1 (a)

$$\gamma_d = \frac{G_s}{1+e}\gamma_w = \frac{2.65}{1+0.5}10 = \underline{17.7\ kN/m^3}$$

$$\gamma_s = \frac{G_s+e}{1+e}\gamma_w = \frac{2.65+0.5}{1+0.5}\times 10 = \underline{21\ kN/m^3}$$

(b)

$$\sigma = \gamma Z \cos^2\beta$$
$$= 17.7 \times 3 \times \cos^2 30$$
$$= 39.8\ kPa$$

$$\tau = \gamma Z \cos\beta \sin\beta$$
$$= 17.7 \times 3 \times \cos 30^\circ \sin 30^\circ$$
$$= 23.0\ kPa$$

$$I_D = \frac{0.85 - 0.65}{0.85 - 0.40} = 0.44$$

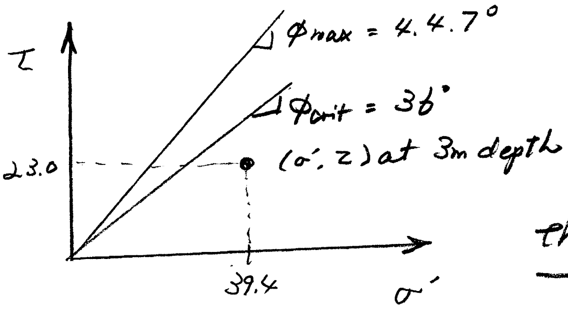
$$I_c = \ln \frac{20.000}{39.8} = 6.22$$

$$I_R = I_D I_c - 1$$
$$= 0.44 \times 6.22 - 1 = \underline{1.74}$$

$\phi_{max} - \phi_{crit} = 5 \times 1.74 = 8.68$ assuming plane strain conditions

$\phi_{crit} = 36^\circ$

$\phi_{max} = 36 + 8.68 = 44.7^\circ$



<u>the Slope is stable</u>.

(c) $\gamma z = 17.7 \times 0.5 + 21 \times 2.5$

$= 61.35\ kPa$

$\sigma = \gamma z \cos^2\beta = 61.35 \times \cos^2 30° = 46.0\ kPa$

$\tau = \gamma z \cos\beta \sin\beta = 61.35 \times \cos 30 \sin 30 = 26.6\ kPa$

$u = \gamma_w \cdot z_w \times \cos^2\beta = 10 \times 2.5 \times \cos^2 30° = 18.8\ kPa$

$\sigma' = \sigma - u = 46.0 - 18.8 = 27.2\ kPa$

$I_D = 0.44$ (as before)

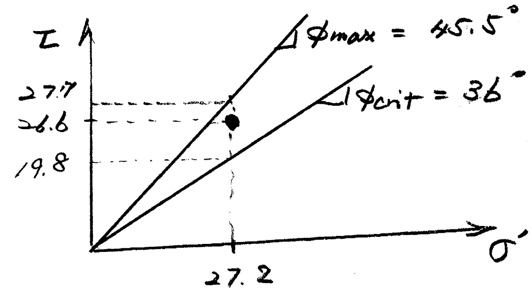
$I_c = \ln \dfrac{20000}{27.2} = 6.60$

$I_R = I_D I_c - 1 = 0.44 \times 6.60 - 1 = 1.90$

$\phi_{max} - \phi_{crit} = 5 \times 1.90 = 9.5$

$\phi_{crit} = 36°$

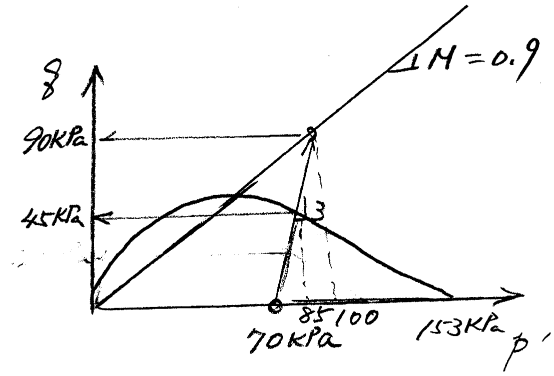
$\phi_{max} = 36° + 9.5° = 45.5°$



The stress state is below the peak failure line. But it is above the critical state line. So maybe safe. If the slope moves and the soil is sheared, the soil may soften and it will be in the metastable state, which may lead to slope failure.

