Load-Deflection Response of Piles in Sand: Performance Prediction Using the DMT

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The accurate prediction of the lateral load deflection response of piles is highly dependent on the proper modeling of the lateral soil stiffness. Recent papers have presented models that incorporate data obtained from the Marchetti dilatometer (DMT) to develop *p-y* curves. As the DMT data are normally obtained at 8-in. depth intervals, these models provide a nearly continuous profile of lateral soil response. This paper presents a comparison of performance predictions made using three of these models with the measured response of two 24-in.-square, 25-ft-long, prestressed concrete piles in sand. The test piles were jetted the first 12 ft and driven the remaining 13 ft into the coastal plain deposits of eastern North Carolina. The measured load-deflection response of the two piles was very similar, and although none of the models that were investigated explicitly permits the consideration of installation effects, the measured response was found to be intermediate between that predicted by a model developed for driven piles and that predicted by a model applied to drilled piers.

A common technique used in the analysis of laterally loaded piles is to idealize the lateral stiffness of the soil adjacent to the foundation as a series of independent nonlinear springs. The Winkler model idealizes the soil-pile interaction mechanism by relating the pile displacement at a point to the soil pressure at that point through a spring constant, referred to as the coefficient of subgrade reaction (k). Therefore, the accurate prediction of the lateral load-deflection response is highly dependent on the determination of the value of the coefficient of subgrade reaction, as well as its variation along the length of the pile.

Methods have been proposed for evaluating k from laboratory-determined soil modulus values and somewhat more directly from pressuremeter pressure-displacement data. Recent papers(*1–4*) have reported the validity of using data obtained from the Marchetti dilatometer test (DMT) in models to generate *p-y* curves. The DMT is capable of providing a nearly continuous profile of the coefficient of subgrade reaction because test data are typically obtained at 8-in. increments. As an in situ test device that involves a lateral displacement of soil somewhat analogous to the lateral displacement of a pile, the DMT has been shown to be a reasonable tool for lateral pile analysis. This paper presents the results of performance predictions made using *p-y* curves generated from three of these models.

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BACKGROUND

The flat dilatometer (DMT) developed by Silvano Marchetti (*5,6*) is essentially a penetration device capable of obtaining an estimate of lateral pressure and soil stiffness. The body of the dilatometer has an approximate width of 3.7 in. (95 mm) and a thickness of 0.6 in. (14 mm). When at rest, the external surface of the approximately 2.4-in.-diameter (60-mm-diameter) membrane is flush with the surrounding flat surface of the blade. The blade is usually pushed into the ground at conventional penetration test rates (1 in./sec). When the desired test depth is reached, the membrane is inflated by means of pressurized gas through a small control unit at the ground surface. Readings are taken of the pressure required to initiate movement of the membrane (related to the lateral stress existing in the ground) and the pressure required to move its center an additional approximate 0.04 in. (1 mm) into the soil (related to the soil stiffness). Both of these pressure readings are corrected for the effect of membrane stiffness. The first of these corrected pressures is called the "p_0" pressure.

Gabr and Borden (*2*) proposed a subgrade reaction model, illustrated in Figure 1, that utilizes the difference between the "p_0" pressure and the existing lateral pressure before penetration, approximating the nonlinear pressure-displacement relationship by a secant during the one-half-blade-thickness lateral displacement of the soil. This model will be referred to as Method A throughout the remainder of this paper. Schmertmann (3) has suggested a similar model, which expresses the coefficient of subgrade reaction in the following form:

$$k = \frac{(K_D - K_0) \cdot \sigma'_v}{0.5 \cdot t_b} \tag{1}$$

where σ'_v is the in situ vertical pressure at the test depth, t_b is the thickness of the dilatometer blade, K_0 is the at-rest earth pressure coefficient, and K_D is the horizontal stress index, determined by the following equation:

$$K_D = \frac{(p_0 - u_0)}{\sigma'_v} \tag{2}$$

where u_0 is the hydrostatic pore water pressure.

Both models require an estimate of the in situ lateral stress. The prediction of lateral stress in this study was based on the model proposed by Gabr and Borden (*2*) based on the evaluation of calibration chamber tests on normally consolidated

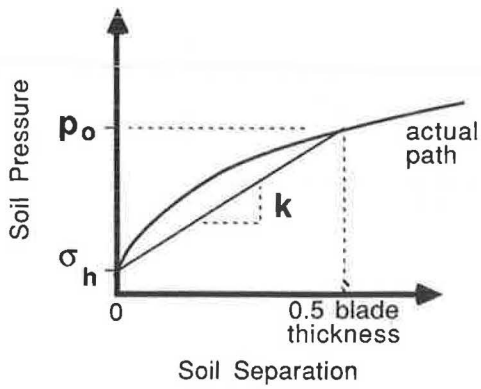

FIGURE 1 DMT subgrade reaction model.

sands. Reasonably good agreement between the predicted and measured response of three drilled piers was obtained (*2*) when this model was used in conjunction with the hyperbolic model for *p-y* curves originally proposed by Parker and Reese (*7*) and modified by Murchison and O'Neill (*8*). The continuous hyperbolic tangent function is expressed as

$$p = \eta \cdot A \cdot p_u \cdot \tanh\left[\left(\frac{k_h}{A \cdot \eta \cdot p_u}\right) \cdot y\right] \tag{3}$$

where p_u is the unmodified ultimate soil resistance, and η and A are empirical adjustment factors. For prismatic piles under static loading, the factor η is equal to 1, and A is a function of the depth from the ground surface to the point in question (z) and of the pile diameter (D), given by the formula $3 - 0.8z/D > 0.9$. The value of k_h in Equation 3 is obtained by multiplying k from Equation 1 by the pile width.

As the observed values of K_D in the sand at the field test site were much larger than K_0 (generally on the order of a factor of 10), any reasonable estimate of K_0 is sufficiently accurate for the determination of the in situ lateral stress used in Equation 1.

