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# Design of Laterally Loaded Driven Piles Using the Flat Dilatometer

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**ABSTRACT:** The nonlinear subgrade reaction method (*P-y* curves) is widely used for the design of laterally loaded piles. This method replaces the soil reaction with a series of independent nonlinear Winkler springs.

A preliminary semi-empirical approach for the determination of *P-y* curves using data obtained from a flat dilatometer test (DMT) is presented and evaluated. A brief description of the equipment, testing procedures, and the theory that enables the family of *P-y* curves to be determined are presented. The *P-y* curves are used as input for an existing finite-difference program, which calculates pile deflection versus depth at various lateral loads.

An evaluation of the proposed method is presented using data from three full-scale laterally loaded test piles. A comparison and discussion are provided between the predicted and measured behavior of the piles during lateral loading.

**KEY WORDS:** flat dilatometer, piles, lateral loading, *P-y* curves, predictions, field test behavior

The nonlinear subgrade reaction method is widely used for the design of laterally loaded piles. This method replaces the soil reaction with a series of independent nonlinear springs. The nonlinear behavior of the soil springs is represented by *P-y* curves, which relate soil reaction and pile deflection at points along the pile length. Most of the existing methods for obtaining *P-y* curves are highly empirical. Often little account is taken of the method of pile installation and the influence that this may have on the soil behavior. Early methods to obtain *P-y* curves used empirical methods based on laboratory data [*1*].

Several methods have recently been proposed for the design of laterally loaded piles using pressuremeter data [*2-5*]. Most of these methods make use of preboring pressuremeter results, using a Ménard type pressuremeter, and do not attempt to model the disturbance caused by a driven pile since the pressuremeters are placed in a prebored hole. However, it is possible to install the pressuremeter in a manner that models the disturbance caused during pile installation. For driven displacement piles, the pressuremeter can be pushed into the soil in a full-displacement manner. For cast-in-place or bored piles, a prebored or self-bored pressuremeter test can model the disturbance caused during pile installation. The method by Robertson et al. [*4*] uses the results from a pressuremeter pushed into the soil to model the installation of a driven displacement pile.

Although the pressuremeter methods have been shown to usually provide adequate results, several problems still exist. Some of the major difficulties relate to the following:

- pressuremeter tests are often difficult and expensive to perform,
- usually only a limited number of tests are performed, and
- the large size of most pressuremeters make it difficult to obtain data close to the ground surface where the lateral response of piles is most influenced.

The flat dilatometer test (DMT), however, is a simple, repeatable and economic in-situ penetration test. Test results are obtained every 200 mm in depth and an increasing amount of experience is being developed to relate DMT data to soil parameters.

The small size of the dilatometer blade enables data to be collected close to the ground surface where the lateral response of piles is most influenced.

Also, since the dilatometer blade is pushed into the soil it can be considered a model of a driven pile [*6*].

The pressuremeter methods, however, have the advantage that the cylindrical expansion can be considered a reasonable model of the lateral movement of the soil during lateral loading of piles [*5*]. Any method that uses the DMT must rely on empirical correlations that relate DMT data to the required geotechnical parameters.

This paper presents a preliminary semi-empirical method for the evaluation of *P-y* curves using data obtained from the DMT.

## Flat Dilatometer Test (DMT)

The flat dilatometer test (DMT) was developed in Italy by Marchetti [*7*]. The dilatometer is a flat plate 15 mm thick, 96 mm wide by 220 mm in length, as shown in Fig. 1. A flexible stainless steel membrane 60 mm in diameter is located on one face of the blade. Beneath the membrane is a measuring device that turns a buzzer off in the control box at the surface when the membrane starts to lift off the sensing disk and turns a buzzer on again after a deflection of 1 mm at the center of the membrane. Readings are made every 200 mm in depth. The membrane is inflated using high pressure nitrogen gas supplied by a tube pre-threaded through the rods. As the membrane is inflated, the pressures required to just

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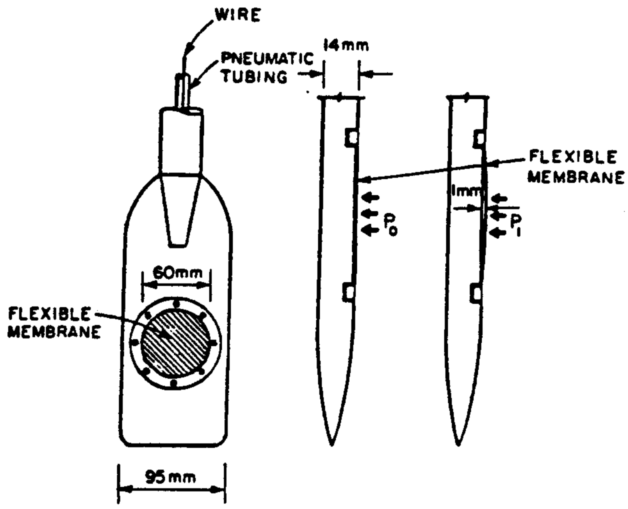


FIG. 1—*Marchetti flat plate dilatometer.*

lift the membrane off the sensing disk (Reading A), and to cause 1-mm deflection at the center of the membrane (Reading B), are recorded. Readings are made from a pressure gage in the control box and entered on a standard data form. Full details of the test procedure are given by Marchetti and Crapps [*8*].

