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Design of Laterally Loaded Single Piles by Using P-Y Curves and the Cone Penetration Test (CPT) in Sandy Soils

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ABSTRACT

The aim of this paper is to present a simple method of construction of the load-transfer P-Y curves for the design of laterally loaded piles in sand based on the cone penetration test (CPT). The proposed method was developed on the basis of interpretation of 5 field tests on single instrumented piles conducted in sandy sites in France and shows a simple relationship linking the P-Y curve parameters, the cone penetration resistance and the lateral pile/soil stiffness ratio.

The validation process was carried out by direct comparison of the predicted load-deflection curves based on the proposed method to those obtained from a worldwide case history of field lateral loading tests on piles and showed a very good quality of the prediction using the proposed method.

KEYWORDS: Piles, Lateral load, Sand, Full-scale loading, P-Y curves, CPT test.

INTRODUCTION

The accumulation of case histories of field lateral loading tests on piles carried out worldwide shows that the load-deflection behaviour is governed by a multitude of parameters involved in such an interaction. The theoretical modeling of laterally loaded piles is a difficult task due to the 3D response of the pile/soil system. In engineering practice, pile/soil interaction analysis is usually undertaken within the framework of a serviceability limit state (SLS) design, the pile deflection being either measured from a full-scale loading test or computed based on a variety of methods, such as elasticity-based methods (Banerjee & Davis, 1978; Budhu & Davies, 1987; Randolph, 1981; Poulos & Hull, 1992), numerical methods (Haouari & Bouafia, 2019; Khedija et al, 2020) and the P-Y curve methods (Matlock & Reese, 1960; Ménard et al., 1969; Baguelin et al, 1978; Reese & van Impe, 2001; Briaud, 2013).

In some particular pile/soil configurations (offshore structures, monopiles of wind turbines, … etc.) working under severe lateral loading conditions, an ultimate limit state (ULS) design should be carried out involving the lateral soil resistance (Ménard, 1969; Reese et al., 1974; Reese & van Impe, 2001).

It is nowadays recognized in engineering practice that the methods based on P-Y curves' concept offer a powerful framework to the analysis of the pile response under lateral load, with the advantage to account for the non-homogeneous distribution of the soil properties and the non-linear response of the pile/soil system. According to the P-Y curves' theory, a series of non-linear springs along the pile is used to model the pile/soil interface, where a spring subjected to the soil reaction P, at a given depth, exhibits a lateral displacement Y. A typical P-Y curve is depicted in Figure 1, where the main parameters are the initial reaction modulus E_{s0} and the lateral soil resistance P_u. According to the experience of full-scale lateral loading tests of piles, the shape of such a curve is usually non-linear and characterized by an initial linear portion corresponding to small pile deflections, as well as by a non-linear portion followed by a horizontal segment BC describing the soil failure around the pile.

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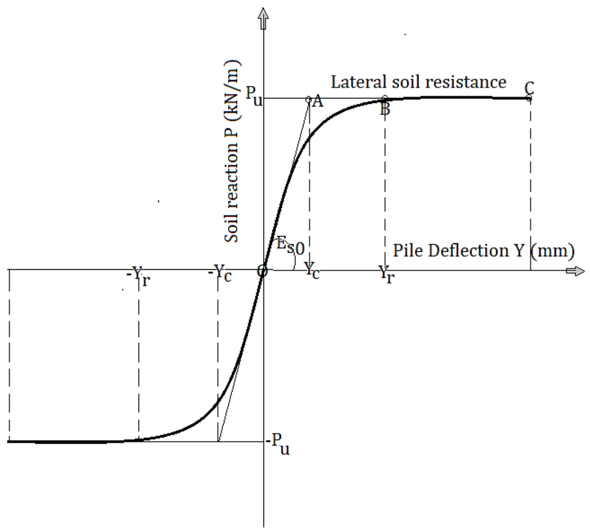


Figure (1): Schematic P-Y curve

In practice, P-Y curves are usually derived from the profiles of bending moment obtained from measurements by strain gauges along the test pile. However, successful derivation of P-Y curves from the process of double differentiation and double integration of the profile of bending moment to obtain respectively P and Y is rarely reported in the literature. Such a fact may be explained by the difficulty to derive the P-Y curves due to the high sensitivity of the lateral soil reaction P to the quality of measurements as well as to the technique of fitting and differentiation of the bending moments (Bouafia and Garnier, 1991).

Although the CPT test is one of the most commonly used *in-situ* Geotechnical tests and widely used as a foundation design tool, it is rarely recommended how to use it as an analysis tool of the pile load-deflection behavior. This paper aims at presenting a simple method to construct the P-Y curves for a single pile embedded in sandy soil based on the cone penetration test (CPT) data. This method was developed from the detailed analysis of full-scale lateral loading tests carried out on 5 fully instrumented piles in 2 sandy sites in France, within the framework of a research work undertaken by the LCPC (Laboratoire Central des Ponts & Chaussées) during the last five decades (Bouafia, 2017). An assessment of the quality of prediction of the proposed method was carried out by a direct comparison of the predicted load-deflection response of a small-sized database of full-scale single piles loaded in sandy soils to the experimental results.

Brief Review of the Methods of P-Y Curves

Reese and Matlock (1956) were the pioneer researchers who studied the P-Y curves by introducing the concept of lateral reaction modulus, initially defined by Winkler (1867). The proposed P-Y curves were bilinearly shaped, assuming an elastic plastic response at the pile soil/interface depicted by the portion OABC in Figure 1 (Reese and Matlock, 1956).

As from the sixties, the *in-situ* tests, such as the pressure meter (PMT) and the CPT tests, have become widely used tools of analysis and design of pile foundations. The PMT test was used by Louis Ménard, the inventor of this test and the pioneer of the pressure meter theory and subsequent studies proposed many refinements and improvements of the basic method (Dunand, 1981; Baguelin et al., 1978; Robertson et al., 1984; Briaud et al, 1985; Bouafia, 2007).