Schmertmann (*3*) also suggested that k determined from Equation 1 should be modified to account for size effects. In order to make this adjustment, Schmertmann estimated that for a reference width of 1 ft the k value is taken as equal to one-half of the value determined from the DMT data (i.e., Equation 1). Terzaghi's equation for size correction is then applied for widths greater than 1 ft. Thus, the corrected equation for k becomes:

$$k_{hsB} = 0.5\left(\frac{B + 1}{2B}\right)^2 \frac{(K_D - K_0)\sigma'_v}{0.5 \cdot t_b} \tag{4}$$

and the value of k_h in Equation 3 is again obtained by multiplying k_{hsB} from Equation 4 by the pile width. This model, including the correction for size effects, will be referred to as Method B.

Robertson et al. (*4*) have suggested using DMT data in the cubic parabola model for *p-y* curves proposed by Matlock (*9*) to predict the lateral response of driven piles. This expression has the form:

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_c}\right)^{1/3} \tag{5}$$

where p_u is the lateral ultimate soil resistance and y_c is the critical deflection. The critical deflection is determined by the following equation:

$$y_c = \frac{4.17 \cdot \sin\phi \cdot \sigma'_{v0} \cdot D}{F_S \cdot E_D \cdot (1 - \sin\phi)} \tag{6}$$

where F_S is an empirical stiffness factor, with a value of 2 suggested, and E_D is the dilatometer modulus. This model will be referred to as Method C.

In each of the three models described, the ultimate soil resistance as a function of depth (p_u) was determined according to the three-dimensional wedge model developed by Reese et al. (*10*).

FIELD TEST SITE

The test site was located at the North Carolina Department of Transportation (NCDOT) marine maintenance facility in Manns Harbor, North Carolina. The current phase of construction at the facility is the bulkhead shown in Figure 2. Prior construction, begun in the late 1970s, had included a warehouse, material handling area, elevated water tank, and dry dock. The load test was conducted as a proof test of the lateral capacity of anchor piles designed to support the bulkhead. The 24-in.-square, 25-ft-long, prestressed concrete test piles were installed approximately 75 ft from the proposed bulkhead. The piles were manufactured using concrete with a compressive strength of 7,000 psi and were prestressed to an average stress of 1,320 psi. The uncracked pile stiffness and cracking moment were calculated to be 0.11×10^{11} lb-in.^{2} and 365 k-ft, respectively. The piles were jetted 12 ft before being driven the remaining 13 ft with a Conmaco 100E5 hammer with an average pile blow count of 20 blows/ft. An enlarged view of the load test arrangement and the CPT and DMT locations is shown in Figure 3.

The site is located in the coastal plain of eastern North Carolina. NCDOT records indicated the typical soil profile

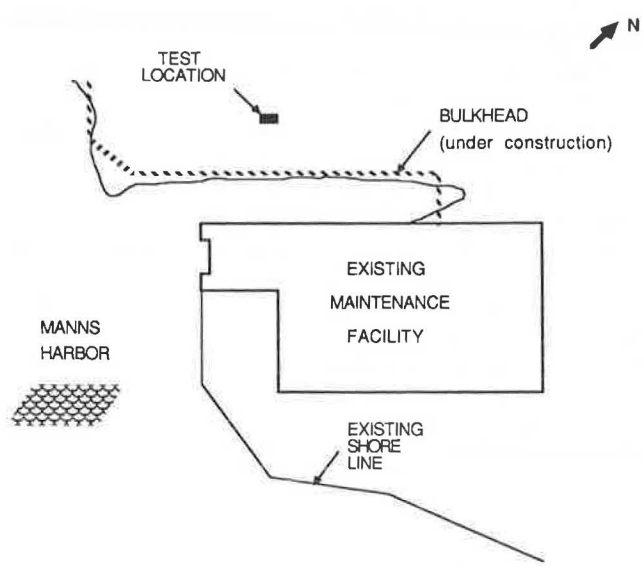

FIGURE 2 Site plan of NCDOT marine maintenance facility.

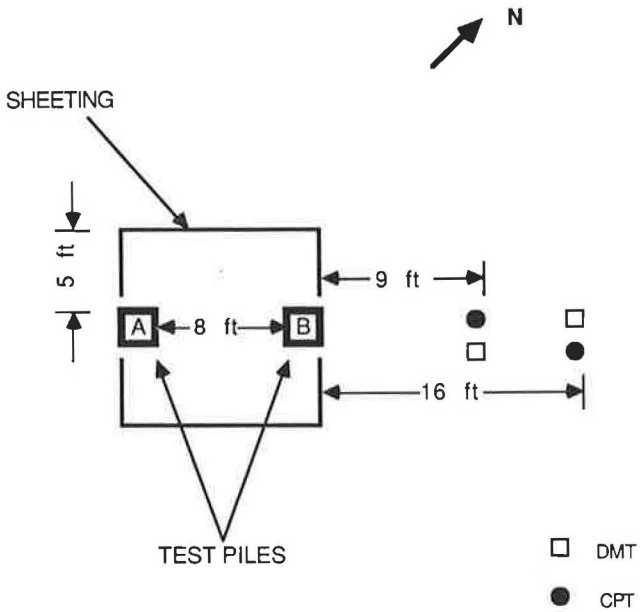

FIGURE 3 Load test arrangement and penetration test locations.

around the facility to generally consist of a 1- to 4-ft-thick layer of hydraulically placed fill, which is composed of loose, uniform fine sand containing some organic material, SP according to the Unified Soil Classification System (USCS). The fill is underlain by a 2- to 7-ft-thick layer of soft organic silty clay with traces of sand. The clay soils were classified as CL. The clay deposit was underlain by medium-dense fine sand, also classified as SP.

