NANYANG TECHNOLOGICAL UNIVERSITY

SEMESTER 2 EXAMINATION 2020-2021

CV2014 – GEOTECHNICAL ENGINEERING

April / May 2021 **Time Allowed: 2½ hours**

INSTRUCTIONS

1. This paper contains **FOUR (4)** questions and comprises **FIVE (5)** pages.
2. Answer all **FOUR (4)** questions.
3. ALL questions carry equal marks.
4. This is a Restricted Open-Book Examination. You may bring in **ONE (1)** piece of A4 size paper with notes written on both sides.
5. All answers must be written in the answer book provided. Answer each question beginning on a **FRESH** page of the answer book.

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1. (a) A direct shear test was carried out on a clean sand under a normal stress of 100 kPa. The shear stress obtained at the peak and the end of the test was 84 and 65 kPa, respectively.

(i) Determine the peak friction angle of this sand.

(ii) Sketch schematically the variation of the void ratio of the sand versus the shear displacement curve.

(iii) If the same sand was tested at a very loose state under the same normal stress of 100 kPa, what would be the anticipated peak friction angle? Explain the reasons behind your answer briefly.

(iv) Explain why the direct shear test is not suitable for the determination of undrained shear strength of clay.

(10 Marks)

(b) An isotropically consolidated drained (CD) triaxial test was carried out on a clean sand sample under an effective cell pressure of 200 kPa. The initial dimensions of the test sample were 50 mm in diameter and 100 mm in height. The maximum axial load measured was 877.4 N. The axial deformation and volumetric change corresponding to the maximum load was 15 mm and 9812.5 mm^3, respectively. No dilation was observed from this test.

(i) Determine the failure deviator stress obtained from this test.

(ii) Determine the friction angle of the sand.

(iii) Was the sand dense or loose? Sketch schematically the deviator stress versus axial strain and the volumetric strain versus axial strain curves for this CD test.

(iv) When the same sand was used for an isotropically consolidated undrained (CU) triaxial test under an effective cell pressure of 200 kPa, the sample collapsed at the end of the test. What is the likely excess pore water pressure at the end of the test? Sketch schematically the deviator stress versus axial strain and the pore water pressure change versus axial strain curves for this CU test.

(v) In the undrained test stated in part (iv), would the maximum axial load obtained be greater or smaller than 877.4 N? Explain your answer briefly.

(15 Marks)

2. (a) Describe the difference between the assumptions of Rankine's earth pressure theory and the assumptions of Coulomb's earth pressure theory, in terms of the properties of the retaining wall.

(4 Marks)

(b) A 6 m high retaining wall is shown in Figure Q2. The soil behind the wall consists of 3 m thick sand overlaying a layer of clay. The water table is at the base of the sand (i.e. 3 m below the ground surface). A uniformly distributed load of 6 kPa is applied at the ground surface. The sand and clay properties are given in Figure Q2.

(i) Calculate the active lateral earth pressures at 3 m and 6 m depths using the Rankine's theory.

(ii) If the water table rises by 1 m (i.e. 2 m below the ground surface), plot the active lateral earth pressure distribution and water pressure distribution along the wall. Assume the unit weight and effective friction angle of the sand do not change due to the rise in water table.

(iii) Will the total thrust acting on the wall change due to the water table rise in part (ii)?

(21 Marks)

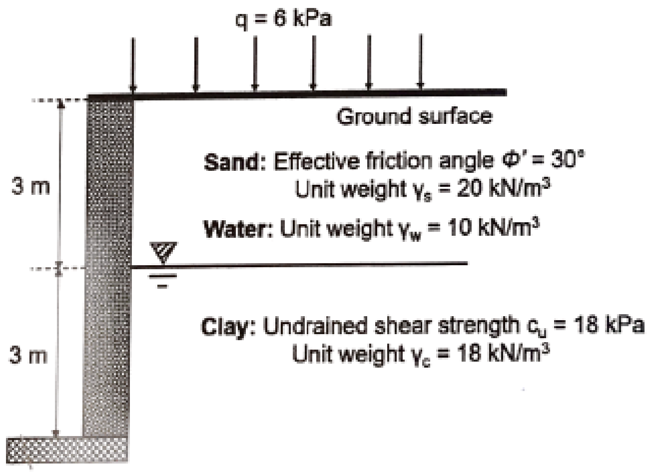


Figure Q2

3. Calculation of factor of safety using the method of slices is illustrated in Figure Q3.

(i) Briefly describe the principles used in the method of slices.

