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by

Barry J. Meyer
Lymon C. Reese

Research Report 244-1

Development of Procedures for the Design of Drilled Foundations
For Support of Overhead Signs

Research Study 3-5-78-244

conducted for

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THE UNIVERSITY OF TEXAS AT AUSTIN

December 1979

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report presents comparisons between results from analytical procedures and results from experiments for a number of cases where deep foundations were subjected to lateral loading. Evaluations are presented of the current procedures that are available for such analyses.

The authors wish to thank the State Department of Highways and Public Transportation for their sponsorship of the work and to express appreciation for the assistance given by many members of their staff. Appreciation is also expressed to Dr. Stephen J. Wright, who made many helpful suggestions during the preparation of the manuscript, and to Mrs. Cathy Collins and Mrs. Kay Lee, who both assisted in the preparation and typing of this manuscript.

Barry J. Meyer
Lymon C. Reese

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ABSTRACT

The behavior of laterally loaded piles was investigated using the finite difference computer program COM623. A thorough search of the literature was undertaken to find the results of lateral load tests performed in clay and sand. The results of these analyses indicate that most of the p-y criteria, where p is the lateral resistance against the pile in force per unit of length and y is pile deflection, are satisfactory in predicting pile behavior. A modification of the p-y criteria of Reese and Welch (1975) was suggested, based on the results of some of the analyses presented in this report.

KEY WORDS: piles, lateral loading, p-y criteria, clay, sand.

SUMMARY

This study is concerned with evaluating the presently available p-y criteria, where p is the lateral resistance against the pile in force per unit of length and y is pile deflection, for analyzing the behavior of piles under lateral loading. The results of a number of tests on piles in clay were analyzed, and the results of tests on piles in sand were analyzed.

From this study it was found that:

(1) The Matlock (1970), Reese, et al. (1974), Reese, et al. (1975), and Sullivan (1977) p-y criteria were all satisfactory in their present form. Based on the results presented in this report, no modifications could be suggested.

(2) The Reese and Welch (1974) p-y criteria for dry, stiff clays were modified based on the results of this report. The currently used exponent in their parabolic equation was too small, which leads to unconservative deflections at small loads, and conservative deflections at large loads. An exponent of 0.4 was recommended.

(3) Single drilled shafts can withstand very large lateral loads. The results of tests reported by Bhushan et al. (1978) indicated that large diameter drilled shafts in hard clay can withstand lateral loads as large as 400 kips.

IMPLEMENTATION STATEMENT

The information presented in this report is recommended for consideration by the Design Office of the State Department of Highways and Public Transportation. The comparison between results of analytical procedures and experimental studies should prove useful to engineers in their design of drilled shafts and other deep foundations subjected to lateral loading. The information should be particularly helpful in the design of foundations for bridge structures for overhead signs. The material that is presented should be of considerable use in computing groundline deflection, maximum bending moment, and required depth of penetration for single drilled shafts supporting overhead signs.

A final report on this project will be submitted in which the problem of the design of foundations for sign structures is discussed in detail.

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NOTATIONS

α	angle defining failure wedge for sand near ground surface
β	angle defining failure wedge for sand near ground surface
γ	unit weight
ε_{50}	strain at 50% of the maximum stress difference, determined from a UU triaxial compression test
$\bar{\sigma}_x$	effective overburden stress
ϕ	angle of internal friction (degrees)
A	empirical adjustment used in p-y criteria for sand
A_c	empirical admustment factor for cyclic loading
A_s	empirical adjustment factor for static loading
A_0	normalized ultimate soil resistance at the ground surface
B	empirical adjustment used in p-y criteria for sand
b	pile width or diameter
C	factor in expression for deflection due to cyclic loading
c_x	undrained shear strength at the depth x
D_r	relative density
E	Young's modulus
E_c	Young's modulus for concrete
E_s	modulus of soil response
E_{sc}	slope of portion of p-y curve for stiff clay
E_{si}	initial slope of p-y curve
E_{ss}	slope of portion of p-y curve for stiff clay
f_c'	compressive strength of concrete
h	increment length
I	moment of inertia
I_g	moment of inertia for gross concrete section
I_s	moment of inertia of reinforcement
J	empirical coefficient in ultimate soil resistance expression

k	coefficient giving increase in initial soil modulus with depth
K_a	coefficient of active earth pressure
k_c	a constant in the expression for soil modulus for cyclic loading of stiff clay
K_o	coefficient of earth pressure at rest
K_p	passive earth pressure coefficient
k_s	a constant in the expression for initial soil modulus for static loading of stiff clay
L	length of pile
m	coefficient used in obtaining p-y curve for sand
N	number of cycles -or- blow count from penetration test
n	coefficient used in obtaining p-y curve for sand
N_p	normalized ultimate soil resistance
$\bar{p}$	effective overburden pressure
p_c	ultimate soil resistance from theory
p_{c1}	ultimate soil resistance near the ground surface
p_{c2}	ultimate soil resistance well below the ground surface
p_{cd}	ultimate soil resistance at depth for p-y curves for sand
p_{ct}	ultimate soil resistance near ground surface for p-y curves for sand
p_m	specific soil resistance on p-y curve for sand
p_R	residual shear resistance (term in unified criteria)
p_u	ultimate soil resistance
$(pu)_c$	experimental ultimate soil resistance for cyclic loading
$(pu)_s$	experimental ultimate soil resistance for static loading
R	=EI=flexural stiffness -or- ratio of soil resistance to ultimate soil resistance
x	depth below the ground surface -or- depth below the pile top
x_r	depth of transition obtained by equating expressions for soil resistance
y	lateral deflection
y_c	deflection due to cyclic loading

y_k specific deflection on p-y curve for sand

y_m specific deflection on p-y curve for sand

y_p specific deflection on p-y curve for stiff clay

y_s deflection due to static loading

y_u specific soil resistance on p-y curve for sand

y_{50} specific deflection on p-y curve for clay ($y_{50} = 2.5\ \varepsilon_{50}\ b$)

CHAPTER 1. INTRODUCTION

Many different methods of analysis have been proposed to solve the problem of a laterally loaded pile, where the problem can be generally defined as computing pile deflection and bending moment as a function of depth below the ground surface. Methods which are based on the theory of elasticity are not generally applicable for design due to the inadmissibility of assigning single values to the required soil parameters. Some methods are based on the theory of subgrade reaction and on simplifying assumptions, such as assuming a variation of the subgrade modulus with depth and that the soil is linearly elastic (Winkler, 1887; Hetenyi, 1946; Terzaghi, 1955; Broms, 1964[a]; Broms, 1964[b]). These simplifying assumptions reduce the difficulty in obtaining a solution to the problem, but errors of an unknown magnitude are introduced into the solution. A more rational approach will be discussed in detail in this report.

The research is important because the design of pile foundations, particularly for the foundations for overhead signs, is a critical method with regard to safety and economy. Work described herein should allow pile foundations to be designed with an adequate factor of safety and a minimum of cost.

METHODS BASED ON THEORY OF ELASTICITY

An elastic solution for the problem of a single pile subjected to lateral loading was presented by Poulos (1971). Poulos assumed the soil to be an elastic, homogeneous, isotropic half-space with a constant Young's modulus and Poisson's ratio. The pile was modeled as a thin, rectangular, vertical strip, with soil pressures constant across the pile width.

Poulos used a method based on Mindlin's equation for modeling the soil behavior in lieu of the generally accepted theory of subgrade reaction employing the Winkler assumption. He stated that the Winkler model of using a series of discrete springs to idealize the soil behavior is incorrect. He compared solutions using his model and the Winkler model and found that deflections computed using the Winkler model were greater than deflections computed using his model based on the theory of elasticity. Vesić (1961) compared solutions from use of the Winkler assumption and theory of elasticity and showed that there is a small difference between solutions using the two methods for the case of an elastic material. The important point is that the theory of subgrade reaction employing the Winkler assumption can be extended to the general case of a nonlinear soil with a variable subgrade modulus, but Poulos' method can only be used for materials which are linearly elastic.

While solutions with the theory of elasticity may be more correct for the case of a linearly elastic soil, most soils behave nonlinearly. Due to the nonlinear behavior of soils, it is difficult to select single values of Young's modulus and Poisson's ratio. Furthermore, these properties cannot easily be obtained directly but must be estimated or obtained from indirect relationships with other soil properties. Poulos' method is sensitive to variations in Young's modulus; thus, his method leads to uncertain results in making design computations.

METHOD OF BROMS

Broms (1964[a]) presented a method for calculating the deflections and moments of piles in a cohesive soil under undrained loading using the theory of subgrade reaction. The procedure was presented in the form of design charts and tables. Using his procedure, a single pile which was either free-headed or perfectly fixed against rotation could be analyzed.

Broms limited his method for calculating deflections to the "working" load range, which is normally considered to be 1/2 of the computed ultimate pile capacity. In the working load range, Broms assumed that the soil was linearly elastic. Even though cohesive soil is not linearly elastic in the working load range, Broms' assumption probably leads to only minor errors. However, Brom's method for cohesive soil is limited because in many instances it is desirable to obtain the response of a pile for a full range of loads. Also, to simplify the analysis, Broms assumed that for cohesive soil the subgrade modulus was constant with depth.

Broms used his method to analyze the results of load tests for piles in clay. The method yielded values for the ratio of measured deflections to computed deflections ranging from 0.33 to 3.75. An important point is that a value of 0.33 means that Broms' method underestimated the actual deflection by a factor of 3.0. These results are instructive in showing that a simplified method cannot be used to analyze such a complex problem as a laterally loaded pile. The method was useful in the period when it was conceived, but the present state-of-the-art is such that the simplifying assumptions of a linearly elastic soil and a constant subgrade modulus do not have to be made.

Broms used the concept of a plastic hinge to compute the collapse load or ultimate lateral load which can be sustained by a long flexible pile. To compute the collapse load, Broms assumed that the ultimate soil resistance, p_u, would be fully mobilized to the depth of the plastic hinge, and that it would have a distribution as shown in Fig. 1.1. Broms obtained ratios of measured maximum moment to calculated maximum moment of 0.84 to 1.13. The method worked well for the small number of cases he analyzed, but more work is needed to prove the validity of Broms' approach.

Broms (1964^b) also presented a method for computing the pile-head deflection at working loads and the lateral load which would induce the formation of a plastic hinge in a flexible pile embedded in a cohesionless material. In his analysis for cohesionless soils, Broms assumed that the horizontal subgrade modulus increased linearly with depth and that the soil was linearly elastic in the working load range. Broms presents values for the coefficient of subgrade reaction, n_h, which are a function of the pile diameter and the relative density of the soil. These reported values of n_h, which are used to calculate the horizontal subgrade modulus, are the same as the values presented by Terzaghi (1955).

The equations that Broms used to compute the lateral ground line deflection were based on work done by Reese and Matlock (1956). Reese and Matlock presented nondimensional curves which can be used to obtain deflections, moments, and shears at any point along the length of a laterally loaded pile. The curves presented by Reese and Matlock can be used to solve the differential equation for a laterally loaded pile if the soil modulus, E_s, increases linearly with depth. The authors point out that the assumption

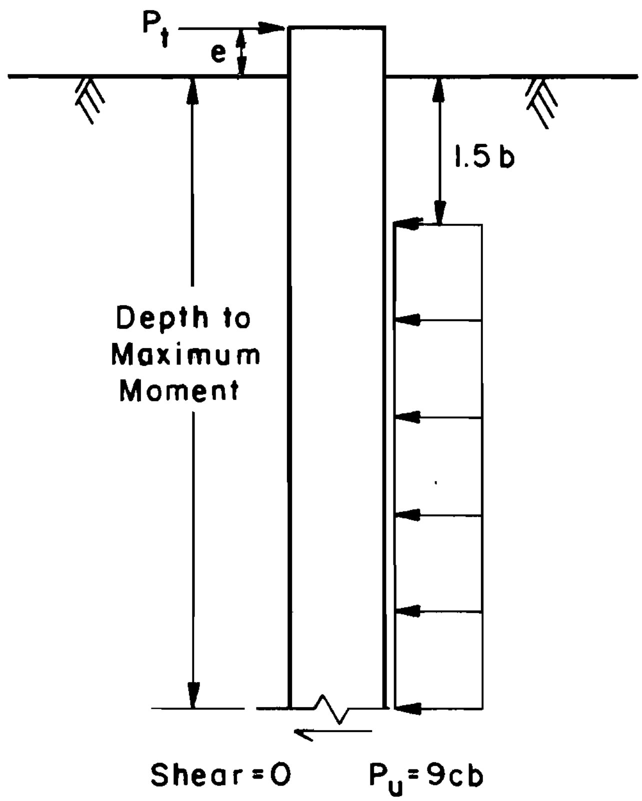


Fig. 1.1. Distribution of ultimate soil resistance for cohesive soils suggested by Broms.

of a linearly varying soil modulus is useful in practice, but that the value of E_s will decrease substantially as the lateral load is increased. No recommendations on selecting the value of k were made in their paper.

Broms used the following equation for the distribution of the ultimate soil resistance with depth in order to compute the collapse load for a pile in cohesionless soil:

$$p_u = 3b\gamma x K_p \tag{1.1}$$

where

p_u = ultimate soil resistance,

b = pile width,

γ = unit height,

x = depth below the ground surface,

K_p = passive earth pressure coefficient,

K_p = $\tan^2(45 + \phi/2)$

He also used this distribution of soil resistance to calculate the maximum bending moments in a laterally loaded pile. His comparisons of measured ultimate collapse loads to computed ultimate collapse loads yielded ratios ranging from 0.63 to 3.09, and his comparison of measured maximum bending moments to computed maximum bending moments yielded ratios ranging from 0.62 to 1.85. The majority of the reported comparisons for both the ultimate collapse load and the maximum bending moment were greater than 1.0. Broms' method of solution is easy to use, and can produce a preliminary estimate of the ultimate collapse load or of the maximum bending moment for a pile in cohesionless soil. If a better estimate of the pile behavior is required, a computer program in conjunction with nonlinear soil resistance-deflection curves should be used.

The limitations suggested earlier to Broms' method for cohesive soil also apply to his method for cohesionless soils. Therefore, the more general method shown in the following section is suggested for most design problems. However, the methods proposed by Broms and others can be useful to the experienced designer in giving an approximate design with a minimum of computation.

GENERALIZED SUBGRADE REACTION

Two problems must be solved to obtain the response of a given pile that is subjected to a lateral load: the soil resistance must be known as a function of depth, pile deflection, pile geometry, and nature of loading; and the equations must be solved that yield pile deflection, bending moment, and shear. These two problems will be discussed separately.

It is stated in the theory of subgrade reaction that the soil around a laterally loaded pile can be replaced by a series of discrete springs as shown in Fig. 1.2. This concept does not imply that the soil is linearly elastic or that a specfic variation of the modulus of subgrade reaction with depth must be used.

Before 1956, analyses were performed assuming that the soil was linearly elastic and that the soil modulus varied in some predetermined manner with depth. These assumptions were necessary so that the solutions could be obtained with the slow-speed calculators available at that time. McClelland and Focht (1956) introduced the concept of the soil resistance deflection curve, "p-y" curve, which can be used to obtain values of the soil modulus with depth. These curves are generally nonlinear and can vary in an arbitrary manner with depth; thus, the soil modulus can vary in an arbitrary manner with depth and with pile deflection. The digital computer allowed for solutions for an arbitrarily varying soil modulus, as will be shown.

The concept of a p-y curve can be defined graphically by considering a thin slice of a pile and surrounding soil, as shown in Fig. 1.3a. The earth pressures which act against the pile prior to loading are assumed to be uniform, Fig. 1.3b. For this condition, the resultant force, obtained by integrating the pressures, is zero. If the pile is given a lateral deflection, y, as shown in Fig. 1.3c, a net soil reaction will be

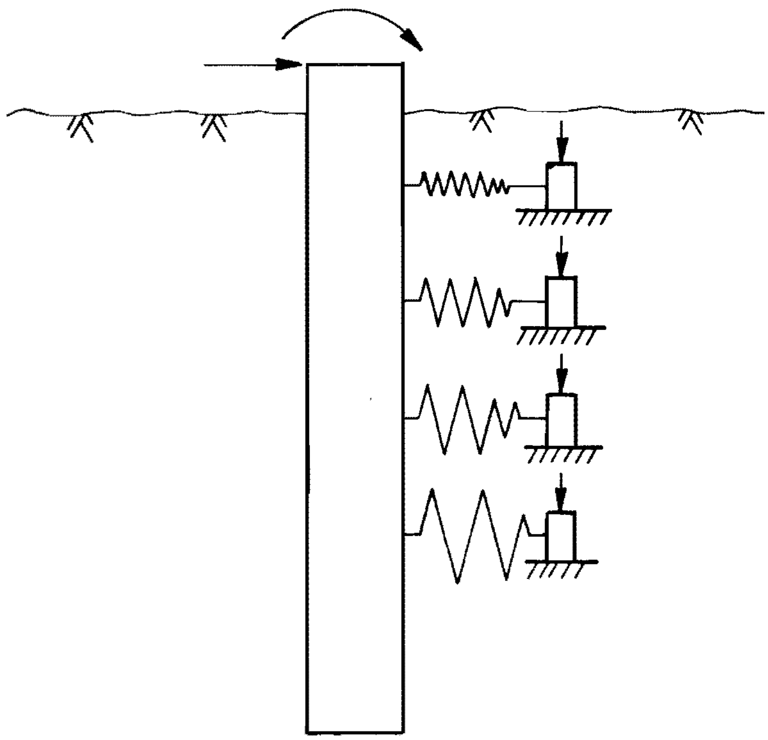


Fig. 1.2. Idealization of soil surrounding a pile.

