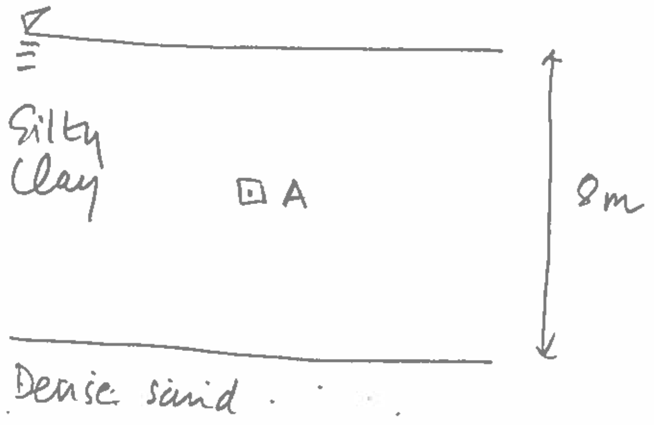
PROBLEM 1



$q_{full} = 400\text{ kPa}$

$G_s = 2.70$

$W = 49\%$

(a) Large settlements could be a problem for the tank because they could result into tilting or breaking of any connections, such as pipelines.
In addition, large settlements are also likely to result in large differential settlements, which will require careful reinforcement of the tank foundation to avoid cracking of the concrete