(d) The clay is likely to be sheared in undrained conditions. The shear stress should be checked against the undrained shear strength, if known. Alternatively excess pore pressure needs to be estimated and the effective stress analysis is performed to check the stability of the slope.

2 (a) (i) $\sigma'_v = \sigma_v - u = 17 \times 10 - 10 \times 10 = 70$ kPa



$q_f = 90$ kPa

$p_f = 70 + \frac{90}{3} = 100$ kPa

$M = q_f / p_f = 90/100 = \underline{0.9}$ //

$q_y = 45$ kPa

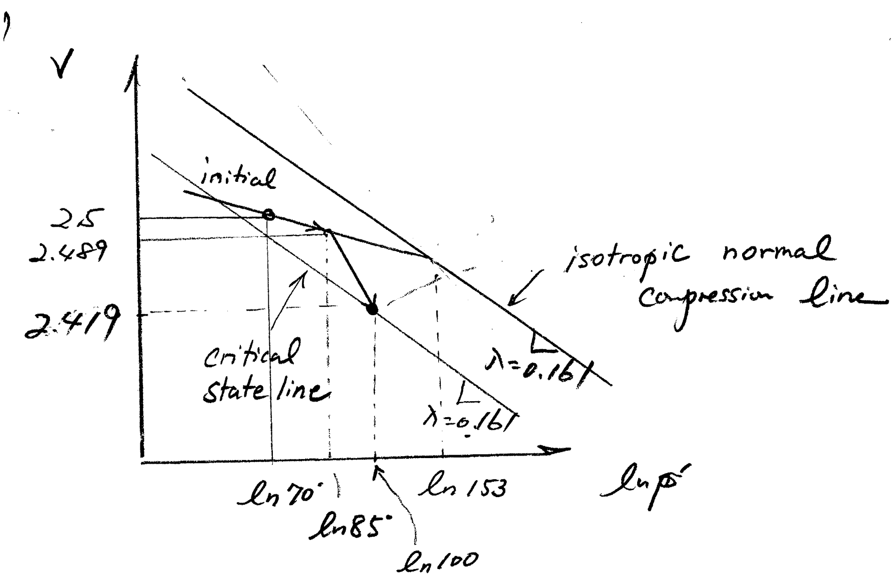
$p_y = 70 + \frac{45}{3} = 85$ kPa

$\frac{q_y}{p_y} = \frac{45}{85} = M \ln\left(\frac{p_c}{p_y}\right) = 0.9 \cdot \ln\left(\frac{p_c}{85}\right)$

$\ln(p_c/85) = 0.588$

$p_c = \underline{153 \text{ kPa}}$ //

(ii), (iii)



$\kappa = \frac{2.5 - 2.489}{\ln(85/70)} = 0.057$

(b)

