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3D2 Question 1 Solution

(a)

(i) From Data Book, elastic expansion $\delta\sigma_c = G.\delta A/A$
where $\delta\sigma_c = \sigma_c - \sigma_0$
and σ_0 = insitu total horizontal stress in the clay prior to start of pressuremeter test

The length of the pressuremeter remains constant, hence $\Delta V/V = \delta A/A$

The clay reaches its yield strength (= undrained shear strength, c_u) at the cavity wall when $\sigma_c = \sigma_0 + c_u$

Hence $\boldsymbol{\Delta V/V = c_u/G}$ [20%]

(ii) As the cavity pressure σ_c increases above ($\sigma_0 + c_u$) a plastic zone begins to develop in the clay around the pressuremeter. In the plastic zone, effective stresses remain constant (the clay has reached its yield strength). The difference between the increased value of σ_c and its value at the point at which yield first occurs results in excess pore pressure Δu:

$\Delta u = \sigma_c - (\sigma_0 + c_u) = \delta\sigma_c - c_u$

For undrained plastic-elastic expansion, $\delta\sigma_c = c_u\,[1 + \ln G/c_u + \ln \delta A/A]$

(from Data Book)

Hence $\Delta u = c_u \ln [G/c_u \,.\, \delta A/A] = \boldsymbol{c_u \ln [G/c_u \,.\, \Delta V/V]}$ [20%]

(b)

(i) By inspection of the data, lift-off occurs somewhere between σ_c = 160 kN/m^2

and σ_c = 180 kN/m^2. Estimate σ_0 = 170 kN/m^2.

Water table is 1.5m below ground level, soil unit weight = 15kN/m^3

At 12m depth, vertical effective stress σ_{v0}' = 12x15 – 10.5x10 = 75 kN/m^2

Pore pressure u_0 = 10.5x10 = 105 kN/m^2

Horizontal effective stress $\sigma_{h0}' = \sigma_0 - u_0$ = 170 – 105 = 65 kN/m^2

Hence $K_0 = \sigma_{h0}'/\sigma_{v0}'$ = 65/75 = **0.87** [20%]

(ii)

Cavity pressure (kN/m^2)	Cavity strain, ε_c (%)	ln ε_c
160	0	
180	0.1	-2.3
190	0.2	-1.6
205	0.5	-0.2 7
230	1.2	0.2
255	2.5	0.9
270	4.1	1.4
290	7.7	2.0
300	10.5	2.4

From plot, for higher cavity pressures tending towards linear relationship
Slope of line $\Delta\sigma_c/\Delta \ln \varepsilon_c \sim 100/3.3 =$ **30 kN/m^2**

[20%]

(iii) $G/c_u = 4500/30 = 150$

$\Delta u = c_u \ln [G/c_u \,.\, \Delta V/V]$

for small strains $\Delta V/V = 2\varepsilon_c$

$\Delta u = 30 \ln [150 \times 0.2] =$ **102 kN/m^2** [20%]

3D2 Q2 2016 Solution

(a) Current in-situ stresses at a depth of 10m:

Total vertical stress, $\sigma_v = 17.5 \times 10 = 175$ kN/m^2

pore pressure, $u_0 = 10 \times 7 = 70$ kN/m^2

Effective vertical stress, $\sigma_v' = \sigma_v - u_0 = 105$ kN/m^2

$K_0 = 0.9$, hence effective horizontal stress, $\sigma_h' = 0.9\sigma_v' = 0.9 \times 105 = 94.5$ kN/m^2

Total horizontal stress, $\sigma_h = \sigma_h' + u_0 = 94.5 + 70 = 164.5$ kN/m^2

$t = (\sigma_v - \sigma_h)/2 = (\sigma'_v - \sigma'_h)/2 = (175 - 164.5)/2 = 5.25$ kN/m^2

$s' = (\sigma_v' + \sigma_h')/2 = (105 + 94.5)/2 = 99.8$ kN/m^2

$s = (\sigma_v + \sigma_h)/2 = (175 + 164.5)/2 = 169.8$ KN/m^2

From Data Book $K_{onc} = 1 - \sin\phi_{crit} = 1 - \sin 28 = 0.53$

(i) Test A:

At failure $t = t_f = c_u = 60$ KN/m^2

$\Delta t = t_f - t = 60 - 5.25 = 54.75$ KN/m^2

Total stress path: $\Delta\sigma_h = 0$

Hence $\Delta t = (\Delta\sigma_v - \Delta\sigma_h)/2 = \Delta\sigma_v/2$

$\Delta s = (\Delta\sigma_v + \Delta\sigma_h)/2 = \Delta\sigma_v/2$

$\Delta t/\Delta s = 1$

$\Delta s = \Delta t = 54.75$ KN/m^2

At failure, $s_f = 169.8 + 54.75 = 224.5$ KN/m^2

Mohr Coulomb model: $t_f = s_f' \sin\phi_{crit}$

$s_f' = 60/\sin 28 = 127.7$ KN/m^2

pore pressure at failure, $u_A = s_f - s_f' = 224.5 - 127.7 = 96.8$ KN/m^2

Effective stress path in elastic region, until Mohr-Coulomb line is reached, is vertical, ie $\Delta s' = 0$

(ii) Test B:

Total stress path: $\Delta\sigma_v = 0$

$\Delta t = (\Delta\sigma_v - \Delta\sigma_h)/2 = -\Delta\sigma_h/2$

$\Delta s = (\Delta\sigma_v + \Delta\sigma_h)/2 = \Delta\sigma_h/2$

$\Delta t/\Delta s = -1$

As for Test A, at failure $t = t_f = c_u = 60$ KN/m^2

$\Delta t = t_f - t = 60 - 5.25 = 54.75$ KN/m^2

$\Delta s = -\Delta t = -54.75$ KN/m^2

At failure, $s_f = 169.8 - 54.75 = 115$ KN/m^2

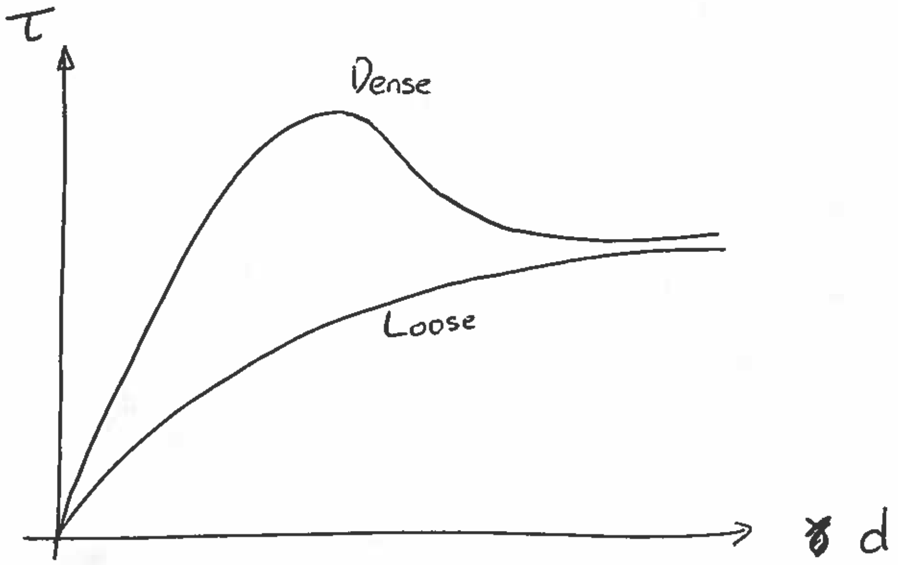
$s_f' = 60/\sin 28 = 127.7$ KN/m^2

pore pressure at failure $= s_f - s_f' = 115 - 127.7 = -12.7$ KN/m^2

3 a) $\gamma_{dry} = \dfrac{G_s \gamma_w}{1+e} = \dfrac{2.75}{1.7} \times 10 = 16.2 \ kN/m^3$

$\gamma_{sat} = \left(\dfrac{G_s + e}{1+e}\right) \gamma_w = 20.3 \ kN/m^3$

b)



