Civilittee
اللجنة الأكاديمية لقسم الهندسة المدنية
www.Civilittee.com

هندسة جيوتقنية

دفتر إسلام حلاحلة

Geotechnical engineering

civilittee.hu
Civilittee HU | لجنة المدني
Civilittee Hashemite
www.civilittee.com

Chapter 1:- تراكم

Soil : any un-cemented (غير متماسكة) accumulation of mineral particles (جزيئات معدنية) formed by the weathering of rocks (تتكون بفعل تجوية الصخور) (كيف يتكون الـ Soil؟)

• Voids spaces between the particles contain (water and or air)

① ماء
② هواء
③ ماء وهواء

تتكون الفراغات اللي تكون بين الجزيئات من

⇒ weathering of rocks
- Physical weathering (Sands, Gravel) (المادة لا تتغير)
- Chemical weathering (clay) (المادة تتغير "صنع جديد")

Size of soil particles (الحجوم) : Sand, Gravel, clay

* Rock + water ⟹ clay "The contact of rocks and water produces clay, either at (تحت السطح) or near the surface of the earth" (أو القريب من سطح الأرض)

clay (جزيء بتكون بالمية جدا)
Sand, Gravel (جزيء بتكون بالمية متوسطة)

1.2 Basic Unit- Silica Tetrahedra → ذرة السليكون (Si) محاطة بـ 4 ذرات من الأكسجين (O)

1:1 semi-basic unit

Al → octahedral (Al)
Si → tetrahedral (Si)

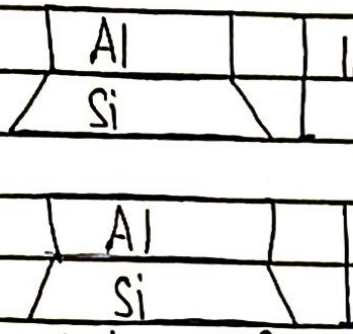
طبقة من السليكون (Si) وطبقة من الألمنيوم (Al)

2:1 semi-basic unit

Si → Tetrahedral
Al → octahedral (Al)
Si → tetrahedral (Si)

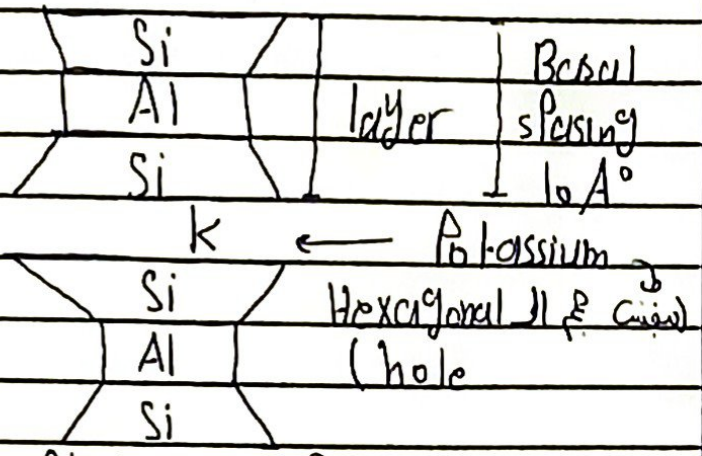
طبقتين من السليكون (Si) وطبقة من الألمنيوم (Al)

1:1 Minerals - Kablinite



→ Flaky shape
→ bonding between layers are "Van der weak forces and hydrogen bonds" (strong bond)
→ no interlayer swelling
layers بين الماء ما يقدر يدخل بين ال
بين ال strong bond وطالع في بعض
(انتفاخ بالماء)
(لا يوجد امتصاص للماء)
(صعبة في البناء)

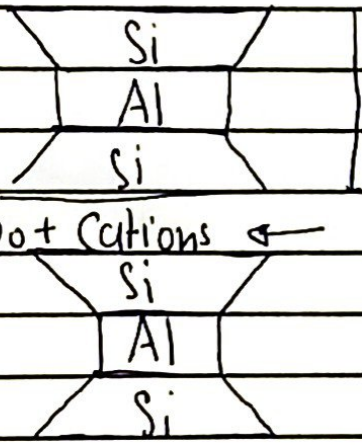
2:1 Minerals - Illite



→ Flaky shape
→ bonding between layers potassium ion (strong bond)

→ no interlayer swelling
strong (الترابط بين الطبقات)
من الماء ما يقدر يدخل بين الطبقات بسبب ال (bond)
(صعبة في البناء)

2:1 Minerals - Montmorillonite (Smectite)



Basal spacing 9.6A°
→ film like shape
→ the bonding between layers Van der waals forces and by cations
n. H2o + cations (weak bonding)
→ interlayer swelling (انتفاخ بين الطبقات)
(الماء يدخل بين الطبقات بسبب ال (weak bond)
→ expansive clay (قابلية الانتفاخ)
(تستخدم في البناء "الطوب")

→ Elementary Particles :

Clay structures → a) dispersed
b) Flocculated
c) bookhous d) turbostratic

a) dispersed =▷ "الجزيئات تتجاذب" (بسبب وجود الماء)
لـ بسبب خاصية الموكيت تقليم بـ كاسحة

b) Flocculated =▷ "dray side" (بسبب عدم وجود الماء) (جزيئة) مجزئة

	Minerals - Kaloinite	Minerals - Illite	Minerals (Smectite) - Montmorillonite
group	1:1	2:1	2:1
shape	Platy shape	Flaky shape	Film-like shape
Bond between layer	"van der waals forces and hydrogen bond" (Strong bond)	" Potassium" bond k (strong bond)	"van der waals forces (weak bond) and by cations" n.H2o + cations
Basal spacing	7.2 A°	10 A°	9.6
interlayer Swelling	no interlayer swelling → strong bond	no interlayer swelling → strong bond	interlayer swelling (expansive clay) weak bond

Soil texture :-
- appearance (Logen) or Feel (Lemsa)
- depends: (1. relative size 2. shapes) of the particles
  3. range or distribution (توزيع) of size

Corse grained Soil: Gravel, sand → well or poor (mix of many) (Almost one size)

Fine grained Soil: silt, clay (0.075 mm) Sieve# 200 نشوف فيها fine or coarse كيفاش اذا كان

Coarse (Gravel, sand)	Fine (Silts, clays)
• can see	• cannot see
• non Plastic (باستثناء)	• Silts (non Plastic or Plastic)
• Un important (water) (exception: loose Sat - dynamic loadings) (مع: متطلبات الثقة)	• clays (Plastic) (اهم اشكال نقارن الطين بإضافة الماء ومع الأيدي نقارن)
Coarse: non-Plastic	• Silts: Important (Plastic كان اذا) وجودها • clays: very important (مع: مدفون بيئة سالبة) Clay: Plastic Silts: non Plastic

Size الحبيبات Gravel, clay ←
clay "Gravel" بحجم اكبر من ال Plastic.

* Clay → -Ve
  - Plastic
  - Size

* sand, Gravel → shape
  - distribution (well or poor)
  - Size

Ps
- clay-size Particles
  example: quartz
- clay minerals
  example: Kaolinite, Illite

note

(sive المنخل) fine ← soil ← مع طرحة اذا كان ال ⇐ sive #200 ⇐ D = 0.075

or

sive اذا هي كل ال coarse

(اللي منخل) Gravel ← مع طرحة اذا كان ⇐ (coarse) soil ← اذا كان ال

or ← استخدم sive #4

Sand ← D = ⇐ 4.75 ← اللي مر من − اللي مر من = نسبة ال sand

sive #200 D = 0.075 sive #4 D = 4.75

(اللي مر من) clay ← مع طرحة اذا كان ⇐ (fine) soil ← اذا كان ال

or ← استخدم sive اللي الـ

Silt ← D = 0.002

نسبة الـ silt = اللي مر من − اللي مر من

sive #200 D = 0.075 D = 0.002

اللي مر من ← sive

اللي مر من ← اللي ما مر من

sive #4 Gravel | Sand | Silt | Clay D = 0.002

D = 4.75

4.75 0.075

⇐ اللي مر من − اللي مر من =	⇐ اللي مر من − اللي مر من =
D = 0.075 D = 4.75	D = 0.002 D = 0.075

Grain size distribution

● طرحة اذا كان (well-graded) or (poor-graded) ؟ حتى تعرف من النوعين

well-graded (for gravel) : $1 < Cc < 3$ and $Cu \geq 4$

well-graded (for sand) : $1 < Cc < 3$ and $Cu \geq 6$

$Cc = D_{60} / D_{10}$

$Cu = (D_{30})^2 / (D_{10})(D_{60})$

D_{10} ← 10 من خلالها حبيبات الـ diameter

D_{30} ← 30 " " " " "

D_{60} ← 60 " " " " "

شرط اذا ما تحقق النوعين او واحد من النوعين نعتبره poor-graded ← اذا لم يتحقق النوعين

Question: what is the (Cu) for a Soil with only one grainsize ??

→ اذا كان الحجم واحد اذا D10 حبة = D60 حبة

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

Example :- Sive Size | mass of soil | Cum mass | Finer

4	0	0	100 → 100 - 0
10	40	0+40 = 40	94.5
20	60	40+60 = 100	86.3
40	89	89+100 = 189	74.1
60	140	189+140 = 329	54.9
80	122	329+122 = 451	38.1
100	210	451+210 = 661	9.3
200	56	661+56 = 717	1.7
Pan	12	717+12 = 729 → total	0

[0/729 * 100%]

→ Calculation and draw a particle size distribution Curve
→ determine the per-centages of Gravel, sand, Silt, and clay size particles.

:Sol

1. وزن احسب الـ Cum mass و Finer

$$100\% - \left(\frac{\text{Cum mass}}{\text{total}} * 100\%\right) \rightarrow 100\% - \left(\frac{40}{729} * 100\%\right) = 94.5$$

100% - ro

كذا احسب لكل الـ sives.

2. اذا كان Sive #200 اكبر من 50% : fine or coarse

Sive #200 = 1.7 → 1.7 < 50% → coarse

3. D10 , D30 , D60 → Well-graded او Poor graded

D10 = 0.15 mm , D30 = 0.17 mm , D60 = 0.27 mm

$C_u = D_{60}/D_{10} = 0.27/0.15 = 1.8$ → $C_u > 4$ or $C_u > 6$ → ما حققت الشرط

$C_c = (D_{30})^2/(D_{60})(D_{10}) = (0.17)^2/(0.15 * 0.27) = 0.71$ → $1 < C_c < 3$ ما حققت الشرط

Poor-graded اذا

(الفلتر الثاني) ← اربع كل نسبة الـ finer

Gravel ← Sive#4 الي بقي كل الـ ← Sive#4 ← Fine = 100

اذا كل البرنامج مرتا ما

Gravel = 0% ← اذا (Sive الي كل الـ ما فيه)

Sand ← 100 - 1.7 = 98.3%

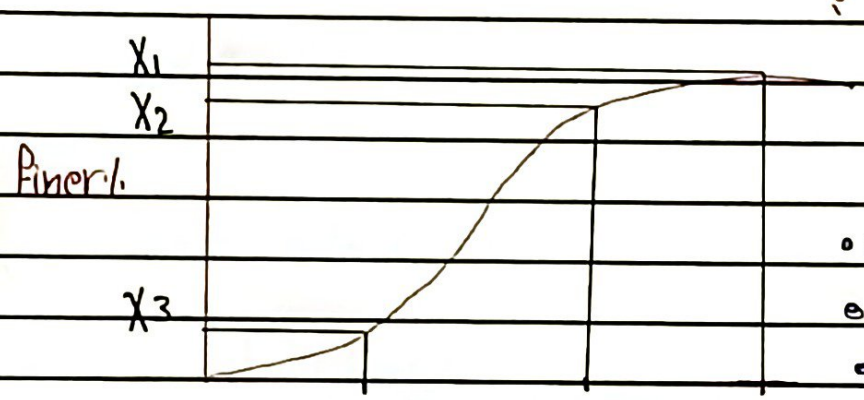
الي مرتا من ما ← الي مرتا من Sive#4 ← Sive#200

Silt and clay ← 1.7 - 0 = 1.7%

Sive#200 ← الي مرتا من ما

D=0.075 من

كيف نوجد الـ Percentages ؟



determine Per-centages:
gravel, sand, clay, silt

- Gravel = 100% - 0 (الفلتر اللي ما مشي Sive#4)
- Sand = $X_1 - X_2$
- Silt = $X_2 - X_3$
- clay = X_3

* Particle shape :- في Coarse grained Soils (الرمل) تتركز اهتمامنا على الـ shape
  - size
  - distribution

⇒ Angular soil → higher friction (الزوايا بتعمل احتكاك)

⇒ Round soil → lower friction (ناعمة الاحتكاك قليل)

- clay particles are (Sheet like)

* Atterberg limits and consistency Indices (Fine soil)

→ PI = LL - PL

- Liquid state → Liquid limit (LL) ⇒ Liquid
- Plastic state → Plastic limit (PL) ⇒ Plastic
- Semisolid state → shrinkage limit (SL) ⇒ water content
- Solid state

• وجود الماء في التربة الناعمة (Fine-Grained Soils) يؤثر على سلوك التربة الهندسي (Engineering behavior)

* Liquid limit (LL) ⇒ Casagrande Method استخدم اختبارين: Multipoint test / one point test

Particle sizes and water → Passing No 40 Sieve D = 0.425 mm
→ Using deionized water

Casagrande method: عدد الضربات N = 25 blows
closing distance = 0.5 in (12.7 mm) عند 25 ضربة بارتفاع 0.5 in (لإغلاق الشق في عينة التربة)

• Multipoint method:
- تجارب 4 نماذج مثلاً لها محتوى رطوبة مختلف
- عند 25 ضربة يكون الـ water content هو الـ Liquid limit (LL)
- عدد الضربات لازم يكون بين (10-40)
(10 < N < 40)
- إذا كانت عدد الضربات أكثر من 25 هذا يكون الـ Liquid state ولازم يكون بين (10-40)

• one-point method:
- أخذ التربة وقلبها واسوي لها حفرة بعمق 0.5 in
- عدد الضربات لازم يكون بين (20-30)
(20 < N < 30)

- N < 25 → water content ↑ → Liquid state
- N = 25 → water content = Liquid limit
- N > 25 → water content ↓ → Plastic state

* Plastic limit (PL) باختصار: اقرب درجة واطيء كلما ماء لما تجيبي العجينة اشكلها شكل كروي واحاول اعلمازي القلم اذا ان اوصل اكو 3.2mm
وبدأ الـ soil يتفتت ← PL

← دقوص تفتت عند 1.5mm ميك بنزل بال Plastic نفسه ← X / 32
← بالشيتفتت عند 3.2mm ميك نكول Plastic limit
↳ 1/8 in

* Indices • Plasticity index PI: هو الـ water content اللي يتغير فيه الـ Plastic
liquid limit •
PI = LL - PL → Plastic limit

• liquidity index LI: natural water (مزاجية) content ← مثلاً وين الـ soil بأي حالة
LI = Water content - PL
PI
LL - PL
of soil sample
• LI < 0 → (brittle)(solid)
• 0 < LI < 1 → (Plastic)
• LI > 1 → (Viscous)(liquid)

• Activity A = PI
size D = 0.002 ← % clay fraction (weight) اللي جوة من
• 0.75 < A < 1.25 (normal)
• A < 0.75 (inactive)
• A > 1.25 (active)

High activity → large volume change when wetted (تزيد الحجم بسبب رطوبة) (انتفاخ)
→ large shrinkage when dried (تنقلص بالحرارة حاف) (انكماش)
→ very reactive (chemically) (مثال)

• Purpose: type and amount of clay in soils will effect

← مثلاً: Kaolinite: (0.5) تفتت بسيط جم قليلاً.
→ Semectite: مثلاً عالي (1-7) انتفاخ يتمدد وتنقلص كبير

مثال كتبه الدكتور :

N (عدد الضربات)	water content (wc)
Plastic state ← 37	w_{c1}
Plastic state ← 28	w_{c2}
liquid state ← 20	w_{c3} → N = 25 (liquid limit)
liquid state ← 15	w_{c4}

• semi solid إلى اليسار Plastic إلى اليمين الحالة التي تكون فيها الرطوبة أكبر من

→ N > 40 (semi-solid)
→ 25 < N < 40 (Plastic) state
→ N = 25 (liquid limit)
→ N < 25 (liquid state)

Chapter 2:- Soil classification

⇒ Plasticity chart • A-line → silt و clay بين يفصل
clay (C) ← (U-line - A-line بين) A-line فوق silt (M) ← A-line تحت

• U-line → upper bound for General (أكبر احتمال للتربة) soil فوق U-line (مستحيل)

• LL < 50 → L • LL > 50 → H

• حالة خاصة معرفة shaded area PI = (4-7)% LL/30 كودها
راح يكون تصنيفها CL. ML

• PI = LL - PL تذكر

تقسيم التربة (معرفة)
1) Sive # 200 → Coarse (أكثر من 50% محجوز)
Fine (أقل من 50%)

2) في حالة Coarse: Sive # 4
Gravel (نسبة المحجوز أكثر من 50%) Sand (نسبة المار أكثر من 50%)

3) Sive #200
(5-12)% المار
أقل من 5% المار → well (or) Poor (المحجوز أكثر من 95%)
أكثر من 12% المار → major minor
major → sand gravel
minor → silt or clay
(حسب Plasticity chart)

2) في حالة Fine: (Plasticity chart على)
PI = LL - PL (الرسم التعبيني)
PL معطى بالسؤال LL
حالة خاصة shaded area PI = (4-7)% LL/30 كودها
(راح يكون التصنيف CL. ML)