(5 Marks)

(ii) Derive the factor of safety equation in terms of effective stress analysis based on moment equilibrium.

(5 Marks)

(iii) Derive the factor of safety equation in terms of total stress analysis based on moment equilibrium.

(5 Marks)

(iv) A homogeneous slope has the following properties $c' = 12$ kPa, $\phi' = 33°$, $c_u = 35$ kPa, $\phi_u = 0$, $\sum_i N_i' = 1055$ kN/m, L_a (arc length AC) $= 33$ m, $\sum_i W_i \sin \alpha_i = 1027$ kN/m. For failure surface AC, calculate the factor of safety (F) of the slope for a long-term loading and the factor of safety (F) of the slope for a short-term loading.

(5 Marks)

(v) What cause the differences between the different methods of slices such as Fellenius, Bishop and Spencer methods? Briefly describe the differences between these three methods.

(5 Marks)

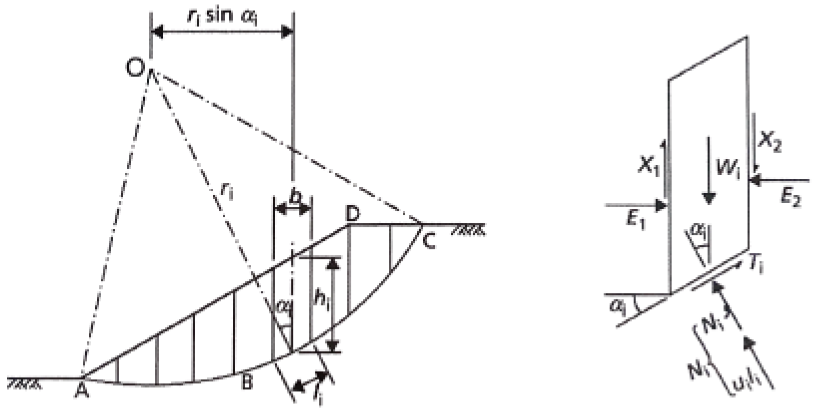


Figure Q3

4. (a) Soil improvement of a 40 m^3 saturated soil requires the following compaction procedure. The saturated soil has a water content of 25% and a specific gravity of the soil solids of 2.7. The saturated soil is first dried to a water content of 14% and then compacted to a dry unit weight of 17.5 kN/m^3. The solid mass in the soil remains constant. Calculate the mass of water that has to be removed from this 40 m^3 of saturated soil and the degree of saturation of the soil after compaction.

(10 Marks)

(b) Different compaction curves of a soil with different optimum conditions are obtained by using different compactive efforts. The different optimum conditions have a degree of saturation ranging from 77% to 82% and a maximum dry unit weight ranging from 16.5 kN/m^3 to 17.8 kN/m^3. Calculate the range of water content corresponding to the different optimum conditions. The specific gravity of the soil solids is 2.7.

(5 Marks)

(c) (i) Briefly describe the mechanism of precompression as soil improvement using soil mechanics principles. Use sketches if necessary.

(ii) Briefly explain the reason why horizontal drains can increase factor of safety of slope using the effective stress principles. Use sketches if necessary.

(iii) Briefly describe the principle behind the use of geotextiles for soil reinforcement.