The dilatometer is pushed into the ground at a constant rate of penetration of 20 mm/s. Before and after each sounding the dilatometer is calibrated for membrane stiffness.

The dilatometer data (Readings A and B) are corrected to pressures $P_0$ and $P_1$ to allow for offset in the measuring gage and for membrane stiffness.

Using $P_0$ and $P_1$, the following three index parameters were proposed by Marchetti [*7*]

$$I_D = \frac{P_1 - P_0}{P_0 - u_0} = \text{material index}$$

$$K_D = \frac{P_0 - u_0}{\sigma'_{vo}} = \text{horizontal stress index}$$

$$E_D = 34.6(P_1 - P_0) = \text{dilatometer modulus}$$

where $u_0$ is generally assumed to be the in-situ hydrostatic water pressure and $\sigma'_{vo}$ is the in-situ vertical effective stress. The data are reduced using a computer program supplied with the instrument. Computer graphics facilities are used to generate the completed plots.

The dilatometer equipment is extremely simple to operate and maintain. The simplicity and low initial cost of the equipment are two of the main advantages of the DMT as an in-situ test method.

Marchetti [*7*] performed DMT at about 10 well documented sites in Italy and developed empirical correlations based on these results. Correlations were developed between the three index parameters, $I_D$, $K_D$, $E_D$, and soil type, soil unit weight $K_0$, overconsolidation ratio (OCR), undrained shear strength, constrained modulus, and friction angle. All of the soil parameters were obtained from laboratory test results. The majority of the sites consisted of clay deposits with only two sites involving sand. At both sand sites the sand was very loose with relative densities around 30 to 40%. Details of the sites and the empirical correlations are given by Marchetti [*7*].

The correlations proposed by Marchetti [*7*] were based on a limited amount of data. However, many studies have recently been performed [*9-16*] to evaluate and improve some of the existing correlations. Many of the improvements have been related to DMT data in sand. In general, experience has shown that the DMT provides a good indication of soil type and reasonable values of undrained shear strength $S_u$, earth pressure coefficient $K_0$, and overconsolidation ratio (OCR) for soft to medium, uncemented, insensitive clays [*9,11,15*]. Reasonable values of these parameters have also been reported in some stiff, cemented clays [*15*]. Good experience has also been reported for evaluating friction angle ($\phi'$) and $K_0$ in silica sands when the dilatometer penetration thrust is recorded [*10,14*].

Recent research using large calibration chambers and pluvially deposited silica sand has shown a good correlation between the dilatometer modulus $E_D$ and the Young's modulus determined at 0.1% axial strain $E'$ [*15*]. For normally consolidated (NC) unaged silica sand the following relationship has been observed

$$\frac{E'}{E_D} = 1.05 \pm 0.25$$

However, for overconsolidated (OC), unaged silica sand the relationship becomes

$$\frac{E'}{E_D} = 3.6 \pm 0.80$$

The ratio of $E'/E_D$ appears to increase with decreasing $K_D$ for OC sands.

Experience has generally shown that for many soil types the DMT provides a reasonable estimate of most major soil properties.

### DMT *P-y* Method

As a first attempt to develop *P-y* curves from DMT data it was decided to adapt the early methods for estimating *P-y* curves that utilized soil properties obtained from laboratory data [*1,17*]. The equivalent laboratory input data to derive *P-y* curves can be estimated directly from the DMT. The following sections briefly outline a first attempt to develop such a method.

Matlock [*1*] proposed the use of a cubic parabola to predict *P-y* curves, which has the form

$$\frac{P}{P_u} = 0.5(y/y_c)^{0.33} \quad (1)$$

where

$$\frac{P}{P_u} = \text{ratio of soil resistance and} \quad (1)$$

$$\frac{y}{y_c} = \text{ratio of pile deflection.}$$

This cubic parabola is valid for short-term, one-way static loading and for soils that behave in a strain hardening manner under this loading. Figure 2 shows the cubic parabolic *P-y* curve. The cubic parabolic *P-y* curve has been used for the DMT *p-y* method. This

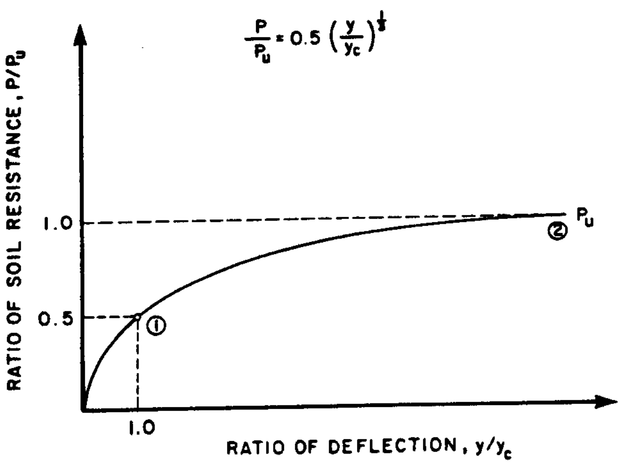


FIG. 2—*Cubic parabolic* P-y *curve for strain hardening soils* [1].

approach requires an evaluation of the ultimate soil resistance $P_u$ and the deflection $y_c$. $P_u$ and $y_c$ are denoted on Fig. 2 as points (2) and (1), respectively.

