NANYANG TECHNOLOGICAL UNIVERSITY SINGAPORE

April / May 2019.

CV2014 - Geotechnical Engineering

Date No.

1(a) $\tau_f = c' + \sigma'_{nf} \tan \phi'$

clean sand : $c' = 0$

normal stress : 800 kPa = 800 kN/m²

$\phi' = 40°$

$\tau_f = (800) \tan \phi' = 800 \tan(40) = 671.3$ kPa

Area of contact = 100 mm × 100 mm = 10000 mm = $0.01 m^2$

shear force = $\tau_f A = 800 \tan(40)(0.01) = \boxed{6.71 \text{ kN}}$

1(b) (i)

effective consolidation pressure = σ'_3 = 200 kPa

$\phi' = 30°$; since $c' = 0$, $\dfrac{\sigma'_1}{\sigma'_3} = \dfrac{1+\sin\phi'}{1-\sin\phi'}$

$\dfrac{\sigma'_1}{200} = \dfrac{1+\sin(30)}{1-\sin(30)} = 3$; $\sigma'_1 = 3(200) = 600$ kPa.

q = deviator stress at failure = $\sigma'_1 - \sigma'_3 = 600 - 200 = \boxed{400 \text{ kPa}}$

1(b) (ii)

The sample used for this consolidated drained triaxial test is likely to be at a loose state. The sample used for the direct shear test is likely to be at a dense state, since the friction angles for peak and ultimate states are distinct and peak friction angle is greater than the ultimate friction angle.

The sample used for the CD triaxial test is 30° which is closer to the ultimate state friction angle of 32° from the direct shear test, thus, the sample is at a loose state.

1(b) (iii)

Refer to Appendix.

1(c) (i)

total consolidation pressure : σ_3 = 200 kPa ; $\phi' = 30°$

σ_3 = 200 kPa ; $\sigma'_{3f} = \sigma_3 - u_f = 200 - 100 = 100$ kPa

Clean sand : $c' = 0$;

$\dfrac{\sigma'_{1f}}{\sigma'_{3f}} = \dfrac{1+\sin\phi'}{1-\sin\phi'} = \dfrac{1+\sin(30)}{1-\sin(30)} = 3$; $\sigma'_{1f} = 3(100) = 300$ kPa

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$q = \sigma'_{1f} - \sigma'_{3f} = 300\text{ kPa} - 100\text{ kPa} = \boxed{200\text{ kPa}}$

1(c) (ii)

undrained shear strength $= c_u = \frac{q_f}{2} = \frac{200}{2} = \boxed{100\text{ kPa}}$

1(c) (iii)

Refer to Appendix

1(c) (iv)

total consolidation pressure σ_3 remains the same : 200 kPa.

$\sigma'_3 = \sigma_3 - u = 200 - 198 = 2\text{ kPa}$

$\frac{\sigma'_1}{\sigma'_3} = \frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1+\sin(30)}{1-\sin(30)} = 3 = \frac{\sigma'_1}{\cancel{\sigma'_3}\ 2}$

$\sigma'_1 = 3(2) = 6\text{ kPa}$

q = deviator stress at the end = $6\text{ kPa} - 2\text{ kPa} = \boxed{4\text{ kPa}}$

1(c) (v)

Refer to Appendix.

2 (a)

1. Rankine's theory : Wall is vertical

   Coulomb's theory : Wall can be inclined ($\alpha \geq 0$)

2. Rankine's theory : Wall is perfectly smooth

   Coulomb's theory : Wall can be rough ($\delta \geq 0$)

3. Rankine's theory : The resultant force is parallel to the ground surface.

   Coulomb's theory : The resultant force is acting at an angle of δ with ~~to~~ the normal to ~~the~~ wall

4. Earth pressure derived from Rankine's theory tend to be more conservative than Coulomb's theory due to the disregard of the soil-structure frictional forces between the retaining wall and the soil body.

5. Rankine's theory is typically used when the top of the soil body is completely ~~horizontal~~ horizontal ($\beta = 0$), whereas ~~the~~ Coulomb ~~theory~~ can be used in situations where $\beta > 0$.

