(a) At point A:

$$\sigma_v = (2\times18) + (1\times21) + (2\times18) = 93 \text{ kPa}$$
$$u_o = 3\times10 = 30 \text{ kPa} \quad (\text{assuming } \gamma_w = 10 \text{ kN/m}^3)$$

$$\therefore \sigma_v' = 93 - 30 = 63 \text{ kPa}$$
$$K_o = 1.0 \quad \therefore \sigma_h' = K_o\sigma_v' = 63 \text{ kPa}$$
$$\sigma_h = \sigma_h' + u_o = 63 + 30 = 93 \text{ kPa}$$

$$q = \sigma_a - \sigma_r = \sigma_v - \sigma_h$$
$$p = \tfrac{1}{3}(\sigma_a + 2\sigma_r) = \tfrac{1}{3}(\sigma_v + 2\sigma_h)$$
$$p' = \tfrac{1}{3}(\sigma_a' + 2\sigma_r') = \tfrac{1}{3}(\sigma_v' + 2\sigma_h')$$

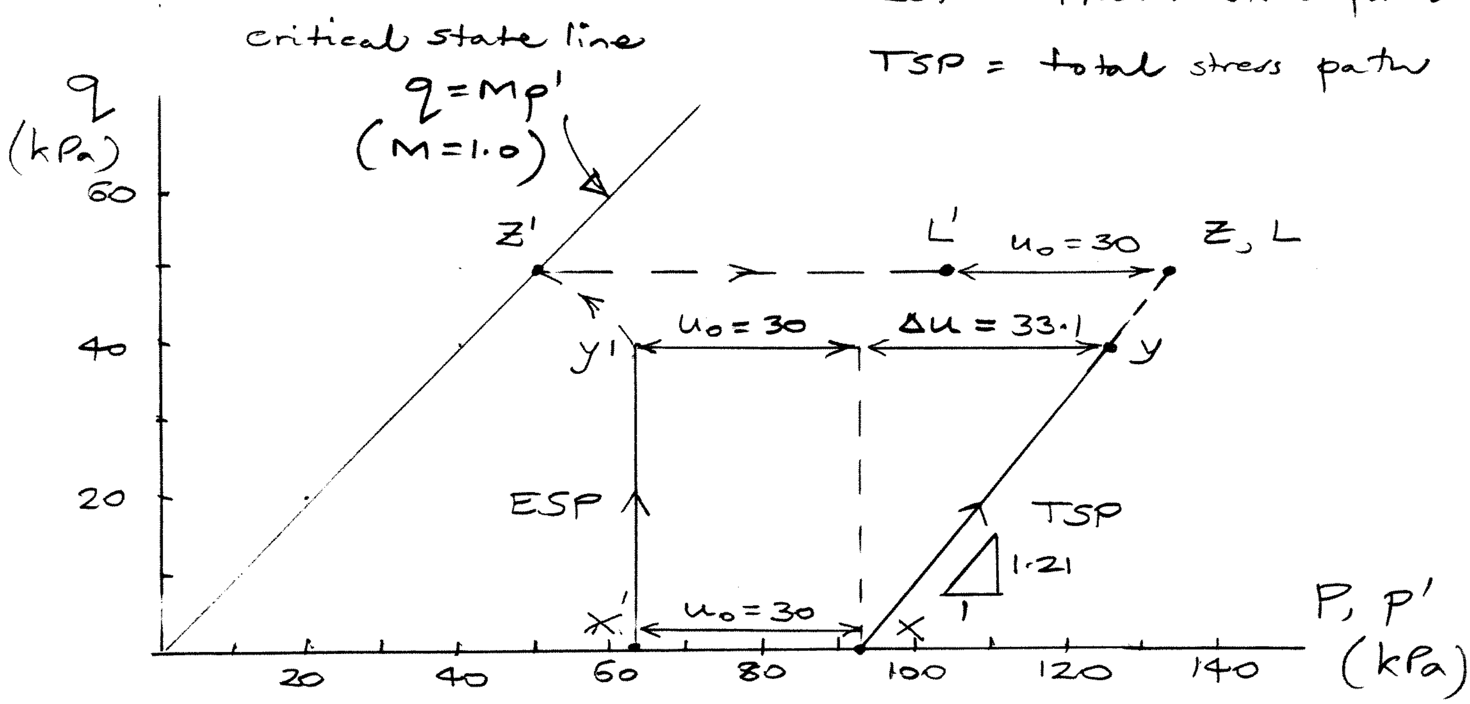
$$\Delta\sigma_v = 0.7\sigma_s$$
$$\Delta\sigma_h = 0.23\sigma_s$$