*Cohesive Soils*

In cohesive soils $y_c$ is a function of the undrained strength of the soil, the in-situ effective stress level, and the soil stiffness. The value of pile deflection $y_c$ is based on a concept proposed by Skempton [*18*] that combines elasticity theory, ultimate strength methods, and laboratory soil properties. Skempton [*18*] showed that the strain $\epsilon_c$ related to $y_c$ is that which occurs at 50% ultimate stress from the laboratory unconfined compression test stress-strain curve. From the work of Skempton, Matlock [*1*] proposed his "soft clay method," which had the form

$$y_c = A\epsilon_c D \tag{2}$$

where

$D$ = pile diameter and
$A$ = empirical coefficient 6.35 for pile diameter in cm and $y_c$, cm.

An important consideration when using this empirical relationship is the potential scale effect. Studies by Stevens and Audibert [*19*], among others, suggest that in cohesive soils the reference deflection $y_c$ is not linearly dependent upon pile diameter but is instead approximately defined as

$$y_c = B\epsilon_c D^{0.5} \tag{3}$$

where

$B$ = empirical coefficient = 14.2 cm and
$D$ = pile diameter in cm.

It is recognized that Eq 3 is not dimensionally correct. However, Stevens and Audibert [*19*] compared Matlock's linear method with their nonlinear approximation on several full-scale lateral load tests with varying pile diameter and showed that their method agreed more closely with observed results. Therefore, Stevens and Audibert's equation has been used for this study to determine $y_c$.

The value of $\epsilon_c$ (or $\epsilon_{50}$) must be evaluated from a stress-strain curve for the soil in question. Using the hyperbolic curve fitting expression proposed by Kondner and Zelasko [*20*], the following relationship can be derived

$$\epsilon_c = \left(\frac{1}{2 - R_f}\right)\frac{\sigma_f}{E_i} \tag{4}$$

where

$R_f$ = ratio of deviatoric failure stress over deviatoric ultimate stress (take equal to 0.8),
$\sigma_f$ = deviatoric failure stress = 2 × $S_u$ for cohesive soil,
$S_u$ = undrained shear strength, and
$E_i$ = initial tangent modulus.

which simplifies to

$$\epsilon_{50} = \frac{1.67S_u}{E_i} \tag{5}$$

The initial tangent modulus $E_i$ can be estimated from the DMT as

$$E_i = F_c E_D \tag{6}$$

where

$F_c$ = empirical stiffness factor and
$E_d$ = dilatometer modulus.

From experience gained at the University of British Columbia (UBC) and by others [*9,11,12,16*] an $F_c$ value of approximately 10 is suggested as a first approximation for cohesive soils ($I_D \leq 1.0$). The undrained strength of the soil $S_u$ can be obtained from the DMT using the existing empirical correlation [*7*].

Therefore, combining Eqs 3, 5, and 6 yields

$$y_c = \frac{23.67S_u D^{0.5}}{F_c E_D} \tag{7}$$

where

$y_c$ is in cm,
$D$ is in cm, and
$F_c$ = 10 (as a first approximation for cohesive soils).

The evaluation of the ultimate static lateral resistance $P_u$ is given by Matlock [*1*] as

$$P_u = N_p S_u D \tag{8}$$

where

$N_p$ = nondimensional ultimate resistance coefficient,
$S_u$ = undrained soil strength (from DMT), and
$D$ = pile diameter.

At considerable depth it is generally accepted that the coefficient $N_p$ should be equal to 9. Near the surface, because of the lower confining stress level and surface boundary effects, the value of $N_p$ reduces to the range of 2 to 4. Matlock [*1*], among others, proposed the following equation to describe this variation

$$N_p = 3 + \frac{\sigma'_{vo}}{S_u} + \left[J\frac{x}{D}\right] \tag{9}$$

where

$N_p \leq 9$,
$x$ = depth,
$\sigma'_{vo}$ = effective vertical stress level at $x$, and
$J$ = empirical coefficient (as given in Table 1).

*Cohesionless Soils*

As for cohesive soils, the values of $P_u$ and $y_c$ must be determined in terms of values obtained from DMT data. The ultimate lateral soil resistance $P_u$ is determined from the lesser value given by the following two equations

$$P_u = \sigma'_{vo}[D(K_p - K_a) + xK_p \tan\phi' \tan\beta] \tag{10}$$

$$P_u = \sigma'_{vo}D[K_p^3 + 2K_oK_p^2 \tan\phi' + \tan\phi' - K_a] \tag{11}$$

where

$\sigma'_{vo}$ = vertical effective stress at depth $x$,
$D$ = pile diameter,
$\phi'$ = angle of internal friction,
$K_a$ = Rankine active coefficient = $\dfrac{1 - \sin\phi'}{1 + \sin\phi'}$,
$K_p$ = Rankine passive coefficient = $1/K_a$,
$K_o$ = coefficient of earth pressure at-rest, and
$\beta = 45° + \phi/2$.

Equations 10 and 11 are after Reese et al. [*21*] and Murchison and O'Neill [*22*]. The value of $\phi'$ can be estimated using the DMT method suggested by Schmertmann [*14*]. However, the DMT pushing force is required for this method. The coefficient of earth pressure at-rest $K_o$ can be estimated using either the DMT method suggested by Schmertmann [*14*] or Marchetti [*13*].