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2(b) (i)

$$K_A = \frac{1-\sin\phi'}{1+\sin\phi'} = \frac{1-\sin(30)}{1+\sin(30)} = \frac{1}{3}$$

active earth pressure at 5m (based on overlying sand layer):

$\sigma'_A = K_A \sigma'_z - 2c'\sqrt{K_A}$

sand: $c' = 0$; $\sigma'_A = K_A \sigma'_z$

$\sigma'_z = \sigma_z - u = [10 + 20(5)] - 5(10) = 110 - 50 = 60$ kPa

$\sigma'_A = (\frac{1}{3})(60) = 20$ kPa

Note: Don't forget the hydrostatic pressure due to the ~~flow~~ groundwater

$u_5 = \gamma_w h_w = (10)(5) = 50$ kPa.

total active earth pressure $= 20 + 50 = \boxed{70 \text{ kPa}}$

active earth pressure at 5m (based on underlying clay layer):

$\sigma_A = \sigma_z - 2c_u$

$\sigma_A = [10 + 5(20)] - 2(55) = 110 - 110 = \boxed{0 \text{ kPa}}$

active earth pressure at 10 m (based on clay layer):

$\sigma_A = \sigma_z - 2c_u$

$\sigma_A = [10 + 5(20) + 5(16)] - 2(55) = 190 - 110 = \boxed{80 \text{ kPa}}$

2(b) (ii)

at 0m depth, $\sigma'_A = (\frac{1}{3})(10) = 10/3$ kPa

↳ plot on the ~~next~~ next page →.

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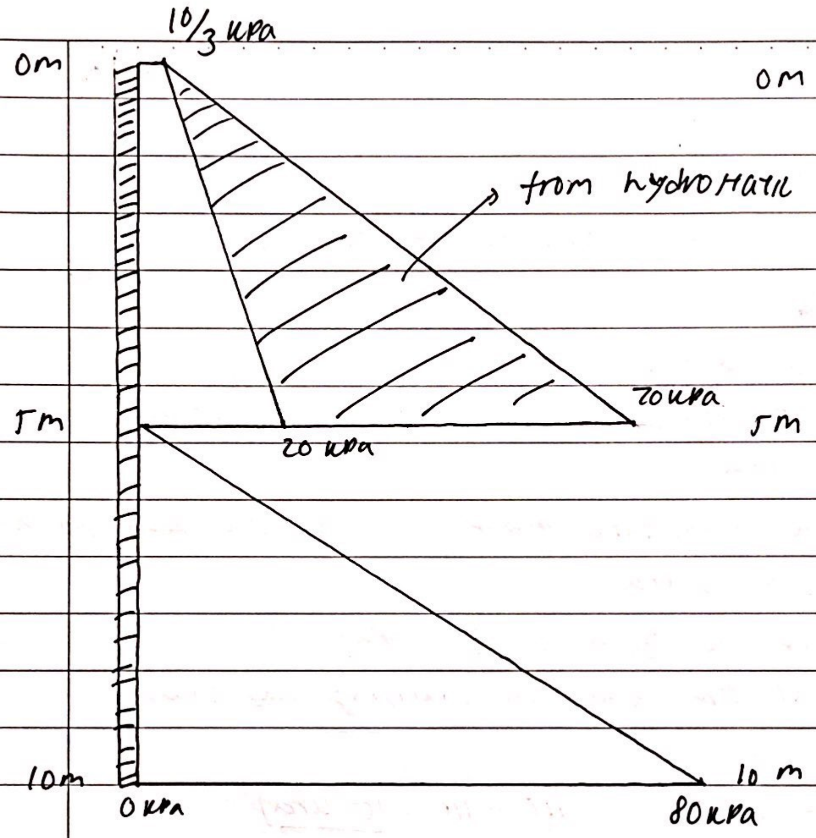


2(b) (iii)

Area of the earth pressure distribution plot = total thrust.

$A_1 = \frac{1}{2}(b_1 + b_2)(h) = \frac{1}{2}(10/3 + 70)(5) = 550/3$

$A_2 = (80)(5)(\frac{1}{2}) = 200$

$A_1 + A_2 = 200 + 550/3 = \cancel{383.33}$ $\boxed{383.33 \text{ kN/m}}$

2(b) (iv)

remove all hydrostatic pressure.

active earth pressure from sand increases by a bit at 5m depth.

