Unified CPT Method for Foundation Design

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2023

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1 CPT Document

1.1 CPT Correlation

This document presents the correlation implemented in the current code.

1.2 Basic Engineering Properties

1.2.1 Unit Weight

(**?**), refer to say

$$\gamma/\gamma_w = 0.27[\log(R_f)] + 0.36[\log(q_t/p_a)] + 1.236 \tag{1}$$

- γ = unit weight of soil
- γ_w = unit weight of water
- R_f = friction ratio, the ratio of f_s over q_t, i.e, $\frac{f_s}{q_t} \times$ 100%
- q_t = corrected cone resistance
- P_a = atmospheric pressure

1.2.2 Relative Density

> Jamiolkowski (2003), Table 5 on Page 9.

The original form of relative density can be expressed as Eq.(2).

$$D_R = \frac{1}{C_2} \tag{2}$$

$$D_r = \frac{1}{3.10} \cdot \ln\left[\frac{q_t/P_a}{17.68 \cdot (\sigma'_{v0}/P_a))^{0.5}}\right] \tag{3}$$

1.2.3 Friction Angle

$$\varphi'_p = 17.6 + 11 \cdot \log_{10}\left[\frac{q_t/P_a}{(\sigma'_{v0}/P_a)^{0.5}}\right] \tag{4}$$

1.2.4 Small Strain Stiffness G_0

$$G_0 = 50 \cdot \sigma_{atm}\left[(q_t - \sigma_{v0}/\sigma_{atm}\right]^{m*} \tag{5}$$

where:

$$m^* = \begin{cases} 0.6 & \text{sand} \\ 0.8 & \text{silt} \\ 1.0 & \text{clay} \end{cases} \tag{6}$$

1.3 Liquefaction Assessment

1.4 Dissipation Tests

1.5 To-dos

1.6 Unified CPT Method

Pile capacity can be calculated as:

$$Q_{r,c} = Q_{f,c} - Q_p = f(z) \cdot A_s + q_{base} \cdot A_{pile} \tag{7}$$

1.7 Pile Capacity within Clay

1.7.1 α Approach

This method will require the input of the undrained shear strength s_u. To directly link this

$$f(z) = \alpha \cdot s_u(z) \tag{8}$$

$$\alpha = \begin{cases} 0.5 \cdot \psi^{-0.5} & \psi <= 1.0 \\ 0.5 \cdot \psi^{-0.25} & \psi > 1.0 \end{cases} \tag{9}$$

$$\psi = \frac{s_u}{\sigma'_{v0}(z)} \tag{10}$$

1.8 Pile Capacity within Sand

1.8.1 Skin Friction

Skin friction within the sand can be determined using Eq.(11). The method implemented is presented in **Section 8.1.4, page 46** of ISO19901-4.

$$f(z) = f_L(\sigma'_{rc} + \Delta\sigma'_{rd}) \cdot \tan(29^0) \tag{11}$$

where

- f_L is the loading coefficient, **0.75** for tension actions and 1.0 for compression actions.
- 29 is the angle of interface friction used for hte calibration of the method, nothing that factors, such as paint, coating or mill-scale varnish, can negatively affect the interface.
- σ'_{rc} is to the radial confined stress, which can be correlated to the CPT cone tip resistance by Eq.(12).

$$\sigma'_{rc} = \frac{q_c}{44} \cdot A_{re}^{0.3} \cdot [\max(1, \frac{h}{D})]^{-0.4} \tag{12}$$

where

- σ'_{rc} is the horizontal effective stress acting on a driven pile at depth z, about **two weeks** after driving.
- σ'_v is the vertical effective stress ast a depth z.
- A_{re} is the effective area ratio, defined in 14, is a measure of soil displacement induced by the driven pile and expressed as a fraction of hte soil displacement induced by a close-ended pile (for which $A_{re} = 1$).

$$\Delta\sigma'_{rd} = \frac{q_c}{10} \cdot \left(\frac{q_c}{\sigma'_v}\right)^{-0.33} \cdot \frac{d_{ref}}{D} \tag{13}$$

where D is the pile outer diameters, d_{ref}

$$A_{re} = 1 - PLR \cdot \left(\frac{D_i}{D}\right)^2 \tag{14}$$

where

- *PLR* is the Plug Length Ratio, which has a maximum value of 1.0, defined as the ratio of the plug length (L_p) to the pile embedment (L), and for which the absence of measurement, *PLR* shall be taken as 1.0 for typical offshore piles.
- $\Delta\sigma'_{rd}$ is the change in horizontal stress acting at a depth of z, arising due to interface shear dilation when the pile is loaded.
- d_ref = 0.0356m
- h is the distance above pile tip at which $f(z)$ acts (=$L - Z$)

