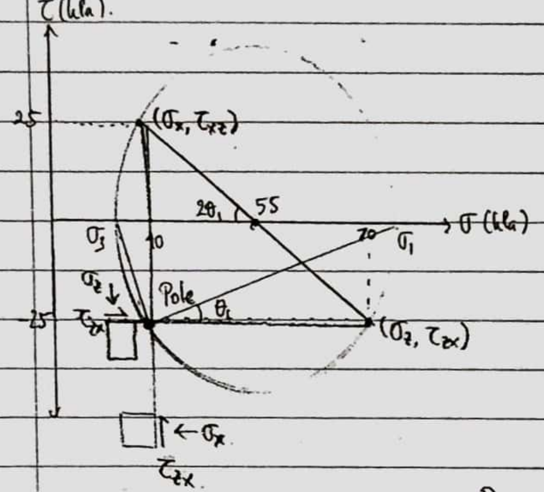
CV2014 : Geotech. Engineering
S2 / 2014-15

Yes, U Can!

Q1. (a) $\sigma_x = 40$ kPa $\quad \tau_{xz} = 25$ kPa.
$\sigma_z = 70$ kPa $\quad \tau_{zx} = -25$ kPa.



Radius of Mohr Circle :

$$r = \sqrt{\left(\frac{70-40}{2}\right)^2 + 25^2}$$

$= 29.15$ kPa.

$\sigma_3 = 55 - 29.15 = 25.85$ kPa.
$\sigma_1 = 55 + 29.15 = 84.15$ kPa.

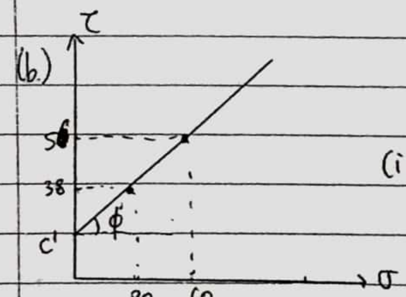
Direction to principal stresses :

$$\theta_1 = \frac{1}{2}\cos^{-1}\sqrt{\frac{1}{1+\left(\frac{50}{70-40}\right)^2}} = 29.52^\circ \text{ (CCW)}$$

Alternative methods :

$$2\theta_1 = \tan^{-1}\left(\frac{25}{55-40}\right) \Rightarrow \theta_1 = 29.52^\circ \text{ (CCW)}.$$

(b.)



(i) $\phi = \tan^{-1}\left[\frac{(56-38)}{(60-30)}\right] = 31^\circ$

Find the equation of the line :

$m = \frac{3}{5}$ $\qquad y = mx + c$

at $\sigma = 30 \rightarrow$ $\quad 38 = \left(\frac{3}{5}\right)(30) + c'$

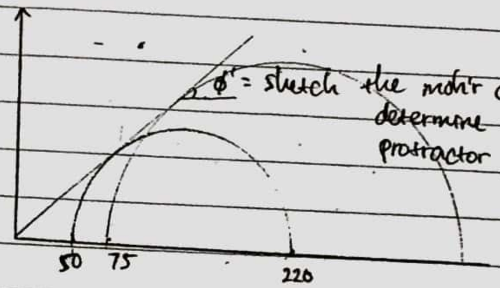
$c' = 20$ kPa.

(ii) Generally, sand doesn't have cohesion in nature. However this considerable amount of cohesion exists could be due to presence of clay / silt in the sand. Other reason could be the sand sample is very dense since it has been undergoing a damping process for a very long time.

(c). $\sigma_3 = 50$ kPa

deviator stress $= \sigma_1 - \sigma_3$ = diameter of Mohr circle. = 170 kPa

(i)



ϕ' = sketch the mohr circle to scale, ~~determine the angle using~~ protractor

(around $\pm 40^\circ$)

(ii) The deviatoric stress can be obtained by plotting another Mohr circle with $\sigma_3 = 75$ kPa where it touches just right at the failure envelope line.

$\sigma_1 \approx \pm 365$ kPa.

$(\sigma_1 - \sigma_3) \approx \pm 290$ kPa.

(d) UU Test should gives a similar size of Moh'r circle regardless of its
(i) confining pressure at test because they are all experiencing the same effective stress prior to the test.

Therefore we could write-off sample 2, since it gave us a smaller deviatoric stress as compared to the other samples.

$$q_u = \frac{60.9 + 61.2}{2} = 61.05 \text{ kPa}.$$

(ii) Advantage: less resource needed for the test. (cheaper and simpler instrument, don't need water to simulate confining pressure).

