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Single piles under monotonic lateral loads-Full scale tests and numerical modelling

Hocine Haouari ✉, Ali Bouafia

Laboratory NEIGE, Department of Civil Engineering, Faculty of Technology, University of Blida, Route de Soumâa, P.O.Box 270, R.P Blida, 09000, Blida, Algeria

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Abstract: This paper aims to present the results of the analysis of full-scale lateral loading tests of a single pile driven into a bi-layered soil located in Plancoet (France). The pile was subjected to a monotonic sequence of loads, and the P-Y curves along the pile were derived and compared to those recommended by the current design methods. Displacement analysis was then undertaken by inputting these P-Y curves, as well as those currently recommended in the literature, into the software SPULL to predict the load-deflection curve. The calibration process was conducted by a 3D finite element model using Abaqus software based on the elastic-perfectly plastic Mohr–Coulomb constitutive model, and the surface-to-surface contact method was used to take into account the nonlinear response at the pile/soil interface. The determined soil elastic modulus was used to predict the pile deflections by using some usual elasticity-based methods, which led to good prediction of the small pile deflections.

1. Background

Experimental observations issued from full-scale pile lateral loading tests clearly show the complexity of the pile response as well as the multitude of parameters involved in such a pile/soil interaction.

Pile lateral load-deflection analysis is usually undertaken within the framework of a serviceability limit state (SLS) design, and pile deflection 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 (finite elements, finite differences methods) and P-Y curve methods (Matlock & Reese, 1960; Ménard et al, 1969; Baguelin et al, 1978; Reese & Van Impe, 2001; Briaud; 2013).

According to the P-Y curve theory, the pile/soil interface is modelled by a series of nonlinear springs along the pile where a spring subjected to the soil reaction P at a given depth exhibits a lateral displacement Y (Bouafia, 2007).

The present paper aims to present the results of the interpretation of full-scale lateral load tests performed on a single pile driven into homogeneous saturated bi-layered soil located in Plancoet (France), with the derivation of high-quality experimental P-Y curves. This test is part of an important experimental research program on the monotonic and cyclic behaviour of piles under lateral loading, undertaken by the University Gustave Eiffel UGE (formerly the IFSTTAR) and the IFP (French Petroleum Institute), based on a series of full-scale tests.

After a brief description of the test pile, its instrumentation and the geotechnical aspects of the experimental site, the main results are presented. Then the article focuses on the construction and analysis of the P-Y curves for monotonic loading conditions, with comparison to the available methods of pile design under lateral loads.

✉Corresponding author. haouari.ce.geo@gmail.com

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2. Description of the Pile/Soil configuration

2.1. Geotechnical description of the experimental site

The experimental site is located in Plancoet (Côtes d'Armor, France), 450 km west of Paris. The soil consists of a bi-layered deposit composed of a low plasticity clayey layer (CL), 4 m thick, overlying a layer of silty sand (SM), 4 m thick. Beyond this depth, the soil consists of a highly plastic clayey layer (CH). Due to its proximity to the Arguenon River, the site is submerged by groundwater (Baguelin et al, 1972; Baguelin and Jezequel, 1972).

A cone penetration test (CPT) was carried out before installation of the test pile by using a Gouda device, with a 10 cm^2 standard electrical cone penetrating at a velocity of 20 mm/s. A prebored Pressuremeter Test (PMT) using an E standard probe to measure the limit pressure p_l and the PMT soil modulus E_M, as well as a Vane shear test (VST), were carried out before the installation of the test pile. The profiles of these tests are compiled in Figure 1.

The average values of the effective shear strength parameters (ϕ', c') of the clayey samples, obtained from consolidated undrained triaxial shear tests (CU), are ϕ'=39.1° and c'=2 kPa.

As illustrated in Figure 1, the (VST) undrained shear strength profile exhibits a linear trend similar to the profiles of the other mechanical properties, which is typical of a normally consolidated clay. Furthermore, this layer is classified as very soft to soft clay based on the margins of the limit pressure, the PMT modulus and the cone resistance (AFNOR, 2012; CCTG, 1993, Notice D60 by Ménard). However, the margin values of 20-75 kPa obtained for c_u, classify this layer as a soil of low to medium resistance (AFNOR, 2005). On the other hand, the ratio E_M/p_l, which is equivalent to a soil rigidity index according to the pressuremeter theory of Menard, inventor of the PMT test, classifies the soil to be an underconsolidated to normally consolidated clay, (Cassan, 1988; CCTG, 1993).

The friction angle value of the sandy layer is equal to 33°, indicating medium density sand. According to the PMT data and the CPT data, the sandy layer is classified as loose to medium dense (CCTG, 1993; AFNOR, 2012).

After pile driving and prior to the pile loading test, the soil layer from the top and down to a one-meter depth, was removed to eliminate this layer overconsolidated by desiccation, which would complicate the interpretation of the results. Indeed, according to previous works (Baguelin et al, 1971; Smith, 1983; Reese & Van Impe, 2001), shallow depths of a clayey layer are usually overconsolidated by seasonal desiccation and exhibit much higher stiffness values than deeper depths, which is not representative of the soil underneath.

The water level was maintained at the ground level during all the experiments to simulate the real conditions of pile foundations in an offshore structure (Baguelin et al, 1985; Hadjadji et al, 2002).

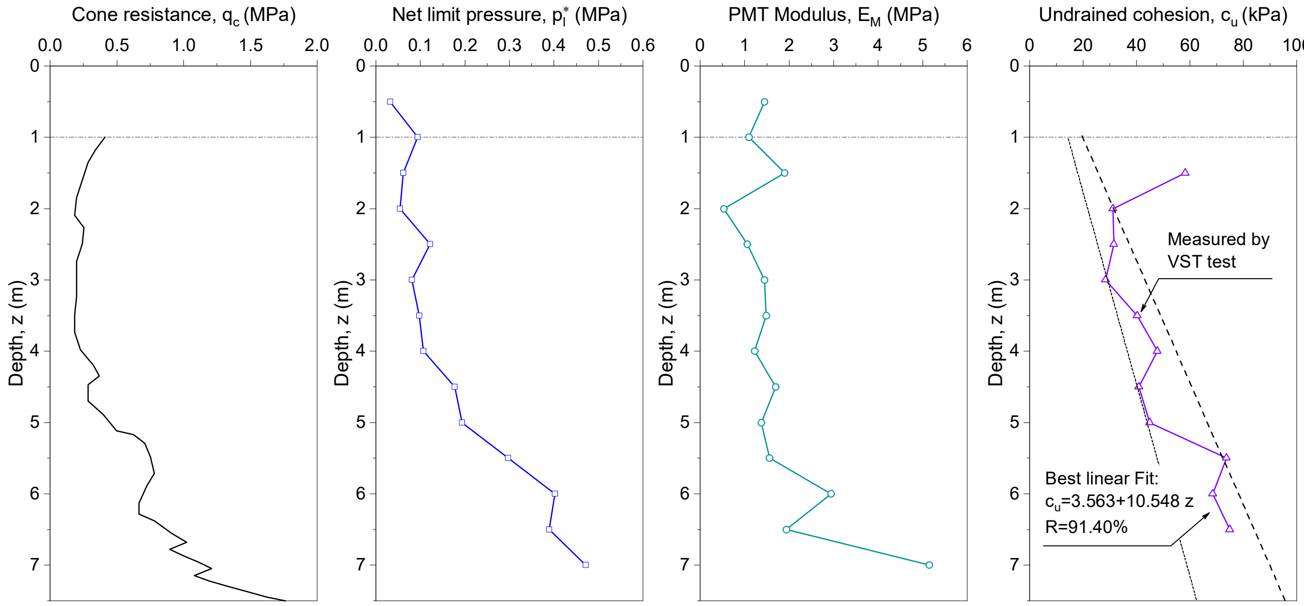

Fig. 1. Typical in-situ properties of the experimental site.

