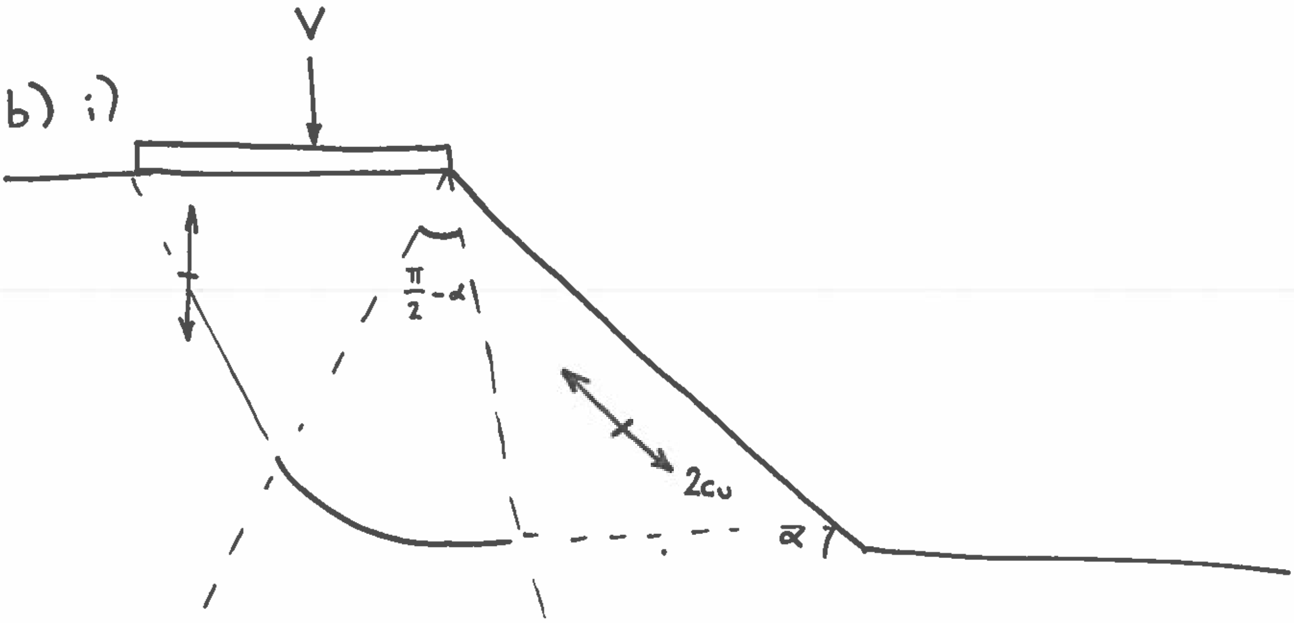
1. a) If a stress field exists in equilibrium with the applied loads which is everywhere within yield, the load can be carried - Lower Bound on collapse.