Disadvantage: Does not represent actual circumstance whereby soil is experiencing confining pressure in nature.

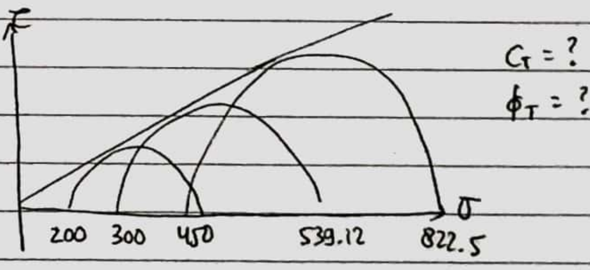
Q2. (a).

σ_3 (kPa)	200	300	450	
Axial load (N)	151.2	248.0	379.1	(1)
Axial compression	5.2	6.5	7.8	
A (mm²)	1056.52	1037.12	1017.72	(2)
$(\sigma_1 - \sigma_3)$ kPa	143.11	239.12	372.5	(1)/(2)
σ_1 (kPa)	343.11	539.12	822.5	
u (kPa)	120	170	250	
σ_3' (kPa)	80	130	200	$\sigma_3 - u$
σ_1' (kPa)	223.11	369.12	572.5	$\sigma_1 - u$

The parameters can be obtained by plotting the mohr circles for each total and effective stress, then manually measure the parameters using ruler and protractor. (Must be drawn to scale).

effective stress :

total stress :

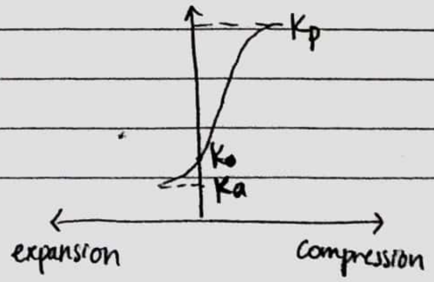


These are not drawn to scale, Therefore I cannot give you the value.

(b.) At-rest : Soil doesn't move, the resultant forces acting at soil is zero. $(\sigma_z = \sigma_x = \sigma_y)$

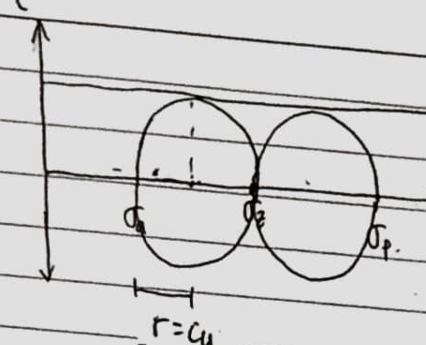
Active : Soil moves apart from the soil mass. (Could be due to the wall moving away from the soil).

Passive : Soil moves towards the soil mass, against other soil. (so called pushing the soil). Could be due to the wall moving into the soil.



Yes, U can!

(c).

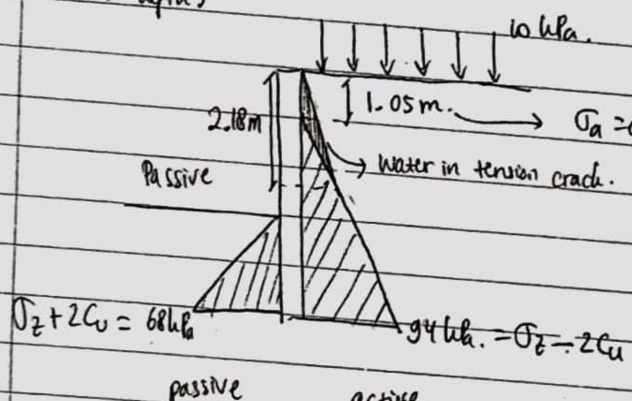


$\sigma_a = \sigma_z - 2c_u$.

$\sigma_p = \sigma_z + 2c_u$.

(d) $z_w = (q - 2c_u)/(\gamma_w - \gamma) = 2.18$ m.
(crack depth)

~~K_a~~



$\sigma_a = 0 \rightarrow \gamma \cdot z + 10 - 2c_u = 0$

$z = 1.05$ m.

Yes, U Can!

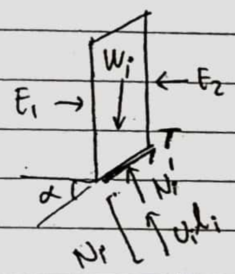
Q3. (a). Fellenius = Assume no interslice forces at all.