The reference pile deflection $y_c$ for cohesionless soils is evaluated from

$$y_c = 2.5\epsilon_{50}D \tag{12}$$

where

$y_c$ = in cm and
$D$ = pile diameter in cm.

The value of $\epsilon_{50}$ is evaluated, as for cohesive soils, using Eq 4. The deviatoric failure stress $\sigma_f$ is taken to be [*23*]

$$\sigma_f = \frac{2\sin\phi'}{(1 - \sin\phi')}\sigma'_{vo} \tag{13}$$

TABLE 1—*Values of J recommended by Matlock* [1].

| Value of $J$ | Soil Type | Soil Tested |
|---|---|---|
| 0.5 | soft clay | Sabine clay |
| 0.25 | stiff clay | Lake Austin clay |

the value of $\phi'$ is estimated from the DMT test [*14*]. As for cohesive soils, $R_f$ is taken to equal 0.8. The initial tangent modulus $E_i$ can be estimated from the DMT as

$$E_i = F_\phi E_D \tag{14}$$

where

$F_\phi$ = empirical stiffness factor and
$E_D$ = dilatometer modulus.

For the prediction of lateral pile response, an $F_\phi$ value of 1 was assumed as a first approximation for cohesionless soils ($I_D > 1.0$). Therefore, combining Eqs 12 through 14 yields

$$y_c = \frac{4.17 \cdot \sin\phi' \cdot \sigma'_{vo}}{E_D \cdot F_S \cdot (1 - \sin\phi')} \times D \tag{15}$$

where

$y_c$ and $D$ are in cm.

The method outlined above is only applicable for static monotonic loading of single piles. It should be possible to estimate *P-y* curves for cyclic loading using the reduction coefficient suggested by Matlock [*1*] and Reese et al. [*21*]. The effect of cyclic loading on the *P-y* curves has not been addressed in this study but is the object of a testing program presently underway at UBC.

The method outlined above also does not address the problem of group effects. This is a basic problem for all *P-y* related methods when piles are closely spaced in a group. The elastic methods described by Poulos and Davis [*24*] provide some guidelines for correction factors.

The proposed DMT method to obtain the *P-y* curves relies on many empirical correlations. In clays the major soil parameters are $S_u$ and $E_i$. In sands the parameters are $\phi'$, $K_o$, and $E_i$. At working lateral loads where pile deflections should be small, the most important parameter is the soil stiffness $E_i$. The proposed analysis is therefore sensitive to changes in $E_i$. For both clays and sands the pile deflection $y_c$ is inversely proportional to $E_i$. Therefore, the major variables in the proposed method are the empirical stiffness factors $F_c$ and $F_\phi$.

The values suggested for $F_c$ and $F_\phi$ are a preliminary attempt to enable an evaluation of the method to be made. Results presented later in the paper will show the influence of changes in these empirical stiffness factors.

**Application of DMT to *P-y* Analysis**

The DMT provides data every 200 mm during a sounding. Therefore, $P_u$ and $y_c$ values are computed correspondingly at 200-mm intervals. In order to use the finite-difference program, LATPILE [*25*], which can handle only up to 20 *P-y* curves, the near continuous record of $P_u$ and $y_c$ must be averaged into a maximum of 20 corresponding layers.

A flowchart describing the steps involved in producing *P-y* curves using DMT data and then predicting lateral pile behavior using LATPILE is presented in Fig. 3. In Fig. 3 it can be seen that engineering judgment is necessary to divide the profiles of $P_u$ and $y_c$ into a maximum of 20 layers. All four computer programs are in FORTRAN with DMT.UBC and LATPILE.UBC being available programs that have been modified by UBC.

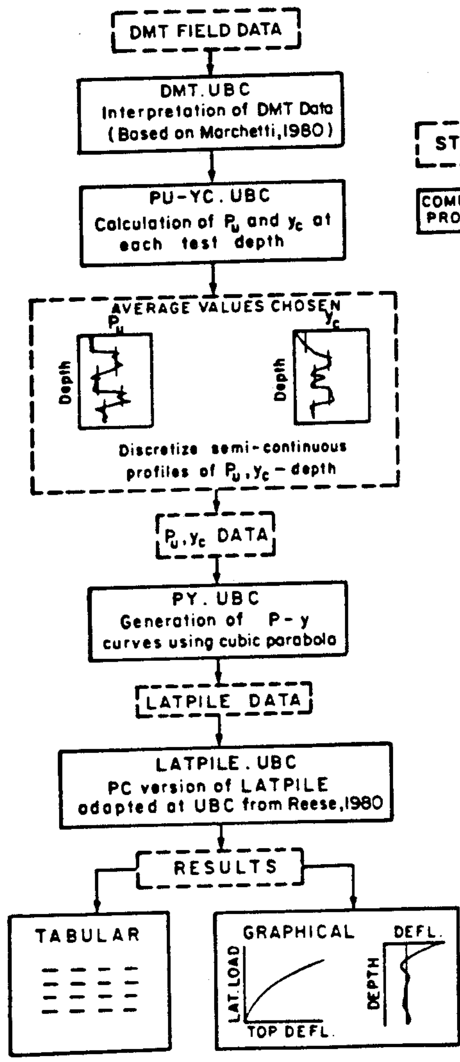


FIG. 3—*Flowchart for determining* P-y *curves from DMT data.*

The DMT data was used to identify soil type using the classification system proposed by Marchetti [7]. Basically, cohesive soil behavior was assumed when $I_D \leq 1.0$ and cohesionless soil behavior assumed when $I_D > 1.0$.