$$\Delta q = \Delta\sigma_v - \Delta\sigma_h = \sigma_s(0.7 - 0.23) = 0.47\sigma_s$$
$$\Delta p = \tfrac{1}{3}(\Delta\sigma_v + 2\Delta_h) = \sigma_s\cdot\tfrac{1}{3}(0.7 + 2\times0.23) = 0.39\sigma_s$$

$$\therefore \frac{\Delta q}{\Delta p} = \frac{0.47\sigma_s}{0.39\sigma_s} = 1.21$$

ESP = effective stress path

TSP = total stress path



Before filling tank (X, X' on p, p' axis):

$q = \sigma_v - \sigma_h = 93 - 93 = 0$

$p = \frac{1}{3}(\sigma_v + 2\sigma_h) = \frac{1}{3}(93 + 2\times 93) = 93$ kPa

$p' = p - u_0 = 93 - 30 = 63$ kPa

Clay is overconsolidated, hence effective stress path (ESP) is vertical at beginning of load test. The total stress path (TSP) has a slope of 1.21, as shown on plot. [7]

(b) At $H_1 = 8.5$m, $\sigma_s = 8.5 \times 10 = 85$ kPa

$\Delta q = 0.47\,\sigma_s = 0.47 \times 85 = 40.0$ kPa

$\frac{\Delta q}{\Delta p} = 1.21 \Rightarrow \Delta p = \frac{40}{1.21} = 33.1$ kPa

Excess pore pressure $\Delta u = \Delta p = 33.1$ kPa

$\therefore$ Pore pressure measured by piezometer at A

$= u_0 + \Delta u = 30 + 33.1 = \underline{\underline{63.1 \text{ kPa}}}$ [3]

(points Y, Y' on P, p' axis)

(c) Tank filled to height H_2 when failure of clay at point A occurs.

Undrained strength, $C_u = 25$ kPa

$\therefore$ At failure $q_f = 2\,C_u = 50$ kPa

$\therefore$ from beginning of test $\Delta q = 50$ kPa

$\Delta q = 0.47\,\sigma_s \Rightarrow \sigma_s = \frac{50}{0.47} = 106.4$ kPa

$\therefore H_2 = \frac{106.4}{10}$ m $= \underline{\underline{10.6 \text{ m}}}$

At failure, point on critical state line (Z')

$q_f = M p'_f \quad \therefore \quad p'_f = \frac{50}{1.0} = 50$ kPa

$\Delta q = 50$ kPa

TSP $\quad \dfrac{\Delta q}{\Delta p} = 1.21$

$\therefore \Delta p = \dfrac{50}{1.21} = 41.3$ kPa

$\therefore$ At point Z, $p_Z = 93 + 41.3 = 134.3$ kPa

$\therefore$ Pore pressure $= p_Z - p'_f = 134.3 - 50$

$= \underline{\underline{84.3 \text{ kPa}}}$

[6]

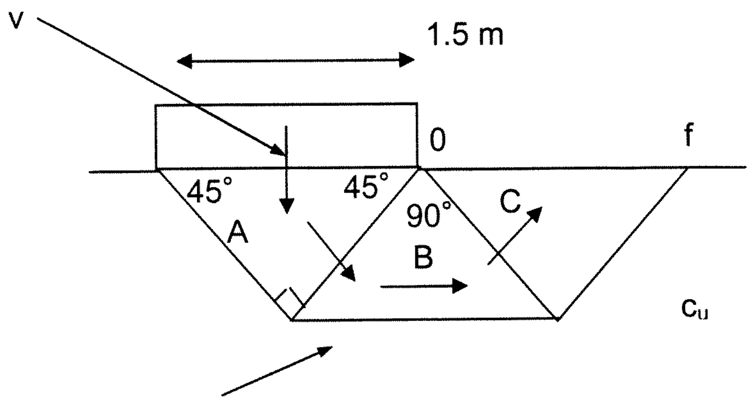
(d) If the water height is maintained at $H_2 = 10.6$m the excess pore pressure ($\Delta u = 84.3 - 30 = 54.3$kPa) will <u>dissipate</u> and the clay will <u>consolidate</u>.