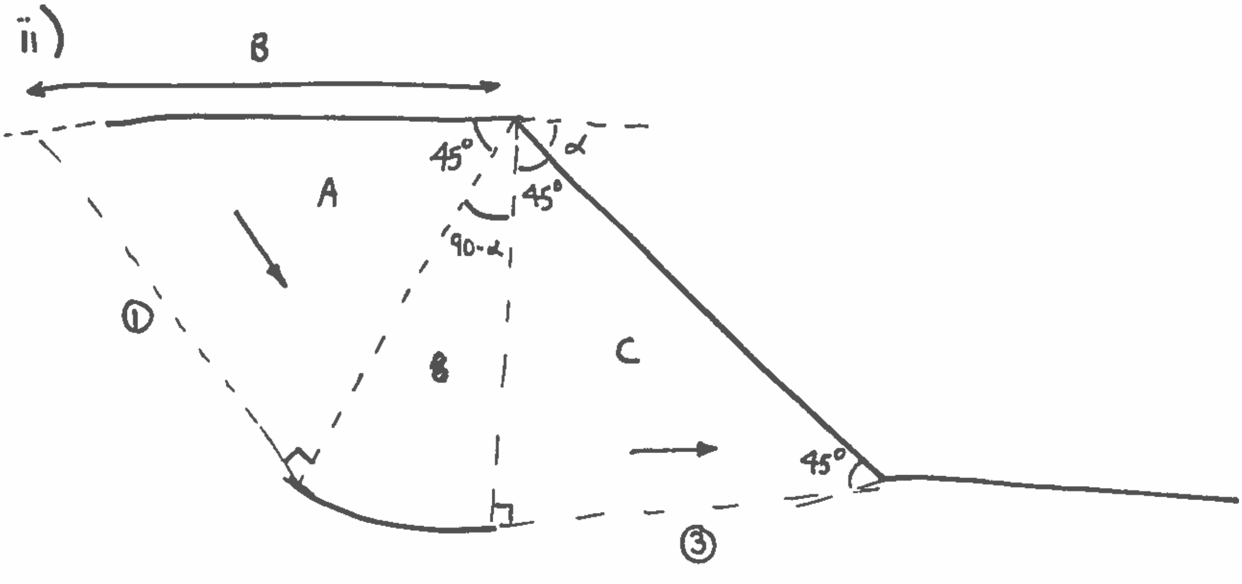
If a mechanism exists which dissipates the work done by the applied loads, the loads cannot be carried - Upper Bound on collapse.

b) i)

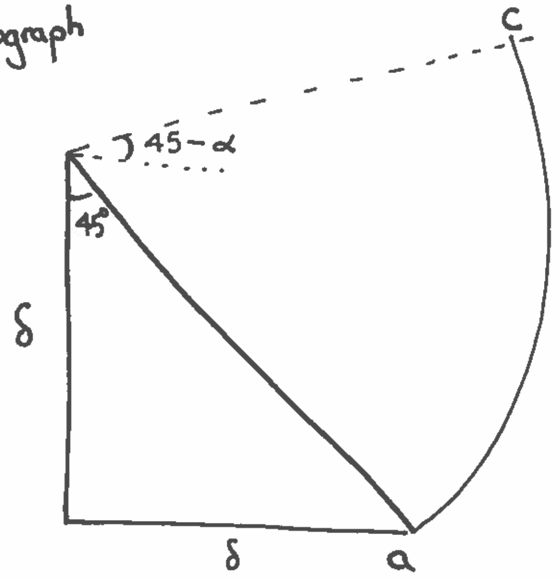


$$V = (2 + \pi - 2\alpha) B s_u$$

ii)



Hodograph



Work in = $V\,\delta$

$$\text{Work done} = s_u\left[\frac{B}{\sqrt{2}}\sqrt{2}\,\delta \;(1) + \frac{B}{\sqrt{2}}\sqrt{2}\,\delta \;(3) + 2\sqrt{2}\,\delta\,\frac{B}{\sqrt{2}}\left(\frac{\pi}{2}-\alpha\right) \text{(fan)}\right]$$

$$= B\,s_u\,\delta\left[2 + \pi - 2\alpha\right]$$

iii) Soil weight will do net work if $\alpha > 0$ as loss of P.E. of block A will be greater than gain of block C.

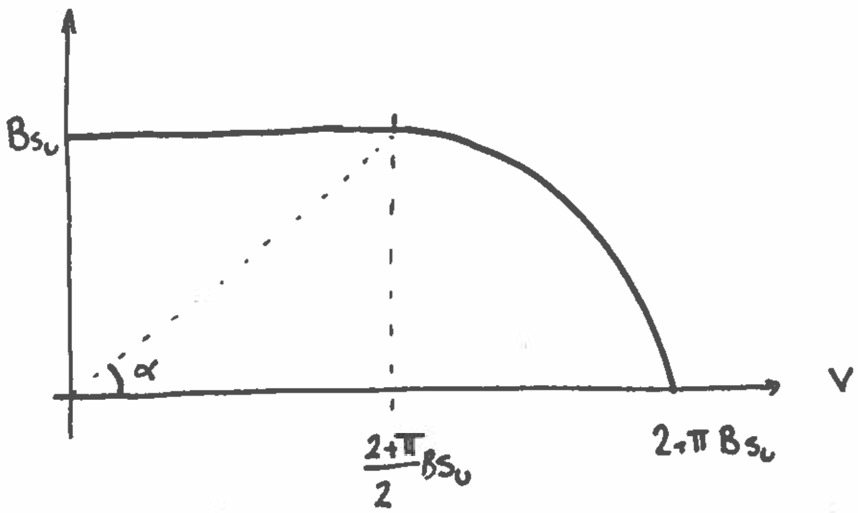
[This was also ignored in lower bound calc which would also reduce].