2.2. Pile description and instrumentation

As illustrated in Figure 2, the test pile is an HEA-280 steel profile on which two steel plates were welded on its lateral sides, which results in a rectangular 0.27x0.28 m pile, and the lateral load is applied parallel to these plates. The pile has a length (L) of 9 m, a width (B) of 0.284 m, an embedded length (D) of 6.5 m and a slenderness ratio (D/B) of 22.9. The overall flexural stiffness of the pile section is 30 MN.m2 and the yielding bending moment of the pile is 285 kN.m (Baguelin et al, 1989).

The test pile was instrumented by 28 pairs of strain gauges fixed along two opposite vertical axes inside the pile. The gauge distribution started 0.5 m above the soil surface with an increment of 0.25 m. The first two gauges were therefore out of the embedded pile length. Moreover, pile deflections were measured by 4 LVDTs (linear variable displacement transducers) fixed on the pile at 1.10 and 1.60 m above the soil surface, as depicted in Figure 2. The average pile displacements measured at two levels above the lateral load were useful for the integration procedure of bending moments to obtain the pile deflections, which requires two boundary conditions.

The lateral loads were applied at 1 m above the soil surface, measured by a load cell incorporated between the hydraulic jack and the pile head (Meimon et al, 1986). The two gauges above the soil surface also served to check the applied lateral load (Degny et al, 1994).

The pile was closed ended and submitted to a driving procedure by a DELMAG-D5 hammer, which likely induced high pressures on the surrounding soil. However, the time elapsed between the pile driving and the first loading tests was 274 days, which is judged sufficient for a total dissipation of the excess pore pressures within the soil (Baguelin et al, 1972).

2.3. Experimental programme of loading

The lateral loading test consists of a series of 4 monotonic loading stages for 2 hours each using a load increment of 5 kN.

3. Analysis of the pile response

3.1. Load-deflection behaviour

The experimental load-deflection curve used to assess the lateral capacity criteria of single piles is given in Figure 3. As summarized in Table 1, the hyperbolic criterion assimilates the curve to a hyperbolic shaped function described by the following equation:

$$H = \frac{Y_0}{\dfrac{1}{K_{H0}} + \dfrac{Y_0}{H_u}} \tag{1}$$

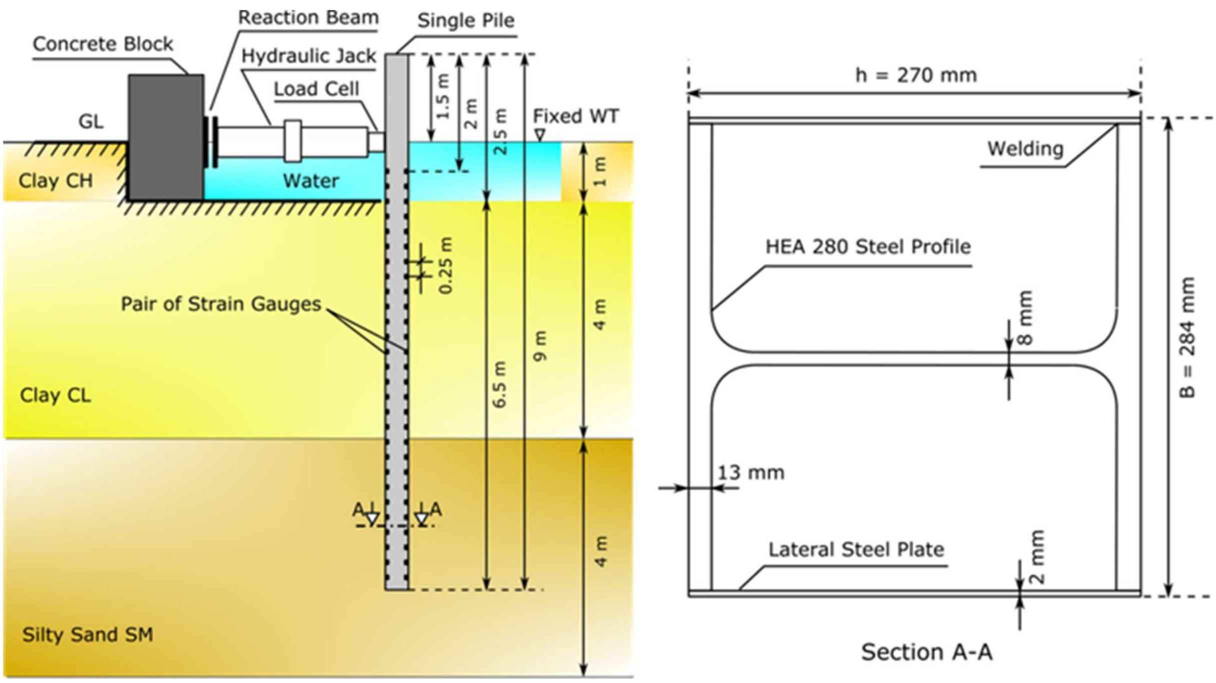

Fig. 2. Test pile and loading device configuration.

Table 1. Summary of usual lateral resistance H_u criteria

No.	Method	Description	Reference
1	Hyperbolic criterion	H_u is the horizontal asymptote	Duncan and Chang (1970)
2	Bi-Logarithmic criterion	H_u corresponds to the change in slope of the Log(H) versus Log(Y_0) curve	Slack and Walker (1970)
3	Graphical procedure	H_u is the intercept of the 1st bisector with the recurring series	Asaoka (1978)
4	Deflection limitation	H_u corresponds to a head deflection of 25 mm	New York City (1981)
5	Deflection limitation	H_u corresponds to a head deflection of 10% of B	Briaud (1981)

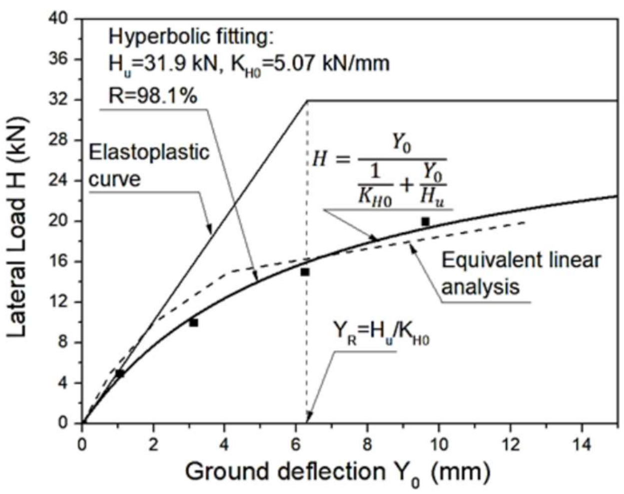

Fig. 3. Load-deflection curve of the test SH-1

H_u and K_{H0} refer to the limit lateral load (horizontal asymptote) and the initial lateral pile stiffness (slope of the initial tangent), respectively.