1.8.2 Base Resistance

When the pile has a length to diameter ratio greater than 5, i.e, $L/D > 5.0$, the base resistance Q_b can be calculated in a similar fashion as closed pile

$$Q_b = \begin{cases} q \cdot A_{pile} & L/D > 5(\text{Plugged Case}) \\ q_{c,avg} \cdot A_{re} \cdot A_{pile} & \text{otherwise}(\text{unplugged Case}) \end{cases} \tag{15}$$

Note that A_{pile} is the gross area of the tubular piles. q is the maximum bearing pressure at the based of the plugged pile that can be calculated using Eq.(16). As the maximum value of A_{re} would be 1.0, the maximum stress allowed would be 0.5 average cone resistance.

In case of the unplugged case, i.e., $L/D < 5.0$, this will be the case for large diameter suction bucket. Assuming PLR is taken as 1.0 for normal offshore piles, the unplugged case will equal to the $q_{avg} * A_{annulus}$.

Base resistance can be calculated as Eq.(16).

$$q = [0.12 + 0.38 \cdot A_{re}] \cdot q_{c,avg} \tag{16}$$

When

where

- A_{re} is defined in Eq.(14).
- q_p can normally be adopted as the average q_c below and above the tip level as Eq.(17), however,

> lower q_p value shall be adopted where spatial variability in the cone resistance indicate potential design sensitivity(ISO/DIS 19901-4:2022(E) 2022 p. 44).

$$q_{c,avg} = \frac{1}{3D} \int_{-1.5D}^{1.5D} q_c \cdot dz \tag{17}$$

> ISO/DIS 19901-4:2022(E) (2022),p.44. The method does not directly apply to **Gravel**, pile capacity within Gravel may be overpredicted, including the method proposed in the main text

1.8.3 Skin Friction and End Bearing for Driven Piles in Intermediate Soils

In intermediate soils, CPT would be partially drained. Neither sand or clay method can produce satisfactory results ((ISO/DIS 19901-4:2022(E) 2022 p. 45)) - Higher shaft resistance when soils are assumed to be **Clay** than **Sand** - Could lead to significantly over-predicted axial capacity compared with offshore load tests - In the absence of more definitive criteria, may consider the minimum of the capacity based on Clay or Sand

2 P-y curves for monotonic actions

This section presents the derivation of the p−y curves using ISO/DIS 19901-4:2022(E) (2022). The origin of this update was not clearly documented in the code. ## Clay

$$p_u = N_p \cdot s_u \cdot D \tag{18}$$

where:

- $N_p = N_{p0} + \frac{\gamma' z}{s_u} \leq N_{pd}$
- $N_p = 2N_{p0} \leq N_{pd}$

$$N_p = \begin{cases} N_{p0} + \frac{\gamma' z}{s_u} \leq N_{pd} & \text{if gapping is assumed} \\ 2\ N_{p0} \leq N_{pd} & \text{if no gapping is assumed} \end{cases} \tag{19}$$

$$N_{p0} = N_1 - (1 - \alpha_{ave}) - (N_1 - N_2)\left[1 - \left(\frac{z}{d \cdot D}\right)^{0.6}\right]^{1.35} \tag{20}$$

where:

- $N_1 = 12$
- $N_2 = 3.22$
- $d = 16.8 - 2.3 \log_1 0(\lambda) \geq 14.5$
- $\lambda = s_{u0}/(s_{u1} D)$
- α_{ave} = the average α over a depth of 20m pile diameter
- D = pile diameter
- z = depth below *original* seafloor

> Note: The is refer to the **DSS strength** For normally consolidated condition and $s_u > 15kPa$ for top 10m, **no gap**

For the wedged failure, i.e., $N_p < N_{pd}$, *triaxial extension* should be used instead of *DSS* strength.