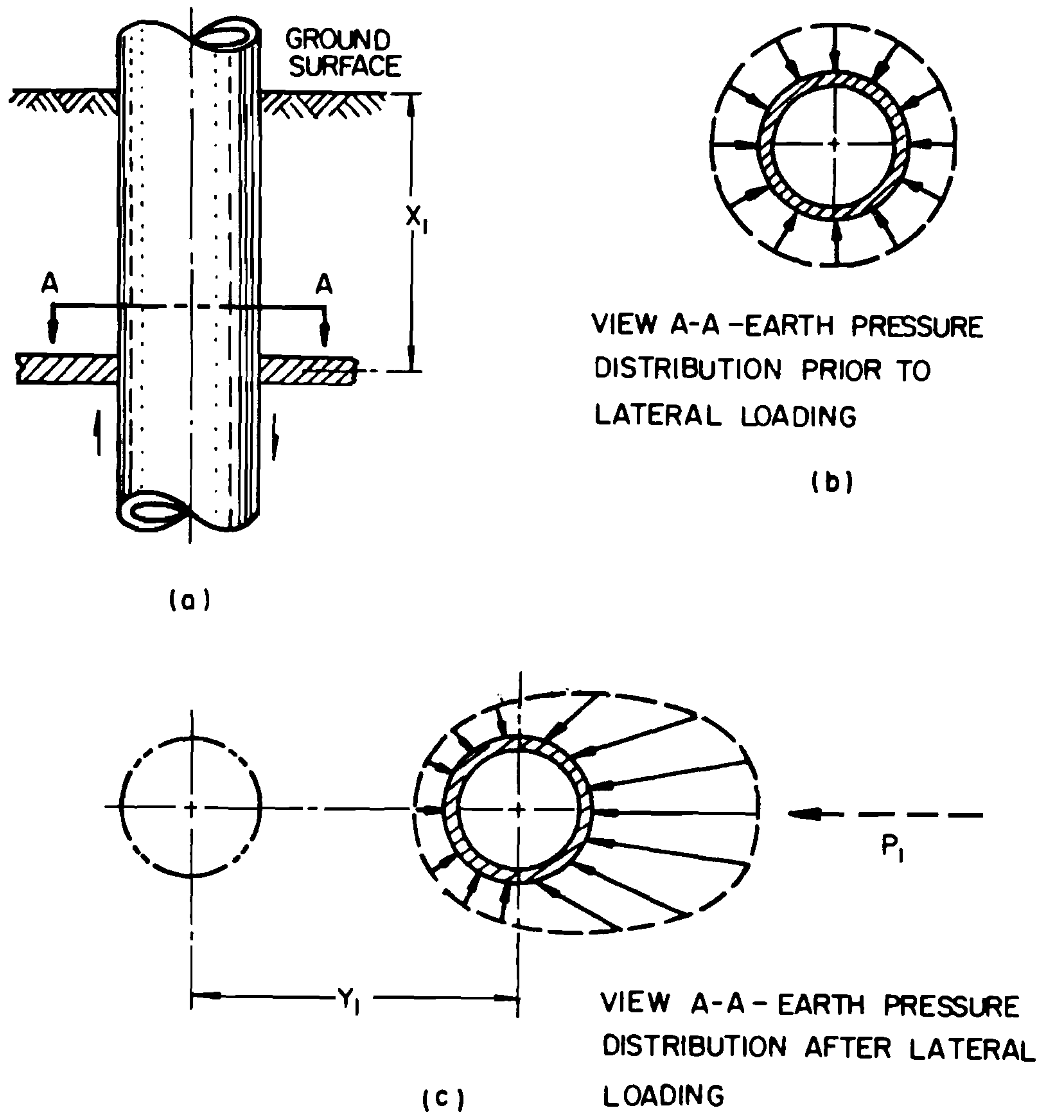


Fig. 1.3. Graphical definition of p and y (Reese and Welch, 1975).

obtained upon integrating the pressures. This process can be repeated in concept for a series of deflections resulting in a series of forces per unit length of pile which may be combined to form a p-y curve. In a similar manner, p-y curves may be generated for a number of depths. A possible family of p-y curves is shown in Fig. 1.4.

Generally, p-y curves are nonlinear, in which case the modulus of soil response, E_s, can be taken as the secant modulus to a point on the p-y curve as shown in Fig. 1.5. The negative sign in the expression shown in the figure indicates that the direction which the pile deflects is opposite to the direction of the soil reaction. The modulus of soil response, or simply, the soil modulus, has the units of force per length squared, which is the force per unit length of the pile per unit of movement of the pile into the soil. The soil modulus should not be confused with Young's modulus, which has the same units but a different meaning.

To obtain a complete solution of deflections, moments, and shears for a pile under lateral loading, an analytical method for solving the following equation must be employed.

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} + E_s y = 0 \tag{1.2}$$

where

E = Young's modulus,

I = moment of inertia,

P_x = axial load,

y = lateral deflection,

x = depth below the pile top.

The soil modulus will vary with deflection and depth, as shown in Fig. 1.4; therefore, iterative techniques must be employed to obtain a correct solution. The following section presents a numerical technique for solving Eq. 1.2.

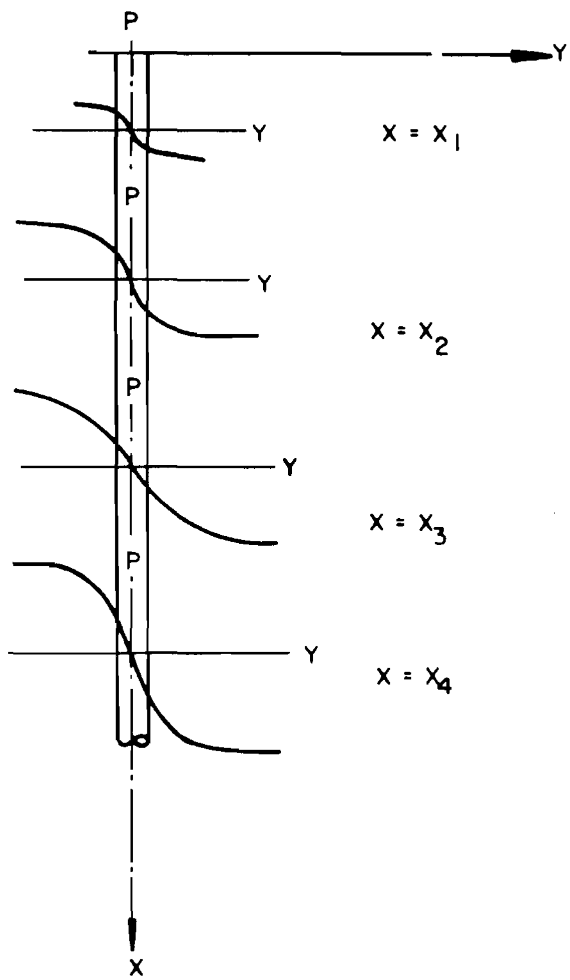


Fig. 1.4. Possible family of p-y curves (Reese and Cox, 1968).

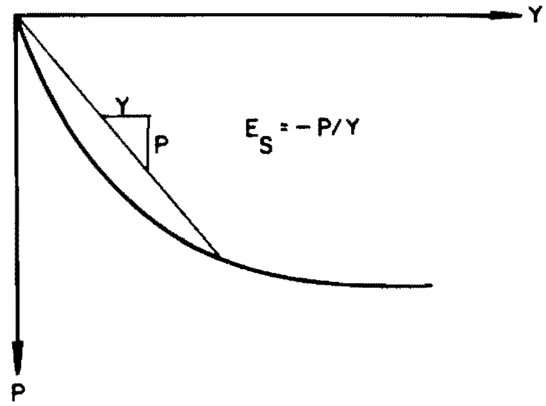


Fig. 1.5. Illustration of secant modulus.

DIFFERENCE EQUATION SOLUTION

The soil modulus which is used in the governing differential equation, Eq. 1.2, usually varies in some complex manner with depth. The variability of E_s with both depth and pile deflection makes it impratical to solve the laterally loaded pile problem using either a closed form solution or a power series solution.

The finite difference method of analysis is very useful in solving the problem of a laterally loaded pile. A solution can be obtained using difference equations when the soil modulus varies with both depth and lateral deflection. The effects of applied axial load and variations in the pile stiffness with depth can also be taken into consideration (Parker and Cox, 1969).

A finite difference model is developed by dividing a pile of finite length into a number of elements of length, h, as shown in Fig. 1.6. The finite difference equations can be written at a node point m on the pile. Writing the differential equation, Eq. 1.2, about point m, the following equation results.

$$y_{m-2}(R_{m-1}) + y_{m-1}\,(-2R_{m-1}\ -2R_m + P_x h^2) + y_m(R_{m-1} + 4R_m + R_{m+1}$$
$$-2P_x h^2 + E_{s_m}\ h^4) + y_{m+1}(-2R_m\ -2R_{m+1} + P_x h^2) + y_{m+2}$$
$$(R_{m+1}) \;=\; 0 \qquad (1.3)$$

where

R = EI (the stiffness of the pile),
h = increment length.

Because two nodal points are needed on either side of the node about which Eq. 1.3 is being written, four imaginary nodes, two at the top and two at the bottom of the pile, are required. These pairs of nodes are shown in

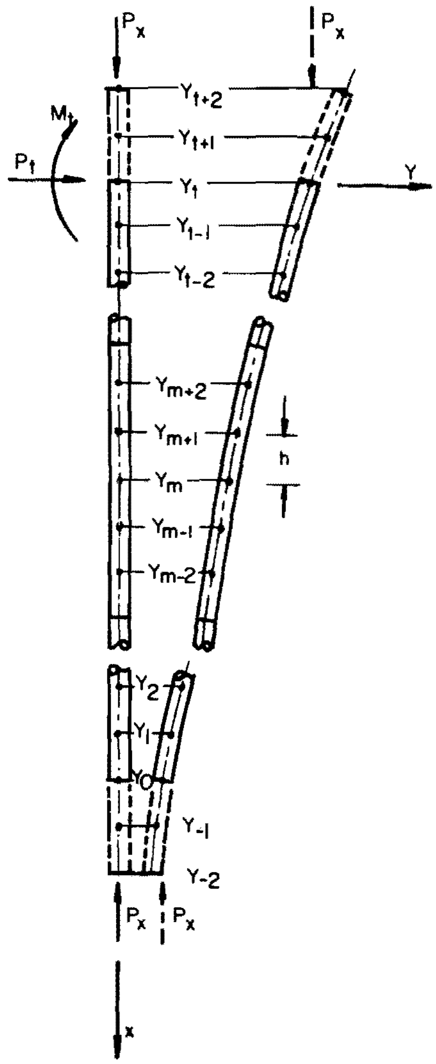


Fig. 1.6. Pile divided into increments.

Figs. 1.7 and 1.8, respectively. Equation 1.3 can be written about every node point on the pile in Fig. 1.6 to obtain n + 1 equations and n + 5 unknowns, where n is the number of pile increments.

Because there are n + 5 unknowns, four boundary conditions are needed to complete the solution. The boundary conditions at the pile top may be of three forms: (1) lateral load P_t and moment M_t; (2) lateral load P_t and slope S_t; (3) lateral load P_t and rotational restraint constant M_t/S_t. The other two boundary conditions are that the shear and moment are zero at the bottom of the pile.

Setting up the problem using the finite difference approach results in a number of simultaneous equations which have to be solved. Gleser (1954) developed a convenient method to solve the system of simultaneous equations by hand, by establishing a systematic procedure for the cases of a free-head and fixed-head pile. Basically, the procedure consists of successively eliminating unknowns beginning with the equations at the bottom of the pile and progressively working upward. At the top of the pile the boundary conditions are used to solve for the deflections y_t, y_{t+1} and y_{t+2}. These deflections can then be used to work back down the pile and solve for the deflections, slopes, moments, shears, and soil reactions for all points along the pile.

Gleser was partially restricted in his method by the number of equations which had to be solved. A more convenient and efficient method employs the use of a computer program. The computer program should be capable of handling the different boundary conditions and should employ an iterative method on account of the nonlinear soil behavior.

A computer program COM622 (Reese, 1977) has been developed to solve the problem of a laterally loaded pile using the Gleser method. The nonlinear soil behavior is accounted for by repeated elastic-theory computations using a secant modulus, E_s, which is obtained from the input p-y curves. The p-y curves are input at different depths and the modulus values are obtained by interpolating between curves. The steps in this iterative procedure are listed below (Reese and Cox, 1968).

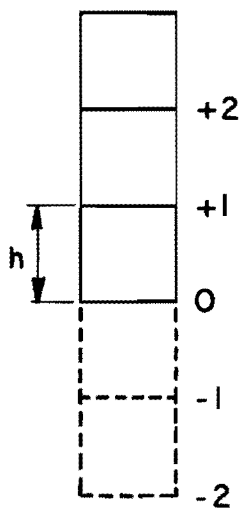


Fig. 1.7. Imaginary nodes at bottom of pile.

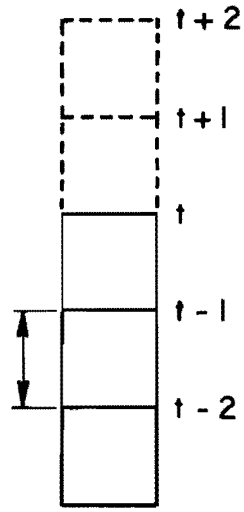


Fig. 1.8. Imaginary nodes at top of pile.

(1) A set of E_s values is assumed for the entire length of the pile.

(2) The set of difference equations are solved to obtain the deflections at each point along the pile.

(3) From the p-y curves and the values of y found in Step 2, a value of p is found at each point.

(4) A new set of E_s values are computed using the p and y values found in steps 2 and 3.

(5) The procedure is continued until convergence is achieved.

The program is written in a general form so that step-changes in the pile stiffness can be input, one of three different boundary conditions at the pile top can be selected, and p-y curves of any arbitrary shape can be input at depths along the pile.

Another computer program, COM623 (Sullivan, 1977), has been developed which employs the same method of solution as COM622 except that p-y curves are generated by the program and do not have to be input. These p-y curves are developed based on five sets of criteria. They are:

(1) submerged soft clay, Matlock (1970),

(2) dry stiff clay, Reese et al. (1975),

(3) submerged stiff clay, Reese et al. (1975),

(4) unified clay criteria, Sullivan (1977), and

(5) sand, Reese et al. (1974).

These criteria will be discussed in detail in Chapter 2. Soil resistance-deflection curves can also be input into the computer program COM623, if the user so desires. It should be pointed out that for a given set of p-y curves, solutions obtained using COM622 and COM623 are identical.

One of the problems with using a computer program such as COM622 or COM623 is that care must be taken to insure that the correct solution is being generated by the computer. The accuracy of the finite difference approximation depends on the values of h which are used to model the pile and the magnitude of the closure tolerance. Generally, for a long, flexible pile a satisfactory increment length is approximately equal to half of the pile diameter, and a satisfactory closure tolerance is 1×10^{-4} in. The closure is achieved by insuring that the difference in deflections between successive iterations is less than the closure tolerance. If p-y curves are being input into the computer program, the curves should be input at close spacing near the ground surface. The exact value of h, the closure tolerance, and the spacing of the p-y curves may vary without having a large affect on the computed solution.

To insure that an unaccounted for error has not occurred, an independent method of solution should be employed (Reese and Allen, 1977). A nondimensional procedure developed by Reese and Matlock (1958), making use of a family of p-y curves, can yield approximate results which may be used as a check on the more accurate computer solution. The procedure for performing the nondimensional analysis is outlined by Reese and Allen (1977).

CHAPTER 2. CRITERIA FOR FORMULATING p-y CURVES

INTRODUCTION

To obtain a complete solution to the laterally loaded pile problem, criteria for formulating p-y curves for a particular soil profile must be obtained. The criteria must be general enough so that basic soil-strength parameters can be used to establish the family of p-y curves. Criteria are usually separated into the two basic categories, those for cohesive soils and those for cohesionless soils. For cohesive soils, the $\phi = 0$ concept is usually employed and deformation of the soil-pile system is assumed to occur under undrained conditions. For cohesionless soils, effective strength parameters are used and it is assumed that the soil-pile system deforms under drained conditions.

