PROBLEM 1

CRIB - 3D1 - 2019

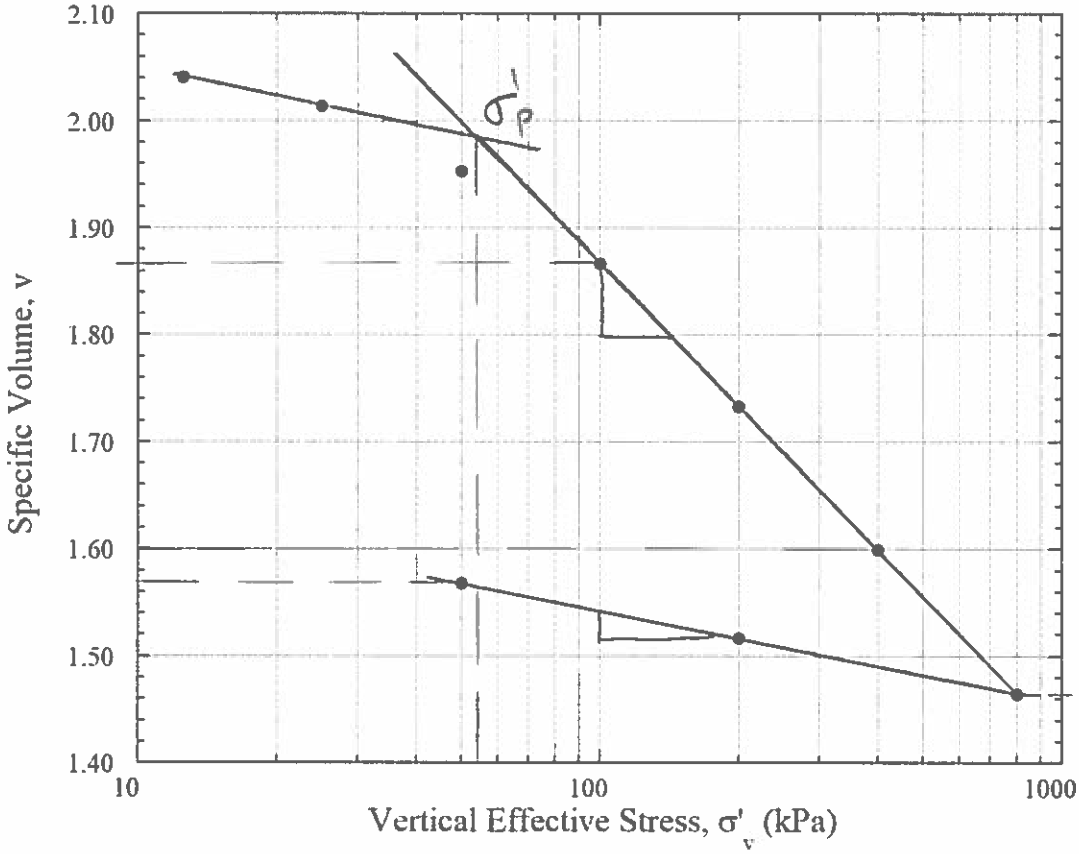
a)

Version GB/1 Candidate Number:

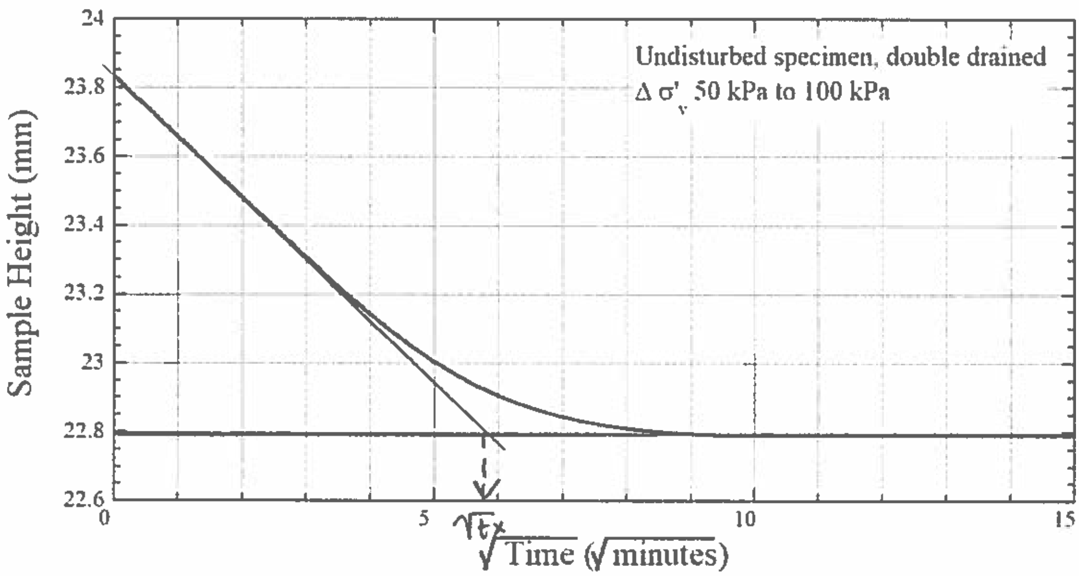
EGT2

ENGINEERING TRIPOS PART IIA

Wednesday 24 April 2019, Module 3D1, Question 2.



Extra copy of Fig. 1: Consolidation curve for question 2.



Extra copy of Fig. 2: Consolidation curve for question 2.

(a)

$$\lambda = -\frac{1.600 - 1.868}{\ln(400/100)} = 0.193$$

$$\kappa = -\frac{1.465 - 1.570}{\ln(400/50)} = 0.050$$

$$\sigma'_p = 55 \text{ kPa}$$

$$\sqrt{t_x} = 5.7 \text{ min} \qquad t_x = 32.49 \text{ min}$$

$$d = \frac{23.82 + 22.8}{2} = 23.3 \text{ mm}$$

$$c_v = \frac{3}{4}\frac{d^2}{t_x} = \frac{3}{4}\frac{(23.3 \text{ mm})^2}{32.49 \text{ min}}\left(\frac{1 \text{ m}}{100 \text{ mm}}\right)^2\left(\frac{1440 \text{ min}}{1 \text{ day}}\right) = 1.48 \frac{\text{m}^2}{\text{d.}}$$

(b) $$e = \frac{G_s w}{S} = \frac{(2.68)(0.391)}{(1)} = 1.047$$

$$\gamma_{sat} = \frac{G_s + e}{1 + e}\gamma_w = \frac{2.68 + 1.047}{1 + 1.047}\left(9.8 \frac{\text{kN}}{\text{m}^3}\right) = 17.8 \frac{\text{kN}}{\text{m}^3}$$

$$\sigma'_v = (17.8 - 9.8)(6) = 48 \text{ kPa}$$

$$\Delta v = (0.05)\ln\left(\frac{55}{48}\right) + (0.193)\ln\left(\frac{148}{55}\right) = 0.198$$

$$\Delta H = \frac{\Delta v}{v_0} H_0 = \frac{0.198}{1 + 1.047}(12 \text{ m}) = 1.16 \text{ m}$$

(c) $t_{90} = \dfrac{T_{90} d^2}{c_v} = \dfrac{(0.848)(6m)^2}{(1.48 m^2/yr)} = 20.6 yrs$

