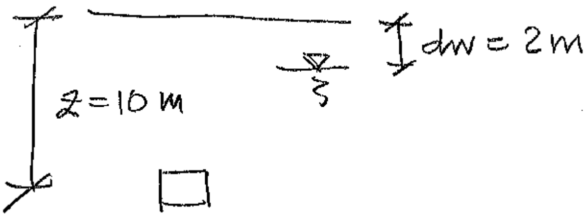


$\gamma = 17\ kN/m^3$

$G_S = 2{,}65$

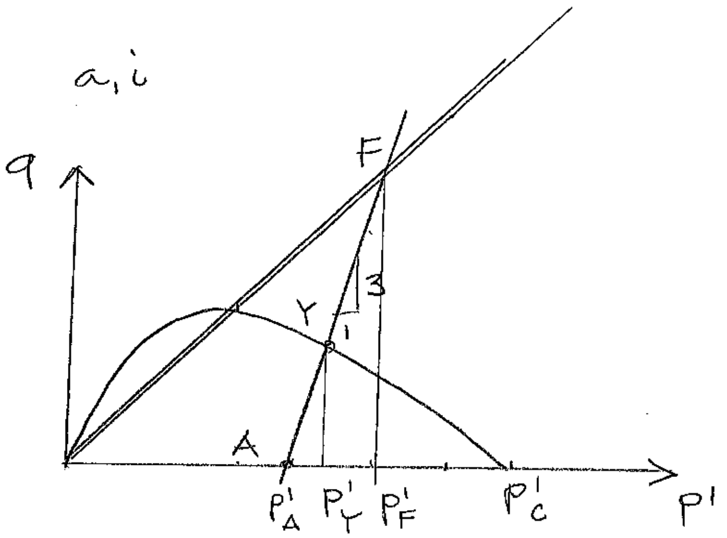
$\gamma_w \hat{=} 10\ kN/m^3$

$\sigma_v = 17 \times 10 = 170\ kPa$

$u = 10 \times 8 = 80\ kPa$

$\sigma_v' = \sigma_v - u = 90\ kPa$

a, i



$p_A = \sigma_v' = 90\ kPa$

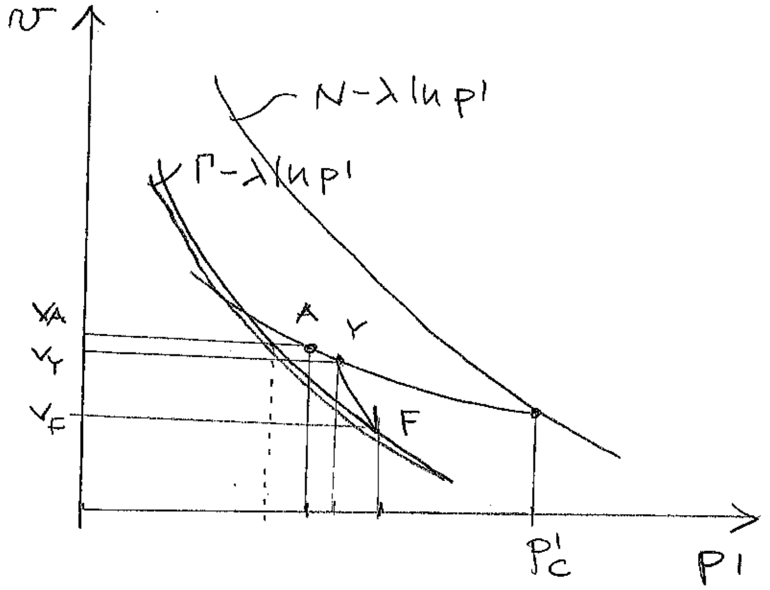
$q_Y = 50\ kPa \qquad p_Y' = 90 + \frac{50}{3} = 106{,}7\ kPa$

$q_F = 120\ kPa \qquad p_F' = 90 + \frac{120}{3} = 130\ kPa$

$M = \frac{q_F}{p_F'} = \frac{120}{130} = 0{,}923$ ✓

$$\frac{q_Y}{p_Y'} = M \ln \frac{p_c'}{p_Y'} \implies p_c' = p_Y' \, e^{\frac{q_Y}{M p_Y'}}$$

$$p_c' = 106{,}7 \, e^{\frac{50}{0{,}923 \times 106{,}7}} = 177{,}3\ kPa$$ ✓



$e_A = wG_s = 0{,}5 \times 2{,}65 = 1{,}325$

$v_A = 1 + e_A = 2{,}325$

$\epsilon_v = \dfrac{\Delta V}{V_0}$

$$\epsilon_{vY} = \frac{v_A - v_Y}{v_A} = \frac{\Delta v_Y}{v_A} = 0{,}002$$

$$\Delta v_Y = v_A \times 0{,}002 = 0{,}002 \times 2{,}325 = 0{,}005$$

$$\Delta v_Y = \kappa \ln \frac{p'_Y}{p_A} \Rightarrow \kappa = \Delta v_Y / \ln(p'_Y/p'_A)$$

$$\kappa = \frac{0{,}005}{\ln(106{,}7/90)} = 0{,}029 \checkmark$$

$$v_A = N - \lambda \ln p'_c + \kappa \ln p'_c/p'_A = \Gamma + \lambda - \kappa - \lambda \ln p'_c + \kappa \ln p'_c/p'_A$$

$$v_F = \Gamma - \lambda \ln p'_F$$

$$v_A - v_F = \lambda - \kappa - \lambda \ln p'_c/p'_F + \kappa \ln p'_c/p'_A$$

$$v_A - v_F = 0{,}025 \times v_A = \lambda(1 - \ln p'_c/p'_F) - \kappa(1 - \ln p'_c/p'_A)$$

$$\lambda(1 - \ln p'_c/p'_F) = 0{,}025 \times v_A + \kappa(1 - \ln p'_c/p'_A)$$

$$\lambda = \frac{0{,}025 \times v_A + \kappa(1 - \ln p'_c/p'_A)}{1 - \ln p'_c/p'_F}$$

