4DS 2025

1\. a) i) Displacement:

pre-formed pile, usually steel tube or solid concrete. Driven into ground with piling hammer or by static jacking

ii) non-displacement.

Hole bored in ground & spoil removed usually with CFA rig. Support fluid may be used to prevent collapse in sands etc. Rebar cage placed and concrete pumped in from base displacing drilling mud.

ii) Displacement

PRO - prestresses soil so gives strong stiff pile

CON - very noisy!

Bored

PRO - non-disruptive installation

CON - soil disturbed & loses strength leading to much softer performance at working loads.

b) Below pile tip high vertical + horizontal stresses exerted

As tip passes σ_V relaxes but σ_H very high

Cyclic hammering of pile leads to friction fatigue as soil contracts away from pile.

Consolidation relaxes σ_H

Increasing σ_{TOTAL} → pore pressure increase in all soils

Shear dense soils — −ve epp

loose soils +ve epp

Loose soils get high epp which relaxes dropping capacity

high $\frac{\sigma_v'}{s_u}$

Dense soils shear cancels stress increase

~~High~~ low $\frac{\sigma_v'}{s_u}$ low e.p.p.

c) $D = 0.5m$ $s_u = 40$ kPa $\gamma = 20$ $\gamma' = 10$ kN/m^3

$\sigma_v' = 10z$ (kPa)

transition @ 4m depth.

0 - 4m $\tau = 40 \times \frac{1}{2}\left(\frac{10z}{40}\right)^{1/4}$

$$F = \pi D \int_0^4 \frac{20}{\sqrt{2}} z^{1/4} = \frac{20\pi D^{1/2}}{\sqrt{2}}\left[\frac{4z^{5/4}}{5}\right]_0^4$$

$= 100.5$ kN

Below 4m

$$F = \cancel{20\pi D} \; 20\pi D \int_4^L 2\left(\frac{z}{4}\right)^{1/2} dz$$

$$= \frac{10\pi}{\cancel{2}} \times \frac{2}{3}\left[z^{3/2}\right]_4^L$$

$$= \frac{10\pi}{3}\left(L^{3/2} - 4^{3/2}\right) = \sim 2\cancel{2}43 \text{ kN}$$

$\cancel{400}$ 35 $- 7 - 100.5$

$$\boxed{L = \cancel{10}9.9\text{m}}$$

Base $\frac{\pi D^2}{4} \times 9c_u = \frac{\pi \times (1/2)^2}{4} \times 9 \times 40$

$= \underline{\underline{7.07 \text{ kN}}}$

d) $L = \cancel{10}15\text{m}$ $D = 0.5\text{m}$ $S_u = 40$ kPa

Only short pile failure $L/D = 30$

$e/D = 0$

$$\frac{H_{ult}}{S_u D^2} = 70$$

$H_{ult} \sim 70 \times 40 \times 0.25 = \underline{\underline{700 \text{ kN}}}$

2. a) Onshore SLS - brittle materials failure

strains much < soil.

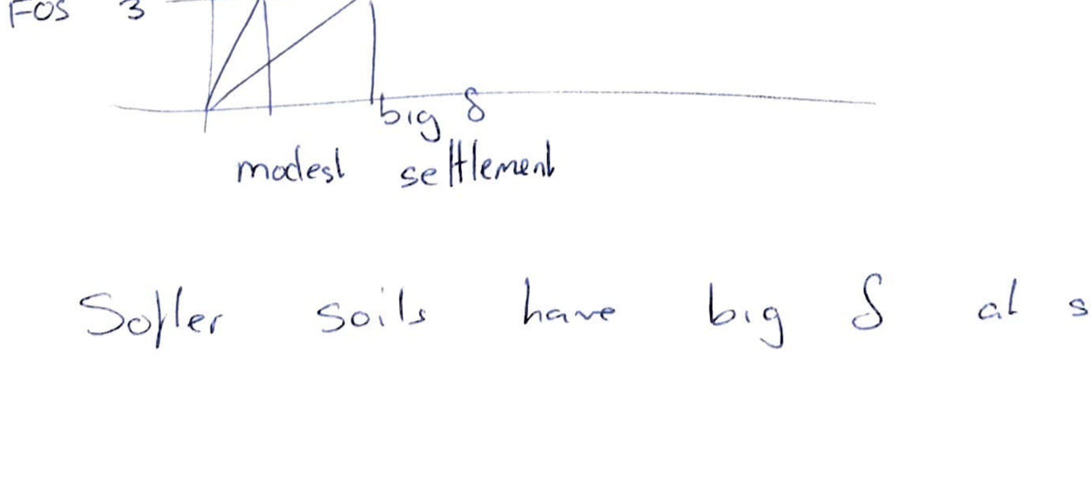
Curvature of buildings leads to structural strains + failure.