(10 Marks)

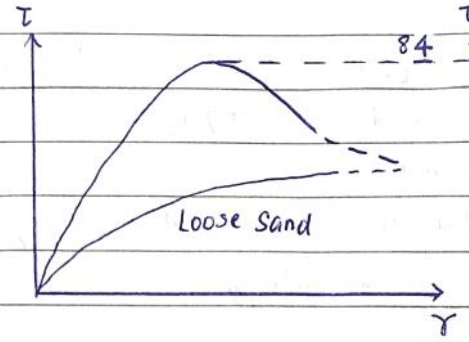
END OF PAPER

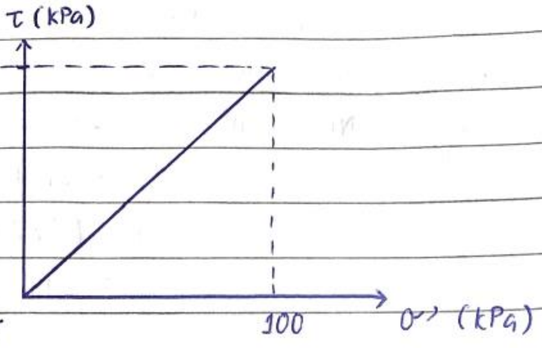
CV2014 Geotechnical Engineering
Semester 2 Examinations 2020-2021
Suggested Solutions by Lim Kai Jian

Q1

Q1(a) Direct Shear Test

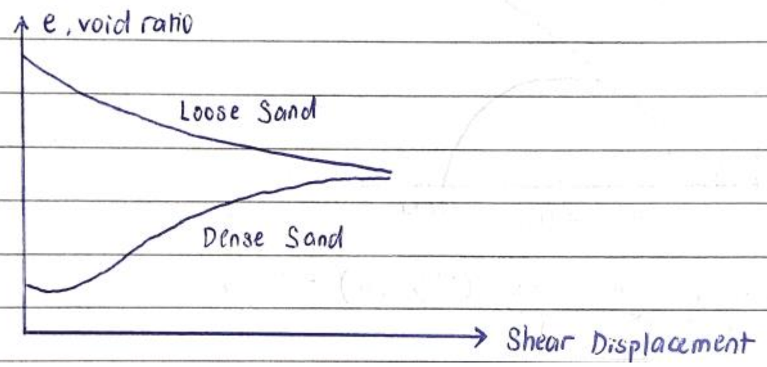
(i)





Peak $\phi' = \tan^{-1}(84/100) = 40.03°$ //

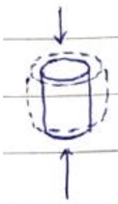
(ii)



(iii) At very loose state, friction ∡ (ϕ') = $\tan^{-1}(65/100) = 33.02°$ //
The peak friction angle for loose sand corresponds to the ultimate state friction angle, which is derived from the shear stress at the end of the shear test.

(iv) This is because an undrained condition cannot be imposed in a direct shear box, however it can be used as a rough method to shear the clay sample quickly and hope that there is no time for pore pressure to dissipate yet.

Q1 (b)(i) CD Triaxial Test



Deviator Stress = F/A

New Cell Volume = $\pi\left(\frac{50}{2}\right)^2(100) - 9812.5 = 186\,537.04\ mm^3$

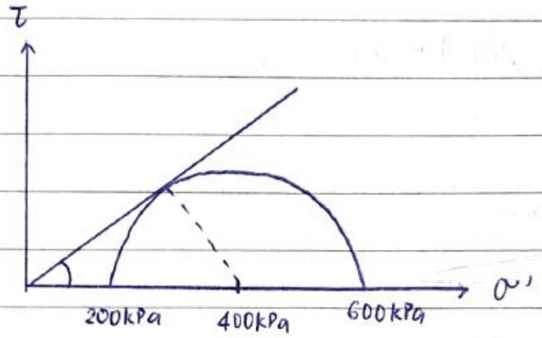
New Cell Radius $= \left\{\left[186\,537.04/(100-15)\right]/\pi\right\}^{0.5}$