Critical state line

$v = \Gamma - \lambda \ln p'$

$2.419 = \Gamma - 0.161 \ln 100$

$\Gamma = 3.16$

p' at undrained failure when $v = 2.5$ is

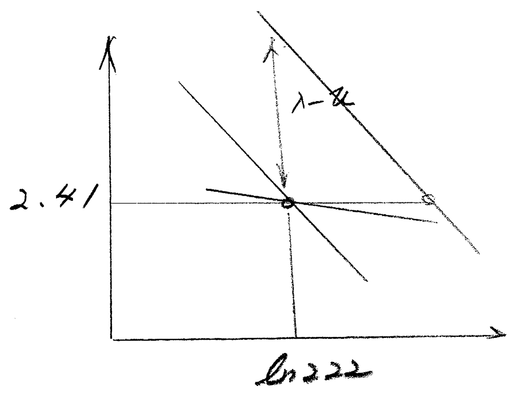
$2.5 = 3.16 - 0.161 \ln p'$

$p_f' = 60.3$ kPa

$q_f = M p_f' = 0.9 \times 60.3 = \underline{54.3 \text{ kPa}}$ //

(c) $200 = 0.9\, p_f'$

$p_f' = 222$ kPa



$v = \Gamma - \lambda \ln p_f' = 3.16 - 0.161 \ln 222 = 2.29$

Normal compression line $v = \Gamma + (\lambda - \kappa) - \lambda \ln p_c'$

$2.29 = 3.16 + (0.161 - 0.057) - 0.161 \ln p_c'$

$p_c = 423$ kPa

Need to add $423 - 70 = 353$ kPa temporary surcharge. If $\gamma = 20\ kN/m^3$, this is about 17.6 m thick soil. Too much. Better to improve the soil by grouting (i.e. mixing with cement)

(a) At depth of 10m, existing in-situ stresses : (present day values)

$\sigma_v = 17.5 \times 10 = 175$ kPa

Pore pressure $u_o = 10 \times 7 = 70$ kPa

$\therefore \sigma_v' = \sigma_v - u_o = 175 - 70 = \underline{\underline{105 \text{ kPa}}}$

$K_o = \dfrac{\sigma_h'}{\sigma_v'} = 0.8 \quad \therefore \sigma_h' = 0.8 \times 105 = \underline{\underline{84 \text{ kPa}}}$

$OCR = \dfrac{\sigma_{vo}'}{\sigma_v'} = 2.0 \quad \therefore \sigma_{vo}' = 2 \times 105 = \underline{210 \text{ kPa}}$

$K_{nc} = \dfrac{\sigma_{ho}'}{\sigma_{vo}'} = 0.6$ where σ_{vo}', σ_{ho}' are previous maximum vertical and horizontal effective stresses when clay was in normally consolidated state

$\therefore \sigma_{ho}' = 0.6 \times 210 = \underline{126 \text{ kPa}}$ [20%]

(b)(i) Existing, present day q, p, p' :

$q = \sigma_v - \sigma_h$; $p = \frac{1}{3}\sigma_v + \frac{2}{3}\sigma_h$; $p' = \frac{1}{3}\sigma_v' + \frac{2}{3}\sigma_h'$

(and $p = p' + u_o$) $u_o = 70$ kPa

$\sigma_v = 175$ kPa $\quad \sigma_h = \sigma_h' + u_o = 84 + 70 = 154$ kPa

$\sigma_v' = 105$ kPa $\quad \sigma_h' = 84$ kPa

$\therefore q = 175 - 154 = 21$ kPa

$p' = \frac{1}{3} \times 105 + \frac{2}{3} \times 84 = 91$ kPa

$p = p' + u_o = 91 + 70 = 161$ kPa

} initial values at start of Tests 1 and 2

In the triaxial test $\sigma_a = \sigma_v$, $\sigma_r = \sigma_h$

$\Delta q = \Delta\sigma_a - \Delta\sigma_r \qquad \Delta\sigma_r = 0 \Rightarrow \Delta q = \Delta\sigma_a$

<u>Yield</u> occurs at $\Delta\sigma_a = \Delta q = 40$ kPa

$\Delta p = \frac{1}{3}\Delta\sigma_a + \frac{2}{3}\Delta\sigma_r = \frac{1}{3}\Delta\sigma_a \Rightarrow \dfrac{\Delta q}{\Delta p} = 3$

Δp at yield $= \dfrac{\Delta q}{3} = \dfrac{40}{3} = \underline{13 \text{ kPa}}$

$\therefore$ at yield $p_y = 161 + 13 = \underline{174 \text{ kPa}}$

Assuming linear elastic, isotropic behaviour prior to yield, effective stress path is <u>vertical</u> : $\Delta p' = 0$

hence at yield (point B')
$$q = 21 + 40 = 61 \text{ kPa}$$
$$p'_y = 91 \text{ kPa (as at start of test)}$$