(d) $R_v = \dfrac{70}{116} = 0.60$

$T_v = 0.30$

$C_v = \dfrac{T d^2}{t} = \dfrac{(0.30)(6m)^2}{(3yr)} = 3.6 \dfrac{m^2}{yr}$

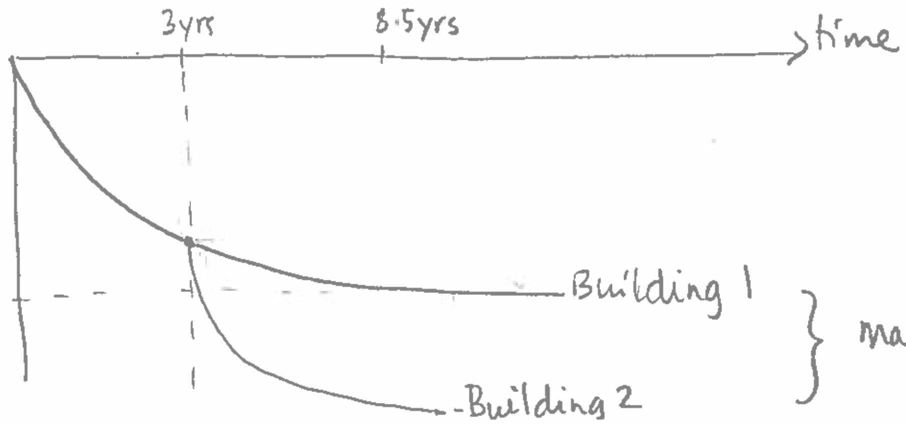
Cv measured in the laboratory does not account for large scale features such as sand or silt seams, or fractures. In general, the hydraulic conductivity is also larger in the horizontal direction than in the vertical one. The test, and the theory, do not account for horizontal drainage.

$t_{90} = \dfrac{(0.848)(6m)^2}{(3.6 m^2/yr)} = 8.5 yrs$

(e) For the second building

$\Delta v_2 = (0.05) \ln\left(\dfrac{55}{48}\right) + 0.193 \ln\left(\dfrac{40+75}{55}\right) = 0.149$

$\Delta H_2 = \dfrac{0.149}{1+1.047}(12m) = 0.87 m$



max $\Delta H = 87 - (116 - 70) = 46cm$

(f) $C_v = \frac{k E_0}{\gamma_w}$

It is likely k (hydraulic conductivity) will not change much - γ_w does not change at all.

The largest difference is caused by the change in stiffness.

$E_0^{(load)} = \frac{v\sigma'}{\lambda}$

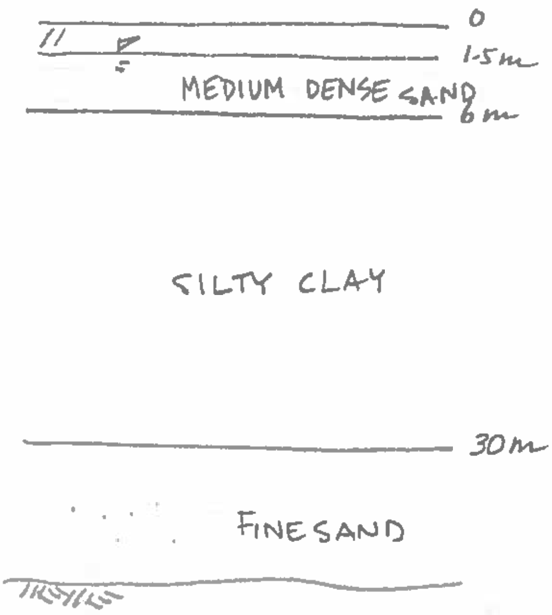
$E_0^{(unload)} = \frac{v\sigma'}{\kappa}$

v and σ' could be a little different, but not as much as λ and κ

$$C_v^{(unload)} = C_v^{(load)} \frac{E_0^{(unload)}}{E_0^{(load)}} = C_v^{(load)} \frac{\lambda}{\kappa} = \left(3.6\frac{m^2}{yr}\right)\left(\frac{0.193}{0.05}\right) = 13.9\frac{m^2}{yr}$$

Most popular question, with all students attempting it. Students displayed good knowledge of basics and were able to derive soil parameters and calculate settlements. Very few students attempted to calculate the coefficient of consolidation in unloading.

