1) a. $h = 60$ mm ; $d = 30$ mm ; $T = 0.49, 1.34, 2.74$ Nm
@ $z = 2.0, 6.0, 12.0$ m

$$T = \frac{\pi d^3}{6}\left(1 + 3\frac{h}{d}\right) S_u \quad \therefore \quad S_u = \frac{6T}{\pi d^3\left(1 + 3\frac{h}{d}\right)}$$

$$= \frac{6T}{\pi \times 0.03^3\left(1 + 3 \times \frac{0.06}{0.03}\right)} \times \frac{1}{1000}$$

$\therefore S_u = 5$ kPa @ 2.0 m
$= 13.5$ kPa @ 6.0 m
$= 27.7$ kPa @ 12.0 m

[20%]

b. $\alpha = 0.2$; $\gamma' = 5$ kN/m^3 ; $\gamma = 15$ kN/m^3 ; $q_c = 53, 155, 322$ kPa
$u_2 = 50, 150, 260$ kPa
@ $z = 2.0, 6.0, 12.0$ m

$$q_t = q_c + (1 + \alpha) u_2$$

$= 93$ kPa @ 2.0 m
$= 275$ kPa @ 6.0 m
$= 530$ kPa @ 12.0 m

$$q_{net} = q_t - \gamma z$$

$= 63$ kPa @ 2.0 m
$= 185$ kPa @ 6.0 m
$= 350$ kPa @ 12.0 m

$$N_{kt} = \frac{q_{net}}{S_u(\text{lab})}$$

$= \frac{63}{5} = 12.6$ @ 2.0 m
$= \frac{185}{13.5} = 13.7$ @ 6.0 m
$= \frac{350}{27.7} = 12.6$ @ 12.0 m

$\}$ $N_{kt} \approx 13$

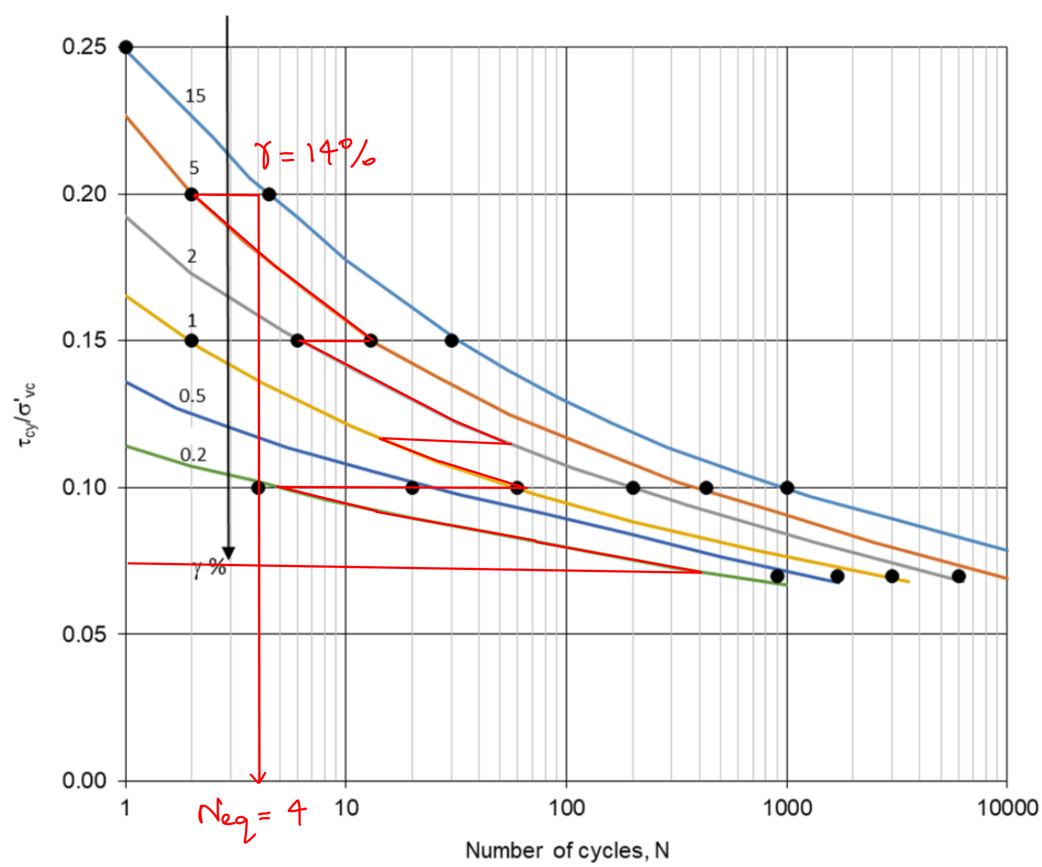
[30%]