<u>excess</u> pore pressure $\Delta u_y = p_y - p'_y - u_0$
$$= 174 - 91 - 70 = \underline{13 \text{ kPa}}$$

(since $\Delta p' = 0$, $\Delta u = \Delta p$)

At failure $q_f = 2c_u = 2 \times 40 = 80$ kPa

$\therefore$ Δq from A' to C' $= 80 - 21 = 59$ kPa

$q_f = M p'_f \Rightarrow p'_f = \dfrac{q_f}{M} = q_f$ ($M = 1.0$)

$\therefore$ $p'_f = 80$ kPa

$\Delta p = \dfrac{\Delta q}{3} = \dfrac{59}{3} \simeq 20$ kPa $\Rightarrow$ at C, $p_f = 161 + 20 = 181$ kPa

$\therefore$ <u>excess</u> pore pressure $= p_f - p_f' - u_0$
$$= 181 - 80 - 70 = \underline{31 \text{ kPa}}$$ [40%]

(b)(ii) If the water level in the tank is kept at the level at which local undrained failure occurred, dissipation of the excess pore pressure will take place. The total stresses remain the same, hence points C and D are coincident. Effective stress path is C' → D', i.e. horizontal. Tank would settle by an additional amount as the excess pore pressure dissipated. [15%]

(b)(iii) Test 2 : drained test $\Delta\sigma_r = 0$, $\Delta u = 0$, $\Delta\sigma_r' = 0$

$\Delta p' = \frac{1}{3}\Delta\sigma_a' + \frac{2}{3}\Delta\sigma_r' = \frac{1}{3}\Delta\sigma_a'$

$\Delta q = \Delta\sigma_a - \Delta\sigma_r = \Delta\sigma_a$

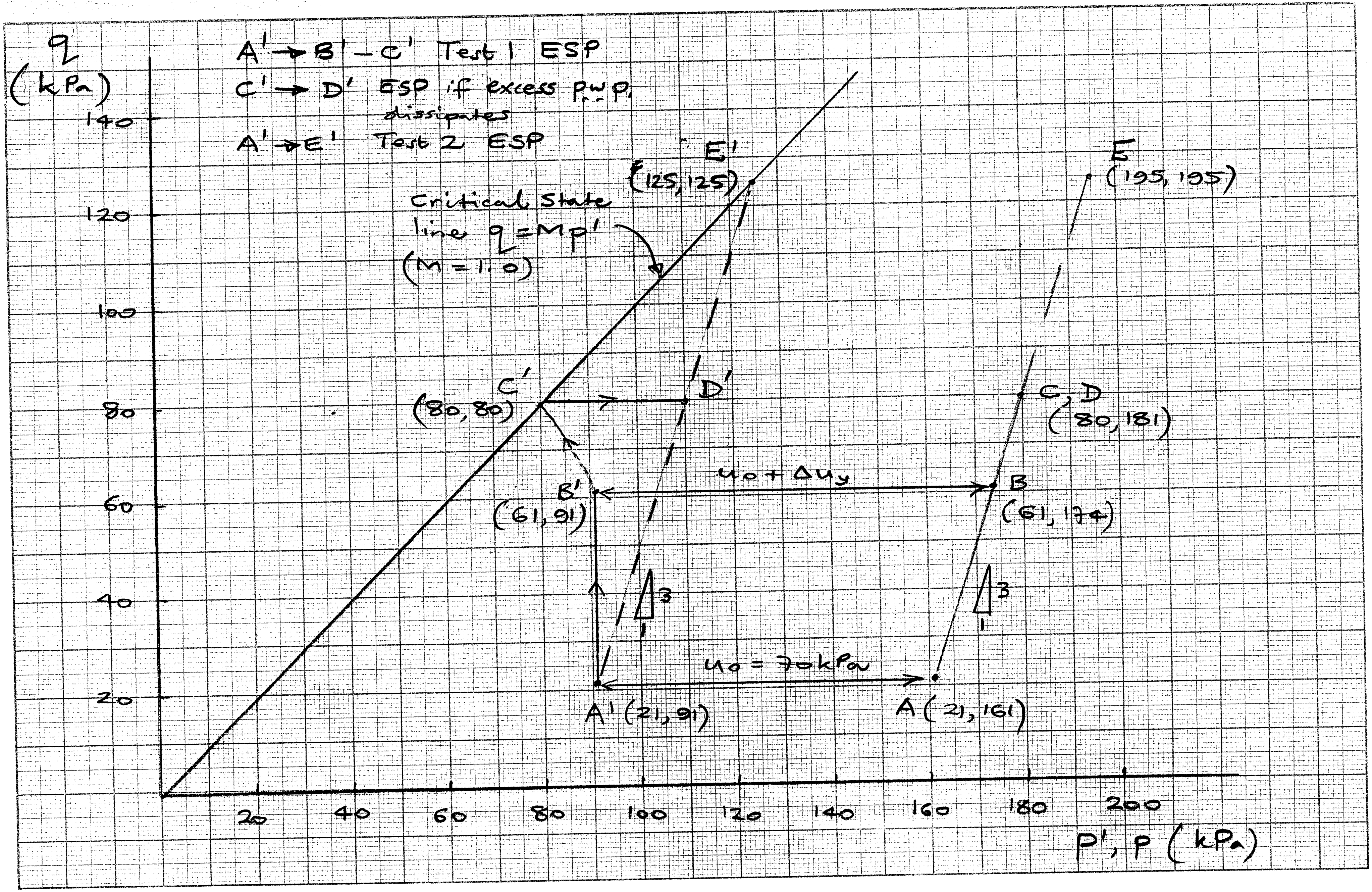
$\therefore$ Effective stress path $\dfrac{\Delta q}{\Delta p'} = \dfrac{\Delta\sigma_a'}{\frac{1}{3}\Delta\sigma_a'} = 3$