After installation of the test piles, the DMT and CPT were performed. The averaged data from the two dilatometer tests, shown in Figure 4, indicated the sand fill layer at the test site location to be about 7.5 ft thick. The angle of internal friction ranged from 33 to 38 degrees, with the average value being approximately 37 degrees, as shown in Figure 5. The clay layer encountered between the depths of 7.5 and 12.5 ft on the basis of DMT data, had a predicted undrained shear strength ranging between 1.6 and 2.9 psi. The friction angles obtained from the DMT for the underlying medium-dense sand ranged from 36 degrees at a depth of 13 ft to 41 degrees at 26 ft. These values were determined using the procedure proposed by Schmertmann (*11*), based on the Durgunoglu and Mitchell bearing capacity theory.

The CPT cone resistance and friction ratio are shown in Figure 6 in conjunction with the interpreted friction angles obtained using the method suggested by Schmertmann (*12*). The cone resistance (q_c) for the fill layer ranged between 66 kg/cm² near the ground surface to 12 kg/cm² near the bottom of the fill layer. The corresponding friction angles for the fill layer ranged from 37 degrees in the upper 5 ft to 33 degrees in the lower 3 ft. Average cone resistances of 3 and 60 kg/cm² were measured for the clay and medium-dense sand layers, respectively. The value of the friction angle ranged from 33 to 38 degrees for the underlying medium-dense sand layer, with an average of approximately 35 degrees.

LOAD TEST RESULTS AND PERFORMANCE PREDICTIONS

The lateral load test was conducted by North Carolina Department of Transportation personnel in accordance with ASTM Standard D3966. The load was applied at a point 3 ft below the top of the pile, as shown in Figure 7. This depth corresponded to the location at which the tiebacks were to be connected to the anchor piles. Sheet piling was installed to allow the excavation of approximately 4 ft of soil below the pile tops. The measured load-deflection response of the two piles is shown in Figure 8.

The computer program LTBASE (*13*) was used to generate the *p-y* curves at 8-in. intervals along the length of the pile and perform the load-deflection prediction. Program options were selected that incorporated the hyperbolic model previously described to generate the *p-y* curves for the sand layers, and the unified method recommended by Sullivan (*14*) to generate the *p-y* curves for the clay layer. Because the stress-strain response of the clay had not been determined from triaxial compression tests, the strain corresponding to the 50 percent stress level was estimated to be 0.02, and on the basis of the in situ determined undrained shear strength, *k* was

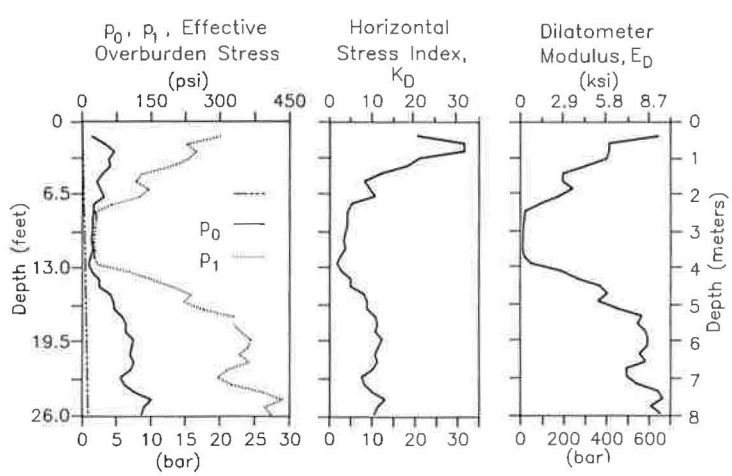

FIGURE 4 Average DMT profiles.

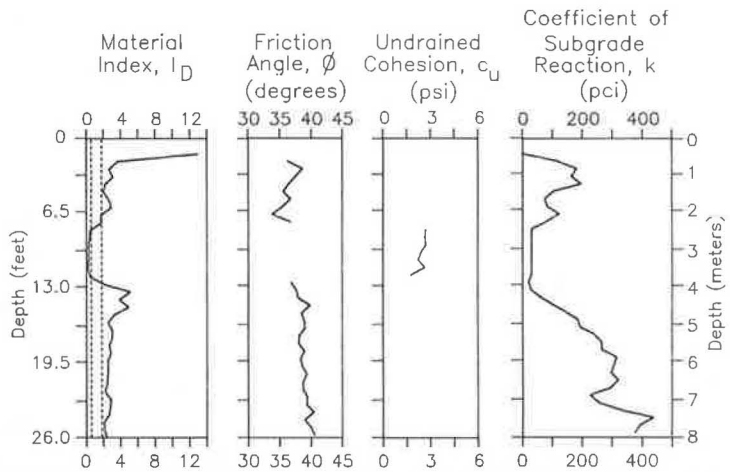

FIGURE 5 Average soil parameters and properties interpreted from DMT.

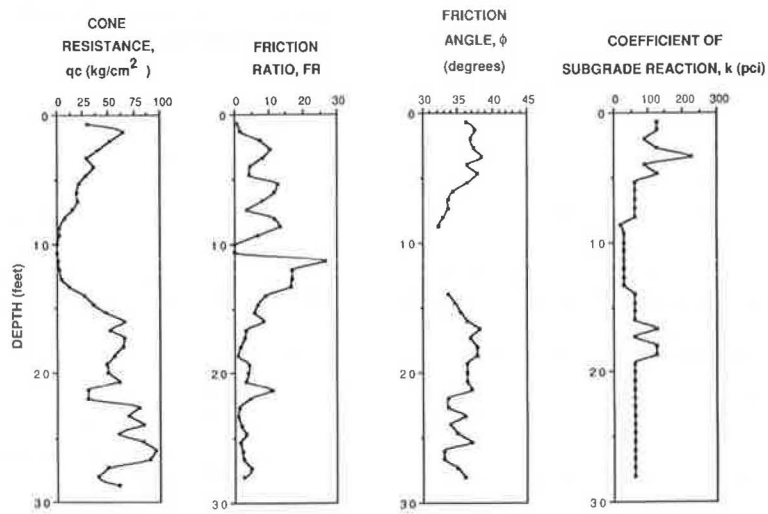

FIGURE 6 Average soil parameters and properties interpreted from CPT.

taken as 30 pci, as suggested by Reese (*15*). The values of the coefficient of subgrade reaction for the sand layers, as determined using Method A, and the estimated k values for the clay layer are presented in Figure 5.