$\sigma'_A = K_A(\sigma'_z)$; $\sigma'_z = \sigma_z = [10 + 5(20)] = 110$ kPa.

$\sigma'_A = (\frac{1}{3})(110) = 110/3$ kPa

$A_1 = \frac{1}{2}(b_1 + b_2)(h) = \frac{1}{2}(10/3 + 110/3)(5) = 100$ ~~kPa~~ kN/m

$A_2 = \frac{1}{2}(80)(5) = 200$

$A_1 + A_2 = 200 + 100 = \boxed{300 \text{ kN/m}}$

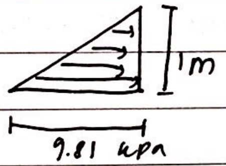
2(c)	segment the area into 3 identical square areas (25m x 25m):
	Apply Fadum's chart:
	$m = B/z = 25/2.5 = 10$
	$n = L/z = 25/2.5 = 10$
	$I_r = 0.2498$ → Refer to page 14's table in "Stress Distribution" lecture slides
	$\Delta\sigma_z = q\,I_r = (100\,kPa)(0.2498) = 24.98\,kPa$
	$\Delta\sigma_{z\,(total)} = 3(24.98) = \boxed{74.94\,kPa}$
3(a)	(i)
→	Total stress analysis: $\tau_f = c_u + \sigma_{nf}\tan\phi_u$, where $\phi_u = 0$ and thus, $\tau_f = c_u$ = undrained shear strength.
→	undrained condition is the condition by which excess porewater pressure is not allowed to dissipate.
→	fully saturated clay: All of our calculation are based off Terzaghi's consolidation theory, which is developed with the assumption that the soil is fully saturated. Clay is a fine-grained soil with low permeability that is most probable to sustain the undrained condition in the short term. [should there be a change in total stress]
→	Because the excess porewater pressure is not allowed to dissipate, the effective stress of the soil remains the same, and thus, the deviator stress remains the same, regardless of what value σ_3 is.
→	example of undrained condition in field: newly cut slope that is predominantly clay.
3(a)	(ii)
	$h_w = 1m$
	~~Factor of safety~~ = F = $\frac{\text{resisting moment}}{\text{driving moment}} = 1.5$
*	Driving moments:
	1) from line load: $P_t = P(7.5)$
	2) from self-weight of soil mass: $W_d = (1300\,kN/m)(4.6) = 5980$

* resisting moments :

1) shear along failure slip surface: $C_u L_a r = (50)(19)(12) = 11400$

2) hydrostatic force (thrust):

$\gamma_w h_w = (9.81)(1) = 9.81$



→ total hydrostatic thrust: $9.81 \times 1 \times \frac{1}{2} = 4.905$

→ hydrostatic thrust acts at $1/3$ from the base of the water : $\frac{1}{3} \times 1 = \frac{1}{3}$

→ distance from center of rotation: $H + h_c - \frac{1}{3} = 8 + 3.5 - \frac{1}{3}$
$= 67/6$

→ moment from hydrostatic thrust: $4.905 \times 67/6 = 54.7725$

* $F = \dfrac{11400 + 54.7725}{5980 + P(7.5)} = 1.5$

$11400 + 54.7725 = 1.5(5980 + P(7.5))$

$11454.7725 = 8970 + 11.25P$

$P = \dfrac{11454.7725 - 8970}{11.25} = \boxed{220.87 \text{ kN/m}}$

3(a) (iii)

$F = \dfrac{\text{resisting moment}}{\text{driving moment}} = 1$ → at failure.

* driving moments:

1) self-weight of soil mass $= Wd = (1300)(4.6) = 5980$

2) line load P : $Pe = P(7.5)$

* resisting moments:

1) shear force : $C_u L_a r = (50)(19)(12) = 11400$

* $F = \dfrac{11400}{5980 + P(7.5)} = 1$

$11400 = 5980 + P(7.5)$

$P = \dfrac{11400 - 5980}{7.5} = \dfrac{5420}{7.5} = \boxed{722.67 \text{ kN/m}}$

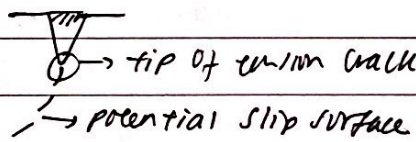
3(a) (iv)

→ tension cracks decreases the factor of safety of the slope and thus, reduces the stability of the clay slope in general.