$= 26.43\ mm$

$q = (877.4 \times 10^{-3}) / (\pi\,(26.43 \times 10^{-3})^2)$

$= 399.8 = 400\ kPa$ //

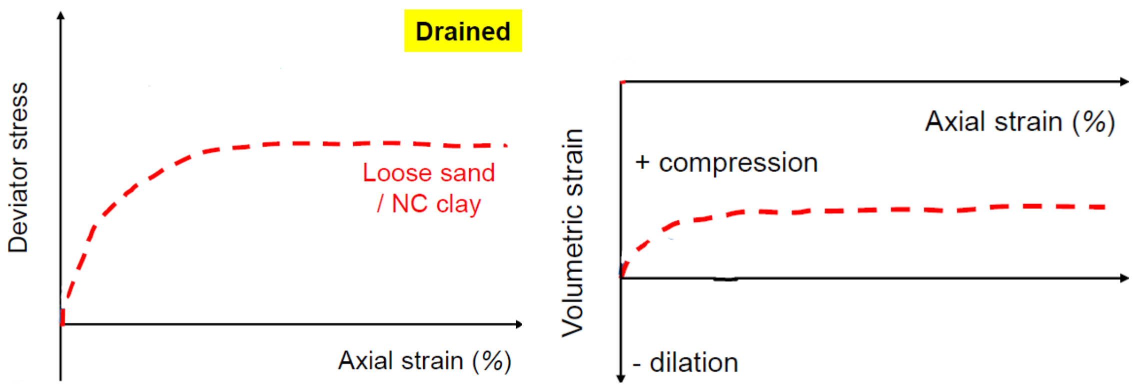
(b)(ii)



Friction Angle $= \sin^{-1}(200/400) = 30°$ //

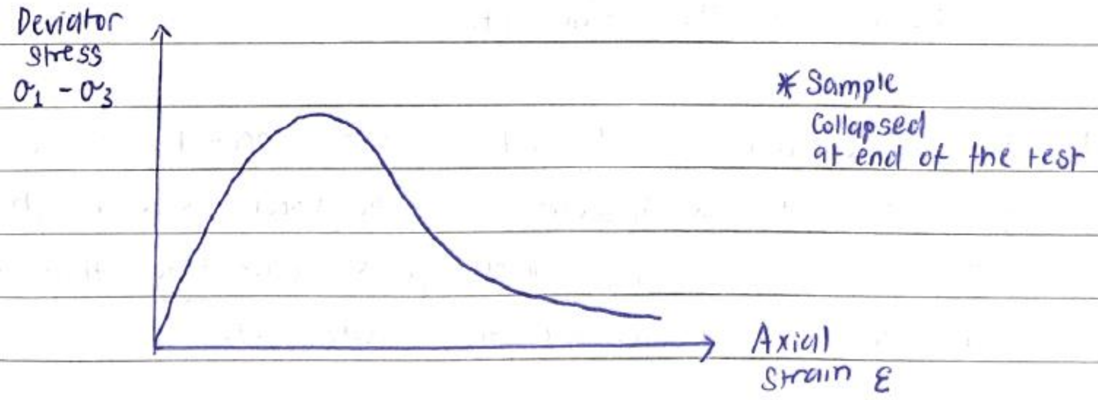
(b)(iii)

The sand was loose since no dilation was observed from the test.



(iv) CU Test: (Loose sand)

Likely excess pore water pressure at end of test
= 200 kPa (Initial consolidation pressure)