Assuming the total stresses remain unchanged (a reasonable approximation), q must remain constant and therefore the effective stress path (ESP) moves horizontally to the right to long term point L'. The long term pore pressure returns to $u_o = 30$ kPa.

This means that the consolidation of the clay leads to a <u>larger</u> yield surface and a <u>stronger</u> clay. When the tank is filled with oil, $\sigma_s = 0.8 \times 10 \times 15 = 120$ kPa (which is equivalent to a height of water of 12m) it is likely that the clay at point A will not reach failure. (even though it did at $H = 10.6$m The water load test has strengthened the clay.

[4]

2



(a) Start with a strip pad

External work = Internal work

F x v = c_u x (1.5sin(45°) x √2v + 1.5 x 2v + 1.5sin(45°) x √2v + 1.5sin(45°) x √2v + 1.5sin(45°) x √2v) = cu x (9v) = 9 cu v

Hence, **F = 9 cu = 9 x 50 = 450 kN/m.**

(b) Use a stress fan as shown below.

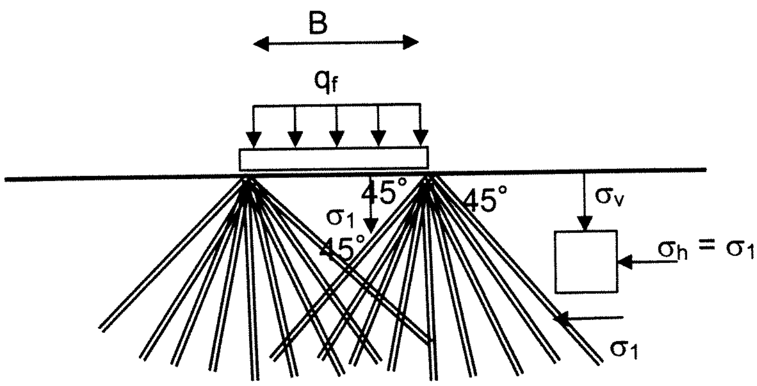
Region I $\sigma_v = \gamma z$, $\sigma_h = \gamma z + 2c_u = \sigma_1(I)$

The fan angle $\theta_f = \pi/2$

Region III $\sigma_1(III) = \sigma_1(I) + 2c_u(\pi/2) = \gamma z + (2+\pi)c_u$

Hence, $\mathbf{q_f = (2 + \pi)c_u}$

The ultimate force is **5.14 x 50 x 1.5 = 385.5 kN/m.**



(c) From the data book, Terzaghi's bearing capacity equation is

$$q_f = cN_c\zeta_c + 0.5\gamma' BN_\gamma\zeta_\gamma + q_s N_q \zeta_q$$

For c = 0 and $\phi = 30°$, $N_\gamma = 20$, $N_q = 21$, $\zeta_\gamma = 1\text{-}0.4(1.5/1.5) = 0.6$ $q_s = \gamma D$, $\zeta_{qs} = 1 + (1.5/1.5)\tan 30 = 1.58$ $\zeta_{qd} = 1+2\tan 30(1-\sin 30)^2 D/1.5 = 1 + 0.193D$ and $qf = P/1.5^2$

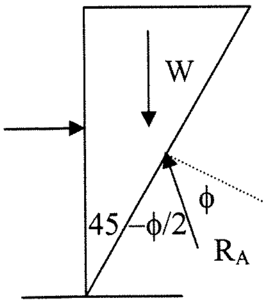
$1000/1.5^2 = 0.5 \text{ x } 16 \text{ x } 1.5 \text{ x } 20 \text{ x } 0.6 + 16 \text{ x } D \text{ x } 21 \text{ x } 1.58 \text{ x } (1+0.193D)$
$444 = 144 + 531(D+0.193D^2)$
$0.193D^2 + D - 0.56 = 0$
$D = 0.51$ m

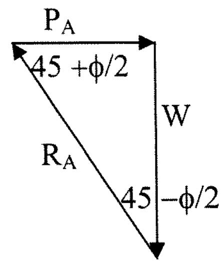
(d) The saturated unit weight γ_{sat} is 20 kN/m^3. Hence, the effective unit weigh γ' becomes 10 kN/m^3.

