A pressuremeter method for laterally loaded piles

Méthode pressiométrique pour pieux chargés latéralement

J. L. BRIAUD, Associate Professor, Civil Engineering Department, Texas A&M University, College Station, Texas, USA
T. D. SMITH, Assistant Professor, Division of Engineering and Applied Science, Portland State University, Portland, Oregon, USA
L. M. TUCKER, Research Associate, Civil Engineering Department, Texas A&M University, College Station, Texas, USA

SYNOPSIS A method is presented to predict the behavior of piles subjected to monotonic lateral loads on the basis of the results of pressuremeter tests performed in prebored holes. The method is used to predict the pile head response of 17 laterally loaded piles including driven and bored piles ranging from 0.32m to 1.37m in diameter and from 3m to 21m in length. The predictions are compared with the load test results.

BASIC MODEL

The F-y/Q-y Mechanism

A laterally loaded pile derives most of its resistance from frontal resistance Q and from friction resistance F at the pile soil interface (Fig. 1).

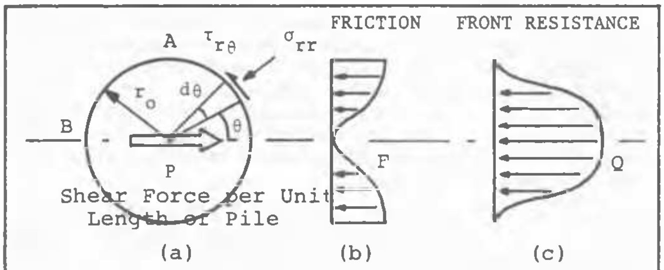

FIG. 1 - Friction and Front Resistance Distribution (elasticity)

Q and F have units of force per unit length. On any elemental area of soil pile interface, a shear stress $\tau_{r\theta}$ and a normal stress σ_{rr} exist (Fig. 1). The elementary force per unit pile length dF due to the component of $\tau_{r\theta}$ in the direction of the shear force P (Fig. 1) is:

$$dF = \tau_{r\theta}\, r_o \sin\theta\, d\theta \qquad (1)$$

where r_o is the radius of the pile and θ is the angle between the direction of the lateral load and the direction of σ_{rr}.

$$F = \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \tau_{r\theta} \sin\theta\, d\theta \qquad (2)$$

Similarly, the elementary forces due to σ_{rr} are:

$$dQ = \sigma_{rr}\, r_o \cos\theta\, d\theta \qquad (3)$$

$$\text{and } Q = \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} \sigma_{rr}\, r_o \cos\theta\, d\theta \qquad (4)$$

No friction nor frontal resistance is considered to exist in the back of the pile (ABC on Fig. 1). Baguelin et al. (1977) gave the expression of σ_{rr} and $\tau_{r\theta}$ for a linear elastic soil:

$$\sigma_{rr} = \sigma_{rr(max)} \cos\theta, \text{ with } \sigma_{rr(max)} = \frac{P}{2\pi r_o} \qquad (5)$$

$$\tau_{r\theta} = \tau_{r\theta(max)} \sin\theta, \text{ with } \tau_{r\theta(max)} = \frac{P}{2\pi r_o} \qquad (6)$$

Use of Eqs. 1-6 leads to:

$$Q = \sigma_{rr(max)} \times 2r_o \times \frac{\pi}{4} \qquad (7)$$

$$\text{and } F = \tau_{r\theta(max)} \times 2r_o \times \frac{\pi}{4} \qquad (8)$$

The total soil resistance P to the lateral movement of the pile element y is the addition of the front resistance Q and the friction resistance F. As a result, the P-y curve is the addition of the Q-y curve and the F-y curve.

Experimental Evidence

Fig. 2 gives an example of existence of the two components. A 0.90m dia. bored pile was loaded laterally in a stiff clay with an undrained shear strength from unconfined compression tests averaging 100 kPa (Kash et al., 1977). Pressure cells were installed on the shaft as shown on Fig. 2 in order to record the front pressure. The shaft was loaded and the resulting load-deflection curve is shown on Fig. 2. At a horizontal load of 47.3

