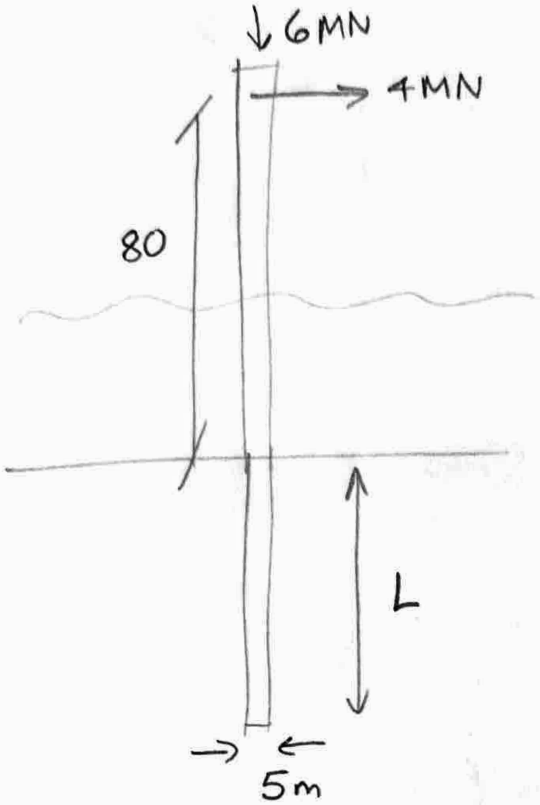
1.



a) High horizontal load. Shallow foundations are poor will high H & M when V is low.

b) Low strength at surface. Piles use strength at depth, shallow foundations reliant on surface

c) Scour. flow past foundation erodes soil and undermines shallow foundations..

b) $D = 5$ $\quad e = 80$ $\quad \frac{e}{D} = 16$ $\quad n = 9 \times 3 = 27$ kPa/m

$\frac{H}{nD^3} = 1.18 \implies$ Short ~~Long~~ mech $\frac{L}{D} = 6.5$

$L \sim 32.5$ m

Long mech $\frac{M_p}{nD^4} = 20$

$M_p = 337,500$ kNm $= D^2 t \sigma_y$

$t = \frac{337,500}{25 \times 400,000} = \underline{\underline{33.75 \text{ mm}}}$

c) $\alpha = 0.5 \max\left[\left(\frac{\sigma_v'}{s_u}\right)^{1/2}, \left(\frac{\sigma_v'}{s_u}\right)^{1/4}\right]$

$\sigma_v' = 6z \qquad s_u = 3z$

$$\frac{\sigma_v'}{s_u} = 2$$

$$\alpha = 0.5\sqrt{2} = 0.707$$

$$V = \cancel{2}(\pi D \cancel{L}) \times 3\frac{L}{\cancel{2}}\,\alpha$$

$$= \pi \times 5 \times 0.707 \times 32.5^2 \times 3$$

$$= \underline{35.2\ \text{MN}}$$

d) Cyclic loading may reduce capacity.
Stiffness must be adequate to prevent resonance.

2 a) $\tau_s R = \tau r$ | @ radius r

$$\gamma = -\frac{\partial w}{\partial r}$$

$$\tau = G\gamma$$

$$\tau_s R = -G r \frac{dw}{dr}$$

$$\int_R^{r_m} \frac{dr}{r} = -\int_{w}^{0} \frac{G}{\tau_s R}\, dw$$

$$\ln \frac{r_m}{R} = \frac{G w}{\tau_s R}$$

$$w = \frac{\tau_s R}{G} \zeta$$

$$\ln \frac{r_m}{r} = \frac{G w_r}{\tau_s R}$$

$$w_r = \frac{\tau_s R}{G} \ln\left(\frac{r_m}{r}\right)$$

$$= \frac{w}{\zeta} \ln\left(\frac{r_m}{r}\right)$$

$$= \frac{w}{\zeta} \ln\left(\frac{r_m}{R}\frac{R}{r}\right)$$

$$= \frac{w}{\zeta}\left[\zeta - \ln \frac{r}{R}\right]$$

$$= w\left[1 - \frac{1}{\zeta} \ln \frac{r}{R}\right]$$

$$F = \tau_s 2\pi R L$$

$$\frac{F}{w} = \frac{2\pi R L \tau_s}{\tau_s R/_G \ln\left(\frac{r_m}{R}\right)} = \frac{2\pi G L}{\zeta}$$