## Evaluation of the DMT *P-y* Method

In order to evaluate the DMT *P-y* method, three full-scale lateral load tests were carried out at the UBC pile research site (UBCPRS) with the assistance of the British Columbia Ministry of Transportation and Highways (BCMOTH). The location of the site is shown in Fig. 4. The geology of the site is predominantly post-glacial Fraser River delta deposits, which are approximately 200 m thick [*26*]. At the site, 2 to 4 m of nonhomogeneous fill exists at the surface. For the purpose of facilitating in-situ testing, making pile driving possible, and studying lateral pile behavior, the nonhomogeneous fill was removed in the general area of the piles and replaced with clean river sand. Beneath the fill there is a deposit of soft organic silty clay extending to a depth of 15 m below ground surface. Underlying the organic silty clay is a deposit of medium dense sand to a depth of about 30 m. The sand is underlain by deep deposit of normally consolidated clayey silt with thin interbedded sand layers. An example of DMT data with intermediate parameters, $I_D$, $K_D$, and $E_D$, to a depth of 33 m at the site is shown in Fig. 5. Complete details of the site and in-situ testing program are given by Davies [*27*].

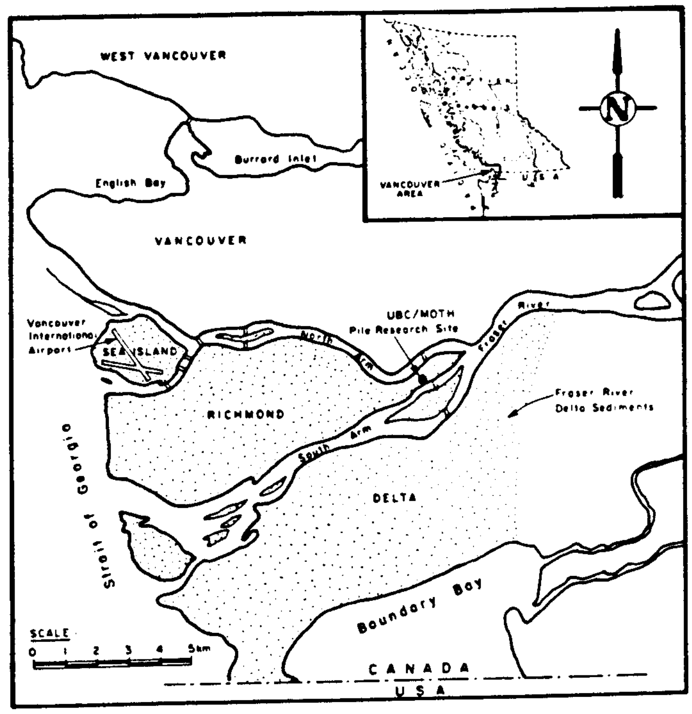


FIG. 4—*General location of pile research site.*

Six steel pipe piles were driven at the site; one pile 914 mm in diameter with a 19-mm wall thickness and five 324-mm-diameter piles by either 9.5-mm (four piles) or 11.5-mm wall thickness (one pile). Static monotonic lateral load tests were performed on three piles. Full details of the test program are given by Davies [*27*].

The averaged $P_u$ and $y_c$ values (for $D = 91.4$ cm) were selected from those computed using the DMT data. Figure 6 shows the average values chosen from the calculated $P_u$ and $y_c$ profiles using the DMT data shown in Fig. 5. These average values were used as input to derive *P-y* curves according to the equations presented earlier.

A summary of the calculated and measured load deflection curves at the pile head is shown in Figs. 7*a*, 8*a*, and 9*a* for three of the test piles. These piles are all of differing sizes as noted. Also, calculated and measured pile deflections versus depth profiles are shown in Figs. 7*b*, 8*b*, and 9*b* for one value of the lateral load.

The results in Fig. 7 for the largest pile (914 mm diameter) show that the predicted deflection agrees well with the measured deflection. The predicted deflection is approximately 25% larger than the measured deflection at the pile head under large load (1100 kN), and agreement is closer at lower loads. The deflected shape versus depth profiles at a load of 1100 kN also agree with the points of contraflexure, both occurring at about 11 m depth. The results in Fig. 8 for the thinner walled (9.5 mm) 324-mm-diameter pile again show reasonably good agreement between predicted and measured deflection. The difference between the predicted and

