Version CNA/3

EGT3
ENGINEERING TRIPOS     PART IIB

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Friday 5 May 2023    09.30 to 11.10

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**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.*

**STATIONERY REQUIREMENTS**
Single-sided script paper

**SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM**
Engineering Data Book
CUED approved calculator allowed
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 A shallowly buried subsea export cable is required to bring electricity back to shore from a proposed floating offshore wind turbine array in the Celtic Sea to be developed at a location called the Celtic Deep, which has a water depth of 120 metres. The route is expected to span sediment types from sands through to transitional silts and soft muds.

(a) Describe the deposition mechanisms that formed the deposits expected to be found along the export cable route and in doing so indicate where along the route they might be expected to be found. [30%]

(b) The site investigation contractor is intending to deploy a survey vessel equipped with a grab sampler, box-corer and cone penetration and full-flow penetrometer testing apparatus. Explain which site investigation tool is best suited for each sediment type expected to be found along the export cable route. [30%]

(c) Describe five offshore geohazards that could potentially be found along the cable export route and for each summarise the danger they could pose to the export cable. [25%]

(d) A bathymetric survey has shown that the slope into the Celtic Deep is approximately 15° where the export cable route is intended to pass and there is evidence of previous slope failure. Using an infinite slope assumption, calculate the minimum critical friction angle and undrained strength ratio of the soil at this location. Is it possible to determine the likely soil type from these simple calculations? [15%]

2 A 0.4 m diameter steel pipeline with specific gravity (SG) of 3 and wall thickness $t$ = 0.02 m, was laid at a location with a water depth of 100 m using a J-lay vessel with firing line inclined at 15° from vertical. The seabed had an undrained shear strength $s_u$ = 15 kNm$^{-2}$, sensitivity $S_t$ = 3 and effective unit weight $\gamma'$ = 5 kNm$^{-3}$.

(a) Calculate the effective weight $W'$ of the pipeline. [5%]

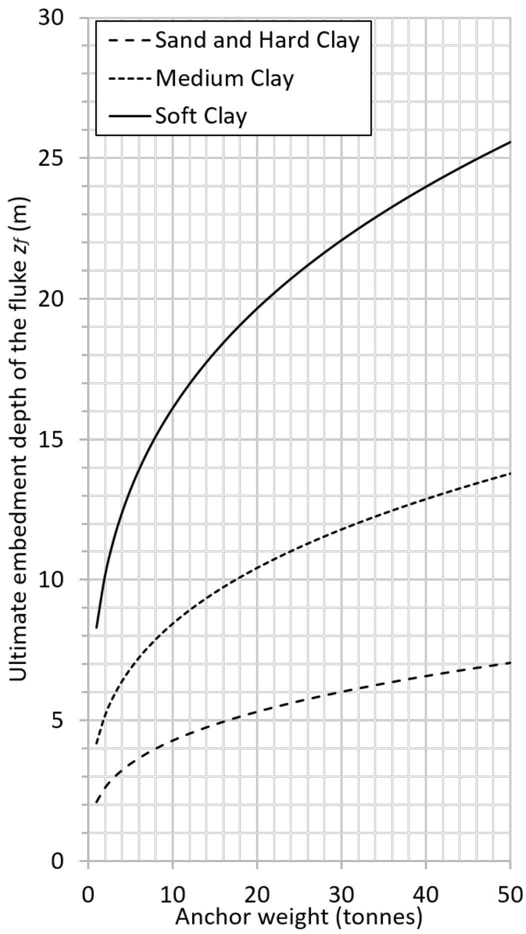
(b) Determine the approximate second moment of area $I$ of the pipeline. [5%]

(c) Given the as-laid embedment was found to be 40% of the pipeline diameter during a subsequent survey, estimate the lumped lay factor $f$ $(= f_{lay} \cdot f_{dyn})$ that must have been experienced by the pipeline during the lay process. [25%]