(b) $\gamma_{clay} = \dfrac{(1+w)G_s\gamma_w}{1+G_s w} =$ Assume $S=1$

$$= \frac{(1+0.49)(2.70)(9.81)}{1+(2.7)(0.49)} = 17\ \text{KN/m}^3$$

$$\sigma'_{A_i} = (17-9.8)(4) = 28.8\ \text{kPa}$$

$$\sigma'_{A_f} = 28.8 + 400 = 428.8\ \text{kPa}$$

$$e_0 = \frac{G_s W}{S} = \frac{(2.7)(0.49)}{(1)} = 1.323$$

Version GB/1

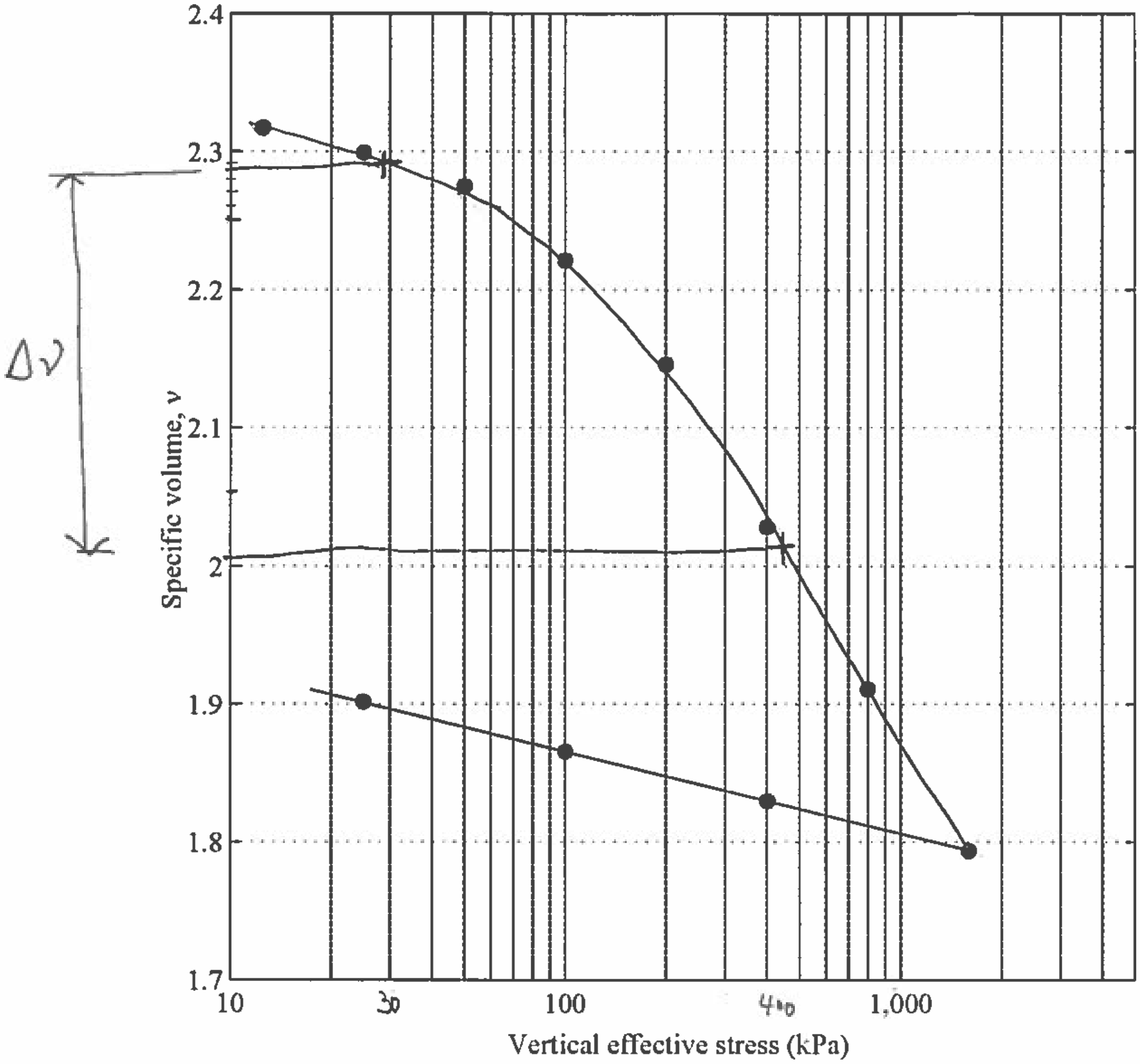


Fig. 1

(TURN OVER

$v_0 = 1 + e_0 = 2.323$

From graph $\Delta v = 2.285 - 2.01 = 0.275$

$$\frac{\Delta v}{v_0} = \frac{\Delta H}{H_0}$$

$$\Delta H = \frac{\Delta v}{v_0} H_0 = \frac{0.275}{2.323}(8\,m) = 0.95\,m$$

The settlement is quite large and likely to cause structural damage to the tank.

(c) The estimate is very rough. The clay seems to be slightly overconsolidated in the center of the clay layer, so it is possibly more overconsolidated in the upper parts. This would lead to lower settlements.

The change in properties of the clay should also be considered.

Dividing the clay layer into smaller sub-layers will improve the estimate.

(d) Estimate k from the test results

$\sigma_i = 25\,kPa$ $\sigma_f = 1600\,kPa$

$\Delta v = 1.90 - 1.79 = 0.11$

$$k = \frac{\Delta v}{\ln\left(\frac{\sigma_f}{\sigma_i}\right)} = \frac{0.11}{\ln\left(\frac{1600}{25}\right)} = 0.026$$

$$v_i = v_0' - \Delta v_{NC} + k \ln\left(\frac{428}{28}\right) = 2.323 - 0.275 + (0.026)\ln\left(\frac{428}{28}\right) =$$

