EGT3
ENGINEERING TRIPOS PART IIB

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11 May 2026 09:30 to 11.10

Module 4D9

OFFSHORE GEOTECHNICAL ENGINEERING

*Answer not more than **three** questions.*

All questions carry the same number of marks.

*The **approximate** percentage of marks allocated to each part of a question is indicated in the right margin.*

*Write your candidate number **not** your name on the cover sheet and at the top of each answer sheet.*

STATIONERY REQUIREMENTS
Write on single-sided paper.

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
Engineering Data Book
CUED approved calculator allowed
Attachment: 4D9 Offshore Geotechnical Engineering Data Book (20 pages)

10 minutes reading time is allowed for this paper at the start of the exam.

You may not start to read the questions printed on the subsequent pages of this question paper until instructed to do so.

You may not remove any stationery from the Examination Room.

1 You are a geotechnical engineer working for a site investigation contractor. For a project aiming to develop a new wind farm in the North Sea, you have the following tasks.

(a) Cone penetrometer tests have been performed at the proposed site. The cone outer diameter D = 35.7 mm and the shaft diameter D_s = 28.0 mm. Estimate the cone area ratio from the geometry. [10%]

(b) Describe an alternative method to obtain the cone area ratio for the cone. [10%]

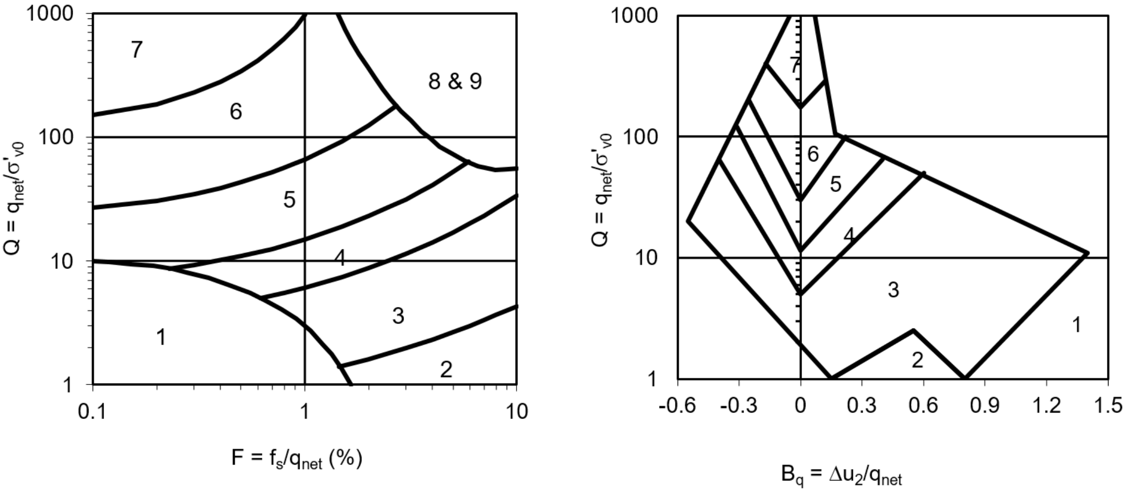
(c) Measured data from one of the cone penetrometer tests is given in Table 1. Use the Robertson charts given in Fig. 1 to indicate the type of sediment at the cone penetrometer test location. [40%]

(d) Describe any additional tests that would be necessary to estimate the appropriate cone factor N_{kt} required to analyse the cone penetration data and derive an undrained shear strength. [10%]

(e) Describe three geohazards likely to be of concern for an export cable from the wind farm back to shore given the North Sea location. [30%]

Table 1

z [m]	q_c $\left[\text{kN m}^{-2}\right]$	f_s $\left[\text{kN m}^{-2}\right]$	u_2 $\left[\text{kN m}^{-2}\right]$	γ $\left[\text{kN m}^{-3}\right]$
20	18000	120	210	19



Zone	Soil behaviour type
1.	Sensitive, fine grained
2.	Organic soils, peats
3.	Clayey clays to silty clay
4.	Silt mixtures, clayey silt to silty clay
5.	Sand mixtures, silty sand to sandy silt
6.	Sands, clean sand to silty sands
7.	Gravelly sand to sand
8.	Very stiff sand to clayey sand
9.	Very stiff fine grained

Fig. 1

2 A steel trunkline pipeline with outer diameter $D = 1.0$ m and specific gravity $SG = 1.7$ is installed on the seabed in soft clay. The clay has a uniform undrained shear strength $s_u = 2$ kPa and effective unit weight $\gamma' = 6$ kN m^{-3}. During installation, the pipeline is embedded to a depth $w = 0.3$ m. The pipe-soil interface may be assumed to be of intermediate roughness.

