Q1 a) Start by taking a known mass of the soil sample, say 100 grams. Place it on
the top sieve of a stack of BS standard seives. Shake the stack for 20 min. Weigh the
soil left on each sized seive. Work out the percentage passing through each seive
size. Plot the seive size on x-axis and percentage passing ~~for~~ on the y axis. For
very fine fractions of the soil, you may have to conduct a sedimentation experiment
to determine the fine silt and clay fractions.

In a PSD, D_{10} represents the average size of the voids, while D_{50} gives
the average size of the solid particles. [10%]

b) Soil can be classed as uniformly graded, poorly graded or gap graded (same
soil fraction sizes are missing) based on a PSD curve. However, it is risky
to classify the soil as clay or sand based on PSD alone. Ground up silica
(rock flour) may have same particle sizes as a clay fraction, but the behaviour
will be totally different. This is due to the water adsorbency of the clay particles
which have surface charges. So in addition to PSD, we perform the liquid, plastic
limit tests and determine the PI = LL-PL. [15%].

c) Plasticity index (PI) is defined as the difference between liquid limit and
plastic limit

$$PI = LL - PL$$

PI indicates the water absorbancy of the soil. Low PI values indicate that
the soil has low water absorbancy and we would expect sand/silt like behaviour
with low plastic fines content. High PI values on the other hand indicate
more clay like behaviour, with water molecules attaching themselves to the
clay particles due to their surface charges.

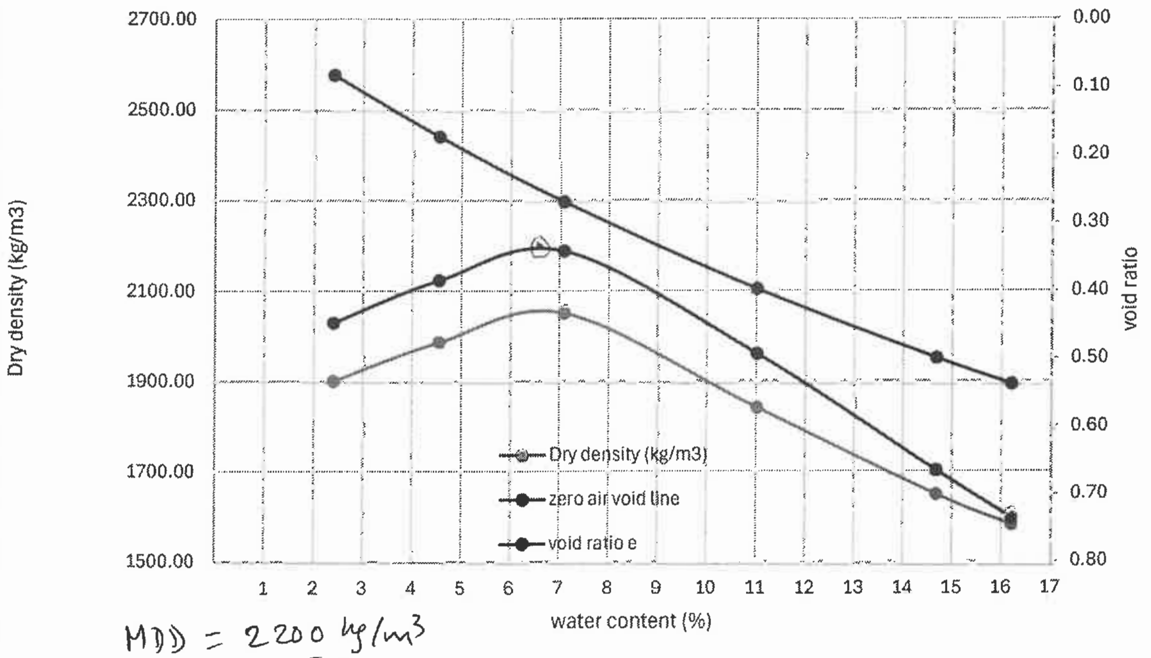
Overall, in addition to the PSD curve, knowing the PI of the soil
helps in classification of the soil correctly and in predicting its behaviour
in various geotechnical applications.