The graphical procedure of Asaoka (1978) is based on the interpretation of a recurrence series of loads built by interpolation of the load deflection curve, which results in a recurrent series of lateral loads H_k for a given deflection increment ΔY. The last points of the series usually converge towards a straight line that intercepts the first bisector $H_{k+1}=H_k$ at the final value of the pile deflection.

The other criteria given in Table 1 are based on the deflection limitation by an absolute value of 25 mm, or a conventional value of 10% of the pile width, equal to 28 mm.

According to the empirical procedure suggested by Slack and Walker (1970), H_u is the ordinate of the pile deflection corresponding to the change in slope of the Log(H) versus Log(Y_0) curve.

As shown in Figure 3, the hyperbolic criterion was applied by fitting the experimental load-deflection curve by a hyperbolic function, based on the least squares technique, leading to H_u=31.9 kN, whereas the bi-logarithmic criterion, illustrated in Figure 4, shows a linear trend without changes in the slope, meaning that H_u is greater than 20 kN. Asaoka's graphical procedure results in a value of H_u=25.1 kN.

The displacement-based criteria were not satisfied because the maximum deflection was 9.6 mm, a value corresponding to 3.4% of B. The three values of H_u determined are in relatively reasonable agreement, and the maximum value of the lateral load (20 kN) is 63% of that given by the hyperbolic criterion, and 80% of that given by Asaoka's criterion. This indicates that the loading test was conducted near the limit lateral load, involving full mobilization of the soil resistance.

3.2. Bending moments

The bending moment profile for a given lateral load is derived from the axial deformation profile measured by strain gauges along the pile. As illustrated in Figure 5, the maximum bending moment increases linearly with the lateral load, and for the maximum load value of 20 kN, it represents only 13% of the yielding bending moment, which leads to the conclusion that the ultimate limit state regarding this type of loading is reached by the soil failure before that of the pile material failure, which is typical of a rather semirigid or rigid pile response.

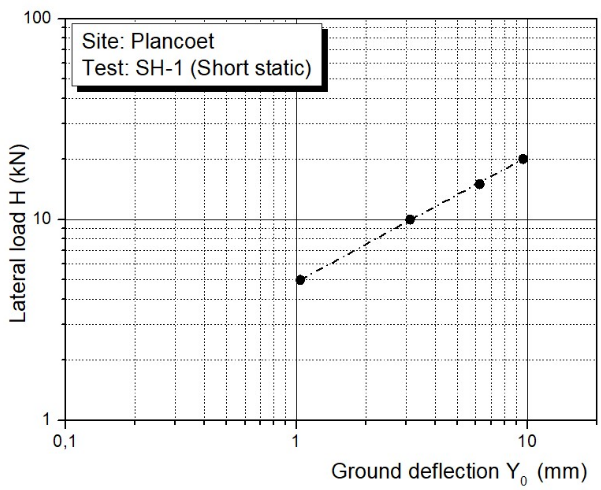

Fig. 4. Bi-Logarithmic load-deflection curve of the test SH1

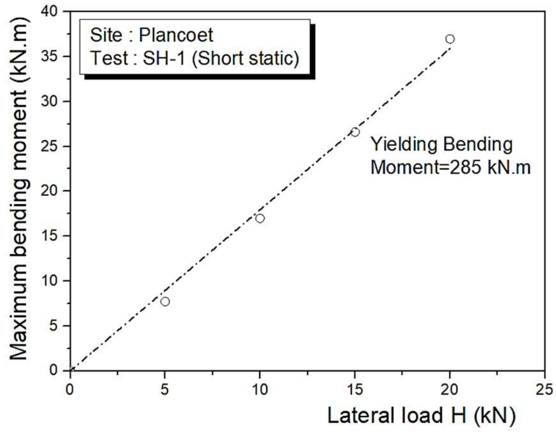

Fig. 5. Maximum bending moment versus the lateral load during test SH-1.

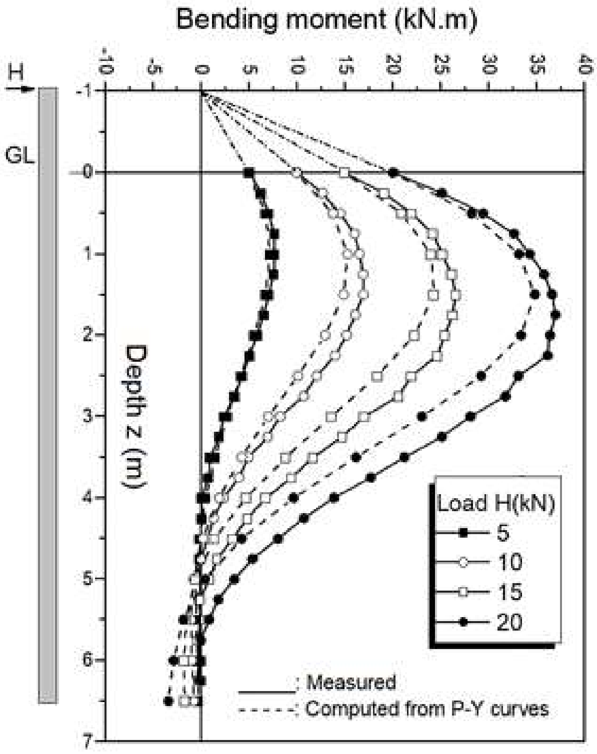

Fig. 6. Profiles of bending moments during test SH-1.

In addition, as illustrated in Figure 6, below the depth of 5.75 m corresponding to 20.5B, the bending moment becomes practically zero, indicating that the pile tip is practically free and does not therefore exhibit any displacement or rotation.

4. Study of the P-Y curves

4.1. Methodology

The bending moment profile M(z) was fitted and then subjected to a process of successive integrations and differentiations. Two successive integrations allow the determination of the profile of deflections Y(z), taking into account some boundary conditions in terms of displacements and rotations. Moreover, two successive differentiations of M(z) lead to determining the soil reaction P(z) and then building the P-Y curves along the pile.

Since the soil reaction P(z) is the curvature of the bending moment profile M(z) at a given depth z, it is therefore marked by a high sensitivity to any variation in the bending moment values and consequently strongly depends on the choice of the fitting curve of the bending moment profile (Bouafia, 2007). The fitting function was chosen according to the criterion of static equilibrium of the test pile under lateral reaction profile P(z) and the loads on the pile top, within a given tolerance (Bouafia and Garnier, 1991). In this study, the fitting procedure is carried out by the Savitzky–Golay method, which performs a local polynomial regression around each point and creates a new smoothed value for each data point. The Savitzky–Golay smoothing method is used with a 3rd order regression polynomial, and 15 points for each local regression. The successive derivatives were determined by using a centred finite differences formula for the first and second derivatives.

4.2. Presentation of P-Y curves

Figure 7 shows the experimental P-Y curves, where these curves at different depths are nonlinearly shaped with a regular increase in soil stiffness with depth. It should be noted that a change in sign of the deflections and the soil reaction is at almost the same depth, approximately 11.5 diameters, which is in accordance with Winkler's hypothesis regarding the soil reaction modulus (Bouafia & Lachenani, 2005). Furthermore, beyond a deflection of approximately 3% of B, a soil limit reaction is reached with asymptotic values in the P-Y curves along the pile.

