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Determination of p-y curves for offshore piles based on in-situ soil investigations

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Abstract. Offshore piles are subjected to complex loads with considerable lateral component. The pile-soil response to lateral loads can be described with the p-y method. For a given depth the load–deflection relationship is built to simulate the surrounding soil stiffness. This state-of-art paper presents a brief discussion of determination methods for the p-y curves using a standard approach based on the soil parameters derived from laboratory and in-situ tests or directly from field tests. The basic relationships for both cohesive and cohesionless soils are discussed. The advantage of direct design methods to describe the p-y curve relies in the reduction of necessary laboratory tests.

1 Introduction

Nowadays, most of offshore wind farms in Europe are located on waters with depths up to 20 m, and the most popular type of foundation structures are monopiles. These structures are appropriate for water depths up to 35 m. They are made of steel, cylindrical pipes with diameters up to 9 m and wall thickness up to 150 mm [1, 2].

This article focuses on free-headed piles subjected to horizontal loads only (no bending moments). The piles under lateral loads can be divided into three categories:

- flexible,
- rigid,
- of intermediate stiffness.

The lateral resistance for piles under horizontal loading can be calculated by constructing non-linear load-deflection (p-y) curves. This method is most common and recommended in e.g. the Offshore Standards DNV-OS-J101 [3] or in the API Recommended Practice 2A-WSD [4]. The API p-y formulation is proper for long flexible piles with diameters of up to 1m and the L/D ratio around 30. A pile is considered to be long flexible if it meets the condition [5, 6]:

$$L > 3 \cdot l_0 \ with \ \ l_0 = \left(\frac{4 \cdot E_p \cdot I}{K} \right)^{1/4} \tag{1}$$

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where:

l_0 transfer length,
E_p modulus of the pile material,
I pile moment of inertia,
K soil stiffness.

The monopiles are relatively short and rigid of 10 times greater diameters and length/diameter (L/D) ratio of values lower than 30. They can be considered short rigid piles if their length satisfies the relationship [5]:

$$L < l_0 \tag{2}$$

The cases between the relations from eq. (1) and (2) are intermediate and can be considered either flexible or rigid. The lateral response of rigid piles is still often modelled with the use of p-y curves, as for flexible piles, which may not accurately describe their behavior. For this reason, many studies and field testing have been carried out to develop a method more suitable for monopiles [6]. The p-y curves are used to model the soil stiffness as non-linear springs applied between beam-column elements - representing pile in foundation analysis. A division of methods used to determine p-y curves is presented in this article.

In the design of offshore support structures it is vital to perform geotechnical tests: both in-situ (CPT/less often DMT test) and laboratory tests (on undisturbed or evenly disturbed samples) [7].

The aim of the work is to collect and discuss the available methods applied to create p-y curves during the offshore pile modelling. The article is the introduction to further research, applying the presented calculation methods and formulas as an input to create an offshore pile model. The presented review of direct and indirect methods for piles subjected to lateral loads considers mainly flexible piles. The next step of the research intends to modify the given approach and adjust it to the behaviour of rigid monopile.

The determination of soil stiffness using CPT and DMT is a starting point for the study of monopile foundation. However, the behaviour mechanism of such a foundation is different from the flexible pile case. Lateral deformations of rigid foundations are more uniform, interaction occurs between the base and shaft mechanism, as the monopiles resembles a block foundation. Additional model tests and numerical analysis are necessary to better analyse this behaviour.

2 Lateral stiffness of piles up to 2,5 m of diameter

2.1 Indirect approach

The non-linear p-y curves presented in API RP 2A-WSD [4] and DNV CN 30.4 [7] are used to design laterally loaded pile structures with diameters up to 2,5 m. They can be also applied in case of not ideally flexible piles.

Construction of p-y curves depends on the soil type and differs for cohesive and cohesionless soils. In the case of cohesionless soils it is necessary to estimate the effective friction angle..