c. The cone is not ideal because of uncertainty associated with the value of Nkt and the dependence on the overburden correction, which requires knowledge of the total unit weight. Full flow penetrometers such as the T-bar and ball penetrometer are better suited to measuring the undrained shear strength of soft clay soils because: (i) the N_{T-bar} and N_{ball} factors are derived using plasticity solutions; (ii) the small shaft area relative to the projected penetrometer area results in an overburden stress correction not being required; and (iii) the sensitivity can be readily measured.

[20%]

d.

τ_{cyc}/σ'_{vc} (-)	N (-)	N_{eq} (-)	γ (%)
0.075	400	400	0.2
0.10	60	64	1.0
0.12	40	55	2.0
0.15	6	12	5.0
0.20	4	4	14.0



[30%]

This question was answered by a little over half of the candidates and was performance was consistently good. There were no particular difficulties that commonly arose, with the most common errors generally being arithmetic.

2) a. $\rho_w = 1020\ kg/m^3$; $D = 0.3m$; $C_D = 0.7$; $C_L = 0.9$; $v = 2m/s$

$$F_H = \frac{1}{2}\rho_w D C_D v|v|$$
$$= \frac{1}{2} \times 1020 \times 0.3 \times 0.7 \times 2 \times 2 \times \frac{1}{1000}$$
$$= 0.43\ kN/m$$

$$F_L = \frac{1}{2}\rho_w D C_L v^2$$
$$= \frac{1}{2} \times 1020 \times 0.3 \times 0.9 \times 2 \times 2 \times \frac{1}{1000}$$
$$= 0.55\ kN/m$$

[20%]

b. $\phi_{cs} = 33°$; $I_D = 0.3$; $\gamma' = 10\ kN/m^3$; $w/D = 0.5$; $Q = 10$

Peak friction angle at pipe invert due to in-situ mean stress:

$$k_0 = 1 - \sin\phi_{cs} = 1 - \sin 33 = 0.45$$

$$p' \approx \frac{1 + k_0}{2}\gamma' w = \frac{1 + 0.45}{2} \times 10 \times 0.5 \times 0.3 = 1\ kPa$$

$$I_R = \min\left(I_D\left(Q - \ln p'\right) - 1, 4\right)$$
$$= \min\left(0.3\left(10 - \ln 1\right) - 1, 4\right)$$
$$= 2$$

$$\psi = \frac{5 I_R}{0.8} = \frac{5 \times 2}{8} = 12.5$$

$$\phi_{peak} = \phi_{cs} + 0.8\psi = 33 + 0.8 \times 12.5 = 43°$$

[20%]

c.