The CPT test has many advantages compared to the PMT test, the main of which are the continuity of measurement of the soil resistance and the simplicity of the experimental procedure. However, the PMT test has the advantage to obtain a complete stress-strain curve at a given depth, which allows a direct evaluation of the soil stiffness. Nonetheless, in light of modern theories, like the theory of cavity expansion (Ladanyi, 1963; Vesic, 1972; Ladanyi and Longtin, 2005) and the theory of strains' path (Baligh, 1985; Baligh and Levadoux, 1986), the cone resistance clearly depends on the soil compressibility and may thus be seen as a tool of indirect evaluation of the soil stiffness. Morover, some experimental research works demonstrated the possibility of correlation of the stiffness of sandy soils to the cone resistance q_c (Jamiolkovski, 1988; Lunne et al., 1997).

The first approach of construction of a P-Y curve based on the cone penetration resistance q_c was proposed by Schmertmann (1978) who recommended an elastic perfectly plastic function for the P-Y curve (curve OABC in Figure 1), where the lateral reaction modulus E_{s0} depends on q_c and the pile diameter B, whereas the lateral soil resistance P_u is taken equal to 11% of q_cB in loose sand and 22% of q_c.B in dense sand.

A series of centrifuge tests on a 0.36-m diameter steel pile driven into calcareous sand led Novello (1999) to propose a description of the P-Y curve by a power function as follows:

$$P = min\left\{2B(\gamma' z)^{1/3} {q_c}^{2/3} \left(\frac{Y}{B}\right)^{1/2}, \mathrm{B}.q_c\right\} \tag{1}$$

where P, Y, z, B and γ' are respectively the lateral soil reaction, the pile deflection, the soil depth, the pile diameter and the submerged unit weight, expressed respectively in kN/m, m, m, m and kN/m^3. The lateral soil resistance was simply defined by the product q_c.B.

The above equation was modified by Dyson and Randolph (2001) to better fit their experimental results of centrifuge tests on laterally loaded piles driven into a calcareous sandy soil, as follows:

$$P = 2.84\gamma' B^2 \left(\frac{q_c}{\gamma' B}\right)^{0.72} \left(\frac{Y}{B}\right)^{0.64}. \tag{2}$$

Based on an extensive centrifuge test on piles embedded in dry dense to very dense sand, Bouafia (2009) proposed hyperbolic shaped P-Y curves and showed the possibility to correlate the lateral reaction modulus E_{s0} to the drained initial constraint modulus M_0 and P_u to q_c, where the modulus M_0 was empirically correlated to q_c as proposed by Eslaamizaad and Robertson (1996) who worked on unaged and uncemented silica sands in a calibration chamber.

Suryasentana and Lehane (2014a) performed a parametric numerical study based on a 3D finite element model of a single pile embedded in sand and proposed the following equation:

$$P = 2.4\gamma z B \left(\frac{q_c}{\gamma z}\right)^{\frac{2}{3}} \left(\frac{z}{B}\right)^{\frac{3}{4}} (1 - exp\left\{-6.2 \left(\frac{z}{B}\right)^{-1.2} \left(\frac{Y}{B}\right)^{0.89}\right\}). \tag{3}$$

For short rigid piles, Kim et al. (2015) proposed an equation of the lateral resistance P_u based on the work of Lee (2010) as follows:

$$\mathrm{P}_u = 2.775 B {q_c}^{0.391} {\sigma_m}'^{0.609} \tag{4}$$

where σ'_m is the mean effective stress. Note that in all these equations, the soil reaction P varies as a power of q_c whereas a simpler approach consists of considering a direct proportionality between P and q_c as will be shown hereafter in light of experimental results obtained from field lateral tests on piles.

The French standard NF P94-262 for deep foundations, accompanying the Eurocode 7, recommends for piles in sand under permanent lateral loading an elastic perfectly plastic P-Y curve defined by a lateral reaction modulus E_{s0} equal to (2.25q_c) and a soil resistance P_u represented by an asymptote equal to (q_c.B/13). Under short-duration loads, E_{s0} is doubled and P_u is kept the same as under permanent loads (AFNOR, 2012).

It should be noted that the previous formulations ignore the effect of the pile/soil stiffness ratio on the load-deflection behaviour of a laterally loaded pile, which implicitly leads to the application of these methods for both short rigid piles and long flexible piles, in contrast with the experimental observations of many full-scale pile tests (Bouafia, 2007). In fact, it is possible to correlate the reaction modulus E_{s0} and the soil resistance P_u at a given depth z to q_c and the pile properties on the basis of the following equation:

$$\mathrm{f}\,(q_c^*, E_p I_p, \mathrm{D}, \mathrm{B}, P_u, E_{s0}) = 0 \tag{5}$$

where q_c* is the net cone resistance taking into account the initial stress state and given by:

$$\text{where} \quad q_c^*(z) = q_c(z) - \sigma_{v0}(z) \tag{6}$$

where q_c and $\sigma_{v0}(z)$ are, respectively, the cone resistance and the initial vertical overburdern pressure at depth z.

The pile flexural stiffness, the pile embedded length and the pile diameter (or the dimension perpendicular to the lateral load direction) are respectively denoted by E_pI_p, D and B. Equation (5) may be transformed by a dimensional analysis based on the Buckingham's theorem which leads to:

$$g\left(\frac{E_{s0}}{q_c^*}, \frac{P_u}{q_c^* B}, \frac{D}{B}, \frac{E_p I_p}{q_c^* D^4}\right) = 0. \quad (7)$$

This equation relates 4 dimensionless variables, whereas Equation (6) relates 6 physical parameters. The first term is called "the modulus number" and hereafter noted K_E as follows:

$$E_{s0} = K_E q_c^*. \quad (8)$$

Similarly, the second term is called "the lateral resistance factor" and noted K_c leading to compute the soil resistance as follows:

$$P_u = K_c q_c^* B. \quad (9)$$

The third term is the pile slenderness ratio and the last one is called "the lateral pile/soil stiffness ratio" and noted K_R. On the basis of equation 7, K_E as well as K_c depend on K_R and D/B. Moreover, since K_E corresponds to small pile displacements and K_c corresponds rather to large displacements, they should be independent. Equation (7) may therefore be uncoupled to the two following equations:

$$K_E = \frac{E_{s0}}{q_c} = h(q_c^*, K_R, \frac{D}{B}); \quad (10)$$

$$K_c = \frac{P_u}{q_c B} = j(q_c^*, K_R, \frac{D}{B}). \quad (11)$$

Determination of the functions h and j leads to practical formulation of the P-Y curves' parameters, as given by Equations (8) and (9). The experimental analysis of the coefficients K_E and K_c on the basis of the lateral loading test on full-scale single piles offers a pragmatic framework.