Q1 Plasticity solutions

32 attempts, Average mark 15.3/20 A relatively straightforward question on upper and lower bound plasticity solutions in clay which was in general answered well by most candidates with several perfect scores.

2.

a)



$$\tan \alpha = \frac{2}{2+\pi} = 0.389 \qquad \alpha = 21.25^\circ$$

$\alpha = 10$:

$$\frac{H}{H_{ult}} = 1 - \left(2\frac{V}{V_{ult}} - 1\right)^2$$

$H_{ult} = Bs_u$ $\qquad H = F \sin 10 = 0.1736 F$

$V_{ult} = (2+\pi) B s_u$ $\qquad V = F \cos 10 = 0.9848 F$

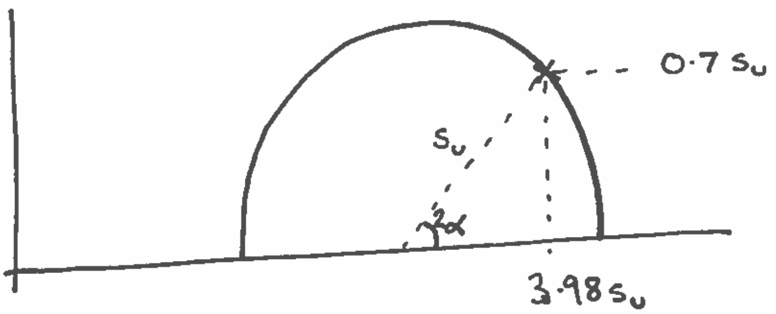
If $x = \frac{F}{Bs_u}$

$$0.1736 x = 1 - (0.383 x - 1)^2$$

$$= -0.1467 x^2 + 0.766 x$$

$x = 0$ or $x = \underline{\underline{4.04}}$

$F = 4.04 Bs_u$ $\qquad H = 0.7 Bs_u$ $\qquad V = 3.98 Bs_u$



$2\alpha = 44.4^\circ$

$\therefore$ max princ stress @ 22.2° to vertical

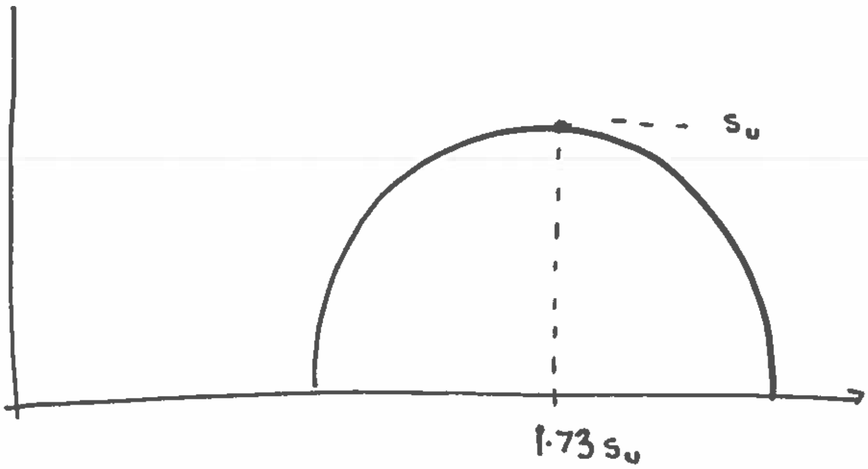
value = $\underline{\underline{4.26\ s_u}}$