[15%]

1d) &e

	Gs=		2.75	1000	
Water content	density (kg	Dry density (kg/m3	void ratio e	Sr (%)	zero air void line
2.4	1948.7	1903.03	0.4451	0.1483	2579.74
4.55	2080.6	1990.05	0.3819	0.3277	2444.17
7.09	2200.7	2055.00	0.3382	0.5765	2301.30
11.02	2050.8	1847.23	0.4887	0.6201	2110.43
14.68	1900.0	1656.78	0.6598	0.6118	1959.11
16.21	1848.0	1590.22	0.7293	0.6112	1902.09

Zero air void line signifies max density possible when the voids are fully saturated. Practising engineers prefer it, as it indicates readily how far the density they achieved is from max theoretical density.



MDD = 2200 kg/m³
OMC = 6.75 %

[40%]

1 f Wet of optimum water content, air is trapped between clumps of grains and can be expelled easily ie with low compactive effort. The pressure induced by hammer in a Proctor test is borne by water and therefore the effective stress drops and there is less frictional resistance. After compaction ($S_r \approx 90\%$) the remaining air cannot be expelled by compaction. As pore fluid cannot get out no further compaction can be achieved. Although this leads lower dry density, the soil texture is more uniform and soil has more ductile behaviour.

Dry of optimum wc, surface tension effects increase as menisci retreat to tight spaces. The soil gets hard to compact, so density reduces.

At optimum wc, the presence of menisci in D_{10} size pores makes grains stick together when hammered giving max. dry density

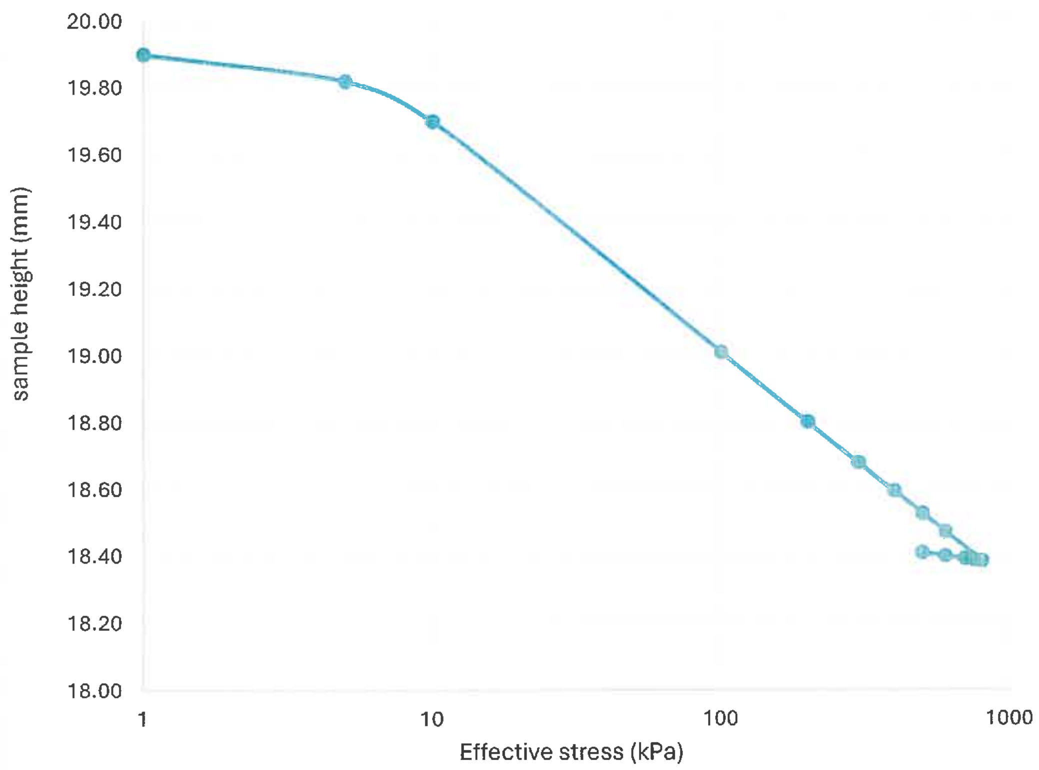
[20%]