b) i) $$\frac{F}{w} = \frac{2\pi \times 10\text{ MPa} \times 20}{4} = 100\pi$$

$= 314$ MN/m $= S$

ii) Each pile carries load $\frac{F}{4}$ Assume $\zeta = 4$

Settlement at pile is w

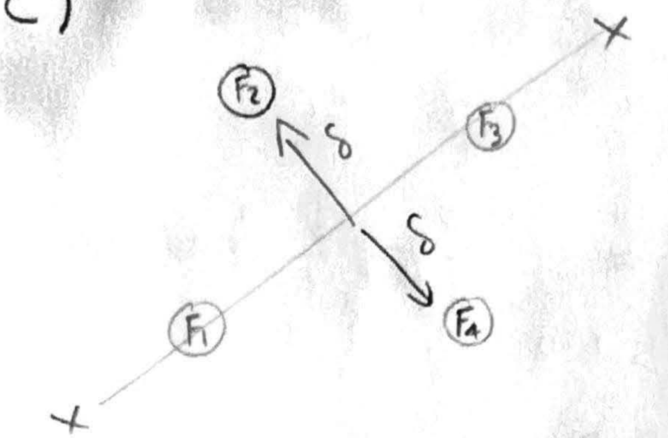
@ $\frac{r}{R} = 16$ Settlement at adjecent pile is $0.31\,w$

$\frac{r}{R} = 16\sqrt{2}$ Settlement at diagonal pile is $0.22\,w$

$1 + 2 \times 0.31 + 0.22 = 1.84$

$$\frac{F_T}{w} = \frac{4}{1.84} \times 314 = 682.6 \text{ MN/m}$$

c)



Taking moments about X-X

$$F_2 \cancel{\delta} = F_4 \cancel{\delta}$$

about other diagonal

$$F_1 = F_3$$

ii) Settlement at 10MN = $\underline{18.7\,mm}$

$\omega_G = 2mm$ $4mm$

(1) F_1 (2) F_2

$F_3 - F_2$ (3) $4mm$ $F_4 = F_1$ (4) $\omega_G = 10\,mm$

$$\omega_1 = \frac{F_1}{s} + 2\frac{F_2}{s} \times 0.31 + \frac{F_1}{s} \times 0.22 + 2\,mm$$

$$\omega_2 = \omega_3 = \frac{F_2}{s} + 2\frac{F_1}{s} \times 0.31 + \frac{F_2}{s} \times 0.22 + 4mm$$

$$\omega_4 = \frac{F_1}{s} + 2\frac{F_2}{s} \times 0.31 + \frac{F_1}{s} \times 0.22 + 10mm$$

rotation about X-X axis $= \dfrac{8mm}{8\sqrt{2}\,m} = 7 \times 10^{-4}$

$= \underline{\underline{0.04^\circ}}$

rotation about Y-Y axis = 0 by symmetry.

As pile cap is rigid $\omega_2 = \dfrac{\omega_1 + \omega_4}{2}$

$$\cancel{4}\,\text{mm} + 1.22\ \cancel{\text{up}}\ \frac{F_2}{S} + 0.62\frac{F_1}{S} = 1.22\frac{F_1}{S} + 0.62\frac{F_2}{S} + \overset{2}{6}\,\text{mm}$$