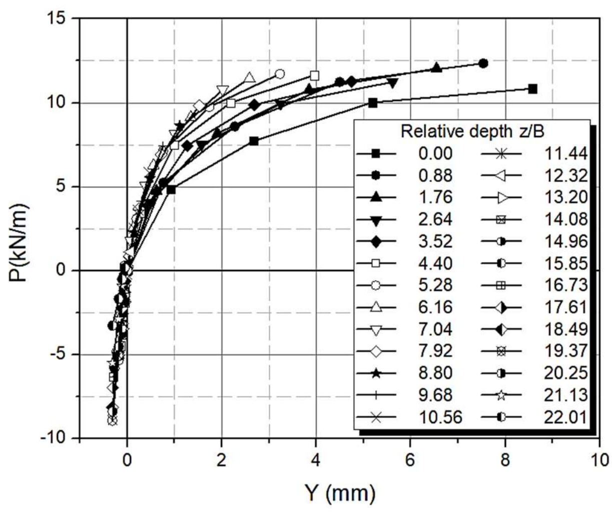

Fig. 7. Monotonic P-Y curves (test SH-1)

4.3. Interpretation of the P-Y curves

Hyperbolic formulation is often used to describe the elastic plastic constitutive laws of soils (Duncan and Chang 1970) as well as the P-Y curves (Reese 1971; Garassino 1976; Georgiadis et al. 1992). Experimental P-Y curves were fitted by the following hyperbolic function on the basis of the least squares technique:

$$P = \frac{Y}{\frac{1}{E_{s0}} + \frac{Y}{P_u}} \quad (2)$$

The coefficient of determination R was found to be greater than 95% for curves corresponding to depths above the zero-displacement depth, approximately 10 diameters. Beyond this depth, the values of E_{s0} seem to be inaccurate, since P and Y become together small and the ratio P/Y has no significance, regarding the inherent uncertainties resulting from experiments, the procedure of interpretation of bending moment curves, and the hyperbolic fitting of the P-Y curves.

As illustrated in Figure 8, the available values of the lateral soil resistance $P_u(z)$ within a depth of 2.8 m are characterized by a very small coefficient of variation, approximately 3.6%, and an average value of 14.2 kN/m. Consequently, a constant lateral resistance of 14.2 kN/m

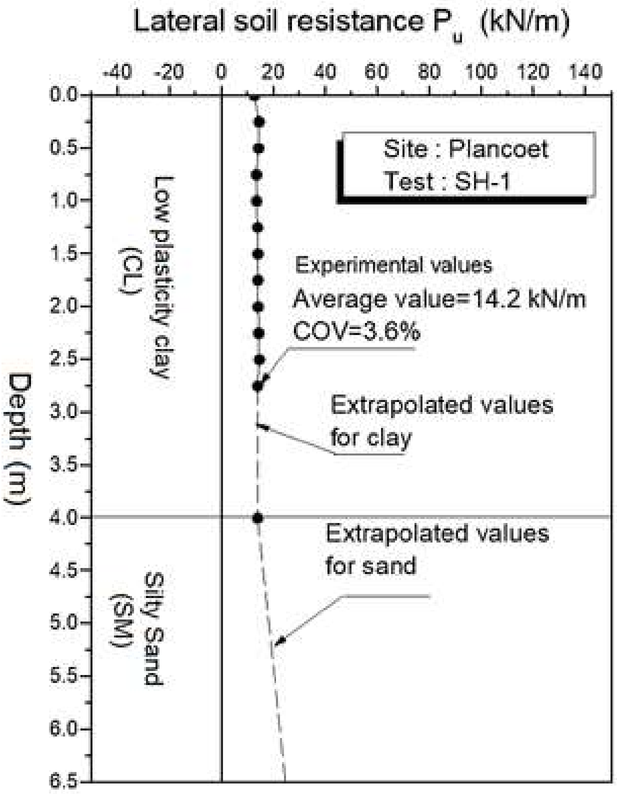

Fig. 8. Profile of the lateral soil resistance.

was assumed until the base of the clayey layer at a depth of 4 m. Within the sandy layer, since no values of P_u are available, they were estimated on the basis of an established fact that at a given depth, P_u is proportional to the soil resistance, quantified by the net limit pressure p_l* measured by the PMT test (Ménard et al, 1969; Baguelin et al, 1977; Gambin, 1979; Dunand,1981; Robertson et al, 1984; Briaud et al, 1985; Bouafia, 2007; Bouafia, 2013), the net cone resistance q_c*measured by the CPT test (AFNOR, 2012, Bouafia, 2014; Bouafia, 2017), or by the undrained strength c_u determined for saturated clays (Randolph & Houlsby, 1984). It is then possible to write that:

$$P_u(z) = K_p\, p_l^*(z)\, B \quad (3)$$

K_p is called the pressuremeter lateral resistance coefficient. Since the profile of the net limit pressure P_l*, is linear, as shown in Figure 1, equalizing the values of P_u at the interface of the clay and sand layers, at a depth of 4 m, led to a value of 0.174 for K_p and to draw the profile of P_u in sand, which is consequently linear, as illustrated in Figure 8.

The same procedure was followed to extrapolate the profile of the soil reaction modulus $E_{s0}(z)$ within the sand, based on the fact that E_{s0} is proportional to the pressuremeter modulus E_M measured by the PMT test at a given depth. The soil reaction modulus $E_{s0}(z)$ can then be written as (Ménard et al, 1969; Baguelin et al, 1977; Gambin, 1979; Bouafia, 2007; Bouafia, 2013):

$$E_{s0}(z) = K_E\, E_M(z) \quad (4)$$

K_E is a dimensionless modulus number. By continuity, equalizing the values of E_{s0} at the clay/sand interface led to obtaining the value of K_E and then drawing the estimated profile of the soil reaction modulus within the sand, as illustrated in figure 9.

The procedure of construction of P-Y curves was validated by back-computation of the test pile. P-Y curves were introduced in the P-Y curve-based computer program SPULL (Single Pile Under Lateral Loads) developed at the University of Blida. This program is based on the theory of beams on elastic foundations combined with the P-Y curves method, and is capable of taking into account the nonlinearity of the P-Y curves as well as the non-homogeneity of the soil stiffness and the soil lateral resistance.

