigma_T = (9.81)(8) + (18.8)(19.35)$

$= 442.26$ kpa

$u = \gamma_w [\Delta H - N_d \Delta h - Z]$

$= 9.81 [8 - 3 * \frac{8}{18} - - 19.35] = 156.75$ kpa

$\sigma'_{C_1} = \sigma_T - u = 285.25$ kpa

At C_2 :-

$\sigma_T = [\gamma z = 18.8 * 19.35 = 363.78$ kpa

$u = \gamma_w (\Delta H - N_d \Delta h - Z)$

$= 9.81 (8 - 15 (\frac{8}{18}) - -19.35)$

$= 32.43$ kpa

$\sigma_{C_2} = 330.57$ kpa

1)

a at 5m

b at 5m

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

$N_d =$ ___

$N_f =$ ___

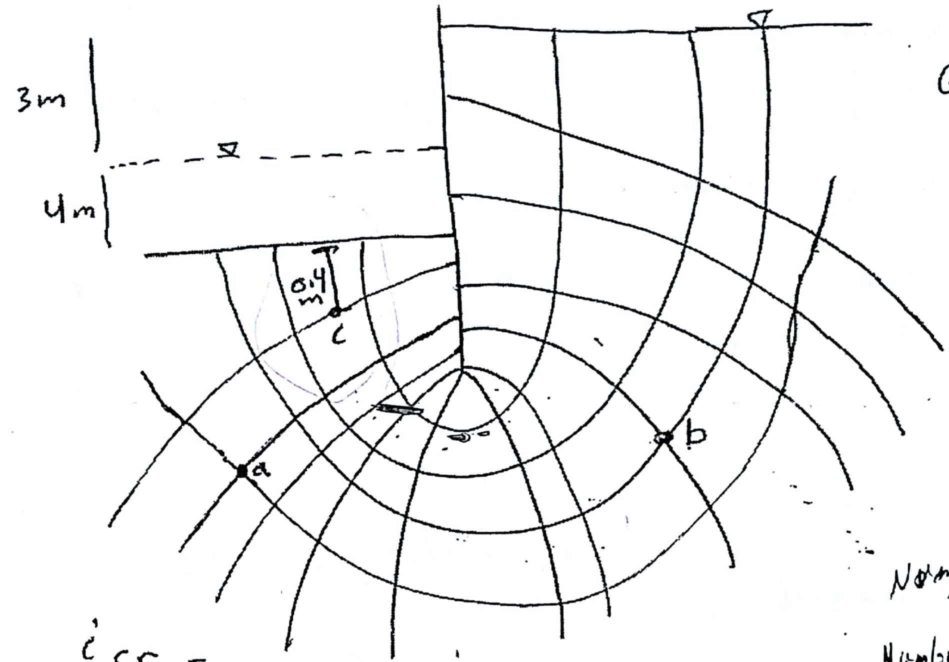
$q =$ ___

σ'_v at a & b.

$i_{cr} =$ ___ , $i_e =$ ___

$G_s = 2.7$

$w = 0.6$



Number of flow

Number of Drop

②

$k = 7 \times 10^{-4}$

$N_f = 6$

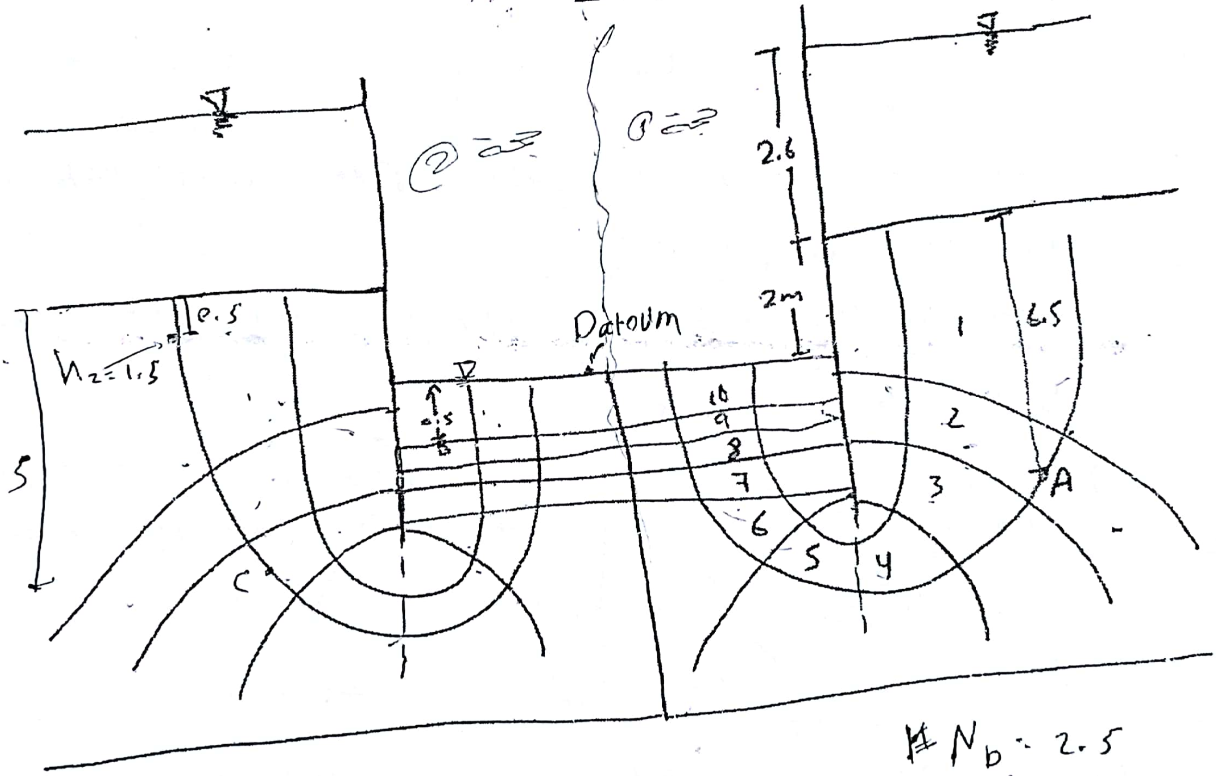
$N_d = 10$

$\gamma_s =$ ___

$q =$ ___ ؟

$\gamma = \frac{G_s - e w}{1+e}$

$= 18.3$



$N_b = 2.5$

$h_z = -?$

① find the flow rate. → ΔH

$N_f = 6$ $N_d = 10$ $\Delta h = \frac{4.6}{10}$

بس خذ جهة وحدة فقط

② find i_B , i_{cr} . $i_B = \frac{0.46}{0.5} = 0.92$

3 find σ'_v at A.

A ⇒ $h_z = -4.5$ $N_d = 1.5$

Consolidation:

- is the gradual (تدريجي) reduction in volume of fully saturated soil of ① low permeability due to ② drainage (تصريف) of some of the pore water.

- swelling: gradual increase in volume of some soil under negative excess pore water pressure.

- شروطه:

1- fully saturated.
2- fine-grained soil.
3- drainge path.
4- load.

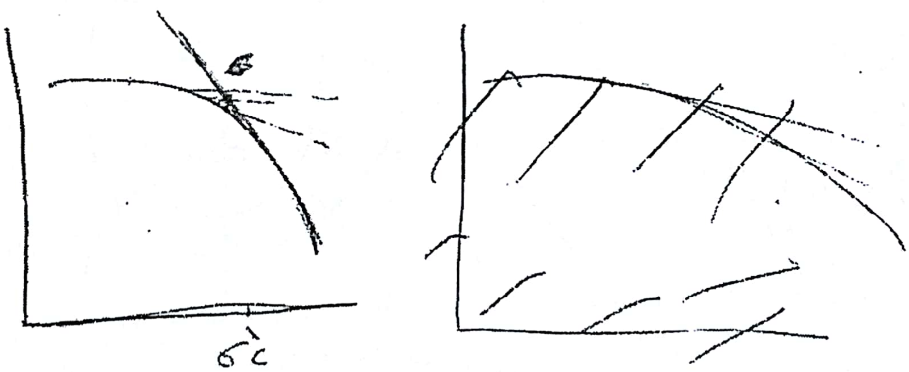
the oedometer test.

شكل السلب

Mv: the volume change per unit volume per unit increse in effective stress. Unit (m^2/MN)

- not constant but depend on stress rang over which it calculated.

ار كانت هذه تقعر بالاتجاه ولا اتجاه حارة في اصل لها اعاده ا:



$$slope = \frac{\Delta e}{\log\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

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

Primary settlement

هي أكبر e_o موجودة بالتربة

① مشكلتنا هي حساب Δe وللتغلب عليها سندرس الشكل في الصفحة التالية

Example1: Seepage Force.

Determine the quantity of seepage under the dam shown in the section in the figure below, both soil layers are isotropic the coefficients of permeability for the upper and lower layers are 2.00 X 10 $^{-6}$ and 1.60 X10 $^{-5}$ m/sec, respectively.

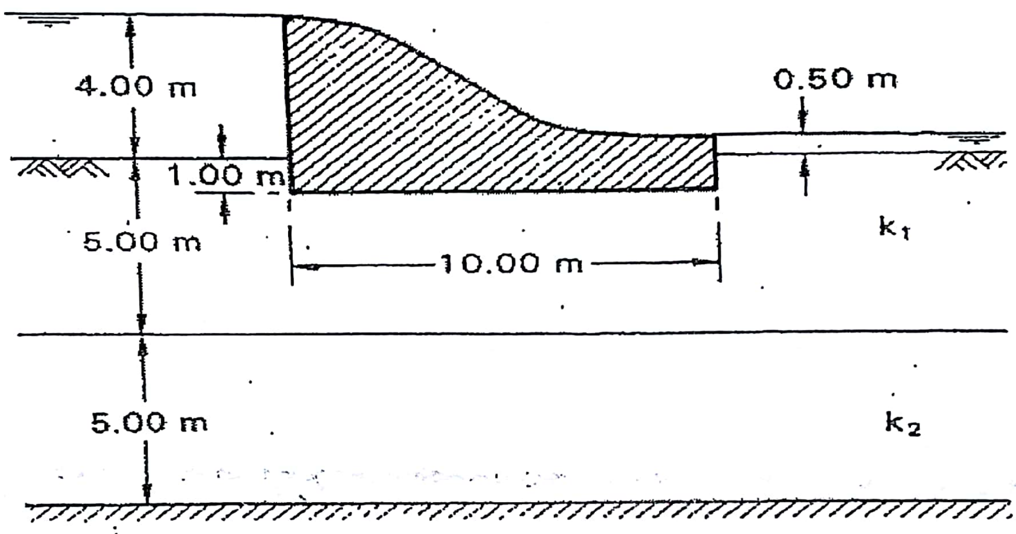


Figure 2.29

****

Example2: Consolidation and Stress increment method.

A raft foundation 60X40 m carrying a net pressure of 145Kpa is located at a depth of 4.5 m below the surface in deposit sand of dens sandy gavel 22 m deep: the water table is at a depth of 7 m. Below the sandy gravel is a layer of clay 5 m thick which in turn is underlined by dens sand, the value of m_v is 0.22 m^2/MN. Determine the settlement below the center of the raft, the corner of the raft, and the center of each edge of the raft, due to consolidation of clay.

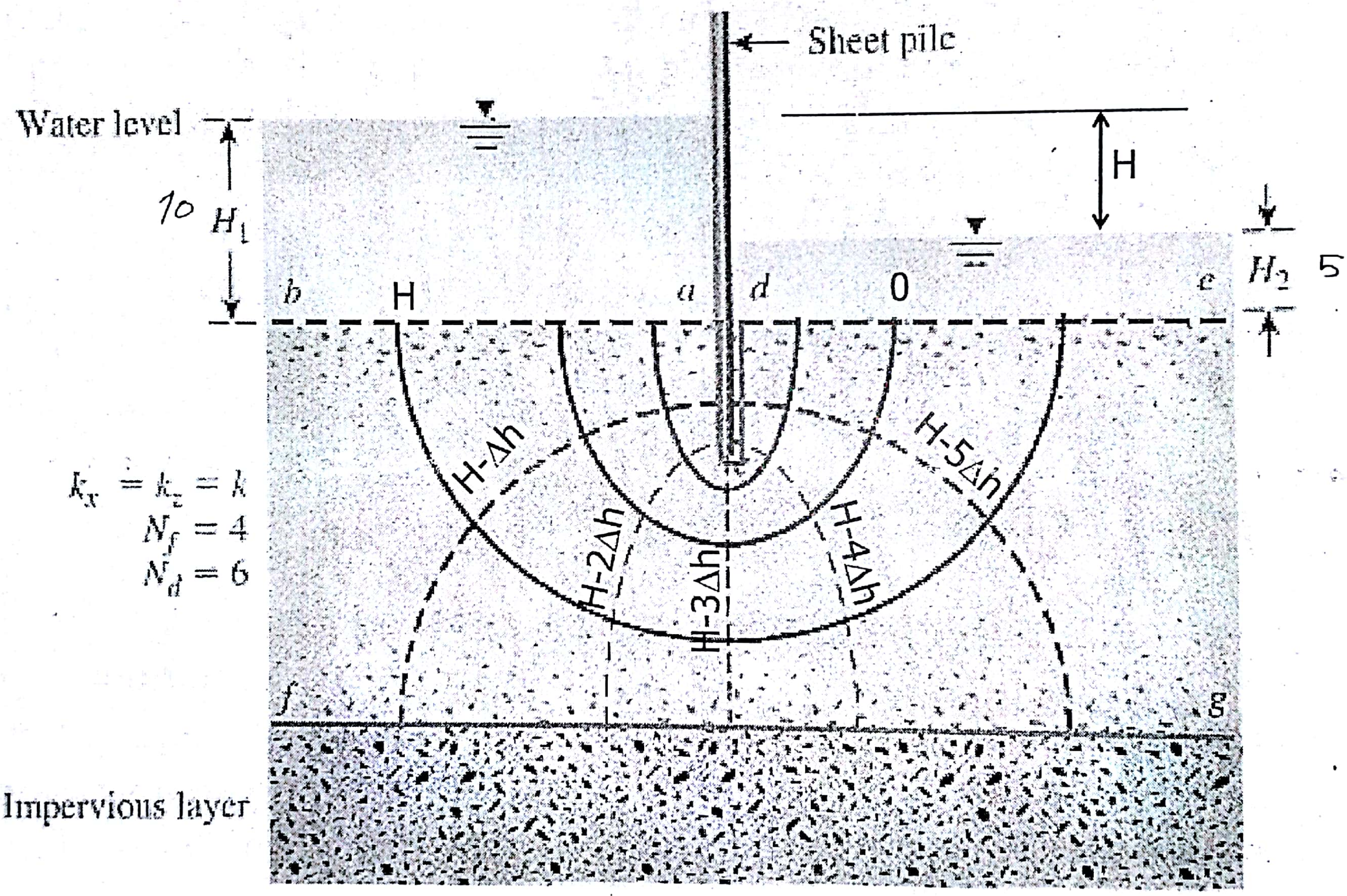
****

Example3: Consolidation theory.

In an oedometer test a specimen of saturated clay 19 mm thick reaches 50% consolidation in 20 min. How long would it take a layer of this clay 5 m thick to reach the same degree of consolidation under the same stress and drainage conditions? How long would it take the layer to reach 30% consolidation?

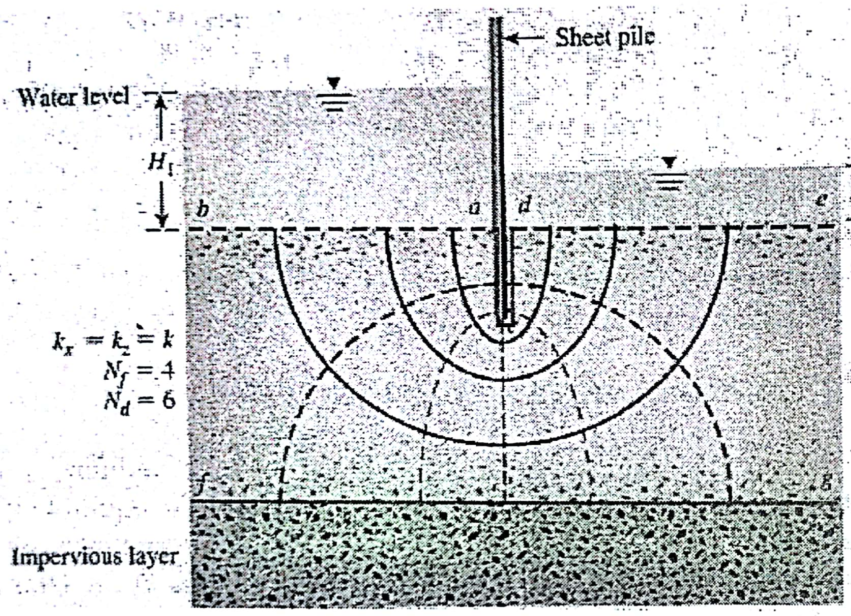
Civilittee
Made by :Saif Hassan

سنوات

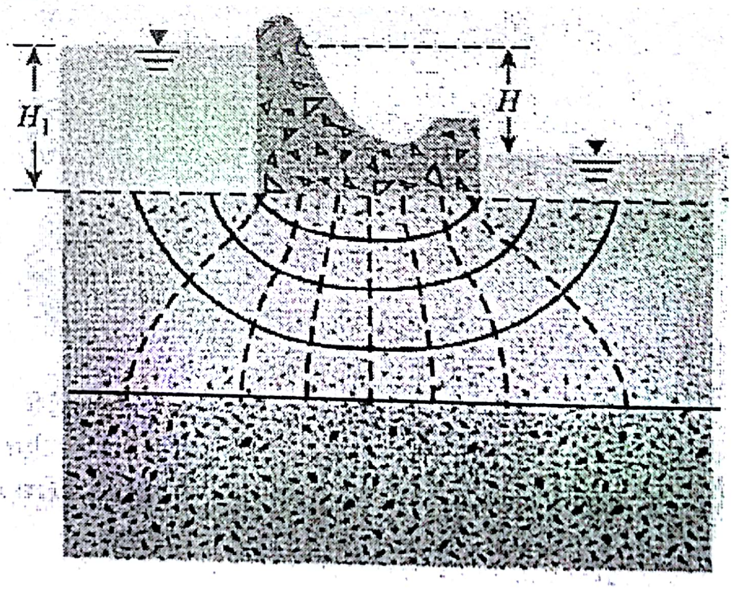


Sheet pile
Water level
10
H_1
H
H_2
5
b
H
a
d
0
e
H-Δh
H-2Δh
H-3Δh
H-4Δh
H-5Δh
$k_x = k_z = k$
$N_f = 4$
$N_d = 6$
f
g
Impervious layer

9 ??



Boundary Conditions



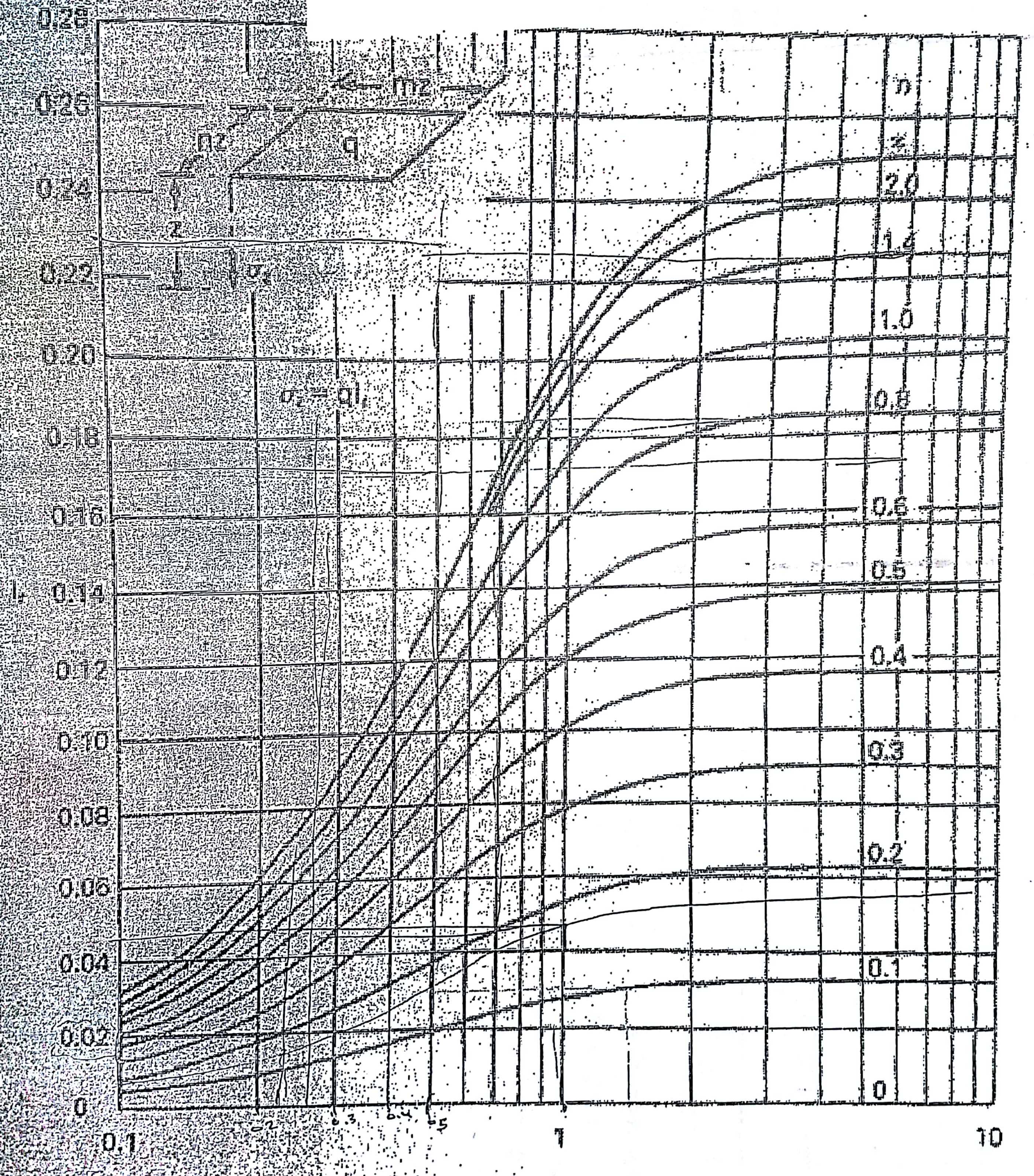


Figure 5.10 Vertical stress under a corner of a rectangular area carrying a uniform press[ure]
(Reproduced from R.E. Fadum (1948) *Proceedings of the 2nd International Confer[ence] of SMFE*, Rotterdam, Vol. 3, by permission of Professor Fadum.)

In a flow net, the strip between any two adjacent flow lines is called a *flow channel*.

The drop in the piezometric level between any two adjacent equipotential lines is the same and is called the *potential drop*.

As we will discussed later, you can draw the flow channels in different separation (to make a rectangular element instead of a square element), but you have to draw the equipotential lines at a constant drop.

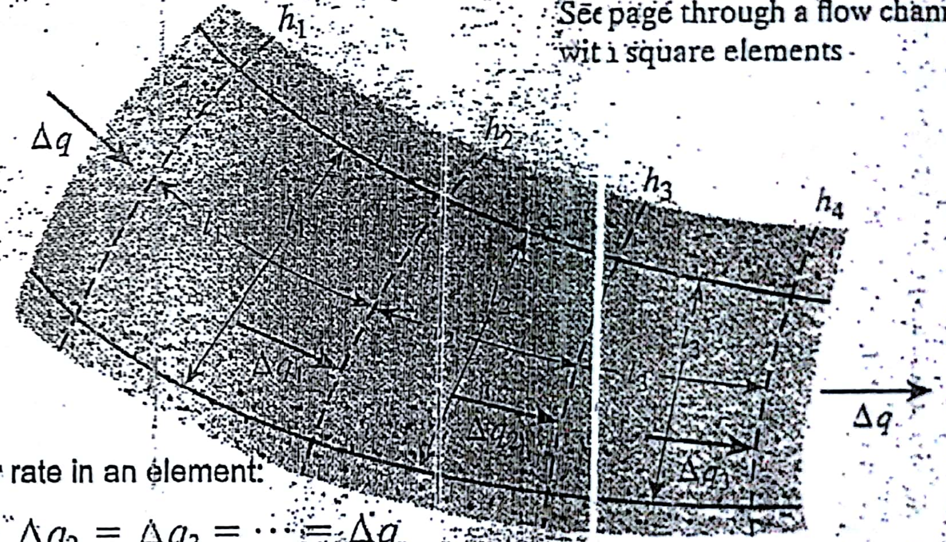


Figure 7.6
Seepage through a flow channel with square elements.

the flow rate in an element:

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

$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)l_1 = k\left(\frac{h_2 - h_3}{l_2}\right)l_2 = k\left(\frac{h_3 - h_4}{l_3}\right)l_3 = \ldots \text{ Darcy's law}$$

$\Delta h =$

$$h_1 - h_2 = h_2 - h_3 = h_3 - h_4 = \cdots = \frac{H}{N_d} \longrightarrow \Delta q = k\frac{H}{N_d}$$

$v = k\,i$

Here, I would like to call your attention for the definition of the Darcy's law. If $q=kiA$, as the originally defined, then q should be in the dimension of volume/time. But here the equation (7.18) is written in a 2D fashion with $q=kiL$, so it is in volume/time/length. It can be interpreted as the hydraulic conductivity per lateral length in the direction perpendicular to the face of the paper (or the slide screen).

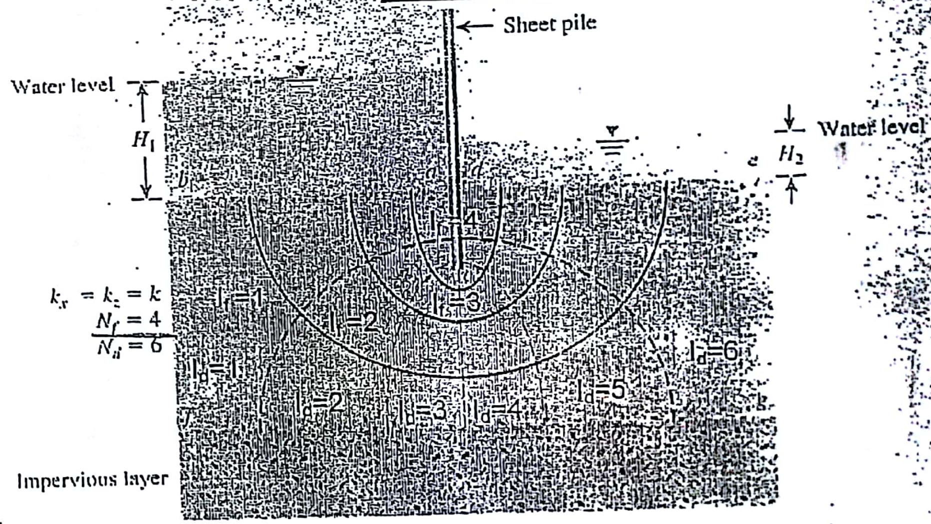
Then, what is Nd?

where H = head difference between the upstream and downstream sides;
N_d = number of potential drops.

In Figure 7.3a, for any flow channel, $H = H_1 - H_2$ and $N_d = 6$.

If the number of flow channels in a flow net is equal to N_f, the total rate of flow through all the channels per unit length can be given by

$$q = k\frac{HN_f}{N_d} \tag{7.21}$$



$q = \frac{V}{t}$

$q = Av$

$= A(ki)$

i : hydraulic gradient

k : coeff. permeability conductivity

A : cross section

$q = kiA$

$= k\frac{H}{N_D}$

N_D - no. of potential drop

N_f : no. of flow channel

— total flow

$kH \frac{N_f}{N_D} \left(\frac{b}{L}\right)$ → n … rec. → = 1 ⇒ square

flow in each channel

$k \frac{H}{N_D} \frac{b}{L}$

Although drawing square elements for a flow net is convenient, it is not always necessary. Alternatively, one can draw a rectangular mesh for a flow channel, as shown in Figure 7.7, provided that the width-to-length ratios for all the rectangular elements in the flow net are the same. In this case, Eq. (7.18) for rate of flow through the channel can be modified to

$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)b_1 = k\left(\frac{h_2 - h_3}{l_2}\right)b_2 = k\left(\frac{h_3 - h_4}{l_3}\right)b_3 = \cdots \tag{7.22}$$

If $b_1/l_1 = b_2/l_2 = b_3/l_3 = \cdots = n$ (i.e., the elements are not square), Eqs. (7.20) and (7.21) can be modified to

$$\Delta q = kH\left(\frac{n}{N_d}\right) \tag{7.23}$$

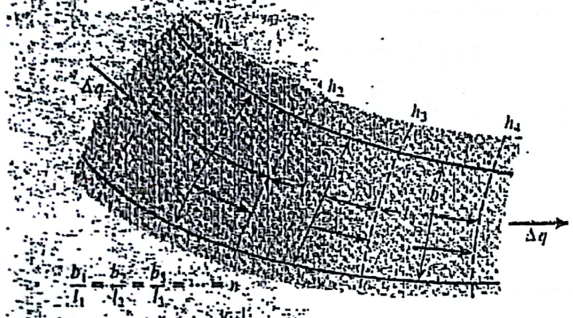
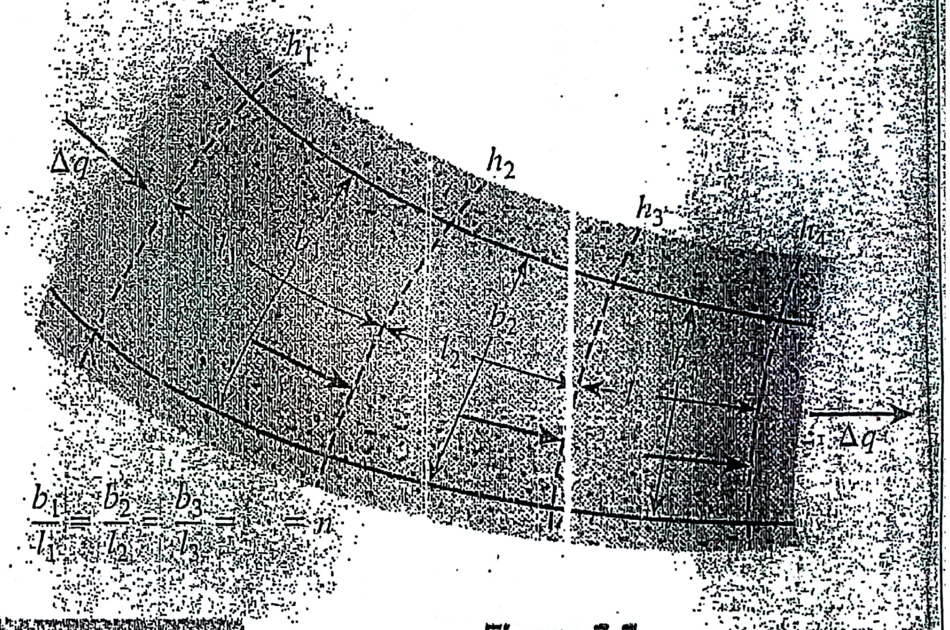


Figure 7.7



$$q = kH\left(\frac{N_f}{N_d}\right)n \tag{7.24}$$

Figure 7.7
Seepage through a flow channel with rectangular elements

$\Delta q = k i$

$\Delta q = k H \frac{N_f}{N_D}$

coef. of cond.

$(b)(k)\left(\frac{\Delta h}{l}\right)$

$\frac{b_1}{l_1} = \frac{b_2}{l_2} = \frac{b_3}{l_3} = n$

$kH\frac{n}{N_D}$

— rate of flow through channel line

$\Delta q = kH \frac{N_f}{N_D}$

- The total rate of flow / Seepage

$q = kH \frac{N_f}{N_D} n$ → $n = \frac{b}{L}$

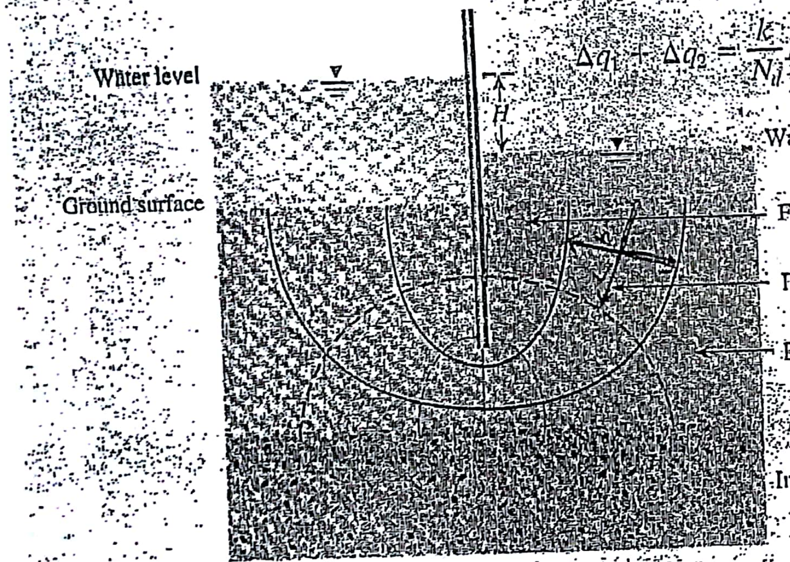
$q_1 = k \frac{H}{N_D}$

$q_2 = k \frac{H}{N_D}$

$q_3 = k \frac{H}{N_D} \frac{b}{l}$

Figure 7.8 shows a flow net for seepage around a single row of sheet piles. Note that flow channels 1 and 2 have square elements. Hence, the rate of flow through these two channels can be obtained from Eq. (7.20):

$$\Delta q_1 + \Delta q_2 = \frac{k}{N_d}H + \frac{k}{N_d}H = \frac{2kH}{N_d}$$

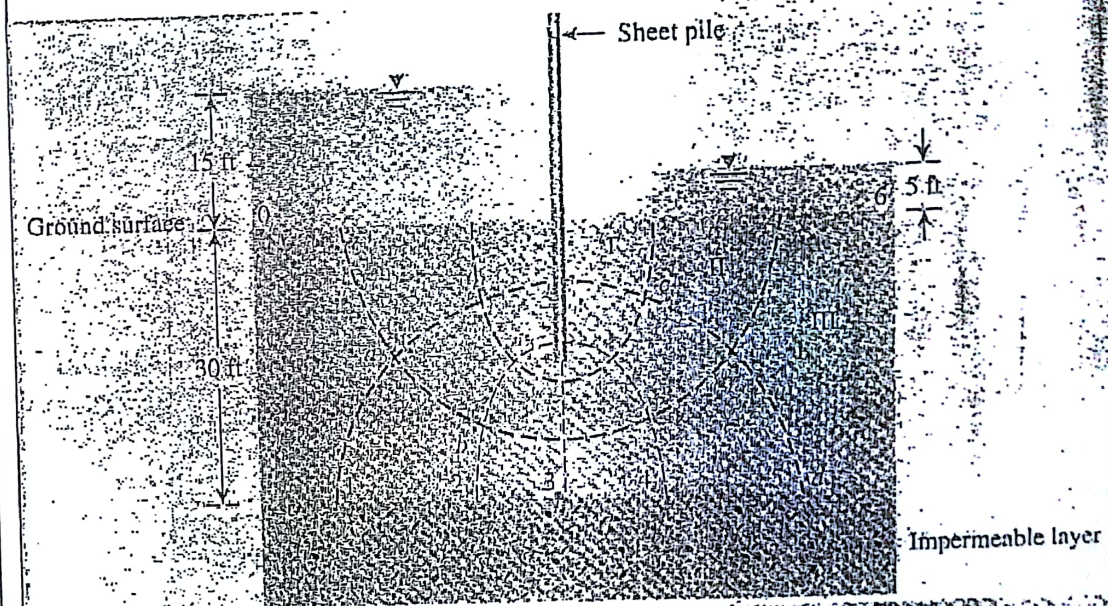


$$\Delta q_3 = \frac{k}{N_d}H(0.38)$$

$$q = 2\Delta q_1 + \Delta q_3$$

Figure 7.8 Flow net for seepage around a single row of sheet piles

Example 7.2, Figure 7.9



$H = 15 - 5 = 10$

$N_D = 6$

$N_f = 3$

$\Delta q = k \frac{H}{N_D}$ loss of head

$q = k H \frac{N_f}{N_D}$ ⇒ total rate

Example 7.2

A flow net for flow around a single row of sheet piles in a permeable soil layer is shown in Figure 7.9. Given that $k_x = k_z = k = 5 \times 10^{-3}$ cm/sec, determine

a. How high (above the ground surface) the water will rise if piezometers are placed at points *a*, *b*, *c*, and *d*.

b. The rate of seepage through flow channel II per unit length (perpendicular to the section shown).

c. The total rate of seepage through the permeable layer per unit length.

Solution

Part a

From Figure 7.9, $N_f = 3$ and $N_d = 6$. The difference of head between the upstream and downstream sides = 10 ft. So, the loss of head for each drop = 10/6 = 1.667 ft. Point *a* is located on equipotential line 1, which means that the potential drop at *a* is 1 × 1.667 ft. So the water in the piezometer at *a* will rise to an elevation of (15 − 1.667) = 13.333 ft above the ground surface.

Similarly, the piezometric level for

$b = (15 - 2 \times 1.667) = 11.67$ ft above the ground surface

$c = (15 - 5 \times 1.667) = 6.67$ ft above the ground surface

$d = (15 - 5 \times 1.667) = 6.67$ ft above the ground surface

$H = 15 - 5 = 10$

$\Delta h = \frac{H}{N_D} = \frac{10}{6} = 1.667$

potential drop $\Delta h = 1 \times 1.667$ ⇒ $h = 15 - 1.667$

Example 7.2 (cont.)

Part b

From Eq. (7.20)

$$\Delta q = k \frac{H}{N_d}$$

$k = 5 \times 10^{-3}$ cm/sec $= 5 \times 10^{-3} \times 0.0328$ ft/sec $= 1.64 \times 10^{-4}$ ft/sec

$\Delta q = (1.64 \times 10^{-4})(1.667) = 2.73 \times 10^{-4}$ ft³/sec/ft

Part c

From Eq. (7.21)

$$q = k \frac{H N_f}{N_d} = (1.64 \times 10^{-4})(1.667)3$$

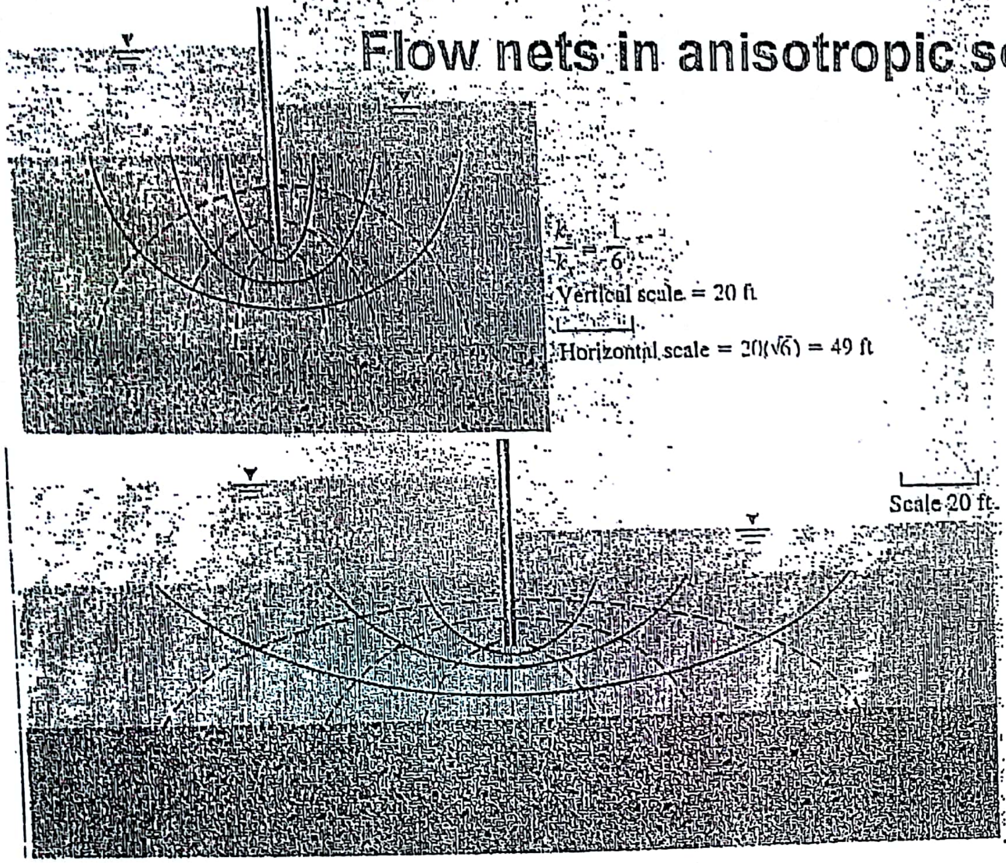
$= 8.2 \times 10^{-4}$ ft³/sec/ft

flow channel

N_f

Δq

Flow nets in anisotropic soil



Flow nets for anisotropic soil

Governing Equation

$$k_H \frac{\partial^2 h}{\partial x^2} + k_V \frac{\partial^2 h}{\partial z^2} = 0$$

+

Transformation

$$x = \alpha \bar{x}$$

$$and \quad z = \bar{z}$$

$$\frac{k_H}{\alpha^2 k_V} \frac{\partial^2 h}{\partial \bar{x}^2} + \frac{\partial^2 h}{\partial \bar{z}^2} = 0 \qquad (7.26)$$

vertical ⇒ z

horizontal ⇒ $\sqrt{\frac{k_z}{k_x}}\, z$

Flow nets for anisotropic soil

$$\frac{k_H}{\alpha^2 k_V}\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial z^2}=0$$

$$\alpha=\sqrt{\frac{k_H}{k_V}}$$

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial z^2}=0 \qquad (7.26)$$

Step 1: Adopt a vertical scale (that is, z axis) for drawing the cross section.

Step 2: Adopt a horizontal scale (that is, x axis) such that horizontal scale = $\sqrt{k_z/k_x} \times$ vertical scale.

Step 3: With scales adopted as in Steps 1 and 2, plot the vertical section through the permeable layer parallel to the direction of flow.

Step 4: Draw the flow net for the permeable layer on the section obtained from Step 3, with flow lines intersecting equipotential lines at right angles and the elements as approximate squares.

The rate of seepage per unit length can be calculated by modifying Eq. (7.21) to

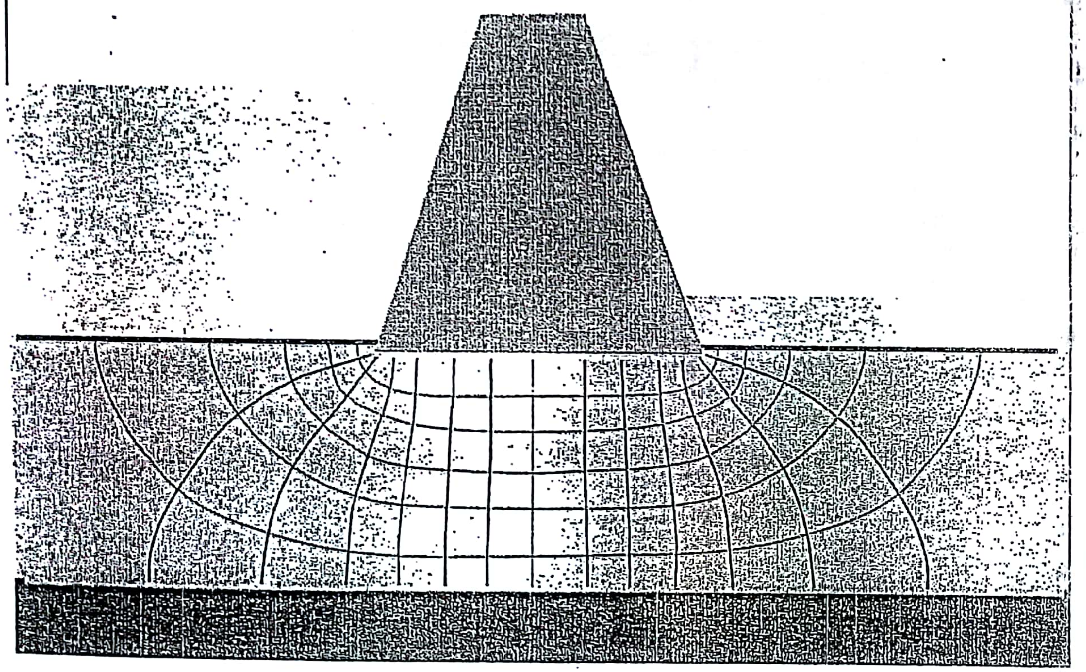
[illegible] (7.27)

anisotropic

$K = \sqrt{k_x k_z}$

Homogeneous

Example: Flow net for anisotropic soil



Flow nets for anisotropic soil

- these are determined as for isotropic soil
- The quantity of flow
  - calculated using $q = kiL$ as before
  - with $k = k_{eq} = \sqrt{k_H k_V}$

Example: Flow net for anisotropic soil

Let us assume that the soil has different horizontal and vertical permeabilities such that $k_H = 4\,k_V$

Transformation

$$\alpha = \sqrt{\frac{k_H}{k_V}} = \sqrt{\frac{4k_V}{k_V}} = 2$$

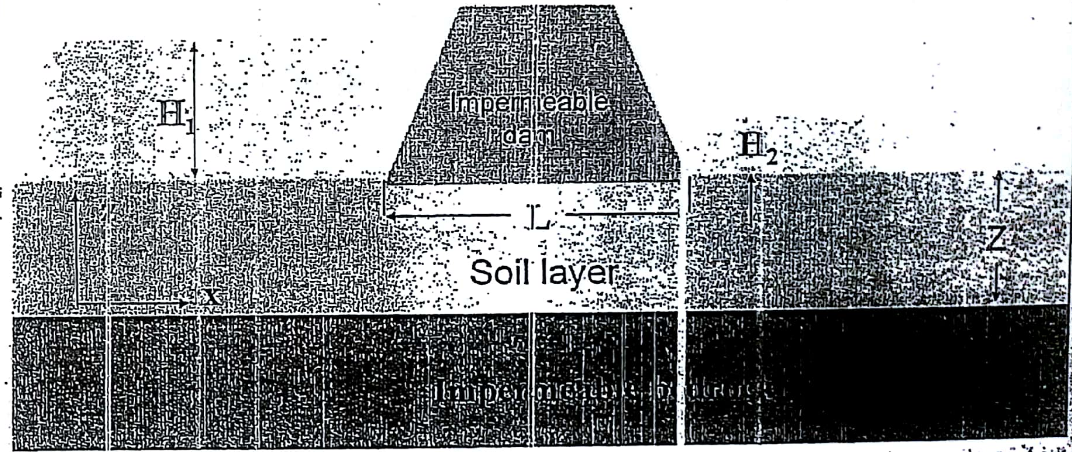
so

$$x = 2\bar{x} \; or \quad \bar{x} = 0.5x$$

$$z = \bar{z}$$

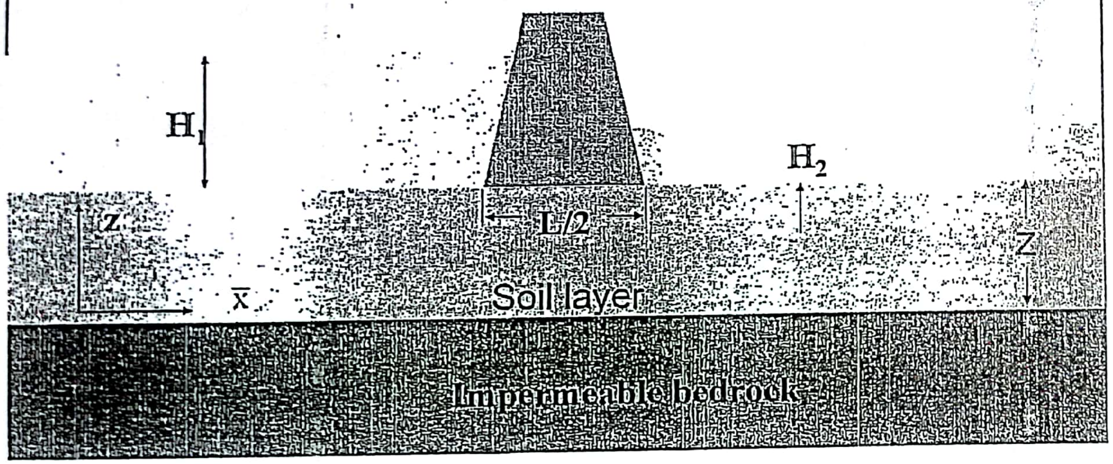
Example: Flow net for anisotropic soil

Fig. 4 Shows the dam drawn at its natural scale

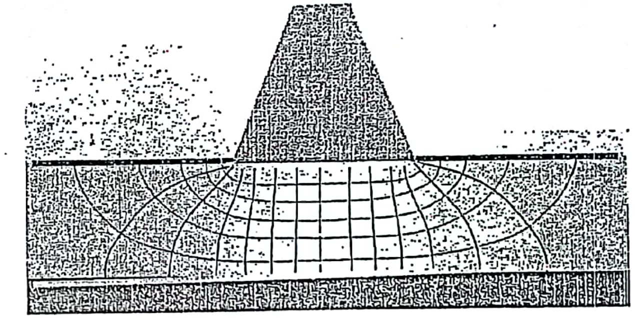


Example: Flow net for anisotropic soil

Fig. 5 Shows the dam drawn to its transformed scale



Example: Seepage under a dam



$h_1 = 13.0$ m
$h_2 = 2.5$ m
$k_V = 10^{-6}$ m/s
$k_H = 4 \times 10^{-6}$ m/s
$N_d = 14$
$N_f = 6$

$$k_{eq} = \sqrt{k_H k_V}$$

$$k_{eq} = \sqrt{(4\times10^{6})\times(10^{6})} = 2\times10^{6}\ \text{m/s}$$

$$\Delta h = \frac{(13-2.5)}{14} = 0.75\ \text{m}$$

$$\Delta q = (2\times10^{-6})\times(0.75) = 1.5\times10^{-6}\ \text{m}^3/\text{s}/\text{m}$$

thus

$$q = 6\times1.5\ \text{m}^3/\text{s}/\text{m} = 9\times10^{-6}\ \text{m}^3/\text{s}/\text{m}$$

$k_{avg} = \sqrt{(4\times10^{-6})(1\times10^{-6})} = 2\times10^{-6}$ m/s

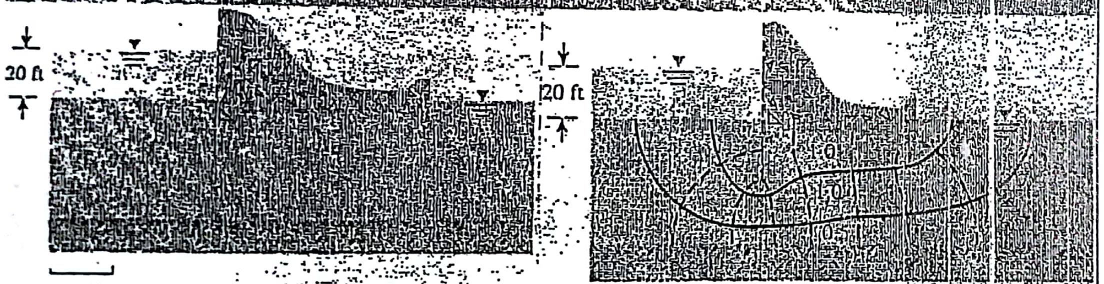
$q = k \frac{H N_f}{N_D}$ => total flow

$= (2\times10^{-6})(13-2.5)\frac{6}{14}$

For one flow channel $\Delta q = k \frac{H}{N_D}$

Example 7.3

A dam section is shown in Figure 7.11a. The hydraulic conductivity of the permeable layer in the vertical and horizontal directions are 2×10^{-2} mm/s and 4×10^{-2} mm/s, respectively. Draw a flow net and calculate the seepage loss of the dam in ft³/day/ft.



Permeable layer   Impermeable layer

Figure 7.11

Solution

From the given data,

$$k_z = 2 \times 10^{-2} \text{ mm/s} = 5.67 \text{ ft/day}$$

$$k_x = 4 \times 10^{-2} \text{ mm/s} = 11.34 \text{ ft/day}$$

and $h = 20$ ft. For drawing the flow net,

$$\text{Horizontal scale} = \sqrt{\frac{2 \times 10^{-2}}{4 \times 10^{-2}}}(\text{vertical scale})$$

$$= \frac{1}{\sqrt{2}}(\text{vertical scale})$$

On the basis of this, the dam section is replotted, and the flow net drawn as in Figure 7.11b. The rate of seepage is given by $q = \sqrt{k_x k_z}\, H(N_f/N_d)$. From Figure 7.11b, $N_d = 8$ and $N_f = 2.5$ (the lowermost flow channel has a width-to-length ratio of 0.5). So,

$$q = \sqrt{(5.67)(11.34)}\,(20)(2.5/8) = 50.12 \text{ ft}^3/\text{day/ft}$$

$k_v = 2 \times 10^{-2}$
$k_h = 4 \times 10^{-2}$ $\Big] \Rightarrow k_{avg} = \sqrt{(4\times10^{-2})(2\times10^{-2})}$

$= 2\sqrt{2} \times 10^{-2}$ mm/s

mm → ft

s → day

$q = k\,(20)\,\frac{2.5}{8}$

Seepage loss ⇒ q below

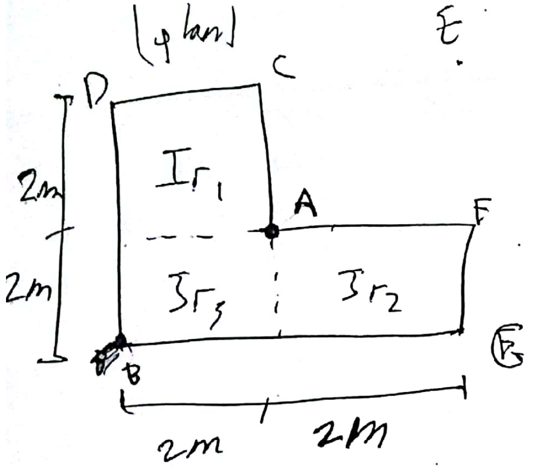
(8)

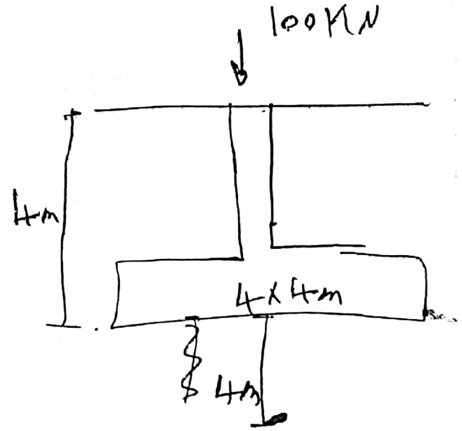
Stress Increment From Elastic Solution

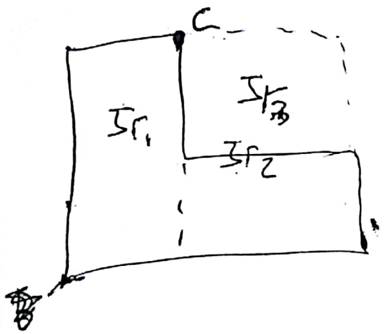
$\sigma_{water} + \sigma_{soil}$

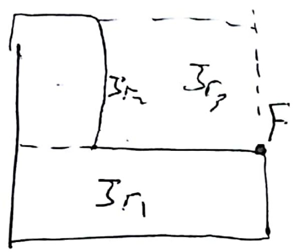
Stresses and displacements
Craig Chapter 5 section 5.2

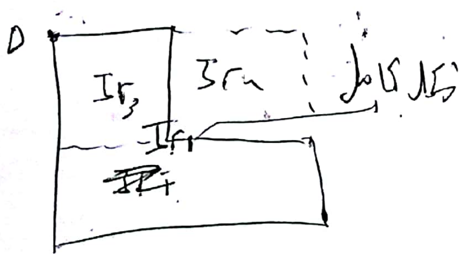
Ex: Rect.