PROBLEM 2



$G_s = 2.65 \qquad \rho_d = 1{,}700\ \mathrm{kg/m^3}$

(a) Medium dense sand

$$\gamma_d = \rho_d g = (1700\ \mathrm{kg/m^3})(9.8\ \mathrm{m/s^2}) = 16.7\ \frac{\mathrm{kN}}{\mathrm{m^3}}$$

$$\gamma_d = \frac{G_s}{1+e}\gamma_w \Rightarrow 1+e = \frac{G_s \gamma_w}{\gamma_d} \qquad 1+e = \frac{(2.65)(9.8\ \mathrm{kN/m^3})}{16.7\ \mathrm{kN/m^3}} = 1.558$$

$$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w = \frac{2.65+0.558}{1.558}\left(9.8\frac{\mathrm{kN}}{\mathrm{m^3}}\right) = 20.2\ \frac{\mathrm{kN}}{\mathrm{m^3}}$$

Use γ_d above water table and γ_{sat} below. Minimal capillary rise is expected.

Silty clay

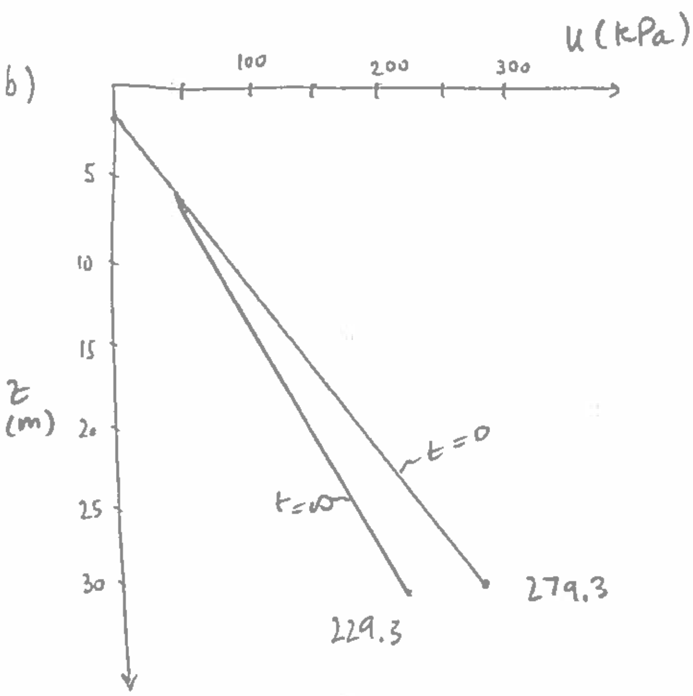
$$w = 54.6\%$$

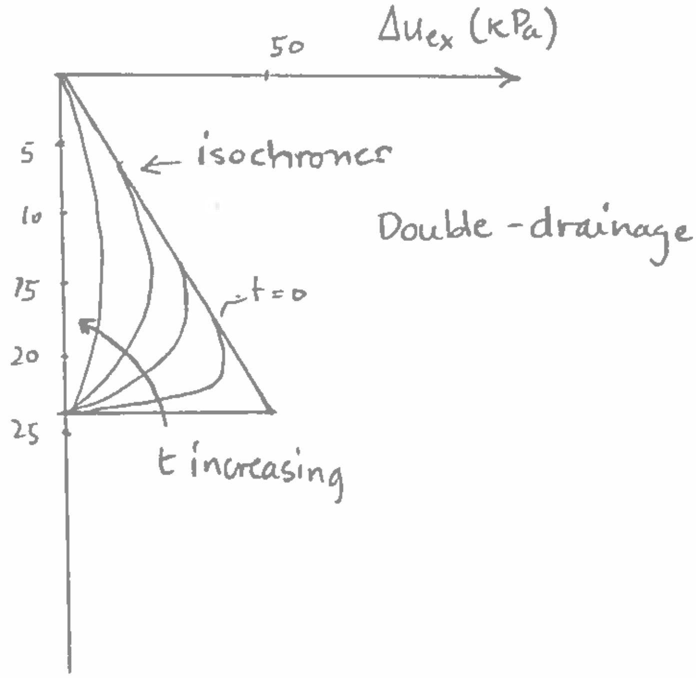
$$e = (0.546)(2.61) = 1.425$$

$$\gamma_{sat} = \frac{2.61+1.425}{2.425}\left(9.8\frac{\mathrm{kN}}{\mathrm{m^3}}\right) = 16.3\ \frac{\mathrm{kN}}{\mathrm{m^3}}$$