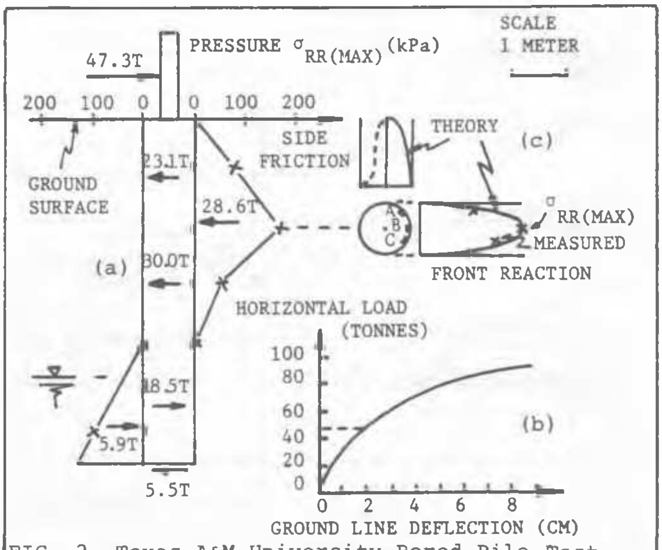

FIG. 2 - Texas A&M University Bored Pile Test

tonnes applied at 0.75m above the ground line, the soil resistance due to front reaction was calculated from the pressure cell readings by using Eq. 7 and calculating the area under the diagram of $\sigma_{rr(max)}$ versus depth. Considering front resistance only, horizontal and moment equilibrium cannot be obtained. The soil resistance due to friction was calculated by using the following equation for F:

$$F = \tau_{r\theta(max)} \times 2r_o \times 1 \tag{9}$$

Eq. 9 allows for enough friction to exist in the back of the pile (dotted line on Fig. 2c) to raise the shape factor $\pi/4$ in Eq. 8 to one in Eq. 9. It was further assumed that $\tau_{r\theta(max)}$ was equal to one half of the unconfined compression strength, in other words, that at that point in the test the full friction resistance was mobilized. After including the friction forces (Fig. 2) corresponding to the full shear strength of the stiff clay, both horizontal and moment equilibrium are approximately satisfied. Other similar case histories have been published that confirm the existence of friction and front resistance (Briaud, et al., 1983). This example tends to indicate two points: 1. the friction resistance is an important part of the total resistance, 2. the friction resistance is fully mobilized before the front resistance because it takes less displacement to mobilize friction than point resistance. Hence, a soil model which distinguishes between friction and front resistance is a proper model.

The Q-y Curve and the Pressuremeter Curve

The theoretical distribution of the elementary forces dQ was found to match the measurements recorded on three pressure cells (A,B,C on Fig. 2) on the shaft of the load test. This validated the use of Eq. 7 provided $\sigma_{rr(max)}$ could be obtained. Pressuremeter tests were performed in a prebored hole and the pressuremeter curves were compared with the response of the pressure cells which measured $\sigma_{rr(max)}$ on the shaft. Fig. 3(b) shows the comparison between pressure cell responses at the front of the pile and the pressuremeter response. For the load cells, P is the cell pressure ($\sigma_{rr(max)}$) and y/R is the lateral movement of the cell y divided by the pile radius R. Fig. 3(b) shows very good agreement between pressure cells and pressuremeter response (Smith, 1983). This tends to prove that the curve obtained from a pressuremeter test performed in a prebored hole simulates well the reaction of the front pressure cell for a bored pile. In the proposed method the front resistance model will be obtained as follows:

$$Q(\text{front}) = p(\text{pmt}) \times B(\text{pile}) \times S(Q) \tag{10}$$

where

Q(front) = the soil resistance due to front reaction (in force/unit length of pile)
p(pmt) = the net pressuremeter pressure
B(pile) = the pile width or diameter
S(Q) = the shape factor = 1.0 for sq. piles
= $\pi/4$ for rnd. piles

$$y(\text{pile}) = y(\text{pmt}) \times \frac{R_{(\text{pile})}}{R_{(\text{pmt})}} \tag{11}$$

where

y(pile) = the lateral deflection of the pile
R(pile) = pile radius
y(pmt) = increase in radius of the soil cavity in the pressuremeter test
R(pmt) = initial radius of the soil cavity in the pressuremeter test

If the pile is driven into the soil and fully displaces it, one would expect that the resulting Q-y curve would be different from the one for a bored pile in the same soil. In the case of a bored pile preboring the hole for the pressuremeter seems to be appropriate; in the case of a closed end driven pile it may be more appropriate to drive the pressuremeter in place. Alternatively the hole can be bored, the pressuremeter expanded a first time to simulate the driving of the pile and then expanded a second time. The Q-y curve for the driven pile can be derived from the reload portion of the pressuremeter curve.