Dense soils dilate, requiring extra work to be done against confining stress. This results in a peak strength at maximum rate of dilation mobilising ϕ_{peak}, a function of stress level. Once sample reaches constant volume the sample mobilises ϕ_{crit}

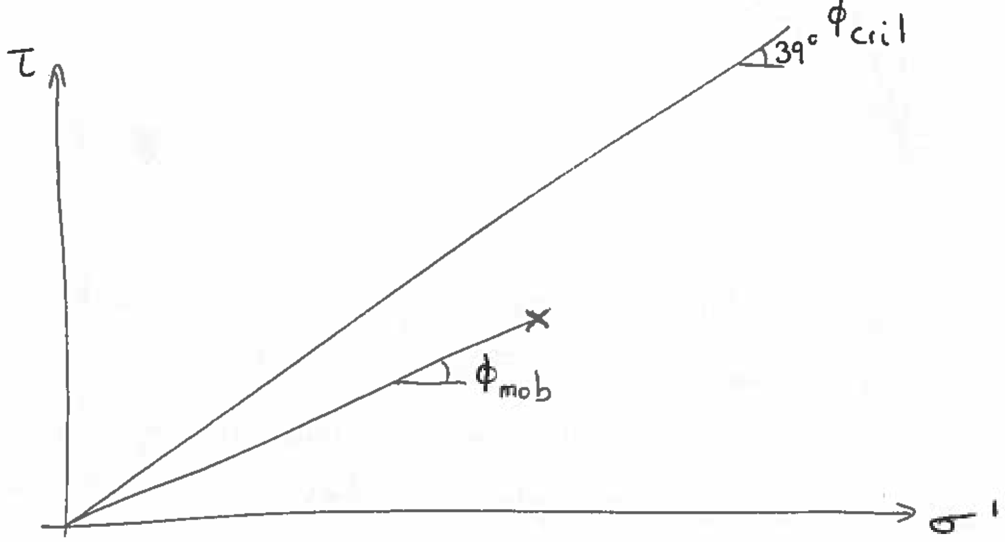
Loose sands collapse to critical state & have no peak strength. At constant volume ~~they~~ also mobilise ϕ_{crit}

ϕ_{crit} should be used in design as ϕ_{peak} is unreliable, especially when cyclic loading causes progressive failure.

c$\cancel{b}$) From databook:

$$\sigma = \gamma z \cos^2\beta$$
$$\sigma' = (\gamma z - \gamma_w z_w)\cos^2\beta$$
$$\tau = \gamma z \cos\beta \sin\beta$$

$$\sigma' = 16.2 \times 5 \times \cos^2(30)$$
$$= 60.75 \text{ kPa}$$
$$\tau = 16.2 \times 5 \times \cos(30) \times \sin(30)$$
$$= 35 \text{ kPa}$$



$$\phi_{mob} = \tan^{-1}\left(\frac{35}{60.75}\right) = 30^\circ$$

$\therefore$ Safe on ϕ_{crit}.