The first p-y criteria were established by McClelland and Focht (1956). They analyzed the results of tests of a full-sized, instrumented, pipe pile which was tested in the Gulf of Mexico (Parrack, 1952). McClelland and Focht attempted to establish a direct relationship between the experimental p-y curves and the stress-strain properties of the clay. The data available to McClelland and Focht did not allow the development of a complete family of experimental p-y curves , and the criteria they proposed have been superceded. However, their work is important because it pointed the way to later field experiments with instrumented piles.

Other criteria which have since been introduced to establish p-y curves in cohesive soils are: Gill and Demars, 1970; Matlock, 1970; Reese and Welch, 1975; Reese et al., 1975; and Sullivan, 1977. Gill and Demars established a completely empirical procedure, based on tests of segmented piles to establish the variation of the ultimate soil resistance with depth. Their procedure does not consider the effects of cyclic loading. The p-y criteria presented by Matlock, 1970; Reese and Welch, 1975; Reese et al, 1975; and Sullivan, 1977; all consider the effects of both short-term static and cyclic loading. These four sets of criteria will be discussed in detail in this chapter and will be used to predict the behavior of laterally loaded piles, as described in Chapter 4.

Methods which have been suggested to obtain p-y curves for piles in cohesionless soils are Kubo, 1967; Gill and Demars, 1970; Parker and Reese, 1970; and Reese et al., 1974. Kubo based his work on model tests and presented a parabolic equation expressing the variation of the soil resistance with deflection, but he did not present methods for obtaining an ultimate soil resistance as a function of depth. Parker and Reese (1971) also performed tests on model piles to determine a method for predicting the behavior of fullscale piles. Their criteria do not treat in sufficient detail the effect of changes in pile width on the deflection at which the ultimate soil resistance is mobilized. Presently, the most widely accepted method for analyzing the behavior of piles in sand is by Reese et al. (1974). The Reese method presents the only published criteria for piles in a cohesionless soil which takes the effect of cyclic loading into consideration, and it will be employed to predict the response of piles which have been tested in a predominately cohesionless media. Chapter 5 presents comparisons between results from experiments and results from analyses using the Reese et al. (1974) p-y criteria.

METHODS FOR PREDICTING p-y CURVES IN COHESIVE SOIL

Soft Submerged Clay

Matlock (1970) developed a procedure for predicting p-y curves in a soft, submerged clay deposit. His procedure was developed from the results of tests on fully instrumented, flexible, pipe piles subjected to short-term static loading and to cyclic loading. Correlations were made with results of field and laboratory tests of "undisturbed" soil samples obtained from the test sites. The actual field testing was performed at two onshore sites, Lake Austin and Sabine but the soils were submerged at both sites and the criteria were mainly developed to design offshore piles.

The criteria for obtaining p-y curves for static loading consist mainly of two parts. The first is to obtain an expression to describe the variation of the ultimate soil resistance, p_u, with depth. The second is to obtain an expression to describe the variation of the soil resistance with lateral deflection at any particular depth along the pile. The basic difference in these parts is that theory can generally be used to determine the variation of p_u with depth, but empiricism must be employed to describe the actual shape of the p-y curves.

The ultimate soil resistance can be obtained by using the equation

$$p_u = N_p c_x b \tag{2.1}$$

where

N_p = normalized ultimate soil resistance,

c_x = undrained shear strength at the depth x,

b = pile width.

The value of N_p has been found to be a function of depth below the ground surface (Matlock, 1970; Reese and Welch, 1975; Reese et al., 1975; Thompson, 1977). The value of N_p increases with depth until it reaches some limiting value, at which point it remains constant for greater depths.

The general function, describing the variation of N_p at shallow depths, is given by

$$N_p = A_0 + \frac{\bar{\sigma}_x}{c_x} + J\frac{x}{b} \tag{2.2}$$

where

A_0 = normalized ultimate soil resistance at the ground surface,

$\bar{\sigma}_x$ = effective overburden stress,

x = depth below ground surface,

J = empirical coefficient.

From the tests he performed, Matlock recommends a value of 3 for A_0 and a value of 0.5 for J. These values were selected on the basis that they gave the best fit of computed to measured ultimate soil resistances. The value of 0.5 for J was obtained from the Sabine tests. A value of 0.25 was obtained for J from the Lake Austin tests, but Matlock recommends the use of the 0.5 value in design.

At depth, a limiting value of ultimate soil resistance is reached corresponding to a plane-strain condition. The value of N_p at depth is difficult to determine experimentally because it is not normally possible to force large deflections in a pile beyond a few diameters below the ground surface. Matlock recommends a value of 9 for N_p at large depths. The depth at which this transition occurs for a deposit with a uniform shear strength can be obtained by using the following equation:

$$x_r = \frac{6b}{\frac{\gamma b}{c_x} + J} \tag{2.3}$$

Matlock states, "Where soil properties undergo considerable variation with depth, it appears reasonable to consider the soil as a system of thin layers with x_r computed as a variable with depth according to the properties of each layer" (Matlock, 1970). However, in performing the analyses presented in this study, the weighted average values of c and γ were used in Eq. 2.3.

To define the shape of the p-y curve, a mathematical expression is selected which fits the experimental p-y curves. Matlock selected the equation

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}}\right)^{1/3} \tag{2.4}$$

where

$$y_{50} = 2.5\varepsilon_{50}b$$

and

ε_{50} = strain at 50% of the maximum stress difference, determined from a UU triaxial compression test.

A nondimensional p-y curve for static loading is shown in Fig. 2.1a.

The effects of cyclic loading are to decrease the ultimate soil resistance to $0.72p_u$ and to reduce the soil resistance at deflections greater than $3y_{50}$ at depths less than x_r. A cyclic p-y curve using the Matlock (1970) p-y criteria is shown in Fig. 2.1b. The shape of the cyclic p-y curve is based on the results of the field tests at Sabine and on laboratory model tests.

Stiff Clay Above the Water Table

Reese and Welch (1975) proposed criteria for predicting the behavior of flexible piles in stiff clays above the water table. The field tests were performed with a drilled shaft, but the criteria are applicable to most deep foundations. Procedures were recommended for constructing p-y curves for the cases of short-term static loading and for cyclic loading.

The criteria that were proposed for static loading are similar to those proposed by Matlock (1970). The equations describing the variation of p_u with depth are nearly the same, except for the manner in which the undrained shear strength is defined. Matlock defined c_x as the undrained shear strength at a depth x; Reese and Welch defined the undrained shear strength as c_a, which is the average undrained shear strength from the ground surface to the depth where p_u is being calculated. Another difference between the two criteria is the exponent describing the shape of the p-y curve. Reese and Welch suggest the following equation for stiff clays above the water table:

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}}\right)^{1/4} \qquad (2.5)$$

A p-y curve for static loading is shown in Fig. 2.2a. The procedure for accounting for the effects of cyclic loading using the Reese and Welch criteria is different than that proposed by Matlock. Reese and Welch found that for clay above the water table repeated load applications do not affect the ultimate soil resistance but do increase the deflection at

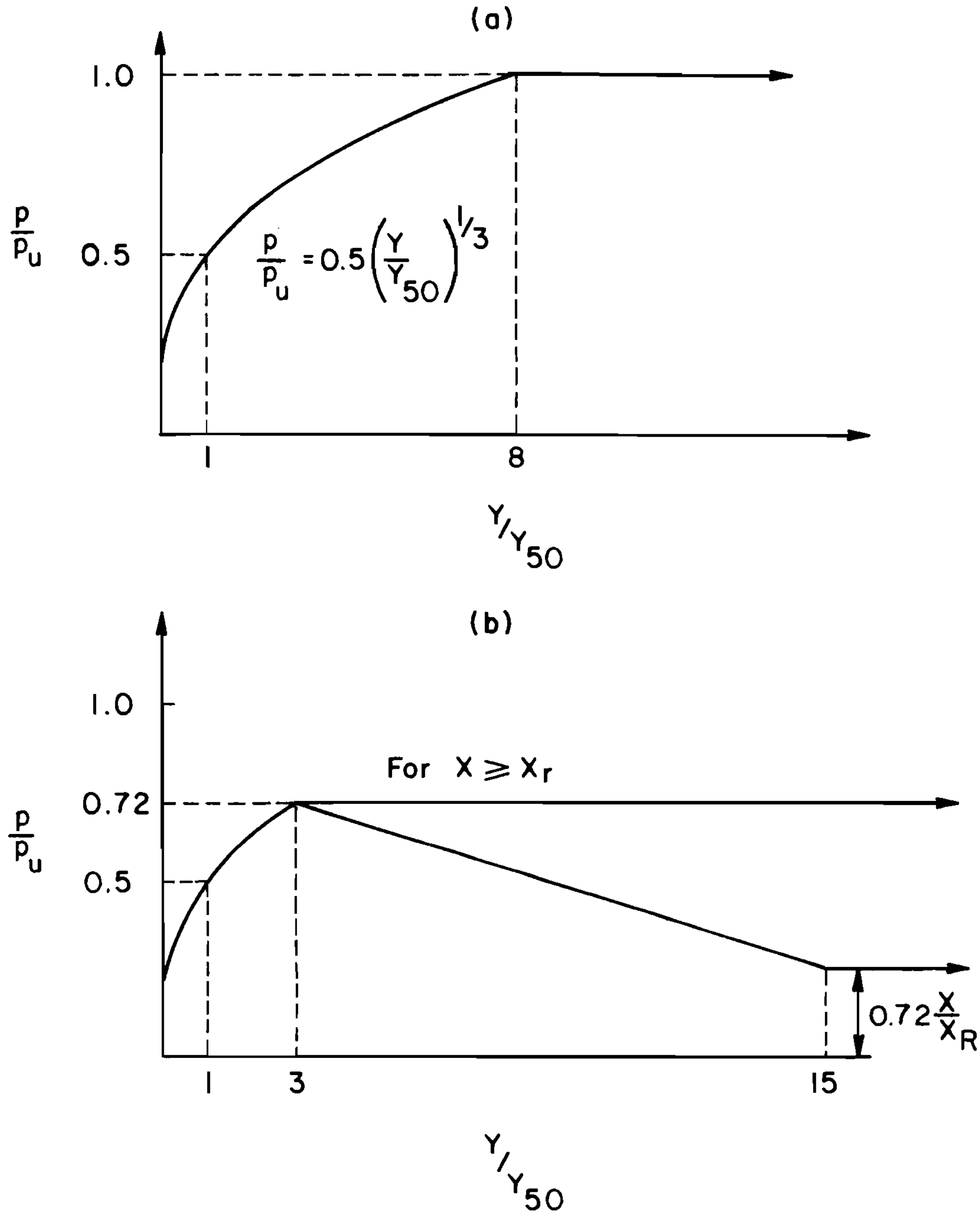


Fig. 2.1. Characteristic shape of the p-y curves for soft submerged clay (Matlock, 1970), (a) static loading. (b) cyclic loading.

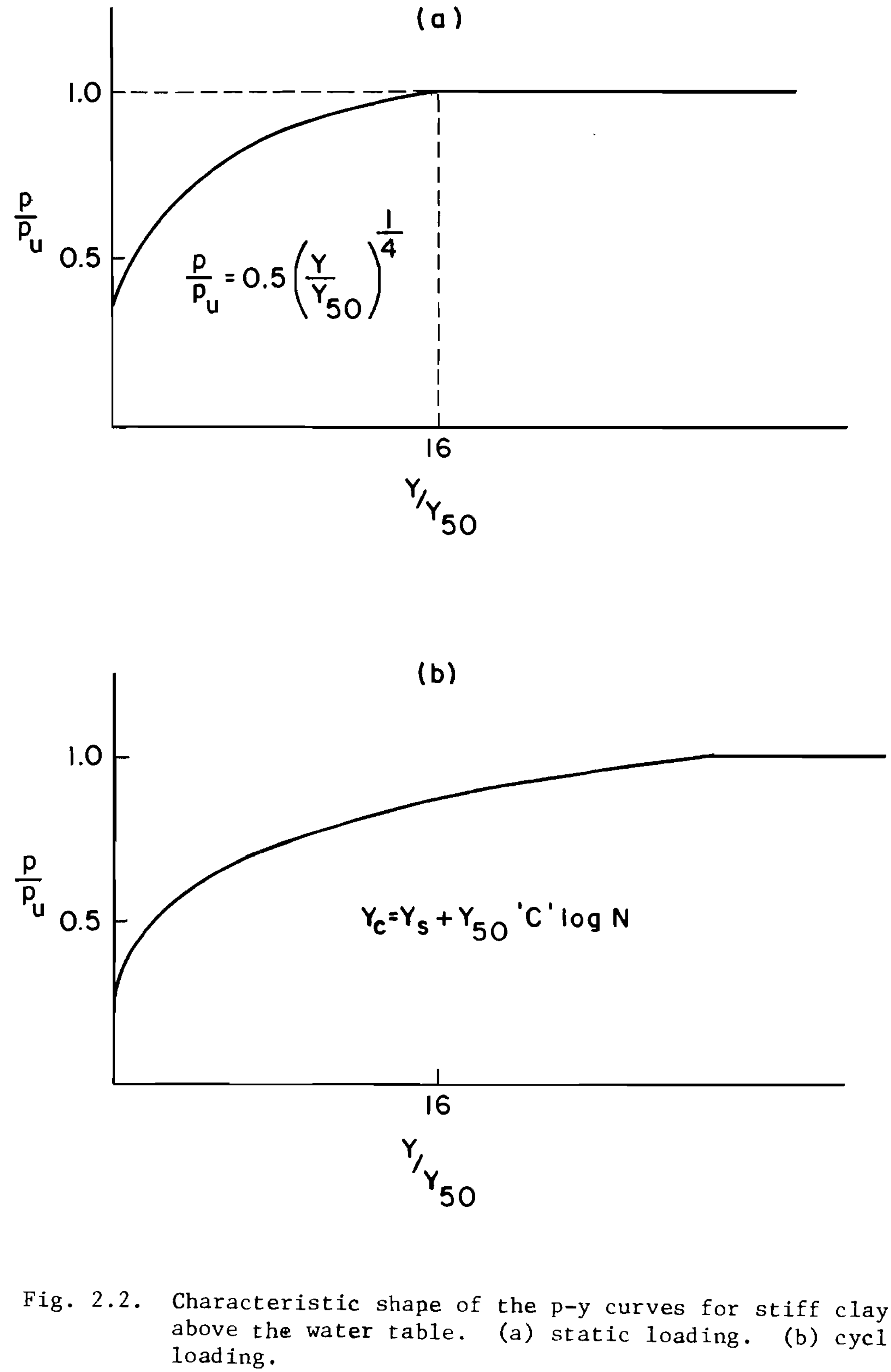


Fig. 2.2. Characteristic shape of the p-y curves for stiff clay above the water table. (a) static loading. (b) cyclic loading.

which p_u occurs. The cyclic deflection, y_c, is computed using the equation

$$y_c = y_s + y_{50} C \log N \tag{2.6}$$

where

y_s = static deflection,

$C = 9.6R^4$,

N = number of cycles,

and

$$R = \frac{p}{p_u} .$$

From the above equations, it is observed that the increase in deflection is not only a function of the number of cycles but also of the stress level.

Stiff Clay Below the Water Table

Reese et al. (1975) performed tests on fully instrumented pipe piles embedded in a submerged, heavily overconsolidated clay deposit. The purpose of the tests was to develop criteria which could be used to predict the behavior of piles under short-term static and cyclic loading.

The variation of the ultimate soil resistance with depth is based on the wedge-type-failure theory and the flow-around failure theory, which were both presented by Reese (1958). The two theoretical expressions which were derived are

$$p_{c1} = 2c_a b + \gamma' bx + 2.83c_a x \tag{2.7}$$

and

$$p_{c2} = 11cb \tag{2.8}$$

where

p_{c1} = ultimate soil resistance near the ground surface,

p_{c2} = ultimate soil resistance well below the ground surface.

Poor agreement was obtained when these theoretical ultimate soil resistances were compared to the ultimate soil resistances from the experiments. It was necessary to adjust the larger theoretical values by using the following empirical adjustment factors:

$$A_s = \frac{(p_u)_s}{p_c} \tag{2.9}$$

$$A_c = \frac{(p_u)_c}{p_c} \tag{2.10}$$

where

A_s = empirical adjustment factor for static loading,

A_c = empirical adjustment factor for cyclic loading,

p_c = ultimate soil resistance from theory,

$(p_u)_s$ = experimental ultimate soil resistance for static loading,

$(p_u)_c$ = experimental ultimate soil resistance for cyclic loading.