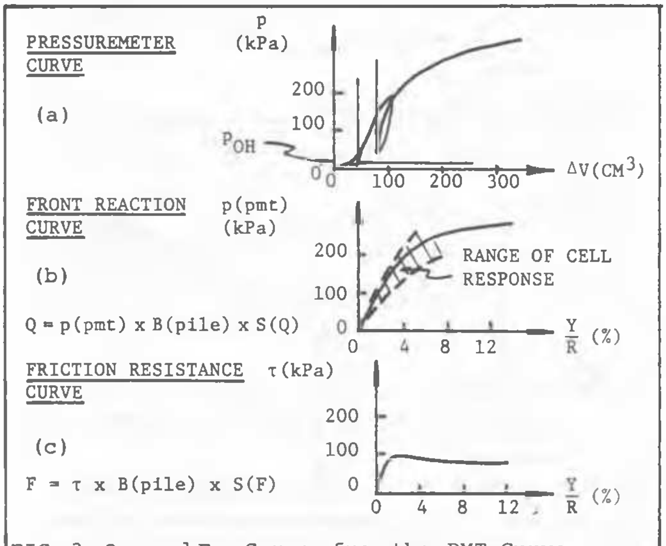

FIG. 3- Q-y and F-y Curves from the PMT Curve

The F-y Curve and the Pressuremeter Curve

Based on the previous theoretical and experimental considerations the friction on the sides of the pile according to the proposed method is:

$$F(\text{side}) = \tau(\text{soil}) \times B(\text{pile}) \times S(F) \tag{12}$$

where F(side) = the soil resistance due to friction resistance
B(pile) = the pile width or diameter
S(F) = the shape factor = 2 for sq. piles
1 for rnd. piles
τ(soil) = the maximum soil shear stress at the soil-pile interface

It has been shown that a shear stress-strain curve can be obtained from the selfboring pressuremeter curve by a theoretical method called the subtangent method (Baguelin et al., 1978). Applying the subtangent method to the curve of a pressuremeter test performed in a prebored hole (preboring pressuremeter test) leads to shear moduli which are too low and peak shear strength which are too high. However, appling the subtangent method to the reload curve from a preboring pressuremeter test (Fig. 3) leads to shear moduli comparable to selfboring shear moduli. As a result, in the proposed approach, the reload portion of the preboring pressuremeter curve is used to obtain the τ(soil) versus y(pmt)/R(pmt) curve (Fig. 3).

CRITICAL DEPTH: SOIL-STRUCTURE INTERACTION

The Phenomenon

When a pile is loaded laterally to failure, there is a zone just below the ground surface where the lateral soil resistance is reduced. This zone of reduced lateral resistance extends to the critical depth, D_c. Above D_c the absence of constraint caused by the stress free ground surface influences the lateral soil resistance. Below D_c, that influence is negligible and the lateral soil resistance is called the deep soil resistance. The basic model described in previous sections refers to the deep soil resistance. Above D_c the shallow soil resistance is obtained by multiplying the deep resistance by a reduction factor. The evaluation of D_c and of the reduction factor is described in this section. The variation of soil resistance along a laterally loaded pile can be approximated by the solid line CBA on Fig. 4. The horizontal pile displacement y increases from

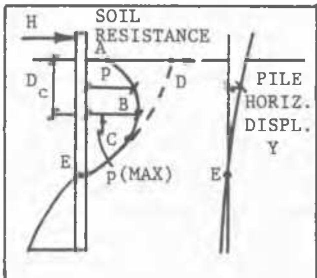

FIG. 4 - Pile Critical Depth

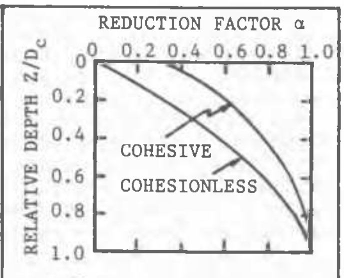

FIG. 6 - Pile Reduction Factor

point E to the ground surface. If there were no weakening influence due to the close proximity of the stress free ground surface, the variation of resistance in a soil of constant strength with depth would be as shown by the dotted line CD. Instead, the soil resistance distribution follows CBA with a maximum resistance $p_{(max)}$, at the critical depth D_c. Within D_c the soil resistance p is less than $p_{(max)}$ and the ratio $p/p_{(max)}$ is the reduction factor α.