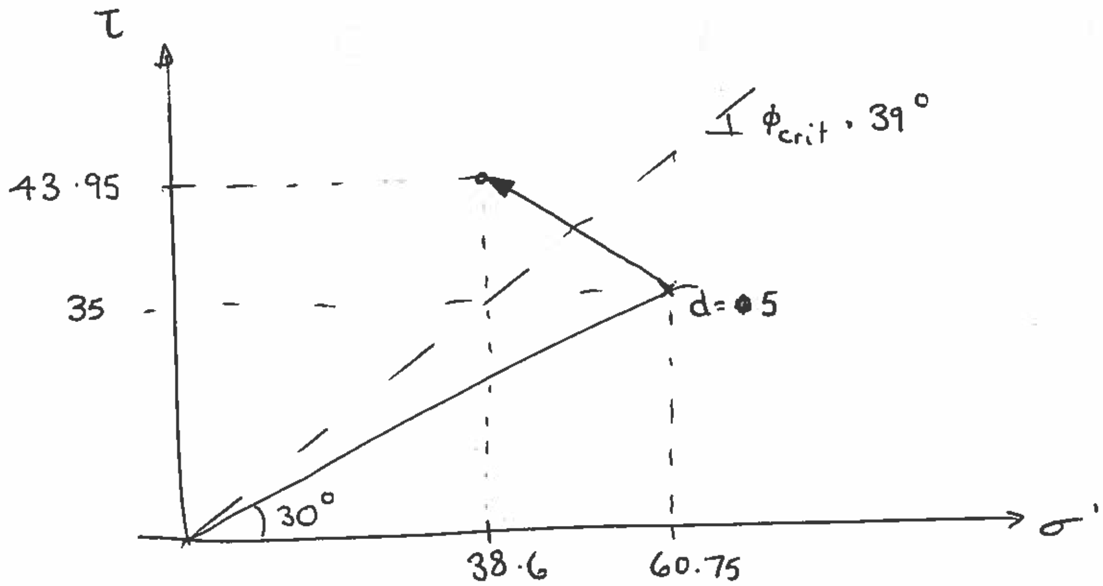
$\therefore$ Slope is safe

d)

$$\cancel{\sigma' = (16.2 \times 5 - 10 d)\cos^2\beta}$$
$$\cancel{\tau = 3}$$

Ignoring capillary rise

$$\sigma = [16.2d + 20.3(5-d)] \cos^2\beta$$
$$= 76 - 3.075d$$
$$\sigma' = \sigma - 10(5-d)\cos^2\beta$$
$$= [16.2d + 10.3(5-d)]\cos^2\beta$$
$$= [51.5 + 5.9d] \cos^2\beta$$
$$= [51.5 + 5.9d] \times 3/4 = 38.6 + 4.4d$$
$$\tau = [16.2d + 20.3(5-d)] \cos\beta \sin\beta$$
$$= [101.5 - 4.1d] \cos\beta\sin\beta$$
$$= [101.5 - 4.1d] \sqrt{3}/4 = 43.95 - 1.775d$$