Description of Field Tests on Piles

Geotechnical Aspect of the Experimental Sites

Within the scope of a research project undertaken during 1987-88, 5 full-scale lateral loads were applied in 2 sandy sites in France. The first site, noted S_1, is located in Châtenay-sur-Seine, 70 km south-east of Paris and the second one, noted S_2, is located in Le-Rheu, 5 km south-west of Rennes. Site S_1 is composed of two layers of a poorly graded sand (S_p) called Fontainebleau sand; the first layer has a thickness of 1.80 m of dense sand (I_D=57%) overlying a loose layer (density index I_D=37%) with a thickness of 1.40 m. Site S_2 is composed of poorly graded clean sand (S_p) and the groundwater level is 10 m deep. The average water content of the sand above water table is 8%, which corresponds to a saturation degree of 31%. Geotechnical *in-situ* tests, notably the PMT, CPT and DPT (dynamic penetration test), were carried out at the location of test piles and analyzed elsewhere (Bouafia, 2007). The CPT test was carried out in both sites with a conic tip having an apex angle of 60°, a diameter of 35.7 mm and a sleeve friction having an area of 150 cm^2, which is in accordance with the international standard ISO-22476-1 (AFNOR, 2013).

The profile of the cone resistance $q_c(z)$, z being the depth of measurement, is usually irregular due to the presence of many peaks registered, which is mainly due to the performance of the penetration device rather than to a relatively high local penetration resistance. In practice, the profile $q_c(z)$ is interpreted according to different approaches, like smoothing, interpolation or filtration. During the development of the proposed CPT-based method, the conventional procedure of interpretation of the profile $q_c(z)$ was adopted, consisting in filtering the profile $q_c(z)$ by suppressing and interpolating the peaks of resistance, which implies the use of a relatively smooth profile of cone resistance. The method presented hereafter is then based on a filtered profile and not on the gross profile of resistance.

Piles

The test piles are open-ended steel pipes and are equipped by pairs of strain gauges along two diametrically opposite axes. Piles T_5, T_{10} and T_{15} were jacked in site S_1 and piles P_1 and P_2 were bored in site S_2. The margin of the ratio embedded length /diameter of the test piles lies between 5.5 and 15.3.

As shown in Figure 2, a hydraulic jack in contact with a concrete block of reaction was used to apply the lateral load which is measured by a high-precision electric load cell.

The program of loading consisted of a series of monotonic lateral loads at the pile top; the duration of each load increment was 15 minutes in site S_1 and 2 hours in site S_2.

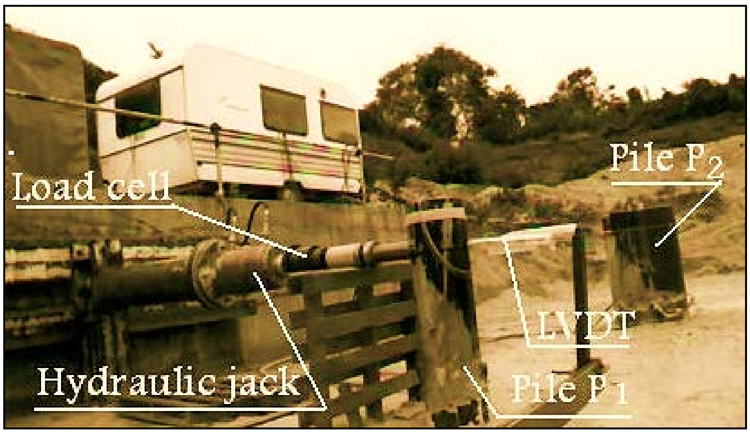
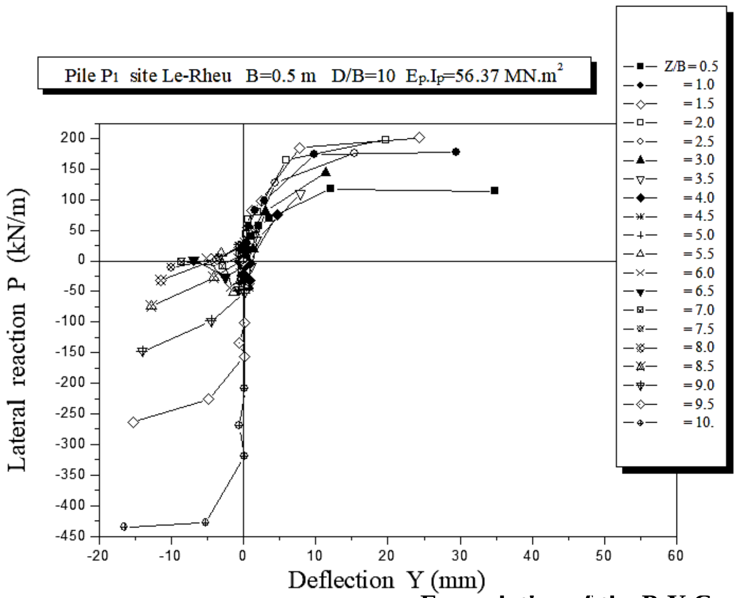


Figure (2): Experimental configuration of the test piles P_1 and P_2 in site S_2

Construction of Experimental P-Y Curves

For a given load increment, the profile of the measured bending moment M(z) was obtained from the axial strains provided by the strain gauges along the pile. The fitting process of the bending-moment profile was carried out by using quintic spline functions or polynomial functions, the choice of the fitting function being governed by the criterion of equilibrium of the pile under the lateral reaction profile P(z) and the load applied at the pile top within a given tolerance (Bouafia & Garnier, 1991). The lateral displacement Y(z) along the pile may easily be obtained by two successive integrations of M(z). Moreover, the soil reaction P(z) is determined by two successive differentiations of M(z) and then the P-Y curve at any depth may be constructed. In Figure 3, a typical set of experimental P-Y curves for the pile P_1 is illustrated, where it can be noticed that there is an inherent non-linearity in the shape of the curves as well as a regular increase in soil stiffness with depth. These curves are marked by a simultaneous change in sign for the deflection Y(z) and the soil reaction P(z) at about 6 diameters of depth, which is in accordance with the Winkler's hypothesis, according to which P(z) and Y(z) have the same sign. Moreover, for relatively large deflections, exceeding a threshold of about 3% of B, the P-Y curves exhibit asymptotic values, indicating a full mobilization of the lateral soil resistance.