$$= 2.119$$

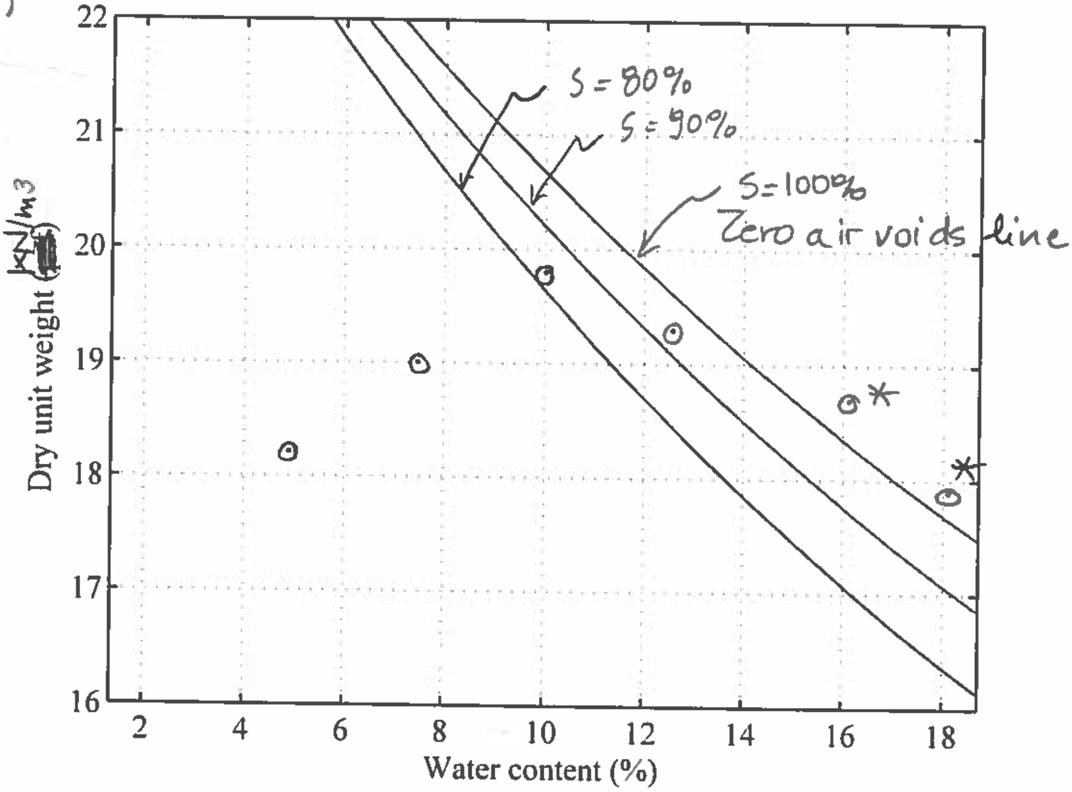
$$\Delta H = \frac{\Delta v}{v_i} H_o = \frac{k \ln\left(\frac{428}{28}\right)}{2.119} (8m) = 0.27 m$$

(e) In order to create a preload of 400kPa we would need ~20m of fill, which is not practical to build. Time rate of settlement may also be too long.
It may be easier to construct a very stiff structure and mitigate the effects of the settlement estimated initially.

Ultimately, deep foundations may have to be used.

PROBLEM 2

(a)



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

The points at water contents of 16% and 18% plot <u>above</u> the zero air voids line - Clearly this indicate an error with the test, or that the material was different.

(b)

