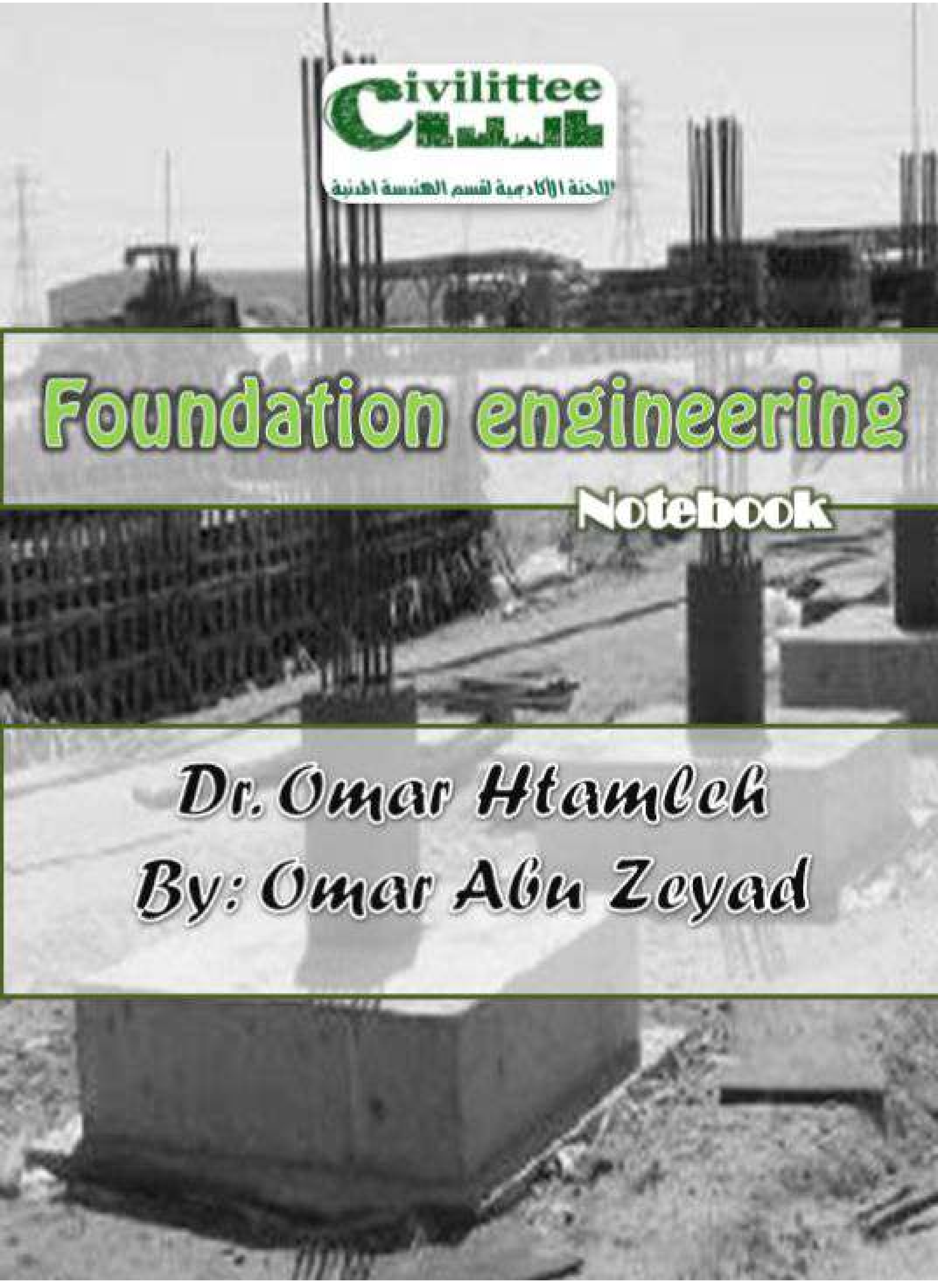

Civilittee

اللجنة الأكاديمية لقسم الهندسة المدنية

Foundation engineering

Notebook

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By: Omar Abu Zeyad

Foundation

هندسة أساسات

إعداد الطالب : عمر زياد أبوزياد

* هذه الدوسية لا تغني عن شرح الدكتور وانما هي وسيلة مساعدة *

Civilittee

اللجنة الأكاديمية لقسم الهندسة المدنية

المحتوى

* الفيرست والسكند والفاينل :

- شرح المادة مع تفصيل الأفكار المهمة .
- الجداول والرسمات المطلوبة .
- حلول أمثلة الكتاب المقترحة .
- امتحان الفصل مع حله .

*First: Ch.4
- Terzaghi's Bearing Capacity.
- General Bearing Capacity.
- Eccentricity Loaded Footing.
- Bearing Capacity For Two Layers.

* Second: ch.6.7,12
- stress Increment.
- settelement
- Lateral Earth Pressure.

*Final: Ch.13
- Tensile Crack
- Retaining Wall
- Piles.

** تذكروا ان هذا الحل هو جهد بشري ، فإن اصبت فمن الله ، وان اخطأت فمن نفسي
في حال وجود اي خطأ او ملاحظة يُرجى التواصل معي .

* هذا الحل اهداء الى دفعة 2014

مع تمنياتي للجميع بالتوفيق
لا تنسوني ووالدي من صالح الدعاء
... أخوكم عمر

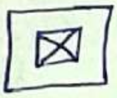
- Footing : Structure element "member" Carry the Loads from Column to underground Soil.

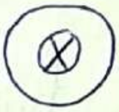
- Types of footing :

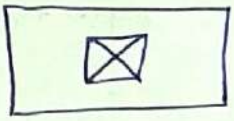
<1> Shallow footings

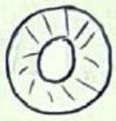
<2> Deep footings → piles * "خابال"

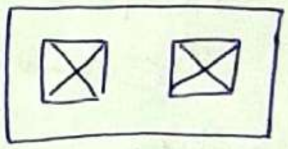
- Footing shape :

1- Squar 

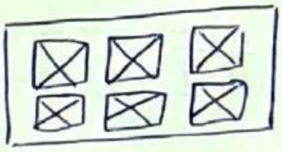
2- Circular 

3- Rectangular 

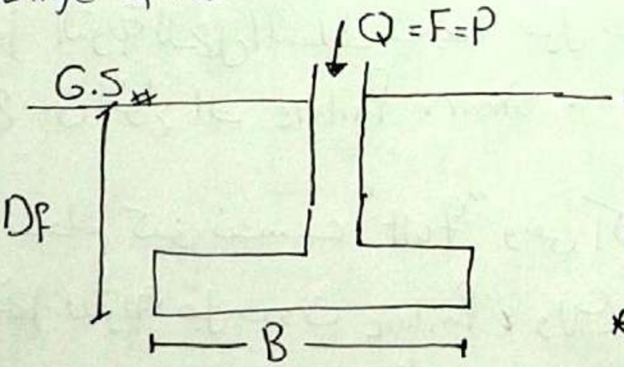
4- Ring footing 

5- Combined : Carry max 3 column (general 2) 

6- Mat (Raft) :
- weak soil.
- Large Load.
- Large Space.



7- strip, continuous, wall

- 

* B : width of footing "must be the shortest dimension on the plan"
* Df : Depth of footing.
* Q : Load.

- Type of failure : "The footing must be safe against" :

<1> Shear failure in soil

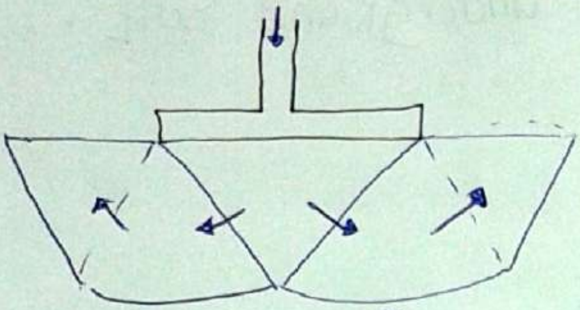
<2> excessive vertical displacement "settlement" }

<3> excessive lateral movement.

*سوف يتم دراسة هذه ، المواضيع بالتفصيل في المادة.

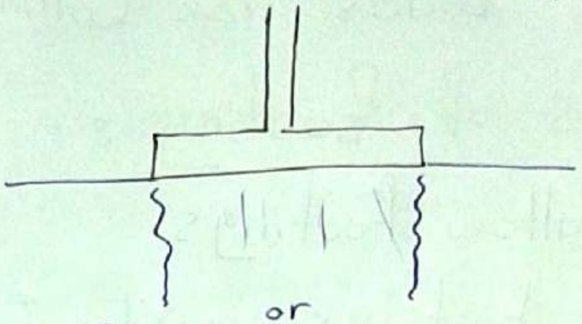
• Type of Shear failure :

1. General shear failure



- Stiff clay or dense sand
- Shallow Depth

2. punching Shear failure



- Soft clay or Loose sand
- great depth

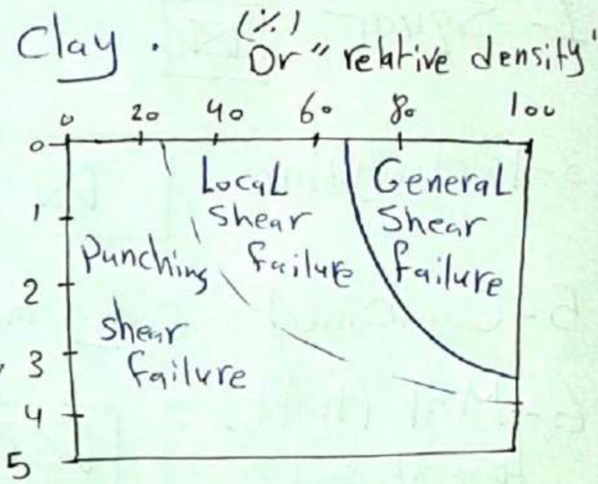
3. Local shear failure : medium clay.

** نستطيع من خلال (B, L, Dr) معرفة نوع الـ failure وهو مهم جداً في الحل. "فكرة سنوات"

* B^* for square = B

$$\frac{D_f}{B^*}$$

$$\boxed{B^* = \frac{2BL}{B+L}}$$



• Bearing Capacity :

* قدرة تحمل التربة للحمل المسلط عليها قبل حدوث اي نوع من انواع الـ shear failure .

$$\boxed{F.S = \frac{q_{ult}}{q_{all}}}$$

$$\boxed{Q_{all} = (q_{all})(Area)}$$

* سوف نتعلم كيف نحسب "qult" وهي اكبر قدرة تحمل للتربة قبل حدوث failure ، ولكن التصميم لا يكون عليها اذ نأخذ معامل أمان وهو "F.S" تحسبا لاي ظرف يمكن ان يحدث ومن خلاله نقوم بحساب قدرة التحمل المسموح بها وهي "qall" وهي التي يتم التصميم والمقارنة بها .

● "qult" طرق حساب CH 4 :

1 Terzagh Bearing Capacity equ. : *يذكر في السؤال استعماله .

Ⓐ $q_{ult} = CN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$ → for wall/Strip/Contin. footing

Ⓑ $q_{ult} = 1.3CN_c + qN_q + 0.4\gamma B N_\gamma$ → for squar footing.

Ⓒ $q_{ult} = 1.3CN_c + qN_q + 0.3\gamma B N_\gamma$ → for Circule footing.

where :

q_{ult} : ultimate bearing Capacity . C : Cohesion "for sand = 0"

q : stress at the bottom ~~&~~ of footing "$D_f * \gamma$"

γ : unit of weight of soil underneath of footing - الطبقة اسفل الاساس التي تتحمل اللود .

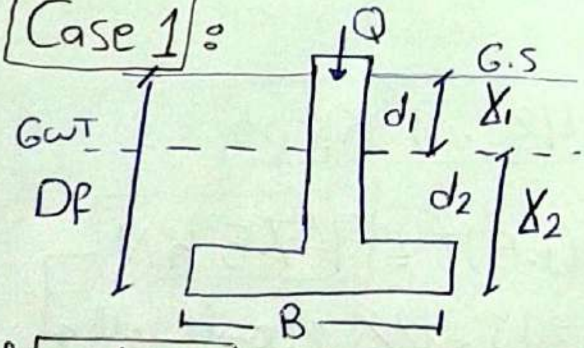
B : width of footing .

N_q, N_γ, N_c : Bearing Capacity ~~coff~~ factor from table .

⊛ Modification for GWT. : عند وجود مستوى مياه في الطبقات يحدث تغيرات على القانون :

$$q_{ult} = \underbrace{CN_c}_{Term(I)} + \underbrace{qN_q}_{Term(II)} + \underbrace{0.5\gamma B \cancel{\gamma} N_\gamma}_{Term(III)}$$

Case 1 :



if $0 \leq d_1 \leq D$

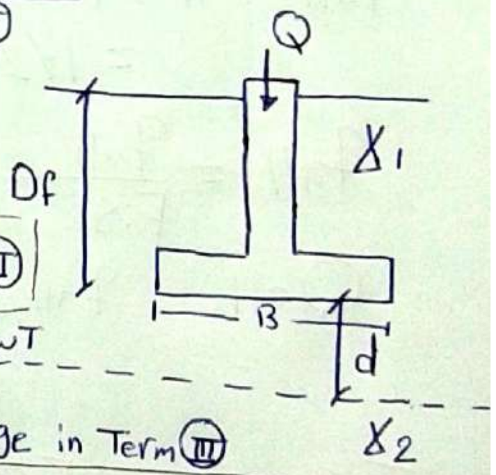
$q = \gamma_1 d_1 + \gamma'_2 d_2$ .... Term (II)

$\gamma'_2 = \gamma_2 - \gamma_w$ used in Term (III)

Case 2 :

$0 \leq d \leq B$

$q = \gamma_1 D_f$ ... Term (I)



use $\bar{\gamma}$ average in Term (III)

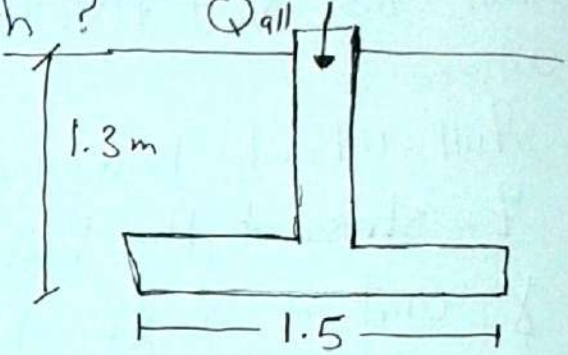
$$\bar{\gamma} = \gamma'_2 + (\gamma_1 - \gamma'_2) d/B$$

Case 3 : as in Case (II) but $d > B$. GWT has no effect .

"لفترة سنوات"

Ex: A square foundation 1.5m x 1.5 m in plane. The supporting soil has a friction angle Ø = 20°, and effective cohesion, C` = 65 Kpa. The unit weight of soil is 19 KN/m³, Determain allawable gross Load on foundation if the fator of safty = 4, and general shear failure occure and ground water table at great depth?

Sol:



~~* خطوات الحل :~~

~~1- حدد [illegible] failure~~

~~2- اوجد q و Ø ثم qult من العلاقات المناسب لشكل القاعدة المعطى~~

$q_{ult} \overset{1.3}{=} CN_c + qN_q + 0.4\gamma B \cancel{\gamma} N_\gamma$

* $q = D_f \gamma = (19)(1.3) = \boxed{24.7 \text{ Kpa}}$ GWT لا يوجد تأثير لمستوى المياه **

* $\varnothing = 20° \Rightarrow N_c = 17.69$, $N_q = 7.44$, $N_\gamma = 3.64$

$q_{ult} = (1.3)(65)(17.69) + (24.7)(7.44) + (0.4)(19)(1.5)(3.64)$
$= 1720.069$

$q_{all} = \frac{q_{ult}}{F.S} = \frac{1720.069}{4} = \boxed{430.01 \text{ Kpa}}$

$Q_{all} = q_{all} * \text{Area} = (430.01)(1.5)^2 = \boxed{967.5 \text{ KN}}$

← اللود المسموح ~~نطابقه~~ تطبيقه على الاساس .

Ex2: Same Ex1, the differance is: square found. B X B
Find B :
$Q_{all} = 1000$ KN
$D_f = 1$ m

Sol: $q = D_f \gamma = (19)(1) = \boxed{19 \text{ Kpa}}$

$Q_{ult} = (Q_{all})(F.S) = (1000)(4) = \boxed{4000 \text{ KN}}$

$q_{ult} = \frac{Q_{ult}}{\text{Area}} = 1.3CN_c + qN_q + 0.4\gamma BN_\gamma$

$\boxed{B = 1.54 \text{ m}}$

$\Rightarrow \frac{4000}{B^2} = (1.3)(65)(17.69) + (19)(7.44) + 0.4(19)B(3.64)$

● Note : if soils fails either in Local or punching shear failur mode use :

$$\boxed{\begin{aligned} C^{*(2)} &= 2/3\, C^{(1)} \\ \phi^{(2)} &= \tan^{-1}(2/3 \tan \phi^{(1)}) \end{aligned}}$$

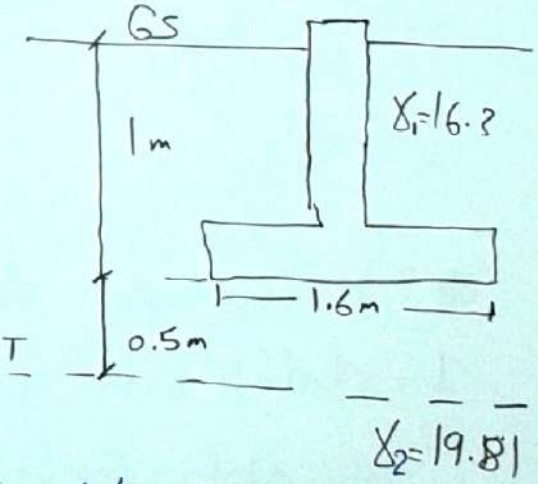
reduction ⇐ مهم سؤال سنوات*

⇐ دائمًا في السؤال لو لم يذكر النوع بكون general .

⇐ ممكن يعطيك (Dr%) * مع الرسمة ، انت بتحسب ال β^* و بتشوف النوع اذا كان Local, punch. بتحل reduction قبل ما تبلش حل (سنوات) .

Ex: a Circular footing of 1.6 m diameter founded on a Soil has a unit weight of 19.81 kN/m^2, below water table and $\gamma = 16.3\ kN/m^3$ above water table, The soil has $C = 50$ Kpa & $\phi' = 25°$, the water table is 0.5 m below the footing, F.S = 3, Soil will fail in Local Shear failure, find gross allowable Load ?

$$q_{ult} = 1.3\, C N_c + q N_q + 0.3\, \gamma B N_\gamma$$



** Local shear failure : reduction

$$C^{(2)} = 2/3\, C^{(1)} = 2/3(50) = \boxed{\frac{100}{3}\ Kpa}$$

$$\phi^{(2)} = \tan^{-1}(2/3 \tan(25)) = 17.26 \simeq \boxed{17°}$$

$\Longrightarrow$ from table : $N_c = 14.6$, $N_q = 5.45$, $N_\gamma = 2.18$

* GWT : Case Ⅱ $0 < d \leq B$

$$\bar{\gamma}_{avg} = \gamma_2' + (\gamma_1 - \gamma_2')\, d/B = (19.81 - 9.81) + (16.3 - (19.81 - 9.81)) * \frac{0.5}{1.6}$$

$$= \boxed{11.96\ KN/m^3}$$

$$q = \gamma D_f = 16.3(1) = \boxed{16.3\ Kpa}$$

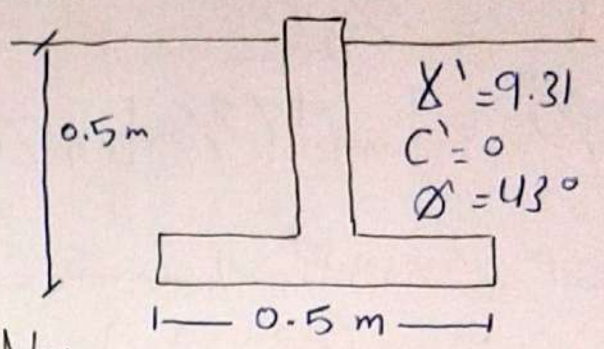
$$q_{ult} = (1.3)(\tfrac{100}{3})(14.6) + (16.3)(5.45) + 0.3(11.96)(1.6)(2.18) = \boxed{734.01\ Kpa}$$

$$Q_{all} = q_{all}\,(Area) = \left(\frac{734.01}{3}\right)^{\frac{q_{ult}}{F.S}} * \frac{\pi}{4}(1.6)^2 = \boxed{491.93\ KN}$$

Ex : Compare Terzaghi bearing capacity for wall footing Versus a measured field test that result $P_{ult} = 1863$ KN?

** مساحة ال Strip, wall, cont. footing $L >> B$

$\boxed{Area = B * 1}$



$$q_{ult} = \overset{0}{\cancel{CN_c}} + qN_q + 0.5\gamma B N_\gamma$$

$$q = D_f \gamma = (0.5)(9.31) = \boxed{4.655 \text{ Kpa}}$$

$$\phi = 43° \rightarrow N_q = 126.5 \quad , \quad N_\gamma = 211.56$$

$$q_{ult} = 0 + (4.655)(126.5) + 0.5(9.3)(0.5)(211.56)$$
$$= \boxed{1081.26 \text{ Kpa}}$$

$$Q_{ult} = q_{ult} * Area = 1081.26 * 0.5 = \boxed{540.63 \text{ KN}}$$

Terzaghi مقارنه ب 1863 جدآ قليلة فنلاحظ بان
محافظ جدآ .

- **Shortcomings of Terzaghi equation :-** عيوب معادلة Terz.

1- didn't take general shape : Like rectangular
⇐ فقط هناك ٣ اشكال محددة عنده ، المستطيل ما بقدر احله عليه

2- didn't take depth into Consideration

3- ~ ~ Load inclination " .

4- ~ ~ base or ground ~ ~ .

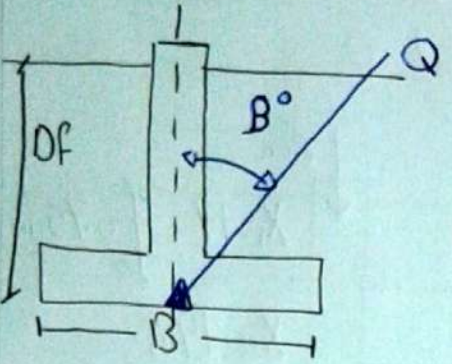
5- ~ ~ eccentricity into Consideration

* لذلك ننتقل للنوع الثاني .

* ~~يطلب بالسؤال~~

2 General Bearing Capacity :

$$q_{ult} = C N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + 0.5 \gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} B$$



where : q_{ult} : Ultimate Bearing Capacity

C : Cohesion , B : width

q : stress at the bottom of footing

γ : unit weight of soil underneath the footing

N_c, N_q, N_γ : General Bearing Capacity. *الحل جدول لطال

* Shape factor ,

$$F_{cs} = 1 + B/L * \frac{N_q}{N_c}$$

$$F_{qs} = 1 + B/L \tan \phi$$

$$F_{\gamma s} = 1 - 0.4 \ B/L$$

* Depth factor :

- if $\frac{Df}{B} \leq 1$:

$$F_{cd} = 1 + 0.4 \frac{Df}{B}$$

$$F_{qd} = 1 + 2 \tan \phi (1 - \sin \phi)^2 \frac{Df}{B}$$

$$F_{\gamma d} = 1$$

- if $\frac{Df}{B} > 1$:

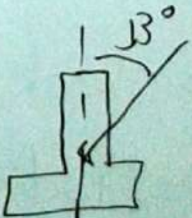
انتبه بال

$$F_{cd} = 1 + 0.4 \boxed{\tan^{-1}\left(\frac{Df}{B}\right)} \Rightarrow rad$$

$$F_{qd} = 1 + 2 \tan \phi (1 - \sin \phi)^2 \boxed{\tan^{-1}\left(\frac{Df}{B}\right)}$$

$$F_{\gamma d} = 1$$

* Load Inclination factor :

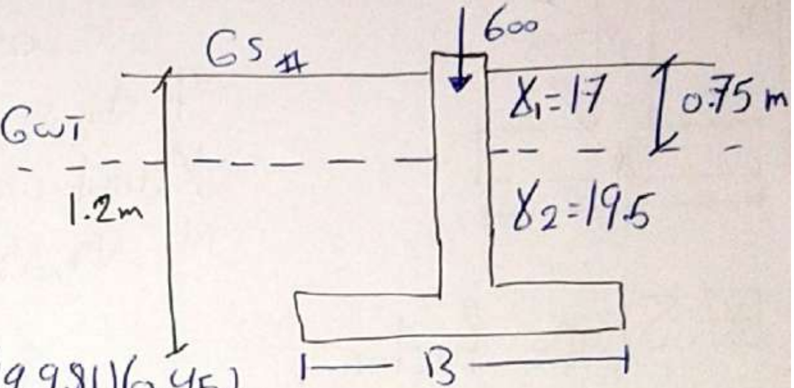


$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90}\right)^2$$

$$F_{\gamma i} = \left(1 - \frac{\beta^\circ}{\phi^\circ}\right)^2 \Rightarrow \beta^\circ < \phi^\circ$$

Ex: A square footing B×B has to be constracted as shown
Assume that: $\gamma_1 = 17\ KN/m^3$, $\gamma_2 = 19.5\ KN/m^3$, $D_1 = 0.75m$
$D_f = 1.2m$, The gross design allowable Load
$Q_{all} = 600\ KN$, $F.S = 3$, find B:
$C = 0$, $\emptyset = 36°$

Sol: from general table



$\emptyset = 36° \rightarrow N_q = 37.75$
$\rightarrow N_\gamma = 56.31$
$\rightarrow$

$$q = \gamma_1 D_1 + \gamma_2' D_2 = (17)(0.75) + (19-9.81)(0.45)$$
$$= \boxed{17.11\ Kpa}$$

* Shape factor:
$$F_{qs} = 1 + \frac{B}{B}\tan(36) = \boxed{1.73}$$
$$F_{\gamma s} = 1 - 0.4\ B/B = \boxed{0.6}$$

*Depth factor: assume $\frac{D_f}{B} \leq 1$:
$$F_{qd} = 1 + 2\tan(36)(1-\sin 36)^2\ 1.2/B = \boxed{1 + \frac{0.29}{B}}$$
$$F_{\gamma d} = 1.0$$

* Load is Vertical so $F_{qi} = F_{\gamma i} = 1.0$

$$Q_{ult} = Q_{all}(F.S) = 600 * 3 = 1800\ KN$$

$\Rightarrow q_{ult} = Q_{ult}/Area$

$$\Rightarrow \frac{1800}{B^2} = 0 + (17.11)(37.75)(1.73)\left(1 + \frac{0.29}{B}\right)(1) + 0.5(9.69)(B)(56.31)(0.6)(1)(1)$$
$$\frac{1800}{B^2} = 1117.4 + \frac{324.04}{B} + 163.69B \Longrightarrow \boxed{B = 1.06\ m}$$

$\Longrightarrow$ Check assumption: $\frac{D_f}{B} \leq 1$?? $\frac{1.2}{1.06} = 1.13 > 1$ ✗

$\Rightarrow$ ~~$F_{qd} = 1 + 2\tan(36)(1 - \sin\emptyset)^2 \tan^{-1}\left(\frac{1.2}{B}\right)$~~

$$\Rightarrow \frac{D_f}{B} > 1$$

$$F_{qd} = 1 + 2\tan(36)(1-\sin(36))^2 \tan^{-1}(1.2/B)$$

$$\Rightarrow \underbrace{\frac{1800}{B^2}}_{LHS} = \underbrace{1117.4 + 163.09B + 275.9\tan^{-1}\left(\frac{1.2}{B}\right)}_{R.H.S}$$

⇒ Use Trial & error

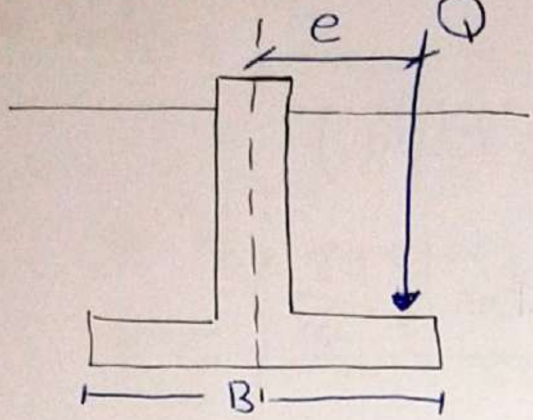
assume :

B(m)	L.H.S	R.H.S	Δ	
1.08	1666.67	1524.62	142.05	Δ_1
1.12	1607.14	1526.15	80.99	Δ_2
~~1.06~~		~~1529.92~~		Δ_3

$$\left|\frac{\Delta_3 - \Delta_1}{\Delta_1}\right| \times 100\% < 2\% \quad ??$$

● Bearing Capacity of Soils on Eccentrically Loaded footing :

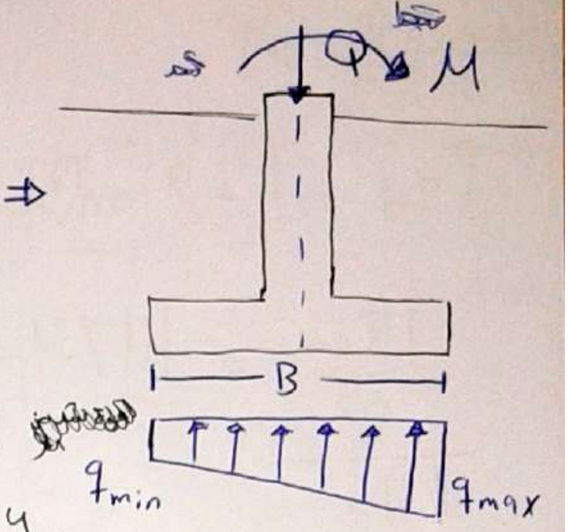
** في المواضيع السابقة كان ال Load في مركز ال footing ، عندما تحصل ازاحة لل Load بمقدار "e" عن مركز الاساس ، سوف يكون جهد اخر الا وهو bending moment = $\frac{My}{I}$ بالاضافة لل axial = P/A وسيغير ذلك من حسابات ال q_{ult} ، q_{all} .