The values of A_s and A_c which were determined are shown in Fig. 2.3. The construction of the p-y curves for the static case involves the use of four functions. The complex definition of the p-y curves is necessary due to the irregular shape of the experimental p-y curves. The initial slope of a p-y curve is defined using the function

$$E_{si} = k_s x \tag{2.11}$$

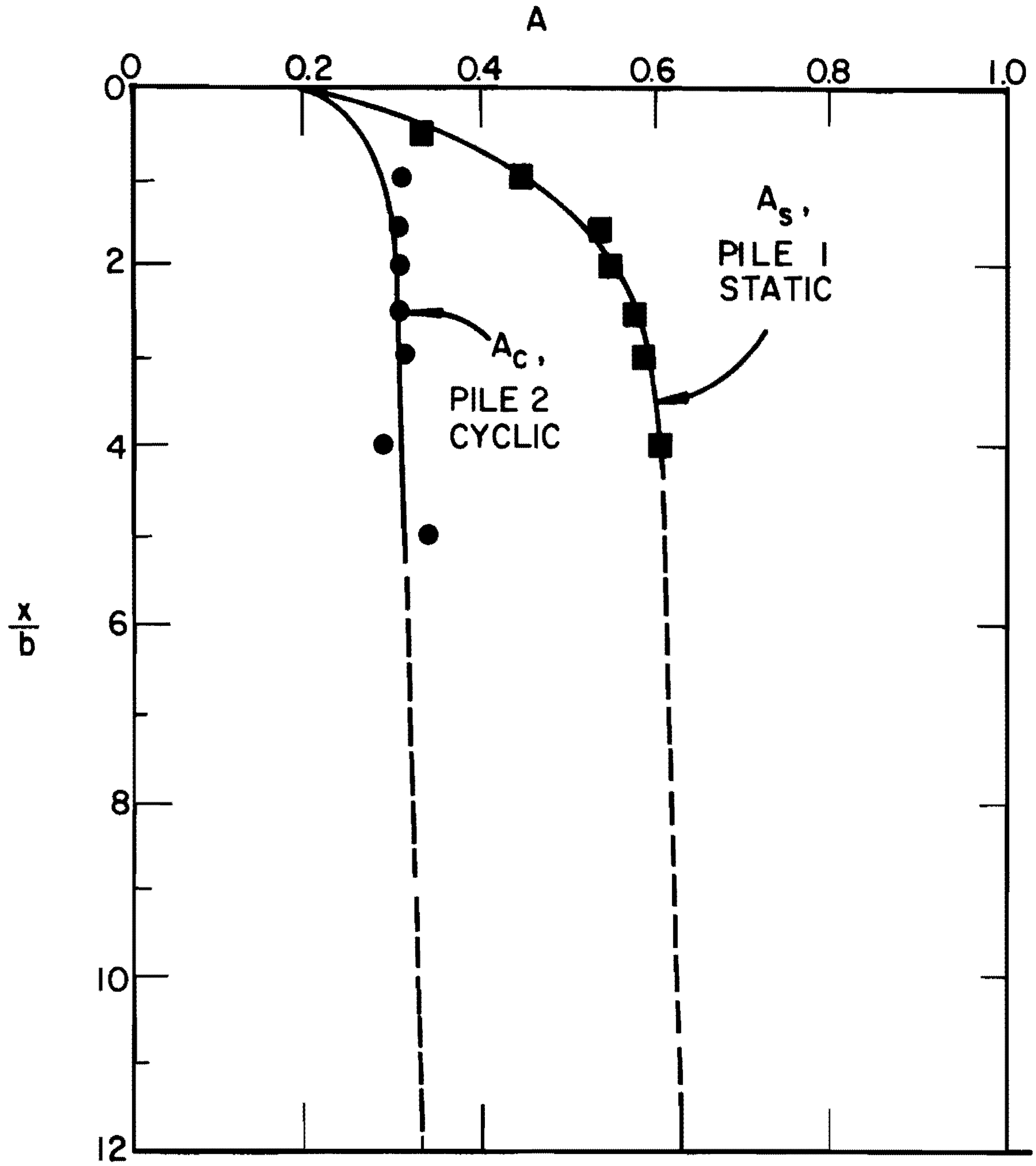


Fig. 2.3. Values of constants A_s and A_c. (Reese et al., 1975).

where

k_s = a constant for static loading.

Values of k_s which were suggested by Reese, et al., (1975) are shown in Table 2.1.

TABLE 2.1 RECOMMENDED VALUES OF k FOR STIFF CLAY

	Average Undrained Shear Strength (ton/ft^2)		
	0.5-1	1-2	2-4
k_s (static) $lb/in.^3$	500	1000	2000
k_c (cyclic) $lb/in.^3$	200	400	800

To define the next portion of the curve, the parameter ε_{50} was used to define y_{50} in the equation

$$p = 0.5p_c \left(\frac{y}{y_{50}}\right)^{0.5} \tag{2.12}$$

where

$$y_{50} = \varepsilon_{50} b$$

The parabolic portion of the curve goes through the origin, but the actual p-y curve starts at the intersection of the straight line, defined with the slope E_{si}, and the parabola, defined by Eq. 2.12. Equation 2.12 continues to the deflection $A_s y_{50}$ where A_s is obtained from Fig. 2.3 for the non-dimensional depth x/b. At this point, the parabola is modified by an offset defined by the equation

$$\text{offset} = 0.55p_c \left(\frac{y-A_s y_{50}}{A_s y_{50}}\right)^{1.25} \tag{2.13}$$

This offset to the p-y curve continues to the deflection corresponding to $6A_s y_{50}$. At this point, the p-y curve assumes a straight line with a slope defined by the equation

$$E_{ss} = \frac{-0.0625p_c}{y_{50}} \tag{2.14}$$

The straight line defined by Eq. 2.14 continues to the deflection $18A_s y_{50}$, where the soil resistance remains constant for increasing deflections. A p-y curve for static loading is shown in Fig. 2.4a.

The effects of cyclic loading are to reduce the ultimate soil resistance and to reduce the deflection at which this ultimate resistance occurs. Three functions are used to describe the cyclic p-y curve. The first function is

$$E_{si} = k_c x \tag{2.15}$$

where

$$k_c = \text{a constant for cyclic loading.}$$

Values for k_c are given in Table 2.1. The parabolic portion of the cyclic p-y curve starts at the intersection of the straight line, defined by Eq. 2.14, and the parabola, defined by the following equation:

$$p = A_c p_c \; 1 - \left(\frac{y-0.45y_p}{0.45y_p}\right)^{2.5} \tag{2.16}$$

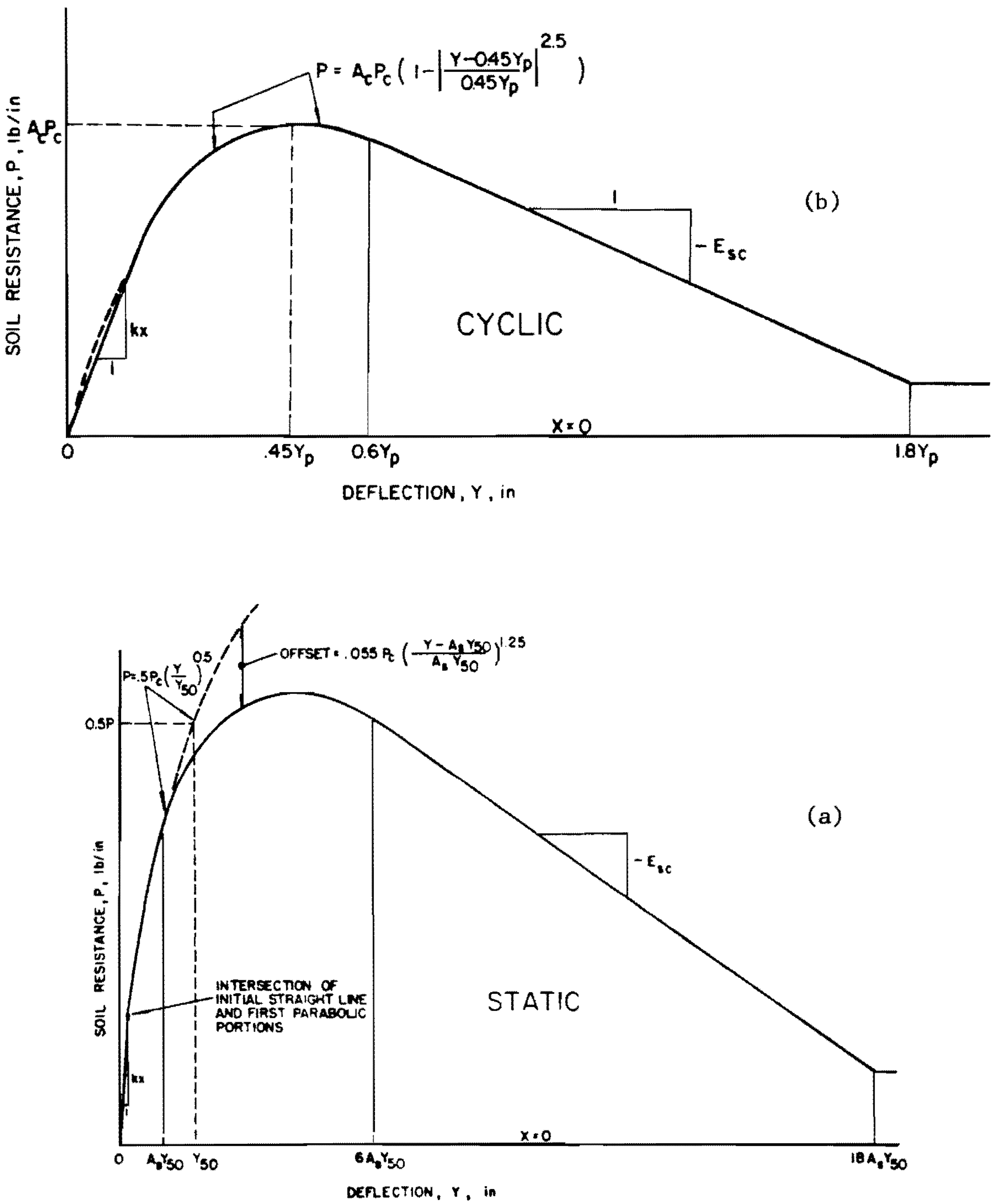


Fig. 2.4. Characteristic shape of the p-y curves for stiff clay below the water table. (Reese et al, 1975). (a) static loading. (b) cyclic loading.

where

$$y_p = 4.1A_s y_{50}$$

The parabola continues to a deflection corresponding to $0.6y_p$. At this point the p-y curve assumes a straight line with a slope defined by the equation

$$E_{sc} = \frac{-0.085p_c}{y} \qquad (2.17)$$

The straight line defined by the slope E_{sc} continues to the deflection $1.8y_p$, where the soil resistance remains constant for increasing deflections. A p-y curve for cyclic loading is shown in Fig. 2.4b.

Unified Criteria

Sullivan (1977) proposed criteria which could be used for all submerged clays, irrespective of the shear strength of the clay. Sullivan used the results from the tests in soft clay at Sabine and the tests in stiff clay at Manor to establish the Unified criteria. Sullivan generalized his criteria by introducing empirical factors obtained from correlations with the test data. The empirical factors depend mainly on the stress-strain properties of the clay.

The expression proposed by Sullivan for N_p as a function of depth is plotted in nondimensional form in Fig. 2.5. His variation differs from both Matlock (1970) and Reese et al. (1975), because he used three equations to describe the variation of p_u with depth. Sullivan's expression is the same as Matlock's for a constant shear strength deposit and for x/b greater than 3, as shown in Fig. 2.5. The equations proposed by Sullivan describing the variation of the ultimate soil resistance with depth are

$$p = \left(2 + \frac{\bar{\sigma}_x}{c_a} + .833\frac{x}{b}\right)c_a b \qquad \text{for } 0 < x < 3b \qquad (2.18)$$

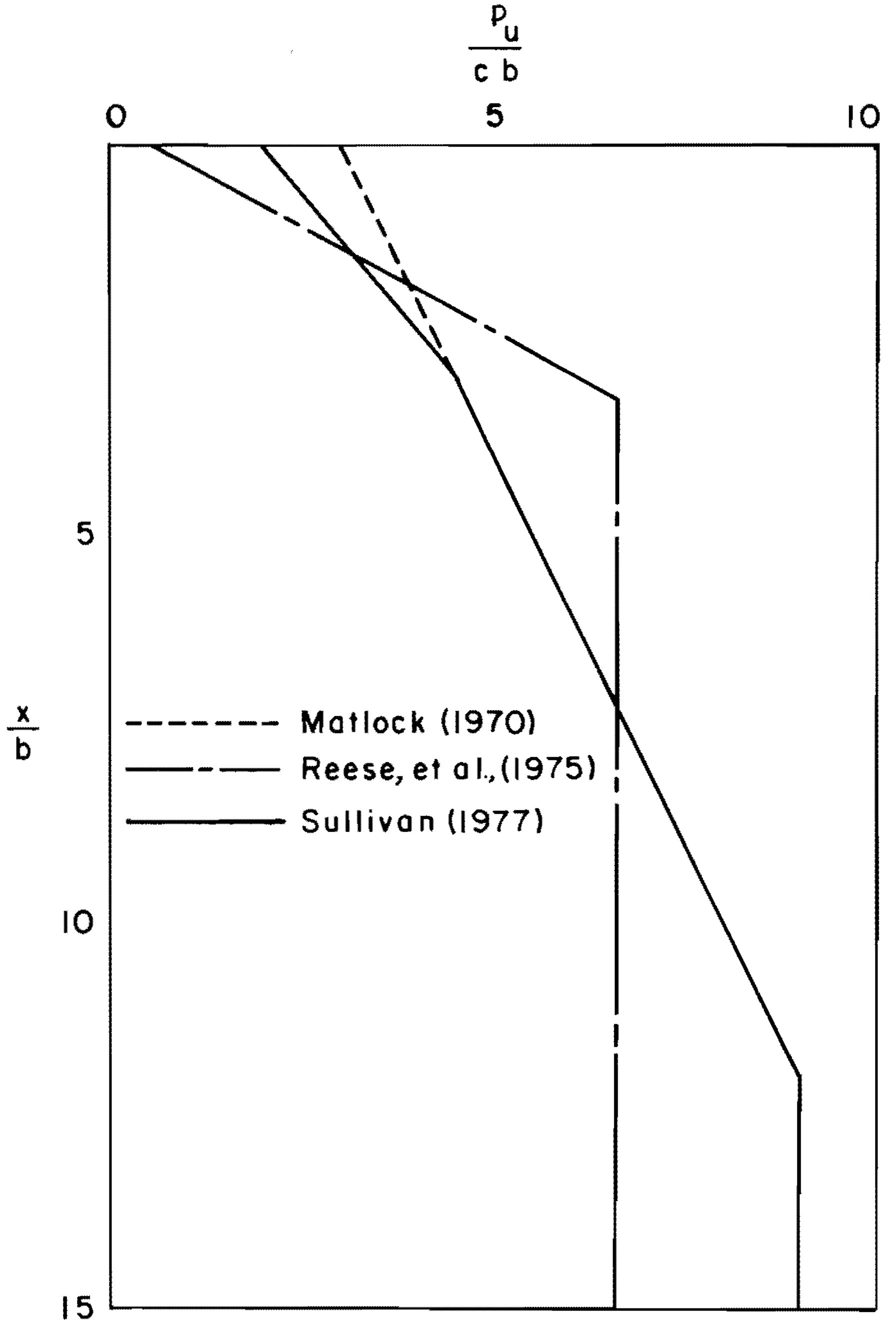


Fig. 2.5. Variation of N_p with depth for a soil deposit with a uniform shear strength.

$$p = (3 + .5\frac{x}{b})\ cb \qquad \text{for } 3b < x < 12B \qquad (2.19)$$

$$p_u = 9cb \qquad \text{for } x > 12b \qquad (2.20)$$

The transition depths are for a clay with a constant shear strength. If c_a is not constant, the smallest value of p_u from the three equations should be selected corresponding to a particular depth.

The shape of the p-y curve was generalized, and correlations were made with the results of both the Sabine tests and the Manor tests. Simple mathematical expressions and empirical factors were used to account for the large difference in behavior of the test piles at the two sites.

The generalized p-y curve for static loading is shown in Fig. 2.6a. The curve is similar to that proposed by Matlock (1970). In the Matlock criteria, the slope of the p-y curve approaches infinity as the deflection approaches zero. Sullivan chose to use a linear function, Eq. 2.21, to define the initial slope of the p-y curve:

$$p = (E_s)_{max} y \qquad (2.21)$$

where

$$(E_s)_{max} = kx \qquad (2.22)$$

The parameter k is a constant whose magnitude depends only on the shear strength of the clay.