(a) Calculate the overload ratio required to achieve the observed embedment of 0.3 m. [35%]

(b) Calculate the ultimate lateral breakout force. [35%]

(c) Describe the laying process, including vessel types and any other issues associated with pipe-laying. [30%]

3 A 35 tonne dry weight fixed-fluke drag anchor is being designed as a catenary mooring for an FPSO vessel in 800 m of water. The anchor has a projected fluke area A_p = 14 m^2, form factor f = 1.25, fluke angle θ_w = 28°, and vertical offset from padeye to fluke centroid of 5 m. Assume a bearing capacity factor N_c = 9 for the anchor and a density of steel ρ_s = 7850 kg m^{-3}. The anchor is connected using a 0.12 m diameter studless chain. Assume the effective width of the chain is 2.5 times the bar diameter, the bearing capacity factor for the chain N_c = 7.5, and the soil-chain friction coefficient μ = 0.4. The seabed comprises soft normally consolidated clay with undrained shear strength:

$$s_u = 1.5z \text{ kPa}$$

where z is the depth below mudline in metres.

(a) Accounting for the anchor weight, chain bearing resistance, and soil-chain friction, show that the ultimate holding capacity at the mudline may be expressed as:

$$T_m = \frac{f A_p N_c k z_f}{\cos\theta'_w} \exp\left[\mu(z_f - d)\sqrt{\frac{b N_{c,chain}\cos\theta'_w}{f A_p N_c z_f}}\right]$$

where k is the shear strength gradient, d is the vertical offset from padeye to fluke centroid, z_f is the final embedment depth, and θ'_w is the adjusted fluke angle accounting for anchor weight. State any assumptions made. [25%]

(b) For a fluke embedment depth z_f = 8 m, calculate the ultimate holding capacity at the mudline T_m and the performance ratio $\eta = T_m/W$, where W is the dry weight. Comment on the result and discuss the implications of three key assumptions inherent in the analysis. [45%]

(c) Describe two alternative anchor types suitable for these soil conditions: a suction caisson and a suction embedded plate anchor (SEPLA). For each, discuss advantages and disadvantages relative to drag-embedded anchors. [30%]

4 A torpedo anchor is being designed for a floating production unit in 1500 m water depth offshore Brazil. The anchor has a dry mass $m = 60$ tonnes, length $L = 12$ m, and external diameter $D = 1.0$ m. The anchor is fitted with four longitudinal fins for directional stability. Assume the density of steel $\rho_s = 7850$ kg m^{-3} and the density of seawater $\rho_w = 1025$ kg m^{-3}. The seabed comprises soft normally consolidated clay with undrained shear strength:

$$s_u = 2z \text{ kPa}$$

where z is the depth below mudline in metres. Take the shaft friction coefficient $\alpha = 0.4$ and bearing capacity factor $N_c = 12$ for deep embedment. Assume a drag coefficient $C_d = 0.24$ for inertial drag during penetration.

(a) Calculate the terminal velocity of the anchor falling through water. [20%]

(b) The anchor is released from 30 m above the seabed and impacts at a velocity $v_{impact} = 20$ m s^{-1}. Calculate the final embedment depth z_{final}. [35%]

(c) Calculate the axial holding capacity at the final embedment depth and hence the holding capacity ratio $\eta = Q/W$, where W is the dry weight. [30%]

(d) Discuss three key factors that introduce uncertainty into the predicted holding capacity. [15%]

END OF PAPER

Module 4D9: Offshore Geotechnical Engineering — Supplementary Data Book —

This supplementary data books contains relationships and associated data that you are not expected to remember, but you will be expected to understand what the parameters and relationships represent and how to apply them in analysis.

1 Offshore environment

1.1 Slope failure

Factor of safety against infinite slope failure:

$$F = \frac{\tau_{ult}}{\tau_{mob}}$$

Undrained strength ratio:

$$k = s_u/\sigma'_v$$

Factor of safety for undrained infinite slope failure:

$$F = \frac{2k}{\sin 2\alpha}$$

Critical slope angle for undrained conditions:

$$\alpha_{ult} = 0.5 \arcsin(2k)$$

Factor of safety for drained conditions:

$$F = \frac{\tan \phi_{cr}}{\tan \alpha}$$

Critical slope angle for drained conditions:

$$\alpha_{ult} = \phi_{cr}$$

Factor of safety for partially drained conditions and an undrained failure criteria:

$$F = k\frac{(\cos^2 \alpha - r_u)}{\sin \alpha \cos \alpha}$$

Factor of safety for partially drained conditions and an drained failure criteria:

$$F = \frac{(\cos^2 \alpha - r_u) \tan \phi_{cr}}{\sin \alpha \cos \alpha}$$

2 Site investigation

2.1 CPT interpretation

Cone total resistance:

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

Cone area ratio from laboratory pressure chamber:

$$\alpha = \frac{A_{shoulder}}{A_{shaft}}$$

Cone area ratio from geometry:

$$\alpha = \frac{q_c}{q_{chamber}}$$

Net cone resistance:

$$q_{net} = q_t - \sigma_{v0} = q_t - \gamma z$$

Undrained strength from cone net resistance:

$$s_u = \frac{q_{net}}{N_{kt}}$$

Normalised cone tip resistance:

$$Q = \frac{q_{net}}{\sigma'_{v0}} = \frac{q_{net}}{\gamma' z}$$

Friction ratio:

$$R_f = \frac{f_s}{q_{net}} \cdot 100 \text{ (i.e. expressed as percentage)}$$

Excess pore pressure ratio:

$$B_q = \frac{u_2 - u_0}{q_{net}} = \frac{\Delta u_2}{q_{net}}$$

Cone relative density:

$$I_D = 0.34 \ln \left(0.04 \frac{q_c}{p'_0} \left(\frac{p'_0}{p_a} \right)^{0.54} \right)$$