Offshore - compliant structures as very large.

foundation movements have limited effect on structure - ULS

b) FoS for material uncertainty
load uncertainty

& to limit deflections.



Softer soils have big δ at same FOS c)

$$\tau \cdot \cancel{2\pi} r \cancel{\ell} = \tau_s \cancel{2\pi} R \qquad \text{eqbm}$$

$$\tau = \tau_s \frac{R}{r}$$

$$\gamma = \frac{\partial w}{\partial \cancel{z} r} \qquad \tau = G\gamma = G\frac{\partial w}{\partial \cancel{z} r}$$

$$G\frac{\partial w}{\partial r} = \tau_s \frac{R}{r}$$

$$\int_{w_0}^{0} \frac{G}{\tau_s R}\, dw = \int_{R}^{r_m} \frac{dr}{r}$$

$$\frac{G}{\tau_s R} w_0 = + \ln\left(\frac{\cancel{R} r_m}{\cancel{R} R}\right)$$

Total stiffness $\quad F = \tau_s 2\pi R L$

$$\frac{2\pi \cancel{R} L G}{\cancel{R} F} w_0 = + \ln\left(\frac{r_m}{R}\right)$$

$$\frac{F}{w_0} = \frac{\cancel{\ \ }}{2\pi G L} \Big/ \ln\left(\frac{r_m}{R}\right) = \cancel{\frac{\ \ }{2\pi G L}}$$

$$= \frac{2\pi G L}{3}$$

d) $\xi = 4$ $\quad G_s = 20\ MPa$ $\quad L = 20 \quad D = 1m$

$t = 20mm$

$G_b = 50MPa$

$G_a = 35$

$$E_p \cdot \frac{\cancel{\pi} D^{\cancel{2}}}{4} = 210\ GPa \times \cancel{\pi}\cancel{D}\, t$$

$$E_p = \frac{4t}{D} \cdot 210\ GPa = \underline{\underline{16.8\ GPa}}$$

$\eta = 1 \qquad \frac{L}{D} = 20 \qquad \rho = \frac{35}{50} = 0.7 \qquad \xi = 1$

$\lambda = 480$

$\mu = 0.065 \qquad \nu = 0.4$

$$\frac{V}{\omega} = D G_L \times \frac{\left(\frac{2 \cancel{\ }}{0.6} + 0.7 \times \frac{2\pi}{4} \times 0.66 \times 20\right)}{\left(1 + \frac{1}{480\pi}\ \frac{8}{0.6} \times 0.66 \times 20\right)}$$

$$\frac{17.85}{1.117}$$

$= 153 \times 1 \times 50MPa$

$= \underline{\underline{7.65\ MN/m}}$

e) As piles become long axial compression of pile means no load reaches pile base. Compression of upper portion of pile dominates behaviour.

Q4

(a) $\gamma_{dry} = \gamma_w \frac{G_s}{1+e}$

$\gamma_{sat} = \gamma_w \frac{(G_s+e)}{1+e}$

upper sand: $\gamma_{dry} = 9.81 \times \frac{2.7}{1.9} = 13.9\ kN/m^3$

$\gamma_{sat} = 9.81 \times \frac{2.7+0.9}{1.9} = 18.6\ kN/m^3$

lower sand: $\gamma_{dry} = 9.81 \times \frac{2.7}{1.7} = 15.6\ kN/m^3$

$\gamma_{sat} = 9.81 \times \frac{2.7+0.7}{1.7} = 19.6\ kN/m^3$

(b) first work out design values of φ' and δ

upper sand $\varphi'_d = \tan^{-1} \frac{\tan 33}{1.25} = 27.5°$

$\delta_d = \tan^{-1} \frac{\tan(33/3)}{1.25} = 8.8°$

lower sand $\varphi'_d = \tan^{-1} \frac{\tan 39}{1.25} = 32.9°$

$\delta_d = \tan^{-1} \frac{\tan(39/3)}{1.25} = 10.5°$

with these values:

upper sand : $k_{a,d} = \frac{1-\sin 27.5}{1+\sin 27.5} = 0.369$

$2\theta_d = \sin^{-1}\left(\frac{\sin 8.8}{\sin 27.5}\right) + \frac{8.8}{180}.\pi = 0.494$