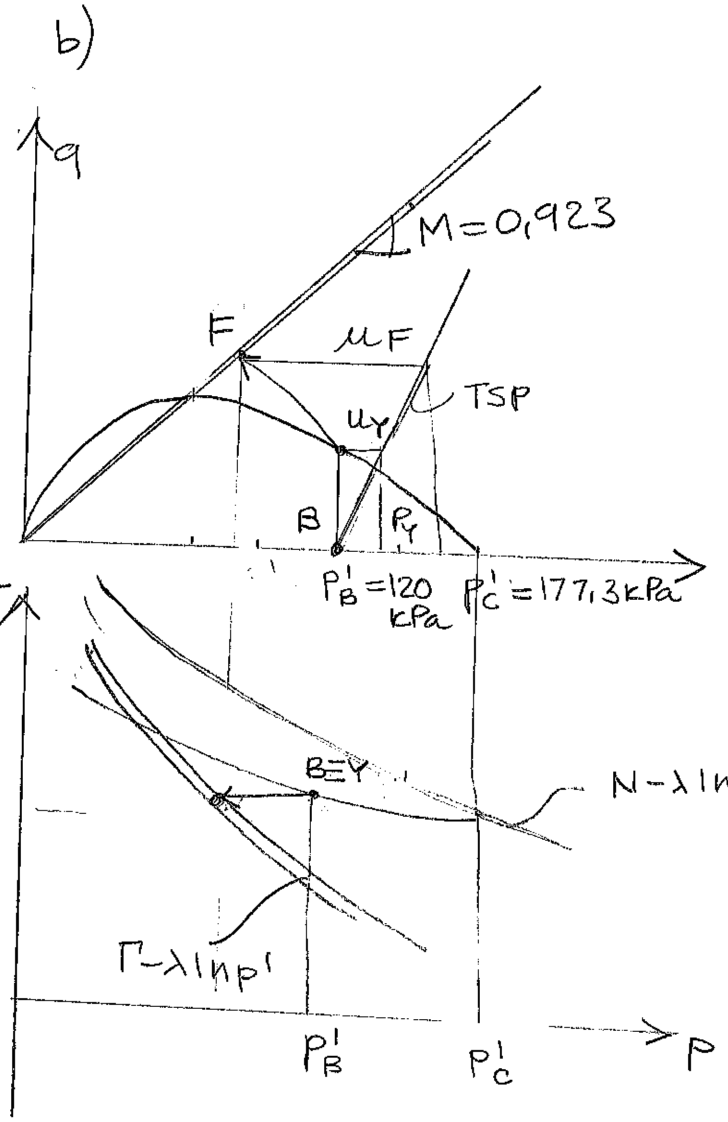
$$\lambda = \frac{0{,}025 \times 2{,}325 + 0{,}029(1 - \ln 177{,}3/90)}{1 - \ln 177{,}3/130} = 0{,}098 \checkmark$$

$$v_F = v_A(1 - 0{,}025) = 2{,}325 \times 0{,}975 = 2{,}267$$

$v_F = \Gamma - \lambda \ln p'_F \qquad \Gamma = v_F + \lambda \ln p'_F = 2{,}267 + 0{,}098 \ln 130$

$\Gamma = 2{,}744$

b)



$\Gamma = 2{,}744$

$N = \Gamma + \lambda - \kappa = 2{,}744 + 0{,}098 - 0{,}029$

$N = 2{,}813$

$S_u = q/2$

$S_{uY} = \frac{q_Y}{2} = \frac{43{,}24}{2} = 21{,}6$ kPa

$u_Y = 14{,}4$ kPa

$S_{uF} = \frac{q_F}{2} = \frac{72{,}02}{2} = 36$ kPa

$u_F = 72$ kPa

$$\frac{q_Y}{p'_Y} = M \ln \frac{p'_C}{p'_Y} \qquad q_Y = p'_Y M \ln \frac{p'_C}{p'_Y}$$

$$q_Y = 120 \times 0{,}923 \times \ln \frac{177{,}3}{120} = 43{,}24 \text{ kPa}$$

$$p_Y = p'_B + \frac{q_Y}{3} = 120 + \frac{43{,}24}{3} = 134{,}41 \text{ kPa} \qquad u_Y = p_Y - p'_B = 14{,}41 \text{ kPa}$$