2.2 Shear vane interpretation

Undrained strength from vane shear:

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

2.3 "Full flow" penetrometer interpretation

Full flow penetrometer undrained shear strength:

$$s_u = \frac{q}{N}$$

3 Pipelines / cables

3.1 Dynamic lay effects

Pipeline / cable tension during laying process:

$$\frac{T_0}{z_w W'} = \left(\frac{\cos\phi}{1-\cos\phi}\right)$$

Stress concentration factor in touchdown zone:

$$f \approx 0.6 + 0.4\left(\frac{\lambda^2 k}{T_0}\right)^{0.25}$$

Seabed secant stiffness:

$$k = \frac{V}{w}$$

Characteristic length:

$$\lambda = \sqrt{\frac{EI}{T_0}}$$

Pipe second moment of area:

$$I_{pipe} \approx \frac{\pi}{8} D^3 t$$

3.2 Pipeline / cable geometry

Effective weight:

$$W' = (SG-1)\left(\frac{\pi D^2}{4}\right)\gamma_w$$

Semi-angle of the embedded pipeline / cable segment:

$$\theta = \cos^{-1}\left(1-\frac{2w}{D}\right)$$

Effective contact diameter:

$$D' = D\sin\theta$$

Embedded area:

$$A' = \left[\frac{\pi D^2}{4}\cdot\frac{\theta}{\pi}\right] - \left[\left(\frac{D}{2}\right)^2 \sin\theta\cos\theta\right] = \frac{D^2}{4}(\theta - \sin\theta\cos\theta)$$

3.3 Undrained bearing capacity

Undrained bearing capacity on effective contact diameter:

$$V_{max} = N_c D' s_u + f_b A'\gamma \text{ (where } N_c = 5 \text{ and } f_b \approx 1.5)$$

Undrained bearing capacity on diameter:

$$V_{max} = N_c D s_u + f_b A'\gamma \text{ (where } N_c \approx 6\,(w/D)^{0.25} \text{ and } f_b \approx 1.5)$$

3.4 Drained bearing capacity

Drained bearing capacity can be estimated from the following system of equations after Tom and White (2019):

$$V_{\max} = A\left(\frac{w}{D}\right)^{B}\gamma' D^{2}$$

$$A = C_1\left(e^{\phi_{peak}C_2}\right)^{C_3\phi_{peak}}$$

$$B = 1.3067 - 0.0123\phi_{\text{peak}}$$

$$C_i = I_{c,i} + \phi_{cs}S_{c,i}$$

Constants for C parameter determination, after Tom and White (2019):

Coefficient		Value
C_1	$S_{C,1}$	0.07
	$I_{C,1}$	1.75
C_2	$S_{C,2}$	0.0163
	$I_{C,2}$	0.6467
C_3	$S_{C,3}$	-5.97e-5
	$I_{C,3}$	0.0030

Bolton's (1986) equations for calculating peak friction and dilation angles relevant to partially buried pipelines and cables:

$$\phi_{peak} = \phi_{cs} + 0.8\psi$$

$$\psi = \frac{5I_R}{0.8}$$

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

$$p' \approx \frac{(1 + K_0)}{2}\gamma' w$$

Jaky's in-situ stress approximation:

$$K_0 = 1 - sin(\phi_{cs})$$

3.5 Axial friction

Axial force:

$$F = \mu\zeta W$$

Wedging factor:

$$\zeta = \frac{2\sin\theta}{\theta + \sin\theta\cos\theta} \leq 1.27$$

Undrained friction coefficient:

$$\mu \approx \left(\frac{s_{u-int}}{\sigma''_{vc}}\right)_{NC} OCR^{0.6}$$

Drained friction coefficient:

$$\mu = \tan\delta$$

3.6 Undrained lateral breakout

Undrained lateral breakout can be calculated using the following system of equations:

$$\frac{H}{H_{max}} = \beta\left(\frac{V}{V_{max}}\right)^{\beta_1}\left(1-\frac{V}{V_{max}}\right)^{\beta_2}$$

$$\beta = \frac{(\beta_1+\beta_2)^{\beta_1+\beta_2}}{\beta_1^{\beta_1}\beta_2^{\beta_2}}$$

$$\beta_1 = (0.8-0.15\alpha)(1.2-w/D)$$

$$\beta_2 = 0.35(2.5-w/D)$$

$$\frac{H_{\max}}{V_{\max}} = \left(0.48-\frac{\alpha}{25}\right)\left(\frac{w}{D}\right)^{\left(0.46-\frac{\alpha}{25}\right)}$$

3.7 Drained lateral breakout

Drained lateral breakout can be calculated using the following system of equations:

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

$$\bar{V} = \frac{V}{\gamma' D^2}; \quad \bar{V}_{max} = \frac{V_{max}}{\gamma' D^2}; \quad \bar{H} = \frac{H}{\gamma' D^2}$$

$$\mu = 0.2w/D + \mu_0$$

$$\mu_0 = -0.00437\phi_{peak} + 0.42$$

$$m = 0.013\phi_{peak} + 0.4$$

3.8 Pipeline thermal expansion

Free end expansion:

$$S_{FE} = 0.5\alpha\Delta TL$$

Fully constrained axial force:

$$P_{FC} = AE\alpha\Delta T$$

Temperature variation due to thermal losses:

$$\Delta T = \Delta T_{\max} - KP\Delta T_{loss}$$

Partially constrained free end expansion:

$$S_{PC} \approx 0.5\left(\alpha\Delta TL - \frac{P_{ave}L}{EA}\right)$$

Axial force:

$$P = \mu\zeta W' L_{FE}$$

Hobb's critical buckling force:

$$P_{buckle} = 3.86\sqrt{\frac{EIH}{D}}$$

4 Piles

Structure natural frequency of a wind turbine on pile:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{1}{m_T\left(\frac{h_T^3}{3EI}+\frac{h_T^2}{k_s}\right)}}$$

Dynamic Amplification Factor:

$$DAF = \frac{1}{\sqrt{\left(1-\beta^2\right)^2+(2\xi\beta)^2}}$$

4.1 Axial Response

4.1.1 Axial capacity: DNV-OS-J101 (DNV 2014) / API (2000) method

Clay

α-Method:

Unit shaft resistance: $\alpha = \frac{\tau_s}{s_u} = 0.5max\left[\left(\frac{\sigma'_{v0}}{s_u}\right)^{0.5}, \left(\frac{\sigma'_{v0}}{s_u}\right)^{0.25}\right]$

Note: it is assumed that equal shaft resistance acts inside and outside open-ended piles.

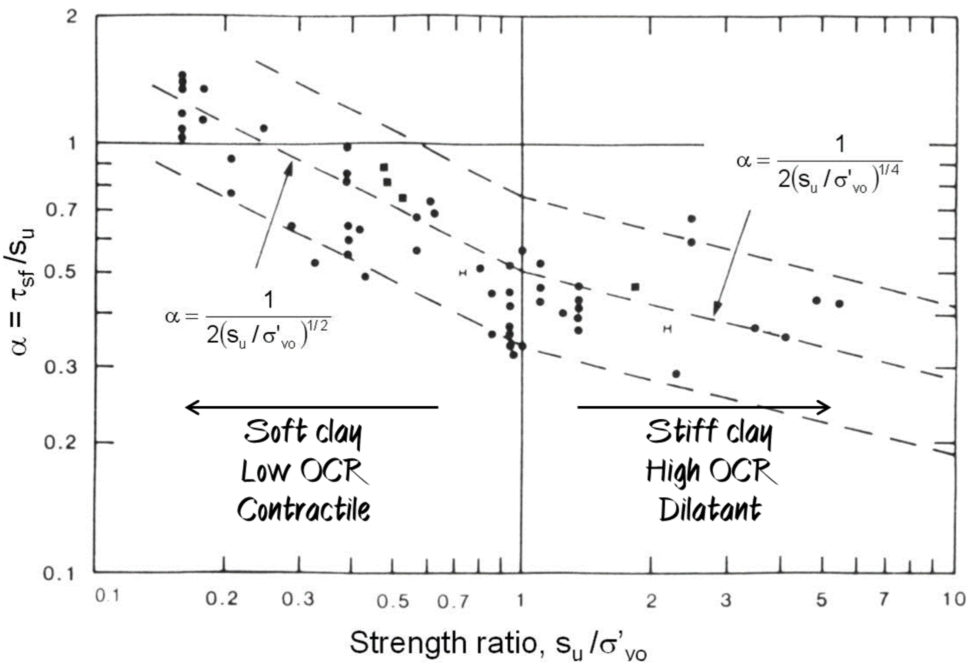


Figure 1: API (2000) α-correlations for ultimate unit shaft resistance in clay (Randolph & Murphy 1985)

β-Method:

$$\tau_{sf} = \beta\sigma'_{v0} = K\sigma'_{v0}tan\delta$$

λ-Method:

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

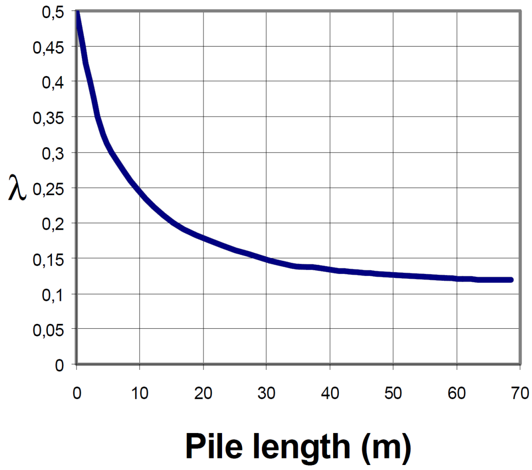


Figure 2: Coefficient λ vs. pile length

Unit base resistance:

$$q_b = N_c s_u; \quad N_c = 9$$

Sand

Unit shaft resistance: $\tau_{sf} = \sigma'_{hf}tan\delta = K\sigma'_{v0}tan\delta \leq \tau_{s,lim}$