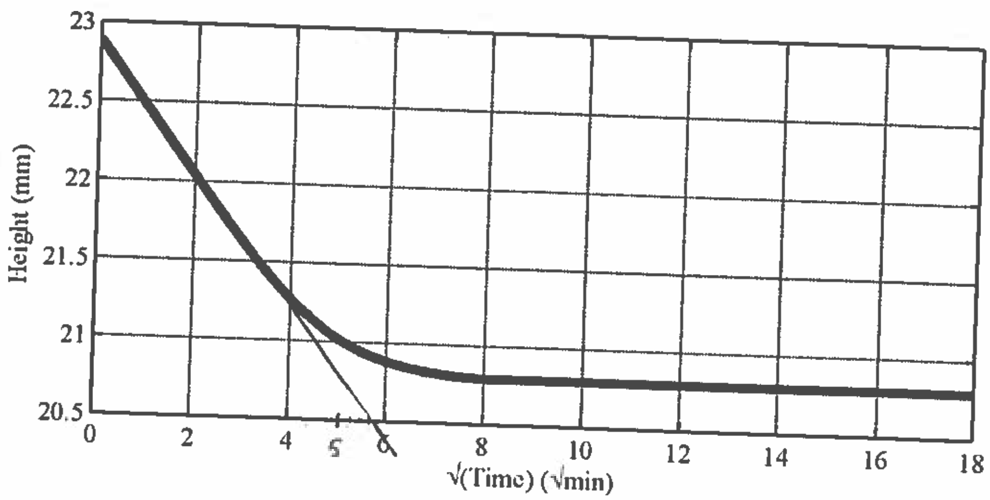


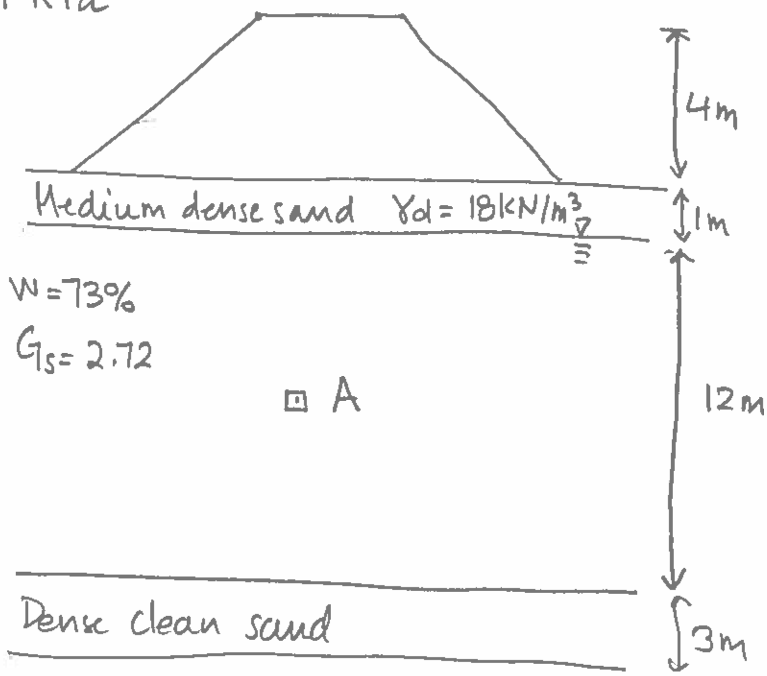
Fig. 2

$H_0 = 22.9$ mm

$H_f = 20.8$ mm

$\Delta H = 22.9 - 20.8 = 2.1$ mm

$$E_0 = \frac{\Delta\sigma}{\Delta\varepsilon_v} = \frac{140-50}{2.1/22.9} = 981 \text{ kPa}$$



For the clay

$$\gamma_t = (1+w)\frac{G_s \gamma_w}{1+G_s w} =$$

$$= (1+0.73)\frac{(2.72)(9.8 \text{ kN/m}^3)}{1+(2.72)(0.73)} =$$

$$= 15.4 \text{ KN/m}^3$$

$$\sigma'_{A_i} = (18)(1) + (15.4-9.8)(6) = 51.6 \text{ kPa}$$

For the embankment, assume $\gamma_d = \gamma_{opt} = 19.8$ kN/m³

$$\gamma_T = (1+w)\gamma_d = (1+0.1)(19.8) = 21.8 \text{ kN/m}^3$$

$$\Delta\sigma = (21.8)(4) = 87.1 \text{ kPa}$$

$$\sigma'_{Af} = 51.6 + 87.1 = 138.7 \text{ kPa}$$

→ The test increment provided is applicable to the loading of the soil due to the embankment

$$\Delta H_{emb} = \frac{\Delta\sigma}{E_0} H_i = \frac{(87 \text{ kPa})}{(981 \text{ kPa})}(12 \text{ m}) = 1.06 \text{ m}$$

Assumptions:

- the embankment does not cause a constant increase in stress with depth, i.e. 1D conditions.
  The 1D solution used may give a settlement estimate that is larger than the actual one.

- It is assumed that point A is representative of the entire clay layer, but it may not be true.
  The variation in initial water content or properties of the clay should also be accounted for.

- The change in initial stress with depth would also lead to lower strains caused by the stress increment. For better predictions the clay layer should be subdivided into thinner layers.

- It was assumed the embankment does not become buoyant

(c)

$$C_v = \frac{T_v d^2}{t}$$

$\sqrt{t_x} = 5.7 \sqrt{min}$ $t = 32.49\ min$

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

Specimen is double-drained $d = \frac{22.9}{2} = 11.45\ mm$

$$C_v = \frac{3}{4}\frac{(11.45\ mm)^2}{32.49\ min} = 3.02\ \frac{mm^2}{min}\left(\frac{m^2}{10^6\ mm^2}\right)\left(\frac{1440\ min}{day}\right)\left(\frac{365\ day}{1\ yr}\right)$$

$$= 1.59\ \frac{m^2}{yr}$$

At $R_d = 90\%$ $T_v = 0.848$ (Fourier solution)

$$t = \frac{T_v d^2}{C_v} = \frac{(0.848)(6\ m)^2}{1.59\ m^2/yr} = 19.2\ yrs$$

(d) $t = 1\ yr$

$$R_d = \frac{38\ cm}{106\ cm} = 0.36$$

Assume $T_v > 1/2$

$$R_v = 1 - \frac{2}{3}\exp\left(1/4 - 3T_v\right)$$

$$T_v = \frac{1}{3}\left[1/4 - \ln\left(\frac{3}{2}(1-R_v)\right)\right] = \frac{1}{3}\left[1/4 - \ln\frac{3}{2}(1-0.36)\right] = 0.096$$

