Q.1.(a)

Total cone resistance, q_t:

$$
\begin{aligned}
q_t &= q_c + (1-\alpha)\,u_2 \\
&= 1525 + (1-0.7)\cdot 720 \\
&= 1741 \text{ kPa}
\end{aligned}
$$

Net cone resistance, q_{net}:

$$
\begin{aligned}
q_{net} &= q_t - \sigma_{v0} \\
&= q_t - \gamma \cdot z \\
&= 1741 - 16\cdot 50 \\
&= 1741 - 800 \\
&= 941 \text{ kPa}
\end{aligned}
$$

Normalised cone resistance, Q:

$$
\begin{aligned}
Q &= \frac{q_{net}}{\sigma'_{v0}} \\
&= \frac{q_{net}}{\gamma' \cdot z} \\
&= \frac{941}{6\cdot 50} \\
&= \frac{941}{300} \\
&= 3.2
\end{aligned}
$$

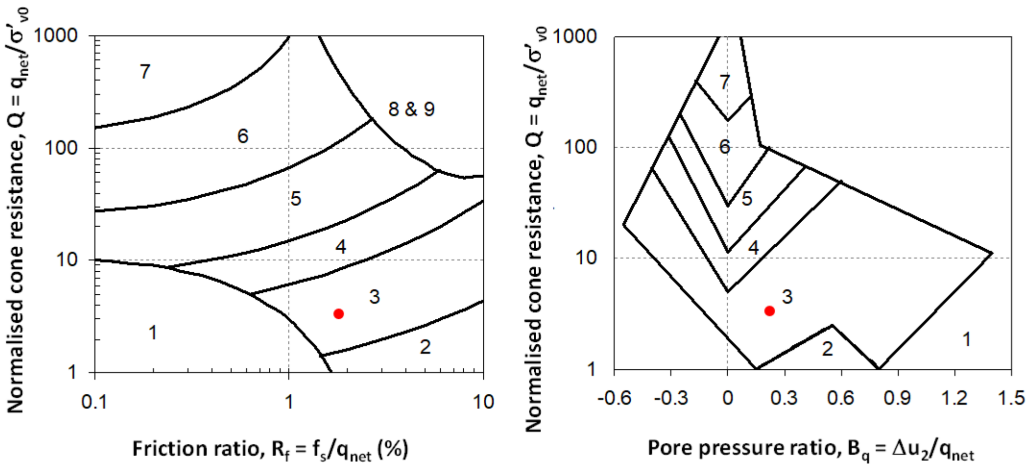
Normalised friction ratio, R_f:

$$
\begin{aligned}
R_f &= \frac{f_s}{q_{net}}\cdot 100 \\
&= \frac{18}{941}\cdot 100 \\
&= 1.9\ \%
\end{aligned}
$$

Pore pressure ratio, B_q:

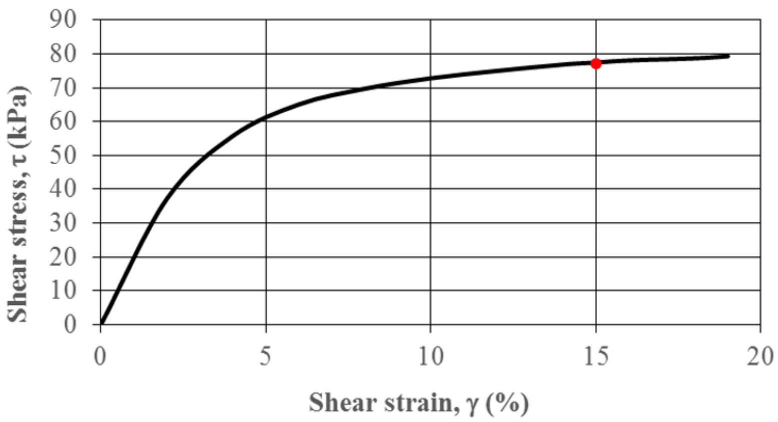
$$
\begin{aligned}
B_q &= \frac{u_2 - u_0}{q_{net}} \\
&= \frac{720 - 10\cdot 50}{941} \\
&= 0.23
\end{aligned}
$$

