EGT3
ENGINEERING TRIPOS PART IIB

Friday 7 May 2021 9 to 10.40

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
CUED approved calculator allowed
Attachment: 4D9 Offshore Geotechnical Engineering Data Book (20 pages)
You are allowed access to the electronic version of the Engineering Data Books.

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

The time taken for scanning/uploading answers is 15 minutes.

Your script is to be uploaded as a single consolidated pdf containing all answers.

1 A deep water location is being considered for the development of a floating offshore wind farm, consisting of an array of floating turbines moored by anchors embedded in soft normally-consolidated clay. An initial site investigation has been performed, which included cone penetrometer testing, simple shear tests performed on samples recovered to a laboratory onshore, and T-bar penetrometer tests.

(a) Measured data from a cone penetrometer test at the location is shown in Table 1, along with the cone area ratio of the instrument, α, and the total unit weight of the soil, γ. Using the Robertson Chart given in Fig. 1, indicate the most likely type of material at the proposed wind farm location. [35%]

(b) The stress-strain response from a simple shear test on a sample recovered from a depth of z = 50 m from a borehole close to the cone penetrometer test location is shown in Fig. 2.

(i) Identify the shear strength from the simple shear data and identify the cone factor, N_{kt}, that ought to be used to interpret the cone penetrometer data in part 1(a). [20%]

(ii) Indicate the stress that the simple shear sample should have been consolidated to and explain why it ought to have been consolidated to that particular stress. [10%]

(c) A T-bar penetrometer test was carried out in a location nearby to the cone penetrometer test, with cycles of penetration performed at a depth z = 50 m in order to measure the remoulded undrained strength. The penetration resistance was measured for the initial penetration as q_{in} = 805 kPa, the initial extraction as q_{out} = 525 kPa, and the steady-state condition after a number of penetration cycles as q_{rem} = 130 kPa.

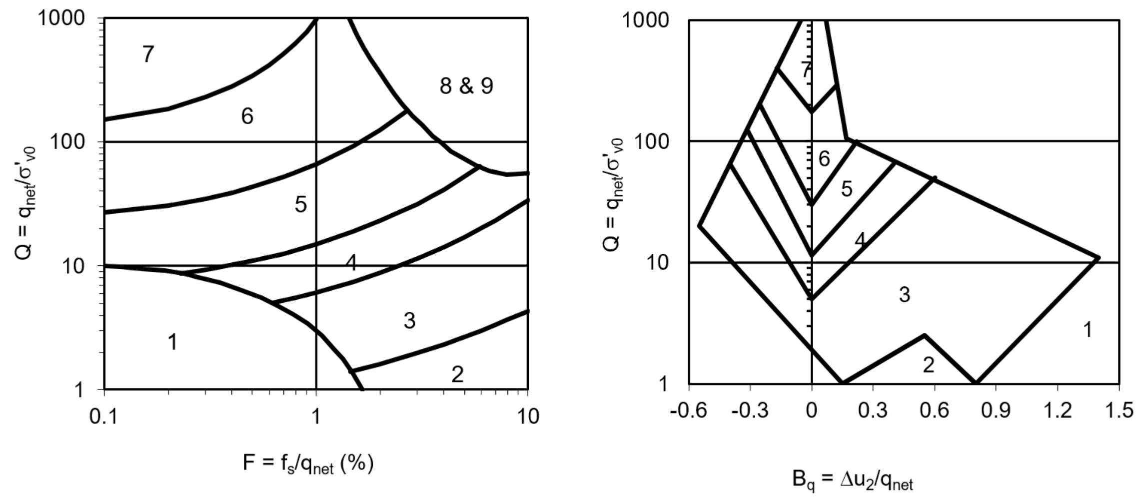
(i) Calculate the fully remoulded strength ratio and comment on the physical meaning of the calculated value. [10%]

(ii) Identify an appropriate choice of T-bar factor, N_{T-bar}, and calculate the intact and fully remoulded undrained shear strength from the T-bar data. [10%]