Pile Critical Depth and Reduction Factor

The critical depth D_c is a soil structure interaction phenomenon. The closed form solution of the interaction problem in linear elasticity makes use of the key interaction parameter:

$$\ell_o = \sqrt[4]{\frac{4EI}{E_{sh}}} \tag{13}$$

where ℓ_o is the pile transfer length, E is the modulus of the pile material, I is the moment of inertia of the pile cross-section perpendicular to the plane of bending and E_{sh} is the modulus of subgrade reaction of the soil. The following interaction parameter called relative rigidity was defined for this study:

$$RR = \frac{1}{B} \sqrt[4]{\frac{EI}{p_L^*}} \tag{14}$$

where B is the pile diameter and p_L^* is the pressuremeter net limit pressure within the critical depth. The correlation of Fig. 5 is a plot of the relative critical depth D_c/B versus soil-pile relative rigidity RR for 10 piles. The data show that, in the same soil, piles of different rigidity generate different relative critical depth (Caisson and H pile in the silt of Plancoet) and that the same pile generates different relative critical depths in different soils (same pipe pile at Sabine and at Lake Austin). Within the critical depth D_c, the reduction factor is defined as $p/p_{(max)}$ (Fig. 5). Fig. 6 shows the recommended values for the variation of α within the critical depth D_c. These recommendations are

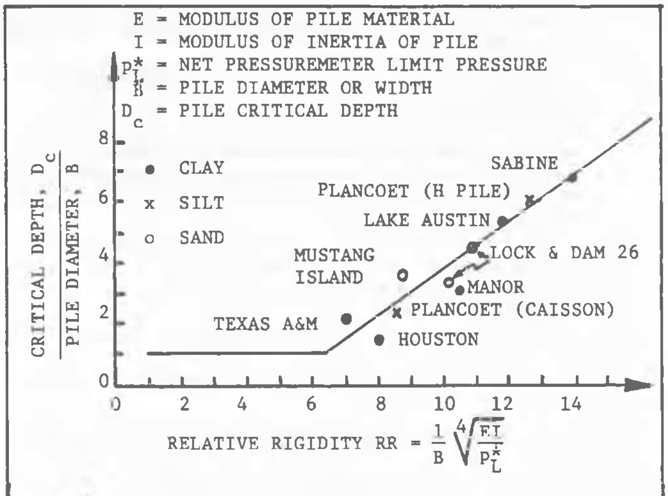

FIG. 5 - Pile Critical Depth vs. Soil-Pile Relative Rigidity

based on experimental data collected for 4 piles in clay and 2 piles in sand.

Pressuremeter Critical Depth and Reduction Factor

The critical depth phenomenon exists also for the pressuremeter. Baguelin et al. (1978) state that the pressuremeter seems to be below its critical depth Z_c if it is at least one meter deep in clay and two meters deep in sand. This would correspond to critical depths Z_c equal to 30 and 60 pressuremeter radii in clay and sand, respectively, for the conventional 35mm radius probes. The statement made by Baguelin et al. (1978) seems to refer to the limit pressure. A finite element study was performed to investigate the pressuremeter critical depth problem at small strain levels (Smith, 1983). The results of that study combined with the above statement lead to the recommendations shown on Fig. 7 for the pressuremeter critical depth and reduction factor β.

SUMMARY OF THE PROPOSED METHOD

More details and background on this method can be found in Briaud et al. (1984) and Smith (1983). The following is a summary:

1. Perform pressuremeter tests in a prebored hole at the site with close spacing near the surface and down to a depth of approximately 20 pile diameters.
2. Correct the pressuremeter curves for membrane resistance, system compressibility and

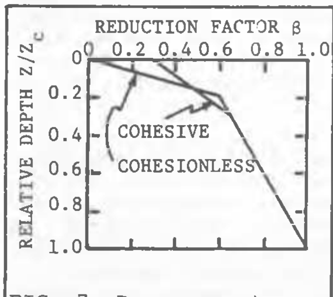

FIG. 7 - Pressuremeter Reduction Factor

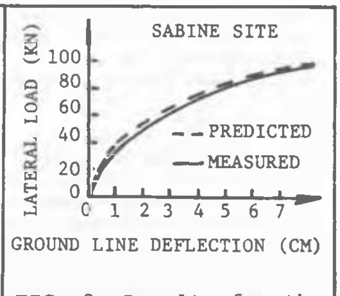

FIG. 8 - Results for the Pipe Pile at Sabine

pressuremeter critical depth effect by using the factor β.