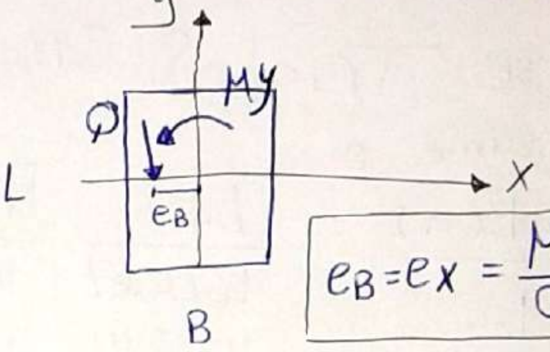
$$\boxed{e = \frac{M}{Q}}$$

$$\boxed{\sigma = P/A + \frac{My}{I}}$$

$\Rightarrow$



▨ Cases of Eccentricity :



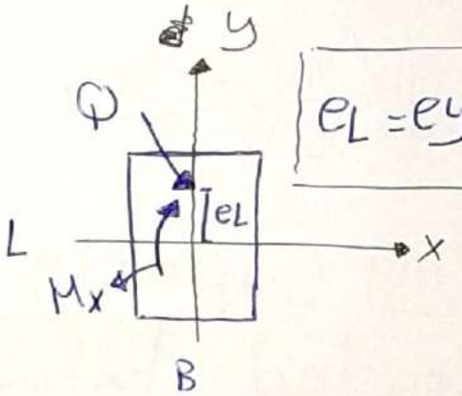
$$\boxed{e_B = e_x = \frac{M_y}{Q}}$$

1 In B - direction :

$$q_{max} = \frac{Q}{BL} + \frac{6M}{B^2L} = \boxed{\frac{Q}{BL}\left(1 + \frac{6e_B}{B}\right)}$$

$$q_{min} = \boxed{\frac{Q}{BL}\left(1 - \frac{6e_B}{B}\right)}$$

* assume $q_{min} = 0 \Rightarrow \boxed{e_B = B/6}$



$$\boxed{e_L = e_y = \frac{M_x}{Q}}$$

2 In L - direction :

$$q_{max} = \boxed{\frac{Q}{BL}\left(1 + \frac{6e_L}{L}\right)}$$

$$q_{min} = \boxed{\frac{Q}{BL}\left(1 - \frac{6e_L}{L}\right)}$$

* assume $q_{min} = 0 \Rightarrow \boxed{e_L = \frac{L}{6}}$

3 In case of 2 way eccentricity :

$$q_{max} = \boxed{\frac{Q}{BL}\left(1 + \frac{6e_B}{B} + \frac{6e_L}{L}\right)}$$

$$q_{min} = \boxed{\frac{Q}{BL}\left(1 - \frac{6e_B}{B} - \frac{6e_L}{L}\right)}$$

* assume $q_{min} = 0 \Longrightarrow \boxed{\frac{6e_B}{B} + \frac{6e_L}{L} = 1}$

● Note : $\boxed{q_{min} \text{ must be } \geq 0}$

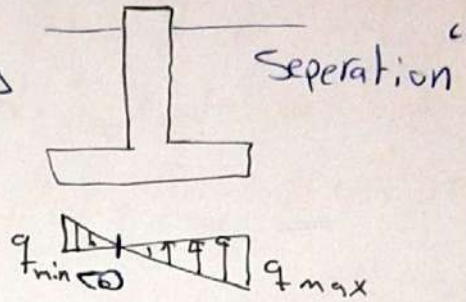
** ⇐ حتى لا يحدث Seperation في التربة .

ولتحقيق ذلك يجب ان تكون كل من

$\boxed{\begin{array}{l} e_B \leq B/6 \\ e_L \leq L/6 \end{array}}$ ⟸

$\boxed{\frac{B}{6} + \frac{L}{6} \leq 1}$

** if $e_B > B/6$ ⟶ يحدث خلل،



☒ Define :

B' = effictive width = $\boxed{B - 2e_B}$

L' = effictive Length = $\boxed{L - 2e_L}$

$\boxed{A' = L' * B'}$

** شرط مهم جداً :
يجب ان تكون $B' < L'$ دائماً
اذا كان العكس بستبدلهم.

Ex : $B = 2m$, $L = 3m$, $e_B = 0.1$ m , $e_L = 0.65$ m

$B' = 2 - 2(0.1) = 1.8m$
$L' = 3 - 2(0.65) = 1.7$ m ⟹ $B' > L'$ so $\boxed{\begin{array}{l} L' = 1.8m \\ B' = 1.7\text{ m} \end{array}}$ ✓

● General Capacity equ. with eccentricity : التغيرات بتصير عليها

(1) N_c, N_q, N_γ ⟶ Same before

(2) Shape factor ⟶ use B', L' in it : $F_{cs} = 1 + \frac{B'}{L'}\frac{N_q}{N_c}$

(3) Depth factor ⟶ Same as before . use L, B original .

(4) Load inclination ⟶ no effect .

(5) $q_{ult} = CN_cF_{cd} \; . \; . \; . \; . + qN_q \; . \; . \; . + \frac{1}{2}\overset{\text{مهم}}{\gamma}(B') \ldots$ use in Last term .

(6) F.s against Bearing capacity ⟶ $F.s = \frac{q_{ult}}{q_{all}}$ ⟹ $Q_{ult} = q_{ult} * \underset{\text{مهم}}{\underline{\underline{A'}}}$ "$L' * B'$"

(7) F.s against q_{max} : $F.S = \frac{q_{ult}}{q_{max}}$ $\qquad Q_{all} = q_{all} * A'$

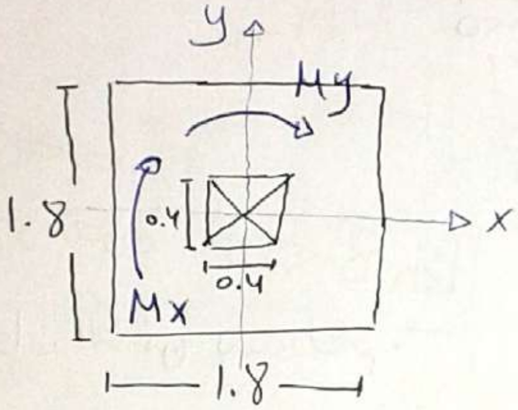
F.s > 1

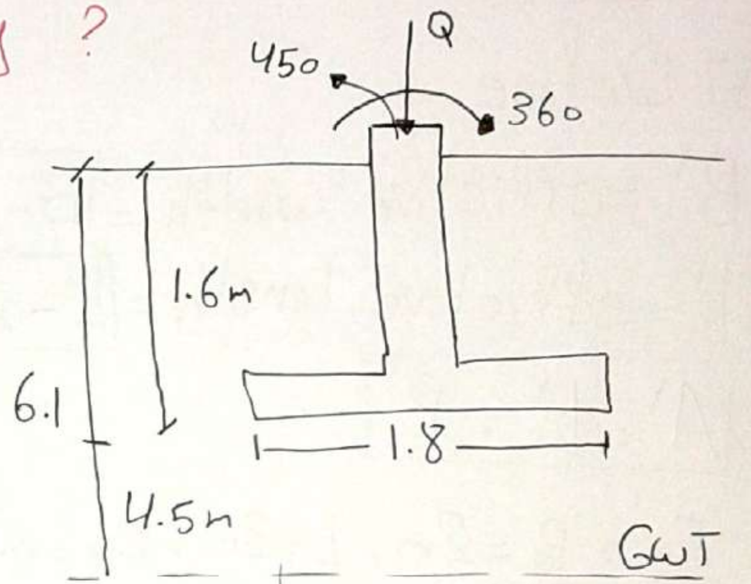
F.s against q_{max} > 1

$q_{min} \geq 0$

**عند وجود الازاحة هناك 3 checks :

Ex: A square footing is 1.8 m X 1.8 m with ~~an~~ a square Column 0.4 m x 0.4 m with an axial Load of 1800 kN and Mx = 450 kN.m and My = 360 kN.m Soil shear strength parameter C = 20 kpa & Ø' = 36° Soil has a γ = 18 kN/m2 and GWT. at depth of 6.1 m below ground surface. Find ~~FS~~ factors of safety ?





4.5 m > 1.8 m
d > B
*لا تأثير للماء

Sol: B = 1.8 m , L = 1.8 m

* $e_B = e_x = \frac{M_y}{Q} = \frac{360}{1800} = \boxed{0.2\ m}$

* $e_L = e_y = \frac{M_x}{Q} = \frac{450}{1800} = \boxed{0.25\ m}$

* $B' = B - 2e_B = 1.8 - 2(0.2) = \boxed{1.4\ m}$

$L' = L - 2e_L = 1.8 - 2(0.25) = \boxed{1.3\ m}$

$\Longrightarrow$ ~~but L' > B' so~~ B' = 1.4 > L' = 1.3

~~must be~~

* لكن يجب ان تكون B' < L' لذلك نستبدلهم

$\Longrightarrow \boxed{B' = 1.3\ m}$ and $\boxed{L' = 1.4\ m}$

* Ø = 36 ⟹ Nc = 50.59 , Nq = 37.75 , Nγ = 56.31

- Shape factor : انتبه

$$F_{cs} = 1 + \boxed{\frac{B'}{L'}} \frac{N_q}{N_c} = 1 + \left(\frac{1.3}{1.4}\right)\left(\frac{37.75}{50.59}\right) = \boxed{1.69}$$

$$F_{qs} = 1 + \frac{B'}{L} \tan\phi = 1 + \left(\frac{1.3}{1.4}\right)(\tan 36) = \boxed{1.67}$$

$$F_{\gamma s} = 1 - 0.4 \frac{B'}{L'} = 1 - 0.4\left(\frac{1.3}{1.4}\right) = \boxed{0.628}$$

- Depth factors : $\frac{D_f}{B} = \frac{1.6}{1.8} < 1$ ✓

$$F_{cd} = 1 + 0.4 \frac{D_f}{\boxed{B}} = 1 + 0.4\left(\frac{1.6}{\underline{1.8}}\right) = \boxed{1.35}$$

انتبه ← B

$$F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D_f}{B} = 1 + 2\tan(36)(1-\sin 36)^2 \frac{1.6}{1.8} = \boxed{1.21}$$

$$F_{\gamma d} = 1.0$$

- no Load Inclination :

$$F_{ci} = F_{qi} = F_{\gamma i} = 1.0$$

$$* \; q = \gamma D_f = (18)(1.6) = \boxed{28.8 \text{ Kpa}}$$

$$\Longrightarrow q_{ult} = (20)(50.59)(1.69)(1.35)(1) + (28.8)(37.75)(1.67)(1.219)(1) * + (0.5)(18)\underline{\underline{(1.3)}}(56.31)(0.628)(1)(1) = \boxed{4930.72 \text{ Kpa}}$$

$$\Longrightarrow Q_{ult} = q_{ult} * A' = 4930.72 * (1.3 * 1.4) = \boxed{8973.91 \text{ Kpa}}$$

①

$$F.S_{BC} = \frac{Q_{ult}}{Q_{all}} = \frac{8973.91}{1800} = 4.98 > 1 \checkmark$$

$$\Longrightarrow q_{max} = \frac{Q}{BL}\left(1 + \frac{6e_B}{B} + \frac{6e_L}{L}\right) = \frac{1800}{(1.8)(1.8)}\left(1 + \frac{6(0.2)}{1.8} + \frac{6(0.25)}{1.8}\right) = \boxed{1388.88 \text{ Kpa}}$$

②

$$F.S_{max} = \frac{q_{ult}}{q_{max}} = \frac{4930.72}{1388.88} = 3.55 > 1 \checkmark$$

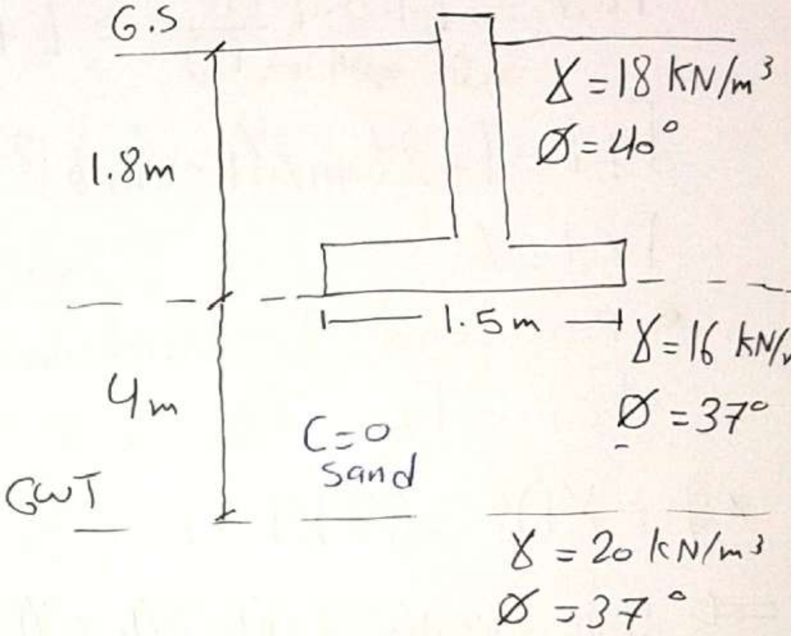
③

$$\Longrightarrow q_{min} = \frac{Q}{BL}\left(1 - \frac{6e_B}{B} - \frac{6e_L}{L}\right) = -277.77 < 0 \; ✗$$

Seperation يحدث

Ex: A rectangular footing (1.5m x 2.0m) founded on Sand, the footing was designed with a factor of Safety = 3, when constructed the contractor was displaced the center of column by 0.26 m away from the center along Long direction, and the Column was inclined by 8° from vertical, Determine actual factor of safety:

$\boxed{1}$ Qdesign before Inc. & eccen. :



* $d > B$

$4 > 1.6 \Rightarrow$ لاتأثير للماء

* $\varnothing = 37^\circ \Longrightarrow$ الطبقة اسفل الـ footing مباشرة، معرضة وتحمل للـ Load اساس

$N_q = 42.92$

$N_\gamma = 66.19$

- Shape factors:

$F_{qs} = 1.59$

$F_{\gamma s} = 0.7$

- depth factors: $\frac{D_f}{B}$ [illegible] > 1

$F_{qd} = 1 + 2\tan(37)(1 - \sin 37)^2 \underline{\tan^{-1}\left(\frac{1.8}{1.5}\right)} = 1.2$ — rad حولها

$F_{\gamma d} = 1.0$

- no Incline $\Rightarrow F_{qi} = F_{\gamma i} = 1.0$

* $q = 18 * 1.8 = \boxed{32.4\ kPa}$

$q_{ult} = \boxed{3159.21\ kPa}$

$Q_{design} = \frac{Q_{ult}}{F.S} = \frac{q_{ult} * A}{F.S} = \frac{3159.21(1.5 \times 2)}{3} = \boxed{3159.21\ kN}$

After Construction :

$e_L = 0.26\ m$ "along Long direction"

$L' = L - 2e_L = 2 - 2(0.26) = \boxed{1.48\ m}$

$B' = B - 2e_B = 1.5 - 2(0) = \boxed{1.5\ m}$

$B' > L'$

لذلك نستبدلهم ⇑

$\boxed{\begin{array}{l} B' = 1.48\ m \\ L' = 1.5\ m \end{array}}$

☒ Shape factors :

$F_{qs} = 1 + \frac{B'}{L'} \tan\phi = 1 + \frac{1.48}{1.5} \tan 37 = \boxed{1.74}$

$F_{\gamma s} = 1 - 0.4 \frac{B'}{L'} = 1 - 0.4 \frac{1.48}{1.5} = \boxed{0.605}$

☒ depth factor : ~~Remain~~ same

$F_{qd} = 1.2$, $F_{\gamma d} = 1.0$

☒ Incline by 8°

$F_{qi} = (1 - \frac{\beta^\circ}{90})^2 = (1 - \frac{8}{90})^2 = \boxed{0.83}$

$F_{\gamma i} = (1 - \frac{\beta^\circ}{\phi})^2 = (1 - \frac{8}{37})^2 = \boxed{0.61}$

$\Rightarrow q_{ult} = 0 + (32.4)(42.92)(1.74)(1.2)(0.83) + 0.5(16)\underline{\underline{(1.48)}}(66.19)(0.605)(1)$ (0.61)

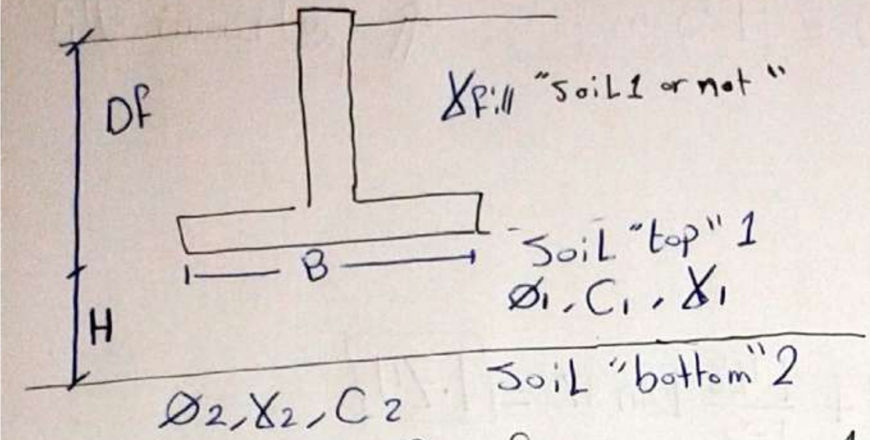
$= \boxed{2699.19\ kPa}$

$\Rightarrow F.S = \frac{Q_{ult}}{Q_{design}} = \frac{q_{ult}\ A}{Q_d} = \boxed{1.89}$

$\Rightarrow$ find q_{max} & check $F.S_{max}$

Bearing Capacity for two Layered footing :

* في الامثلة السابقة كان ال failure يحدث في طبقة واحد ، سوف ندرس بهذا الموضوع حدوث ال failure في طبقتين .



Ca ≡ Adhesion

Ks ≡ punching Coeficient

H ≡ البعد من قاعده الاساس لبداية الطبقة الثانية

Check if failure in 1 or 2 Layer :

* $H \geq 2B \Rightarrow$ one Layer الحل مثل السابق على القوانين $ الطبقة الواحدة

$\boxed{*}$ $H < 2B \Rightarrow 2$ Layer $\Rightarrow$ قوانين خاصة

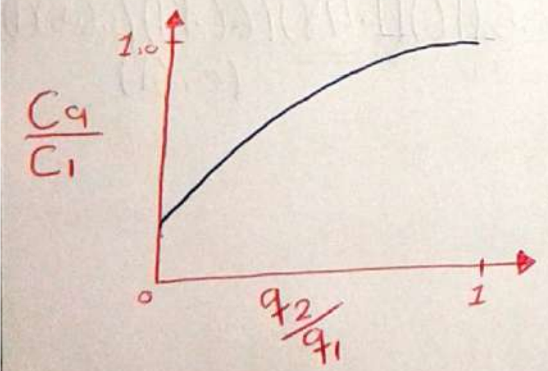
اول طبقة يحدث بها punching

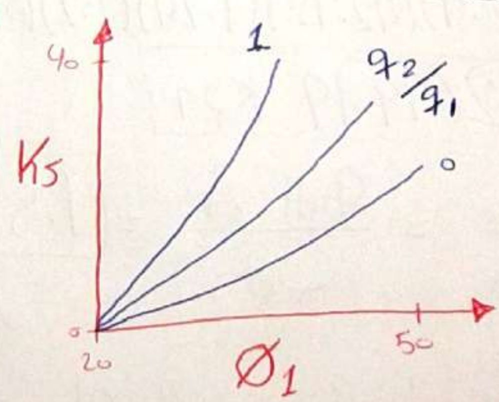
$$q_{ult} = \overset{bottom}{q_b} + \left(1+\frac{B}{L}\right)\left(\frac{2C_a H}{B}\right) + \gamma_1 H^2 \left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H \leq \underset{top}{q_t}$$

$$\text{bottom } q_b = C_2 N_{c2} F_{cs2} + \gamma_1 (D_f + H)(N_{q2})(F_{qs2}) + \tfrac{1}{2}\gamma_1 B N_{\gamma 2} F_{\gamma s2}$$

$$q_t = C_1 N_{c1} F_{cs1} + \gamma_1 (D_f) N_{q1} F_{qs1} + \tfrac{1}{2}\gamma_1 B N_{\gamma 1} F_{\gamma s1}$$

⇐ Ca ، Ks من الرسمات :-





** في حال كانت $1 < \frac{q_2}{q_1}$ نأخذ منحنى ال 1 .

$q_1 = C_1 N_{c1} + \frac{1}{2}\gamma_1 B N_{\gamma 1}$

$q_2 = C_2 N_{c2} + \frac{1}{2}\gamma_2 B N_{\gamma 2}$

** undrained Soil $\Rightarrow \phi = 0$

** في الحل :

$q_{ult} \leq q_t$

⇐ نأخذ الاصغر قيمه دائما .

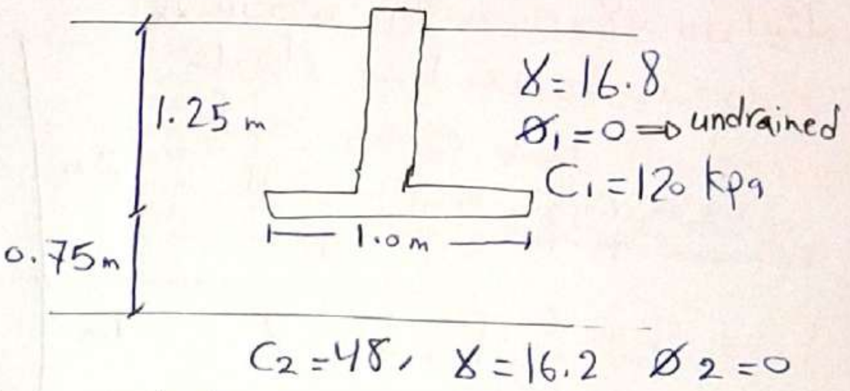
Ex: A Foundation 1.5 m x 1.0 m placed at depth of 1.25 m in a stiffy clay. A softe clay Layer is Located at depth of 0.75 m, measured from the bottom of the foundation. * For Top Layer, The undrained shear strength is 120 kpa & $\gamma = 16.8\ kN/m^3$ * For the bottom layer - the undrained shear strength is 48 kpa and $\gamma = 16.2\ kN/m^3$, Find the allowable bearing capacity if factor of safty = 3 ?

Sol :

Check :

$H = 0.75 < 2B = 2$

Two Layer



$$q_{ult} = q_b + (1+B/L)\left(\frac{2C_aH}{B}\right) + \underbrace{\cancel{\gamma H^2(1+B/L)\left(1+\frac{2D_f}{H}\right)\left(\frac{k_s\tan\phi_1}{B}\right)}}_{=0 \Rightarrow \text{Since } \cancel{\phi}\ \tan\phi_1 = 0} - \gamma_1 H < q_t$$

$$q_b = C_2 N_{c2} F_{cs2} + \gamma_1 (D_f + H) N_{q2} F_{qs2} + \cancel{0.5\gamma_2 B N_{\gamma 2} F_{\gamma s2}} \rightarrow 0$$

$\phi_2 = 0 \Rightarrow N_c = 5.14,\ N_q = 1,\ N_\gamma = 0$

$$F_{cs2} = 1 + B/L\ \frac{N_{q2}}{N_{c2}} = 1 + 1/1.5 * \frac{1}{5.14} = \boxed{1.13}$$

$$F_{qs2} = 1 + \cancel{B/L\ \tan\phi_2} \rightarrow 0 = \boxed{1}$$

$$\Rightarrow q_b = (48)(5.14)(1.13) + (16.8)(1.25 + 0.75)(1)(1) + 0 = \boxed{312.4\ kpa}$$

$$q_t = C_1 N_{c1} F_{cs1} + \gamma_1 D_f N_{q1} F_{qs1} + \cancel{\tfrac{1}{2}\gamma_1 B N_{\gamma 1} F_{\gamma s1}} \rightarrow 0$$

$$\Rightarrow q_t = (120)(5.14)(1.13) + (16.8)(1.25)(1)(1) = \boxed{717.98\ kpa}$$

$$q_1 = C_1 N_{c1} + \cancel{\tfrac{1}{2}\gamma_1 B N_{\gamma 1}} \rightarrow 0 = (120)(5.14)$$

$$q_2 = C_2 N_{c2} + \cancel{\tfrac{1}{2}\gamma_2 B N_{\gamma 2}} \rightarrow 0 = (48)(5.14)$$

$\Rightarrow \frac{q_2}{q_1} = \frac{48}{120} = \boxed{0.4} \xrightarrow{\text{من الرسمه}} \frac{C_a}{C_1} = 0.9$

$\Rightarrow \boxed{C_a = 108 \text{ Kpa}}$

$q_{ult} = 312.4 + (1 + 1/1.5)\left(\frac{2 * 108 * 0.75}{1}\right) + 0 - (16.8)(0.75) \leq 717.9$

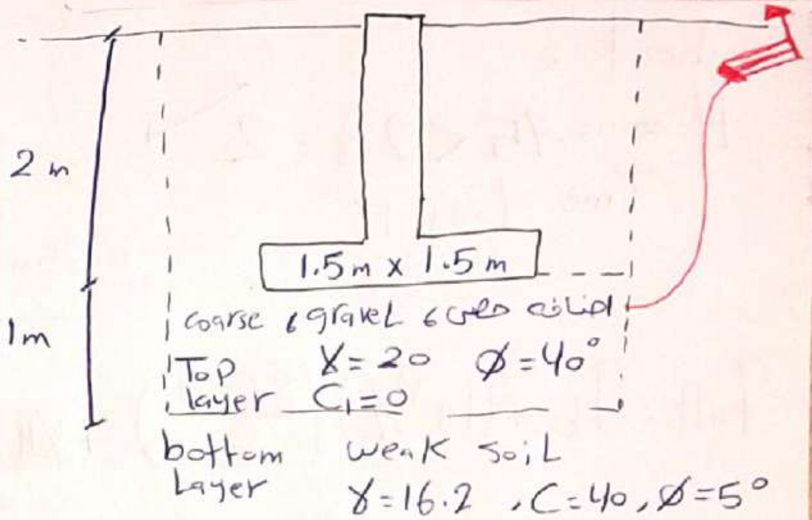
$\Rightarrow \underline{\underline{569.8}} < 717.98$ نأخذ الاصغر

$\Longrightarrow q_{ult} = 569.8 \Longrightarrow q_{all} = \frac{q_{ult}}{3} = \boxed{189.9 \text{ Kpa}}$

Ex :

بالمثال راح تزيد الـ bearing capacity من 426.9 ← 686.6

** عند وجود تربة ضعيفة ولا بد من البناء عليها، قمنا بعمل حفريات ووضع طبقة Coarse اسفل الاساس لزيادة الـ bearing capacity



Find q_{all} if F.S = 3 ?

Sol :

$\phi = 5° \rightarrow N_c = 6.49$, $N_q = 1.57$, $N_\gamma = 0.45$

$F_{cs_2} = 1.24$, $F_{qs_2} = 1.08$, $F_{\gamma s_2} = 0.6$

$\Longrightarrow q_b = (40)(6.49)(1.24) + (20)(2+1)(1.57)(1.08) + 0.5(16.2)(1.5)(0.45)(0.6) = \boxed{426.92 \text{ kpa}}$

$q_2 = (40)(6.49) + 0.5(16.2)(1.5)(0.45) = \boxed{265 \text{ Kpa}}$

$\frac{q_2}{q_1} = \frac{265}{1641} = 0.16$ → من الرسمه ⇐ $\boxed{K_s = 2.5}$

$\phi = 40° \rightarrow N_c = ~~64.2~~$, $N_q = 64.2$, $N_\gamma = 109.2$

$\Longrightarrow F_{qs_1} = 1.84$, $F_{\gamma s_1} = 0.6$

$\Longrightarrow q_t = 0 + (20)(2)(64.2)(1.84) + 0.5(20)(1.5)(109.4)(0.6) = \boxed{5709.72 \text{ K}}$

$q_1 = 0 + 0.5(20)(1.5)(109.2) = \boxed{1641 \text{ kpa}}$

$q_{ult} = 426.92 + 0 + (20)(1)^2\left(1 + \frac{1.5}{1.5}\right)\left(1 + \frac{2*2}{1}\right)\left(\frac{2.5 \tan 40}{1.5}\right) - (20)(1) \leq 5709.72$

$\Rightarrow \underline{686.6 \text{ Kpa}} < 5709.72 \text{ kpa} \Rightarrow$ بوخذ الاقل

$\Rightarrow q_{all} = \frac{686.6}{3} = \boxed{228.66 \text{ Kpa}}$

Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6)

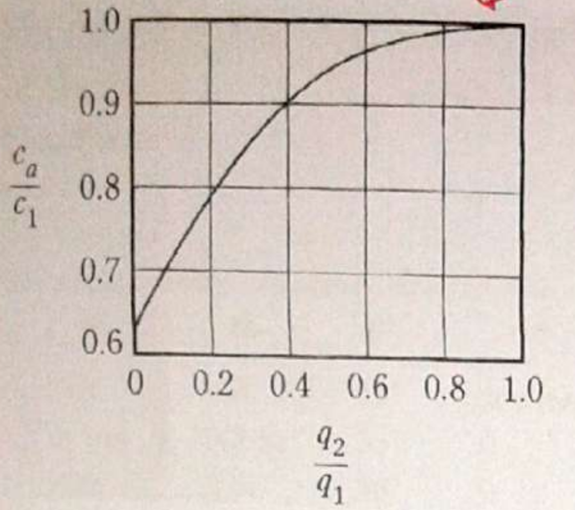
ϕ'	N_c	N_q	N_γ[*]	ϕ'	N_c	N_q	N_γ[*]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

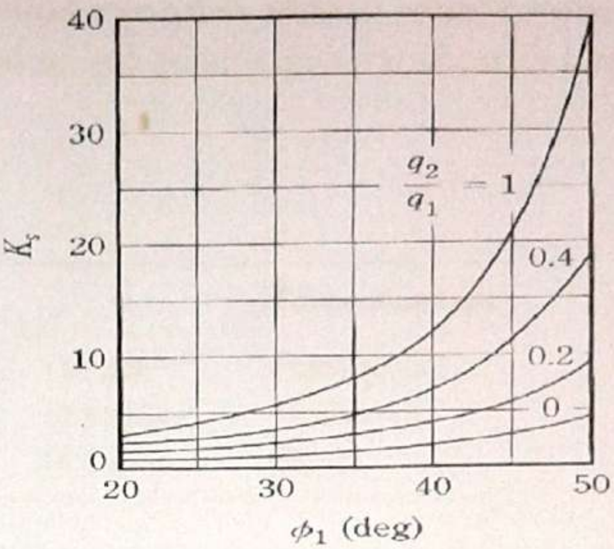
[*] From Kumbhojkar (1993)

Table 3.3 Bearing Capacity Factors

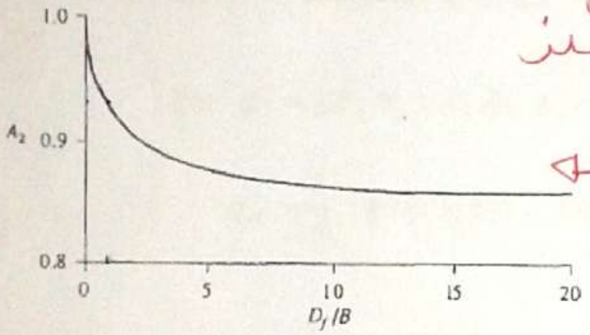
ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

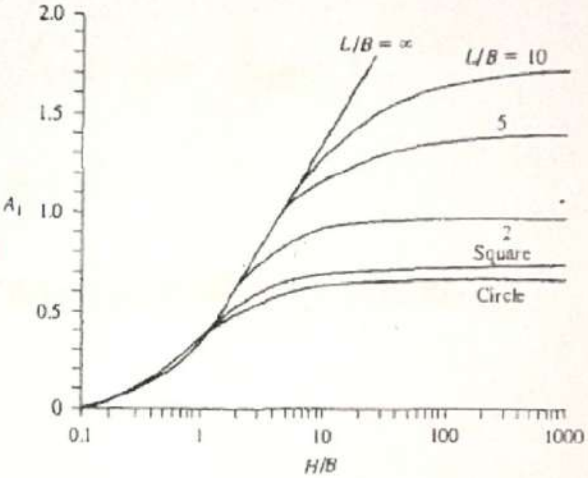
فيرست





سكنز





$$E_{s(\text{rectangle})} = \left(1 + 0.4 \log\frac{L}{B}\right) E_{s(\text{square})}$$

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}}$$

- I_z at $z = 0$

$$I_z = 0.1 + 0.0111\left(\frac{L}{B} - 1\right) \leq 0.2$$

- Variation of z_1/B for $I_{z(m)}$

$$\frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) \leq 1$$

- Variation of z_2/B

$$\frac{z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right) \leq 4$$

4.1 For the following cases, determine the allowable gross vertical load-bearing capacity of the foundation. Use Terzaghi's equation and assume general shear failure in soil. Use FS = 4.