→ Tension ~~crack~~ cracks develops at the crest of the slope as the ~~soil~~ soil mass tends to slip downwards due to its own self-weight, establishing tensile forces at the crest of the slope.

→ These tension ~~cracks~~ cracks opens up space for foreign material such as water to fill up. When water fills up the tension crack, it imposes hydrostatic pressure on the soil mass, increasing the driving moment and thus, decreasing the factor of safety.

→ The formation of a failure slip surface may begin from the tip of these tension cracks. The presence of water in the tension crack may cause the tension crack to grow in size, which will then result in more water entering the tension crack, resulting in a debilitating positive feed back loop.



3(b) 1) Drill one borehole at the toe and another one at the crest. If you have more funds then you can drill another borehole mid-height of the slope. This will be relatively more expensive as the ground surface is inclined and you have to build a platform for the drilling rig.

2) In order to ensure that the soil sample ~~is~~ collected is undisturbed for residual soil, drilling fluid (typically a foam / polymer) is used when advancing the borehole. This ensures that friction is minimal, disturbance due to mechanical vibrations are minimal and the water content is kept relatively constant.

3) Use piezometers to detect the location of the groundwater table, and use tensiometers to detect the negative porewater pressure above the GWT.

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4) Take the soil to the laboratory:
→ conduct appropriate triaxial tests to determine the strength parameters c', ϕ' and c_u;
→ Conduct soil classification based on USCS
→ Find out average unit weight for every soil type and each respective water density.

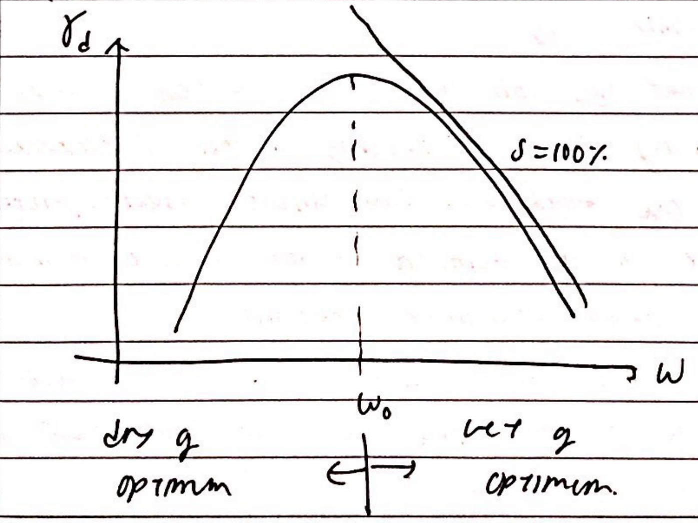
5) generate a generalised soil profile.

6) input the soil profile, GWT and strength parameters into software for slope stability analysis.

4(a) → Soft clay is improved through precompression. Essentially, in order for the soft clay to increase in strength, we allow it to undergo consolidation, where the effective stress of the soil will increase, and thus, the shear strength of the soil. Precompression uses a surcharge fill and prefabricated vertical drains to ensure that the consolidation occurs at a manageable time period (not too long).

→ Loose sand is improved via vibrations, through methods such as vibro-compaction and dynamic compaction. Vibratory compaction methods will cause the soil to increase in density, and increasing the density of the loose sand will increase the friction angle from the ultimate state to the peak state.

4(b)



→ Dry of optimum: clays have a flocculated fabric
↳ attractive forces is greater than repulsive forces, thus, the face of one clay particle is in contact with the edge of another clay particle.

→ Wet of optimum: Clays have an oriented fabric
↳ attractive forces are weaker than repulsive forces, thus, clay particles orient themselves in a more parallel manner.

* Clay compacted dry of optimum has a higher shear strength and higher permeability than clay compacted wet of optimum.