Pick off appropriate zone from Robertson chart, giving 'Zone 3: Clayey clays to silty clays':



Q.1.(b)(i)

Failure taken to have occurred at $\gamma = 15$ % (but could be 5-15 %, if adequately justified), thus $\tau_f = s_u =$ 78 kPa:



Therefore, the cone factor, N_{kt}, is calculated as follows:

$$
\begin{aligned}
N_{kt} &= \frac{q_{net}}{s_u} \\
&= \frac{941}{78} \\
&= 12
\end{aligned}
$$

Q.1.(b)(ii)

The test should be performed on a sample consolidated to an effective stress equal to the in situ vertical effective stress:

$$
\begin{aligned}
\sigma'_{v0} &= \gamma' \cdot z \\
&= (\gamma - \gamma_w) \cdot z \\
&= (16 - 10) \cdot 50 \\
&= 6 \cdot 50 \\
&= 300 \text{ kPa}
\end{aligned}
$$

This is so that the sample is tested in the same stress state as it was found in situ, thus the mechanical response will be as representative of the field conditions as possible.

Q.1.(c)(i)

The undrained strength ratio is simply the ratio of penetration resistance for the remoulded steady-state, q_{rem}, and the first pass, q_{in}, because the T-bar factors cancel, therefore:

$$\frac{s_{u-rem}}{s_{u-in}} = \frac{q_{rem}}{q_{in}} = \frac{130}{805} = 0.16$$

Thus the remoulded undrained strength is only 16% of the intact strength - in other words the undrained soil strength reduces by a factor of more than 6 during significant undrained loading events.

Q.1.(c)(ii)

The T-bar factor, N_{T-bar}, can be taken as any value between 9 and 12, if justified adequately. Typically 10.5 is chosen, since that is the value for intermediate roughness conditions from plasticity solutions derived using the strain path method, thus:

$$\begin{aligned} s_u &= \frac{q_{in}}{N_{T-bar}} \\ &= \frac{805}{10.5} \\ &= 76.7 \text{ kPa} \end{aligned}$$

This compares extremely well with the cone penetrometer / simple shear test derived measurement.

Q.1.(d)

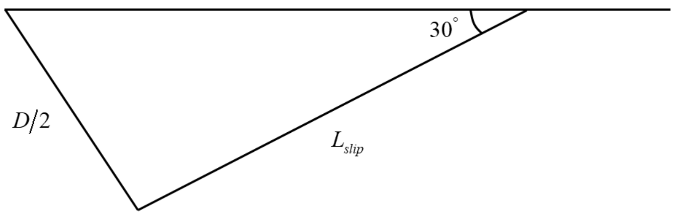
Three key benefits of full-flow penetrometers:

1. No need for overburden correction;
2. Can cycle up and down to measure remoulded undrained strength; and
3. Larger projected area, thus better for measuring low undrained strengths in soft normally-consolidated seabeds.

Comments: This question was generally reasonably well-answered by most candidates. The early parts of the question relating to CPT interpretation were almost always answered correctly. The part that caused the most confusion was surprisingly related to T-bar remoulded strength ratio determination, as many students conflated sensitivity and remoulded strength ratio (one being the reciprocal of the other).

Q.2.(a)

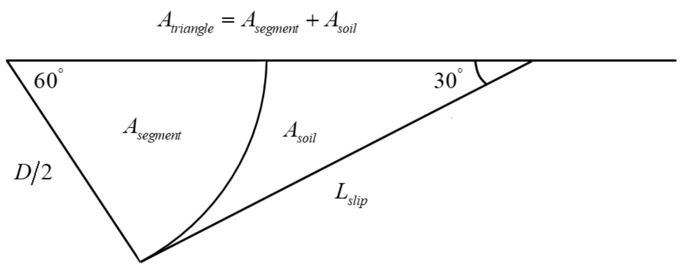
Geometry of slip surface in assumed mechanism (right angled triangle):



$$
\begin{aligned}
L_{slip} &= \frac{D}{2tan\theta_{slip}} \\
&= \frac{1}{2tan30} \\
&= \frac{\sqrt{3}D}{2} \\
&= 0.346 \text{ m}
\end{aligned}
$$