• إذا كانت نسبة المار من sive #200 في الربط أكثر من 50% Fine وإلا إذا أقل من 50% Coarse وإلا

• حالات Fine ← (1) و (2)
• حالات Coarse ← (1) و (2) و (3)
• حالة إذا طلعت بالربط 50% Fine (1) و (2) Coarse
حالة (1) و (2) و (3)

Example 1: Passing No.200 sieve 30% LL = 33
Passing No.4 Sieve 70% PI = 12

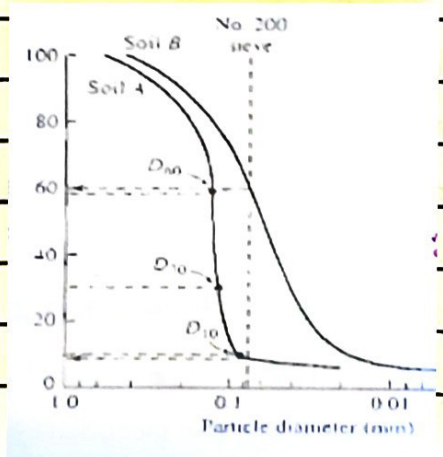
Sol. 1) Sive #200 = 30% < 50% → Coarse
2) Sive #4 = 70% > 50% → Sand
3) Sive #200 = 30% > 12% → major (Sand)
→ minor (Chart الجدول) PI=12
LL=33

Clay اذا (U-line) و (A-line) تقاطعت بين [S C] اذا التصنيف

Example 2:	Soil A	Soil B
Liquid limit	30	26
Plastic limit	22	20

Soil A:
1. Sive#200 = 60% > 50% (Coarse ~~Gravel~~)
2. Sive#4 = 100% > 50% (Sand)
D=4.75 في الشكل اللي معطي إياكم اللي
Sive#4 D=4.75 مش كامل (1) → 100%
100 = (1) فاكرين اللي بالعشرة اذا موجودة
نأخذ 100 كمان (4) اللي موجودة
3. Sive#200 = 6% (بين 12 - 5)
< 5% (Coarse) > 12% (Plasticity chart)
PI = LL - PL = 8%
(SC)
D10, D60, D30 من الشكل
0.085 و 2 و 0.135
Cu = D60/D10 = 1.59 → (Cu > 6 ماكنش) → [SP-SC]
Cc = (D30)² / (D10 D60) = 1.25 (1 < Cc < 3 ✓)
→ (SP)
Sand → Poor (ماخنش)

Soil B:
1. Sive #200 = 60% > 50% (Fine)
2. Plasticity chart
PI = LL - PL = 26 - 20 = 6 (U-7 اللي)
LL = 26 (30 under)
Shaded area (CL-ML)



Example 3:

Sive no	% Passing (Fine)
4	94
10	63
20	21
40	10
60	7
100	5
200	3

1. Sive #200 = 31. < 50%. ⇝ Coarse
2. Sive #4 = 94 > 50%. ⇝ Sand (S)
3. Sive #200 = 31. < 5%.

↳ D_{10}, D_{30}, D_{60} من الجدول

= 0.41 = 0.96 = 1.41

$C_u = \frac{D_{60}}{D_{10}} = \frac{1.41}{0.41} = 3.44$ $C_u > 6$ X → Poor

$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = 1.59$ $1 < C_c < 3$ ✓ (SP)

Example 4: Passing No 200 Sive 50%. Passing No 4 sieve 70%. LL = 25 PL = 20. Soil classified as:

1. Sive #200 = 50% ⇒ Coarse: 2. Sive # 4 = 70% > 50% (Sand) S
   3. Sive #200 = 50% > 12% [major (sand), minor] ⇒ Plasticity chart
   ⇒ Fine ? Plasticity chart
   - PI = 25 − 20 = 5% (4–7 coal)
   - LL = 25 (30% coal)

   PI = 5%. LL = 25 (b)

Shaded area (L) ⇒ CL-ML (Sand S)

Fine (CL-ML) | Coarse (SC-SM) ⇒ Shaded area (CL-ML) (SC-SM)

⇒ Clay ... Swelling ... -Ve ... Clay ...

CL - SC

Example 5: Passing No 200 sieve 20%. Passing No.4 sieve 40%.
LL = 25 PL = 20 Soil classified as:

1. Sive #200 = 20% < 50% → Coarse
2. Sive #4 = 40% < 50% → Gravel
3. Sive #200 = 20% > 12%

major (Gravel G) | minor (Plasticity chart) PI = LL - PL = 25 - 20 = 5%

LL = 25

Shaded area

(GC-GM) ∴ L ← CL-ML (Gravel (G))

For major divisions:

1) Coarse-grained
2) Fine-grained
3) Organin Soils
4) Peat

(Fine ...) ← ...

• Highly organic Soils - Peat (PT)

Composed Primarily (...) → Vegetable in various stages of decomposition

(...) has a fibrous to amorphous texture

(... dark-brown to black color

(... organic odor

• highly organic Soil and shall be classified as Peat (PT)

organic clay or silt (OL or OH):-

LL(oven dried) / LL(not dried) → LL(after oven dry) < 75% ⇒ O

→ Silts and clays (LL (less than 50)):
- Inorganic → PI > 7 (Plots on or above A line) (CL)
  - PI < 4 (or Plots below A line) (ML)
- organic: liquid limit(oven dry) / liquid limit(not dried) < 0.75 (OL)

→ Silts and clays (liquid limit (50 or more):-
- Inorganic → PI Plots on or above A line (CH)
  - PI Plots below A line (MH)
- organic: → liquid limit(oven dry) / liquid limit(not dry) < 0.75 (OH)

Chapter 4: Compaction

To compact a soil, that is, to place it in a dense state.
دمك التربة يعني وضعها في حالة كثيفة

Compaction: عملية الدمك

Dense state: الحالة الكثيفة الناتجة عن الدمك

كيف يتم تحقيق الـ dense state؟
1. Reduction of the air voids in the soil
2. whith little or no reduction in the water content.

Consolidation و Comfaction الدمك
الفرق بين الـ
الفراغات الموائية تكون مليانة بالهوا والماء
الفراغات الموائية تكون مليانة بالهوا والماء بيقل من الـ air voids
الماء من التربة
الماء بيخرج من التربة والماء بيحمل ثابت.
water is squeezed out under the action of a continuous static load

- which type of earth construction require soil Compaction ??
dams, retaining walls, highways and airfort.

objects الأهداف:
- Decrease future settlements تقليل الهبوط المستقبلي
- Increase shear strength زيادة
- Decrease Permeability التقليل من نفاذية الماء

- General Compaction Methods (طرق الدمك):

Vibration (اهتزاز)
- Coarse
  - Vibrating hammer (BS) [laboratory]
  - hand-operated Vibration Plates
  - Motorized Vibratory rollers
  - Rubber-tired equipment
  - Free-falling weight
  - [Field]

Fine kneading (تدريجة الوزن)
- falling weight and hammers
- kneading compactors
- static loading and Press

→ (laboratory)

- Hand-operated tampers
- sheepsfoot rollers
- Rubber-tired rollers

→ (Field)

Laboratory compaction (Purpose) → Proper amount of mixing water to use when compacting the soil
(Compaction لل ماء المناسبة لاداء الاساسية)
→ resulting degree of denseness
(density الى [redacted])

Proctor test is an impact compaction
دك التربة بالدم

Standard and Modified Proctor compaction test.
- Standard: مستخدم لمشاريع الطرق (الاكوام ضخمة)
- Modified: مستخدم لمشاريع المطارات (الاكوام ضخمة)

Compaction is a function of four variables
- Dray density, Dray unit weight
- water content
- compactive effort (energy)
- Soil type

Compaction effort (energy E)

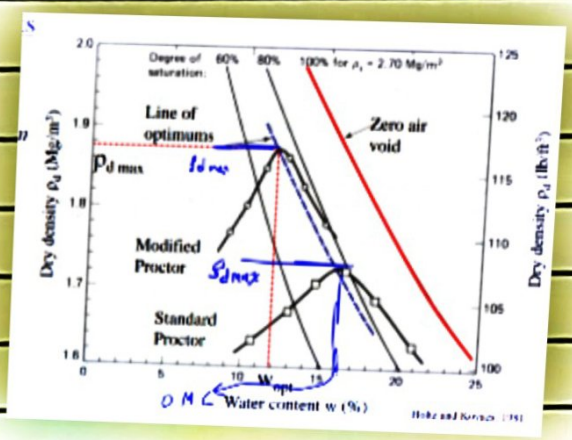
$$E = \frac{\text{weight of hammer} \times \text{Hight of drop of hammer} \times \text{number of layer} \times \text{number of blows for layer}}{\text{Volume of mold}}$$

For standard Proctor test
$E = (2.495)(9.81)(0.3048\,m)(3\text{layers})(25\ \text{blows/layer}) / 0.944 \times 10^{-3}\ m^3$

2.495 kg هي 5.5 lb
5.5 lb * 0.4536 = 2.495 kg

Method A: 4 inch mold, 10 lb rammer, 18 inch drop, 5 layers, 25 blows per layer, uses material passing #4 sieve
Method B: 4 inch mold, 10 lb rammer, 18 inch drop, 5 layers, 25 blows per layer, uses material passing 3/8 inch sieve
Method C: 6 inch mold, 10 lb rammer, 18 inch drop, 5 layers, 56 blows per layer, uses material passing #4 sieve

Peak Point is the Point with maximum dray density ($\rho_{d\,max}$)
$\rho_{d\,max}$ عند OMC أو Wopt هيك بيكون
Wopt (دائماً) بتجيب ρ_d أعلى
zero air voids curve: fully saturated condition $S_r = 1$ (هيك بيكون الموضوع)
(Compaction لـ
line of optimums: لـ $\rho_{d\,max}$ بيمر فيه
for same soil
Parallel (بيوازي) curve



fully saturated $S_r = 1$ zero air voids كيف ممكن أوصل لـ
$\rho_d = \frac{G_s}{1+e}$ ∴ $wG_s = eS_r$ ($S_r = 1$) $e = wG_s$. comp

$\rho_d = \frac{G_s}{1+wG_s}$ (Zero)

curve لـ zero air void هيك رسم لـ (OMC بعد ما بتزيد الرطوبة) بعد ما curve لـ saturated بيمشي رسم لـ

(Higher Swelling Potential) (w_{opt} Points) (Higher Shrinkage Potential)
ρ_b (استيعاب الماء في الـ soil Particles)
في كتابة الماء بتعمل
$\rho_w < \rho_s$

dry wet
w
زيادة الماء (بتخلي من V_c الموجودة) ⇒ ρ_b زيادة الكثافة
Fine (clay) → بتعطي انتفاخ كبير

مثال: a Soil sample has a WC = 0.12 G_s = 2.72 . Possible to compact to Soil sample to a dray unit w (γ = 23.5) ?

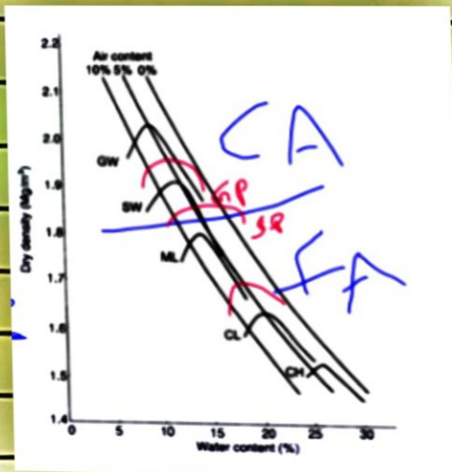
$S_r = wG_s$ → $e = wG_s$ = 1 لما تكون
$\rho_d = \frac{G_s}{1+wG_s}$ → $\frac{2.72}{1+0.12(2.72)}$ = 2.0566 × 9.81 = γ = 20.11 < γ = 23.5 zero air void $S_r = 1$

لا يمكن الوصول إلى γ_{dray} أعلى من zero air void

راح تكون بعد الـ zero air void إذا أردت تكون بأعلى من

→ طبقا الـ grain size طبقا الـ
dry density ρ_d

→ dry density طبقا عند اكبر الـ



→ coarse-grained (GW, GP, SW, SP)
↩ يعطي اعلى ρ_d (اقل WC) والـ
(كبير grain size)

→ fine-grained (ML, MH, CL, CH)
↩ يعطي اقل ρ_d (أعلى WC) الـ
(صغير grain size)

coarse (ρ_d ↑, WC ↓) fine (ρ_d ↓, WC ↑)

- Relative compaction (R.C) = $\frac{\rho_{d\text{-field}}}{\rho_{d\,max\text{-laboratory}}} \times 100\%$

$\rho_d = \frac{\rho_{bulk}}{1+WC}$] من المعطيات

R.C = 80 + 0.2 Dr نعوض فيها
(الـ coarse وفي حالة Dr)

typical required R.C = 90% - 95% ← لازم الكثافة تكون عالية بعد الـ comp

$\rho = \frac{M}{Volume}$] ← كيف أجد الـ volume ؟؟ Destructive method
→ Sand cone
→ Balloon
→ Oil (or water)

→ sand cone: بحفر حفرة (أوزنها ثم أعبيها بالرمل المعلومة كثافته) 1. $\rho = \frac{M}{V?}$
2. $\rho_{bulk} = \frac{M}{V}$ 3. $\rho = \frac{\rho_{bulk}}{1+WC}$

- error: من عمل الـ vibration الناتجة عن الـ
nearby working equipment ← حجم أكبر وراح يكون الحجم المحسوب أكبر الـ density راح تقل الـ
لأنه أكبر lower field density.

→ Balloon: الماء يرتبط بالـ Ballon الموضوع في الحفرة مباشرة
وحجم الحفرة من خلال كمية الماء

- error: من كون الـ walls of the hole فيها نتوءات (حادة) حيث تسبب
خطأ في الـ Ballon (ممكن تمزق)

اعلى كل عزل الحفرة واعميها في الـoil or water وهم oil (or water) method.
الحفر يكون عبئ الـoil or water.
Error: اذا كانت التربة Coarse ماراح تعمل الطريقة بشكل جيد الا اذا كانت الحفرة كبيرة جدًا والمادة العازلة تكون Ethylene.

* ممكن اضيف في الموقع مادة خشنة او حفرة كبيرة اعدم كيف اكمل اعلاها وهالوزنها وله اذا لقيت ارجعها على الحفرة مثلما اكون حاسبلها الـ Volume او mass.

Chapter 5: water in Soil

→ Soils are Permeable materials (الاختلافات) (Interconnected pores between solid Particles)

→ You must know how much water is flowing through a soil per unit time (كمية المياه المتدفقة خلال التربة)

1. design earth dams (لتصميم السدود الترابية)
2. Determine the quantity of seepage under hydraulic structures (لتحديد كمية التربة)
3. dewater foundations before and during their construction (تجفيف الأساسات قبل وأثناء البناء)

water table هو reference لحساب Pore water Pressure ← لإيجاد ال

↳ WT : atmospheric Pressure = zero ($P = \gamma_w \nabla$ كلما نزلنا لتحت زاد الضغط)

↓

Fully saturated ← soil (حتى لا تكون)

- Below the water table (atmospheric Pressre = 0)
  - → Soil is assumed to be fully saturated
  - → Pore water may be static (يتغير hydrostatic Pressure مع depth; static Porewater = $\gamma_w h$ ← below the water table)
  - → may be seeping (تتسرب) through the soil under hydraulic gradient.

سرعة التدفق $V = ki$ (ميل الهيدروليك hydrolic gradient) (معامل النفاذية hydrolic conductivity)

$i = \frac{\Delta h\,(m)}{L\,(m)}$ (فرق الضغط / المسافة)

→ Volume of water

$q = AKi$ | $K \rightsquigarrow$ Temperature

Cross-Section Area | $L \leftarrow V$

كل ما زادت الحرارة Temp قلت اللزوجة Viscosity

K (measured at 20°C is taken

100%)

- مهم: اذا اعطى الـ K عند درجة حرارة غير الـ 20 لازم اعالج الـ correction

$K = \frac{\gamma_w}{\eta} K$ ← الحقيقية

$\eta \rightarrow$ Viscosity

γ_w: unit weight of water

η: Viscosity of water

K: Viscosity of water absolute (نسبي) coefficient depending only on the characteristics of soil skeleton

يعتمد على خصائص هيكل التربة

- مهم: قيمة الـ (K) تعتمد على نوع التربة في الـ coarse قيمة الـ K كبيرة من الـ fine لأن الفراغات اللي تكون موجودة في الـ coarse (interconnected) الماء يقدر يمر بشكل أسهل.