$$0.6\left(\frac{F_2}{S} - \frac{F_1}{S}\right) = 2\,\text{mm}$$

$$\frac{F_2 - F_1}{S} = 3.33\ [\text{mm}]$$

$$F_2 = 5\ \text{MN} - F_1$$

$$\frac{\overset{[\text{MN}]}{5} - 2F_1}{314^{8}\ [\text{MN/m}]} = 3.33\ [\text{mm}]$$

$$5 - 2F_1 = 1046\ [\text{kN}]$$

$$F_1 = 1977\ \text{kN}$$

$$F_2 = 3023\ \text{kN}$$

$$\omega_{avg} = 1.22\frac{F_2}{S} + 0.62\frac{F_1}{S} + 4\,\text{mm}$$

$$= \frac{1.22 \times 3023}{314} + \frac{0.62 \times 11977}{314} + 4$$

$$= \underline{\underline{19.6\,\text{mm}}}$$

5mm ove initial

Question Retaining wall

(a) Embedded retaining walls are flexible structures constructed from ground level with their toe embedded below the final excavation level to mobilise passive resistance. The main types include soldier piles and lagging walls (Berlin wall), steel sheet pile walls, and embedded reinforced concrete walls such as contiguous or secant pile walls, and diaphragm walls.

Soldier pile and lagging walls consist of driven or bored steel H-piles with timber, steel, or pre-cast concrete lagging installed as excavation proceeds. They are quick and economical to construct but are not watertight, making them unsuitable when groundwater control is required.

Steel sheet pile walls are formed from interlocking steel sections driven or pressed into the ground to create a continuous wall. They can be used as temporary or permanent works and may be made watertight by sealing or welding the interlocks although the joints remain the weakest point for water ingress.

Embedded reinforced-concrete walls can be constructed using bored piles. Contiguous and tangent pile walls have gaps between piles and are not watertight, whereas a secant pile wall uses overlapping piles to form a groundwater barrier.

Diaphragm walls are constructed by excavating panels under support fluid and casting reinforced concrete in-situ; with water bars at panel joints they provide a watertight retaining system suitable for deep excavations below the water table.

(b)

(i) $\gamma_{dry} = \gamma_w \frac{Gs}{(1+e)}$ $\quad \gamma_{sat} = \gamma_w \frac{(Gs+e)}{(1+e)}$

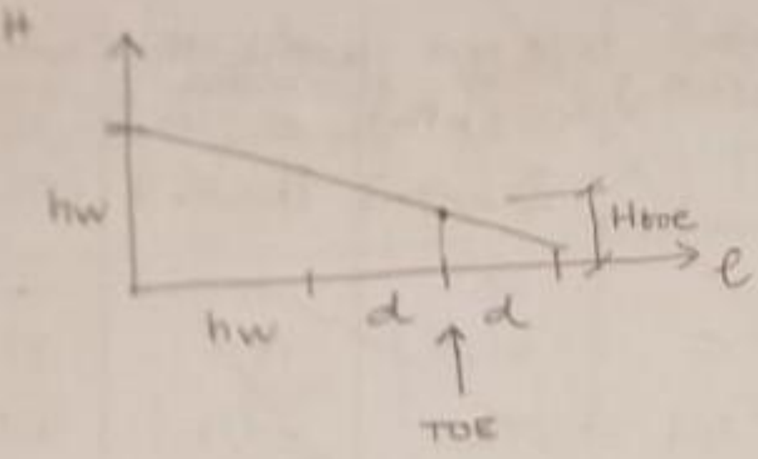
upper sand $\quad \gamma_{dry} = 9.81 \times \frac{2.65}{1.9} = 13.7 \text{ kN/m}^3$

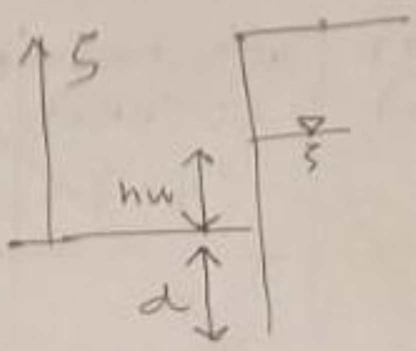
$\gamma_{sat} = 9.81 \times \frac{2.65+0.9}{1.9} = 18.3 \text{ kN/m}^3$

lower sand $\quad \gamma_{dry} = 9.81 \times \frac{2.7}{1.7} = 16.6 \text{ kN/m}^3$