When $d = 0$ $\tau = 43.95$ kPa $\sigma' = 38.6$ kPa $\phi_{mob} = 48.7°$

Greater than ϕ_{crit} so potentially unstable

$$I_D = \frac{0.95-0.8}{0.95-0.6} = 43\%$$

$$p' \sim 38.6 \times \frac{2}{3}$$

assuming $K_0 = \frac{1}{2}$

$$= 26 kPa$$

$$I_c = \ln\left(\frac{5000}{26}\right) = 5.26$$

$$I_R = I_D I_c - 1 = 1.26$$

$$\phi_{max} - \phi_{crit} = 5 I_R = 6^\circ$$

$$\phi_{max} \sim 45^\circ$$

Slope will definitely fail if water table reaches surface

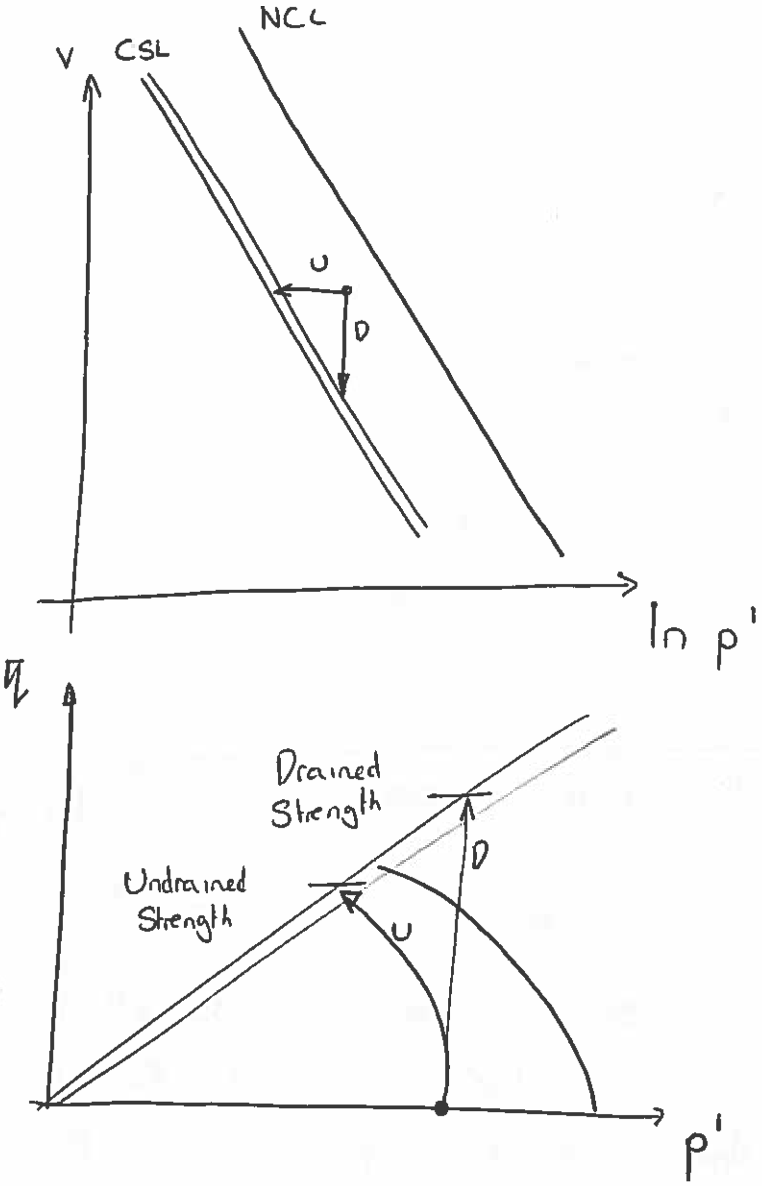
~~Mag z ber stable~~

Definitely stable if $\dfrac{(101.5 - 4.1d)}{(51.5 + 5.9d)\sqrt{3}} < \tan 39$

$$\sim d > 2.35m$$

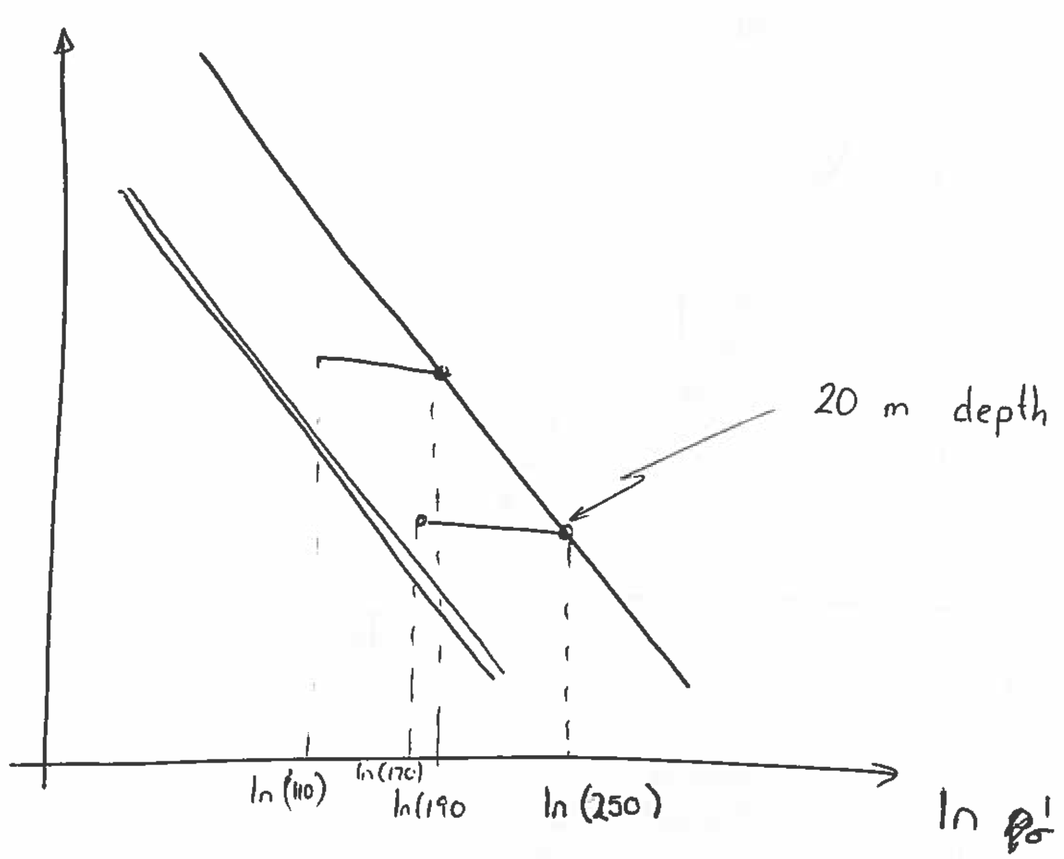
Definitely unstable if $d < 0.8m$

4. a)