- Laboratory Permeability tests:

→ constant head في حالة الـ coarse ← مهم جدًا

→ Falling head في حالة الـ fine

Hazen showed that approximate value of k is given by → $K = 10^{-2} D_{10}^2$ (m/s)

10% من الـ fine — الـ sieve اللي بيمثل الـ (Symbol)

- Seepage Theory نظرية التسرب (two dimensions)

← عدد الماء من خلال الـ Soil Particles من خلال الفراغات الموجودة

two dimensions ←

Soil is homogeneous and isotropic ← التربة متجانسة

Generalized Darcy law will be used ← وقانون دارسي معمول به في التربة

$V = 1$

- Correction في حالة homogeneous and isotropic تكون التربة متجانسة K

(Laplace Equation and Flow Net (Graphical representation)

Flow Net:

Equipotential line: مقدار Pressure الـ Head الـ Equipotential lines

Nd

Equipotential line أي

Flow line (Flow paths of particles of water)

Flow Channel (between two lines)

NF → between two lines

Flow rate الـ Constant

Flow lines الـ (الاتجاه دايما بيكون عكس الـ Flow)

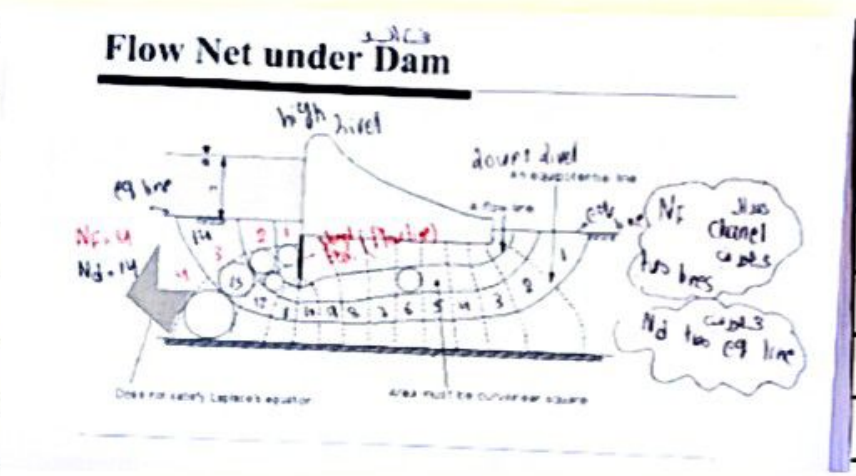
Velocity الـ Equipotential line الـ (normal) عمودية

Head loss الـ هو الفرق بين two Equipotential lines

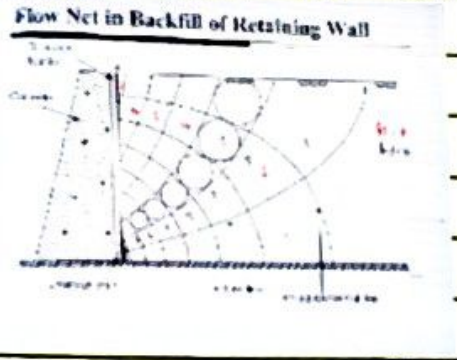
Flow الـ high level الـ low level

Flow net الـ Laplace في اد من الـ Flow net كل ما زادت الـ Equipotential lines الـ Flow lines (كل ما زاد عدد الـ Flow net كل ما زادت دقة الحل في الـ lines)

* Flow net under Dam:



* Flow net in Backfill of Retaining wall:



* Interpertation of Flow net: $\Delta h = \Delta H$ → high الفرق بين اعلى واقل
between two ← Nd lower
equipotential line

$q = k \Delta H \frac{N_f}{N_d}$ → between two
between two equipotential Nd Flow line
line

hydrolic gradient over each square $i = \frac{\Delta h}{L}$

$i_{max} = \frac{\Delta h}{L_{min}}$ ← اللي بين اصغر دائرة (عندما اصغر قيمة)

$i_{cr} = \frac{G_s - 1}{1+e}$: $i = i_{cr}$ معنى الوصول لقيمة ال icr وقت ازدياد i Failure حدث

OR $i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{G_{sat} - G_w}{1+e}$ الوصول الى حالة

Safe $i < i_{cr}$ or $i_{ext} < i_{cr}$

اذا كان معامل الامان (Factor) بين الواحد وال icr
اكبر من 1

↳ pressure head

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

↳ between two equipotential lines

(pressure head)

$$U_j = (h_P)_j \gamma_w$$

↳ Pore water pressure

$$P_w = \frac{\Delta x}{3}\left[U_1 + U_n + 2\sum_{i=3,5,\dots}^{n} U_i + 4\sum_{i=2,4,\dots}^{n} U_i\right]$$

→ calculate the uplift force per unit length

Δx: طول القسم

n: عدد القسم الزوجية

b: عدد القسم الفردية

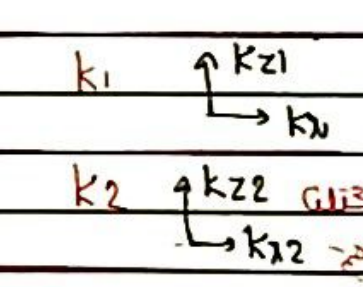
ملاحظة: الـ odd هي الأقسام الفردية و الـ even هي الأقسام الزوجية

- $K_y = K_{max}$: direction of stratification اتجاه الطبقات
- $K_z = K_{min}$: value in the direction normal to that of stratification

$$\text{Dam} - X_1 = X\sqrt{\frac{K_z}{K_x}} \qquad K' = \sqrt{(K_x K_z)} \text{ : correction}$$

$$K_x = \frac{H_1 K_1 + H_2 K_2 \dots n}{H_1 + H_2 \dots n} \text{ (Parallel)}$$

$$K_z = \frac{H_1 + H_2 \dots n}{(H_1/K_1) + (H_2/K_2) \dots n} \text{ (series)}$$



* وبعدما اوجد الـ resulting (avg) $K = \sqrt{(K_x K_y)}$ بيكون K_z و K_x (المتوسط الحسابي)

Chapter 6: Flow Nets Example

Coefficient of Permeability $k = 1\times10^{-4}$ cm/sec Void ratio $e = 0.8$, $n_{min} = 0.6$

1) Find q ?? $q = k\,\Delta H\,\frac{N_f}{N_d}$

$k = 1\times10^{-4}$ cm/sec $\times \frac{1}{100}$ m/cm $= 1\times10^{-6}$ m/sec

$\Delta H = 8$ → lowest to upper (الارتفاع)

$N_f = 8$ → (between two flow line) $N_d = 18$ (between two equipotential line)

$q = 1\times10^{-6} * 8 * \frac{8}{18} = 3.5\times10^{-6}$ m³/sec/m

2) Find i_{max} ?? $i_{max} = \frac{\Delta h}{L_{min}}$; $\Delta h = \frac{\Delta H}{N_d} = \frac{8}{18}$ (اقصر مسار للتدفق)

$i_{max} = \frac{8}{18 * 0.6} = 0.74$

3) Find the Pore water Pressure at (J , G) ??

$u_j = (hP)_j\,\gamma_w$; $(hP)_j = \Delta H - (N_d)_j\,\Delta h - h_z$

$\Delta H = 8$ → الارتفاع بين الـ lower والـ upper

$(N_d)_j$ → بدي N_d عند نقطة معينة (بعني الـ N_d اللي بتمثل هاي النقطة باتجاه الـ upper)

Δh → $\Delta H / N_d$ (بشكل كامل)

h_z → ارتفاع النقطة المطلوبة من الـ water table (بالنسبة للـ lower) ($P = 0$)

عند النقطة J:

$(hP)_j = 8 - 0 - 0 = 8$

$u_j = (8)(9.81) = 78.48$

عند النقطة G: طرح ارتفاع الـ water table

$(hP)_j = 8 - 1\left(\frac{8}{18}\right) - (1\,2) = 9.56$

$u_j = (9.81)(9.56) = 129.71$

$$\int w = \frac{\Delta x}{3}\left[u_1 + u_n + 2\sum u_{odd} + 4\sum u_{even} \right]$$

u_1 اول — u_n آخر — u_2+u_5 — u_2+u_n line — $+u_6$

u: ultra الشكل من ال equipotential عن كل

$u = \frac{x}{\Delta x}$ حيث : $y_j = (hP)_j \, \delta w$

اذا الخطأ الارتفاع راح يعطي scale (اقيسه بالمسطرة cm وبعدها حولها الى m)

مثال: 20 m ← 1 cm

المسافة بينهما ← x ← 2 cm (اقيس الارتفاع اللي بيا المسطرة)

Example 2.2

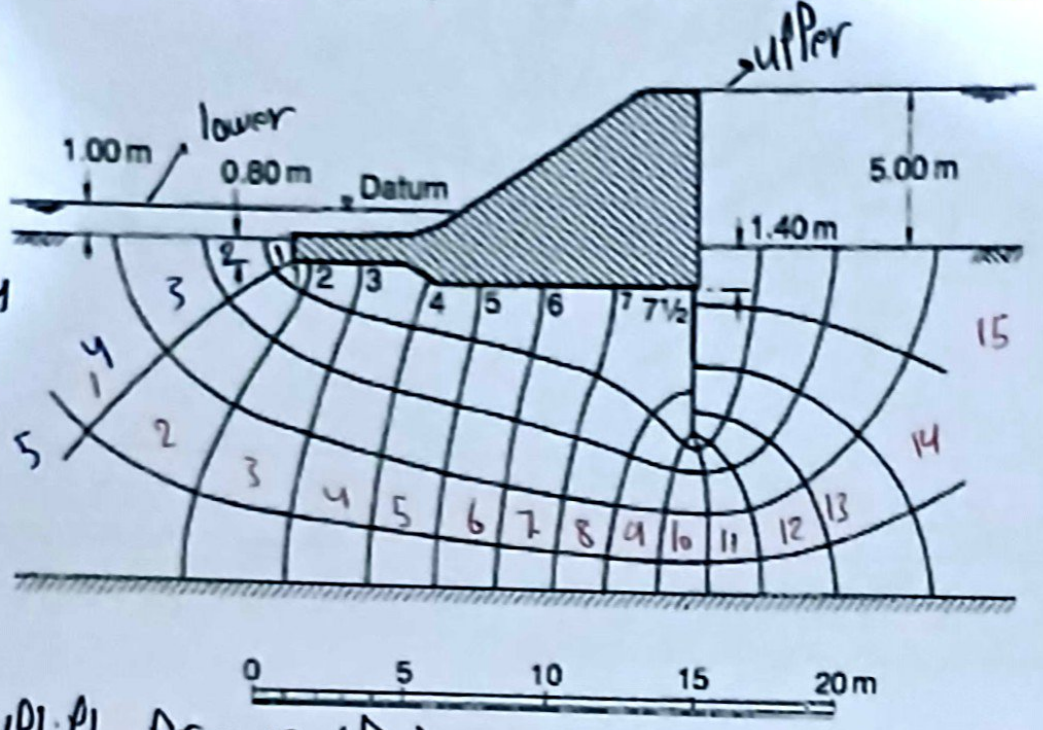
q: quantity of seepage

- 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 2.5×10^{-5} m/s.

$N_f = 5$

$N_d = 15$

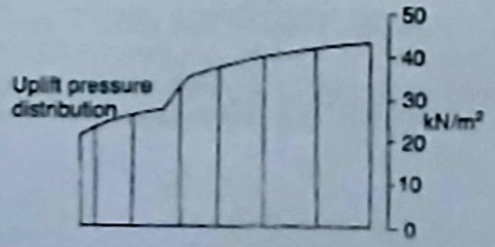
$\Delta H = 5 - 1 = 4$



Force uplift Pressure (P_w)

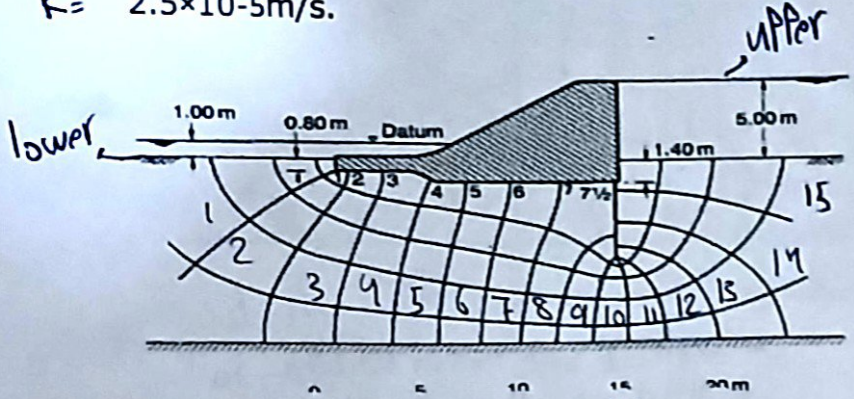
$$q = kh\frac{N_f}{N_d} = 2.5 \times 10^{-5} \times 4.00 \times \frac{4.7}{15}$$
$$= 3.1 \times 10^{-5}\ \text{m}^3/\text{s} \quad \text{(per m)}$$

Point	h (m)	z (m)	$h - z$ (m)	$u = \gamma_w(h - z)$ (kN/m²)
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



Example 3

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



$$q = kh\frac{N_f}{N_d} = 2.5 \times 10^{-5} \times 4.00 \times \frac{4.7}{15}$$
$$= 3.1 \times 10^{-5}\ \text{m}^3/\text{s} \quad \text{(per m)}$$

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

$\Delta H = 5 - 1 = 4$

$N_F = 5 \quad N_d = 15$

$q = (2.5 \times 10^{-5})(4)\left(\frac{5}{15}\right) =$

$= 3.1 \times 10^{-5}\ \text{m}^3/\text{s}/\text{m}$

k: coefficient of permeability

i: hydraulic gradient immediately

Example 4

- A river bed consists of a layer of sand 8.25 m thick overlying impermeable rock; the depth of water is 2.50 m. A long cofferdam 5.50 m wide is formed by driving two lines of sheet piling to a depth of 6.00 m below the level of the river bed and excavation to a depth of 2.00 m below bed level is carried out within the cofferdam. The water level within the cofferdam is kept at excavation level by pumping. If the flow of water in to the cofferdam is 0.25 m3 /h per unit length, what is the coefficient of permeability of the sand? What is the hydraulic gradient immediately below the excavated surface

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

$k = \frac{q}{h(N_f/N_d)}$

$= \frac{0.25}{4.50 \times 6/10 \times 60^2} = 2.6 \times 10^{-5}\ \text{m/s}$

$q = 0.25\ \text{m}^3/\text{h} \times \frac{1}{60 \times 60}$

=

The distance (Δs) between the last two equipotentials is measured as 0.9 m. The required hydraulic gradient is given by

$$i = \frac{\Delta h}{\Delta s}$$
$$= \frac{4.50}{10 \times 0.9} = 0.50$$

two lowers, two upper انتبه كذه

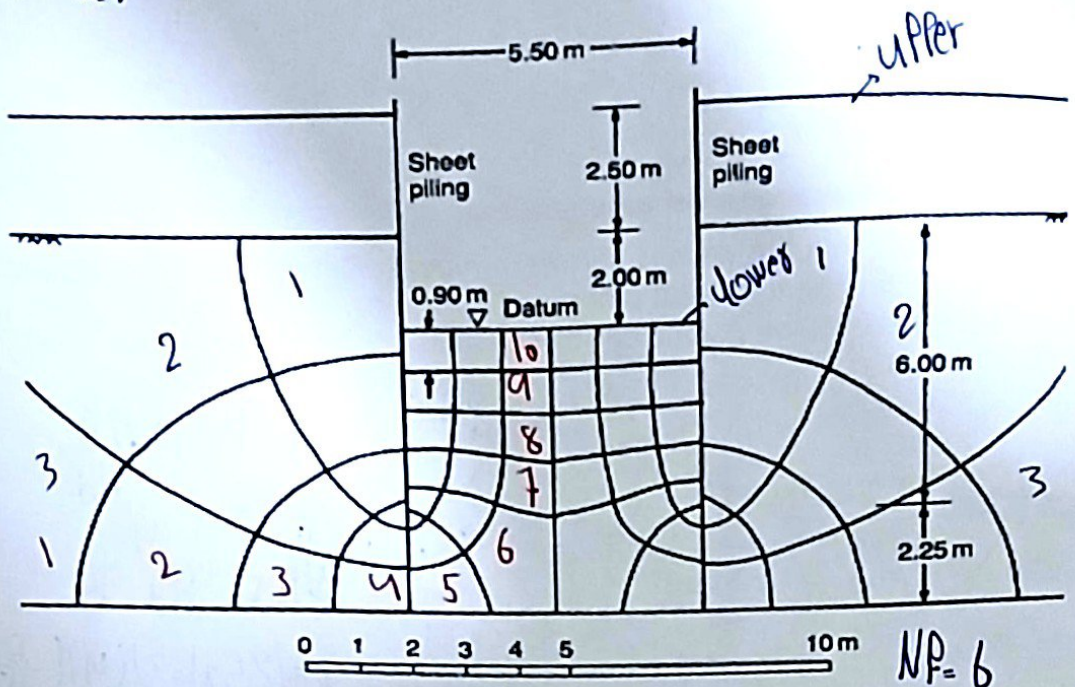


Figure 2.10 Example 2.3.