$\gamma_{sat} = 9.81 \times \frac{2.7+0.7}{1.7} = 20.2 \text{ kN/m}^3$

(iii) assuming a constant gradient for the hydraulic head loss:





$$\frac{H_{toe}}{d} = \frac{h_w}{h_w + 2d}$$

$$H_{toe} = \frac{d\,h_w}{h_w + 2d}$$

$$\frac{u_{toe}}{\gamma_w} + \zeta_{toe} = \frac{d\,h_w}{h_w + 2d} \qquad \zeta_{toe} = -d$$

$$u_{toe} = \gamma_w \left[\frac{h_w d}{h_w + 2d} + d\right] = \gamma_w \frac{h_w d + h_w d + 2d^2}{h_w + 2d} =$$

$$= 2\gamma_w d \, \frac{h_w + d}{h_w + 2d}$$

it follows that the gradient of u on the active side is:

$$g_a = \frac{u_{toe}}{h_w + d} = \frac{2\gamma_w d}{h_w + 2d} = \frac{2 \times 9.81 \times 3{,}20}{2 + 6{,}40} = 7{,}474 \text{ kPa/m}$$

while the gradient of u on the passive side is:

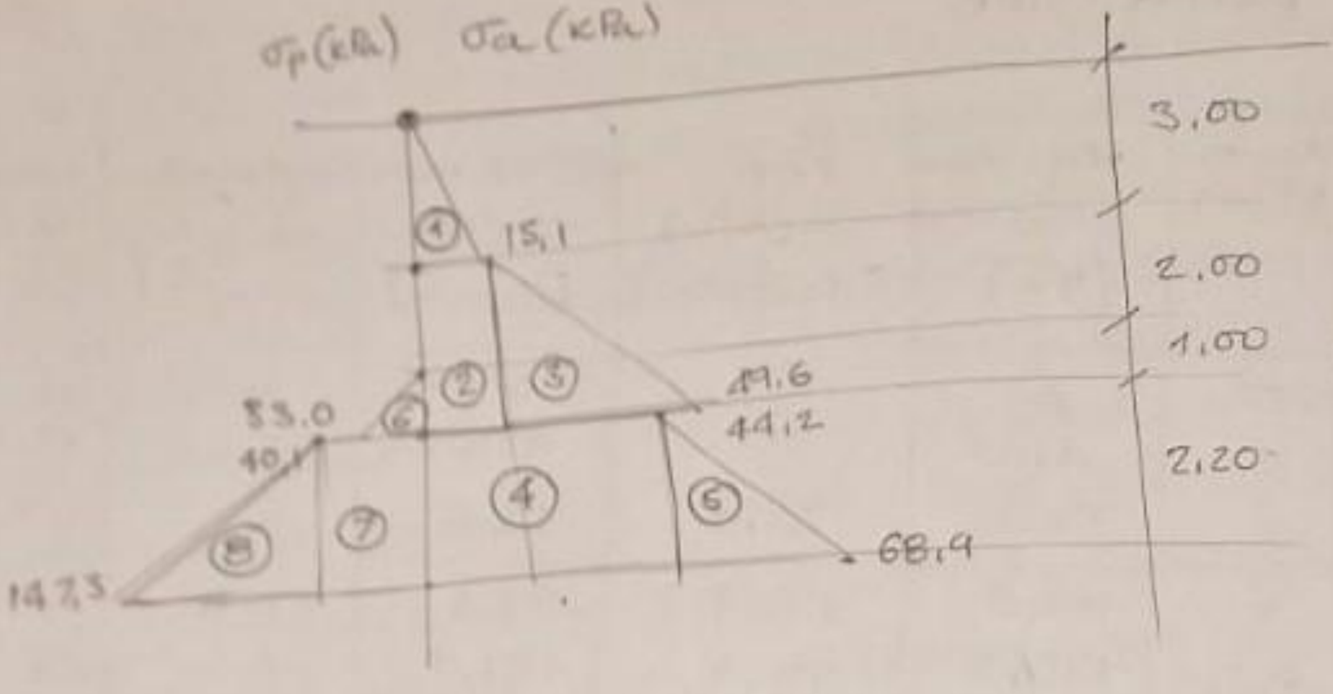
$$g_p = \frac{u_{toe}}{d} = 2\gamma_w \frac{h_w + d}{h_w + 2d} = 12{,}146 \text{ kPa/m}$$