Except for the initial slope, the shape of the static p-y curve, up to a deflection of $8y/y_{50}$, is the same as the shape suggested by Matlock (1970) and is given by Eq. 2.4. However, Sullivan redefined y_{50} as

$$y_{50} = A\varepsilon_{50} b \qquad (2.23)$$

Based on the results of the Sabine and Manor tests, Sullivan suggests

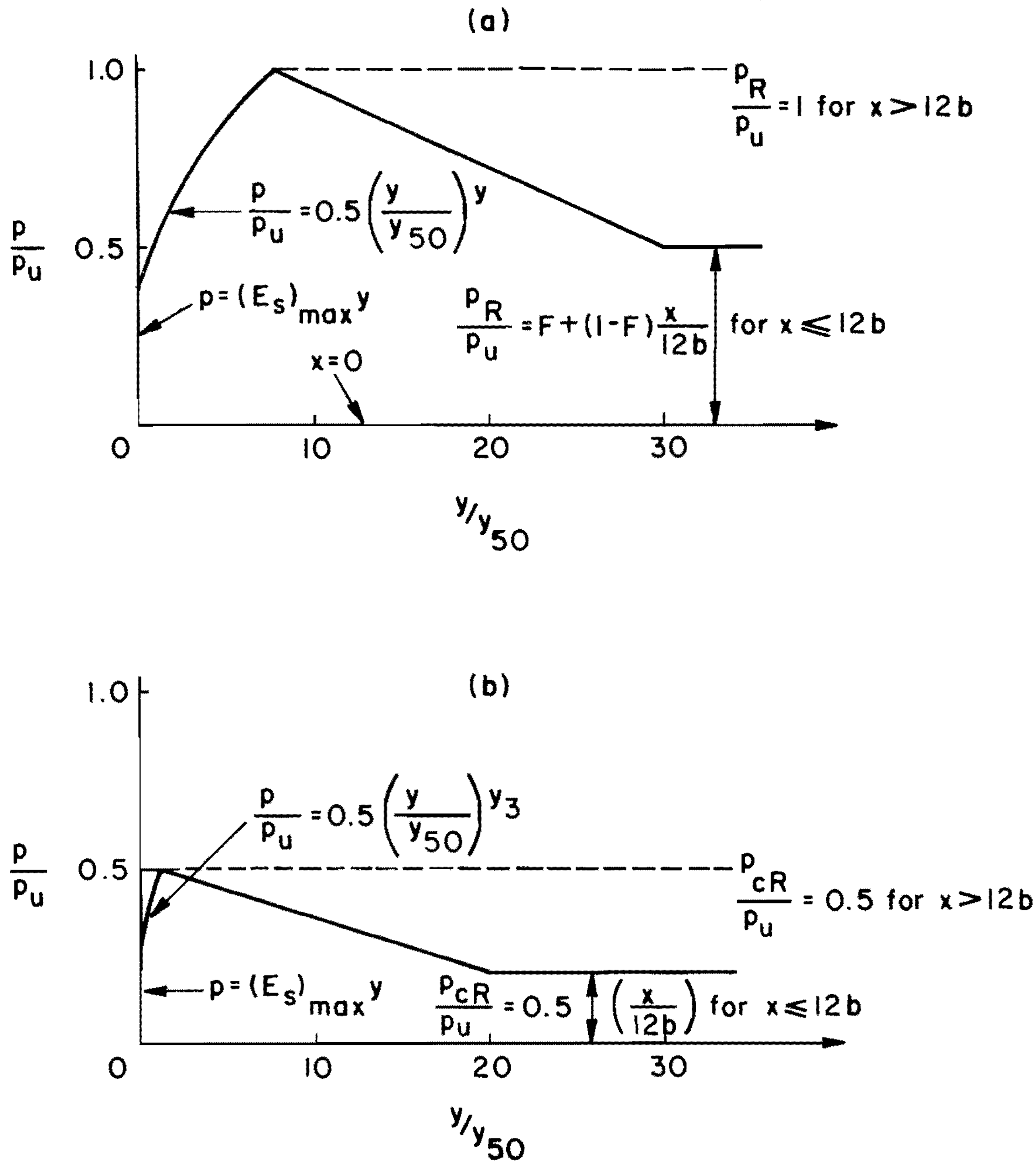


Fig. 2.6. Characteristic shape of the p-y curves for clay, Unified criteria (Sullivan, 1977). (a) static loading. (b) cyclic loading.

values for A of 2.5 and 0.35, respectively.

The residual shear resistance, p_R, illustrated in Fig. 2.6a, for static loading is defined by the equation

$$p_R = p_u\left(F + (1-F)\frac{x}{12b}\right) \qquad \text{for } x < 12b \qquad (2.24)$$

or

$$p_R = p_u \qquad \text{for } x > 12b \qquad (2.25)$$

Values of F, which were determined for the Sabine and Manor sites, are 1.0 and 0.5, respectively. For static loading, the residual soil resistance is reached at a deflection of $30y_{50}$. In the analyses presented in Chapter 4, the A and F parameters suggested by Sullivan were used for soils which were similar to the soils at Sabine and Manor.

The p-y curve for cyclic loading, shown in Fig. 2.6b, is similar in shape to the curve for static loading, but the ultimate soil resistance is reduced. Matlock (1970) found that p_u was reduced to about 72% of the static value; however, Reese et al. (1974) found that p_u was reduced to about 50% of the static value for their tests. Sullivan used a 50% reduction of p_u in his criteria. The reduction in p_u and the use of 2 for N_p at the ground surface would lead to a conservative estimate of p_u for the Sabine tests. In Fig. 2.6b, it can be seen that the soil resistance at deflections larger than $20y/y_{50}$ is zero at the ground surface and increases to $0.5p_u$ at a depth of 12b.

METHODS FOR PREDICTING p-y CURVES IN COHESIONLESS SOILS

Reese, et al., (1974) proposed criteria for cohesionless soils for analyzing the behavior of piles under static and cyclic loading. The procedures were developed from the results of tests at Mustang Island on 24-in. diameter, flexible piles embedded in a deposit of submerged, dense, fine sand (Cox, et al., 1974). Experimental p-y curves were obtained from the results of tests on the fully instrumented piles. Both theory and empiricism were then employed to obtain mathematical expressions that fit the experimentally derived p-y curves.

The ultimate soil resistance near the ground surface is based on a wedge-type-failure theory. The passive force, F_p, which results from the formation of the wedge, can be differentiated with respect to depth, to yield the equation

$$p_{ct} = \bar{\gamma} x \frac{K_o x \tan \phi \sin \beta}{\tan (\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan (\beta - \phi)} (b + x \tan \beta \tan \alpha)$$

$$+ K_o \tan \beta (\tan \phi \sin \beta - \tan) - K_a b \qquad (2.26)$$

where

K_o = coefficient of earth pressure at rest,

ϕ = angle of internal friction (degrees),

β = $45 + \phi/2$

α = $\phi/2$

K_a = $\tan^2 (45 - \phi/2)$.

The ultimate soil resistance at some distance below the ground surface was derived theoretically and is given by the following euqation:

$$p_{cd} = K_a \bar{\gamma} x (\tan \beta - 1) + K_o \bar{\gamma} x \tan \phi \tan^4 \beta \qquad (2.27)$$

When the measured soil properties were used in Eqs. 2.26 and 2.27, it was found that the calculated ultimate resistance was much smaller than the experimental ultimate soil resistance. Therefore, it was necessary to use an empirical adjustment factor to bring the two quantities into agreement:

$$p_u = A p_c \qquad (2.28)$$

where

$$p_c \;=\; \text{ultimate soil resistance from theory,}$$

$$A \;=\; \text{empirical adjustment.}$$

The value of A depends on depth and whether the pile is subjected to short-term, static or to cyclic loads. In the former case A_s is used and in the latter case A_c is used. Plots of A_s and A_c versus x/b are shown in Fig. 2.7. Values of the ultimate soil resistance were obtained from the experiments only to a relatively shallow depth (Reese et al. 1974). Below this depth, it can only be assumed that the theoretical ultimate soil resistance is correct.

The construction of p-y curves for both static and cyclic loading involves the use of a number of functions. The coordinates at the beginning and end of these functions are shown in Fig. 2.8. They are p_k y_k; p_m, y_m; and p_u, y_u. The computation of these coordinates will be discussed in the following paragraphs. The coordinates p_m, y_m and p_u, y_u depend on empirical adjustment factors and the pile width. The value of p_m is given by

$$p_m = Bp_c \tag{2.29}$$

where

$$B \;=\; \text{an empirical adjustment factor, shown in Fig. 2.9.}$$

The value of y_m is given by the equation

$$y_m = b/60 \tag{2.30}$$

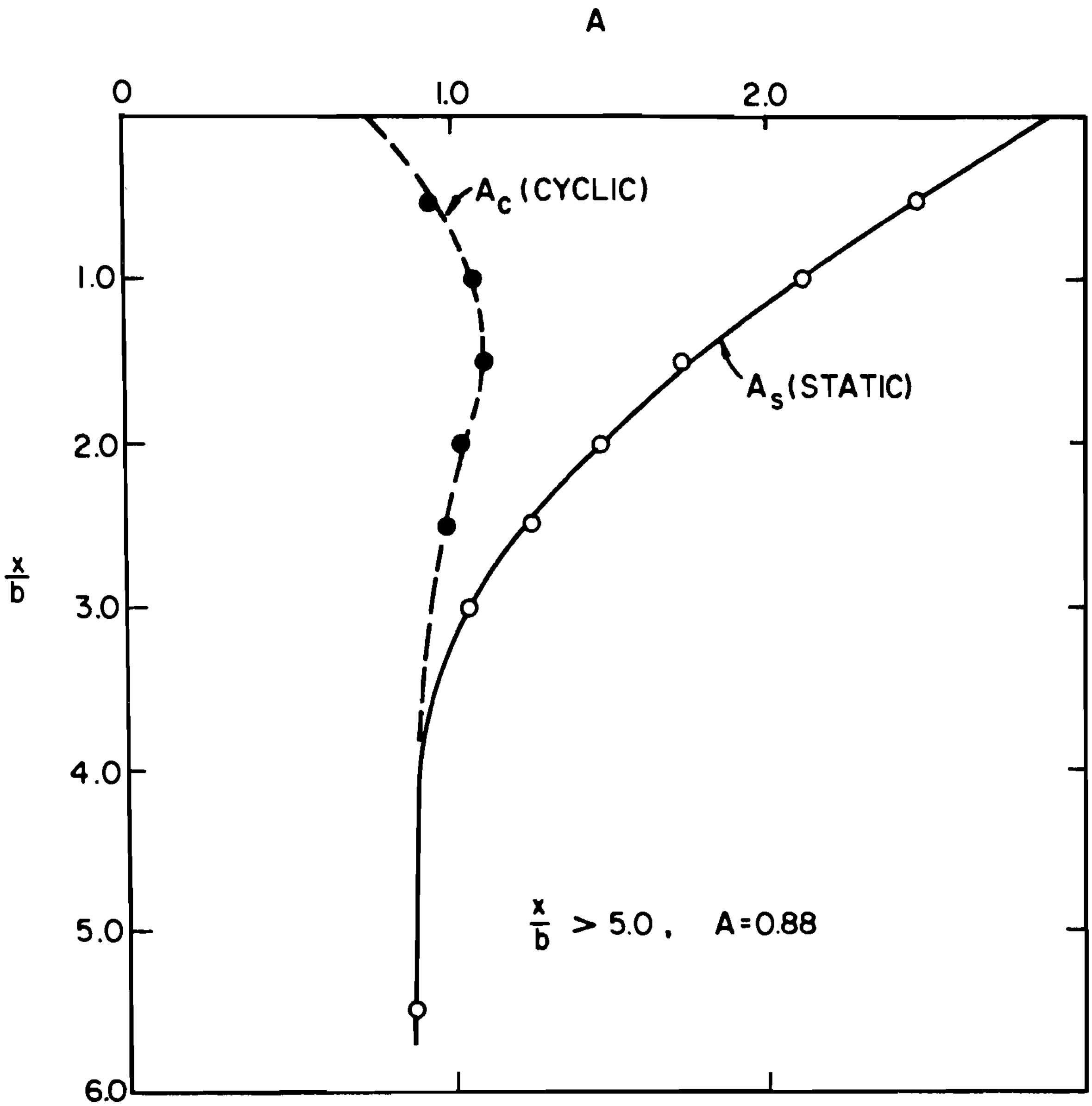


Fig. 2.7. Nondimensional coefficient A for ultimate soil resistance versus depth. (Reese et al., 1974).

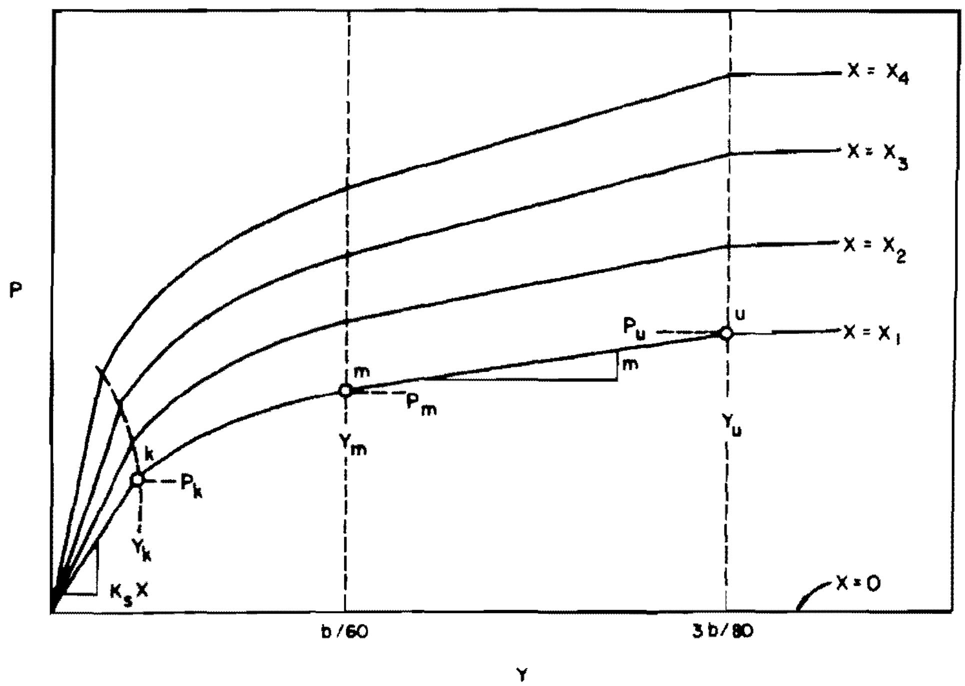


Fig. 2.8. Characteristic shape of a family of p-y curves in sand. (Reese, et al., 1974).