Q.2.(b)

Split triangle by subtracting a segment of the cable in order to isolate the soil area encapsulated by the mechanism:



Area of segment of cable:

$$
\begin{aligned}
A_{segment} &= \frac{\pi D^2}{4}\frac{\theta_{segment}}{360} \\
&= \frac{\pi 0.4^2}{4}\frac{60}{360} \\
&= 0.0209 \text{ m}^2
\end{aligned}
$$

Area of triangle encapsulating the segment of the cable and soil within the failure mechanism:

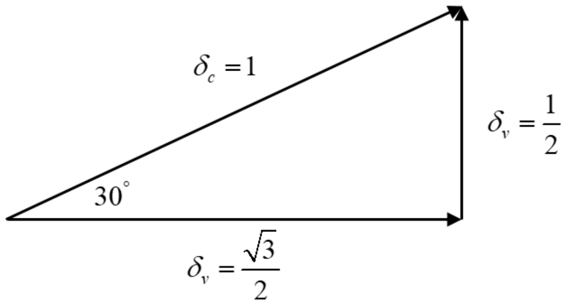
$$
\begin{aligned}
A_{triangle} &= \frac{1}{2}\frac{D}{2}L_{slip} \\
&= \frac{1}{2}\frac{0.4}{2}0.346 \\
&= 0.0346 \text{ m}^2
\end{aligned}
$$

Area of soil involved in the mechanism:

$$
\begin{aligned}
A_{soil} &= A_{triangle} - A_{segment} \\
&= 0.0346 - 0.0209 \\
&= 0.0137 \text{ m}^2
\end{aligned}
$$

Q.2.(c)

Displacement hodograph where δ_c is the displacement of the cable:



Work equation:

$$
\underbrace{F_L \cdot \delta_h - W' \cdot \delta_v}_{\substack{\text{Work input from external forces (cable} \\ \text{self-weight and lateral breakout force)}}} = \underbrace{\gamma' \cdot A_{soil} \cdot \delta v}_{\substack{\text{Potential energy of soil lifted by} \\ \text{the assumed mechanism}}} + \underbrace{s_u \cdot L_{slip} \cdot \delta_c}_{\substack{\text{Work dissipated in shear along the slip} \\ \text{plane in the assumed mechanism}}}
$$

Therefore:

$$
\begin{aligned}
F_L &= \frac{2}{\sqrt{3}} \left[\frac{W'}{2} + \frac{\gamma' \cdot A_{soil}}{2} + \frac{s_u \cdot \sqrt{3} \cdot D}{2} \right] \\
&= \frac{1}{\sqrt{3}} \left[W' + \gamma' \cdot A_{soil} + s_u \cdot \sqrt{3} \cdot D \right]
\end{aligned}
$$

Q.2.(d)

Substitute in numerical values from the Table:

$$
\begin{aligned}
\frac{F_L}{W'} &= \frac{1}{\sqrt{3}} \left[1 + \frac{\gamma' \cdot A_{soil}}{W'} + \frac{s_u \cdot \sqrt{3} \cdot D}{W'} \right] \\
&= \frac{1}{\sqrt{3}} \left[1 + \frac{6 \cdot 0.0137}{1} + \frac{1 \cdot \sqrt{3} \cdot 0.4}{1} \right] \\
&= 0.577 \, [1 + 0.082 + 0.69] \\
&= 1.022
\end{aligned}
$$

Comments: This question was less popular than the others, but those that answered it generally did a very good job. The main difficulty was in determining the work equation in part (c), with a handful of candidates seemingly forgetting that work done is force multiplied by displacement! In a number of instances the displacement component of the work done was inexplicably ignored.