$k_{p,d} = \frac{\cos(8.8)}{1-\sin(27.5)}\left[\cos(8.8) + \sqrt{(\sin 27.5)^2 - (\sin 8.8)^2}\right] e^{0.494 \tan 27.5} =$

$= 3.372$

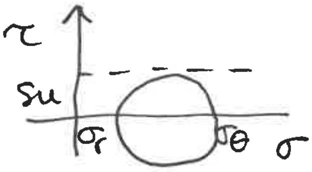
(c) in the elastic zone:

$$\sigma_r = \sigma_0 - \frac{G\delta A}{\pi r^2}$$

$$\sigma_\theta = \sigma_0 + \frac{G\delta A}{\pi r^2} \qquad \therefore \quad \sigma_\theta - \sigma_r = 2G\frac{\delta A}{\pi r^2}$$

in the plastic zone:

$$\sigma_\theta - \sigma_r = 2su$$



at the boundary between the elastic zone and the plastic zone ($r = r_p$)

$$\cancel{2}su = \cancel{2}G\frac{\delta A}{\pi r_p^2}$$

$$r_p = \sqrt{\frac{G\delta A}{\pi su}} = \sqrt{\frac{10000 \cdot 0{,}024 \cdot \cancel{\pi} \cdot 2{,}5^2}{\cancel{\pi} \cdot 80}} = 4{,}3\text{m}$$

In order to be outside the plastic zone, the foundation should be at a distance, c, of at least

$$c = r_p - r_c = 4{,}3 - 2{,}5 = 1{,}80 \text{ m}$$

Q3

(a) uniform radial contraction, ρ_c

tunnel lining radial stress, σ_c

initial total stress in the ground, σ_0

change of stress, $\delta\sigma_c = \sigma_0 - \sigma_c$

from Data Book:

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

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

$G = 10\ MN/m^2$

$s_u = 80\ kN/m^2$

$$\therefore \quad \frac{G}{s_u} = \frac{10 \times 10^3}{80} = 125$$

$\rho_c = 30\ mm$

$r_c = 2.5\ m$

$$\therefore \quad \frac{\delta A}{A} = \frac{2 \times 30}{2500} = 0.024$$

$$\delta\sigma_c = 80\left(1 + \ln 125 + \ln 0.024\right) = 167.9\ kPa$$

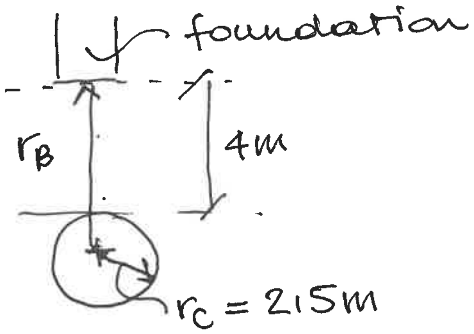
tunnel axis @ 20 m depth $\quad \gamma = 20\ kN/m^3$

$$\sigma_{vo} = \sigma_{ho} = \sigma_0 = 20 \times 20 = 400\ kPa$$

$$\sigma_c = \sigma_0 - \delta\sigma_c = 400 - 167.9 = 232.1\ kPa$$

(b) undrained kinematics : $2\pi r \rho = \text{const.}$

with ρ radial displacement at radius r



$$r_c \rho_c = r_B \rho_B$$

$$\rho_B = \frac{r_c}{r_B} \cdot \rho_c = \frac{2.5}{6.5} \cdot 30 = 11.5\ mm$$

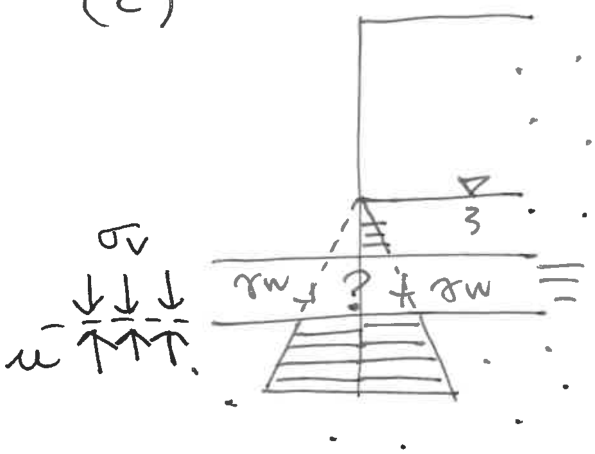
lower sand: $K_{a,d} = \dfrac{1-\sin 32.9}{1+\sin 32.9} = 0.296$