(or $T_v = 0.102$ using Fourier solution)

$$C_v = \frac{T_v d^2}{t} = \frac{(0.096)(6m)^2}{(1yr)} = 3.46\ m^2/yr$$

+ horizontal drainage
+ possibly shorter drainage path due to sand/silt seams

$$t_{90} = \frac{(0.848)(6m)^2}{(3.46\ m^2/yr)} = 8.8\ yrs$$

(e) The loading of the soil due to the embankment is mostly normally consolidated.
The clay can be preloaded to a level exceeding the the final embankment load. Once sufficient excess pore pressure has been dissipated, the preload can be removed and the embankment constructed. The reload will result in much smaller settlements and therefore potential differential settlements.
However, it will still take years for consolidation to occur.

1. a) $\phi = 33^\circ$

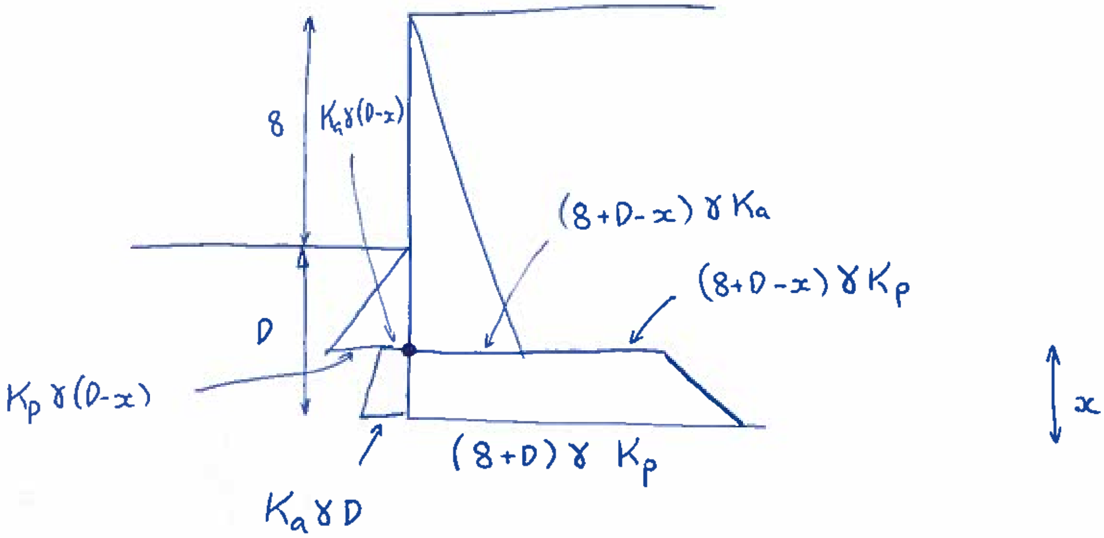
$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.295 \qquad K_p = 3.39$$

$e = 0.8 \qquad G_s = 2.65$

$$\gamma_{dry} = \frac{2.65}{1.8} \times 10 = 14.7 \ \text{kN/m}^3$$

$$\gamma_{sat} = \frac{2.65 + 0.8}{1.8} \times 10 = 19.2 \ \text{kN/m}^3$$

b)



Horizontal eqbm:

$$\frac{K_p \cancel{\gamma}(D-x)^2}{2} + K_a \cancel{\gamma}(D-x)x + \frac{K_a \cancel{\gamma} x^2}{2}$$

$$=$$

$$\frac{(8+D-x)^2 \cancel{\gamma} K_a}{2} + (8+D-x)\cancel{\gamma} K_p x + \frac{K_p \cancel{\gamma} x^2}{2}$$