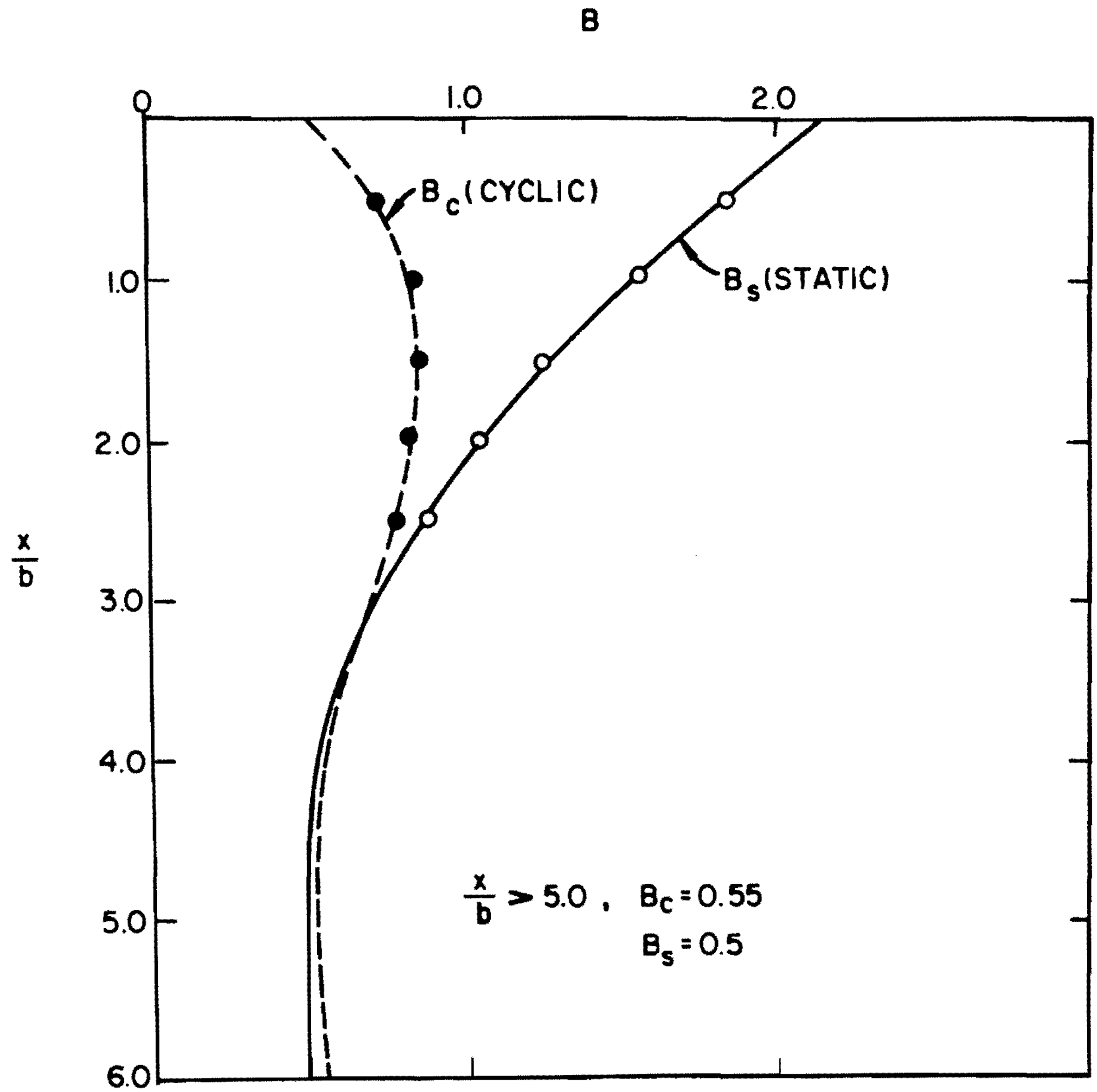


Fig. 2.9. Nondimensional coefficient B for soil resistance versus depth. (Reese et al., 1974).

and the value of y_u is given by the equation

$$y_u = 3b/80 \tag{2.31}$$

The p-y curves were assumed to vary in a parabolic form between p_k, y_k and p_m, y_m. The equation describing the shape of the typical curve is

$$p = Cy^{1/n} \tag{2.32}$$

The constants c and n must be evaluated by using the following equations:

$$m = \frac{p_u - p_m}{y_u - y_m} \tag{2.33}$$

$$n = \frac{p_m}{my_m} \tag{2.34}$$

$$C = \frac{p_m}{y_m^{1/n}} \tag{2.35}$$

The point y_k can be determined by using the equation

$$y_k = \left(\frac{C}{kx}\right)^{\frac{n}{n-1}} \tag{2.36}$$

The initial straight-line portion of the curve up to y_k can now be determined by the equation

$$p = E_{si}y \tag{2.37}$$

where

$$E_{si} = kx$$

Values of k will be given in Chapter 5 for submerged sands and sands above the water table as a function of relative density. In some instances, k is so small that the initial straight-line portion of the p-y curve does not intersect the parabolic portion of the p-y curve at the deflection y_k. In this case, the initial straight line should simply be extended until it intersects the p-y curve, and the p-y curve remains unchanged for deflections greater than the deflection at which the intersection occurs. Therefore, the initial straight line defined by E_{si} should always be the largest soil modulus.

Another straight-line portion of the curve can be established by connecting coordinates p_m, y_m and p_u, y_u with a line segment. Beyond a value of y_u, the soil resistance is constant.

CHAPTER 3. PARAMETRIC STUDY

INTRODUCTION

The behavior of a laterally loaded pile is a complex function of a number of parameters. In this chapter, a number of soil and pile parameters were varied to determine the effect that these variations had on the computed pile behavior. In each case, one input parameter was varied while the other parameters were held constant. It was then determined what effect the variation of this single parameter had on the pile behavior. The results of these analyses, using different p-y criteria, would be different if a different set of initial values were assumed for the soil and pile parameters, but for each set of criteria the relative difference in results would be small.

In conducting this study, the criteria for stiff clay above the water table were termed StiffA, the criteria for stiff clay below the water table were termed StiffB, and the criteria for sand were not further identified. Each of the sets of criteria were used to determine what effect changes in soil parameters would have on the computed pile behavior. The initial soil parameters which were selected for each set of criteria are shown in Table 3.1. The initial pile parameters which were used in all of the analyses are shown in Table 3.2. The pile head was free to rotate in these analyses, and the lateral load was applied at the ground surface.

In general, the soil parameters which were varied are: c, ε_{50}, γ, and k for clay and ϕ, γ, K_o, and k for sand. The percentage change of the input parameters was computed from the following equation:

$$\Delta = \frac{\text{New Value - Initial Value}}{\text{Initial Value}} \times 100 \qquad (3.1)$$

In most cases, a $\pm 50\%$ change in each parameter was used in the analysis. However, a small percentage change was used in cases where the results were sensitive to changes in the parameter, and a larger percentage change was used in cases where the results were relatively insensitive to changes.

TABLE 3.1. INITIAL SOIL PARAMETERS

Soil Properties	StiffA	StiffB	Sand
c (lb/ft^2)	2500	2500	—
ϵ_{50} (%)	0.5	0.5	—
ϕ	—	—	39
γ (lb/ft^3)	120	60*	66*
K_o	—	—	0.4
k_s $(lb/in.^3)$	—	1000	130
k_c $(lb/in.^3)$	—	400	130

*For these cases the soil was completely submerged

TABLE 3.2. INITIAL PILE PARAMETERS

b (in.)	30
EI $(lb\text{-}in.^2)$	$5x10^{10}$
L (ft)	50

VARIATION IN SOIL PROPERTIES

StiffA Criteria

The results of the analyses using the static StiffA criteria are plotted in Figs. 3.1, 3.2, and 3.3. The results of the analyses for a $\pm$50% variation in c are shown in Fig. 3.1. A $\pm$50% variation in c would correspond to values for c of 1250 and 3750 lb/ft^2. The results of these analyses indicate that the lateral deflection is much more sensitive to variations in c than is the maximum moment. Also, the results of the analyses were more sensitive to a decrease in c than to an increase in c.

The results of the analyses for a $\pm$50% variation in ε_{50} are plotted in Fig. 3.2. The effects of variations in ε_{50} on the results of the analyses are much less than the effect that c had in the previous analyses. The changes in maximum moment due to a change in ε_{50} are very small. The lateral deflection is more sensitive to changes in ε_{50}, but the overall effect on the pile behavior is small. These results are useful because ε_{50} is frequently not reported, and a value must be selected based on the shear strength of the clay.

The results of the analyses for a $\pm$50% change in γ are shown in Fig. 3.3. Almost no change in the results of the analyses occurred when γ was varied. The $\pm$50% variation in this case corresponds to a change of between 60 lb/ft^3 and 180 lb/ft^3 from the initial value of 120 lb/ft^3. From these results, it is evident that γ does not have to be known with any degree of accuracy to analyze the behavior of piles using the StiffA criteria. Selection of a reasonable value for γ will suffice.

The results of the analyses using the cyclic StiffA criteria for 100 cycles of loading are presented in Figs. 3.4 and 3.5. Comparing the 0% curves in Figs. 3.1 and 3.4, it is seen that cyclic loading causes a 40% increase in lateral deflection, but only causes an 11% increase in the maximum moment. The effect of variations in c and ε_{50} on the results of the analysis using the cyclic StiffA criteria is similar to the results obtained using the static StiffA criteria. The pile behavior was sensitive to changes in c, but not very sensitive to changes in ε_{50}.

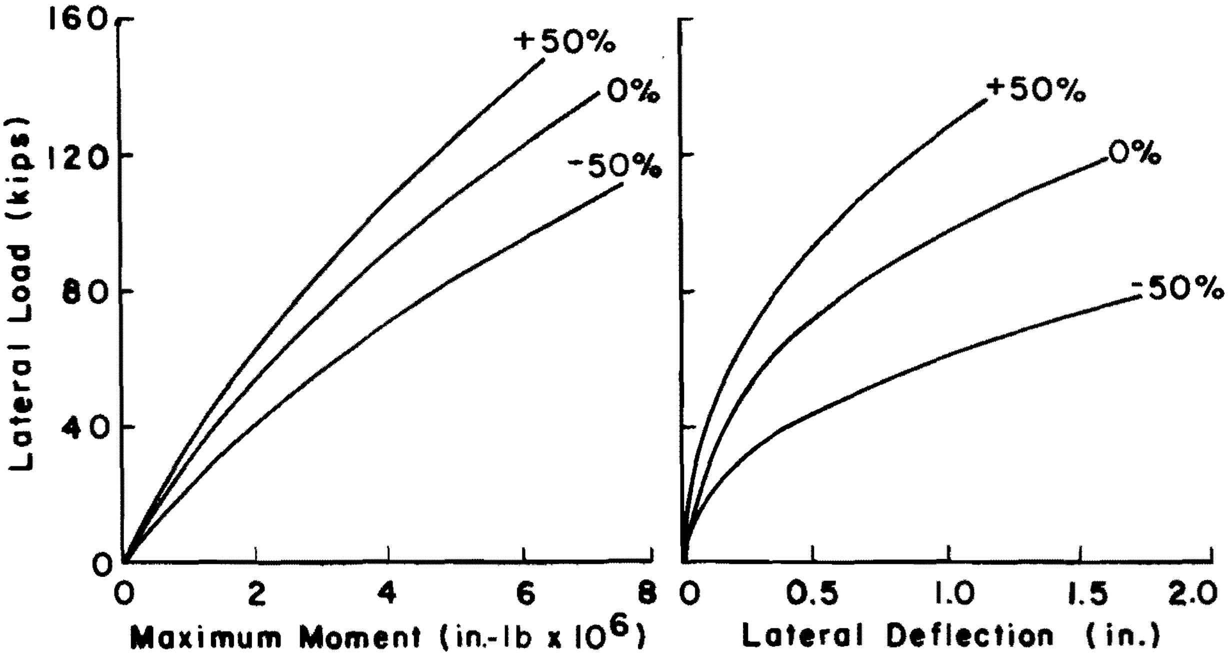


Fig. 3.1. Comparison between results for $\pm 50\%$ variation in c using StiffA criteria for static loading.

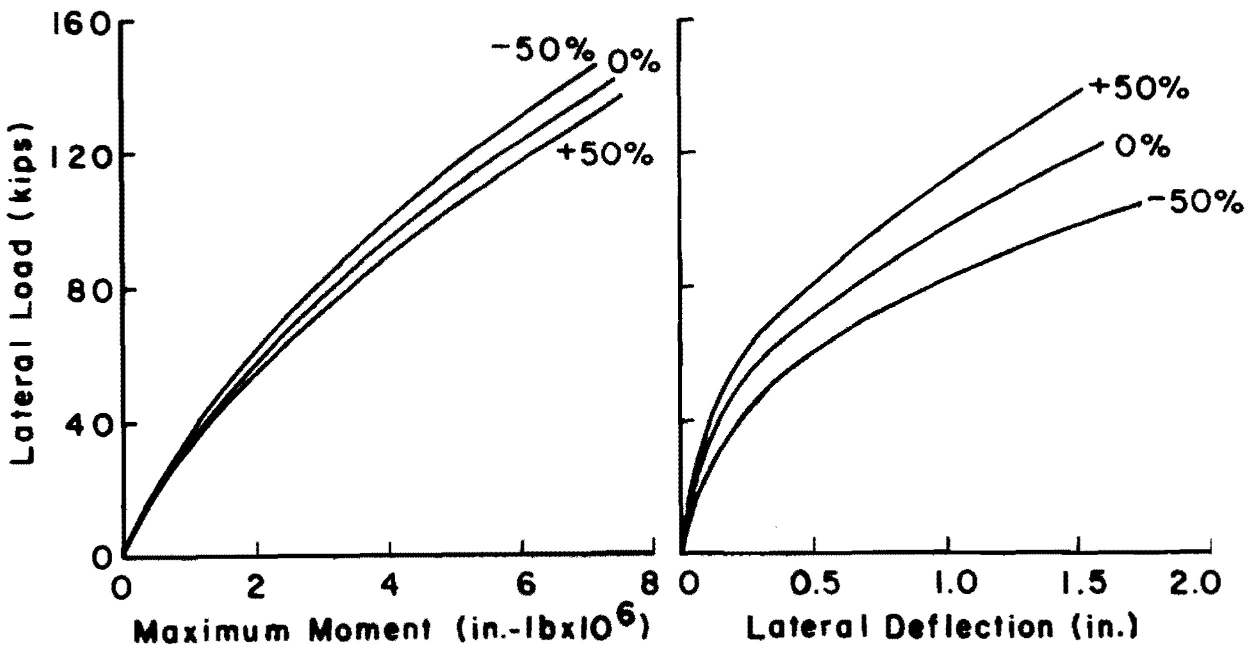


Fig. 3.2. Comparison between results for $\pm 50\%$ variation in ε_{50} using the StiffA criteria for static loading.

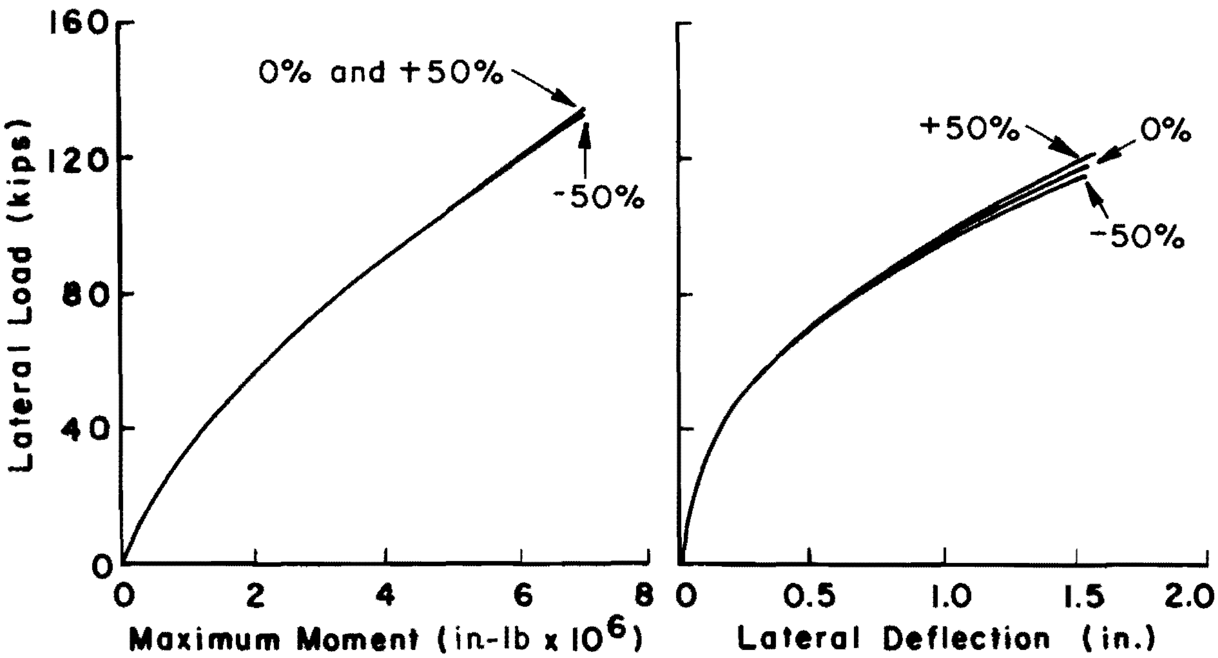


Fig. 3.3. Comparison between results for ± 50 variation in γ using StiffA criteria for static loading.

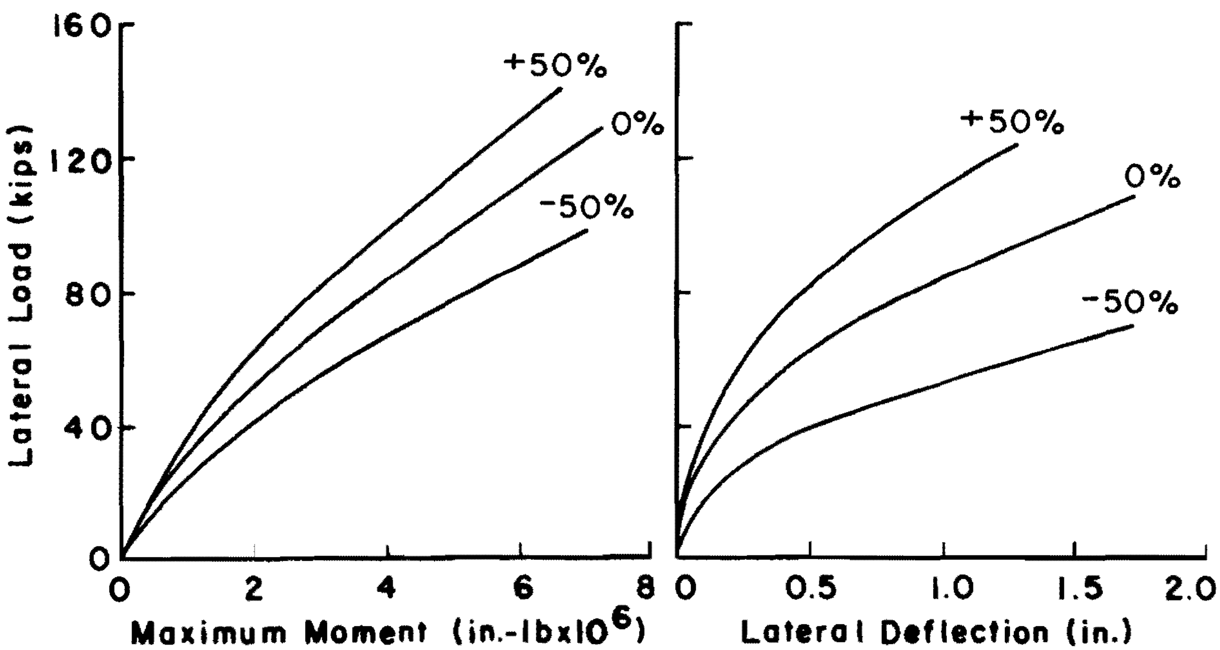


Fig. 3.4. Comparison between results for $\pm 50\%$ variation in c using StiffA criteria for cyclic loading.

