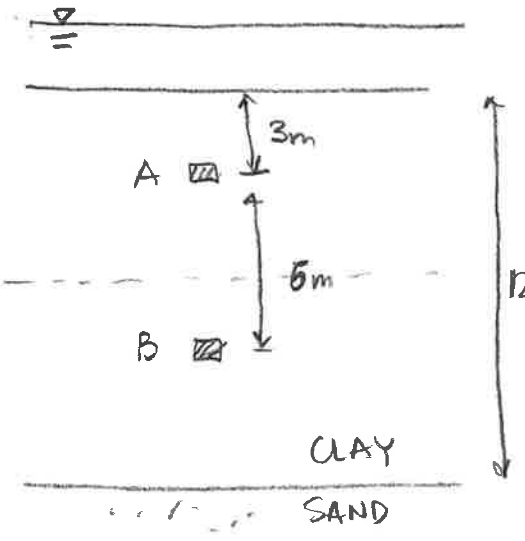
PROBLEM 1



At A:

$W_0 = 63\%$

$G_s = 2.70$

$e_0 = (0.63)(2.7) = 1.701$

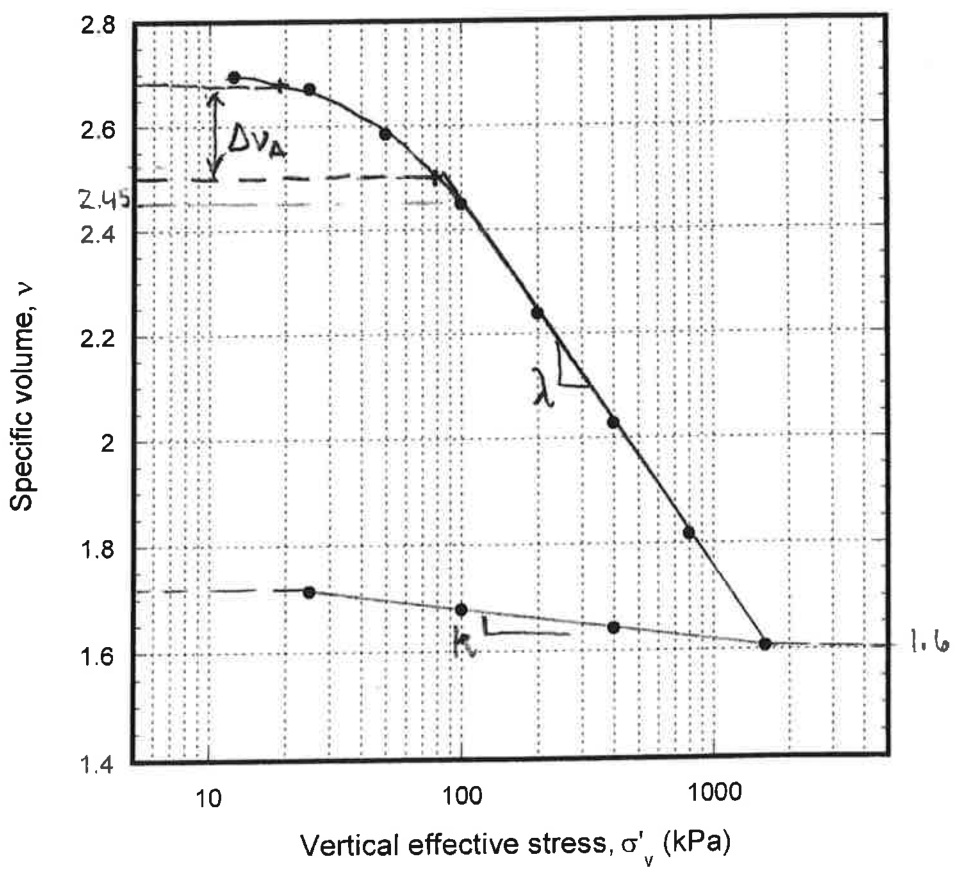
Assume $S = 1$

(a) (i) $\gamma_T = \dfrac{G_s + Se}{1+e}\gamma_w = \dfrac{2.7 + (1)(1.701)}{2.701}(9.81\ kN/m^3) = 16\ kN/m^3$ [10%