$1000/1.5^2 = 0.5 \text{ x } 10 \text{ x } 1.5 \text{ x } 20 \text{ x } 0.6 + (16 \text{ x } 0.5 + 10 \text{ x } (D - 0.5)) \text{ x } 21 \text{ x } 1.58 \text{ x } (1+0.193D) + 10 \text{ x } (D-0.5)$
$444 = 90 + (3+10D)33.18(1+0.193D)+10D-5$
$64D^2 + 361D-259=0$
$D = 0.64$ m

3

(a)

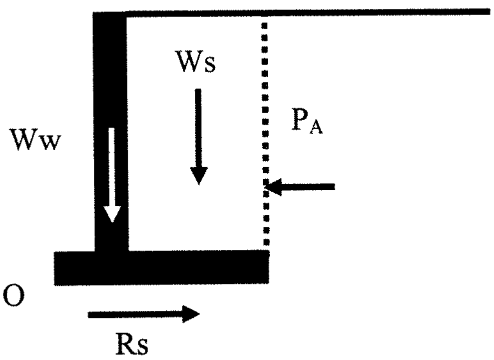




$W = (\gamma H^2/2)\tan(45 - \phi/2)$

$P_A = (\gamma H^2/2)\tan^2(45 - \phi/2) = (18 \times 7^2/2) \times \tan^2(45 - 35/2) = 120$ kN/m

(b) Rs = width of the wall x c_u = 3.5 m x 50 kPa = 175 kN/m. F.S. = 175/120 = 1.46.



(c) W_{wv} = weight of the vertical section of the wall = 24 x 0.5 x 6.5 = 78 kN/m acting at 1.05 m from O.

W_{wh} = weight of the base of the wall = 24 x 0.5 x 3.5 = 42 kN/m acting at 1.75 m from O.

Ws = weight of the soil = 18 x 2.2 x 6.5 = 257.4 kN/m acting at 2.4 m from O.

Find the eccentricity

(78 x 1.05 + 42 x 1.75 + 257.4 x 2.4) = (78 + 42 + 257.4)x C

The combined load of 377.4 kN/m is acting at C = 2.05 m.

The eccentricity is 2.05 – 1.75 = 0.30. Hence, the base length is modified as 3.5 – 2x0.3 = 2.9 m.
The horizontal load is 120 kN/m. Hence, the load inclination is 17.6 degrees to the vertical. The correction factor for inclination becomes $(1\text{-}17.6/90)^2 = 0.65$.

Hence, the bearing capacity qf = 5.14 x 50 x 0.65 = 167.1 kN/m^2 acting on 2.9 m modified base length. The factor of safety for bearing capacity failure becomes 167.1x2.9/377.4 = <u>1.28</u>

(d) When the water table is at 4 m from the base, the active force now becomes
$P_A = (1/2) \times \tan^2(45\text{-}35/2) \times 18 \times (7\text{-}4)^2 + (1/2) \times 10 \times 4^2 + 3 \times \tan^2(45\text{-}35/2) \times 18 \times 4 + (1/2) \times \tan^2 (45\text{-}35/2) \times 11 \times 4^2 = 21.9 + 80 + 58.3 + 23.8 = 184$ kN/m < 3.5 x 50 = 175 kN/m. The wall may slide (F.S. = 175/184 = 0.95)

The weight of the wall
W_{wv} = weight of the vertical section of the wall = 24 x 0.5 x 6.5 = 78 kN/m acting at 1.05 m from O.
W_{wh} = weight of the base of the wall = 24 x 0.5 x 3.5 = 42 kN/m acting at 1.75 m from O.
Ws = weight of the soil = 21 x 2.2 x 3.5 + 18 x 2.2 x 3 = 281 kN/m acting at 2.4 m from O.