In the case of cohesive soils the ultimate lateral resistance is based on undrained shear strength (gained from laboratory testing) to be calculated as [4]:

$$p_u = \begin{cases} (3 \cdot c_u + \gamma' \cdot X) \cdot D + J \cdot c_u \cdot X & for \quad 0 < X < X_R \\ 9 \cdot c_u \cdot D & for \quad X > X_R \end{cases} \tag{3}$$

where:

X	depth below soil surface,
X_R	transition depth, below which the value $((3c_u+\gamma' X)D+Jc_uX)$ exceeds $9c_u$ [m],

$$X_R = \frac{6 \cdot D}{\gamma' \cdot \dfrac{D}{c_u} + J} \tag{4}$$

where:

D	pile diameter,
c_u	undrained shear strength for undisturbed soil samples,
γ'	effective unit weight of soil,
J	empirical coefficient with values from 0,25 to 0,50 (the upper limits value is assumed for soft, normally consolidated cohesive soils).

In static loading:

$$p_u = \begin{cases} \dfrac{p_u}{2} \cdot \left(\dfrac{y}{y_c}\right)^{\frac{1}{3}} & for\ y \le 8 \cdot y_c \\ p_u & for\ y > 8 \cdot y_c \end{cases} \tag{5}$$

In cyclic loading and $X>X_R$:

$$p_u = \begin{cases} \dfrac{p_u}{2} \cdot \left(\dfrac{y}{y_c}\right)^{\frac{1}{3}} & for\ y \le 3 \cdot y_c \\ 0{,}72 \cdot p_u & for\ y > 3 \cdot y_c \end{cases} \tag{6}$$

In cyclic loading and $X \le X_R$:

$$p = \begin{cases} \dfrac{p_u}{2} \cdot \left(\dfrac{y}{y_c}\right)^{\frac{1}{3}} & for\ y \le 3 \cdot y_c \\ 0{,}72 \cdot p_u \cdot \left(1 - \left(1 - \dfrac{X}{X_R}\right)\right) \cdot \dfrac{y - 3 \cdot y_c}{12 \cdot y_c} & for\ 3 \cdot y_c < y \le 15 \cdot y_c \\ 0{,}72 \cdot p_u \cdot \dfrac{X}{X_R} & for\ y > 15 \cdot y_c \end{cases} \tag{7}$$

with $y_c = 2{,}5 \cdot \varepsilon_c \cdot D$

where:

ε_c	strain which occurs at one-half the maximum stress in laboratory undrained compression tests.

In the case of cohesionless soils, p-y curves are also non-linear but can be approximated [4]:

$$p = A_i \cdot p_u \cdot \tanh\left(\frac{k_{init} \cdot X}{A_i \cdot p_u} \cdot y\right) \tag{8}$$

where:

- A_i factor to account for static or cyclic loading: $A_i=(3-0{,}8(\cdot z/D))>0{,}9$ for static and $A_i=0{,}9$ for cyclic loading,
- p_u static ultimate resistance at depth z,
- Y lateral deflection,
- k_{init} initial modulus of subgrade reaction dependent on the friction angle (see [3]).

The value of ultimate lateral resistance is soil depth dependent. It has been found that in the case of shallow waters it can be calculated from eq. (9) in the case of deeper waters from eq. (10). At a particular depth, the equation leading to lower value should be considered decisive [4].

$$p_{us} = (C_1 \cdot X + C_2 \cdot D) \cdot \gamma' \cdot X \tag{9}$$

$$p_{ud} = C_3 \cdot D \cdot \gamma' \cdot X \tag{10}$$

where:

- C_1, C_2, C_3 coefficients dependent on the friction angle ϕ' (see [3]).

The friction angle can be obtained from laboratory tests or in-situ data. If the laboratory data is not available, the values of effective friction angle in sands can be evaluated from CPTU using eq. (11) [8] or eq. (12) [9]:

$$\phi'_{CPT} = \arctan\left[0{,}1 + 0{,}38 \cdot \log\left(\frac{q_c}{\sigma'_{v0}}\right)\right] \tag{11}$$

$$\phi'_{CPT} = 17{,}6° + 11{,}0 \cdot \log(q_{c1}) \tag{12}$$

where:

$$q_{c1} = \left[\frac{q_c}{Pa}\right] \cdot \left[\frac{Pa}{\sigma'_{v0}}\right]^{0{,}5} \tag{13}$$

with $Pa = 100\ kPa$, σ'_{v0} - vertical effective stress [kPa].