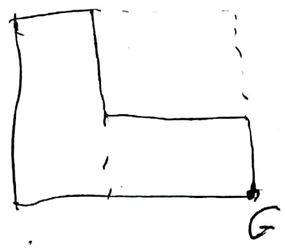


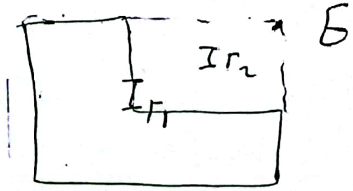












$$\sigma_{zA} = \frac{100}{[2\cdot2+2\cdot4]} * [I_{r_1} + I_{r_2} + I_{r_3}]$$

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

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

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

$I_{r_1} = \longrightarrow m = \frac{4}{4}$, $n = \frac{4}{4}$ $\qquad$ $I_{r_3} \rightarrow m = \frac{2}{4}$, $n = \frac{2}{4}$

$I_{r_2} \rightarrow m = \frac{4}{4}$, $n = \frac{2}{4}$

$$\sigma_{zE} = q[I_{r_1} - I_{r_2}]$$

$I_{r_1} \rightarrow m = \frac{4}{4}$, $n = \frac{4}{4}$ $\qquad$ $I_{r_2} \rightarrow m = \frac{2}{4}$, $n = \frac{2}{4}$

$$\sigma_{zF} = q[I_{r_1} + I_{r_2} - I_{r_3}] ??$$

$$\sigma_G = q[I_{r_1} - I_{r_2} + I_{r_3}]$$

$I_{r_1} \rightarrow m = \frac{4}{4}$, $n = \frac{4}{4}$

$I_{r_2} \rightarrow m = \frac{2}{4}$, $n = \frac{2}{4}$

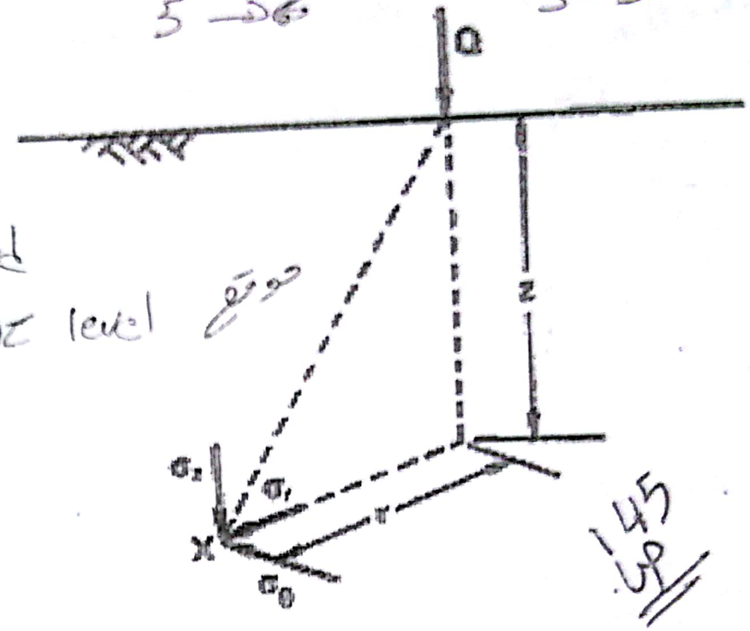
$I_{r_3} \rightarrow m = \frac{2}{4}$, $n = \frac{2}{4}$

$\frac{3-1}{3-5} = \frac{4-2}{4-6}$

Point load

Q = load

z = point level



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

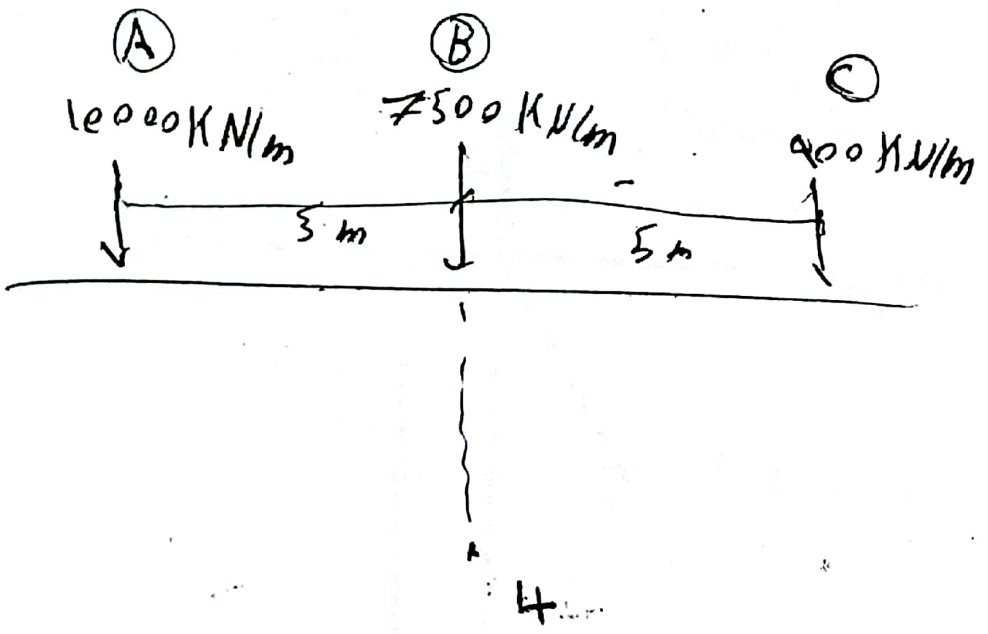
$$I_p = \frac{3}{2\pi}\left\{\frac{1}{1+(r/z)^2}\right\}^{5/2}$$

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

Table 5.1 Influence factors for vertical stress due to point load

r/z	I_p	r/z	I_p	r/z	I_p
0.00	0.478	0.80	0.139	1.60	0.020
0.10	0.466	0.90	0.108	1.70	0.016
0.20	0.433	1.00	0.084	1.80	0.013
0.30	0.385	1.10	0.066	1.90	0.011
0.40	0.329	1.20	0.051	2.00	0.009
0.50	0.273	1.30	0.040	2.20	0.006
0.60	0.221	1.40	0.032	2.40	0.004
0.70	0.176	1.50	0.025	2.60	0.003

Ex:-



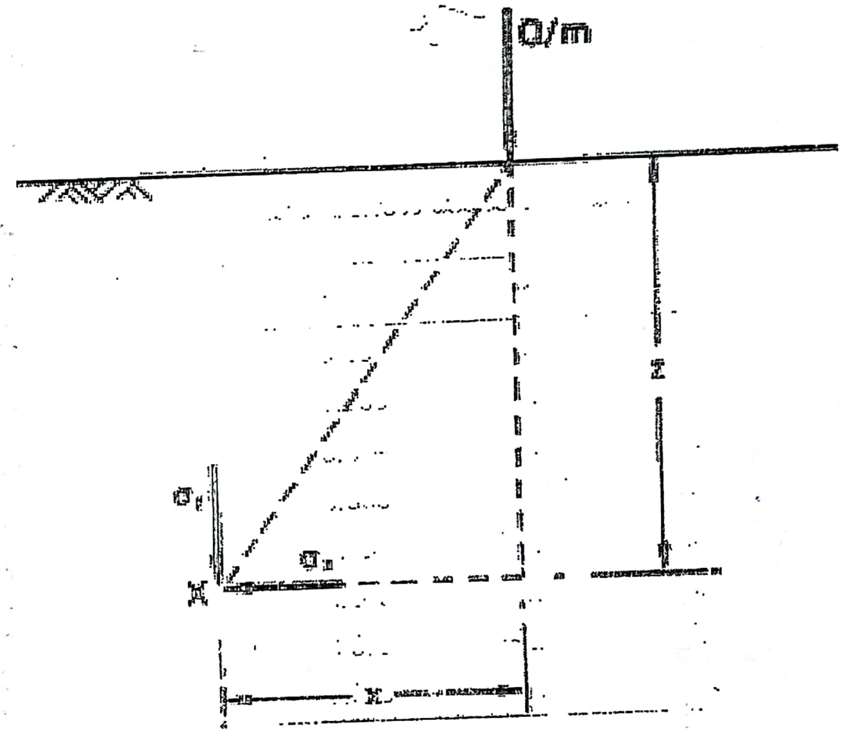
$$\sigma_z = \sigma_{z_A} + \sigma_{z_B} + \sigma_{z_C}$$

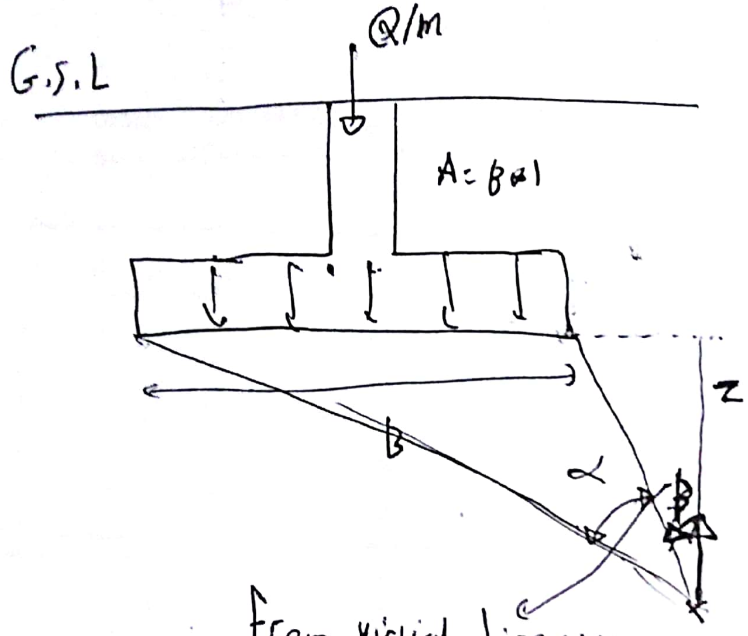
$$= \frac{2\times 10000}{\pi}\left[\frac{4^3}{(3^2+4^2)^2}\right] + \frac{2\times 7500}{\pi}\left[\frac{4^3}{(0^2+4^2)^2}\right] + \frac{2\times 900}{\pi}\left[\frac{4^3}{(5^2+4^2)^2}\right]$$

Line load (Q/m)

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

$x = r$

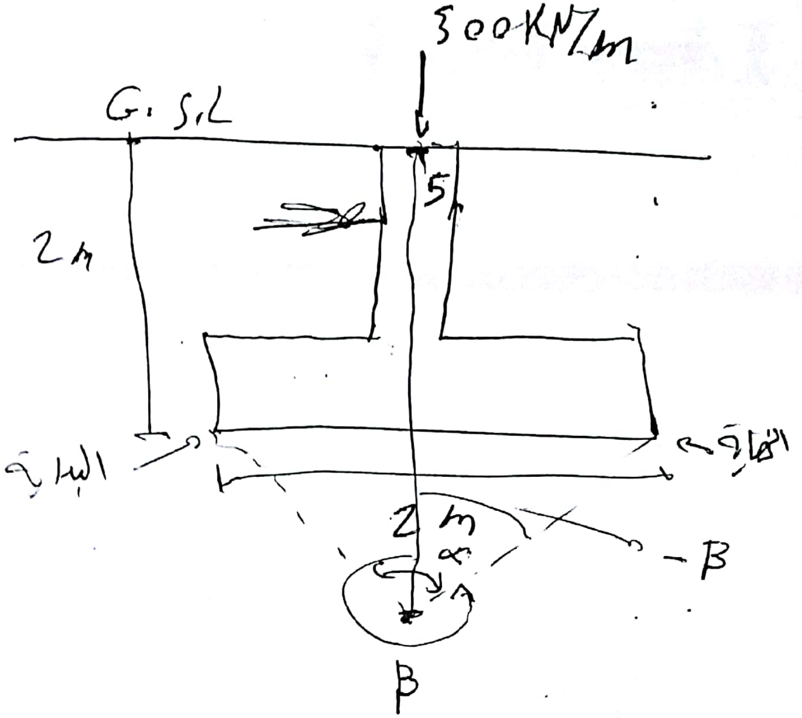




from virtical direction
toward end arraw
Counter clokwise

$\pi \rightarrow 180^\circ$
$?? \rightarrow$

Ex:



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

$\beta = \tan^{-1}\left(\frac{1}{3}\right) = 18^\circ \Rightarrow -18^\circ$

$\alpha = |2\beta| = 2 * 18^\circ = 36^\circ$

$\alpha_{rad} = 0.64$

$\sigma_z = \frac{\frac{500}{2*1}}{\pi}\left[0.64 + \sin 36^\circ \cos(36^\circ - 36^\circ)\right]$

$= 49 \text{ KN/m}^2$

Rectangular area carrying uniform pressure

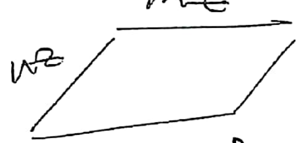
$q = \frac{Q}{B \times L}$, $I_r \rightarrow$ fig 5.10 , $f(m, n)$

شروط الاستخدام

① point should be corner

② Rect. shape زاوية

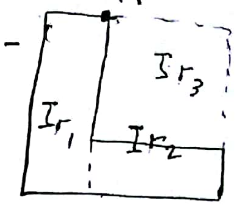
$\sigma_z = q I_r$



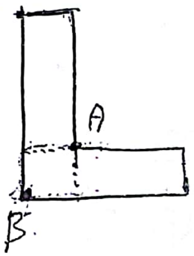
$m = \frac{B}{z}$

$n = \frac{L}{z}$

$\neq$ corner

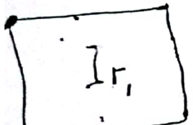


$\sigma_z = q\,[I_{r_1} + I_{r_2} - I_{r_3}]$

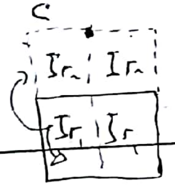


See example 5.2

$\sigma_x' = K_0\, \sigma_z'$



$\sigma_z = q\, I_{r_1}$



$\sigma_z = q\,[2 I_{r_1} - 2 I_{r_2}]$ X

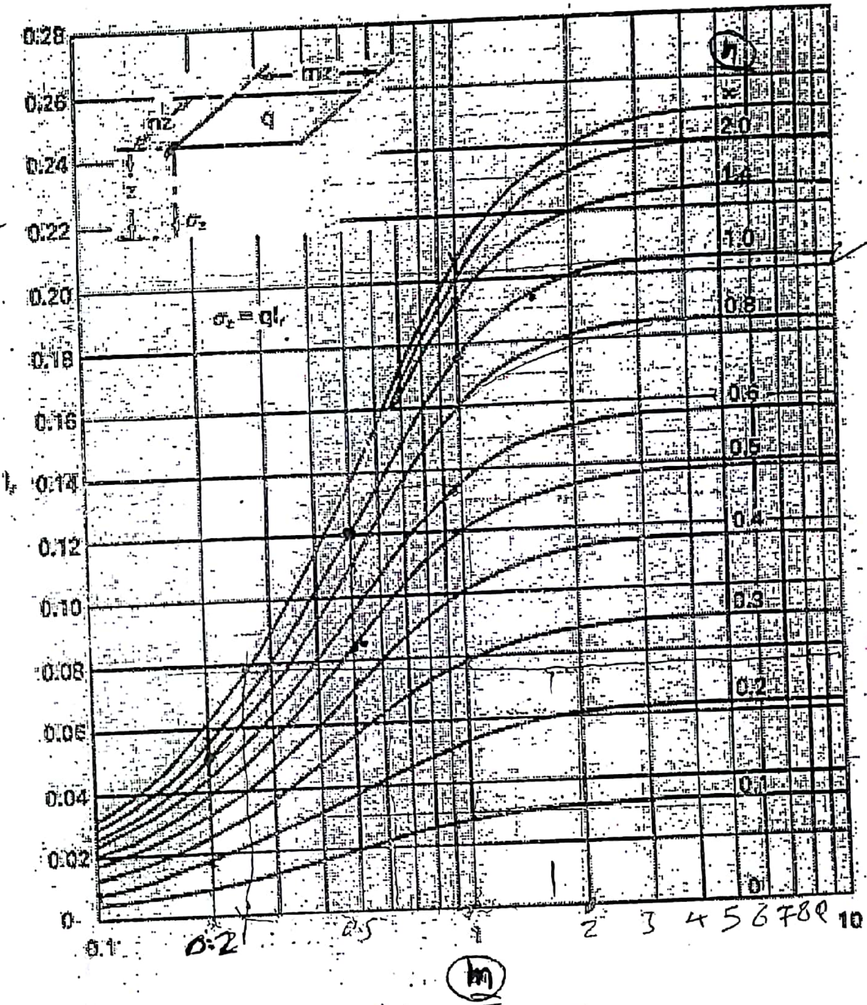


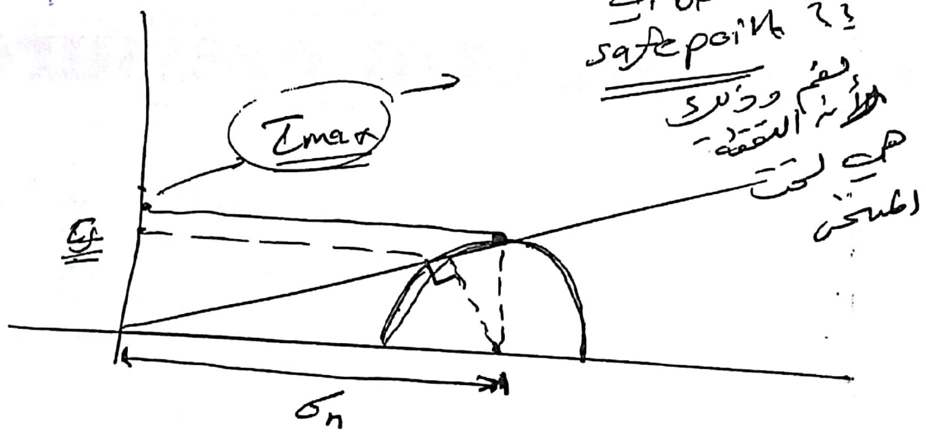
Figure 5.10 Vertical stress under a corner of a rectangular area carrying a uniform pressure. (Reproduced from

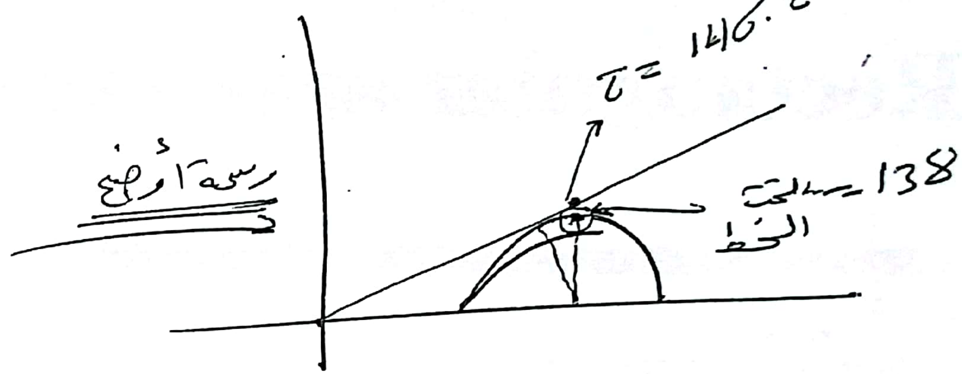
Rectangle

على الزاوية Corner

شروط استخدام

(*) لیس τ_{max} بالشرط انه تلمس τ_f





$$\tau_f = 0 + \sigma_n \tan\phi' = 146.2 \text{ KN/m}^2$$

$$\sigma_n \text{ at } \tau_{max} = 414 \text{ KN/m}^2$$

*أي قيمة تحت الخط هي آمنة.

$$\tau_{max} = \frac{552 - 276}{2} = 138 \text{ KN/m}^2$$

$$\sigma_n \text{ at } \tau_{max} = \frac{552 + 276}{2} = 414 \text{ KN/m}^2$$

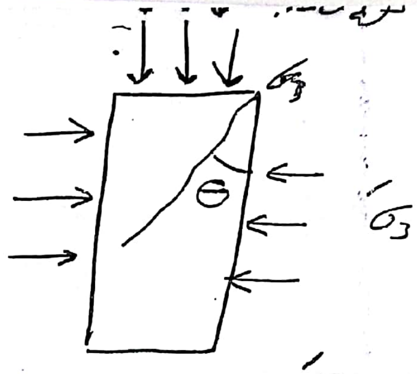
Example 7.1

* CD test
* $\Delta u = 0.0$ because CD
* $\sigma_3 = \sigma'_3$

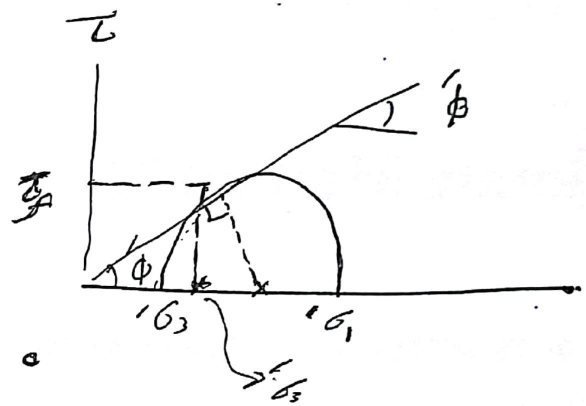
(a) angle of ... (ϕ')

(b) θ

(c) σ' & τ at failure

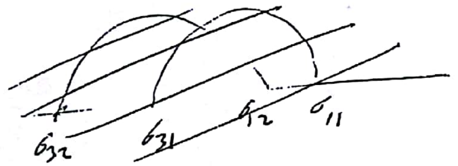


N.C $\Rightarrow c' = 0.0$



$\sigma'_1 = \sigma'_3 + (\Delta\sigma_d)_f = 276 + 276 = 552 \; kN/m^2$

$\sin\phi = \dfrac{552 - 276}{552 + 276} = 0.333 \rightarrow \phi' = 19.45^\circ$



(2) $\theta = 45^\circ + \dfrac{\phi'}{2} = 54.73^\circ$

$\sigma'_{nf} = \dfrac{\sigma'_1 + \sigma'_3}{2} + \dfrac{\sigma'_1 - \sigma'_3}{2} \cos 2\theta = \dfrac{552 + 276}{2} + \dfrac{552 - 276}{2} \cos \text{---}$

$= 368.03$

(4) $\tau_f = \overset{0.0}{\cancel{c}} + \sigma'_{nf} \tan\phi' = \dfrac{\sigma'_1 - \sigma'_3}{2} \sin 2\theta = \dfrac{552 - 276}{2} \sin 2 \times 54.73 = 130.12 \; kN/m^2$

(*) friction angle is constant

نستفيد من ذلك في ايجاد اي قيمة
للاي ستريس عند تغيير القيم

$\sigma_1 = \underbrace{\sigma'_3}_{K_0 \sigma'_{vo}} + \underbrace{\Delta\sigma_d}_{\Delta\sigma_z}$

$\tan^2(45 + \frac{\phi'}{2})$

Approximate Method 2:1 method

under the center

+Just for:
① strip, Rectangular footing
② under the center of foot

~~Just for~~:

① **Strip Load Width B** Q

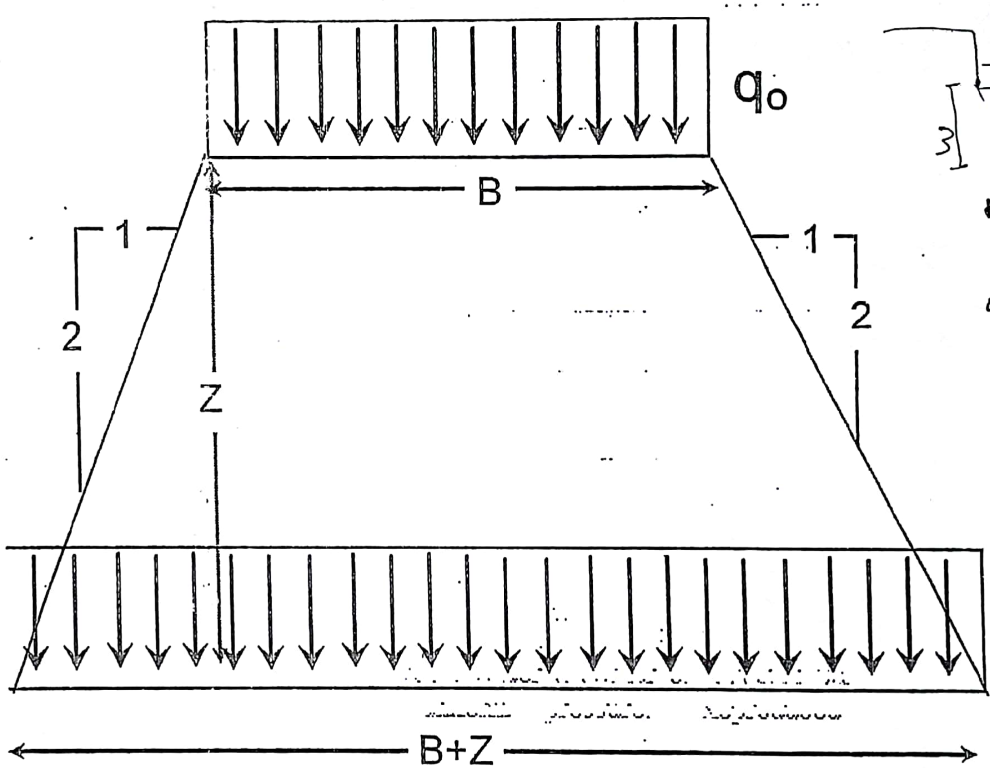
$$\Delta\sigma'_v = \frac{q_o(Bx1)}{(B+Z)}$$

Square Load Width B Q

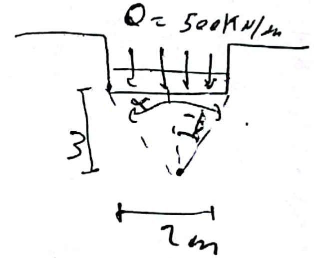
$$\Delta\sigma'_v = \frac{q_o(BxB)}{(B+Z)(B+Z)}$$

Rectangular Load B X L Q

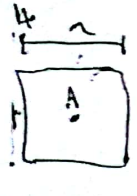
$$\Delta\sigma'_v = \frac{q_o(BxL)}{(B+Z)(L+Z)}$$



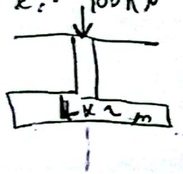
Ex:



$\sigma_z = \frac{500}{(2+3)} = 100$

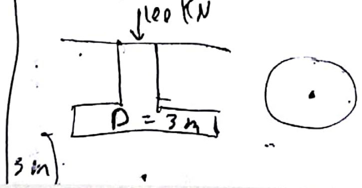


Ex: 100 kN



$\sigma_z = \frac{100}{[2+4][4+4]} = \frac{100}{6 \times 8}$

Ex: 100 kN



$\sigma_z = q\, I_c$, $q = \frac{100}{\frac{\pi}{4}\cdot(3)^2}$

Stress Increment

Q : Load (KN)

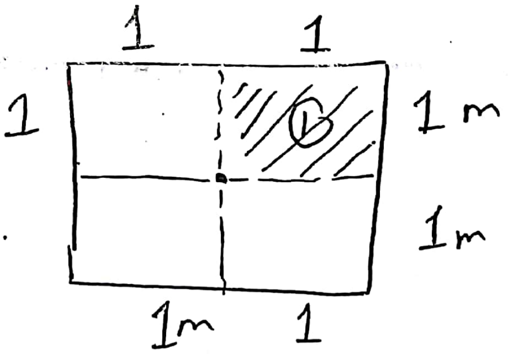
q : stress (KN/m^2) $\Rightarrow$ or KN/m

$\hookrightarrow$ take care

Ex1: A load of 1500 KN carried on a foundation 2m square at a shallow depth in a soil mass. Determine the vertical stress at a point 5m below the center of the foundation

(a) assuming that the load is uniform pressure

solu



(a) $mz = nz = 1$

$m = n = \frac{1}{z} = \frac{1}{5} = 0.20$

From Figure 5.10

$I_r = 0.018 \Rightarrow \sigma_{z_1} = q\, I_{r_1}$

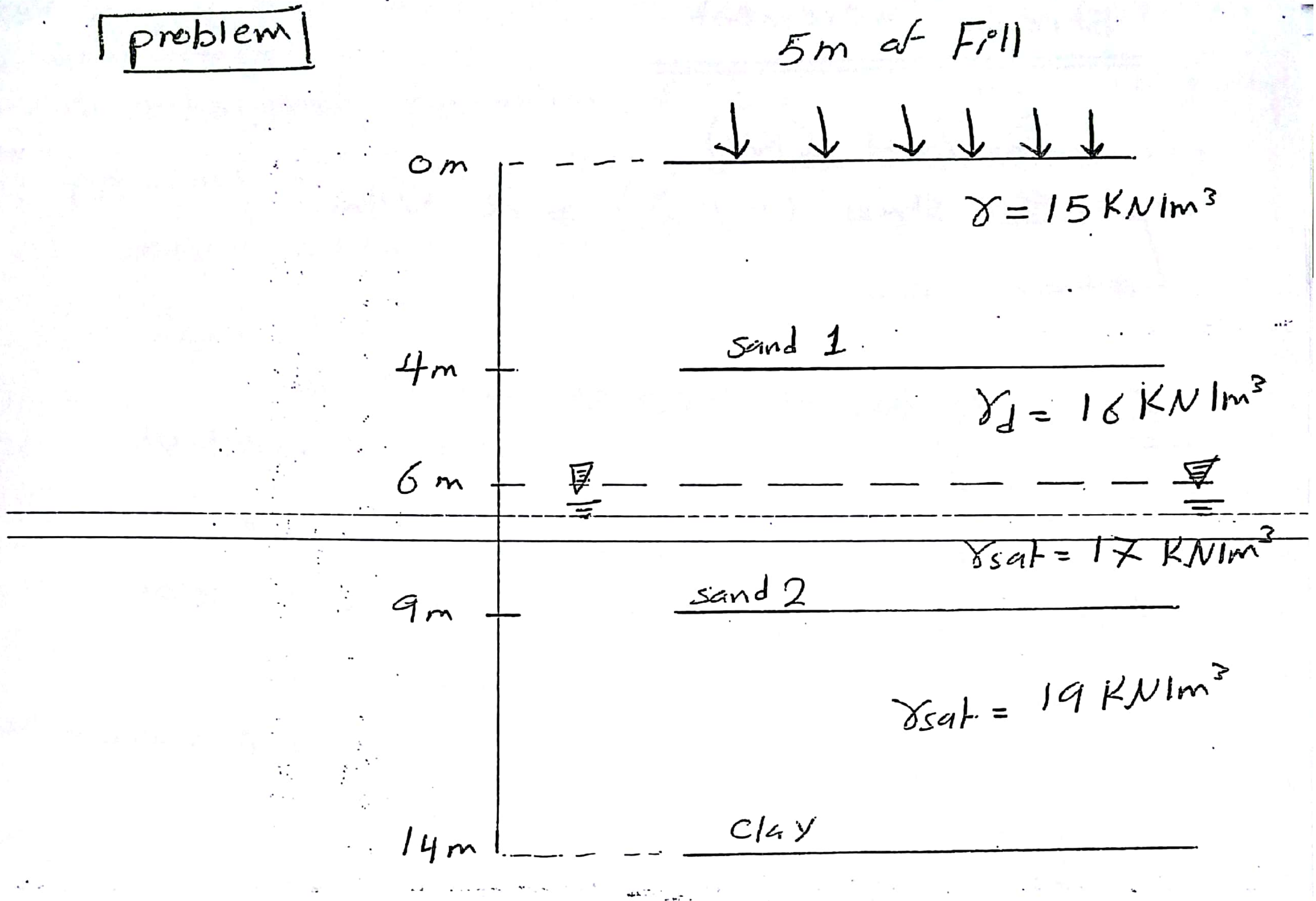
But $q = \frac{Q}{A} = \frac{1500}{2^2} * 0.018 =$

$\therefore$ $\sigma_{total} = 4\,(6.75) = 27\ KN/m^2$

(b) assuming that the load act as a point load

$\sigma_z = \frac{Q}{z^2} I_p \Rightarrow \frac{r}{z} = \frac{0}{5} = 0.0 \Rightarrow I_p = 0.478$

$\boxed{6}$ $\therefore \sigma_z = \frac{1500}{5^2} * 0.478 = 29$ K problem



[1] cal. effective stress at mid. of sand 2 immediatly after the fill has been placed

[2] cal. overburden pressure at mid of clay layer many years after the fill has been placed

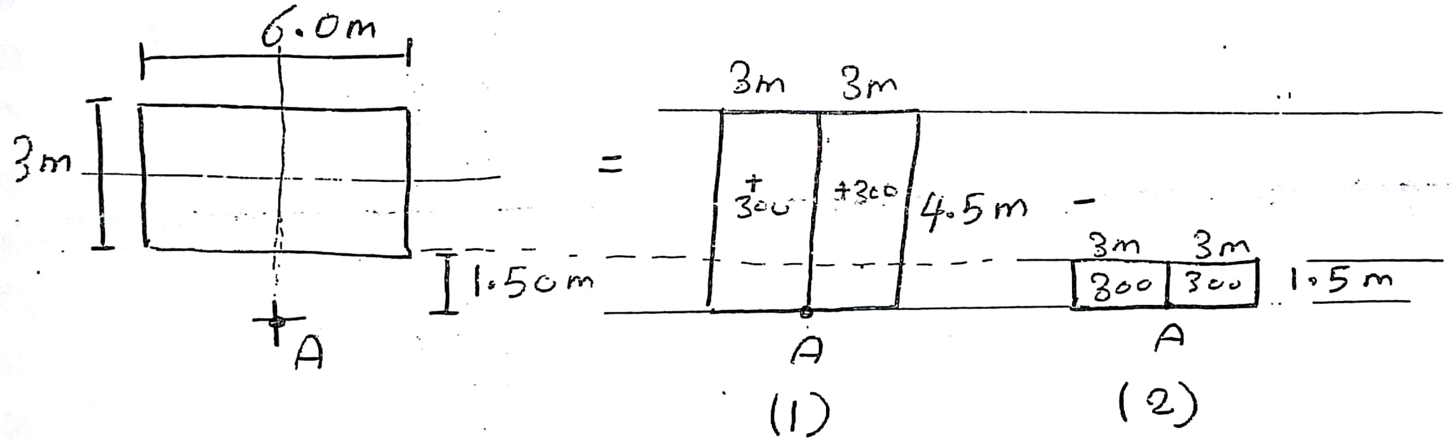
[3] cal. pore water pressure at depth = 10 m

[4] cal. total stress at depth = 10 m many years after the fill has been placed

Example 2,

A rectangular foundation 6 × 3 m carries a uniform pressure of 300 KN/m^2 near the surface of soil mass. Determine the vertical stress at a depth of 3 m below a point (A) on the center line 1.5 m outside a long edge ~~of foundation~~.

solution



For (1) ; $nz = 3n = 4.5 \Rightarrow n = 1.50$

$mz = 3m = 3 \Rightarrow m = 1.00$

From Fig 5.10 $\Rightarrow I_r = 0.193$

$\sigma_z = 2(300 \times 0.193) = 115.8$ Kpa

For (2)

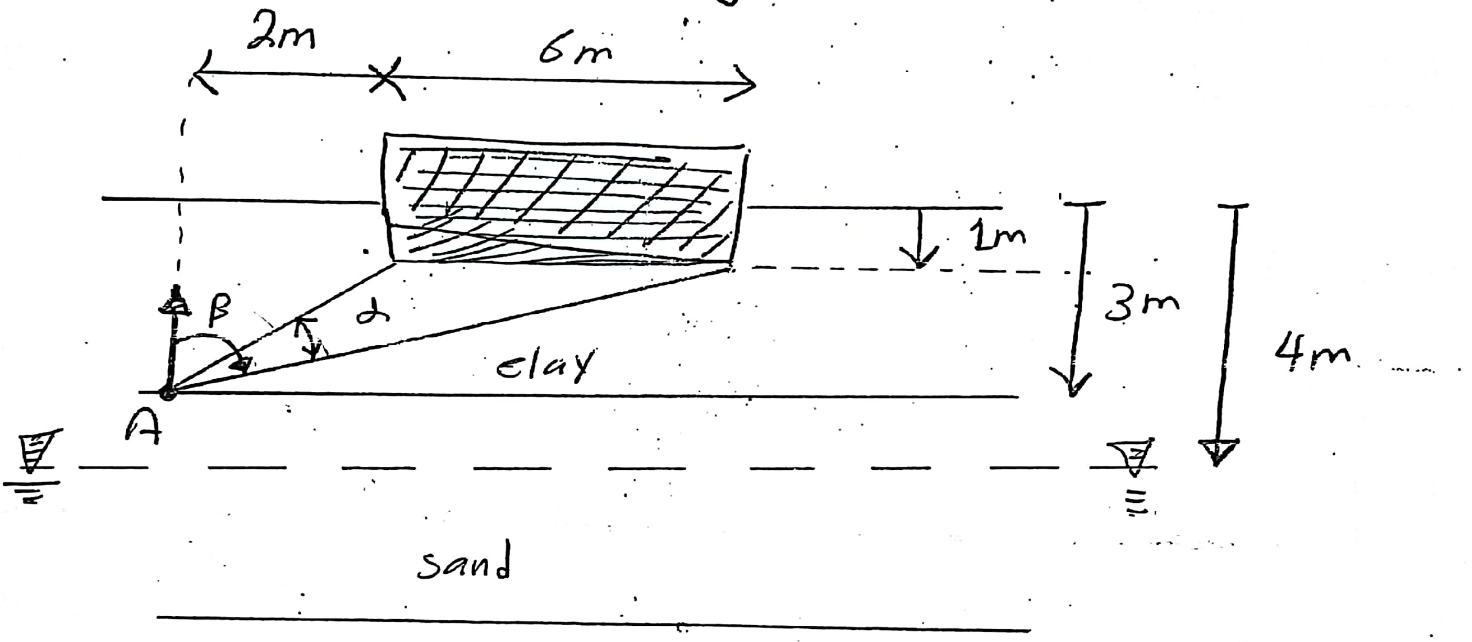
$3n = 1.5 \Rightarrow n = 0.5$
$3m = 3 \Rightarrow m = 1.00$ $\Rightarrow I_r = 0.120$

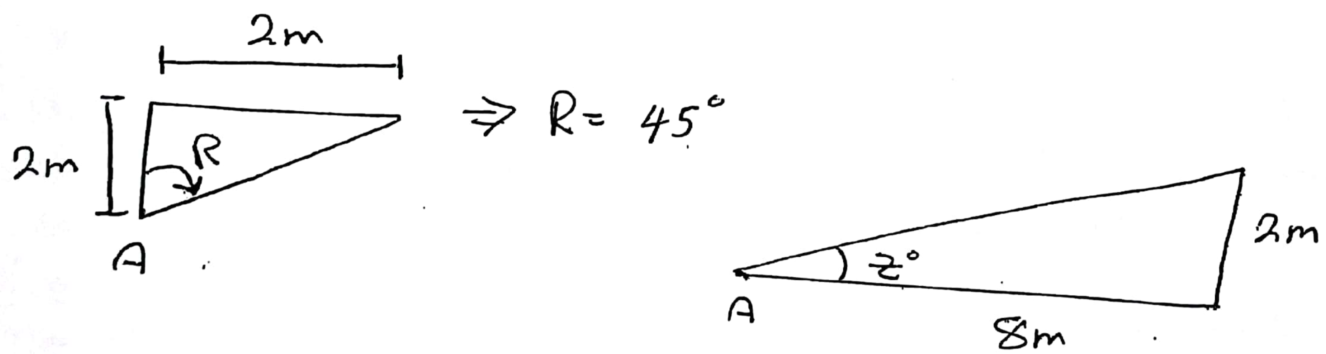
$\sigma_z = 2(300 \times 0.120) = 72$ KPa

$\sigma_{net} = 115.8 - 72 = 43.8 \approx 44$ Kpa

Example 3

Find the vertical stress at point A due to stress $q = 1200$ KN/m^2 on the given strip footing





$\Rightarrow R = 45°$

$\tan^{-1}\left(\frac{2}{8}\right) = z° = 14°$

$\therefore \alpha° = 90° - (14 + 45)$
$= 31° = 0.54$ Rad

$\beta = R + \alpha = -(45 + 31) = -76°$ (clock wise)

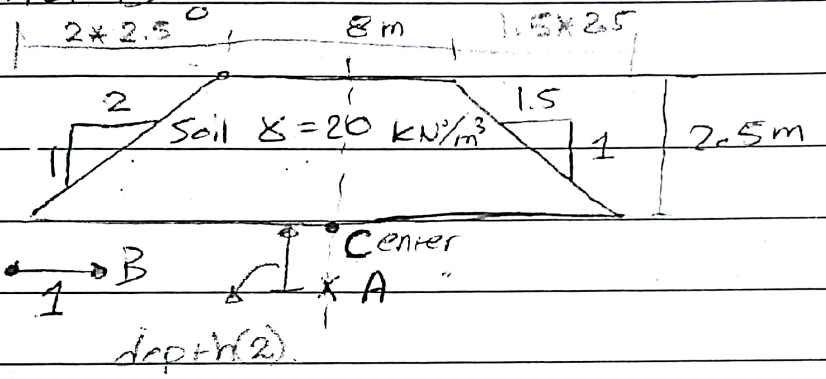
$\sigma_z = \frac{1200}{\pi}\left(0.54 + \sin 31 \cos(31 - 2(76))\right)$

Point Load :- مثالو الـ Colume

Line Load :- مثالو انابيب النفط والغاز .

Embankment Load :-

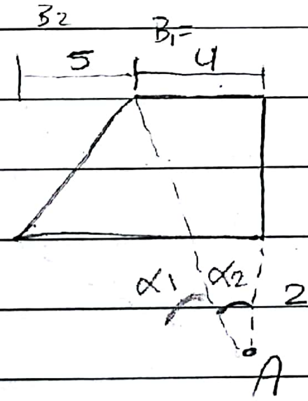
Ex :- Find $\Delta\sigma$ at depth $Z = 2m$ at point A and B?



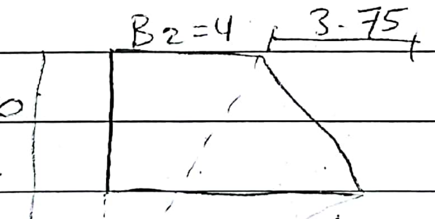
صنولي
مـ

$q_o = \gamma H = (20 * 2.5) = 50\ kPa$

At point A $\Delta\sigma = \Delta\sigma_L + \Delta\sigma_R$ ــاليمين ــاليسار



$q_o = 50$



$\alpha_2 = \tan^{-1}\frac{4}{2} = 1.1$

$\alpha_2 = \tan^{-1}\frac{B_1}{Z} = \tan^{-1}\left(\frac{4}{2}\right) = 1.1$ &

$\alpha_1 = \tan^{-1}\left(\frac{B_1+B_2}{Z}\right) - \tan^{-1}\frac{B_1}{Z}$

$= \tan^{-1}\left(\frac{4+5}{2}\right) - 1.1 = 0.25$

$\Delta\sigma_L = \frac{50}{\pi}\left[\left(\frac{4+5}{5}\right)(1.1+0.25) - \frac{4}{5}(1.1)\right]$

$= 24.68\ kPa$

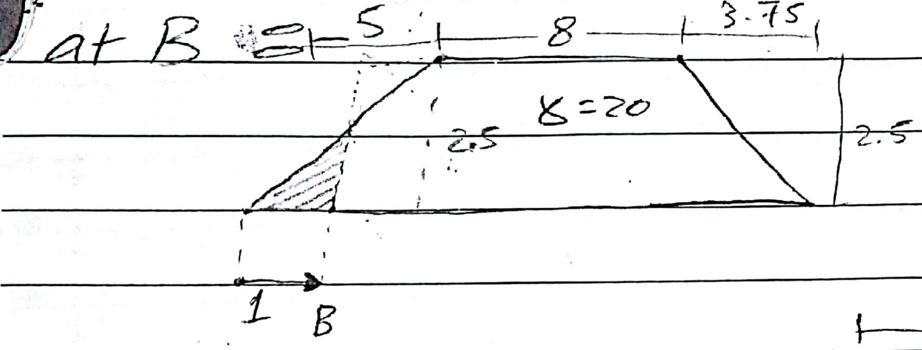
$\alpha_1 = \left[\tan^{-1}\frac{4+3.75}{2} - 1.1\right] = 0.2$

$\Delta\sigma_R = \frac{50}{\pi}\left[\left(\frac{4+3.75}{3.75}\right)(1.3) - \frac{4}{3.75}(0.2)\right]$

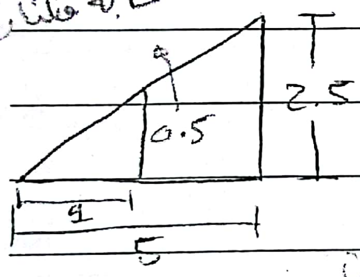
$39.38\ kPa$

$\Delta\sigma = 63.98\ kPa$

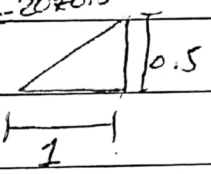
at B



تشابه مثلثات



$\Rightarrow$ $q_o = 20 * 0.5 = 10$



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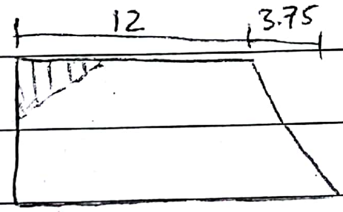

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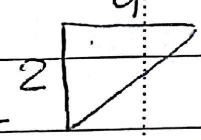


$B_1 = 0$ للمربع

$B_2 = 1$ للمثلث

لا يوجد α

نعوض على القانون *

$q_o = 50$ kPa

$B_1 = 12$

$B_2 = 3.75$

$q = 40$ kPa

$B_1 = 0$

$B_2 = 4$

Subject # Consolidation & consolidation settlement :-

$$\sigma'_f = \sigma'_o + \Delta\sigma$$

$q = \gamma_f H_{fill}$

∞ ~~~~~~~~ $\to \infty$

#

$S = 100\% \to \gamma_{SAT}$

G_s, w

$e_o = G_s w$

* at time $t = 0$

$\Delta\sigma = q$; $\sigma_{Tf} = \sigma_{To} + q$

$u = u_o + \Delta u = u_o + q$

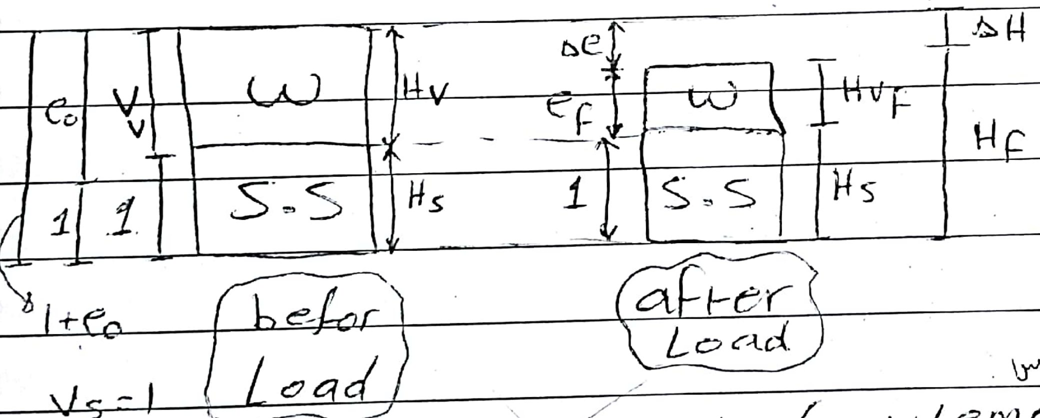
$\sigma' = \sigma'_o$

* at time $t = \infty$

$\Delta\sigma = q$; $\sigma_{Tf} = \sigma_T + q$

$u = u_o + \Delta u = u_o$

$\sigma'_f = \sigma'_o + \Delta\sigma = \sigma'_o + q$



تكريس

change in hieght (settlement)

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

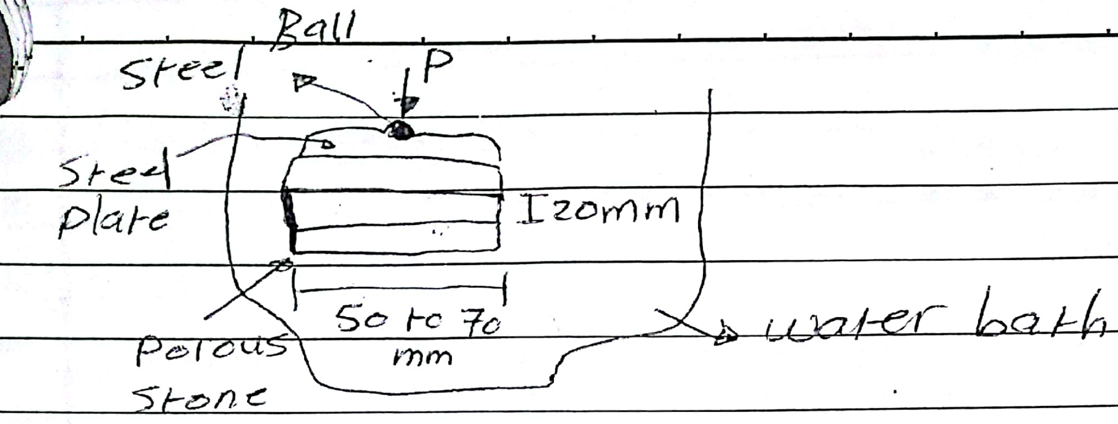
Δe → change in Void ratio

$1 + e_o$ → Initial void ratio

H_o → initial Height

$$\Delta H = \frac{\Delta e}{1 + e_o} H_o$$ } consolidation settlement

Subject Date No.



$k >> k_{soil}$

$\gamma_{SAT} = 19.81 \ kN/m^3$

$\sigma_o' = \gamma' * Z \rightarrow$ عمق 20

$= (19.81 - 9.81)\left(\frac{10}{1000}\right) = 0.1 \ kN/m^3$

$\sigma_f' = \sigma_o' + \Delta\sigma$

$\Delta\sigma = \frac{P}{A_o}$

$\sigma_f' = \frac{P}{A_o}$

day	Load	stress	Void ratio
0	0	0	e_o
1	P_1	σ_1'	e_1
2	$2P_1$	σ_2'	e_2
3	$4P_1$	σ_3'	e_3
4	$8P_1$	σ_4'	e_4
5	$16P_1$	σ_5'	e_5
6	$32P$	σ_6'	e_6
7	$16P$	σ_7'	e_7
8	$8P$	σ_8'	e_8
9	$4P$	σ_9'	e_9

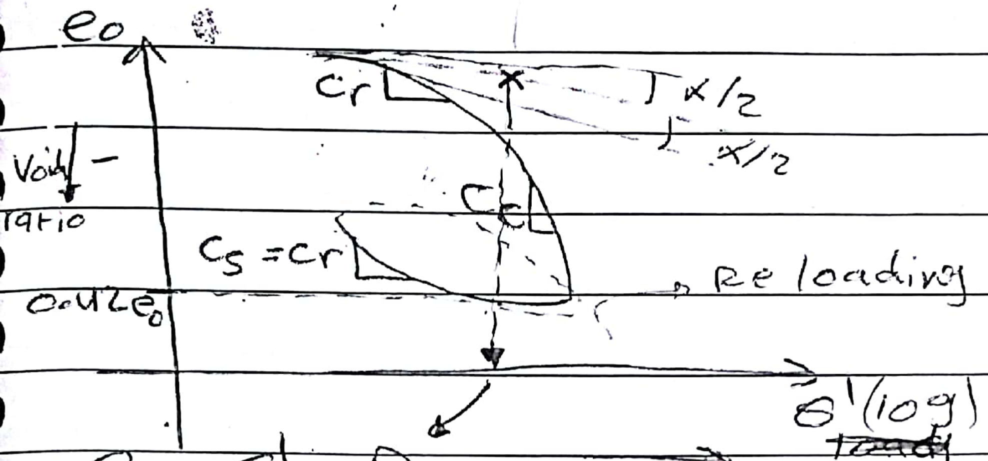
* كل ما زدت القوة راح تزيد σ و يقل Void ratio

N O T E B O O K

Subject Date No.

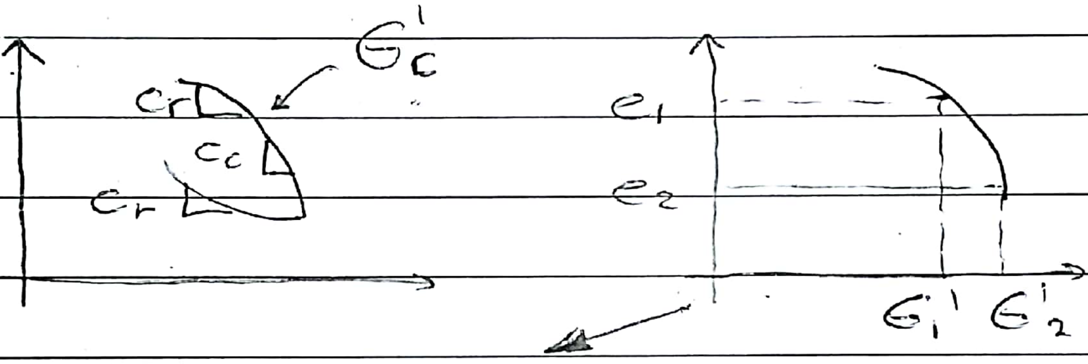
Recompresion | Virigon Compression.