3. Obtain the front reaction curves (Q-y) by using Eqs. 10 and 11 together with the pressuremeter curves of step 2 as shown on Fig. 3 for bored piles. For driven piles use the reload pressuremeter curves.
4. For any test within the pile critical depth apply the proper α reduction factor to obtain the true Q-y curves.
5. Obtain the friction resistance curves (F-y) by applying the subtangent method to the reload pressuremeter curves and then by using Eqs. 11 and 12.
6. Obtain the P-y curves by adding at each depth the Q-y curve to the F-y curve.
7. Run the finite difference program to obtain the pile response.

COMPARISON OF MEASURED AND PREDICTED BEHAVIOR

The proposed method was used to predict the lateral load versus ground line deflection of 17 piles ranging from 0.32m to 1.37m in diameter and from 3m to 21m in length. These piles were loaded with time to failure varying from a few hours to a few days. Fig. 8 shows a case where a very good prediction was obtained. The results of the other comparisons between the predictions and the load test results are shown in Table 1. The comparisons were made by

TABLE 1: COMPARISON OF PREDICTED AND MEASURED BEHAVIOR

SITE	PILE TYPE	PILE DIAM. (m)	EMBED. LENGTH (m)	PRED. LOAD / MEAS. LOAD (at 2%*)	PRED. LOAD / MEAS. LOAD (at 10%*)	SOIL TYPE
LACKLAND	BORED	0.457	10.5	1.14	1.37	CLAY
DELTA	BORED	0.610	3.0	1.36	----	CLAY
BAYTOWN	BORED	0.610	11.9	1.13	1.24	CLAY
VIRGINIA	BORED	1.370	3.51	1.21	----	CLAY
CAROLINA	BORED	1.370	4.54	0.59	----	SAND
IOWA	BORED	1.370	4.57	0.58	0.75	CLAY
TEXAS A&M '77	BORED	0.915	6.10	0.87	1.00	CLAY
TEXAS A&M '78	BORED	0.76	4.57	0.60	0.87	CLAY
TEXAS A&M '79	BORED	0.76	4.57	0.71	1.06	CLAY
HOUSTON	BORED	0.762	13.0	1.08	----	CLAY
MUSTANG ISLAND	DRIVEN	0.610	21.0	0.95	----	SAND
LAKE AUSTIN	DRIVEN	0.324	12.2	2.00	1.16	CLAY
SABINE	DRIVEN	0.324	12.2	1.15	1.02	CLAY
LA BAULE 1	DRIVEN	0.609	6.0	1.11	1.04	CLAY
LA BAULE 2	DRIVEN	0.609	6.0	1.00	0.93	CLAY
PLANCOET CAISSON	JACKED	0.95	4.40	0.80	----	SILT
PLANCOET H	DRIVEN	0.357	6.10	1.15	0.92	SILT

* AT GROUND LINE DEFLECTION EQUAL TO 2% OR 10% OF PILE DIAM.

comparing predicted and measured loads (Q_p and Q_m respectively) at two displacement levels: 2% of the pile diameter and 10% of the pile diameter. For the 2% displacement comparison, the average of the ratio Q_f/Q_m was 1.030 and the standard deviation 0.337. For the 10% displacement comparison, the average of the ratio Q_p/Q_m was 1.003 and the standard deviation 0.190.

CONCLUSION

A pressuremeter method to predict the behavior of laterally loaded piles is described. The soil resistance model is the addition of a friction model and a front resistance model. It is shown on one case history that the pressuremeter curve gives the front resistance model directly and that the friction model can be obtained from the pressuremeter curve. The critical depth effect for the pile is handled through a soil structure interaction approach; the critical depth effect for the pressuremeter is also included. The method is used to predict the behavior of 17 piles. The predictions are compared to the results of load tests. These comparisons allow to quantify the accuracy of the load predictions by a coefficient of variation of 33% at a pile head deflection equal to 2% of the pile diameter and of 16% at a pile head deflection equal to 10% of the pile diameter.

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