b) $30^\circ > 21.25$

$\Rightarrow \quad H = B s_u$

$$V = \frac{H}{\tan 30} = 1.73 B s_u$$

$$F = \underline{\underline{2 B s_u}}$$



Max principale stress @ 45° to vertical value $\underline{\underline{2.73 s_u}}$

Q2 V-H loading on shallow foundation
29 attempts, Average mark 10.5/20

This question had many very good answers but also many that were substantially incomplete, probably because of time, reducing the average mark substantially. Many candidates chose to make life hard for themselves by calculating from first principles, often successfully but at the expense of time taken. The most common errors were forgetting that the foundation could also fail in sliding (in part c) and not calculating both principal stress and orientation in parts b and d.

PROBLEM 3

(a) (i) $M_m = 4.25 kg$ $\qquad V_T = 944 \, cm^3$

$M_{w+m} = 6.38 kg$

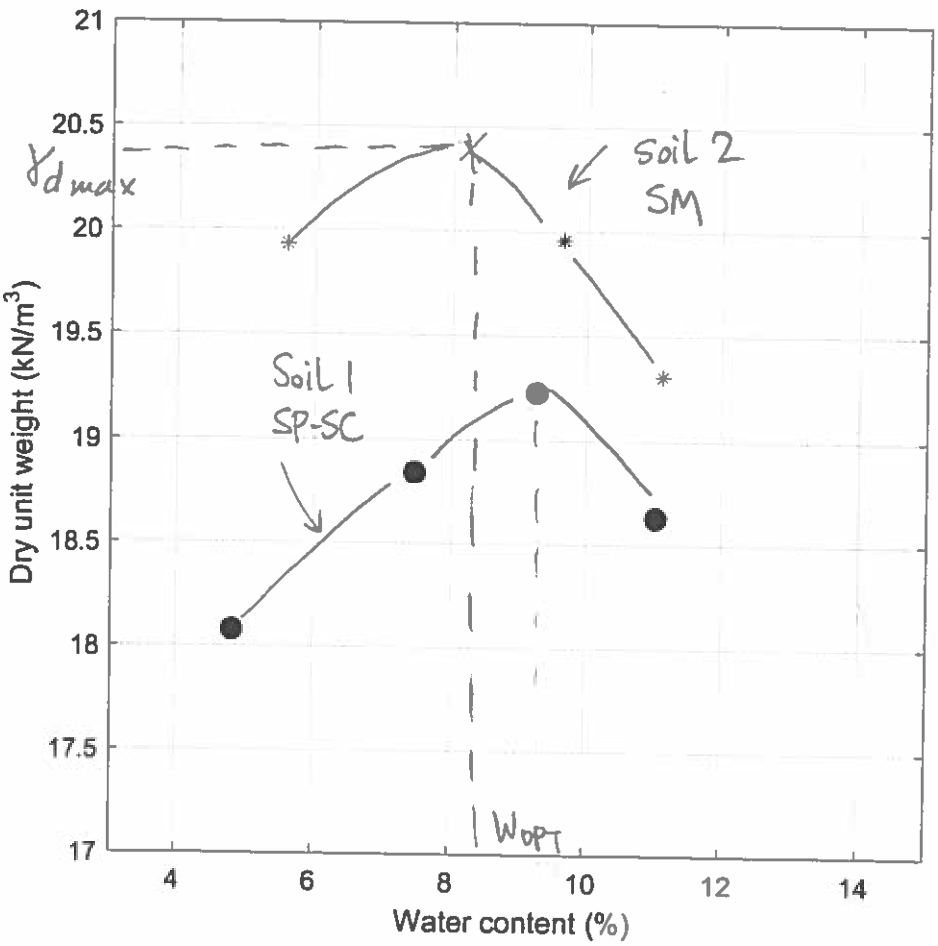
$M_d = 1.97 kg$

$$W = \frac{(6.38 - 4.25) - 1.97}{1.97} = 8.12\%$$

$$\gamma_T = \frac{(6.38 - 4.25)}{944 \, cm^3} \left(9.8 \frac{m}{s^2}\right) \frac{(100 \, cm)^3}{1 m^3} = 22.1 \frac{kN}{m^3}$$

$$\gamma_d = \frac{\gamma_T}{1+w} = \frac{22.1 \, kN/m^3}{1+0.0812} = 20.45 \frac{kN}{m^3}$$