Closed-ended piles: $K = 1$
Open-ended piles: $K = 0.8$

Unit base resistance: $q_b = N_q\sigma'_{v0} < q_{b,limit}$

Soil category	Soil density	Soil type	Soil-pile friction angle, δ (°)	Limiting value $\tau_{s,lim}$ (kPa)	Bearing capacity factor, N_q	Limiting value, $q_{b,lim}$ (MPa)
1	Very loose Loose Medium	Sand Sand-silt Silt	15	48	8	1.9
2	Loose Medium Dense	Sand Sand-silt Silt	20	67	12	2.9
3	Medium Dense	Sand Sand-silt	25	81	20	4.8
4	Dense Very dense	Sand Sand-silt	30	96	40	9.6
5	Dense Very dense	Gravel Sand	35	115	50	12

Figure 3: DNV-OS-J101 (DNV 2014) recommendations for driven pile capacity in sand (following API, 2000).

t-z curves: DNV-OS-J101 (DNV 2014)

Governing equation:

$$\frac{d^2w}{dz^2} = \frac{\pi D}{(EA)_p}\tau_s$$

The t-z curves can be generated with a nonlinear relation between the origin and the point where the maximum skin resistance t_{max} is reached:

$$z = t\frac{R}{G_0}ln\left(\frac{z_{IF} - r_f\frac{t}{t_{max}}}{1 - r_f\frac{t}{t_{max}}}\right) \quad for\ 0 \leq t \leq t_{max}$$

in which:

R	Radius of the pile
G_0	Initial shear modulus of the soil
z_{IF}	Dimensionless zone of influence (defined as the radius of the zone of influence around the pile divided by R)
r_f	curve fitting factor

4.2 Lateral Response

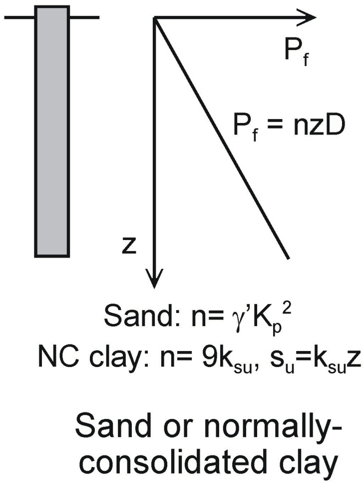
4.2.1 Lateral capacity: linearly increasing lateral resistance with depth

Lateral soil resistance (force per unit length): $P_f = nzD$

In sand: $n = \gamma' K_p^2$

In normally consolidated clay with strength gradient k: $s_u = kz; n = 9k$

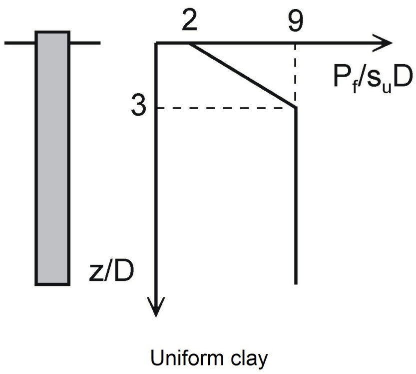
H_{ult} Ultimate horizontal load on pile
D Pile diameter
L Pile length
γ' Effective unit weight
K_p Passive earth pressure coefficient, $\approx (1 + sin(\phi))/(1 - sin(\phi))$



4.2.2 Lateral capacity: uniform clay

Lateral soil resistance (force per unit length), P_f, increases from $2s_uD$ at surface to $9s_uD$ at $3D$ depth then remains constant.

H_{ult} Ultimate horizontal load on pile
D Pile diameter
L Pile length
s_u Undrained shear strength



4.2.3 DNV-OS-J101 (DNV 2014) / API (2000) p-y curves method

Governing equation:

$$E_p I_p \frac{\mathrm{d}^4 y}{\mathrm{d}z^4} + V \frac{\mathrm{d}^2 y}{\mathrm{d}z^2} + k_{py} y = 0$$

Clay

The static ultimate lateral resistance is recommended to be calculated as:

$$P_f = \begin{cases} (3s_u + \gamma' z)D + J s_u z & for\ 0 < z < z_R \\ 9 s_u D & for\ z > z_R \end{cases}$$

where:

z	Depth below soil surface
z_R	Transition depth
	below which the value of $(3s_u + \gamma' z)D + J s_u z$ exceeds $9 s_u D$
D	Pile diameter
	defined as the radius of the zone of influence around the pile divided by R
s_u	Undrained shear strength of the soil
γ'	Effective unit weight of soil
J	Dimensionless empirical constant
	with values in the range of 0.25 to 0.50
	with 0.50 recommended for soft normally consolidated clay.