Analysis of P-Y Curve Parameters

Formulation of the P-Y Curve

To derive the P-Y curve parameters; namely, E_{s0} and P_u, the experimental P-Y curves were fitted on the basis of a simple analytical function called **PARECT** (**PA**rabola-**RECT**angle), which is able to capture the non-linear shape of the curve (portion OB in Figure 1), as well as the mobilization of the soil resistance described by an asymptote (portion BC in Figure 1) as from a threshold deflection $Y_r(z)$ called "reference deflection". The **PARECT** function is defined as follows:

$$P(z) = E_{s0}(z).Y(z).\left(1 - \frac{Y(z)}{2Y_R\ (z)}\right). \quad (12)$$

Since at the deflection Y_R the soil reaction P reaches P_u, the P-Y curve may be written as:

$$P(z) = 2P_u(z)\frac{Y(z)}{Y_R\ (z)}\left(1 - \frac{Y(z)}{2Y_R\ (z)}\right). \quad (13)$$

From the two previous equations, the reference deflection should be equal to:

$$Y_R(z) = 2\frac{P_u(z)}{E_{s0}(z)}. \quad (14)$$

Compared to some functions used in practice to describe the P-Y curve (hyperbolic, exponential, hyperbolic tangent, … etc) giving an infinite value to Y_R, the **PARECT** function provides rather a finite value and consequently offers a more realistic description of the pile load-deflection behaviour.

Experimental P-Y curves were fitted by the **PARECT** function on the basis of the least-squares technique, which led to construct the profiles of E_{s0} and $P_u(z)$ of each pile tested. Further correlations of these profiles with that of the net cone resistance $q_c*(z)$ will lead, as will be seen hereafter, to determine the functions h and J defined respectively in Equations (10) and (11).

The Lateral Pile/Soil Stiffness Ratio

In most cases, the profile $q_c*(z)$ is marked by a non-negligible variation with depth. To account for such a non-homogeneity, an equivalent net cone resistance $q_{ce}*$ is defined as follows:

$$q_{ce}^* = \frac{1}{D_e}\int_0^{D_e} q_c^*.dz \quad (15)$$

where D_e, called "the effective embedded length" of the pile, is the depth below which the pile segments do not respond to the lateral loading at the pile top and defined as:

$$D_e = \min\{D, \pi L_0\} \quad (16)$$

where the elastic length (or transfer length) L_0 is defined by:

$$L_0 = \sqrt[4]{\frac{E_p.I_p}{K_E.q_{ce}^*}}. \quad (17)$$

The lateral pile/stiffness ratio already defined in Equation (7) is written as follows:

$$K_R = \frac{E_p.I_p}{q_{ce}^*.D^4}. \quad (18)$$

The Lateral Reaction Modulus E_{s0}

It was found for all the test piles that the profile of the reaction modulus $E_{s0}(z)$ is linear, corresponding to a usual distribution of the soil stiffness in homogeneous granular soils belonging to the category of Gibson's soils. In Figure 4, a typical profile of $E_{s0}(z)$ for the pile P_2 is illustrated.

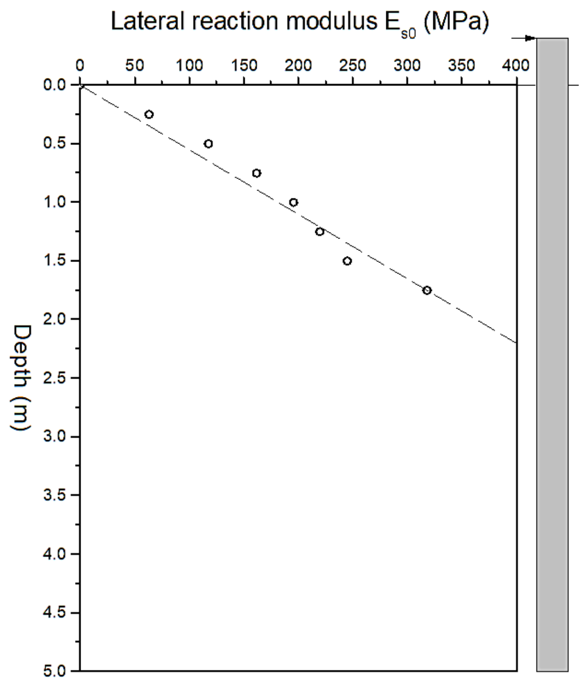


Figure (4): Typical profile of the lateral reaction modulus for pile P_2

Having a slenderness ratio D/B equal to 5.5, the pile P_2 should be considered rather as a pier exhibiting a different behaviour than that of a pile. The analysis of the experimental values of the modulus number K_E showed a singular value of K_E of P_2 with respect to other piles, which leads to limit the analysis of K_E to piles characterized by D/B greater than or equal to 10. Moreover, according to the values of K_R for the piles tested, they are classified as semi-rigid piles ($K_R \geq 2.9x10^{-2}$). Due to limited data regarding the flexible piles, it was suggested to extend the value of K_E=5.1 corresponding to K_R=$2.9x10^{-2}$ for all the flexible piles, provided that this extension will be subsequently validated by analyzing the response of a representative case history of experimental flexible piles.