In order to perform the load-deflection analysis for the situation encountered in this load test where the lateral load was applied 3 ft below the ground surface, the following analytical procedure was used:

1. The load-deflection response of the 22-ft portion below the point of load application was determined. To obtain p-y curves appropriate for the actual 25-ft pile, the effect of the upper 3 ft of soil was modeled using a proportionately higher unit weight in the first 8-in. soil layer below the point of load application. Because of the installation of sheet piling and the excavation of soil from within the test pit (Figures 3 and 7), it was considered appropriate to model the stress reduction due to excavation by reducing the full height of sand above the top of the model pile by 50 percent.

2. Because the analysis described did not include the lateral resistance provided by the upper 3 ft of fill, it was necessary to modify the predicted pile displacements. This modification was made by first performing an analysis using the entire 25-ft pile to determine the lateral resistance provided by the upper 3 ft of fill as a function of deflection. For example, the ultimate lateral resistance, (p_u) of each pile increment above the point of loading is shown in Figure 9. This curve was then integrated to determine the equivalent shear and moment at the point of loading. Next, an analysis was made applying this shear and moment to the top of the model 22-ft pile to determine the resulting deflection. An iterative procedure was employed until the assumed deflection of the pile at the point of load application, from which the soil resistance in the upper 3 ft was determined, was approximately equal to the subsequently calculated displacement. The significance of this correction is shown in Figure 8 in conjunction with the uncorrected prediction. A similar correction procedure was applied to each of the subsequent predictions.

As shown in Figure 10, the predicted deflected shape of the pile below the point of loading suggests that the pile is behaving as a relatively rigid member. At a pile top deflection of 3 in., a maximum moment of 175 k-ft was predicted to

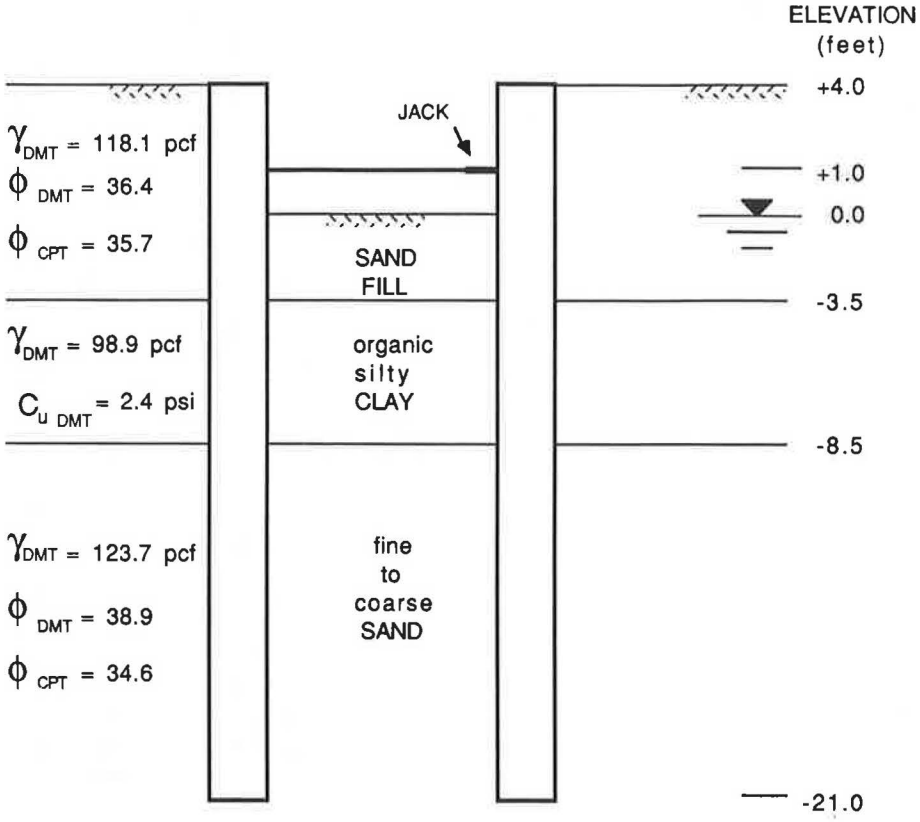

FIGURE 7 Load test configuration and soil profile.

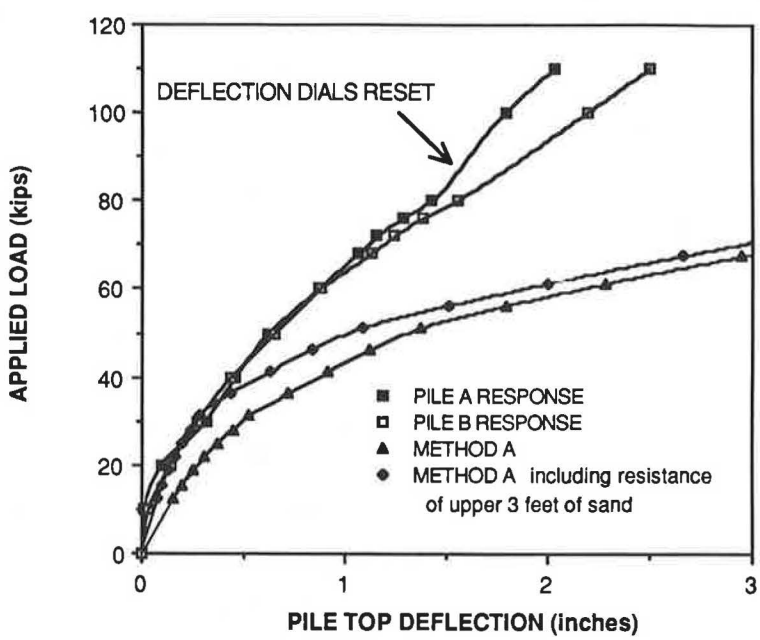

FIGURE 8 Comparison of measured pile response with predicted pile response using Method A.