Part	B	D_f	ϕ'	c'	γ	Foundation type
a.	3 ft	3 ft	28°	400 lb/ft^2	110 lb/ft^3	Continuous
b.	1.5 m	1.2 m	35°	0	17.8 kN/m^3	Continuous
c.	3 m	2 m	30°	0	16.5 kN/m^3	Square

a Eq. (3.3) and Table 3.1: $q_{all} = \dfrac{q_u}{\text{FS}} = \dfrac{1}{\text{FS}}\left(c'N_c + qN_q + \dfrac{1}{2}\gamma BN_\gamma\right)$

For $\phi' = 28°, N_c = 31.61, N_q = 17.81, N_\gamma = 13.7$

$$q_{all} = \frac{1}{4}\left[(400)(31.61) + (110)(3)(17.81) + \frac{1}{2}(110)(3)(13.7)\right] = \mathbf{5195\ lb/ft^2}$$

b. $q_{all} = \dfrac{q_u}{\text{FS}} = \dfrac{1}{\text{FS}}\left(c'N_c + qN_q + \dfrac{1}{2}\gamma BN_\gamma\right)$

For $\phi' = 35°$, from Table 3.1, $N_c = 57.75, N_q = 41.44, N_\gamma = 45.41$

$$q_{all} = \frac{1}{4}\left[0 + (1.2 \times 17.8)(41.44) + \frac{1}{2}(17.8)(1.5)(45.41)\right] = \mathbf{372.8\ kN/m^2}$$

c. Table 3.1: $\phi' = 30°, N_q = 22.46, N_\gamma = 19.13$

Eq. (3.7), with $c' = 0$

$$q_{all} = \frac{q_u}{\mathrm{FS}} = \frac{1}{\mathrm{FS}}(qN_q + 0.4\gamma BN_\gamma)$$

$$= \frac{1}{4}[(2 \times 16.5)(22.46) + (0.4)(16.5)(3)(19.13)] = \mathbf{280\ kN/m^2}$$

.2 A square column foundation has to carry a gross allowable load of 1805 kN (FS = 3). Given: $D_f = 1.5$ m, $\gamma = 15.9$ kN/m³, $\phi' = 34°$, and $c' = 0$. Us Terzaghi's equation to determine the size of the foundation (B). Assume genera shear failure.

Eq. (3.7); $c' = 0$

$$q_{all} = \frac{q_u}{\mathrm{FS}} = \frac{1}{\mathrm{FS}}(qN_q + 0.4\gamma BN_\gamma)$$

$$q_{all} = \frac{Q_{all}}{B^2} = \frac{1805}{B^2}$$

From Table 3.1, for $\phi' = 34°, N_q = 36.5, N_\gamma = 38.04$

$$\frac{1805}{B^2} = \frac{1}{3}[(1.5)(15.9)(36.5) + (0.4)(15.9)(B)(38.04)]$$

By trial and error, $B = \mathbf{2\ m}$

4.5 A column foundation (Figure P4.5) is 3 m × 2 m in plan. Given: $D_f = 1.5$ m, $\phi' = 25°$, $c' = 70$ kN/m^2. Using Eq. (4.26) and FS = 3, determine the net allowable load [see Eq. (4.22)] the foundation could carry.

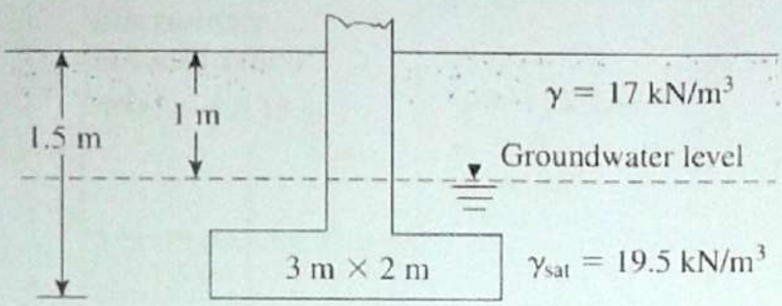


Figure P4.5

From table :

$\varnothing = 25 \rightarrow 20.72$

$\rightarrow 10.66$

$\rightarrow 10.88$

$\boxed{*}$ Shape factors :

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} = 1 + \frac{2}{3}\cancel{\left(\frac{20.72}{18.66}\right)}\left(\frac{10.66}{20.72}\right) = \boxed{1.34}$$

$$F_{qs} = 1 + \frac{2}{3}\tan\varnothing = 1 + \frac{2}{3}\tan 25 = \boxed{1.31}$$

$$F_{\gamma s} = 1 - 0.4 * \frac{B}{L} = 1 - 0.4\left(\frac{2}{3}\right) = \boxed{0.73}$$

$\boxed{*}$ Depth factors : $\frac{D_f}{B} = \frac{1.5}{2} < 1$

$$F_{cd} = 1 + 0.4\frac{D_f}{B} = 1 + 0.4\left(\frac{1.5}{2}\right) = \boxed{1.3}$$

$$F_{qd} = 1 + 2\tan\varnothing(1-\sin\varnothing)^2\frac{D_f}{B} = 1 + 2\tan(25)(1-\sin 25)^2\frac{1.5}{2} = \boxed{1.23}$$

$$F_{\gamma d} = \boxed{1}$$

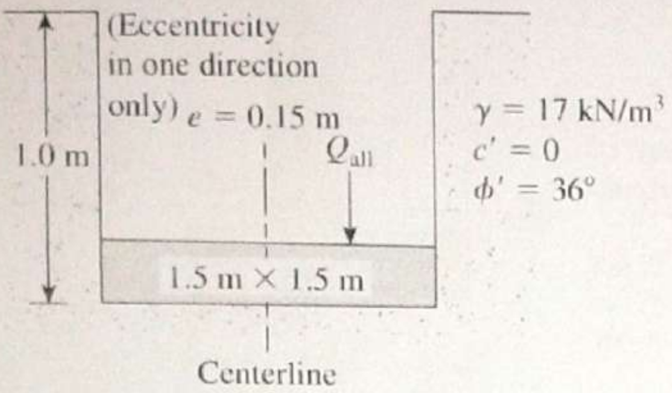
$q = (1)(17) + 0.5(19.5 - 9.81)$

$$q_{ult} = 70(20.72)(1.34)(1.3)(1) + \underline{(21.845)}(10.66)(1.31)(1.23) + 0.5(19.5 - 9.81)(10.88)(0.73)(1)(2)$$

$$= 2978.78 \text{ kpa}$$

$$Q_{all} = q_{all} * Area = \frac{q_{ult}}{F.S} * Area = \frac{2978.78}{3} * 2 * 3 = \boxed{5957.58 \text{ KN}}$$

4.8 An eccentrically loaded foundation is shown in Figure P4.8. Use FS of 4 and determine the maximum allowable load that the foundation can carry. ~~Use Meyerhof's effective area method~~.



$e_B = 0.15$ m

$B' = B - 2e_B = 1.5 - 2(0.15) = \boxed{1.2\ m}$

$L' = L - 2e_L = 1.5 - 2(0) = \boxed{1.5\ m}$

Eccent. in one direction only

** from table Ø = 36 → $N_q = 37.75$
→ $N_\gamma = 56.31$

*shape :

$$F_{qs} = 1 + \frac{\overset{B'}{1.2}}{1.5}\tan 36 = \boxed{1.58}$$

$$F_{\gamma s} = 1 - 0.4 * \frac{1.2}{1.5} = \boxed{0.68}$$

*depth : $\frac{D_f}{B} = \frac{1}{\underset{\to B}{1.5}} < 1$

$$F_{qd} = 1 + 2\tan 36(1 - \sin 36)^2 \frac{1}{1.5} = \boxed{1.16}$$

$$F_{\gamma d} = 1$$

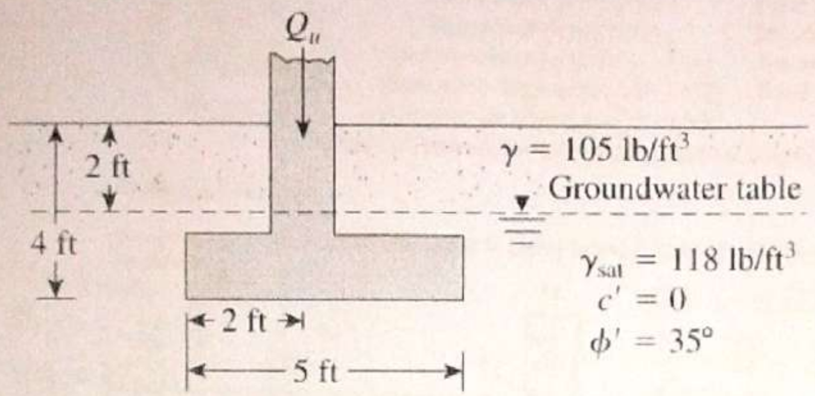
$$q_{ult} = 0 + \overset{q}{17} * 37.75 * 1.58 * 1.16 + 0.5(17)(56.31)(0.68)(1)(\overset{B'}{1.2})$$

$$= \boxed{1566.77\ kpa}$$

$$q_{all} = \frac{1566.77}{4} = 391.69$$

$$Q_{all} = q_{all} * \grave{A}rea = 391.69 * 1.5 * 1.2 = \boxed{705.04\ kN}$$

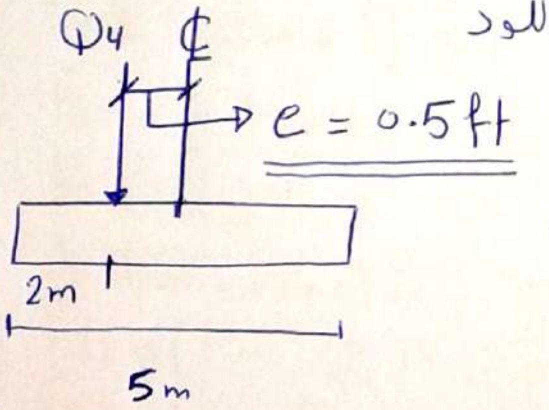
4.11 An eccentrically loaded continuous foundation is shown in Figure P4.11. Determine the ultimate load Q_u per unit length that the foundation can carry. Use the reduction factor method [Eq. (4.63)].



** فكرة السؤال :-

يجب ان يكون اللود " العمود " في مركز الـ footing

و الـ e هي البعد بين المركز واللود



واكمل حل على قوانين الـ eccentricity

وانتبه للـ water table .

اختبر نفسك "40 min"

الرقم التسلسلي 55

Student Name: عمر زياد فريد ابو نجاد Student Number: 1432740

25

Hashemite University
Faculty of Engineering
Department of Civil Engineering
Foundation Engineering (0401435)
Instructor: Dr. Omar Al-Hattamleh

First Exam
Monday: 31/10/2016
Attempts All Problems
Write Only in the designated space

Note all calculation are made to nearest 2 significant digits

Problem # 1 (8 points)

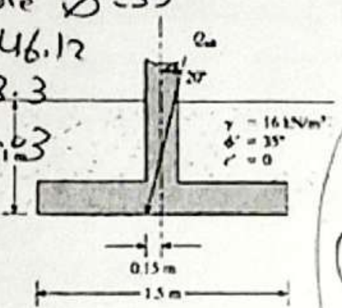
A continuous (wall) foundation is shown in Figure below. Estimate the ultimate load, per unit length of the foundation.

from table $\phi = 35$

$N_c = 46.12$

$N_q = 33.3$

$N_\gamma = 48.03$



$e_B = 0.15$ m

$B' = B - 2e_B = \boxed{1.2 \text{ m}}$

~~$\frac{B}{L} = 1m$~~

$\frac{B}{L} = 0$

$L >> B$ wall

Q 40

Shape factor :

$F_{qs} = 1 + \underbrace{\frac{B}{L}}_{=\boxed{0}} \tan\phi \Rightarrow \boxed{1}$

$F_{\gamma s} = 1 - 0 = \boxed{1}$

Depth factor : $\frac{D_f}{B} \leq 1$

$F_{qd} = 1 + 2\tan(35)(1 - \sin(35))^2 * \frac{1}{1.5} = \boxed{1.17}$

$F_{\gamma d} = \boxed{1.0}$

inclination factor : $\beta° = 20°$

$F_{qi} = (1 - \frac{20}{90})^2 = \boxed{0.605}$

$F_{\gamma i} = (1 - \frac{20}{35})^2 = \boxed{0.184}$

$q = \cancel{16} * \gamma D_f = 16 * 1 = \boxed{16 \text{ kPa}}$

$\gamma = 16$

$q_{ult} = \cancel{C N_c F_{cs} F_{cd} F_{ci}} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma N_\gamma F_{\gamma s} F_{\gamma i} B'$

$= 0 + 16 * 33.3 * 1 * 1.17 * 0.605 + 0.5 * 16 * (48.03) * (1) * (1) * (0.184) * (1.2)$

$= \boxed{461.98 \text{ kPa}}$

يتبع $\Longrightarrow$

$Q_{ult} = q_{ult} * \text{Area}$

$= 461.98 * B' = 461.98(1.2)$

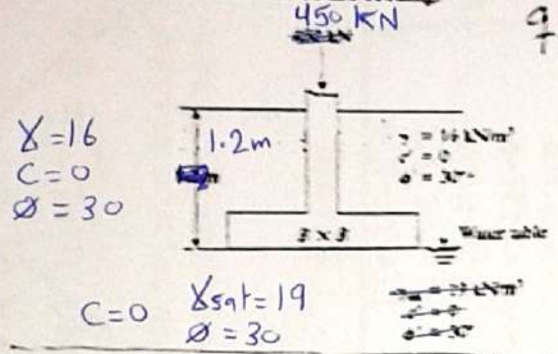
$= \boxed{554.38 \text{ kN}}$

Student Name: .... Student Number: ....

Problem # 2 (7 points)

A square footing is shown in Figure below. Use Factor of Safety = 3 and Terzaghi bearing capacity equation determine the size of the footing.

$q_{ult} = 1.3\,C N_c + q N_q + 0.4\,\gamma B \cancel{*} N_\gamma$ for square



for G.W.T :

Case 2 : $0 \le d \le B$

$q = \gamma_1 D_f = 16 * 1.2 = \boxed{19.2 \text{ kpa}}$

use $\bar{\gamma} = \gamma'_2 + \cancel{(\gamma_1 - \gamma'_2)\, d/B} \rightarrow 0$

$= \gamma_{sat} - \gamma_w = 19 - 9.81$

$= \boxed{9.19 \text{ KN/m}^3}$

from Table "Terzaghi"

$\varnothing = 30$

$N_c = 37.16$

$N_q = 22.46$

$N_\gamma = 19.13$

$q_{ult} = F.S * q_{allowable}$

$= \dfrac{3 * 450}{Area} = \dfrac{1350}{B^2}$

$\dfrac{1350}{B^2} = 1.3 * (0)(37.16) + 19.2 * 22.46 + 0.4 * 9.19 * B * 19.13$

$B^2 * \dfrac{1350}{B^2}) = (0 + 431.23 + 70.32\,B) * B^2$

$\Rightarrow 70.32\,B^3 + 431.23\,B^2 - 1350 = 0$

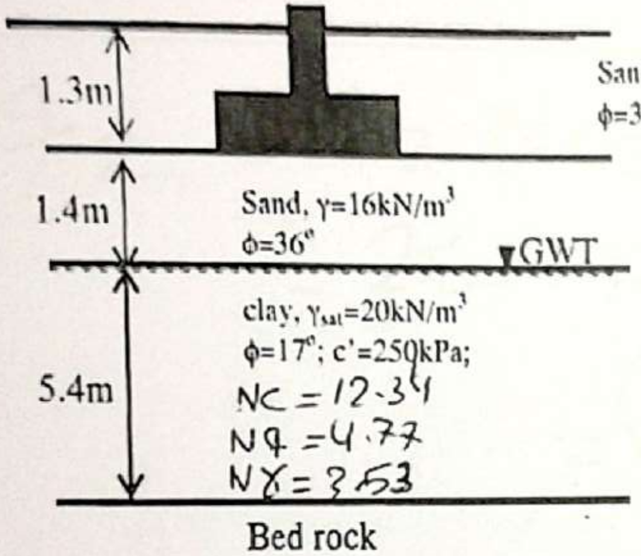
$\boxed{B = 1.58 \text{ m}}$

2

Student Name:……………………………………Student Number:……………………………

Problem # 3: (5 points)

A rectangular footing (1.6m x 2.0m) founded on soil profile shown. The footing was designed using factor of safety equal to 3.0.



Solg

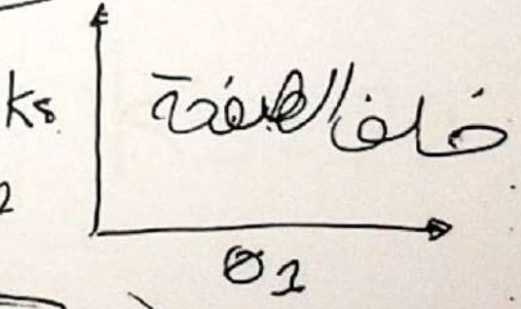
$\boxed{1}$ $H = \boxed{1.4}$ m

$2B = 2 * 1.6 = \boxed{3.2\ m}$

So: $H < 2B \Rightarrow$ ~~fail~~ footing will fail in <u>two Layers</u>.

$\boxed{2}$

1. Is this footing will fail in two layers or in a single layers? Why?
2. If your answer is "yes" determine the punching shear coefficient Ks?



$$\cancel{\overset{c}{q}_2 = C_2 N_{c2} F_{cs2} + \gamma_1 (D_f + H) N_{q2} F_{qs2} + \frac{1}{2} \gamma_2 B N_{\gamma 2} F_{\gamma s2}}$$

~~E2 = 250~~, $F_{cs2} = 1 + \frac{1.6}{2} * \frac{4.77}{12.34} = \boxed{1.31}$

Df

سؤال ③ فرع 2

$\phi = 36$

$N_c = 50.5$

$N_\gamma = 56.31$

$$q_1 = C_1 N_{c1} + \tfrac{1}{2} \gamma_1 B N_{\gamma 1}$$

$$= 0 + 0.5 * 16 * 1.6 * 56.31$$

$$= \boxed{720.768} \text{ Kpa}$$

$$q_2 = C_2 N_{c2} + 0.5 \gamma_2 B N_{\gamma 2}$$

N_{c2}

$N_\gamma =$

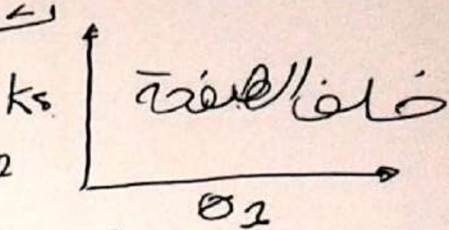
$$= 250 * 12.34 + 0.5 (20 - 9.81) * 1.6 * 3.53$$

$$= \boxed{3113.77} \text{ Kpa}$$

$$\frac{q_2}{q_1} = 4.32$$ من الرسمة

$K_s = 22$

1. Is this footing will fail in two layers or in a single layers? Why?
2. If your answer is "yes" determine the punching shear coefficient Ks?



~~$q_2 = C_2 N_{c2} F_{cs2} + \gamma_1 (D_f + H)(N_{q2}) F_{qs2} + \frac{1}{2} \gamma_2 B N_{\gamma 2} F_{\gamma s2}$~~

~~$C_2 = 250$~~ , ~~$F_{cs2} = 1 + \frac{1.6}{2} * \frac{4.77}{12.34} = \boxed{1.31}$~~

$\frac{D_f}{B} < 1$

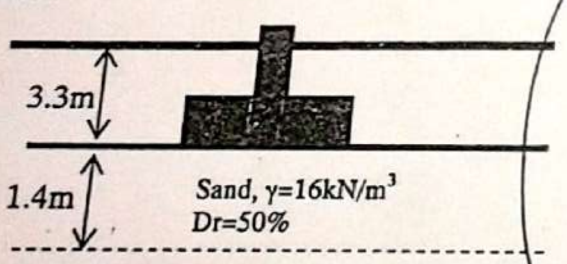
~~$F_{qs2} = 1 + \frac{1.6}{2} \tan(17) = \boxed{1.25}$~~

~~$F_{\gamma s} = 1 - 0.4 (\frac{1.6}{2}) = \boxed{0.68}$~~

~~$q_2 = 250 * 12.34 * 1.31 + 16(1.3 + 1.4) * 4.77 * 1.25 + 0.5(20 - 9.81) * 1.6 * 3.53 * 0.68$~~

خلف الصفحة $= \boxed{4318.5 \text{ kPa}}$ تتبع

Problem # 4: (5 points)
A rectangular footing (1.6m x 2.0m) founded on soil profile shown. Find only the bearing capacity factors if φ=30°.



ال نوع fail

$\frac{D_f}{B^*} = \frac{3.3}{1.77} = \boxed{1.864}$

$B^* = \frac{2 * 1.6 * 2}{1.6 + 2} = \boxed{1.77}$

Local shear failure

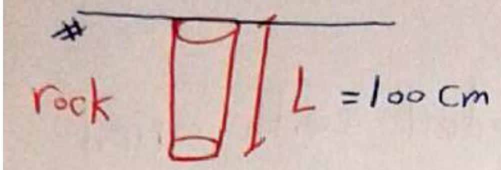
$\Rightarrow \phi'' = \tan^{-1}(\frac{2}{3} \tan \phi')$

$= \tan^{-1}(\frac{2}{3} \tan 30) = \boxed{21^\circ}$ من الرسمة ←

3/4

So $\boxed{N_c = 15.82 \;,\; N_q = 7.07 \;,\; N_\gamma = 6.2}$

• Bearing Capacity of Rock :



RQD : Rock Quality Designation

$$= \frac{\sum(\text{all pieces} > 100\ \text{mm})}{\text{Advancement Length}}$$

$RQD \rightarrow 0 \equiv$ weak

$RQD \rightarrow 1 \equiv$ exellent

$$q_{ult} = CN_c + qN_q + 0.5\gamma B N_\gamma$$

• الموضوع لم يتم شرحه إلا في شعبة واحده 3:

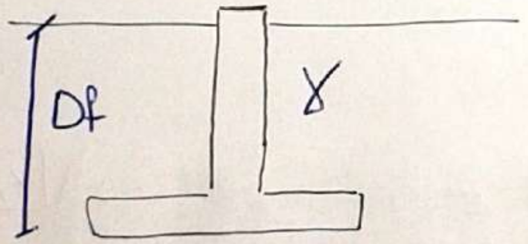
$\emptyset \Rightarrow$ Rock

- $\rightarrow N_c = 5\tan^4(45+\emptyset/2)$
- $\rightarrow N_q = \tan^6(45+\emptyset/2)$
- $\rightarrow N_\gamma = N_q + 1$

• Fully Compensation footing :

هناك عمق معين يصبح عنده ال Settlement تقريباً صفر ، نقوم مرات بالابراج العاليه بالحفر له حتى نلغي تأثير الهبوط .

ويتحقق عندما تكون $\boxed{q_{net} = 0}$.



*Net Bearing Capacity : ⇐ مهم

$$\boxed{q_{net} = q_{ult} - q}$$

$$\boxed{q = \gamma D_f}$$

$q_{ult} \rightarrow$ or "gross allowable pressure"

"مسنوات"

** if $q_{net} = 0 \rightarrow$ fully compensation footing

$$0 = q_{ult} - q \Longrightarrow D_f\gamma = q_{ult}$$

$$\Rightarrow D_f = \frac{Q_{ult}}{(A)(\gamma)} \Rightarrow$$

العمق بكون عند الهبوط تقريبا صفر .

● Foundation Settlement : CH6 , CH 7 : السكند

▪ Introduction :

* Foundation settlement must be estimated with great Care for building.

* For other structure such as : fill, earthdam, a greater margen of error in settlement can usually be tolerated.

▪ problems with soil settlement analysis : "سنوات"

1. Obtaining reliable Value of the elastic parameters: E, ν
2. Obtaining reliable stress profile from the applied Load.

▪ Type of settlement :

1. Elastic "Immediate" or distortion settlement for (sand , clay).
2. Consolidation settlement only for clay :

a) primary consolidation.

b) Secondary consolidation.

[redacted] **

Stress Increments : CH6

* الضغط المضاف/الزائد ينتج عند وضع الاساس أو المبنى.

$$\boxed{\sigma'_T = \sigma'_o + \Delta\sigma}$$

σ'_T : Total (final) effictive stress)

σ'_o : Initial overburden pressure : الحمل الاولي قبل وضع الاساس من التربة نفسها ويحسب مثل السابق

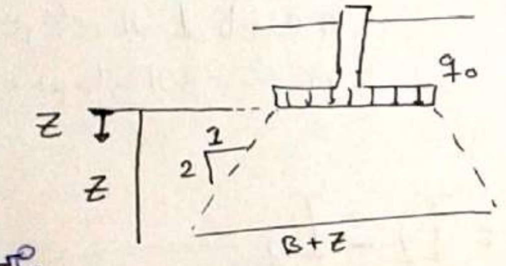
$$\boxed{\sigma'_o = \gamma Z}$$

$\Delta\sigma$: Induced (increment) stress.

****** لحساب $\Delta\sigma$ هناك طريقتان :

1. Approximate method
2. Elastic solution

Approximate method :



* strip footing

$$\boxed{\Delta\sigma = \frac{q_o \times B}{B+Z}}$$

* Square

$$\boxed{\Delta\sigma = \frac{q_o \times B \times B}{(B+Z)(B+Z)}}$$

* rectangular

$$\boxed{\Delta\sigma = \frac{q_o \times B \times L}{(B+Z)(L+Z)}}$$

q_o : net applied pressure
مهم

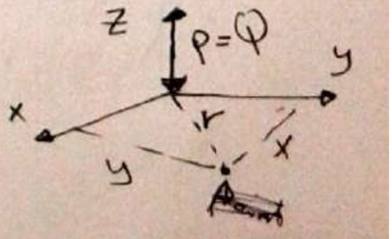
Z : العمق من قاعدة ~~أسفل~~ الـ footing للاسفل

Elastic Solution :

1. Point Load :

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

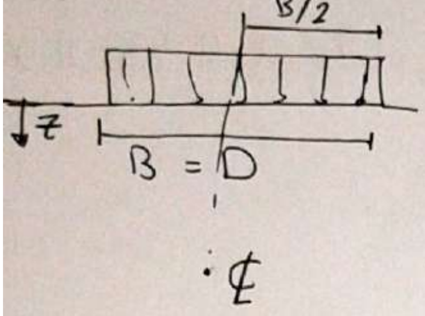
$$\boxed{r = \sqrt{x^2+y^2}}$$



2 Circular Loaded Area :

$$\boxed{\Delta\sigma = q_0\, I}$$

I: Influence factor

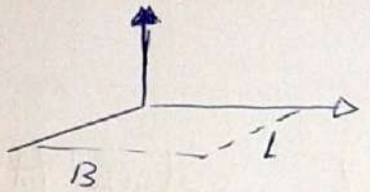


$$\boxed{I_{¢} = 1 - \frac{1}{\left[1+\left(\frac{B}{2z}\right)^2\right]^{3/2}}}$$

under the center of footing

* for other point (r, z) use table in book CH6.

3 Stress under Corner of Load rectangular area :



$$\boxed{\Delta\sigma = q_0\, I}$$

**شروط حساب ال I :-

① ان يكون الشكل مستطيل .

② ان تكون النقطة corner .

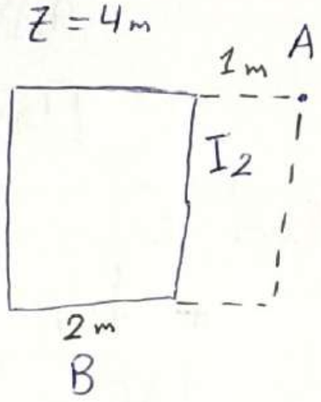
** خطوات الحل : مثال

① الضلع الاكبر L ← n

② الضلع الاصغر B ← m

⇐ اي تقسيم على ال L تأخذه n

واي تقسيم على الB " m .



$I_¢ \equiv$ للشكل كامل

$$\boxed{m = \frac{B}{z} = \frac{3}{4} \qquad n = \frac{L}{z} = \frac{4}{4}}$$

$I_1 = \square$ ← من جدول بالكتاب

$I_2 \equiv$

$m = \frac{1}{4}$

$n = \frac{4}{4}$ $\Rightarrow I_2 = \square$

$I_{total} = I_1 - I_2$ ← مهم

$$\boxed{\Delta\sigma = q_0\, I_{total}}$$

*هكذا نكون حسبنا الحمل المؤثر من الاساس عند النقطة A .

* ~~[illegible]~~ راجع الموضوع بمادة السويل

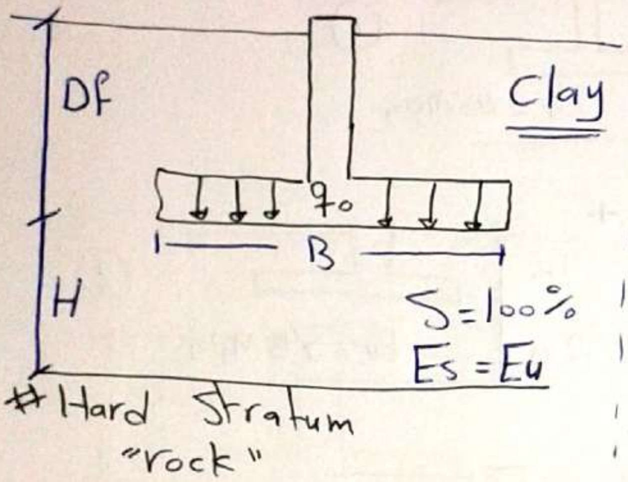
** غالباً اسئلة الامتحان تكون على ال (approximate methode) .