The study of the function h formulated by Equation (10) showed that the modulus number K_E depends slightly on the slenderness ratio D/B and varies as a power function of the lateral pile/soil stiffness ratio K_R as follows:

$$K_E = \begin{cases} \frac{1}{10} K_R^{-1.1} & if\ K_R > 0.029; \\ 5.1 & if\ K_R < 0.029. \end{cases} \qquad (19)$$

According to this equation, the modulus number decreases with the pile/soil stiffness ratio, which means that a flexible pile behaves with a greater value of K_E than that of a rigid pile. Moreover, based on Equation (18), K_E decreases with the flexural stiffness E_pI_p and increases with the pile length D.

The equation of K_E is only valid for long piles having D/B greater than or equal to 10. Figure 5 illustrates in a bi-logarithmic scale the variation of the modulus number as a function of K_R.

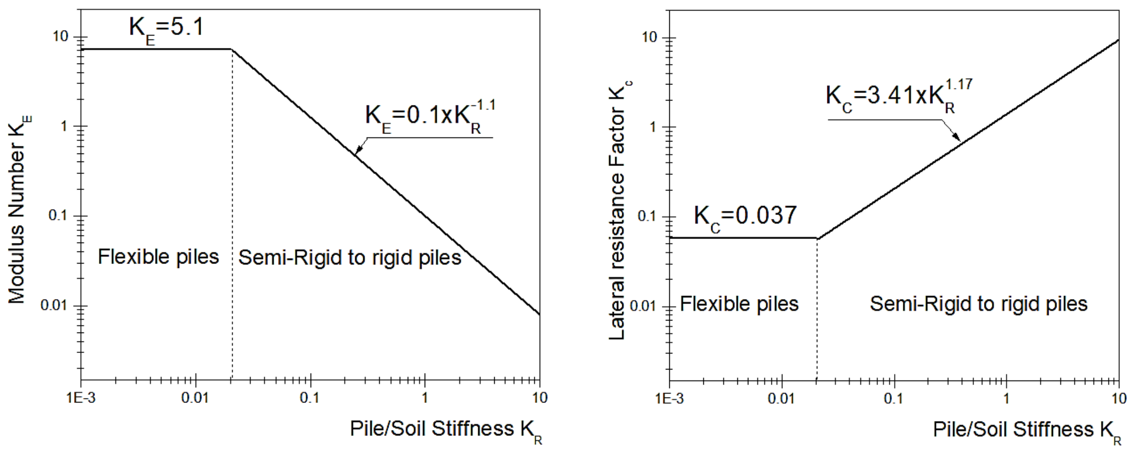


Figure (5): Diagrams of the P-Y curve parameters K_E and K_c

Since the test piles were jacked or bored, it seems that the installation procedure has a relatively little effect on the reaction modulus for semi-rigid piles, which is in accordance with the findings of Bouafia and Garnier (1991) from centrifuge tests.

The Lateral Soil Resistance P_u

Similarly, the function j defined by Equation (11) was found depending on the lateral pile/soil stiffness and was formulated by fitting its experimental values by a power function as follows:

$$K_c = \begin{cases} 3.41xK_R^{1.17} & if\ K_R > 0.029; \\ 0.037 & if\ K_R < 0.029. \end{cases} \qquad (20)$$

It is interesting to highlight that this equation shows an increase of K_c with the pile/soil stiffness ratio K_R. Consequently, the soil around a rigid pile is characterized by a greater value of K_c than that of a flexible pile and mobilizes a greater soil resistance. Furthermore, according to Equation (18), K_c increases with E_pI_p and decreases with length D.

This formula, illustrated by Figure 5, is also limited to long piles having D/B greater than or equal to 10 and provides a substantial improvement of previous studies (Bouafia, 2009; 2010; 2014; 2017).

The values of K_c proposed by Schmertmann (1978) are 0.11 and 0.22, respectively in loose and dense sand independently of the pile stiffness, whereas Novello (1999) suggested a value of 1.0 which is much greater than that of Schmertmann. Moreover, on the basis of Equation (4) proposed by Kim et al. (2015), the coefficient K_c may be derived with a somewhat laborious formulation depending on q_c as well as on the depth. Finally, the French standard prescribes a value of 0.077 for K_c. Compared to these values, those given by Equation (19) are smaller, particularly for flexible piles, but they offer the advantage to take into consideration the pile/soil stiffness and provide relatively conservative values of K_c.

Regarding the modulus number K_E, previous studies (Novello, 1999; Disan and Randolph, 2001; Suryasentana and Lehane, 2014) proposed power functions expressed by Equations (1) to (3) serving to construct the P-Y curve point-by-point, but not allowing to derive the lateral reaction modulus. The French standard prescribed a value of 2.25 for K_E regardless of the value of the lateral pile/soil stiffness K_R, which means that it is applicable to flexible as well as to rigid piles.

The Reference Deflection Y_R

On the basis of Equations (14), (8) and (9), the reference deflection may be written as follows:

$$Y_R(z) = 2\frac{P_u(z)}{E_{s0}(z)} = 2\frac{K_c.B}{K_E}. \tag{21}$$

According to Equations (19) and (20), Y_R for flexible piles (K_R<2.9x10^{-2}) does not depend on K_R and takes a constant value of 1.5%. For semi-rigid to rigid piles, fitting the experimental values of K_c and K_E leads to the following power function with a value of the correlation coefficient R equal to 99.5%:

$$\frac{Y_R}{B} = 109[K_R]^{2.5}. \tag{22}$$

For comparison, the French standard P94-262 specifies that Y_R/B is equal to 3.4%, whereas in the method of Reese et al. (1974), Y_R/B is equal to 3.75%. Finally, a widely used design criterion is the conventional value of 10% of B. As can be seen from Equation (22) and Figure 6, the reference deflection increases as a power of the lateral pile/soil stiffness in contrast with the traditional methods of P-Y curves ignoring this parameter, thereby providing a reference deflection applicable to flexible as well as to rigid piles.