[redacted]



$\sigma_p = \sigma'_c = P_c$ +

↳ maxiumum past pressure



Cr = recompression index

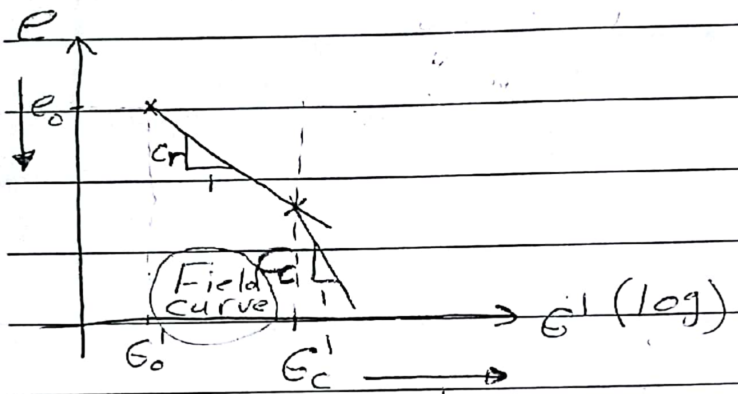
$$= \frac{\Delta e}{\Delta \log \sigma} = \frac{\Delta e}{\log \frac{\sigma'_f}{\sigma'_0}} = \left| \frac{e_2 - e_1}{\log \frac{\sigma'_2}{\sigma'_1}} \right|$$ = موجبة

Cc = compression index

$$= \frac{\Delta e}{\log \frac{\sigma'_f}{\sigma'_0}}$$

$$\frac{C_r}{C_c} \simeq \frac{1}{5} \Rightarrow \boxed{C_c = 5 C_r}$$

in



$e_o, \sigma'_o, C_r, \sigma'_c, C_c$ } معروفات من اللاب

* Conslidation settlement :-

#

هون الـ كله

w
G_s
e
γ
C_c
C_r
σ'_c

clay

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

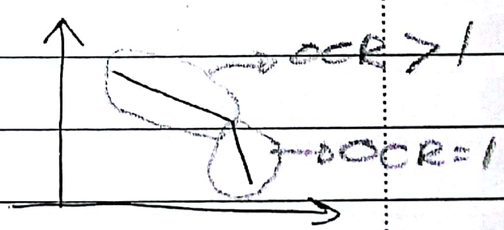
$\Delta H = S_c$ = Consolidation settlement

$$S_c = \frac{\Delta e}{1+e_o} H_o$$

over Consolidation Ratio :- $OCR = \frac{\sigma'_p = \sigma'_c}{\sigma'_o}$

$OCR > 1 \longrightarrow$ O.C clay (over cons...)

$OCR = 1 \longrightarrow$ N.C clay (Normal cons...)



$OCR < 1 \longrightarrow$ meta stable.

$$O.C.R = \frac{\text{الاجهاد السابق}}{\text{الاجهاد الحالي}} = \frac{\sigma_c}{\sigma_o}$$

د. كند

سهيل

Consolidation theory

* is the gradual reduction in volume of fully saturated soil of low permeability due to drainage of some of the pore water.

* التعريف مهم جداً

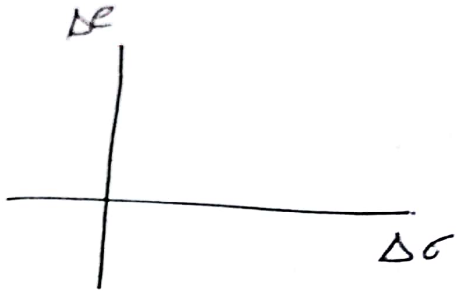
* يجب حفظه كامل دون نقصانه

---

* وعكس هذه العملية الـ swelling ⇐ التعريف في السلايدات

* ما تبقى من المقدمة تمت مناقشته في المراجعة

Oedometer test



خلال هذه التجربة يتم تعريض عينة من التربة لقوة بحيث يتم زيادة هذه القوة بمقدار ثابت (load increment)

* في نهاية التجربة نقيس نسبة الماء (ω_1)

ثم نحسب "e" ⇐ $e_1 = \omega_1 G_s$ ⇐ let $S_r = 100\%$

وبما أنه سمك العينة الابتدائي (H_o) معلوم

$$\therefore \quad \Delta e = \frac{1 + e_o}{H_o} \Delta H$$

e_o: Void Ratio at start of test

H_o = thickness of sample at start of test

* لكننا نعلم ان العينة مكونة من حبيبات صلبة وهواء وماء ولو فرضنا انه سمك الجزء الصلب هو H_s

فإن :

$$H_s = \frac{M_s}{A\, G_s\, \rho_w}$$

A : مساحة مقطع العينة

M_s : وزن العينة وهي جافة

coeff. of compressibility (m_v)

* هذا الثابت يدل على قابلية التربة للإنضغاط فيما اذا تعرضت لقوة معينة

⇐ التعريف حفظ من السلايد

⇐ الوحدة $\frac{m^2}{kN}$

* يمكننا التعبير عن التغير في حجم التربة بطريقتين :

1- change in void Ratio

2- change in thickness

ولهذا يوجد قانونين لحساب m_v :

$$a)\quad m_v = \frac{1}{1+e_0}\left(\frac{e_0 - e_1}{\sigma'_1 - \sigma'_0}\right)$$

$$b)\quad m_v = \frac{1}{H_0}\left(\frac{H_0 - H_1}{\sigma'_1 - \sigma'_0}\right)$$

* H_0 : طول العينة قبل وضع الحمل (load)
H_1 : " " بعد " "

* $\acute{6}_0$: الضغط ع العينة قبل وضع الحمل
$\acute{6}_1$: " " " " بعد " "

→ we deal with effective stress
تعامل
Take care

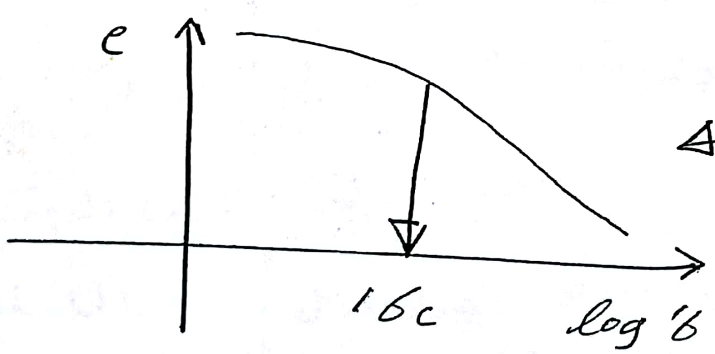
---

preconsolidation pressure ($\acute{6}_c$) ⇒ effective stress

* هو أكبر ضغط تعرضت له الطبقات الطينية في الماضي

* مثلا لو انه طبقة من الطين تم البناء عليها في الماضي ثم تمت ازالة البناء لوضع بناء جديد وبالتالي عند بناء الجديد يؤخذ بعين الاعتبار الحمل الذي كان عليها من البناء القديم

* يتم ايجاده بالمختبر باستخدام طريقة Casagrande methode المشروحة بالسلايد



* رسم تقريبي فقط
والتفاصيل في المختبر

Compression and recompression indices

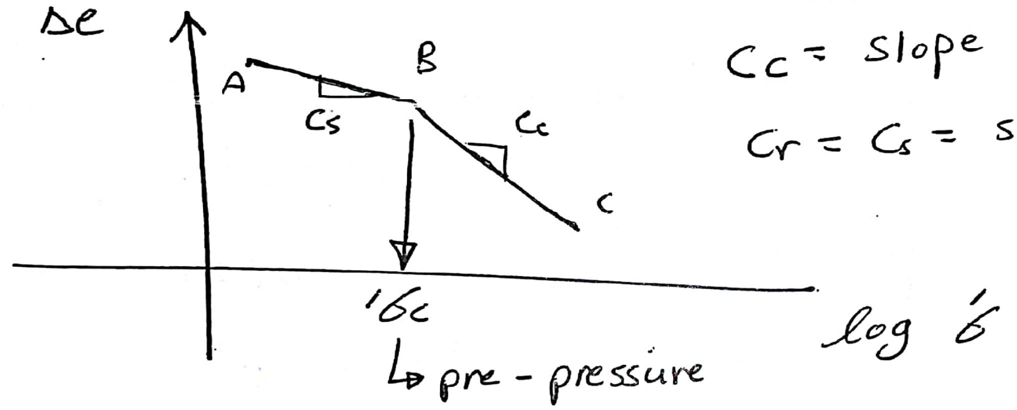
* C_c = compression index

* $C_s = C_r$ = recompression index
  = swelling index

← * ا لحفظ
تعريف كل
منهم

* $(C_s = C_r) \approx (\frac{1}{4} \text{ to } \frac{1}{5}) * C_c$

↲ هذه علاقة تقريبية يتم استخدامها
حين عدم توافر المعطيات الكافية



C_c = slope of $\overline{BC}$

$C_r = C_s$ = slope of $\overline{AB}$

Settlement

* هو هبوط يحدث في طبقات
التربة عند تعرضها للضغط أو حمل (load) وهي
وهو نوعان ⇐ ١- الابتدائي : وهو الاكبر وله وقت نهاية
٢- الثانوي : أقل و يمتد الى فترات
طويلة

A. primary consolidation settlement

1 لإيجاد مقدار الهبوط في هذا النوع يجب حساب

ال $OCR = \dfrac{\sigma'_c}{\sigma'_o} = \dfrac{\text{pre-pressure}}{\text{initial Effictive stress}}$

2 IF $OCR = 1 \Rightarrow$ called Normally consolidation

Then primary settlement $= \delta_c = \dfrac{C_c}{1+e_o} H \log\left(\dfrac{\sigma'_f}{\sigma'_o}\right)$

* H: سمك طبقة الطين (clay) المراد حساب الهبوط فيها

* $\sigma'_f \Leftarrow$ الضغط النهائي في منتصف طبقة الطين المراد حساب الهبوط فيها

* $\sigma'_o \Leftarrow$ الضغط الابتدائي فيها

* $\sigma'_o , \sigma'_f \Leftarrow$ Effective stress not Total stress

3 ~~[illegible]~~ اذا كانت قيمة ال OCR اكبر من واحد

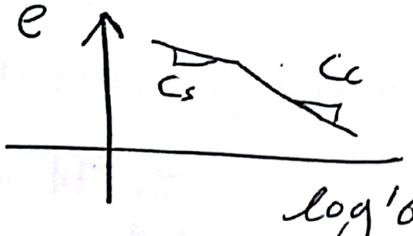
يوجد حالتين تحت هذا البند :

I. $\sigma'_o < \sigma'_f < \sigma'_p \Rightarrow$ اذا كان الضغط النهائي على التربة أعلى من الضغط الابتدائي ولكن بنفس الوقت أقل من الضغط الذي تعرضت له الطبقة في الماضي $(\sigma'_c = \sigma'_p)$

$$S_c = \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$: في هذه الحالة

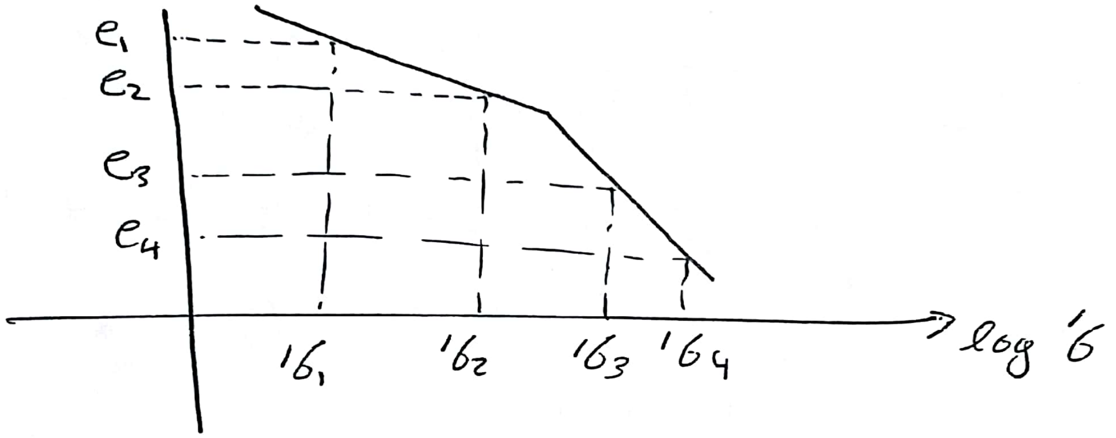
II. if $\sigma'_0 < \sigma'_p < \sigma'_f$

Then $$S_c = \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_p}{\sigma'_0}\right) + \left(\frac{C_c}{1+e_0}\right) H \log\left(\frac{\sigma'_f}{\sigma'_p}\right)$$



C_r ، C_c ⟸ من المخطط

بصورة أوضح



$$C_c = \frac{e_3 - e_4}{\log\left(\frac{\sigma'_4}{\sigma'_3}\right)} \quad , \quad C_s = \frac{e_1 - e_2}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)}$$

= positive slope   = positive slope

Secondary settlement

الهبوط الثانوي

* بعد انتهاء الهبوط الإبتدائي سيتوقف الهبوط دائما ليبدأ الثانوي ليمتد عبر الزمن الطويل ويحسب بالعلاقة التالية:

$$\delta_t = C'_\alpha H \log\left(\frac{t}{t_p}\right)$$

$$= \frac{C_\alpha}{1+e_p} * H * \log\left(\frac{t}{t_p}\right)$$

انتبه الى الفرق بين C_α و C'_α

t : الوقت المراد حساب الهبوط الثانوي عنده ويبدأ من بعد انتهاء الأولي

t_p : الوقت اللازم لانتهاء الهبوط الأولي

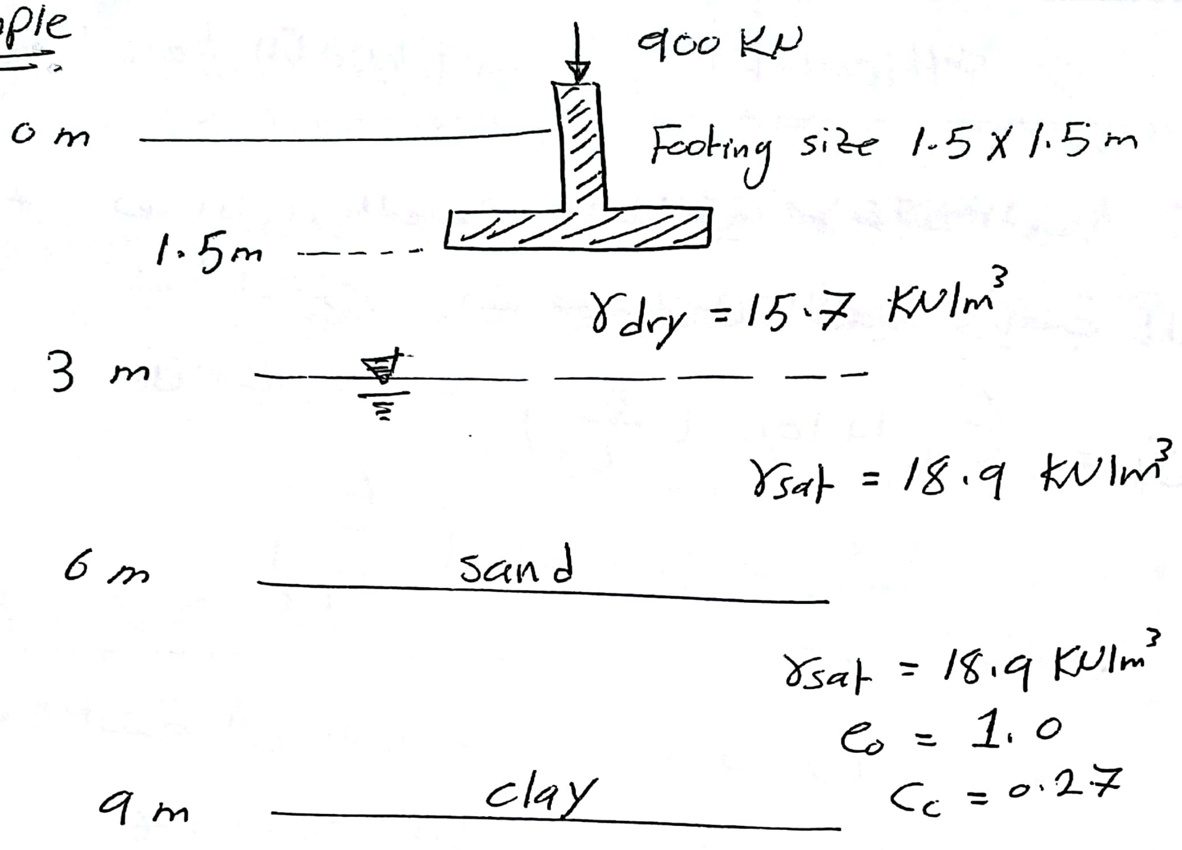
H : سمك طبقة ال clay المراد حساب الهبوط فيها

e_p : قيمة ال Void Ratio في نهاية الهبوط الاولي وتساوي $e_o - \Delta e$ حيث انه Δe هي مقدار الهبوط في قيمة ال Void Ratio في نهاية الهبوط الأولي

* يوجد مجموعة من الارقام الخاصة بالنسبة $\frac{C_\alpha}{C_c}$ لكل نوع من انواع التربة

← إحفظها

Example



* The clay is normally consollidated.

calculate the settlement at clay layer

الحل

$\boxed{1}$ من معطيات السؤال لدينا $OCR = 1.0$

لذلك:

$$S_c = \frac{C_c}{1+e_o} H \log\left(\frac{\sigma_f}{\sigma_o}\right)$$

$\boxed{2}$

σ_o at mid of clay layer

= stress befor Footing constructed

$$= 3(15.7) + 3(18.9 - 9.81) + \frac{3}{2}(18.9 - 9.81)$$

$$= 88 \text{ KN/m}^2$$

← قمنا بحساب الضغط المتوسط في طبقة الـ clay وعادة ما يكون في منتصف الطبقة أو نقوم بحساب الضغط عند بداية ونهاية

Degree of Consolidation

* في الشرح السابقة كنا نقوم بحساب خصائص الـ Consolidation فقط عند بداية ونهاية العملية حيث كنا نستخدم الـ:

$\acute{\sigma}_0$, $\acute{\sigma}_f$, e_0 , e_f --

فإما بداية أو نهاية

* هذا الموضوع لحساب تلك الخصائص عند أي قيمة لل

Stress $\acute{\sigma}$ أو أي قيمة لل e = Void Ratio التي تتغير خلال العملية

* U_z = degree of consolidation at specific time

$$= \frac{e_0 - e}{e_0 - e_1} = \frac{\acute{\sigma} - \acute{\sigma}_0}{\acute{\sigma}_1 - \acute{\sigma}_0}$$

e_0 = at starting point (befor conso. strats)

e_1 = at end of conso.

and so on For $\acute{\sigma}_0$ and $\acute{\sigma}_1$

But "e" = Void Ratio at which we want to calculate U_z

and so on For "$\acute{\sigma}$"

* Take care : $\acute{\sigma}_1 = \acute{\sigma}_0 + \underline{\underline{u_e}}$ → usuall = $\Delta\acute{\sigma}$ as we disscused

* $U_z = 1 - \dfrac{u_e}{u_i} \Rightarrow u_e = [1 - U_z]\, u_i$

هذا مهم

at $t = 0.0 \Rightarrow U_z = 0.0$
at $t = \infty \Rightarrow U_z = 1.0$ $\Bigg] \rightarrow 0 \leq U_z \leq 1$

* new term called coefficient of conso. $= C_v$
and it's unit is m^2/year

$\hookrightarrow$ This constant used to calculate something called $T_v = \dfrac{C_v * t}{(d)^2}$

where T_v = Time Factor

and $d = \dfrac{H}{2}$ For double drainge
$= H$ For single drainge

sand	sand
single drainge	double drainge
Roke	sand

هنا الانتقال مستوى 2
من طبقة

حيث الانتقال للماء......
مستوى 2 من طبقتين

* مجموعة من المعادلات المستخدمة خلال الحل :

$$* \text{ IF } U \leq 60\% \Rightarrow T_v = \frac{\pi}{4} U^2$$

$$\text{OR } U_i = \sqrt{\frac{4T_i}{\pi}} \quad \text{where} \quad 0 \leq T \leq 0.197$$

كلا المعادلتين لنفس الغرض لكنه الاولى بدلالة ال U وعند عكس المتغيرات تنتج المعادلة الثانية وللك الحرية باستخدام اي منها لكن

يجب الانتباه لل Range الخاص بكل معادلة

$$* \text{ IF } U > 60\% \Rightarrow T_v = -0.933 \log(1-U) - 0.085$$

$$\text{OR } U_i = 1 - \frac{8}{\pi^2} e^{-\pi^2 T/4} \quad \text{if } T > 0.197$$

لكن لماذا نقوم بحساب U_z ؟؟

وذلك لايجاد قيمة الهبوط عند أي مرحلة وأي وقت حسب العلاقة

$$\delta_c(t) = \text{settlement at specific time} = U_z * S_c$$

S_c: Full value of settlement (at $U = 100\%$ OR at t_{100})

t_{100}: time at which $U = 100\%$

* $$C_v = \frac{0.196\, d^2}{t_{50}}$$ ⇒ هذا ليس بقانون جديد

ارجع للقوانين في الصفحة

السابقة ونستطيع اشتقاقه

هذا القانون عند $U = 50\%$

where $T_v = \frac{C_v\, t_{50}}{d^2}$ ⇒ $t_{50} = \frac{\pi}{4}(0.5)^2 = 0.196$

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

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

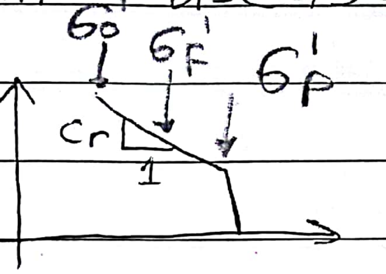
هذا تم اشتقاقه من قوانين الصفحة

السابقة وهو ليس بجديد

Consolidation settlement, 4-cases:-

① $\sigma'_o < \sigma'_f < \sigma'_p$

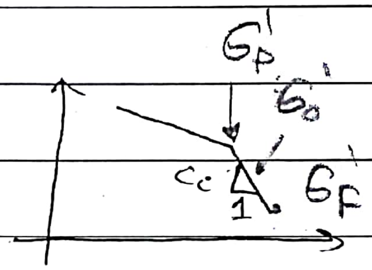
$$S_c = \frac{C_r}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_o}$$



$OCR > 1$

② $\sigma'_p < \sigma'_o < \sigma'_f$

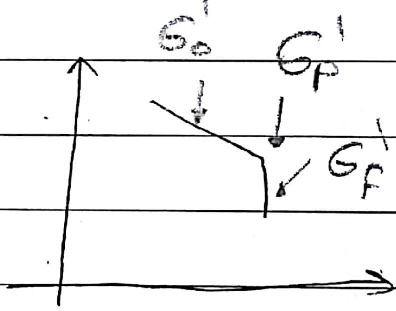
$$S_c = \frac{C_c}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_o}$$



$OCR = 1$

③ $\sigma'_o < \sigma'_p < \sigma'_f$

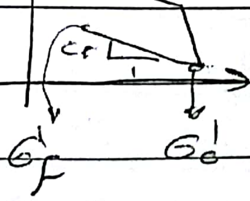
$$S_c = \frac{C_r}{1+e_o} H_o \log \frac{\sigma'_p}{\sigma'_o} + \frac{C_c}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_p}$$



$OCR > 1$

④ $\sigma'_f < \sigma'_o$

$$S_c = \frac{C_r}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_o} \quad (-ve)$$



Ho → فقط لطبقة ال clay

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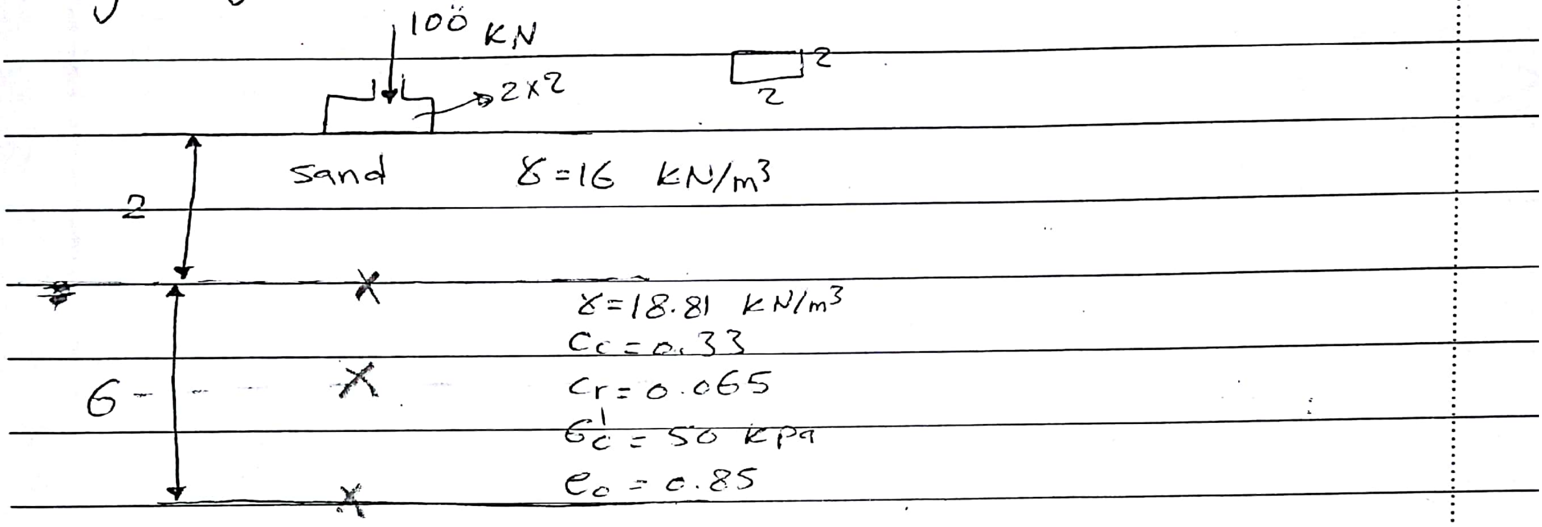
Ex :- Determine Consolidation settlement on clay layer due to footing Loading :-



Sol :- (at middle of clay layer)

$\sigma'_o = \Sigma \gamma z$

$= (16*2) + (18.81 - 9.81)*(3) = 59$ kpa

$\sigma'_f = \sigma'_o + \Delta\sigma$

$$\Delta\sigma = \frac{1}{6}\left[\overbrace{\Delta\sigma_{top} + 4\,\Delta\sigma_{middle} + \Delta\sigma_{bottom}}^{1+4+1=6}\right]$$

$\Delta\sigma_{top} = \dfrac{q_o\, B \times B}{(B+z)(B+z)}$ use 2:1 method

$q_o = \dfrac{P}{A} = \dfrac{100}{2*2} = 25$ kpa

$\Delta\sigma_{top} = \dfrac{25*2*2}{(2+2)(2+2)} = 6.25$ kpa

$\Delta\sigma_{mid} = \dfrac{25*2*2}{(2+5)(2+5)} = 2.04$ kpa

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$\Delta \sigma_{bottom} = 1\ kPa$

$\Delta \sigma = \frac{1}{6} \left[6.25 + 4 * 2.04 + 1 \right] = 2\cancel{4}.56\ kPa$

$\sigma'_f = \sigma'_o + \Delta \sigma = 59 + \overset{2.56}{\cancel{4.56}} = \overset{61.59}{\cancel{63.56}}\ kPa$

$\underset{=50}{\sigma'_c} < \underset{=59}{\sigma'_o} < \sigma'_f = 63.56 \Rightarrow$ Case II (N.C)

$S_c = \frac{C_c}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_o}$

$= \frac{0.3}{1+0.85} (6) \log \frac{63.56}{59} = 0.032\ m = 32.4\ mm$

سماكة طبقة الـ clay

Time Rate of Consolidation :-

S_c = Consolidation Settlement.

$\dot{S}_c = \dot{u}$ excess

At time $t=0$ σ_T, u, σ' known بالبداية معروفة

At time $t=\infty$ σ_T, u, σ' known بالنهاية معروفة

$\Rightarrow \sigma_T$ at any time ✓

$u_f = u_o + \Delta u$ $\Delta u = u_e$ at any time ?? مجهول

σ' at any time ? مجهول

* Terzaghi Theory :-

Assumption :-

1) clay is homogeneous.

2) $S = 100\%$

*اكي كليهم اسكم عبارة عن الفرق بين Lablace ونظرية Terzaghe

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تفريغ

3) Drainage at both top and bottom of compression layer.

4) Dary's law is valid.

5) soil grains and water are incompressible

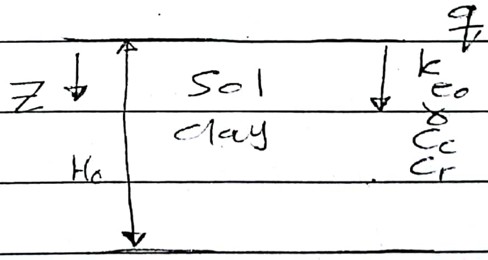
6) compression and flow are 1-D

7) Small Load increment → change in thickness is negligible $k = constant$.

8) unique relation between Δe and $\Delta \sigma'$ $(de = -a_v \, d\sigma')$

9) soil is ~~...~~ Isotropic.

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



$$C_v = \left(\frac{k}{\gamma_w}\right)\left(\frac{1+e_o}{a_v}\right)$$

where :-

u_e : excess (زيادة) pore water pressure at $0^+ < t < \infty^-$

t : time (since appling the Load).

z : depth $0 < z < H_o$

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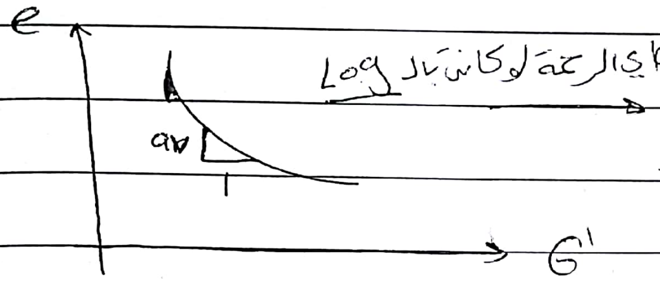
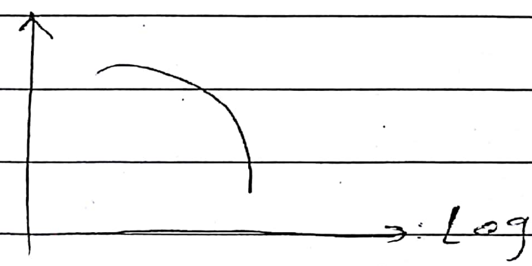
C_v : Coeficent of consolidation (m^2/s) [illegible]

k : hydraulic conductivity

γ_w : unit weight of water

e_0 : Initial void ratio





$$a_v = \frac{\Delta e}{\Delta \sigma'} \rightarrow a_v = \frac{de}{d\sigma'}$$

$$\Rightarrow de = a_v \, d\sigma'$$

m_v = Coeficient of Volume change

$$* \quad \frac{\Delta V}{V_0} = m_v = \frac{a_v}{1+e_0}$$

Sand or air $u = 0$

clay

H_0

- Initial condition $t=0$ $t=\infty$
- Bondary condition

$$u_e = f(\text{time}) f(\text{space})$$

$$u_e = \sum_{m=0}^{m=\infty} \left(\frac{2 \, u_{exes\,(at\ time\ 0)}}{M} \right) \left(\sin \frac{M \cdot z}{d} \right) \left(e^{-M^2 T} \right)$$

$m = 0, 1, 2, 3, 4, \ldots$

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

$u_{exe\ at\ time} = \Delta\sigma$

$d = H_0$ — sand or air / H_0 / sand

$d = \frac{H_0}{2}$ — sand/air / H_0 / sand

$$T = \frac{C_v t}{d^2} \quad \text{Time Factor}$$

N O T E B O O K

$H_0 \rightarrow$ shortest drning distance

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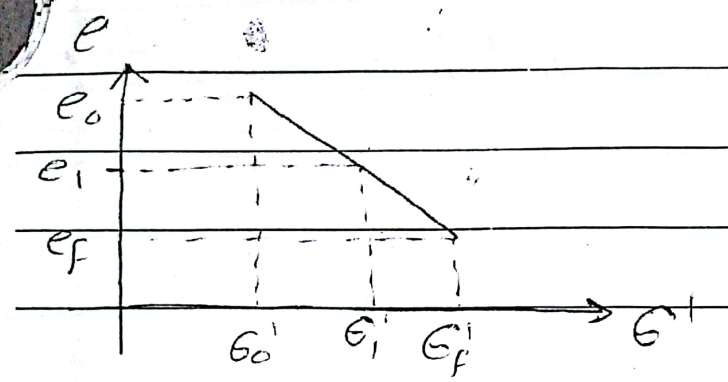


Degree of consolidation , U_Z :-

$$U_Z = \frac{e_o - e_1}{e_o - e_f}$$

عند أي زمن U ـه

$$= \frac{\sigma_f' - \sigma_1'}{\sigma_f' - \sigma_o'} = \frac{u_o - u_e}{u_o} = 1 - \frac{u_e}{u_o} = \boxed{1 - \frac{u_e}{\Delta\sigma}}$$

الي حكيناه قبل

اصلا هدول بريحنا من هاي العلاقة :-

Excessive pore water pressure Isochrones.

$T = \frac{C_v t}{d^2}$

$\frac{Z}{d} = \frac{Z}{H}$

T بنحسب *

$\frac{Z}{d}$ بنحسب *

U_Z بنجيب *

* هسا احنا بدنا نحسب لكل نقطة فبدنا نوخذ ال avg

→ Average Degree of consolidation :-

$$\bar{U} = U = \int_0^d U_Z \, dz$$

$\Rightarrow$

* If $T < 0.197 \longrightarrow U < 0.6$

نسبة 60 %

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

$$\Rightarrow U = \sqrt{\frac{4T}{\pi}}$$

* If $T = 0.197 \longrightarrow U = 60\%$

* If $T > 0.197 \longrightarrow U > 60\%$

$$T = 1.781 - 0.933 \cdot \text{Log}(1 - U)$$

$$U = 1 - \frac{8}{\pi^2} e^{-\frac{\pi^2 T}{4}}$$

$$S = U_e$$

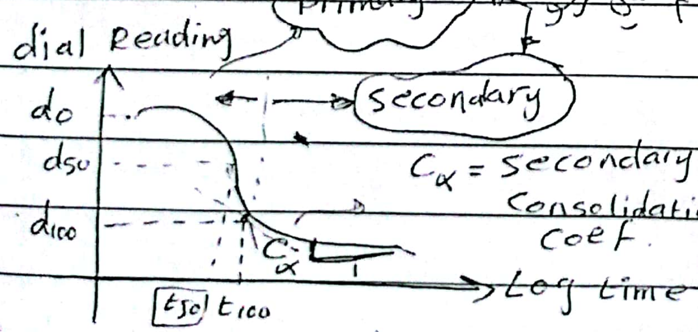
$$\bar{U} = U_{avg} = U = \frac{S_t}{S_c}$$

S_t = Consolidation settlement at any time, t

S_c = primary consolidation settlement

U = average degree of consolidation.

primary — اهم اشي تعرفو



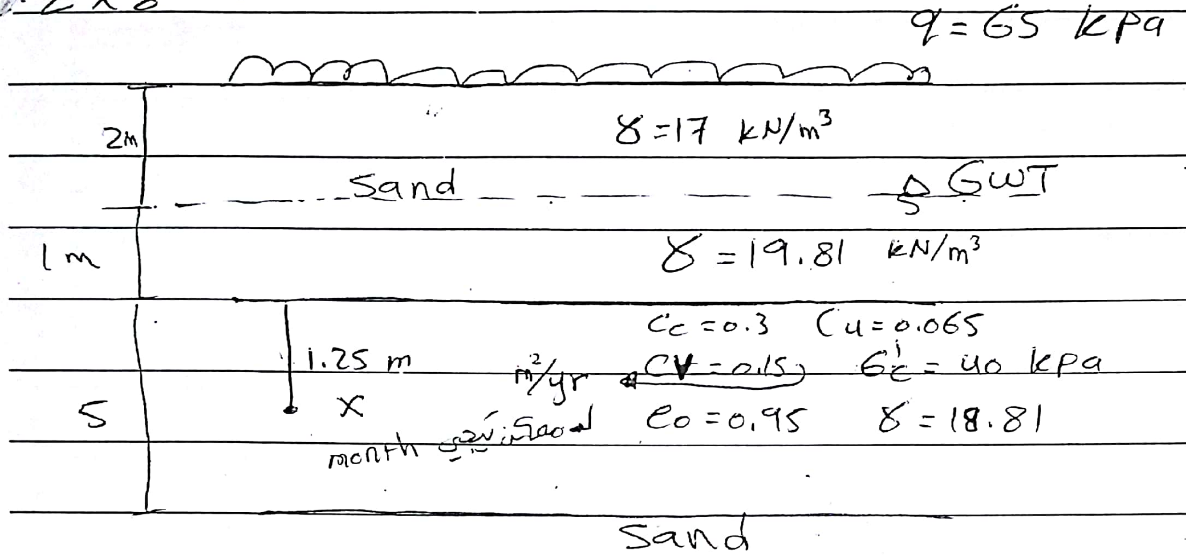
C_α = secondary consolidation coef.

بدو مدوز

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

Subject صلوات Date No.

Ex 8-



For soil profile shown Determain :

أساسي/مبدئي

1) primary Consolidation settlement on clay layer due to surcharge load. القوة الخارجية

$\Delta\sigma = q$

خلال وضع القوة t = 0 يعني

والدكتور عرف t هو الزمن عند بدء وضع القوة.

2) settlement after ~~layers~~ 1 yrs ?

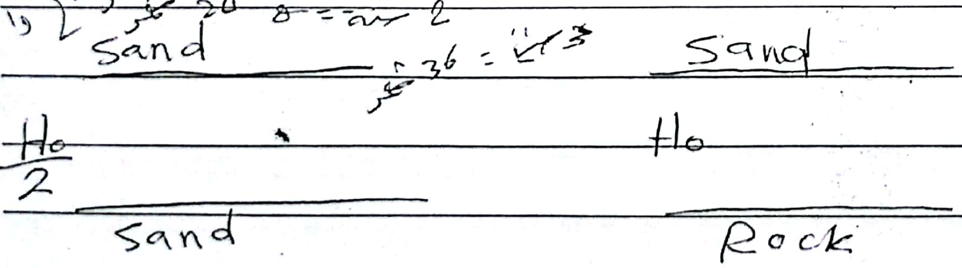
3) if settlement measured ... he 20 mm, estimate time of applying the load

4) σ' at point X shown after 3yrs ?

إذا الطالب معطيك Cv وحدته month قبل تحول 1 سنة = 12 شهر

2 سنة = 24 شهر

3 سنة = 36 شهر



Sol:

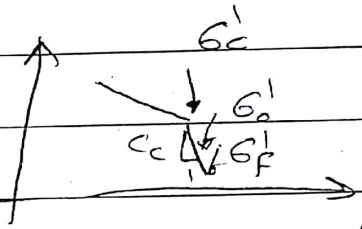
① $\sigma'_o = \sum \gamma' Z$ عند منتصف طبقة ال clay

$= (17)(2) + (19.81-9.81)(1) + (18.81-9.81)(2.5) = 66.5\ kpa$

$\sigma'_f = \sigma'_o + \Delta\sigma$ → at time ∞

$\Delta\sigma = q$

$\sigma'_f = 66.5 + 65 = 131.5\ kpa$



$\therefore \sigma'_c < \sigma'_o < \sigma'_f$

$40 < 66.5 < 131.5$

→ Normal consolidation clay

$\therefore S_c = \frac{C_c}{1+e_o} H_o \log \frac{\sigma'_f}{\sigma'_o}$

$= \frac{0.3}{1+0.95} * (15) \log \frac{131.5}{66.5} = 0.2277\ m = 227.7\ mm$

② $T = \frac{C_v t}{d^2} = \frac{0.15 * 1}{(\frac{5}{2})^2} = 0.024$

→ $T = 0.024 < 0.197$

$U = \sqrt{\frac{4T}{\pi}} = \sqrt{\frac{4 * 0.024}{\pi}} = 0.175$

③ $U = \frac{S_t}{S_c}$ → $S_t = U S_c = 0.175 * 227.7 = 39.8\ mm$

③ $U = \frac{20}{227.7} = 0.087 < 0.6$

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$$T = \frac{\pi}{4} U^2 = \frac{\pi}{4} * (0.087)^2 = 5.9 * 10^{-3}$$

$$T = \frac{C_v t}{d^2}$$

$$t = \frac{(5.9 * 10^{-3})(\frac{5}{2})^2}{0.15} = 0.039 \text{ yr} \xrightarrow{*360} 14.35 \text{ day}$$

(4) $\sigma' = \sigma_T - u$

$\sigma_T = \sigma_{To} + \Delta\sigma$

$\sigma_{To} = \sum \gamma z = (17*2) + (19.81*1) + (18.81*1.25) = 77.32$ kPa

X مهر

$\sigma_{Tf} = 77.32 + 65 = 142.32$ kPa

$u = u_o + \Delta u$

$u_o = \gamma_w h_w = (9.81)(1 + 1.25) = 22.07$ kpa

ابو كرشين

$$\Delta u_e = \sum_{m=0}^{m=\infty} \frac{2 u \sigma}{M} \sin\left(\frac{MZ}{d}\right) e^{-M^2 T} \longrightarrow \text{لا نستخدمه}$$

or

$$U_z = 1 - \frac{u_e}{u_{e_o}} = 1 - \frac{u_e}{\Delta\sigma}$$

$$T = \frac{C_v t}{d^2} = \frac{0.15 * 3}{(\frac{5}{2})^2} = 0.072$$

$$\frac{z}{H_{dr}} = \frac{z}{d} = \frac{1.25}{(\frac{5}{2})} = 0.5$$

رسم اقل من ١.٠

From chart $T = 0.07$ and $\frac{z}{d} = 0.5$ isochrone

$\Rightarrow U_z = 0.2$

N O T E B O O K

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$$U_z = 1 - \frac{U_e}{\Delta\sigma} \longrightarrow U_e = (1-0.2)(65) = 52 \text{ kpa}$$

$\therefore U = U_o + U_e = 22.07 + 52 = 74.07$ kpa

$\therefore \sigma'_{3yr} = 142.32 - 74.07 = 68.25$ kpa

نهاية مادة السكند

الصفحة التالية اسئلة الكتاب ←

قبل ال Shear strength

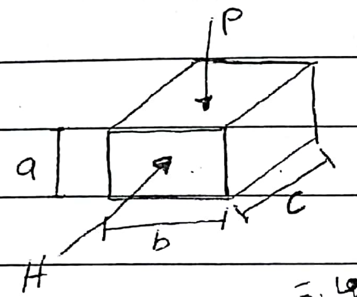
Subject Soil Final Date No.

Shear strength - Failure criterion

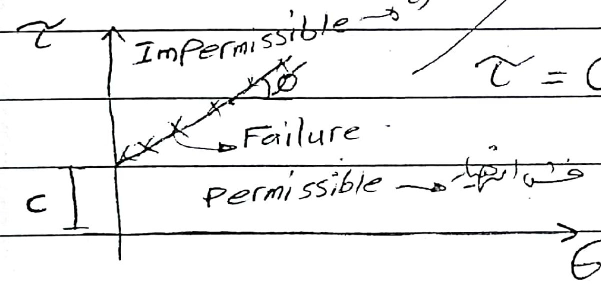
Mohr - Calame :-

σ_n = Normal stress

τ = shear stress



$\sigma_n = \frac{P}{bc}$ المساحة العمودية $\tau = \frac{H}{bc}$ المساحة الموازية للقطع



معادلة الخط

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

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

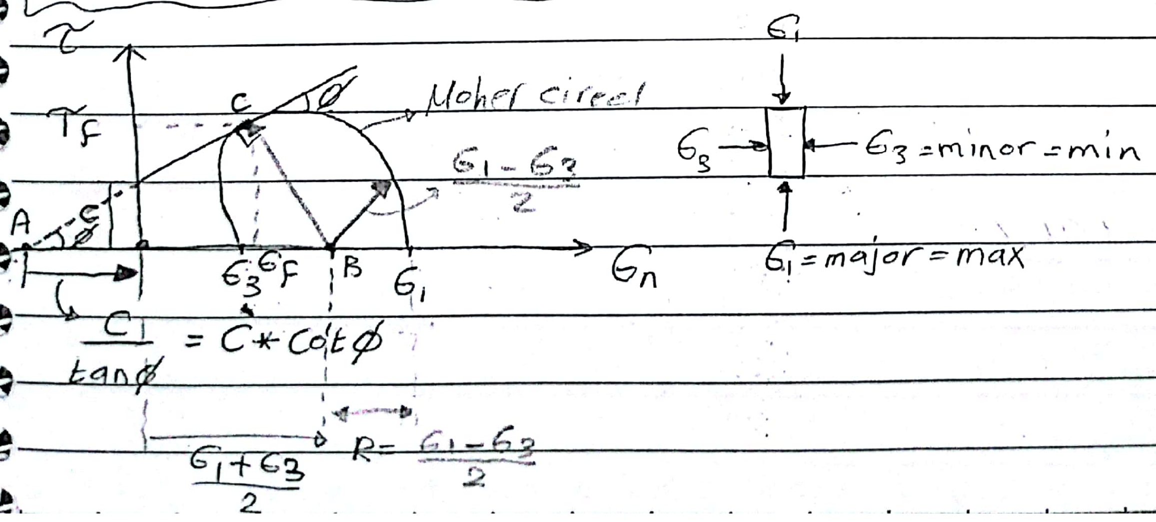
where :-

τ : Shear stress

C : cohesion التماسك

σ_n : normal stress

ϕ : Internal friction angle



9

6.7 Compression Index (C_c)

The compression index for the calculation of field settlement caused by consolidation can be determined by graphic construction (as shown in Figure 6.11) after one obtains the laboratory test results for void ratio and pressure.

Terzaghi and Peck (1967) suggest the following empirical expressions for compression index:

$$\text{Undisturbed clays: } C_c = 0.009(LL - 10) \tag{6.24}$$

$$\text{Remolded clays: } C_c = 0.007(LL - 10) \tag{6.25}$$

where LL = liquid limit, in percent

In the absence of laboratory consolidation data, Eq. (6.24) is often used for approximate calculation of primary consolidation in the field.

6.8 Swell Index (Cs)

The swell index is appreciably smaller in magnitude than the compression index and generally can be determined from laboratory tests. In most cases,

$$C_s \simeq \tfrac{1}{5} \text{ to } \tfrac{1}{10} C_c \tag{6.26}$$

The liquid limit, plastic limit, virgin compression index, and swell index for some natural soils are given in Table 6.1.

Example 6.1

A soil profile is shown in Figure 6.13a. Laboratory consolidation tests were conducted on a specimen collected from the middle of the clay layer. The field consolidation curve interpolated from the laboratory test results is shown in Figure 6.13b. Calculate the settlement in the field caused by primary consolidation for a surcharge of 48 kN/m² applied at the ground surface.

Solution

$$\sigma'_0 = (5)(\gamma_{sat} - \gamma_w) = 5(18.0 - 9.81)$$
$$= 40.95 \text{ kN/m}^2$$

Table 6.1 Compression and Swell of Natural Soils

Soil	Liquid limit	Plastic limit	Compression index, C_c	Swell index, C_s
	41	20	0.35	0.07

أسئلة كتاب مهمة

6.7 Compression Index (C_c)

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Table 6.1 Compression and Swell of Natural Soils

Soil	Liquid limit	Plastic limit	Compression index, C_c	Swell index, C_s
	41	20	0.35	0.07

$$e_o = 1.1$$

$$\Delta\sigma' = 48 \text{ kN/m}^2$$

$$\sigma'_o + \Delta\sigma' = 40.95 + 48 = 88.95 \text{ kN/m}^2$$

$\sigma'_{vo} = \sigma_t - u$

$= \frac{10}{2}[18 - 9.8]$

$= 40.95$

$\sigma'_F = \sigma'_{vo} + \Delta\sigma_z$

$= 88.95$

(1) $\delta_c = \frac{\Delta e}{1 + e_o} H$

$= \frac{[1.1 - 1.045] * 10}{1 + e_o} = 0.262$ m

1.1

(2) $OCR = \frac{\sigma'_c}{\sigma'_{vo}}$

$\frac{70}{40.95}$

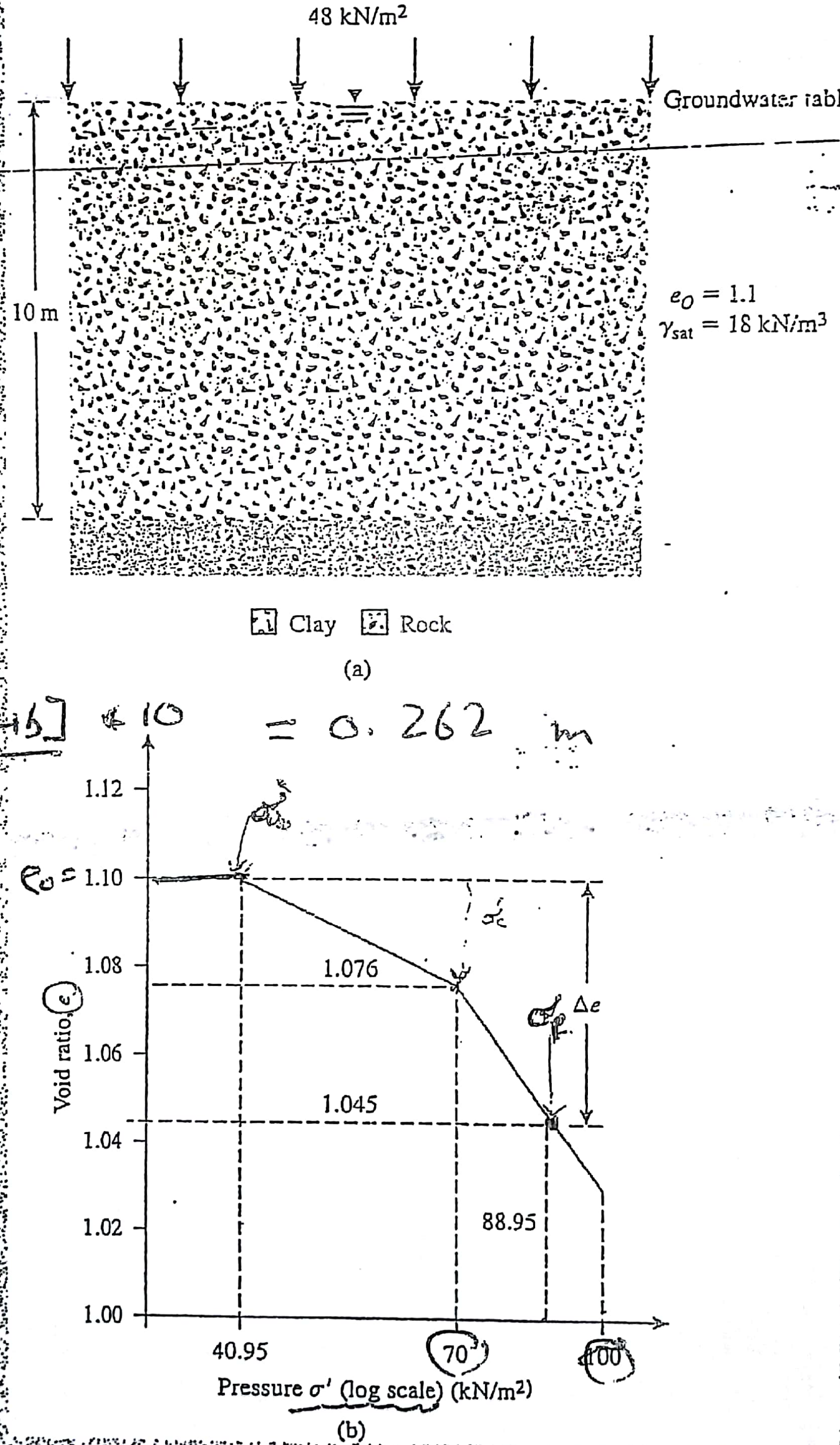


Figure 6.13 (a) Soil profile (b) field consolidation curve

The void ratio corresponding to 88.95 kN/m² (see Figure 6.13b) is 1.045. Hence, $\Delta e = 1.1 - 1.045 = 0.055$. We have

$$\text{Settlement } (S_c) = H\frac{\Delta e}{1 + e_O} \qquad \text{[Eq. (6.18)]}$$

so,

$$S_c = 10\frac{(0.055)}{1 + 1.1} = 0.262 \text{ m} = 262 \text{ mm}$$

Example 6.2

A soil profile is shown in Figure 6.14. If a uniformly distributed load, $\Delta\sigma$, is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation if

a. The clay is normally consolidated
b. The preconsolidation pressure $(\sigma_c') = 190 \text{ kN/m}^2$
c. $\sigma_c' = 170 \text{ kN/m}^2$

Use $C_s \approx \frac{1}{6}C_c$

Solution

Part a.