Find the eccentricity
(78 x 1.05 + 42 x 1.75 + 281 x 2.4) = (78 + 42 + 281)x C
The combined load of 401 kN/m is acting at C = 2.07 m.

The eccentricity is 2.07– 1.75 = 0.32. Hence, the base length is modified as 3.5 – 2x0.32 = 2.86 m.
The horizontal load is 184 kN/m. Hence, the load inclination is 24.6 degrees to the vertical. The correction factor for inclination becomes $(1\text{-}24.6/90)^2 = 0.52$.

Hence, the bearing capacity qf = 5.14 x 50 x 0.52 = 134 kN/m^2 acting on 2.86 m modified base length. The factor of safety for bearing capacity failure becomes 134x2.86/401 = <u>0.96.</u> The clay foundation may fail.

Put drainage holes and improve the clay foundations.

Question 4

(a) Lift off occurs somewhere between a cavity pressure, σ_c, of 50 and 75 kN/m² (without detailed plot of pressure - displacement it is not possible to be precise).

Estimate lift-off at $\sigma_c = 60$ kN/m² (say).

$\therefore \sigma_{ho} = 60$ kPa (ignoring any corrections for membrane)

At 3m depth, pore pressure $u_o = 2.5 \times 10 = 25$ kPa

$$\therefore \sigma_{ho}' = 60 - 25 = 35 \text{ kPa}$$

$$\sigma_{vo} = 3 \times 16 = 48 \text{ kPa}$$

$$\therefore \sigma_{vo}' = 48 - 25 = 23 \text{ kPa}$$

$$\therefore K_o = \frac{\sigma_{ho}'}{\sigma_{vo}'} = \frac{35}{23} = \underline{\underline{1.52}}$$

[4]

(b) $$K_o = K_{onc}(OCR)^{0.9}$$

$\phi' = 29°$

$K_{onc} = 1 - \sin\phi' = 0.52$

Taking $K_o = 1.52$ from above

$$OCR^{0.9} = \frac{1.52}{0.52} = 2.92$$

$\therefore OCR = 3.3$

Max. vertical effective stress $= \sigma_{vc}'$

$$OCR = \frac{\sigma_{vc}'}{\sigma_{vo}'} \Rightarrow \sigma_{vc}' = 3.3 \times 23 = \underline{\underline{76 \text{ kPa}}}$$

[4]

(c)

Volume of pressuremeter cavity

$V = A.h$

where A = cross-sectional area = πr^2

h = length of pressuremeter (fixed)

$\therefore \ln V = \ln A + \ln h$

$$\frac{\Delta V}{V} = \frac{\Delta A}{A} + \frac{\Delta h}{h} = \frac{\Delta A}{A} \quad \left(\text{since } \frac{\Delta h}{h} = 0\right)$$