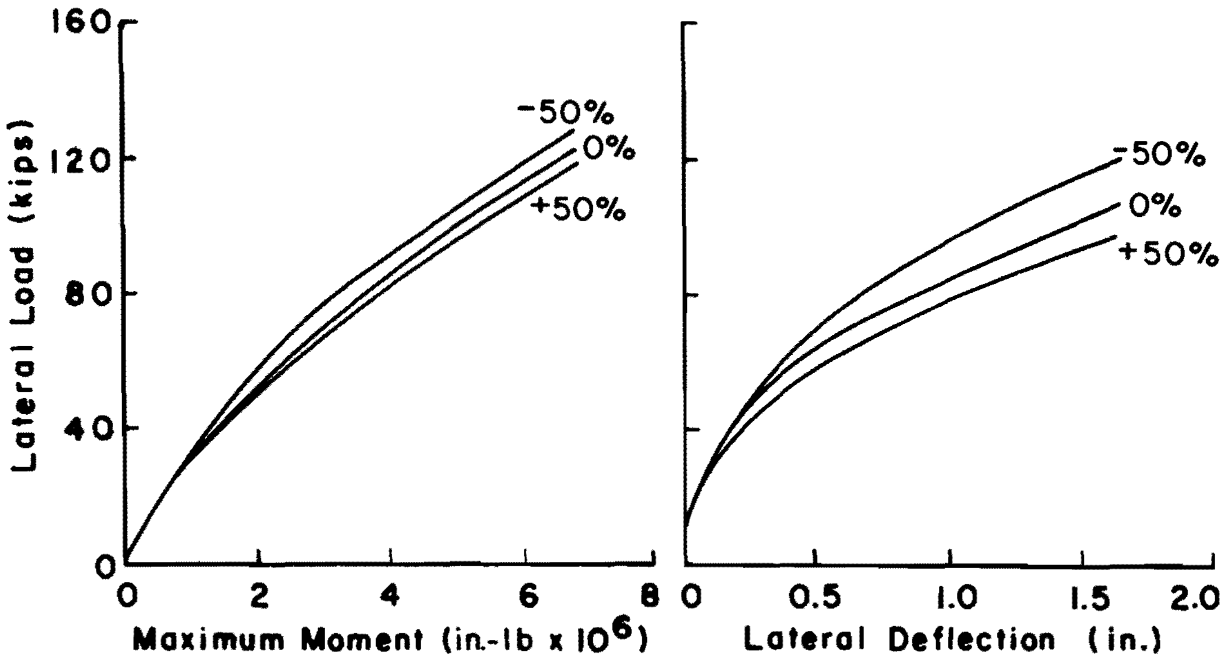


Fig. 3.5. Comparison between results for $\pm 50\%$ variation in ε_{50} using StiffA criteria for cyclic loading.

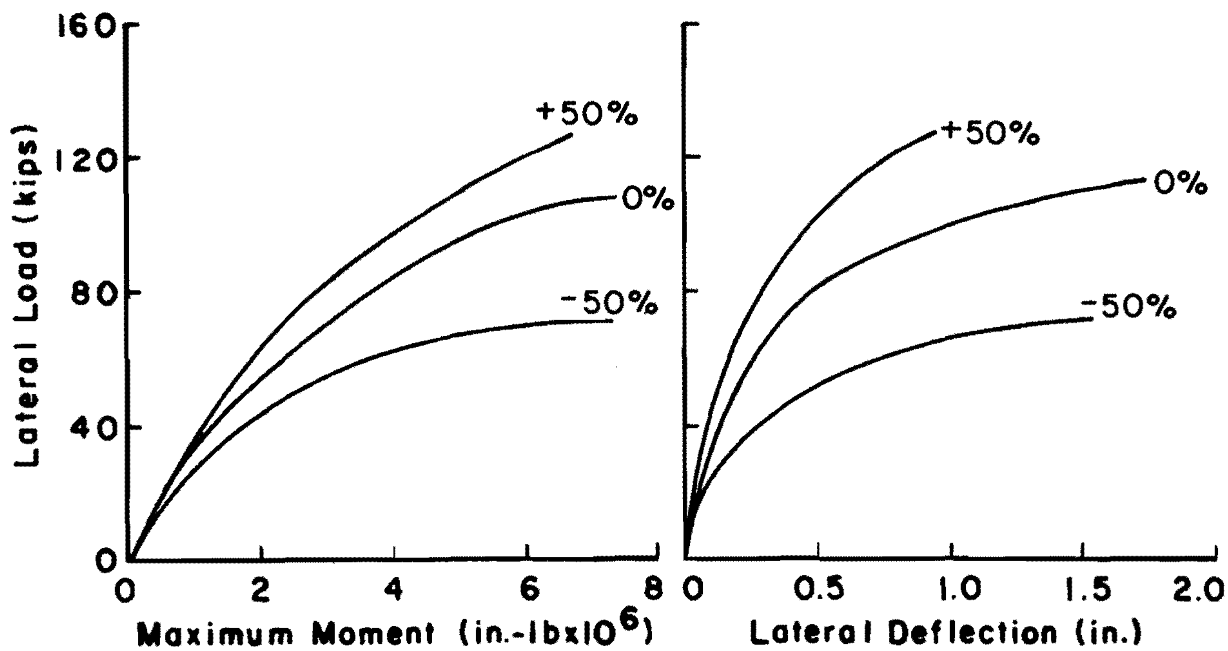


Fig. 3.6. Comparison between results for $\pm 50\%$ variation in c using StiffB criteria for static loading.

StiffB Criteria

The results of the analyses using the static StiffB criteria are plotted in Figs. 3.6, 3.7, and 3.8. Comparing the 0% curves in Figs. 3.6 and 3.1, it is observed that the results obtained using the static StiffB criteria and the Static StiffA criteria are similar, but the ultimate lateral load capacity is larger when the StiffA criteria are used. The results of the analyses for a $\pm$50% variation in c using the static StiffB criteria are shown in Fig. 3.6. The results using the StiffB criteria are also very sensitive to variations in c. Both the maximum moment and lateral deflection increase substantially for a 50% decrease in c, but do not decrease by the same percentage for a 50% increase in c.

The results of the analyses for a $\pm$50% variation in ε_{50} are shown in Fig. 3.7. The results of these analyses are similar to those obtained for the StiffA criteria. The main difference is the variation in the ultimate pile capacity due to changes in ε_{50}. For the StiffA criteria a decrease in ε_{50} caused an increase in pile capacity. This result is expected because a lower ε_{50} stiffens the p-y curve. For the StiffB criteria, the lower ε_{50} increases the pile capacity initially, but at larger loads the pile capacity is reduced. The reason for this reversal in behavior is due to the general shape of the StiffB p-y curve. A p-y curve which has a peak and residual soil resistance will exhibit the type of behavior shown in Fig. 3.7.

The results of the analyses for a $\pm$75% variation in k_s are shown in Fig. 3.8. In this case, a $\pm$75% change was used because the results of the analyses were so insensitive to variations in k_s. As shown, the maximum moment is unaffected by variations in k_s. The lateral deflection is affected by a decrease in k_s, but is unaffected by increases in k_s. As k_s, increases, the intersection between the initial straight-line portion of the p-y curve, given by Eq. 2.11, and the parabolic portion of the curve, given by Eq. 2.12, moves closer to the origin, thus decreasing the length of the initial straight-line portion of the curve. Therefore, an overestimate of k_s will have no ill effects on the results of the analysis, but selection of a k_s which is too small may reduce the computed pile capacity.

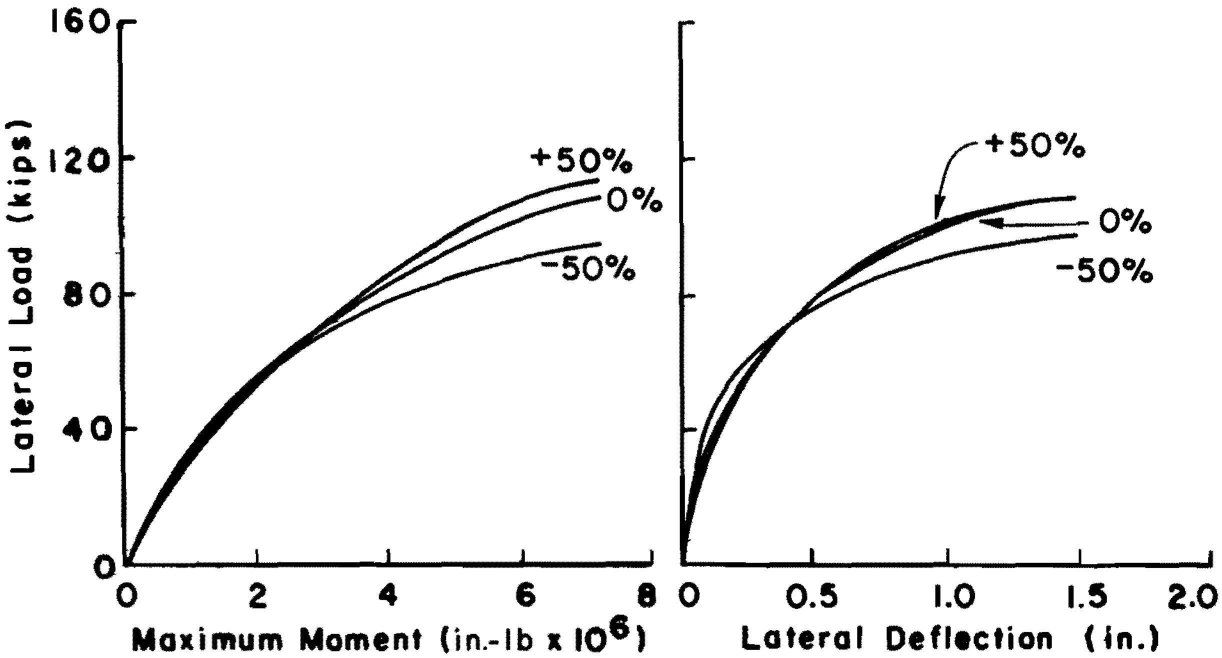


Fig. 3.7. Comparison between results for ±50% variation in ε_{50} using StiffB criteria for static loading.

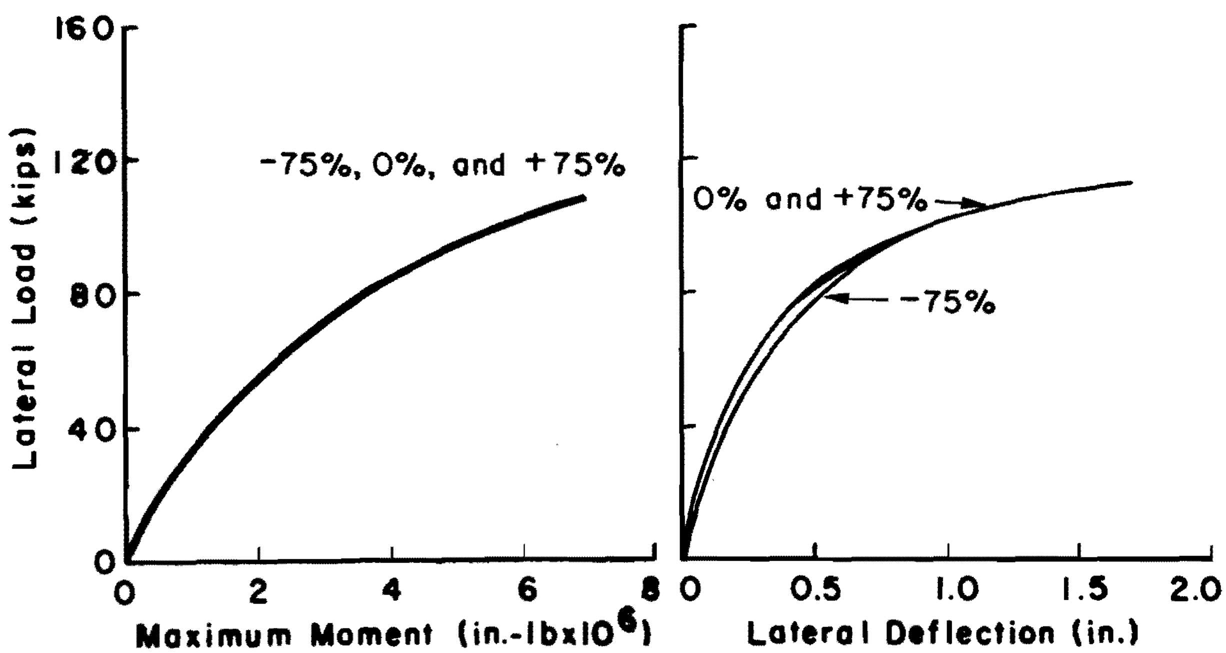


Fig. 3.8. Comparison between results for ±75% variation in k_s using StiffB criteria for static loading.

The results of the analyses using the cyclic StiffB criteria are plotted in Figs. 3.9 and 3.10. A striking difference between the results obtained using the cyclic and static criteria is the large reduction in pile capacity for the case of cyclic loading. Comparing the 0% curves in Figs. 3.6 and 3.9, the ultimate lateral load, using cyclic criteria, is approximately 50% less than the ultimate lateral load, using static criteria.

Sand Criteria

The results of the analyses using the static sand criteria are shown in Figs. 3.11, 3.12, 3.13, 3.14. The results of the analyses for a $\pm 25\%$ variation in ϕ are shown in Fig. 3.11. The variation in ϕ has a greater effect on the lateral deflection than on the maximum moment. The lateral deflection is very sensitive to variations in ϕ. The $\pm 25\%$ variation was selected to illustrate clearly the effect of ϕ on the results of the analysis, but this parameter can generally be obtained with much greater accuracy.

The results of the analyses for a $\pm 25\%$ variation in γ are shown in Fig. 3.12. The unit weight has a larger effect on the behavior of piles in sand than on the behavior of piles in clay. Variations in γ had a small effect on the maximum moment, but a larger effect on the lateral deflection. The effect of variations in γ are less than the effect of variations in ϕ. Because γ is generally known within $\pm 10\%$, the analysis would be only moderately sensitive to normal variations in γ.

The results of the analyses for a $\pm 50\%$ variation in K_o are shown in Fig. 3.13. A variation of $\pm 50\%$ was selected because the pile capacity did not appear to be sensitive to changes in this parameter. As shown, almost no change in the maximum moment occurred due to a 50% change in K_o.

However, some difference in lateral deflection resulted, but the difference between the 0% curve and the $\pm 50\%$ curve is small enough that the analysis could be considered to be insensitive to small changes in K_o.