$$2\theta_d = \sin^{-1}\left(\frac{\sin 10.5}{\sin 32.9}\right) + \frac{10.5}{180} \cdot \pi = 0.523$$

$$K_{p,d} = \frac{\cos 10.5}{1-\sin 32.9}\left[\cos 10.5 + \sqrt{(\sin 32.9)^2 - (\sin 10.5)^2}\right] e^{0.523 \tan 32.9} =$$

$$= 4.524$$

(c)



The pore water pressure on either side of the wall remains hydrostatic, both above and below the clay layer.

The pore water pressure under the clay layer is $u = 3 \times 9.81 = 29.4$ kPa

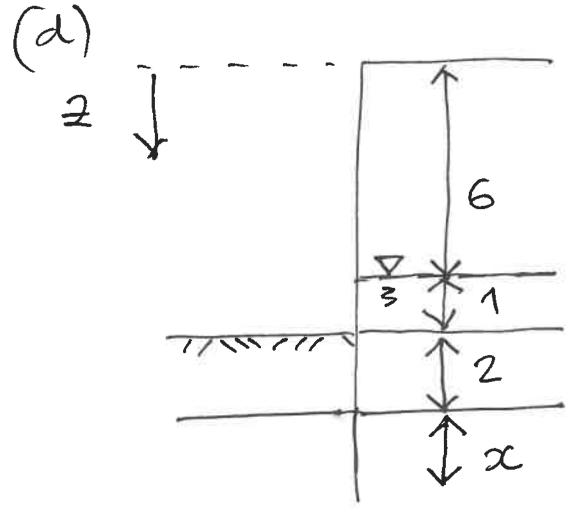
Inside the excavation this is contrasted by the self weight of the clay, with a total vertical stress $\sigma_v = 20 \times 2 = 40$ kPa

The safety factor against base heave instability is therefore

$$F_s = \frac{\sigma_v}{u} = \frac{40}{29.4} = 1.36 > 1$$

This is probably not large enough and it may be a good idea to carry out construction with phased excavation and casting of the base slab in segments

(d)



$K_{ad} = 0.369$
$\gamma_{dry} = 13.9$ kN/m³

$\gamma_{sat} = 18.6$ kN/m³

$\gamma_{sat} = 20$ kN/m³ $S_{ud} = 28.6$ kPa

$\gamma_{sat} = 19.6$ kN/m³
$K_{ad} = 0.296$
$K_{pd} = 4.524$

$S_{ud} = \dfrac{S_{uk}}{1.4} = \dfrac{40}{1.4}$

ACTIVE SIDE

z(m)	σ_v (kPa)	u (kPa)	σ'_v (kPa)	σ'_a (kPa)	σ_a (kPa)
0	0	0	0	0	0
6	83.6	0	83.6	30.9	30.9
7	102.2	9.81	92.4	34.1	43.9
7	102.2	?	?	?	45.1
9	142.2	?	?	?	85.1
9	142.2	29.4	112.8	33.3	62.8
9+x	142.2+19.6x	29.4+9.81x	112.8+9.8x	33.3+2.9x	62.8+12.7x

In the table above $\sigma_a = \sigma'_a + u = K_a \sigma'_v + u$, in the sand layers and $\sigma_a = \sigma_v - 2 s_u$, in the clay.