- Settlement :

$$\underset{\text{Total}}{S_T} = S_e + \underset{\text{Primary}}{S_c} + \underset{\text{Secondary}}{S_s}$$

$$S_e \begin{cases} \rightarrow S_e \Rightarrow \text{Sand} \\ \rightarrow S_d \text{ "distortion settle." } \Rightarrow \text{Clay} \end{cases} \rightarrow \text{Elastic settlement}$$

1 Distortion Settlement on saturated Clay : متوقع



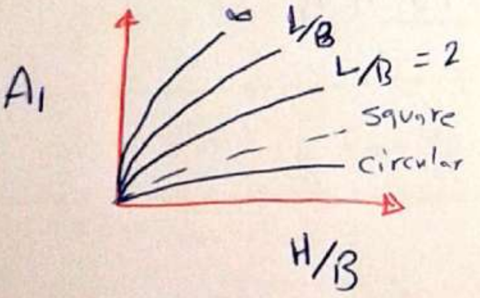
$$\boxed{S_d = A_1 A_2 \frac{q_o}{E_u} B}$$

q_o : net applied pressure

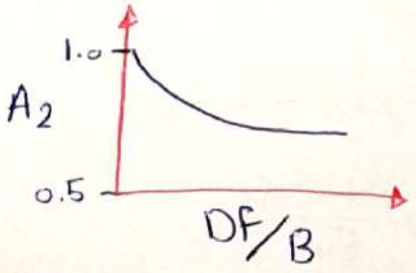
E_u : undrained Elastic modulus

B : width

A_1 : Shape factor



A_2 : Depth factor رسمات في كتاب :



** في حال اعطى Es ، لايجاد Eu نستخدم المعادلة التالية :

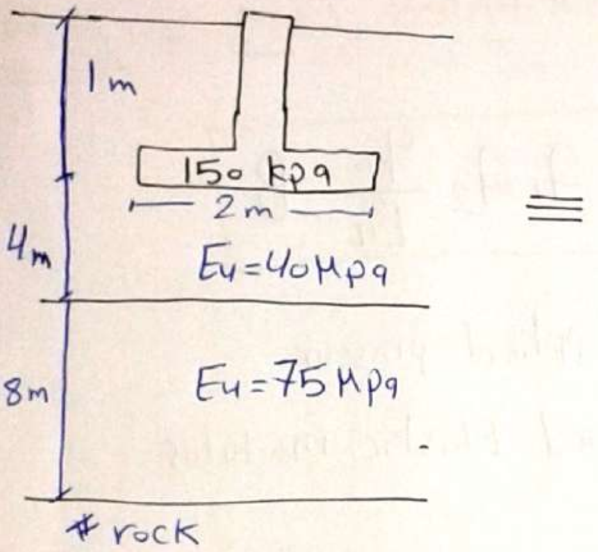
$\boxed{E_u = \beta E_s}$ ⇒ β : جدول في الكتاب يعطي كل من

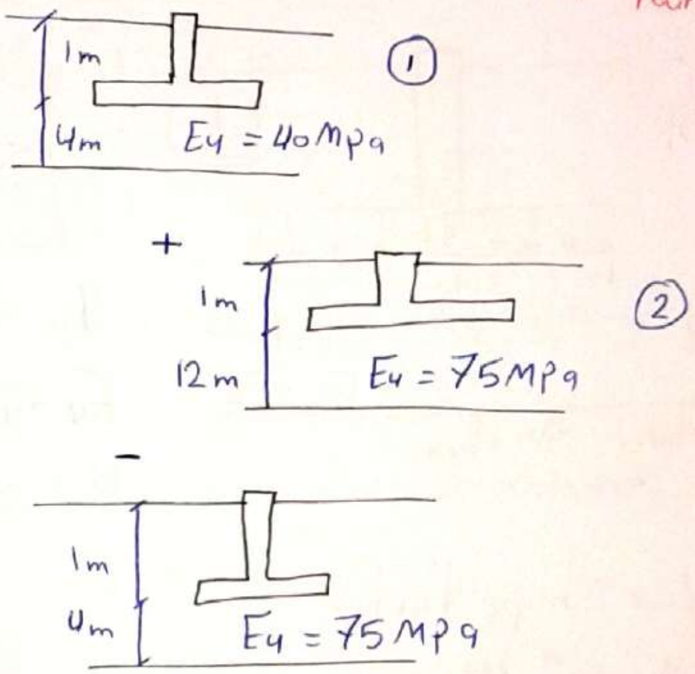
⇒ $E_u = (\boxed{} —)$ ما نأخذ الاقل

C_u | $\overset{\alpha R}{\boxed{}}$ → $\beta (—)$ على شكل فترة

23

Ex : A foundation 4m x 2m , Carring a net uniform pressure of 150 kpa is Located at a depth of 1m in a Layer of clay 5m thick for which the Value of Eu = 40 Mpa , The Layer in underLain by a Second Layer 8m thick for which the Eu = 75Mpa A hard stratum is Located under the second Layer Determine the average immediate Settlement under fund. ?





Sol :

$S_d = S_{d1} + S_{d2} - S_{d3}$

① $S_{d1} = A_1 A_2 \frac{q_o}{E_u} B$

$A_1 \left(\frac{H}{B} = 4/2 = \boxed{2},\ L/B = 4/2 = \boxed{2}\right) \xrightarrow{\text{from fig}} A_1 = 0.65$

$A_2 \left(\frac{Df}{B} = 1/2\right) \xrightarrow{\text{from fig}} A_2 = 0.925$

$S_{d1} = (0.65)(0.925)\left(\frac{150}{40*10^3}\right)(2) = \boxed{4.51 \text{ mm}}$

② $S_{d2} = A_1 A_2 \frac{q_o}{E_u} B$

$A_1 \left(\frac{H}{B} = \frac{12}{2} = 6,\ L/B = 2\right) \xrightarrow{\text{from fig}} A_1 = 0.85$

$A_2 = 0.925$

$Sd_2 = (0.85)(0.925)\left(\frac{150}{75*10^3}\right)(2)^{*10^3} = \boxed{3.145 \text{ mm}}$

---

③

$A_1 = 0.65$

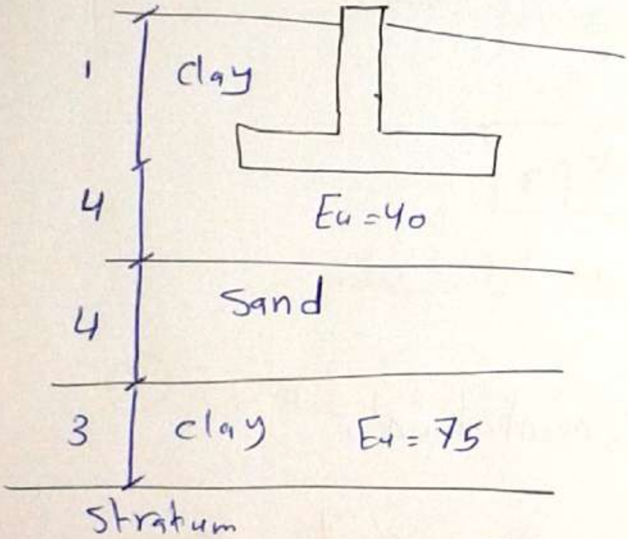
$A_2 = 0.925$

$Sd_3 = (0.65)(0.925)\left(\frac{150}{75*10^3}\right)(2)^{*10^3} = \boxed{2.405 \text{ mm}}$

$\therefore Sd = Sd_1 + Sd_2 - Sd_3 = 4.51 + 3.145 - 2.405$

$= \boxed{5.25 \text{ mm}}$

⇐ في حال اعطى في السؤال طبقات clay وبينهم sand مثلاً :

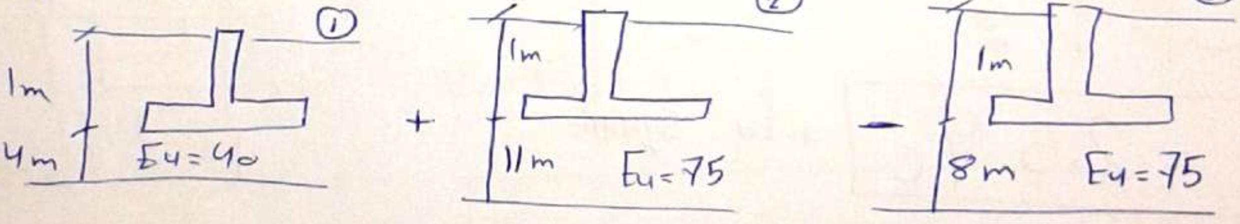


بعتبر انوما في طبقة sand وبحسب عادي لل clay وبطلع Sd .

بعدين بحسب Se لل sand

وبحسب St

$St = Se + Sd$



$\Rightarrow Sd = Sd_1 + Sd_2 - Sd_3$

● Elastic Settlement on Sandy Layer "E_s"

* In situ Testing : ** هناك اختبارين للحصول على Es :

① Standard penetration Test (SPT)

② Cone penetration Test (CPT)

$\boxed{1}$ SPT :

$$\boxed{N_{60} = \frac{N\,\eta_H\,\eta_B\,\eta_S\,\eta_R}{60}}$$

N : number of blows that the Sampler can enter 1 ft.

N_{60} : spt N corrected for field procedure.

η_H : Hammer efficiency

η_S : Sample Correction

η_B : Bore Hole

η_R : rod Correction

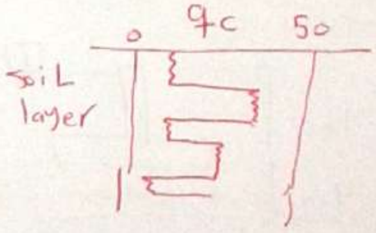
$$\boxed{E_s = \alpha\,(N_{60})\,P_a}$$

مهم ⇐ "سنوات"

P_a : atmospheric pressure ≡ $\boxed{100 \text{ kpa}}$

α
- → 10 Sand Clean. ⇐ نظيفة
- → 5 Sand with fine.
- → 15 Sand with ~~Cemetat~~ Cementation.

$\boxed{2}$ CPT :



$$\boxed{E_s = 2.5\,q_c}$$ ⇒ for Square

$$\boxed{E_s = 3.5\,q_c}$$ ⇒ for wall $L/B > 10$

$$\boxed{E_s = \left(1 + 0.4 \log L/B\right) E_{s\,square}}$$ ⇒ for rectangular $1 < \frac{L}{B} < 10$

$$\boxed{S_e = C_1 C_2 q_{net} \sum_{i=1}^{i=n} \frac{I_{zi}\,\Delta z_i}{E_{si}}}$$

Se : Elastic Settlement on Sand Layer.

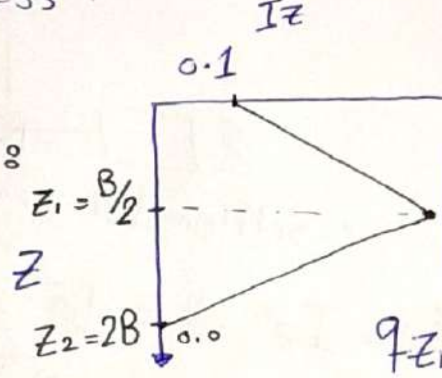
C_1 : depth correction $= \boxed{1 - 0.5 \frac{q}{q_{net}}}$ $q \rightarrow D_f * \gamma$

C_2 : Creep factor $= \boxed{1 + 0.2 \log t/0.1}$ t: time since construction

ΔZ : Layer thickness.

I_z :

① Square :



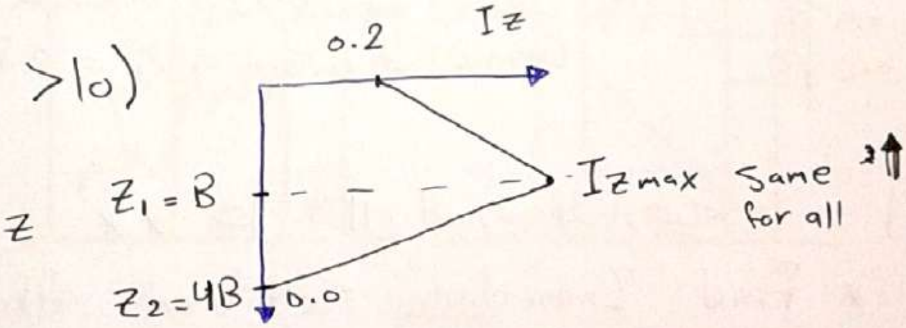
$\rightarrow Z = 0$ (من قاعدة الاساس) مهم*

$$\boxed{I_{z\,max} = 0.5 + 0.1 \sqrt{\frac{q_{net}}{q_{Z_1}}}}$$

q_{Z_1} = Stress at Z_1

*تذكر ان Z_1 مقاسه من نهاية ال footing يعني بدنا نضيف عليها Df .

② wall ($L/B > 10$)



③ Rectangular :

* Iz at Z = 0 $\Longrightarrow$ $\boxed{I_z = 0.1 + 0.0111 (L/B - 1)} \leq 0.2$

* Izmax at Z_1 : $\boxed{\frac{Z_1}{B} = 0.5 + 0.0555 (L/B - 1)} \leq 1$

Same for all

قبل الضرب بـ B

* Iz = 0 at Z_2 : $\boxed{\frac{Z_2}{B} = 2 + 0.222 (L/B - 1)}$

■ خطوات الحل :

☒ $S_e = C_1 C_2 q_{net} \sum \frac{I_{zi} \Delta Z_i}{E_{si}}$

$\boxed{1}$ نحسب كل من C_1 ، C_2 ، q_{net} من القوانين .

$\boxed{2}$ بحسب ال E_s : لكل طبقة حسب نوع الاختبار المطلوب

$\boxed{3}$ برسم رسمة ال I_z و بحدد عليها فبتم I_z عن $z =$

$z_1 = \square$

$z_2 = \square$

$\boxed{4}$ بعمل جدول بحتوي على :

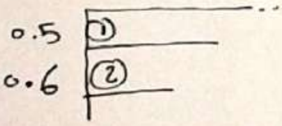
Layer no.	ΔZ(m)	E_s	I_z	$\frac{I_z \Delta Z}{E_s}$
1				⋮
2				
⋮				

$\boxed{5}$ بعوض بقانون S_e $\sum \square \Rightarrow$

و بوجد ال Settlement .

- ملاحظة مهمة : عند قراءة ال I_z من الرسمة لكل طبقة بأخذ ال Avg. للطبقة ، " يعني عند منتصف كل طبقة "

⟸ مثلاً :



Layer① $\Rightarrow 0.5/2 = 0.25$ m $\Rightarrow$ بوخذ قراءة I_z

layer② $\Rightarrow 0.5 + 0.6/2 = 0.8$ m

وهكذا

Sol : ** حل مثال الورقة الخارجية

* Find Immediate Settlement after 1 year from construction

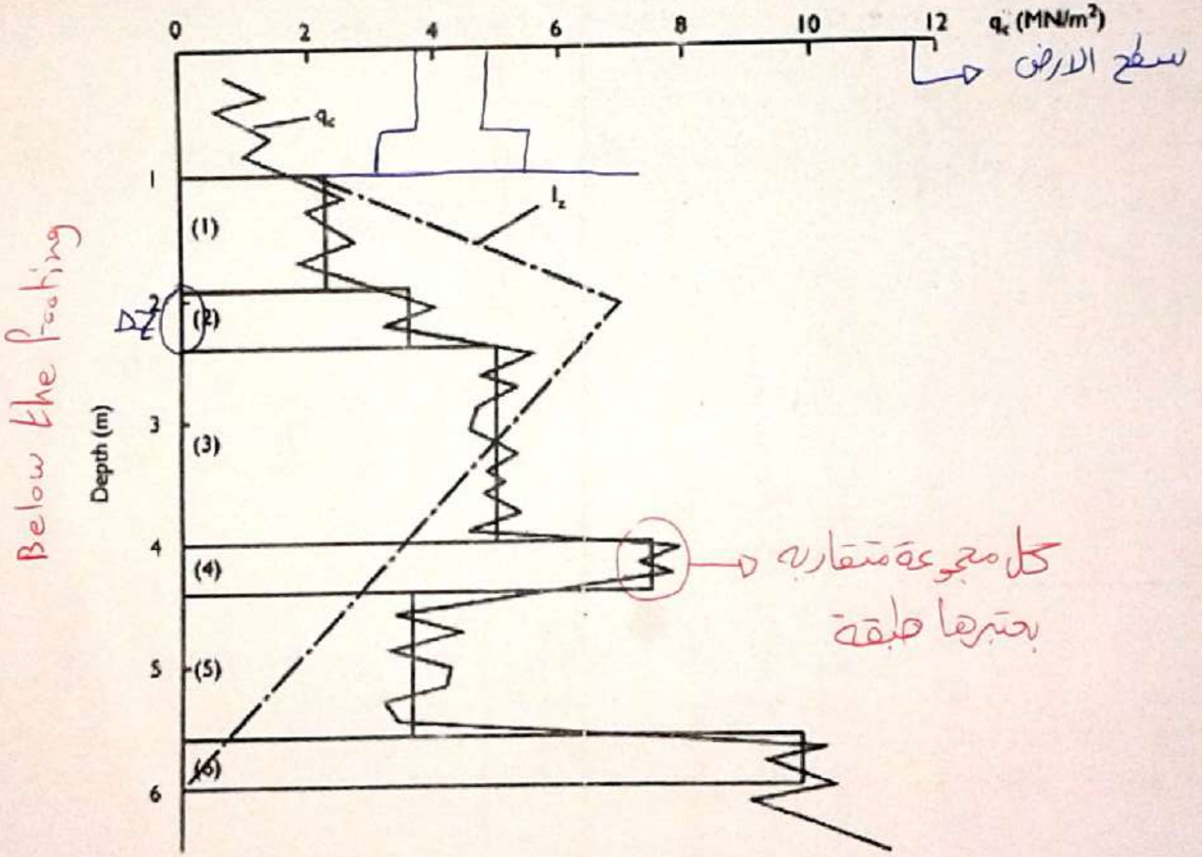
$C_1 = 1 - 0.5\left(\frac{q}{q_{net}}\right) = 1 - 0.5\left(\frac{1*17}{150}\right) = \boxed{0.94}$

$C_2 = 1 + 0.2 \, Log\left(t/0.1\right) = 1 + 0.2 \, Log\left(1/0.1\right) = \boxed{1.2}$

$q_{net} = \boxed{150 \text{ kpa}}$

Example

A footing 2.5×2.5m supports a net foundation pressure of 150kN/m^2 at a depth of 1.0m in a deep deposit of normally consolidated fine sand of unit weight 17kN/m^3.



$I_z \Rightarrow$ Square :

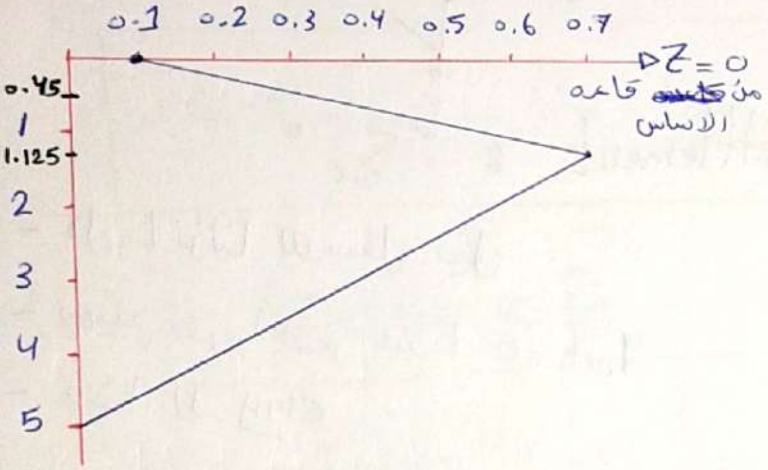
* $Z = 0 \Longrightarrow I_z = 0.1$

* $Z_1 = B/2 = 2.5/2 = \boxed{1.25\ m} \Rightarrow$ من قاعدة الاساس

$q_{z_1} = (17)(1 + 1.25) = \boxed{38.25\ Kpa}$

$\Rightarrow I_{z\,max} = 0.5 + 0.1\sqrt{\frac{q_{z_1}}{q_{net}}} = 0.5 + 0.1\sqrt{\frac{38.25}{150}} = 0.698$ ي $\boxed{0.7}$

* $Z_2 = 2B = 5m \Longrightarrow I_z = 0$



** لاحظ بالنسبه الـ square footing فقط يهمنا الحسابات لعمق 2B من اسفل الاساس اي اشي بعدها بهمله و ما بدخله بالحسابات .

* اما الـ wall footing لعمق 4B .

Layer no.	ΔZ (m)	q_c (mpa)	$E_s = 2.5 q_c$	I_z	$\frac{I_z \Delta Z}{E_s}$
1	0.9	2.4	6	0.3	0.045
2	0.45	3.8	9.5	0.6	0.022
3	1.65	5	12.5	0.58	0.0765
4	0.3	7.5	18.75	0.35	0.0056
5	1.25	3.8	9.5	0.22	0.088
6	0.1	10	25	0.04	2.4×10^{-4}
					Σ 0.237

$\therefore S_e = (0.94)(1.2)(150 \times 10^3)(0.237 \times 10^{-6}) = 0.0401\ m = \boxed{40.1\ mm}$

** الاعماق التي تم اخذ قراءات لل I_z عندها هي :

Layer 1 = $0.9/2 = 0.45$ m

Layer 2 = $0.9 + 0.45/2 = 1.125$ m

Layer 3 = $0.9 + 0.45 + 1.65/2 = 2.175$ m

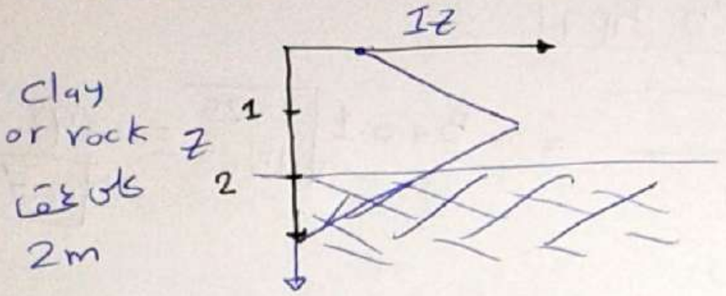
Layer 4 = 3.15 m

Layer 5 = 3.925 m

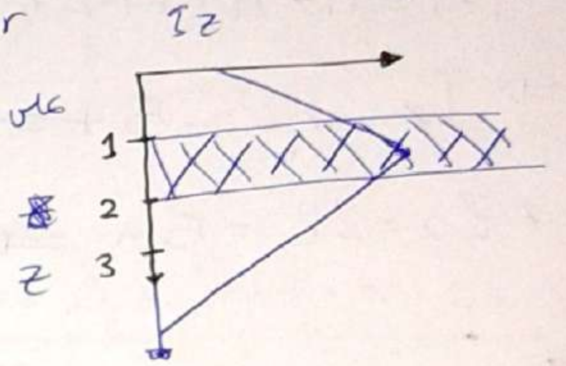
Layer 6 = 4.6 m

● ملاحظة هامة: في حال كان هناك تغير في الطبقات، مثلا كان هناك صخر او clay

⟸ نرسم الرسمة زي كأنها كلها sand بعدين نحذف الجزء اللي فيه clay or rock.



clay or rock على أعماق بين (1-2)m



● Primary Consolidation Settlement: "δ_c"

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

- كل ما نزلنا للاسفل يقل
- يصبح تقريباً صفر عندما $q_{net} = 0$
- فقط لل clay

σ'_o: Initial effective overburdend pressure at middle of Clay Layer. مهم

$$\Delta\sigma = \text{Induced stress}$$

$$\boxed{= \frac{1}{6}\left[\Delta\sigma_{\text{top of clay Layer}} + 4\,\Delta\sigma_{\text{mid of Clay Layer}} + \Delta\sigma_{\text{bottom of clay Layer}}\right]}$$

▨ Cases:

1 Normally Consolidated Clay: OCR = 1.0

$$\boxed{\delta_c = \frac{C_c}{1+e_o} H_o \operatorname{Log}\frac{\sigma'_f}{\sigma'_o}}$$

$$\boxed{OCR = \frac{\sigma'_p}{\sigma'_o}}$$

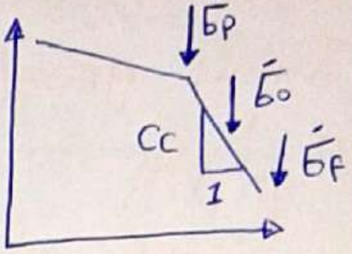
$$\boxed{\begin{array}{l} \sigma'_p < \sigma'_o < \sigma'_f \\ \text{او} \quad \sigma'_p = \sigma'_o < \sigma'_f \end{array}}$$

⟵ لما تكون $\sigma'_p < \sigma'_o$ ⟸ بعتبر انه σ'_p هو ال σ'_o.

H_o : thickness of clay Layer

σ_p' : preconsolidation Pressure "maximum past pressure"

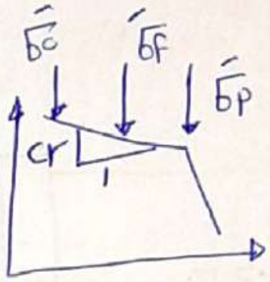
$$\bar{\sigma}_p < \bar{\sigma}_o < \bar{\sigma}_f$$



2 OCR > 1

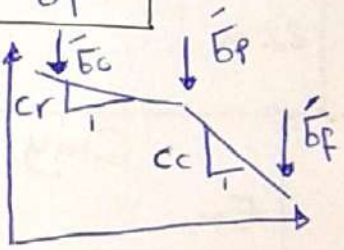
(a) $\bar{\sigma}_o < \bar{\sigma}_f < \bar{\sigma}_p$

$$\boxed{S_c = \frac{Cr}{1+e_o} H_o \operatorname{Log} \frac{\bar{\sigma}_f}{\bar{\sigma}_o}}$$

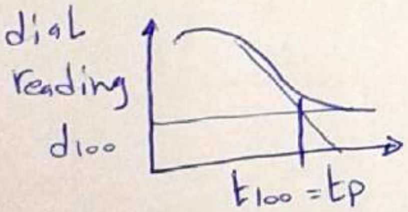


(b) $\bar{\sigma}_o < \bar{\sigma}_p < \bar{\sigma}_f$

$$\boxed{S_c = \frac{Cr}{1+e_o} H_o \operatorname{Log} \frac{\bar{\sigma}_p}{\bar{\sigma}_o} + \frac{Cc}{1+e_o} H_o \operatorname{Log} \frac{\bar{\sigma}_f}{\bar{\sigma}_p}}$$



Secondary Consolidation settlement : "S_s"



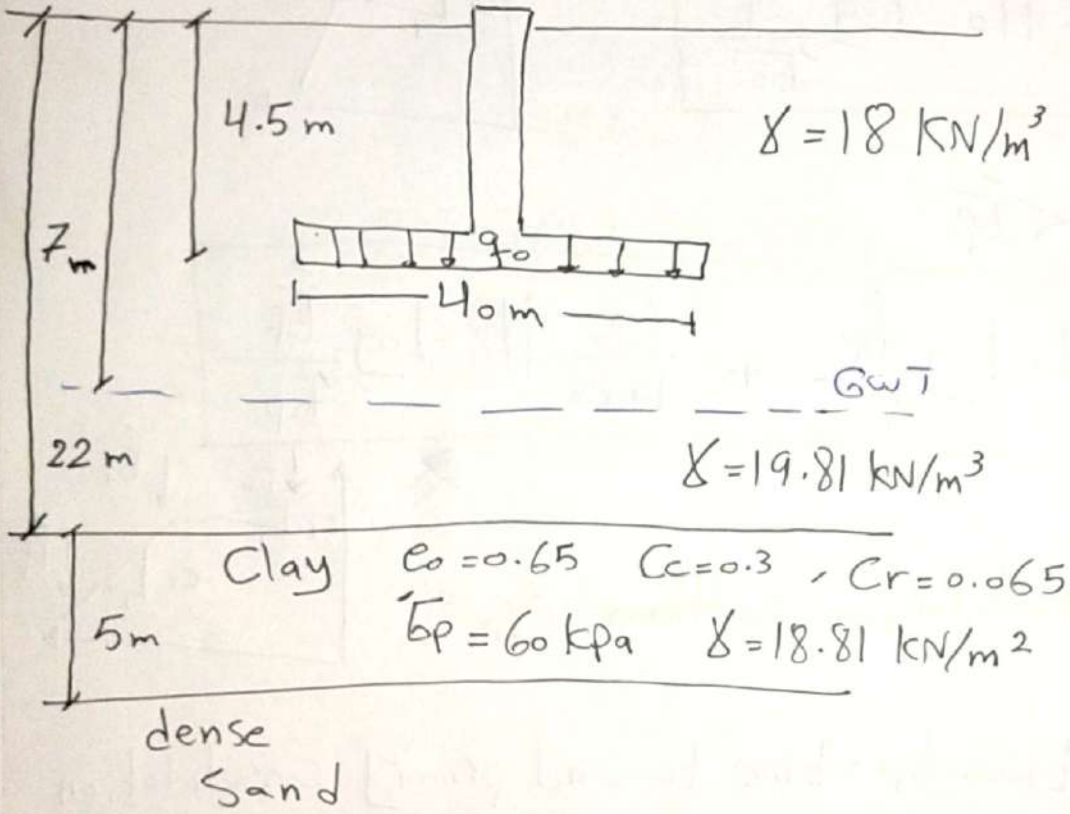
$t_{100} = t_p$ = time to end primary Consolidation

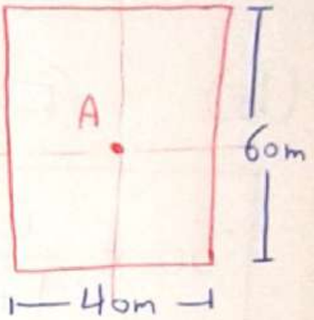
$$\boxed{\frac{C\alpha}{C\varepsilon} = 0.05 \pm 0.01}$$

$$\boxed{S_s = C\alpha\, H_o \operatorname{Log} (t/t_p)}$$

t = time after ending primary Consolidation

Ex : A Raft foundation (40 x 60) m Carring a net pressure of 145 kpa is Located at depth of 4.5 m below the surface in a deposits of dense Sandy gravel 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 inturn is underLine by dense Sand, determain the settlement below the Center of Raft ~~the~~ ?