z (m)	σ_v (kPa)	u (kPa)	σ'_v (kPa)
0	0	0	0
1.5	20.05	0	20.05
6	115.95	44.1	71.85
11	197.45	93.1	104.35
23	393.05	210.7	182.35
30	507.15	279.3	227.85

b)





c) $\lambda = 0.26$ $\kappa = 0.05$

$\sigma'_{p}(11m) = OCR\,\sigma'_v = (1.75)(104.35) = 182.6\,kPa$

At 11m

$\Delta\sigma'_v = \left(\frac{50kPa}{24m}\right)(5m) = 10.42\,kPa$

The silty clay will be overconsolidated after pumping

$\Delta v = (0.05)\ln\left(\frac{104.35 + 10.42}{104.35}\right) = 0.005$

At 23m

$\Delta\sigma'_v = \left(\frac{50kPa}{24m}\right)(17m) = 35.42\,kPa$

$\Delta v = \lambda \ln\left(\frac{\sigma'_v + \Delta\sigma'_v}{\sigma'_v}\right) = (0.26)\ln\left(\frac{182.35 + 35.42}{182.35}\right) = 0.046$

Assuming the changes in specific volume can be representative for the whole soil layer

$\Delta h = \left(\frac{0.005}{1+1.425}\right)(10m) + \left(\frac{0.046}{1+1.425}\right)(14m) = 28.6\,cm$

d) When pumping is stopped the pore pressure profile eventually returns to its original state and the effective stress decreases. Unloading, however, only partially reverses the settlements. Ideally swelling in the upper part of the clay layer is elastic, but only κ/λ portion of the settlement is recovered in the lower part of the soil.

Students confidently calculated the stresses with depth and identified the boundary conditions. Only a portion of the students recognised that water pumping will create a decrease in pore water pressure which varies with depth. In the calculation of settlements a few students did not recognise the overconsolidated vs normally consolidated regimes.

PROBLEM 3

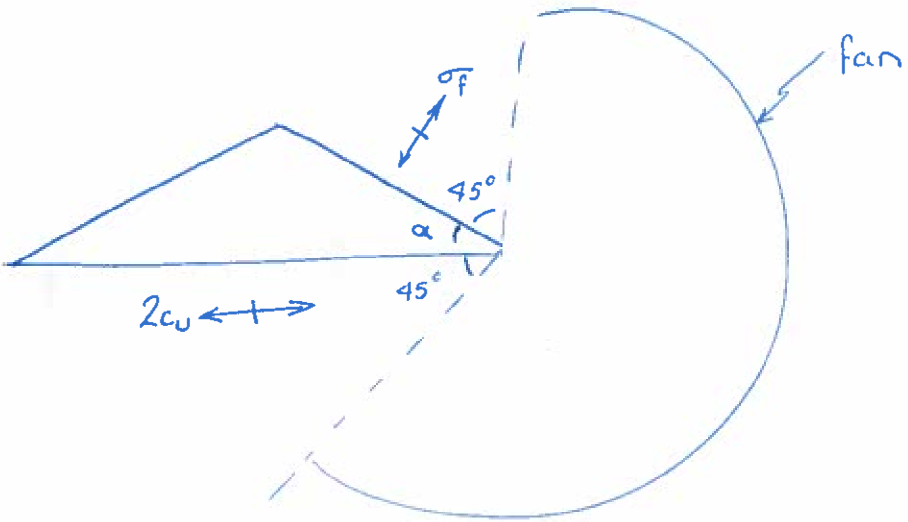
a) Lower bound

If a state of stress exists which is in equilibrium and does not violate yield this is a lower bound on loads that can be carried.