For standard Proctor test:



Soil 1 SP-SC (poorly graded sand)

$W_{OPT} = 9.2\%$

$\gamma_{d\,max} = 19.3 \, kN/m^3$

Soil 2 SM (Silty Sand)

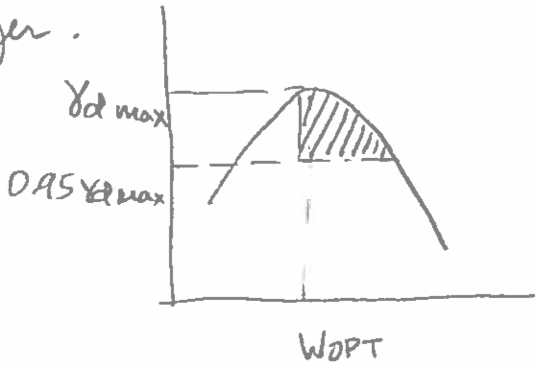
$W_{OPT} = 8\%$

$\gamma_{d\,max} = 20.5 \, kN/m^3$

(aii) Soil 2 has significantly more fines than Soil 1. The fines can pack more effectively in the voids between sand particles, resulting in a denser compacted soil. The fines in Soil 2 are also silty, therefore non-plastic, while those in soil 1 are clayey. Silt particles are also more likely to pack more effectively than clay particles.

Earth dams are usually constructed with different zones, fulfilling different functions: a core, providing a barrier against flow, and shells, providing strength and stability.

Soil 2 is more likely to be more effective as shell material because it can be compacted at a denser state. A silty sand is also likely to be stronger.



$\gamma_d \geq 0.95\,\gamma_{d\,max} \simeq 19.4\ \frac{kN}{m^3}$

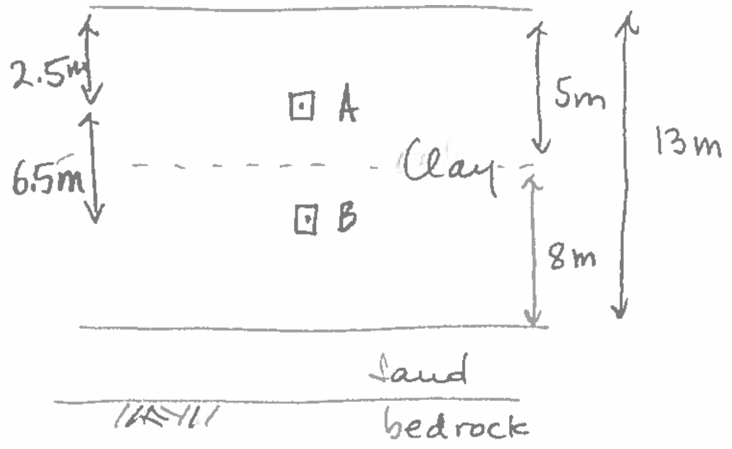
$7\% \leq W \leq 10\%$

compact on wet side to avoid brittle failure

Soil 1 may be acceptable as a material for the core, but its hydraulic conductivity would need to be checked first. The specifications would follow a similar reasoning

$\gamma_d \geq 18.3\ kN/m^3$, $9\% \leq W \leq 12\%$

(b)



A: $w_0 = 41.7\%$

OCR = 4

$e_{o_A} = (2.75)(0.417) = 1.147$

B: $w = 37.5\%$

OCR = 1.5

$e_{o_B} = (2.75)(0.375) = 1.031$

(i) $\gamma_{sat_A} = \dfrac{G_s + Se}{1+e}\gamma_w = \dfrac{(2.75)+(1)(1.147)}{1+1.147}\left(9.8\,\dfrac{kN}{m^3}\right) = 17.8\,\dfrac{kN}{m^3}$