$$N_{pcor} = c_w N_{p0} + \frac{\gamma' z}{s_u} \leq N_{pd} \tag{21}$$

where

$$c_w = 1 + \left(\frac{s_{uTE}}{s_{uDSS}} - 1\right) \frac{N_{pd} - N_p}{N_{pd} - N_{p|z=0}} \tag{22}$$

where:

- $\frac{s_{uTE}}{S_{uDSS}} = 0.9$ based on ISO/DIS 19901-4:2022(E) (2022),pp.52

Table 1: Normalised p-y curves for monotonic actions in clay I_P>30%

p/pu	OCR<=2	OCR=4	OCR=10
0	0	0	0
0.05	0.0003	0.004	0.0005
0.2	0.003	0.004	0.005
0.3	0.0053	0.008	0.001
0.4	0.009	0.015	0.021
0.5	0.014	0.024	0.034
0.6	0.022	0.036	0.052
0.7	0.032	0.055	0.078
0.8	0.05	0.084	0.12
0.9	0.082	0.14	0.19
0.975	0.15	0.23	0.3
1.0	0.25	0.3	0.4
1.0	∞	∞	∞

2.1 P-y curve for cyclic actions

$$\begin{aligned} p_{cy} &= p_{mod} \cdot p_{mo} \\ y_{cy} &= y_{mod} \cdot y_{mo} \end{aligned} \tag{23}$$

$$h_f = \begin{cases} \frac{p_{mo}}{p_u} - \left(\frac{z}{z_{rot}}\right)^2 & \text{if } z \leq z_{rot} \\ \frac{p_{mo}}{p_u} - 1 & z > z_{rot} \end{cases} \tag{24}$$

$$N_{eq} = \left(\frac{2}{1-h_f}\right)^g \leq 25 \tag{25}$$

$$g = \begin{cases} 1.0 & \text{Gulf of Mexico Condition} \\ 1.25 & \text{North Sea Soft Clay Condition} \\ 2.5 & \text{North Sea stiff Clay condition} \end{cases} \tag{26}$$

$$p_{mod} = \begin{cases} 1.47 - 0.14 \cdot \ln N_{eq} & \text{Gulf of Mexico} \\ 1.63 - 0.15 \cdot \ln N_{eq} & \text{North Sea soft Clay} \\ 1.45 - 0.17 \cdot \ln N_{eq} & \text{North Sea stiff Clay} \end{cases} \tag{27}$$

$$y_{mod} = \begin{cases} 1.2 - 0.14 \cdot \ln N_{eq} & \text{Gulf of Mexico} \\ 1.2 - 0.17 \cdot \ln N_{eq} & \text{North Sea soft Clay} \\ 1.2 - 0.17 \cdot \ln N_{eq} & \text{North Sea stiff Clay} \end{cases} \tag{28}$$

2.2 p-y curves for fatigue action

$$p_{fa} = p_u A_s \cdot \left(\frac{y_{fa}}{D}\right)^{-B_s} \tag{29}$$

where

- y_{fa} = lateral displacement for fatigue actions
- A_s = 0.45 if s_u < 40kPa, 0.19 otherwise
- B_s = 0.05

2.3 p-y curves for earthquake

2.4 Lateral Capacity for sand

The ultimate lateral resistance of for pile within sand is the minimum of the Eq.(30)

$$p_r = \min\left(p_{rs}, p_{rd}\right) \tag{30}$$

$$p_{rs} = C_1 z + C_2 D \cdot \gamma' z \tag{31}$$

$$p_{rd} = C_3 \cdot D \cdot \gamma' \tag{32}$$

$$C_1 = \frac{(tan\beta)^2 \tan\alpha}{\tan\left(\beta - \phi'\right)} + K_0\left(\frac{\tan\phi' \tan\beta}{\cos\alpha \tan\left(\beta - \phi'\right)} + \tan\beta(\tan\phi' \sin\beta - \tan\alpha)\right) \tag{33}$$

$$C_2 = \frac{\tan\beta}{\tan\left(\beta - \phi'\right)} - K_a \tag{34}$$

$$C_3 = K_a((\tan\beta)^8 - 1) + K_0 + \tan\phi'(\tan\beta)^4 \tag{35}$$

ISO/DIS 19901-4:2022(E). (2022). *Petroleum and natural gas industries — Specific requirements for offshore structures —Part 4: Geotechnical design considerations*.