4(c) (i)

$\gamma = \gamma_d (1+w)$; $w = 0.22$;

$15.5 \text{ kN/m}^3 = \gamma_d (1 + 0.22)$

$$\gamma_{d0} = \frac{15.5}{1+0.22} = \frac{15.5}{1.22} = 12.705 \text{ kN/m}^3 = \frac{M_s g}{V_0}$$

$$\gamma_{dA} = \frac{M_s g}{V_A} = \frac{M_s g}{1 \text{ m}^3} = 16.5 \text{ kN/m}^3$$

$M_s g$ is the same in both γ_{d0} and γ_{dA}, since the mass of soil solids and gravitational accelerations are constants.

$M_s g = 16.5$ kN

$$\frac{16.5 \text{ kN}}{V_0} = 12.705 \text{ kN/m}^3$$

$V_0 = 16.5 / 12.705 = 1.298709677 \text{ m}^3 \approx \boxed{1.3 \text{ m}^3}$

4(c) (ii)

* Original:

density of water $= 1000 \text{ kg/m}^3$

$G = 2.7$; density of soil solids $= 2.7 \times 1000 = 2700 \text{ kg/m}^3$

$$\text{volume of soil solids} = \frac{\text{Mass of soil solids}}{\text{density of soil solids}} = \frac{(16.5 \times 1000)/9.81}{2700} = 0.6229471061$$

$$e = \frac{V_v}{V_s} = \frac{V_T - V_s}{V_s} = \frac{1.3 - 0.623}{0.623} = 1.086677368$$

$$S_r = \frac{w G_s}{e} = \frac{(0.22)(2.7)}{1.0847} = \boxed{0.548}$$

$$n = \frac{V_v}{V_T} = \frac{V_T - V_s}{V_T} = \frac{1.3 - 0.623}{1.3} = \boxed{0.520}$$

* Compacted:

Volume of soil solids: same as mass doesn't change.

$= 0.623$

$$e = \frac{V_T - V_s}{V_s} = \frac{1 - 0.623}{0.623} = 0.60527272 73$$

$$S_r = \frac{w G_s}{e} = \frac{\left(\frac{13.5}{100}\right)(2.7)}{0.605} = 0.6022078702.$$

$$\approx \boxed{0.602}$$

$$n = \frac{V_T - V_s}{V_T} = \frac{1 - 0.623}{1} = \boxed{0.377}$$

4(c) (iii)

If the soil is clay, dry of optimum, because the flocculated fabric of clay will give rise to a higher shear strength.

~~is consistent with [illegible]~~

~~My~~

Drop me an email if you have any questions → Sam (~~SA0001 JA@~~ [illegible])

(SA0001JA@e.ntu.edu.sg)