(ii) $\sigma_A = (16)(3) + (2)(9.81) = 67.62\ kPa$

$\sigma'_A = 67.62 - (9.81)(5) = 18.57\ kPa$ [10%]

(b) (i) See figure



$$\lambda = \frac{2.45 - 1.6}{\ln(1600/100)} = 0.307$$

$$k = \frac{1.72 - 1.6}{\ln(1600/25)} = 0.029$$ [10%]

(ii) $e_B = (0.72)(2.7) = 1.944$

$OCR_B = 1.0$

$$\gamma_T = \frac{2.7 + 1.944}{2.944}(9.81) = 15.5 \text{ kN/m}^3$$ [5%]

$\sigma'_{Bi} = (16 - 9.81)(6) + (15.5 - 9.81)(3) = 54.21$ kPa

$\Delta\sigma = (20 \text{ kN/m}^3)(4\text{m}) - (9.81 \text{ kN/m}^3)(2\text{m}) = 60.4$ kPa [5%]

Layer A

$\sigma_{Af} = 18.6 + 60.4 = 79$ kPa

$H_A = 6$m

$\Delta V_A = 2.68 - 2.50 = 0.18$ Directly from graph

$$\Delta h_A = \frac{\Delta V_A}{V_A} H_A = \frac{0.18}{1 + 1.701}(6\text{m}) = 0.40 \text{ m}$$ [8%]

Layer B

$\sigma_{Bf} = 54.2 + 60.4 = 114.6$ kPa

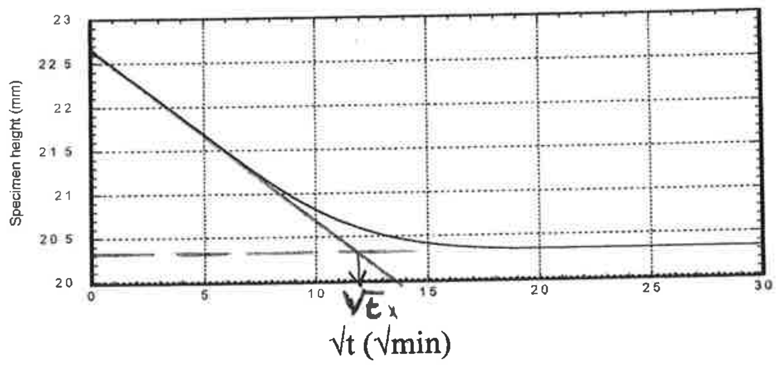
$H_B = 6$m

$$\Delta V_B = (0.307)\ln\left(\frac{114.6}{54.2}\right) = 0.23$$ [7%]

$$\Delta h_B = \frac{0.23}{1+1.944}(6m) = 0.47\ m$$

$$\Delta H_{TOT} = 0.40 + 0.47 = 0.87\ m$$

(c)



From graph $\sqrt{t_x} = 12\sqrt{min}$

$$C_v = \frac{3}{4}\frac{d^2}{t_x}$$

During the test $h_o = 22.6\ mm$, $h_{100} = 20.3\ mm$ $\Big\}\ h_{50} = 21.45\ mm$

$d \approx 10.7\ mm$ assuming double drainage

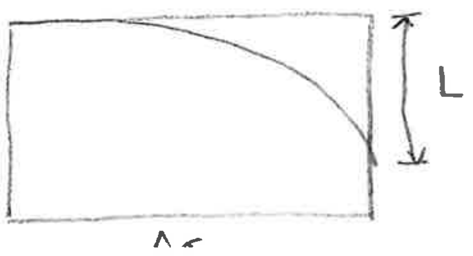
$$C_v = \frac{3}{4}\frac{(10.7)^2}{12^2} = 0.6\ \frac{mm^2}{min}\ \frac{m^2}{10^6 mm^2}\ \frac{60 min}{1 hr}\ \frac{24 hr}{1 day}\ \frac{365 day}{1 yr} =$$

$$C_v = 0.32\ m^2/yr$$

[10%]

(d) t = 6 month = 0.5 yr

$$T_v = \frac{(0.32)(0.5)}{(6m)^2} = 0.0044 < \frac{1}{12}$$ ⇒ Advancing isochrones phase.