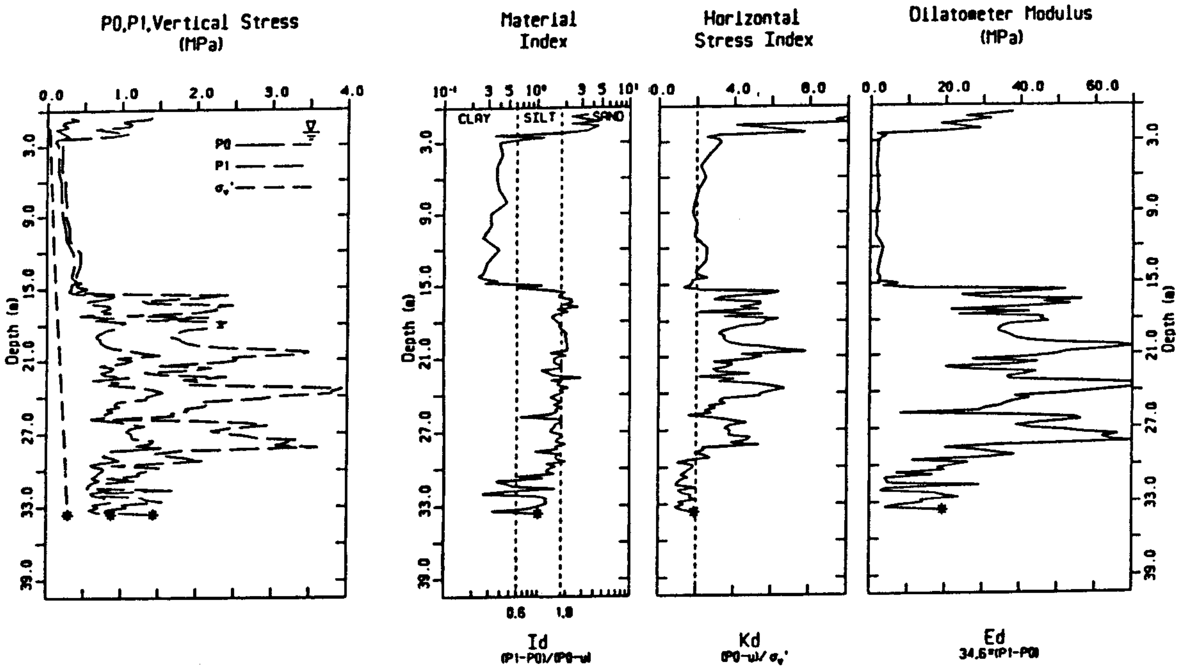


FIG. 5—*Intermediate geotechnical parameters from DMT: pile research site.*

measured results is approximately 30% under large loads with the predicted values being higher. The deflected shape versus depth profiles for a 120-kN load are of similar shape, with a point of fixture at a depth of about 5 m.

The results in Fig. 9 for the other 324-mm (11.5-m-wall) diameter pile show a similar agreement between predicted and measured deflection.

In all cases the calculated bending moments predicted using the DMT derived *P-y* curves are larger than and at a similar depth to those calculated from the measured pile deflection profile.

The behavior of the smaller 324-mm-diameter piles is controlled by the response of the 2-m-thick surface sand fill. To improve the DMT *P-y* method prediction and to evaluate the sensitivity of the method to variations in input data, the empirical stiffness factor for sand $F_\phi$ was increased to a value of 2. The results of the modified analyses using $F_\phi = 2$ are also shown in Figs. 7, 8, and 9. The stiffer response of the sand using $F_\phi = 2$ produces a better overall prediction for all three piles. This result appears to be consistent with the studies reported by Jamiolkowski et al. [*15*], whereby sands show a significant increase in stiffness because of previous stress or strain history, and the stiffness factor $F_\phi$ can be close to or greater than 2 for OC sands. The sand adjacent to the piles is probably overconsolidated because of the stress and strain history caused during pile installation.

Any reasonable variation in stiffness factor for clays $F_c$ produced only a small change in the predicted response for the three piles. This illustrates the importance of the 2-m-thick surface sand fill, especially at small loads.

## Summary and Conclusions

A method for designing single, laterally loaded piles under static, one-way loads using DMT data has been briefly summarized. Results from lateral load tests on three piles of differing sizes have also been presented to illustrate the method.

The flat dilatometer test has been shown to be a viable in-situ penetration test to obtain the data necessary to generate *P-y* curves. For the displacement piles investigated, the proposed method provided reasonably good predictions of their behavior under on-way lateral loading. The proposed method was generally able to predict the lateral deflection at the ground surface and the overall deflected shape of the test piles to within about 25% of the measured values for deflections up to about 10% of the pile diameter.

Further field studies are necessary in order to evaluate the proposed DMT method and to refine the empirical stiffness factors $F_c$ and $F_\phi$ for other soil profiles and pile types. The proposed method must be used with caution until further validation has taken place. However, because of the ability of the dilatometer to obtain economic, repeatable, and near continuous data of soil response, the DMT offers a promising means of obtaining considerable data even at shallow depths below the ground surface. The ability to obtain data close to the ground surface can be very important for the design of laterally loaded piles since very little deflection occurs below a depth of approximately ten pile diameters under typical design loads [*24*].