ACTIVE SIDE

	depth z (m)	vert. stress σ_v (kPa) $[\gamma z]$	pwp u (kPa) $[g_a(z-z_w)]$	vert. eff. stress σ'_v (kPa) $[\sigma_v - u]$	active eff. stress σ'_a (kPa) $[K_a \sigma'_v]$	active stress σ_a (kPa) $[\sigma'_a + u]$
upper sand	0	0	0	0	0	0
	3	41.0	0	41.0	15.1	15.1
	6	96.0	22.4	73.6	27.2	49.6
lower sand	6	96.0	22.4	73.6	21.8	44.2
	8.2	140.5	38.9	101.7	30.1	68.9

PASSIVE SIDE

			pwp h $[g_p(z-z_w)]$		σ'_p (kPa) $[K_p \sigma'_v]$	σ_p (kPa) $[\sigma'_p + u]$
upper sand	5	0	0	0		0
	6	18.3	12.1	6.2	20.8	33.0
lower sand	6	18.3	12.1	6.2	28.0	40.1
	8.2	62.8	38.9	24.0	108.5	147.3



DESIGN DRIVING MOMENT

	F	b	M
1	$\frac{15{,}1\times3}{2}$	2	$15{,}1\times3 = 45{,}4$
2	$15{,}1\times3$	4,5	$15{,}1\times3\times4{,}5 = 204{,}4$
3	$(49{,}6-15{,}1)\times3/2$	5	258,3
4	$44{,}2\times2{,}20$	$6+\frac{2{,}2}{2}$	690,1
5	$(68{,}9-44{,}2)\times\frac{2{,}20}{2}$	$6+2{,}2\times\frac{2}{3}$	203,2

$M_{Dd} = 1401.4$

DESIGN RESTORING MOMENT

6	$\frac{33\times1}{2}$	5,7	93,5
7	$40{,}1\times2{,}20$	$6+\frac{2{,}2}{2}$	515,4
8	$(147{,}3-40{,}1)\cdot\frac{2{,}20}{2}$	$6+2{,}2\times\frac{2}{3}$	880,6