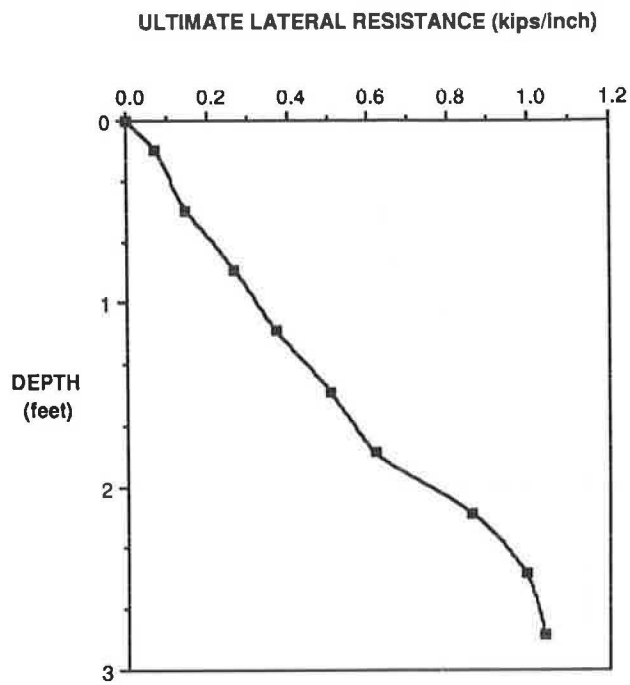

FIGURE 9 Ultimate lateral resistance in upper 3 ft.

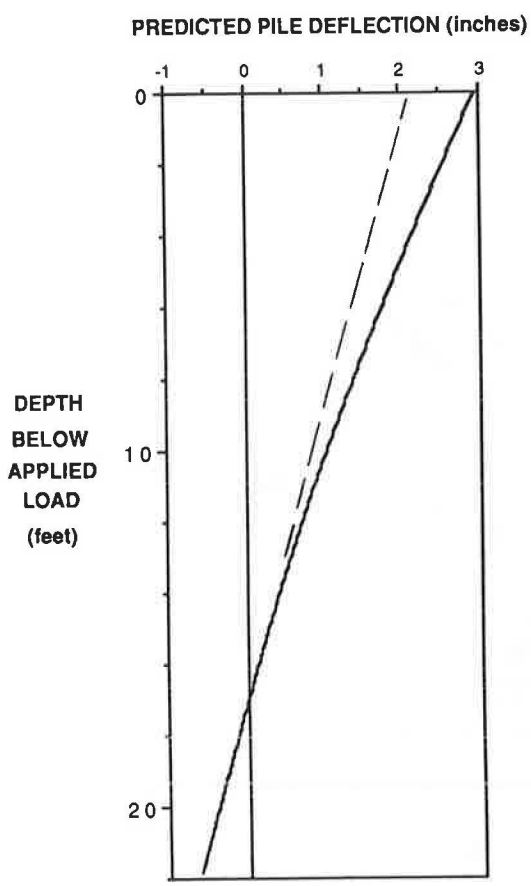

FIGURE 10 Pile deflection predicted using Method A.

occur in the pile. This is less than one-half of the calculated cracking moment of the pile.

Similarly, an analysis was made using Method B in conjunction with the hyperbolic *p-y* curve formulation, as shown in Figure 11. As previously reported by Schmertmann (*3*), the *k* values from this model tend to underestimate *k*. J. H. Schmertmann (personal communication, 1988) also suggested that these *k* values might logically be used in bi-linear *p-y* curves as they represent secant values. Figure 12 shows the improved predicted response obtained by using these *k* values in bi-linear *p-y* curves.

Figure 13 shows the predicted load-deflection response based on the cubic parabola model of Method C, in conjunction with the measured response. With respect to the measured response, the predicted pile response is significantly stiffer. Because this method was developed for driven piles, this difference may be due to a reduction in lateral stress adjacent to the piles caused by jetting the piles the first 12 ft during installation and the subsequent excavation of the soil within the test pit.

For each of the predictions presented, the shape of the *p-y* curves as a function of depth is the controlling factor, as the ultimate lateral resistance at large deflections is the same. In order to more clearly demonstrate the difference in these *p-y* curves, the following comparison is presented. The *p-y* curves generated by each of the above models for depths of 3 and 18 ft are shown in Figure 14. As evidenced in the preceding performance predictions, the softest *p-y* curves are generated using Method B in hyperbolic *p-y* curves. Modeling the *p-y* curves as bi-linear significantly reduces the deflection for a given load, particularly once the load is greater than one-half of the ultimate. The stiffer *p-y* curves generated using Method A in hyperbolic *p-y* curves results in a slightly improved performance prediction when viewed over the first few inches of deflection. Method C produces a *p-y* curve that is much stiffer than any of the other models. Because of the stiffness of the *p-y* curves generated by this model, the deflection at any given load was underpredicted. From this information it appears that the stiffness of the *p-y* curves should be between that generated by Method A and that generated by Method C.