The average effective stress at the middle of the clay layer is

$$\sigma_O' = 2\gamma_{dry} + 4[\gamma_{sat(sand)} - \gamma_w] + \tfrac{4}{2}[\gamma_{sat(clay)} - \gamma_w]$$

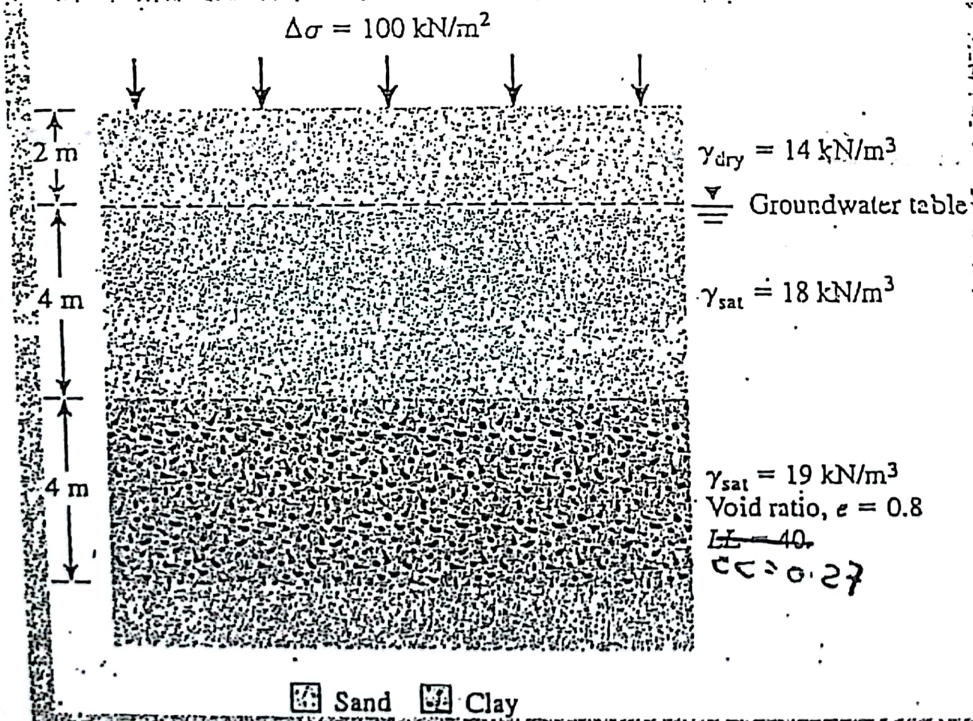


Figure 6.14

or

$$\sigma_O' = (2)(14) + 4(18 - 9.81) + 2(19 - 9.81) = 79.14\,\text{kN/m}^2$$

From Eq. (6.20),

$$S_c = \frac{C_c H}{1 + e_O} \log\left(\frac{\sigma_O' + \Delta\sigma'}{\sigma_O'}\right)$$

From Eq. (6.24),

$$C_c = 0.009(LL - 10) = 0.009(40 - 10) = 0.27$$

So,

$$S_c = \frac{(0.27)(4)}{1 + 0.8} \log\left(\frac{79.14 + 100}{79.14}\right) = 0.213\,\text{m} = \mathbf{213\ mm}$$

Part b.

$$\sigma_O + \Delta\sigma' = 79.14 + 100 = 179.14\ \text{kN/m}^2$$

$$\sigma_c' = 190\,\text{kN/m}^2$$

Because $\sigma_O' + \Delta\sigma > \sigma_c'$, use Eq. (6.22) to get

$$S_c = \frac{C_s H}{1 + e_O} \log\left(\frac{\sigma_O' + \Delta\sigma}{\sigma_O'}\right)$$

$$C_s = \frac{C_c}{6} = \frac{0.27}{6} = 0.045$$

$$S = \frac{(0.045)(4)}{1 + 0.8} \log\left(\frac{79.14 + 100}{79.14}\right) = 0.036\,\text{m} = \mathbf{36\ mm}$$

Part c.

$$\sigma_O' = 79.14\ \text{kN/m}^2$$

$$\sigma_O' + \Delta\sigma' = 179.14\ \text{kN/m}^2$$

$$\sigma_c' = 170\ \text{kN/m}^2$$

Because $\sigma_O' < \sigma_c' < \sigma_O' + \Delta\sigma'$, use (Eq. 6.23),

$$S = \frac{C_s H}{1 + e_O} \log\frac{\sigma_c'}{\sigma_O'} + \frac{C_c H}{1 + e_O} \log\left(\frac{\sigma_O' + \Delta\sigma}{\sigma_c'}\right)$$

$$= \frac{(0.045)(4)}{1.8} \log\left(\frac{170}{79.14}\right) + \frac{(0.27)(4)}{1.8} \log\left(\frac{179.14}{170}\right) = 0.0468\ \text{m}$$

$$= \mathbf{46.8\ mm}$$

■

Secondary consolidation settlement is more important than primary consolidation in organic and highly compressible inorganic soils. In overconsolidated inorganic clays, the secondary compression index is very small and of less practical significance.

There are several factors that might affect the magnitude of secondary consolidation, some of which are not yet very clearly understood (Mesri, 1973). The ratio of secondary to primary compression for a given thickness of soil layer is dependent on the ratio of the stress increment ($\Delta\sigma$) to the initial effective stress (σ'_O). For small $\Delta\sigma'/\sigma'_O$ ratios, the secondary-to-primary compression ratio is larger.

Example 6.3

Refer to Example 6.2(a). Assume that the primary consolidation will be complete in 3.5 years. Estimate the secondary consolidation that would occur from 3.5 years to 10 years after the load application. Given $C_\alpha = 0.022$. What is the total consolidation settlement after 10 years?

Solution

From Eq. (6.29)

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

The value of e_p can be calculated as

$$e_p = e_O - \Delta e_{\text{primary}}$$

From Eq. (6.19)

$$\Delta e = C_c[\log(\sigma'_O + \Delta\sigma') - \log\sigma'_O]$$

Thus,

$$\begin{aligned} e_p &= e_O C_c[\log(\sigma'_O + \Delta\sigma') - \log\sigma'_O] \\ &= 0.8 - 0.27[\log(79.14 + 100) - \log(79.14)] \\ &= 0.8 - 0.096 = 0.704 \end{aligned}$$

Hence,

$$C'_\alpha = \frac{0.022}{1 + 0.704} = 0.0129$$

Again, from Eq. (6.28)

$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right) = (0.0129)(4)\log\left(\frac{10}{3.5}\right) = 0.0235 \text{ m} = 23.5 \text{ mm}$$

Total consolidation settlement = primary consolidation settlement (S_c) + secondary consolidation settlement (S_s). From Example 6.2a, $S_c = 213$ mm. So total consolidation settlement = 213 + 23.5 = 236.5 mm ∎

Eq. (6.20), (6.22), or (6.23). However, the increase of stress $\Delta\sigma'$ in these equations should be the average increase of pressure, or

$$\Delta\sigma' = \Delta\sigma'_{av} \simeq \frac{\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b}{6} \qquad (6.46)$$

where $\Delta\sigma'_t, \Delta\sigma'_m$, and $\Delta\sigma'_b$ are increases of pressure at the top, middle, and bottom of the layer, respectively. The values can be determined by using the procedure described in Chapter 5.

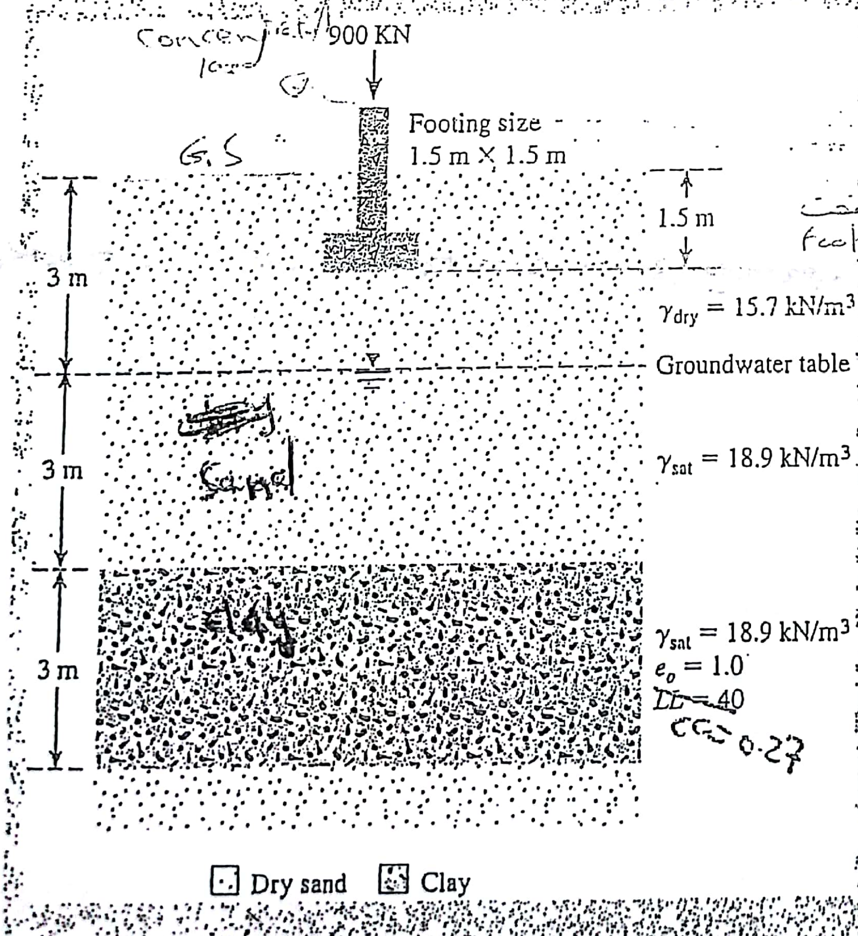
Example 6.7

Calculate the settlement of the 3 m thick clay layer (Figure 6.22) that will result from the load carried by a 1.5 m square footing. The clay is normally consolidated. ~~Use the weighted average method [Eq. (6.46)] to calculate the average increase of pressure in the clay layer.~~

Solution

For normally consolidated clay, from Eq. (6.20)

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}$$



$\sigma'_0 = 88.01$

$\Delta\sigma'_z = q I_r$

$\Delta\sigma'_z = \frac{900}{(1.5+6)(1.5+6)} = 16$ kN/m²

$\sigma'_f = \sigma'_0 + \Delta\sigma'_z = 100.01$ kN/m²

$S_c = \frac{0.27 \times 3}{1+1} \log \frac{100.01}{88} \simeq 25$

Figure 6.22

$S_c = \frac{\Delta e}{1 + e_o} H = \frac{\Delta e}{1 + 1.0} \times 3.0$

where $C_c = 0.009(LL - 10) = 0.009(40 - 10) = 0.27$

$H = 3\text{m}$

$e_o = 1.0$

$$\sigma'_o = 3 \times \gamma_{dry(sand)} + 3[\gamma_{sat(sand)} - 9.81] + \frac{3}{2}[\gamma_{sat(clay)} - 9.81]$$

$$= 3 \times 15.7 + 3(18.9 - 9.81) + 1.5(18.9 - 9.81)$$

$$= 88.01 \text{ kN/m}^2$$

~~From Eq. (6.46)~~

$$\cancel{\Delta\sigma' = \frac{\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b}{6}}$$

$\Delta\sigma'_t$, $\Delta\sigma'_m$, and $\Delta\sigma'_b$ below the center of the footing can be obtained from Figure 5.28.

$$\cancel{\Delta\sigma'_t\left(\text{at } z = \frac{4.5}{1.5}B = 3B\right) = 0.055q}$$

0.125 = $m = \frac{B}{2} = 0.75/6$

0.125 = $n = \frac{L}{2} = 0.75/6$

$\Rightarrow I_2 = 0.0075$

$$\cancel{\Delta\sigma'_m\left(\text{at } z = \frac{6}{1.5}B = 4B\right) \approx 0.028q}$$

$$\cancel{\Delta\sigma'_b\left(\text{at } z = \frac{7.5}{1.5}B = 5B\right) \approx 0.02q}$$

So

$$\cancel{\Delta\sigma' = \frac{[0.055 + (4 \times 0.028) + 0.02]q}{6} = 0.03116q}$$

However,

$$q = \frac{900}{1.5 \times 1.5} = 400 \text{ kN/m}^2$$

So

0.0075 × 4   12.0

$$\Delta\sigma' = (\cancel{0.03116})(400) = \cancel{12.46} \text{ kN/m}^2$$

Substituting these values in the settlement equation

12.0

$$S_c = \frac{0.27 \times 3}{1 + 1} \log \frac{88.01 + \cancel{12.46}}{88.01} \approx \cancel{0.023 \text{ m} = 23 \text{ mm}}$$

= 0.0225 m = 22.5 mm

Table 11.10 Comparison Between the Coefficients of Consolidation Determined in the Laboratory and Those Deduced from Embankment Settlement Analysis, as Observed by Leroueil (1988)

Site	$c_{v(\text{lab})}$ (m^2/sec)	$c_{v(\text{in situ})}$ (m^2/sec)	$c_{v(\text{in situ})}/c_{v(\text{lab})}$	Reference
Ska-Edeby IV	5.0×10^{-9}	1.0×10^{-7}	20	Holtz and Broms (1972)
Oxford (1)			4–57	Lewis, Murray, and Symons (1975)
Donnington			4–7	Lewis, Murray, and Symons (1975)
Oxford (2)			3–36	Lewis, Murray, and Symons (1975)
Avonmouth			6–47	Lewis, Murray, and Symons (1975)
Tickton			7–47	Lewis, Murray, and Symons (1975)
Over causeway			3–12	Lewis, Murray, and Symons (1975)
Melbourne			200	Walker and Morgan (1977)
Penang	1.6×10^{-8}	1.1×10^{-6}	70	Adachi and Todo (1979)
Cubzac B	2.0×10^{-8}	2.0×10^{-7}	10	Magnan, *et al.* (1983)
Cubzac C	1.4×10^{-8}	4.3×10^{-7}	31	Leroueil, Magnan, and Tavenas (198
A-64	7.5×10^{-8}	2.0×10^{-6}	27	Leroueil, Magnan, and Tavenas (198
Saint-Alban	1.0×10^{-8}	8.0×10^{-8}	8	Leroueil, Magnan, and Tavenas (198
R-7	6.0×10^{-9}	2.8×10^{-7}	47	Leroueil, Magnan, and Tavenas (198
Matagami	8.0×10^{-9}	8.5×10^{-8}	10	Leroueil, Magnan, and Tavenas (198
Berthierville		4.0×10^{-8}	3–10	Kabbaj (1985)

Example 11.11

During a laboratory consolidation test, the time and dial gauge readings obtained from an increase of pressure on the specimen from 50 kN/m^2 to 100 kN/m^2 are given

Time (min)	Dial gauge reading (cm × 10^4)	Time (min)	Dial gauge reading (cm × 10^4)
0	3975	16.0	4572
0.1	4082	30.0	4737
0.25	4102	60.0	4923
0.5	4128	120.0	5080
1.0	4166	240.0	5207
2.0	4224	480.0	5283
4.0	4298	960.0	5334
8.0	4420	1440.0	5364

Using the logarithm-of-time method, determine c_v. The average height of the specimen during consolidation was 2.24 cm, and it was drained at the top and bottom.

Solution

The semi-logarithmic plot of dial reading versus time is shown in Figure 11.2 this, $t_1 = 0.1$ min, $t_2 = 0.4$ min to determine d_0. Following the procedure out Figure 11.25, $t_{50} \approx 19$ min. From Eq. (11.66)

$$C_v = \frac{0.197 H_{dr}^2}{t_{50}} = \frac{0.197\left(\frac{2.24}{2}\right)^2}{19} = 0.013 \text{ cm}^2/\text{min} = 2.17 \times 10^{-4} \text{ cm}$$

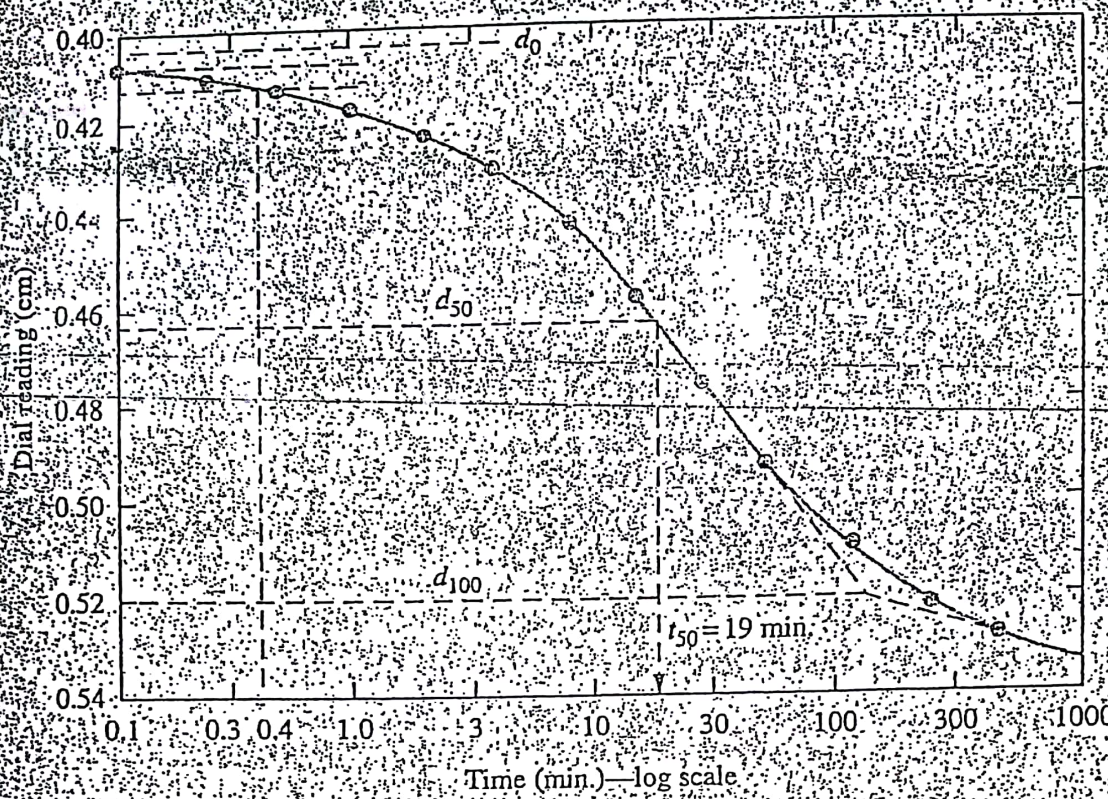


Figure 11.29

11.14 Calculation of Consolidation Settlement Under a Foundation

Chapter 10 showed that the increase in the vertical stress in soil caused by a load applied over a limited area decreases with depth z measured from the ground surface downward. Hence to estimate the one-dimensional settlement of a foundation, we can use Eq. (11.31), (11.33), or (11.34). However, the increase of effective stress, $\Delta\sigma'$, in these equations should be the average increase in the pressure below the center of the foundation. The values can be determined by using the procedure described in Chapter 10.

Assuming that the pressure increase varies parabolically, using Simpson's rule, we can estimate the value of $\Delta\sigma'_{av}$ as

$$\Delta\sigma'_{av} = \frac{\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b}{6} \tag{11.70}$$

where $\Delta\sigma'_t$, $\Delta\sigma'_m$, and $\Delta\sigma'_b$ represent the increase in the effective pressure at the top, middle, and bottom of the layer, respectively.

The swell index was expressed by Nagaraj and Murty (1985) as

$$C_s = 0.0463\left[\frac{LL(\%)}{100}\right]G_s \tag{11.41}$$

Based on the modified Cam clay model, Kulhawy and Mayne (1990) have shown that

$$C_c \approx \frac{PI}{370} \tag{11.42}$$

Typical values of the liquid limit, plastic limit, virgin compression index, and swell index for some natural soils are given in Table 11.7.

Table 11.7 Compression and Swell of Natural Soils

Soil	Liquid limit	Plastic limit	Compression index, C_c	Swell index, C_s
Boston blue clay	41	20	0.35	0.07
Chicago clay	60	20	0.4	0.07
Ft. Gordon clay, Georgia	51	26	0.12	—
New Orleans clay	80	25	0.3	0.05
Montana clay	60	28	0.21	0.05

Example 11.3

The following are the results of a laboratory consolidation test:

Pressure, σ' (kN/m^2)	Void ratio, e	Remarks	Pressure, σ' (kN/m^2)	Void ratio, e	Remarks
25	1.03	Loading	800	0.71	Loading
50	1.02		1600	0.62	
100	0.98		800	0.635	Unloading
200	0.91		400	0.655	
400	0.79		200	0.67	

a. Draw an e-log σ'_O graph and determine the preconsolidation pressure, σ'_c.
b. Calculate the compression index and the ratio of C_s/C_c.
c. On the basis of the average e-log σ' plot, calculate the void ratio at σ'_O = 1200 kN/m^2.

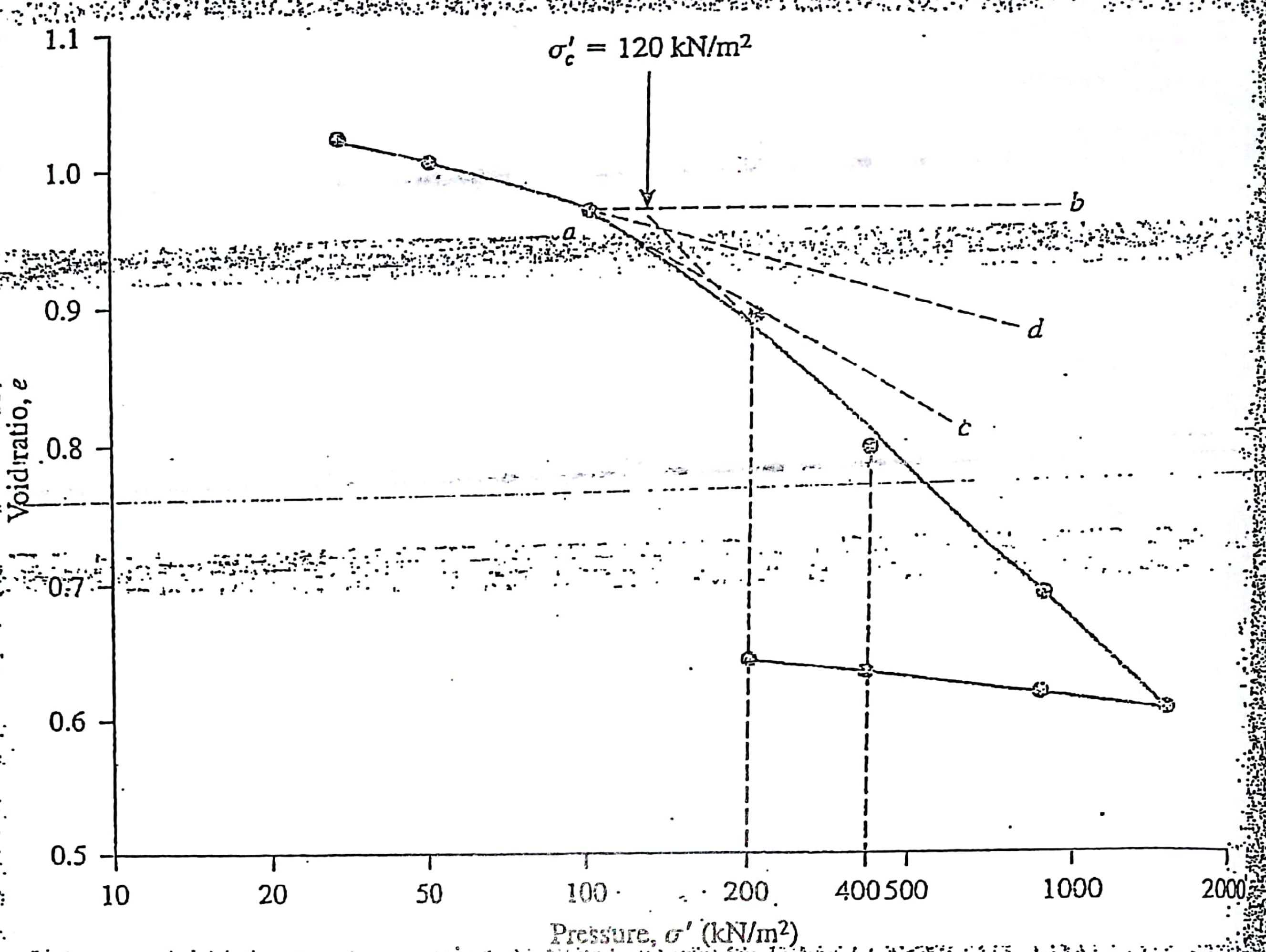


Figure 11.17 Plot of e versus log σ'

Solution

Part a

The e versus log σ' plot is shown in Figure 11.17. Casagrande's graphic procedure used to determine the preconsolidation pressure.

$$\sigma'_o = 120\ \text{kN/m}^2$$

Part b

From the average e-log σ' plot, for the loading and unloading branches, the following values can be determined.

Branch	e	σ'_o (kN/m²)
Loading	0.9	200
	0.8	400
Unloading	0.67	200
	0.655	400

From the loading branch,

$$C_c = \frac{e_1 - e_2}{\log \frac{\sigma'_2}{\sigma'_1}} = \frac{0.9 - 0.8}{\log\left(\frac{400}{200}\right)} = 0.33$$

From the unloading branch,

$$C_s = \frac{e_1 - e_2}{\log \frac{\sigma'_2}{\sigma'_1}} = \frac{0.67 - 0.655}{\log\left(\frac{400}{200}\right)} = 0.05$$

$$\frac{C_s}{C_c} = \frac{0.05}{0.33} = 0.15$$

Part c

$$C_c = \frac{e_1 - e_3}{\log \frac{\sigma'_3}{\sigma'_1}}$$

We know that $e_1 = 0.9$ at $\sigma'_1 = 200$ kN/m² and that $C_c = 0.33$ [part (b)]. Let $\sigma'_3 = 1200$ kN/m². So

$$0.33 = \frac{0.9 - e_3}{\log\left(\frac{1200}{200}\right)}$$

$$e_3 = 0.9 - 0.33 \log\left(\frac{1200}{200}\right) = 0.64$$

Example 11.4

A soil profile is shown in Figure 11.18. If a uniformly distributed load, $\Delta\sigma$, is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation if

a. The clay is normally consolidated
b. The preconsolidation pressure (σ'_c) = 190 kN/m²
c. $\sigma'_c = 170$ kN/m²

Use $C_s \approx \frac{1}{6} C_c$.

Solution

Part a

The average effective stress at the middle of the clay layer is

$$\sigma'_o = 2\gamma_{dry} + 4[\gamma_{sat(sand)} - \gamma_w] + \frac{4}{2}[\gamma_{sat(clay)} - \gamma_w]$$

$$\sigma'_o = (2)(14) + 4(18 - 9.81) + 2(19 - 9.81) = 79.14 \text{ kN/m}^2$$

From Eq. (11.31),

$$S_c = \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

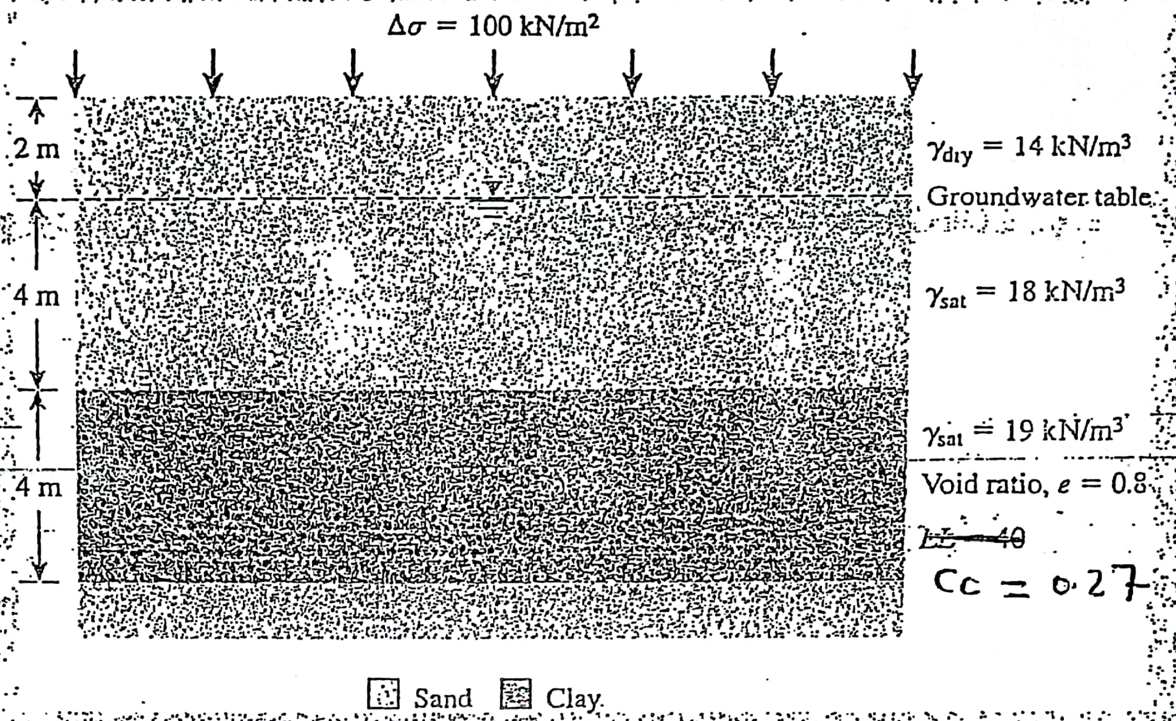


Figure 11.18

From Eq. (11.35),

$$C_c = 0.009(LL - 10) = 0.009(40 - 10) = 0.27$$

So,

$$S_c = \frac{(0.27)(4)}{1 + 0.8} \log\left(\frac{79.14 + 100}{79.14}\right) = 0.213 \text{ m} = \mathbf{213 \text{ mm}}$$

Part b

$$\sigma'_o + \Delta\sigma' = 79.14 + 100 = 179.14 \text{ kN/m}^2$$

$$\sigma'_c = 190 \text{ kN/m}^2$$

Because $\sigma'_o + \Delta\sigma' > \sigma'_c$, use Eq. (11.33):

$$S_c = \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

$$C_s = \frac{C_c}{6} = \frac{0.27}{6} = 0.045$$

$$S_c = \frac{(0.045)(4)}{1 + 0.8} \log\left(\frac{79.14 + 100}{79.14}\right) = 0.036 \text{ m} = \mathbf{36 \text{ mm}}$$

Part c

$$\sigma'_o = 79.14 \text{ kN/m}^2$$

$$\sigma'_o + \Delta\sigma' = 179.14 \text{ kN/m}^2$$

$$\sigma'_c = 170 \text{ kN/m}^2$$

Because $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'$, use Eq. 11.34:

$$S_c = \frac{C_s H}{1+e_o}\log\frac{\sigma'_c}{\sigma'_o} + \frac{C_c H}{1+e_o}\log\left(\frac{\sigma'_o+\Delta\sigma'}{\sigma'_c}\right)$$

$$= \frac{(0.045)(4)}{1.8}\log\left(\frac{170}{79.14}\right) + \frac{(0.27)(4)}{1.8}\log\left(\frac{179.14}{170}\right)$$

$$\approx 0.0468 \text{ m} = 46.8 \text{ mm}$$

Example 11.5

A soil profile is shown in Figure 11.19a. Laboratory consolidation tests were conducted on a specimen collected from the middle of the clay layer. The field consolidation curve interpolated from the laboratory test results is shown in Figure 11.19b. Calculate the settlement in the field caused by primary consolidation for a surcharge of 48 kN/m² applied at the ground surface.

Solution

$$\sigma'_O = (5)(\gamma_{sat} - \gamma_w) = 5(18.0 - 9.81)$$

$$= 40.95 \text{ kN/m}^2$$

$$e_O = 1.1$$

$$\Delta\sigma' = 48 \text{ kN/m}^2$$

$$\sigma'_O + \Delta\sigma' = 40.95 + 48 = 88.95 \text{ kN/m}^2$$

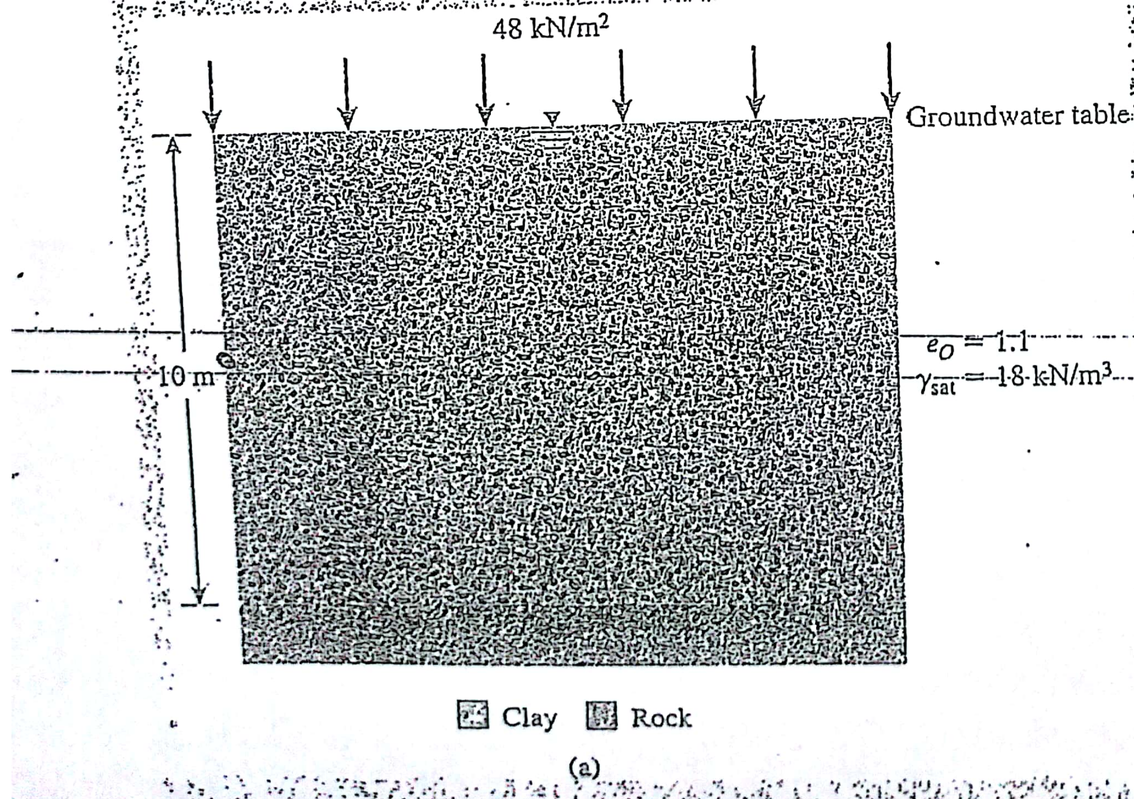
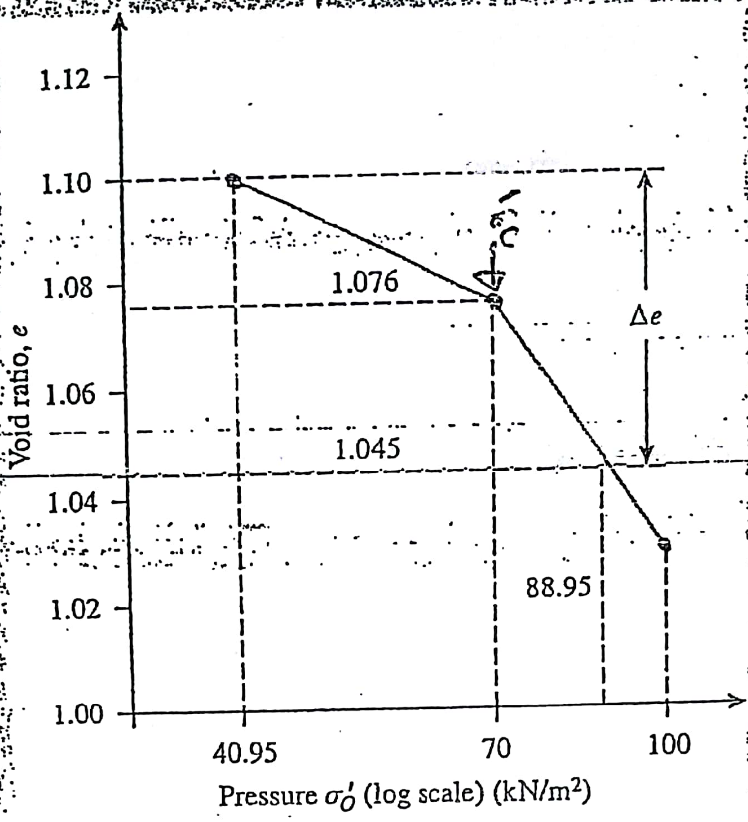


Figure 11.19 (a) Soil profile



(b)

Figure 11.19 (b) field consolidation curve

The void ratio corresponding to 88.95 kN/m^2 (see Figure 11.19b) is 1.045. Hence $\Delta e = 1.1 - 1.045 = 0.055$. We have

$$\text{Settlement } (S_c) = H\frac{\Delta e}{1+e_o} \qquad \text{[Eq. (11.2}$$

so,

$$S_c = 10\frac{(0.055)}{1+1.1} = 0.262 \text{ m} = 262 \text{ mm}$$

11.11 Secondary Consolidation Settlement

Section 11.4 showed that at the end of primary consolidation (that is, after com dissipation of excess pore water pressure) some settlement is observed because of plastic adjustment of soil fabrics. This stage of consolidation is called *secondary solidation.* During secondary consolidation the plot of deformation against the log time is practically linear (see Figure 11.8). The variation of the void ratio, e, with t for a given load increment will be similar to that shown in Figure 11.8. This varia is shown in Figure 11.20. From Figure 11.20, the secondary compression index ca defined as

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log(t_2/t_1)} \qquad (11$$

72

double drainge دائما عينة المختبر تكون

sand

2.5mm clay $t_{50} = 2$ min 20 sec

sand

Example 11.7

The time required for 50% consolidation of a 25-mm-thick clay layer (drained at both top and bottom) in the laboratory is 2 min. 20 sec. How long (in days) will it take for a 3-m-thick clay layer of the same clay in the field under the same pressure increment to reach 50% consolidation? In the field, there is a rock layer at the bottom of the clay. ~~terzaghi~~

Solution

$$T_{50} = \frac{c_v t_{\text{lab}}}{H^2_{\text{dr(lab)}}} = \frac{c_v t_{\text{field}}}{H^2_{\text{dr(field)}}}$$

or

$$\frac{t_{\text{lab}}}{H^2_{\text{dr(lab)}}} = \frac{t_{\text{field}}}{H^2_{\text{dr(field)}}}$$

$$\frac{140\,\text{sec}}{\left(\frac{0.025\,\text{m}}{2}\right)^2} = \frac{t_{\text{field}}}{(3\,\text{m})^2}$$

$$t_{\text{field}} = 8{,}064{,}000\,\text{sec} = 93.33\,\text{days}$$

Example 11.8

Refer to Example 11.7. How long (in days) will it take in the field for 30% primary consolidation to occur? Use Eq. (11.62).

Solution

From Eq. (11.62)

$$\frac{c_v t_{\text{field}}}{H^2_{\text{dr(lab)}}} = T_v \propto U^2$$

So

$$t \propto U^2$$

$$\frac{t_1}{t_2} = \frac{U_1^2}{U_2^2}$$

or

$$\frac{93.33\,\text{days}}{t_2} = \frac{50^2}{30^2}$$

$$t_2 = 33.6\,\text{days}$$

Example 11.9

A 3-m-thick layer (double drainage) of saturated clay under a surcharge loading underwent 90% primary consolidation in 75 days. Find the coefficient of consolidation of clay for the pressure range.

Solution

$$T_{90} = \frac{c_v t_{90}}{H_{dr}^2}$$

Because the clay layer has two-way drainage, $H_{dr} = 3\text{ m}/2 = 1.5\text{ m}$. Also, $T_{90} = 0.848$ (see Table 11.8). So,

$$0.848 = \frac{c_v(75 \times 24 \times 60 \times 60)}{(1.5 \times 100)^2}$$

$$c_v = \frac{0.848 \times 2.25 \times 10^4}{75 \times 24 \times 60 \times 60} = \mathbf{0.00294\ cm^2/sec}$$

Example 11.10

For a normally consolidated laboratory clay specimen drained on both sides, the following are given:

- $\sigma'_o = 150\text{ kN/m}^2$, $e = e_o = 1.1$
- $\sigma'_o + \Delta\sigma' = 300\text{ kN/m}^2$, $e = 0.9$
- Thickness of clay specimen = 25 mm
- Time for 50% consolidation = 2 min

a. Determine the hydraulic conductivity (min) of the clay for the loading range.

b. How long (in days) will it take for a 1.8 m clay layer in the field (drained on one side) to reach 60% consolidation?

c) Find the primary consolidation settlement

d) Find the excess pore water pressure after 42.06 days

e) what is U_z after 30 days

f) Find the settlement after 30 days

$$\boxed{U_z = \frac{S_t}{S_c}}$$

g) Find K

Solution

Part a

$S_c = \frac{1.1 - 0.9}{1 + 0.9} \times 1.8$

$u_z = [1 - U_z] \times \Delta\sigma'_z = 0.4 \times 150\text{ kN/m}^2$

The coefficient of compressibility is

$$m_v = \frac{a_v}{1 + e_{av}} = \frac{\left(\frac{\Delta e}{\Delta\sigma'}\right)}{1 + e_{av}} \quad \boxed{\sigma'_f - \sigma'_o}$$

$$\Delta e = 1.1 - 0.9 = 0.2$$

$$\Delta\sigma' = 300 - 150 = 150\text{ kN/m}^2$$

$$e_{av} = \frac{1.1 + 0.9}{2} = 1.0$$

So

$$m_v = \frac{\frac{0.2}{150}}{1 + 1.0} = 6.35 \times 10^{-4} \text{ m}^2/\text{kN}$$

From Table 11.8, for $U = 50\%$, $T_v = 0.197$; thus,

$$c_v = \frac{(0.197)\left(\frac{25}{2 \times 1000}\right)^2}{2} = 1.53 \times 10^{-5} \text{ m}^2/\text{min}$$

$$k = c_v m_v \gamma_w = (1.53 \times 10^{-5})(6.35 \times 10^{-4})(9.81)$$

$$= 95.3 \times 10^{-9} \text{ m/min} = 95.3 \times 10^{-7} \text{ cm/min}$$

Part b

$$T_{60} = \frac{c_v t_{60}}{H_{\text{dr}}^2}$$

$$t_{60} = \frac{T_{60} H_{\text{dr}}^2}{c_v}$$

From Table 11.8, for $U = 60\%$, $T_v = 0.286$,

$$t_{60} = \frac{(0.286)(1.8)^2}{1.53 \times 10^{-5}} = 60,565 \text{ min} = \mathbf{42.06 \text{ days}}$$

11.13 Coefficient of Consolidation

The coefficient of consolidation c_v generally decreases as the liquid limit of soil increases. The range of variation of c_v for a given liquid limit of soil is wide.

For a given load increment on a specimen, two graphical methods commonly are used for determining c_v from laboratory one-dimensional consolidation tests. The first is the *logarithm-of-time method* proposed by Casagrande and Fadum (1940), and the other is the *square-root-of-time method* given by Taylor (1942). More recently, at least two other meth-

Subject لمش حفظ!! Date

AB C [illegible]*

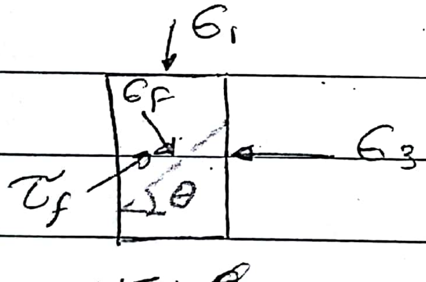
$$\sin\phi = \frac{\frac{\sigma_1-\sigma_3}{2}}{C*\cot\phi + \frac{\sigma_1+\sigma_3}{2}} = \frac{\sigma_1-\sigma_3}{2C*\cot\phi+\sigma_1+\sigma_3}$$

$$\sigma_1 = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)\sigma_3 + 2C\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

$$\sigma_3 = \left(\frac{1-\sin\phi}{1+\sin\phi}\right)\sigma_1 - 2C\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

$$\Rightarrow \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45-\frac{\phi}{2}\right)$$

$$\rightarrow \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45+\frac{\phi}{2}\right)$$



$45+\frac{\phi}{2}$

$$\tau_f = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta$$

$$\sigma_f = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\theta$$

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

Subject Date No.

– Total stress

$\sigma, C, \underbrace{\phi}_{???}, \tau$ → هدول بدنا نجيبهم

– Effective stress

$\sigma', \underbrace{C', \phi'}_{???}, \tau_f$

$\sigma' = \sigma - U_f$

* How to get shear strength parameters :-

Two wons :-

1) using lab tests :-

1– Direct shear test ✓ راح نشرح هدول

2– Tria axial Test ✓

3– plane strain Test.

4– True Trai axial Test.

2) using in site Test (Field) مش راح نوخذهم هون بس بالفاونديشن.

– SPT

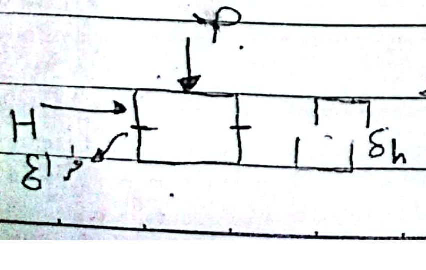
– CPT

– UST

– PMT

Direct shear test :-

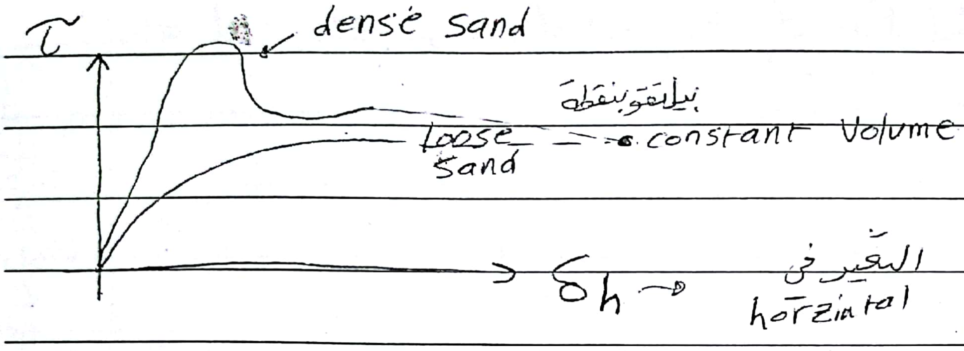
– used for sand and Gravel.

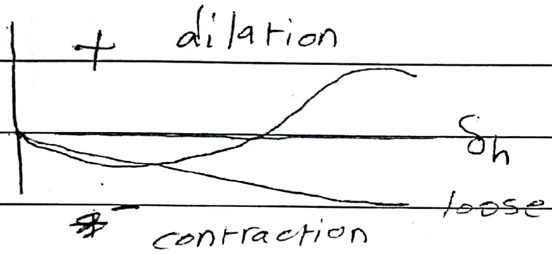


1) $\boxed{P_1, \sigma_{n1}}$ H, τ_1

2) P_2, σ_{n2} H, τ_2

3) P_3, σ_{n3} H, τ_3

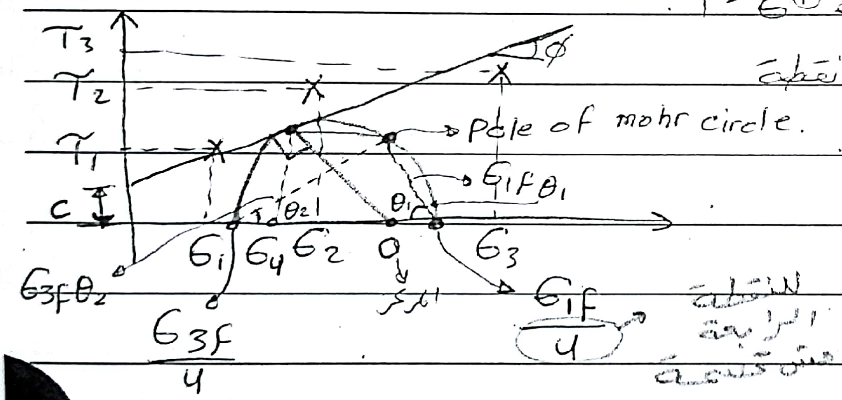




* لو كانت effective نفس الزاوية بس بدل σ1 ① - شحطات فوق كل القيم .

* بعكس السؤال انو σ4 هي نقطة

الـ faliuar او جد الزاوية .



Direct shear

Ex :- The following results are from D.S test conducted on clean sand.

	σ_n (kpa)	τ_{max} (kpa)
1)	50	25
2)	75	32.5
3)	90	41

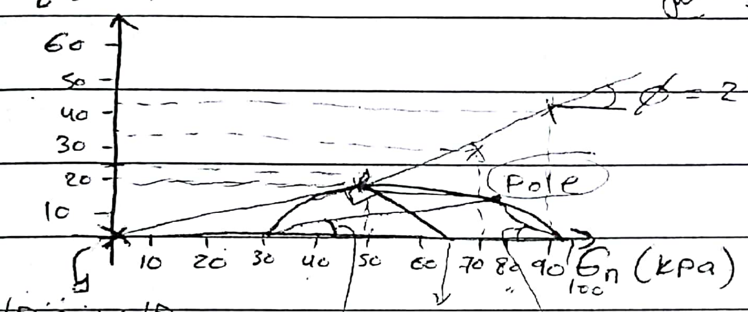
① Determine shear strength parameters? c, ϕ

2) For sample ~~#2~~ 1 Find σ_1 and σ_3 at faliar and Thier arientation ؟

Sol :-

① — Clean sand , C = 0

بالنقطة او من طريق الميل $\tan^{-1}\left(\frac{41-25}{90-50}\right) = 21.8$



شوف الدكتورة دائرة كيف او عدي

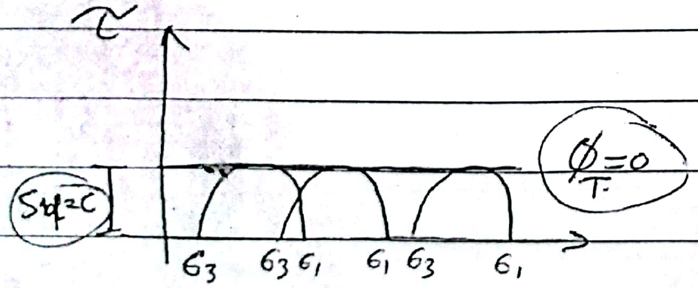
من نقطة الـ failure احط خط مستقيم لحتى يعطي (pole)

$\sigma_{3_f} = 30$ kPa

$\sigma_{1_f} = 95$ kPa

القبل كنا السابق → UU Test :- Q Test

Sample	confining pressure	$\Delta\sigma$	σ_1
1	$(\sigma_3)_1$	$(\Delta\sigma)_1$	$(\sigma_3+\Delta\sigma)_1$
2	$(\sigma_3)_2$	$(\Delta\sigma)_2$	$(\sigma_3+\Delta\sigma)_2$
3	$(\sigma_3)_3$	$(\Delta\sigma)_3$	$(\sigma_3+\Delta\sigma)_3$



$S_u = C$ = ~~Und. Shear~~
→ undrained shear strength

liquifaction ⟵ $\bar{\sigma} = \sigma_t - \Delta\gamma$

rate of Loading < cv

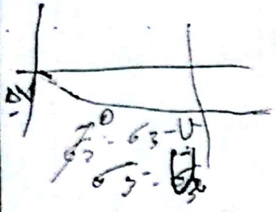
failure عشان نقطع الحي قبل ال

coefficient of Consolidation

عشان هيك هو أصعب أنواع Test

عملنا تشبع بـ Cn بالك

CD – S Test

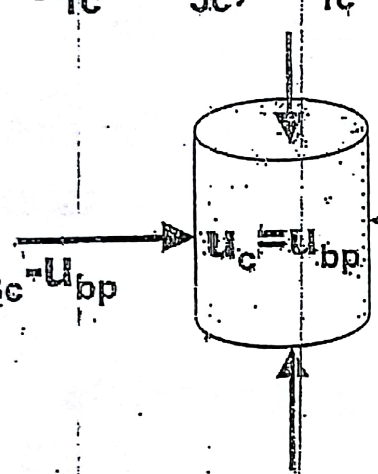


$\sigma_3^0 \cdot \sigma_3 - U$

$\sigma_3 \cdot \sigma_3$

$\sigma_{1c} = \sigma_{3c};\ \sigma'_{1c} = \sigma'_{3c}$

$\sigma_{3c};$
$\sigma'_{3c} = \sigma_{3c} - u_{bp}$



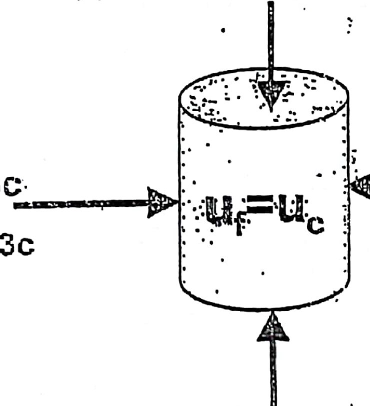
$\sigma_{3c};$
$\sigma'_{3c} = \sigma_{3c} - u_{bp}$

$\sigma_{1c} = \sigma_{3c};\ \sigma'_{1c} = \sigma'_{3c}$

Stage 1 Confinement Stage

$\sigma'_{1c} = \sigma'_{3c} + \Delta\sigma_d$

$\sigma_{3c} = \sigma_{3c}$
$\sigma'_{3c} = \sigma'_{3c}$



$\sigma_{3c} = \sigma_{3c}$
$\sigma'_{3c} = \sigma'_{3c}$

$\sigma'_{1c} = \sigma'_{3c} + \Delta\sigma_d$

Stage 2 Shear Stage

Coarse soil ⟶ $\bar{\sigma}$ مباشرة

fine soil ⟶ σ_t ① من بداية الانشاء حتى الانتهاء من الانشاء t_{90}

② $\bar{\sigma}$ ⟹ بعد الانتهاء من البناء t_{90} — نصمم على الرقم الأعلى

Elastic settlement (reduction in volume) ...

Con. & drained except e(cath cyclic case.

slow - test

Types of Tests

3. CD or S Test

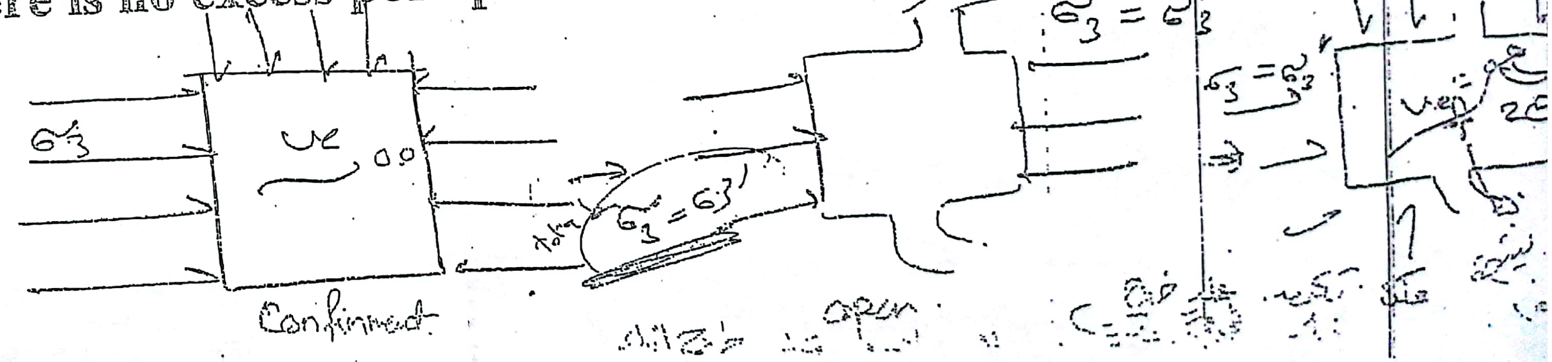
Effective stress

❑ Consolidated-Drained (CD) test, also called slow test → effective stress
(abbreviated CD or S) → Slow

❑ In this test, the drain valve is opened and is left open for the duration of the test, with complete sample drainage prior to application of the vertical load.

❑ The load is applied at such a slow strain rate that particle readjustments in the specimen do not induce any excess pore pressure. u = 0.0

❑ Since there is no excess pore pressure total stresses will equal effective stresses.