$M_{Rd} = 1489{,}5$

$M_{Rd} > M_{Dd}$ ✓ OK

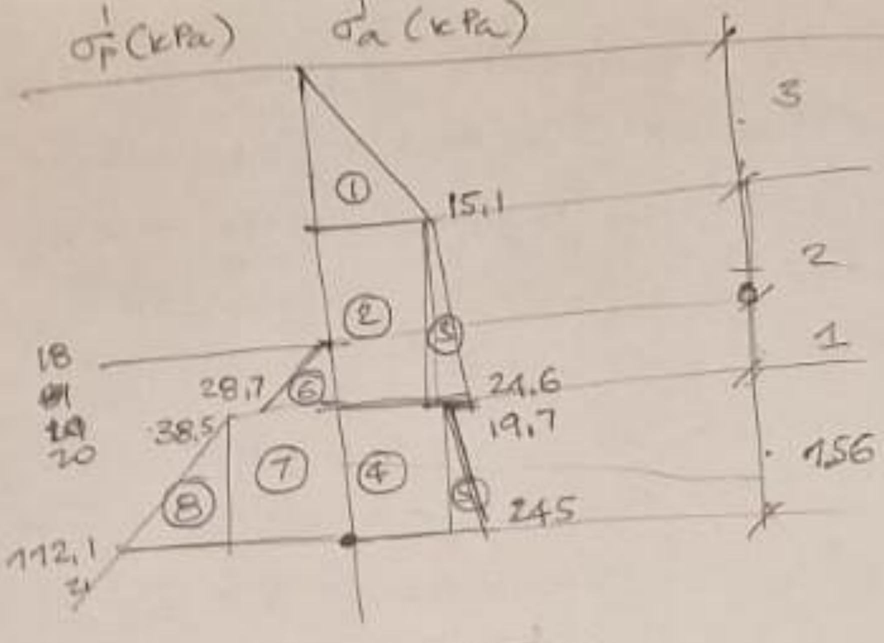
(iv)
if the excavation is carried out submerged, the pore water pressures on both the active and passive side are hydrostatic and hence cancel out. To check that the prop is not needed, we shall consider rotation (and equilibrium) about a pivot point at $d^* = 0.8d = 0.8 \times 3.2 = 2.6$ m. Moreover, because in this case the pwp on the active and passive side cancel out we shall only consider effective stress.

ACTIVE SIDE

	depth z (m)	vert. stress σ_v (kPa) $[\gamma z]$	pwp u (kPa) $[\gamma_w(z-z_w)]$	vert. eff. str. σ'_v (kPa) $[\sigma_v - u]$	active/passive eff. stress $\sigma'_{a,p}$ (kPa) $[\sigma'_{ap} = K_{a,p}\sigma'_v]$
upper sand	0	0	0	0	0
	3	41.0	0	41.0	15.1
	6	96.0	29.4	66.6	24.6
lower sand	6	96.0	29.4	66.6	19.7
	7.6	127.6	44.7	82.9	24.5

PASSIVE SIDE

upper sand	5	19.6	19.6	0	0
	6	37.9	29.4	8.5	28.7
lower sand	6	37.9	29.4	8.5	38.5
	7.6	69.5	44.7	24.8	112.1



DRIVING MOMENT

	F	b	M
1	22,7	5,6	127,2
2	45,4	3,1	140,8
3	14,1	2,6	36,8
4	30,7	0,8	24,0
5	3,75	0,52	1,9
			330,7

RESTORING MOMENT

6	14,4	2,1	29,6
7	44,8	0,8	35,0
8	57,4	0,5	29,8
			94,4

NO, The prop is still needed.

Question tunnel

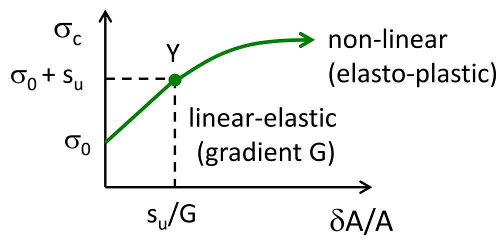
(a) In clay soils, tunnels are typically constructed as bored tunnels using either conventional mining methods or mechanised tunnelling. The choice depends on ground strength, groundwater conditions, tunnel size, and control of ground movements.

Conventional mining is commonly used in stiff clays. Excavation is carried out using road-headers or backhoes and requires the ground to be stable enough to stand temporarily. The tunnel may be excavated either full face, where the entire section is excavated at once, or by sequential excavation, where the section is divided into smaller parts excavated in a controlled sequence. Temporary support such as sprayed concrete and steel ribs is installed, followed later by a permanent lining, typically cast in situ concrete. This method is flexible and well suited to complex tunnel geometries and short tunnel lengths.

Mechanised tunnelling is widely used in soft clays, particularly in urban areas where control of ground movements is critical. Excavation is carried out using a tunnel boring machine (TBM) operating in full face. Closed-shield machines, such as Earth Pressure Balance (EPB) or slurry shield TBMs, provide continuous support to the tunnel face to maintain stability in clay. Precast segmental linings are installed immediately behind the shield, forming the permanent tunnel lining. Mechanised tunnelling allows rapid construction, good control of settlement, and is particularly suitable for long tunnels with uniform cross-section.