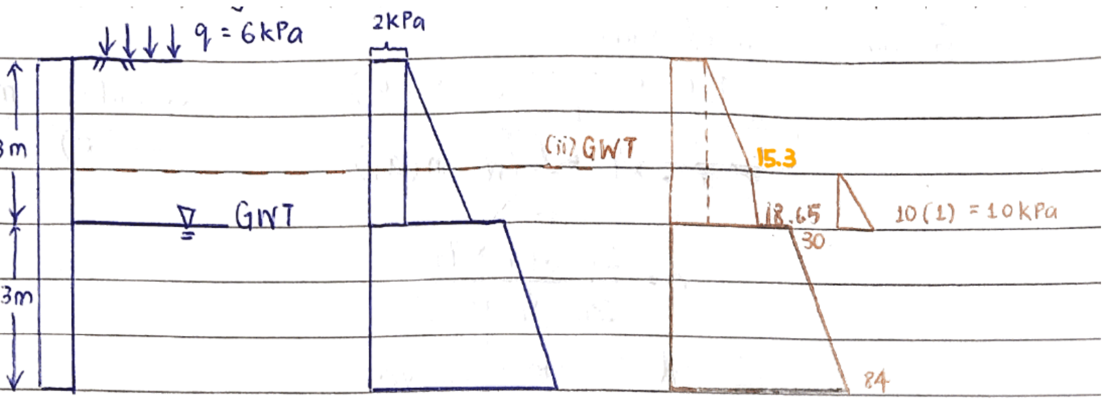
(v) The maximum axial load obtained for CU test will be smaller than 877.4 N (max. axial load for CD test). This is due to the nature of the CU test which results in pore water pressure build up and thus resulting in liquefaction or instability.

Q2 (a)

Difference between Rankine's earth pressure theory and Coulomb's earth pressure theory:

Assumptions	**Rankine**	**Coulomb**
Wall Back	Smooth wall back	Interface friction between wall and soil
Failure Surface	No assumption about failure surface	Assumes failure surface to be a plane
Soil Type	No assumption	Cohesionless soil (c' =0)

Q2 (b)



(i) Sand : $K_a = {1 - \sin 30°}/{1 + \sin 30°} = 0.333$

At 3m depth, $\sigma_A' = (6)(0.333) + (20)(3)(0.333)$

$= 22$ kPa

At 6m depth, $\sigma_A = (6 + 3 \times 20 + 3 \times 18) - 2(18)$

$= 84$ kPa

(ii) If GWT rises to 2m below ground surface:

At 2m, $\sigma_A' = (6)(0.333) + (20)(2)(0.333) = 15.3\,kPa$

At 3m, $\sigma_A' = 15.3 + (10)(0.333) = 18.65$

(iii) Initial Total Thrust $= 0.5(3)(2+22) + 0.5(30+84)(3)$

$= 207\,kN/m$

Total Thrust in (ii) $= 0.5(2)(2+15.3) + 0.5(1)(15.3+18.65)$

$+ 0.5(30+84)(3) + 0.5(10)(1)$

$= 210.3\,kN/m$

Δ in total thrust $= 210.3 - 207 = 3.3\,kN/m$

$\Rightarrow$ Total Thrust Increases by $3.3\,kN/m$

Q3 (i)

- The potential failure surface of the slope is assumed to be a circular arc with centre O and radius r.
- Analysis is based on a lumped factor of safety whereby it is equals to the ratio of shear strength at failure to shear strength mobilised.
- The factor of safety for each slice is taken to be the same
- There must be forces acting between the slices due to mutual support, thus making the slope stability problem statically indeterminate

Q3 (ii) Method of Slices:

$$\sum_i T_i r_i = \sum_i W_i (r_i \sin \alpha_i) \quad \text{(Considering moments about O)}$$

$$\sum_i \frac{T_{f,i}}{F} \times l_i = \sum_i W_i \sin(\alpha_i)$$

$$F = \frac{\sum_i T_{f,i} \times l_i}{\sum_i W_i \sin(\alpha_i)}$$

For effective stress analysis:

$$F = \frac{\sum_i (c_i' + \sigma_i' \tan \phi_i') \times l_i}{\sum_i W_i \sin(\alpha_i)}$$

$$= \frac{c' L_a + \tan\phi' \sum_i N_i'}{\sum_i W_i \sin \alpha_i} \quad (\because L_a = \sum_i l_i)$$