QUESTION 3

3(a) *Calculate the ultimate lateral soil resistance of the pile* p_{u1} *and* p_{u2} *at depths* $z_1 = 0.1L$ *and* $z_2 = 0.8L$ *respectively, assuming an idealised linear increase of the soil lateral resistance with depth.*

Data Book; Lateral capacity of piles: linearly increasing lateral resistance with depth

$$p_u = \gamma' K_p^2 z D \tag{1}$$

with:

$$K_p = \frac{1 + sin(\phi')}{1 - sin(\phi')} = 3.25 \tag{2}$$

And therefore:

$$p_{u1} = 20 \times 3.25^2 \times 0.1 \times 35 \times 8.75 = 6.5 MN/m \tag{3}$$

And:

$$p_{u2} = 20 \times 3.25^2 \times 0.8 \times 35 \times 8.75 = 52 MN/m \tag{4}$$

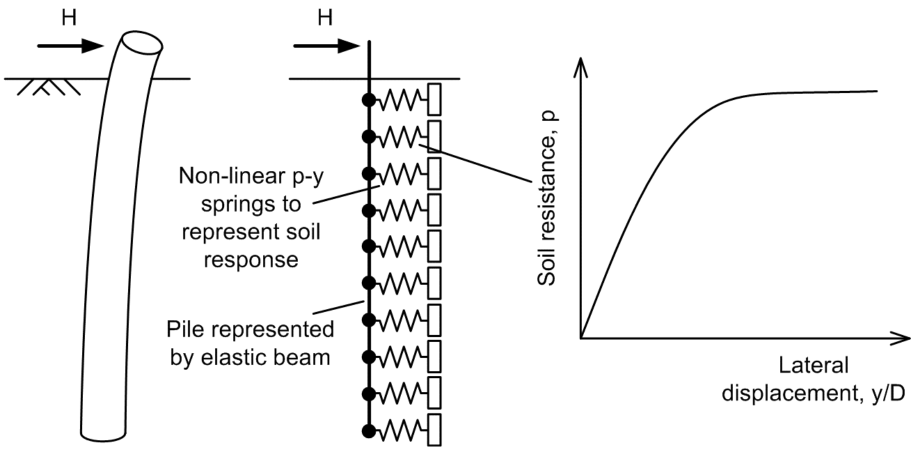
Suggested Marking: K_p = [5%]; $Pu1, 2$ = [5%] - TOTAL = [10%]

3(b) *Compare the p-y method and the 1D PISA method and discuss their application to the design of offshore wind monopiles.*

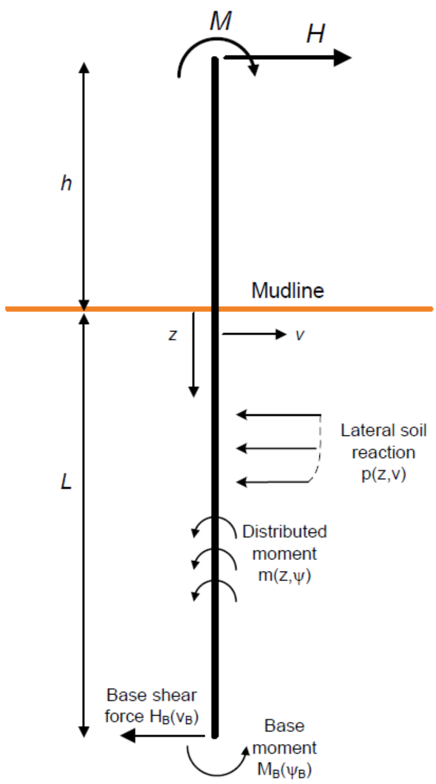
The soil lateral resistance in the *p-y* method is captured through one unique component: the lateral soil reaction p as a function of pile displacement y. The relationship between the two is cpatured through an empirical function (hyperbolic tangent for sand) that was originally calibrated for piles of aspect ratio L/D of 30 to 100, i.e. long piles. However, monopiles used for offshore wind turbine foundations are usually short rigid piles of aspect ratio L/D of 4 to 6.