$A = \pi r^2$

$$\therefore \frac{\Delta A}{A} = 2\frac{\Delta r}{r}$$

Δr = change of radius

r = current radius of cavity

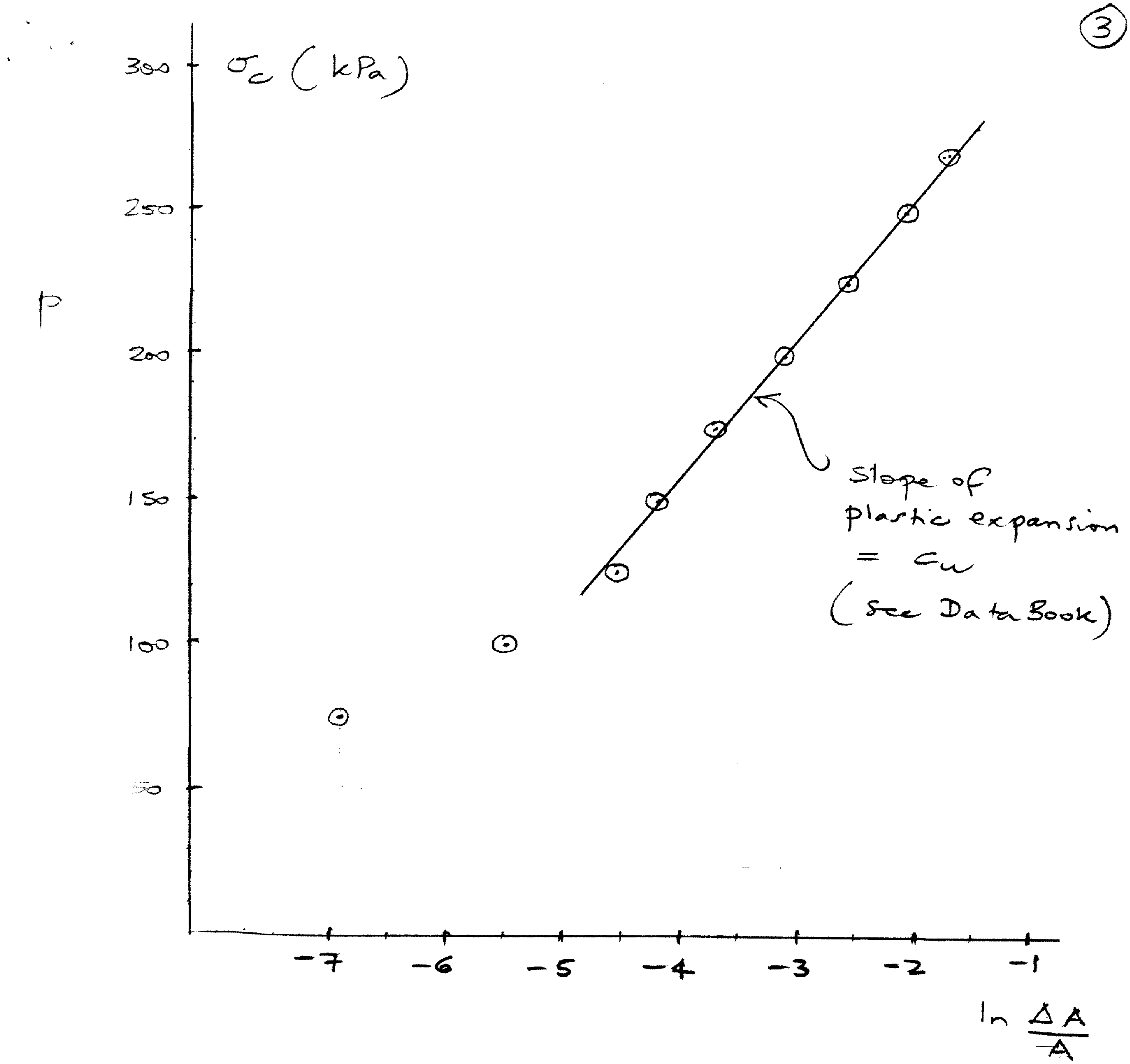
Initial radius of cavity $r_0 = 41$ mm $(r = r_0 + \Delta r)$

σ_c (kPa)	Δr (mm)	$\frac{\Delta A}{A} = 2\frac{\Delta r}{r}$	$\ln \frac{\Delta A}{A}$
0	0	0	
25	0	0	
50	0	0	
75	0.02	0.0009	−6.9
100	0.08	0.0039	−5.5
125	0.21	0.010	−4.6
150	0.31	0.0150	−4.2
175	0.57	0.025	−3.7
200	0.90	0.043	−3.1
225	1.64	0.077	−2.6
250	2.67	0.122	−2.1
270	4.10	0.182	−1.7

From Data Book $\delta\sigma_c = c_u\left[1 + \ln\frac{G}{c_u} + \ln\frac{\delta A}{A}\right]$

$\therefore$ plot σ_c against $\ln\frac{\delta A}{A} \Rightarrow$ slope c_u

(a)



$$c_u = \frac{280 - 120}{-1.4 - (-4.8)} = \frac{160}{3.4} = \underline{\underline{47 \text{ kPa}}}$$

[8]