Total stress هنا أصعب

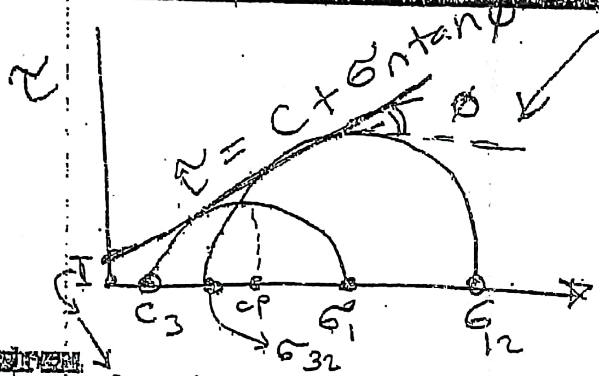
تتجاهلوا الا ... مع ال

M-C for CU Test

calculated / measure in terms of total stress

x علينا ... c' و ϕ'

Mohr's Circle.



tangent ال of Both Circles يمثل ال envelope Line.

more practical

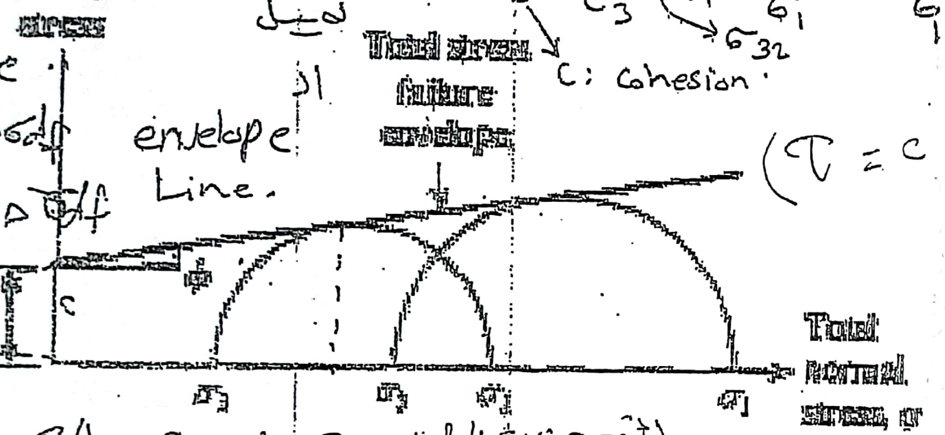
نوكانت sand لعرضه

Calculates NOT measures

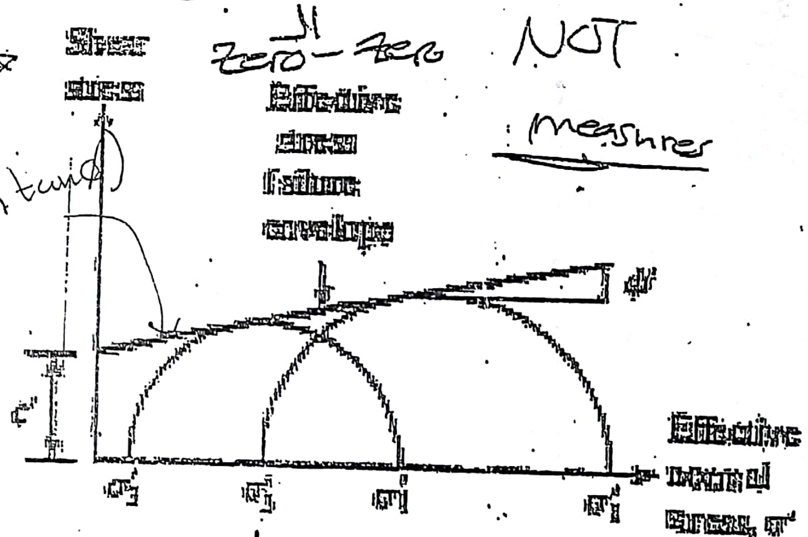
$\bar{\sigma}_3 = \sigma_3 - \Delta u_f$

$\bar{\sigma}_1 = \bar{\sigma}_3 + \Delta\sigma_{df}$

$\sigma_1 = \sigma_3 + \Delta\sigma_{df}$



$(\tau = c + \sigma_n \tan\phi)$



ال zero - zero

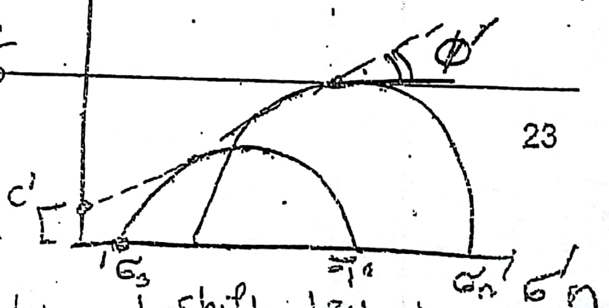
Consolidated-undrained test

Consolidated (CU test) ← ...

$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$ .... simple

لحساب $\sigma_3' = \sigma_3 - \Delta u$

$\sigma_1' = \sigma_1 - \Delta u$



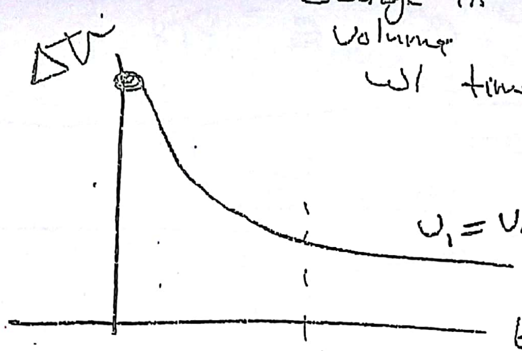
measured ال ... له ملاك ثم نوجد المحور ال

more pressure on soil ⇒ takes water away.

23

CU – R Test

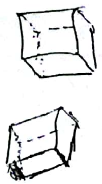
Change in Volume w/ time.

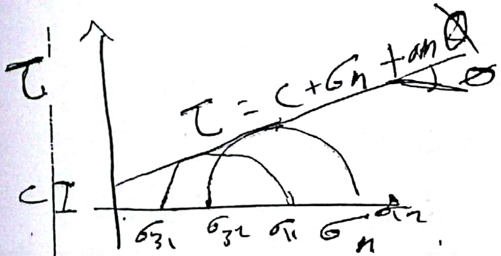


becomes V = Constant

axial gauge steps

excess pore water pressure steps.

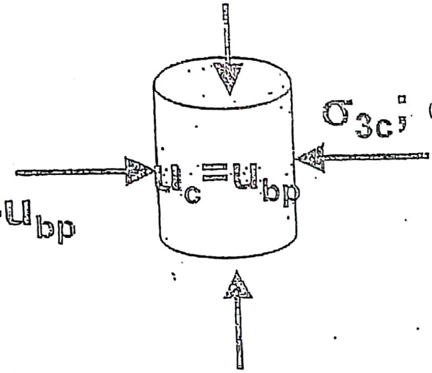




$\bar{\sigma}_1 = \sigma_1 - \Delta U_c$

$\bar{\sigma}_3 = \sigma_3 - \Delta U_c$

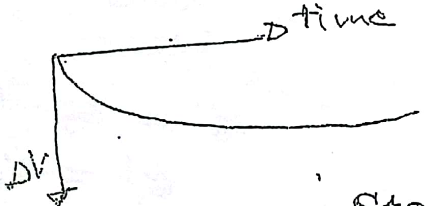
$\sigma_{1c} = \sigma_{3c}$; $\sigma'_{1c} = \sigma'_{3c}$



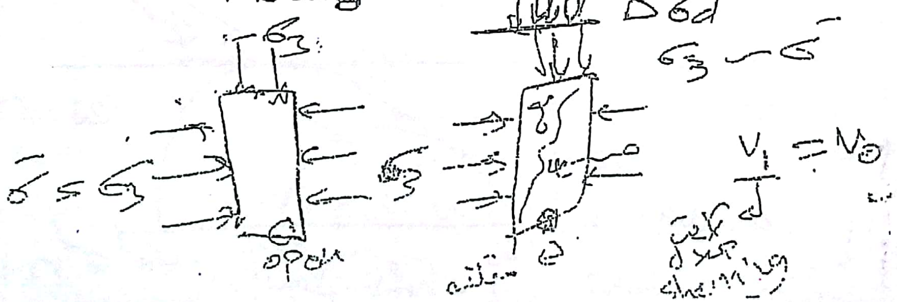
σ_{3c}; $\sigma'_{3c} = \sigma_{3c} - u_{bp}$

σ_{3c}; $\sigma'_{3c} = \sigma_{3c} - u_{bp}$

$\sigma_{1c} = \sigma_{3c}$;
$\sigma'_{1c} = \sigma'_{3c}$



Stage 1 Confinement Stage

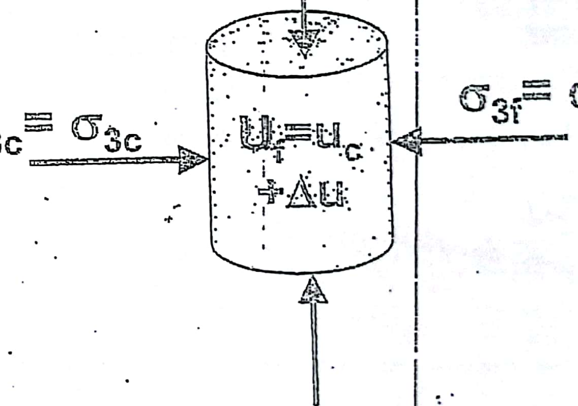


$V_1 = V_0 - \Delta V$

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

U_e load excess pore water

$\sigma_{1c} = \sigma_{3c} + \Delta\sigma_d$

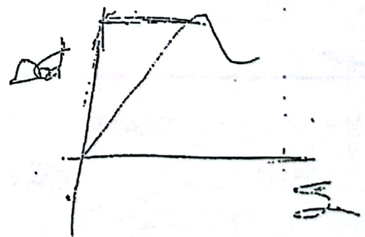


$\sigma_{3c} = \sigma_{3c}$

$\sigma_{3f} = \sigma_{3c}$

$\sigma_{1c} = \sigma_{3c} + \Delta\sigma_d$

Stage 2 Shear Stage



محار mass نصير Force مما يكون قطع نصير

Sn shear strength ال مباشرة

Dessipation

Types of Tests

unconso. & undrained

الاستثناء الحالات الأولى

اذا تعرض ال sand لـ خلال فترة زمنية قليلة

CU or R Test

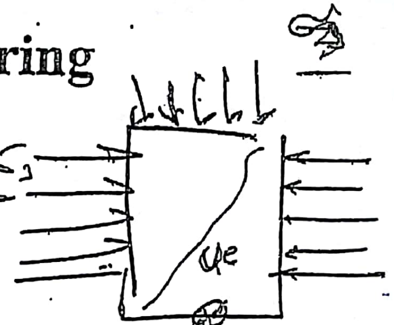
: In term of total stress

Hche =P

■ Consolidated-Undrained (CU) test, also termed consolidated-quick test or R test (abbreviated CU or R).

لأنه جاي بين ال slow وال Q → R

□ In this test, drainage or consolidation is allowed to take place during the application of the confining pressure σ3.



□ Loading does not commence until the sample ceases to drain (or consolidate).

□The axial load is then applied to the specimen, with no attempt made to control the formation of excess pore pressure.

open measures the change in volume

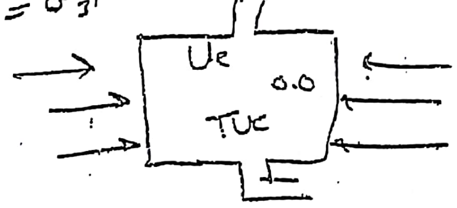
$\sigma_3 = \sigma'_3$

□ For this test, the drain valve is closed during axial loading, and excess pore pressures can be measured.

Structure ال خلال بناء ال في مرحلة البناء

unco. & undrai.

$\sigma_3 = \sigma'_3$

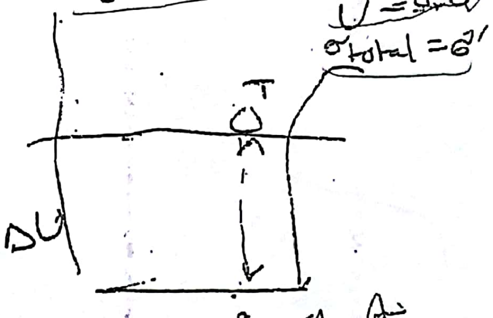


Con finement

$D_0 = 38$ mm

$L_0 = 76$ mm

$\sigma'_3 = \sigma_3$



$\sigma_{total} = \sigma'$

Constant

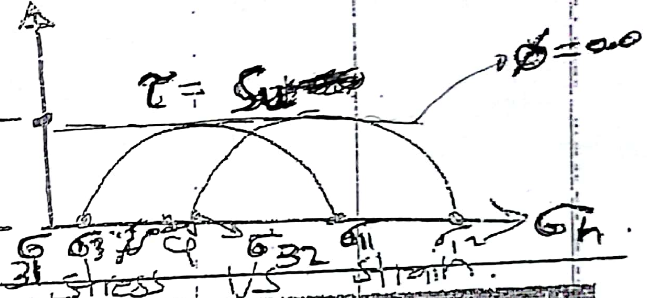
Mohr-Coulomb of UU

$\phi = 0$ fully saturated so No Friction

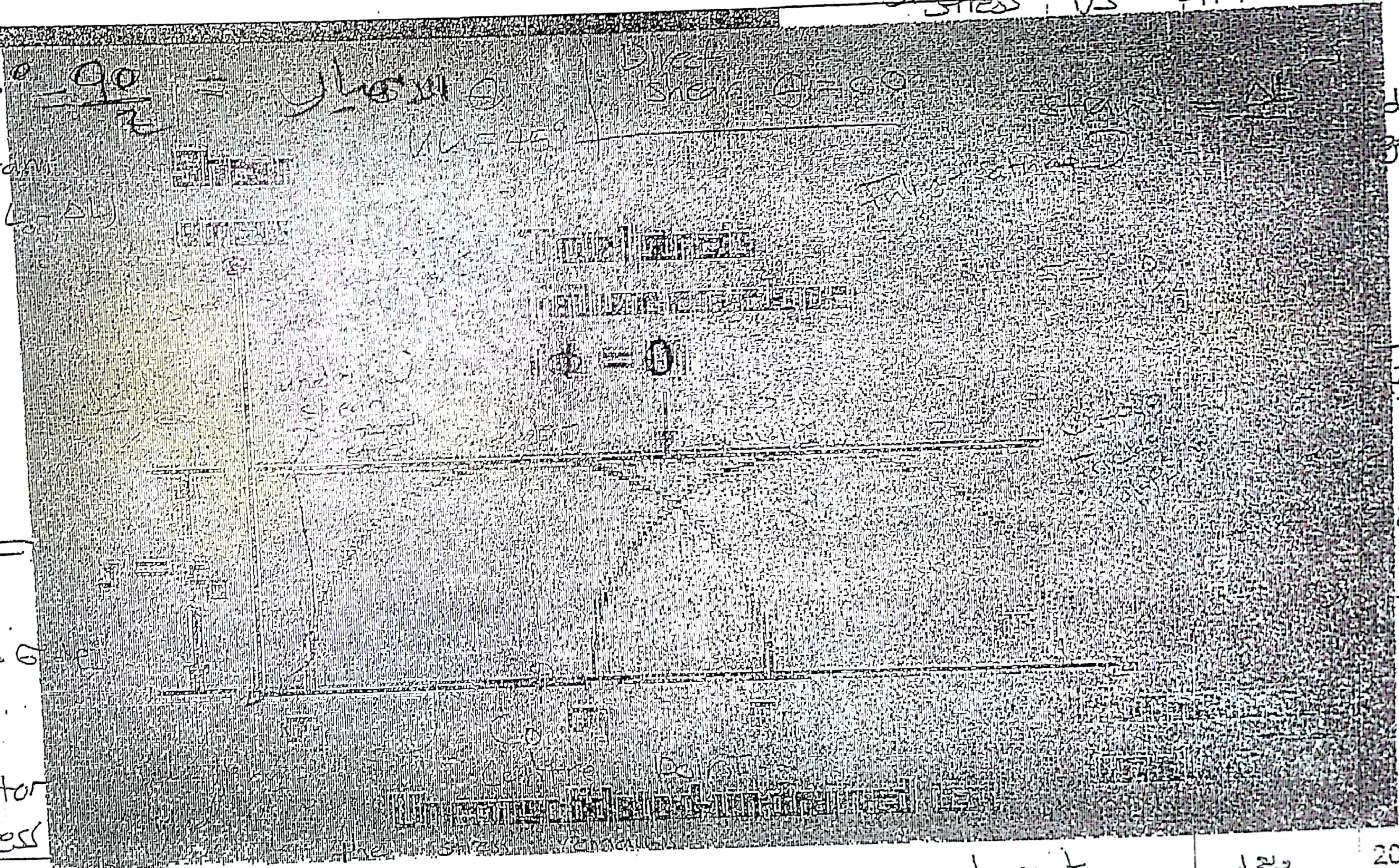
Sand doesn't have cohesion

$\Delta\sigma_d$ (Deviator stress)

S_u: Undrained shear strength parameter S_u



$45° - \frac{90}{2}$ = الانهيار



volume → Constant

$A_o L_o = A_c * (L_o - \Delta L)$

$A_c = \frac{A_o L_o}{L_o - \Delta L}$

$= \frac{A_o}{1 - \epsilon}$

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

$\Delta\sigma_d \neq$

Diviator stress

Constant

tangent

envelope line

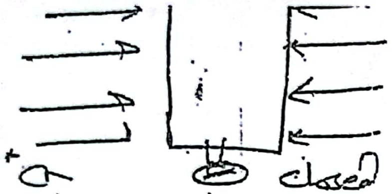
$\phi = 0.0$

لأنه ال water تمنع احتكاك friction Angle

C & ϕ

ال dial gauge

20 σ_1



$\Rightarrow$ No drainge path.

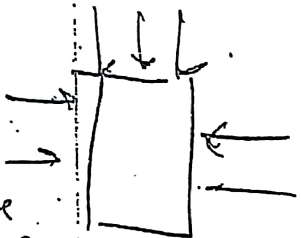
UU – Q Test

σ_1 major principle stress

σ_3 major

cylinderical.

sand
$D = 3.8$ cm
$L_o = 7.6$ cm
test
sand
sand test
consolidation
~~consolidatio~~
sattelment
consolidation
adually
decreased
and
Pressure



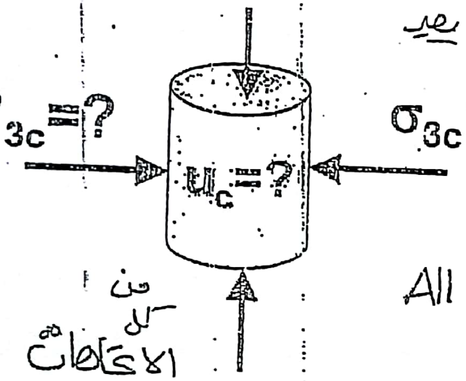
$\Delta\sigma_z$

deflection ال

Stage No. 2

$\sigma_{1c} = \sigma_{3c}$

$\Delta\sigma_z$ failure



All drainage Paths are ommetted

$\sigma_{1c} = \sigma_{3c}$

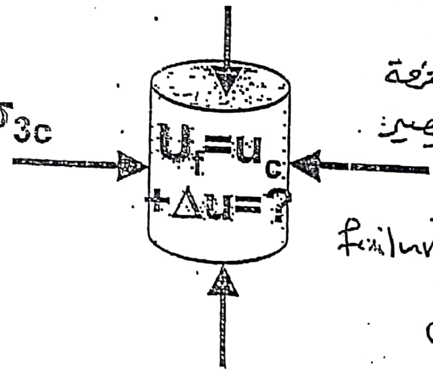
unconsolidaded.

Stage 1 Confinement Stage (σ_3)

Check the 1st page
Has a photo

$\boxed{\text{volume remains constant}}$

$\sigma_{1c} = \sigma_{3c} + \Delta\sigma_d$



ال $\sigma_{3f} = \sigma_{3c}$ failure.

$\sigma_{1c} = \sigma_{3c} + \Delta\sigma_d$ (σ_f)

deformation ال using a dial ~~guage~~ gauge

Stage 2 Shear Stage

Load ل

$\boxed{\sigma_3}$

$\Delta\sigma_d = \frac{P}{A}$

$A_c \Rightarrow A_o L_o = A_c [L_o - \Delta L)$

Types of Tests

There are 3 types of tests

1. UU Quick Q Test

In term of total stress.

Looks like of an "Everyday Test".

- Unconsolidated-Undrained (UU) test which is also called the quick test (abbreviations commonly used are UU and Q test).

1st stage confinement — 2nd stage sheared — as a total stress

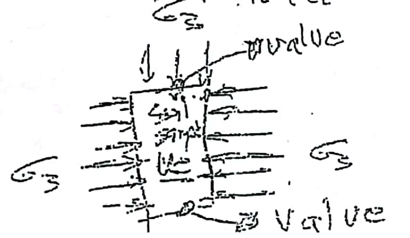
⇒ Total

- This test is performed with the drain valve closed for all phases of the test.

Total stress NOT the effective stress

- Axial loading is commenced immediately after the chamber pressure σ3 is stabilized.

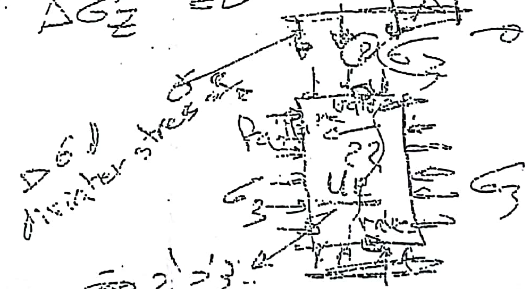
unconsolidated



excess pore water pressure

load

$u_e = \left(1 - \dfrac{u_p}{\cdot}\right) \Delta\sigma_z$ ⇒ confinement



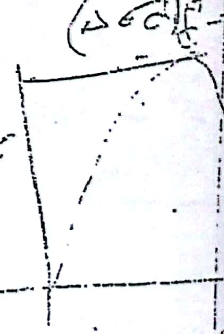
$\sigma_{total} = \dfrac{P}{A_{corrected}}$

$A_c L_o = A_c [L_o - \Delta L]$ ⇒ $\dfrac{A_o}{1-\varepsilon}$

$A_c = \dfrac{A_o L_o}{L_o - \Delta L}$

σ_3 ⇒ minor principle stress

σ_1 ⇒ major principle



Comparison Of Triaxial with Direct Shear Test

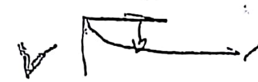
$\Delta V \rightarrow$ constant
total stress = effective stress
No consolidation
drainage
pore water $\rightarrow \Delta V$ constant $\rightarrow$ يعني ماكو

The advantages of the triaxial test over the direct shear test are:

- Progressive effects are less in the triaxial.

- The measurement of specimen volume changes are more accurate in the triaxial. consolidation

dial gage



- The complete state of stress is assumed to be known at all stages during the triaxial test, whereas only the stresses at failure are known in the direct shear test. $\rightarrow$ ~~I~~

  I can know principle stresses (σ_1, σ_3) at any point in a test.

- The triaxial machine is more adaptable to special requirements.

shear stress

case

C_v, C_c

consolidation test.

Final

(2)

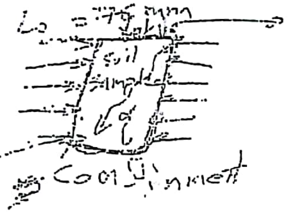
(11)

has a previous Complement ... =))

Triaxial Tester

عشان تقدر أن أعمل الـ stress غير متساوي

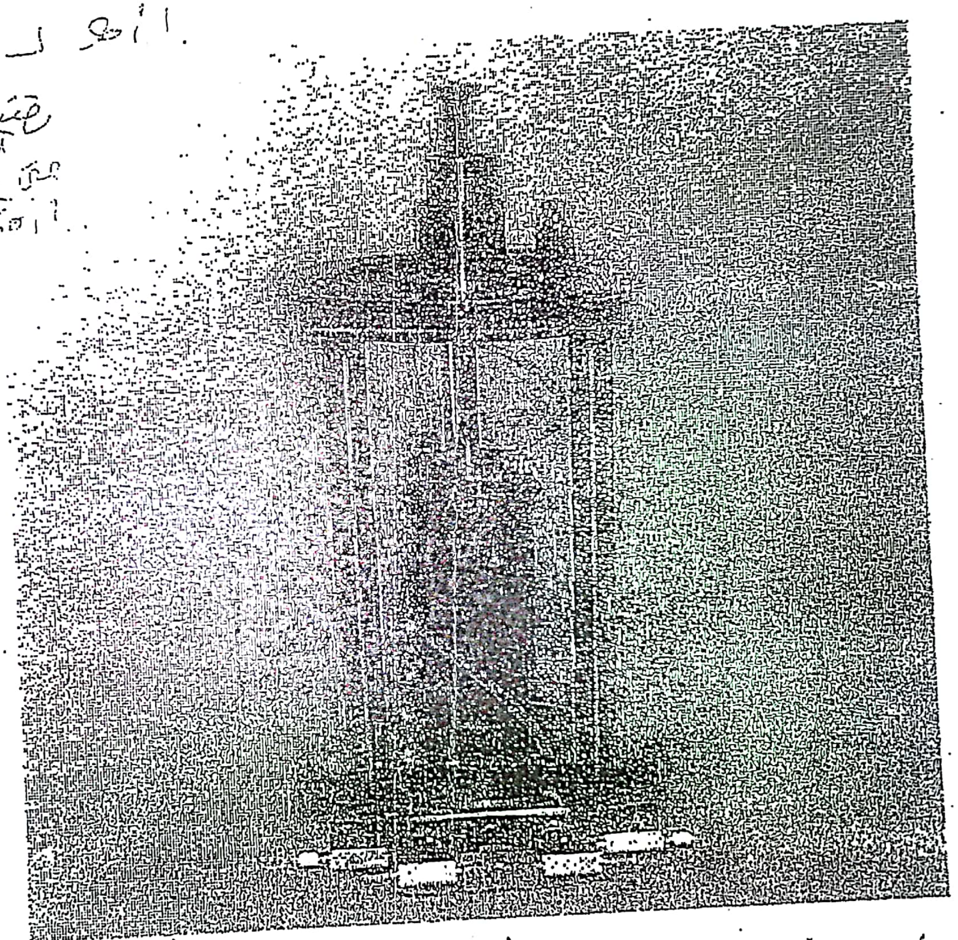
$D_o = 38mm$
$L_o = 76mm$



membrane

confinement

load



Triaxial Test on Soil Sample in Laboratory

بتعرض السويل

لـ normal load

بعدين بحط

as a shear load.

Direct shear

Subject Date No.

CU – R Test :- مهم جداً

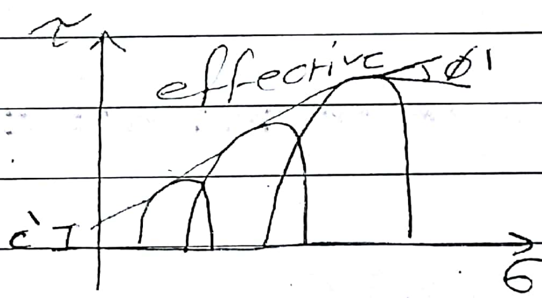
u_{bp} = back pressure

Sample	confining pressure	$u_c = u_{bp}$	σ_3'	$\sigma_1 = \sigma_3 + \Delta$
1	$(\sigma_3)_1$	u_c	$\sigma_3 - u_c$	
2	$(\sigma_3)_2$	u_c	$\sigma_3 - u_c$	
3	$(\sigma_3)_3$	u_c	$\sigma_3 - u_c$	نفس القيم

مع العناوين

$\Rightarrow \sigma_1' = \sigma_1 - u_T$

$\Rightarrow u_T = u_c + \Delta u$ من Load



$\phi' > \phi_T$

$c_T > c'$

CD – S Test :-

$\sigma_{3_T} = \sigma_3'$ $\sigma_{1_T} = \sigma_1'$

U	CU	CD
لا يوجد	$\phi' <$	ϕ'
$c >$	$c >$	c'

N O T E B O O K

Subject Date ٢٠ جمعة ١١ No.

Ex :- consolidated Undrained (CU) shear strength test with pore water pressure measrment was performed on normally consolidted clay sample. If the consolidrion pressure was 200 kpa and deviater الفرق stress at failure was 120 kpa and pore water pressure at failure 100 kpa

A) Find the following:

① Undrained shear strength ?

② Undrained shear strength parameters ?

③ effectife drained angle of friction ?

④ Effective major principal stress at failure ?

B) If an identical soil specimen was consolidated under 100 kpa and shemed under drained condition Find: CD

① Drained shear strength parameters ?

② Effective ~~major~~ Normal stress at failure ?

③ angle of failure plane ?

④ effective Cohession ?

Sol :- (A)

$\sigma_3 = 200$ kpa

$\Delta\sigma = \sigma_d = 120$ kapa

$u_f = 100$ kpa

① $\sigma_3 = 200$ kpa

$\sigma_1 = \sigma_3 + \Delta\sigma$

$= 200 + 120$

$= 320$ kpa

$$S_u = \frac{\sigma_1 - \sigma_3}{2} \quad \frac{\Delta\sigma}{2}$$

$$= \frac{320 - 200}{2} = 60 \text{ kpa}$$

2) C_T, ϕ_T

$C = C' = 0 \Rightarrow$ Normally consolidated clay

$$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 + 2C\cancel{*\cot\phi}}$$

13.34

$$= \frac{320 - 200}{320 + 200} \Rightarrow \phi = \sin^{-1}\left(\frac{120}{520}\right) = 12.9 = 13^\circ$$

3) $\sigma'_3 = \sigma_3 - u = 200 - 100 = 100$ kpa

$\sigma'_1 = \sigma_1 - u = 320 - 100 = 220$ kpa

$C' = 0$

$$\Rightarrow \sin\phi = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3 + 2C'\cancel{*\cot\phi}}$$

$$\phi' = \sin^{-1}\left(\frac{220 - 100}{220 + 100}\right) = 20.55^\circ$$

4) $\sigma'_1 = 220$ kpa

B) $\sigma'_3 = \sigma_3 - \sigma'_3 = 400$ kpa

1) $C' = 0 \quad \phi = 20.55^\circ \longrightarrow$ لأن لنفس العينة

2) $$\sigma'_f = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma'_1 - \sigma'_3}{2} \cos 2\theta$$

$$\theta = 45 + \frac{\phi}{2} = 45 + \frac{20.55}{2} = 55.275^\circ$$

Subject Date No.

$$\sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c \tan\left(45 + \frac{\phi'}{2}\right)$$

$$\sigma'_1 = 400 \tan^2(55.275) + 0 = 832.713 \text{ kpa}$$

$$\sigma'_f = \frac{832.713 + 400}{2} + \frac{832.713 - 400}{2} \cos(2 * 55.225)$$

$$= 540.4 \text{ kpa}$$

3) $\theta = 55.275$

4) $c' = 0$

(5) shear stress at failure ? sol

$$\tau_f = \cancel{c'_f} + \sigma'_f \tan \phi'$$

$$\tau_f = \sigma'_f \tan \phi' = 540.4 \tan(20.55) = 202.58 \text{ kpa}$$

or

$$\tau = \frac{\sigma'_1 - \sigma'_3}{2} \sin 2\theta$$

$$= \frac{832.713 - 400}{2} \sin(2 * 55.275)$$

$$= 202.54 \text{ kpa}$$

أسئلة كتاب

$\tau = \sigma_n \tan\phi$

$\sigma_n = \frac{P}{A}$

Solution :))

① σ_n : Angle of shearing resistence ϕ.

$\frac{\sigma_1+\sigma_3}{2}$

$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$

$= \frac{[420 + 350] - 420}{[420 + 350] + 420}$

$\phi = 17.1°$

$\sin\phi' = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}$

$\sigma_1' = \sigma_1 - \Delta u$

$= [770 - 289.45] - [420 - 289.45]$

$[770 - 289.45] + [420 - 289.45]$

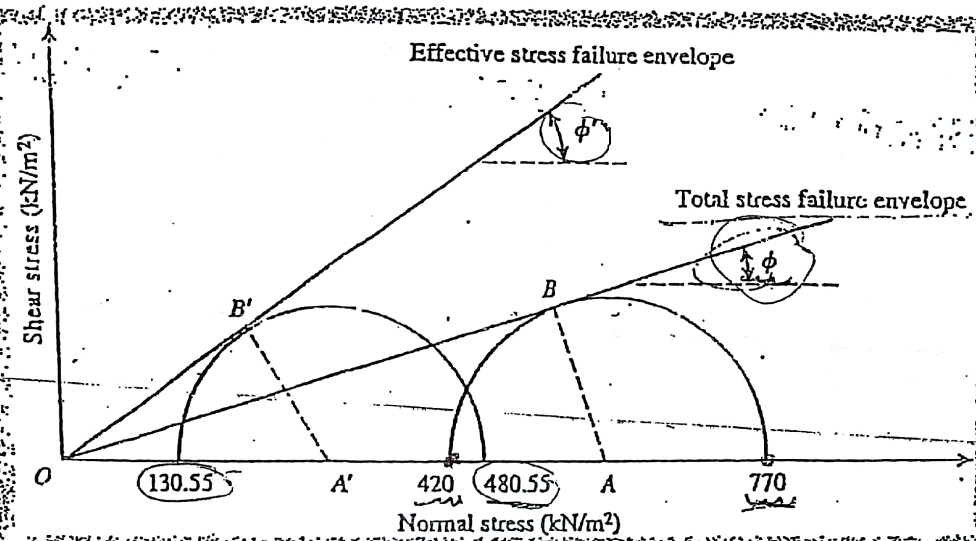


Figure 7.19

Solution

Part a

At failure, $\sigma_3 = 420\ \text{kN/m}^2$

$$\sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 420 + 350 = 770\ \text{kN/m}^2$$

From Figure 7.19,

$$\sin\phi = \frac{AB}{OA} = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \frac{770 - 420}{770 + 420} = \frac{350}{1190} = 0.294$$

or

$$\phi = 17.1°$$

Part b

$$\sigma_3' = \sigma_3 - (\Delta u_d)_f = 420 - 289.45 = 130.55\ \text{kN/m}^2$$

$$\sigma_1' = \sigma_1 - (\Delta u_d)_f = 770 - 289.45 = 480.55\ \text{kN/m}^2$$

$$\sin\phi' = \frac{A'B'}{OA'} = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'} = \frac{480.55 - 130.55}{480.55 + 130.55} = \frac{350}{611.1} = 0.5727$$

or

$$\phi' = 34.94°$$

Examples on shear strength

22/11/2011

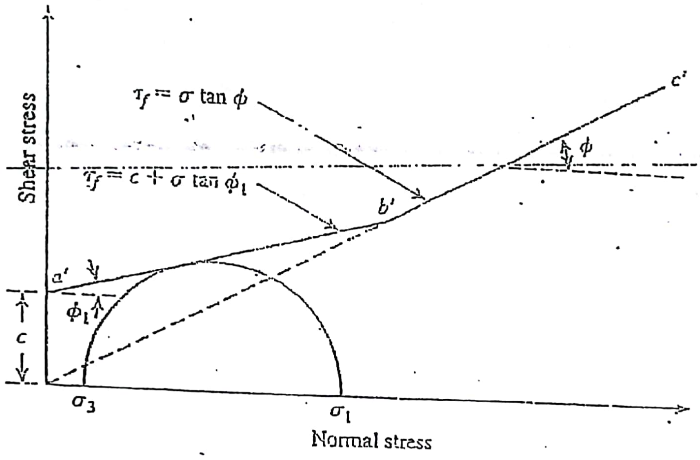


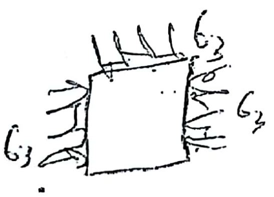
Figure 7.18 Total stress failure envelope obtained from consolidated-undrained tests in over-consolidated clay

and the straight line $b'c'$ follows the relationship given by Eq. (7.16). The effective stress failure envelope drawn from the effective stress Mohr's circles will be similar to that shown in Figure 7.18.

Consolidated-drained tests on clay soils take considerable time. For this reason, consolidated-undrained tests can be conducted on such soils with pore pressure measurements to obtain the drained shear strength parameters. Because drainage is not allowed in these tests during the application of deviator stress, they can be performed quickly.

Skempton's pore water pressure parameter A has been defined in Eq. (7.15). At failure, the parameter A can be written as

$$A = A_f = \frac{(\Delta u_d)_f}{(\Delta \sigma_d)_f} \tag{7.18}$$



The general range of A_f values in most clay soils is as follows:

- Normally consolidated clays: 0.5 to 1
- Overconsolidated clays: -0.5 to 0

$\sigma_3' = \sigma_3 - \Delta u = 420 - 289.4 = 130.55$

Sand :. $c = 0$

$\sigma_1' = \sigma_1 - \Delta u$

$= 770 - 289.45$

$= 480.55$

Example 7.4

CU

A specimen of saturated sand was consolidated under an all-around pressure of 420 kN/m². The axial stress was then increased without permitting drainage. The specimen failed when the axial stress reached 350 kN/m². The pore water pressure at failure was 289.45 kN/m². Determine

a. the consolidated-undrained angle of shearing resistance;
b. the drained friction angle, ϕ'.

$\sigma_3 = 420$

$\sigma_1 = \sigma_3 + \Delta\sigma_d = 420 + 350 \Rightarrow 770$

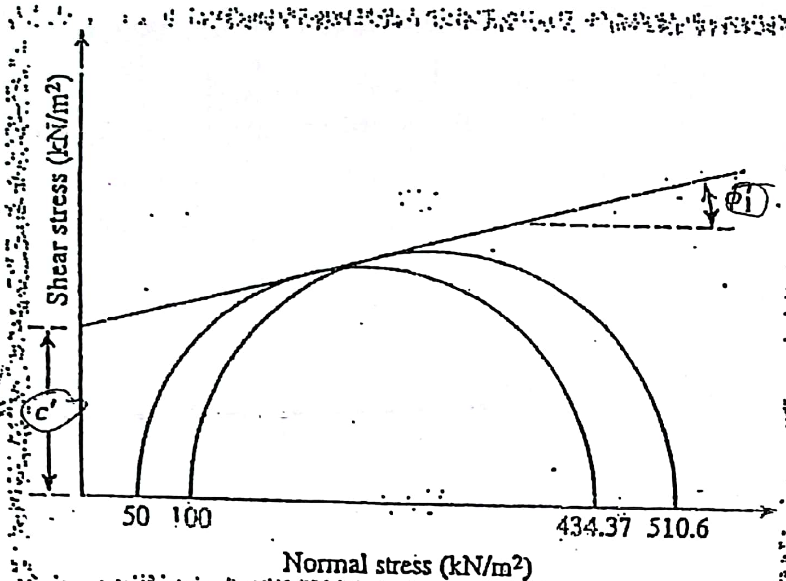


Figure 7.15

Solution

For Specimen 1, the principal stresses at failure are

$$\sigma_3' = \sigma_3 = 100 \text{kN/m}^2$$

and

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 100 + 410.6 = 510.6 \text{ kN/m}^2$$

Similarly, the principal stresses at failure for Specimen 2 are

$$\sigma_3' = \sigma_3 = 50 \text{ kN/m}^2$$

and

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 50 + 384.37$$
$$= 434.37 \text{ kN/m}^2$$

These two specimens are overconsolidated. Using the relationship given by Eq. (7.7)

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi_1'}{2}\right) + 2c' \tan\left(45 + \frac{\phi_1'}{2}\right)$$

Thus, for Specimen 1

$$510.6 = 100 \tan^2\left(45 + \frac{\phi_1'}{2}\right) + 2c' \tan\left(45 + \frac{\phi_1'}{2}\right) \qquad (7.14a) \quad \text{—①}$$

and for Specimen 2

$$434.37 = 50 \tan^2\left(45 + \frac{\phi_1'}{2}\right) + 2c' \tan\left(45 + \frac{\phi_1'}{2}\right) \qquad (7.14b) \quad \text{—②}$$

Mohr's Coulomb criteria Failure

let me Live that Fantacy

Example 7.2

For the triaxial test described in Example 7.1, Ayat

a. Determine the effective normal stress on the plane of maximum shear stress.

b. Explain why the shear failure occurred along a plane with $\theta = 54.73°$ and not along the plane of maximum shear stress.

Solution

Part a

From Eq. (5.22), we can see that the maximum shear stress will occur on the plane with $\theta = 45°$. From Eq. (5.21),

$$\sigma' = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma'_1 - \sigma'_3}{2}\cos 2\theta$$

Substituting $\theta = 45°$ into the preceding equation, we get

$$\sigma' = \frac{552 + 276}{2} + \frac{552 - 276}{2}\cos 90 = 414\ \text{kN/m}^2$$

Part b

safe I do NOT knows P

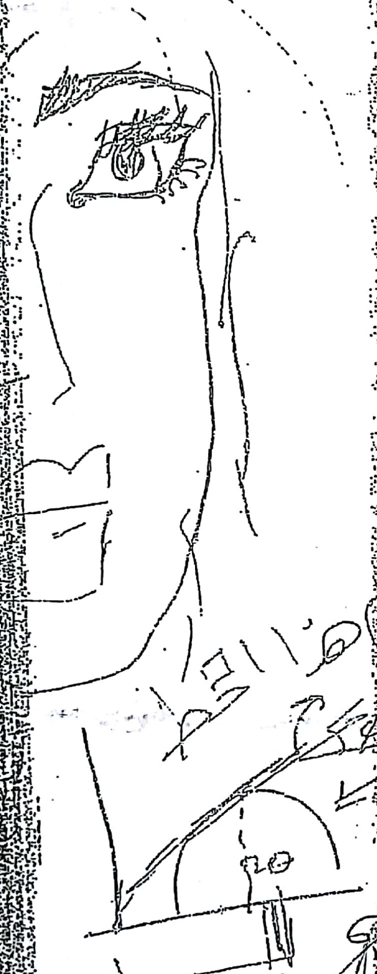
The shear stress that will cause failure along the plane with $\theta = 45°$ is

$$\tau_f = \sigma' \tan\phi' = 414 \tan(19.45) = 146.2\ \text{kN/m}^2$$

However, the shear stress induced on that plane is

$$\tau = \frac{\sigma'_1 - \sigma'_3}{2}\sin 2\theta = \frac{552 - 276}{2}\sin 90 = \boxed{138\ \text{kN/m}^2}$$

Because $\tau = 138\ \text{kN/m}^2 < 146.2\ \text{kN/m}^2 = \tau_f$, the specimen did not fail along the plane of maximum shear stress. ■



$C + \sigma \tan\phi'$ τ_{max} = Radius safe

((Here))! Be careful

Example 7.3

Two similar clay soil specimens were preconsolidated in triaxial equipment under a chamber pressure of 600 kN/m². Consolidated-drained triaxial tests were conducted on these two specimens. Following are the results of the tests:

Specimen 1: $\sigma_3 = 100\ \text{kN/m}^2$

$(\Delta\sigma_d)_f = 410.6\ \text{kN/m}^2$

Specimen 2: $\sigma_3 = 50\ \text{kN/m}^2$

$(\Delta\sigma_d)_f = 384.37\ \text{kN/m}^2$

Danger

Determine the shear strength parameters for the samples (see Figure 7.15).

$\theta = 45°$ Failure

!!! لازم نتجاوز $\Delta\sigma_d = 99\ \text{kN}$

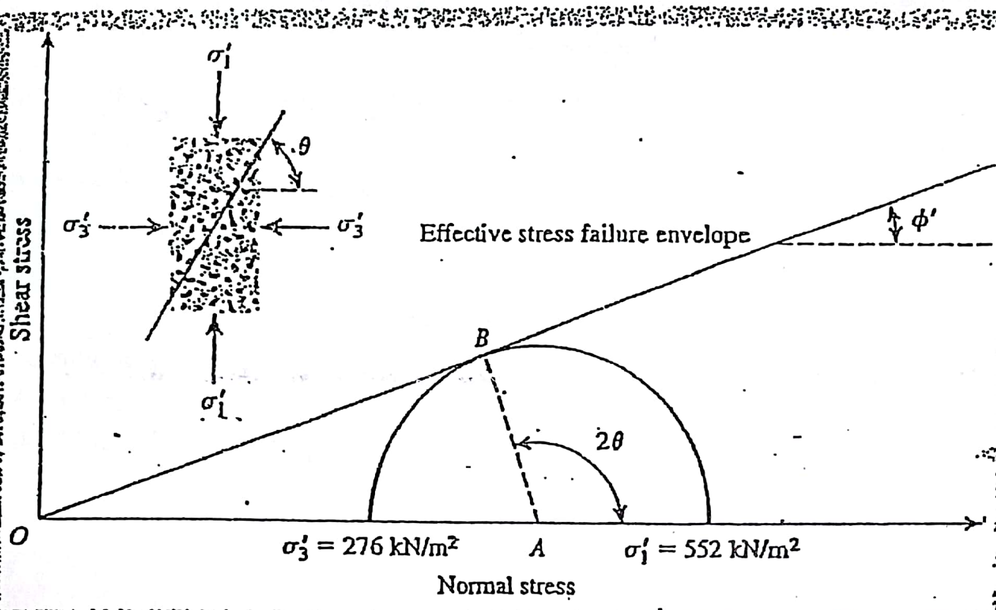


Figure 7.14 Mohr's circle and failure envelope for a normally consolidated clay

Part b
From Eq. (7.6),

$$\theta = 45 + \frac{\phi'}{2} = 45° + \frac{19.45°}{2} = 54.73°$$

Part c
Using Eqs. (5.21) and (5.22), we get

$$\sigma' \text{ (on the failure plane)} = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

and

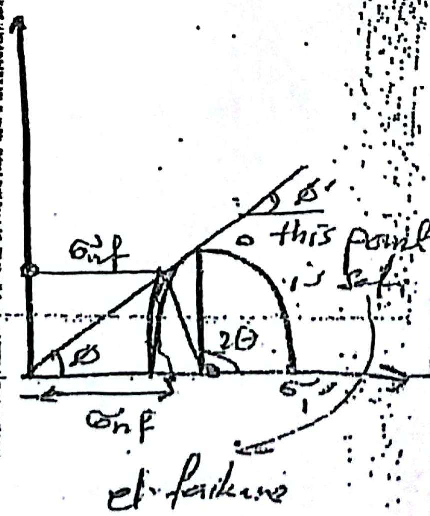
$$\tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$$

Substituting the values of $\sigma_1' = 552$ kN/m², $\sigma_3' = 276$ kN/m², and $\theta = 54.73°$ into the preceding equations, we get

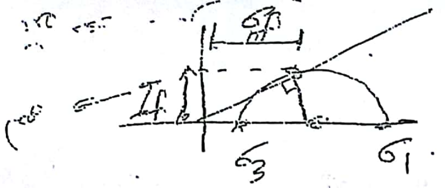
$$\sigma' = \frac{552 + 276}{2} + \frac{552 - 276}{2} \cos(2 \times 54.73) = 368.03 \text{ kN/m}^2$$

and

$$\tau_f = \frac{552 - 276}{2} \sin(2 \times 54.73) = 130.12 \text{ kN/m}^2$$



$\tau_{max} \neq \tau_f$



Friction Angle.

ليت الذي بيني وبينك عامر

A consolidated-drained triaxial test on a clayey soil may take several days to complete. This amount of time is required because deviator stress must be applied very slowly to ensure full drainage from the soil specimen. For this reason, the CD type of triaxial test is uncommon.

N.C Clay

$C = 0.0$

Example 7.1

A consolidated-drained triaxial test was conducted on a normally consolidated clay. The results are as follows:

- $\sigma_3 = 276\ \text{kN/m}^2$
- $(\Delta\sigma_d)_f = 276\ \text{kN/m}^2$

Determine

a. Angle of friction, ϕ'

b. Angle θ that the failure plane makes with the major principal plane

c. Normal stress, σ', and shear stress, τ_f, on the failure plane

Solution

For normally consolidated soil, the failure envelope equation is

$$\tau_f = \sigma' \tan\phi' \qquad (\text{because } c' = 0)$$

For the triaxial test, the effective major and minor principal stresses at failure are as follows:

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 276 + 276 = 552\ \text{kN/m}^2$$

and

$$\sigma_3' = \sigma_3 = 276\ \text{kN/m}^2$$

Part a. The Mohr's circle and the failure envelope are shown in Figure 7.14, from which we find that

$$\sin\phi' = \frac{AB}{OA} = \frac{\left(\dfrac{\sigma_1' - \sigma_3'}{2}\right)}{\left(\dfrac{\sigma_1' + \sigma_3'}{2}\right)}$$

or

$$\sin\phi' = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'} = \frac{552 - 276}{552 + 276} = 0.333$$

وكل الذي فوق التراب تراب!

Friction Angle.

Subtracting Eq. (7.14b) from Eq. (7.14a),

$$76.23 = 50 \tan^2\left(45 + \frac{\phi_1}{2}\right)$$

$$45 + \frac{\phi_1}{2} = \tan^{-1}\left[\sqrt{\frac{76.23}{50}}\right] = 51°$$

or

$$\phi_1 = 12°$$

Substituting $\phi_1 = 12°$ in Eq. (7.14a),

$$510.6 = 100 \tan^2[45 + (12/2)] + 2c \tan[45 + (12/2)]$$

or

$$510.6 = 152.5 + 2.47c$$

So,

$$c = 145 \text{ kN/m}^2$$

good Luck..!

CU

Consolidated-Undrained Test

over Consolidated ??

The consolidated-undrained test is the most common type of triaxial test. In this test, the saturated soil specimen is first consolidated by an all-around chamber fluid pressure, σ_3, resulting in drainage. After the pore water pressure generated by the application of confining pressure is completely dissipated (that is, $u_c = B\sigma_3 = 0$), the deviator stress, $\Delta\sigma_d$, on the specimen is increased to cause shear failure. During this phase of the test, the drainage line from the sample is kept closed. Since drainage is not permitted, the pore water pressure, Δu_d, will increase. During the test, simultaneous measurements of $\Delta\sigma_d$ and Δu_d are made. The increase of pore water pressure, Δu_d, can be expressed in a nondimensional form as

$$A = \frac{\Delta u_d}{\Delta \sigma_d} \tag{7.15}$$

where A = Skempton's pore pressure parameter (Skempton, 1954)

The general patterns of variation of $\Delta\sigma_d$ and Δu_d with axial strain for sand and clay soils are shown in Figures 7.16d through 7.16g. In loose sand and normally consolidated clay, the pore water pressure increases with strain. In dense sand and overconsolidated clay, the pore water pressure increases with strain to a certain limit, beyond which it decreases and becomes negative (with respect to the atmospheric ... of a tendency of the soil to dilate.

... ed test, the total and effective principal stresses ... undrained test. Because the pore water pressure ... the principal stresses may be analyzed as follows:

... (total): $\sigma_3 + (\Delta\sigma_d)_f = \sigma_1$

... (effective): $\sigma_1 - (\Delta u_d)_f = \sigma_1'$

establish a reasonable pattern of the change of c_u with depth. However, if the clay deposit at a given site is more or less uniform, a few unconsolidated-undrained triaxial tests on undisturbed specimens will allow a reasonable estimation of soil parameters for design work. Vane shear tests also are limited by the strength of soils in which they can be used. The undrained shear strength obtained from a vane shear test also depends on the rate of application of torque T.

Bjerrum (1974) also showed that, as the plasticity of soils increases, c_u obtained from vane shear tests may give results that are unsafe for foundation design. For this reason, he suggested the correction

$$c_{u(\text{design})} = \lambda c_{u(\text{vane shear})} \tag{7.49}$$

where

$$\lambda = \text{correction factor} = 1.7 - 0.54\log(PI) \tag{7.50}$$

PI = plasticity index

Problems

7.1 A direct shear test was conducted on a specimen of dry sand with a normal stress of 191.5 kN/m². Failure occurred at a shear stress of 119.7 kN/m². The size of the sample tested was 50.8 mm × 50.8 mm × 25.4 mm (height). Determine the angle of friction, ϕ'. For a normal stress of 144 kN/m², what shear force would be required to cause failure in the sample?

7.2 Following are the results of four drained direct shear tests on a normally consolidated clay:
Sample size: diameter of sample = 50 mm
height of sample = 25 mm

Test No.	Normal force (N)	Shear force at failure (N)
1	271	120.6
2	406.25	170.64
3	474	204.1
4	541.65	244.3

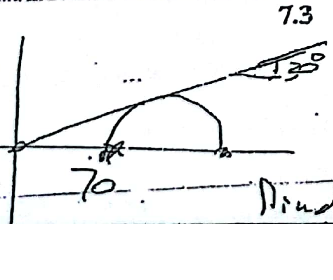
70 $45 + \frac{\phi}{2}$ ⇒ $45 + \frac{30}{2}$

$\sin\phi' = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$ ⇒ $\sin 30 = \frac{\sigma_1 - 70}{\sigma_1 + 70}$ ⇒ find σ_1 ⇒ $\sigma_1 = (\sigma_3 + \Delta\sigma_d)$

Draw a graph for shear stress at failure against normal stress. Determine the drained angle of friction from the graph.

fail the angle

7.3 The equation of the effective stress failure envelope for a loose sandy soil was obtained from a direct shear test as $\tau_f = \sigma' \tan 30°$. A drained



σ_1 is $\sigma_3 + \Delta\sigma_d$

70 find the Diviator stress??

test was conducted with the same soil at a chamber confining pressure of 70 kN/m². Calculate the deviator stress at failure.

For the triaxial test described in Problem 7.3:

a. Estimate the angle that the failure plane makes with the major principal plane;

b. Determine the normal stress and shear stress (when the specimen failed) on a plane that makes an angle of 30° with the major principal plane. Also explain why the specimen did not fail along the plane during the test.

For a normally consolidated clay, the results of a drained triaxial test are as follows:

- Chamber confining pressure = 140 kN/m²
- Deviator stress at failure = 263.5 kN/m²

Determine the soil friction angle, ϕ'.

The results of two drained triaxial tests on a saturated clay are given below:

Specimen I: chamber confining pressure = 69 kN/m²
deviator stress at failure = 213 kN/m²

Specimen II: chamber confining pressure = 120 kN/m²
deviator stress at failure = 258.7 kN/m²

Calculate the shear strength parameters of the soil.

If a specimen of clay described in Problem 7.6 is tested in a triaxial apparatus with a chamber confining pressure of 200 kN/m², what will be the major principal stress at failure? Assume full drained condition during the test.

A drained triaxial test on a normally consolidated clay showed that the failure plane makes an angle of 58° with the horizontal. If the specimen was tested with a chamber confining pressure of 103.5 kN/m², what was the major principal stress at failure?

A deposit of sand is shown in Figure P7.9. Find the shear resistance in kN/m² along a horizontal plane located 10 m below the ground surface.

find shear resistance at point A ??

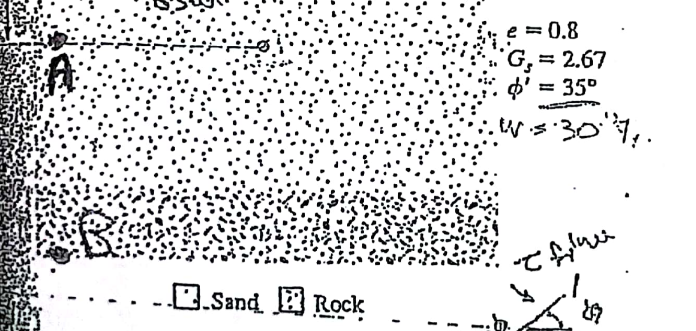


Figure P7.9

$\tau = \bar{c} + \bar{\sigma}_n \tan\phi'$

sand

$\tau = 0.0 + \bar{\sigma}_n \tan 35$

$\bar{\sigma}_n = \sigma_n - u$

$= \gamma_{sat} \times 10 - \ldots$

$[9.8 \times 10]$

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

$\bar{\sigma}_n = \gamma' h$

$\tau = \bar{\sigma}_n \tan\phi$

τ_f

$\tau_D > \tau_{DR}$

at B. σ_n نفس

الكلام

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

$G_s w_s = S e$

CD test

11

Find the pore water pressure ??