In comparison, the 1D PISA method includes 4 components of the soil lateral resistance: (1) The distributed lateral load p as a function of pile displacement y, similar to the *p-y* method; (2) The distributed moment to capture shear stress on side of the pile; (3-4) The base shear and base moment to capture pile base resistance to lateral loading. There are three additional components in comparison to the *p-y* method, which makes the model more precise to capture short rigid pile behaviour.

In addition, the model calibrated on a data base relevant to soil profiles in the North Sea and pile aspect ratio L/D of 4 to 6, with dimensiosn relevant to piles currently being installed in the North Sea.



(a)



(b)

Figure 1: Comparison between *p-y* method and 1D PISA method for short rigid piles

Suggested Marking: drawing *p-y* curves = [5%]; drawing PISA = [5%]; discussion on additional components and their function with regards to the soil resistance = [10%]; discussion on calibration = [10%] - TOTAL = [30%]

3(c) *Using the DNV-OS-J101/API method recommended for $p - y$ curves in sand, calculate the static and cyclic lateral resistance of the sand at depths $z_1 = 0.1L$ and $z_2 = 0.8L$ for a pile ground level displacement of $v_G = 0.1D$. Assume that the pile failed by pure rigid body rotation, with pivot point located at a depth of $z_{rot} = 0.7L$ to deduce the value of the pile displacement at depth.*

Use Data book: DNV-OS-J101/API method for laterally loaded piles

Lateral resistance at depth:

$$p(x, y) = Ap_u \cdot tanh\left(\frac{k_y.z}{Ap_u}y\right) \tag{5}$$

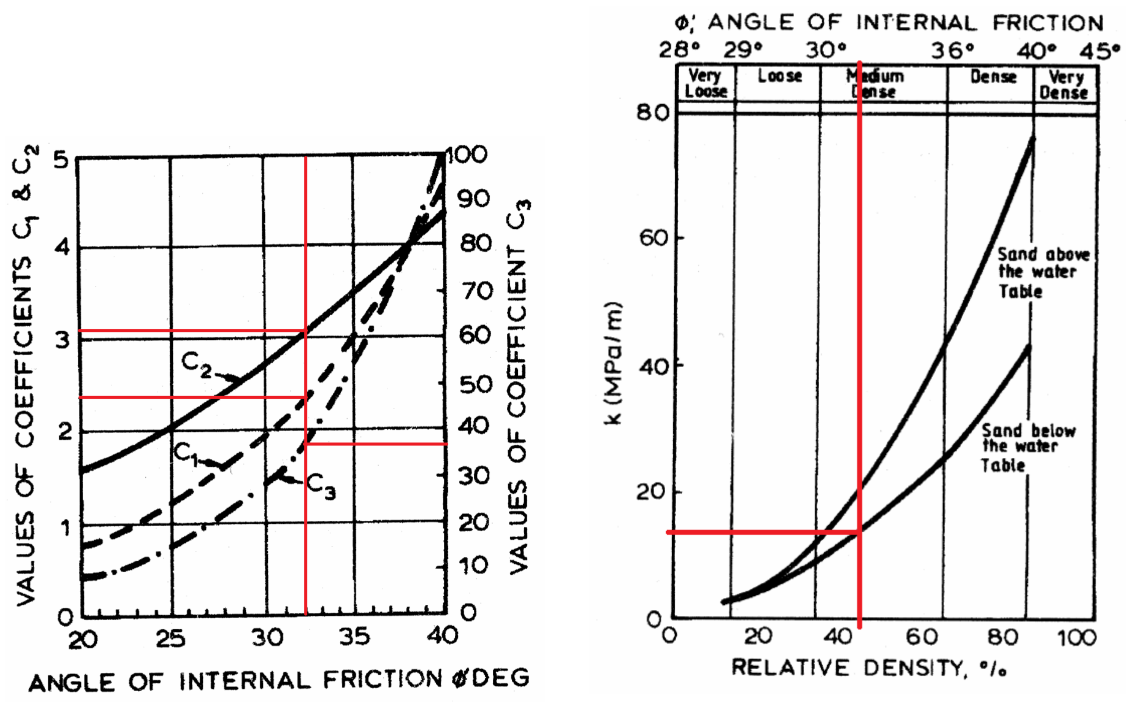
Coefficient A

$$\begin{cases} A_{static} = max\left(0.9, \left(3 - 0.8\frac{z}{D}\right)\right) \\ A_{cyclic} = 0.9 \end{cases} \tag{6}$$