For static loading, the $p - y$ curve can be generated according to:

$$p = \begin{cases} \frac{p_u}{2}\left(\frac{y}{y_c}\right)^{1/3} & for\ y < 8y_c \\ p_u & for\ y > 8y_c \end{cases}$$

For cyclic loading and $z > z_R$, the p-y curves can be generated according to:

$$p = \begin{cases} \frac{p_u}{2}\left(\frac{y}{y_c}\right)^{1/3} & for\ y < 3y_c \\ 0.72 p_u & for\ y > 3y_c \end{cases}$$

For cyclic loading and $z \le z_R$, the p-y curves can be generated according to:

$$p = \begin{cases} \frac{p_u}{2}\left(\frac{y}{y_c}\right)^{1/3} & for\ y < 3y_c \\ 0.72 p_u \left(1 - \left(1 - \frac{z}{z_R}\right) \frac{y - 3y_c}{12 y_c}\right) & for\ 3y_c < y < 15y_c \\ 0.72 p_u \frac{z}{z_R} & for\ y > 15y_c \end{cases}$$

Here, $y_c = 2.5\epsilon_c D$, in which D is the pile diameter and ϵ_c is the strain which occurs at one-half the maximum stress in laboratory undrained compression tests of undisturbed soil samples.

Sand

For piles in cohesionless soils, the static ultimate lateral resistance is recommended to be calculated as:

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

where the coefficients C_1, C_2 and C_3 depend on the friction angle ϕ as shown in Figure 2 (RHS), and where:

z	Depth below soil surface
z_R	Transition depth below which the value of $(C_1 z + C_2 D)\gamma' z$ exceeds $C_3 D\gamma' z$
D	Pile diameter defined as the radius of the zone of influence around the pile divided by R
γ'	Effective unit weight of soil

The p-y curve can be generated according to:

$$p(z, y) = Ap_u \cdot tanh\left(\frac{k.z}{Ap_u} y\right)$$

in which k is the initial modulus of subgrade reaction and depends on the friction angle ϕ as given in the Figure below (LHS), and A is a factor to account for static or cyclic loading conditions as follows:

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

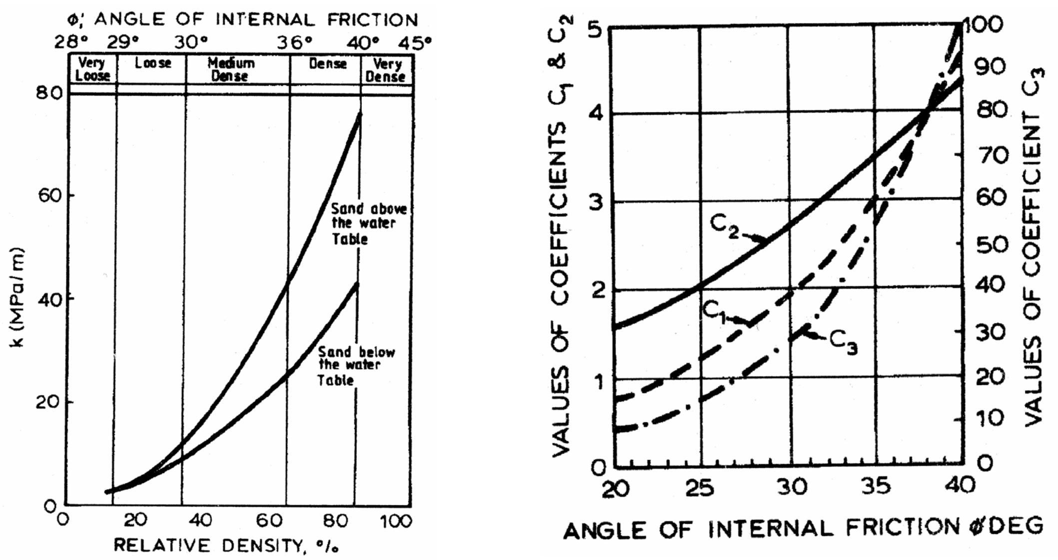


Figure 4: Modulus of subgrade reaction and Empirical coefficients in DNV as a function of friction angle.

5 Anchors

5.1 Equilibrium equations for an embedded anchor line

Change in line tension:

$$\frac{dT}{ds} = F + wsin\theta$$

Change in mooring line angle:

$$\frac{d\theta}{ds} = \frac{-Q + wcos\theta}{T}$$

Unit soil friction acting on the anchor line (parallel to the line):

$$F = A_s \alpha s_u$$

Limiting normal force transmitted to the anchor line from the soil:

$$Q = A_b N_c s_u$$

5.2 Analytical solution for embedded anchor line

Line tension at padeye:

$$\frac{T_a}{2}\left(\theta_a^2 - \theta_m^2\right) = z_a Q_{av}$$

Bearing resistance:

$$z_a Q_{av} = bN_c \int_0^{z_a} s_u dz$$

Angle at padeye:

$$\theta_a = \sqrt{\frac{2}{T^*}}$$

where:

$$T^* = \sqrt{\frac{T_a}{z_a Q_{av}}}$$

Relationship between the anchor line tension at the mudline T_m and that at the padeye T_a:

$$\frac{T_m}{T_a} = e^{\mu(\theta_a - \theta_m)}$$

Normalised profile of the anchor line:

$$z^* = e^{x^* \theta_a}$$

Effective bearing resistance:

$$Q_{eff} = Q - w$$

5.3 Analytical solution for drag anchors

Holding capacity:

$$T_a = A_f N_c s_u$$

Force acting on the anchor parallel to the direction of travel:

$$T_p = (fA_p)N_c s_u$$

Anchor capacity at any embedment:

$$T_w = \frac{T_p}{cos\theta} = \frac{fA_pN_cs_u}{cos\theta_w}$$

Resultant force in the anchor chain at the anchor attachment point:

$$T_a = \frac{T_p}{cos\theta'_w} = \frac{fA_pN_cs_u}{cos\theta'_w}$$

Angle of the resultant anchor line tension T_a to the fluke for a weighty anchor:

$$\theta'_w = tan^{-1}\left(\frac{W + T_p tan\theta_w}{T_p}\right)$$

Anchor holding capacity:

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

Bearing resistance:

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

Relationship between the anchor line tension at the mudline T_m and that at the padeye T_a:

$$\frac{T_m}{T_a} = e^{\mu(\theta_a - \theta_m)}$$

Performance ratio:

$$\eta = \frac{T_m}{W}$$

5.4 Drop anchors

Impact velocity:

$$V_{terminal} = \sqrt{\frac{2mg}{C_{drag} A_{end} \rho_{water}}} \quad where \;\; C_{drag} \approx 0.035 + 0.01\frac{L}{D}$$

Equation of motion:

$$m\frac{d^2 z}{dt^2} = W_s - F_{bear} - F_{fric} - F_d$$

where:

m	anchor mass
z	depth
t	time

Shaft resistance :

$$F_{fric} = Q_{sf} = \alpha s_u A_{shaft} = \alpha k_{su} z A_{shaft}$$

Front and rear ”base resistance”:

$$F_{bear} \approx 2 N_c s_u A_{tip}$$

Inertial drag:

$$F_d = \frac{1}{2} C_d \rho_s A_{tip} v^2$$

where:

C_d	drag coefficient, estimated as 0.24
ρ_s	soil density
A_{tip}	projected anchor area
v	current anchor velocity

Final penetration depth:

$$z_{final} = \frac{W_s + \sqrt{(W_s)^2 + m v_{impact}^2 k_{su}(\alpha A_{shaft} + 2N_c A_{tip})}}{k_{su}\left(\alpha A_{shaft} + 2N_c A_{tip}\right)}$$

5.5 Suction caissons

5.5.1 Installation resistance

Clay:

$$Q = A_s \alpha \bar{s}_u + A_{tip}\left(N_c s_u + \gamma' z\right)$$

Required under-pressure:

$$\Delta u_{req} = \frac{Q - W'}{A_i}$$

Allowable under-pressure:

$$\Delta u_a = \frac{A_i N_c s_u + A_{si}\alpha \bar{s}_u + W'_{plug} - \gamma' d A_{plug}}{A_i} = \frac{A_i N_c s_u + A_s \alpha \bar{s}_u}{A_i}$$

Factor of safety:

$$F = \frac{\Delta u_a}{\Delta u_{req}}$$

Soil plug stability criterion:

$$\left(\frac{L}{D}\right)_{limit} \simeq \frac{1}{4\alpha_e}\left[N_c + \left(N_c^2 + \frac{32 W \alpha_e}{\pi k D^3}\right)\right]$$

Sand:

$$W' + 0.25\pi D_i^2 p = F_o + (F + Q_{tip})\left(1 - \frac{p}{P_{crit}}\right) \text{ for } \quad p \leq p_{crit}$$

5.5.2 Vertical capacity

Without suction:

$$V_{ult} = W' + A_{se}\alpha_e \bar{s}_{u(t)} + A_{si}\alpha_i \bar{s}_{u(t)}$$

Or:

$$V_{ult} = W' + A_{se}\alpha_e \bar{s}_{u(t)} + W'_{plug}$$

With suction:

$$V_{ult} = W' + A_{se}\alpha_e \bar{s}_{u(t)} + N_c s_u A_e$$

5.5.3 Maximum horizontal resistance

$$H_{max} = L D_e N_p \bar{s}_u$$

5.5.4 Inclined loading

$$\left(\frac{H}{H_{ult}}\right)^a + \left(\frac{V}{V_{ult}}\right)^b = 1$$

6 Shallow foundations

6.1 Clay

Ultimate capacity:

$$V_{ult} = A' \left(s_{u0} \left(N_c + kB'/4 \right) \frac{F K_c}{\gamma_m} + p'_0 \right)$$

Dimensionless undrained strength gradient:

$$\kappa = \frac{kB'}{s_{u0}}$$

Modification factor:

$$K_c = 1 - i_c + s_c + d_c$$

where:

$$\begin{aligned} i_c &= 0.5(1 - \sqrt{1 - H/A's_{u0}}) \\ s_c &= s_{cv} \left(1 - 2i_c\right) B'/L \\ d_c &= 0.3e^{-0.5kB'/s_{u0}} \arctan\left(d/B'\right) \end{aligned}$$

Shape factor s_{cv} after Salençon and Matar (1982):