$$V_{max} = A\left(\frac{w}{D}\right)^{B} \gamma' D^2 \quad \text{where} \quad \begin{cases} A = C_1\left(e^{\phi_{peak}^{C_2}}\right)^{C_3 \phi_{peak}} \\ B = 1.3067 - 0.0123\,\phi_{peak}. \\ C_i = I_{c,i} + \phi_{cs} S_{c,i} \end{cases}$$

$$C_1 = 1.75 + 33 \times 0.07 = 4.04$$

$$C_2 = 0.6467 + 33 \times 0.0163 = 1.18$$

$$C_3 = 0.0030 + 33 \times -5.97e^{-5} = 1.03 \times 10^{-3}$$

$$\therefore A = 4.04\left(e^{43^{1.18}}\right)^{1.03\times10^{-3} \times 43} = 171.44$$

$$B = 1.3067 - 0.0123 \times 43 = 0.78$$

$$\therefore V_{max} = 171.44\,(0.5)^{0.78} \times 10 \times 0.3^2$$

$$= 89.86 \text{ kN/m}$$

[20%]

d. $SG = 3$; $\gamma_w = \rho_w g = 10$ kN/m^3

$$\frac{\bar{H}}{\bar{V}_{max}} = \mu\left(\frac{\bar{V}}{\bar{V}_{max}} + \beta\right)^{n}\left(1 - \frac{\bar{V}}{\bar{V}_{max}}\right)^{m}$$

where :

$$\bar{V}_{max} = \frac{V_{max}}{\gamma' D^2} = \frac{89.86}{10 \times 0.3^2} = 99.84$$

$$\mu_0 = -0.00437\,\phi_{peak} + 0.42 = -0.00437 \times 43 + 0.42 = 0.23$$

$$\mu = 0.2\frac{w}{D} + \mu_0 = 0.2 \times 0.5 + 0.23 = 0.33$$

$$m = 0.013\,\phi_{peak} + 0.4 = 0.013 \times 43 + 0.4 = 0.959$$

$$n = 0.64$$

$\beta = 0$ ~ no uplift resistance.

$$W' = (SG-1)\left(\frac{\pi D^2}{4}\right)\gamma_w = (3-1) \times \left(\frac{\pi \times 0.3^2}{4}\right) \times 10 = 1.41 \text{ kN/m}$$

$$V = W' - F_L = 1.41 - 0.55 = 0.86 \text{ kN/m}$$

$$\bar{V} = \frac{V}{\gamma' D^2} = \frac{0.86}{10 \times 0.3^2} = 0.88$$

$$\frac{\bar{V}}{\bar{V}_{max}} = \frac{0.88}{99.8} = 8.81 \times 10^{-3}$$

$$\frac{\bar{H}}{\bar{V}_{max}} = 0.33 \left(8.81 \times 10^{-3} + 0\right)^{0.64} \left(1 - 8.81 \times 10^{-3}\right)^{0.959}$$

$$= 0.0158$$

$$\therefore \bar{H} = 0.0158 \times 99.8 = 1.57$$

$$H = \bar{H} \gamma D^2 = 1.57 \times 10 \times 0.3^2 = 1.42 \text{ kN/m} \gg F_H \therefore \text{SAFE}.$$

[30%]

e. The mean stress is almost certainly under-estimated because the cable weight has been ignored. The lift force the cable is subjected to would also influence the operative mean stress at failure, as would the process of the cable loading the sand. Under-estimation of the operative mean stress at failure will result in over-estimation of the peak friction angle. A better estimate would account for cable weight, lift force and loading direction, because the mean stress will vary as a function of all three.

[10%]

This question was answered by most students. Performance was generally good, with the most common mistakes being in the main vertical bearing capacity calculation in the calculation of the C parameters, which require degrees as an input, whereas a few used radians.

QUESTION 3

3(a) *Describe the two critical driveability issues that can be encountered during pile hammering and their consequences.*

Driveability issues include:

- **Refusal**: when the required penetration cannot be reached due to the resistance exceeding the hammer capacity.
- **Tip damage**: if the pile tip is damaged before or during driving then the pile tip may buckle and collapse during driving (Barbour & Erbrich 1995). The respullouting loss of pile shape may reduce the capacity or it may lead to premature refusal, requiring significant remedial works.