$Np = 6$

$Nd = 10$ (اكبر من واحدة لان الفقد من الجهة اليمنى = الفقد من الجهة اليسار)

$$= \frac{0.25}{4.50 \times 6/10 \times 60^2} = 2.6 \times 10^{-3}\ \text{m/s}$$

الارتفاع بين الـ $\Delta H = 2 + 2.5 = 4.5$

1. $K = \dfrac{q}{\Delta h(Np/Nd)} = \dfrac{25}{4.5(6/10)*60\times60} = 2.6\times10^{-3}$ m/s

lower و upper

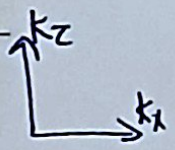
2. $i = \dfrac{\Delta h}{L_{min}} = \dfrac{\Delta H}{N_d L_{min}} = \dfrac{4.5}{10 * 0.90} = 0.5$

حد ماطار ما ال

isotropic $K_x = K_z$

correction على الـ K_x

Example 5

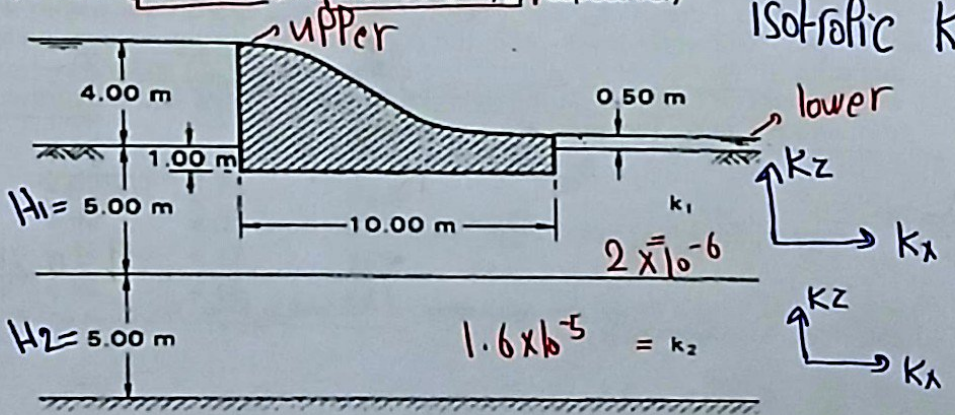


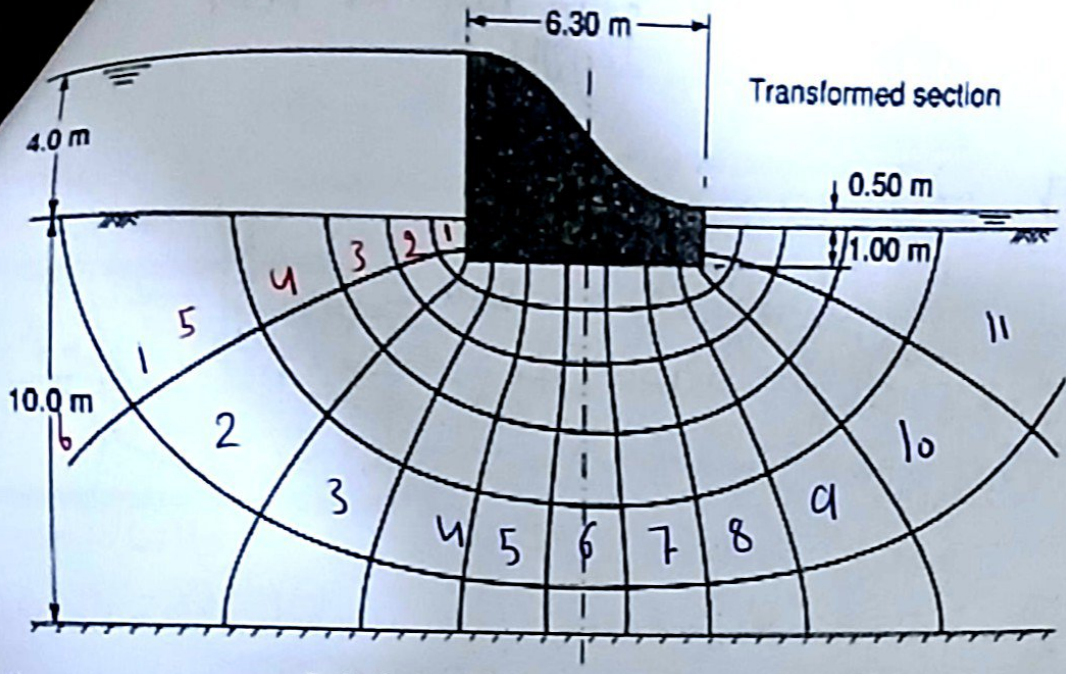
- Determine the quantity of seepage under the dam shown in section in Figure Both layers of soil are isotropic, the coefficients of permeability of the upper and lower layers being 2.0 ×10-6and 1.6 ×10-5 m/s, respectively

isotropic $K_x = K_z$

$q = K\Delta H \dfrac{Nf}{Nd}$

$\Delta H = 4 - 0.5 = 3.5$





$N_f = 6$

$N_d = 11$

$$\bar{k}_x = \frac{H_1k_1 + H_2k_2}{H_1 + H_2}$$

$$= \frac{5(2\times10^{-6}) + 5(1.6\times10^{-5})}{5+5} = 9\times10^{-6}$$

$$\bar{k}_z = \frac{H_1 + H_2}{\left(\frac{H_1}{k_1}\right) + \left(\frac{H_2}{k_2}\right)} = \frac{5+5}{\left(\frac{5}{2\times10^{-6}}\right) + \left(\frac{5}{1.6\times10^{-5}}\right)}$$

$$= 3.55\times10^{-6}$$

$$q = (5.65\times10^{-6})(3.5)\left(\frac{6}{11}\right) = 1.079\times10^{-5}\ \text{m}^3/\text{s/m}$$

$$k' = \sqrt{(k_x k_z)} = \sqrt{(9\times10^{-6})(3.55\times10^{-6})}$$

$$= 5.65\times10^{-6}$$

$$\bar{k}_x = \frac{H_1k_1 + H_2k_2}{H_1 + H_2} = \frac{10^{-6}}{10}\{(5\times2.0) + (5\times16)\} = 9.0\times10^{-6}\ \text{m/s}$$

$$\bar{k}_z = \frac{H_1 + H_2}{\frac{H_1}{k_1} + \frac{H_2}{k_2}} = \frac{10}{\frac{5}{(2\times10^{-6})} + \frac{5}{(16\times10^{-6})}}$$

$$= 3.6\times10^{-6}\ \text{m/s}$$

Then the scale transformation factor is given by

$$x_1 = x\frac{\sqrt{\bar{k}_z}}{\sqrt{\bar{k}_x}} = x\frac{\sqrt{3.6}}{\sqrt{9.0}} = 0.63x$$

$$x_1 = x\sqrt{\frac{k_z}{k_x}} = 10\sqrt{\frac{3.55\times10^{-6}}{9\times10^{-6}}}$$

$$= 6.30\ \text{m}$$

$$k' = \sqrt{(\bar{k}_x\bar{k}_z)} = \sqrt{(9.0\times3.6)}\times10^{-6} = 5.7\times10^{-6}\ \text{m/s}$$

Then the quantity of seepage is given by

$$q = k'h\frac{N_f}{N_d} = 5.7\times10^{-6}\times3.50\times\frac{5.6}{11}$$

$$= 1.0\times10^{-5}\ \text{m}^3/\text{s per m}$$

Example 6 q: Flow rate Per meter length

ΔH=5
Nf=5
Nd=11

q

Example 1):

For the system shown in Fig., find

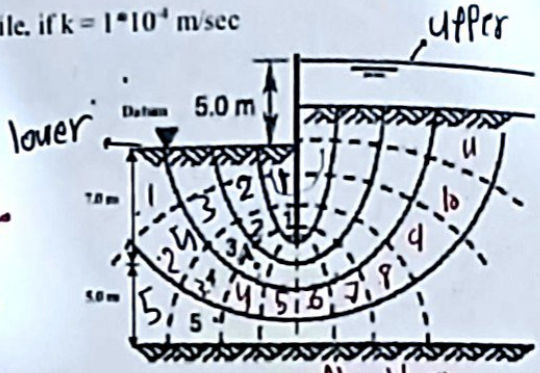
1. Flow rate per meter length of sheet pile. if $k = 1*10^{-4}$ m/sec
2. The FS against piping. L=1.5m

$FS = \frac{i_{cr}}{i_{ex}} \geq 1$ safe

Solution:

1) $q = k H \frac{N_f}{N_d}$ $i_{cr} > i$ safe

$q = 1*10^{-4}*5*\frac{5}{11} = 2.273*10^{-4}$ m³/sec/m



2) $FS = \frac{i_{cr}}{i_{ex}}$, $i_{cr} = \frac{G_s - 1}{1+e} = \frac{2.68-1}{1+0.57} = 1.07$

$i_{ex} = \frac{\Delta h}{L}$, $\Delta h = \frac{H}{N_d} = \frac{5}{11} = 0.455$

$i_{ex} = \frac{0.455}{1.5} = 0.303$

$FS = \frac{1.07}{0.303} = 3.531$

: الحل بالكود

$G_s = 2.68$ $e = 0.57$

Factor of safety $FS = \frac{i_{cr}}{i_{ex}} \geq 1$ Uplift Pressure = u

Example 7

Example 2):

The dam and flow net shown below, the dam is 120m long and has two sheet piles. Datum is at the impervious layer. Required the

1. Quantity of the seepage loss under the dam, $k = 20*10^{-4}$ cm/sec
2. Factor of safety against boiling.
3. Uplift pressure at point A & F.

u

$k = 20 \times 10^{-4} * \frac{1}{100} = 20 \times 10^{-6}$ m/s

Solution:

1) $q = k H \frac{N_f}{N_d}$

$q = 20*10^{-6}*12*\frac{3}{10.4}*120 = 8.308*10^{-3}$ m³/sec

2) $FS = \frac{i_{cr}}{i_{ex}}$

$i_{ex} = \frac{\Delta h}{L}$, $\Delta h = \frac{H}{N_d} = \frac{12}{10.4} = 1.154$

$i_{ex} = \frac{1.154}{8.5} = 0.136$

$FS = \frac{1.00}{0.136} = 7.353$

3) point A

$h_p = h_t - h_e$

$h_t = 42 - 3.5*1.154 = 37.961$ m

$h_e = 28.0$ m

$h_p = 37.961 - 28.0 = 9.961$ m

$u_A = 9.961 * 9.81 = 97.717$ kN/m²

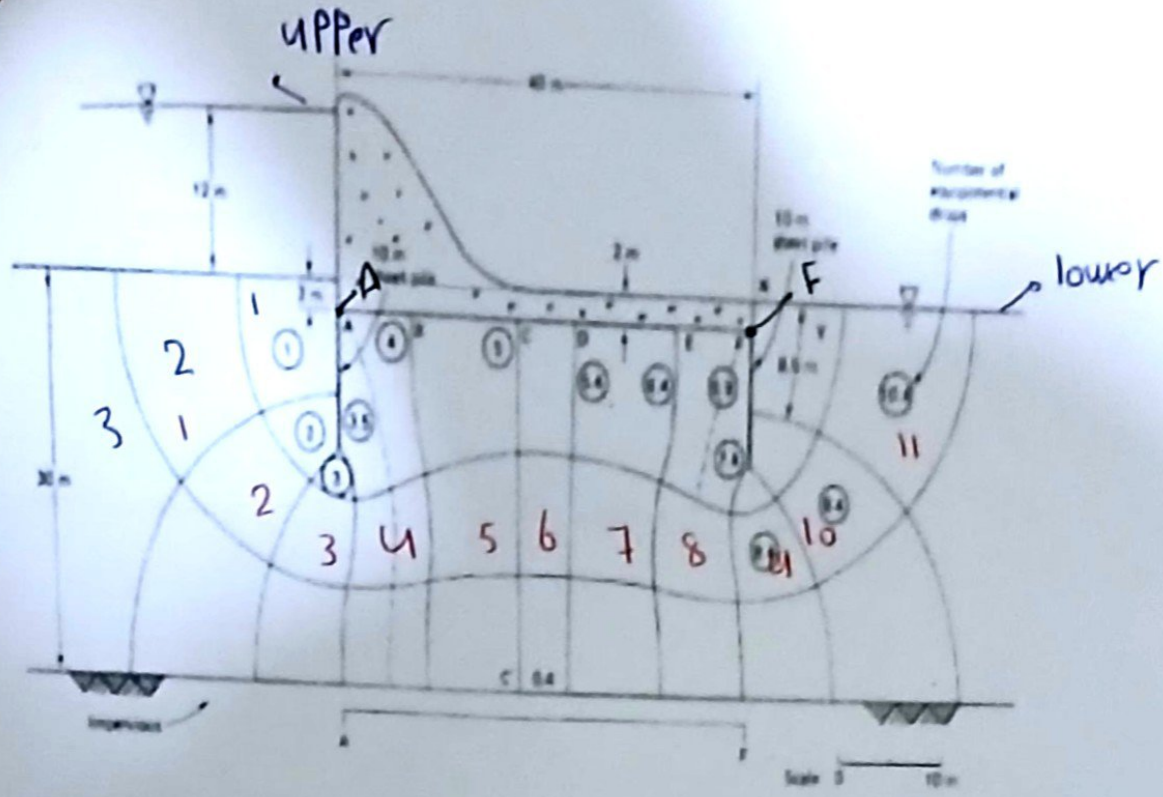
point F

$h_t = 42 - 6.9*1.154 = 34.037$ m

$h_e = 28.0$ m

$h_p = 34.037 - 28.0 = 6.037$ m

$u_F = 6.037 * 9.81 = 59.223$ kN/m²



$\Delta H = 12$
$N_F = 3$
$N_d = 11$

1. $q = k \Delta H \frac{N_F}{N_d} = 20 \times 10^{-6} * 12 * \frac{3}{11} * 120$

$= 7.85 \times 10^{-3} \, m^3/sec$

معم: انا دافتا احيد q وو مستا $m^3/sec/m$
لكلو وحدة طولو افقي
صون في السؤال كتر لكل 12m

2. $FS = \frac{i_{cr}}{i_{ex}} = \frac{1}{0.128} = 7.8 \geq 1$ ✓ safty

: $i_{ex} = \frac{\Delta h}{n_f} = \frac{12}{11 * 8.5} = 0.128$

3. $U = (hp) \gamma_w$

Point A:

$(hp)_A = \Delta H - (N_d)_A \Delta h - h_z = 12 - (3)\left(\frac{12}{11}\right) - -2) * 9.81 = 105.23$

Δh بدي حساب $\frac{\Delta H}{N_d}$

Point F:

$U = \left(12 - 8\left(\frac{12}{11}\right) - -2\right) 9.81) = 51.725$

Chapter 7: Effective stresses

→ Soil: skeleton (هيكل) of solid particles enclosing (يحيط) continuous voids (فراغات متصلة) which (air and or water)

→ For range of stresses (individual solid particles and water can be considered incompressible (غير قابلة للانضغاط))
(air is highly compressible (قابلة للانضغاط))

The volume of the soil skeleton as whole can change
→ due to rearrangement of the soil particles (إعادة ترتيب الجزيئات) into new positions (مواضع جديدة) (by rolling and sliding)
→ with a corresponding change in the forces acting between particles

actual compressibility of the soil skeleton will depend on the structural arrangement of the solid particles

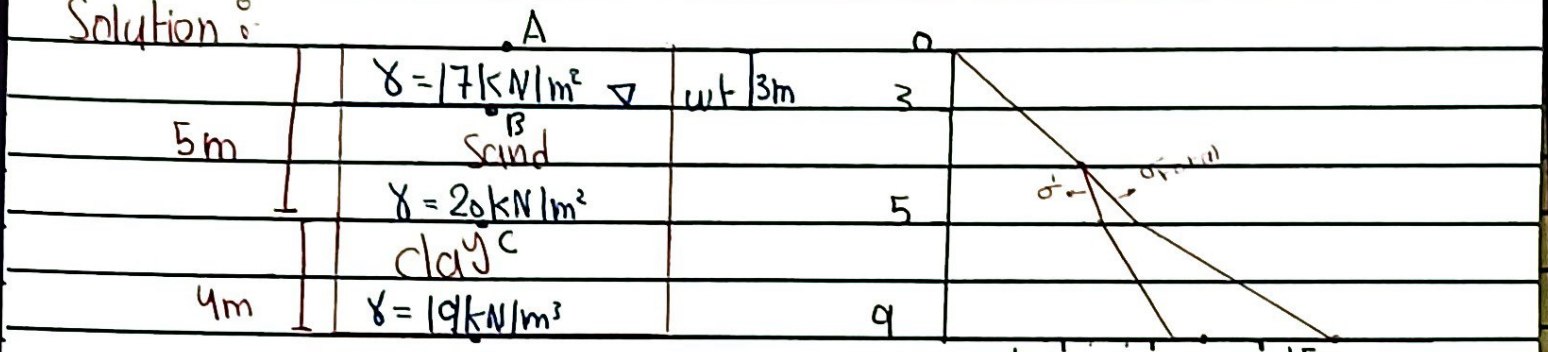
→ fully saturated soil (water incompressible) so the reduction the volume is possible only (إذا ~ some of the water can escape (يهرب) from the void)

→ dry or partially saturated soil reduction the volume is always possible (بسبب compression of the air in the voids) provided (بشرط) scope for particle rearrangement (إعادة ترتيب الجزيئات)

Example

- A layer of saturated clay 4m thick is overlain by sand 5m deep, the water table being 3m below the surface. The saturated unit weights of the clay and sand are 19 and 20 kN/m³, respectively; above the water table the unit weight of the sand is 17 kN/m³. Plot the values of total vertical stress and effective vertical stress against depth. If sand to a height of 1m above the water table is saturated with capillary water, how are the above stresses affected?