Q2 a) An oedometer test is conducted on a small soil sample that is placed in a cylindrical container. The d/h ratio of the samples is normally about 2.5 to 3 and sample diameters can be 50 mm to 75 mm. The soil sample is placed in the container and loading platen is placed on top. The whole sample container can be placed in a water bath if consolidation and swelling tests are to be performed. Loading is applied through a lever mechanism so that the applied dead weight loading can be amplified. During the experiments, the applied load (and hence vertical stress) and the sample height at each load increment is recorded. For clayey soils, normally 24 hours is allowed between load application and taking the sample height reading, to allow for consolidation of the soil to take place.

From the oedometer results, we can obtain the λ and κ values for the soil. We may also be able to say the pre-consolidation pressure that the soil has previously seen.

[20%]

b) Plot of the data given is shown below. Change in sample height is proportional to change in void ratio, in an oedometer test.



Pre consolidation pressure ~ 10 kPa

Calculate the slope of the NCL and rebound lines.

$\lambda = 0.3$

K = 0.05

[30%]

2 c) Unit weight of embankment = 23.2 kN/m³.

∴ Vertical stress applied to the clay layer = 23.2×4
= 92.8 kPa.

Clay layer 8 m thick; $\lambda = 0.3$ & $\kappa = 0.05$ from part b).

Vertical stress at 4m depth :

Clay layer $e = 0.71$ initially and $G_s = 2.71$.

$$\therefore \gamma_{sat} = \left(\frac{G_s + e}{1+e}\right)\gamma_w = \frac{2.71+0.71}{1.71} \times 9.81 = 19.62 \text{ kN/m}^3.$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.62 - 9.81 \approx 9.81 \text{ kN/m}^3.$$

at 4m depth $\sigma_v' = 9.81 \times 4 = 39.24$ kPa.

∴ Vertical stress at 4m = $92.8 + 39.24 = 132.04$ kPa.

∴ change in sample height = $19.7 - 0.3 \ln(132.04 - 10)$

$\Delta h = 18.2589$ mm.

$$\therefore \varepsilon_v = \frac{\Delta h}{h} = \frac{(19.7 - 18.2589)}{19.7} \times 100 = 7.315\%$$

∴ Settlement of 8m clay layer = $8 \times \frac{7.315}{100} = 0.585$ m

[30%]

2 d) If the water table is at half the height of the embankment, then

Vertical stress due to embankment = $23.2 \times 2 + (23.2 - 9.81)\,2$
= 73.18 kPa

∴ Vertical stress at 4m depth in clay layer

= $73.18 + 39.24 = 112.42$.

∴ change in sample height = $19.7 - 0.3 \ln(112.42 - 10)$
= 18.311 mm

$$\varepsilon_v = \frac{19.7 - 18.311}{19.7} = 7.049\%.$$

∴ Settlement of 8m clay layer = 0.564 m

So the settlement is smaller if the water table is higher, due to lower effective stress

(20%)

3 a) for centre equivalent to 4 squares 5×5 m

$$I_r = \frac{0.25}{4} = 0.0625$$

$$m = n = 0.41$$

$$\text{Depth} = \frac{5}{0.41} = 12.2\,m$$

b) corner $I_r = 0.25$

$$m = n = \infty$$

$$\text{Depth} = 0!$$

c) Corner All on K

2 layers 0 – 5 m & 5 – $\cancel{15}$ 20 m (say)