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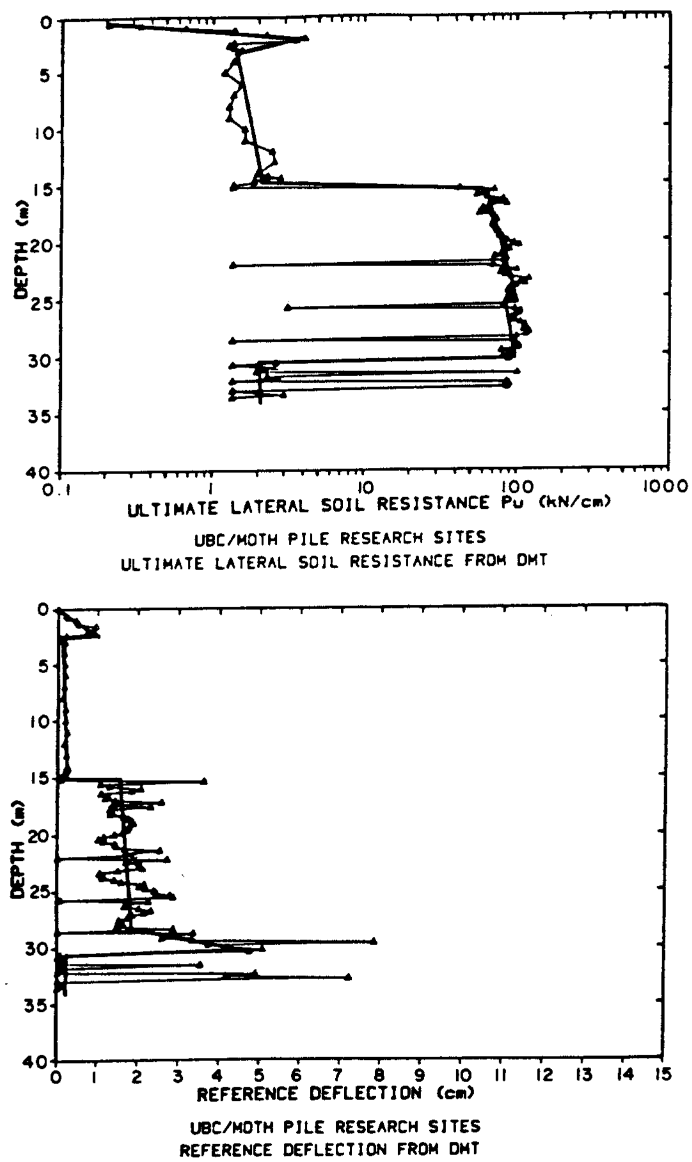