7.10 A consolidated-undrained triaxial test on a normally consolidated clay yielded the following results:

- $\sigma_3 = 84$ kN/m²
- Deviator stress: $(\Delta\sigma_d)_f = 64$ kN/m²
- Pore pressure: $(\Delta u_d)_f = 48$ kN/m²

Calculate the consolidated-undrained friction angle and the drained friction angle.

7.11 The shear strength of a normally consolidated clay can be given by the equation $\tau_f = \sigma' \tan 31°$. A consolidated-undrained triaxial test was conducted on the clay. Following are the results of the test:

- Chamber confining pressure = 112 kN/m² σ_3
- Deviator stress at failure = 100.14 kN/m² $(\Delta\sigma)_f$

Determine:

a. The consolidated undrained friction angle (ϕ);

b. The pore water pressure developed in the clay specimen at failure.

$$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

$$\sigma_1' = \sigma_1 - \Delta u$$

$$\sigma_3' = \sigma_3 - \Delta u$$

[112 + 100.14]

$$\sin\phi_f = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

7.12 A silty sand has a consolidated-undrained friction angle of 22° and a drained friction angle of 32° ($c' = 0$). If a consolidated-undrained test on such a soil is conducted at a chamber confining pressure of 115 kN/m², what will be the major principal stress (total) at failure? Also calculate the pore pressure that will be generated in the soil sample at failure.

7.13 The following are the results of a consolidated-undrained triaxial test in a clay:

Sample No.	σ_3 (kN/m²)	σ_1 at failure (kN/m²)
1	191.67	375.67
2	383.34	636.33

$$\sin 32 = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 + 2\Delta u}$$

Draw the total stress Mohr's circles and determine the shear strength parameters for consolidated-undrained conditions.

$$\sin 31 = \frac{(\sigma_1 - \Delta u) - (\sigma_3 - \Delta u)}{(\sigma_1 - \Delta u) + (\sigma_3 - \Delta u)}$$

$$\sin 31 = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 - 2\Delta u}$$

7.14 The unconsolidated-undrained test results of a saturated clay specimen are as follows:

- $\sigma_3 = 96$ kN/m²
- σ_1 at failure = 144 kN/m²

What will be the axial stress at failure if a similar sample is subjected to an unconfined compression test?

$212.14 = \sigma_1 - \Delta u$

$112.0 = \sigma_3 - \Delta u$

7.15 The friction angle, ϕ', of a normally consolidated clay sample collected during field exploration was determined from drained triaxial tests to be equal to 25°. The unconfined compression strength, q_u, of a similar sample was found to be 100 kN/m². Determine the pore water pressure at failure for the unconfined compression test.

7.16 A soil profile is shown in Figure P7.16. The clay is normally consolidated. Its liquid limit equals 68%, and its plastic limit equals 27%. Estimate the unconfined compression strength of the clay at a depth of 10 m measured from the ground surface.

$$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

$\Delta\sigma_d = \sigma_1 - \sigma_3$

Pro (7.12)

1) CU test → $\phi_f = 22°$ at $\sigma_3 = 115$

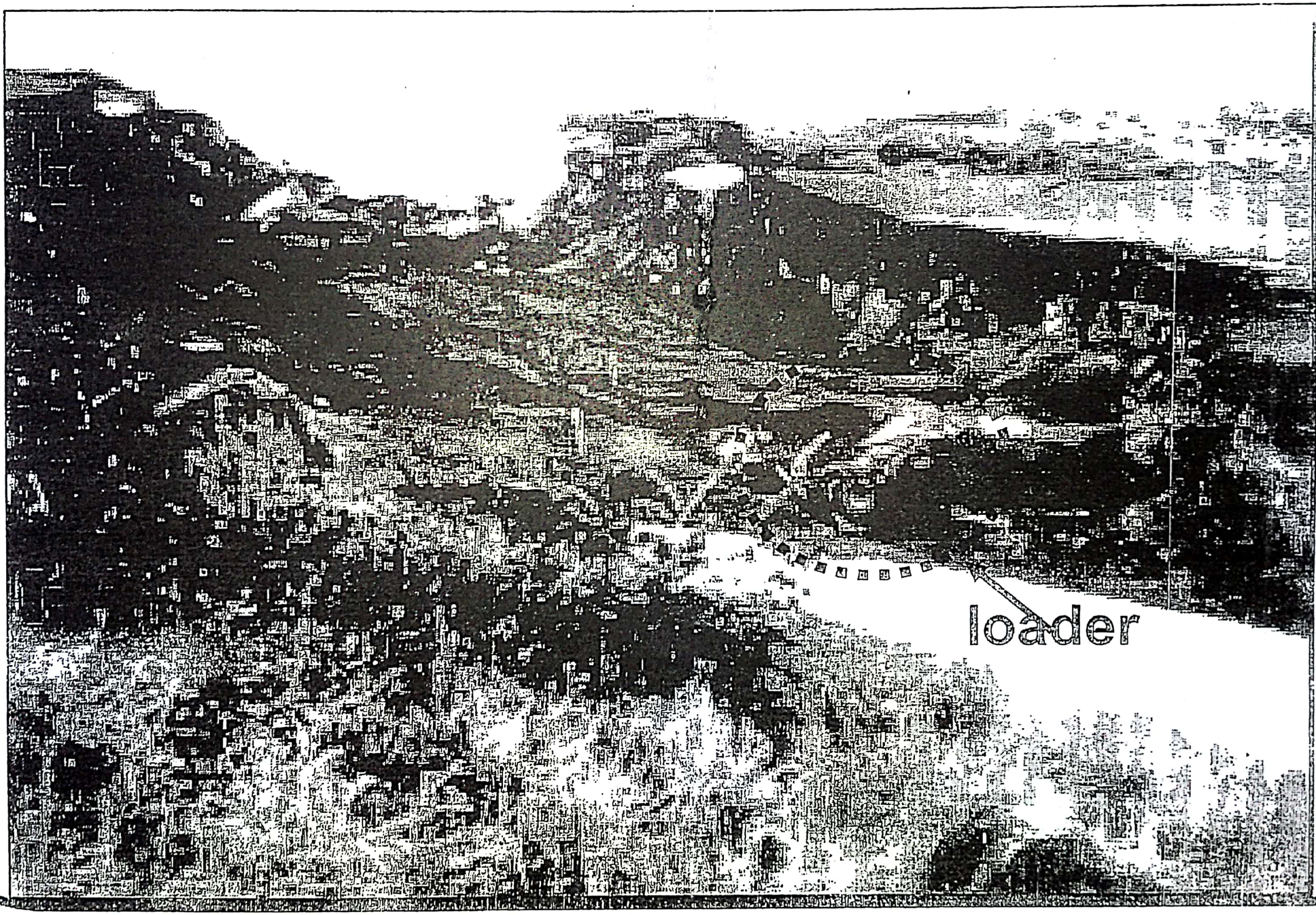
CD test → $\phi = 32°$ σ_1 ??

$$\sin\phi_f = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} \Rightarrow \sin 22 = \frac{\sigma_1 - 115}{\sigma_1 + 115}$$

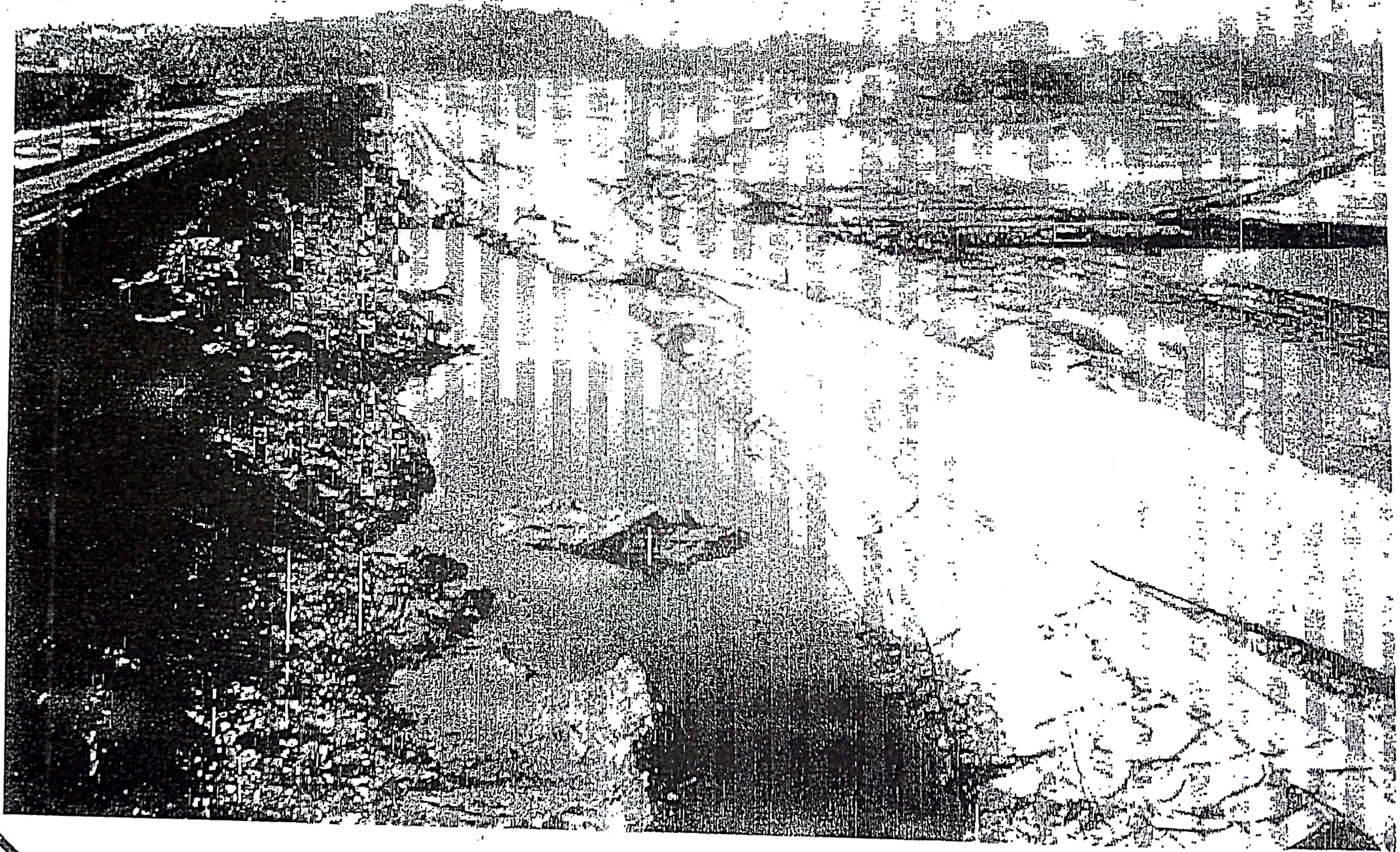
13

10

Slope Stability



Lower San Fernando Dam Failure, 1971



Outlines

- Introduction
- Definition of key terms
- Some types of slope failure
- Some causes of slope failure
- Shear Strength of Soils
- Infinite slope
- Two dimensional slope stability analysis

Introduction II

- In this session we will discuss a few methods of analysis from which you should be able to :
  1) Estimate the stability of slopes with simple geometry and geological features
  2) Understand the forces and activities that provoke slope failures
  3) Understand the effects of geology, seepage and pore water pressures on the stability of slopes

Introduction I

- Slopes in soils and rocks are ubiquitous in nature and in man-made structures.
- Highways, dams, levees, bund-walls and stockpiles are constructed by sloping the lateral faces of the soil
  - Slopes are general less expensive than constructing a walls.
- Natural forces (Wind, water, snow, etc.) change the topography on Earth often creating unstable slopes.
- Failure of such slopes resulted in human loss and destruction.
- Failure may be sudden and catastrophic; others are insidious;
- Failure wither wide spread or localized.

Definitions of Key Terms

- Slip or Failure Zone: A thin zone of soil that reaches the critical state or **residual state** and results in movement of the upper soil mass
- Slip plane; failure plane; Slip surface; failure surface: Surface of sliding
- Sliding mass: mass of soil within the slip plane and the ground surface
- Slope angle: Angle of inclination of a slope to the horizontal
- Pore water pressure ratio (r_u): The ratio of pore water force on a slip surface to the total weight of the soil and any external loading.

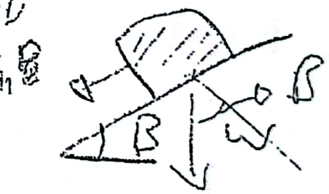
$$r_u = \frac{\gamma_w * Z * A}{\gamma Z_1 + \gamma_2 Z_2 * A}$$

Common Type of Slope Failure

سنوات

- Slope failures depends on
  - The Soil Type, $[\phi, c]$ — $\tau = c + \sigma_n \tan\phi$
  - Soil Stratification, طبقات — weak layer → $\tau = c + \sigma_n \tan\phi$ قليلة
  - Ground Water, زاد الوزن
  - Seepage and
  - Geometry.

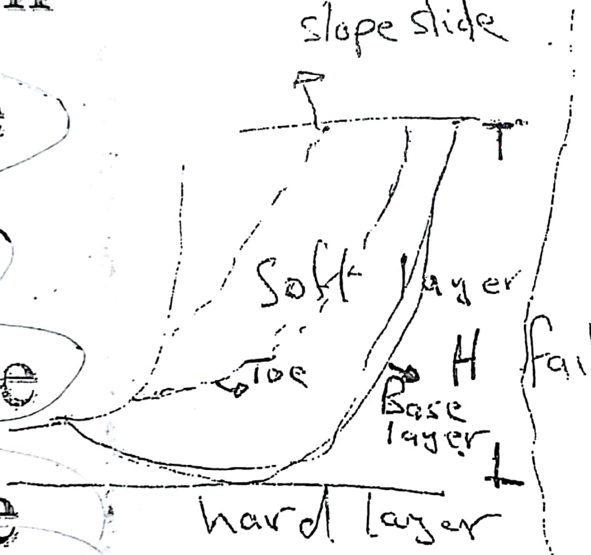
زاد $W \sin\beta$



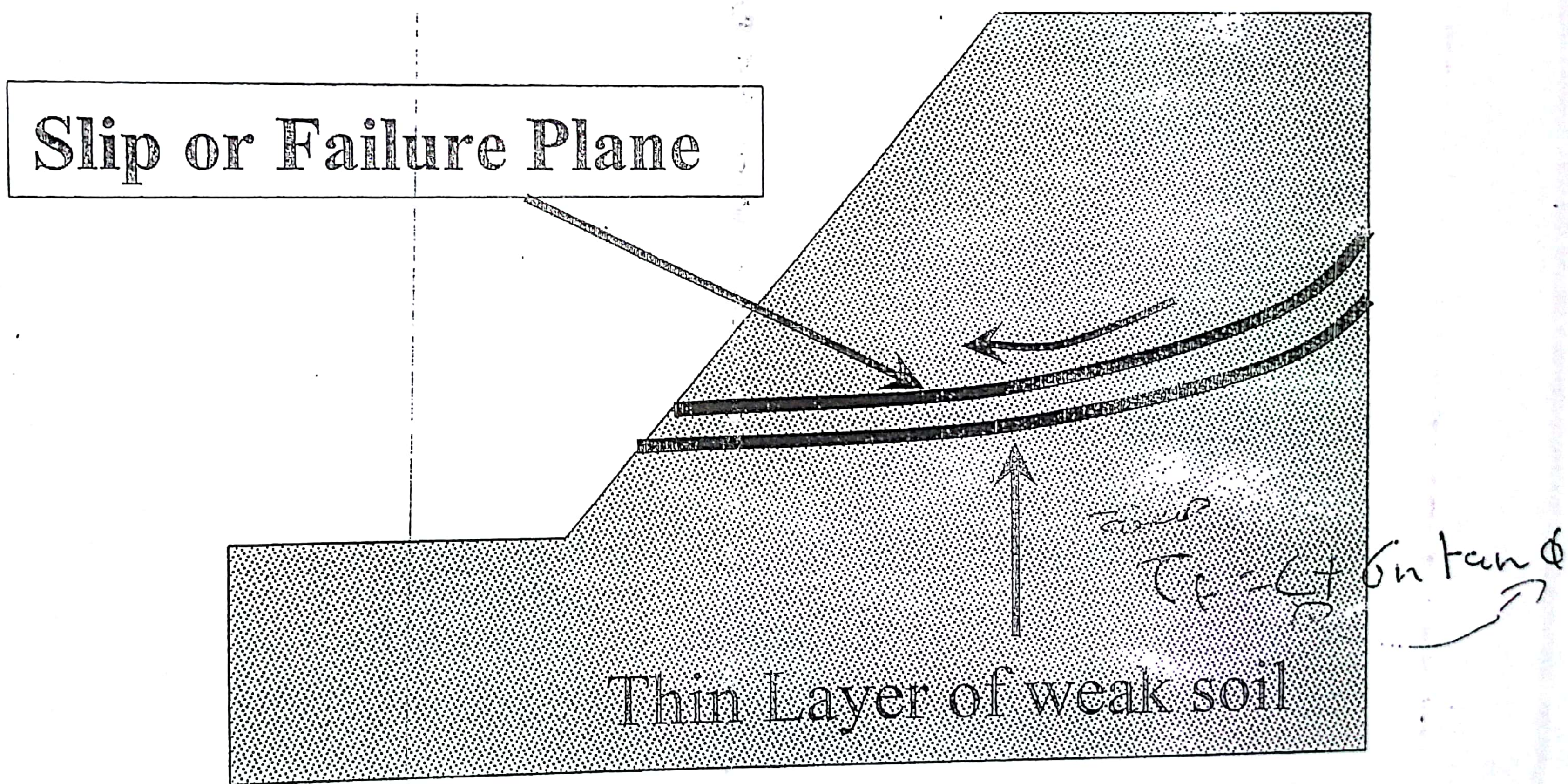
Common Type of Slope failures

- **Common Type**
  - ☐ Movement of Soil Mass Along a Thin Layer of Weak Soil
  - ☐ Base Slide
  - ☐ Toe Slide
  - ☐ Slope Slide
  - ☐ Flow Slide
  - ☐ Block Slide

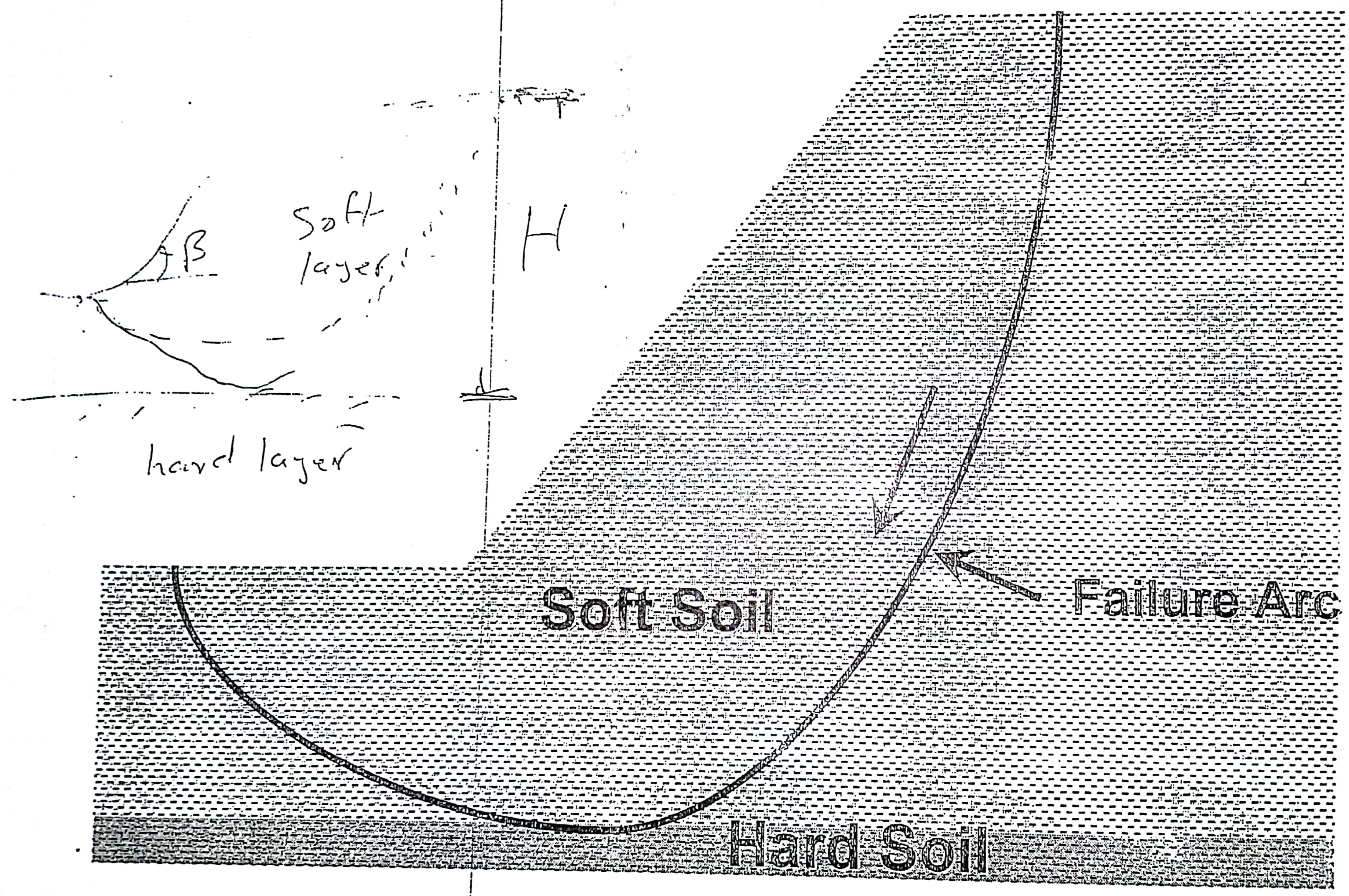
locally



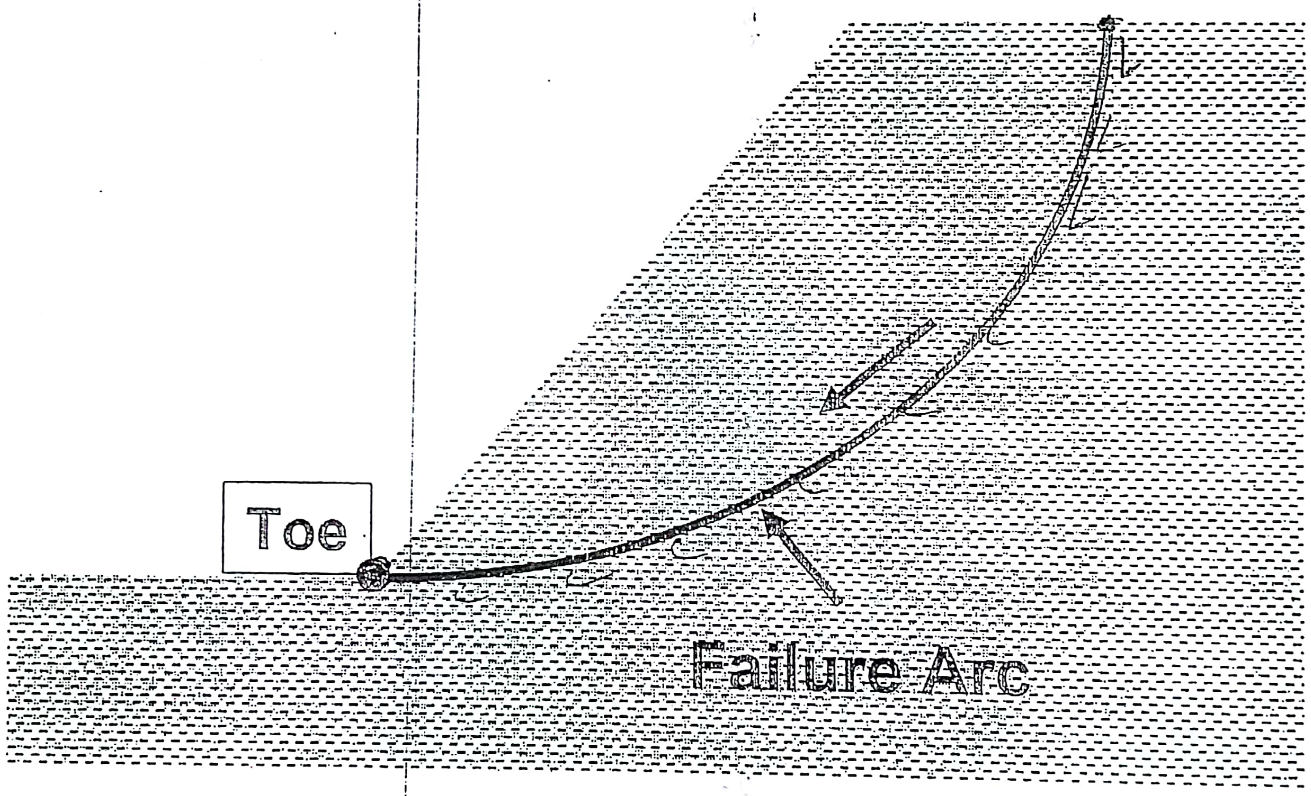
Movement of soil mass along a thin layer of weak soil



Base Slide

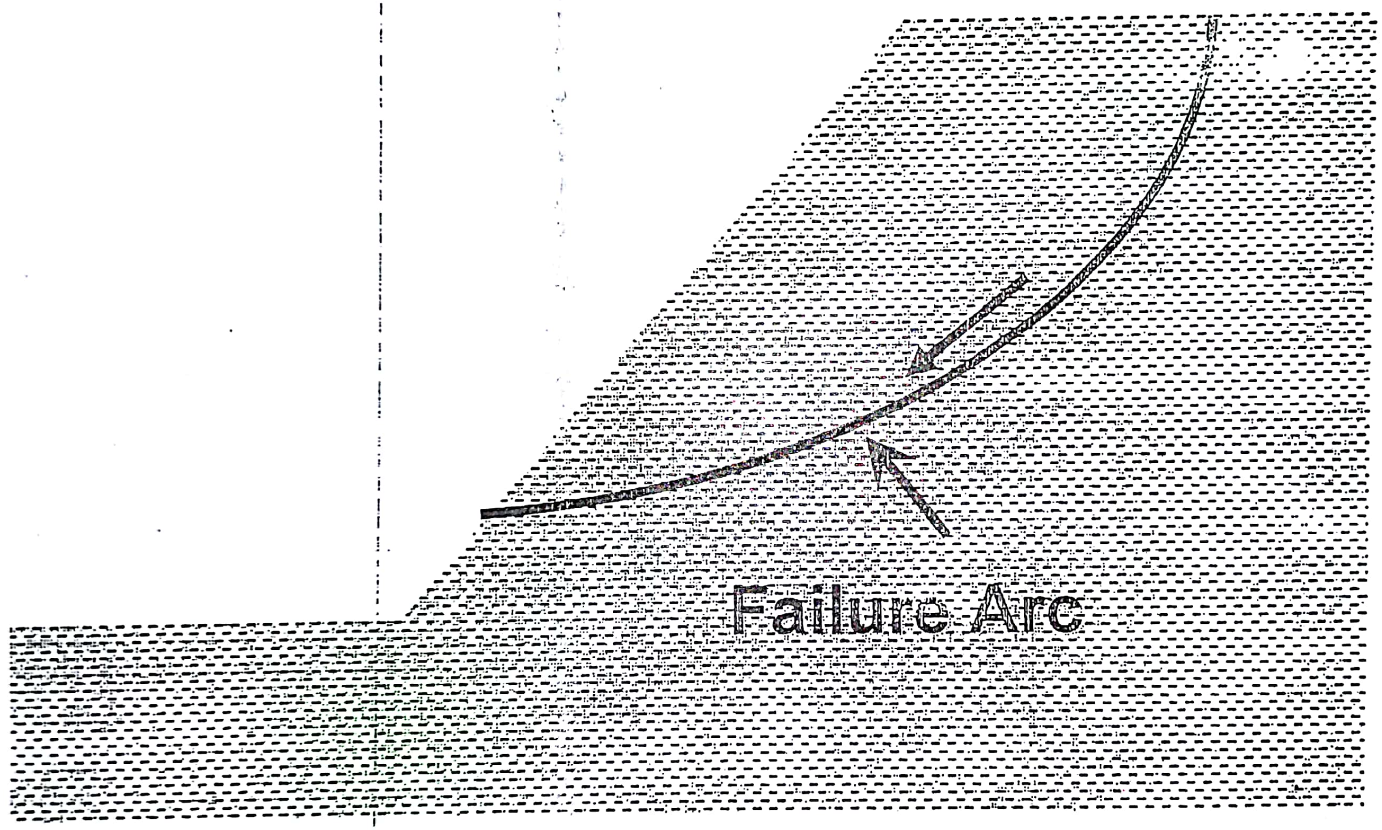


Toe Slide



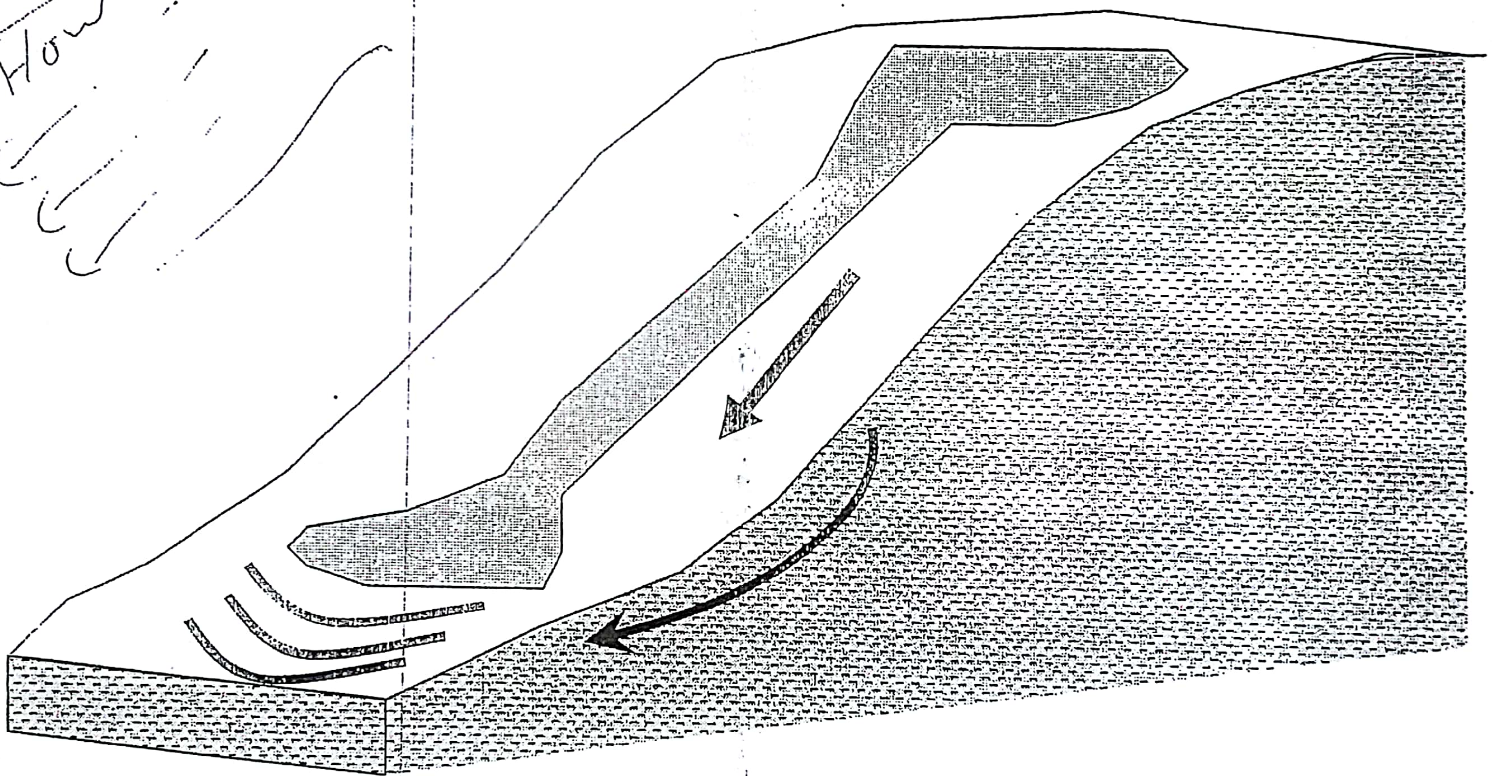
soft
hard.

Slope Slide



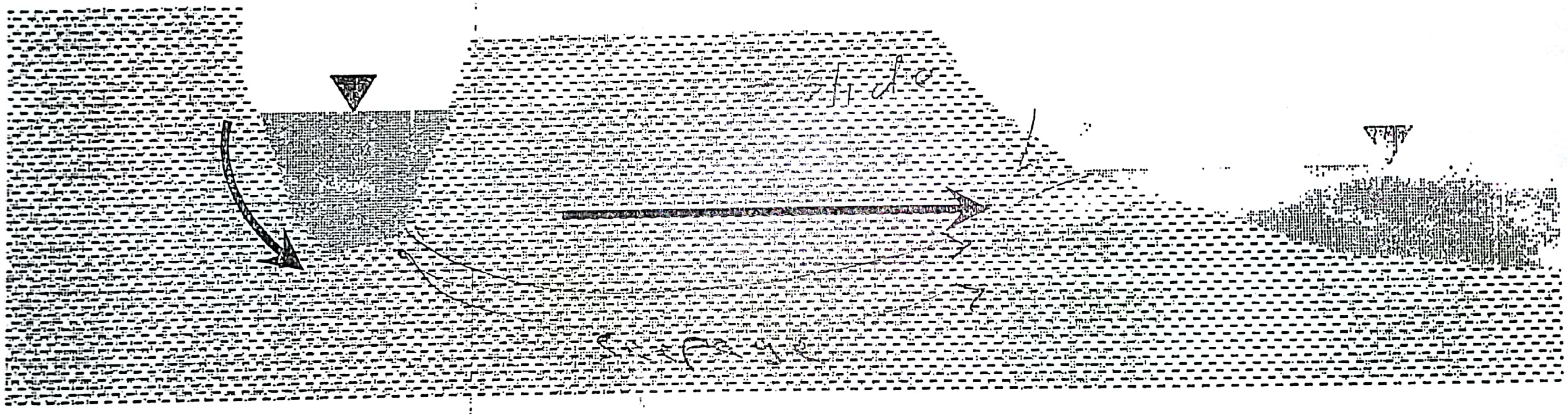
Soft
hard

Flow Slide



Flow

Block Slide



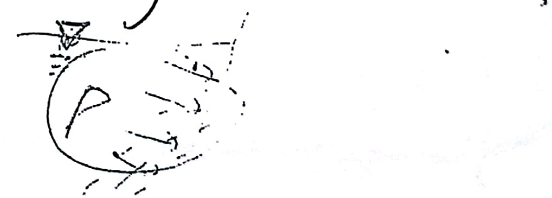
Some causes of slope failure

- Erosion انجراف water and wind erode natural and main
- Rainfall
- Earthquake
- Geological factures
- External loading
- Construction activity
- Excavated slope الحفر
- Fill Slope الطمر
- Rapid draw Down

زيادة في ال load

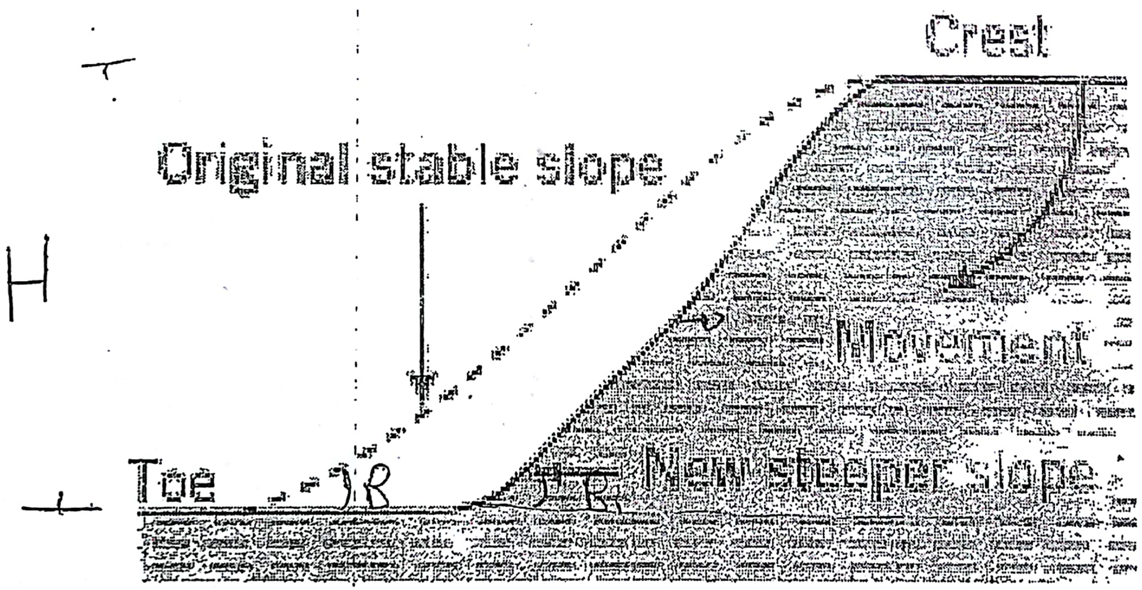
تغير في Geometry

Slope failure depending :
1) Soil type
2) Soil stratification
3) Ground water
4) seepage
5) gematry



Steepening by Erosion

- Water and wind continuously erode natural and man made slopes.
- Erosion changes the geometry of the slope, ultimately resulting in slope failures or, more aptly, landslide.

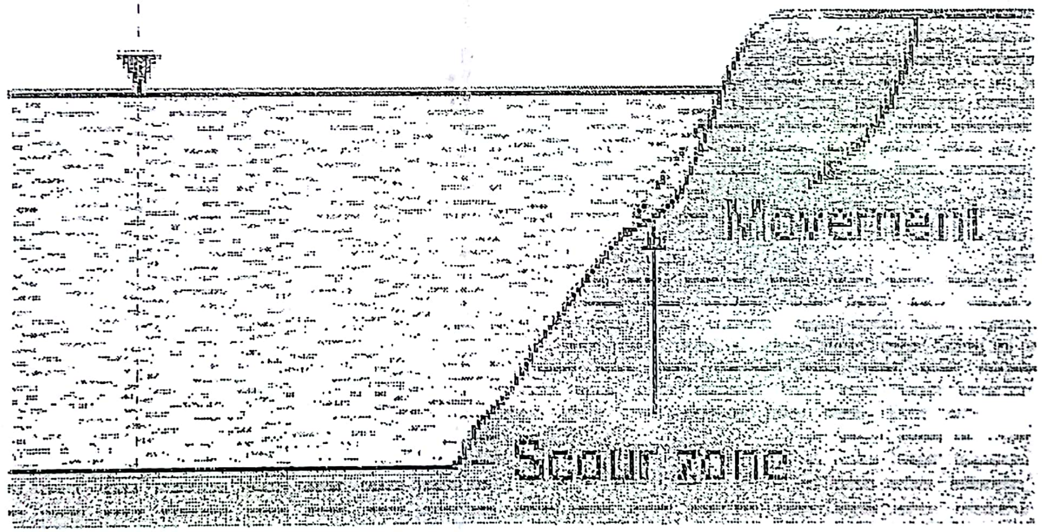


New steeper slope

$\beta_1 > \beta$

Water Scouring

- Rivers and stream continuously scour their banks undermining their natural or man made slopes



Scouring by water movement

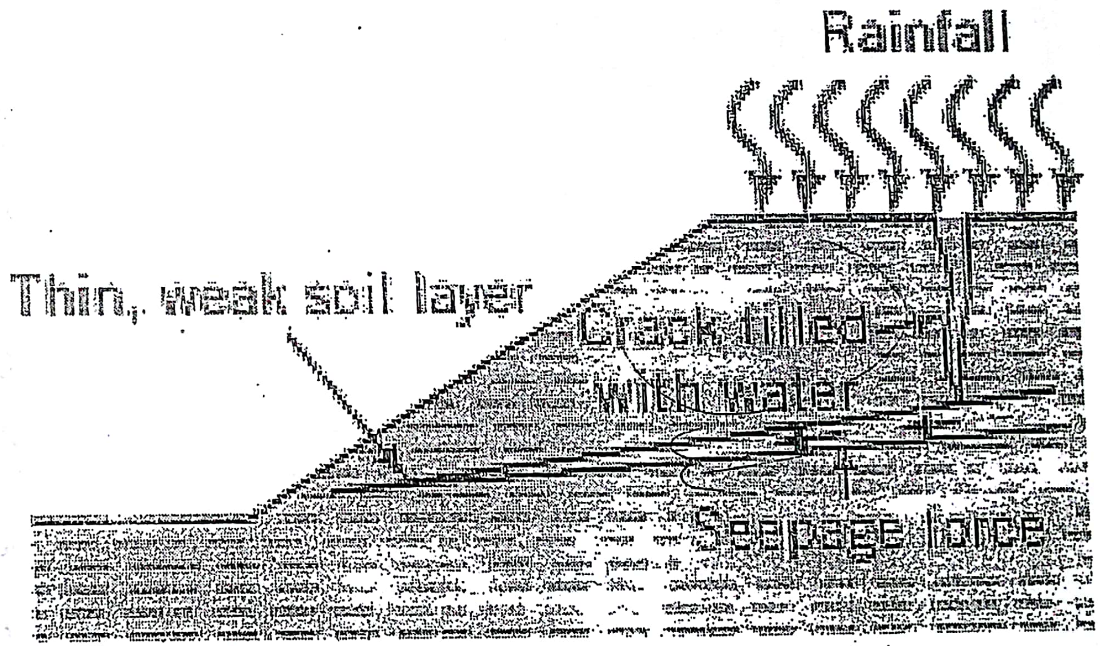
Rainfall

Long period of rainfall saturate, soften and erode soils. Water enter into exiting crack and may weaken underlying soil layers leading to failure e.g. mudslides

sliding

stone

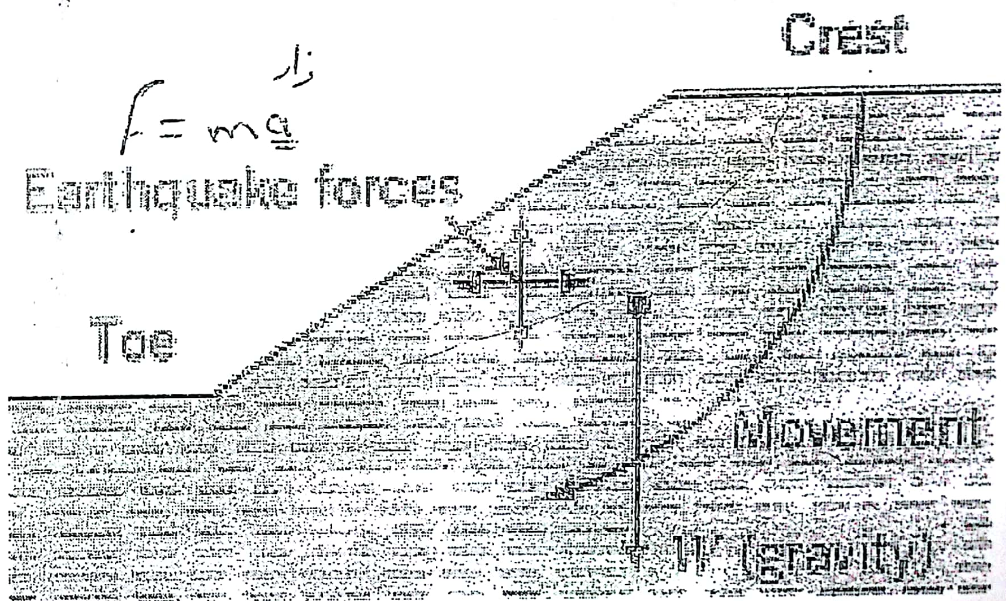
صخور



Rainfall fills crack and introduces seepage forces in the thin, weak soil layer

Earthquake

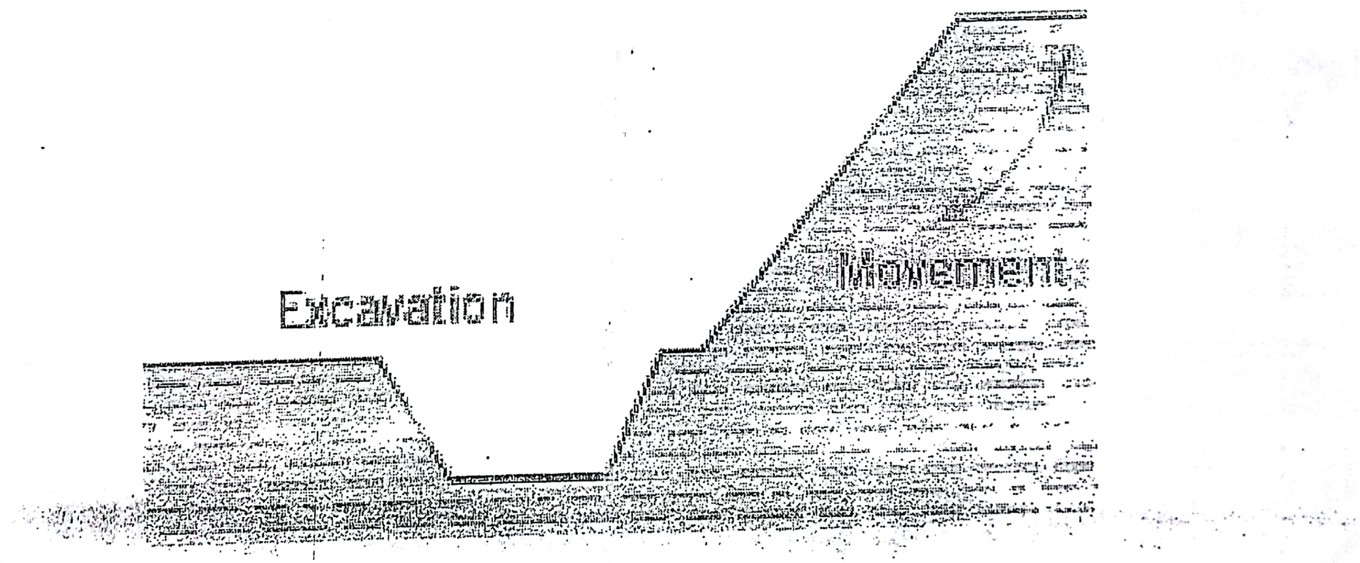
- Earthquake introduced dynamic forces. Especially dynamic shear forces that reduce the shear strength and stiffness of the soil. Pore water pressures rise and lead to liquefaction



Gravity and Earthquake forces

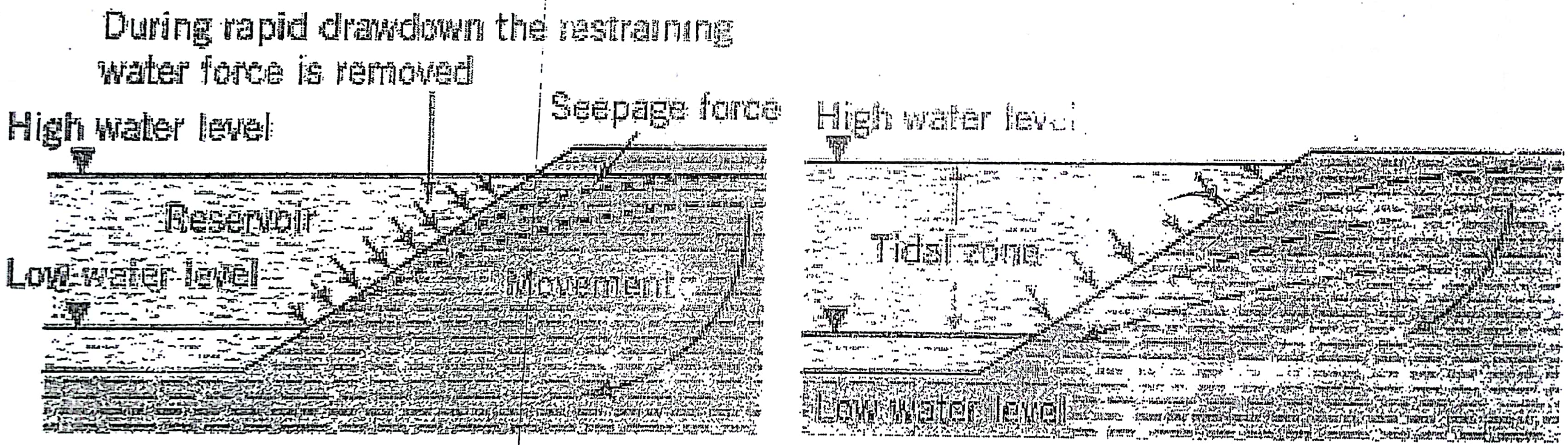
Construction Activity

- Excavated slopes: If the slope failures were to occur, they would take place after construction is completed.
- Fill slopes: failure occur during construction or immediately after construction.



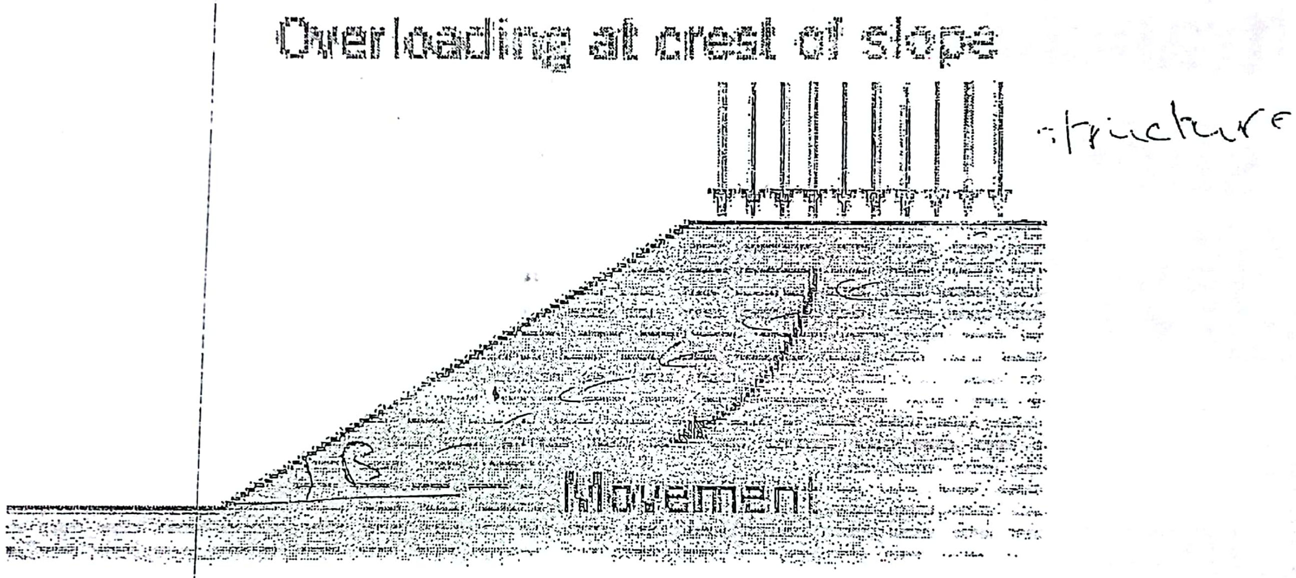
Rapid Draw Down

- Later force provided by water removed and excess p.w.p does not have enough time to dissipated



External loading

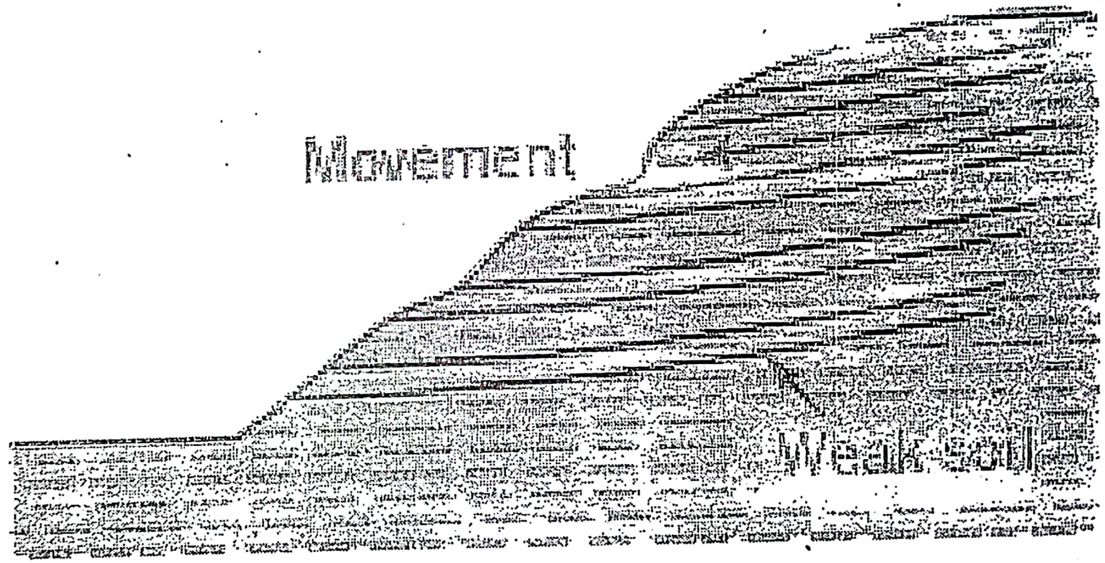
- Loads placed on the crest of a slope add to the gravitational load and may cause slope failures.



- Load places at the toe called a berm, will increase the stability of the slope. Berms are often used to the remediate problem slopes.