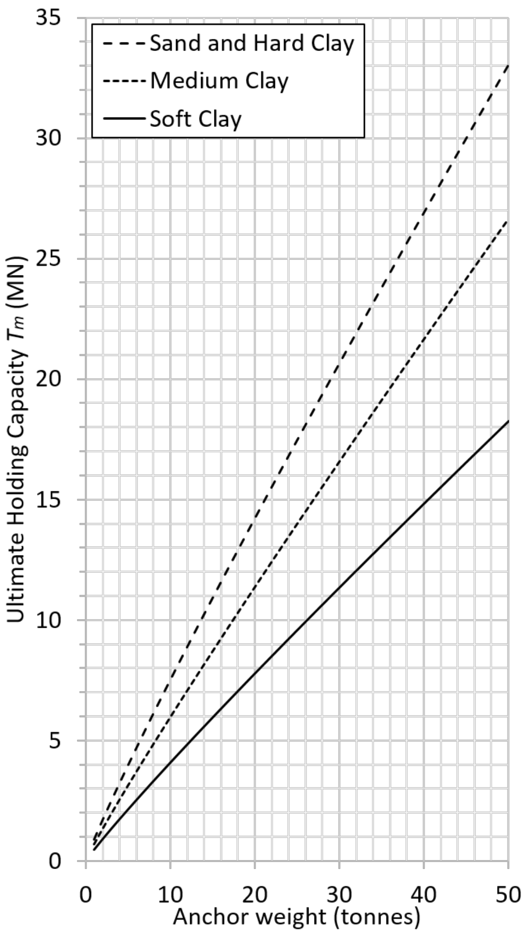
(d) Estimate the "static lay" and "dynamic lay" components, $f_{lay}$ and $f_{dyn}$, respectively, of the lumped lay factor $f$ calculated in part (c). [50%]

(e) Comment on the weather conditions likely to have been experienced during the lay process given the preceding calculations in parts (c) and (d). [15%]

3 A wind turbine manufacturer is considering a drag anchor to moor a prototype floating wind turbine. The selected anchor has a dry weight $W_{dry}$ = 40 tonnes, a height from fluke to padeye of $z_{a-f}$ = 6.2 m, a projected area $A_p$ = 16.4 m$^2$, a shape factor $f$ = 1.1 and a resultant angle for the net resistance (ignoring the anchor weight) of $\theta_w$ = 50 degrees. The density of steel is equal to $\rho_{steel}$ = 7850 kg/m$^3$. The anchor manufacturer provides the design charts in Figs. 1(a) and 1(b) to facilitate the design.




(a)




(b)

Fig. 1

The anchor is to be installed at a soft clay site with undrained shear strength $s_u = 4+1.5z$ kPa, where $z$ is the depth in metres. The bearing capacity factor on anchor resistance is $N_c$ = 9.

The turbine will be moored to the ground using a semi-taut bar-link chain of 0.5 m in diameter, with an effective width in bearing of 3.0 times the bar link diameter, a friction factor of 0.35 and bearing capacity factor $N_{cl}$ = 7.6. The semi-taut line forms an angle of $\theta_m$ = 20 degrees with the mudline.

The designer estimates that the ultimate tension load exerted by the turbine onto each mooring line is equal to $T_{line}$ = 21 MN. The designer's objective is to estimate whether the selected drag anchor is able to withstand this load with a factor of safety greater than $FOS$ = 1.35.

(a) Sketch and describe a fixed fluke drag anchor and describe their use for mooring of offshore structures. [15%]

(b) Using the designer's chart to obtain the anchor embedment depth $z_f$ (Fig. 1(a)), estimate the anchor resisting force $T_p$ and calculate the resultant force in the anchor chain at the attachment point $T_a$ and its angle of inclination to the fluke $\theta_a$. [25%]

(c) Using the bearing resistance for the embedded anchor chain, show that the estimation of the anchor embedded depth from the manufacturer's chart in Fig. 1(a) for this anchor is correct. [35%]