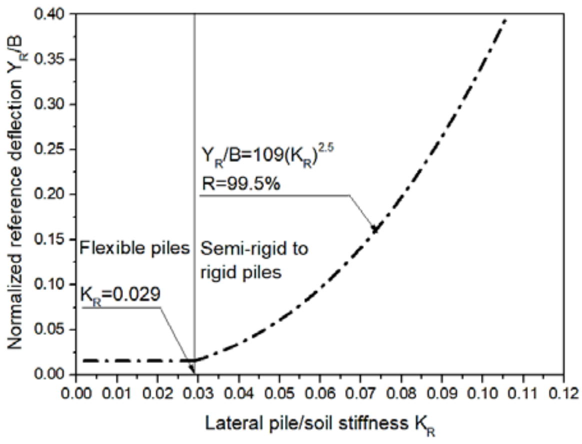


Figure (6): Chart of the normalized reference deflection Y_R/B

Step-by-Step Procedure of Construction of P-Y Curves

The following methodology allows a practical analysis of a laterally loaded pile in sand on the basis of P-Y curves constructed from the CPT data:

1. Filter the profile of cone resistance by eliminating the peaks of resistance and simply replacing them by a linear interpolation.
2. Sub-divide the soil along the pile into N thin horizontal segments in such a way that q_c* exhibits a linear variation within any segment. The value of q_c* at the mid-segment is therefore representative of all the segment.
3. Assume that the pile is semi-rigid or rigid. From Equation (16), one obtains: $D_e = D$.
4. Compute q_{ce}* according to Equation (15) by replacing the integration formula by the summation of trapezes.
5. Compute K_R according to Equation (18).
6. Compute K_E by Equation (19) or determine it from Figure 5.
7. Compute L_0 by Equation (17).
8. Compute D_e of the pile from Equation (16). If D_e < D (case of flexible pile), then an iterative process is carried out by repeating steps from 4 to 8 until convergence of K_R.
9. Compute the values of E_{so} and P_u for each segment from Equations (8) and (9), respectively.
10. Use Equation (12) or Equation (13) to construct the P-Y curve for each segment along the pile.
11. Use a software based on the theory of P-Y curves to analyze the pile response, like the software SPULL

(**s**ingle **p**ile **u**nder **l**ateral **l**oads) developed at the University of Blida since 1999 (Bouafia, 2007). This software is a freeware available upon request.

Validation of the Proposed Method

Description of the Case Histories

The predictive capability of the proposed CPT-based P-Y curves' method was assessed by predicting the load-deflection behaviour of test piles subjected to monotonic loading in sandy deposits. Summarized in Table 1 are the main Geo-technical and physical properties of the pile/soil systems. For easy access to the data, the piles analyzed are identified as mentioned in their original references.

Table 1. Features of the pile/soil systems used in full-scale tests

Site	Pile	D (m)	D/B	E_pI_p ($MN.m^2$)	q_{ce}* (MPa)	D_e (m)	K_R	Reference
S_3	3-12	20.4	57.3	61	10.24	3.94	$3.44x10^{-5}$	Bouafia (2007)
	3-13	20.4	57.3	61	10.24	3.94	$3.44x10^{-5}$	Bouafia (2007)
S_4	DS-4	2.73	10.0	10.35	21.77	2.10	$8.56x10^{-3}$	Ross et al. (2019)
S_5	PS-3	4.35	12.8	40.0	12.30	3.40	$9.10x10^{-3}$	Li et al. (2014)
S_6	PS-5	5.00	14.7	40.0	13.90	3.28	$4.60x10^{-3}$	Li et al. (2014)
	PS-6	7.00	20.6	40.0	13.90	3.28	$1.20x10^{-3}$	Li et al. (2014)
S_7	UWA-1	3.50	15.56	2.42	3.43	2.30	$4.70x10^{-3}$	Suryasentana & Lehane (2014b)
S_8	UWA-2	6.00	17.65	11.50	13.30	2.40	$6.70x10^{-4}$	Suryasentana & Lehane (2014b)
S_9	PD	11.5	35.5	30.0	3.96	4.68	$4.30x10^{-4}$	Rollins et al. (2005)
S_{10}	PF	13.8	23.0	291.8	2.81	9.00	$2.82x10^{-3}$	Rollins et al. (2005)
S_{11}	#2	15.0	31.2	70.0	6.42	5.10	$2.10x10^{-4}$	Reese & van impe (2001)

Site S_3 is located at the Lock & Dam 26 near the Mississipi river and composed of 20-m thick poorly graded medium-to-coarse sand with gravel overlying a bedrock of hard limestone. Two identical HP-14x73 piles socketed in the limestone bedrock were subjected to lateral loading increments, where each pile served during tests as a reaction beam to the other one.

Site S_4 is located at Dunkirk, about 300 km north of Paris, being composed of fine-to-medium sand and the test pile is a steel pipe installed by vibratory driving into the site.

Sites S_5 and S_6 are both located at Blessington, about 35 km south-west of Dublin, Ireland and composed of heavily over-consolidated very dense sand with a density index nearly equal to 100%. The test piles are three identical open-ended steel pipe piles driven into embedded lengths varying from 4.3 to 7 m.

Site S_7 is located at Shenton Park, Perth, Western Australia and composed of a deposit of a dry loose-to-medium dense sand and the test pile is a grouted pile.

Site S_8 is located at north Perth, Western Australia and composed of dense sandy soil deposit, while the test pile is a CFA bored pile.

Site S_9 is located at Treasure Island, San-francisco Bay, California and composed of poorly graded silty sand along the pile with the presence of a silty clay layer with a thickness of 1.60 m and outside the effective length D_e of the pile. The test pile is a driven steel pipe pile loaded under a controlled displacement rate of 9 mm/s.

Site S_{10} is located at Treasure Island, but composed of a saturated poorly graded loose sand with the presence of a low-plasticity clayey layer with a thickness of 1.2 m above the pile base. This layer does not affect the pile response, since it is outside the effective length D_e. The test pile is a cast-in-steel-shell bored pile.

The last site, noted S_{11}, is located at Arkansas River, Mississipi and composed of poorly-graded sand with small traces of fines and the test pile is a steel pipe pile.