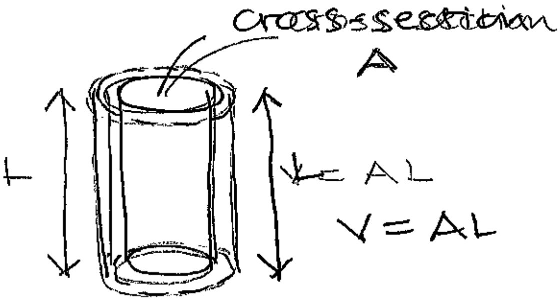
$$v_B = N - \lambda \ln p'_C + \kappa \ln \frac{p'_C}{p'_B} = 2{,}813 - 0{,}098 \ln 177{,}3 + 0{,}029 \ln \frac{177{,}3}{120} =$$

$$v_B = 2{,}317 = v_F \qquad v_F = \Gamma - \lambda \ln p'_F \qquad p'_F = e^{\frac{\Gamma - v_F}{\lambda}}$$

$$p'_F = e^{\frac{2{,}744 - 2{,}317}{0{,}098}} = 78{,}03 \text{ kPa} \qquad q_F = M p'_F = 0{,}923 \times 78{,}03 = 72{,}02 \text{ kPa}$$

$$p_F = p'_B + \frac{1}{3} q_F = 120 + \frac{72{,}02}{3} = 144 \text{ kPa} \qquad u_F = p_F - p'_F = 144 - 72 = 72 \text{ kPa}$$

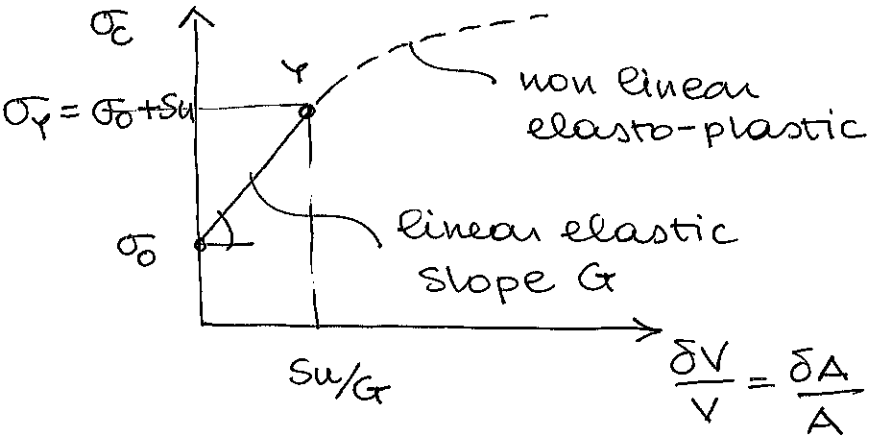
(a) Expansion of pressuremeter caused by increase of cavity pressure:



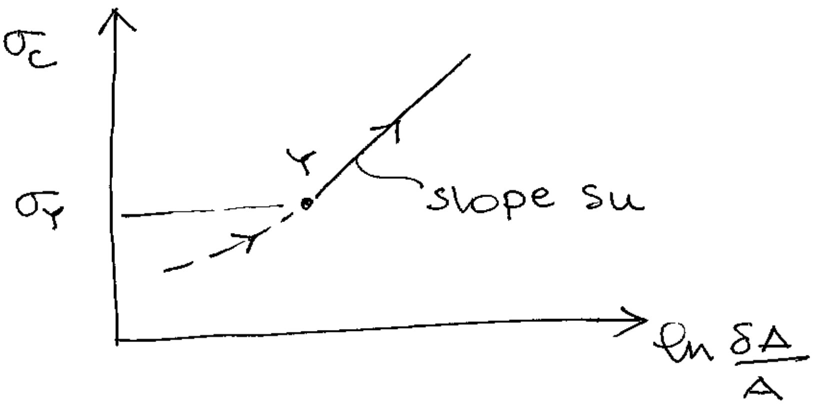
$$\delta\sigma_c = \sigma_c - \sigma_0$$

is $\delta A = A - A_0$. Note that $\frac{\delta A}{A} = \frac{\delta V}{V}$

Plot cavity pressure, σ_c, vs change of volume $\frac{\delta V}{V}$



Also plot cavity pressure, σ_c, vs $\ln \frac{\delta A}{A}$



G can be determined from the slope of first part of $\left(\sigma_c : \frac{\delta A}{A}\right)$ plot. If unload reload loop carried out at later stage in pressuremeter test G can be found as slope of unload reload loop (more reliable, get rid of disturbance effects)

continued →

from data book:

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

If point Y can be clearly identified in the $(\sigma_c : \frac{\delta A}{A})$ plot (as the end point of linear part of plot), then:

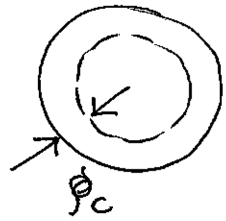
$$\sigma_Y = \sigma_0 + su = \gamma z + su \Rightarrow su = \frac{\sigma_Y}{\gamma z}$$

In the graph of $(\sigma_c : \ln\frac{\delta A}{A})$, su can be estimated from the slope of the linear portion of the plot, because of equation (1). Generally, this is more reliable than first method (we rely on many points, not just the stress at yield)