$L^2 = 12 c_v t$

$L = \sqrt{12(0.32)(0.5)} = 1.18 \text{ m}$

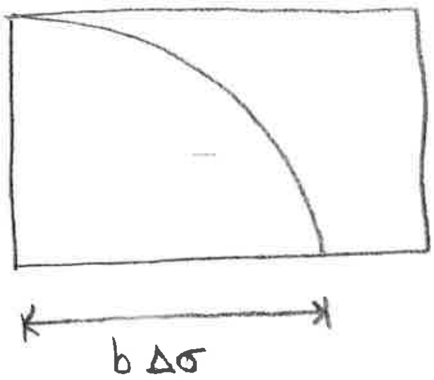
$\Rightarrow$ No or minimal pore pressure changes should be observed at the centre of the clay layer.

[10%]

(ii) $\Delta u_{ex}(t = 0.5 yr) = 132 - (9.8)(6+2) = 53.6 \text{ kPa}$

$b \approx \frac{53.6}{60.4} = 0.89$

$\Rightarrow$ Retreating isochrones phase



[10%]

(iii) $R_v = 1 - \exp\left(\frac{1}{4} - \frac{2}{3} b\right) = 0.29$

$\Delta h = (0.29)(0.87) = 0.25 \text{ m}$

measured value is consistent with pore pressure dissipated.

[10%]

Q1. Examiner's Comment:

A popular and straightforward question, well-•-answered by most candidates. The most difficulties were related to the time-•-rate of consolidation and comparing predicted vs measured values. When comparing measured settlements at a certain time, no student advanced the hypothesis that the final settlement estimate could be incorrect. Some students were not able to recognize whether a soil was overconsolidated or normally consolidated by comparing with the experimental results.

PROBLEM 2

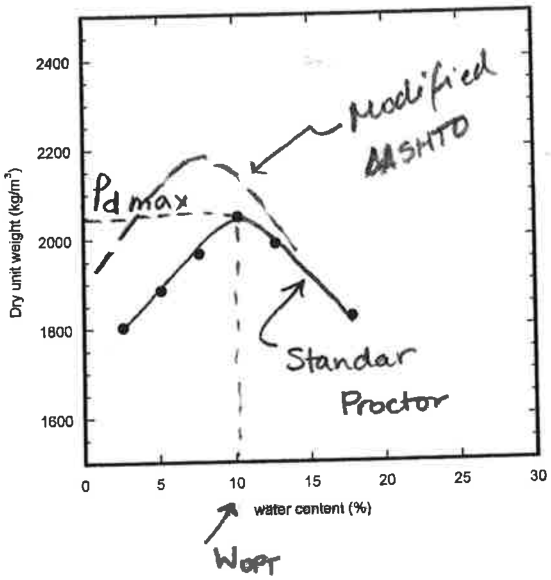
(a) (i) $M_T = 2.13\ kg$ $V_T = 9.44 \times 10^{-4}\ m^3$

$M_s = 1.933\ kg$

$$W = \frac{M_T - M_s}{M_s} = \frac{2.13 - 1.933}{1.933} = 0.102 \quad \text{or} \quad 10.2\%$$

$$\rho_d = \frac{1.933\ kg}{9.44 \times 10^{-4}\ m^3} = 2048\ kg/m^3$$

(ii)



$\rho_{d\,max} = 2048\ kg/m^3$

$W_{OPT} = 10.2\%$

(iii) The modified test imparts more energy into the soil than the standard test. Therefore, we expect the compaction curve to plot to the left and above of the curve from part (a)(ii)

(b) (i) The source material at the lower water content should be selected. It is much easier to add water to the soil than it is to dry it out. The water content at the other source location is too high and would result in low densities.