b) Upper bound

If a compatible mechanism dissipates the work done by the external loads, this is an upper bound on the load.

b)



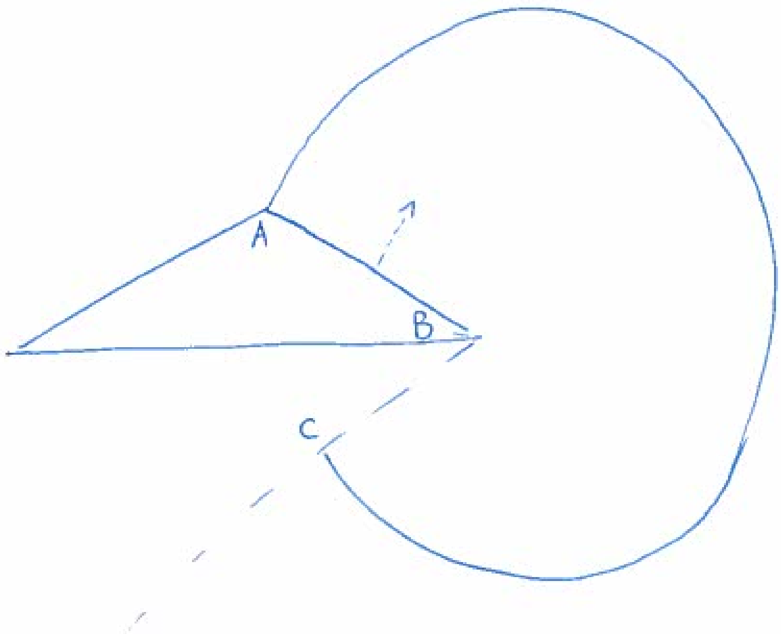
$$\sigma_f = 2c_u + \left(\frac{3\pi}{2} - \alpha\right) \times 2c_u$$

$$= (2 + 3\pi - 2\alpha)\, c_u$$

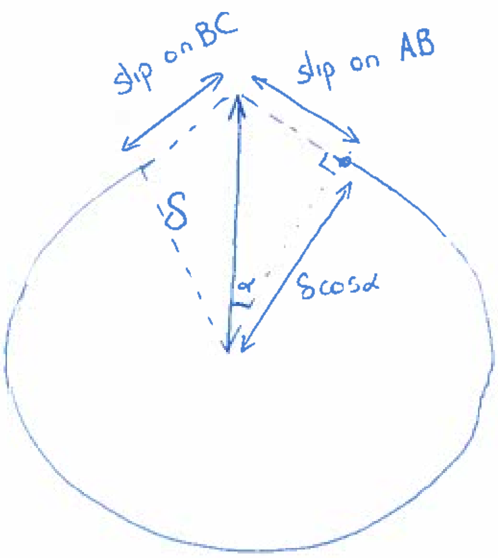
$$F = \left(\frac{x}{\cancel{\cos\alpha}} \cancel{\cos\alpha}\right) \times \sigma_f$$

$$= (2 + 3\pi - 2\alpha)\, x\, c_u$$

c) Upper bound



Hodograph



on BC

$$f\cancel{\delta} = 2\left[2c_u\,\cancel{\delta}\,\cancel{\cos\alpha}\,(2\pi-2\alpha)\frac{x}{2\cancel{\cos}\alpha} + c_u\cancel{\delta}\sin\alpha\,\frac{x}{\cos\alpha}\right]$$

$$\frac{f}{x} = 2c_u\left(2\pi - 2\alpha + \tan\alpha\right)$$

or similar mechanism.