Suggested Marking: Refusal = [5%]; Tip damage = [10%] - TOTAL = [15%]

3(b) *For a soil profile of $s_u = 2z$ kPa, where z (m) is the depth below the mudline, explain why the shaft capacity of a pile increases with the square of the pile embedment.*

- If embedment increased by a factor of e.g. 3 ⇒ the mean shaft resistance triples and so does the surface area.
- Capacity = product of mean shaft resistance and surface area ⇒ capacity increases with square of embedment.

Suggested Marking: Line 1 = [5%]; Line 2 = [5%] = [10%]

3(c) *The selected piles have a diameter of $D = 3$ m, are driven to an embedded depth of $L = 30$ m. The site comprises normally-consolidated soft clay, with an undrained strength $s_u = 2z$ kPa, where z is the depth below the mudline in meters. The effective unit weight of the soil is $\gamma' = 7.3$ kN.m^{-3}. Estimate the uplift capacity available at the head of each pile when used to anchor the floating platform, , explaining your calculations and assumptions. Assume that the pile is plugged.*

The pile is plugged ⇒ no shaft resistance on the internal walls of the pile. The pile is in tension loading ⇒ no base resistance contributing to equilibrium.

Uplift capacity given by shaft resistance only:

$$Q_{pullout} = \underbrace{Q_{pullout,s}}_{shaft} + W_{plug}$$

The site mostly consists of normally consolidated clay ⇒ use λ-method to calculate the shaft resistance with:

$$\tau_{sf} = \lambda\left(\sigma'_{0,m} + 2s_{u,m}\right)$$

$$\Rightarrow Q_{pullout,s} = (\pi DL)\lambda\left(\sigma'_{0,m} + 2s_{u,m}\right)$$

with σ'_{0m} is the average effective overburden pressure between the pile head and the pile tip, s_{um} is the average undrained shear strength along the pile shaft, and λ depends on the pile length, read on Figure 1 ⇒ $\lambda \approx 0.15$. The pile is closed-ended ⇒ use a coefficient $K = 1$

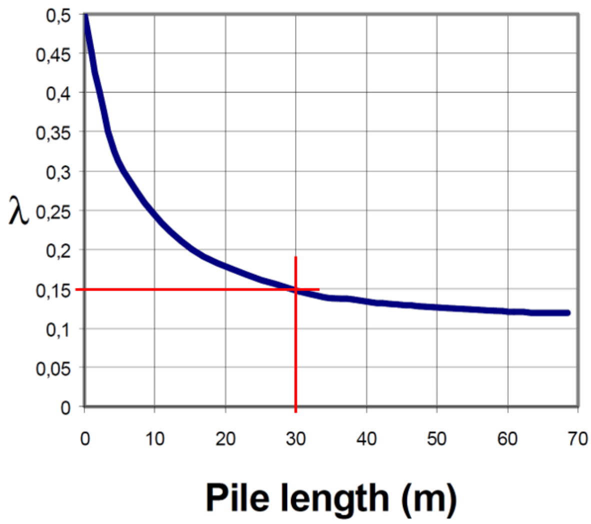


Figure 1: Coefficient λ vs. pile length

Therefore:

$$Q_{pullout,s} = (\pi DL)\lambda\left(\gamma'\frac{L}{2} + 2k_{su}\frac{L}{2}\right) = (\pi \times 3 \times 30) \times 0.15 \times (\frac{30}{2} \times 7.3 + 2 \times \frac{30}{2} \times 1.7)$$

$$Q_{pullout,s} = 7.2MN$$

$$W_{plug} = \gamma' V_{plug} = 7.3 \times \frac{\pi \times 3^2}{4} \times 30 = 1.5kN$$

And therefore:

$$Q_{pullout} = \underbrace{Q_{pullout,s}}_{shaft} + W_{plug} = 8.7MN$$

Note: The α and β methods can also be used here but are less appropriate for normally consolidated clays. Using the alpha or beta method will be accepted, but 5% of the mark will be taken out (for correct calculation).
Also, answers where the calculated capacity include the pile weight and the plug weight will be awarded full mark, as well as if undrained base resistance component is accounted for, if included by the student.