Geological factures

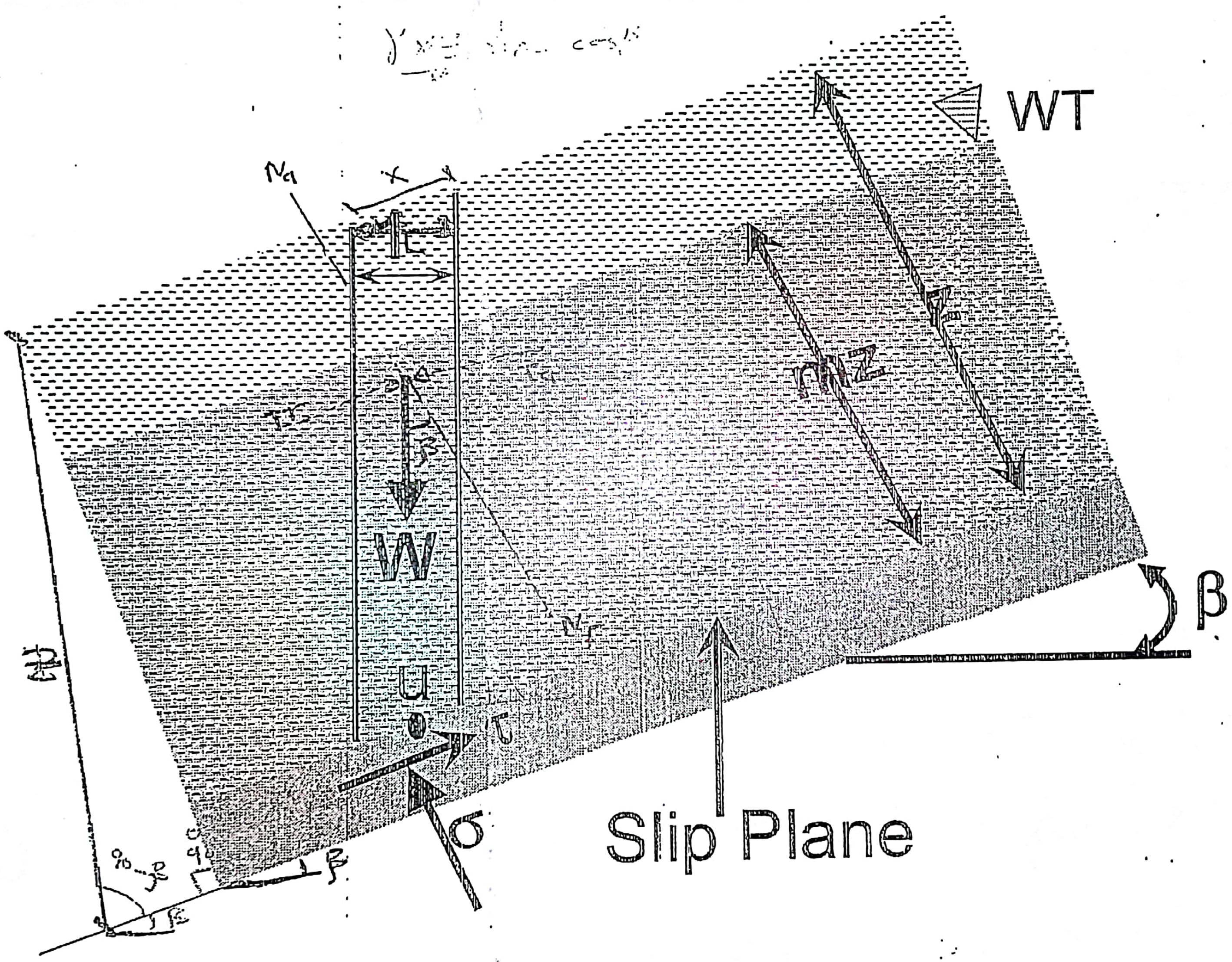
- Sloping stratified soils are prone to translational slide a long weak layer



Infinite Slope II

- Assumption Continued:
  - The failure mass moves as an essentially rigid body, the deformation of which do not influence the problem
  - The shearing resistance of the soil mass at various point along the slide of the failure surface is independent of orientation
  - The Factor of safety is defined in term of the average shear strength along this surface.

Infinite Slope III



Infinite Slope IV

Stress in the soil mass and Available Shear Strength

$$\sigma = [(1-m)\gamma + m\gamma_{sat}]z\cos^2\beta$$

$$\tau = [(1-m)\gamma_d + m\gamma_{sat}]z\sin\beta\cos\beta$$

$$u = mz\gamma_w\cos^2\beta$$

$$\underline{\tau_f} = c' + (\sigma - u)\tan\phi'$$

τ_f

Infinite Slope V

Effective stresses (Three Scenarios)

1) 0<m<1

$$\boxed{F.S = \frac{\tau_f}{\tau_m} = \frac{c' + (\sigma - u)\tan\phi'}{[(1-m)\gamma + m\gamma_{sat}]z \sin\beta \cos\beta}}$$

2) m=0 & c'=0.

$$\boxed{F.S = \frac{\tan\phi'}{\tan\beta}}$$

3) m=1 & c'=0.

$$\boxed{F.S = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\phi'}{\tan\beta}}$$

$$\tan\beta = \frac{\gamma' \tan\phi}{\gamma_{sat}}$$

$$\beta \approx \frac{1}{2}\phi$$

Total stresses: $c' \longrightarrow c_u$ and $\phi' \longrightarrow \phi_u$ and $u=0$

Infinite Slope VI

- Summary:
  1) The maximum stable slope in a coarse grained soil, in the absence of seepage is equal to the friction angle
  2) The maximum stable slopes in coarse grained soil, in the presence of seepage parallel to the slope, is approximately one half the friction angle
  3) The critical slip angle in fine grained soil is 45° for an infinite slope mechanisms

with $\phi = 0.0$

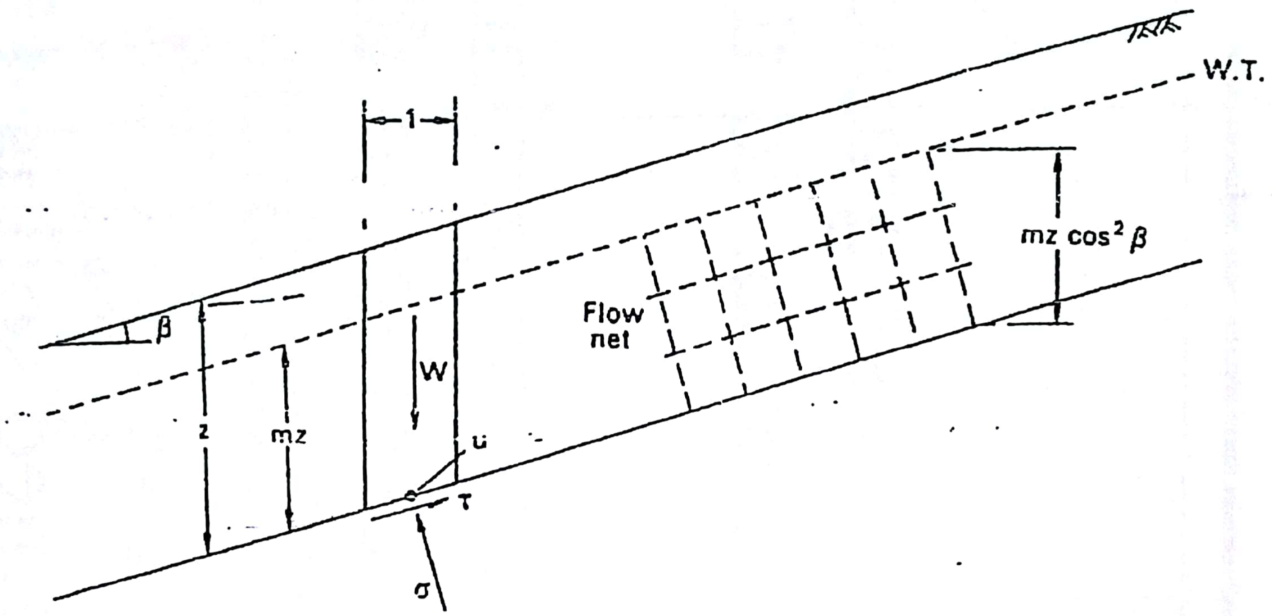


Figure 9.7 Plane translational slip.

and the factor of safety is

$$F = \frac{\tau_f}{\tau}$$

The expressions for σ, τ and u are

$$\sigma = \{(1-m)\gamma + m\gamma_{sat}\}z\cos^2\beta$$

$$\tau = \{(1-m)\gamma + m\gamma_{sat}\}z\sin\beta\cos\beta$$

$$u = mz\gamma_w\cos^2\beta$$

If the soil between the surface and the failure plane is not fully saturated (i.e. $m = 0$) then

$$F = \frac{\tan\phi'_{cv}}{\tan\beta} \tag{9.11}$$

If the water table coincides with the surface of the slope (i.e. $m = 1$) then

$$F = \frac{\gamma'\tan\phi'_{cv}}{\gamma_{sat}\tan\beta} \tag{9.12}$$

For a total stress analysis the shear strength parameter c_u is used (with $\phi_u = 0$) and the value of u is zero.

Example 9.3

A long natural slope in an overconsolidated fissured clay of saturated unit weight $20\,kN/m^3$ is inclined at $12°$ to the horizontal. The water table is at the surface and seepage is roughly parallel to the slope. A slip has developed on a plane parallel to the surface at a depth of 5 m. (1) Determine the factor of safety along the slip plane using (a) the critical-state parameter $\phi'_{cv} = 28°$ and (b) the residual strength parameter $\phi'_r = 20°$. (2) Analyse the stability of the slope by the limit state method.

1 Equation 9.12 applies in both cases.

(a) In terms of critical-state strength

$$F = \frac{10.2 \tan 28°}{20 \tan 12°} = 1.28$$

(b) In terms of residual strength

$$F = \frac{10.2 \tan 20°}{20 \tan 12°} = 0.87$$

2 In the limit state method the characteristic values of the ϕ' parameters are divided by the partial factor 1.25. Thus the design values are

$$\phi'_{cv} = \tan^{-1}\left(\frac{\tan 28°}{1.25}\right) = 23°$$

$$\phi'_r = \tan^{-1}\left(\frac{\tan 20°}{1.25}\right) = 16°$$

With the water table at the surface the value of $m = 1$

(a) The design disturbing force per m^2 is

$$\begin{aligned} S_d &= \gamma_{sat} z \sin \beta \cos \beta \\ &= 20 \times 5 \times \sin 12° \cos 12° \\ &= 20.3\ \text{kN} \end{aligned}$$

The design resisting force per m^2 is

$$\begin{aligned} R_d &= (\sigma - u) \tan \phi'_{cv} \\ &= (\gamma_{sat} - \gamma_w) z \cos^2 \beta \tan 23° \\ &= 10.2 \times 5 \times \cos^2 12° \tan 23° \\ &= 20.7\ \text{kN} \end{aligned}$$

The design disturbing force is less than the design resisting force; therefore, in terms of critical-state strength, the limit state for overall stability is satisfied.

(b) With $\phi'_r = 16°$ the design resisting force becomes 14.0 kN. The design disturbing force remains 20.3 kN; therefore, in terms of residual strength, the limit state is not satisfied.

آخر شابتر

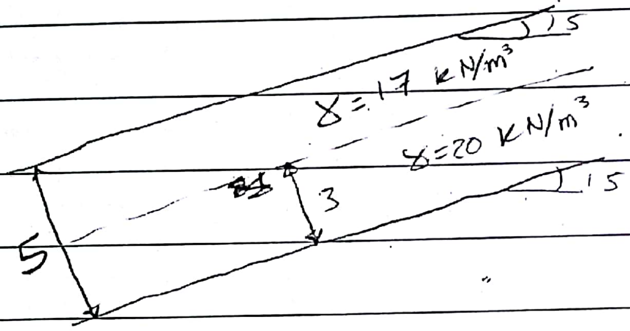
Ex:- For the Infinite slope with slope angle of (15°), soil has $c' = 0$ $\phi = 33^\circ$ and $C_u = S_u = 25$ kpa

$$\sigma = [(1-m)\gamma + m\gamma_{SAT}] Z \cos^2\beta$$

$$\tau = [(1-m)\gamma + m\gamma_{SAT}] Z \sin\beta\cos\beta$$

$$u = m Z \gamma_w \cos^2\beta$$

$$\tau_f = c' + (\sigma - u)\tan\phi'$$



effective

① For drained case if $H_w = 3$ m and $H = 5$ m
Find the factory of safty for the slope shown?

② IF $H_w = 0$, find F.S for drained case?

③ IF $H_w = 5$, " " " " " ?

④ Find the factor of safety of slope in case of undrained case?

Sol:-

$$m = \frac{H_w}{H} = \frac{3}{5}$$

$$\sigma = [(1-m)\gamma + m\gamma_{SAT}] Z \cos^2\beta$$

$$= \left[\left(1 - \frac{3}{5}\right)(17) + \frac{3}{5}(20)\right] 5 \cos^2 15 = 87.7 \text{ kPa}$$

N O T E B O O K

$\tau = \left[(1-m)\gamma + m\,\gamma_{SAT}\right] Z \sin\beta \cos\beta$

$= \left[\left(1-\frac{3}{5}\right)(17) + \frac{3}{5}(20)\right](5) \sin 15 \cos 15$

$= 23.5$ kpa

$u = m\,\gamma_w\,Z \cos^2\beta$

$= \left(\frac{3}{5}\right)(10)(5) \cos^2 15 = 27.99$ kpa

1) Drained $\rightarrow$ Effective stress

$F.S = \frac{\tau}{\tau_m} = \frac{c' + (\sigma - u)\tan\phi}{\tau_m}$

$= \frac{0 + (87.7 - 27.99)\tan 33}{23.5} = 1.65 > 1$

ok safe

2) $m = 0$, $c' = 0$ $\rightarrow$ Drained $\rightarrow$ effective.

$F.S = \frac{\tan\phi'}{\tan\beta} = \frac{\tan 33}{\tan 15} = 2.42 > 1$ ok safe

3) $m = \frac{5}{5} = 1$ $c' = 0$

$F.S = \frac{\gamma'}{\gamma_{SAT}} * \frac{\tan\phi'}{\tan\beta} = \left(\frac{20-10}{20}\right)(2.42) = 1.21 > 1$

safe

Subject Date No.

4) undrained $\longrightarrow$ Total $u=0$

$$\tau = [(1-m)\gamma + m\gamma_{SAT}]\, Z \cos\beta \sin\beta$$

$$= [0 + 1(20)](5) \cos 15 \sin 15 = 25 \text{ kPa}$$

$$F.S = \frac{c^{25} + (\sigma - u^{0}) \tan\phi^{0}}{\tau} = \frac{25}{25} = 1$$

slope on verge of failure.

انتهت المادة

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Geotechnical Engineering
Final Exam (Out of 50 points)
Thursday 17/5/2012
Duration: 2 H: 00

يحيى أبو عجمية

Hashemite University
Department of civil Engineering

Student Name:……………………………………………
Student Number:…………………………………………

Ch

All the answers should be in the given space and ***no partial credit*** will be given

Problem # 1 (15 points)

A normally consolidated clay was consolidated under a stresses of 169kPa, then sheared undrained in axial compression. The deviator stresses at failure were 118kPa, and the induced pore water pressures at failure were 96kPa. Determine:

$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$

1. The Mohr-Coulomb strength parameters in terms of total stresses (C and ϕ)

أبو عجمية

$c =$

$\phi =$

σ_3 σ_1

$\boxed{\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}}$

$\sigma_3 = 169$ $\quad \sigma_1 = 196 + 118 = 287$

$\Delta\sigma = 118$

$\sin\phi = \frac{\frac{1}{2}(\sigma_1 - \sigma_3)}{c\cot\phi + \frac{1}{2}(\sigma_1 + \sigma_3)}$ $\quad 118 + 169$

c $\quad \phi$

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

2. The Mohr-Coulomb strength parameters in terms of effective stresses (C' and ϕ')

أبو عجمية

$\sigma_1 = 287 - 96$ $\quad \sigma_3 = 169 - 96$ $\quad \sigma'_1 =$

$\sin\phi = \frac{\frac{1}{2}(\sigma_1 - \sigma_3)}{\frac{1}{2}(\sigma_1 + \sigma_3)}$

3. The [illegible] θ

$\theta = 45 + \frac{\phi}{2}$ $\quad \theta = 45 + \frac{\phi}{2}$

4. [illegible] σ_f

أبو عجمية

$\sigma_f = \frac{1}{2}(\sigma_1 + \sigma_3) + \frac{1}{2}(\sigma_1 - \sigma_3)\cos 2\theta$ $\quad \sigma_f =$

5. Shear stress at failure at failure plane τ_f

أبو عجمية

$\tau = \frac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta$

τ

$\frac{\phi}{2}$

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Problem # 2 (10 points)

In an oedometer test a specimen of saturated clay 20 mm thick reaches 50% consolidation in 20 minutes.

1. How long would it take a layer of this clay 3.0 m thick to reach the same degree of consolidation under the same stress and drainage conditions?

$$T_v = \frac{c_v \times t}{(H_{dr})^2} \Rightarrow 0.196 = \frac{c_v \times (1200)}{\left(\frac{0.02}{2}\right)^2} \Rightarrow c_v = 1.63 \times$$

$$0.196 = \frac{1.63e^{-8} \times T}{(1.5)^2} = 5313.13 \text{ youm}$$

2. How long would it take the layer to reach 30% consolidation?

4 $T_v = \frac{\pi}{4}(0.3)^2 = 0.07$ / $0.07 = \frac{1.63e^{-8} \times T}{(1.5)^2}$

$= 111.83$ youm.

Problem # 3 (10 points)

A long slope is to be formed in a soil of unit weight 19 kN/m3 for which the characteristic shear strength parameters are C' = 0 and φ' = 34. A firm stratum lies below the slope. It is to be assumed that the water table may occasionally rise to the surface, with seepage taking place parallel to the slope.

(1) Determine the maximum slope angle to ensure a factor of safety along the slip plane 1.5, assuming a potential failure surface parallel to the slope

4 $m = 1$

$$1.5 = F.S = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\phi}{\tan\beta}, \quad \gamma' = 19 - 9.8$$

(2) What would be the factor of safety of the slope formed at this angle if water table if the water table were well below the bed rock level?

4

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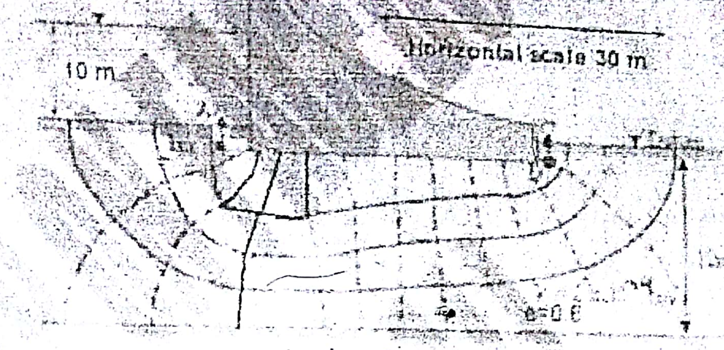
Problem # 4 (5 points)

For the flownet drawn for a dam shown.



1. Determine the amount of seepage per unit length of the dam in m³/sec.

$q = k \, \Delta H \, \frac{N_F}{N_D}$

2. Determine the pore water pressure at point D?

$\frac{\Delta H}{N_d}$

$U = (\Delta H - N_{dp} \, \Delta h + Z) \gamma_w$

Problem # 5 (10 points)

Determine the total ... pore water pressure and effective stress ...

$\gamma_{eff} + \gamma_w = \gamma_{sat}$

2.5 γ_w

$(\gamma_{clay}) + 1.5 (\gamma$ ...$) +$

~~4 (...) + 3.2 (...)~~

total

1.5 (γ_{clay}) + ... ($\gamma_w + \gamma_{eff}$) ...

+ ... 11.5 γ_w ...

ابو عجمية

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

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

(4 − 1.5)

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Hashemite University
Department of civil Engineering

Geotechnical Engineering
Final Exam (Out of 50 points)
Tuesday 15/5/ 2011
Duration: 2 H: 00

Student Name: ....
Student Number: ....

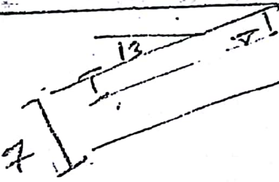
All the answers should be in the given space and *no partial credit* will be given

Problem # 1 (10 points)

A long natural slope in an overconsolidated fissured clay of saturated unit weight 19.8 kN/m^3 and a dry unit weight of 16.5 kN/m^3 is inclined at 13° to the horizontal. A slip has developed on a plane parallel to the surface at a depth of 7 m.

(1) Determine the factor of safety along the slip plane using effective stress parameter $c'=10$ kPa $\phi'=24°$ If water table is at depth 2 m below the surface and seepage is roughly parallel to the slope.

أبو عجمية



10

$$F.S = \frac{c' + (\sigma_n - u)\tan\phi'}{[(1-m)\gamma + m\gamma_{sat}]Z \sin\beta\cos\beta}$$

$\frac{2}{7}$

10 - 7

$0 < m < 1$

$m = 1 \quad \frac{\gamma}{\gamma_{sat}} \quad \frac{\tan\phi}{\tan\beta}$

(2) [illegible] strength parameter [illegible]

أبو عجمية

$m = 1$

$m = 0 \quad \frac{\tan\phi}{\tan\beta}$

$$\boxed{m = 1 \quad \frac{\gamma}{\gamma_{sat}} \quad \frac{\tan\phi}{\tan\beta}}$$

$\sigma =$

$F.S = \frac{T_R}{T_D}$

$\sigma = \quad \cos^2\beta$

$u = m\gamma_w Z \cos^2\beta$

الصغير

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Problem # 2 (10 points)

Saturated clay samples were tested in a series of consolidated undrained tests with pore water pressure measurement. The results shown in the following table:

σ_3 All round pressure (kPa)	σ_1	Principal stress difference (kPa) σ_d		Pore water pressure (kPa) σ_u
50 ← 150	250 ← 350	+ 200		60
125 ← 300	475 ← 650	+ 350		175
200 ← 450	750 ← 1000	+ 550		250

Determine:

1. The Mohr Coulomb strength parameters in terms of effective stresses (C' and ϕ)

$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$

$\sigma_f = \sigma_3 \tan^2(45 + \frac{\phi}{2}) + 2c \tan(45 + \frac{\phi}{2})$

$\sigma_f \quad \tau_f$

2. Orientation of failure plane for the last sample tested

$\sigma_f = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta$ / $\tau_f = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta$

Problem # 3 (20 points)

The slope shown is 6.0 m height and has a ratio of 1.00 vertical on [illegible] Horizontal. The slope was excavated [illegible] over consolidated clay. The undrained shear strength of soil is [illegible] kPa. The residual effective cohesion is 10 kPa and the internal residual friction angle is 25°, (γ = 20 kN/m³) [illegible] using fellinius' method [illegible] (Swedish method) or simplified Bishop)

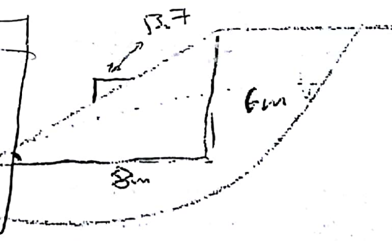
$\phi = 25°$ 90 kPa

1. Determine the Factor of safety at the end of excavation
2. Determine the Factor of safety at a very long time after excavation

[illegible] use only a four slice and the information for these slice are [illegible]

Slice	b m	h m	W kN	θ deg
1	3.5	[illegible]	[illegible]	[illegible]
2	3.5	2	[illegible]	30
3	[illegible]	[illegible]	[illegible]	[illegible]
4	4.5	3.5	[illegible]	[illegible]

W W sin



$FS = \frac{\sum [c' b_i + \sum (W_i (1 - r_u)) \tan\phi'] / m}{\sum W_i \sin\theta_i}$

$FS = \frac{\sum (S_u) \frac{b_i}{\cos\theta_i}}{\sum W_i \sin\theta_i}$

$FS = \frac{\sum [c' b_i + \sum W_i (1 - r_u) \tan\phi'] / m}{\sum W_i \sin\theta_i}$

$FS = \frac{\sum (S_u b)}{\sum (W_i \tan\theta_i)}$

$r_u =$

65

a x

Do not write back of the page; it will not be graded

Geotechnical Engineering
Final Exam (Out of 50 points)
Tuesday 4/1/ 2009
Duration: 2 H:00

Hashemite University
Department of civil Engineering

Student Name: ... .
Student Number: ... ..

All the answers should be in the given space and *no partial credit* will be given

Problem # 1 (16 points)

Saturated normally consolidated clay was consolidated under a stress of 165 kPa, then sheared undrained in axial compression. The principal stress difference at failure was 124 kPa, and the induced pore water pressure at failure was 107kPa Determine:-

1. The Mohr-Coulomb strength parameters in terms of total stresses (C and ϕ_T)

$\Delta\sigma = 124 \quad \bar{\sigma}_3 = 165 \quad \sigma_1 = 165 + 124 = 289$

$c = \bar{c} = 0$

$\sin\phi = \dfrac{\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{c}\cot\phi + \frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)} = 15.56 = \phi$

2. The Mohr-Coulomb strength parameters in terms of effective stresses (C' and ϕ')

$\bar{\sigma} = \sigma - U \Rightarrow 289 - 107 = 182 = \bar{\sigma}_1$

$\sigma_3 = 165 - U = 58$

أبو عجمية

$\boxed{\sin\theta = \dfrac{\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{c}\cot\phi + \frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)}}$

124

[illegible]

$\sin^{-1} = \phi$

3. The angle of failure plane

$\theta = 45 + \dfrac{\phi}{2} \Rightarrow 113.6$

[illegible]

Normal stress at failure

4. Effective normal stress at failure

$\bar{\sigma}_f = \frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3) + \frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)\cos 2\theta$

5. Shear stress at failure

$\tau_f = \frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)\sin 2\theta$

effective [illegible]
total [illegible]

6. pore water pressure parameters A and B at failure

$\tau = \bar{c} + (\sigma_{Total} - U)\tan\phi$

7. the slope of the effective K line

8. the intercept of the effective k_f line

Do not write back of the page; It will not be graded

Problem # 2 (10 points)

Saturated clay samples were tested in a series of consolidated undrained tests, with pore water pressure measurement. The results shown in the following table:

All round pressure (kPa)	Principal stress difference (kPa)	Pore water pressure (kPa)
150	160	100
300	330	175
450	560	250

Determine:

1. The Mohr-Coulomb strength parameters in terms of effective stresses (C' and ϕ')

$\sigma \to \sigma'$

2. Orientation of failure plane for the last sample tested $u = 0$

$\tau = 0$

$\tau = C + \sigma$

Do not write back of the page; It will not be graded

Problem # 3 (10 points)

Saturated ~~clay~~ samples were tested in a series of consolidated undrained tests, with pore water pressure measurement. The results indicated an angle of internal friction of 15° and an cohesion of 50 kPa.

$\phi = 15$

~~Determine:~~

3. Normal stress at failure if identical soil sample tested under confining pressure of 200 kPa.

$\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi}{2}) + 2c\tan(45 + \frac{\phi}{2}) \to \sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\theta$

4. Shear stress at failure if identical soil sample tested under confining pressure of 200 kPa.

$\tau = \frac{\sigma_1 - \sigma_3}{2}\sin 2\theta$

$\phi \quad c = 50$

Problem # 4 (10 points)

A layer of normally consolidated clay 8m thick lies below a 4m depth of sand, the water table being at the surface. moreover anther layer of sand present below the clay layer. The clay has a compression index of 0.2500, recompression Index of 0.0225, water content=40% and specific gravity of 2.7. The saturated unit weight for both soils (sand and clay) is 19kN/m³. A 4m depth of fill of unit weight 21 kN/m³ is placed on the sand over an extensive area.

1- Determine the final settlement due to consolidation of the clay

2. If the fill were to be removed some time after the completion of consolidation, what heave would eventually take place due to swelling of the clay?

sand 4

clay 8

أبو عجمية

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

$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\theta$

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

Problem 1)

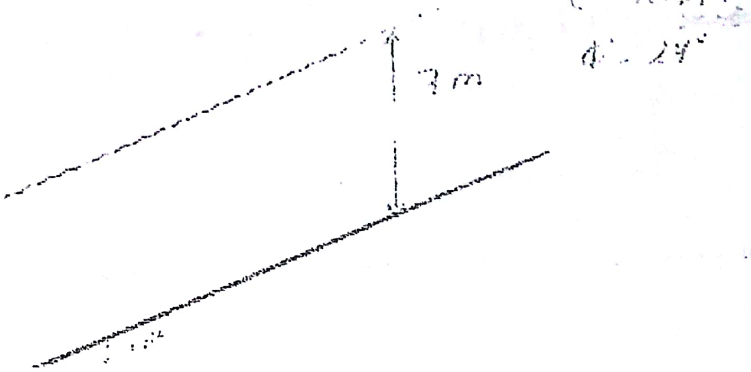
Soil → C C Fissured Clay

$\gamma_{sat} = 19.5\ kN/m^3$

$\gamma_{dry} = 16.5$

$\beta = 13^\circ$

$H = 7\ m$



1] WT @ 2m ⇒ find F.S

2] S [illegible] ⇒ find F.S

rob.lem (2)

CU test

1

2

Abu ajameyeh

Abu ajameyeh

Problem (3)

→ CU test

$\phi = 15^\circ$

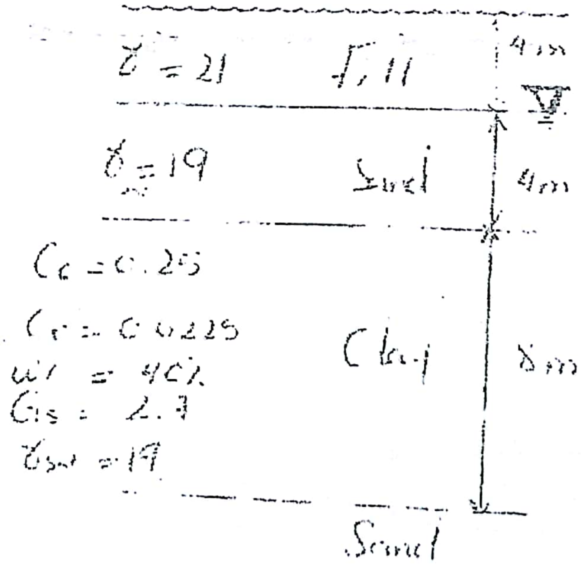
$C = 50$ kPa

3

4

Problem(4)

→ NC Clay



Problem (5)

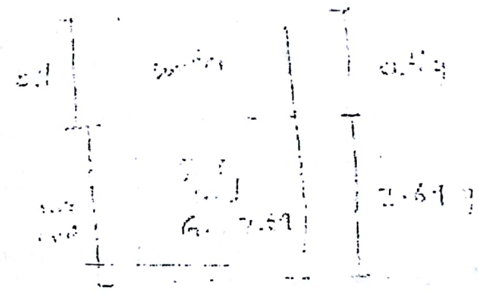
$\boxed{1}$ γ_{sat} Sand

$G_s = 2.69$ [illegible]

[illegible]

[illegible]

$\gamma_{sat} =$ [illegible] $\times 9.81 = \boxed{21.65}$



$\boxed{2}$

Abu ajameyeh

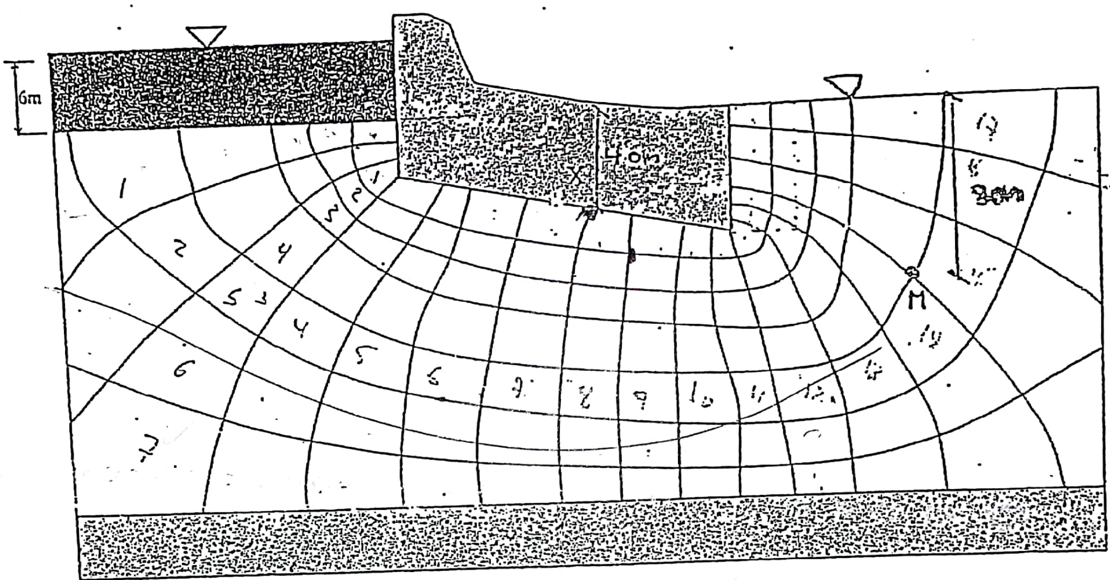
Karam M. Azzeh

Faculty of Engineering
Department of Civil Engineering

Geotechnical Engineering Laboratory
Instructors: Eng. Hussien Aldeeky
Second Exam first Semester 2012
Duration 60 Minutes
Monday Dec 05, 2012

Student Nam : KEY Solution. Student No. First 2012|2013
Section No: Set NO:

Q1): (5 point)
e dam and flow net shown below .The dam is 50 m long and has 9.6 m Sheet piles driven into the granular l layer with (Gs=2.65 , e= 0.82 ,) and coefficient of permeability = 2×10^{-6} m/sec). The datum at tail water vation.



$N_f = 7$
$N_D = 17$

1- Find the quantity of seepage (q) (2 point)

$q = \Delta H \times k \times \frac{N_f}{N_D} = 6.0 \times 2\times10^{-6} \times \frac{7}{17} = \boxed{4.94\times10^{-6}\ m^3/sec/m}$

2- Find the pore water pressure at point X (1 point)

$U = (h_p - h_z)\gamma_w = [\Delta H - N_D \frac{\Delta H}{N_D} - h_z]\gamma_w = [6.0 - 7 \times \frac{6}{17} + 4.5]\times 9.806 = \boxed{78.833\ kN/m^2}$

↳ total

3- Find the critical hydraulic gradient (1 point)

$i_{cr} = \frac{G_s - 1}{1+e} = \frac{2.65 - 1}{1 + 0.82} = \boxed{0.907}$

4) Find the effective stress (σ') at point M (1 point)

$\gamma_{sat} = \gamma_w \left(\frac{G_s + e}{1+e}\right) = 9.806\left(\frac{2.65 + 0.82}{1.82}\right) = 18.70\ kN/m^3$

$U_M = (6.0 - \frac{14}{17} \times 6.0 + 8.0) \times \gamma_w = 88.83\ kN/m^2$ 0.5

$\therefore \sigma' = (18.7 \times 8) - 88.83 = \boxed{60.77\ kN/m^2}$

0.5

Q2) (2 point)

The uniformly distributed loads of 80 kN/m^2 applied on the following footing .Determine the vertical :ess($\Delta\sigma v$) At a depth of 4 m below point A

$\Delta\sigma v = q \times [I_{r1} - I_{r2} + I_{r3}]$

$I_{r1} \Rightarrow m = \frac{16}{4} = 4.0$

$n = \frac{20}{4} = 5.0 \Rightarrow I_{r1} = 0.25$

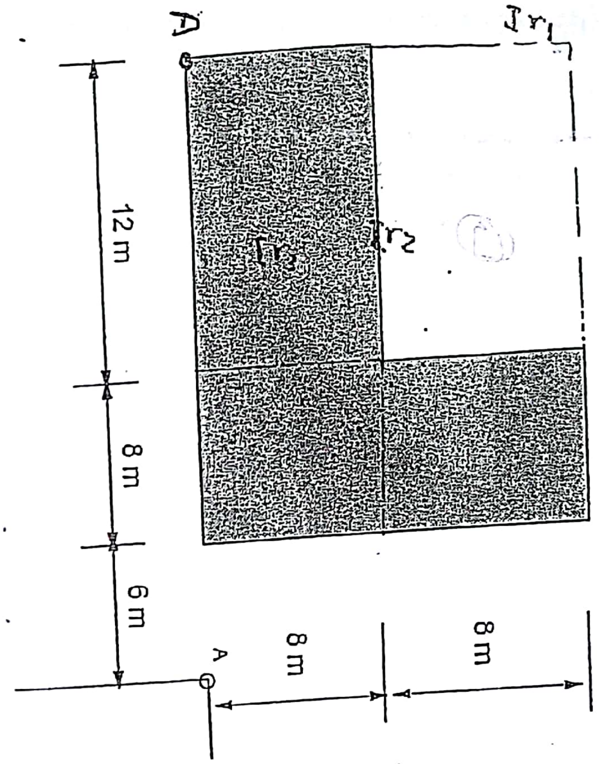
$I_{r2} \Rightarrow m = \frac{16}{4} = 4.0$

$n = \frac{12}{4} = 3.0 \Rightarrow I_{r2} = 0.25$ 1

$I_{r3} \Rightarrow m = \frac{8}{4} = 2.0$

$n = \frac{12}{4} = 3.0 \Rightarrow I_{r3} = 0.24$

$\Delta\sigma v = 80 \times [0.25 - 0.25 + 0.24]$ 1 $= 19.2\ kN/m^2$



Q3 (2 point)

For the figure shown below . Determine the vertical stress ($\Delta\sigma v$) due to the Strip footing at point P.

$\tan\beta = \frac{3}{5} \Rightarrow \beta = 30.96^\circ$

$\tan(\alpha+\beta) = 8/5 \Rightarrow \alpha+\beta = 57.99^\circ$

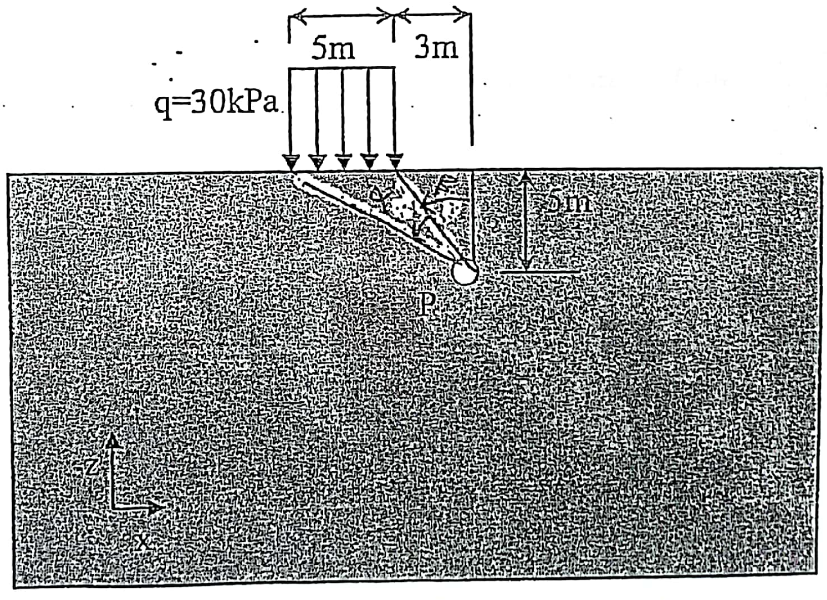
$\therefore \alpha = 27.05^\circ \Rightarrow \alpha = 0.472\,rad.$

$\therefore \Delta\sigma v = \frac{q}{\pi}[\alpha + \sin\alpha\cos(\alpha+2\beta)]$

$= \frac{30}{\pi}[0.472 + \sin 27.05 \cos(27.05 + 2\times(30.96))]$ 1

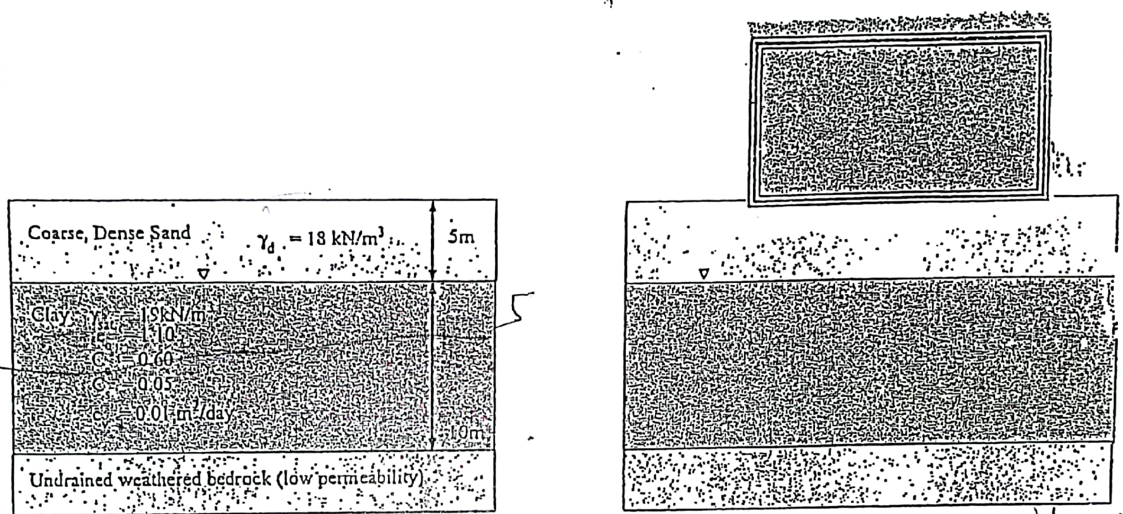
1

$\boxed{= 4.585\ kpa}$



Q3 : (11 points)

ɡure 2a shows a two-layered soil system in which a dry sandy soil overlies a
ʏ soil with preconsolidation stress(maximum past pressure) level of (σc') 300 kN/ m², which in turn overlies
.yer of low-permeability, un drained bedrock.An ethanol storage tank of diameter 15 m and gross stress(Δq)= 140
) kN/ m² is to be constructed on the site. shown in Figure 2b.



a) b)

σ'_{v0} stress من نفس التربة

$\Delta\sigma_v$ stress من الحمل الخارجي

A) Find the OCR and soil type. (1 point)

$$\sigma'_{vo} = 18 \times 5 + \frac{10}{2} \times (19 - 9.806) = 135.97 \text{ kN/m}^2$$

0.5

$$OCR = \frac{\sigma'_c}{\sigma'_{vo}} = \frac{300}{135.97} = 2.2071 \ldots \text{O.C clay}$$

0.5

B). What is the average vertical stress increase(Δσv) in the clay layer directly beneath the centere of the
ooting at the middle of clay layer . (2 point)

تحت

$$\Delta\sigma_v = q \times I_c = 140 \times I_c = 140 \times 0.6 = 84 \text{ kN/m}^2$$

1

$$I_c \Rightarrow \frac{D}{Z} = \frac{15}{10} = 1.5 \Rightarrow I_c = 0.6$$

1

$$I_c = \frac{D}{Z}$$

C) Find the expected primary consolidation settlement (2 point) :

$\sigma'_{v0} = 135.97$ $\sigma'_c = 300$

$$\sigma_f = \sigma'_{vo} + \Delta\sigma_v = 135.97 + 84 = 219.97 \text{ kN/m}^2$$

$$\therefore S_c = \frac{C_r H}{1+e_0} \log \frac{\sigma_f}{\sigma'_{vo}} = \frac{0.05 \times 10}{1+1.10} \cdot \log \frac{219.97}{135.97}$$

1

$$= 4.97 \text{ cm}$$

1

D) Find the length of time in years required for settlement about 40% from primary settlement. (2 point)

$$TV = \frac{\pi}{4} U^2 = \frac{\pi}{4}(.4)^2 = 0.1257 \quad 1$$

$$\therefore TV = \frac{Cv \times t}{(Hdr)^2} = \frac{0.01 \times t}{(1100)^2} \Rightarrow t = 1257\ day \quad 1 \quad 3.54\ years$$

??

E) .Find the length of time in years required for settlement about 80% from primary settlement. (2 point)

$$TV = -0.933 \log(1-U) - 0.085 = -0.933 \log(1-0.8) - 0.085 \quad 1 \quad 0.5671$$

$$0.5671 = \frac{Cv \times t}{(Hdr)^2} \Rightarrow t = 5671.39\ day \quad 1 \quad 15.54\ years$$

(F). Answer with a TRUE (T) or a FALSE (F) to the following statements. If you answer with a false, state the **correct statement.. (2 point)**

1- Taylar Construction method (The square-root of time method) is a graphical procedure used for determination of preconsolidation pressure.(F) 0.5

For determination the coeff. of consolidation (CV). 0.5

2- The settlement of a soil in an over consolidated condition is grater than that in the normally consolidated condition (F) 0.5

settlement in o.c clay less than N.C clay. 0.5

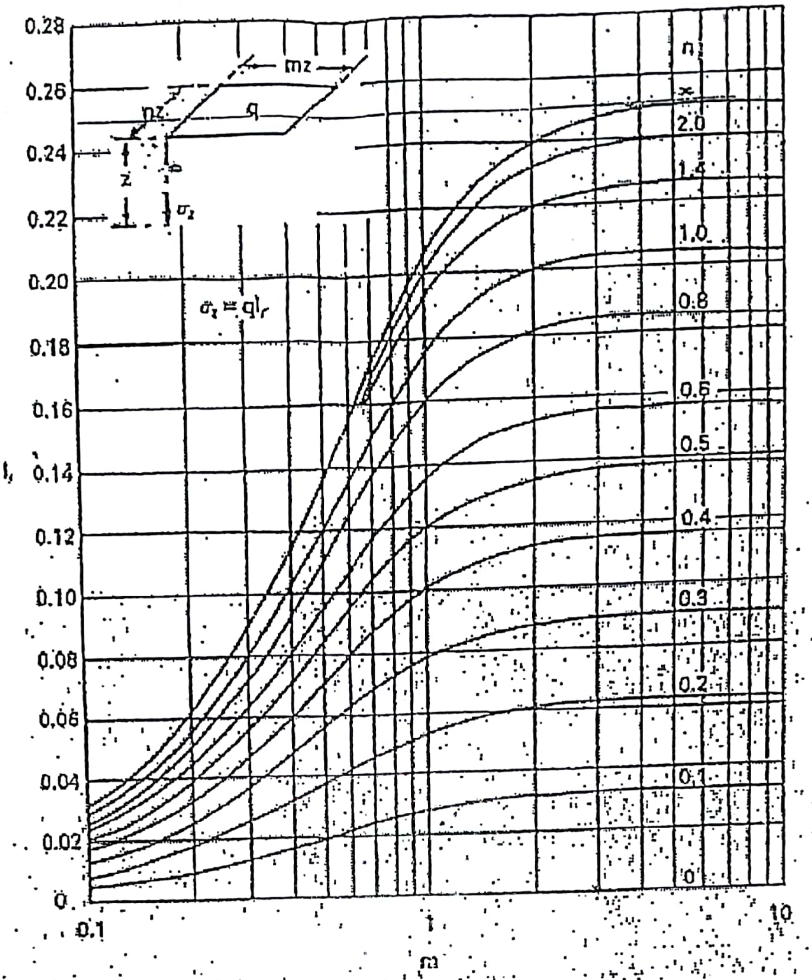


Figure 5.10 Vertical stress under a corner of a rectangular area carrying a uniform pressure. (Reproduced from

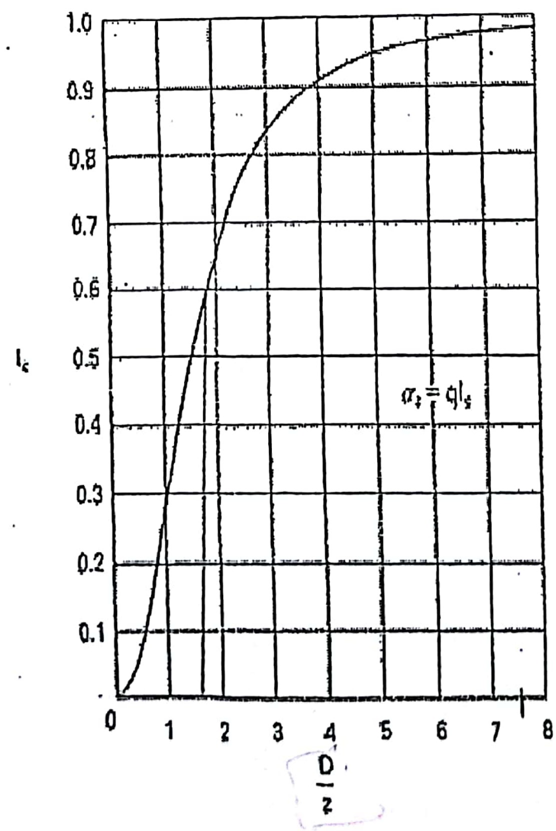


Figure 5.9 Vertical stress under the centre of a circular area carrying a uniform pressure.

Faculty of Engineering
Department of Civil Engineering

Geotechnical Engineering [illegible]
Instructors: Eng. [illegible]
Second Exam Fall Semester 2011
Duration 50 Minutes
Sunday Des 11, 2011

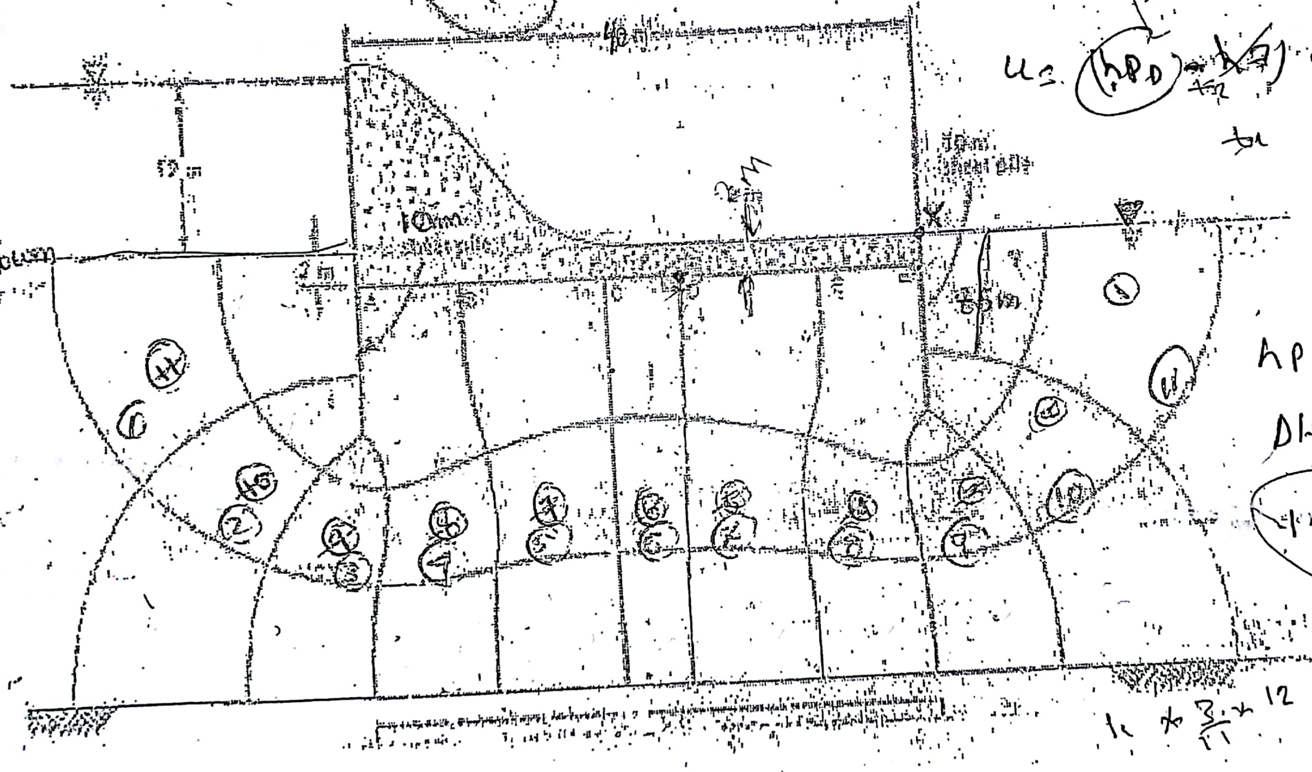
Student Name: ______
Section No:

Student No.
Set NO:

19.5/20

Q1):
The dam and flow net shown below. The dam is 120 m long and has 10 m Sheet pile driven partially into the granular soil layer with (Gs=2.68 , e=0.9) and coefficient of permeability = 20×10⁻⁴ cm/sec. The datum at tail water elevation.