Safe estimation of effective angle of internal friction in sands can be also determined from DMT [10]:

$$\phi'_{DMT} = 28° + 14{,}6 \cdot \log K_D - 2{,}1 \cdot \log^2 K_D \tag{14}$$

In the case of undrained shear strength, the following equations can be used for clays:

$$c_{u,CPT} = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{15}$$

$$c_{u,DMT} = 0{,}22 \cdot \sigma_{v0} \cdot (0{,}5 \cdot K_D)^{1{,}25} \tag{16}$$

where:

- N_{kt} cone factor for clays (default to 15); $N_{kt}=10{,}5+7\cdot log(F_r)$
- F_r normalized friction ratio,

$$F_r = \frac{f_s}{q_t - \sigma'_{v0}} \cdot 100\% \quad (17)$$

K_D horizontal stress index; $K_D=(p_0-u_0)/\sigma_{v0}$,
p_0 pressure applied to the soil at the start of the expansion,
u_0 hydrostatic pore water pressure.

2.2 Direct approach

This approach uses the results of advanced in-situ soil investigations to predict the p-y curve and pile response to lateral loading. It reduces the errors due to inaccurate estimation of undrained shear strength and friction angle with the use of eq. (11),(12),(15), it also eliminates the need for laboratory testing on undisturbed soil samples [11]. Based on API and DNV methods Lehane and Truong [12] determined the construction of p-y curves directly from CPT data. They suggested the following equations for soft clay [12]:

$$\frac{p}{p_u} = \tanh \cdot \left[(026 \cdot I_r + 3{,}98) \left(\frac{y}{D} \right)^{0{,}85} \left(\frac{X}{D} \right)^{-0{,}5} \right] for\, 0 < \frac{X}{D} < 3 \quad (18)$$

$$\frac{p}{p_u} = \tanh \cdot \left[(0{,}151 \cdot I_r + 2{,}3) \left(\frac{y}{D} \right)^{0{,}85} \right] for\, \frac{X}{D} > 3 \quad (19)$$

where:

$$I_r = \frac{1}{3} \left(\frac{E}{q_n} \right) \cdot N_{kt} \quad (20)$$

and
I_r rigidity index G/c_u,
G shear modulus (from eq. (22))
q_n net cone resistance; $q_n=q_t-\sigma_{v0}$,
q_n cone tip resistance; $q_t=q_c+(1-a)\cdot u_2$,
a cone coefficient,
E Young's modulus (from eq. (21)).

$$E = (q_t - \sigma_{v0}) \cdot 0{,}015 \cdot \left[10^{0{,}55 \cdot I_c + 1{,}68} \right] \quad (21)$$

$$G = \frac{\gamma}{9{,}8} \cdot (51{,}6 \ln f_s + 18{,}5)^2 \quad (22)$$

Regarding sands, Lehane and Suryasentana (2014) [13] proposed:

$$\frac{p}{y \cdot X \cdot D} = 2{,}4 \cdot \left(\frac{q_c}{y \cdot X} \right)^{0{,}67} \cdot \left(\frac{x}{D} \right)^{0{,}75} \cdot \left(1 - e^{-0{,}62 \cdot \left(\frac{X}{D} \right)^{-1{,}2} \cdot \left(\frac{y}{D} \right)^{0{,}89}} \right) \quad (23)$$

Another method (Robertson, 1989) [14] to obtain the p-y curves for laterally loaded piles is based on DMT data. For cohesive soils, the equation of p-y curves is the same as in the API Recommendations (eq. (3),(4)), however the equation for the pile deflection changes [15,16] and y_c can be evaluated from:

$$y_c = \frac{23{,}67 \cdot c_u \cdot D^{0{,}5}}{F_c \cdot E_D} \tag{24}$$

where:

F_c empirical factor $F_c=10$,

E_D dilatometer modulus.

$$E_D = 34{,}7 \cdot (p_1 - p_0) \tag{25}$$

and p_0 and p_1 – corrected readings from DMT.