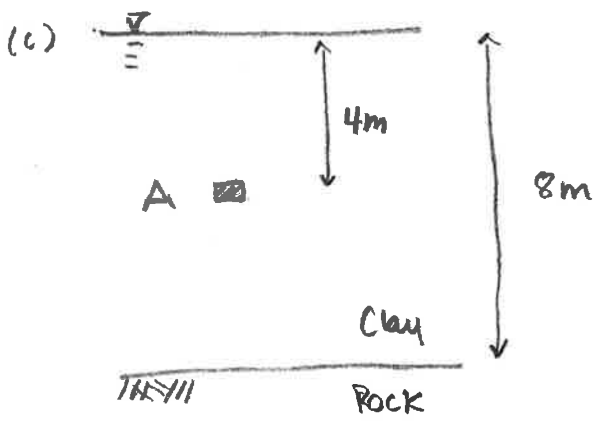
(ii) The variation in compaction effectiveness may be due to:

(a) the soil is placed in layers that are too thick and compaction is not effective at depth

OR

(b) the compaction effort needs to be increased by using larger equipment or more passes

(c)



$\lambda = 0.25$

$\kappa = 0.03$

$OCR_A = 1.5$

$w_o = 65\%$

$G_s = 2.72$

for $S=1$

(i) $e_o = w_o G_s = (0.65)(2.72) = 1.768$

$v_o = 2.768$

$$\gamma_T = \frac{G_s + Se}{1+e}\gamma_w \quad \text{from Data book}$$

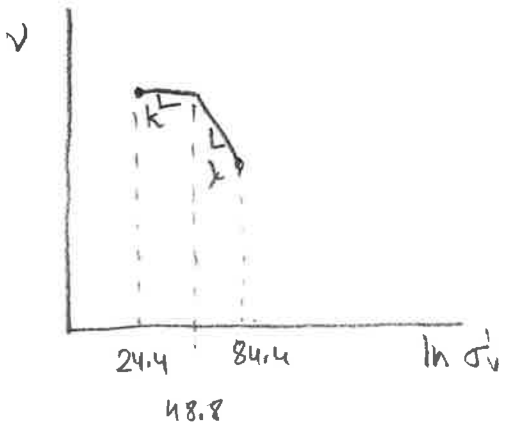
$$= \frac{2.72 + (1)(1.768)}{2.768}(9.81\,kN/m^3) = 15.9\,kN/m^3$$

$$\sigma'_A = (15.9\,kN/m^3 - 9.81\,kN/m^3)(4\,m) = 24.4\,kPa$$

$$\Delta\sigma = (3)(20\,\frac{kN}{m^3}) = 60\,kPa$$

$$\sigma'_{c_A} = (2)(24.4) = 48.8\,kPa$$

7/12



$$\Delta\nu = \left[0.03 \ln\left(\frac{48.8}{24.4}\right) + 0.25 \ln\left(\frac{84.4}{48.8}\right)\right]$$

$$= 0.1577$$

$$\Delta h = \frac{\Delta\nu}{\nu} H_o = \frac{0.1577}{2.768}(8m) =$$

$$= 0.46\ m$$

Q2. Examiner's Comment:

Most students were able to answer the questions on unit weight and water contents to build a compaction curve. Most students had difficulties responding to the qualitative questions on using a soil that is too wet or one that is slightly dry and compaction effectiveness. The vast majority could identify the benefits and problems of compacting dry vs wet of optimum, but only one realized that you can easily add water, while removing it is problematic. Erratic compaction effectiveness was mostly attributed to soil heterogeneity, with some considering sampling disturbance a problem. A few could identify layer thickness as an issue. The consolidation problem with an overconsolidated clay seemed to be generally well understood.

(a)

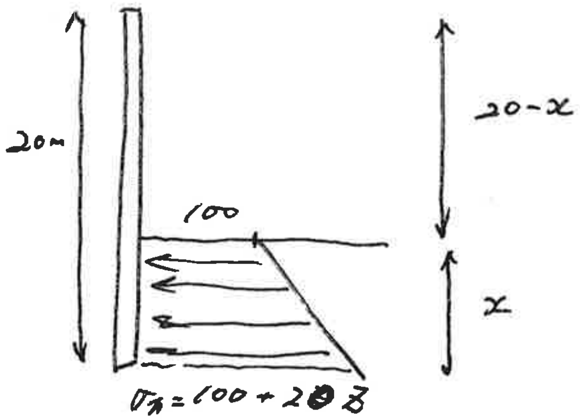
$\sigma_v = 20z$ z is from the original ground surface