Solution:



A عند:

$\sigma_{total} = \gamma h = 0$ لأنه على السطح $u = \gamma_w h = 0$ $\sigma' = \sigma_{Total} - u = 0$

لأن لا يوجد أي وزن فوق النقطة

B عند: $0 = wt$ لأنه على السطح

$\sigma_{total} = \gamma h = 17(3) = 51$ $u = \gamma_w h = 9.81 \times 0 = 0$ $\sigma' = \sigma_{total} - u = 51 - 0 = 51$

C عند:

$\sigma_{Total} = (17)(3) + (20)(8) = 211$ $u = \gamma_w h = 9.81 \times 2 = 19.62$ $\sigma' = 211 - 19.62 = 191.38$

D عند:

$\sigma_{total} = (17)(3) + 20(2) + 19(4) = 167$ $u = (9.81)(6) = 58.86$ $\sigma' = 167 - 58.86 = 108.14$

Solution: a) لا يوجد ماء $\sigma' = \sigma_{Total} - u$

fill (أول طبقة) water عند

Take $u = 0$

$\sigma' = \sigma_{total}$

$= (5 * 9.2) + (3 \times 10.2) = 76.6$

b) fill الثانية

يوجد ماء $\sigma' = (4 * 20) + (5 \times 9.2) + (3 \times 10.2) = 156.6 kN/m^2$

Solution :-

Point N:

$\sigma' = \sigma_{Total} - u$

$\sigma_{Total} = \gamma h + \gamma_w h = (20)(10) + (9.81)(8) = 278.48$

$u = h_p \gamma_w = (\Delta H + (N_d)_j * \Delta h - \Delta z)\ \gamma_w$

$= \left(8 + 3 * \frac{8}{18} - 10\right) 9.81$ $\quad \because \Delta h = \Delta H$ $\quad (105.85 - 11)\ N_d$

$u = 163.5$

$\sigma_{total} = 278.48 - 163.5 = 114.98$

Point M:

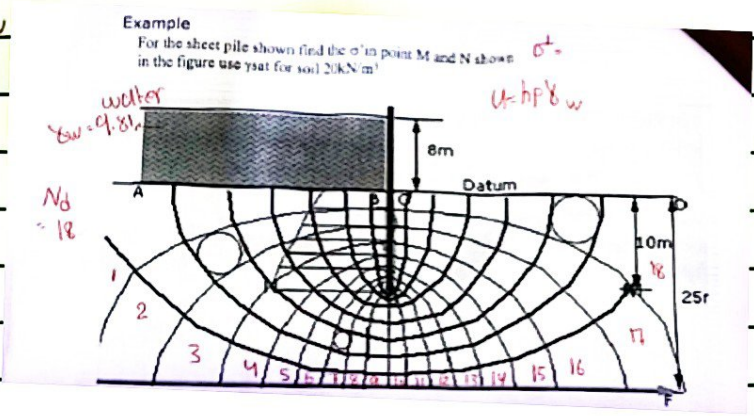
$\sigma_{total} = \gamma h = (20)(10) = 200$

$u = \left(8 - 17 * \frac{8}{18} - 10\right) 9.81 = 102.46$

$\sigma_{Total} = 200 - 102.46 = 97.54$

Example

For the sheet pile shown find the σ' in point M and N shown in the figure use γ_{sat} for soil 20kN/m³



calculate the total and effective stress at 1m, 3m, 6m:

1m A

2m — sand $e = 0.4$ $G_s = 2.65$

3m B

3m — clay $\gamma_{sat} = 20\ kN/m^3$

6m

$\gamma_{dray} = \frac{G_s}{1+e} * \gamma_w = \frac{2.65}{1+0.4} * 9.81 = 18.569$

$\gamma_{sat} = \frac{G_s + e}{1+e} * \gamma_w = \frac{2.65 + 0.4}{1+0.4} * 981 = 21.3726\ mC$

at A:	at B:	at C:
$\sigma_{Total} = 18.569 * 1 = 18.569$	$\sigma_{total} = (21.372)(2) + 18.591(1)$	$\sigma_{Total} = (20)(3) + (21.372)(2)$
$u = 0$ (at water surface)	$= 61.313$	$+ 18.591(1) = 121.313$
$\sigma' = \sigma_{total} = 18.569$	$u = \gamma_w h = (9.81)(2) = 19.62$	$u = 9.81 * 5 = 49.05$
	$\sigma' = 61.313 - 19.62 = 41.693$	$\sigma' = 121.313 - 49.05$
		$= 72.263$

Chapter 8: stress Increment From Elastic Solution:

- Point load
- line load
- strip load : strip area carrying uniform / strip area carrying linearly
- circular loaded area
- Rectangular loaded area
- APProximate Method

* Point load:

Q: load (kN) : عن طريق المعادلة

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

z: surface عن النقطة عن ال

r: المسافة الافقية من تأثير ال load (البعد horizontal بين ال load واللي عندنا) distance النقطة اللي احنا عندها

: عن طريق اكبرون

$$\sigma z = \frac{Q}{z^2} IP : \quad IP = \frac{3}{2\pi}\left[\frac{1}{1+(r/z)^2}\right]^{5/2}$$

لو اذا كان ال $r/z = 0.43$ كيف نجد IP ؟؟ راح نعمل interPolation

لما راح نكون بين ال 0.4 و 0.5

r/z	IP
0.4	0.329
0.43 →	X
0.5	0.273

$$\frac{0.5-0.4}{0.273-0.329} = \frac{0.5-0.43}{0.273-X}$$

* Line load:

كيف فرق بين ال Point load وال line load ؟؟

Point load: عن طريق الوحدة Q

line load: Q/m

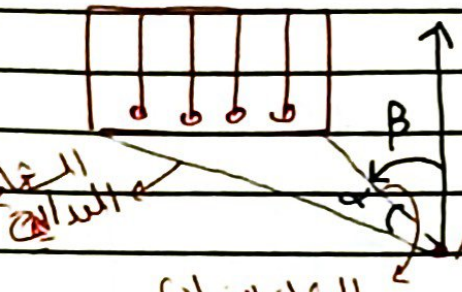
$$\sigma z = \frac{2Q}{\pi}\frac{z^3}{(x^2+z^2)^2}$$

horizontal ↳

Strip area carring uniform Pressure:

$$\sigma_z = \frac{q}{\pi}\left[\alpha + \sin\alpha\cos(\alpha+2\beta)\right] \quad : q = \text{Force/area}$$

rad * π / 180 — deg

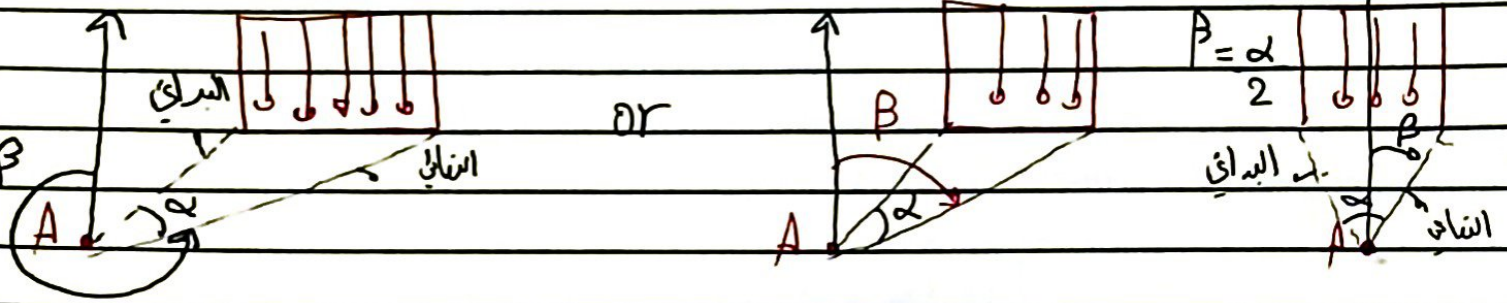


- اعلى شعاع لوحاله طرفين الـ distributed
- الزاوية اللي هندور عليها α
- اعلى Vertical line عند النقطة A
- من الـ Vertical لعند الشعاع الثاني عند زاوية β الـ line

هم كيفاش ؟ (كأني اكتب بالانجليزي بخصوص المثال اللي تحت)

الشعاع الثاني من البداية

لبنجيب α لازم الـ β نبدأ من الـ Vertical ونرسم الشعاع الثاني line (حسب موقع النقطة لو كيكون على يمين ولا يسار الساحة)



Strip area carring linearly increasing Pressure

$$\sigma_z = \frac{q}{\pi}\left(\frac{\chi}{\beta}\alpha - \frac{1}{2}\sin 2\beta\right)$$

rad * π / 180 — deg

ازاد الـ β و α دفت على طريقة الـ Uniform

Circular area carrying uniform Pressure :

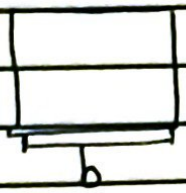
$\sigma_z = q \left[1 - \left[\frac{1}{1+(R/Z)^2} \right]^{3/2} \right] = q\, I_c$: q = Force / area (πr^2)

I_c : معامل التأثير — من chart

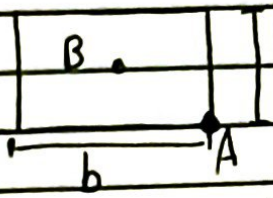
Rectangular area carrying uniform Pressure :

$\sigma_z = q\, I_r$ ← chart من (m و n من الرسم) : q = Force / area ($b \times L$)

$n = \frac{L}{Z}$ (العمق) $m = \frac{b}{Z}$



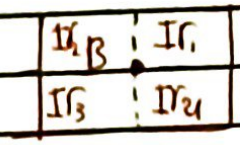
عند النقطة A



$\sigma_z = q\, I_r$ (نحسب I_r من chart بعد ما نحدد m و n لكل مستطيل لوحده)

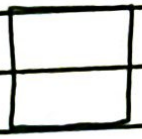
q بالمساحة الكلية

عند النقطة B (corner) لأن الـ stress الأكبر يكون عند الـ corner



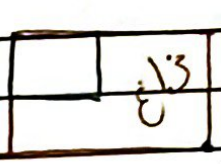
$\sigma_z = q(I_{r_1} + I_{r_2} + I_{r_3} + I_{r_4})$

I_r من الـ chart



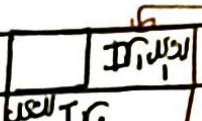
النقطة A في الشكل ولازم أعمل corner لأنها

أعمل extend



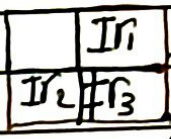
σ_z = أجيب stress الشكل الأصلي لازم

أطلعه من I_r الزائد (إذا قسم زائد)



$I_r - I_{r_1} - I_{r_2}$

(الشكل كامل)



$\sigma = (I_r - I_{r_1} - I_{r_2} + I_{r_3})q$

I_{r_1} هون المستطيل I_{r_1} موجب مرتين (مرة لـ I_r ومرة دخلت بالشكل كامل)

كيف أعرف النقطة اللي مار؟؟ لازم أزيد

APProximate Method 2:1 method: ← التقريبية الطريقة

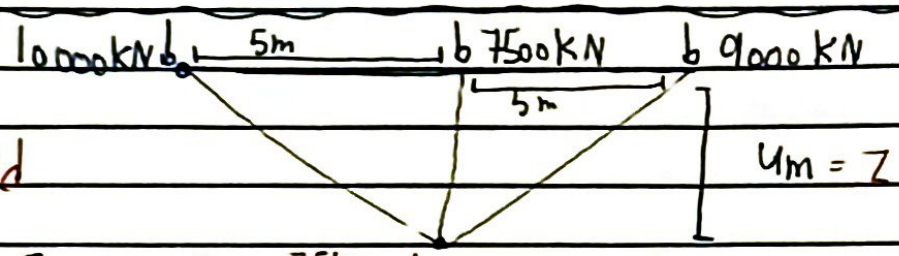
→ strip load $\Delta\sigma_v' = \frac{q_0(B \times 1)}{(B+z)}$ — stress عند نقطة تحت ال center

→ Square load

$$\Delta\sigma_v' = \frac{q_0(B \times B)}{(B+z)(B+z)}$$

→ Rectangular load

$$\Delta\sigma_v' = \frac{q_0(B \times L)}{(B+z)(L+z)}$$



Example 1:
Three Point load

$$\sigma_A = \frac{3(10000)}{2\pi(4)^2}\left[\frac{1}{1+(5/4)^2}\right]^{5/2} \quad (10000)$$

$$\sigma_A = \frac{3(7500)}{2\pi(4)^2}\left[\frac{1}{1+(6/4)^2}\right]^{5/2} \quad (7500)$$

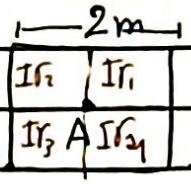
$$\sigma = \frac{3(9000)}{2\pi(4)^2}\left[\frac{1}{1+(5/4)^2}\right]^{5/2} \quad (9000)$$

$\sigma_{Total} = \sigma_{10000} + \sigma_{7500} + \sigma_{9000}$

$= 27\,kN/m^2$

Example 2: Load = 1500kN Foundation 2m-Square
Determine vertical stress at a Point 5m below center
a) load uniformly distributed over the Foundation
b) Load acts as a Point load

Sol. a)



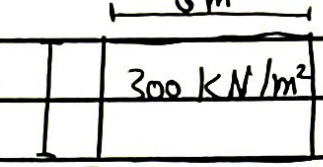
$n = \frac{2}{2} = 1$ $\quad m = \frac{2}{2} = 1$ $\quad$ Figure 11 (a) b

$Ir = 0.018$

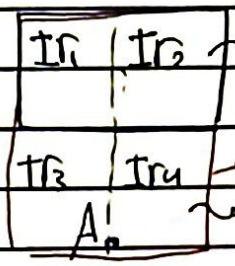
$\sigma = (q.Ir)y = \frac{4(1500)}{(2*2)}(0.018) = 27\ kN/m^2$

b) $\sigma_z = \frac{3(1500)}{2\pi(5)^2}\left[\frac{1}{1+(0/5)^2}\right]^{5/2} = 28.68 \simeq 29$

Example 3:



A rectangular Foundation
Determine the vertical stress at a depth of 3m below a Point A ??



$\sigma = (2Ir - 2Ir)q$ $\quad m = 6$ $\quad n = 3$

$Ir = 0.120$

Ir_3 : $m = 3/3 = 1$ $\quad n = 1.5/3 = 0.5$ $= Ir_{2\&8}$ $\quad m = 3/3 = 1$ $\quad n = 0.5 \sim 2Ir$ (للوراء)

Ir_1: $m = 3/3 = 1$ $\quad n = 4.5/3 = 1.5$ $\quad Ir_4$: $m = 3/3 = 1$ $\quad n = 4.5/3 = 1.5$ (للأمام)

$Ir = 0.193$

$\sigma_z = (2(0.193) - 2(0.120)) * 300$

$= 44\ kN/m^2$

Example 4: strip footing

$q = 250\ kN/m^2$ — 2m — ▽ water table — $\gamma_{sat} = 20\ kN/m^3$ — 3m — A

before and after ??

Sol. Before loading $\sigma_{(soil\ \text{الإجهاد})} = \sigma_{total} - u_{internal}$

▽

3m — A

$\sigma_{Total} = \gamma_{sat} * 3 = 20 * 3 = 60$

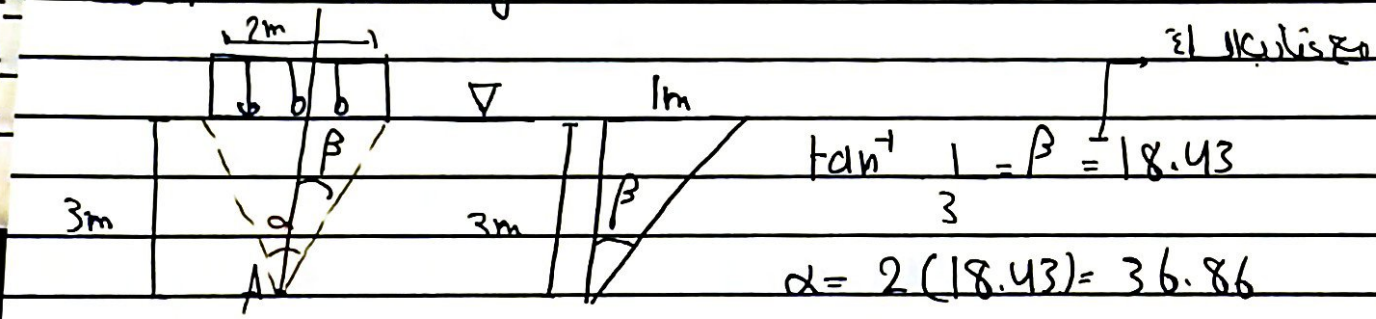
(الإجهاد الكلي) (الماء)

A ||

$u = \gamma_w h = 9.8 * 3$

$\Rightarrow \sigma_0 = \sigma_{total} - u = 30.6\ kN/m^2$

after loading



$\tan^{-1}\frac{1}{3} = \beta = 18.43$

$\alpha = 2(18.43) = 36.86$

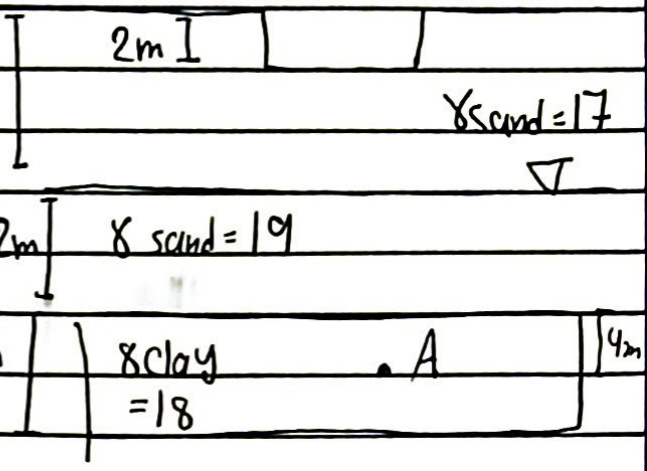
$\sigma_z = \frac{q}{\pi}\left(\alpha + \sin\alpha \cos(\alpha + 2\beta)\right)$

$= \frac{250}{\pi}\left(\frac{36.86 * \pi}{180} + \sin(36.86)\cos(36.86 + -(2*18.43)\right) = 98.94 \approx 99$ (0)

after $\sigma = 99 + 30.6 = 129.6\ kN/m^2$

Example 5:

Determine effective stress before 4m and after applying the load at the load at the middle of clay layer: 2m



→ before $\sigma_D = \sigma_{Total} - u$

$\sigma_{total} = (18)(4) + 19(2) + (17)(4) - u_A = \gamma_w h = 9.81 * 6 = 58.86$

$= 178$

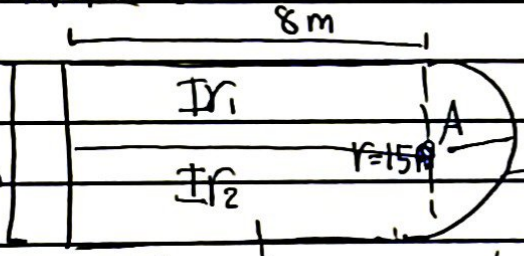
$= 119.14\ kN/m^2$

→ after $\Rightarrow \sigma_z = q\left[1 - \frac{1}{1+(R/Z)^2}\right]^{3/2} \Rightarrow$

$= \frac{2000}{\frac{\pi}{4}(4)^2}\left[1 - \frac{1}{1+(2/8)^2}\right]^{3/2} = 2.269$

$\rightarrow \sigma = 2.269 + 119.14 = 121.4$

Example 6:

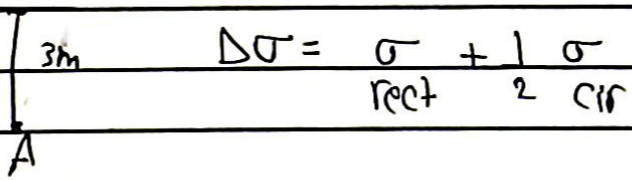


$q = 150\ kN/m^2$

Circular $\sigma_z = q I_c$

rectangular → $\sigma = (I_{r1} + I_{r2})q$

$= \frac{1}{2}\sigma_z$



$\Delta\sigma = \underset{rect}{\sigma} + \frac{1}{2}\underset{cir}{\sigma}$

Chapter 9: consolidation thory

Consolidation : reduction in Volume (انضغاط في الحجم) of a fully saturated low Permeability soil (such as: fine soil(clay)) due to drainage of some of the pore water (when the soil is subjected to stress)

باختصار: الماء اللي بالفراغات راح يطلع نتيجة تؤثر التربة stress وراح يصير consolidation الانضغاط بالحجم reduction in Volume هذا اللي بيصير
↳ اذا التربة فراغاتها تكون مشبعة
. fully saturated بالماء

- Swelling is the reverse (عكس) of consolidation
  - increase in Volume (زيادة) ↳ reduction in Volum (انضغاط)
- stress ↑ Pressure ↑ Because the hydraulic conductivity of clay very small

. consolidation جزء من الثانية Collects (يجمع) settlement ما يصير

Mv : Volume change (ΔV) Per unit Volume (V_0) Per unit increase in effective stress

$$= \frac{\frac{\Delta V}{V_0}}{\sigma_1' - \sigma_0'} = \frac{\Delta V}{V_0(\sigma_1' - \sigma_0')} = m^2/MN$$

- $m_v = \frac{\Delta e}{1+e_0(\sigma_1' - \sigma_0')} = \frac{1}{1+e}\left(\frac{e_0 - e_1}{\sigma_1' - \sigma_0'}\right)$; H او e بدل V لانه بالاساس الـ $V_0 = 1 + e_0$

- $m_v = \frac{\Delta H}{H_0(\sigma_1' - \sigma_0')} = \frac{1}{H_0}\left(\frac{H_0 - H_1}{\sigma_1' - \sigma_0'}\right)$

→ Preconsolidation Pressure $OCR = \frac{\sigma'_c}{\sigma'_o}$

OCR > 1 : $\sigma'_c > \sigma'_o$	OCR = 1 : $\sigma'_c = \sigma'_o$
كان عندي structure وتم ازالته والتربة تعرضت stress وتعرضت consolidation (over consolidation)	اذا ماكان عندي structure بعض التربة تعرضت لـ stress و consolidation (normally consolidation)

• لاجد ادخال Cc من العلاقة:

$$C_c = \frac{C_1 - C_2}{\log \sigma'_2 - \log \sigma'_1} = \frac{C_1 - C_2}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)}$$

→ آخر نقطة بنزل منها الـ curve لعمل خط horizontal وبعدها اعمل خط مماس ثم اخدهم واعمل امتداد لخط الـ linear الى ان يقطع الخط اللي رسمته ثم نعمل خط ع ودي من النقطة اللي قطعت الخطين الـ 2 ورا والنقطة في σ_c.

→ هبوط الـ Clay

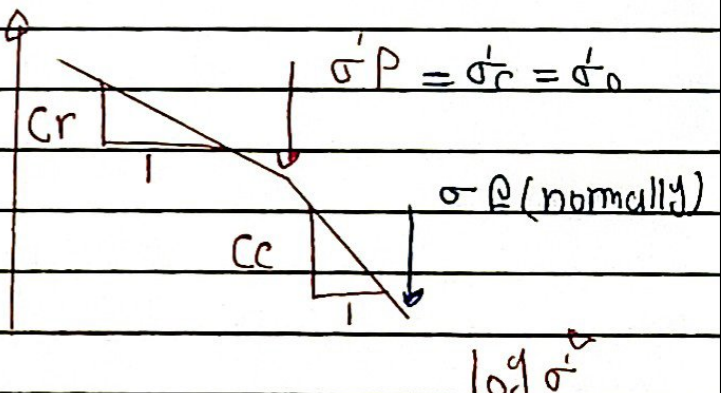
$$S_c \text{ (consolidation settlement)} = \frac{C_c}{1+e_o} H$$

→ Primary consolidation Settlement

• Normally consolidation Soil

$$S_c = \frac{C_c}{1+e_o} H \log\left(\frac{\sigma'_p}{\sigma'_o}\right)$$

→ $OCR = 1 : \sigma'_c = \sigma'_o$

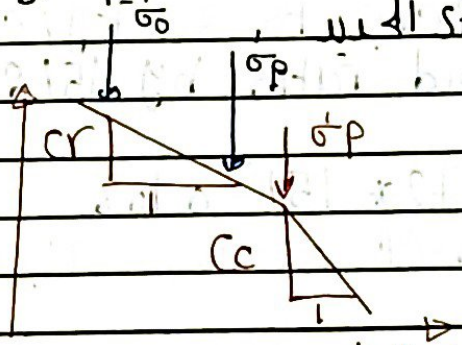


slope $C_c = \frac{\Delta e}{\log \sigma_c - \log \sigma_o}$ هنا الـ σ_c كانت في منطقة الـ C_c واخدنا الـ slope C_c (منفرد)

(OCR) ؛ σ'_c هي σ'_o هي over consolidation

1. $\sigma'_o < \sigma'_p < \sigma'_f$

اذا كان الـ structure القديم اكبر من الـ structure الجديد

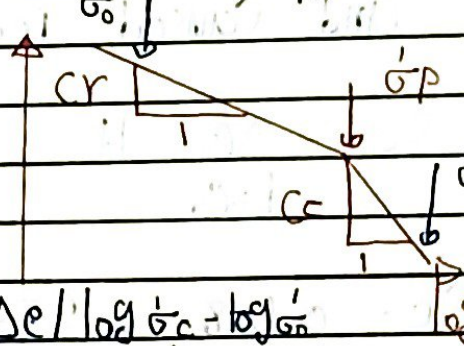


$Cr = \Delta e / \log \sigma'_p - \log \sigma'_o$

$Sc = \frac{Cc \cdot H}{1+e_o} \log\left(\frac{\sigma'_p}{\sigma'_o}\right)$

2. $\sigma'_o < \sigma'_p < \sigma'_f$

اذا كان الـ structure الجديد اكبر من الـ structure القديم



$Cr = \Delta e / \log \sigma'_c - \log \sigma'_o$

$Cc = \Delta e / \log \sigma'_f - \log \sigma'_c$

$Sc = \frac{Cr \cdot H}{1+e_o} \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + \frac{Cc \cdot H}{1+e_o} \log\left(\frac{\sigma'_f}{\sigma'_p}\right)$

Degree of consolidation $U\% = \frac{e_o - e}{e_o - e_f}$ وقت

Tv (dimensionless time) $= \frac{Cv \cdot t}{d^2}$: $Cv = \frac{k}{mv \cdot \gamma_w} = \frac{m^2}{year}$

$d = H/2$ double drainage

$d = H$ single drainage

sand
clay
sand

طبقة الـ clay تكون محصورة بين طبقتين من الـ sand

sand
clay
rock

طبقة واحدة من الـ sand فوق طبقة الـ clay

- $U \leq 0.60 : Tv = \frac{\pi}{4} U^2$
- $U > 0.60 : Tv = -0.933 \log(1-U) - 0.085$

Ex1: Compute the expected settlement of a building constructed on a soil layer of 2.5m thick if $e_o = 0.72$ the pressure increased from 1.5 kg/cm² to 3.8 kg/cm² and caused reduction in void ratio by 15%.

Sol.

$$S_c = \frac{\Delta e}{1+e_o} H \quad : \quad \Delta e = 0.72 * 15\% = 0.108$$

$$\Delta e = e_o - e_f \rightarrow 0.108 = 0.72 - e_f$$

$$S_c = \frac{0.108}{1+0.72} * 2.5 = 0.157m = 15.7cm \qquad \boxed{e_f = 0.612}$$

Ex3: The laboratory consolidation data for an undisturbed clay specimen are as follows:

$e_1 = 1.1$ $\sigma_1' = 95$ kN/m²

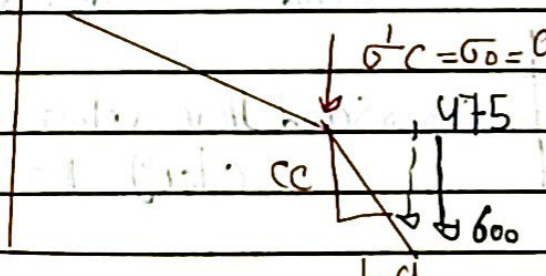
$e_2 = 0.9$ $\sigma_2' = 475$ kN/m²

what will be the void ratio for a pressure of 600 kN/m²?

Sol.

$$OCR = \frac{\sigma_c'}{\sigma_o'} = \frac{95}{95} = 1 \text{ (normally)}$$

Note $\sigma_c' / 95$ kN/m²



$$C_c = \frac{\Delta e}{\log \sigma_2' - \log \sigma_o'} = \frac{1.1 - 0.9}{\log 475 - \log 95} = 0.286$$

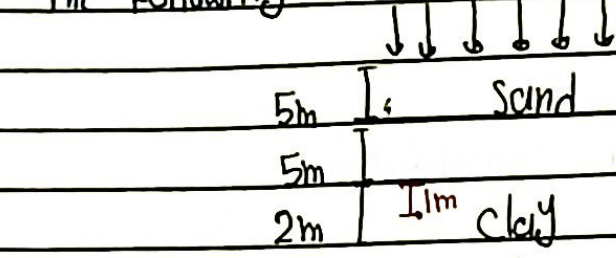
$$C_c = \frac{\Delta e}{\log 600 - \log 95} \rightarrow 0.286 = \frac{\Delta e}{0.8}$$

$$\Delta e = 0.228$$

$$\Delta e = e_o - e_1$$

$$0.228 = 1.1 - e_1 \rightsquigarrow \boxed{e_1 = 0.87}$$

EX2: determine the Primary consolidation settlement if $C_c = 0.5$ for the following 2) $\sigma'_c = 150$ kPa 3) $\sigma'_c = 175$ kPa 4) $\sigma'_c = 250$ kPa



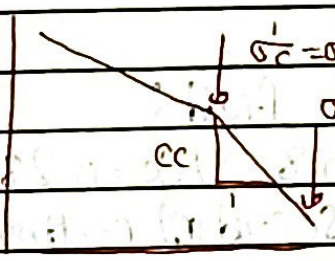
Solution: $\sigma'_c = 150$ kPa

1) $OCR = \frac{\sigma'_c}{\sigma'_o} = \frac{150}{142.834} = 1.05 > 1$ ∴ normally

$\sigma_o = \sigma_T - u = 201.7 - 58.86 = 142.834$

$\sigma_T = (9.7)(1) + (19.7)(5) + (18.7)(5) = 201.7$

$u = \gamma_w h = 9.81 \times 6$ (wt من سطح الماء) $= 58.86$



$\sigma_p = \sigma'_o + \Delta\sigma = 142.84 + 50 = 192.84$

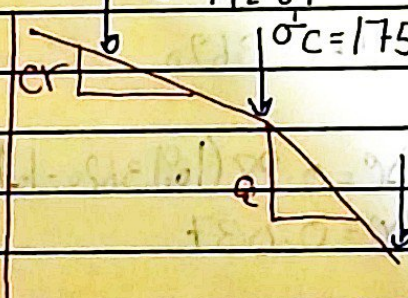
$C_c = \frac{\Delta e}{\log\sigma_p + \log\sigma_c}$ ∴ $\Delta e = 0.83(\log 192.84 - \log 150)$

$\Delta e = 0.0906$

$S_c = \frac{\Delta e}{1+e_o} H = \frac{0.0906}{1+1.1} \times 2 = 0.0863$

$\sigma'_c = 175$ kPa

1) $OCR = \frac{\sigma'_c}{\sigma_o} = \frac{175}{142.84} = 1.225 > 1$ (over consolidation)



$C_c = \frac{\Delta e}{\log\sigma_p - \log\sigma_c}$, $\Delta e = 0.83(\log 192.84 - \log 175) = 0.035$

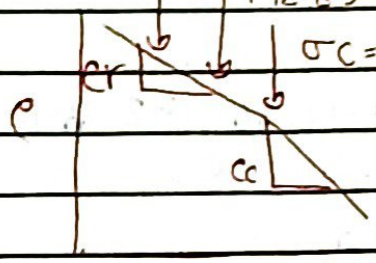
$C_R = \frac{\Delta e}{\log\sigma'_c - \log\sigma_o}$, $\Delta e = 0.05(\log 175 - \log 142.84) = 0.0041$

$\Delta e = 0.035 + 0.0041 = 0.0391$

$S_c = \frac{\Delta e}{1+e_o} H = \frac{0.0391}{1+1.1} \times 2 = 0.037$ m

$\sigma'_c = 250$ M.Pa ..... EX2 حل مثال

1. $OCR = \frac{\sigma'_c}{\sigma'_0} = \frac{250}{142.84} = 1.75 > 1$ (over consolidation)



$C_R = \frac{\Delta e}{\log\sigma'_f - \log\sigma_0}$ → $\Delta e = 0.83(\log(192.85) - \log(142.85))$

$\Delta e = 0.1082$

$S_c = \frac{\Delta e}{1+e_0} H = \frac{0.1082}{1+1.1} \times 2 = 0.103m$

EX4: for anormally consolidated clay layer in the field:

1) thickness of clay layer = 8.5 ft 2) $e_0 = 0.8$ 3) $C_c = 0.28$

4) $\sigma'_0 = 2650$ lb/ft² 5) $\Delta\sigma' = 970$ lb/ft² 6) secondary compression $C_\alpha = 0.02$

OCR=1

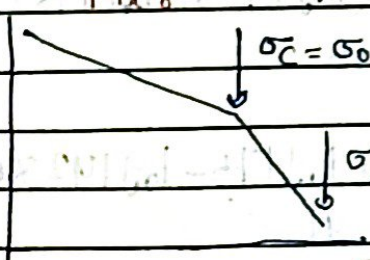
what is the total consolidation settlement of the clay layer five years after the completion of primary consolidation settlement?

Note: Time for completion of primary settlement = 1.5 year.

Primary settlement $\Delta e \sim$ $OCR = \frac{\sigma'_c}{\sigma_0} = 1$ (normally loaded)

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

$\therefore \sigma'_f = \Delta\sigma' + \sigma'_0 = 970 + 2650 = 3620$



$C_c = \frac{\Delta e}{\log 3620 - \log 2650}$ → $\Delta e = 0.28(\log 3620 - \log 2650)$

$\Delta e = 0.037$

$S_c = \frac{0.037}{1+0.8}(8.5) = 0.1794$ ft

$= (0.1794 \times 12)$ in $= 2.1528$ in

Secondary Settlement (S_s) $= \frac{\Delta e}{1+e_o} H$: EX4 دخلت

(Primary) e_o (secondary) $= e_1$ (Primary) $\Delta e = e_o - e_1$

$0.038 = 0.8 - e_1 \rightarrow e_1 = 0.762 = e_o$ (Primary) secondary

$\therefore C_\alpha = 0.02$

$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} \rightarrow 0.02 = \frac{\Delta e}{\log 5 - \log 1.5} \rightarrow \Delta e = 0.01046$

$S_c = \frac{\Delta e}{1+e_o} H = \frac{0.01046}{1+0.762} * (8.5) = 0.05 ft$

$= (0.05)(12)$ in $= 0.6$ in

total consolidation settlement $= 0.1794 + 0.05 = 0.2294$ ft

$= 2.1528 + 0.6 = 2.7528$ in

$d = H/2$ double

EX5: The time required for 50% consolidation of a 25-mm thick clay layer (drained at both top and bottom) in the laboratory is 2min 20sec. 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. $d = H$, single

Lab $\sim T_v = \frac{C_v t_{50}}{d^2}$: U=50% 60% $T_v = \frac{\pi}{4}(50\%)^2 = 0.196$

$0.196 = \frac{C_v (140)}{(12.5)^2}$ $t_{50} = 2min\ 20sec = 2(60) + 20 = 140sec$

$C_v = 2.1875 \times 10^{-7}\ m^2/sec$ double $d = \frac{H}{2} = \frac{25}{2} = 12.5$

Field ~~U=50% 60%~~ t_{50} ~~double~~

Field $\sim T_v = \frac{C_v t_{50}}{d^2}$: U=50% 60% $T_v = 0.196$

$C_v = 2.1875 \times 10^{-7}$ (for the same clay)

$0.196 = \frac{(2.1875 \times 10^{-7}) t_{50}}{3^2}$ single $d = H = 3$

$\therefore t_{50} = (8064000 sec) / 60*60*24 = 93.33$ days

Ex6: refer Ex10.7 How long (in days) will it take in the field for 30% primary consolidation to occur?
(Ex5 نفس المثال)

$t_{30} = 22$

$T_v = \frac{C_v t_{30}}{d^2}$: $C_v = 2.1875 \times 10^{-7}$ من نفس المسألة

$U = 30\% \rightarrow 60\%$ ∴ $T_v = \frac{T_1}{4}(30\%)^2 = 0.71$

$\Rightarrow 0.71 = \frac{2.1875 \times 10^{-7} \, t_{30}}{3^2}$ field ال 3-m thic من المسألة السابقة

$\therefore t_{30} = (29211942.857 \text{ sec})$ (single)

$60 \times 60 \times 24$

$t_{30} = 338.6$ days

$\rightarrow d = H/2$

Ex7: 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.