(d) Identify three key benefits of "full flow" in situ profiling tools, such as the T-bar or ball penetrometer, over conventional tools such as the cone penetrometer. [15%]

Table 1

z (m)	q_c (kPa)	f_s (kPa)	u_2 (kPa)	α (-)	γ (kN/m^3)
50	1525	18	720	0.7	16



Zone	Soil behaviour type
1.	Sensitive, fine grained
2.	Organic soils, peats
3.	Clayey clays to silty clay

Zone	Soil behaviour type
4.	Silt mixtures, clayey silt to silty clay
5.	Sand mixtures, silty sand to sandy silt
6.	Sands, clean sand to silty sands

Zone	Soil behaviour type
7.	Gravelly sand to sand
8.	Very stiff sand to clayey sand
9.	Very stiff fine grained

Fig. 1

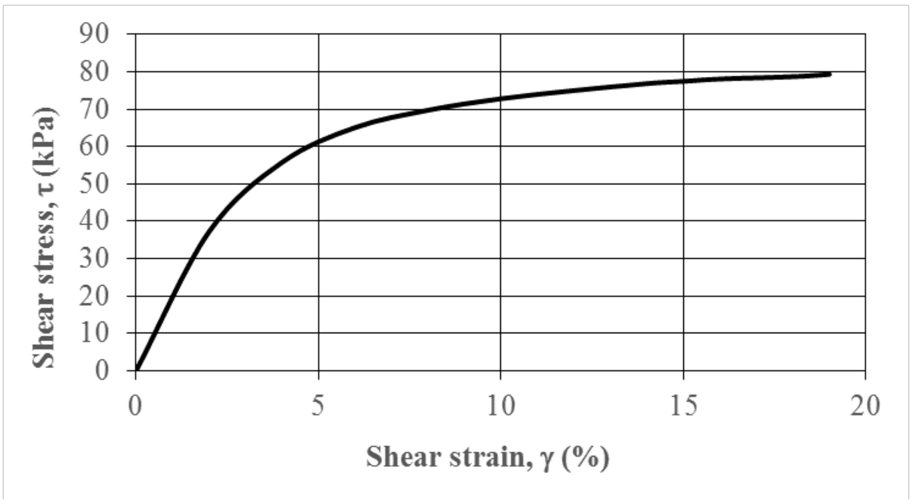


Fig. 2

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2 A subsea cable, with outer diameter D and a submerged weight per unit length of W', is embedded in uniform clay as shown in Fig. 3. The cable is subjected to a lateral force (per unit length) of F_L due to hydrodynamic loading, and breaks out by failing the soil on the slip plane shown. The clay is characterised by an undrained shear strength, s_u, and a submerged unit weight, γ'.

(a) Calculate the length of the slip plane as a multiple of the cable diameter, D. [15%]

(b) Calculate the area of the soil above the slip plane (shown hatched in Fig. 3) as a multiple of D^2. [20%]

(c) The lateral breakout resistance, F_L, can be calculated for the failure mechanism shown using a work equation. Determine F_L as a function of W', s_u, γ' and D. [50%]

(d) For the properties given in Table 2, what is the lateral breakout resistance expressed as an equivalent friction factor, F_L/W'? [15%]

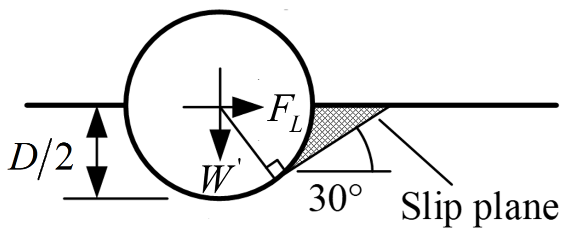


Fig. 3

Table 2

Cable properties	Outer diameter, D (m)	0.4
	Submerged weight, W' (kN/m)	1
Seabed properties	Undrained shear strength, s_u (kPa)	1
	Submerged unit weight, γ' (kN/m^3)	6

3 A rigid monopile with diameter $D = 8.75$ m and length $L = 35$ m is being considered to support 8 MW offshore wind turbines for a site in the North Sea. The site consists of sand with friction angle $\phi' = 32°$ and effective unit weight $\gamma' = 20$ kN/m^3.