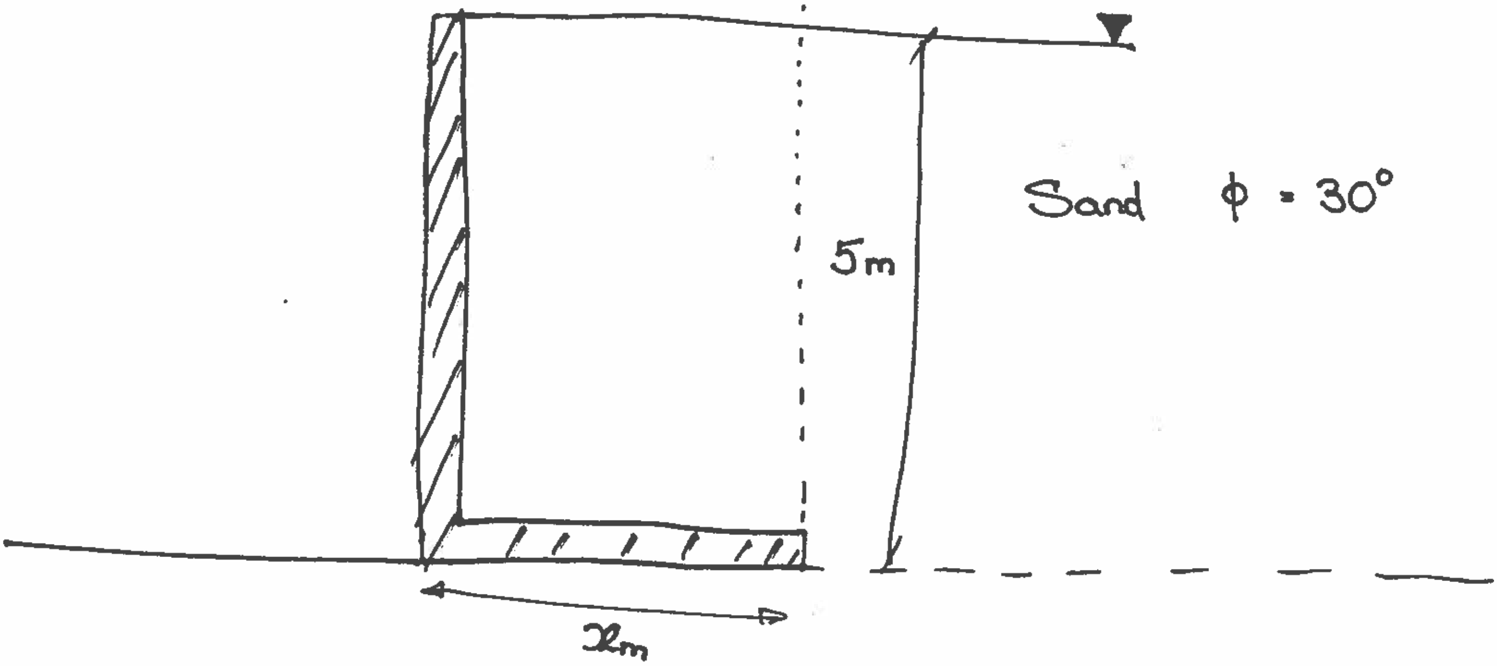
d) friction – more work (on AB) more rotation of stress in lower bound due to direction of principal stress due to friction.

3D effects compare to shallow foundation. Increased capacity with a shape factor ~ 1.2.

A fairly unpopular question on plasticity mechanisms only tacked by 14 students. In general the candidates understood what was required and how to tackle the problem. There were several very good solutions. The main mistake made was in invoking mechanisms that were not compatible for the upper bound or rotating stresses between the two upper surfaces of the anchor for the lower bound rather than rotating stresses to the unloaded base of the anchor.

PROBLEM 4

$= 30^\circ$ $\gamma = 20\,kN/m^3$ sand + concrete.