$\sigma_h = 20z - 2 \times 50 = 20z - 100$

It is assumed that a dry crack develops when $\sigma_h \leq 0$

$\sigma_h = 0 \rightarrow \underline{z = 5 (m)}$ //

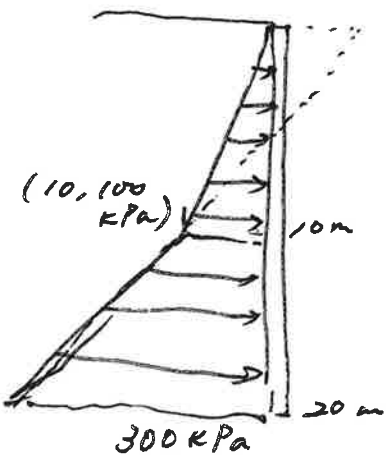
(b)



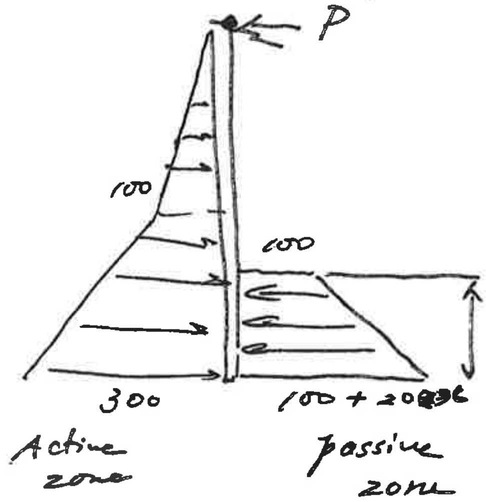
(c) water pressure profile is $p = 10z$

crack will further develop until $p = \sigma_h$

$10z = 20z - 100 \rightarrow z = 10m$



(d)



Moment around P.

Counter clockwise

$$M_{cc} = \frac{1}{2} 100 \cdot 10 \left(10 \cdot \frac{2}{3}\right) + 100 \cdot 10 \cdot \left(10 + 10 \cdot \frac{1}{2}\right)$$

$$+ \frac{1}{2} \cdot 200 \cdot 10 \cdot \left(10 + 10 \cdot \frac{2}{3}\right) = 35000 \quad kN \cdot m/m \quad ①$$

clock wize

$$M_c = 100 \cdot x \cdot \left(20 - \frac{1}{2}x\right) + \frac{1}{2} \cdot 20x \cdot x \cdot \left(20 - \frac{1}{3}x\right)$$

$$= 2000x - 50x^2 + 200x^2 - \frac{10}{3}x^3$$

$$= 2000x + 150x^2 - \frac{10}{3}x^3 \quad kNm/m \quad ②$$

① = ②

$$\frac{10}{3}x^3 - 150x^2 - 2000x + 35000 = 0$$

$$x^3 - 45x^2 - 600x + 10500 = 0$$

$$\rightarrow x = 10.8 \text{ (m)}$$

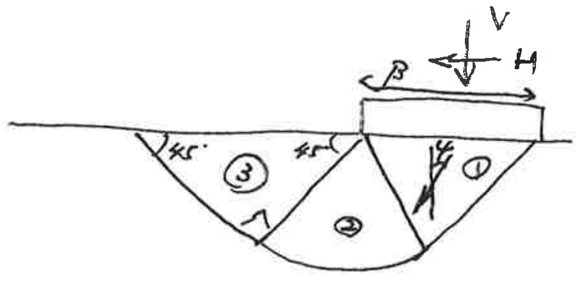
The maximum excavation depth is therefore <u>9.2m</u> //

(e) By introducing friction between the wall and the soil, the active pressure will decrease and the passive pressure will increase. Hence it will increase the maximum excavation depth.

Q3. Examiner's Comment:

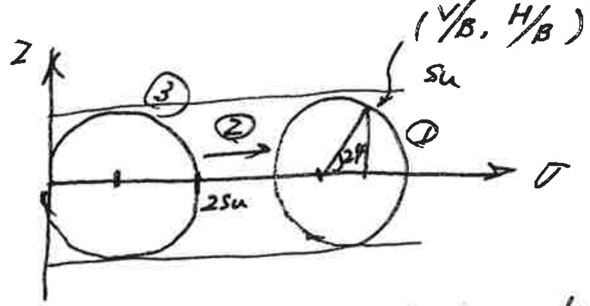
This question was answered easily by a good number of students and generally quite well. Some had difficulties in calculating moment arms. Most students could not clearly identify the effect of friction on the forces acting on the wall.