FIG. 6—*Average value of* $P_u$ *and* $y_c$ *chosen from DMT.*

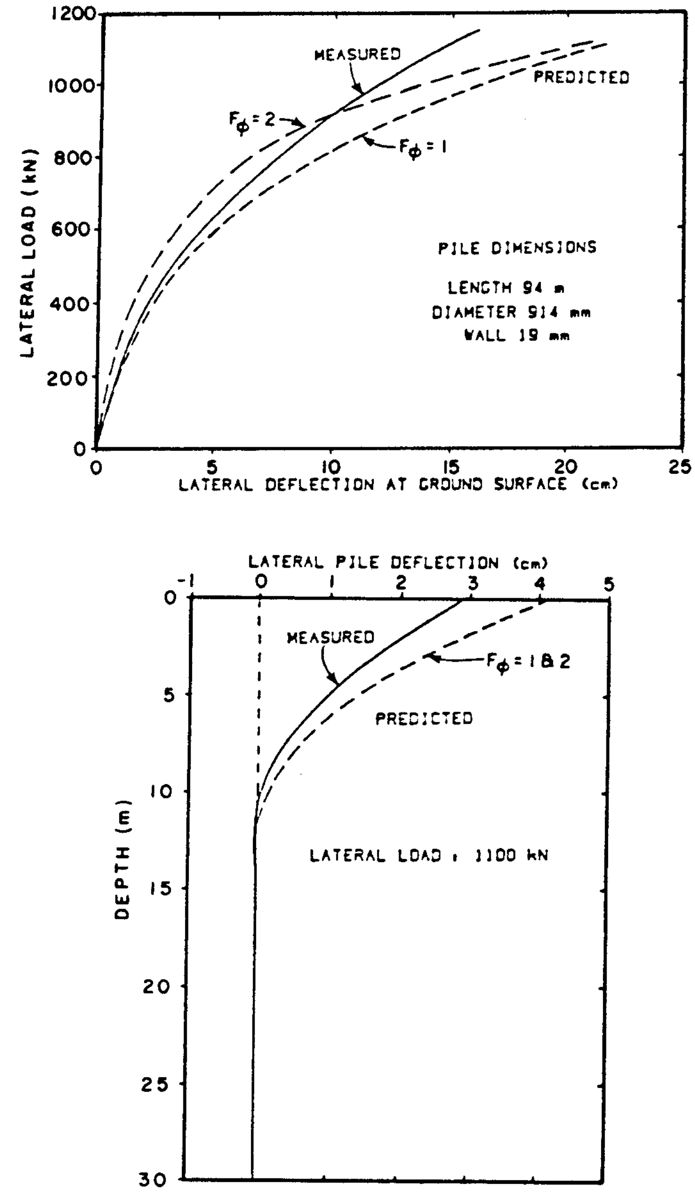


FIG. 7—*Predicted versus measured lateral pile behavior: MOTH test pile.*

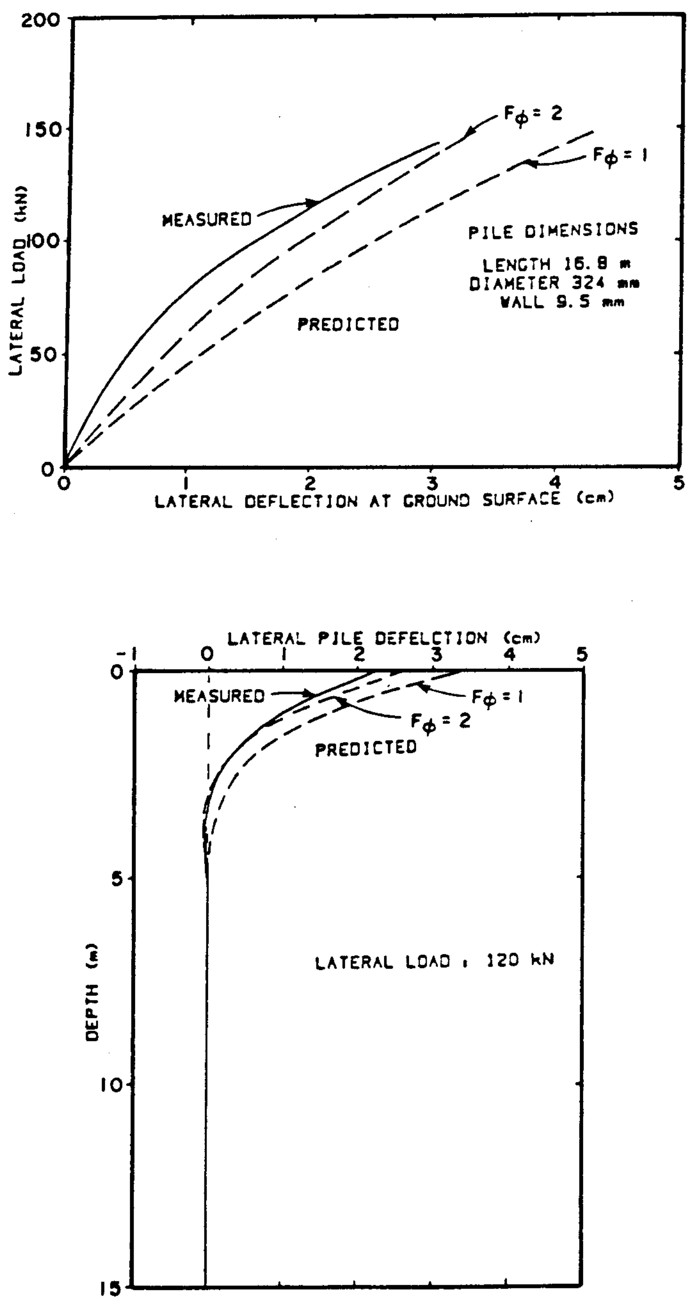


FIG. 8—*Predicted versus measured lateral pile behavior: UBC Pile 3.*

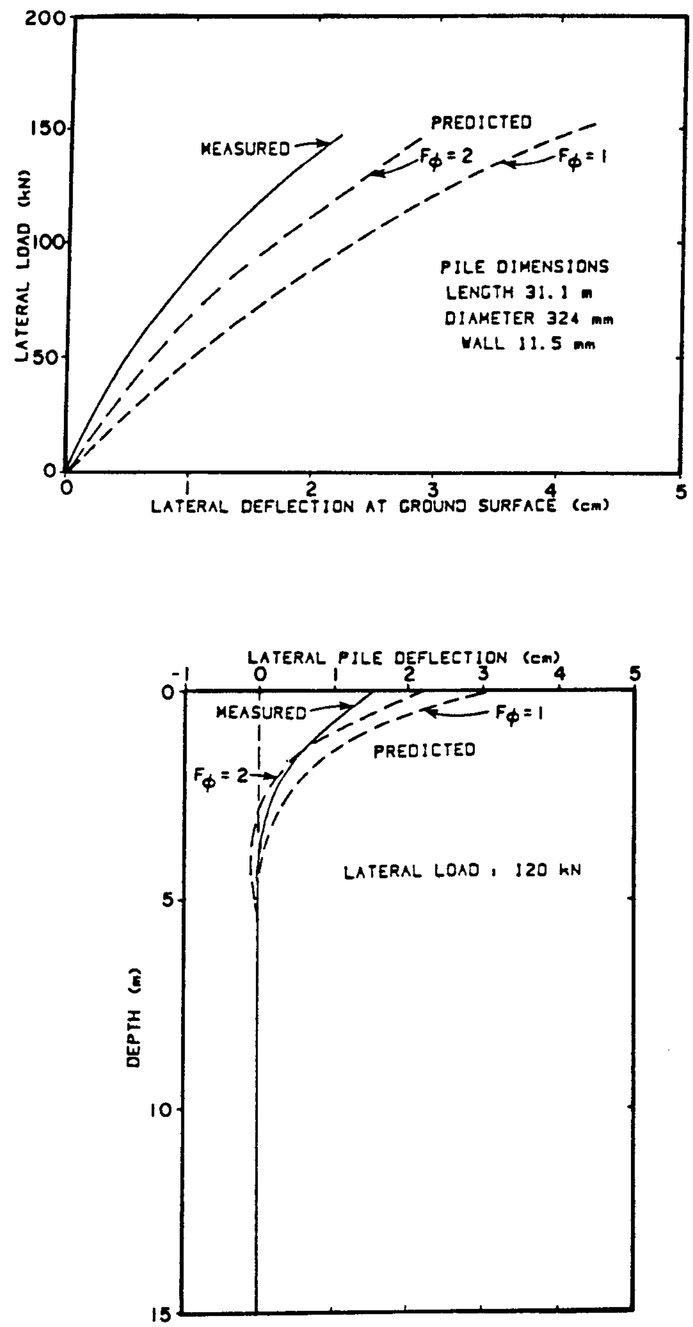


FIG. 9—*Predicted versus measured lateral pile behavior: UBC Pile 5.*

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