Sol : Using 2:1 Approximate method : ** لإيجاد ال $\underline{\underline{\Delta\sigma}}$ ← increment stress

$$\bar{\sigma}_o = \Sigma \gamma' Z$$

$$= 18(7) + (19.81 - 9.81) * (22-7) + (18.81 - 9.81)\underline{(2.5)}$$

$$= \boxed{298.5\ kpa}$$

لو عند منتصف طبقة Clay

" middle "

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

$$\Delta\sigma = \frac{1}{6}\left[\Delta\sigma_{top} + 4\,\Delta\sigma_{mid} + \Delta\sigma_{bottom}\right]$$

$$\Delta\sigma_{top} = \frac{q_o\ B \times L}{(B+\underline{\underline{Z}})(L+Z)} = \frac{(145)(40)(60)}{(40+22-4.5)(60+22-4.5)} = \boxed{78.09\ \text{kpa}}$$

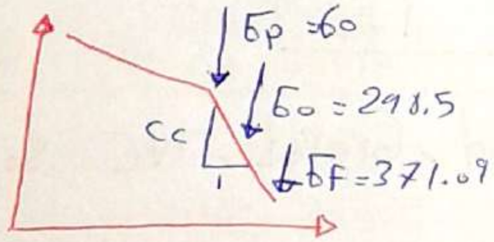
← من اسفل ال footing الى top طبقة ال clay

$$\Delta\sigma_{mid} = \frac{(145)(60)(40)}{(40+22-4.5+2.5)(60+22-4.5+2.5)} = \boxed{72.5\ \text{Kpa}}$$

$$\Delta\sigma_{bottom} = \frac{(145)(40)(60)}{(40+22-4.5+5)(60+22-4.5+5)} = \boxed{67.5\ \text{Kpa}}$$

$$\Delta\sigma = \frac{78.09 + 4(72.5) + 67.5}{6} = \boxed{72.59\ \text{kpa}}$$

$$\sigma'_f = \sigma'_o + \Delta\sigma = 298.5 + 72.59 = \boxed{371.09\ \text{kpa}}$$



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

(NC clay)

Case Ⓘ

$$S_c = \frac{C_c}{1+e_o}\,\underline{H_o}\,\text{Log}\,\frac{\sigma'_f}{\sigma'_o}$$

له عرض طبقة ال clay

$$= \frac{0.3}{1+0.65}(5)\,\text{Log}\left(\frac{371.09}{298.5}\right) = 0.0859\ \text{m} = \boxed{85.9\ \text{mm}}$$

\#

- لو طلب ال Settlement عند نقطة اخرى غير ال center نستخدم elastic method، $\Delta\sigma = q_o I_c$ "مادة سويل"

■ Lateral Earth pressure : CH 12

* Geotechnical application : retaining wall

** سوف نقوم بحساب (Lateral earth pressure and force) الذي يؤثر على ال wall حتى نضمن ان
يكون آمن ضد ⇓
excessive Lateral movement

[**] 3 Cases :

[1] Eearth pressure at rest

* no strain (no deformation)

Coefficient of Lateral earth pressure

$$\boxed{K = \frac{\sigma_h'}{\sigma_v'}}$$

horizontal

Vertical

* K ~~with~~ will be $\boxed{K_o}$ at rest

$$\boxed{K_o = \frac{\sigma_h'}{\sigma_v'}}$$

[*] K_o from elastic theory : $\boxed{K_o = \frac{\nu}{1-\nu}}$ ν : poissions ratio

[*] K_o for Granular Soil " Sand , Gravel , NC Soil "

$$\boxed{K_o = 1 - \sin\phi'} \Leftarrow \text{مهم}$$

** for undrainged Soil " $\phi = 0$ " $K_o = 1$

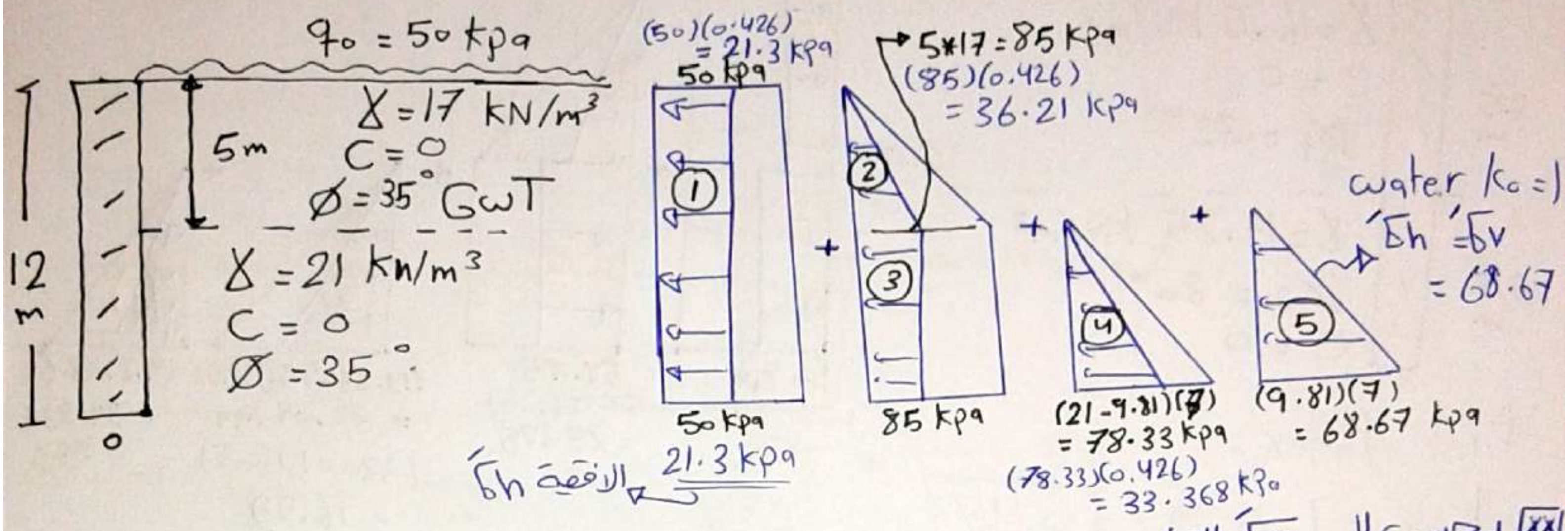
[*] For over Consolidated Soil :

$$\boxed{K_o = (1 - \sin\phi')(OCR)^{\sin\phi'}}$$

** $\boxed{K_o \text{ for water } = 1} \Leftarrow$ مهم

Ex : if the wall is prevented from movement ? → at rest

Determin the total force acting on the wall and Line of action of the total force?



XX بحسب ال σ'_v القادم من كل طبقة بعدين بحولها ال σ'_h عن طريقة k_o من ثم بحول ال σ'_h إلى Load $\underline{KN/m}$.

$\sigma'_v \rightarrow \sigma'_h$ use k_o →

$$k_o = 1 - \sin Ø' = 1 - \sin(35) = \boxed{0.426} \Rightarrow = \frac{\sigma'_h}{\sigma'_v}$$

** بقسم الاشكال فوق إلى Seg وكل Seg عبارة عن شكل بقدر احسب مساحته و ال center اله .

مساحة ال Seg

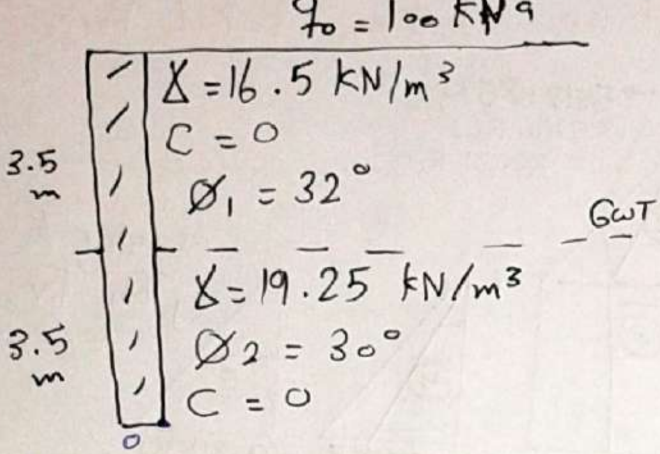
البعد من center seg للنقطة o

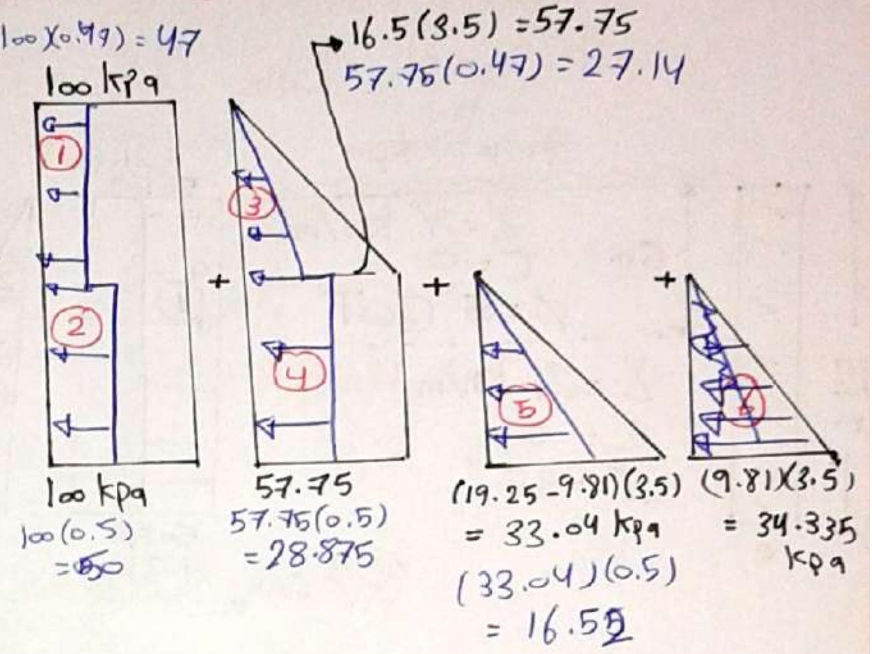
Seg #	phi/m "KN/m"	Arm(m) Xi	phi * Xi
1	21.3 * 12 = 255.6	12/2 = 6	1533.6
2	½ (36.21)(5) = 90.525	7 + 5/3 = 8.67	784.506
3	36.21 * 7 = 253.4	7/2 = 3.5	886.9
4	½ * 33.368 * 7 = 116.788	7/3 = 2.33	272.4
5	½ * 7 * 68.67 = 240.345	7/3 = 2.33	560.805
	Σ $\boxed{956.62 \text{ KN/m}}$ ①		Σ 4038.21

② $$\bar{X} = \frac{\Sigma p_i X_i}{\Sigma p_i} = \frac{4038.21}{956.62} = \boxed{4.22 \text{ m}}$$

Ex : Same the past example ,

الاختلاف $ حدوث تغير في الطبقات ، الزاويه ∅





first Layer :

$K_{o1} = 1 - \sin 32 = \boxed{0.47}$

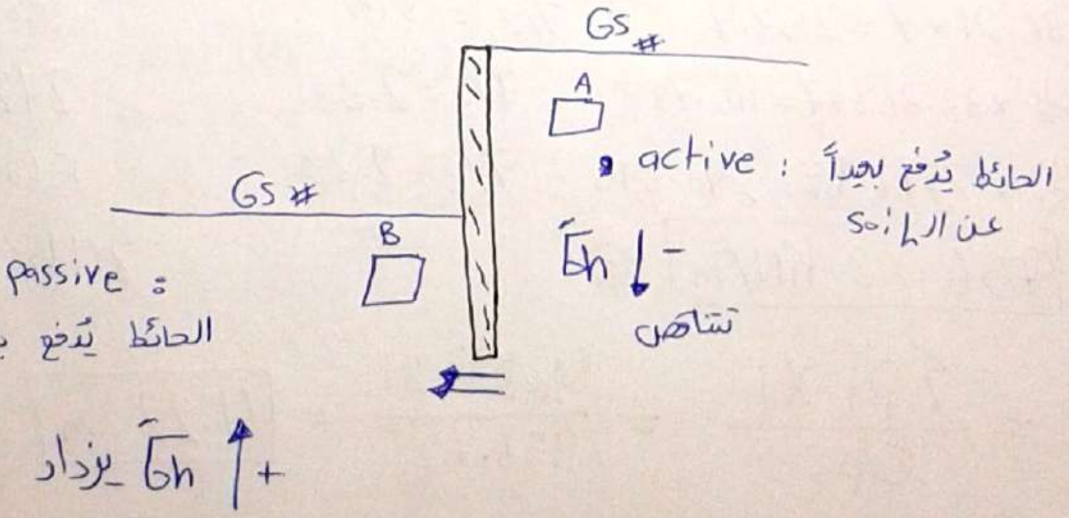
Second Layer

$K_{o2} = 1 - \sin 30 = \boxed{0.5}$

$\bar{X} = \dfrac{1672.48}{577.09} = \boxed{2.89\,m}$

seg #	Phi/m	arm Xi	Phi·Xi
1	47 * 3.5 = 164.5	5.25	863.625
2	50(3.5) = 175	1.75	306.25
3	0.5(27.14)(3.5) = 47.5	4.67	221.825
4	28.875(3.5) = 101.1	1.75	176.925
5	0.5(16.52)(3.5) = 28.91	1.167	33.74
6	0.5(34.335)(3.5) = 60.08	1.167	70.11
	Σ 577.09		Σ 1672.48

▨ Cases 2 & 3 :



• active : الحائط يدفع بعيداً عن الsoil

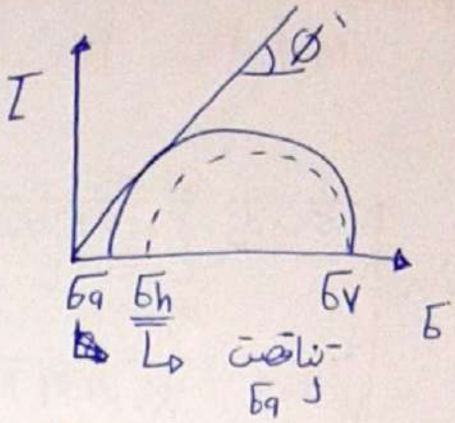
$\sigma_h \downarrow -$ تناقص

passive : الحائط يدفع باتجاه الsoil

يزداد $\sigma_h \uparrow +$

$\boxed{2}$ Active Case "A"

⊛ for granular soil "C = 0":



$$\sin\phi' = \frac{\sigma'_v - \sigma'_a}{\sigma'_v + \sigma'_a}$$

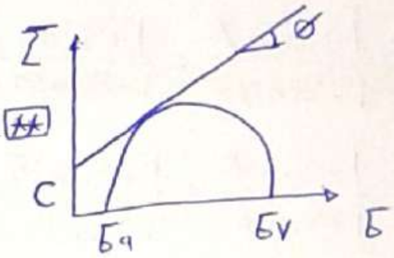
$$\boxed{\sigma'_a = \frac{1-\sin\phi'}{1+\sin\phi}\,\sigma'_v} \Rightarrow \boxed{\sigma_a = K_a\,\sigma_v}$$

↳ in case "active"

$$\boxed{K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right)}$$

⊛ for OC clay "C = □"

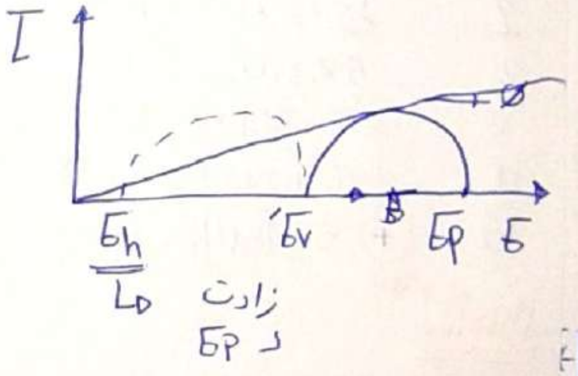
↖ Cohesion يوجد ⊠



$$\boxed{\sigma_a = K_a\sigma_v - 2C\sqrt{K_a}}$$

$\boxed{3}$ passive Case "B"

⊛ for granular soil "C = 0"



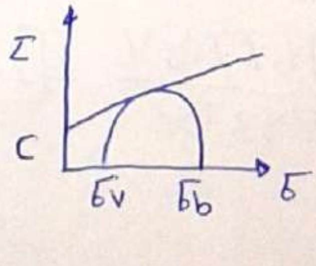
$$\sin\phi' = \frac{\sigma'_p - \sigma'_v}{\sigma'_p + \sigma'_v}$$

$$\boxed{\sigma_p = \frac{1+\sin\phi}{1-\sin\phi}\,\sigma'_v} \Rightarrow \boxed{\sigma'_p = K_p\,\sigma'_v}$$

↳ in case passive

$$\boxed{K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\phi/2)}$$

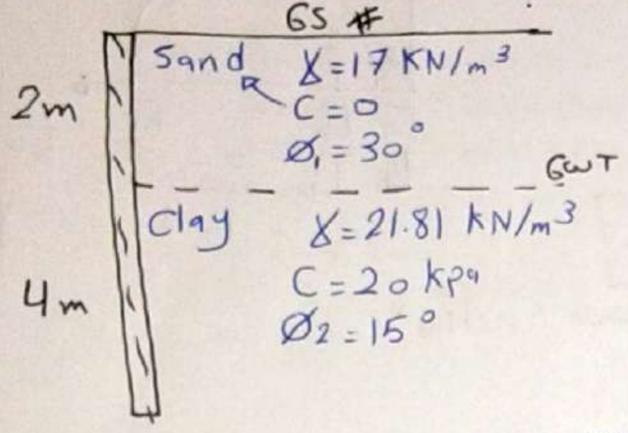
⊛ for OC clay "C = □"

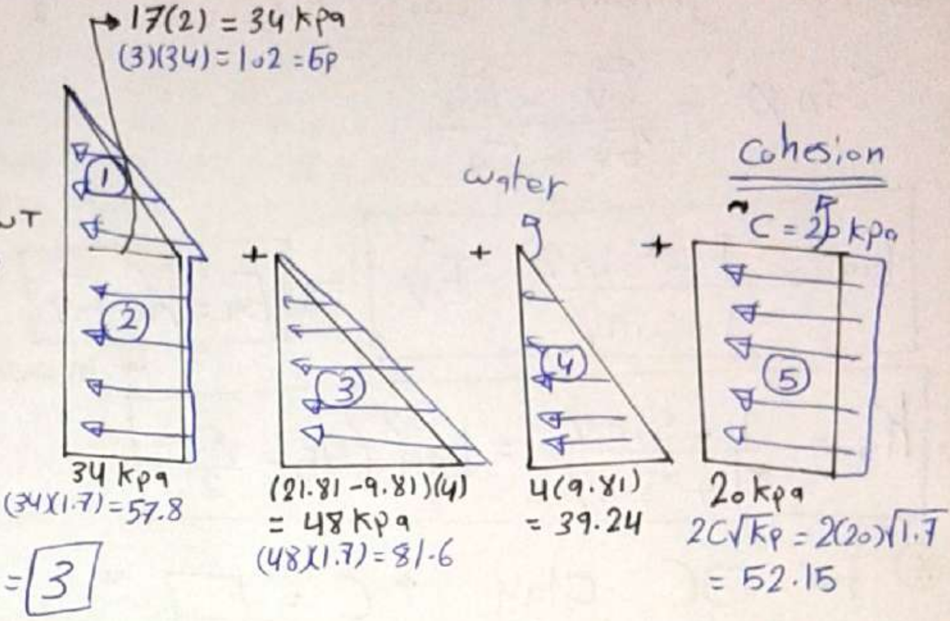


$$\boxed{\sigma_p = K_p\sigma_v + 2C\sqrt{K_p}}$$

$$\boxed{K_a < K_o < K_b} \quad , \quad \boxed{K_a = \frac{1}{K_p}}$$

Ex: Determine the total passive force/m of the wall and the Location of Line action of total force ?





$$Kp_1 = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \frac{1+\sin 30}{1-\sin 30} = \boxed{3}$$

$$Kp_2 = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \frac{1+\sin 15}{1-\sin 15} = \boxed{1.7}$$

$C = 20 \equiv \sigma'_v$

$2C\sqrt{Kp} \equiv \sigma'_h$

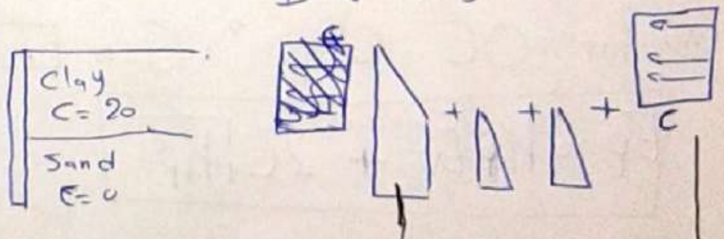
Seg #	Ppi/m "KN/m"	Xi (m)	Ppi * Xi
1	½ * 2 * 102 = 102	4.67	476
2	57.8(4) = 231.2	4/2	462.4
3	0.5 * 81.6 * 4 = 163.2	4/3	217.6
4	½ (39.24)(4) = 78.48	4/3	104.64
5	⊕ 52.15(4) = 208.6	4/2	417.2
			Σ 1677.84

لانه Passive

$Pp = \Sigma = 783.5\ KN/m$

$$\bar{X} = \frac{1677.84}{783.5} = \boxed{2.14\ m}$$

** بالنسبه لل Cohesion فهي خاصية متعلقة بكل طبقة فقط تكون بالطبقة الواحدة مثلاً



لاحظ فقط موجودة بالطبقة العليا لا تنتقل للاسفل على خلاف ال σ "stress" فهو ينتقل .

Example 7.4

Consider a rectangular foundation 2 m × 4 m in plan at a depth of 1.2 m in a sand deposit, as shown in Figure 7.11a. Given: $\gamma = 17.5 \text{ kN/m}^3$; $\bar{q} = 145 \text{ kN/m}^2$, and the following approximated variation of q_c with z:

t=10 year

z (m)	q_c (kN/m²)
0–0.5	2250
0.5–2.5	3430
2.5–6.0	2950

$$q_{net} = \bar{q} - \underset{\hookrightarrow D_f \gamma}{q}$$

Estimate the elastic settlement of the foundation using the strain influence factor method.

$$q_{net} = \bar{q} - q = 145 - 17.5(1.2) = \boxed{124 \text{ kN/m}^2}$$

$$S_e = C_1 C_2 \, q_{net} \sum_{i=1}^{i=n} \frac{I_{zi} \, \Delta z_i}{E_{si}}$$

$$C_1 = 1 - 0.5 \frac{q}{q_{net}} = 1 - 0.5\left(\frac{21}{124}\right) = \boxed{0.93}$$

$$C_2 = 1 + 0.2 \log t/0.1 = 1 + 0.2 \log 10/0.1 = \boxed{1.4}$$

$\boxed{**}$ معلومات لرسم رسمه ال I_z :-

$$Z = 0 \longrightarrow I_z = 0.1 + 0.0111\left(L/B - 1\right) = 0.1 + 0.0111\left(\tfrac{4}{2} - 1\right) = \boxed{0.1111}$$

$$\frac{Z_1}{B} = 0.5 + 0.0555\left(L/B - 1\right) \leq 1 \Rightarrow \frac{Z_1}{2} = 0.5 + 0.0555(2-1)$$

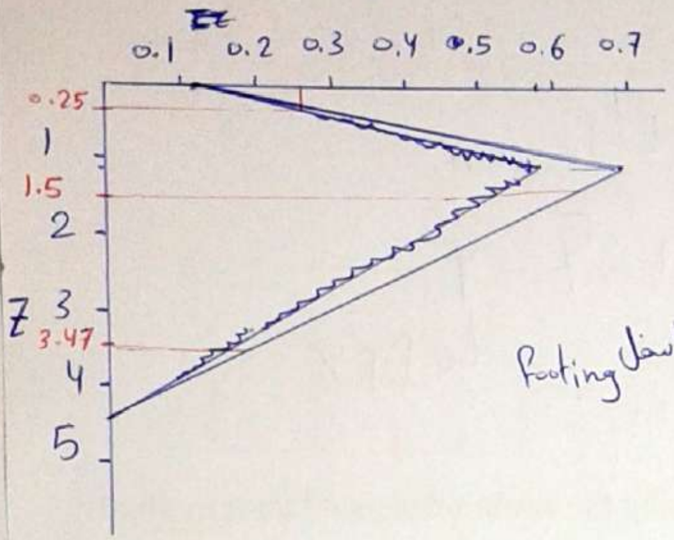
$$\Rightarrow \frac{Z_1}{2} = 0.5555 < 1 ✓ \Rightarrow \boxed{Z_1 = 1.11 \text{ m}}$$

$$q_{Z_1} = (\underset{\overset{\nwarrow}{D_f}}{\underline{1.2}} + 1.11)(17.5) = \boxed{40.425 \text{ kpa}}$$

$$I_{zmax} \, @ Z_1 = 0.5 + 0.1\sqrt{\frac{124}{40.425}} = \boxed{0.68}$$

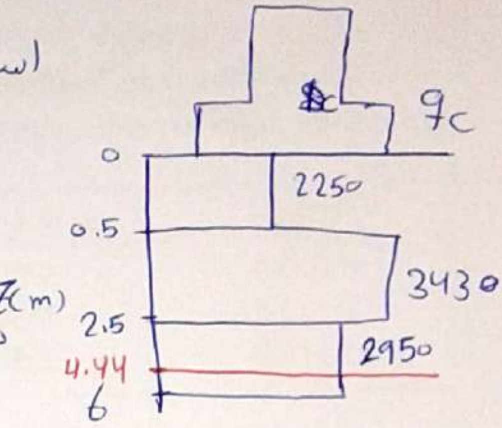
$I_z = 0$ @ $\frac{Z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right)$

$\Rightarrow \frac{Z_2}{2} = 2 + 0.222(2-1) \Rightarrow \boxed{Z_2 = 4.44 \text{ m}}$



Z=0 → اسفل

Z(m) معطي Z يعني من اسفل Footing



$E_s = \left(1 + 0.4 \text{ Log } \frac{L}{B}\right) E_{s94} = \left(1 + 0.4 \text{ Log } \frac{L}{B}\right)(2.5 q_c)$

$E_{s1} = \left(1 + 0.4 \text{ Log } \frac{4}{2}\right)(2.5 * 2250) = 6302.32$

$E_{s2} = \left(1 + 0.4 \text{ Log } \frac{4}{2}\right)(2.5 * 3430) = 9607.53$

$E_{s3} = \left(1 + 0.4 \text{ Log } \frac{4}{2}\right)(2.5 * 2950) = 8263.04$

Layer no	ΔZ	E_s	I_z	$\frac{I_z \Delta Z}{E_s}$
1	0.5	6302.32	0.26	$2.06 * 10^{-5}$
2	2	9607.53	0.64	$1.33 * 10^{-4}$
3	1.94	8263.04	0.195	$4.58 * 10^{-5}$
				$\Sigma\ 1.994 * 10^{-4}$

$S_e = 0.93(1.4)(124 * 10^3)(1.994 * 10^{-4}) = \boxed{32.19 \text{ mm}}$

7.10 A continuous foundation on a deposit of sand layer is shown in Figure P7.10 along with the variation of the cone penetration resistance q_c. Assuming $\gamma = 18\ \text{kN/m}^3$ and creep is at the end of ten years after construction, calculate the elastic settlement of the foundation using the strain influence factor method. Use Eqs. (7.20) and (7.26).

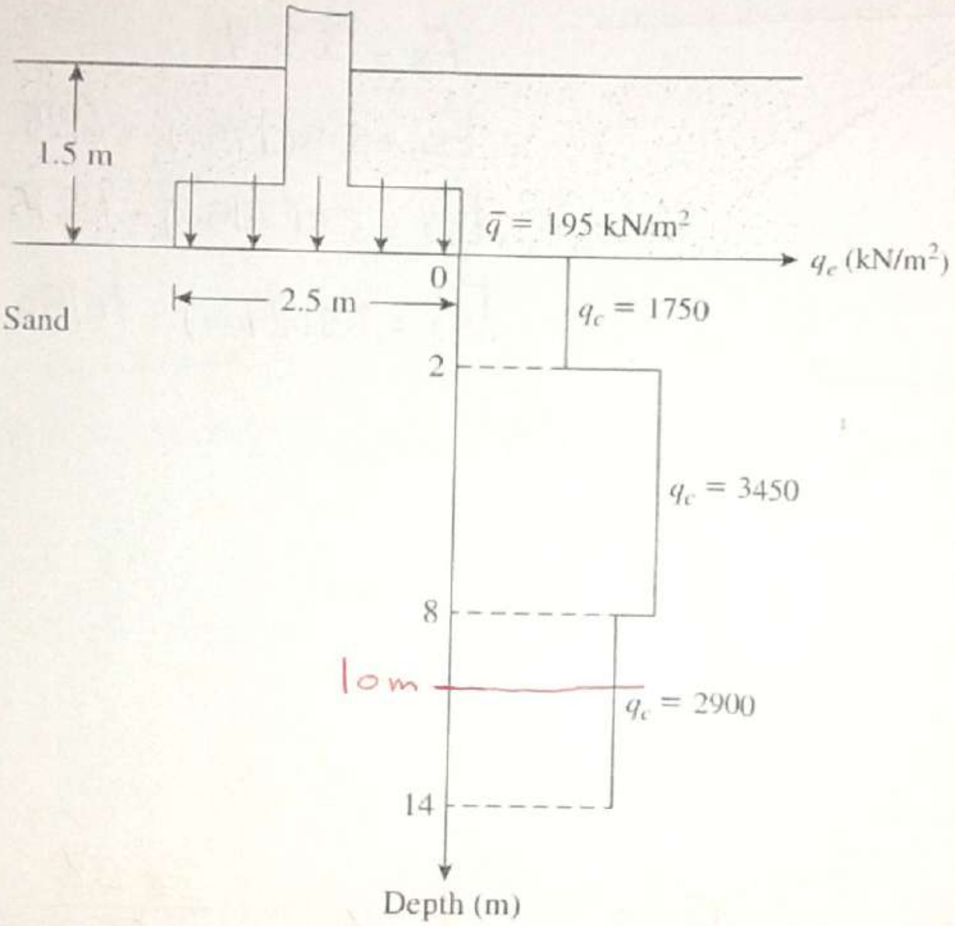


Figure P7.10

$q_{net} = \bar{q} - q = 195 - (18)(1.5) = \boxed{168\ \text{Kpa}}$

$C_1 = 1 - 0.5\left(\frac{27}{195}\right) = \boxed{0.93}$

$C_2 = 1 + 0.2 \log \frac{10}{0.1} = \boxed{1.4}$

$\boxed{**}$ معلومات لرسم رسمة I_z :-

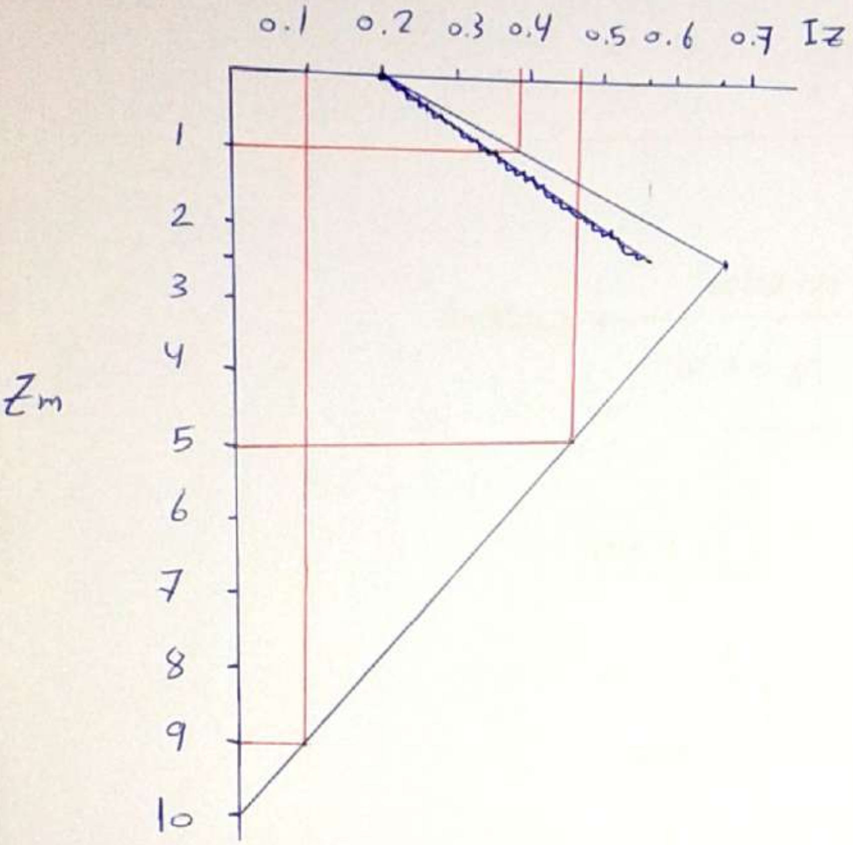
$Z = 0 \longrightarrow I_z = 0.2$

$Z_1 = \cancel{2B}\ B = 2.5\ m$

$q_{z_1} = (2.5+1.5)(18) = \boxed{72\ kpa}$

$I_{z\,max} = 0.5 + 0.1\sqrt{\frac{195}{72}} = \boxed{0.66}$

$Z_2 = 4B = 4(2.5) = \boxed{10\ m} \longrightarrow \boxed{I_{z2} = 0}$



$E_s = 3.5\ q_c$

$E_{s1} = 3.5(1750) = 6125$

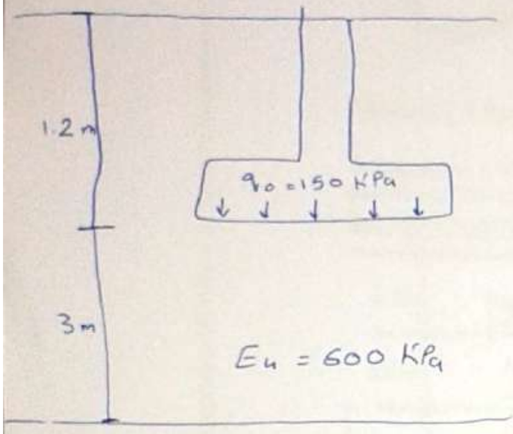
$E_{s2} = 3.5(3450) = 12075$

$E_{s3} = 3.5(2900) = 10150$

Layer no	ΔZ	E_s	~~Z~~·I_z	$\frac{I_z \Delta Z}{E_s}$
1	2	6125	0.39	$1.27 * 10^{-4}$
2	6	12075	0.47	$2.34 * 10^{-4}$
3	2	10150	0.1	$1.97 * 10^{-5}$
				$\Sigma\ 3.81 * 10^{-4}$

$S_e = 0.93(1.4)(168*10^3)(3.81*10^{-4}) = \boxed{83.34\ mm}$

7.1 Refer to Figure 7.1. A flexible foundation measuring 1.5 m × 3 m is supported a saturated clay. Given: D_f = 1.2 m, H = 3 m, E_s (clay) = 600 kN/m^2, and q_o 150 kN/m^2. Determine the average elastic settlement of the foundation. Use Eq. (7.