$\gamma_{sat_B} = \dfrac{(2.75)+(1)(1.031)}{1+1.031}(9.8) = 18.3\,kN/m^3$

At the centre of the dam, assume 1D conditions

$\Delta\sigma_v = \left(20\,\dfrac{kN}{m^3}\right)(5m) = 100\,kPa$

	σ'_{vo} (kPa)	σ'_{vf} (kPa)	σ'_c (kPa)	Δv_{OC}	Δv_{NC}
A	20	120	80	0.086	0.065
B	74	174	111	0.025	0.072

$\Delta v_{OC} = \kappa \ln\left(\dfrac{\sigma'_c}{\sigma'_{vo}}\right)$

$\Delta v_{NC} = \lambda \ln\left(\dfrac{\sigma'_{vf}}{\sigma'_{vc}}\right)$

$\Delta h_A = \dfrac{h_A}{v_{oA}}\left(\Delta v_{OC} + \Delta v_{NC}\right) = \dfrac{5m}{2.147}(0.086 + 0.065) = 0.351\,m$

$$\Delta h_B = \frac{(8m)}{2.031}(0.025 + 0.072) = 0.382m$$

$$\Delta h_{TOT} = 0.73m$$

(ii) $t = 6mo = 0.5yr$

$$T = \frac{C_v t}{d^2} = \frac{(2.3\ m^2/yr)(0.5yr)}{(13/2)^2} = 0.027 < \frac{1}{12}$$

Therefore phase (i), advancing isochrones

In the middle of the clay layer, at 6mo the pore pressure is the same

$u_{ex} = 100kPa$

$$R_d = \sqrt{\frac{4T_v}{3}} = \sqrt{\frac{(4)(0.027)}{3}} = 0.19 \quad \Rightarrow \rho = 14cm$$

(iii) At 6 mo measured $u_{ex} = 55kPa \Rightarrow b = 0.55$

$\Delta h = 37.5cm \Rightarrow R_d = \frac{37.5}{73} = 0.51$

$b = 0.55$ corresponds to $R_d = 1 - \frac{2}{3}\underbrace{\exp(0.25 - 3T_v)}_{b} =$

$$= 1 - \frac{2}{3}(0.55) = 0.63$$

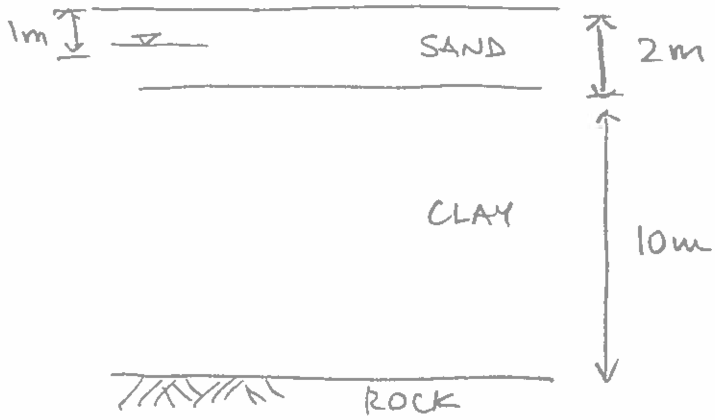
Therefore the field values indicate faster consolidation than expected. It is likely C_v measured in the lab is not fully representative of field drainage condition

Q3 Compaction and consolidation

33 attempts, Average mark 13.3/20

Overall, students answered correctly the initial questions on compaction in large proportion. Most students that actually attempted the second part of the question on consolidation also produced credible answers or good attempts. Low scores are associated with blank answers, rather than major mistakes in any section. Most students demonstrated sufficient competence in both subjects included in the question as indicated by the relatively good average score.