Discussion of the Results of Validation

P-Y curves of each pile analysed were generated from the CPT data of the experimental site according to the methodology mentioned above and input into the software SPULL to simulate the load-deflection curve of each pile. Figure 7 illustrates a direct comparison of the predicted pile deflections Y_0 at ground level to the ones measured up to 16% of the pile diameter. It can be noticed that there is a very good accordance between predicted and measured deflections, particularly for normalized pile deflections less than 8%. The ratio of predicted to measured deflection Y_0^{pred}/Y_0^{meas} is characterized by an average value of 1.07 and a coefficient of variation of 30%. Moreover, Figure 8 illustrates the histogram of the 81 points of comparison through the random variable Y_0^{pred}/Y_0^{meas}. Fitting this histogram by the Gauss' density function leads to a mean value μ of 0.91, a standard deviation σ of 0.14 and a confidence interval [0.81, 1.14] for the mean value at a level of 95% of confidence, which is very encouraging seeing the multitude of approximations made during the process of development of this method.

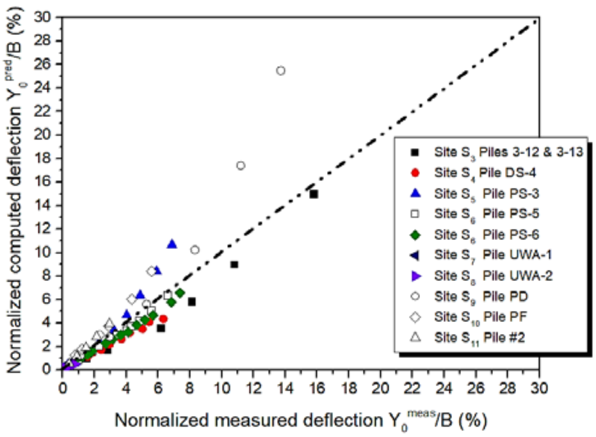


Figure (7): Comparison of predicted and measured pile deflections

In light of this process of validation, it can be seen that the proposed method of construction of P-Y curves on the basis of the CPT tests has a good predictive capability capturing many aspects of the pile/soil interaction; namely, the variability of the soil properties, the effect of different techniques of the pile installation and the non-linear response of the pile.

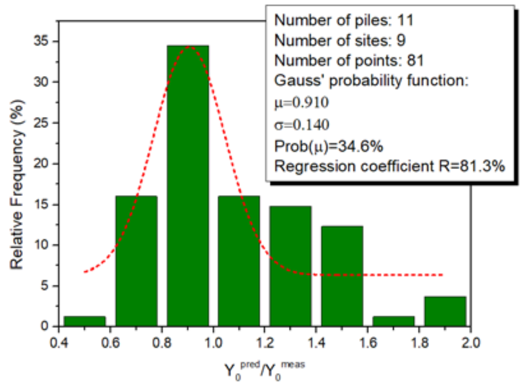


Figure (8): Histogram of the ratio Y_0^{pred}/Y_0^{meas}

CONCLUSIONS

In this paper, an empirical method to construct the P-Y curves for single piles in sand based on the cone penetration test CPT was proposed. A detailed interpretation of 5 full-scale lateral loading tests of instrumented piles in 2 sandy soils led to derive the parameters of the experimental P-Y curves in correlation with cone resistance q_c. It was found that the lateral reaction modulus and the lateral soil resistance vary as power functions of the lateral pile/soil stiffness ratio, on the basis of which a parabola-rectangle function was proposed to describe the P-Y curves in sand and a step-by-step procedure of analysis of the load-deflection response of a single pile under lateral load in sand was proposed.

An assessment of the predictive capability of the proposed method undertaken by predicting the pile/soil response of 11 test piles in full-scale tests in 9 sites worldwide showed a very good accordance between the predicted pile deflections and the measured ones, which makes encouraging seeing the multitude of approximations made during the process of development of this method.

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Conflict of Interest

The author declares that there is no conflict of interest regarding this work.

REFERENCES

AFNOR. (2012). "Justification des ouvrages géotechniques-fondations profondes (in French)". Norme d'application Nationale de l'Eurocode 7, NF, 94-262, 205 p.

AFNOR. (2013). "Reconnaissance et essais géotechniques - essais en place - Partie 1: Essais de pénétration au cône électrique et au piézocône (in French)". NF EN ISO 22476-1, 52 p.

Baguelin, F., Jézéquel J-F., and Shields, D-H. (1978). "The pressure meter and foundation engineering". Series on Rock and Soil Mechanics, 2 (1). 1st edition, 1978 Trans. Tech. Publications, Germany, 615 p.

Baligh, M. (1985). "Strain path method". Journal of Geotechnical Engineering, 111 (9). ASCE, Paper No. 19988, 1108-1136.

Baligh, M., and Levadoux, J-l. (1986). "Consolidation after undrained piezocone penetration, Part II: Interpretation". Journal of Geotechnical Engineering, 112 (7). ASCE, Paper No. 20785, 727-745.

Banerjee, P., and Davies, T. (1978). "The behaviour of axially and laterally loaded single piles embedded in non-homogeneous soils". Géotechnique, 28 (3), 309-326.

Bouafia, A., and Garnier, J. (1991). "Experimental study of P-Y curves for piles in sand". Proceedings of the International Conference CENTRIFUGE'91, Boulder, Colorado, 13-14 June 1991, 261-268, A A Balkema.

Bouafia, A. (2007). "Single piles under horizontal loads: Determination of the P-Y curves from the prebored pressuremeter". International Journal of Geotechnical and Geological Engineering, Springer-Verlag, ISSN: 0960-3182, 25, 283-301.

Bouafia, A. (2009). "Analysis of P-Y curves for single piles from the cone penetration test". Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, October 2-9, 2009, Alexandria, Egypt, 1103-1106.

Bouafia, A. (2010). "P-Y curves for single piles in sand from the *in-situ* tests". Proceedings of the 1st International Conference on Geotechnical Engineering ICGE-2010, November 5-6, 2010, Lahore Pakistan.

Bouafia, A. (2014). "P-Y curves from the CPT test for laterally-loaded single piles in sand". Proceedings of the 3rd International Symposium on Cone Penetration Testing, May 12-14, 2014, Las Vegas, Nevada, 7 p.