Sol.

$$S_d = A_1 A_2 \frac{q_o}{E_u} \times B$$

$$* \ A_1 \rightarrow \left(H/B = \frac{3}{1.5} = \underline{2} \ \& \ L/B = \frac{3}{1.5} = \underline{2} \right) \longrightarrow A_1 = \underline{0.65}$$

$$* \ A_2 \rightarrow \left(D_f/B = \frac{1.2}{1.5} = \underline{0.8} \right) \longrightarrow A_2 = \underline{0.935}$$

$$S_d = 0.65 \times 0.935 \times \frac{150 \times 10^3}{600 \times 10^3} \times 1.5 = 0.228 \text{ m} = \boxed{228 \text{ mm}}$$

[redacted] "40 min"

22.5

Student Name: [illegible] Student Number: 1438854
Student Section: 2 Student New Serial Number: 4

Hashemite University
Faculty of Engineering
Department of Civil Engineering
Foundation Engineering (0401435)
Instructor: Dr. Omar Al-Hattamleh

Second Exam
Wed: 30/11/2016
Attempts All Problems
Write Only in the designated space

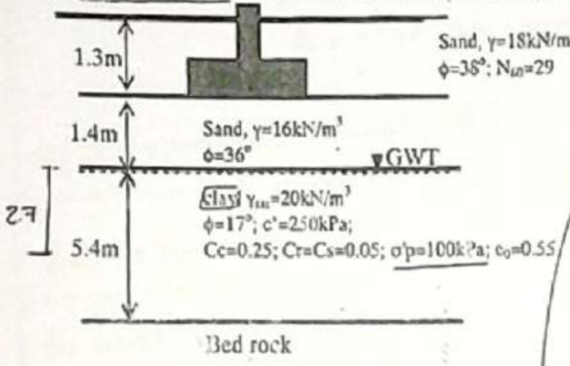
Note all calculation are made to nearest 2 significant digits

Problem # 1 (7.5 points)

A rectangular footing (1.6m x 2.0m) founded on soil profile shown. The footing was carried a net uniform pressure of 200 kPa. Determine, only, the primary consolidation settlement due to footing loading? Hint: Use 2:1 approximate method to find the induced (increment) of stress under the footing.

B L

$q_{net} = 200 KPa$



7.5/7.5

$\sigma'_f = \bar{\sigma}_o + \Delta\sigma$

$* \sigma'_o = 18\times1.3 + 16\times1.4 + 2.7\times(20-9.81) = \boxed{73.31\ KPa}$

$OCR = \frac{100}{73.1} > 1$

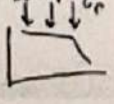
$* \Delta\sigma' \longrightarrow \quad \sigma_{Top} = \frac{q_o \times B \times L}{(B+z)(L+z)} = \frac{200\times1.6\times2}{(1.6+1.4)(2+1.4)} = \boxed{62.75\ KPa}$

$\sigma_{mid} = \frac{200\times1.6\times2}{(1.6+4.1)(2+4.1)} = \boxed{18.41\ KPa}$

$\sigma_{bot} = \frac{200\times1.6\times2}{(1.6+6.8)(2+6.8)} = \boxed{8.66\ KPa}$

$\Delta\sigma = \frac{1}{6}(8.66 + 4\times18.41 + 62.75) = \boxed{24.18\ KPa}$

$* \sigma_f = 73.31 + 24.18 = \boxed{97.5\ KPa}$



$* S_c = \frac{C_r}{1+e_o} \times H_o \log\frac{\sigma_f}{\sigma_o} = \frac{0.05}{1.55} \times 5.4 \times \log\frac{97.5}{73.31}$

$= 0.0216 m = \boxed{21.6}$

Student Name: ... ... علاء علي ... ... Student Number: ... ... 1438854 ... ...
Student Section: ... ... 2 ... ... Student New Serial Number: ... ... 4 ... ...

Problem # 2 (7.5 points)

q_{all} $\alpha = 10$

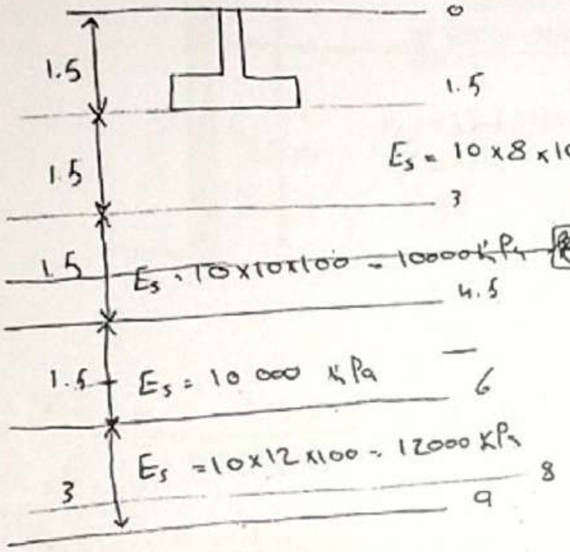
Consider a square foundation 2 m x 2 m in plan at a depth of 1.5 m in a clean sand deposit, given: γ=16kN/m³, gross allowable load 150 kPa. The value of N60 with depth given in the table below:

Depth (m)	0.0-1.50	1.5-3.0	3.0-4.50	4.5-6.0	6.0-9.0	9.00	
N_{60}	6	8	10	10	12	50	

Estimate the elastic settlement of the foundation using the strain influence factor method after 5 years?

$4B = 8m$

$E_s = \alpha \times N60 \times P_a$

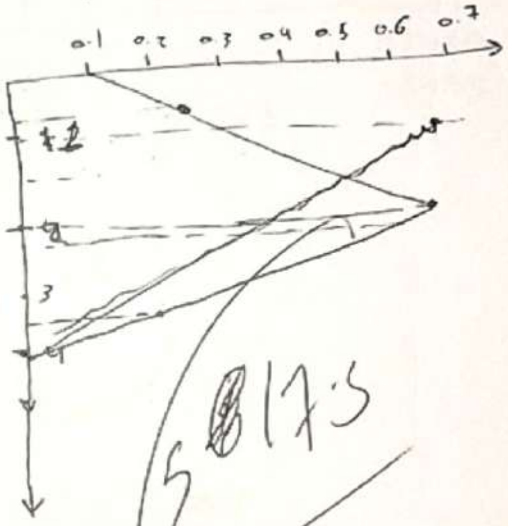


$E_s = 10 \times 8 \times 100 = \underline{8000 kPa}$

~~$E_s = 10 \times 10 \times 100 = 10000 kPa$~~

$E_s = 10\,000$ kPa

$E_s = 10 \times 12 \times 100 = 12000$ kPa



5 17.5

$$c_1 = 1 - 0.5 \times \frac{1.5 \times 16}{150} = 0.92$$

$$c_2 = 1 + 0.2 \log \frac{5}{0.1} = \boxed{1.34}$$

$0 = z_0 \rightarrow I_z = 0.1$

seg	Δz	Es	Iz	$\frac{\Delta z I_z}{E_s}$
1	1.5	8000	0.25	4.69×10^{-3}
2	1.5	10000	0.6	9×10^{-5}
3	1	10000	0.2	2×10^{-5}
				15.7×10^{-5}

$$z_1 = 2 \rightarrow I_z = 0.5 + 0.1\sqrt{\frac{150}{0.2 \times (2+1.5) 16}} = 0.66$$

$z_2 = 4$

2/3

$$S_e = 0.92 \times 1.34 \times 150 \times 10^3 \times 15.7 \times 10^{-5} \times 10^{-3} = 0.029$$

$$= \boxed{29 mm}$$

Student Name: [illegible] Student Number: 1438854

Student Section: 7 Student New Serial Number: 4

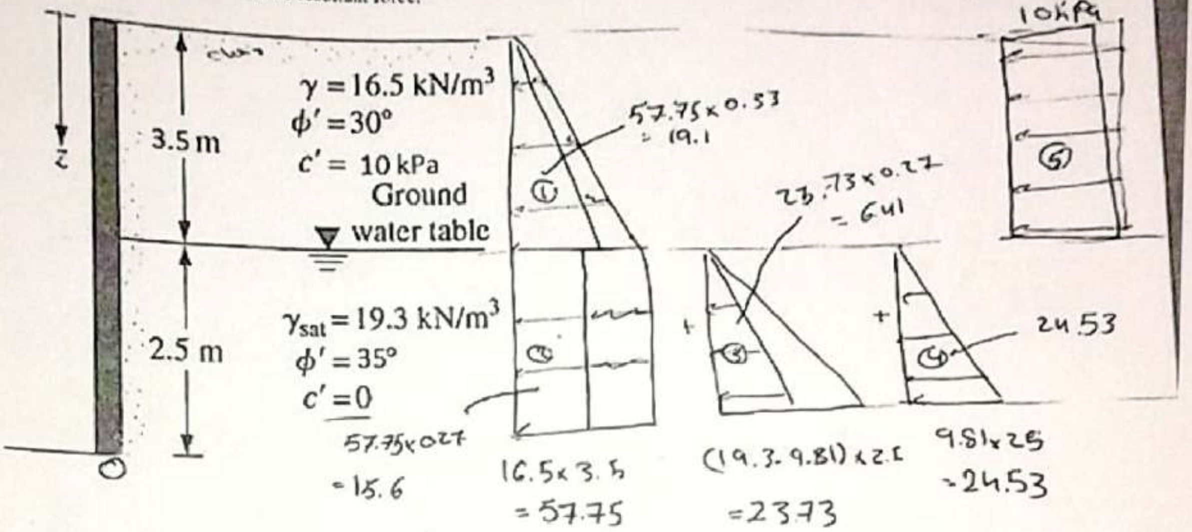
Problem # 3 (10 points)

For the retaining wall shown in Figure below, determine the lateral earth force in active state per unit length of the wall. Also determine the location of the resultant force.

$= 11.5$

$- 2 \times 10 \times \sqrt{0.33}$

10kPa



$$K_{a_1} = \frac{1 - \sin 30}{1 + \sin 30} = 0.33$$

$$K_{a_2} = \frac{1 - \sin 35}{1 + \sin 35} = 0.27$$

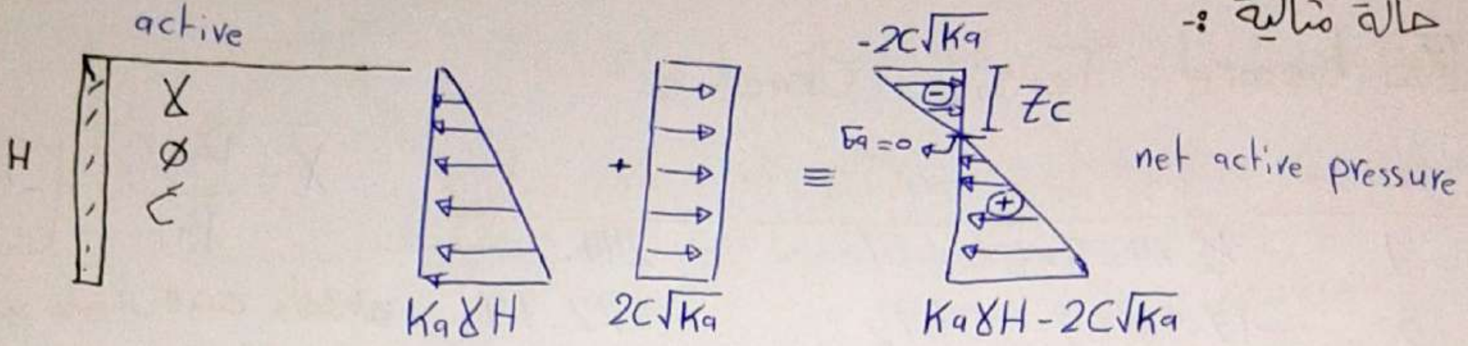
10

No	P_i (kPa)	X_i	$P_i \times X_i$
1	19.1×3.5×0.5 = 33.43	$2.5 + \frac{3.5}{3}$	122.58
2	15.6×2.5 = 39	1.25	48.75
3	0.5×6.41×2.5 = 8.1	2.5/3	6.75
4	24.53×0.5×2.5 = 30.66	2.5/3	25.55
5	− 11.5×3.5 = 40.25	2.5+1.75	− 171.1
Σ	$\boxed{70.94\ \text{KPa}}$		32.48

$$\bar{X} = \frac{32.48}{70.94} = \boxed{0.45\ \text{m}}$$

- Tensile Cracks occur in active case only :

حالة مثالية :-



Zc : depth of tensile Crack :

Z_c @ $\sigma_a = 0$

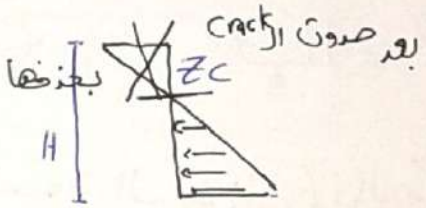
$0 = K_a \gamma' H - 2C\sqrt{K_a}$

$0 = K_a \gamma' Z_c - 2C\sqrt{K_a}$ $\Longrightarrow$ $Z_c = \dfrac{2C\sqrt{K_a}}{\gamma' k_a} = \boxed{\dfrac{2C}{\gamma' \sqrt{K_a}}}$

⊛ Horizontal force Pa/m before Tensile Crack :

$$\boxed{P_a = \tfrac{1}{2}(\gamma' K_a H^2) - 2C\sqrt{K_a}\, H}$$

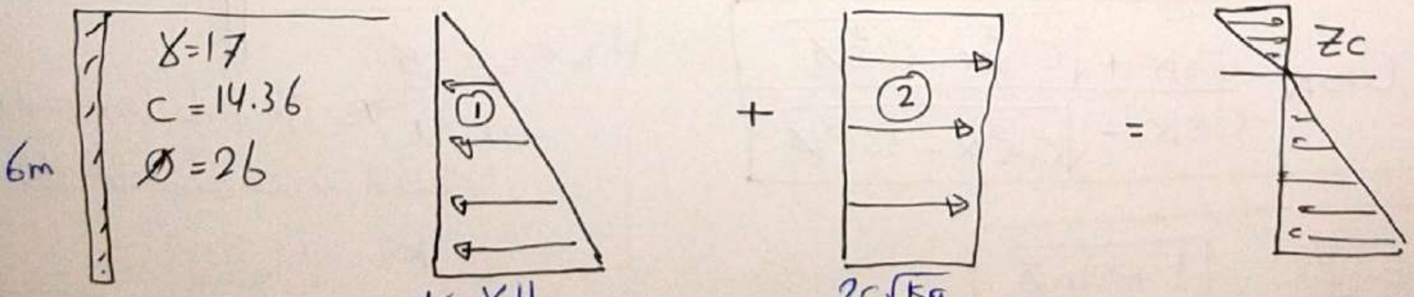
⊛ after tensile cracks occur : ~~m~~



$$\boxed{P_a = \tfrac{1}{2}(K_a \gamma' H - 2C\sqrt{K_a})(H - Z_c)}$$

$$\boxed{\bar{X} = \frac{H - Z_c}{3}}$$

Ex: A 6m high retaining wall is to support a soil with a unit weight $\gamma = 17\ kN/m^3$, $\phi = 26°$, $C = 14.36$ kpa, ~~Determ~~ Determine retaning active force per unit Length of wall befor and after tensile cracks occure, and the Line of action of the resultent in both cases :



$K_a \gamma H$
$= (0.39)(17)(6)$
$= 40.71$ kPa

$2C\sqrt{K_a}$
$= 2(14.36)(\sqrt{0.39})$
$= 17.93$ kPa

$$* K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 26}{1+\sin 26} = \boxed{0.39}$$

** before Tensile Crack :

Seg #	Pai/m	X_i	$P_{ai} * X_i$
1	$\frac{1}{2} * 40.71 * 6$	6/3	244.26
2	$-17.93(6)$	6/2	-322.74
	Σ 14.55 kN/m		$\Sigma = -78.48$

$$\bar{X} = \frac{P_{ai} X_i}{P_{ai}} = \boxed{-5.39 \text{ m}}$$

لأن اللود على الـ الوس والمحصلة اسفل الجدار بسبب الـ Cohesion.

** after Tensile crack :

$$Z_c = \frac{2C}{\gamma\sqrt{K_a}} = \frac{2 * 14.36}{17.4 * \sqrt{0.39}} = \boxed{2.64 \text{ m}}$$

$$P_a = \frac{1}{2}(H - Z_c)(\gamma K_a H - 2C\sqrt{K_a}) = \frac{1}{2}(6-2.64)((17.4)(0.39)(6) - 2(14.36)\sqrt{0.39}$$

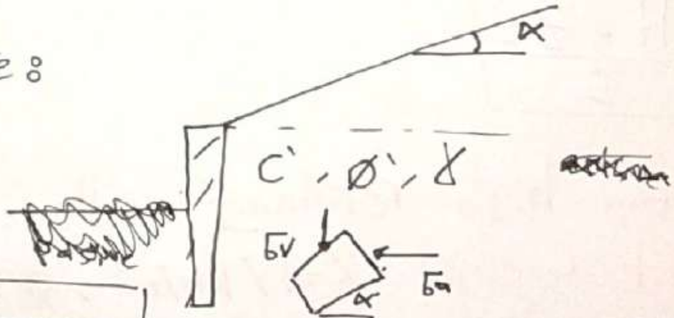
$$= \boxed{38.27 \text{ kN/m}}$$

$$\bar{X} = \frac{H - Z_c}{3} = \frac{6-2.64}{3} = \boxed{1.12 \text{ m}}$$

- في حال كان هناك ميلان في الـ soil في منطقة الـ active (or passive) بزاوية "α"

1 Rankine earth pressure :

$$\sigma_a = K_a \sigma_v \quad , \sigma_p = K_p \sigma_v$$

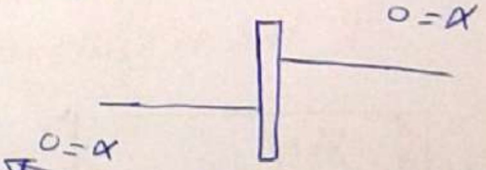


$$\boxed{K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}}$$

$$\boxed{K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}}$$

$$\boxed{Z_c = \frac{2C}{\gamma}\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}}$$

** في الامثلة السابقة كان السويل افقي يعني

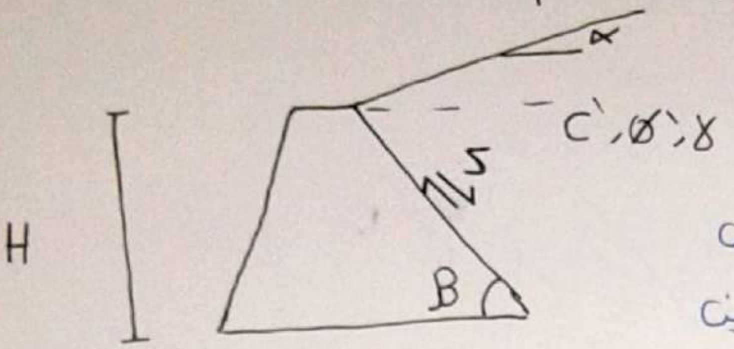


ولو تعوضها بالقوانين راح نجد انه نفس القانون السابق

$$k_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$k_b = \frac{1+\sin\phi}{1-\sin\phi}$$

2 Colombs earth pressure Coefficients :



δ : Interface friction angle

*نلاحظ ان Rankine لم يهتم بشكل ال wall
او زاويه ميلانه ، بينما Colombs اخذها بعين
الاعتبار هي و زاوية الاحتكاك .

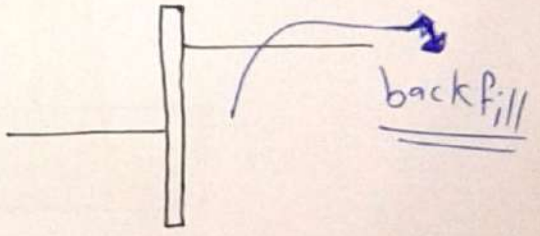
$$K_a = \frac{\sin^2(\beta+\varnothing)}{\sin^2\beta \; \sin(\beta-\delta)\left[1+\sqrt{\dfrac{\sin(\varnothing+\delta)\sin(\varnothing-\alpha)}{\sin(\beta-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$K_p = \frac{\sin^2(\beta-\varnothing)}{\sin^2\beta \; \sin(\beta+\delta)\left[1+\sqrt{\dfrac{\sin(\varnothing+\delta)\sin(\varnothing+\alpha)}{\sin(\beta+\delta)\sin(\alpha+\beta)}}\right]^2}$$

Ex : Compare Rankine & ~~Cot~~ Colomb Lateral earth pressure Coefficient for a wall retains a granular backfill Soil with ($\varnothing' = 35°$, $\delta = 15°$, $\alpha = 20°$, $\beta = 90°$)

** Rankine : $K_a = 0.322$

$K_p = 2.74$



* Colomb : $K_a = 0.323$

$K_p = 24.46$

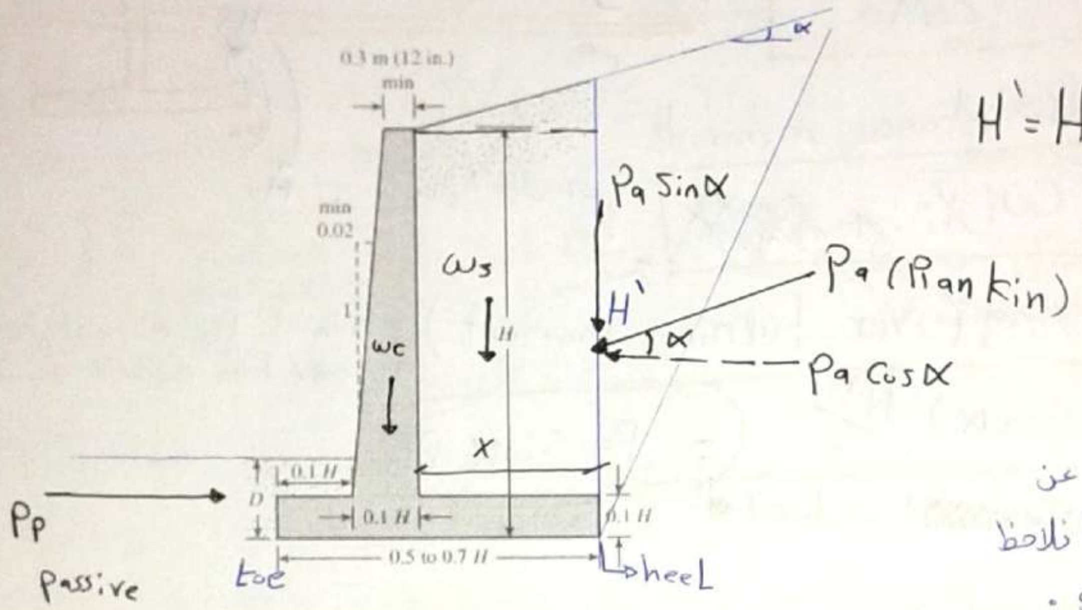
⇒ Colomb أدق

▨ Lateral earth pressure application :

** : Conventional retaining walls (RW) :

1 Cantilever R.w "reinforced concrete"

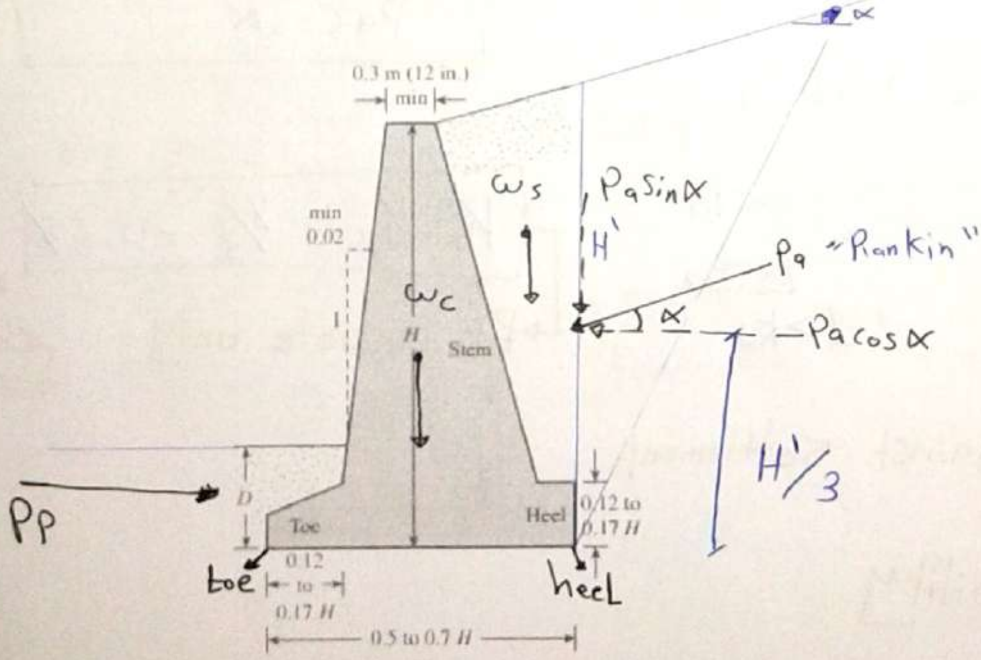
$$H' = H + X \tan \alpha$$



** هذه الامثلة عبارة عن
Rankine Theory لان ال Pa نلاحظ
انها على ال Soil .
توثر ما

2 Gravity wall "plain concrete only"

** بالنسبه لل Colomb Theory فتكون
Pa موثره مباشرة على
الحائط (سوف ندرسها
لاحقاً)

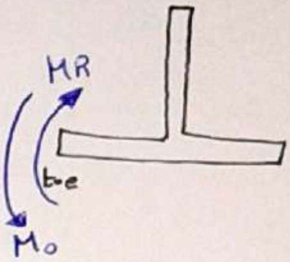


- The R.w must satisfied the following stability checks :

1. Safe against over turning about the toe (point o) :

$$\boxed{F.S_{ot} = \frac{\sum M_R}{\sum M_o}} \geq 3$$

مهم



* $\sum M_R$: all Resistance moment

$$= \boxed{\sum w_i \bar{X_i} + [illegible]}$$

نهمل ال P_P

* $\sum M_o$: Driving (over turning moment) : sum of the moment tend to over turn the wall around toe

$$= \boxed{(P_a \cos\alpha)\frac{H'}{3} - P_a \sin\alpha \, B}$$

[illegible]" اختياري

2. Safe against sliding aLong the base "friction, adhesion, passive force"

$$F.S_{sliding} = \frac{\text{Resistance force}}{\text{driving force}} = \boxed{\frac{\sum V \tan\delta + B C_a + P_p}{P_a \cos\alpha}} \geq 1.5$$

مهم

* $\sum V \rightarrow$ sum of all vertical force (w)

$\delta = K_1 \phi_2$

$C_a = K_2 C_2$

$o < k_1$

$1 > k_2$

$\boxed{K_1 = K_2 = 2/3 = 0.67}$ → for concrete only

مهم

4. footing safe against settlement

5. over all stability

3 Footing safe against Bearing Capacity failure :

$$\boxed{e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V}} \leq B/6$$

$$\boxed{F.S_{BC} = \frac{q_{ult}}{q_{mat}}} \geq 3$$

$$\boxed{q_{toe} = q_{max} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right)}$$

$$\boxed{q_{heel} = q_{min} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) \geq 0}$$

q_{ult} : تعطى بالسؤال
او تحسب كمادة الفيرست
for eccentricity footing.

** If sliding not satify :

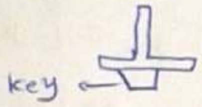
* اخطر مشكل و يمكن حلها
من خلال :

(1) use stronger back fill soil : $\phi \uparrow_+$ $K_a \downarrow^-$

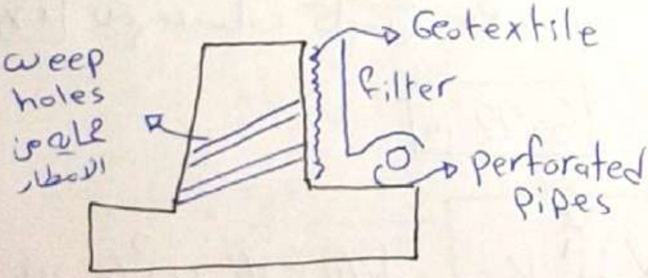
(2) Install Tie back anchor.