As shown in Figure 10, the computed deflections were found to be in excellent agreement with the experimental results. Moreover, Figure 6 shows very good agreement

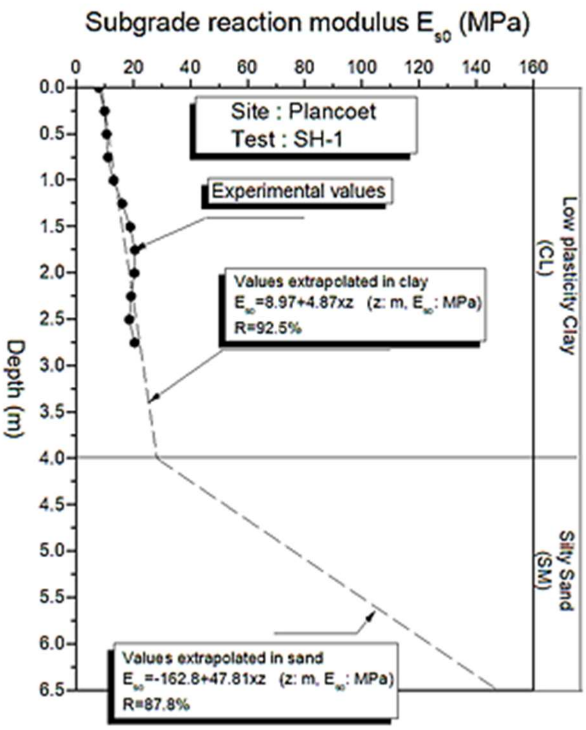

Fig. 9. Profile of the soil reaction modulus.

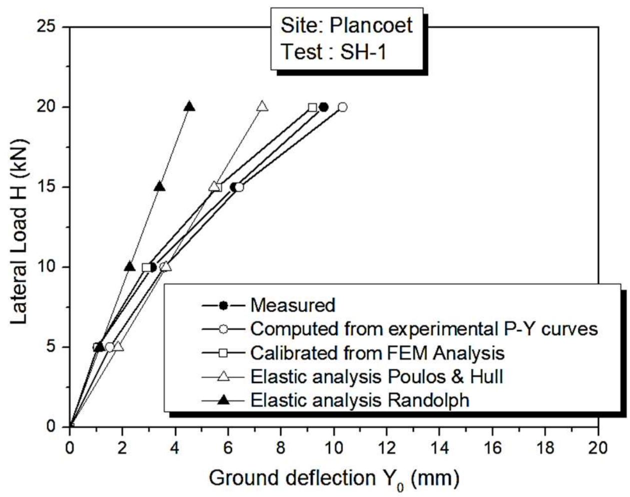

Fig. 10. Comparison of the pile deflections Y_0.

between the measured bending moment profiles and the profiles predicted from the experimental P-Y curves. It is then possible to accurately describe the lateral load-deflection of the test piles by means of these experimental P-Y curves.

4.4. Comparison with current P-Y curves methods

Typical P-Y curves at depths of 1 and 2 m within the clayey layer were directly compared to those recommended in the literature through Figures 11 and 12.

According to Ménard et al. (1969), the P-Y curves are trilinear shaped, where the first portion describing the

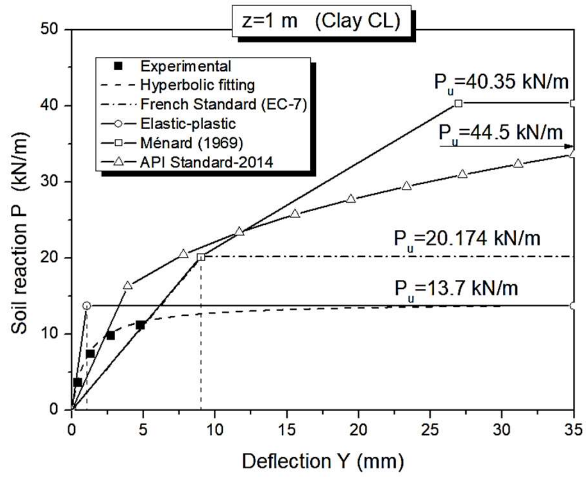

Fig. 11. Comparison of P-Y curves at 1 m of depth.

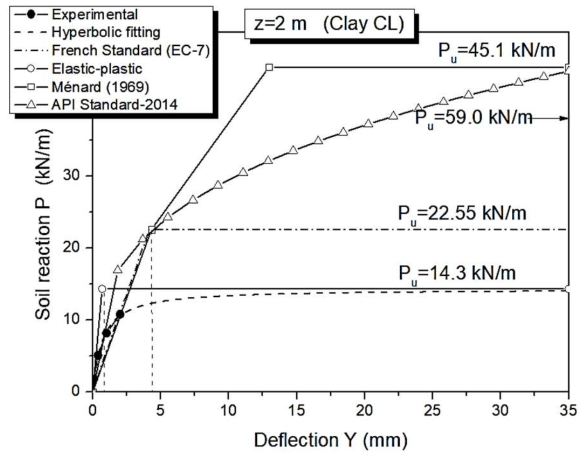

Fig. 12. Comparison of P-Y curves at 2 m of depth.

linear response has a slope equal to the lateral reaction modulus E_{s0}, the second portion has a slope equal to $E_{s0}/2$ and the third portion, describing the plastic behaviour, corresponds to the soil resistance P_u, which is equal to the net limit pressure P_l* multiplied by the pile diameter (or the frontal width) B.

The lateral reaction modulus E_{s0} was evaluated by Ménard on the basis of the settlement formula of strip foundations, by assuming the pile is an infinitely long rigid foundation whose "settlement" is horizontal and equal to the pile deflection Y.

Ménard's method was improved and integrated in the French geotechnical code NF P94-262 accompanying Eurocode 7, with a reduction in the lateral soil resistance to the net creep pressure p_f* multiplied by B (AFNOR, 2012). This adaptation was dictated by the necessity to obtain conservative prediction of the pile response at large deflections (Baguelin et al, 1978).

For the analysis of the short-term response of soft clay in the presence of free water, the API method recommends a cubic root function to describe the P-Y curves until a reference deflection of $8Y_{50}$, which is equal to $2.5\varepsilon_{50}B$, ε_{50}being the strain corresponding to one-half the maximum principal stress difference (ANSI/API, 2014). An unconsolidated undrained triaxial compression test on samples of clay led to a value of 0.014 for ε_{50} (Baguelin and Jezequel, 1971), which is in accordance with typical values for normally consolidated clays (Reese and Van Impe, 2001). Beyond the reference deflection, P is equal to the lateral resistance P_u, which is the smallest value of the following quantities:

$$P_u = c_u\ B\ min\left\{\left(3 + \frac{\gamma' z}{c_u} + \frac{J}{B} z\right), 9\right\} \quad (5)$$

J is a dimensionless factor usually taking a value of 0.5 for soft clays.

For the sand, the API P-Y curves are described by a hyperbolic tangent function, having an asymptotic value corresponding to the lateral resistance P_u, given as the smallest value as follows:

$$P_u = min\{(C_1\ z + C_2\ B)\gamma' z, C_3\ B\ \gamma' z\} \quad (6)$$

C_1, C_2 and C_3 are dimensionless factors depending on the drained angle of internal friction.

As shown in Figures 11 and 12, all the methods predict an initial portion with a slope less than the experimental portion, leading to overprediction of the small pile deflections, which is confirmed by previous comparative studies based on full-scale pile loading tests, as stated in paragraph 5.

As summarized in Table 2, compared to the experimental values, the lateral soil resistance is overestimated by all the methods, and the API and Ménard's method led to the same order of magnitude in clay. The value of P_u for sand is too high, which is expected since the API indicates that equation (6) might be unconservative in the case of sandy soil overlain by soft clay (ANSI/API, 2014).

The reference deflection Y_R is defined as the threshold of mobilization of the lateral soil resistance. According to Ménard et al. (1969) and the French standard, Y_R at a given depth is equal to ($1.5p_l^*B/E_{s0}$) and ($0.5p_l^*B/E_{s0}$), whereas the API prescripts a value of $8Y_{50}$ for clays and 3B/80 for sands.