2(b)

$$\delta\sigma_c = s_u \left[1 + \ln \frac{G}{s_u} + \ln \frac{\delta A}{A} \right]$$

tunnel radius R



$$\frac{\delta A}{A} = \frac{s_u}{G} \exp\left(\frac{\delta\sigma_c}{s_u} - 1 \right)$$

contraction of cavity with complete unloading $\delta\sigma_c = \gamma z$

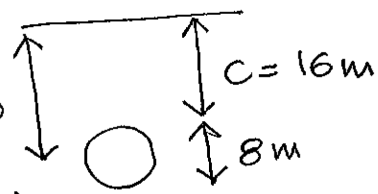
radial ground movement ρ_c

$$\delta A \sim 2\pi R \rho_c$$
$$A = \pi R^2$$

$$\frac{\delta A}{A} = \frac{2\pi R \rho_c}{\pi R^2} = 2\frac{\rho_c}{R}$$

$$\rho_c = \frac{R}{2} \frac{s_u}{G} \exp\left(\frac{\gamma z}{s_u} - 1 \right)$$

2(c) $s_u = 80$ kPa $\gamma = 20$ $D = 8$ m



$$\sigma_T = \gamma D \left(\frac{C}{D} \right) - 2 s_u \ln\left(2\frac{C}{D} + 1 \right) =$$

$$= 20 \times \cancel{8} \left(\frac{16}{\cancel{8}} \right) - 2 \cdot 80 \cdot \ln\left(2 \times \frac{\overset{2}{\cancel{16}}}{\cancel{8}} + 1 \right) =$$

$$= 320 - 160 \ln 5 = 62.5 \text{ kPa} > 0$$

tunnel requires support

OR

$$N = \frac{\gamma z}{s_u} = \frac{20 \times 20}{80} = \frac{400}{80} = 5 = N_c$$

Tunnel requires support

2(d) $\delta\sigma_c = s_u\left[1 + \ln\frac{G}{s_u} + \ln\frac{\delta A}{A}\right]$

$\therefore \ln\frac{\delta A}{A} = \frac{\delta\sigma_c}{s_u} - 1 - \ln\frac{G}{s_u}$

or $\frac{\delta A}{A} = \exp\left[\frac{\delta\sigma_c}{s_u} - 1 - \ln\frac{G}{s_u}\right]$

$\sigma_0 = \gamma z = 400\text{ kPa}$

$\delta\sigma_c = \sigma_0 - \sigma_r = 400 - 240 = 160\text{ kPa}$

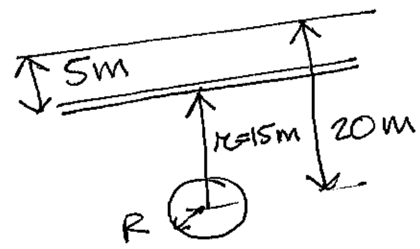
$\frac{\delta A}{A} = \exp\left[\frac{160}{80} - 1 - \ln\frac{20'000}{80}\right] =$

$= 0{,}011$

$\frac{\delta A}{A} = 2\frac{\rho_c}{R}$ (see 2(b))

$\rho_c = \frac{R}{2}\frac{\delta A}{A} = \frac{4}{2} \times 0{,}011 = 0{,}022\text{ m}$

or 22 mm



in undrained conditions

$\cancel{2\pi} r\rho = \cancel{2\pi} R\rho_c$

$\rho = \frac{R}{r}\rho_c$ $R = 4\text{ m}$ $r = 15\text{ m}$

$\rho = \frac{4}{15} \times 0{,}022 \cong 0{,}006\text{ m} = 6\text{ mm}$

Until the soil is elastic, the effective stress path is vertical upwards in (s', t) space ($\Delta s' = 0$), and then the stress point climbs up the Mohr Coulomb failure criterion.

In (σ'_v, σ'_h) space, until the soil is elastic it follows the path with $\frac{\Delta\sigma'_v}{\Delta\sigma'_h} = -1$ and then it climbs the M.C failure criterion.