Graphical construction of ESP at 3:1 slope intersects critical state line ($q = Mp'$) at E' at which q_f = 125 kPa.

Under drained conditions, corresponding to slow filling of the tank, the increase in vertical total stress $\Delta\sigma_a$ necessary to cause local failure is 125 − 21 = 104 kPa.

Under undrained conditions, corresponding to rapid filling of the tank, $\Delta\sigma_a$ to cause local failure is 80 − 21 = 59 kPa.

Hence almost twice the amount of vertical stress can be applied if the water test in the tank is undertaken slowly — the soil strengthens as the stress is steadily and slowly applied. [25%]



A' → B' – C' Test 1 ESP
C' → D' ESP if excess p.w.p. dissipates
A' → E' Test 2 ESP
q (kPa)
140
120
100
80
60
40
20
Critical State line q = Mp' (M = 1.0)
E' (125, 125)
E (195, 195)
C' (80, 80)
D'
C, D (80, 181)
u0 + Δuy
B' (61, 91)
B (61, 174)
3
1
u0 = 70 kPa
A' (21, 91)
A (21, 161)
20
40
60
80
100
120
140
160
180
200
p', p (kPa)

Q.4

(a) see later

(b) From Geotechnical Engineering Data Book, undrained plastic - elastic <u>expansion</u> given by

$$\delta\sigma_c = c_u \left[1 + \ln \frac{G}{c_u} + \ln \frac{\delta A}{A} \right] \quad \dots \quad (1)$$

where δA is expansion $= A - A_0$ caused by <u>increase</u> of pressure $\delta\sigma_c = \sigma_c - \sigma_0$

σ_c = cavity pressure
σ_0 = initial total stress in ground

Tunnel construction can be thought of as undrained plastic - elastic <u>contraction</u> in an exactly analagous way, with <u>$\delta\sigma_c = \sigma_0 - \sigma_c$</u> where σ_c = radial lining pressure, supporting the cavity.