Ultimate soil resistance:

$$p_u = min\left((C_1 z + C_2 D)\gamma' z; C_3 D\gamma' z\right) \tag{7}$$

Where k_y is the initial modulus of subgrade reaction, also called the soil spring constant, and is a function of the angle of friction ϕ' and can be found from the graph in the data book (Figure 2(b)). C_1, C_2 and C_3 are three empirical factors given by Figure 2(a).



(a)

(b)

Figure 2

And therefore: Assuming that the pile is rigid, at $v_G = 0.1D$, we have (Fig. **??**):

$$y_1 = v_G\frac{0.6}{0.7} = 0.75m \tag{8}$$

$$y_2 = v_G\frac{0.1}{0.7} = 0.125m \tag{9}$$

Reading from Figure 2(a), we have: $C_1 = 2.4$; $C_2 = 3.1$; $C_3 = 37$ and $k_y \sim 14MPa/m$.
Using Equation 7, this gives:

$$p_{u,1} = min(2.5, 2.5) = 2.5MN/m \tag{10}$$

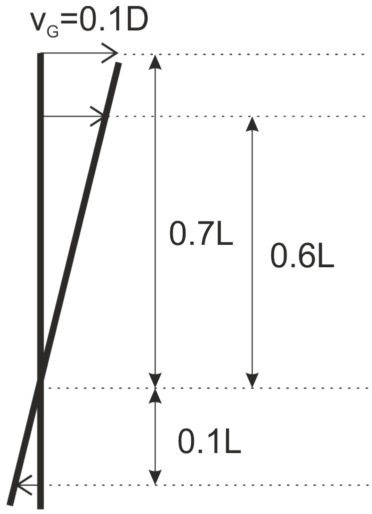


Figure 3

and:

$$p_{u,2} = min(52.8, 181.3) = 52.8 MN/m \quad (11)$$

Because of reading on the curves, values between the following brackets are accepted:
$p_{u,1} = 2.2 - 2.6 MN/m$; $p_{u,2} = 49.2 - 55.4 MN/m$.

Using Equation 6, we have:

$$A_{static,1} = max(0.9, 2.68) = 2.68 \quad (12)$$

$$A_{cyclic,1} = 0.9 \quad (13)$$

And:

$$A_{static,2} = max(0.9, 0.44) = 0.9 = A_{cyclic,2} \quad (14)$$

$$A_{cyclic,2} = 0.9 \quad (15)$$

And therefore, using Equation 5, we have:
$p_{static,1} = 6.7 MN/m$; $p_{cyclic,1} = 2.2 MN/m$
$p_{static,2} = 36.8 MN/m$; $p_{cyclic,2} = 36.8 MN/m$

Values between the following brackets are accepted:
$p_{static,1} = 6.3 - 7.1 MN/m$; $p_{cyclic,1} = 2.1 - 2.4 MN/m$
$p_{static,2} = 35.5 MN/m$; $p_{cyclic,2} = 35.5 - 37.6 MN/m$

Suggested Marking: Use graphs to get $C_{1,2,3}$ and k_y = [10%]; Calculate $y_{1,2}$ = [10%]; Calculate $A_{static/cyclic,1,2}$ = [10%]; Calculate $p_{u,1,2}$ = [10%]; Calculate $p_{static/cyclic,1,2}$ = [10%] = [50%]

3(d) *Comment on the expression you used to calculate the cyclic response.*

When considering cyclic loading, at shallow depths, the value of A is fixed to 0.9, regardless of the **number of cycles**, loading conditions, cyclic load history, cyclic soil properties or frequency of the load (inertial effects). The response is therefore the same whether a short storm load history of 10 cycles or the entire wind turbine lifetime of 3×10^8 cycles are applied on the pile.
Suggested Marking: TOTAL = [10%]

Comments: This question was attempted by most students and was reasonably well-answered. Most candidates used the correct method and provided correct calculations for both questions (a) and (d). The most common mistake was an error in the reading of the diagram for the value of C_3. The main difficulty was with question (b), and a lack of understanding of the two key differences between PISA and the p-y curves. Most candidates described the methods but did not explain how they compare.