C_v: ؟؟؟؟ $T_v = \frac{C_v t_{90}}{d^2}$: $U = 90\% \setminus 60\%$

$T_v = -0.933 \log(1 - 90\%) - 0.085$

$T_v = 0.848$

t_{90} (75 days) $* 60 * 60 * 24$

$t_{90} = 6480000$ sec

double $d = H/2 = 3/2 = 1.5$

$\Rightarrow 0.848 = \frac{C_v (6480000)}{1.5^2}$ m → cm

$C_v = 0.00294 \text{ m}^2/\text{sec}$

$= 2.94 \times 10^{-7} \text{ m}^2/\text{sec} * (100)^2$

$= 0.00294 \text{ cm}^2/\text{sec}$

Ex8: For a normally consolidation laboratory clay specimen drained on both sides the following given:

$\sigma'_o = 3000\ lb/ft^2$ $e = e_o = 1.1$ thick of clay = 1in

$\sigma'_f = \sigma'_o + \Delta\sigma' = 6000\ lb/ft^2$ $e = 0.9$ Time for 50% consolidation = 2min

a) Determine the hydraulic conductivity (ft/min) of the clay for the loading range:

$C_v = \frac{k}{m_v \gamma_w}$: $\gamma_w = 62.4\ pf$ $m_v = \frac{\Delta e}{1+e_o}\left(\frac{1}{\sigma'_f - \sigma'_i}\right)$

$= \frac{1.1-0.9}{1+1.1}\left(\frac{1}{6000-3000}\right)$

$= 3.17 \times 10^{-5}$

$T_v = \frac{C_v t_{50}}{d^2}$ ⇝ $U = 50\% / 60\%$ $T_v = \frac{\pi}{4}(50\%) = 0.196$ double drainage $d = \frac{H}{2} = \frac{1}{2} = 0.5$

$0.196 = \frac{C_v (2*60)}{(0.5)^2}$ ⇒ $C_v = 4.833 \times 10^{-4}\ in^2/sec \times \frac{1}{12}$

$= 2.8356 \times 10^{-6}\ ft^2/sec$

$\approx 2.836 \times 10^{-6}\ ft^2/sec$

→ $K = C_v m_v \gamma_w$

$= (2.836 \times 10^{-6})(3.17 * 10^{-5})(62.4) = 5.617 \times 10^{-9}\ ft^2$

$\frac{sec}{min}$ $(5.617 \times 10^{-9} \times 60)\ ft^2$

60 $= 3.37 \times 10^{-7}\ ft^2$ min

min

b) How long (in days) will it take for a 6-ft clay layer in the field (drained on one side) to reach 60% consolid??

$d = H$ t_{60} obtain

$T_v = \frac{C_v t_{60}}{d^2}$ ⇝ $U = 60\% \leq 60\%$ $T_v = \frac{\pi}{4}(60\%)^2 = 0.28$

$0.282 = \frac{(2.836 \times 10^{-6})\ t_{60}}{(6)^2}$, $t_{60} = 3570680.7\ sec = 41.4$ days

$60 \times 60 \times 24$

Single H=d → OCR=1

Ex9: a strata normally consolidation clay of thickness 10ft is drained one side only it has a hydraulic conductivity of $K = 1.863\times10^{-8}$ in/sec and a coefficient of volume compressibility $mv = 8.6\times10^{-4}$ in²/lb Determine the ultimate value of the compression of the strata by assuming a uniform distributed load of 5.250 lb/ft² and also determine the time required for 20% and 80% → $\sigma_f - \sigma_o$

80% / 20% / Sc. : 0.1 0.6

1. $Sc = \frac{\Delta e}{1+e} H$ ∴ $mv = \frac{\Delta e}{1+e}\left(\frac{1}{\sigma_f - \sigma_o}\right)$ $mv = \frac{8.6\times10^{-4}\ in^2/lb}{(12)^2}$

$= 5.972\times10^{-6}\ ft^2$

$Sc = 0.031353 * 10 = 0.31$ ft

$5.972\times10^{-6} = \frac{\Delta e}{1+e}\left(\frac{1}{5.250}\right)$

$\frac{\Delta e}{1+e} = 0.031353$

2. T_{20} : $U = 20\%/60\%$ $Tv = \frac{\pi}{4}(20\%)^2 = 0.0314$

$Tv = \frac{Cv\, t_{20}}{d^2}$

$Cv = \frac{K}{mv\,\gamma_w} = \frac{1.55\times10^{-7}}{(5.972\times10^{-6})(62.4)} = 4.159\times10^{-4}$

$0.0314 = \frac{(4.159\times10^{-4})t_{20}}{(10)^2}$

$K = 1.86\times10^{-6}\ in/sec \times \frac{1}{12} = 1.55\times10^{-7}$ ft/sec

$t_{20} = 7549.89$ sec

$60*60*24 \rightarrow t_{20} = 0.0873$ days

Normally Consolidation → OCR=1

A small but heavily loaded utility building has dimensions of 20 m x 20 m. It applies a uniform load on its mat foundation of 100 kN/m^2. Its mat foundation sits 1 m below the surface. The soil profile consists of 3 m of a dry sand, with $\gamma = 17.5\ kN/m^3$ under laid by a 5 m thick clay layer with a $\gamma = 18.5\ kN/m^3$, a moisture content of 22%, $C_c = 0.30$ and a $G_s = 2.70$. The clay stratum is under laid by another sand stratum, and the phreatic surface coincides with the top of the clay stratum.

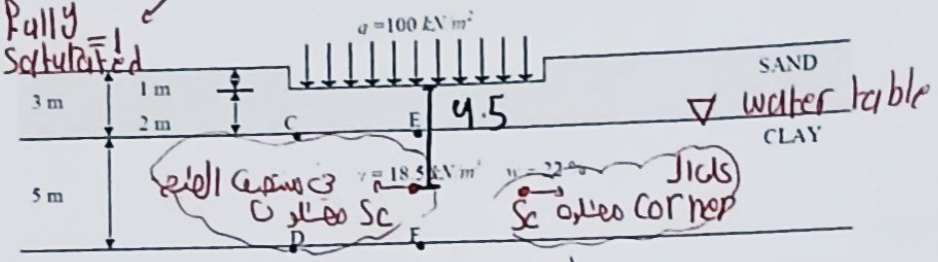
water table

What is the expected differential settlement between the building's center and one of its corners, in mm?

If a laboratory sample 4" thick of the field clay attained 50% consolidation in 5 hours, what time will the clay layer in the field attain 60% consolidation?

$S_c = \frac{\Delta e}{1+e_o} H$: $e_o S_r = w G_s \rightarrow e_o = w G_s = 0.22 * 2.70 = 0.594$

Fully Saturated = 1



$\rightarrow OCR = \frac{\sigma'_c}{\sigma'_o} = 1$

sand

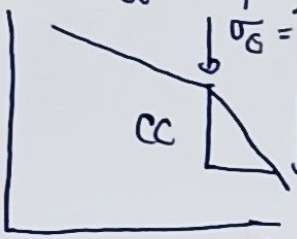
$\sigma'_o = \sigma_T - u = 98.75 - 24.515 = 74.235$

$\sigma_T = (18.5)(2.5) + (17.5)(3) = 98.75$

$u = \gamma_w h = 9.81 * 2.5 = 24.515$

$\sigma'_c = 74.235 + \Delta\sigma = 96.8 + 74.235$

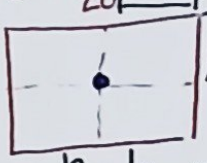
$\sigma_f \rightarrow = 171.035$



$\Delta e = 0.30(\log 171.035 - \log 74.235)$

$\Delta e = 0.1087$

$\Delta\sigma = (4Ir)q = b_o(4 * 0.242)$ = 96.8 (من طريقة الشكل)



20

$n = \frac{l_o}{4.5}$ $m = \frac{l_o}{4.5}$

The in-situ stress at mid-clay stratum before the building is built is.

$$p_o = \gamma_{sand} h_{sand} + \gamma_{clay} h_{at\ mid-clay} = (3m)(17.5) + (2.5m)(18.5 - 9.8) = 74.3 \frac{kN}{m^2}$$

Average stress in the clay stratum beneath the center $= \frac{(100 + 87.5)}{2} = 93.8 \frac{kN}{m^2}$

Average stress in the clay stratum beneath the corner $= \frac{(50 + 48)}{2} = 49 \frac{kN}{m^2}$

$S_c(center) = \frac{0.1087}{1+0.594} * 5 = 0.341\ m$

Corner

$m = \frac{20}{4.5}$ $n = \frac{20}{4.5}$

20

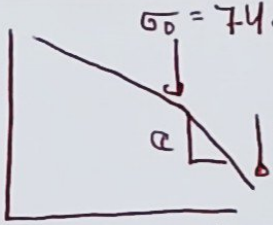
20

OCR=1 normally من السؤال

$\sigma'_o = \sigma_T - u = 74.235$

98.75 - 24.515

$\sigma'_o = 74.235$



(متساويان قيم كل نص المنحنى)

$\Delta e = 0.30(\log(99.235) - \log(74.235))$

$\Delta e = 0.0378$

$\Delta\sigma = qIr = 100(0.25) = 25$

$\sigma'_f = \sigma'_o + \Delta\sigma$

$= 74.235 + 25$

$= 99.235$

$S_c(corner) = \frac{0.0378}{1+0.594} * 5 = 0.118m$

$S_c(differential) = 0.341 - 0.118 = 0.223$

The differential settlement between the center and a corner of the building is.

$C_c = 0.30$ *and* $G_s = 2.70$

$$Se_o = wG_s \;\therefore\; e_o = \frac{wG_s}{S} = \frac{(0.22)(2.70)}{1} = 0.59$$

The in-situ stress at mid-clay stratum before the building is built is.

$$p_o = \gamma_{sand} h_{sand} + \gamma_{clay} h_{at\ mid-clay} = (3m)(17.5) + (2.5m)(18.5 - 9.8) = 74.3 \frac{kN}{m^2}$$

(لا تعتمد حل الملايدان)

Therefore,

the settlement at the center $\Delta H = \frac{C_c H}{1+e} \log\left(\frac{p_o + \Delta\sigma}{p_o}\right) = \frac{(0.30)(5m)}{1+(0.59)} \log\left(\frac{(74.3)+(93.8)}{(74.3)}\right) = 0.33m$

the settlement at the corner $\Delta H = \frac{C_c H}{1+e} \log\left(\frac{p_o + \Delta\sigma}{p_o}\right) = \frac{(0.30)(5m)}{1+(0.59)} \log\left(\frac{(74.3)+(49)}{(74.3)}\right) = 0.21m$

the differential settlement is $\Delta(\Delta H) = 0.33\,m - 0.21\,m = 0.12\ m = 120\ mm$

الفرع (2): معلوم t_{60}

$T_v = \frac{C_v t_{50}}{d^2}$: $U = 50\% \Delta 60\%$ $T_v = \frac{\pi}{4}(50\%)^2 = 0.196$

$t_{50} = 5$ hours

$H = 4in * 2.54 = 10.16$ (lab)

double drainage $d = H/2 = 10.16 = 5.08$

$0.196 = \frac{C_v(5)}{(5.08)^2}$

$\rightarrow$ $C_v = 1.0116 \approx 1.012$ $cm^2/hours$

$T_v = \frac{C_v t_{60}}{d^2}$: $U = 60 \angle 60\%$ $T_v = \frac{\pi}{4}(60\%)^2 = 0.2827$

Clay سماكة الطبقة: H

double $= \frac{H}{2} = \frac{5}{2} = 2.5 \times 100$ $H = 5m$

$= 250\ cm$

$\rightarrow 0.2827 = \frac{(1.012)\, t_{60}}{(250)^2}$

$t_{60} = 17459.24$ hours

The time required to attain 60% consolidation in the field is.

$$c_v = \frac{T_v H^2_{drained}}{t} = \frac{(0.26)(2.500mm)^2}{t_F} = \frac{(0.18)(2x25.4mm)^2}{(5\ hours)}$$

Solving for the time of settlement in the field t_f.

$$t_F = \frac{(0.26)(2.500mm)^2}{(0.18)(2x25.4mm)^2} \frac{(5\ hours)}{1} \left(\frac{1\,day}{24\,hours}\right)\left(\frac{1\,year}{365\,days}\right) \approx 2\ years$$

$\frac{t_{60}(17459.24)}{24*365} = 1.99$ year ≈ 2 year

Chapter 10: Soil shear Parameters

Type of shear strength parameters
- → total normal stress $S = T = C + \sigma \tan\phi$
- → effective normal stress $S = T = C' + \sigma' \tan\phi'$ ↓ $\sigma' = \sigma_{Total} - u$

* laboratory strength test (two common tester)
- → 1. Direct Shear test
- → 2. tricaxial test

1. Direct shear test:
- يتأثر shear force الموازية
- as a shear

2. tricaxial test:
- يتأثر normal force الموازية
- as a shear
  1. a confining stress is applied ← يتكون من الاتجاهين مرحلتين
  (الأولى العينة الاتجاهات جميع الاتجاهات)
  2. Second involves shearing the specimen

normally consolidated or sand or Gravel → C = 0

over consolidated or dense sand → C = قيمة

⇒ UU Quick Q test: (total stress) * الطريقة بالإجهاد
- يتم فتح صمام الماء (Drainage valve) يتم غلق صمام الماء

UU: unconsolidated - undrained) test
Q: Quick test
- → stage 1: Confinement stage ⇐ يتكون من مرحلتين
(يتم تطبيق الـ confining pressure على العينة)
- → stage 2: Shear stage
Increase the axial load until shear تحدث $\Delta\sigma$
Δu ← يحدث بسبب ان الماء لا يستطيع الخروج

UU Quick Q test: (total stress) اختبار بدون

UU: Un Consolidated - Un drained
Q: Quick test

stage 1: confinement stage	stage 2: shear stage
• يتم تطبيق σ_3 على العينة (confining Pressure)	• increase the axial load until shear → $\Delta\sigma$ تتغير • Δu → يحدث بسبب ان الماء لا يستطيع الخروج من العينة

• في هذا الاختبار (UU, Q test) لا يخرج الماء من العينة. Cap (Drainage Valve)

CU or R test: effecting total stress اختبار بدون → CU: Consolidated - Undrained

Stage 1: confinement stage	Stage 2: shear stage
• يتم تطبيق σ_3 على العينة confining Pressure • في هذه الحالة الماء يستطيع الخروج لان الـ (Drainage valve) مفتوح • يحدث consolidation (انضغاط) بسبب التي تتوقف عند النقطة Consolidated	• في هذه الحالة الماء لا يستطيع الخروج لان الـ Drainage Valve مغلق. • نبدأ بزيادة $\Delta\sigma$ التي يحدث عندها Shear • Δu يحدث بسبب ان الماء لا يستطيع الخروج.