(3) extend the heel

(4) provide key : increase pp by increas Df $\Rightarrow (D_f + D_{key})$



** Backfill drainage :



* Two conditions must met :

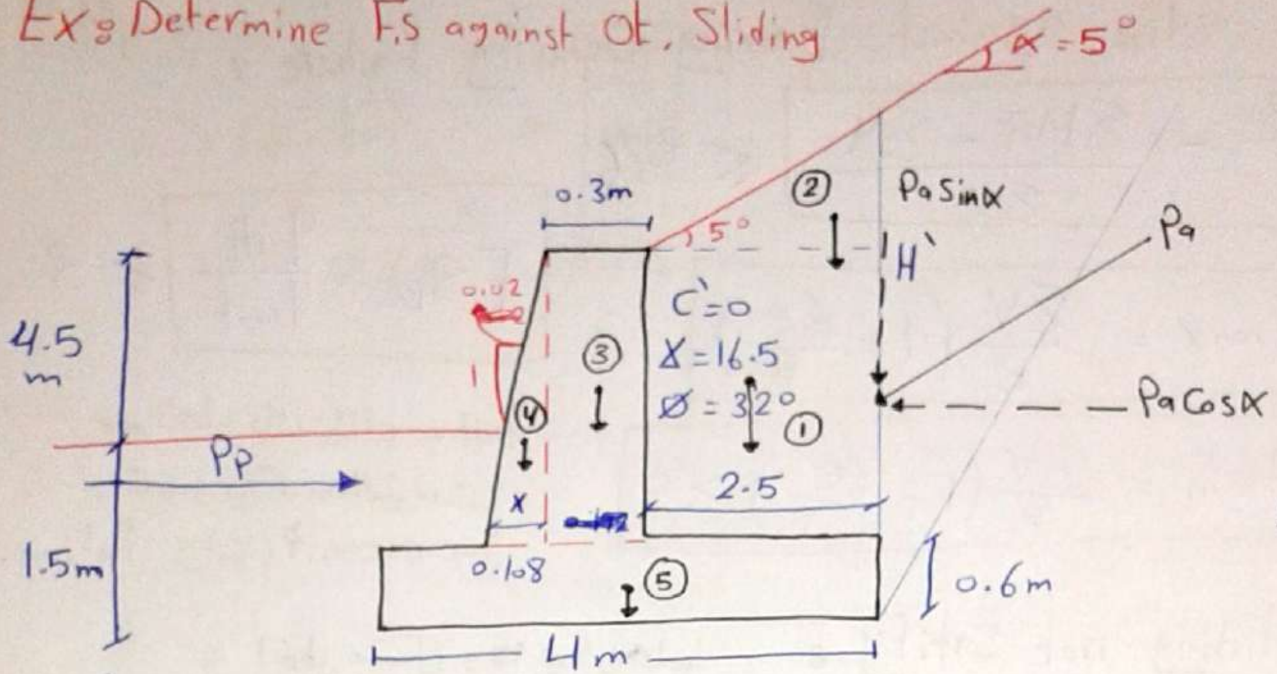
$$\frac{D_{15}\ \text{filter}}{D_{85}\ \text{backfill}} < 5$$ حتى لا تدخل اجزاء ال backfill الى ال filter.

$$\frac{D_{15}\ \text{filter}}{D_{15}\ \text{backfill}} > 5$$ يمنع تشكل ال U
↳ pore water pressure

D_{15} → 15% passing
↳ Diameter

Ex: Determine F.S against O.t, Sliding

$\gamma_{concrete} = 24\ kN/m^3$



$$\frac{0.02}{1} = \frac{x}{6-0.6}$$

$$\boxed{x = 0.108m}$$

$C' = 40\ kpa$, $\gamma = 18\ kN/m^3$
$\text{Ø} = 22°$

Sol: $K_{a1} = \cos\alpha \dfrac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\text{Ø}}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\text{Ø}}} = \cos 5 \dfrac{\cos 5 - \sqrt{\cos^2 5 - \cos^2 32}}{\cos 5 + \sqrt{\cos^2 5 - \cos^2 32}}$

$= \boxed{0.31}$

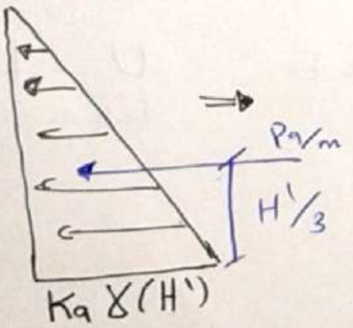
$K_{p2} = \dfrac{1 + \sin\text{Ø}}{1 - \sin\text{Ø}} = \dfrac{1 + \sin 22}{1 - \sin 22} = \boxed{2.19} \Rightarrow$ لان ال Soil مش مايل افقي


$\rightarrow \alpha = 0°$

⊠ يتم حساب كل من Pa ، Pp مثل السابق :-

$H' = H + 2.5\tan\alpha = 6 + 2.5\tan 5 = \boxed{6.22\ m}$

- Pa :



H' = 6.22

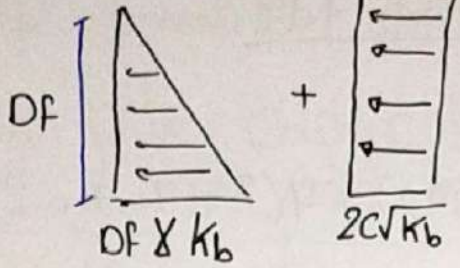
$Pa/m = \boxed{\frac{1}{2}\gamma H'^2 K_a} = \frac{1}{2}(16.5)(6.22)^2(0.31)$

$= \boxed{98.94\ KN/m}$

* $P_{ah} = P_a \cos\alpha = 98.94 \cos 5 = \boxed{98.56\ KN/m}$

* $P_{av} = P_a \sin\alpha = 98.94 \sin 5 = \boxed{8.62\ KN/m}$

- P_p :



$$\Rightarrow \boxed{P_p/m = \tfrac{1}{2}\gamma D_f^2 K_p + 2C\sqrt{K_b}\,D_f}$$

$$= 0.5(18)(1.5)^2(2.19) + 2(40)(\sqrt{2.19})(1.5)$$

$$= \boxed{221.93\ KN/m}$$

Seg#	A_i	V_i (KN/m)	X_i	M_{Fi}
1	$(2.5)(6-0.6) = 13.5$	$(13.5)(16.5) = 227.75$	$4-\frac{2.5}{2} = 2.75$	626.31
2	$\frac{1}{2}(2.5)^2 \tan 5 = 0.27$	$0.27(16.5) = 4.455$	$4-\frac{2.5}{3} = 3.167$	14.11
3	$0.3(6-0.6) = 1.62$	$1.62(\overset{\gamma_{concrete}}{\underline{24}}) = 38.88$	$4-2.5-\frac{0.3}{2} = 1.35$	52.488
4	$\frac{1}{2}(0.108)(6-0.6) = 0.2916$	$0.2916(24) = 6.998$	$4-2.5-0.3-\frac{0.108}{3} = 1.164$	8.145
5	$4 \times 0.6 = 2.4$	$2.4(24) = 57.6$	$4/2 = 2$	115.2
6	$P_{aV} =$	8.62	4	34.48

$\Sigma V = \cancel{2}\ 344.303$

ΣM_R with P_{aV} $= 850.733\,(KN/m)\cdot m$

ΣM_R without P_{aV} $= 816.253\,(KN/m)\cdot m$

X_i : المسافة الافقية بين PV
وال toe

* $F.S_{ot} = \dfrac{\Sigma M_R}{\Sigma M_{ot}}$

$\Sigma M_o = (P_{ah})\,H'/3 = 98.56\left(\frac{6.22}{3}\right) = 204.34\ kN.m/m$

$F.S_{ot} = \dfrac{850.733}{204.34} = 4.163 > 3$ ok ~~without~~ $\Rightarrow$ حل على هاض

$F.S_{ot} = \dfrac{816.25}{204.34 - \underline{\underline{34.48}}}$ "without Pav" $= 4.805 > 3$ ok"

$P_a \sin\alpha\, B$ ملاحظة

** معظم الاسئلة كان الدكتور يهمل ال PP بال M_R . في حال لم تهمل

$\Sigma M_R = \Sigma w_i X_i + P_p \bar{X}$

**

$$* \ F.S_{sliding} = \frac{\Sigma V \tan\delta + B\,C_a + P_P}{P_{ah}}$$

$$= \frac{344.303 \tan(\frac{2}{3}(22)) + 4(\frac{2}{3}(40)) + 221.93}{98.94}$$

$$= 4.23 > 1.5 \text{ OK}$$

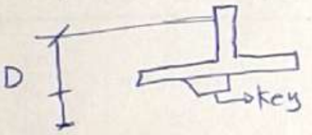
[**] If GWT Rise To the Surface :

$$P_{ah} = P_{ah_1} + P_{a\,hydrostatic}$$

$$= 98.94 + \tfrac{1}{2}(\gamma_w)(H_w)^2 = 98.94 + \tfrac{1}{2}(9.81)(6)^2 = \boxed{275.52 \text{ KN/m}}$$

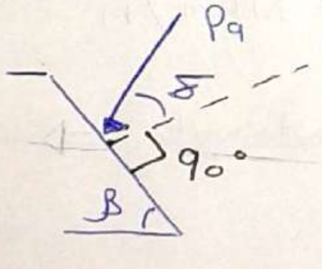
$$F.S_{sliding_{new}} = \frac{344.303 \tan(\frac{2}{3}(22)) + 4(\frac{2}{3}(40)) + 221.93}{275.52} = 1.519 > 1.5 \checkmark$$

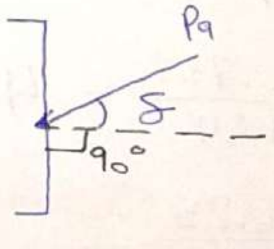
■ في حال كانت F.S اقل من 1.5 ، نفرضها انها 1.5 ونحسب من خلالها Pp ومن
قانون ال Pp نحسب ال D ، ثم اطرح ال D الاصلية من ال D المحسوبة
والفرق سوف يكون طول ال Key كحل للـ sliding.



[**] for Colomb Theory :

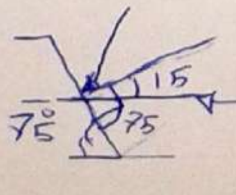
القوة مؤثرة مباشرة على الحائط





[**] يوجد مثال عليها بالكتاب Ex 13.2 صفحه 669 *.

* اهم اشي عند تحليل Pah في حال الجدار المائل

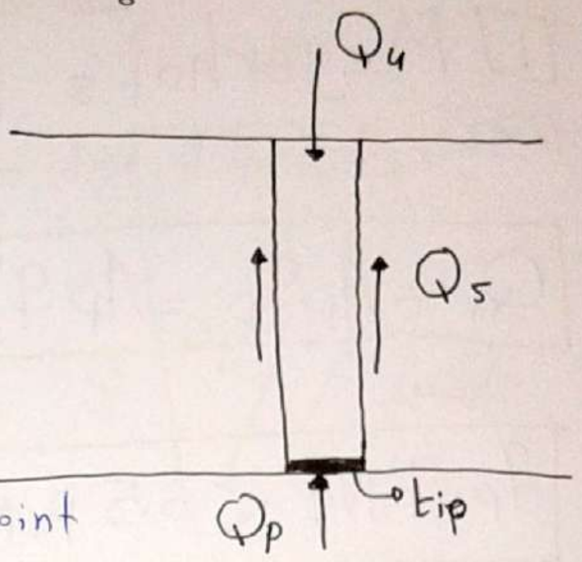


$$P_{ah} = P_a \cos(15 + \delta)$$

Deep Foundation "Piles" :

- Static pile capacity :

$$\boxed{Q_u = Q_p + Q_s}$$



Q_u : ultimate Load carrying capacity

Q_p : Load-carring capacity of the pile point

Q_s : Frictional resistance (skin friction) derived from the soil-pile interface.

$$\boxed{Q_p = A_p q_p = A_p(c' N_c^* + q' N_q^*)}$$

A_p : area of pile tip

c' : Cohesion of the Soil supporting the pile tip تحتها مهم

q_p : unit point resistance

q' : effective vertical stress at level of the pile tip $= \boxed{\gamma' z}$

N_c^*, N_q^* = Bearing capacity factors.

$$\boxed{Q_s = \sum p \Delta L f}$$

p : perimeter of the pile section المحيط

ΔL : incremental pile length over which p & f are taken to be constant : بقسمه لاجزاء تكون كلاهن p, f ثابتان

f : unit friction resistance at any depth.

$$\boxed{Q_{All} = \frac{Q_u}{F.S}}$$

Q_{all} : allowable Load-carrying capacity

F.S : factor of safety

● for Estimating Q_p & Q_s in Sand :

1 Meyerhof's Method :

** for tip pile rest on sand

$$\boxed{Q_p = A_p q_p = A_p q' N_q^*}$$

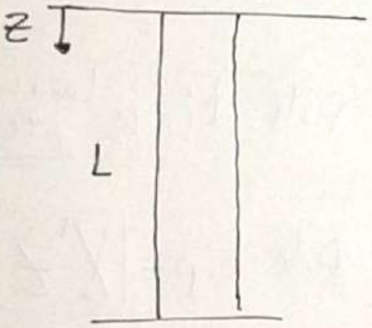
بشرط

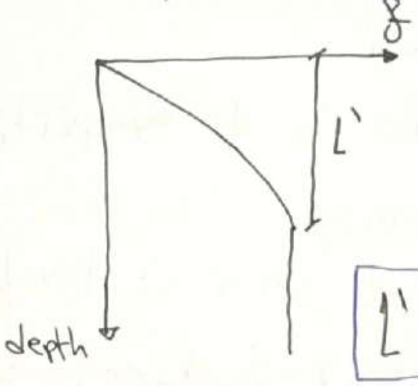
$$\boxed{q_p = q' N_q^* < 0.5\, P_a N_q^* \tan\phi'}$$

بيختار القيمة الاقل منهم وبعتمدها

$P_a = 100\ kN/m^2 \Rightarrow$ atmospheric pressure

** frictional resistance Q_s :





** معامل الاحتكاك بضل يزيد لعمق بسميه critical depth L' بعدين بثبت .

$\boxed{L' = 15 D}$ مهم

● $Z = 0 \rightarrow L'$

$$\boxed{f = K_o \bar{\sigma}_o \tan\delta'}$$

$Z = L' \_ L$ بثبت

$$\boxed{f = f_z @ L'}$$

$$\boxed{Q_s = \sum f \Delta L P}$$

عند الحل بقسم الطبقات من $0 - L'$ ، $L - L'$.

$K_o \Rightarrow$ * جدول من الكتاب

$\bar{\sigma}_o \Rightarrow$ effictive stress at each level — له طريقة حل توضح بالامثلة

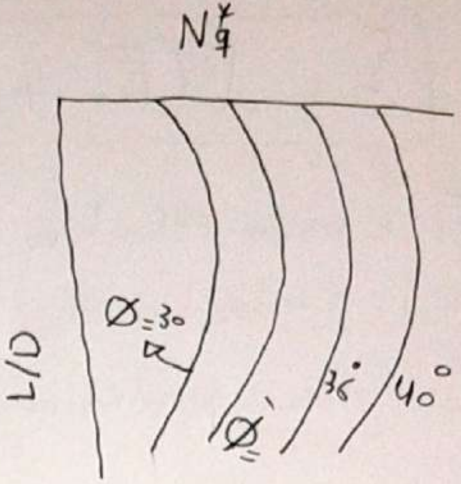
$\delta' \Rightarrow$ friction angle $(0.5\phi' - 0.8\phi')$

* medium-displacement driven ⟸ ** فكرة سنوات

✓ $K_o = 1.6(1 - \sin\phi')$

[2] Coyle & Castello's Method :

[**] For estimating Q_p on sand :



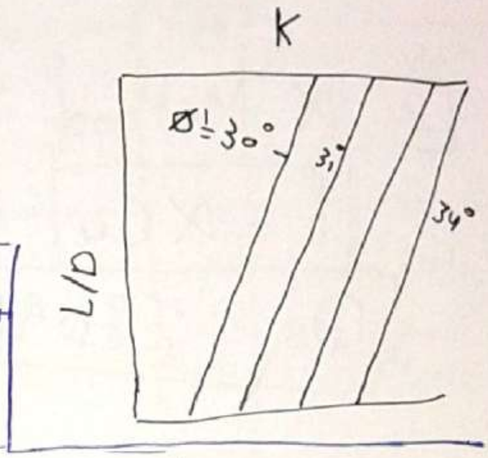
$$\boxed{Q_p = q' N_q^* A_p}$$

q' : effictive vertical stress at pile tip.

[**] Frictional resistance Q_s :

$$\boxed{Q_s = f_{av}\, pL = \sum (K \bar{\sigma}_o \tan\delta')\, pL}$$

K ⇐ from chart ⟹



● for estimating Q_p & Q_s in Clay

▨ Q_p for pile in saturated clays under undrained conditions ($\phi = 0$)

$$\boxed{Q_p = 9 C_u A_p}$$

C_u : undrained cohesion of the soil below the tip of the pile.

▨ frictional (skin) Resistance in Clay :

* Three method exist :

1. λ method
2. α method
3- β method

$$\boxed{Q_s = \sum f \Delta L P}$$

for all

1 λ method :

$$\boxed{f = \lambda(\bar{\sigma}'_o + 2C_u)}$$

$\bar{\sigma}_o$ = mean effective vertical stress for the entire ~~length~~ embedment Length.

C_u = mean undrained shear strength "$\phi = 0$"

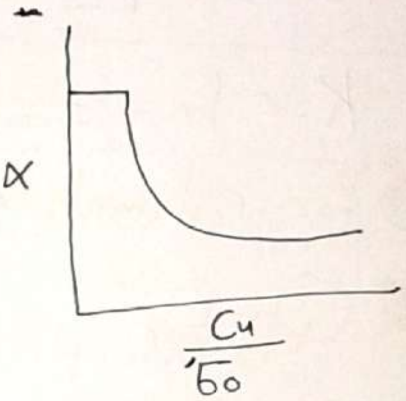
embedment Length ΔL	λ

← سنوا ن "interpolation"

2 α Method "total stresses"

$$\boxed{f = \alpha C_u}$$

$$\boxed{Q_s = \sum f p \Delta L = \sum \alpha C_u p \Delta L}$$



3 β method (effictive stress) :

$$\boxed{f = \beta \bar{\sigma}'_o}$$

$$\boxed{f = (1 - \sin\phi'_R)\tan\phi'_R \sqrt{OCR}\ \sigma'_o}$$

σ'_o = vertical effective stress

$\beta = K \tan\phi'_R$

ϕ'_R = drained friction angle of remoleded Clay

K = earth pressure ~~coeff~~. Coefficient

OCR = overconsolidation ratio

$Q_u = Q_p + Q_s - \underbrace{W}_{\text{وزن كونكريت}}$

* ملاحظة : في حل الاسئلة مهم

$$\boxed{W = (A_p L)\gamma_c}$$

Ex: for square precast prestress driven pile of 0.25 m

Estimate:

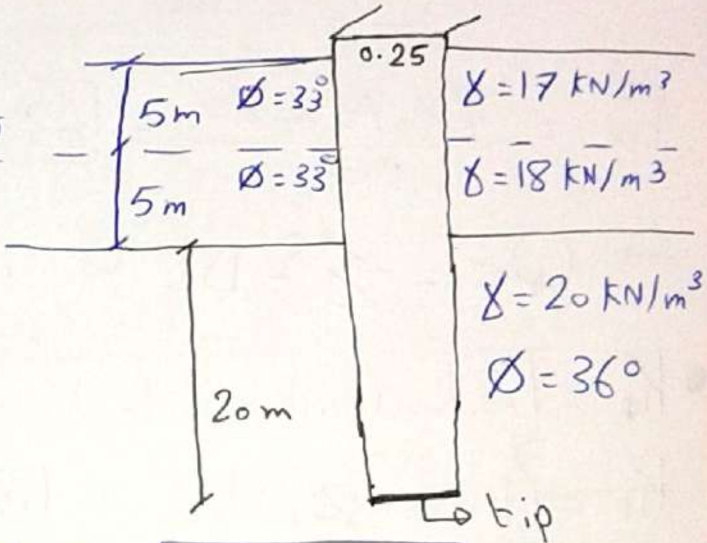
(1) Point resistance capacity by using both meyerhoff & Coyle & Castellos method.

(2) Shaft (skin) resistance by using both methods "high-displacement"

(3) Ultimate carrying capacity if $\gamma_c = 25\ kN/m^3$

Sol:

(1)



☒ **Meyerhoff:**

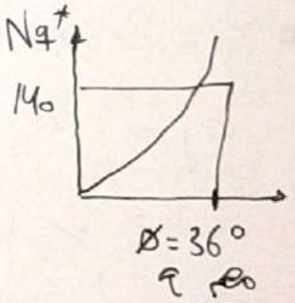
$Q_p = q_p A_p$

$q_p = q' N_q^* < 0.5 p_a N_q^* \tan\phi'$

$q' = (17*5) + (18-9.81)(5) + (20-9.81)(20) = \boxed{329.75\ kpa}$

$q_p = 329.75 \underline{\underline{(140)}} \nless 0.5(100)(140)(\tan 36)$

$46165 < \underline{5085.8} \rightarrow$ نأخذ الاقل مقداراً



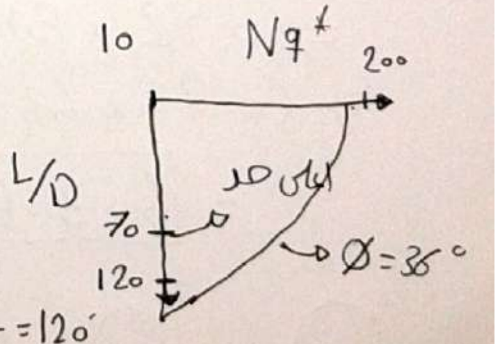
$Q_p = (5085.8)(0.25)^2 = \boxed{317.8\ KN}$

☒ **Coyle & Castello:**

$Q_p = q' (N_q^*) A_p$

$= 329.75(10)(0.25)^2$

$= \boxed{206.1\ KN}$



$\frac{30}{0.4} = 120$

* $L/D = 120$ تعدت اعلى حد بالرسمة

$\boxed{N_q^* = 10}$ بوخذ اقل قيمه

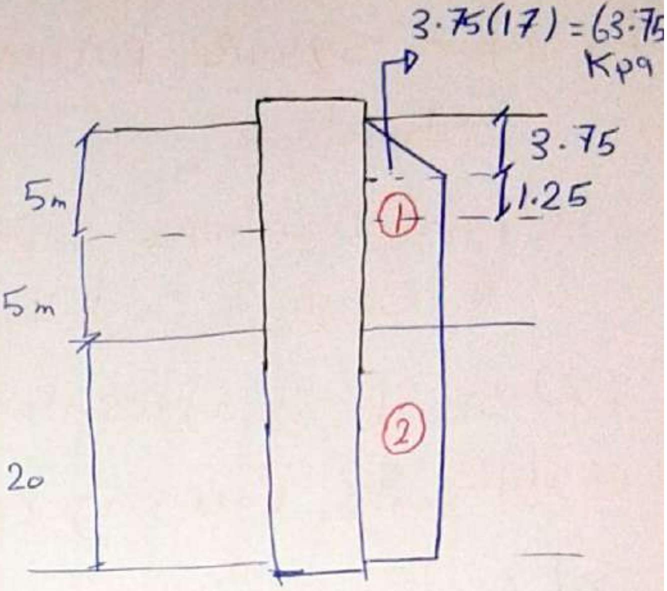
Scanned by CamScanner

(2)

☒ Meyerhoff :

$f \rightarrow$ increase $0 - L'$ "critical depth"

$L' = 15 * 0.25 = 3.75\text{m}$



$$\bar{\bar{\sigma}}_1 = \frac{\left[\frac{1}{2} * 3.75 * 63.75 + 63.75(10 - 3.75)\right]}{10}$$

① طبقة

$$= \boxed{51.8 \text{ Kpa}}$$

$$\bar{\sigma}_2 = \frac{63.75 * 20}{20} = \boxed{63.75 \text{ kpa}}$$

$\Rightarrow Q_s = \sum f \Delta L\, P$ if ① high - displacement driven
② $\delta' = \frac{2}{3}\phi'$

- $K = 1.8\, K_o$ (high - displ.)

$K_1 = (1 - \sin\phi_1)1.8 = 1.8(1 - \sin 33) = \boxed{0.82}$

$K_2 = 1.8(1 - \sin\phi_2) = 1.8(1 - \sin 36) = \boxed{0.72}$

- $\delta = \frac{2}{3}\phi'$

$\delta_1 = \frac{2}{3}(33) = \boxed{22^\circ}$

$\delta_2 = \frac{2}{3}(36) = \boxed{24^\circ}$

محيط

P for square = 4 * طول الضلع

$Q_s = \sum (K \sigma \tan\delta\, pL$

$= (0.82 * 51.8 * \tan 22 * (\underline{4 * 0.25} * 10))$ *

$+ (0.72 * 63.75 * \tan 24 * (4 * 0.25 * 20))$

$= \boxed{603.65 \text{ kN}}$

$\boxed{**}$ $W = (A_p L)\, \gamma_c$

$= (0.25)^2 (30)(25)$

$= \boxed{46.8 \text{ kN}}$

$Q_u = Q_p + Q_s - W$

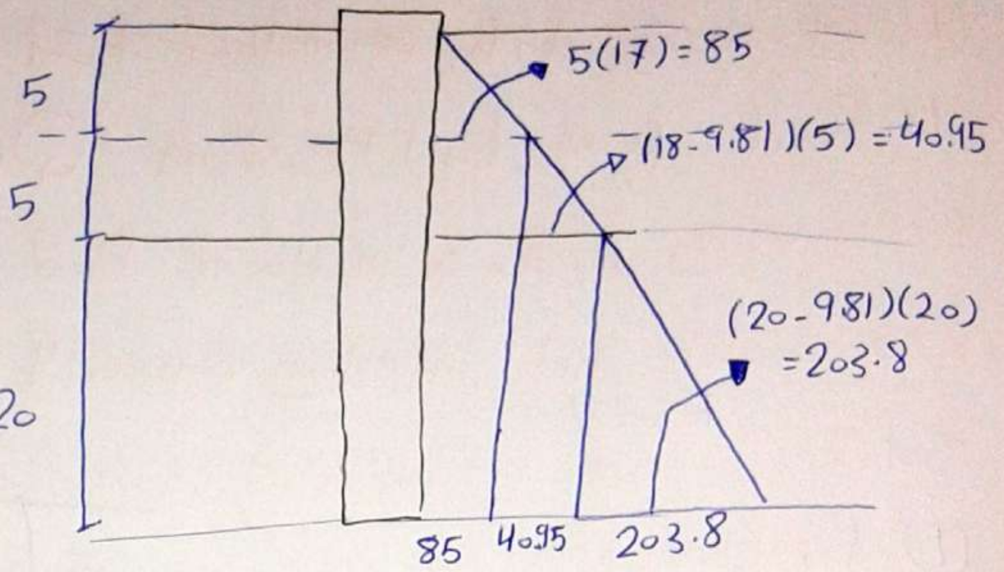
Meyerhoff $= 317.8 + 603.65 - 46.8$

$= \boxed{874.65 \text{ kN}}$

[*] Coyle & Castells 8

$Q_s = \sum f_s A_s$

$= K \bar{\sigma}_0 \tan\delta \, pL$



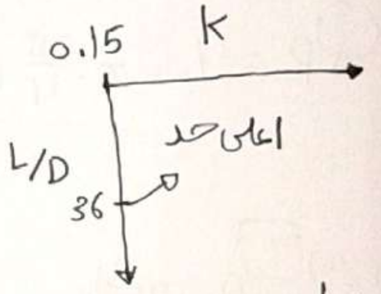
$\bar{\sigma}_1 = \frac{[\frac{1}{2} * 85 * 5 + 85(5) + \frac{1}{2}(40.95)(5)]}{10} = \boxed{73.98 \text{ kpa}}$

$\bar{\sigma}_2 = \frac{(85 + 40.95)(20) + \frac{1}{2}(203.8)(20)}{20} = \boxed{277.85 \text{ kpa}}$

$Q_s = [0.15 * 73.98 * \tan(\frac{2}{3}(33)) * (4 * 0.25 * 10)]$

$+ [0.15 * 277.85 * \tan(\frac{2}{3}(36)) * (4 * 0.25 * 20)]$

$= \boxed{349.17 \text{ kN}}$



$L/D = 120$ تعدت اعلى حد لذلك

اختار اقل قيمه ل $\boxed{0.15 = K}$

$Q_u = Q_p + Q_s - W$

Coyle & castle

$= 349.17 + 206.1 - 46.8 = \boxed{508.47 \text{ kN}}$

Ex : For the pile shown , Estimate :

~~(x) By using λ, β, α method find~~

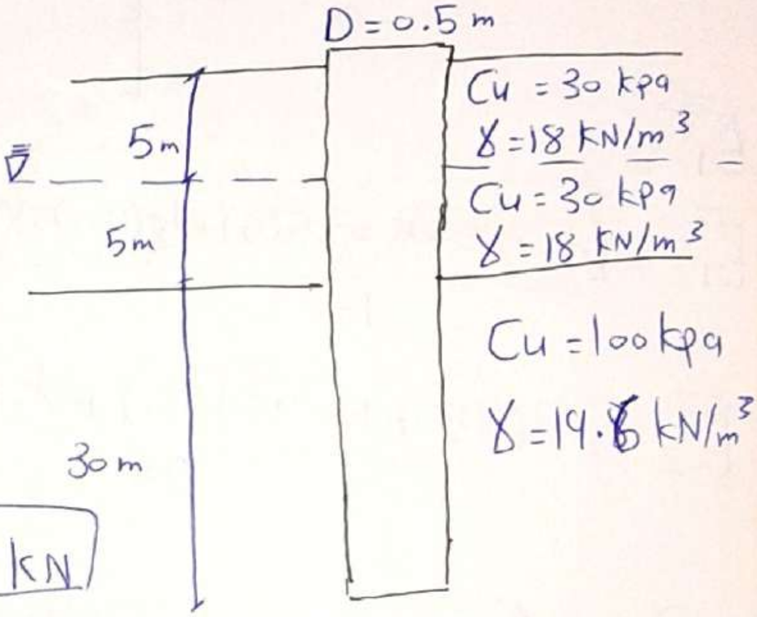
(1) find point (tip) resistance Q_p .

(2) By using λ, β, α method find Q_s .