Tunnel diameter 4m $\Rightarrow$ radius r_{co} = 2m

at 25m depth $\sigma_0 = \gamma z = 20 \times 25 = 500$ kPa

σ_c = average lining stress = 150 kPa

$\therefore \delta\sigma_c = 500 - 150 = 350$ kPa

$c_u = 125$ kPa

from eqn (1) $\dfrac{\delta A}{A} = \dfrac{c_u}{G} \exp\left(\dfrac{\delta\sigma_c}{c_u} - 1 \right)$

radial deformation at tunnel boundary ρ_c = 20mm

$$\frac{\rho_c}{r_{co}} \simeq \frac{1}{2} \frac{\delta A}{A} = \frac{1}{2} \frac{c_u}{G} \exp\left(\frac{\delta\sigma_c}{c_u} - 1 \right)$$

$$\therefore \quad \frac{0.020}{\cancel{2}} = \frac{1}{\cancel{2}} \times \frac{125}{G} \exp\left(\frac{350}{125} - 1\right)$$

$$\therefore \quad G = \frac{125}{0.020} \exp 1.8 \quad kN/m^2 = 37.8 \times 10^3 \ kN/m^2$$

$$= \underline{37.8 \ MN/m^2}$$

[50%]

(~~b~~ c) tunnel diameter $= 10m$, $r_{co} = 5m$

$$\rho_c = r_{co} \frac{1}{2} \frac{c_u}{G} \exp\left(\frac{\delta\sigma_c}{c_u} - 1\right)$$

$$\delta\sigma_c = 500 - 200 = 300 \ kN/m^2$$

$$\therefore \rho_c = 5 \cdot \frac{1}{2} \cdot \frac{125}{37.8 \times 10^3} \exp\left(\frac{300}{125} - 1\right) \ m$$

$$= 33.5 \times 10^{-3} \ m = \underline{33 mm}$$

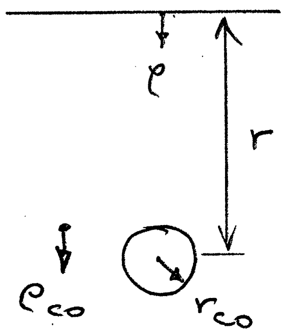
[20%]

(d) Constant volume (undrained behaviour in clay)

$$\cancel{2\pi} \cdot r \cdot \rho = \cancel{2\pi} r_{co} \cdot \rho_{co}$$

At ground surface $r = 25m$

$$\therefore \quad \rho = 33 \cdot \frac{5}{25} = \underline{6.6 \ mm}$$



[15%]

(~~e~~ a) Self-boring pressuremeter test minimizes soil disturbance and can give profile of properties with depth as it drills downwards through the ground.

Properties measured: σ_{ho} (and hence K_o if pore pressure known), G, c_u

[15%]

3D2 Geotechnical Engineering II

1 (a) 17.7 kN/m^3, 21 kN/m^3, assuming the unit weight of water is 10 kN/m^3

(b) σ = 39.8 kPa, τ = 23.0 kPa, ϕ_{crit} = 36°, ϕ_{peak} = 44.7°

(c) σ = 46.0 kPa, τ = 26.6 kPa, u = 18.8 kPa, σ'= 27.2 kPa, ϕ_{crit} = 36°, ϕ_{peak} = 45.5°

(d) –

2(a) (i) M = 0.9, p_c = 153 kPa, (ii) κ = 0.057, (iii) –

(b) q_f= 54.3 kPa

(c) p = 423 kPa

3 (a) σ_v' = 105 kPa, σ_h' = 84 kPa, σ_{v0}' = 210 kPa, σ_{h0}' = 126 kPa

(b) (i) Δu at yield = 13 kPa, Δu at ultimate failure = 31 kPa

(ii) –

(iii) drained strength at failure = 125 kPa

4(a) –

(b) G = 37.8 MN/m^2

(c) 33 mm

(d) 6.6 mm