APPENDIX

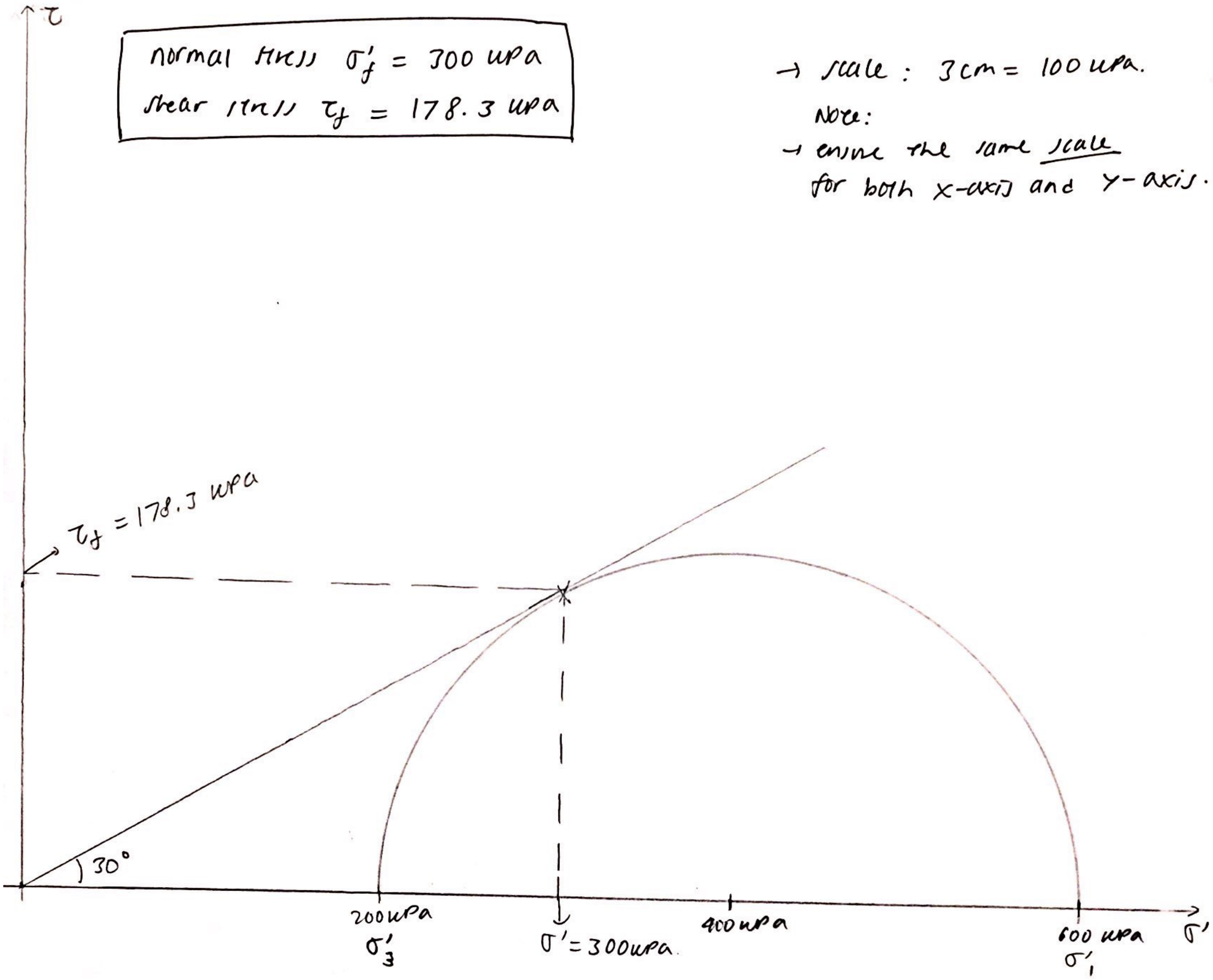
1(b)(iii)

normal stress σ'_f = 300 kPa
shear stress τ_f = 178.3 kPa

→ scale : 3cm = 100 kPa.