Moment eqbm about $\frac{x}{3}$ above base:

$$(8+D-x)\gamma K_p \frac{x^2}{6} + \frac{(8+D-x)^2}{2}\gamma K_a \left(\frac{\cancel{2}x + 8 + D}{3}\right)$$

$$=$$

$$K_a\gamma(D-x)\frac{x^2}{6} + \frac{K_p\gamma(D-x)^2}{2}\left(\frac{\cancel{2}x}{3} + \frac{D}{\cancel{3}}\right)$$

At limit of stability $x = 0$

$$\therefore \quad \frac{K_p D^2}{\cancel{2}} = \frac{(8+D)^2 K_a}{\cancel{2}}$$

$$D^2 (K_p - K_a) - 16 D K_a - 64 K_a = 0$$

$$D = \frac{16 K_a \pm \sqrt{256 K_a^2 + 4(K_p - K_a) \times 64 K_a}}{2(K_p - K_a)}$$

$$= \frac{4.72 \pm \sqrt{256.01}}{6.19}$$

$$= \underline{\underline{3.35 m}} \checkmark$$

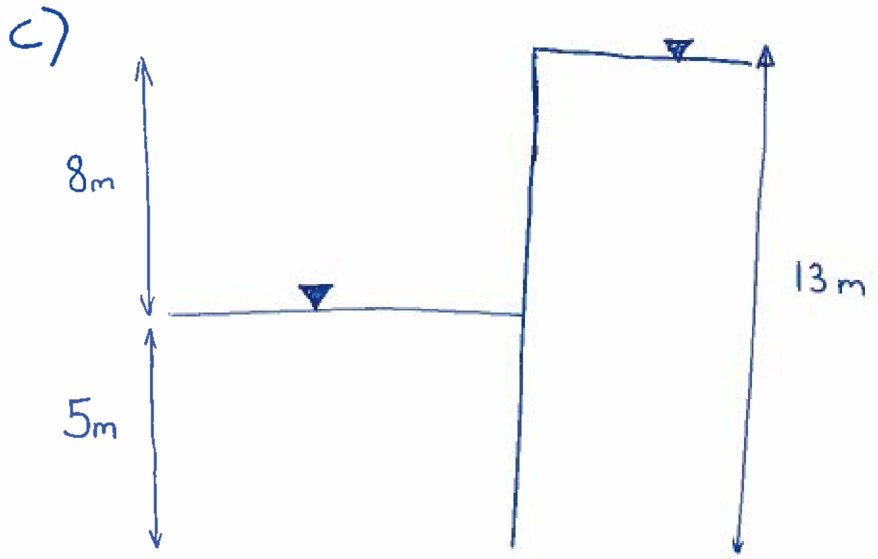
Check horizontal eqbm:

~~At~~ resisting | driving

$$\frac{3.35^2 K_p \cancel{\gamma}}{\cancel{2}} \qquad \frac{11.35^2 K_a \cancel{\gamma}}{\cancel{2}}$$

38.04 | 38.0 | ok

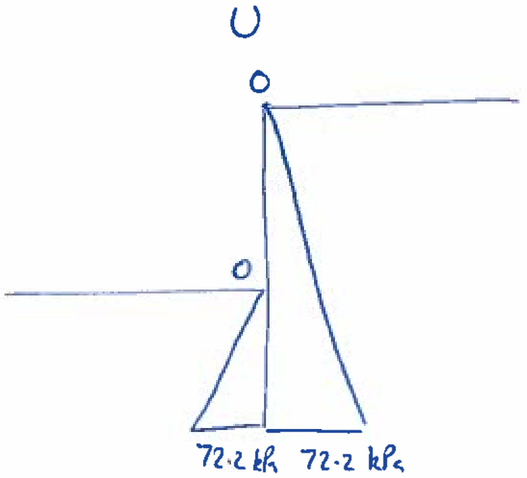
c)