Suggested Marking: Use of lambda method and Read lambda= [5%]; Calculate member 1 = [10%]; Calculate member 2= [10%]; TOTAL = [25%]

3(d) *The hammers considered for installations are the IGC-280 and the IHC-S500, with a maximum rated energy on the pile of 280 kJ and 500 kJ respectively. These both have a blowrate of 42 bl/min. At this location, refusal is encountered when 600 bl/m is reached. Determine which hammer is most suited to install the pile to the required penetration depth and an upper-bound estimate of the installation time needed for installation. Assume a hammer efficiency of* $\eta_H = 0.82$ *for both hammers.*

If pile is driven unplugged: $Q_{ult} \sim 2Q_s = 14.4MN$
If pile driven plugged: $Q_{ult} = Q_s + Q_b - W_{plug} = 9.5MN$
Pile is driven plugged $\therefore$

(i) To calculate Q_{ult} during driving, one need to add the Base resistance:

$$Q_{ult} = \underbrace{Q_{ult,s}}_{shaft} + \underbrace{Q_{ult,b}}_{base}$$

Pile closed-ended $\Rightarrow Q_{ult,b} = N_c s_u(z = L)A_b = N_c k_{su} L \frac{\pi D^2}{4}$

$$Q_{ult,b} = 9 \times 2 \times 30 \times \frac{\pi 3^2}{4} = 3.8MN$$

Therefore:

$$Q_{ult} = 7.189 + 3.817 + 1.548 = 9.457MN = 9.5MN$$

(ii) Number of hammer blows:

$$log(s) = 2.4 - \frac{9561}{104.5\sqrt{0.82 \times E_H}}$$

$\Rightarrow log(s) = -2.07$ for IHC-S500
$\Rightarrow log(s) = -3.57$ for IHC-S280

And therefore:

$s = 10^{-2.12} = 8.52$ mm/bl for IHC-S500 $s = 10^{-3.64} = 0.27$ mm/bl for IHC-S500
This gives 117 bl/m when the pile reaches full penetration for IHC-S500 < 600 bl/m $\Rightarrow$ OK
And 3740 bl/m when the pile reaches full penetration for IHC-S280 > 600 bl/m $\Rightarrow$ Not OK
$\rightarrow$ Select **IHC-S500**

Uper-bound estimate of the installation Time : $t = \frac{17 \times 30}{42} = 84$ minutes
Note: Very likely to be less than that as 131 bl/m is when the pile is at its final embedded depth. The hammer will require much less blows per meter at the beginning of installation as shaft resistance increases with the square of the pile embedment (question b).

Suggested Marking: Recalculate Q_{ult} = [10%]; Calculate s = [5%] each= [10%]; Calculate no. of blows and select hammer = [10%]; Calculate time = [5%] TOTAL = [35%] / Bonus point of [5%] for complete note on installation time.

3(e) *List two counteractive effects caused by cyclic loading on the axial response of the pile.*

Cyclic loadings (including inertial loadings) developed by environmental conditions such as storm waves and earthquakes can have two potentially counteractive effects on the static axial capacity (API 2000):

- Repetitive loadings can cause a temporary or permanent decrease in load-carrying resistance, and/or an accumulation of deformation caused by remoulding of the clay.
- Rapidly applied loadings can cause an increase in load-carrying resistance and/or stiffness of the pile. Very slowly applied loadings can cause a decrease in load-carrying resistance and/or stiffness of the pile.

Suggested Marking: Each point = [7.5%] TOTAL = [15%]

Comments: This question was attempted by most students and relatively well succeeded, with the lowest mark being 1/20 and the highest 19/20. The most common error was to take the pullout capacity for the calculation of the driving force and time, leading to no hammer being suited for driving.