For comparison purposes, a performance prediction was made using the CPT data. Predictions made using the CPT data will be referred to as generated by Method D. The relative density (D_r) of the sand at 8-in. increments was estimated from Schmertmann's (*12*) correlation of D_r with σ'_v and q_c. The effective unit weight of the sand was estimated using typical values of the maximum and minimum unit weights of uniform fine sands (*16*). Values of *k* as a function of depth were chosen according to relative density as suggested by Reese and Allen (*15*) and are shown in Figure 6. Figure 15 presents a comparison of the prediction made using Method D with the prediction made using Method A. A review of Figures 5 and 6 indicates that the DMT soundings produced *k* and ϕ values higher than those obtained from the CPT. However, the subgrade modulus profile for the CPT shown in Figure 16 is calculated by multiplying the Reese-determined value of *k* by the depth, while the DMT-developed *k* values are multiplied by the pile width. Although at shallow depths the DMT subgrade modulus values are somewhat greater, at increasing depths and in the sand underlying the clay layer

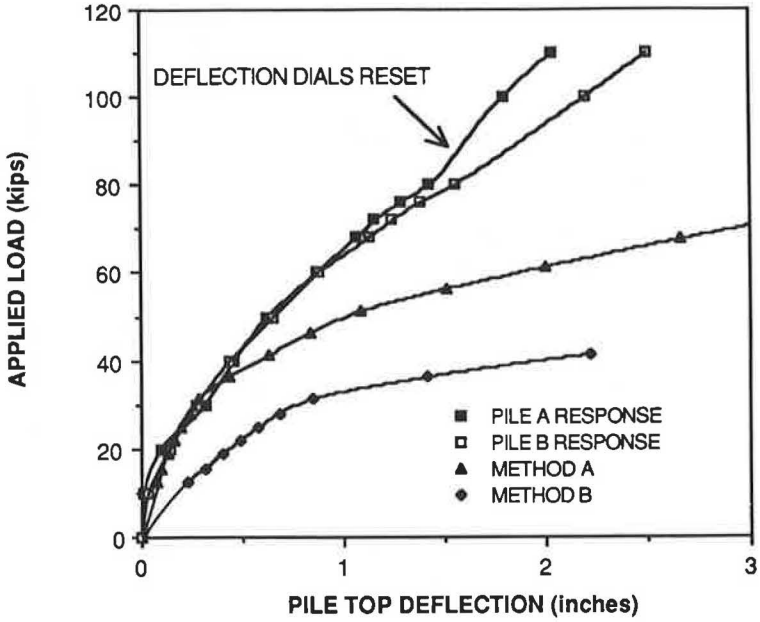

FIGURE 11 Pile response predicted using Method B.

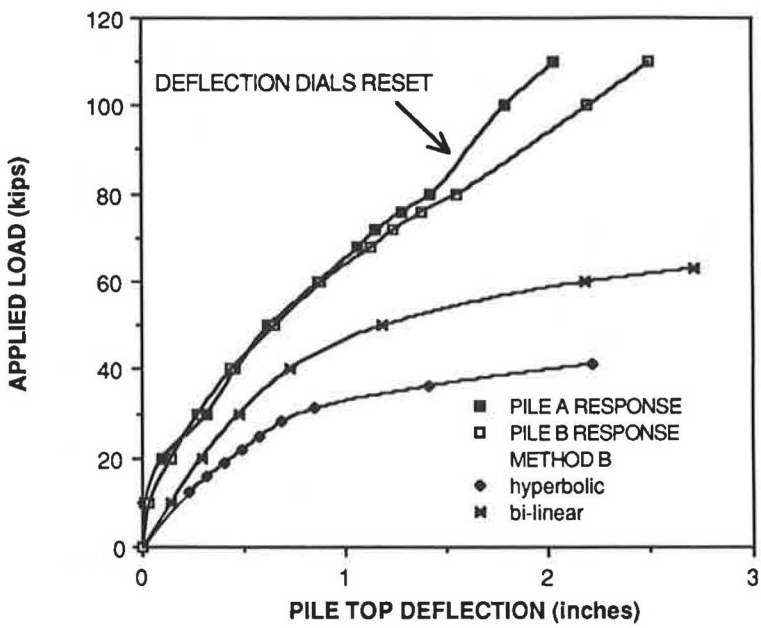

FIGURE 12 Pile response predicted using Method B in hyperbolic and bi-linear *p-y* curves.

the values interpreted from the CPT become almost twice as large as those from the DMT. The *p-y* curves at depths of 3 and 18 ft resulting from the CPT data are plotted in Figure 17 in conjunction with those previously shown in Figure 14. Because of the relatively rigid response of the pile, the effect of the stiffer *p-y* curves at depth from the CPT data very prominently influences the resultant load-deflection response.

In order to illustrate the insignificant influence of small variations in the angle of internal friction on the predicted load-deflection response in the first few inches of deflection, an analysis was performed utilizing the somewhat lower CPT ϕ values in conjunction with k values from Method A. A comparison of this prediction, the prediction using Method D, and the prediction using Method A is shown in Figure 15. As there is virtually no difference in the predicted response as a result of the different friction angle profile produced by the CPT and DMT, this figure illustrates the significance of the early portion of the *p-y* curve in predicting the lateral response of these piles up to a displacement of 10 percent of the pile width. The value of an instrumented test pile is obvious when one tries to evaluate the likely deflected shape and actual soil response as a function of depth.