(iii) For total stress analysis:

$$\phi_u = 0, \quad \tan \phi_u = 0$$

$$F = \frac{c_u L_a}{\sum_i W_i \sin \alpha_i}$$

(iii) For total stress analysis :

$\phi_u = 0$, $\tan \phi_u = 0$

$$F = \frac{c_u L_a}{\Sigma_i W_i \sin \alpha_i}$$

(iv) F_s for long term loading (drained)

$$= \frac{12(33) + \tan 33^\circ (1055)}{1027}$$

$= 1.053$

F_s for short term loading (undrained)

$$= \frac{35(33)}{1027}$$

$= 1.125$

(v)

- The difference between these methods is due to the way in which the forces N'_i are estimated.
- Fellenius method assumes that for each slice, the resultant of the interslice forces is zero, and it is based on moment equilibrium.
- Bishop assumes that the resultant forces of the slices are horizontal and there are not shear forces.
- Spencer assumes that the resultant interslice forces are parallel in which both force and moment equilibrium are satisfied.

Q4 (a)

Initial State :

$w = 0.25$, $\gamma_d = 9.81(2.7) / 1 + \frac{0.25(2.7)}{1} = 15.81 \text{ kN/m}^3$

(since $S_r = 1$)

$\gamma = \gamma_d(1+w) = 15.81(1+0.25) = 19.766 \text{ kN/m}^3$

$\gamma_d = M_s g / V$

$M_s = \gamma_d \times V \times 1/g = 15.81 \times 40 \times 10^3 \times 1/9.81$

$= 64464.83 \text{ kg}$

$M_{w_i} = 0.25(64464.83) = 16116 \text{ kg}$

After Soil Improvement :

$w = 0.14$, $\gamma_d = 17.5 \text{ kN/m}^3$

$M_{w_f} = 0.14(64464.83) = 9025.08 \text{ kg}$

Mass of water req'd to remove

$= 16116 - 9025 = 7091 \text{ kg}$ //

$$\gamma_d = \frac{\gamma_w G_s}{1 + \frac{w G_s}{S_r}}$$

$$17.5 = \frac{9.81(2.7)}{1 + \frac{0.14(2.7)}{S_r}}$$

$S_r = 0.736$

$= 73.6\%$ //

Q4 (b)

$\gamma_d = 16.5\,kN/m^3$

$S_r = 0.82$

$$\gamma_d = \frac{\gamma_w G_s}{1 + \frac{w G_s}{S_r}}$$

$$1 + \frac{w G_s}{S_r} = \frac{\gamma_w}{\gamma_d} G_s$$

$$\frac{w G_s}{S_r} = \frac{\gamma_w}{\gamma_d} G_s - 1$$

$$w = \left(\frac{\gamma_w}{\gamma_d} G_s - 1\right) \times \frac{S_r}{G_s}$$

$$= 0.184$$

$\gamma_d = 17.8\,kN/m^3$

$S_r = 0.77$

$$w = \left(\frac{9.81}{17.8} \times 2.7 - 1\right) \times \frac{0.77}{2.7}$$

$$= 0.139$$

$\therefore$ Range of w is 13.9 % to 18.4 % //

Q4(c)

(i) Precompression is performed by covering the soil layer with a temporary surcharge fill, causing soil to consolidate thus improving both their settlement and strength properties. It is useful for clayey and silty soils. The additional surcharge results in dissipation of porewater pressure over the long run and hence the effective stress of the soil increases since effective stress = total stress – porewater pressure.

(ii) Horizontal drains help to provide a pathway for water to drain out thus preventing build up of porewater pressure in the soil. Therefore, this helps to ensure that the effective stress of the soil remains high, thus ensuring that the factor of safety for slope stability to be increased, since effective stress contributes to the resistance.

(iii) Geotextiles help to increase the tensile strength of the soil. Soil is stronger in compression than in tension. The use of geotextiles helps to increase the strength and hence allowing slopes to be built more steeply.

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