(3) allowable net carrying capacity if $\gamma_c = 23\ kN/m^3$

F.S = 4

Sol :



(1) for clay :

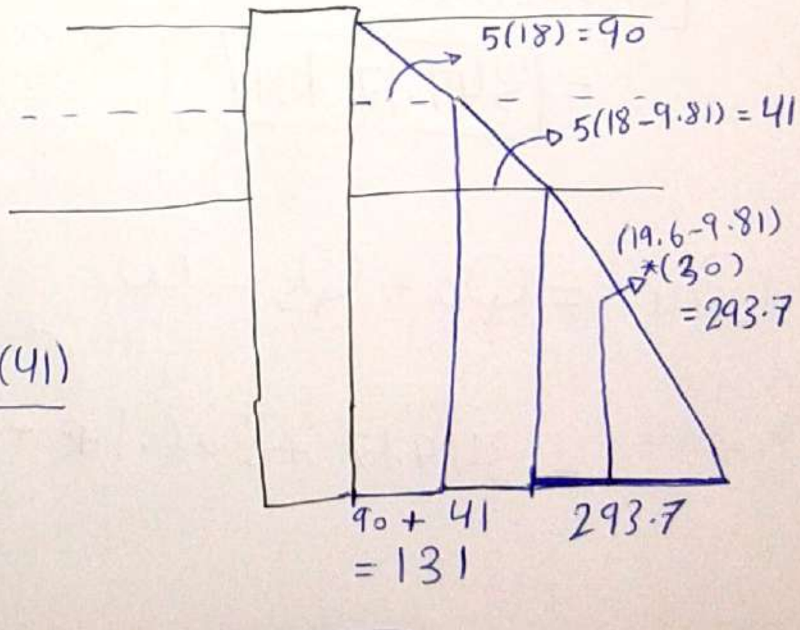
$Q_p = \underline{\underline{9C_u}} * A_p$ الطبقة الي يتحمل ال tip

q_p له

$q_p = 9(100) = 900\ kpa$

$$Q_p = 900 * \underbrace{\frac{\pi}{4}(0.5)^2}_{A_p} = \boxed{176.71\ kN}$$

(2) $\boxed{1}$ λ method :



$Q_s = \sum f_s A_s$

$f = \lambda(\bar{\sigma}_o + 2C_u)$

$$\bar{\sigma}_{o1} = \frac{(\frac{1}{2}(5\times 90)) + 90(5) + \frac{1}{2}(5)(41)}{10}$$

$$= \boxed{77.75\ kpa}$$

$$\bar{\sigma}_{o2} = \frac{131 * 30 + \frac{1}{2}(30)(293.7)}{30} = \boxed{277.85\ kpa}$$

⇒ from table

$L = 10 \rightarrow \lambda = 0.245$

$L = 30 \rightarrow \lambda = 0.136$

$$Q_s = \sum \lambda_i(\sigma_{oi} + 2C_{ui}) * \overset{\text{محيط دائره}}{\underline{\pi D}} L_i$$

$$= [0.245(77.75 + 2(30)) * \pi * (0.5) * 10]$$

$$+ [0.136(277.85 + 2(100)) * \pi(0.5)(30)]$$

$$= 529.86 + 3060.92 = \boxed{3590.78 \text{ kN}}$$

$$W = (A_p L)\gamma_c = \frac{\pi}{4}(0.5)^2 * 40 * 23 = \boxed{180.32 \text{ kN}}$$

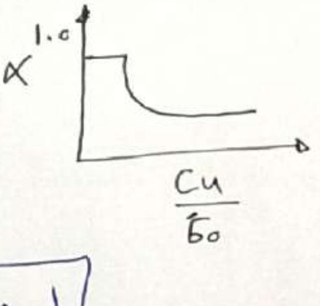
$$Q_u = Q_s + Q_p - W = 3590.78 + 176.71 - 180.32$$

$$= \boxed{3587.17 \text{ kN}}$$

$$Q_{all} = \frac{Q_u}{F.S} = \frac{3587.17}{4} = \boxed{896.79 \text{ kN}}$$

$\boxed{2}$ α - method :

$$f_s = \alpha C_u$$



* first Layer :

$$\frac{C_{u1}}{\sigma_{o1}} = \frac{30}{77.75} = 0.386 \simeq \boxed{\alpha_1 = 0.1}$$

* Second layer :

$$\frac{C_{u2}}{\sigma_{o2}} = \frac{100}{277.85} = 0.36 \simeq \boxed{\alpha_2 = 0.1}$$

$$Q_s = \sum f_s A_s = (1 * 30 * \pi * (0.5) * 10) + (1 * 100 * \pi * 0.5 * 30)$$

$$= \boxed{5181 \text{ kN}}$$

$\boxed{3}$ β - method :

$$\left[\begin{array}{ll} \phi_{R_1} = 30^\circ & , C_1 = 0 \\ \phi_{R_2} = 35^\circ & , C_2 = 0 \\ OCR = 1 & \end{array}\right]$$ تعطى بالسؤال

$$f_s = \beta \bar{\sigma}_o$$

$$\beta = (1 - \sin \phi_R) \tan \phi_R \sqrt{OCR}$$

$$f_{s_1} = (1 - \sin 30) \tan 30 \sqrt{1} * 77.75 = 22.44$$

$$f_{s_2} = (1 - \sin 35) \tan 35 \sqrt{1} * 277.85 = \text{[illegible]}\ 82.96$$

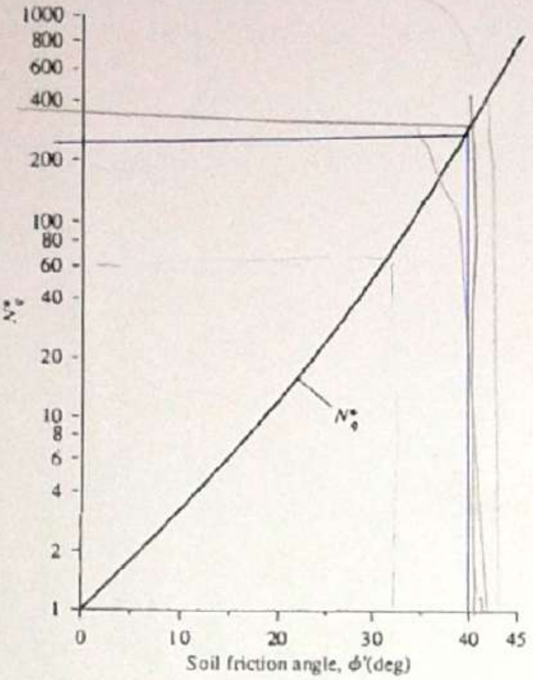
$$Q_s = \sum f_s \, P \, \Delta L$$

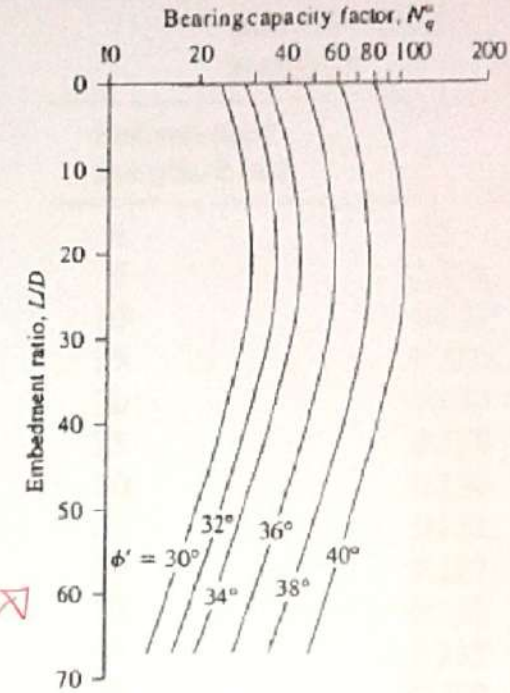
$$= 22.44 * \pi(0.5)(10) + \overset{82.96}{\text{[illegible]}} * \pi(0.5)(30)$$

$$= 352.308 + \cancel{9163.305}\ 3907.416$$

$$= \boxed{\cancel{9515.613\ kN}}$$

$$\text{[illegible]}\ \boxed{4259.724\ kN}$$





$$Q_p = q' N_q^* A_p$$

$$f = K\sigma_o' \tan\delta'$$

❖ L' = 15D

Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

$$Q_s = f_{av} pL = (K\bar{\sigma}_o' \tan\delta')pL$$

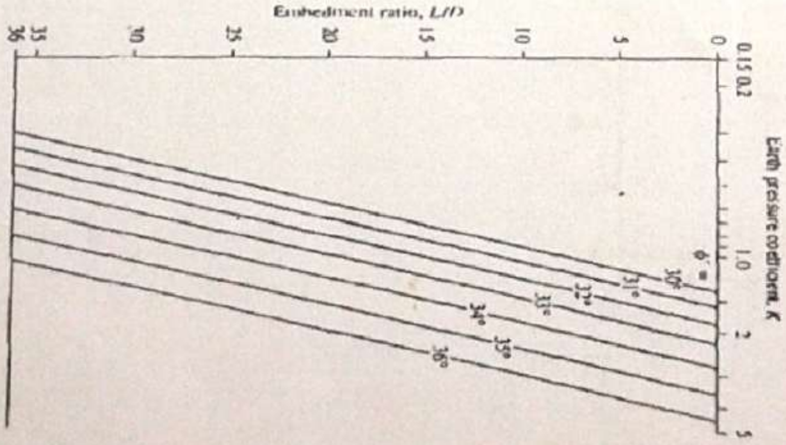
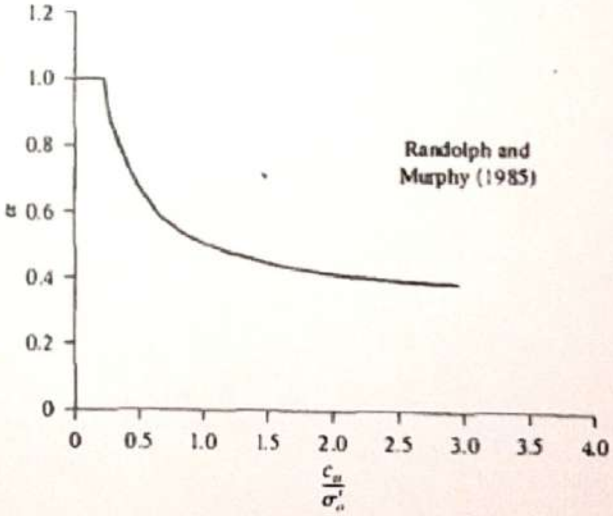


Table 11.7 Variation of λ with pile embedment length, *L*

Embedment Length, *L* (m)	**λ**
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110



In Problems 13.1 through 13.4, use $\gamma_{concrete} = 23.58$ kN/m³. Also, in Eq. (13.11), use $k_1 = k_2 = 2/3$ and $P_p = 0$.

13.2 لـ
13.3
مقترحات
ايضاً

13.1 For the cantilever retaining wall shown in Figure P13.1, let the following data be given:

Wall dimensions: $H = 8$ m, $x_1 = 0.4$ m, $x_2 = 0.6$ m, $x_3 = 1.5$ m, $x_4 = 3.5$ m, $x_5 = 0.96$ m, $D = 1.75$ m, $\alpha = 10°$

Soil properties: $\gamma_1 = 16.8$ kN/m³, $\phi'_1 = 32°$, $\gamma_2 = 17.6$ kN/m³, $\phi'_2 = 28°$, $c'_2 = 30$ kN/m²

Calculate the factor of safety with respect to overturning, sliding, and bearing capacity.

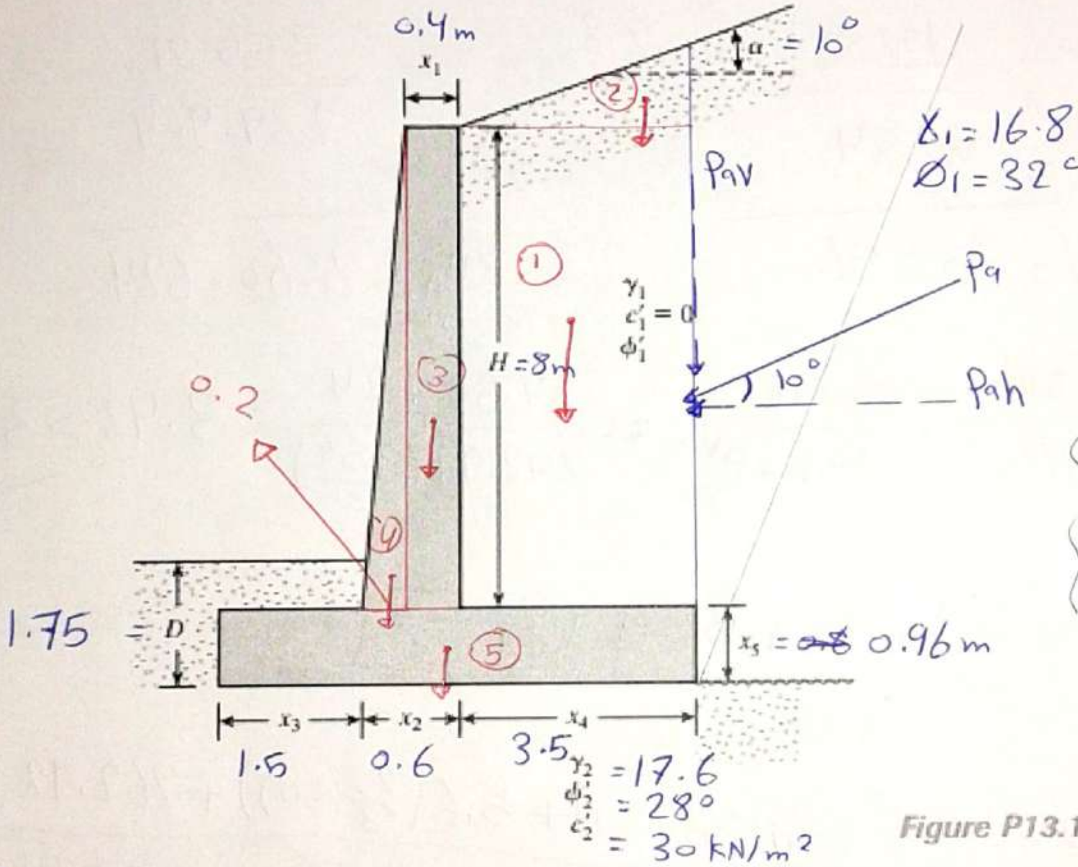


Figure P13.1

$$H' = H + X_4 \tan\alpha$$
$$= 8.96 + 3.5 \tan 10$$
$$= \boxed{9.58\text{ m}}$$

$$k_{a_1} = \cos 10 \frac{\cos 10 - \sqrt{\cos^2 10 - \cos^2 32}}{\cos 10 + \sqrt{\cos^2 10 - \cos^2 32}} = \boxed{0.32}$$

$$k_{p_2} = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1 + \sin 30}{1 - \sin 30} = \boxed{3}$$

② P_a :

$$P_a = \frac{1}{2}\gamma H'^2 k_a = \boxed{246.69\text{ kN/m}}$$

$$P_{ah} = P_a \cos\alpha = \boxed{242.94\text{ kN}} \quad // \quad P_{aV} = P_a \sin\alpha = \boxed{42.84\text{ kN/m}}$$

- Pp :

$$P_p = 0.5\,\gamma\,D_f^2\,K_p + 2C\sqrt{K_p}\,D_f$$

$$= 0.5(17.6)(1.75)^2(3) + 2(30)\sqrt{3}\,(1.75)$$

$$= \boxed{262.72\ \text{kN}}$$

Seg #	A_i	V_i	X_i	M_{F_i}
1	28	470.4	3.85	1811.04
2	1.08	18.14	4.43	80.36
3	3.2	75.46	1.9	143.37
4	0.8	18.86	1.63	30.74
5	5.38	126.86	2.8	355.21
6		PaV = 42.84	5.6	239.904

$\Sigma V = 752.56$

$\Sigma M_R = 2660.624$

$$\boxed{*}\ F.S_{ot} = \frac{\Sigma M_R}{\Sigma M_o} = \frac{\Sigma M_R}{P_{ah} * \frac{H'}{3}} = \frac{2660.624}{242.94\left(\frac{9.58}{3}\right)} = 3.43 > 3\ ✓$$

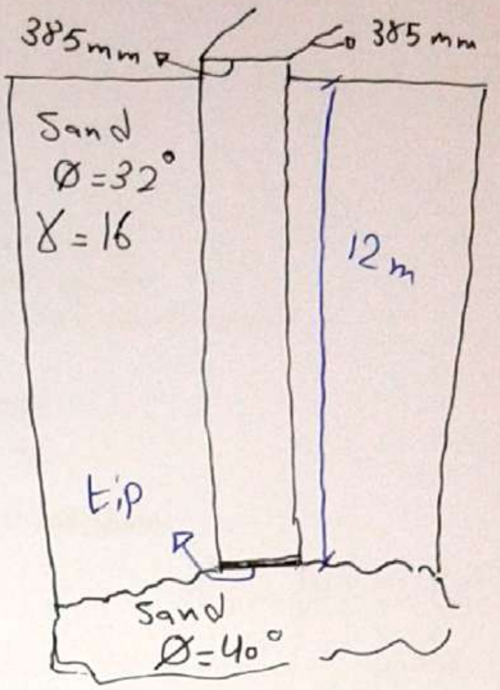
$$F.S_{\text{sliding}} = \frac{\Sigma V \tan\delta + B\,C_a + P_p}{P_{ah}}$$

$$= \frac{752.56 \tan\left(\frac{2}{3}(28)\right) + 5.6\left(\frac{2}{3}(30)\right) + 262.72}{242.94}$$

$$= 2.59 > 1.5 \text{ ok } ✓$$

Q :   فكرة سنوات فصل ماضي 2016

① Find Q_p :   لاحظ ان هناك طبقة صخيرة بالاسفل مختلفة عن اعلى هي التي تحمل ال tip لذلك نستخدم ال $\phi = 40°$



Using Meyerhofs :

$$Q_p = A_p\, q'\, N_q^*$$

$$A_p = (0.385)^2 = 0.14823\ m^2$$

$$q' = 16 \times 12 = 192\ kPa$$

N_q^* from chart @ $\phi = 40°$

$= 250$

$$Q_p = \boxed{7115.04\ kN}$$

** فكرة
medium-displacement Driven
$K_o = 1.6(1-\sin\phi)$

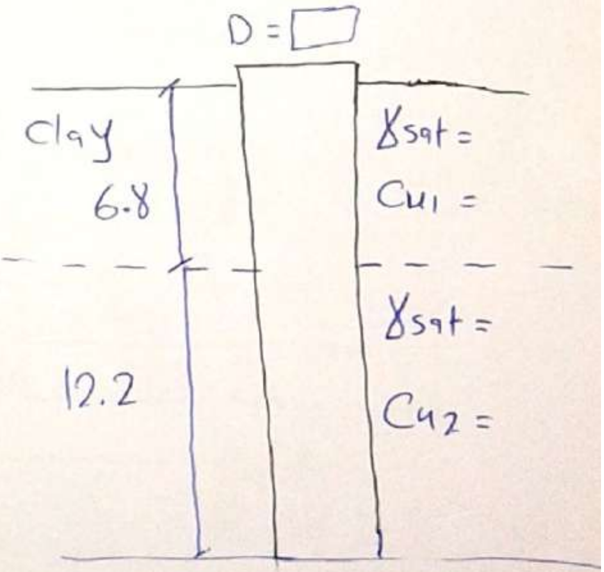
② كانت ايجاد Q_s بالطريقتين ⇒

③ اوجد Q_u و Q_{all}

Q : * كان فيه γ_{sat} اعلى ال water table : 3

* اسأل الدكتور عنها

⇐ طلب بالسؤال :



① $Q_p = 9C_{u2} =$ مباشرة قانون

② $Q_s \rightarrow$ 3 method λ, β, α

③ → سؤال فكرة

** اعطى Q_{all} وحسبنا Q_u وسأل هل تستطيع التحمل؟ اذا لا؟ احسب كم لازم نزيد طول ال pile بالطبقة الثانية عشان يتحمل .

$$Q_{all} = \frac{Q_p + [Q_{s1} + f_{s2} * P * \underline{L}] - W}{F.S} \rightsquigarrow \text{or } (A_p\, \underline{\underline{L}}\, \gamma_c)$$

معطى بالسؤال

لكن تأثيرها قليل

امتحان فصل ماضي "اول"
2016

Student Name.. Student Number..

Hashemite University
Faculty of Engineering
Department of Civil Engineering
Foundation Engineering (0401333)
Instructor: Dr. Omar Al-Hattamleh

First Exam
Thursdays: 30/05/2009
Attempts All Problems
Write Only in the designated space

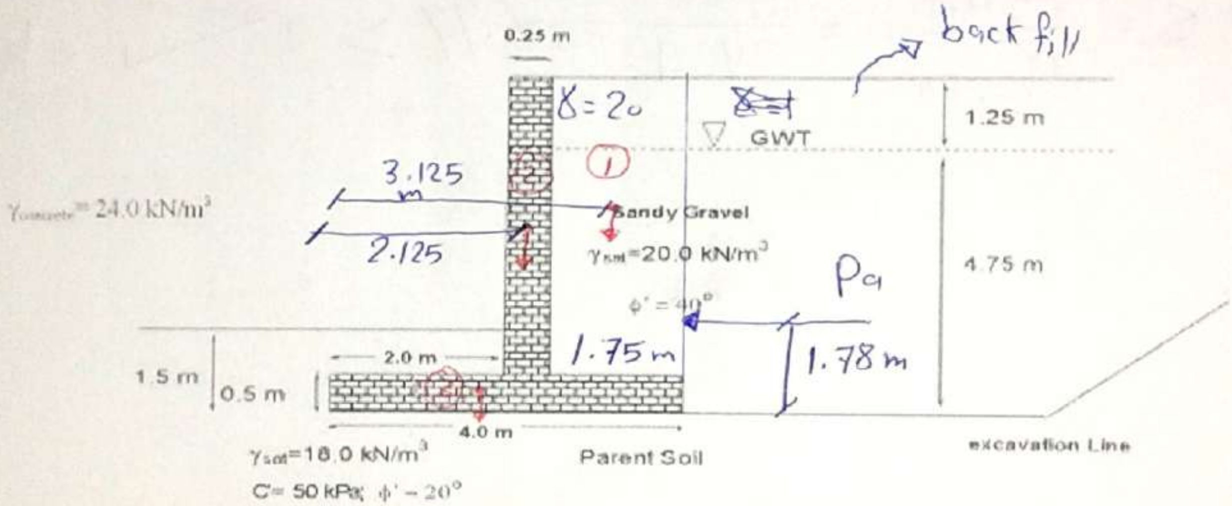
No Partial Credit Will Be Given

Question # 1: (12 Points)
Determine if the retaining wall shown with straight stem in the following figure is safe against:

1) Sliding (F.S.>1.5)
2) Overturning (F.S.>2.0)
3) Bearing capacity failure, use ultimate bearing capacity, q_{ult}=750.0 kPa.

(Note ignore passive contribution; consider $\gamma_{dry}=\gamma_{sat}$ for backfill soil)

$\gamma_c = 24\ kN/m^3$
معطى



Section no	Area(m2)	Weight/m (kN/m)	Moment arm(m)	Moment (kN.m)/m
1	1.75(5.5)=9.63	9.63(20)=192.6	3.125	601.875
2	0.25(5.5)=1.38	1.38(24)=33.12	2.125	70.38
3	0.5(4) = 2	2(24) = 48	½ ~~2~~ = 2	96
	ΣV = 273.72		ΣMR = 768.26	

$$\boxed{*}\ F.S_{ot} = \frac{\Sigma M_R}{\Sigma M_o} = \frac{768.26}{P_{ah} * 1.78} = \frac{768.26}{\cancel{165.415}\ 289.48} = \overset{2.65}{\cancel{[illegible]}} > 2\ ✓$$

$$\boxed{*}\ F.S\ sliding = \frac{\Sigma V \tan\delta + B\,C_a + P_P}{P_{ah}} = \frac{273.72 \tan(\frac{2}{3}(20)) + 4(\frac{2}{3}50) + 244.09}{\cancel{2}\ 165.415}$$

$$= 2.67 > 1.5\ ✓$$

$\boxed{**}$ B.S failure :

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} \leq \frac{B}{6}$$

$$= 2 - \frac{768.26 - 289.48}{273.72} = \boxed{0.25\ m}$$

$$q_{toe} = q_{max} = \frac{\Sigma V}{B}\left(1 + \frac{6eB}{B}\right) = \frac{273.72}{4}\left(1 + \frac{6(0.25)}{4}\right)$$

$$q_{heel} = q_{min} \Rightarrow 0 \checkmark \qquad = \boxed{94.09\ kpa}$$

$$F_{B.S} = \frac{q_{ult}}{q_{max}} = \frac{750}{94.09} = 7.97 \geq 3 \checkmark\ ok$$

Q1 (12 point) :

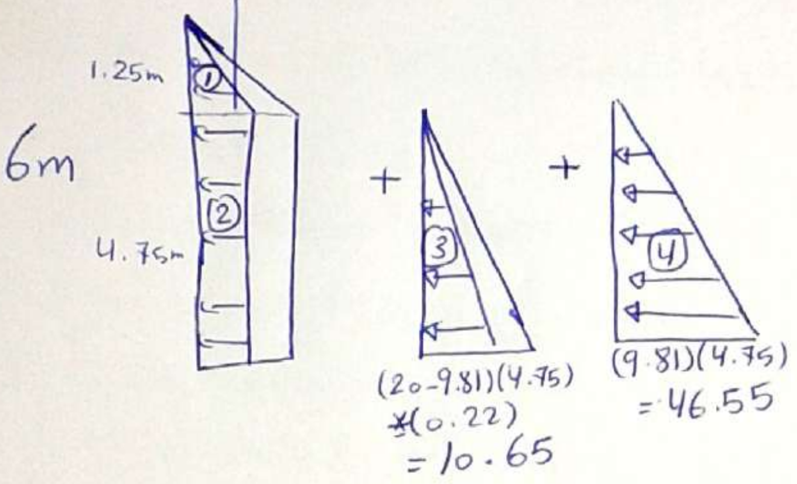
$$K_{a1} = \frac{1-\sin\phi_1}{1+\sin\phi_1} = \boxed{0.22}$$

$$K_{p2} = \frac{1+\sin\phi_2}{1-\sin\phi_2} = \frac{1+\sin 20}{1-\sin 20} = \boxed{2.04}$$

[**] يتم حساب كلمن Pa ، Pp مثل السابق :

- Pa :

$1.25(20)(0.22) = 5.5$



$(20-9.81)(4.75)$
$*(0.22)$
$= 10.65$

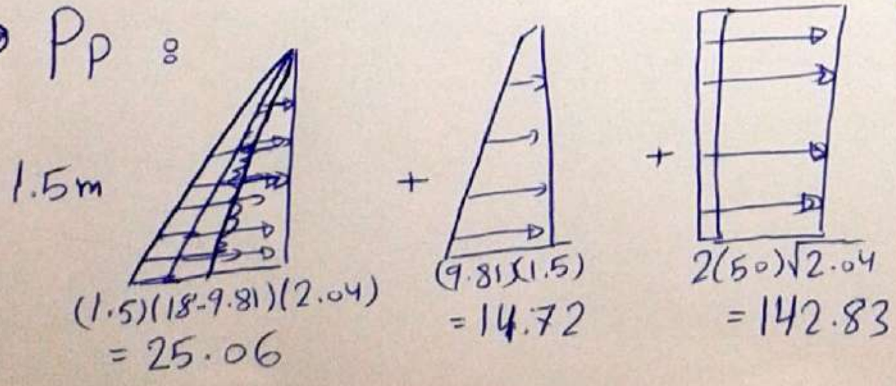
$(9.81)(4.75)$
$= 46.55$

seg #	Ph_i/m	X_i	Ph_i*X_i
1	3.44	5.167	17.77
2	26.125	2.375	62.05
3	25.29	1.58	39.96
4	110.56	1.58	174.68
	Σ		Σ 294.46

Pa/m = 165.415

$$\bar{X} = \frac{\Sigma Ph_i X_i}{\Sigma Ph_i} = \boxed{1.78\,m}$$

- Pp :



$(1.5)(18-9.81)(2.04)$
$= 25.06$

$(9.81)(1.5)$
$= 14.72$

$2(50)\sqrt{2.04}$
$= 142.83$

$$P_p/m = 0.5(1.5)(25.06) + 0.5(1.5)(14.72) + 142.83(1.5)$$
$$= 18.8 + 11.04 + 214.25$$
$$= \boxed{244.09\ KN/m}$$

Scanned by CamScanner

سؤال فاينال حفظ 10 علامات

1. Piles are Commonly used for : اذكر 5 نقاط

1- to carry structure Loads into or through a soil stratum

2- To resist uplift ~~or~~ overturning forces.

3- To control earth movements such as Landslides

4- To control settlement when spread footing are on marginal or highly compressible soil.

5- To control scour problems on bridge abutment or piers

2. footing should be carried below : 5 نقاط

<1> The frost Line خط التجمد

<2> Zones of high Volume change due to moisture fluctuation

<3> Top soil or organic matter

<4> Peat or muck

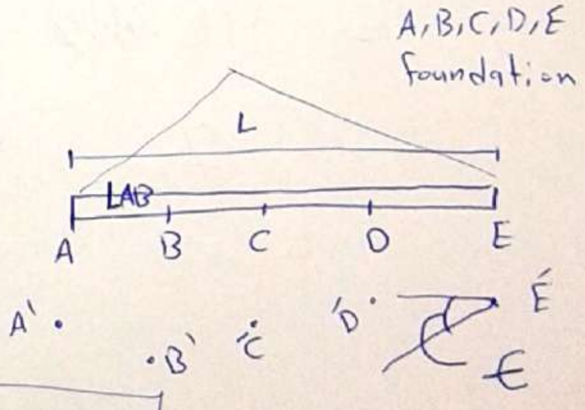
<5> unconsolidated materials such as abounded dumps

3. problems with soil settlement analysis ع موجودة بالسلايدز

4. Servicibility Limit : اشرح المفاهيم

$\overline{AA'}$: Total settlement "S_{TA}".

$\overline{BB'}$: 〃 〃 "S_{TB}".



ΔS_{TAB} : differential settlement bettween A & B.

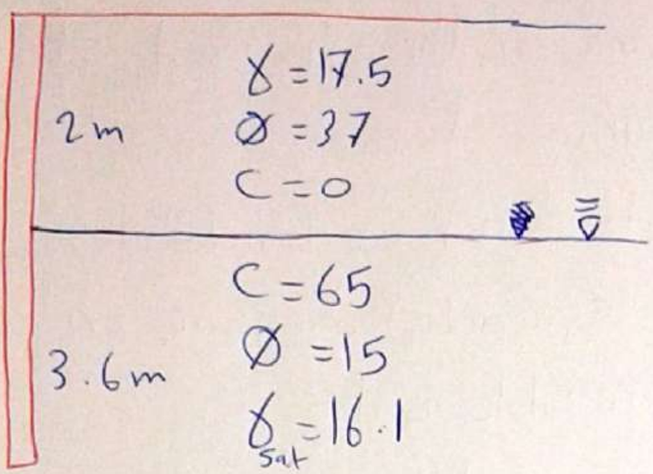
α = gradient between any Two footing.

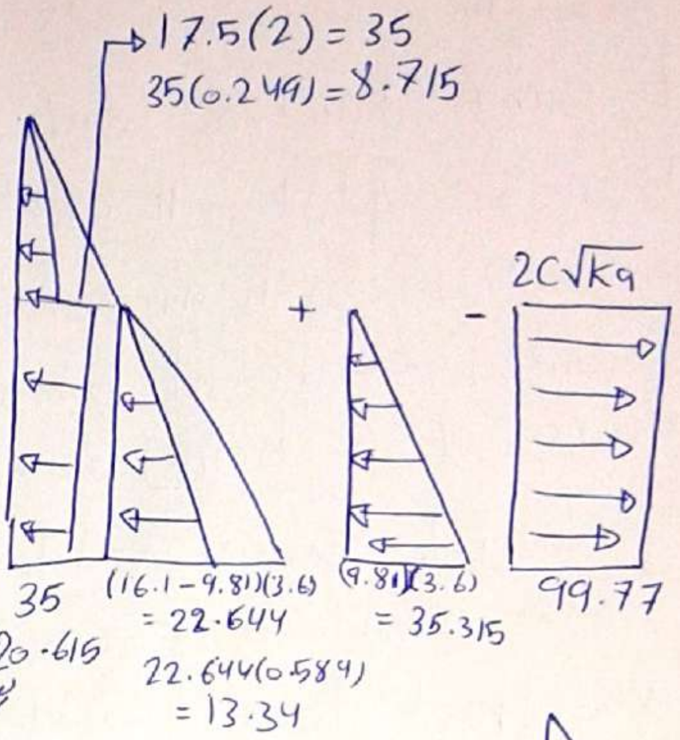
β = angular distortion = $\boxed{\frac{\Delta ST_{ij}}{L_{ij}} \text{ (Rad)}}$ $\Rightarrow \frac{\Delta ST_{AB}}{L_{AB}}$

ε : tilt angle (Rad)

Scanned by CamScanner

Tensile crack : فكرة سنوات $\boxed{**}$

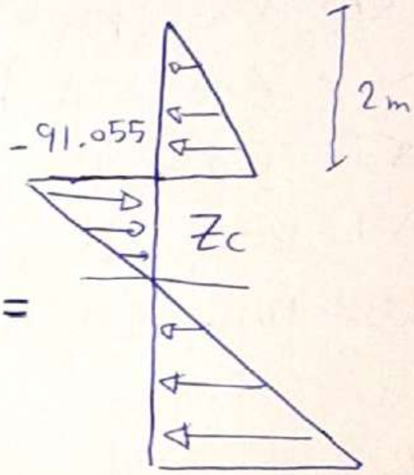




$Ka_1 = 0.249$

$Ka_2 = 0.589$

8.715 - 99.77 = -91.055



Tensile crack بعد =

لإيجاد Zc :

$0 = (13.34\,H + 35.315\,H) - 91.055$

$\Rightarrow 48.\overset{655}{\cancel{4415}}\,H = 91.055$

$Zc = 1.87m$

تأكد من الحل .