@ 2.5 m $\sigma_{v0}' = 15$ kPa $\sigma_{vf}' = 115$ kPa

$\Delta v = 0.102$ $\epsilon_v = 0.037$

$\rho = 0.185\,m$

@ 12.5 m $\sigma_{v0}' = 75$ kPa $\sigma_{vf}' = 133$ kPa

$\Delta v = 0.0286$

$\epsilon_v = 0.01$

$\rho = 0.156\,m$

$\rho_T = \underline{0.341\,m}$

Centre

| layer on λ | layer on κ

@ 6.1m $\sigma'_{v0} = 36.6\ kPa$ $\sigma'_{vf} = 275\ kPa$

$\Delta v = 0.248$ $\epsilon = 0.09$

$\rho = 1.1\ m$

@ 16.1m $\sigma'_{v0} = 96.6\ kPa$ $\sigma'_{vf} = 160\ kPa$

$\Delta v = 0.025$ $\epsilon = 0.009$

$\rho = 0.072\ m$

$\rho_T = 1.17\ m$

d) Overall settlement large once consolidation completes - problems will services.

Very large differential settlement leads to cracking of brittle structure especcally at bottom where it is in tension.

SVI

4 a) $\rho_{ult} = \frac{80}{\overset{2}{\cancel{10}},000} \times 10 = \underline{\underline{0.4m}}$

$R_v = 0.9 \rightarrow T_v = 0.716$ (parabolic)

or $T_v \sim 0.84$ (fourier)

$\frac{C_v t}{d^2} = T_v \qquad t = \frac{T_v d^2}{C_v} = 17.9$ years

20.9 years (fourier

b) Increase load to $\frac{1\cancel{8}}{5} \times 80 = \overset{160}{\cancel{128}}$ kPa

n.c soil so on λ

$\rho_{ult} \simeq 0.4 \times \ln$

Centre of clay layer $\sigma'_{vo} = 80kPa - 30kPa$

$\sim \underline{\underline{50 kPa}}$

$\rho_{ult} = 0.4 \times \frac{\ln\left(\frac{\overset{210}{\cancel{178}}}{50}\right)}{\ln\left(\frac{130}{50}\right)} = \underline{\underline{0.5\cancel{8}m}}$

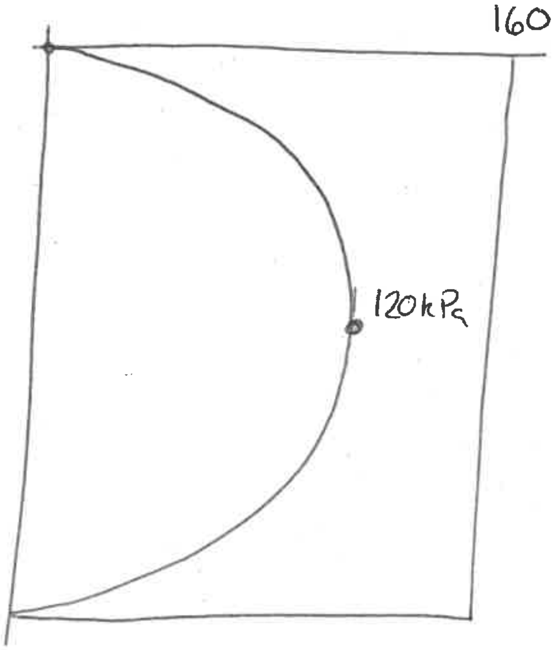
$R_v \sim 2/3 \qquad T_v = \cancel{\ln\left[-\tfrac{3}{4}(R_v - 1)\right]}\ \underline{\underline{0.314}}$