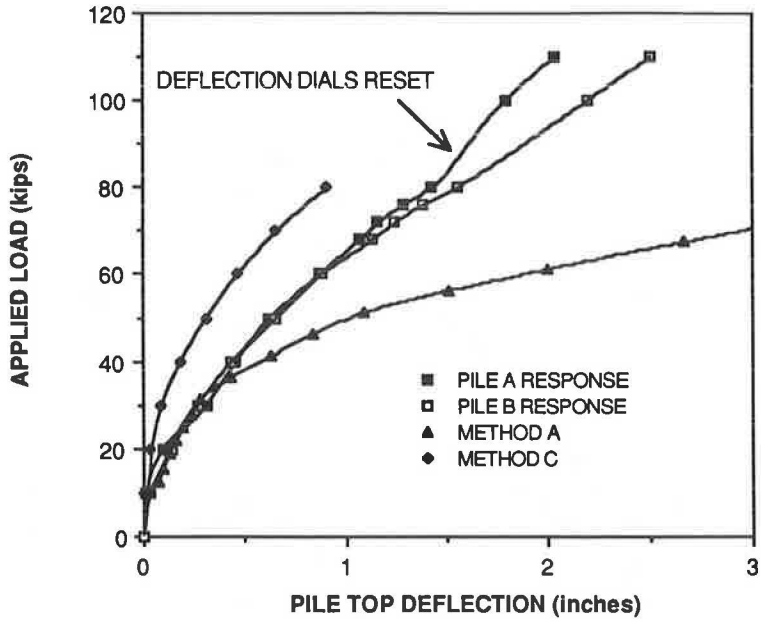

FIGURE 13 Pile response predicted using Method C.

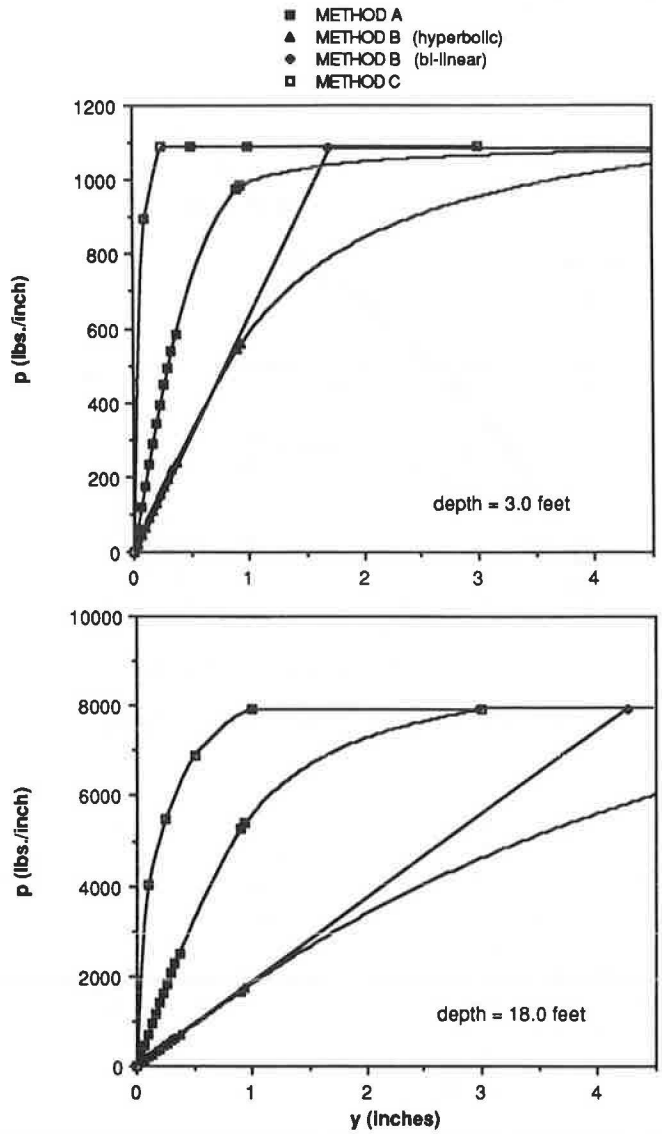

FIGURE 14 DMT-based *p-y* curves at depths of 3 and 18 ft.

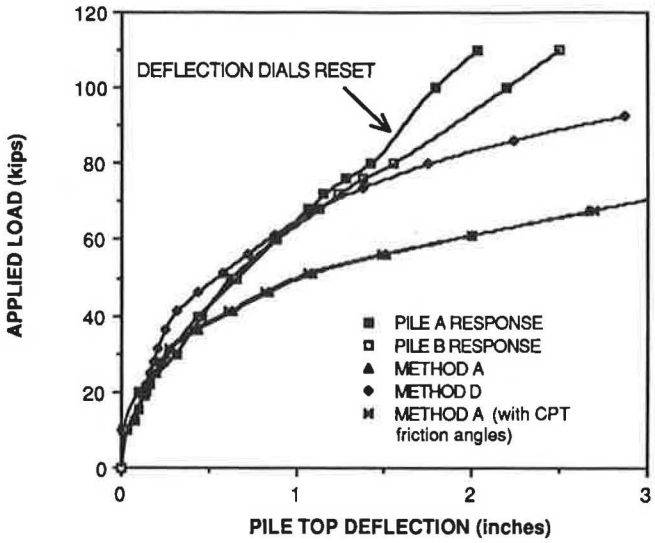

FIGURE 15 Pile response predicted using Method D.

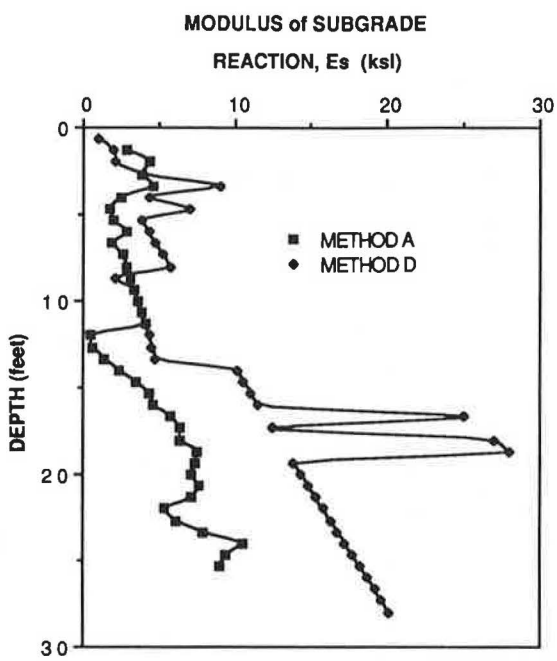

FIGURE 16 Modulus of subgrade reaction as determined by Methods A and D.