[scale : 3]



$u = (h_{P_D}) \cdot \gamma_w$

$h_P = \Delta H - N_D \dfrac{\Delta H}{N_D}$

$12 - 6 \dfrac{12}{11}$

1- Find the quantity of seepage loss under the dam (3 point)

$q = k \dfrac{\Delta H}{N_d} N_f = 20 \times 10^{-4} \times 10^{-2} \times \dfrac{12}{11} \times 3 = \boxed{6.55 \times 10^{-5}}$ m³/s/m

2- Find the exit gradient (at point x) (2 point)

$i_{exit} = \dfrac{\Delta h = \frac{\Delta H}{N_d}}{L_{exit}} \quad \dfrac{12/11}{8.5} = \boxed{0.128}$

$\dfrac{\Delta L}{L} \qquad \dfrac{\Delta h \to \frac{12}{11}}{L} \qquad \dfrac{\frac{12}{11}}{8.5}$

3- Find the pore water pressure at point D (2 point)

$u = (\Delta H - N_d \dfrac{\Delta H}{N_d} - z)\gamma_w = 12 - 6\left(\dfrac{12}{11}\right) - 2 = (7.45) \times 9.81 = \boxed{73.13}$

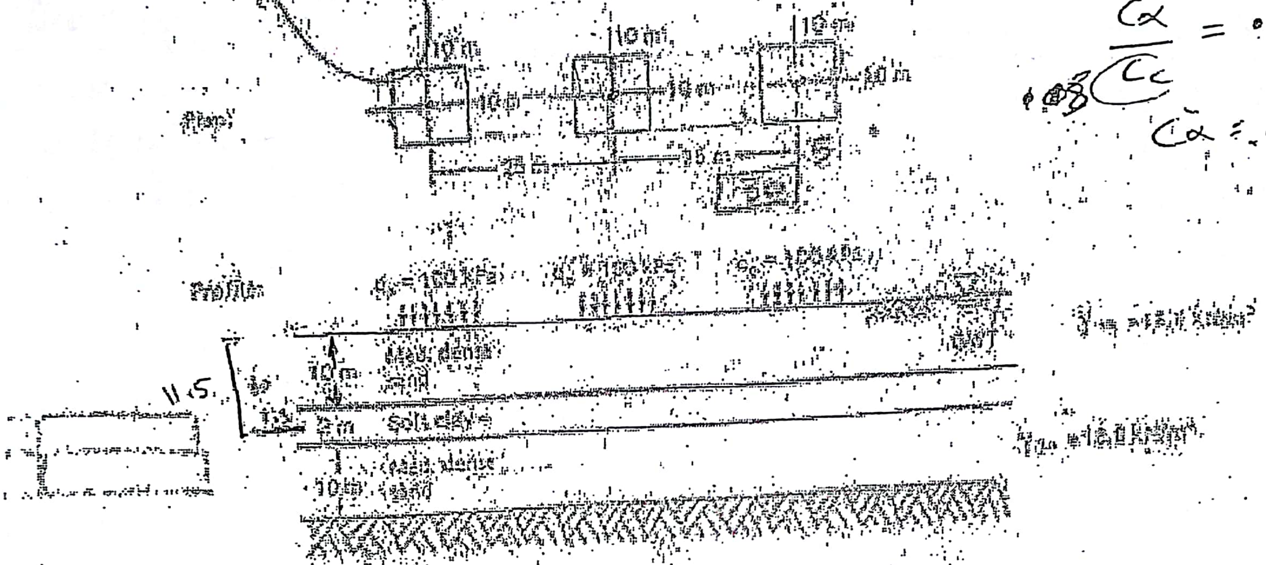
4- Find the critical hydraulic gradient (2 point)

$i_{cr} = \dfrac{G_s - 1}{1 + e} = \dfrac{2.68 - 1}{1 + 0.9} = \boxed{0.88}$

Q2) Three uniformly distributed loads of $\boxed{100\ kpa}$ each are applied to 10X10 m square areas on the soil profile shown below. Undisturbed sample of the clay were taken prior to construction, and consolidation tests indicated that ($\sigma_c' = 200$ Kpa , $Cc = 0.5$, $Cr = 0.02$, $Cv = 0.89\ m^2/yr$, $e_0 =$ [illegible] , $C_\alpha/C_c = 0.04$, $\gamma_{sat} =$ 17 kn/m3) — 10 yr

$\frac{C_\alpha}{C_c} = .04$

$C_\alpha = .02$



A) Determine the stress under the center of middle loaded area due to the three footing at the middle of clay layer ($\Delta\sigma'$). (2 point)

$\boxed{z = 11.5}$

$4\left(\begin{matrix} m = 20/11.5 = 2.61 \\ n = 5/11.5 = .43 \end{matrix}\right) - 4\left(m = 5/11.5 = .43\right) + 4\left(n = 5/11.5 = .43\right)$

2

$4(100)(0.12) - 4(100)(0.117) + 4(100)(0.112)$

$48 - 46.8 + 44.8$

$\boxed{\sigma_z = 46}$

B) Determine the effective overburden pressure at the middle of clay layer (σ_{vo}'). (2 point)

$\sigma_{vo}' = 10(18.8 - 9.81) + 1.5(17 - 9.81) =$

$89.9 + 10.785 = \boxed{100.69}$

c) Find the value of OCR and soil type (1 point)

$OCR = \frac{\sigma_c'}{\sigma_{vo}'} = \frac{200}{100.69} = 1.986 > 1.0$ over Consolidated Soil

D) Estimate the primary consolidation settlement (use $\Delta\sigma = 150$ kpa) (3 point)

$S_c = \sigma'_c \ldots \Delta\sigma = \ldots \times 150 = \ldots$

$\sigma'_o < \sigma'_c < \sigma'$

$S_c = \frac{C_r}{1+e_o} H \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c}{1+e_o} H \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_c}\right)$

$= \ldots = 0.11 \ldots$

E) Estimate the secondary consolidation settlement that would occur from 2.5 years to 10 years. (2 point)

$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right)$ $C'_\alpha = 0.04(\ldots)$ $= 0.04(0.5) = 0.02$

$= (0.02)(3)\log\left(\frac{10}{2.5}\right)$

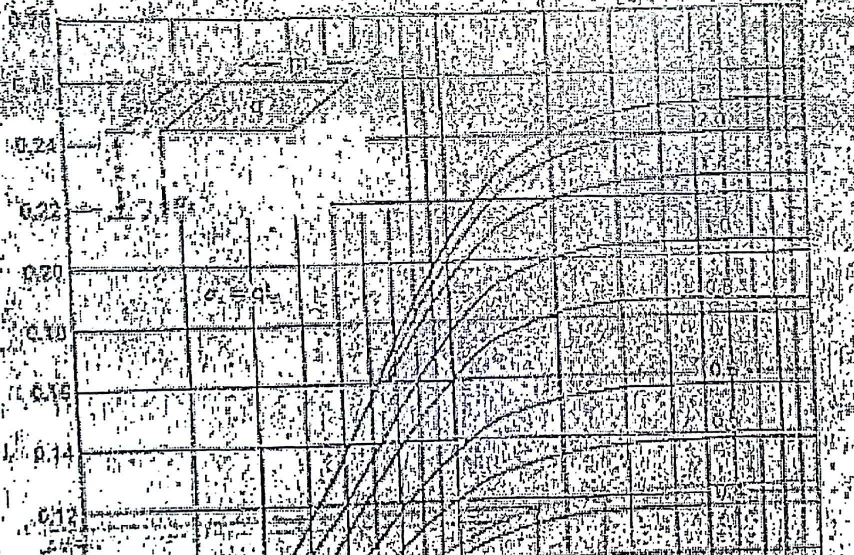
$= (0.02)(3)(0.602) = \boxed{0.027\ m}$

F) How long of time it needs to reach 90% of total settlement (2 point)

$S_t = 0.9(S_{tot}) = 0.9(0.11 + 0.027) = 0.123$

$U = \frac{S_c}{S_t} = \frac{0.11}{0.123} = 0.89$ $T = \frac{c_v t}{H_{dr}^2} = 0.627 = \frac{\ldots}{(3/2)^2}$

$T = \frac{\pi}{4} U^2 \ldots$ $\boxed{t = 1.5\ldots}$



20.5

28

Faculty of Engineering
Department of Civil Engineering

Geotechnical Engineering Laboratory
Instructors: Eng. Hussien Aldeeky
Second Exam Second Semester 2013
Duration 50 Minutes

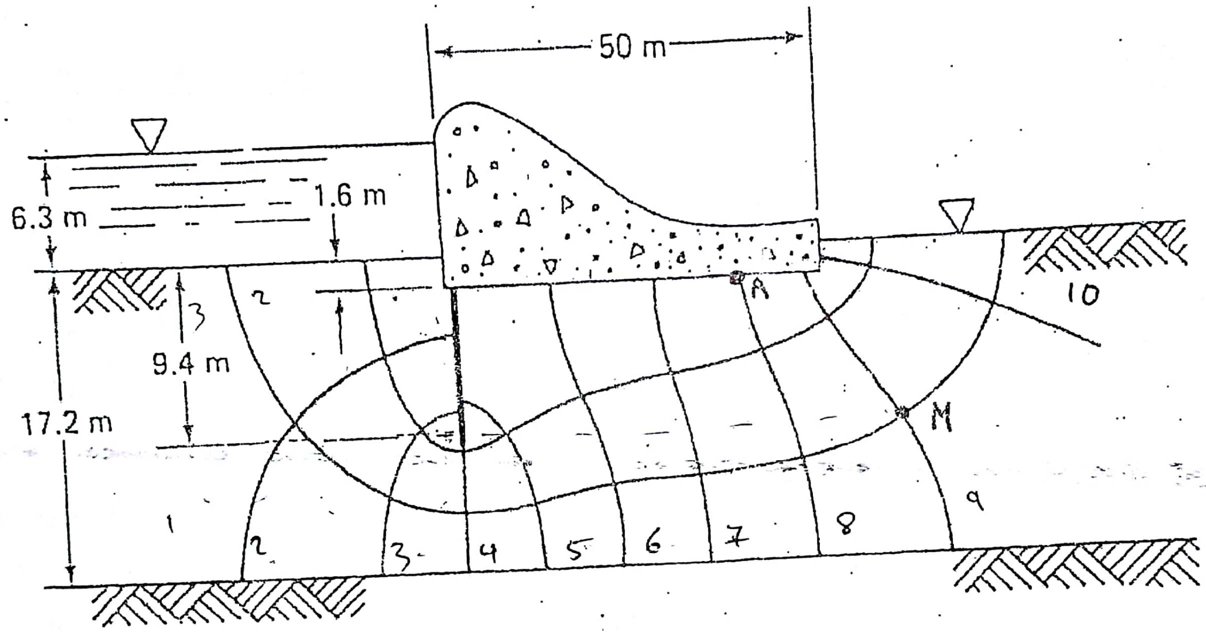
Student Name: علاء محمد عيسى حسون 90

Student No. 114166

Section No: 6

Set NO: 62

Q1): (8 point)
The dam and flow net shown below .The dam is 50 m long and has 9.6 m Sheet piles driven into the granular soil layer with $\gamma sat = 20 KN/m^3$ and coefficient of permeability ($K_x = 2\times10^{-6}$ m/sec $K_z = 0.5\times10^{-6}$ m/sec)
The datum at tail water elevation.



1- Find the quantity of seepage (q) (2 point)

$K = \sqrt{K_x K_z} \Rightarrow \sqrt{2\times10^{-6} \times 0.5\times10^{-6}}$

$\bar{K} = 1\times10^{-6}$

2 $q = K \Delta H \frac{N_f}{N_d} \Rightarrow 1\times10^{-6} \times 6.3 \times \frac{3}{10} \Rightarrow \boxed{1.89\times10^{-6}}$

2- Find the pore water pressure at point A (2 point)

$\Rightarrow (\Delta H - N_d \cdot \Delta h - h_z)\gamma_w \Rightarrow (6.3 - (7 \times \frac{6.3}{10}) + 1.6) \times 9.81$

2 $\Rightarrow \boxed{34.23}$

3- Find the critical hydraulic gradient (2 point)

$i_{cr} = \frac{G_s - 1}{1+e}$ 0.0

4) Find the effective stress (σ') at point M (2 point)

2 $\Delta \sigma_v = (20 \times 9.4) = 188$ KN/m

$U = [6.3 - (8 \times \frac{6.3}{10}) + 9.4] \times 9.81 \Rightarrow 104.5$

$\Rightarrow \sigma' = 188 - 104.5 = 83.4$

Q2) (3 point)

The uniformly distributed loads of **80 kpa** applied on the following footing .Determine the vertical stress($\Delta\sigma v$) At a depth of 10 m below point A

$m_1 = \frac{B}{z} = \frac{8}{10} = 0.8$ $I_{r_1} = 0.19$

$n_1 = \frac{L}{z} = \frac{20}{10} = 2$

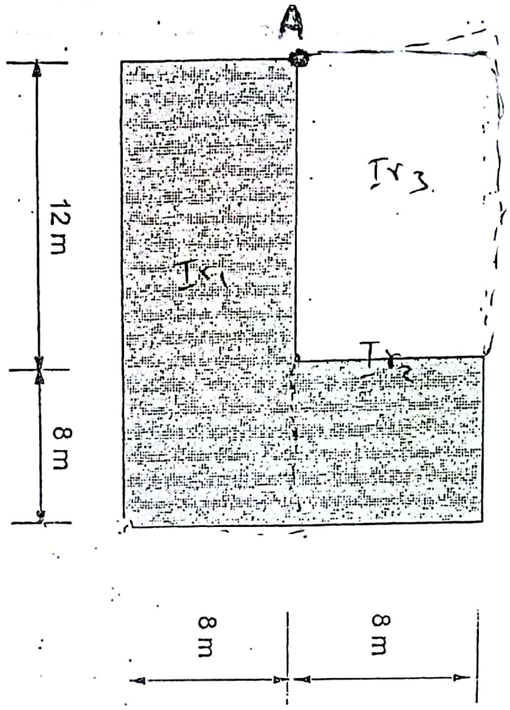
3

$m_2 = \frac{B}{z} = \frac{8}{10} = 0.8$ $I_{r_2} = 0.19$

$n_2 = \frac{L}{z} = \frac{20}{10} = 2$

$m_3 = \frac{B}{z} = \frac{8}{10} = 0.8$

$n_3 = \frac{B}{z} = \frac{12}{10} = 1.2$ $I_{r_3} = 0.17$



$\sigma \Rightarrow q(I_{r1} + I_{r2} - I_{r3})$

$\Rightarrow 80(0.19 + 0.19 - 0.17) \Rightarrow 16.8$

Q3 (3 point)

An ethanol storage tank with a circular foundation diameter (D)= 15m , find the vertical ~~depth~~ stress at level 10m Below the ground surface . if the footing carrying a uniform stress = $140 KN/m^2$.

$\sigma = q * I_c$ 3

$\sigma = 140 \times 0.45 \Rightarrow 63$

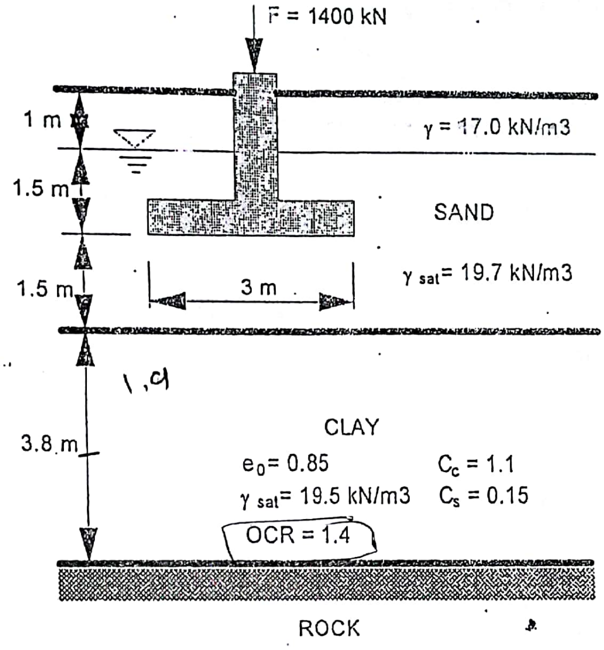
$\frac{D}{z} = \frac{15}{10} = 1.5$

at 1.5 $I_c = 0.45$

8.5

Q3 : (12 points)

The figure below show a square footing with dimension 3m×3m using the coefficient of consolidation (CV = 4.3 m^2/year)



A) Find. the effective overburden pressure at the middle of clay layer (σ'vo) (2 point)

$\sigma_{v_o} = (17\times1)+(19.7-9.81)(3)+(1.9)(19.5-9.8) \Rightarrow$

2 $17+\cancel{84.01}+18.43=65.1$
29.67

B). What is the average vertical stress increase(Δσv) in the silt clay layer directly beneath the centerline of the strip loading using approximate (2:1 method)? (2 point)

$q=\frac{1400}{3\times3}=155.5$ 1

$\Delta\sigma_v=\frac{q(B\times B)}{(B+z)(B+z)} \Rightarrow \frac{155\times3\times3}{2.5\times2.9} \Rightarrow \boxed{224}$

$(3.0+3.4)^2$

C) Find the expected primary consolidation settlement (2 point)

$\sigma_f=224+65.1=289.1$

$S_c=\frac{C_s H}{1+e}\log\left(\frac{\sigma_f}{\sigma_o}\right)=\frac{0.15\times3.8}{1+0.85}\log\frac{289.1}{65.1}\times$ Cr

0.75

$\Rightarrow 0.31\times0.65=\boxed{0.20}$

D) Find the length of time in years required for settlement about 40% from primary settlement. (2 point)

$T_v = \frac{C_v t}{d^2} \Rightarrow$ $T_v = \frac{\pi}{4} U^2$

$T_v = 0.125$

$0.125 = \frac{4.3 \times t}{(3.8)^2}$

2

$\Rightarrow \frac{1.805}{4.3} = \frac{4.3 t}{4.3} \Rightarrow 0.419$ year ~~years~~ $\Rightarrow$

E) Find the consolidation settlement after 6 month. (2 point)

$U = \frac{S_t}{S_c} \Rightarrow$

2

$0.43 = \frac{S_t}{0.20}$ $\boxed{0.086}$

$T_v = \frac{C_v t}{d^2} \Rightarrow \frac{4.3 \times 0.5}{(3.8)^2}$

$\Rightarrow 0.148$

$T_v = \frac{\pi}{4} U^2 \Rightarrow 0.148 = \frac{\pi}{4} U^2$

$U = 0.43$

F) Find the secondary consolidation settlement after 10 years, if the primary settlement occurs during 5 years use $C\alpha/Cc = 0.04$

(2 point)

$S_c = C_\alpha H \log \frac{t}{t_p} \Rightarrow \frac{0.044 * 3.8 * \log \frac{10}{5}}{1+e_p}$

$\Rightarrow \boxed{0.05}$

0.75

$\frac{C_\alpha}{1.1} = 0.04 \Rightarrow C_\alpha = 0.044$

$e_p = e_0 - \Delta e$??

Geotechnical Engineering Laboratory
Instructors: Eng. Hussien Aldeeky
Second Exam fall Semester 2011
Duration 50 Minutes
Sunday Dec 11, 2011

19.5/20

Faculty of Engineering
Department of Civil Engineering

Student Name:
Section No:

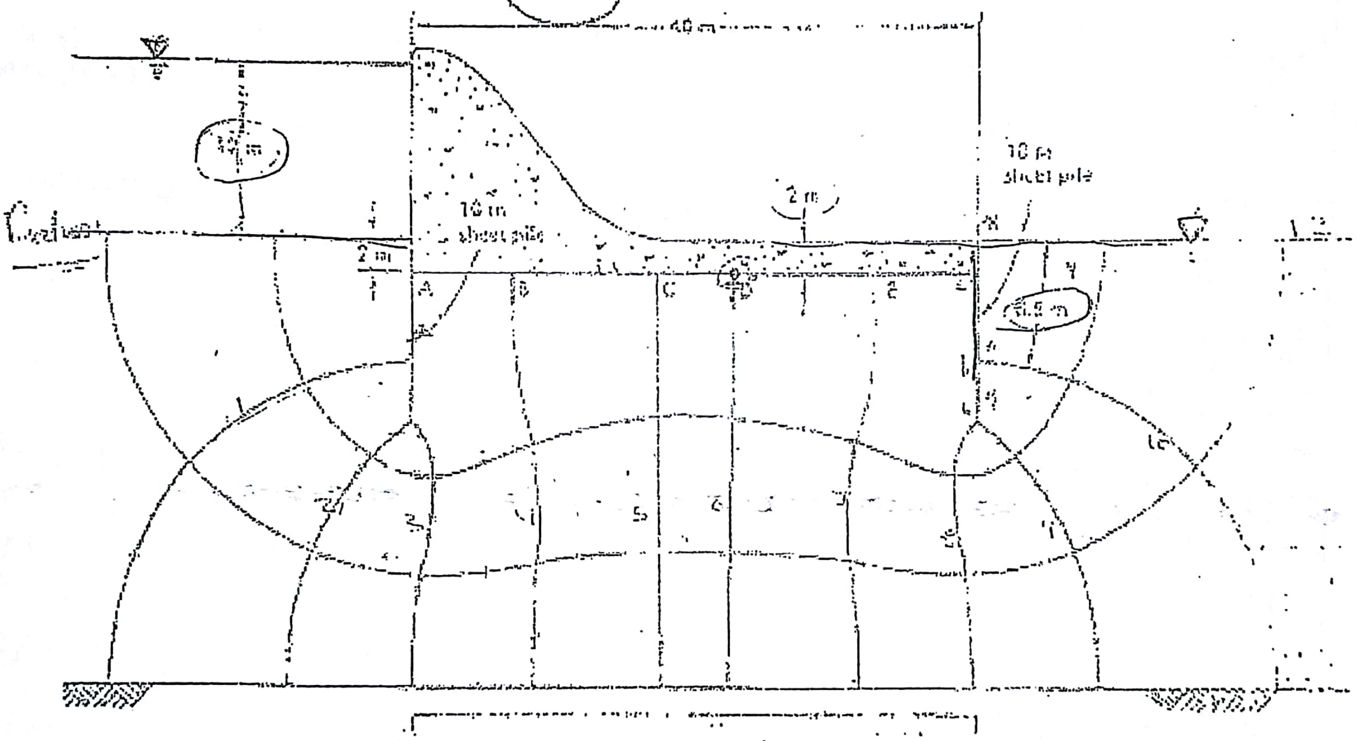
Excellent

Student No.
Set NO:

Q1):

The dam and flow net shown below. The dam is 120 m long and has tow 10 m Sheet piles driven vertically into the granular soil layer with (Gs=2.68, e= 0.9,) and coefficient of permeability = 20×10⁻⁴ cm/sec. Take datum at tail water elevation.

9



$q = k \frac{\Delta H}{N_d} * N_f$

$\Delta H - \left(\frac{\Delta H}{N_d} * N_{dj}\right) - 2$

1- Find the quantity of seepage loss under the dam (3 point)

$q = k \frac{\Delta H}{N_d} N_f = 20\times10^{-4}\times10^{-2} \times \frac{12}{11} \times 3 = 6.55\times10^{-5}$ m³/s/m

3

2- Find the exit gradient (at point x) (2 point)

$i_{exit} = \frac{\Delta h}{L_{exit}} = \frac{12/11}{8.5} = \boxed{0.128}$

2

$(h_p - h_z)\,\gamma_w$

3- Find the pore water pressure at point D (2 point)

$u = (\Delta H - N_{dj} \frac{\Delta H}{N_d} - z)\,\gamma_w = (12 - 6(\frac{12}{11}) - -2)$

$= (7.45)\,9.81 = \boxed{73.13}$

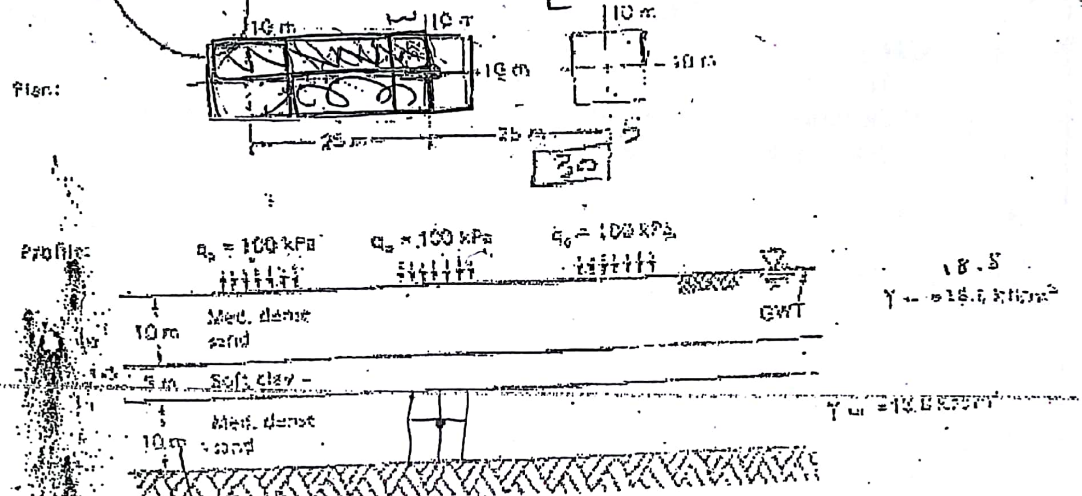
2

4- Find the critical hydraulic gradient (2 point)

$i_{cr} = \frac{G_s - 1}{1 + e_o} = \frac{2.68 - 1}{1.9} = \boxed{0.88}$

2

(Q2) Three uniformly distributed loads of 100 kpa each are applied to 10x10 m square areas on the ... shown below. Undisturbed sample of the clay were taken prior to construction and consolidation test indicated that ($\sigma c'$ = 200K pa, Cc = 0.5, Cr = 0.02, Cv = 0.89 m2/ yr, eo = 0.5..., ... 17 kn/m3,)



$m_1 = 15/11.5$
$n_1 = 35/11.5$
2.6

$m_2 = 10/11.5$
$n_2 = 10/11.5$

$\Rightarrow I_1 + (I_1 - I_2)$

A) Determine the stress under the center of middle loaded area due to the three loading ... layer ($\Delta\sigma$) (2 point)

$4\left(m = 25/11.5,\ n = 5/11.5\right) - 4\left(m = 15/11.5,\ n = 5/11.5\right) + 4\left(m = 5/11.5,\ n = 5/11.5\right)$

$4(100)(0.12) - 4(100)(0.117) + 4(100)(0.112)$

$48 - 46.8 + 44.8$

$\boxed{\sigma_z = 46}$

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

$\frac{100 \times (10 \times 10)}{(10 + 11.5)^2} = 21.63$

$21.63 \times 3 = 64.86$

B) Determine the effective overburden pressure at the middle of clay layer ($\sigma vo'$) (1 point)

$\sigma_{vo}' = 10(18.8 - 9.81) + 1.5(17 - 9.81) =$

$89.9 + 10.785 = 100.69$

$10(18.8 - 9.81) + 1.5(...)$

c) Find the value of OCR and soil type (1 point)

$\frac{\sigma_c'}{\sigma_{vo}'} = \frac{200}{100.69}$

$OCR = \frac{\sigma_c'}{\sigma_{vo}'} = \frac{200}{100.69} = 1.986 > 1.0$ over consolidated soil

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hussien

$C_\alpha H \log \frac{t_2}{t_1} = S_s$

D) Estimate the primary consolidation settlement (use $\Delta\sigma$ =150kpa) (2 point)

$\sigma_f = \sigma'_o + \Delta\sigma = 100.69 + 150 = 250.69$ kPa

$\sigma'_o < \sigma_c < \sigma'_f \Rightarrow 3$

$S_c = \frac{C_r}{1+e_o} H \log \frac{\sigma_c}{\sigma_o} + \frac{C_c}{1+e_o} H \log \left(\frac{\sigma_f}{\sigma_c}\right)$

$= \frac{0.02}{1+0.5} (3) \log \left(\frac{200}{100.69}\right) + \frac{0.5}{1+0.5} (3) \log \left(\frac{250.69}{200}\right) = 0.019 + 0.098 = 0.11$ m

E) Estimate the secondary consolidation settlement that would occur from 3.5 years to 10 years. (2 point)

$S_s = C_\alpha H \log \left(\frac{t_2}{t_1}\right)$

$(C_\alpha = 0.04 (C_c))$

$0.04 (0.5) = 0.02$

$= (0.02)(3) \log \left(\frac{10}{3.5}\right)$

$\frac{C_\alpha}{1+e_p}$

$S_s = C_\alpha H \log \left(\frac{t_2}{t_1}\right)$

$= (0.02)(3)(0.456) = 0.027$ m

F) How long of time it needs to reach 90% of total settlement? (2 point)

$S_{total} (S_{t_{90}}) = 0.9 (0.11 + 0.027) = 0.1233$ m

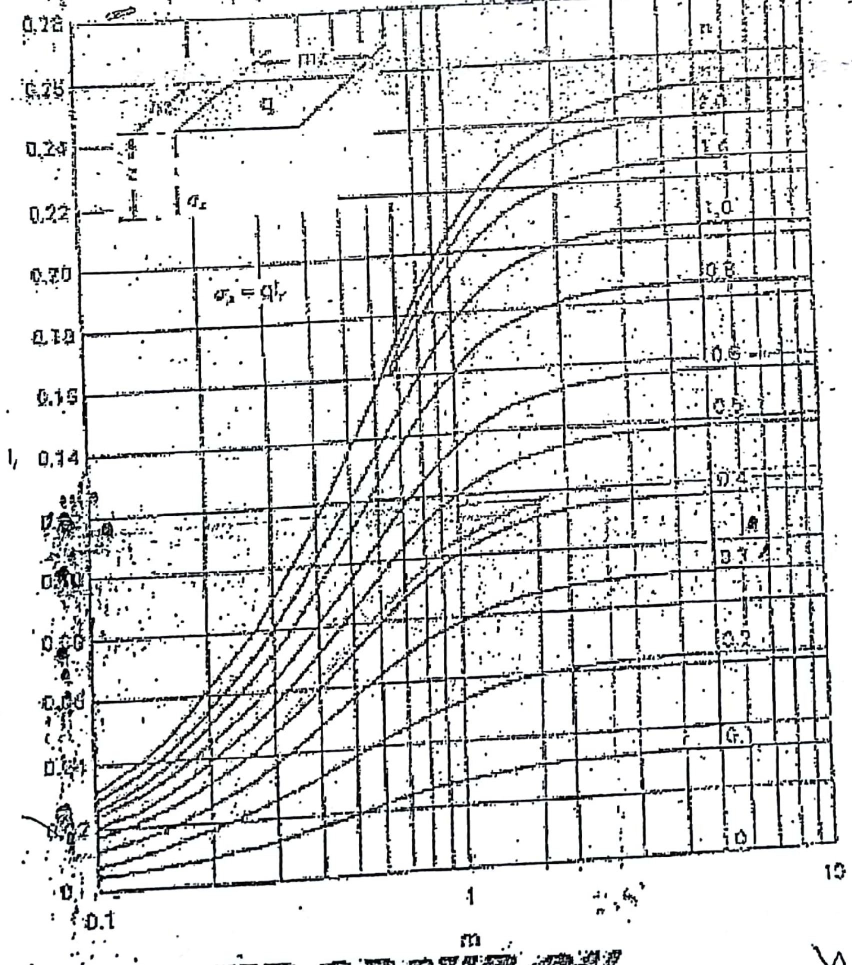
$U = \frac{S_t}{S_c} = \frac{0.1233}{0.138} = 0.89$

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

$0.628 = \frac{0.89 (t)}{(3/2)^2}$

$T = \frac{\pi}{4} U^2 = \frac{\pi}{4} (0.89)^2 = 0.622$

$t = 1.588$ years



$\frac{S_c}{S_{ct}} = 0.9$

$= 0.1233$

total sett.

$= S_c + S_s$

$= 0.11 + 0.027$

$U = \frac{S_{ct}}{S_{ct}}$

$U = \frac{0.12}{0.13}$

$U = \frac{0.1233}{0.11}$

$T_v = C_v t$

$U = \frac{S_c(t)}{S_{ct}} = \frac{0.1233}{0.11}$

$U = \frac{S(t)}{S(t)}$

$U = 0.9 \, S_t$

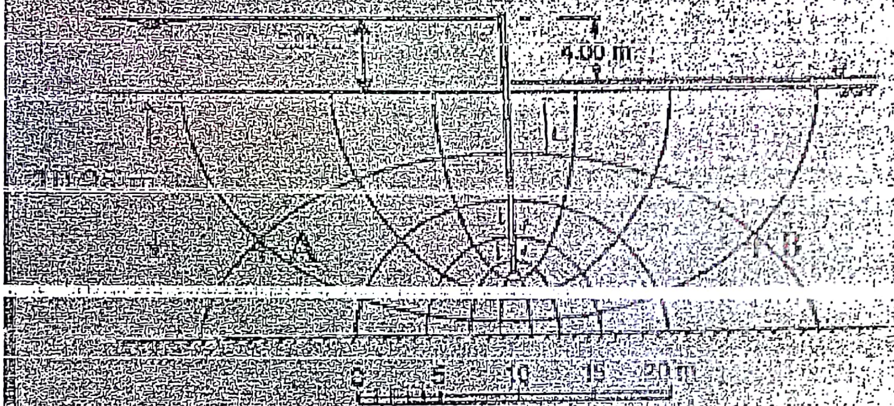
$C_v = \frac{T_v \times H_{dr}^2}{t}$

Geotechnical Engineering
Second Exam 45 Minutes
, 2009
All answers should be in the given pages

Faculty of Engineering Student Name: [redacted] Student Number
Department of Civil Engineering

Problem #1

The completed transformed flow net for the cross-section for a sheet pile is shown below. Th... in a broad alluvial valley with moderately dense sands and gravels that have an average coe... permeability in x direction of $K_x=3.5\times10^{-6}$ m/s and in z direction $K_z=4.5\times10^{-6}$ m/s. The bed... dense shale formation and is considered to be impervious relative to the alluvium. Determine



1. The quantity of seepage under the sheet pile in $m^3/s/m$. (2 p...

$H = 4$ $q = H\, k_{avg} \frac{N_f}{N_d}$

$N_d = 11$

$N_f = 4$

$k = \sqrt{k_x k_z} = \sqrt{3.5\times10^{-6} \times 4.5\times10^{-6}} = 1.25\times10^{-6}$

$= (4)(1.25\times10^{-6})\left(\frac{4}{11}\right)$

$= 1.81 \ldots m^3/s/m$

2. Exit gradient (2 p...

$\frac{\Delta h}{\Delta L} = \frac{4/11}{\ldots} = 0.833$

3. The factor of safety against piping, if the average saturated unit weight of the sands and gravels is about 21 kN/m³ (2 p...

$i_{critical} = \frac{\gamma'}{\gamma_w}$ $i_{cr} = \frac{21}{9.81} = 2.14$

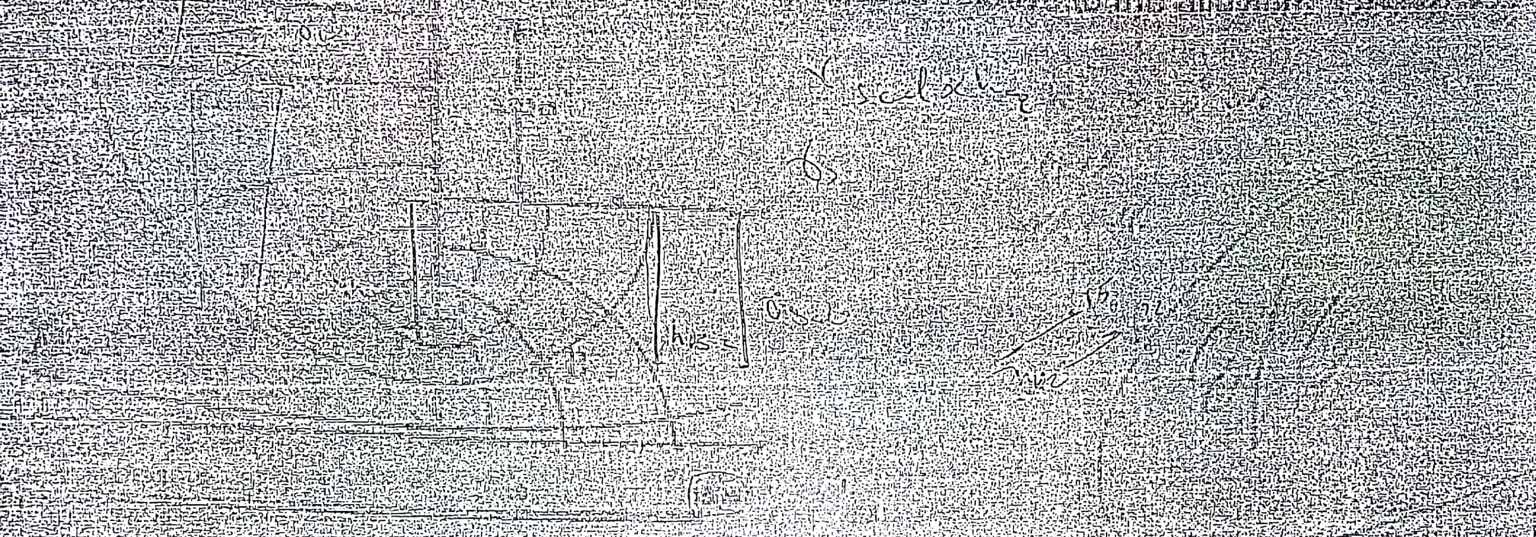
$F.S = \frac{i_{cr}}{i_{max}} \geq 1$

4. The effective stress at point A and B. (2 p...

$\sigma' = \sigma - u$

...ed for the ...

... assumed to be impervious relative to the alluvium ...



... pressure under the sheet pile ...

... of safety against piping, if the ... saturated unit weight of the ...

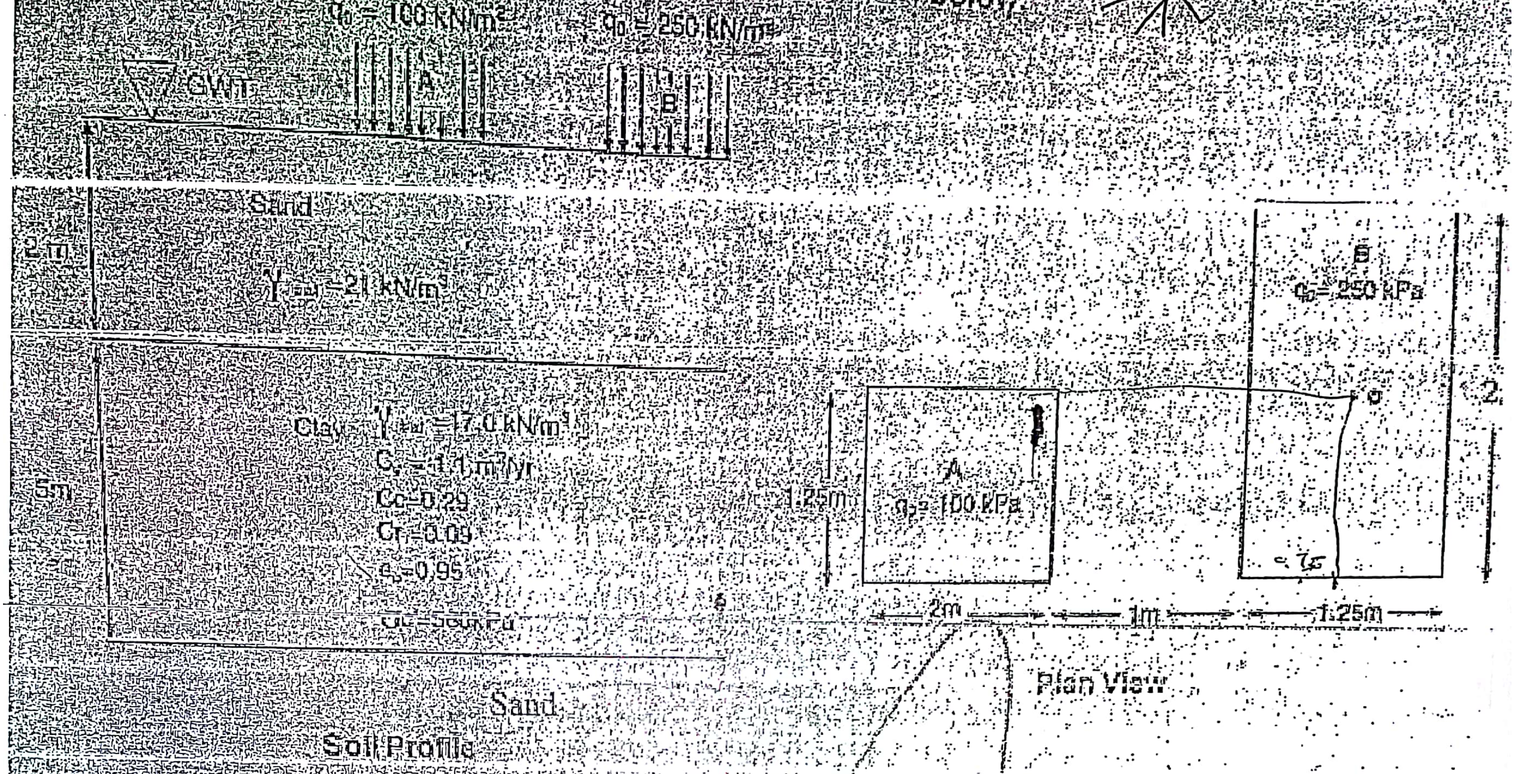
... at point A and B.

$h = (8) - 2\left(\frac{8}{9}\right)$

$B = (8) - 7\left(\frac{8}{9}\right)$

Problem # 2 (8 points)

Two footing is resting on the given soil profile shown below.



5) Determine the stress imposed under the center of footing B (point O) at depth 2.5 m. (3 points)

$= 0.57$

$100\left(\begin{matrix} m=1.25/2.5 \\ n=1.75/2.5\end{matrix}\right) - 100\left(\begin{matrix} m=1.25/2.5 \\ n=1.75/2.5\end{matrix}\right) + \frac{250 \cdot (1.25)(2.5)}{(1.25+2.5)(2.5+2.5)}$

41.6

$= 26.4$ kPa

47.166 kPa

6) Find the consolidation settlement in the clay layer? Suppose the stress increment imposed at middle of clay layer is 125kPa. (3 points)

$\boxed{59.5}$

$\sigma_f = 26.4 + 125 = 151.409$ kPa

41.166 152.166

172.166

$S_c = \frac{C_r H}{1+e_o}\log\left(\frac{\sigma_f}{\sigma_o}\right) = \frac{0.09}{1+0.95}(5)\left(\log\left(\frac{151.409}{125}\right)\right) = 0.0192$ m

$= 19.2$ mm

$S_c = 0.0867$

7) What would be the consolidation settlement after 9 months if the primary consolidation settlement was 0.0867m (3 points)

$U = \frac{S_c(t)}{S_{c_t}}$ $T = \frac{S_c}{S_t}$

$\frac{1.1 \times 0.75}{(2.5)^2} = 0.1337$

$U = \frac{S_c(t)}{S_{C_t}}$

12m → 12
x → 9

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

$0.132 = 0.933\log(1-U) - 0.085$

$U = 0.705 = 70.5\%$

$T_v = \frac{C_v t}{H_{dr}^2}$ $T_v = \frac{(1.1)(0.75)}{(2.5)^2} = 0.132$ → $S_c = 0.0614$

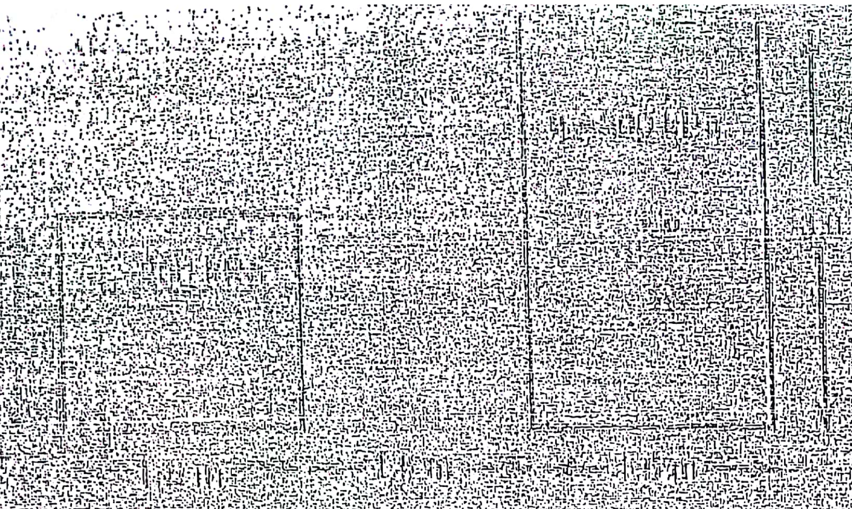
$T_v = \frac{1.1 t}{(2.5)^2}$

8) The coefficient of permeability for the clay layer is (3 points)

$m_v = \frac{De}{D\sigma}$

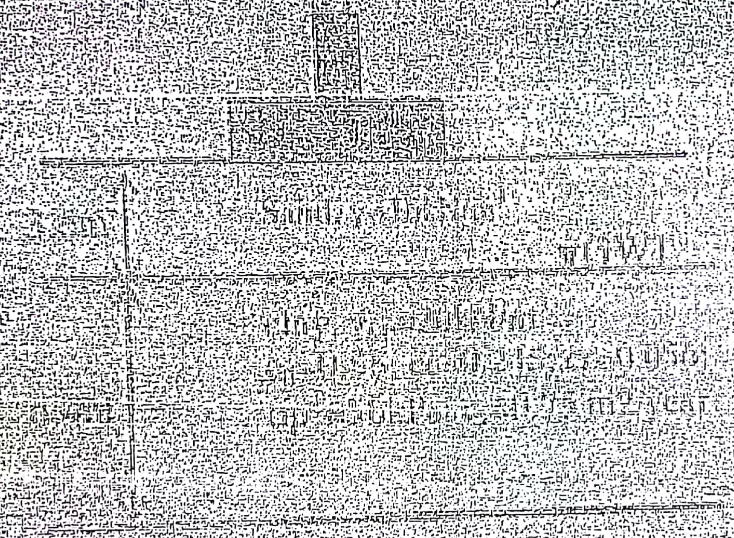
$m_v = \frac{DH}{D\sigma}\frac{1}{H_o}$

$= 0.0185$ 4.9133



Problem # 3 (10 points)

[illegible] foundation [illegible] founded on sand [illegible]



G_s (3.6)(18) + 22(22 − 9.81)

5.76

[illegible] the consolidation settlement in the clay layer. [illegible]

[illegible] What would be the consolidation settlement after [illegible] years

[illegible] at the middle height of clay layer equal to [illegible] kPa. Estimate the [illegible]

12.5 / 20

Student Name.

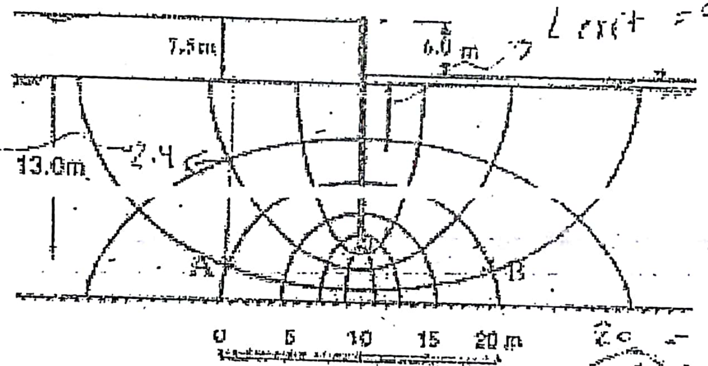
Student Number

Geotechnical Engineering
Second Exam 40 Minutes
Monday 12th , 2010
All answers should be in the given pages

Faculty of Engineering
Department of Civil Engineering

Problem # 1

The flow net for the cross-section for a sheet pile is shown below. The site is in a broad alluvial valley w... moderately dense sands and gravels that have an average coefficient of permeability in x and z direction is $k_x = 4.5\times10^{-5}$ m/sec and $\gamma_{sat}=20.8$ kN/m³ dense shale formation and is considered impervious relative to alluvium. Determine: use $\gamma_w=9.8$ kN/m³



$N_f = 4$
$N_d = 11$

1. The quantity of seepage under the sheet pile in m³/sec/m. (2 points)

$q = K H \frac{N_f}{N_d} = (1\times10^{-5})(7.5)\frac{4}{11} = 2.72\times10^{-6}$ m³/sec/m

2. Exit gradient (2 points)

$= i = \frac{\Delta h}{L_{exit}} \Rightarrow \Delta h = \frac{H}{N_d} = \frac{7.5}{11} = 0.6818$

$i = \frac{0.6818}{5.549} = 0.1228$

3. Factor of safety against piping (2 points)

Factor of safety $= \frac{i_{cr}}{i_{max}}$ $F.S = \frac{i_{cr}}{i_{max}}$ $i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{20.8}{9.8} = 2.12$

4. The pore water pressure at point A (2 points)

$\Rightarrow u = h_p \gamma_w$

$h_p = 7.5 - 0.6818\times2 + 13 = 19.1364$

$u = 19.1364 \times 9.8 = 187.5367$ kPa

5. The effective stress at point B. (2 points)

$\sigma' = \sigma_t - u$

$\sigma_t = 13 \times 20.8 = 270.4$ kN/m² = kPa

$u = h_p \gamma_w \Rightarrow h_p = 7.5 - 0.6818\times9 + 13 = 14.3638$

$u = 140.76524$ kPa

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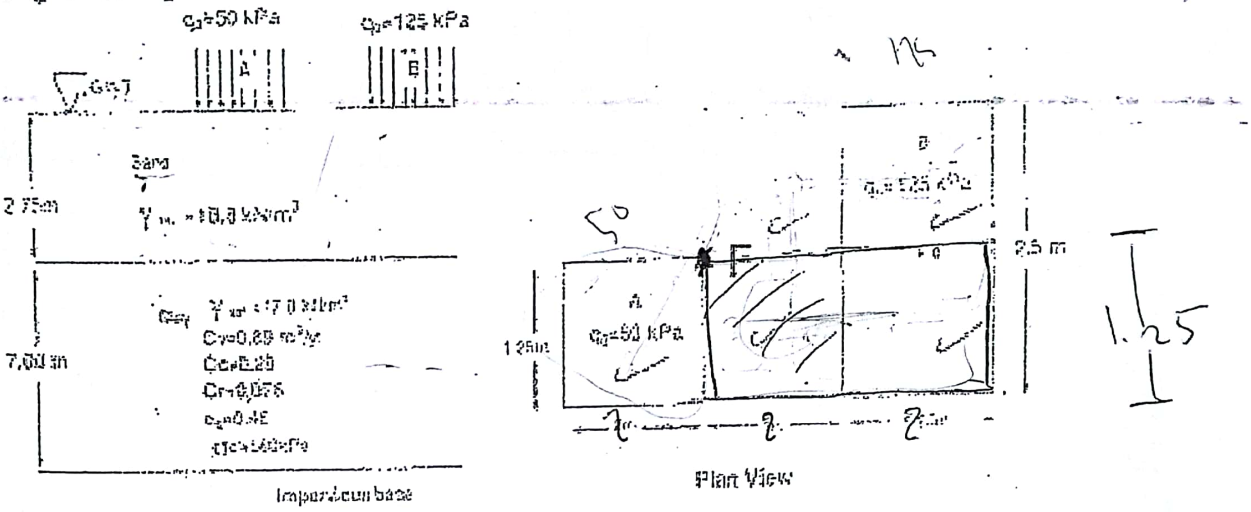
Student Name: Student Number

Problem # 2 (8 points)

The two footing is resting on the given soil profile shown below.



Soil Profile

6) Determine the stress imposed under point F from both footing at depth 1.25 m. (3 points)

at F from A => $M = \frac{2.5}{1.25} = 1.6$, $N = 1$ -> $I = 0.195$

$\boxed{\Delta\sigma = 50 \times 0.195 = 9.75 \text{ kPa}}$

at F from B => $M = 1$, $N = 1$, $I = 0.216$ => $\Delta\sigma = 44.5$

=> $M =$, $N = 1$, $I =$ => $\Delta\sigma = 40$

$S = 9.75 + 44.5 - 40 = \boxed{14.25 \text{ KPa}}$

7) Find the consolidation settlement in the clay layer? Suppose the stress increment imposed at middle of clay layer from the two footing is 50 kPa. (2 points)

$\sigma_t = (18.8 \times 2.75) + (17 \times 7) = 170.7$ KPa

$U = 109.15$ KPa => $\sigma' = 60.55$ KPa

$\sigma'_f = \sigma' + \Delta\sigma = 110.55$ KPa

$S_c = \frac{C_c}{1+e_0} H \log\left(\frac{110.55}{60.55}\right) = 0.3534$ m

8) What would be the consolidation settlement after 20 months if the primary consolidation settlement was 0.118m (2 points)

$U_{avg} = \frac{S_c}{S_t}$ => get U_{avg} => $T_v = \frac{c_v t}{H^2} = \frac{0.89 \times 1.6667}{49}$

$U = \sqrt{\frac{4T_v}{\pi}} = 0.1963$ => $S_c = 0.023$ m

8) Determine the effective stress at the middle of the clay layer if imposed increment of the stress was 100kPa after 10 years (3 points).

$\sigma_T = 18.8 \times 2.75 + 7 \times 17 + 100 = 270.7$ KPa

$U = 9.81(2.75 + 3.5) = 61.25$ KPa

$\sigma' = \sigma_T - U = 209.45$ KPa

$\sigma'_f = \sigma'_0 + \Delta\sigma = 24.72 + 100 = 124.72$ kPa

Student Name: Student Number

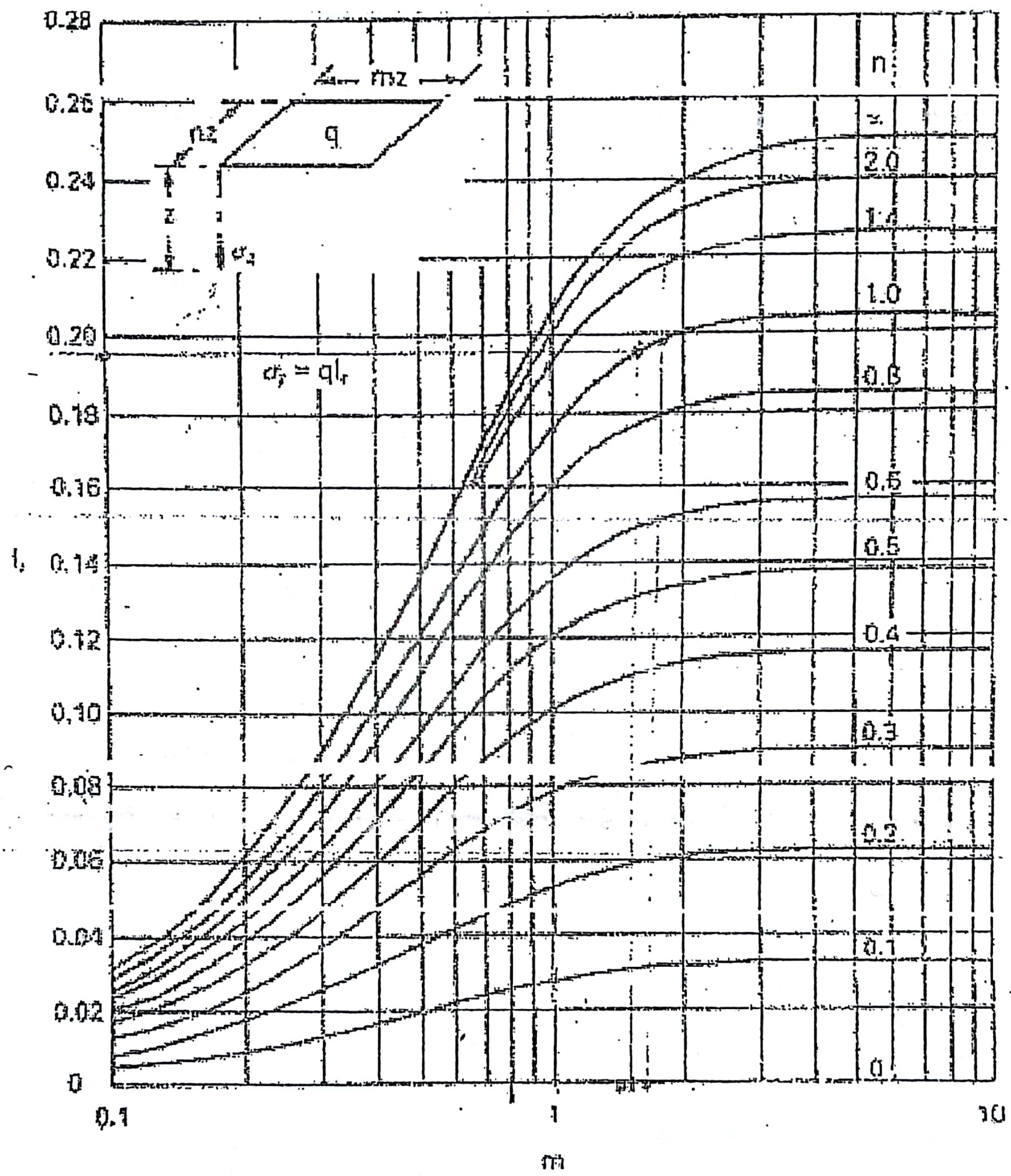


Figure 5.10 Vertical stress under a corner of a rectangular area carrying a uniform pressure. (Reproduced from R.E. Fadum (1948) *Proceedings of the 2nd International Conference of SMFE*, Rotterdam, Vol. 3, by permission of Professor Fadum.)

$$u_e = \sum_{m=0}^{m=\infty} \frac{2u_i}{M}\left(\sin\frac{Mz}{d}\right)\exp(-M^2T_v)$$

$U < 0.60, \quad T_v = \frac{\pi}{4}U^2$ $\qquad U_v = \sqrt{\frac{4T_v}{\pi}}$ $\qquad (0 \le T \le 0.197)$

$U > 0.60, \quad T_v = -0.933\log(1-U) - 0.085$ $\qquad U_v = 1 - \frac{8}{\pi^2}e^{-\pi^2 T/4}$ $\qquad T > 0.197$