The experimental value of Y_R was estimated by adapting the hyperbolic function and fitting the experimental P-Y curves, to an elastoplastic function, which implies that Y_R is equal to

Table 2. Comparison of the P-Y curves parameters

Depth (m)	1		2		5	
Material	Clay		Clay		Sand	
	P_u (kN/m)	Y_R/B (%)	P_u (kN/m)	Y_R/B (%)	P_u (kN/m)	Y_R/B (%)
Ménard et al.	40.35	9.6	45.10	4.62	120.00	4.00
French standard (EC-7)	20.17	3.2	22.55	1.54	60.00	1.34
API	44.50	28.6	59.0	28.6	448.0	3.75
Hyperbolic fitting	13.70	0.37	14.30	0.25	18.40	0.086

P_u/E_{s0}. According to Table 2, the values of Y_R given by Ménard's method are 3 times those of the French standard. Moreover, those of the API are remarkably high for the clay. All the methods predict much greater values than those interpreted from the elastoplastic function, with a margin of 0.08 to 0.3% of B.

4.5. Lateral reaction modulus

As illustrated in Figure 13, the profile of the pressuremeter modulus number K_E^{PMT}, computed according to equation 4, exhibits a regular increase with depth, whereas the values recommended by Ménard et al. (1969), the French standard NF P94-262 and Briaud (1997), are rather constant and much less than the experimental values. Consequently, it is expected that these methods lead to an overestimation of the pile deflections.

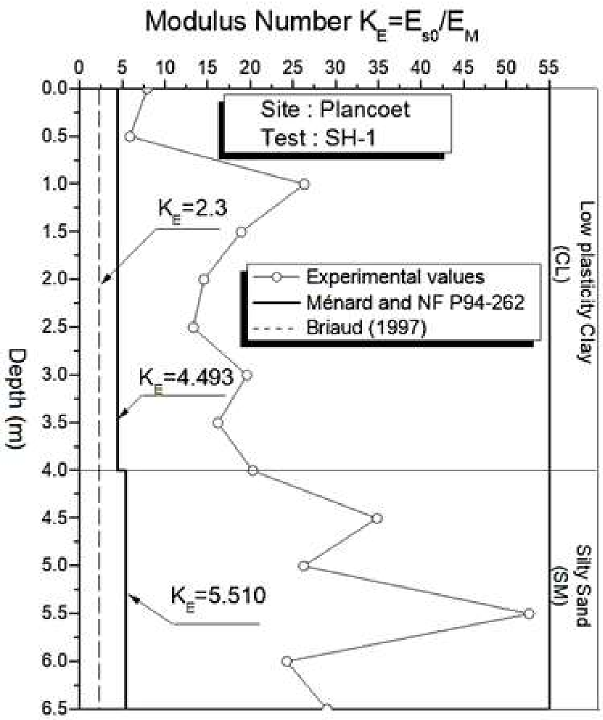

Fig. 13. Comparison of the values of the pressuremeter modulus number.

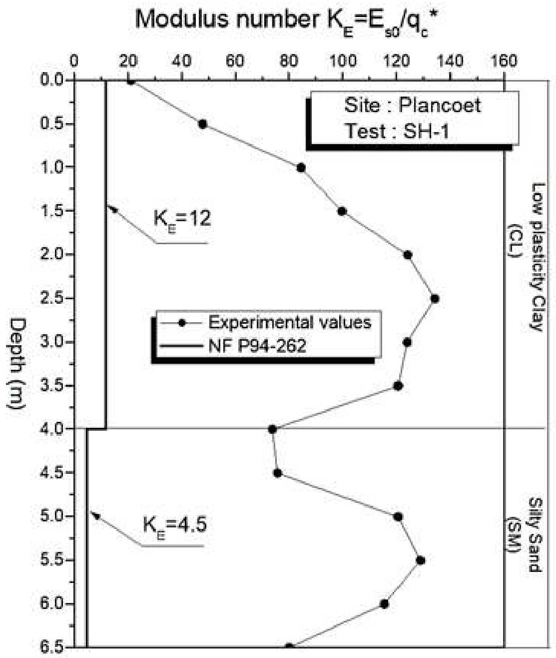

Fig. 14. Comparison of values of the cone modulus number

Although it is a measure of the soil resistance, the cone resistance q_c measured by the CPT test is usually correlated to the soil stiffness, which makes it possible to correlate E_{s0} to q_c. Figure 14 illustrates the profiles of the cone modulus number K_E^{CPT} defined as follows:

$$K_E^{CPT} = E_{s0} / q_c^* \tag{7}$$

q_c^* is the net cone resistance measured by the CPT. It can be stated that K_E varies with depth contrary to the values prescribed by the French standard NF P94-262, which are constant with depth and much less than the experimental values, which leads to an overestimation of the pile deflections by such prescribed values.

4.6. Lateral soil resistance

According to equation (3), and as summarized in Table 3, the values of K_p range in a margin between 0.3 and 3, which shows some uncertainty in predicting the lateral resistance (Bouafia, 2007). In the methods of Briaud et al. (1982, 1985), Baguelin (1978, 1982), and Robertson et al. (1984, 1985), it is suggested to construct the P-Y curves point by point from the experimental PMT expansion curves. As illustrated in Figure 15, the values of K_p computed from the experimental curves vary slightly along the depth and may be characterized by average values of 0.27 and 0.17 for clay and sand, respectively. These values are less than those found elsewhere according to Table 3, which leads to an overprediction of the lateral soil resistance compared to those interpreted from the experimental P-Y curves.

Table 3. Comparison of values of K_p

Method	Value of K_p	Remarks
Present study	0.27 in clay 0.17 in sand	
Ménard et al. (1969)	1.0	
French standard NF P94-262	0.5	Usual correlation $P_f^*/P_l^* \simeq 0.5$
Dunant (1981)	1.0	
Briaud et al. (1982)	0.83	Bored pile in sand
Baguelin et al. (1978, 1982)	0.3-3.0	
Robertson et al. (1984, 1985)	1.50	Beyond a critical depth of 4B in sand

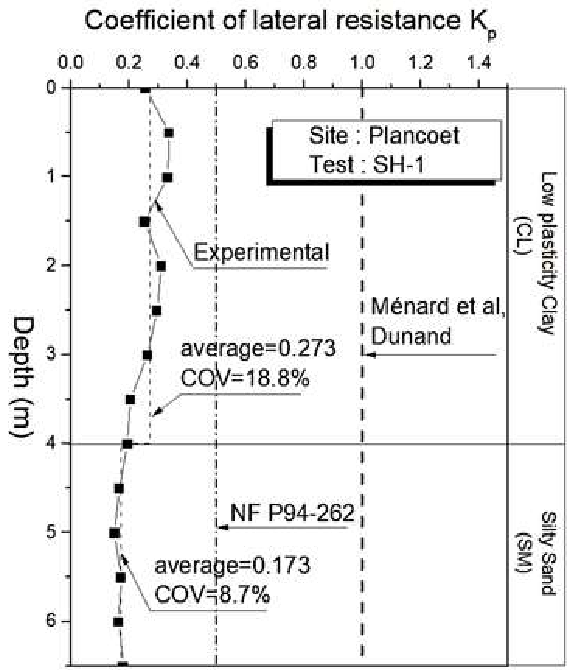

Fig. 15. Comparison of values of K_p.