QUESTION 4

4(a) *Prove that it is safe to apply suction to install the caisson.*

Problem geometry:
$D_i = D_e - 2t_w = 3 - 2 \times 0.06 = 2.88$ m

Values for the relevant area required for calculations:

- Cross sectional area of soil plug = Internal cross-sectional area of the foundation:
$A_{plug} = A_i = \frac{\pi D_i^2}{4} = \frac{\pi 2.88^2}{4} = 6.51$ m^2

- Skirt tip bearing area: $A_{tip} = \frac{\pi \left(D_e^2 - D_i^2\right)}{4} = \frac{\pi \left(3^2 - 2.88^2\right)}{4} = 0.55$ m^2

- Internal skirt wall surface area: $A_{si} = \pi D_i L = \pi \times 2.88 \times 12 = 109$ m^2

- External skirt wall surface area: $A_{se} = \pi D_e L = \pi \times 3 \times 12 = 113$ m^2

- Skirt (internal + external) wall surface area: $A_s = \pi L(D_e + D_i) = \pi \times 25 \times (3 + 2.88) = 222$ m^2

(i) Calculation of the FoS on plug stability:

All the equations are in the data book and in the lecture notes

Installation resistance:
$Q = (N_c s_u + \gamma' z)A_{tip} + \alpha \bar{s}_u A_s = (7.5 \times 16 + 6.3 \times 12) \times 0.55 + 0.3 \times 3 \times 222 = 1172$ kN

Required suction:
$Q - W' = 1172 - 720 = 452$ kN

Allowable suction:
$(N_c s_u A_i) + (\alpha \bar{s}_u A_{si}) = (9 \times 16 \times 6.51) + (0.3 \times 3 \times 108) = 1459$ kN

Factor of safety against plug failure:

$$FoS = \frac{1459}{452} = 3.2$$

∴Safe to apply suction required for installation.

Suggested Marking: Q=[5%]; Required suction=[5%]; Allowable suction=[10%]; FoS=[5%]; TOTAL = [25%]

4(b) *Describe the possible uplift failure mechanisms of a suction caisson. Draw a diagram indicating the components that contribute to capacity for each of them.*

Reverse end bearing relies on passive suction and can be relied upon if the top cap is sealed. If passive suction can be mobilised, axial pullout resistance V_{ult} consists of outer skirt friction and reverse end bearing.

If the top plate is not sealed, or sustained load is applied, reverse end bearing will either not be relevant, or will be reduced in value. In the extreme, the vertical capacity V_{ult} consists of outer skirt friction plus the lesser of inner skirt friction and the soil plug self weight, in addition to the caisson submerged weight.

Components of undrained uplift resistance of a suction anchor = Caisson (buoyant) weight + shaft resistance + upward (or reverse end bearing) resistance

Figure 2 illustrates the various components of pullout resistance for sealed and vented conditions.

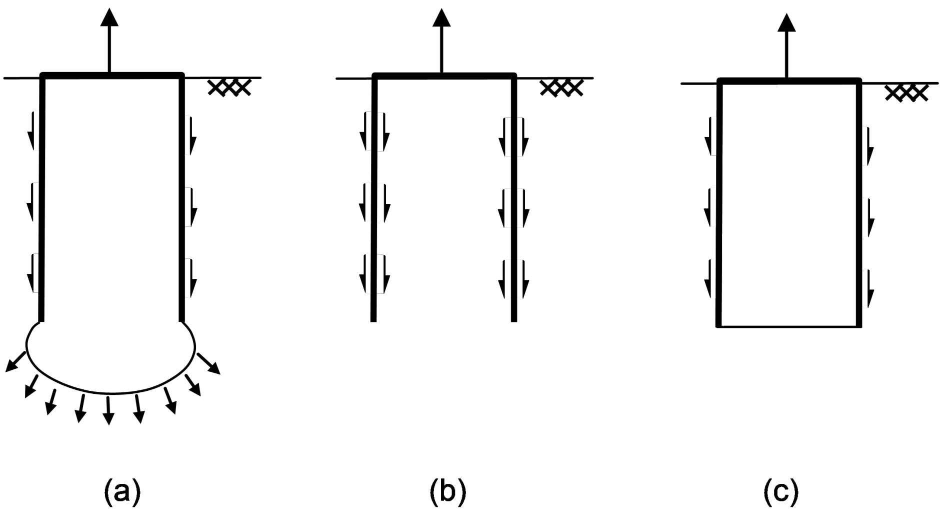


Figure 2: Failure models for vertical pullout resistance (a) reverse end bearing, i.e. with passive suction and (b & c) without passive suction (b) caisson pull out and (c) caisson and plug pull out