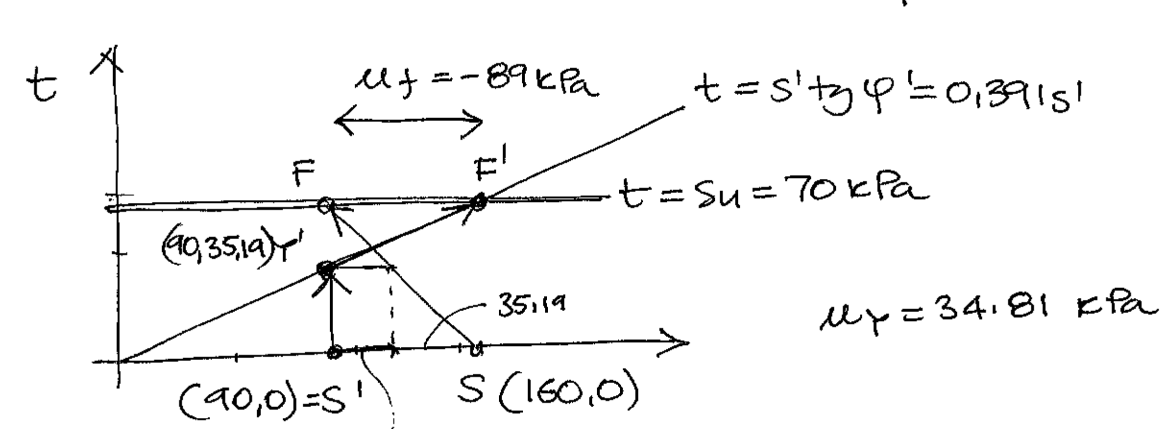
The final value of s' can be found from

$t' = s' \sin\varphi' \Rightarrow \quad s' = 70/\sin 23° = 179$ kPa

the pore water pressure at failure is

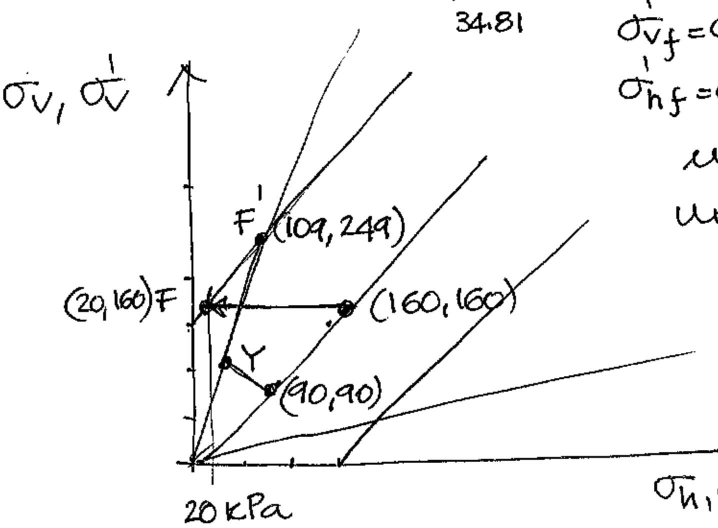
$u_f = s - s' = 90 - 179 = -89$ kPa

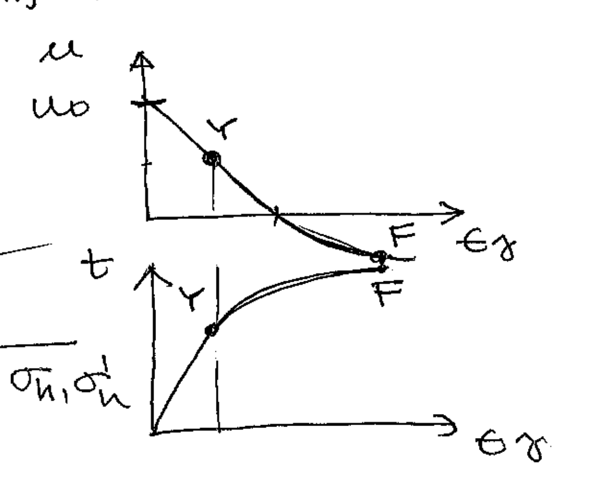
$t_Y = 0.391 \times s' = 0.391 \times 90 =$
$= 35.19$ kPa



$u_Y = 34.81$ kPa

$\sigma'_{vf} = \sigma_{vf} + u_f = 160 + 89 = 249$ kPa
$\sigma'_{hf} = \sigma_h + u_f = 20 + 89 = 109$ kPa





3b

as the excess pore water pressure dissipates the point representative of the state of effective stress at A will move horizontally to the left in (s, t) space and at $45°$ left and down in the (σ_h, σ_v) space.

In fact, total stress constant

$\Delta\sigma_v = 0 \quad \Delta\sigma_h = 0 \quad \Delta s = 0 \quad \Delta t = 0$

$\Delta\sigma'_v = -\Delta u \qquad \Delta s' = -\Delta u$

$\Delta\sigma'_h = -\Delta u \qquad \Delta t' = 0$

$$\frac{\Delta\sigma'_v}{\Delta\sigma'_h} = +1$$

Before the excess pore water pressure is fully dissipated the effective stress path will cross the MC failure criterion and the soil will fail in active state. This is because the final effective stress state is incompatible with the failure criterion (see plots above)

3c initial state @ A same as in point (a) above. Final state @ A:

$\sigma_v = \gamma z = 160$ kPa $\qquad s = \frac{\sigma_v + \sigma_h}{2} = \frac{180}{2} = 90$ kPa

$\sigma_h = 20$ kPa $\qquad t = \frac{\sigma_v - \sigma_h}{2} = \frac{140}{2} = 70$ kPa