(a) A lower bound theorem with a stress fan concept can be used to derive the solution.



At the foundation boundary, the normal and shear stress are $\sigma_v = V/B$ and $\tau = H/B$

The failure zone has three regions and the stress states of the three regions are shown in the stress diagram below.



The inclination of the principal stress direction beneath the footing is ψ. From geometry $H/B = S_u \sin 2\psi$ — (a)

In Region ②, the stress fan concept is used to rotate the principal stress direction to the horizontal, so that it becomes Region ③.

From geometry

$$\frac{V}{B} = S_u + 2S_u(\pi/2 - \psi) + S_u \cos 2\psi \quad \text{— (b)}$$

Combining (a) & (b) and eliminating ψ will give the equation given in the question.

(b)

(i) $\frac{H}{BS_u} = \frac{350}{7 \times 100} = 0.5$

$$\frac{\sigma_v}{S_u} = \frac{V}{BS_u} = 1 + \pi - \sin^{-1}(0.5) + \sqrt{1 - 0.5^2}$$

$$= 4.48$$

σ_v at the bottom of the foundation will be

$$\sigma_v = 4.48 \cdot S_u + \underbrace{3 \times 10}_{\text{surcharge effect in Total stress}}$$

$$= 448 + 30 = 478 \text{ kPa}$$

The weight of the concrete is $23 \times 4 = 92$ kPa

Hence the maximum stress that can be applied to the Top of the foundation is

$$478 - 92 = 386 \text{ kPa}$$

The maximum vertical load is $386 \times 7 (m)$

$$= \underline{2702 \text{ kN/m}}$$

(ii) No horizontal load

so $\frac{\sigma_v}{S_u} = 5.14$.

σ_v at the bottom of the foundation is

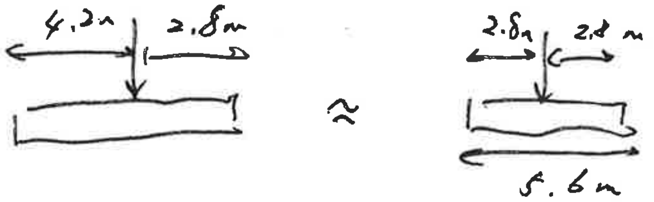
$$\sigma_v = 5.14 \cdot S_u + 3 \times 10 = 514 + 30 = 544 \text{ kPa}$$

Subtracting the weight of the concrete (92 kPa)

The maximum vertical stress that can be applied to the Top of the foundation is

$$544 - 92 = 452 \text{ kPa}$$

Because it is eccentrically loaded, the effective foundation width is 5.6 m (= 2.8 × 2 m) using Meyerhof's assumption.



The maximum vertical load that can be applied to the structure is

$$452 \times 5.6\,m = \underline{2531\ kN/m}$$

(iii) Use the bearing capacity formula for the drained case.

$$N_q = \tan^2(\pi/4 + \phi'/2)\exp(\pi \cdot \tan\phi')$$

$$N_\gamma = 2(N_q - 1)\tan\phi'$$

$$\phi' = \pi \cdot \frac{28}{180} = 0.489$$

$$N_q = \tan^2\left(\pi/4 + \frac{0.489}{2}\right)\exp(\pi \cdot \tan(0.489))$$

$$= 2.772 \times 5.321$$

$$= 14.75$$

$$N_\gamma = 2(14.75 - 1)\cdot\tan(0.489)$$

$$= 14.63$$

$$q_f = 14.63 \cdot \frac{8 \times 7}{2} = 409.64\ kPa$$

Subtracting the concrete weight

$$409.64 - 92 = 317.64$$

$$V = 317.64 \times 7 = \underline{2,224\ kN/m}$$

The long term bearing capacity is likely to be more crucial.

Q4. Examiner's Comment:

The main difficulty on this problem was related on how to account for the water around the foundation in the total stress analysis questions. Meyerhof’s approach was generally adopted and carried out correctly. Some students included the pore pressure as a resisting force in the drained analysis. A good number of students was able to solve the problem correctly.