Clays above CSL are "loose" and collapse on drained shearing due to large void spaces. On undrained shearing water cannot escape & hence pore pressure rises to allow stress path to approach CSL.
This rise in pore pressure reduces σ' and hence strength.
Drained strength thus exceeds undrained strength.

b)



Guess ~~bo~~ bulk unit weight $\sim 16 \ kNm^{-3}$

@ 20m $\quad \sigma_v \sim$ ~~5x~~ $20 \times 16 = 320$ kPa

$\sigma_v' \sim 320 - 150 = 170$ kPa

Approx 80 kPa of "surcharge" has been removed

@ 10m $\quad \sigma_v \sim 10 \times 16 = 160$ kPa

$\sigma_v' \sim 160 - 50 = 110$ kPa

$\sigma_v'{}_{max} \sim \underline{190 \text{ kPa}}$

$v \sim (3.767 + 0.26 - 0.05) - 0.26 \ln(190) + 0.05 \ln\left(\frac{190}{110}\right)$

$= 2.64$

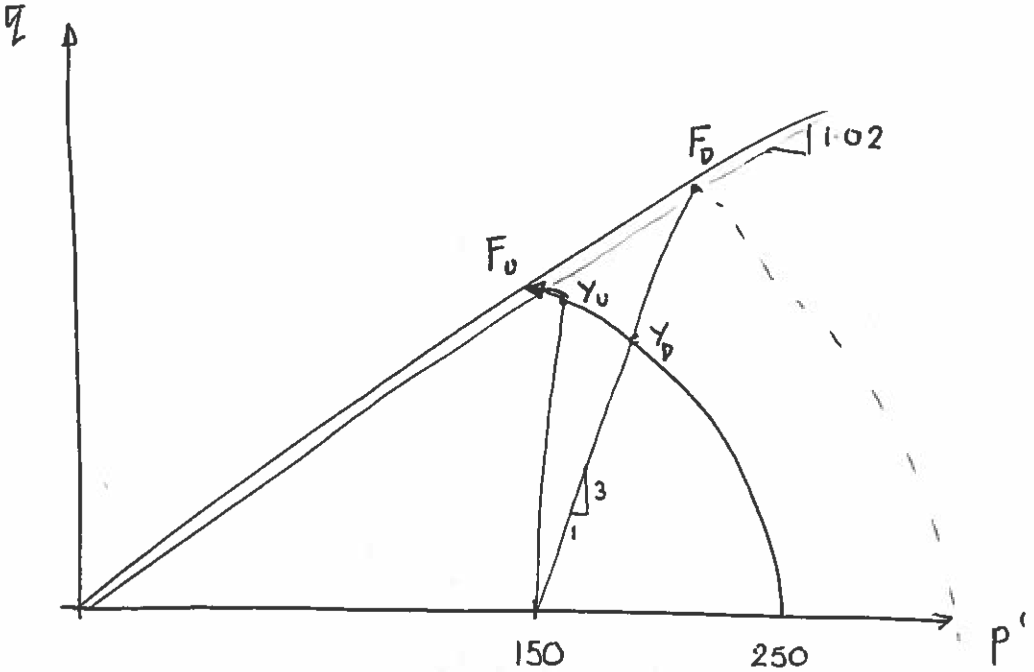
$\gamma = \frac{2.61 + 1.64}{2.64} \times 10 = 1610 \ kg/m^3 = 16.1 \ kN$