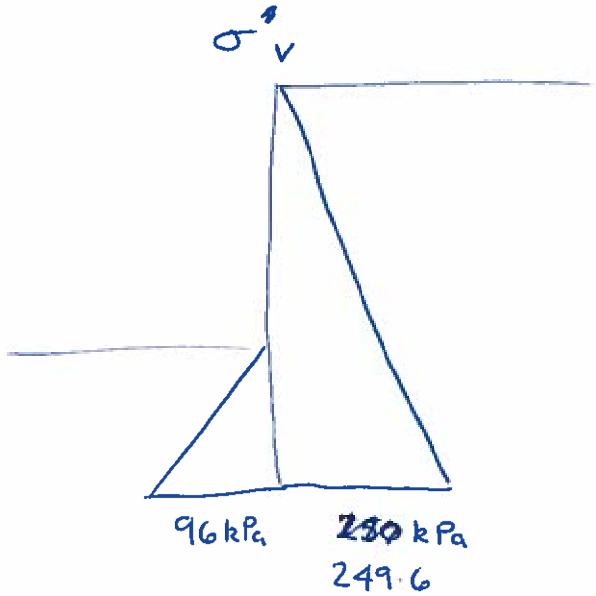


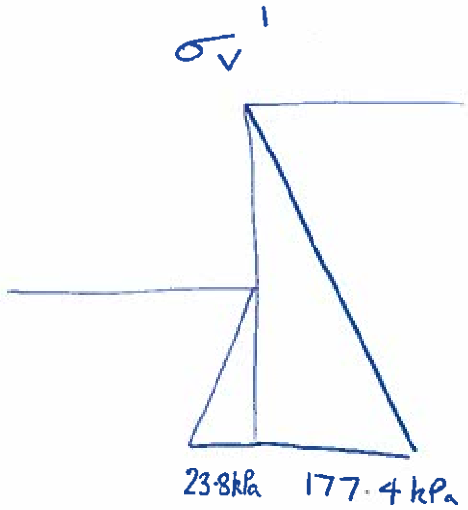
8m of head difference is distributed over an 18m long flow line.

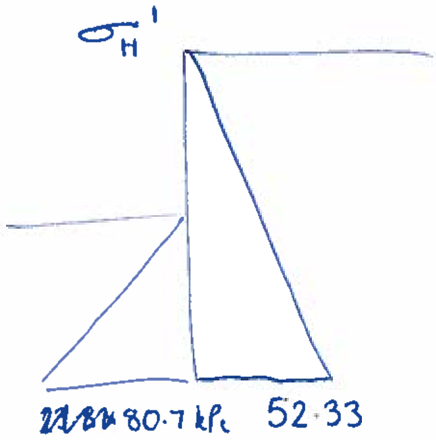
$\therefore$ pore pressure at base $= 50 + \frac{5}{18} \times 80$

$= 72.2$ kPa









Horizontal eqbm:

Driving

$(52.3 + 72.2) \times \frac{13}{2} = 809.25$ kN/m

Resisting

$(80.7 + 72.2) \times \frac{5}{2} = 382.25$ kN/m

failure

d) Taking moments about prop:

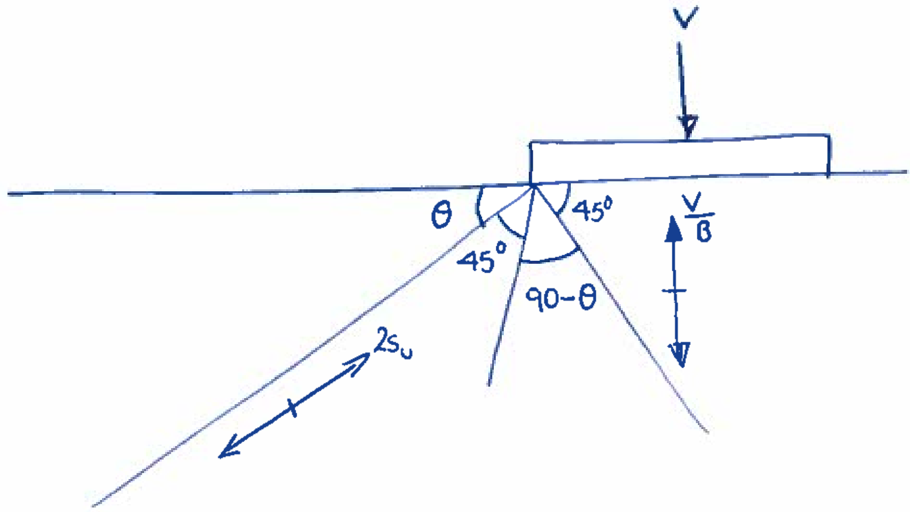
$382.25 \times \left(13 - \frac{5}{3}\right) = 4332$ kNm/m resisting

$809.25 \times 13 \times \frac{2}{3} = 7013$ kN/m driving

failure

2.