(d) From the result of part (b), calculate the ultimate holding capacity $T_m$ of the anchor and its efficiency $\eta$. Is the design requirement from the designer met for this application? [15%]

(e) Compare your results to the ultimate holding capacity recommended by the manufacturer (Fig. 1(b)) and comment on any differences. [10%]

4 A mobile jack-up platform is commissioned to install an offshore wind farm at a clay site. The legs of the installation vessel are supported by spudcan foundations of diameter $B = 10$ m and effective bearing area $A' = 76$ m$^2$. Following pre-loading, the spudcan has become embedded to a depth of $D = 10$ m in the clay whose strength is $s_{uD} = 30$ kPa at this depth, increasing at a rate $k_{su} = 1.5$ kPa/m. The effective unit weight at this site is $\gamma'_{clay} = 7.3$ kN/m$^3$. The submerged weight of the spudcan and the weight of the soil infill are assumed to be negligible.

(a) Describe the installation steps of a mobile jackup platform. [15%]

(b) Calculate the increase in bearing capacity $Q_v$ of a spudcan following pre-loading compared with the initial phase where the spudcan is simply resting at the soil surface. [15%]

(c) Estimate which soil failure mechanism occurs around the spudcan. Sketch and describe this mechanism. [20%]

(d) An unexpected layer of sand is discovered to be overlying the strata of clay at this site. What risk does this represent for the jackup? [15%]

(e) The sand effective unit weight is equal to $\gamma'_{sand} = 10$ kN/m$^3$ and the layer height of sand is equal to $h_{layer} = 5$ m. Consider first the case where the spudcan punches through the harder layer of sand, beyond the sand-clay interface and has penetrated deep into the clay, to a depth $D = 10$ m from the mudline. The undrained shear strength of the clay at this depth is $s_{uD} = 30$ kPa. Assume that the upper sand layer contributes to the overburden stress and calculate the bearing capacity $Q_v^{deep}$ of the spudcan. Why would the real capacity be higher than the one predicted? [15%]

(f) Consider the case where the spudcan is embedded within the harder layer, at a depth of $h = 2$ m. The punching shear mechanism shown in Fig. 2 is proposed to calculate the bearing capacity. The punching shear coefficient $K_s \tan(\phi')$ that characterises the shear band is equal to:

$$K_s \tan(\phi') = \frac{3s_u}{\gamma'_{sand} B} \tag{1}$$

The undrained shear strength of the clay at the sand-clay interface is $s_u = 22.5$ kPa and the bearing capacity factor $N_c = 9$. Write an expression for the bearing capacity $Q_v^{shallow}$ and calculate its value. [20%]

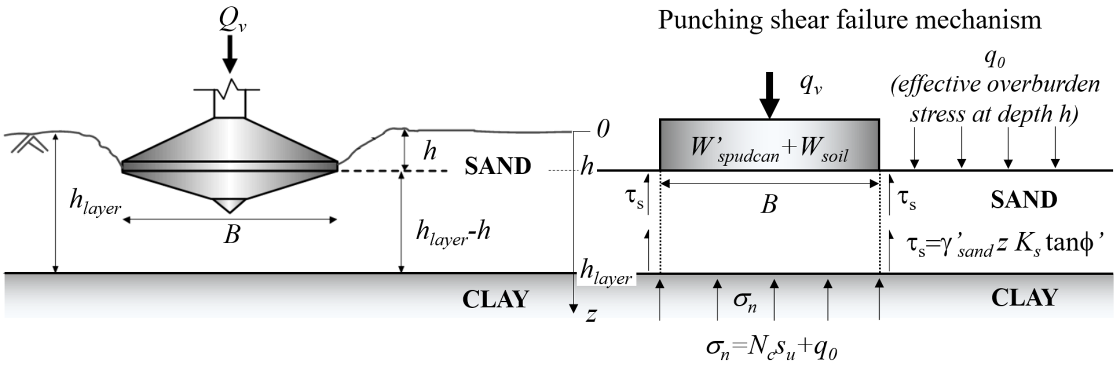



Fig. 2

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# 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:

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

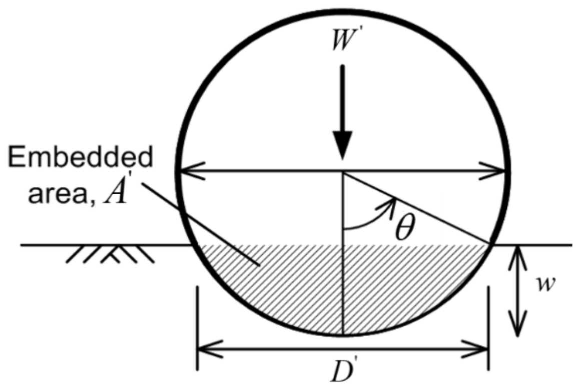
Characteristic length:

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

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:

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

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

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

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

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:

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

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**

*$\alpha$-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.

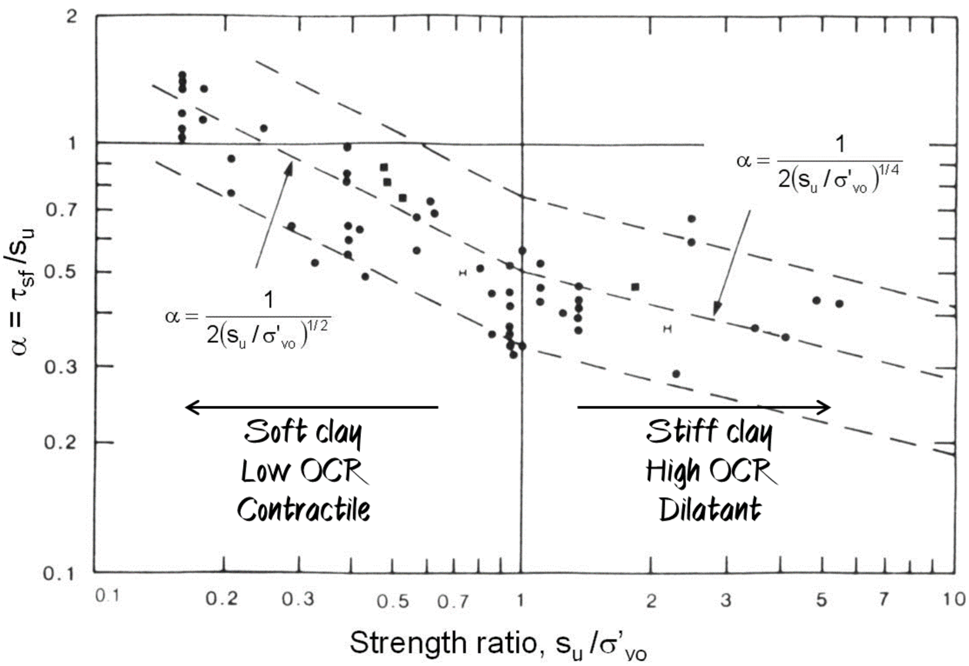



Figure 1: API (2000) $\alpha$-correlations for ultimate unit shaft resistance in clay (Randolph & Murphy 1985)

*$\beta$-Method*:

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

*$\lambda$-Method*:

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

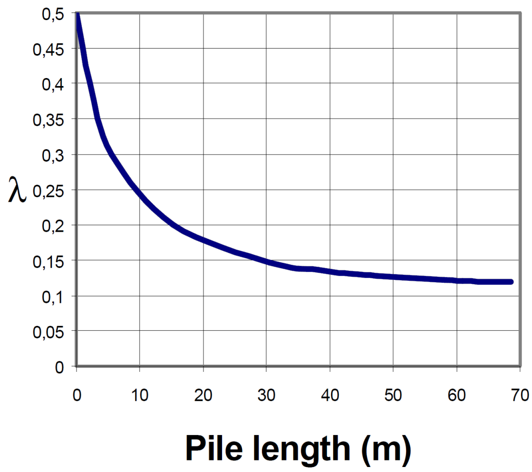



Figure 2: Coefficient $\lambda$ vs. pile length

*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, $\delta$ (°) | Limiting value $\tau_{s,lim}$ (kPa) | Bearing capacity factor, $N_q$ | Limiting value, $q_{b,lim}$ (MPa) |
|---|---|---|---|---|---|---|
| 1 | Very loose<br>Loose<br>Medium | Sand<br>Sand-silt<br>Silt | 15 | 48 | 8 | 1.9 |
| 2 | Loose<br>Medium<br>Dense | Sand<br>Sand-silt<br>Silt | 20 | 67 | 12 | 2.9 |
| 3 | Medium<br>Dense | Sand<br>Sand-silt | 25 | 81 | 20 | 4.8 |
| 4 | Dense<br>Very dense | Sand<br>Sand-silt | 30 | 96 | 40 | 9.6 |
| 5 | Dense<br>Very dense | Gravel<br>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:

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

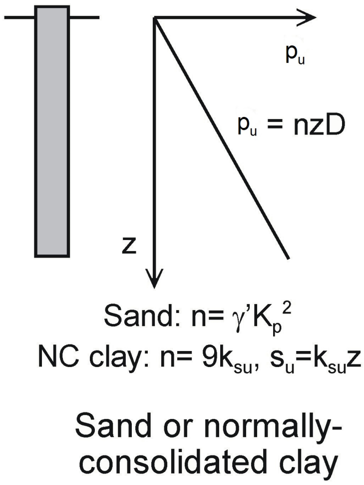
in which:

| | |
|---|---|
| $R$ | Radius of the pile |
| $G_0$ | Initial shear modulus of the soil |
| $z_{IF}$ | Dimensionless zone of influence<br>(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

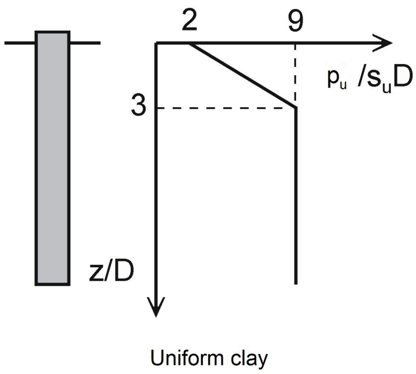
| | |
|---|---|
| Lateral soil resistance (force per unit length): | $p_u = nzD$ |
| In sand: | $n = \gamma' K_p^2$ |
| In normally consolidated clay with strength gradient $k$: | $s_u = kz; n = 9k$ |




| | |
|---|---|
| $H_{ult}$ | Ultimate horizontal load on pile |
| $D$ | Pile diameter |
| $L$ | Pile length |
| $\gamma'$ | 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 |
| $\gamma'$ | 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 $z \leq z_R$, 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 $\epsilon_c$ 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 $\phi$ as shown in Figure 2 (RHS), and where:

| | |
|---|---|
| $z$ | Depth below soil surface |
| $z_R$ | Transition depth<br>below which the value of $(C_1 z + C_2 D)\gamma' z$ exceeds $C_3 D\gamma' z$ |
| $D$ | Pile diameter<br>defined as the radius of the zone of influence around the pile divided by $R$ |
| $\gamma'$ | 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 $\phi$ as given in the Figure below (LHS), and $A$ is a factor to account for static or cyclic loading conditions as follows:

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

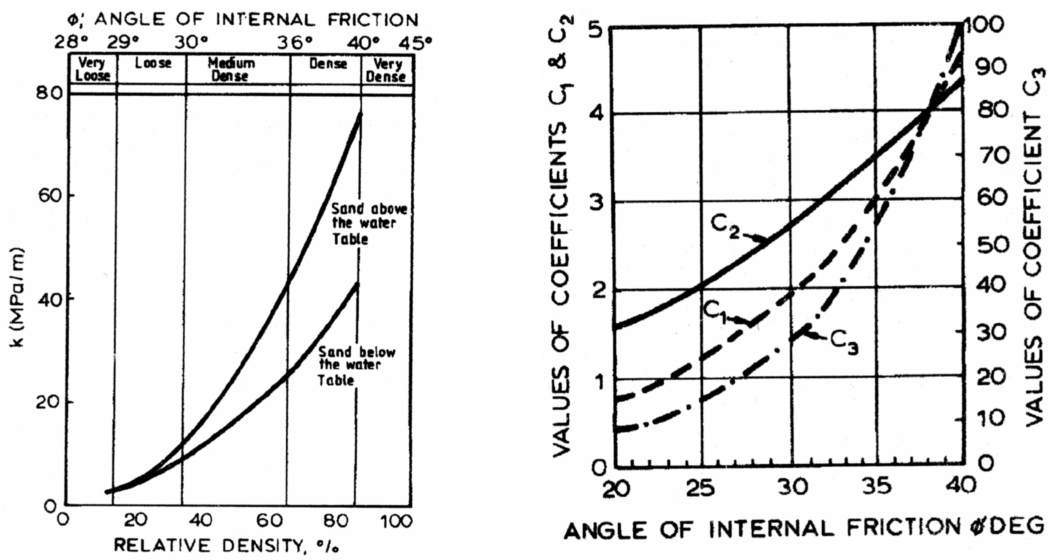



Figure 4: Modulus of subgrade reaction and Empirical coefficients in DNV as a function of friction angle.

# 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:

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

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 $T_a$:

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

Anchor efficiency:

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

## 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 |
| $\rho_s$ | 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:

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

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

$$\left( \frac{H}{H_{ult}} \right)^a + \left( \frac{V}{V_{ult}} \right)^b = 1; \quad a = \frac{L}{D} + 0.5; \quad b = \frac{L}{3D} + 4.5$$

# 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 |

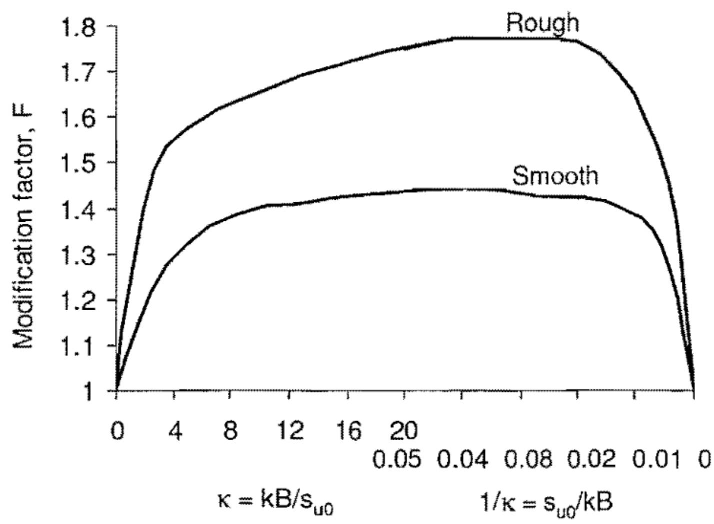



Figure 5: Modification factor $F$, after Davis and Booker (1973).

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 |
| $\gamma'$ | Effective unit weight of the soil |
| $B'$ | Effective width of the foundation |
| $N_\gamma,\ N_q$ | Bearing capacity factors for self-weight and surcharge |
| $K_\gamma, K_q$ | 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. |
| $\phi$ | Effective internal friction angle of the soil |
| $\gamma_m$ | material factor on shear strength |

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

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

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)

| $Embedment\ depth/B$ | Bearing factor, $N_c$ |
|---|---|
| 0 | 6 |
| 0.1 | 6.3 |
| 0.25 | 6.6 |
| 0.5 | 7.1 |
| 1.0 | 7.7 |
| $\geq 2.5$ | 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

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

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

| Friction angle $\phi$ (degrees) | Bearing factor, $N_\gamma$ |
|---|---|
| 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:

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