Clay $s_u =$? ~~50 kPa~~

false back :

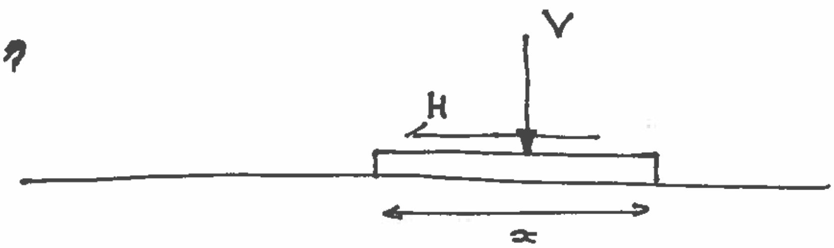
$$5x \times 20 \times \frac{x}{2} > \left(5 \times 20 \times \tfrac{1}{3}\right) \times \frac{5}{2} \times \frac{5}{3}$$

$$50x^2 > \frac{2500}{18}$$

$$x^2 > \frac{50}{18} = 2.78$$

$$x > 1.67m$$

If $x = 1$?



$V = 100x$ kN/m $H = \frac{250}{3} = 83\tfrac{1}{3}$ kN/m

What is required s_u for bearing failure to dominate?

from databook:

$$\frac{H}{H_{ult}} = 1 - \left(2\frac{V}{V_{ult}} - 1\right)^2 \quad \text{if } \frac{V}{V_{ult}} > 0.5$$

$$H_{ult} = s_u x$$

$$V_{ult} = (2+\pi)\, s_u\, x$$

$$\frac{V}{V_{ult}} = \frac{100\, \cancel{x}}{(2+\pi) s_u \cancel{x}}$$

$$\frac{H}{H_{ult}} = 1 - \left(\frac{200}{\cancel{50}(2+\pi)} - 1\right)^2 = \cancel{0.949} \quad \cancel{0.95} \quad \cancel{0.106}$$

$\cancel{30}$ 25

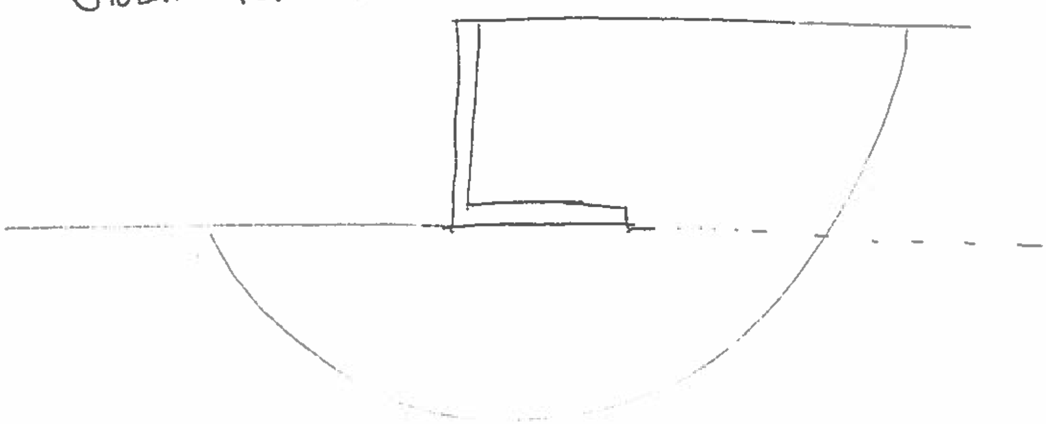
$$0.69$$

$$\frac{83\tfrac{1}{3}}{2\cancel{0}\,x} = \cancel{0.106}\ 0.69$$

5

$$x = \cancel{39\,m} \quad \therefore \quad \underline{\underline{4.83\,m}}$$

c) Global rotation.



Check moments of shear stress on global slip surfaces vs driving moment from self-weight. In sand need to assess confining stress on slip plane.

d) Increases unit weight of fill and applies hydrostatic driving pressure. Seepage will give upthrust on base of wall.

A popular question tackled by almost all candidates. Overturning of the wall was generally well tackled. Many students in part ii did not explicitly determine whether the wall might slide and hence whether the combined V-H envelope was applicable. Wall structural failure was often suggested in part iii which is a valid answer, few thought of global failure beneath the wall. The influence of water was well handled, some considered the water table to be level across the wall rather than following the wall surface, marks were still given for good discussion with this reading of the problem.