A similar interpretation of the lateral soil resistance was undertaken by using the net cone resistance q_c^*, based on the following equation:

$$P_u(z) = K_c\, q_c^*(z)\, B \tag{8}$$

K_c is a dimensionless factor called the cone lateral resistance coefficient. As shown in Figure 16, reasonable agreement of the average values of K_c with those recommended by the French standard NF P94-262: 0.2 for clay and 0.08 for sand, is obvious.

5. Deflection analysis by the P-Y curves methods

As illustrated in Figure 17, Ménard's method overpredicts the pile deflections with a margin of the ratio predicted

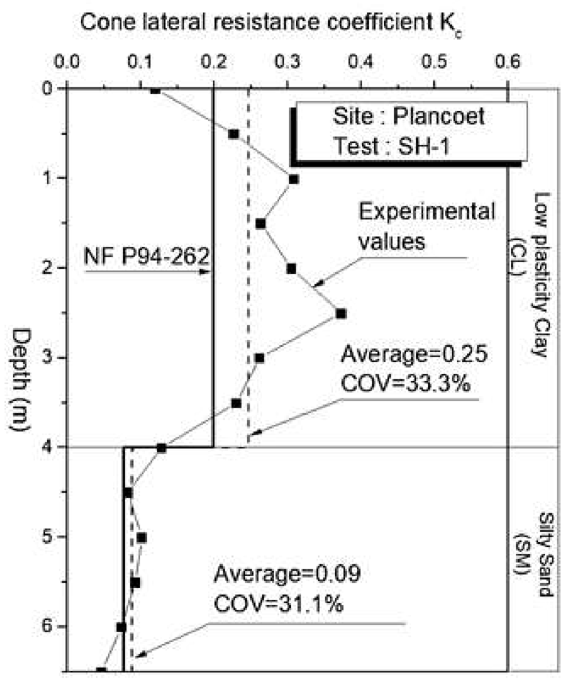

Fig. 16. Comparison of values of K_c.

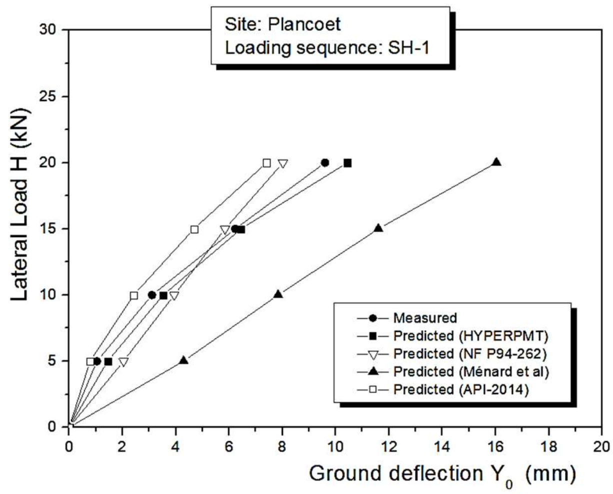

Fig. 17. Comparison of pile deflection based on P-Y curves.

deflection to the measured deflection, varying between 1.67 and 4.10, which may be explained, according to Figures 11 and 12, by smaller values of the lateral reaction modulus E_{s0} as well as by greater values of the lateral resistance P_u, with respect to the experimental values. This fact is confirmed by many investigators who have demonstrated from the analysis of full-scale pile loading tests that this method overpredicts small deflections (Frank 1984; Briaud 1986; Baguelin and Jézéquel 1972; Baguelin et al. 1990). What can be obvious from Figure 18 is that the French standard NF P94-262 provides a relatively good prediction of the deflections, although it overpredicts at small deflections and underpredicts at larger deflections (Hadjadji

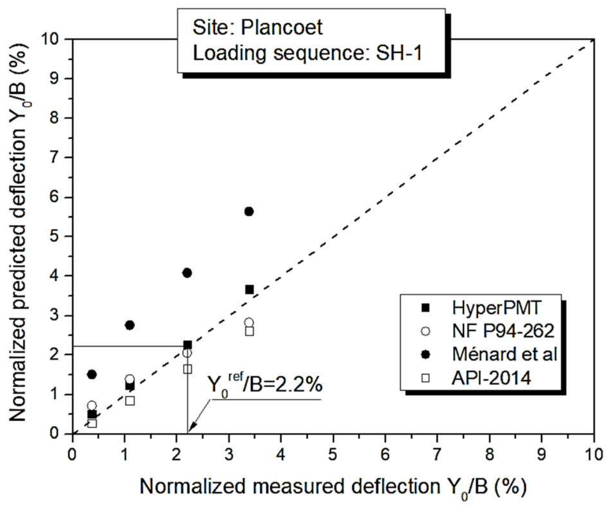

Fig. 18. Comparison of normalized pile deflections based on P-Y curves.

et al, 2002). Furthermore, it can be stated that the API standard leads to a pessimistic prediction by default of approximately 23% with respect to the experimental pile deflections.

Findings from full-scale lateral loading tests show that at the same site, the behavior of piles having different pile/soil stiffness ratios K_R could not be characterized by a unique lateral reaction modulus, as prescribed by the previously mentioned P-Y curve methods based on pressuremeter data (Bouafia 1990; Bouafia 1997; Bouafia 2002a). Moreover, tests on centrifuged models of instrumented piles showed rather a variation of the modulus E_{s0} as a power of K_R (Bouafia 2002b).

Research undertaken at the University of Blida suggested a simple semi-empirical P-Y method called HYPERPMT to determine the P-Y curve parameters from the PMT data as a power function of K_R. The PMT modulus number K_E and the PMT lateral resistance coefficient K_p may be written as (Bouafia, 2005; Bouafia, 2013):

$$K_E = a\, K_R^{\;n} \tag{9}$$

$$K_p = b + c\, K_R^{\;m} \tag{10}$$

Coefficients a, b, c, n and m are summarized for the clay and sand in Table 4. Figure 18 shows an excellent prediction of

Table 4. Values of coefficients a, b, c, n and m

Soil	D/B	K_R	a	n	b	c	m
Sand	D/B ≥ 10	≥ 0.01	0.33	- 0.5	0.0	3.0	0.5
		< 0.01	3.40	0.0	0.0	0.31	0.0
Clay	D/B ≥ 5		1.85	- 0.2	0.3	1.0	1.0

the pile deflections made by the HYPERPMT, which accounts for the lateral pile/soil stiffness ratio when defining the P-Y curve parameters.

The HYPERPMT method of constructing P-Y curves has a very good predictive capability, as demonstrated by comparing the predicted pile deflections to the measured deflections of several single piles loaded in a variety of soils (Bouafia, 2013).

If the elastoplastic scheme of the load-deflection curve, depicted in Figure 3, is adopted, one can conventionally define the domain of small displacements as bounded by the pile top reference deflection Y_R (equal to 2.2% of the diameter), and one notice therefore in Figure 18 that the French standard and the HYPERPMT method predict relatively well the measured deflections, whereas the API code method underestimates them, and Menard's method overestimates them.