QUESTION 4

4(a) *Calculate the ultimate holding capacity of the anchor system at the mudline, the anchor system efficiency and the final depth of the anchor fluke.*

From the data book, the drag anchor solution for the geotechnical resisting force acting on the anchor parallel to the direction of travel:

$$T_p = fA_pN_cs_u = 1.2 \times 16 \times 9 \times 15 = 2,592kN \tag{16}$$

And:

$$W = 40 \times 9.81 = 392kN \tag{17}$$

And therefore:

$$W' = W\left(\frac{\rho_s - \rho_w}{\rho_s}\right) = 392\left(\frac{7,850 - 1,000}{7,850}\right) = 342kN \tag{18}$$

$30° = 0.52$ radians. And therefore, using the formulae in the Data book:

$$\theta'_w = tan^{-1}\left(\frac{W' + T_p tan\theta_w}{T_p}\right) = tan^{-1}\left(\frac{342 + 2,592tan0.52}{2,592}\right) = 0.62 \text{ radians} \tag{19}$$

And:

$$T_a = \frac{T_p}{cos\theta'_w} = \frac{2,592}{cos0.62} = 3,178kN \tag{20}$$

From the chain solution in the data book:$b = 2.5d = 0.375m$ And:

$$z_aQ_{av} = bN_c\int_0^{z_a} s_u dz \tag{21}$$

Hence:

$$Q_{av} = bN_cs_u = 0.375 \times 7.5 \times 15 = 42kN \tag{22}$$

And:

$$\frac{T_a}{2}\left(\theta_a^2 - \theta_m^2\right) = z_aQ_{av} \tag{23}$$

Therefore:

$$z_a = \frac{T_a}{2Q_{av}}\left(\theta_a^2 - \theta_m^2\right) = \frac{3,178}{2 \times 42}\left(0.62^2 - 0^2\right) = 14.5m \tag{24}$$

This leads to

$$z_f = z_a + 6 = 20.5m \tag{25}$$

Finally, the system capacity is:

$$T_m = e^{\mu(\theta_a - \theta_m)}T_a = e^{0.3(0.62-0)}3,178 = 3,824kN \tag{26}$$

Leading to an efficiency of:

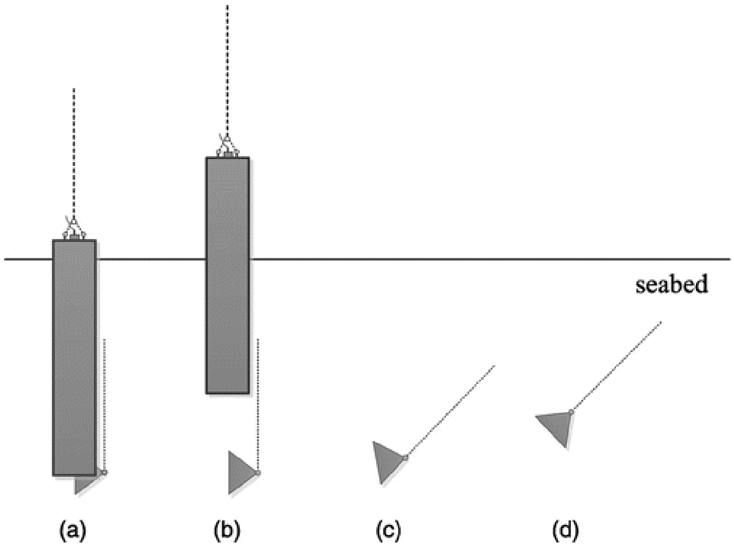
$$\eta = \frac{T_m}{W} = \frac{3,824}{392} = 9.8 \tag{27}$$