(a) Calculate the ultimate lateral force per unit length 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. [10%]

(b) Compare the p-y method and the 1D PISA method and discuss their application to the design of offshore wind monopiles. [30%]

(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 fails by pure rigid body rotation, with the pivot point located at a depth of $z_{rot} = 0.7L$ to deduce the value of the pile displacement at depths z_1 and z_2. [50%]

(d) Comment on the limitations of the method you used to calculate the cyclic response. [10%]

4 A 40 tonne dry weight fixed-fluke anchor is being designed as a catenary mooring for a semi-submersible structure in 1,000 m of water. The anchors have a projected area of 16 m^2, form factor f of 1.2, $\theta_w = 30°$ and vertical offset from padeye to fluke of 6 m. Assume that the bearing capacity factor $N_c = 9$ and the density of steel $\rho_s = 7850$ kg/m^3.

The anchor is tethered to the semi-submersible structure using a 0.15 m diameter bar link chain at the ocean surface. Assume that the effective width of the chain link is 2.5 times the bar link diameter, the bearing capacity factor $N_c = 7.5$ and the friction factor $\mu = 0.3$.

The seabed has soil with approximately uniform undrained shear strength, s_u, equal to 15 kPa.

(a) Calculate the ultimate holding capacity of the anchor system at the surface of the seabed, the anchor system efficiency and the final depth of the anchor fluke. [60%]

(b) Describe other anchor options with accompanying sketches. Describe their installation methods and summarise potential advantages and disadvantages, relative to fixed-fluke anchors. [40%]

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:

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

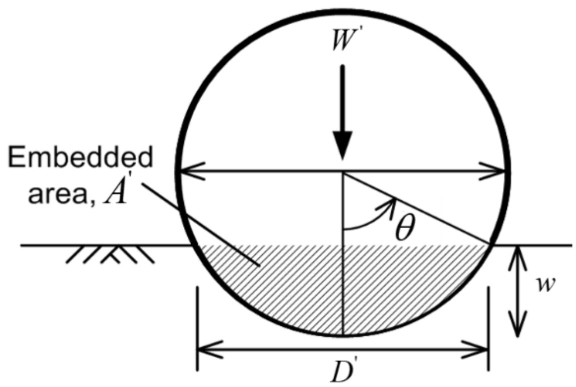
Characteristic length:

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

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):

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

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

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

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

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:

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

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

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

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

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:

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

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

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

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

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

$$n = 0.64$$

3.8 Pipeline thermal expansion

Free end expansion:

$$S_{FE} = 0.5\alpha\Delta T L$$

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:

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

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

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

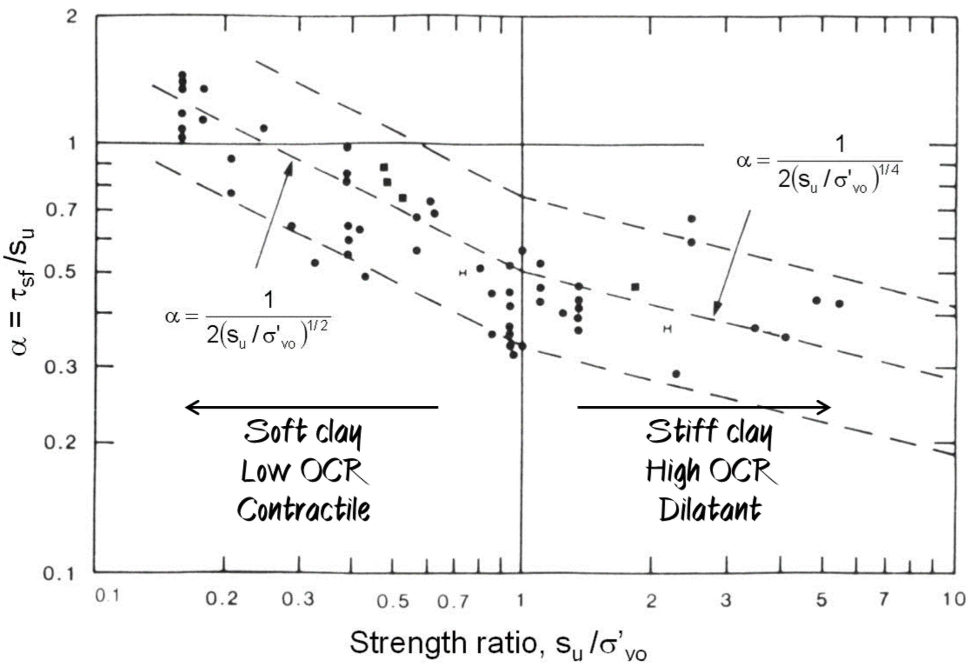


Figure 1: API (2000) α-correlations for ultimate unit shaft resistance in clay (Randolph & Murphy 1985)

β-Method:

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

λ-Method:

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

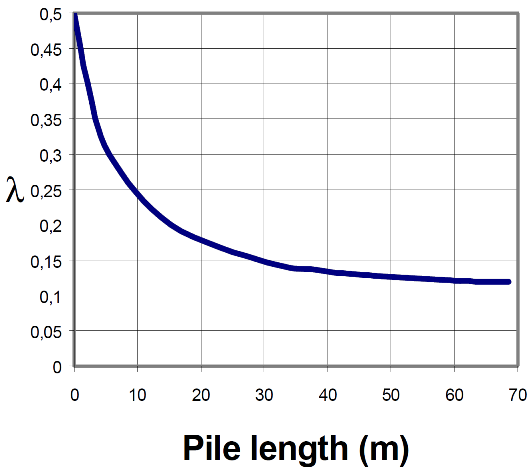


Figure 2: Coefficient λ vs. pile length

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:

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

in which:

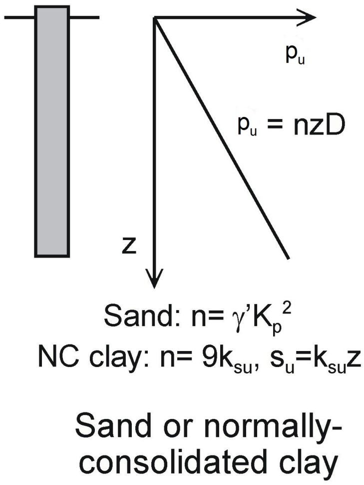
R	Radius of the pile
G_0	Initial shear modulus of the soil
z_{IF}	Dimensionless zone of influence (defined as the radius of the zone of influence around the pile divided by R)
r_f	curve fitting factor

4.2 Lateral Response

4.2.1 Lateral capacity: linearly increasing lateral resistance with depth

Lateral soil resistance (force per unit length):	$p_u = nzD$
In sand:	$n = \gamma' K_p^2$
In normally consolidated clay with strength gradient k:	$s_u = kz; n = 9k$

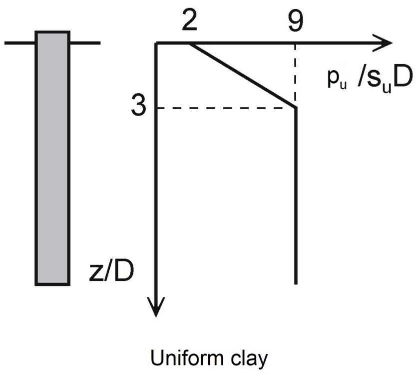
H_{ult} Ultimate horizontal load on pile
D Pile diameter
L Pile length
γ' Effective unit weight
K_p Passive earth pressure coefficient, $\approx (1 + sin(\phi))/(1 - sin(\phi))$



4.2.2 Lateral capacity: uniform clay

Lateral soil resistance (force per unit length), p_u, 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:

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

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:

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

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

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

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

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

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:

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

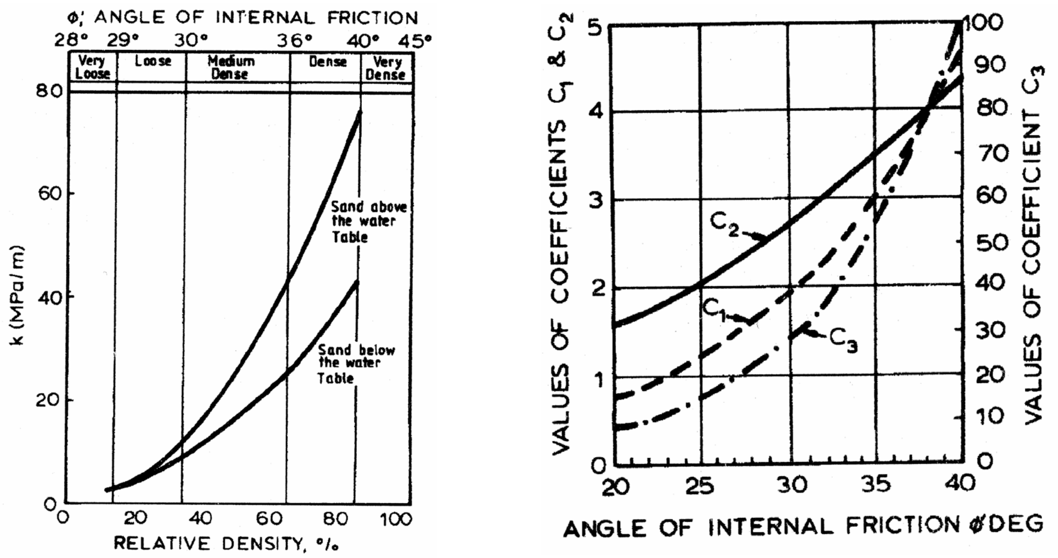


Figure 4: Modulus of subgrade reaction and Empirical coefficients in DNV as a function of friction angle.

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:

$$Q = A_b N_c s_u$$

A_b = Effective surface area of the anchor line per unit length.
For wire or polyester rope: $A_b = d$.
For a standard link chain: effective width $b = 2.5d$; $A_b = b = 2.5d$

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:

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

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:

$$T_p = (fA_p) N_c s_u$$

f = form factor for the anchor.

Anchor capacity at any embedment:

$$T_w = \frac{T_p}{cos\theta} = \frac{fA_p N_c s_u}{cos\theta_w}$$

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

$$T_a = \frac{T_p}{cos\theta'_w} = \frac{fA_p N_c s_u}{cos\theta'_w}$$

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:

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

Bearing resistance:

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

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:

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

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”:

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

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:

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

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:

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

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:

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

Dimensionless undrained strength gradient:

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

Modification factor:

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

where:

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

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

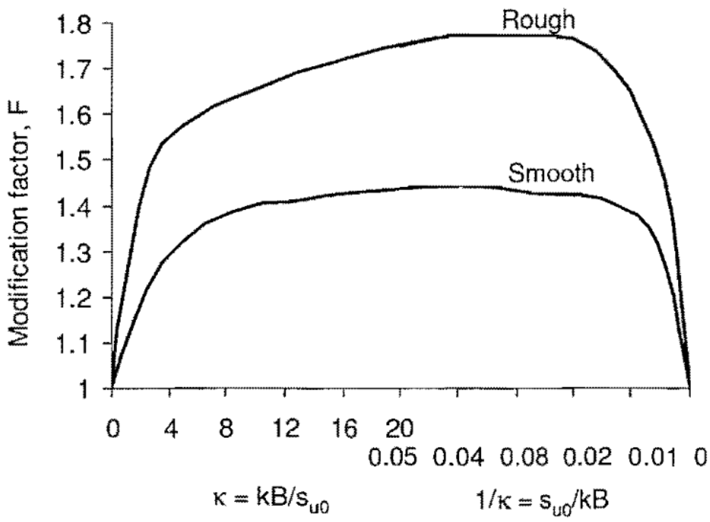


Figure 5: Modification factor F, after Davis and Booker (1973).

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:

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