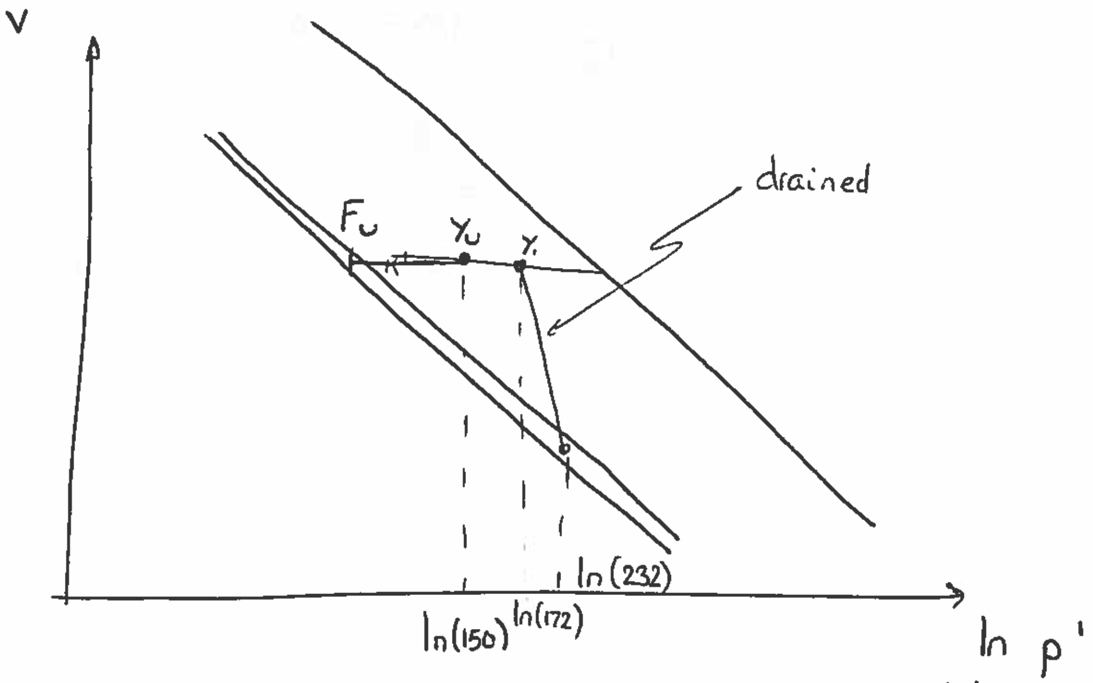
$\therefore$ assumption ok

@ 20 m $\quad v = 2.56$

$\gamma = 16.3 \ kNm^{-3}$

c)





At yield: (Y_D) $\quad 3(p' - 150) = 1.02\, p' \ln\left(\frac{250}{p'}\right)$

$\Rightarrow \quad p' \sim 172 \text{ kPa}$

$q \sim 66 \text{ kPa} \qquad \tau_y \sim \underline{33 \text{ kPa}}$

At failure (F_D) $\quad 3(p' - 150) = 1.02\, p'$

$p' = \frac{450}{1.98} = 227 \text{ kPa}$

$q = 232 \text{ kPa} \qquad \tau_f = 116 \text{ kPa}$

$v = 3.767 - 0.26 \times \ln(227) = 2.357$

$\varepsilon_v = \frac{0.2}{2.56} = \underline{\underline{8\%}}$

d) At yield Y_u

$$p' = 150\text{ kPa}$$

$$q = 1.02 \times 150 \ln\left(\frac{250}{150}\right)$$

$$= 78.2 \text{ kPa}$$

$$\tau_y = 39.1 \text{ kPa}$$

At failure F_u

$$v = 2.56$$

$$\Rightarrow 3.767 - 0.26 \ln(p') = 2.56$$

$$p' = \underline{\underline{103.8 \text{ kPa}}}$$

$$q = Mp' = 105.9 \text{ kPa}$$

$$c_u = \underline{\underline{52.9 \text{ kPa}}}$$