Bouafia, A. (2017). "Laterally-loaded single piles: Construction of P-Y curves from the cone penetration test". Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering ICSMGE, 17-22 September, Seoul, Korea, 4 p.

Briaud, J.L., Smith, T.D., and Tucker, L.M. (1985). "A pressuremeter method for laterally-loaded piles". Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, San Francisco 1985, 3, 353-356.

Briaud, J.-L. (2013). "Geotechnical engineering: Unsaturated and saturated soils". Edited by John Wiley & Sons, Inc., 1022 p.

Budhu, M., and Davies, T. (1987). "Non-linear analysis of laterally-loaded piles in cohesionless Soils". Canadian Geotechnical Journal, 24, 289-298.

Dunand, M. (1981). "Etude expérimentale du comportement des fondations soumises au renversement (in French)". Doctorate Thesis, University of Grenoble IMG, France.

Dyson, G., & Randolph, M. (2001). "Monotonic lateral loading of piles in calcareous sand". Journal of Geotechnical Engineering, ASCE, 127 (4), 346-352.

Eslaamizaad, S., and Robertson, R.K. (1996). "Cone penetration test to evaluate bearing capacity of foundation in sands". Proceedings of the 49th Canadian Geotechnical Conference, St-John's, Newfoundland.

Haouari, H., and Bouafia, A. (2019). "A centrifuge modeling and finite element analysis of laterally-loaded single piles in sand with focus on P-Y curves". Periodica Polytechnica Civil Engineering, 2020, 3. https://doi.org/10.3311/PPci.14472

Jamiolkovski, M. (1988). "Research applied to geotechnical engineering". James Forrest Lecture, Proceedings ICE, Part 1, 1988, 84, 571-604.

Khadidja, S., Djarir, Y., and Khelifa, A. (2020). "Inelastic response of soil-pile-structure interaction system under lateral loading: A parametric study". Jordan Journal of Civil Engineering, 14 (2), 250-266.

Kim, G., Kyung, D., Park, D., and Lee, J. (2015). "CPT-based P-Y analysis for monopiles in sands under static and cyclic loading conditions". Geo-mechanics and Engineering, 9 (3), 313-328.

Ladanyi, B. (1963). "Expansion of a cavity in a saturated clay medium". Journal of Soil Mechanics and Foundations Division, ASCE, 89 (SM4/3577), 127-161.

Ladanyi, B., and Longtin, H. (2005). "Short-and long-term sharp cone tests in clay". Canadian Geotechnical Journal, 42, 136-146.

Lee, J., Kim, M., and Kyung, D. (2010). "Estimation of lateral capacity of rigid short piles in sands using CPT results". Journal of Geotechnical Engineering, ASCE, 136 (1), 48-56.

Li, W., Igoe, D., and Gavin, K. (2014). "Evaluation of CPT-based P-Y models for laterally-loaded piles in siliceous sand". Geotechnique Letters, 4, 110-114.

Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997). "Cone penetration testing in geotechnical practice". Blackie Academic & Professionals, 340 p.

Matlock, H., and Reese, L.C. (1960). "Generalized solutions for laterally-loaded piles". Journal SMFE, ASCE, SM5, Oct., 83-91.

Ménard, L., Gambin, M., and Bourdon, G. (1969). "Méthode générale de calcul d'un rideau ou d'un pieu sollicité horizontalement en fonction des résultats pressiométriques (in French)". Journal Sols/Soils, 1 (20/23), 16-28.

Novello, E.A. (1999). "From static to cyclic P-Y data in calcareous sediments". Proceedings of the 2nd International Conference on Engineering for Calcareous Sediments, Perth, 17-27.

Poulos, H., and Hull, T.S. (1992). "Analytical geomechanics in foundation engineering: A study of laterally-loaded piles". Research Report no. R667, University of Sydney, Australia, 40.

Randolph, M.F. (1981). "The response of flexible piles to lateral loading". Géotechnique, 81 (2), 247-259.

Reese, L.C., Cox, W.R., and Koop, F.D. (1974). "Analysis of laterally-loaded piles in sand". Proceedings of the 6th Annual Offshore Technology Conference, Houston, Texas, 473-485.

Reese, L., and Matlock, H. (1977). "Non-dimensional solutions for laterally-loaded piles with soil modulus assumed proportional to depth". Proceedings of the 8th Texas Conference on Soil Mechanics and Foundations Engineering, Austin.

Reese, L.C., and van Impe, W. (2001). "Single piles and pile groups under lateral loading". Edited by Francis and Taylor/Balkema, 463 p.

Robertson, P.K., Hughes, J.M.O., and Campanella, R.G. (1984). "Design of laterally-loaded displacement piles using a driven pressuremeter". In: Laterally-loaded Deep Foundations: Analysis and Performance, ASTM Special Technical Publications, No. 835, 1984, 229-238.

Rollins, K.-M., Gerber, T.-M., Lane, J.-D., and Ashford, S.-A. (2005). "Lateral resistance of full-scale pile group in liquefied sand". Journal of Geo-tech. & Geo-envir. Engg., 131 (1), 115-125.

Ross, Mc., Byron, W., and Houlsby, G.-T. (2019). "Monotonic laterally loaded pile testing in a dense marine sand at Dunkirk". Géotechnique, 7 (11), 986-998.

Schmertmann, J.H. (1978). "Guidelines for CPT: Performance and design". Report FHWA-TS-78-209, Florida Departement of Transportation.

Suryasentana, S.K., and Lehane, B.M. (2014a). "Numerical derivation of CPT-based P-Y curves for piles in sand". Géotechnique, 64 (3), 186-194.

Suryasentana, S.K., and Lehane, B.M. (2014b). "Verification of numerically-derived CPT-based P-Y curves for piles in sand". Proceedings of the 3rd International Symposium on Cone Penetration Testing, May 12-14, 2014, Las Vegas, Nevada, 8 p.

Vesic, A.S. (1972). "Expansion of cavities in infinite soil mass". Journal of Soil Mechanics and Foundations Division, ASCE, 98 (SM3), 265-290.