For cohesionless soils, also the eq. (3) is used, however the ultimate lateral resistance p_u is the lower value from eq. (26) (Reese, 1974) [17] and eq. (27) [18]:

$$p_u = \sigma'_{v0} \cdot \left[D \cdot \left(K_p - K_a \right) + x \cdot K_p \cdot \tan \phi' \cdot \tan \beta \right] \tag{26}$$

$$p_u = \sigma'_{v0} \cdot D \cdot \left[K_p^3 + 2 \cdot K_a \cdot K_p^2 \cdot \tan \phi' - \tan \phi' - K_a \right] \tag{27}$$

where:

$$\beta = 45° + \frac{\phi'}{2} \tag{28}$$

$$y_c = \frac{4{,}17 \cdot \sin \phi' \cdot \sigma_{v0}'}{E_D \cdot F_\phi \cdot (1 - \sin \phi')} \cdot D \tag{29}$$

and

$F_\phi = 1$,

K_a active earth pressure coefficient,

K_p passive earth pressure coefficient.

Parameters such as friction angle and undrained shear strength are calculated with the help of equations (14) and (16), applying the data from the DMT test [19].

3 Monopile lateral stiffness

Application of the p-y method, according to the API and DNV guidelines overestimates the subgrade modulus thus produces enormous results if applied to monopiles [1,20]. As a result, many solutions based on both analytical and three-dimensional analysis have been developed [21]. For the purpose of the article, only some of the developed methods are briefly presented. One of the methods, formulated for cohesive soils, was proposed by Dunnavant & O'Neil [22,23]. The main idea of API Recommendation formulas remains the same, however the expression for the static ultimate lateral resistance p_u and critical deflection y_c changes [24]:

$$p_u = c_u \cdot D \cdot \min \cdot \left\{ 2 + \frac{\sigma'_v(X)}{c_{ua}} + \frac{J}{D} \cdot X, 9 \right\} \tag{30}$$

$$y_c = 0{,}0063 \cdot \left(\frac{E_s \cdot L_r^4}{E \cdot I} \right)^{0{,}875} \tag{31}$$

where:

c_{ua} average shear strength between the layer surface and depth X,
E_s secant modulus,
L_r representative pile length.

$$L_r = \min \left\{ \left(\frac{E \cdot I}{E_s \cdot D^4} \right)^{0{,}286}, 3D, L \right\} \tag{32}$$

Similar simplified approach was proposed by Mayne et al. (1992) [25].

These alternative formulations improve the monopile design mainly in overconsolidated clays.

For cohesionless soils a new approach was developed by Thieken et al. (2015) [20]. In this method, a deflection-dependent correction factor, is applied to increase the p_u value at shallow depths. Calculations are made by means of an iterative procedure. In the first step, deformations are calculated with the use of basic p-y curves. Next, the correction factor is applied to the curve and the calculation of deflection is repeated. These steps are reiterated until no significant change is noticed [20].

Apart from analytical approaches, numerical methods are also applied to verify their results [26]. Moreover, model tests in a reduced scale are carried out for a better understanding of the pile – soil interaction. Geotechnical centrifuge is used to investigate the response of model monopiles subjected to different loads [27]. Next, model observations can be extrapolate to analyse the prototype behaviour [28].

Despite many attempts, there is still no specific method for estimating lateral stiffness for large diameter piles. This problem remains unsolved and requires more analysis and systematic research.

4 Conclusions

The advantage of direct method to establish p-y curve is the reduction of extent of laboratory tests on high quality samples. The formulations should be however well calibrated and adjusted to given soil conditions. Following direct approach it is impossible to completely reject laboratory testing and the use of correlation between soil strength characteristics and parameters measured by in-situ soil investigation. The well-developed direct and indirect approaches exists for flexible piles up to 2,5 m in diameter. The monopiles used in offshore structures are generally rigid piles, so they cannot be analysed using any standard p-y approach for flexible piles. The case of rigid or intermediate piles needs further efforts considering analytical and numerical solutions combined with advanced physical modelling of centrifuge tests.

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