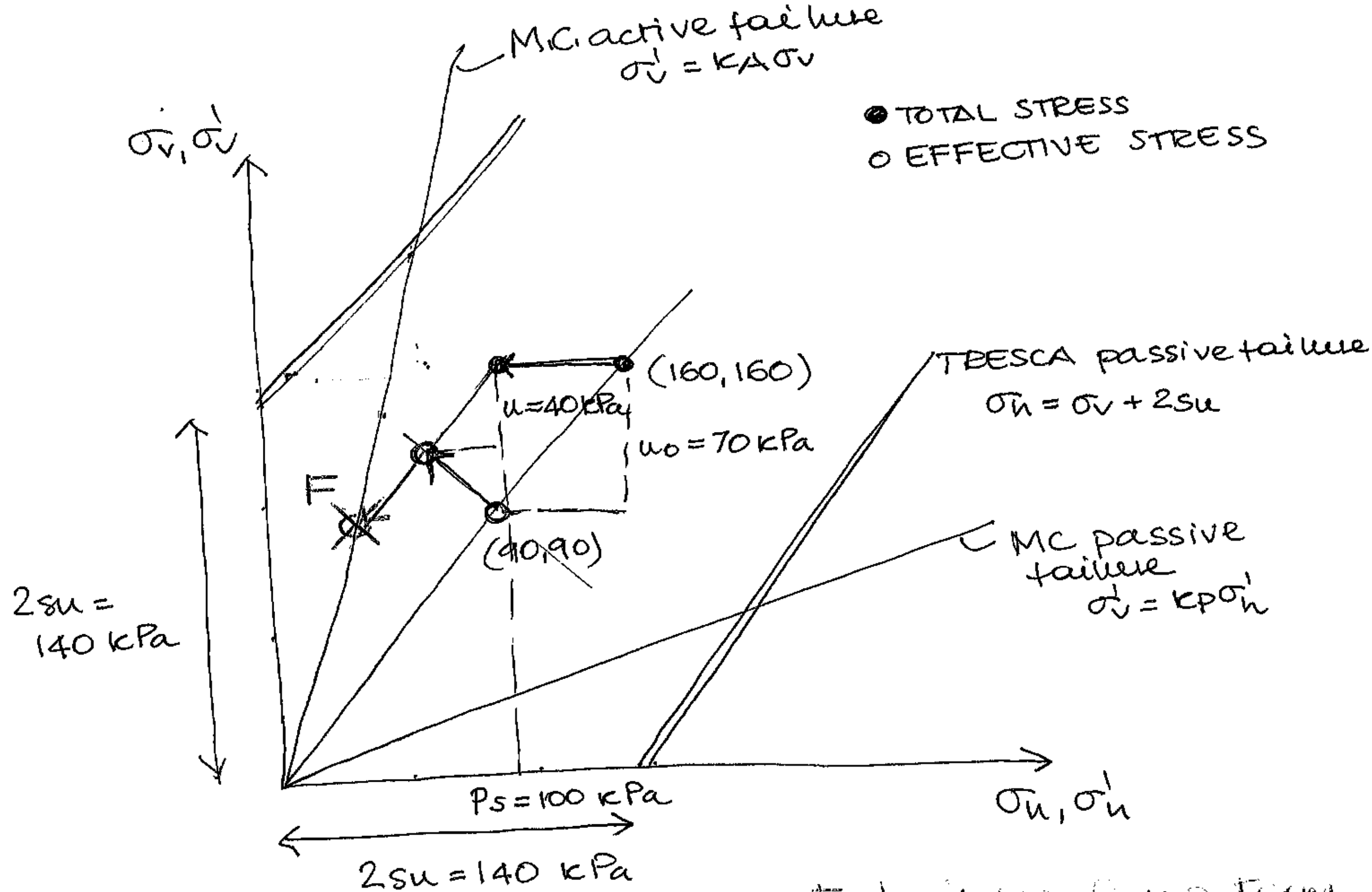
$t = s_u = 70$ kPa

the soil fails in undrained conditions

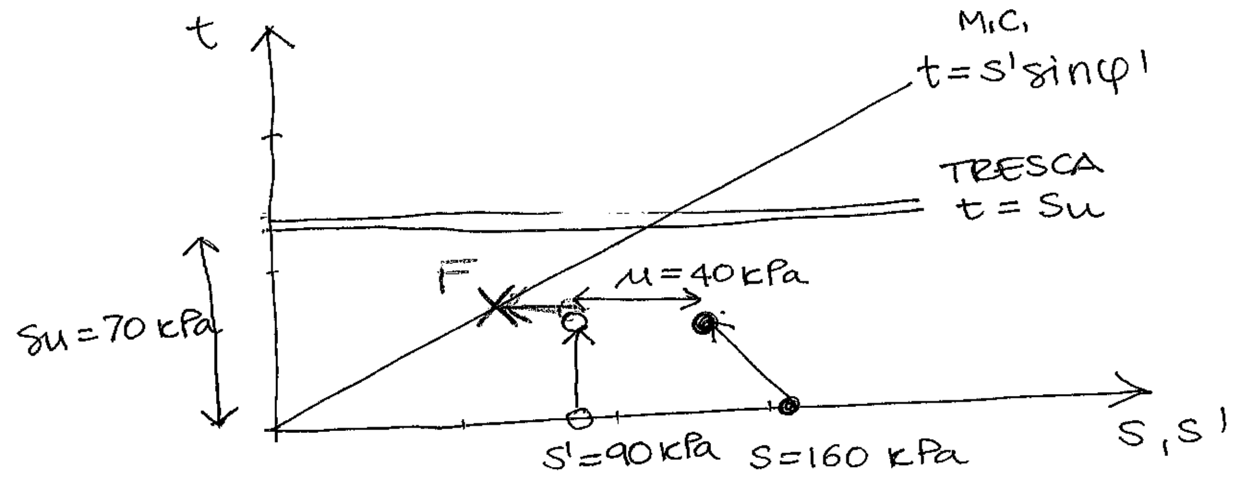
continued →

$\sigma'_v = \sigma_v - u = 160 - 40 = 120$ kPa

$\sigma'_h = \sigma_h - u = 100 - 40 = 60$ kPa

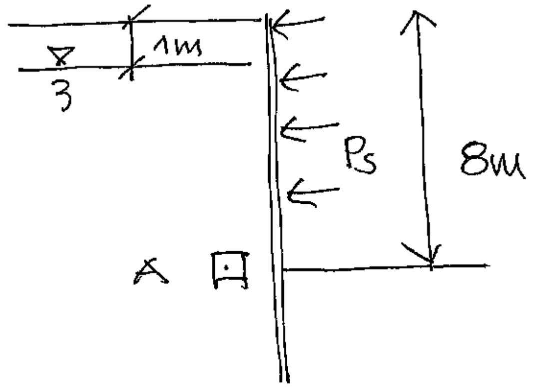


F, failure long term



$$SF = \frac{S_u}{t} = \frac{70}{30} = 2{,}33$$ ✓

(3)