PROBLEM 4



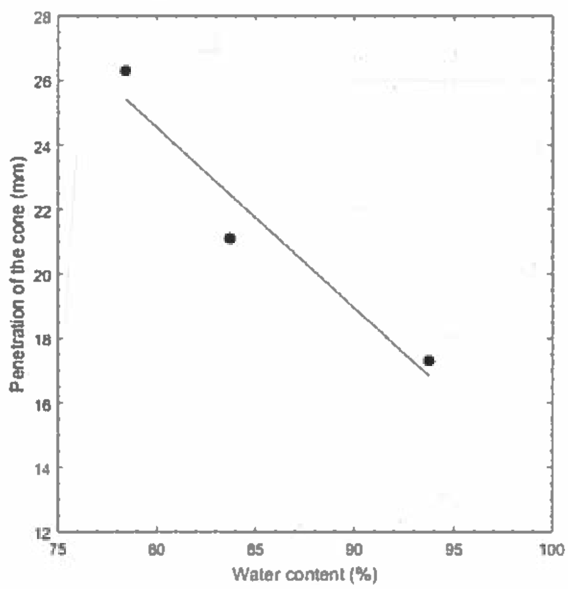
$\gamma_d = 17 \frac{kN}{m^3}$

$\gamma_{sat} = 19.5\ kN/m^3$

$\gamma_{sat} = 17\ kN/m^3$

(a) $w = \frac{M_{wet} - M_{dry}}{M_{dry} - M_{cont}}$

Specimen no.	w (%)
1	78.44
2	83.72
3	93.76



At $d = 20$ mm

$w = 88\%$

LL = 88

(b) $w_{PL_1} = \frac{35.62 - 32.88}{32.88 - 26.55} \times 100\% = 43.3\%$

$w_{PL_2} = \frac{30.51 - 28.36}{28.36 - 23.14} \times 100\% = 41.2\%$

PL = 42.

PI = 46

The plastic limit test is particularly influenced by the skill of the operator. In general, two specimens are required and their water contents need to be within 0.5% of each other. Therefore the results given will not be acceptable.

(c)

$$\sigma'_{v0} = (1)(17) + (1)(19.5) + (5)(17) - (9.8)(6) = 62.7\ kPa$$

$$\Delta\sigma = (5)(19.5\ \frac{kN}{m^3}) = 97.5\ kPa$$

$$\sigma'_{vf} = 160.2\ kPa$$

$$v_0 = 1 + e_0 = 1 + wG_s = 1 + (0.697)(2.68) = 2.868$$

$$\Delta h = \frac{\Delta v}{v_0} H_0 = \lambda \ln\left(\frac{\sigma'_{vf}}{\sigma'_{v0}}\right) \frac{H_0}{v_0} = (0.35)\ln\left(\frac{160.2}{62.7}\right)\left(\frac{10m}{2.868}\right) = 1.16m$$

(d) $C_v = 0.7\ \frac{m^2}{yr}$

$R_d = 90\%$ for $T_v = 0.848$

clay layer is single drainage

$$t = \frac{T_v d^2}{C_v} = \frac{(0.848)(10m)^2}{(0.7 m^2/yr)} = 121\ yrs$$

(e) The construction of an embankment on such soft clay creates two major issues: the settlement is very large and time rate of settlements is very slow.

The compacted soil making up the embankment would not be able to accommodate over 1m of deformation over time, resulting in cracks. Since an embankment does not produce a uniform pressure on the soil surface, differential settlement will also occur in the plane of the cross-section. Any soil heterogeneity will also result into differential settlement along the longitudinal direction. The time rate is such that the embankment would continue to deform for a very long time, and the problems outlined above would continue until the end of consolidation.

It is possible to use preloading to pre-consolidate the soil before final construction of the roadway. This is likely to still take too long and it will be necessary to use methods to speed up consolidation rate, i.e. vertical drains.
If the section of embankment on soft soil is relatively short, piles could be used. Ground improvement with stone columns or deep mixing (with cement) are additional alternatives.

Q4 Soil characterisation

32 attempts, Average mark 12.6/20 Students seemed to be surprised by the question on Atterberg limits, with a good number of them leaving the two questions blank, or making very little progress. Generally, students answered the relatively straightforward questions in part c) and d) well. Most students identified the issues with the estimates of amount of settlements and time for settlement to occur and were able to propose reasonable mitigation strategies. Overall, the students answered quite competently.