Bishop = Assume no Interslice shear force.

Spenser : Assume the resultant of interslice forces are parallel.

The assumptions were made because method of slices generates statically indeterminate problem. Therefore, the interslices force must be assumed to a particular value in order to be able to solve the problem statically

(b.) Bishop assumed no interslice shear force at each slice.



$$W_i = N_i \cos\alpha + T_i \sin\alpha$$

$$W_i = N_i' \cos\alpha + u_i l_i \cos\alpha + T_i \sin\alpha$$

$$N_i' = \frac{W_i - u_i l_i \cos\alpha - T_i \sin\alpha}{\cos\alpha}$$

More complete solution : substitute this

$$T_i = \frac{\tau_i l_i}{F} \rightarrow T_i = \frac{1}{F}(c_i' l_i + N_i' \tan\phi')$$

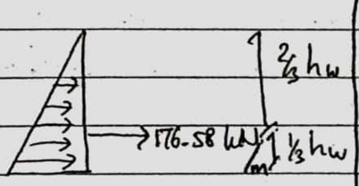
F = factor of safety.

$$N_i' = \frac{[W_i - (c_i' l_i / F)\sin\alpha - u_i l_i \cos\alpha]}{[\cos\alpha + (\tan\phi' \cdot \sin\alpha)/F]}$$

(c). (i) $F = \frac{\text{Resistance moment}}{\text{Driving Moment}}$ $\longrightarrow$ $1.5 = \frac{(C_u \times 18.9 \times 12.1) + (176.58 \times 9.5)}{(1300 \times 4.5) + (100 \times 7)}$

$C_u = 35.63$ kPa

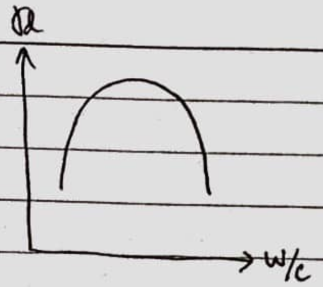
Hydrostatic Force.



9.81×6
$= 58.86$ kPa

Area of Δ = 176.58

(b)



The dry density of soil increases as the water content in the soil increases. This water provides lubrication, soften clay bonds, and reduce surface tension forces in the soil. However if the soil is too wet, then water starts to replace the soil particles, leaving less voids, thus becomes difficult to compact. The peak of the curve is known as the optimum dry density and the corresponding w/c is known as the optimum water content.

At dry of optimum, clays have more attractive force, hence giving a flocculated particles. Such soils have a higher permeability and strength. However at the wet of optimum, clays have more repulsive force, hence giving a dispersed particles (more oriented). Such soils have lower permeability because it has less tortuosity.

(c). For S = 75%. (upper limit).

$$w = 75\left(\frac{9.81}{17.7} - \frac{1}{2.7}\right) = 13.79\,\%$$

For S = 80%.

$$w = 80\left(\frac{9.81}{16.7} - \frac{1}{2.7}\right) = 17.36\%$$

(d). •) Precompression : Introduce consolidation to make the soil denser. The vertical drains are installed to enhance the reduction of pore water, thus accelerating the consolidation and settlement process. It works by providing more paths for water to drain from the soil.

(Clay / Silty Soil)

•) Dynamic Compaction : A method used in in-situ densification where it uses a special crane to lift (5-30 tons) weight (pounders) then drop it onto the ground several times with a certain pattern. The objective is to increase density, treat liquefaction-prone soil, collapsible soil, excess settlement

(loose sand, soils contain large boulders, rocks, etc.)

only effective to a shallow depth.

Yes, U Can!

Inject the grouts into the soil.

(e.) Grouting includes two primary kinds namely cementitious grouts (PCC) and Chemical grouts (other chemical substances). The principles in grouting are intrusion grouting (filling joints / fractures), permeation grouting (thin grouts to permeate voids), compaction grouting (stiff grout to compact the adjacent soils, supporting structures), Jet grouting (forming a column of treated soil inside the ground.)

mix the substance with the soil.

Admixtures are used to be mixed with soil, providing artificial cementation increasing strength, reducing compressibility. Most common : Surface mixing, where the upper soil is ripped and mixed with the admixtures, then compacted to form a hard and durable soils. New method is In-situ deep mixing : use a rotating mixer shaft, paddles, or jet that penetrates into the ground while injecting and mixing Portland Cement or other stabilizing agent.

All the Best !!

God Bless.

Kevin Janiardy.