(b)

(i) Expansion of pressuremeter $\delta A = A - A_0$ (and $\delta V = V - AV_0$) caused by increase of pressure $\delta\sigma_c = \sigma_c - \sigma_0$



The *in-situ* horizontal stress σ_{h0} is taken to be equal to "lift-off" pressure of curve σ_0 obtained extrapolating backwards to zero cavity strain using initial few data:

$$k_0 = \frac{\sigma'_{h0}}{\sigma'_{v0}} = \frac{\sigma_0 - u_0}{\sigma'_{v0}}$$

The elastic modulus, G, is obtained from the slope of the first part of the plot or from unload reload loop later in the test (initial stiffness, G_i, unloading/reloading stiffness, G_{ur}, due to disturbance $G_i < G_{ur}$). Remembering that $\frac{\delta A}{A} = \frac{\delta V}{V} = \gamma = 2\varepsilon_c$:

$2G = d\sigma_c/d\varepsilon_c$.

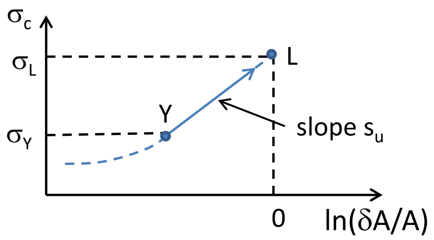
Yield can be identified from the graph of σ_c : $\delta A/A$ as the point at the onset of non-linearity, and the undrained shear strength estimated from $s_u = \sigma_{cY} - \sigma_0$.

The more reliable method however is to use a plot of $\sigma_c : \ln(\delta A/A)$.

From the databook:

$$\sigma_c = \sigma_0 + s_u\left[1 + \ln\frac{G}{s_u} + \ln\frac{\delta A}{A}\right]$$

so that the undrained shear strength can be determined from the slope of the linear part of the plot.



(ii) the stability ratio is defined as:

$$N = \frac{\sigma_s + \gamma z - \sigma_t}{s_u}$$

Where σ_s is the surcharge at surface, γ is the unit weight of the soil, z is the depth of the tunnel axis, and σ_t is the tunnel support pressure. In this case, $\sigma_s = \sigma_t = 0$, so:

$$N = \frac{20 \times 15}{100} = 3$$

From the chart in Fig. 1, for a cover $C/D = (z - R)/D = (15 - 3)/6 = 2$ and $P/D = \infty$, $N_C \approx 4$.

Because $N < N_C$, the tunnel does not collapse if unlined. The safety factor can be obtained as $SF = N_C/N = 4/3 = 1.33$

(iii) from data book:

$$\delta\sigma_c = \sigma_c - \sigma_0 = s_u\left[1 + \ln\frac{G}{s_u} + \ln\frac{\delta A}{A}\right]$$

hence

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

Contraction of cavity with complete unloading:

$\delta\sigma_c = \gamma z$

Radial ground movement ρ_c:

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

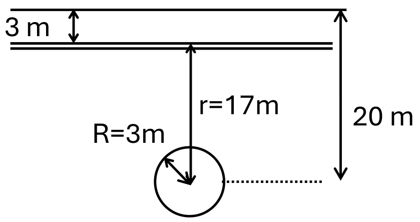
$A = \pi R^2$

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

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

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

$\rho_c = \frac{3}{2}\frac{100}{30 \times 10^3} \exp\left(\frac{20 \times 15}{100} - 1\right) = 0.037 \text{ m } (= 37 \text{ mm})$



Constant volume condition (undrained behaviour) implies that $2\pi r \rho = 2\pi R \rho_c$, hence:

$\rho = \frac{R}{r}\rho_c = \frac{3}{17} 37 = 6.53 \text{ mm}$