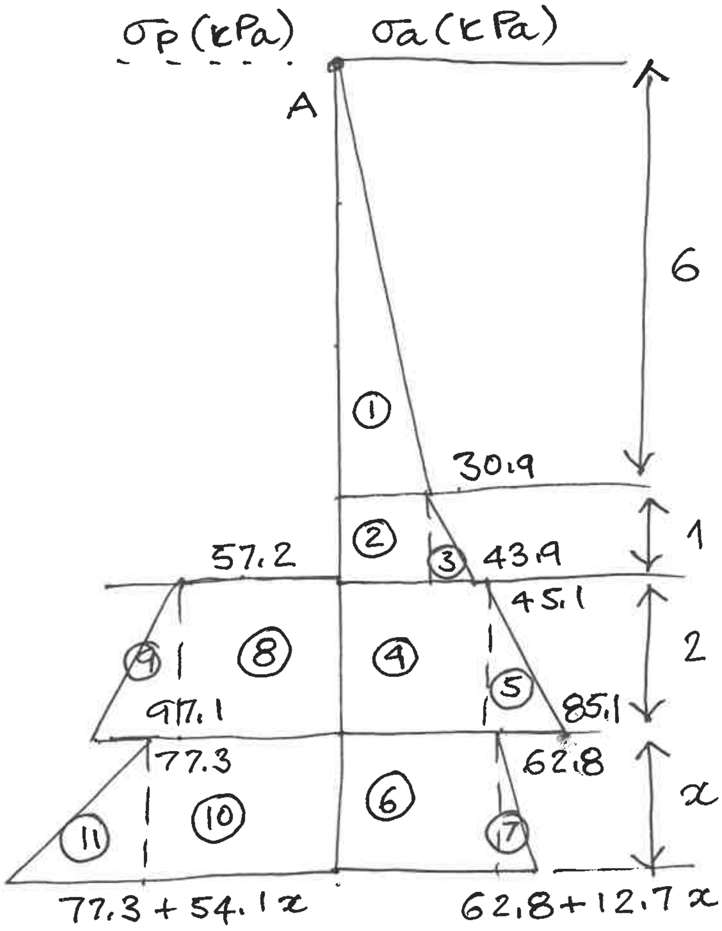
PASSIVE SIDE

z(m)	σ_v (kPa)	u (kPa)	σ'_v (kPa)	σ'_p (kPa)	σ_p (kPa)
7	0	?	?	?	57.2
9	40	?	?	?	97.1
9	40	29.4	10.6	47.8	77.3
9+x	40+19.6x	29.4+9.8x	10.6+9.8x	47.8+44.29x	77.3+54.1x

Again $\sigma_p = \sigma'_p + u = K_p \sigma'_v + u$ in the sand layers and $\sigma_p = \sigma_v + 2 s_u$ in the clay layer.

With the values in the table it is possible to sketch the total earth pressure distribution

(d) continued



moment equilibrium about A

	F (kN/m)	b (m)	M (kNm/m)
1	$\frac{1}{2}(30.9\times 6) = 92.7$	$\frac{2}{3}\times 6 = 4$	370.8
2	$30.9\times 1 = 30.9$	$6+\frac{1}{2} = 6.5$	200.8
3	$\frac{1}{2}(13\times 1) = 6.5$	$6+\frac{2}{3}\times 1 = 6.7$	43.5
4	$45.1\times 2 = 90.2$	$7+\frac{1}{2}\times 2 = 8$	721.6
5	$(40\times 2)\times\frac{1}{2} = 40$	$7+\frac{2}{3}\times 2 = 8.33$	333.3
6	$62.8x$	$9+x/2$	$565.2x+31.4x^2$
7	$\frac{1}{2}12.7x^2 = 6.35x^2$	$9+\frac{2}{3}x$	$57.15x^2+4.23x^3$

$$M_a = 4.23x^3+88.55x^2+565.2x+1670$$

	F (kN/m)	b (m)	M (kNm/m)
8	$57.2\times 2 = 114.4$	8	915.2
9	$40\times 2\times\frac{1}{2} = 40$	$7+\frac{2}{3}\times 2 = 8.33$	333.3
10	$77.3x$	$9+x/2$	$695.7x+38.65x^2$
11	$\frac{1}{2}54.1x^2 = 27.1x^2$	$9+\frac{2}{3}x$	$243.9x^2+18.07x^3$

$$M_p = 18.07x^3+282.55x^2+695.7x+1248.5$$

$$M_p - M_a = 13.84x^3+194x^2+130.5x-421.5$$

(d) continued.

$$Mp - Ma = 13{,}84x^3 + 194x^2 + 130{,}5x - 421{,}5$$

solve for x

x	$Mp - Ma$
2	726,22
1	−83,16
1,2	38,38
1,15	6,189 ✓

depth of embedment $d = 2 + x = 3{,}15 \; (\simeq 3{,}2m)$