Consolidate → close drainage, shear

CD or S test: effective اختبار بدون → CD: Consolidated - Drained

Stage 1: confinement	Stage 2: shear stage
• الماء يستطيع الخروج لان (Drainage Valve) مفتوح • يتم تطبيق σ_3 على العينة. • يحدث consolidation بسبب التي تتوقف عند النقطة Consolidated	• الماء يستطيع الخروج لان (Drainage Valve) مفتوح • نبدأ بزيادة الـ $\Delta\sigma$ لان الماء يستطيع الخروج $\Delta u = 0$ ← Total stress = Effective stress ← $\sigma = \sigma_{Total}$

* During Construction and End of Construction

- Free draining Soils
  - Drained shear strength (effective stress)
- Low Permeability Soils
  - Undrained shear strength (total strength)

* Steady - state seepage conditions

- Drained shear strength (effective stress)

* Sudden Drawdown Conditions

- Free draining Soils
  - Drained shear strength (effective stress)
- low Permeability Soils
  - three stage
    - stage 1: Drained shear strength (effective stress)
    - stage 2: undrained Shear consolidation Pressure from the first stage
    - stage 3: Drained shear strength (effective stress) or Undrained

(مهم جدًا) القوانين الخاصة بالمنحنى كلها

$\theta = 45 + \frac{\phi}{2}$ → normally consolidated / over consolidated — استخدامها في هذه الحالات

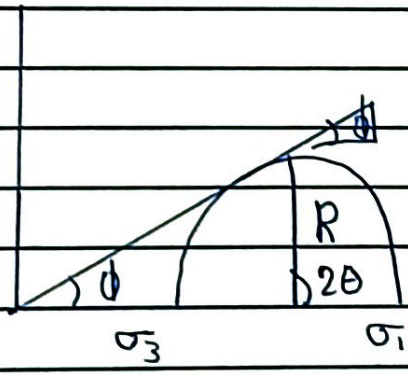
$TR = R \sin 2\theta$ → Shear stress (failure) استخدامها في حالة

$\sigma R = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta$ → normal (failure) stress استخدامها في حالة

$\phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$ → normally consolidated في حالة $C = 0$

$\sigma max = \frac{\sigma_1 + \sigma_3}{2}$ Shear max $= \frac{\sigma_1 - \sigma_3}{2}$

$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi}{2}\right)$ → over consolidated في حالة C and ϕ عندها



دائرة موهر $= \sigma_1 - \sigma_3$ نصف القطر $R = \frac{\sigma_1 - \sigma_3}{2}$

$C = 0$ normally consolidated إذا
$C =$ قيمة ما over consolidated

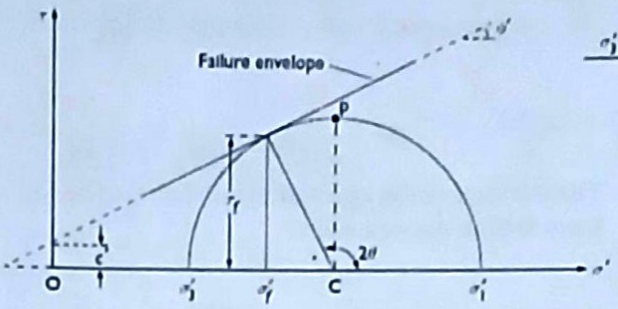
total ← undrained في حالة
effective ← drained في حالة

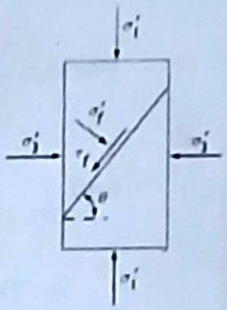
$\sigma' = \sigma - \Delta U$ فإن
eff total

Mohr–Coulomb failure criterion

$$\theta = 45^\circ + \frac{\phi'}{2}$$

$$\sin\phi' = \frac{\frac{1}{2}(\sigma'_1 - \sigma'_3)}{c'\cot\phi' + \frac{1}{2}(\sigma'_1 + \sigma'_3)}$$





$$\sigma'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3) + \frac{1}{2}(\sigma'_1 - \sigma'_3)\cos 2\theta$$

$$\tau_f = \frac{1}{2}(\sigma'_1 - \sigma'_3)\sin 2\theta$$

θ is the theoretical angle between the major principal plane and the plane of failure

$$\sigma'_1 = \sigma'_3 \tan^2\left(45^\circ + \frac{\phi'}{2}\right) + 2c'\tan\left(45^\circ + \frac{\phi'}{2}\right)$$

total = undrained في حالة ال
effective = drained في حالة ال

Example

C = 0

For a normally consolidated clay, the results of a drained triaxial test are as follows:

Chamber confining pressure = 16 lb/in.^{2} → σ_3 ← effective : $\sigma_1 = \sigma_3 + \Delta\sigma$

Deviator stress at failure = 25 lb/in.^{2} → $\Delta\sigma$

$= 16 + 25 = 41$

a. Find the angle of friction, ϕ'.

b. Determine the angle θ that the failure plane makes with the major principal plane.

a: لاحظ قانون

$$\phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$$

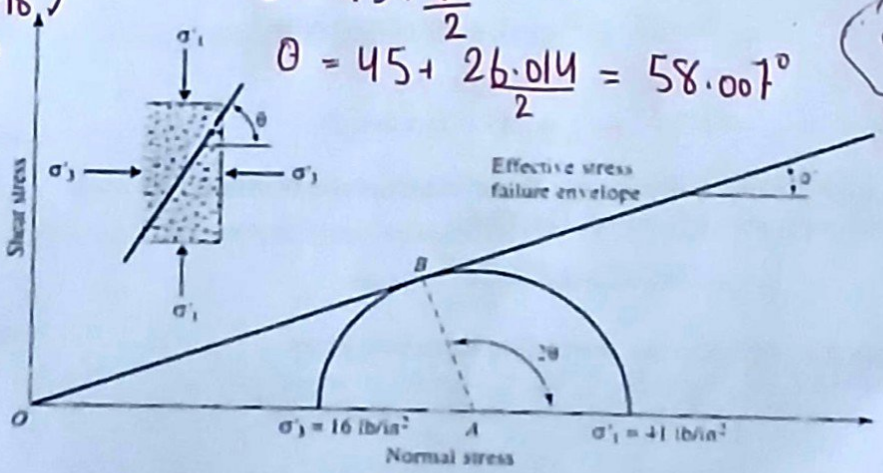
$$\sin^{-1}\left(\frac{41-16}{41+16}\right)$$

$\phi = 26.014^\circ$

$$\theta = 45 + \frac{\phi}{2} \rightarrow \text{a عكس}$$

$$\theta = 45 + \frac{26.014}{2} = 58.007^\circ$$

٥ لاتنسى حل البنود في جميع الـ Example



Solution

For a normally consolidated soil, the failure envelope equation is

$$\tau_f = \sigma' \tan \phi' \qquad (\text{since } c' = 0)$$

For the triaxial test, the effective major and minor principal stresses at failure are

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 16 + 25 = 41 \text{ lb/in.}^2$$

and

$$\sigma_3' = \sigma_3 = 16 \text{ lb/in.}^2$$

a. The Mohr's circle and the failure envelope are shown in Figure 11.20, from which we can write

$$\sin \phi' = \frac{AB}{OA} = \frac{\left(\frac{\sigma_1' - \sigma_3'}{2}\right)}{\left(\frac{\sigma_1' + \sigma_3'}{2}\right)}$$

or

$$\sin \phi' = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'} = \frac{41 - 16}{41 + 16} = 0.438$$

$$\phi' = 26°$$

b. $\theta = 45 + \frac{\phi'}{2} = 45° + \frac{26}{2} = 58°$

$\sigma = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\theta$ $\quad \sigma = 23.176$

$\sigma = R \sin 2\theta \quad ; \quad R = \frac{\sigma_1-\sigma_3}{2} = \frac{41-16}{2} = 12.5$

$= 12.5 \sin 2(58.007)$

$= 11.23$

Refer to Example 11.2.

a. Find the normal stress σ' and the shear stress τ_f on the failure plane.

b. Determine the effective normal stress on the plane of maximum shear stress.

$\sigma_{max} = \frac{\sigma_1+\sigma_3}{2}$ $\quad$ Shear $_{max} = \frac{\sigma_1-\sigma_3}{2}$

Solution

a. From Eqs. (9.8) and (9.9),

$$\sigma' \text{ (on the failure plane)} = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

and

$$\tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$$

Subsituting the values of $\sigma_1' = 41 \text{ lb/in.}^2$, $\sigma_3' = 16 \text{ lb/in.}^2$, and $\theta = 58°$ into the preceding equations, we get

$$\sigma' = \frac{41 + 16}{2} + \frac{41 - 16}{2} \cos(2 \times 58) = \mathbf{23.0 \text{ lb/in.}^2}$$

and

$$\tau_f = \frac{41 - 16}{2} \sin(2 \times 58) = \mathbf{11.2 \text{ lb/in.}^2}$$

b. From Eq. (9.9), it can be seen that the maximum shear stress will occur on the plane with $\theta = 45°$. From Eq. (9.8),

$$\sigma' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

Substituting $\theta = 45°$ into the preceding equation gives

$$\sigma' = \frac{41 + 16}{2} + \frac{41 - 16}{2} \cos 90 = \mathbf{28.5 \text{ lb/in.}^2}$$

Example

effective

The equation of the effective stress failure envelope for normally consolidated clayey soil is $\tau_f = \sigma' \tan 25°$. A drained triaxial test was conducted with the same soil at a chamber confining pressure of 80 kN/m². Calculate the deviator stress at failure.

C=0

σ_3

$\Delta\sigma$

$\rightarrow \sigma_1 = \Delta\sigma + \sigma_3$

Solution

For normally consolidated clay, $c' = 0$. Thus, from Eq. (11.8),

$T = c + \sigma' \tan\phi$

$T = \sigma' \tan 25$

$\phi = 25$

so

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

$$\phi' = 25°$$

$$\sigma_1' = 80 \tan^2\left(45 + \frac{25}{2}\right) = 197 \text{ kN/m}^2$$

$$(\Delta\sigma_d)_f = \sigma_1' - \sigma_3' = 197 - 80 = 117 \text{ kN/m}^2$$

$\rightarrow \phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$

$25 = \sin^{-1}\left(\frac{\sigma_1 - 80}{\sigma_1 + 80}\right) \rightarrow \sin 25 = \frac{\sigma_1 - 80}{\sigma_1 + 80}$

$\rightarrow \sigma_1 = 197.24 \text{ kN/m}^2$

$\Rightarrow \sigma_1 = \Delta\sigma + \sigma_3$

$197.24 = \Delta\sigma + 80$

$\rightarrow \Delta\sigma = 117.24 \text{ kN/m}^2$

Example

effective

The results of two drained triaxial tests on a saturated clay are as follows:

normally اس كلاي ممكن يكون اذا نوع over consolidated

Specimen I: $\sigma_3 = \sigma_3' = 70 \text{ kN/m}^2$, $(\Delta\sigma_d)_f = 130 \text{ kN/m}^2$

Specimen II: $\sigma_3 = \sigma_3' = 160 \text{ kN/m}^2$, $(\Delta\sigma_d)_f = 223.5 \text{ kN/m}^2$

$\sigma_1' = \sigma_3' + \Delta\sigma'$

$\sigma_1' = 70 + 130 = 200$

$\sigma_1' = \sigma_3' + \Delta\sigma' = 160 + 223.5 = 383.5$

Determine the shear strength parameters c' and ϕ'. $\rightarrow \sigma_1' = \sigma_3' \tan^2(45 + \frac{\phi}{2}) + 2c' \tan(45 +$

Solution

For specimen I, the principal stresses at failure are

$$\sigma_3' = \sigma_3 = 70 \text{ kN/m}^2$$

and

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 70 + 130 = 200 \text{ kN/m}^2$$

Similarly, the principal stresses at failure for specimen II are

$$\sigma_3' = \sigma_3 = 160 \text{ kN/m}^2$$

and

$$\sigma_1' = \sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 160 + 223.5 = 383.5 \text{ kN/m}^2$$

Using the relationship given by Eq. (11.8), we get

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

Thus, for specimen I,

$$200 = 70 \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

and for specimen II,

$$383.5 = 160 \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

Solving the two preceding equations, we obtain

$$\phi' = 20° \qquad c' = 20 \text{ kN/m}^2$$

Example

A consolidated-undrained test on a normally consolidated clay yielded the following results:

$\sigma_3 = 12$ lb/in.^{2}

Deviator stress, $(\Delta\sigma_d)_f = 9.1$ lb/in.^{2}

Pore pressure, $(\Delta u_d)_f = 6.8$ lb/in.^{2}

Calculate the consolidated-undrained friction angle and the consolidated-drained friction angle.

Solution

$$\sigma_3 = 12 \text{ lb/in.}^2$$

$$\sigma_1 = \sigma_3 + (\Delta\sigma_d)_f = 12 + 9.1 = 21.1 \text{ lb/in.}^2$$

From Eq. (11.9), for normally consolidated clay with $c = 0$,

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

$$21.1 = 12 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\phi = 2\left[\tan^{-1}\left(\frac{21.1}{12}\right)^{0.5} - 45\right] = 16°$$

Solution
From Figure 11.31,

$$\sin\phi' = \frac{AB}{AC} = \frac{AB}{CO + OA} = \frac{\left(\frac{\sigma_1' - \sigma_3'}{2}\right)}{c'\cot\phi' + \left(\frac{\sigma_1' + \sigma_3'}{2}\right)}$$

so

$$\frac{\sigma_1' - \sigma_3'}{2} = c'\cos\phi' + \left(\frac{\sigma_1' + \sigma_3'}{2}\right)\sin\phi' \qquad \text{(a)}$$

or

$$q' = m + p'\tan\alpha \qquad \text{(b)}$$

Comparing Eqs. (a) and (b), we find that

$$m = c'\cos\phi'$$

and

$$\tan\alpha = \sin\phi'$$

or

$$\alpha = \tan^{-1}(\sin\phi')$$

$$\boxed{\phi' = \sin^{-1}(\tan\alpha')}$$

$$\boxed{c' = \frac{a'}{\cos\phi'}}$$

$\sigma_1 = \sigma_3 + \Delta\sigma = 3150 + 2100$

$\sigma_3' = \sigma_3 - u = 3150 - 1848 =$ $\qquad \sigma_1' = \sigma_1^{total} - u = 2100 - 1848$

σ_3 total → $C = 0$

A normally consolidated clay was consolidated under a stress of 3150 lb/ft², then sheared undrained in axial compression. The principal stress difference at failure was 2100 lb/ft², and the induced pore pressure at failure was 1848 lb/ft². Determine (a) the Mohr-Coulomb strength parameters, in terms of both total and effective stresses analytically, (b) compute $(\sigma_1/\sigma_3)_f$ and $(\sigma_1'/\sigma_3')_f$, and (c) determine the theoretical angle of the failure plane in the specimen.

σ_3 — $\Delta\sigma$ — θ — effective

Solution

The parameters required are: effective parameters c' and ϕ', and total parameters c and ϕ.

(a) Given $\sigma_{3f} = 3150$ lb/ft², and $(\sigma_1 - \sigma_3)_f = 2100$ lb/ft². The total principal stress at failure σ_{1f} is obtained from

$$\sigma_{1f} = (\sigma_1 - \sigma_3)_f + \sigma_{3f} = 2100 + 3150 = 5250 \text{ lb/ft}^2$$

Effective $\sigma'_{1f} = \sigma_{1f} - u_f = 5250 - 1848 = 3402$ lb/ft²

$\sigma'_{3f} = \sigma_{3f} - u_f = 3150 - 1848 = 1302$ lb/ft²

Now $\sigma_1 = \sigma_3 \tan^2(45° + \phi/2) + 2c\tan(45° + \phi/2)$

Since the soil is normally consolidated, $c = 0$. As such

$$\frac{\sigma_1}{\sigma_3} = \tan^2(45° + \phi/2) = \frac{1 + \sin\phi}{1 - \sin\phi}, \quad \text{or} \quad \sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

Total $\phi = \sin^{-1}\dfrac{2100}{5250 + 3150} = \sin^{-1}\dfrac{2100}{8400} = 14.5°$

Effective $\phi' = \sin^{-1}\dfrac{2100}{3402 + 1302} = \sin^{-1}\dfrac{2100}{4704} = 26.5°$

a) $C = 0$ $\qquad \phi = \sin^{-1}\left(\dfrac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$

$\phi = \sin^{-1}\left(\dfrac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}\right)$

b) $\dfrac{\sigma_1}{\sigma_3} = \dfrac{3150}{}$ $\qquad \dfrac{\sigma_1'}{\sigma_3'} =$

(b) The stress ratios at failure are

$$\frac{\sigma_1}{\sigma_3} = \frac{5250}{3150} = 1.67, \quad \frac{\sigma_1'}{\sigma_3'} = \frac{3402}{1302} = 2.61$$

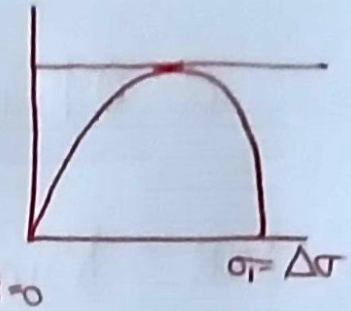
c) $\theta = 45 + \dfrac{\phi}{2}$

(c) From Eq. (8.18)

$$\alpha_f = 45° + \frac{\phi'}{2} = 45° + \frac{26.5}{2} = 58.25°$$

normally الكثير من الـ
اذا الـ

unconfined/qu



A soil has an unconfined compressive strength of 120 kN/m². In a triaxial compression test a specimen of the same soil when subjected to a chamber pressure of 40 kN/m² failed at an additional stress of 160 kN/m². Determine:

(i) The shear strength parameters of the soil, (ii) the angle made by the failure plane with the axial stress in the triaxial test.

C and ϕ

$\sigma_1 = \sigma_3 \tan^2$

θ

Solution

لا تعتمد على البلاديات

There is one unconfined compression test result and one triaxial compression test result. Hence two Mohr circles, C_1, and C_2 may be drawn as shown in Fig. Ex. 8.11. For Mohr circle C_1, $\sigma_3 = 0$ and $\sigma_1 = 120$ kN/m², and for Mohr circle C_2, $\sigma_3 = 40$ kN/m² and $\sigma_1 = (40 + 160) = 200$ kN/m². A common tangent to these two circles is the Mohr envelope which gives

(i) $c = 43$ kN/m² and $\phi = 19°$

(ii) For the triaxial test specimen, A is the point of tangency for Mohr circle C_2 and C is the center of circle C_2. The angle made by AC with the abscissa is equal to twice the angle between the failure plane and the axis of the sample = 2θ. From Fig. Ex. 8.11, $2\theta = 71°$ and $\theta = 35.5°$. The angle made by the failure plane with the σ-axis is $\alpha = 90° - 35.5° = 54.5°$.

٥٣

Boreholes reveal that a thin layer of alluvial silt exists at a depth of 50 ft below the surface of the