Suggested Marking: for each mode = [6.7%] ; description = [3.7%]; diagram = [3%]; TOTAL = [20%]

4(c) *Evaluate the uplift capacity of each caisson during operation and conclude on the mode of failure expected at this site.*

From data book, three modes of failures possible:

(a) Sealed: $V_{ult} = W' + A_{se}\alpha_e \overline{s}_{u(t)} + N_c s_u A_{plug}$
(b) Vented: $V_{ult} = W' + A_{se}\alpha_e \overline{s}_{u(t)} + A_{si}\alpha_i \overline{s}_{u(t)}$
(c) Plug break-away: $V_{ult} = W' + A_{se}\alpha_e \overline{s}_{u(t)} + W'_{plug}$

Check all three and use the values of area calculated in Question 10:

(a) **Sealed**:
$V_{ult,sealed} = W' + A_{se}\alpha_e \overline{s}_{u(t)} + N_c s_u A_{plug} = 720 + 113 \times 0.3 \times 16 + 9 \times 16 \times 7.07$
$V_{ult,sealed} = 2281$ kN

(b) **Vented**:
$V_{ult,vented} = W' + A_{se}\alpha_e \overline{s}_{u(t)} + A_{si}\alpha_i \overline{s}_{u(t)} = 720 + 113 \times 0.3 \times 16 + 109 \times 0.3 \times 16$

$V_{ult,vented} = 1784$ kN

(c) **Plug break-away**:

$$V_{ult,breakaway} = W' + A_{se}\alpha_e\bar{s}_{u(t)} + W'_{plug} = W' + A_{se}\alpha_e\bar{s}_{u(t)} + \gamma' L\frac{\pi D^2}{4}$$

$$V_{ult,breakaway} = 720 + 113 \times 0.3 \times 16 + 6.3 \times 12 \times \frac{\pi \times 2.88^2}{4}$$

$$V_{ult,breakaway} = 1755 \text{ kN}$$

Conclusion: $V_{ult,sealed} > V_{ult,vented} > V_{ult,breakaway} \Rightarrow$ The critical mode is (c):
$\therefore$Plug break-away failure. The caisson will fail without passive suction, with caisson and plug pull out.

Suggested Marking: each mode calc = [12%] each; comparison = [4%]; TOTAL = [40%]

4(d) *Why can you neglect the overburden term in the calculation of uplift resistance with passive suction?*

The overburden from the soil column outside the caisson above tip level, and the weight of the soil plug within the caisson are equal and opposite and so their effects cancel out.

TOTAL = [10%]

4(e) *How would scour affect the results obtained in question 4.c?*

- External skirt wall surface area now becomes smaller: $A_{se,scour} = \pi D_e L = \pi \times 3 \times L_{scour} < 113$ m^2 $\Rightarrow$ reduces capacity of the caisson.

This does not change the height of the core/plug that has formed inside the caisson during installation. In the case of uplift resistance with passive suction, the overburden from the soil column outside the caisson above tip level, and the weight of the soil plug within the caisson are no longer equal and opposite and so their effects need to be accounted for.

Reduction of capacity = [2%]; Change on uplift resistance / plug/core = [3%] - TOTAL = [5%]

Comments: This question was well attempted by the students, and very well succeeded. Marks ranged from 9/20 to 20/20 with the most common mistakes arising from not choosing the correct surface areas for the calculations.Question (d) was only succeeded by about half the cohort.