$t = \underline{\underline{7.85}}$ years

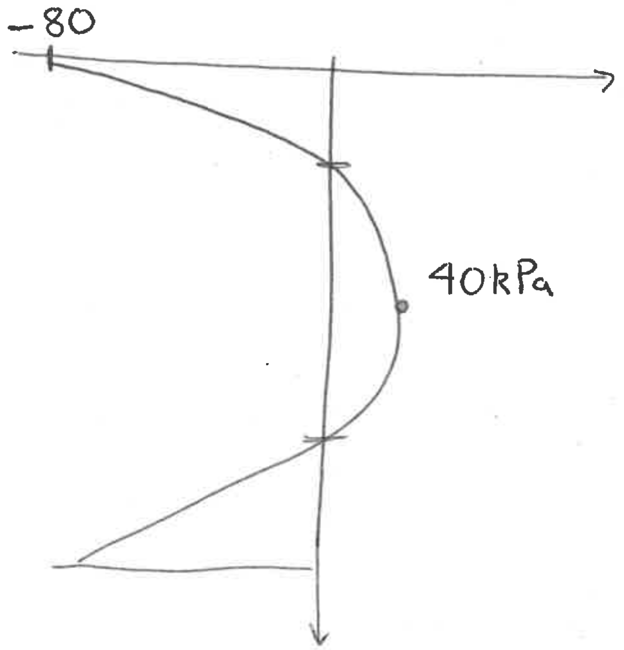
$b = \underline{\underline{0.5}}$

c)

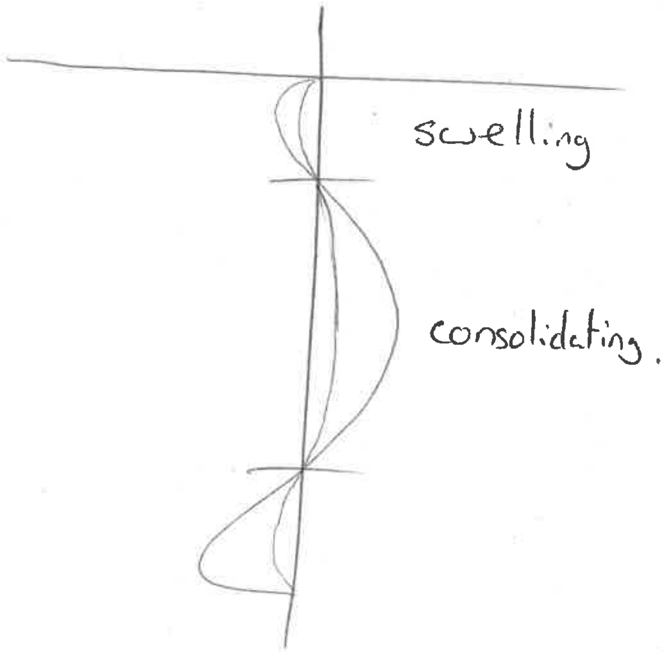
Before removal



After removal



Reconsolidation



b) 160 kPa so settlement doubles

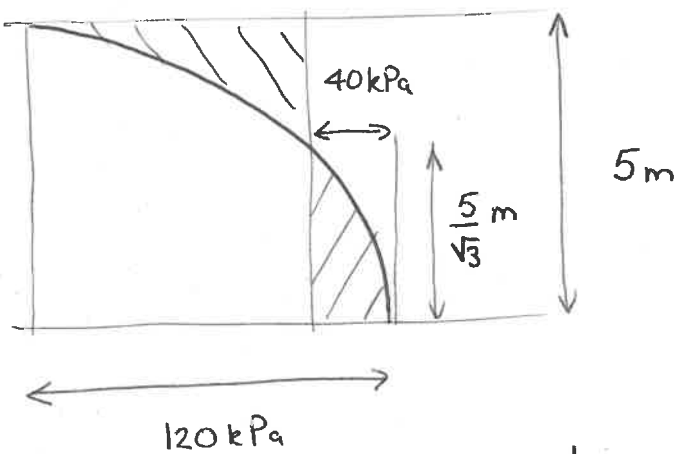
$R_v = 1/2 \rightarrow T_v = 0.18$

$t = 4.5$ years.

$b = \underline{0.75}$

d) Consolidation will continue on λ but swelling zones will be on κ so recover only $0.16\times$ compression.

Assuming parabola



Shaded areas equal

each $A = \frac{2}{3} \times 40 \times \frac{5}{\sqrt{3}}$

$= \underline{77}$ kPa m

If consolidation had continued,

extra settlement $= \frac{2}{3} \times 120 \times 5$

$= \underline{400 \text{ kPa m}}$

further settlement $= \frac{77 \times (1-0.16)}{400} \times 0.4 = \underline{\underline{0.065\text{m}}}$