$\kappa = kB/s_{u0}$	s_{cv}
0	0.20
2	0.00
4	-0.05
6	-0.07
8	-0.09
10	-0.10

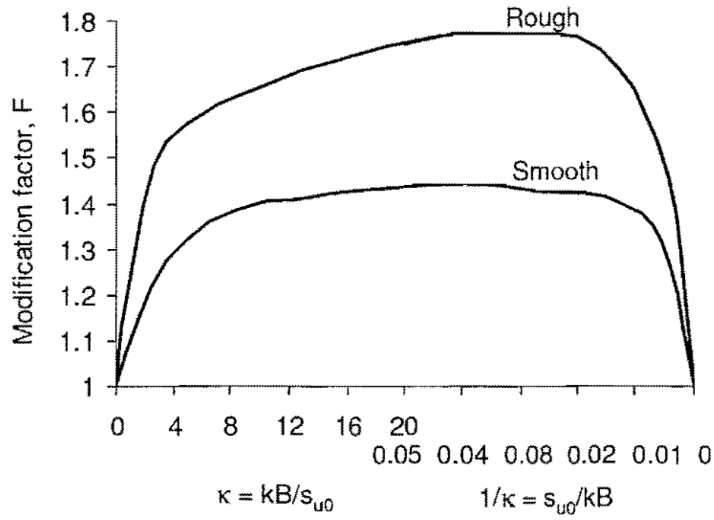


Figure 5: Modification factor F, after Davis and Booker (1973).

Horizontal failure criterion gives:

$$\frac{H_{ult}}{A's_{u0}} = 1$$

Ultimate moment:

$M_{ult} = 0.64A'Bs_{u0}$ for a strip foundation
$M_{ult} = 0.61A'Ds_{u0}$ for a circular foundation

6.2 Sand

$$V_{ult} = A'\left(\frac{1}{2}\gamma' B' N_\gamma K_y + (p_0 + a)N_q K_q - a\right)$$

Where:

V_{ult}	Ultimate vertical load
A'	Effective bearing area of the foundation
γ'	Effective unit weight of the soil
B'	Effective width of the foundation
$N_\gamma,\ N_q$	Bearing capacity factors for self-weight and surcharge
K_γ, K_q	Modification factors to account for foundation shape, embedment and load inclination
p_0'	Effective overburden acting to either side of the foundation
a	Soil attraction factor which accounts for cementation equal to the point of interception of the tangent to the Mohr Circle and the normal stress axis.
ϕ	Effective internal friction angle of the soil
γ_m	material factor on shear strength

$$N_q = tan^2\left(\frac{\pi}{4} + \frac{1}{2}tan^{-1}\left(\frac{tan\phi}{\gamma_m}\right)\right) e^{\dfrac{\pi tan\phi}{\gamma_m}}$$

$$N_\gamma = 1.5(N_q - 1)tan\left(\frac{tan\phi}{\gamma_m}\right)$$

Modification factors:

$$K_q = s_q d_q i_q$$

and:

$$K_\gamma = s_\gamma d_\gamma i_\gamma$$

$$s_q = 1 + i_q\frac{B'}{L}sin\left(tan^{-1}\left(\frac{tan\phi}{\gamma_m}\right)\right)$$

$$d_q = 1 + 2\frac{d}{B'}\left(\frac{tan\phi'}{\gamma_m}\right)\left\{1 - sin\left(tan^{-1}\left(\frac{tan\phi}{\gamma_m}\right)\right)\right\}^2$$

$$i_q = \left\{1 - 0.5\left(\frac{H}{V + A'a}\right)\right\}^5$$

$$s_\gamma = 1 - 0.4 i_\gamma \frac{B'}{L}$$

$$d_\gamma = 1$$

$$i_\gamma = \{1 - 0.7(\frac{H}{V + A'a})\}^5$$

6.3 Spudcan Foundations

6.3.1 Clay

Bearing capacity:

$$V = (N_c s_u + \sigma'_{v0})A$$

Table 1: Bearing capacity factors for rough circular plate in homogeneous soil (Houlsby & Martin 2003)

$Embedment\ depth/D$	Bearing factor, N_c
0	6
0.1	6.3
0.25	6.6
0.5	7.1
1.0	7.7
≤ 2.5	9.0

Conditions for backflow:

$$Flow\ failure\ occurs\ if\ : \quad \frac{D}{B} > \left(\frac{s_{uD}}{\gamma' B}\right)^{0.55} - \frac{1}{4}\left(\frac{s_{uD}}{\gamma' B}\right)$$

6.3.2 Sand

$$V = \gamma' N_\gamma \frac{\pi D^3}{8} \tag{1}$$

Table 2: Bearing capacity factors for a flat, rough circular footing (from use of ABC software of Martin (2003))

Friction angle ϕ (degrees)	Bearing factor, N_γ
20	2.42
25	6.07
30	15.5
35	41.9
40	124
45	418

7 Hydrodynamics / scour

Surface elevation:

$$\eta(x,t) = \frac{H}{2} \sin\left[2\pi\left(\frac{t}{T} - \frac{x}{L}\right)\right]$$

Dispersion equation:

$$\omega^2 = gk\tanh(kh)$$

Particle velocity:

$$v_x = \omega\frac{H}{2}\frac{\cosh[k(h-z)]}{\sinh(kh)}\sin(\omega t - kx)$$
$$v_z = \omega\frac{H}{2}\frac{\sinh[k(h-z)]}{\sinh(kh)}\cos(\omega t - kx)$$

Drag force due to fluid flow:

$$F_D = \frac{1}{2}\rho v^2 C_D D$$

Lift force due to fluid flow:

$$F_L = \frac{1}{2}\rho v^2 C_L D$$