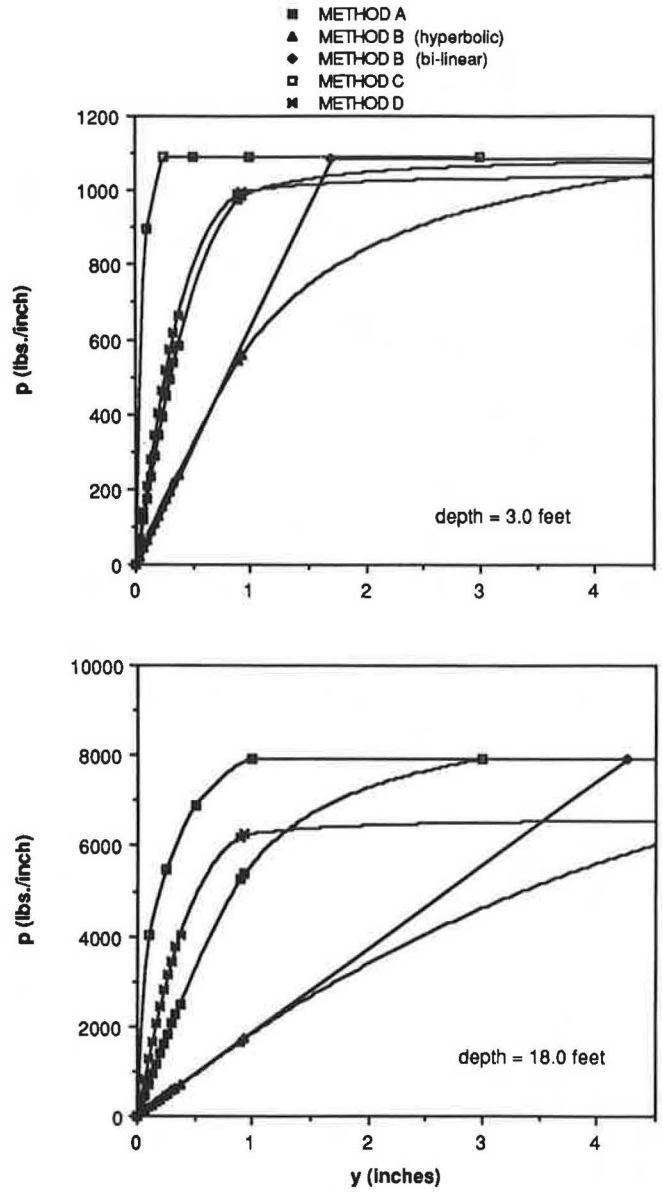

FIGURE 17 CPT-based *p-y* curves at depths of 3 and 18 ft.

Finally, Figure 18 presents a comparison of the measured and predicted load-deflection responses over the first 1 in. of deflection. During the first 0.5 in. of deflection, the measured response was most closely approximated by the hyperbolic *p-y* curves using the DMT secant model without consideration for size effects. The actual response was bounded by the stiffer response predicted using Method C and the softer response produced using Method B in conjunction with bi-linear *p-y* curves. It is not the intention of this comparison to suggest that in all cases the best agreement should be expected to be obtained by the models showing the best performance in this study. However, in cohesionless profiles, the stiffness of the predicted load-deflection responses of the three DMT-based models, with respect to each other, is expected to remain the same.

SUMMARY AND CONCLUSIONS

The results of performance predictions made using three proposed models for developing *p-y* curves from DMT data are presented in conjunction with the measured response from lateral load tests on two 24-in.-square prestressed concrete piles that were installed by jetting the first 12 ft followed by driving to a depth of 25 ft. Over the first 1-in. deflection, a method for developing *p-y* curves using a simple secant approximation based on the pressure increase needed to dis-

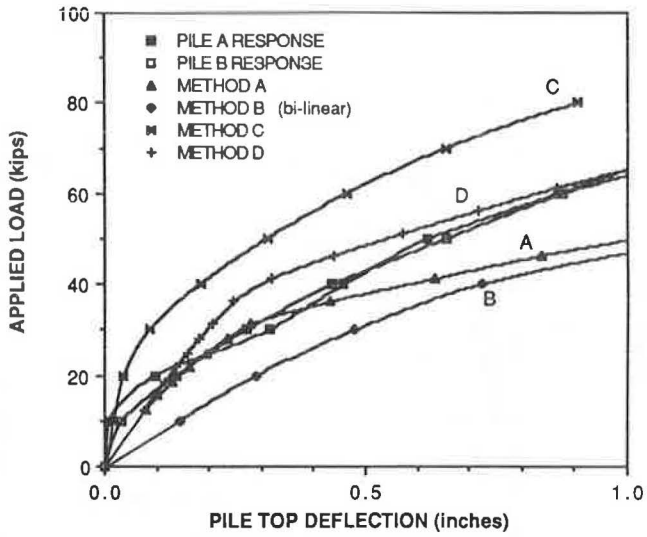

FIGURE 18 Predicted pile response over 1-in. deflection.

place the soil a distance equal to one-half of the DMT blade thickness produced a predicted response closest to that measured. The use of size effect corrections produced a significantly softer response than that measured, whereas the model of Robertson et al. produced a significantly stiffer response. It should be noted that this method was developed for driven piles and that the stiffer predicted response may in part be due to a lateral stress reduction due to the jetting. Although none of the models investigated allowed for the explicit consideration of the installation procedure, it is quite reasonable that the observed response was bracketed by procedures that previously had been applied to driven piles and drilled piers, respectively.

Although DMT soundings produced friction angles, based on the Durgunoglu and Mitchell bearing capacity theory, that were somewhat higher than those obtained from the CPT, this difference was shown to be of relatively minor importance in the first several inches of pile displacement in comparison to that of the inferred k values used to generate the p-y curves.

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