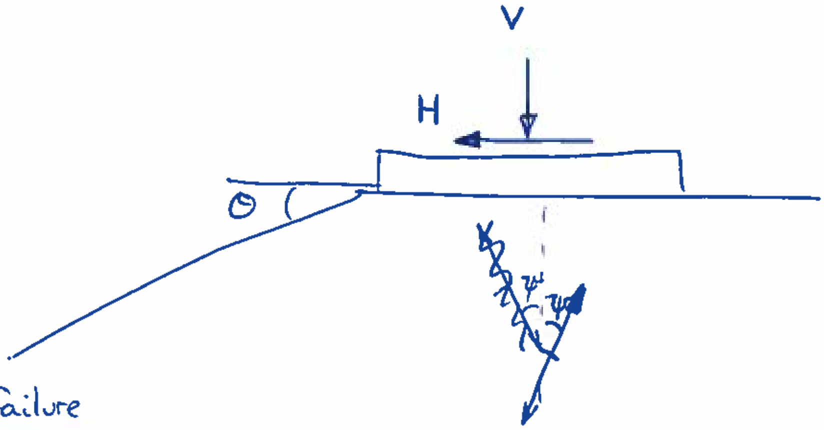
a)



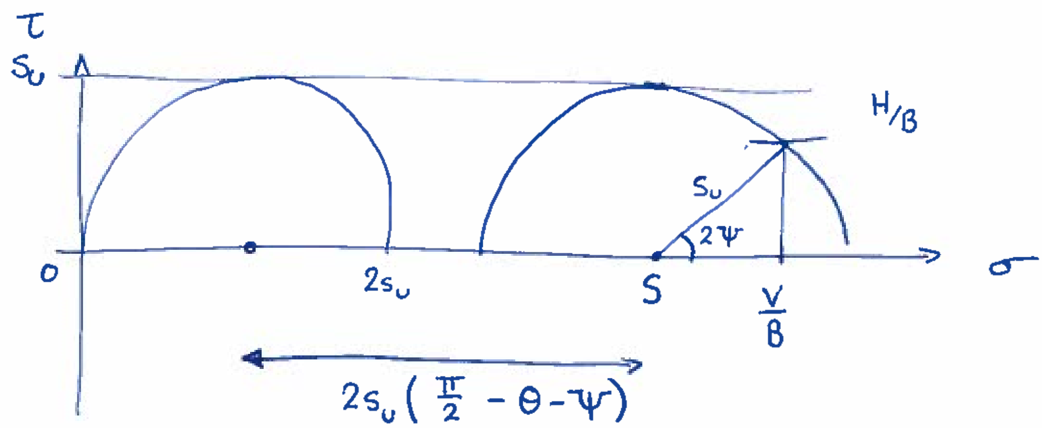
$$\frac{V}{B} = 2s_u + 2s_u\left(\frac{\pi}{2} - \theta\right)$$

$$= (\pi + 2 - 2\theta)\, s_u$$

b)



Bearing failure



$$\sin(2\psi) = \frac{H}{Bs_u} \qquad S = \frac{V}{B} - s_u\cos(2\psi)$$

$$\frac{V}{B} = s_u + 2s_u\left(\frac{\pi}{2} - \theta - \psi\right) + s_u \cos 2\psi$$

$$= s_u\left[\pi + 1 - 2\theta - \sin^{-1}\left(\frac{H}{Bs_u}\right) + \sqrt{1 - \left(\frac{H}{Bs_u}\right)^2}\,\right]$$

Sliding

$$\underline{H = Bs_u}$$

c) $$\frac{V}{B} = s_u\left[\pi + 1 - 2\theta - \frac{\pi}{2}\right]$$

$$\underline{V = Bs_u\left[\frac{\pi}{2} + 1 - 2\theta\right]}$$

d) i) If $\theta = 0$ with incompressible soil, in the upper bound analysis work done against gravity is zero as soil punched into surface by foundation is heaved up vertically. No net work is done. As upper + lower bounds match, no change in failure loads

ii) If $\theta > 0$, soil heaving at the sloping surface can move sideways as well as vertically. The soil weight thus contributes to failing the foundation and the stress that can be carried will drop. Calcs here will be unconservative.