$\gamma = 20\,kN/m^3$

$S_u = 70\,kPa$

$\varphi' = 23°$

$K_A = \dfrac{1-\sin\varphi'}{1+\sin\varphi'} = 0{,}438$

$K_P = 1/K_A = 2{,}283$

$\sin\varphi' = 0{,}391$

(a) initial state @ A

- $\sigma_v = \gamma z = 20 \times 8 = 160\,kPa$

  $u = \gamma_w z_w = 10 \times 7 = 70\,kPa$
- $\sigma'_v = \sigma_v - u = 160 - 70 = 90\,kPa$
- $\sigma'_h = K_0 \sigma'_v = \sigma'_v = 90\,kPa$
- $\sigma_h = \sigma'_h + u = 160\,kPa$

- $s = \dfrac{\sigma_v + \sigma_h}{2} = 160\,kPa$ • $s' = s - u = 160 - 70 = 90\,kPa$

- $t = \dfrac{\sigma_v - \sigma_h}{2} = 0\,kPa$

final state @A, short term

$\sigma_v = 160\,kPa$

$\sigma_h = P_s = 100\,kPa$

$s = \dfrac{\sigma_v + \sigma_h}{2} = \dfrac{260}{2} = 130\,kPa$

$t = \dfrac{\sigma_v - \sigma_h}{2} = \dfrac{160-100}{2} = 30\,kPa$

$\Delta s = 130 - 160 = -30\,kPa$

$\Delta t = 30 - \cancel{8}0 = 30\,kPa$

$\dfrac{\Delta t}{\Delta s} = -1$

$\Delta s' = 0 \quad (\Delta s' = K \underbrace{\delta \varepsilon_v}_{\text{0, undrained conditions}} = 0)$

$\Delta\sigma'_v + \Delta\sigma'_h = 0$

$\dfrac{\Delta\sigma'_v}{\Delta\sigma'_h} = -1$

$s' = 90\,kPa \qquad u = s - s' = 130 - 90 = 40\,kPa$

continued →

3 (e) excavation is a process that causes a reduction of total mean stress $\Delta s < 0$ typically associated with the development of negative excess pore water pressure $\Delta u < 0$.

As the excess pore water pressure dissipate the mean effective stress s' reduces causing a reduction of the sole effective strength - The safety factor reduces with time

4 (a)

layer ①

$$\gamma_d = \frac{G_s}{1+e} \gamma_w = \frac{2.65}{1.5} \times 10 = 17.7 \text{ kN/m}^3$$

$$\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w = \frac{2.65 + 0.5}{1.5} \cdot 10 = 21.0 \text{ kN/m}^3$$

base

$$\gamma_d = \frac{2.65}{1.65} \times 10 = 16.1 \text{ kN/m}^3$$

$$\gamma_s = \frac{2.65 + 0.65}{1.65} \times 10 = 20.0 \text{ kN/m}^3$$

4(b) $\sigma = \gamma z \cos\beta = 17.7 \times 3 \times \cos^2 27° = 42.2$ kPa

$\tau = \gamma z \sin\beta\cos\beta = 17.7 \times 3 \times \cos 27 \times \sin 27 = 21.5$ kPa

$$I_D = \frac{0{,}85 - 0{,}65}{0{,}85 - 0{,}40} = 0{,}44$$

$$I_C = \ln \frac{20'000}{42.2} = 6.16$$

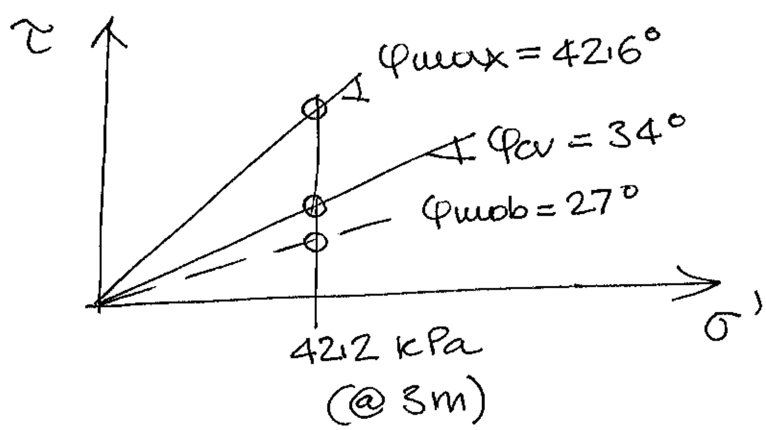
$$I_R = I_D I_C - 1 = 0{,}44 \times 6.16 - 1 = 1{,}71$$

$$\Delta\varphi = \varphi_{max} - \varphi_{cv} = 5 \times 1.71 = 8.6$$

assuming plane strain conditions

$\varphi_{cv} = 34°$

$\varphi_{max} = 34 + 8.6 = 42.6°$



The slope is stable

4 (c)

$$\gamma z = 17.7 \times 0.5 + 21 \times 2.5 = 61.35 \text{ kPa}$$

$$\sigma = \gamma z \cos^2\beta = 61.35 \times \cos^2 27° = 48.7 \text{ kPa}$$

$$\tau = \gamma z \sin\beta\cos\beta = 24.8 \text{ kPa}$$

$$u = \gamma_w z_w \cos^2\beta = 2.5 \times 10 \times \cos^2 27 = 19.8 \text{ kPa}$$