Suggested Marking: T_p=[5%]; T_p=[10%]; θ'_w=[5%]; T_a=[5%]; Q_av=[10%]; T_a=[10%]; z_f=[5%]; T_m=[5%]; η=[5%]; TOTAL = [60%]

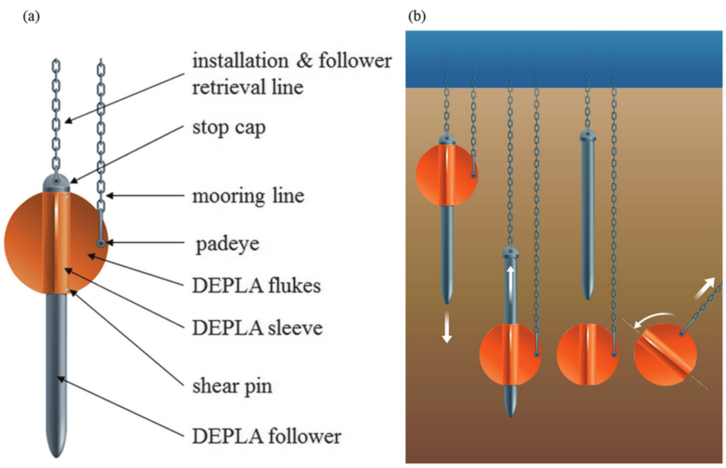
4(b) *Describe other anchor options with accompanying sketches. Describe their installation methods and summarise potential advantages and disadvantages, relative to fixed-fluke anchors.*

The 4 anchor systems to list are:

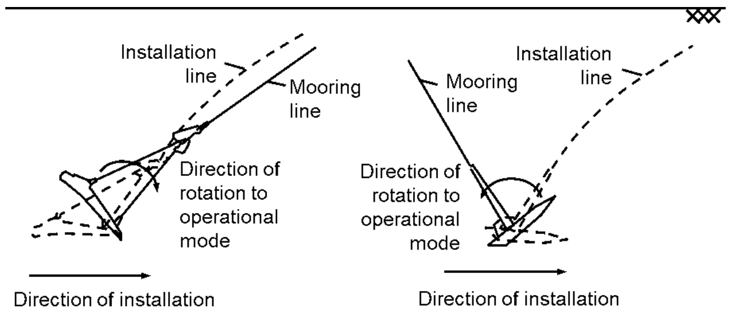
1. Suction embedded plate anchor (or SEPLA). Installed via a caisson penetration after which the caisson is removed leaving the plate anchor in place. This is then keyed during initial loading as the anchor rotates toward the direction of applied loading. Diagram should be similar to this with labels for top marks:



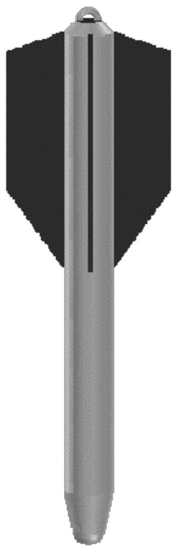
2. Dynamically embedded plate anchor. Plate anchor as for the SEPLA except the anchor is installed using Earth's gravity and a removable follower. Diagram should be similar to this with the major components labelled for top marks:



3. Vertically loaded anchor (or VLA). Similar to the drag anchor but with a camming mechanism that orients the plate anchor perpendicular to the direction of loading. The diagram should be similar to either the left or right hand side of the diagram below with the major components labelled for top marks:



4. Torpedo anchor (or drop anchor). Similar to the DEPLA but with a fixed follower. Accompanying sketch to look similar to this with major components labelled for top marks:



Suggested Marking: drawings = [5%] each; comparison = [5%]; TOTAL = [40%]

Comments: This question was less popular than the others, but those who answered it generally did well, in particular with question (a). The main difficulty with this question was to use the correct method and not make algebraic mistakes. Question (b) was answered relatively well, with the most common mistake being to list the different anchors but not detailing the advantages and disadvantages.