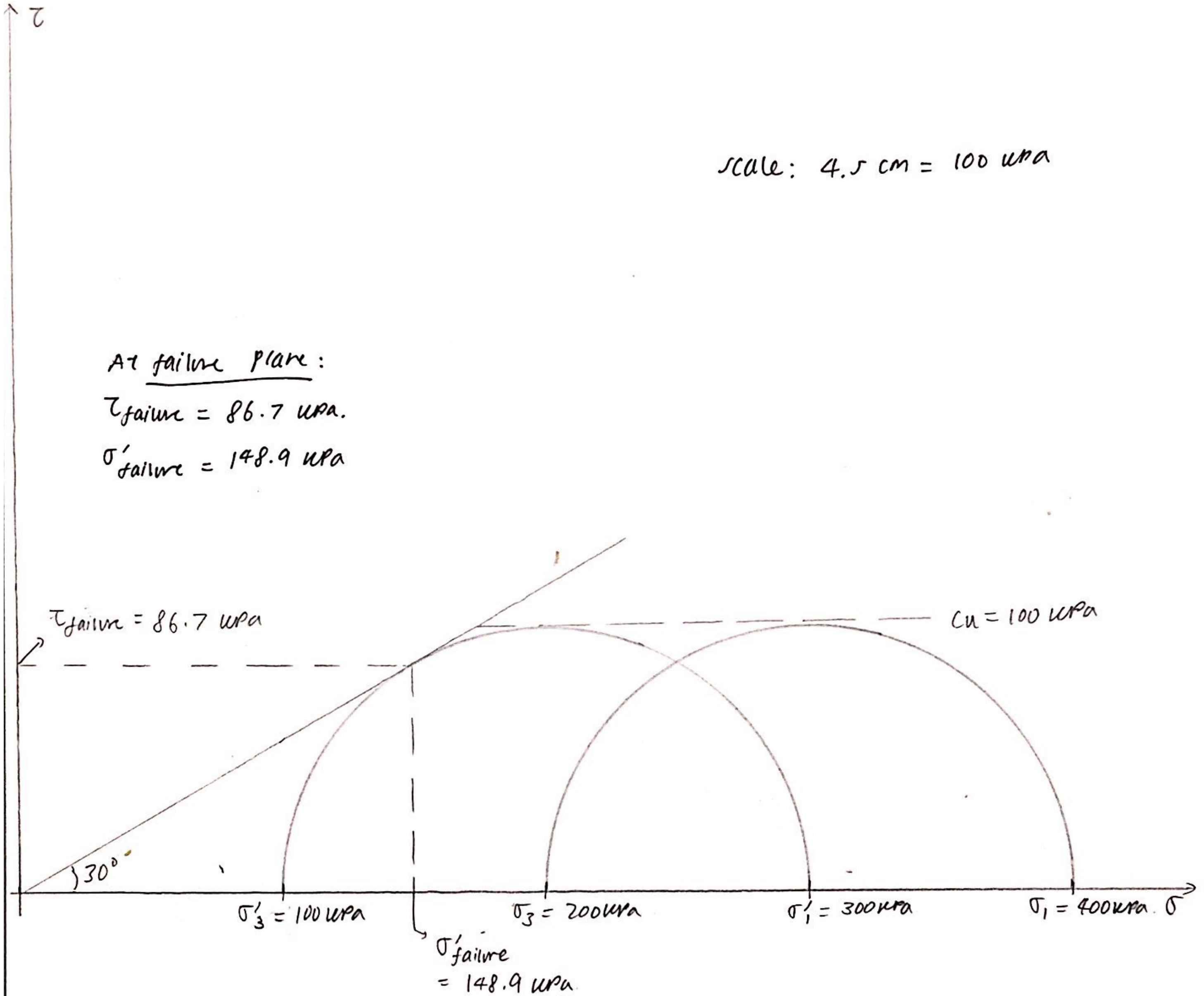
Note:

→ ensure the same scale for both x-axis and y-axis.

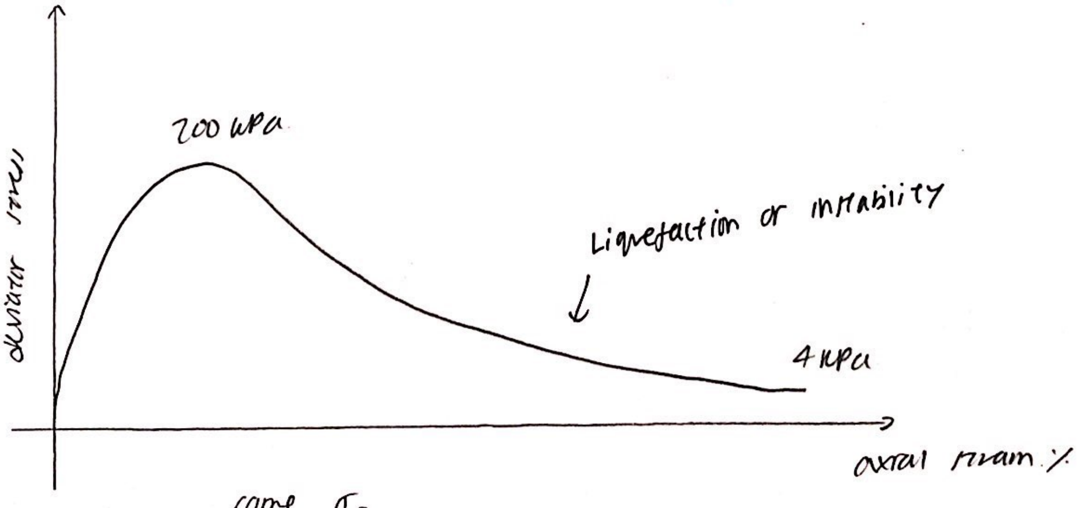


APPENDIX

1(c)(iii)



1(c)(v)



same σ_3.