6. Finite elements modelling

The calibration procedure was undertaken by 3D finite element analysis to determine the soil elastic modulus E within the scope of elastic–plastic modelling of the soil behavior. As shown in Figure 19, the FEM mesh encompasses a half-cylinder mass high of 2D, with a radius of 2.3D, D being the embedded pile length. The model dimensions were adopted based on the work of Lachenani (2003), according to which these dimensions are the minimum values leading to results independent of the model size. The mesh is regular and symmetric with respect to the neutral axis of the pile, but it is more refined in the vicinity of the pile.

The Mohr–Coulomb elastic-perfectly plastic constitutive model was used to describe the soil response. The surface-to-surface contact method of Abaqus software was used to model the nonlinear response at the pile/soil interface.

The soil is modeled as a tri-layered mass, where the first layer is the clayey layer. The intermediate layer is formed of silty sand characterized by an effective friction angle $\phi'=33°$, cohesion $C'=7.5$ kPa, and dilation angle ψ equal to 3°, computed according to Bolton (1986) by taking a constant volume friction angle ϕ_{cv} equal to 30° and a Poisson's ratio equal to 0.3.

The third layer consists of high plasticity clay. Both clayey layers are supposed to exhibit undrained behavior characterized by an internal friction angle equal to 0, a dilation angle equal to 0, a Poisson's ratio of 0.45, and an

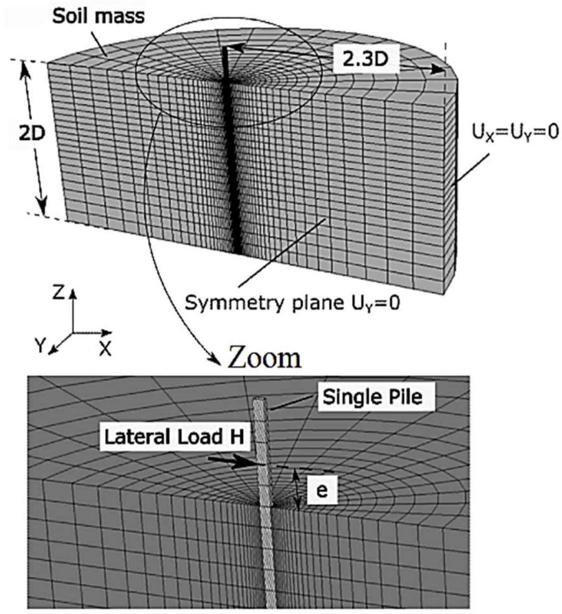

Fig. 19. ABAQUS 3D FEM Mesh and Dimensions.

undrained shear resistance C_u estimated by Skempton's correlation as follows (Skempton, 1944):

$$C_u = (0.11 + 0.37\, I_p)\, \sigma'_{v0} \tag{11}$$

I_p and σ'_{v0} are the plasticity index and the effective vertical overburden stress, respectively.

To simulate the linear variation of E with the depth and of the undrained cohesion c_u in the clayey layers, the soil material was defined in Abaqus through the "user-defined field" option. First, a "field variable" as depth z was added through the Material Editor of Abaqus/CAE, and then a user-defined USDFLD subroutine was written in Fortran Language within the Microsoft Visual Studio environment. This subroutine, defining the linear variation of E with vertical coordinates, was implemented in Abaqus through the Job Editor, and finally, the compilation and calculation were carried out.

The pile model was modelled by 4 noded quadrilateral stress-displacement shell elements with reduced integration and a large-strain formulation S4R, whereas the soil was modelled by 3D continuum stress/displacement and 8 noded reduced-integration elements C3D8R (Haouari & Bouafia, 2020).

The pile/soil interface was taken into account according to the "contact pairs" (surface-to-surface) approach of Abaqus using the "basic Coulomb friction model", which assumes that the friction coefficient μ is the same in all directions (isotropic friction). It was adopted hereafter a value of 0.25 for μ.

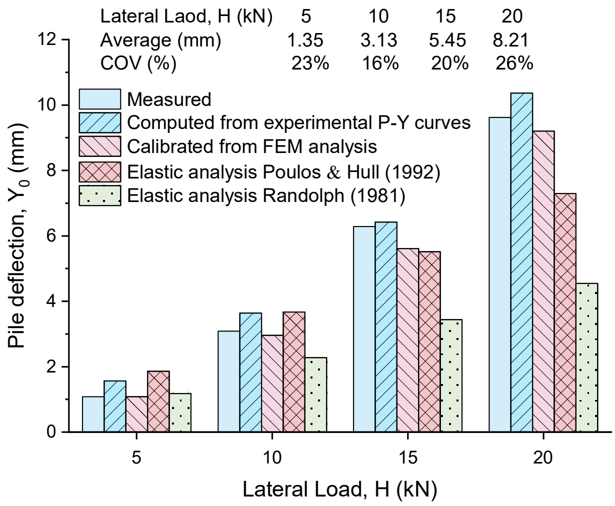

Fig. 20. Comparison of pile deflections.

Table 5. Measured vs elasticity-based pile deflections

H (kN)	5	10	15	20
Y_0 (mm) Measured	1.04	3.11	6.23	9.60
Y_0 (mm) Poulos & Hull (1992)	1.82	3.64	5.46	7.27
Y_0 (mm) Randolph (1981)	1.13	2.26	3.40	4.52

Two surfaces are required for defining the surface-to-surface contact, namely, the master surface and the slave surface. The outside surface of the pile is defined as the master surface, whereas the inner side surface of the soil, which is directly in contact with the pile, is defined as the slave surface.

The calibration of the FEM model consists of varying the gradient m of the linear profile E(z) until good matching of the load-deflection curve as well as the bending moment profiles with the experimental curves. As shown in Figure 10, excellent matching was found for a gradient m of 6.4 MPa/m.

Moreover, the determined soil elastic modulus was used to predict the pile deflections by using elasticity-based methods, namely, Poulos and Hull (1992) and Randolph (1981). As shown in Figures 10 and 20 and summarized in Table 5, these two methods are capable of accurately predicting the small deflections, the domain for which these methods were developed, and exhibit an underprediction for high levels of lateral loads.

7. Conclusions

In this paper, a full-scale analysis of the pile response under monotonic lateral loads in bi-layered soil consisting of a soft clayey layer overlying a sandy soil is presented, which is an original case study focusing on the P-Y curves and the numerical modelling of the monotonic response of this particular pile/soil configuration.

After description of the experimental conditions as well as the geotechnical aspects of the experimental site, the analysis of the bending moment profiles led to the construction of P-Y curves, which allowed a detailed comparative study of the current P-Y curve-based methods recommended in the literature.

The procedure of construction of the experimental P-Y curves was validated by back-computation of the test pile by using the P-Y curve-based computer program SPULL, which led to an excellent agreement of the predicted deflections and bending moments with the experimental results.

The calibration procedure of the elastic soil modulus was undertaken on the basis of a 3D FEM analysis within the scope of elastic–plastic modelling of the soil behaviour, where the soil was modelled as a Gibson's mass surrounding the pile. The determined soil elastic modulus was used to predict the pile deflections by using some usual elasticity-based methods, which led to good prediction of the small pile deflections.

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9. Data Availability Statement

Due to the nature of this research, participants of this study do not agree for their data to be shared publicly, so supporting data are not available.

Disclosures

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