$$\sigma' = \sigma - u = 48.7 - 19.8 = 28.9 \text{ kPa}$$

$I_D = 0.44$ (as before)

$$\tan\varphi_{mob} = \frac{\tau}{\sigma} = \frac{24.8}{28.9} = 0.858$$

$$\varphi_{mob} = 40.6°$$

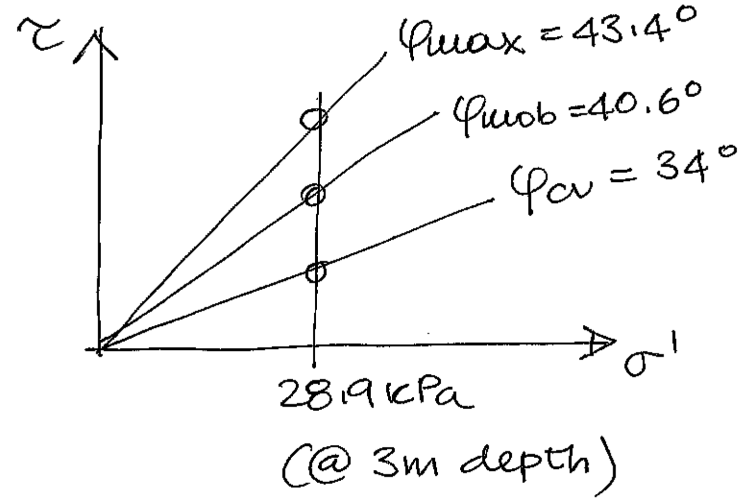
$$I_C = \ln \frac{20'000}{28.9} = 6.54$$

$$I_R = I_C I_D - 1 = 0.44 \times 6.54 - 1 = 1.88$$

$$\Delta\varphi = \varphi_{max} - \varphi_{cv} = 5 \times 1.88 = 9.4$$

$$\varphi_{cv} = 34°$$

$$\varphi_{max} = 34 + 9.4 = 43.4°$$



This time $\varphi_{cv} < \varphi_{mob} < \varphi_{max}$

so the stress state is below the peak failure conditions. However, the stress state is above the critical state line.

cont.

If, for any reason, the slope moves and the soil is sheared the soil may soften towards critical state - soil is in a metastable state which may lead to slope failure.

4 (d) The clay is likely to be sheared in undrained conditions

The mobilised shear stress should be checked against undrained shear strength. Potential failure surfaces different from contact between sand and clay may have to be considered (?). ~~[illegible]~~ Unlikely to be stable in the long term with seepage.

4(d)

~~The Mohr~~ We need to work out the peak friction angle of the sand and the mobilized friction angle as a function of z_w

$$\sigma = [\gamma_d(z - z_w) + \gamma_{sat} z_w]\cos^2\beta$$

$$u = \gamma_w z_w \cos^2\beta$$

$$\sigma' = [\gamma_d z - \gamma_d z_w + \gamma_{sat} z_w - \gamma_w z_w]\cos^2\beta$$

$$\sigma' = [\gamma_d \cdot z + (\gamma_{sat} - \gamma_d - \gamma_w) z_w]\cos^2\beta$$

$$\tau = [\gamma_d(z - z_w) + \gamma_{sat} z_w]\sin\beta\cos\beta$$

$$tg\,\varphi_{mob} = \frac{\tau}{\sigma'} = \frac{\gamma_d z + (\gamma_{sat} - \gamma_d) z_w}{\gamma_d z + (\gamma_{sat} - \gamma_d - \gamma_w) z_w} \cdot tg\,\beta =$$

$$= \frac{17.7 \times 3 + (21 - 17.7) z_w}{17.7 \times 3 + (21 - 17.7 - 10) z_w} \cdot tg\,27 =$$

$$= 0.51 \cdot \frac{53.1 + 3.3 z_w}{53.1 - 6.7 z_w}$$

$$\varphi_{mob} = tg^{-1}\left[0.51 \times \frac{53.1 + 3.3 z_w}{53.1 - 6.7 z_w}\right]$$

$$\varphi_{peak} = \varphi_{cs} + \Delta\varphi = 34^\circ + 5 I_R = 34^\circ + 5\left[I_D I_C - 1\right] =$$

$$34^\circ + 5\left[0.44 \cdot \ln \frac{20'000}{[17.7 \times 3 + (21 - 17.7 - 10) z_w]\cos^2 27} - 1\right] =$$

$$= 34^\circ + 5\left[0.44 \cdot \ln \frac{20'000}{42.16 - 5.32 z_w} - 1\right]$$

z_w	φ_{mob}	φ_{peak}
3.0	44.2	43.60
2.9	43.5	43.56

for $z_w = 3.0$ (fully saturated slope) the mobilised friction angle is larger than the peak friction angle

$$\varphi_{mob} > \varphi_{peak}$$

for $z_w = 2.9$ $\quad \varphi_{mob} \approx \varphi_{peak}$

and the slope will fail

for $z_w < 2.9$ $\quad \varphi_{mob} < \varphi_{peak}$ and the slope is metastable.

4(e) The clay is likely to be sheared (and fail) in undrained conditions. The mobilised shear stress should be checked against undrained shear strength. If undrained shear strength is a function of depth explore critical depth. Unlikely to be stable in the long term at slope > φ_{cs}.