The results of the analyses for a $\pm 50\%$ difference in k are shown in Fig. 3.14. As shown, no discernable difference resulted between maximum moments for a $\pm 50\%$ change in k. The 50% decrease in k caused a difference

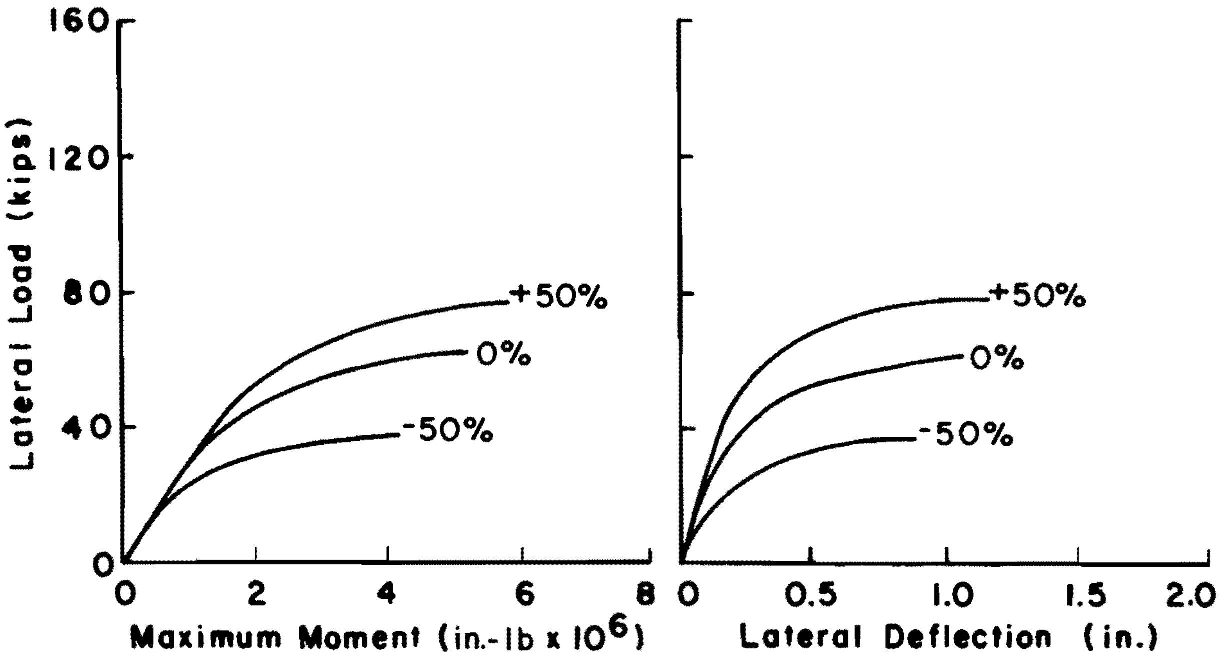


Fig. 3.9. Comparison between results for $\pm 50\%$ variation in c using StiffB criteria for cyclic loading.

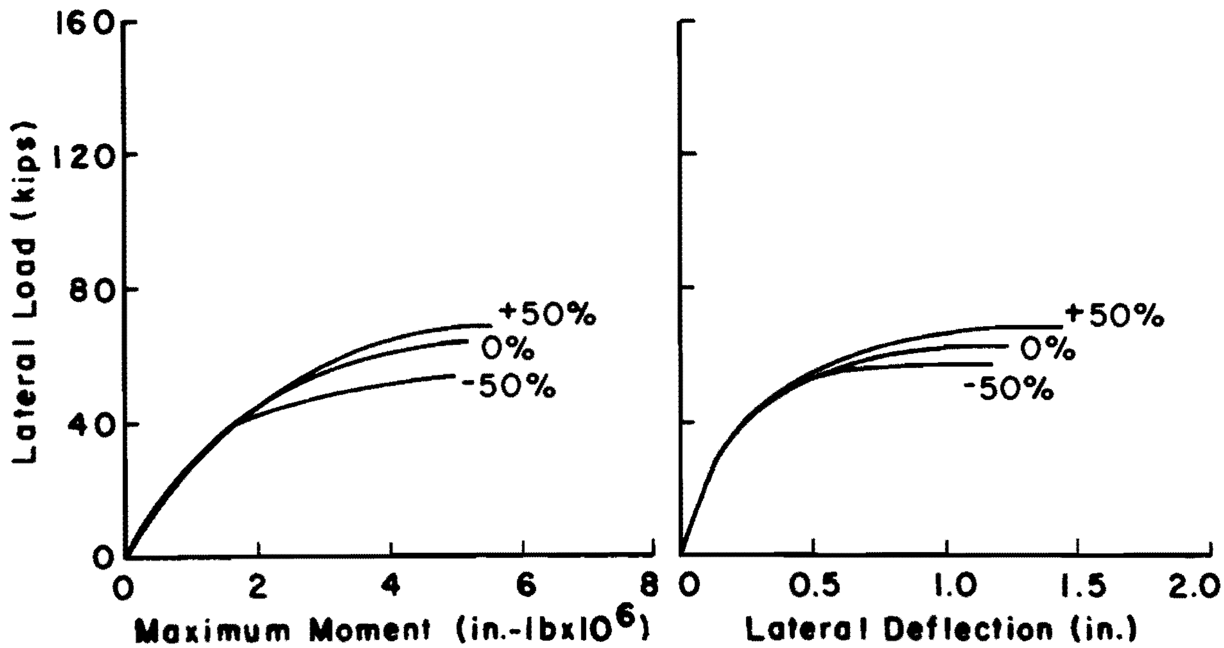


Fig. 3.10. Comparison between results for $\pm 50\%$ variation in ε_{50} using StiffB criteria for cyclic loading.

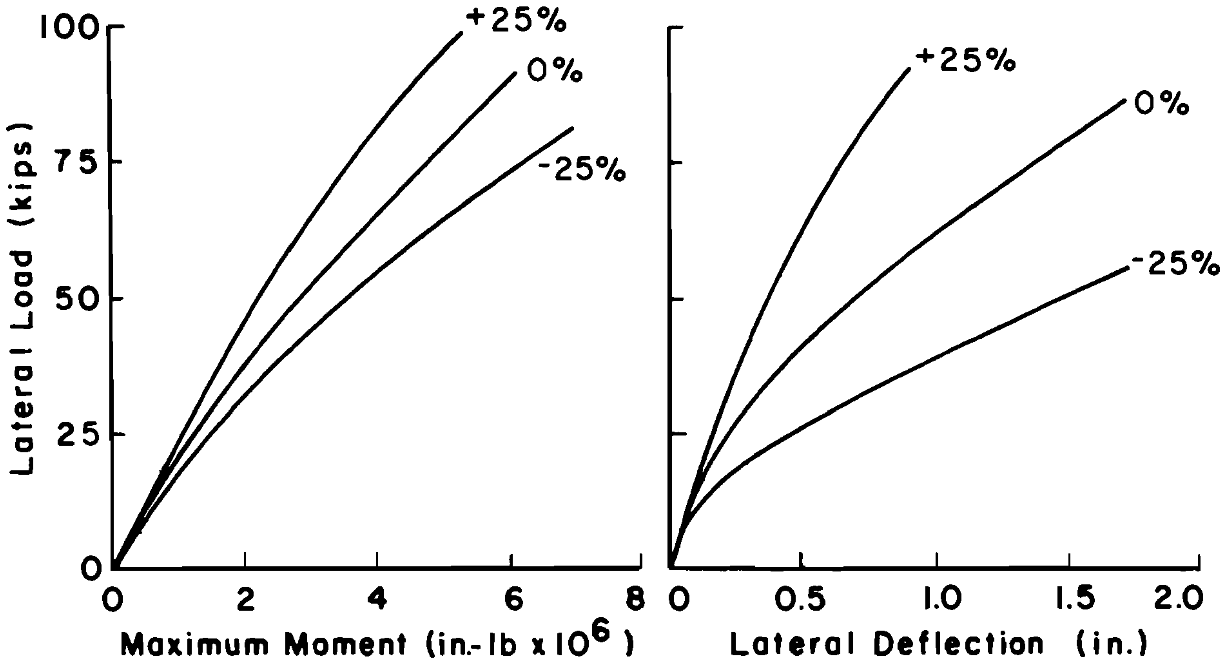


Fig. 3.11. Comparison between results for ±25% variation in ϕ using sand criteria for static loading.

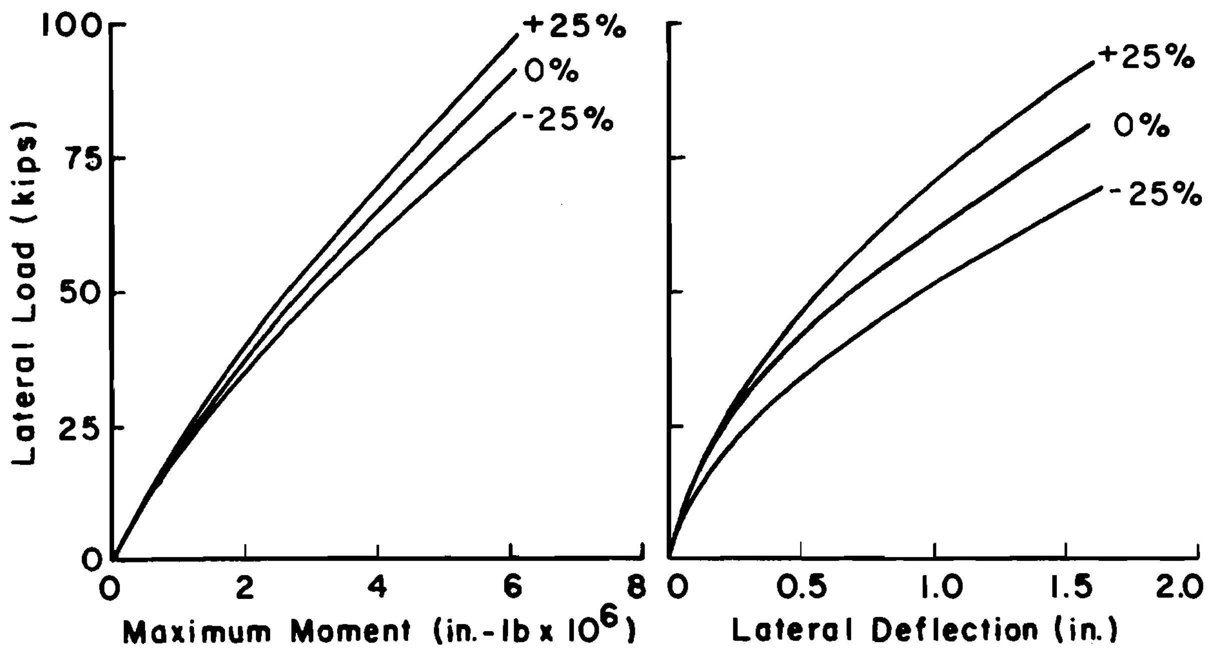


Fig. 3.12. Comparison between results for ±25% variation in γ using sand criteria for static loading.

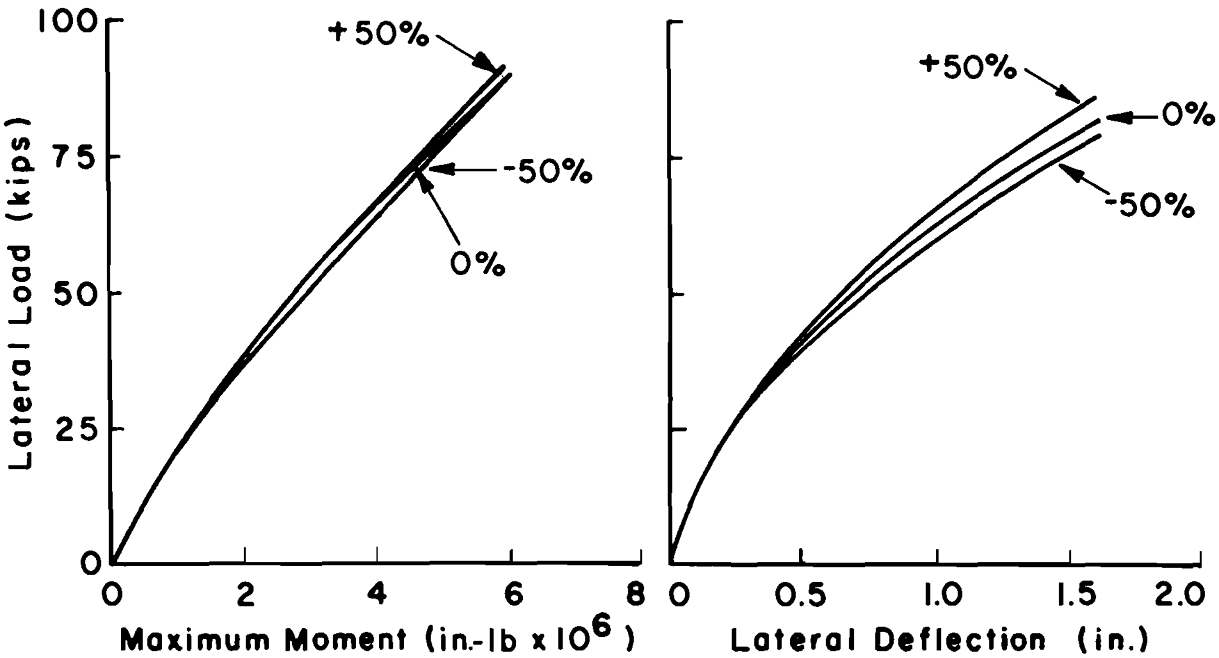


Fig. 3.13. Comparison between results for ±50% variation in K_o using sand criteria for static loading.

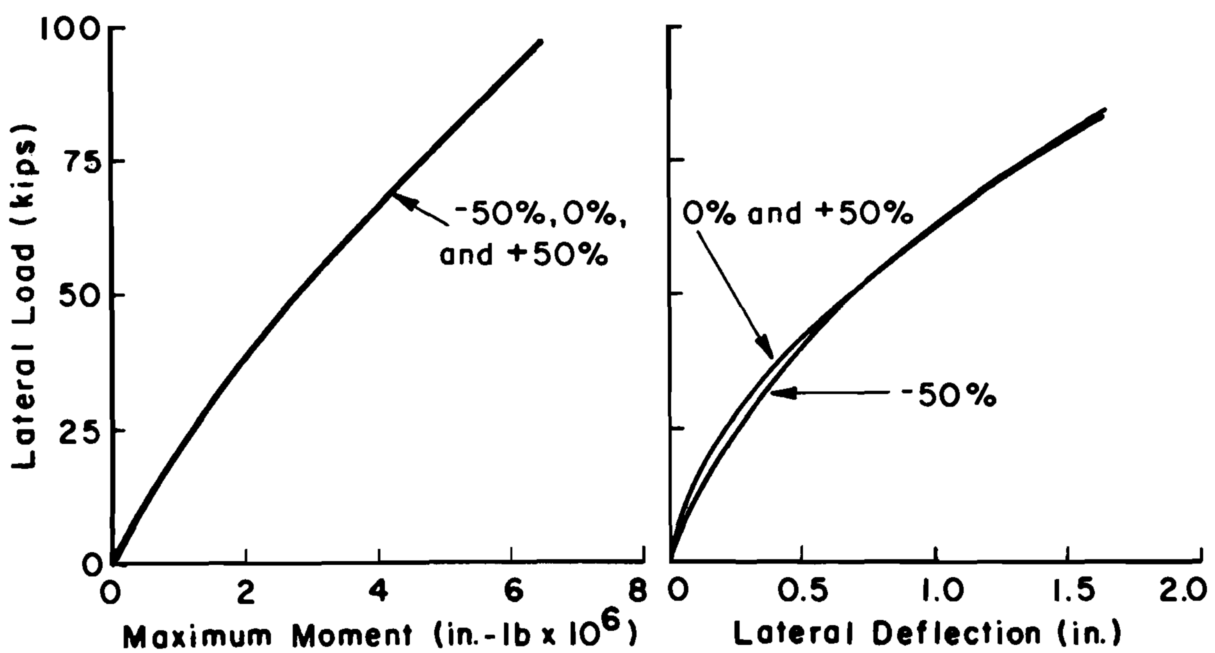


Fig. 3.14. Comparison between results for ±50% variation in k using sand criteria for static loading.

in the initial portion of the load-deflection curve, but the difference between the 0% curve and the -50% curve is very small for larger loads. Increasing k by 50% had practically no effect on the lateral deflection.

The results of the analyses using the cyclic sand criteria are plotted in Figs. 3.15 and 3.16. The comparison of the 0% curves for cyclic and static loading indicates that cyclic loading does not have a very large effect on the pile behavior. The maximum moments and the lateral deflections increase by 20 and 25%, respectively. The influence of variations in ϕ and γ on the pile behavior for cyclic loading are similar to that obtained for static loading. In obtaining p-y curves for sand, the angle of internal friction is definitely the most important parameter. As shown in Fig. 3.15, a decrease in ϕ has a much larger influence on the pile behavior than does a corresponding increase in ϕ.

VARIATION OF PILE PROPERTIES

The analyses described in the following paragraphs were performed using the static StiffA criteria and the soil properties in Table 3.1. The results of the analyses are plotted in Figs. 3.17 and 3.18 for variations in EI and in Fig. 3.19 for variations in L.

Two separate cases were used to determine what effect variations in EI had on the pile behavior. In the first case, the pile was given a sufficient depth of embedment, L, so that it would behave as a flexible member. A flexible member is defined as a member which has at least two points of zero deflection along its elastic shape when the member is loaded to the maximum lateral load. In the second case, the depth of embedment was short enough so that there was only one point of zero deflection. To be classified as rigid, the pile can have a change in slope, but the sign of the slope along its deflected shape must remain the same.

The results of the analyses for a $\pm$50% variation in EI for the flexible pile are shown in Fig. 3.17. As shown, the effects of increasing EI are to increase the maximum moment and to decrease the lateral deflection. The maximum moment is not very sensitive to variations in EI, but the

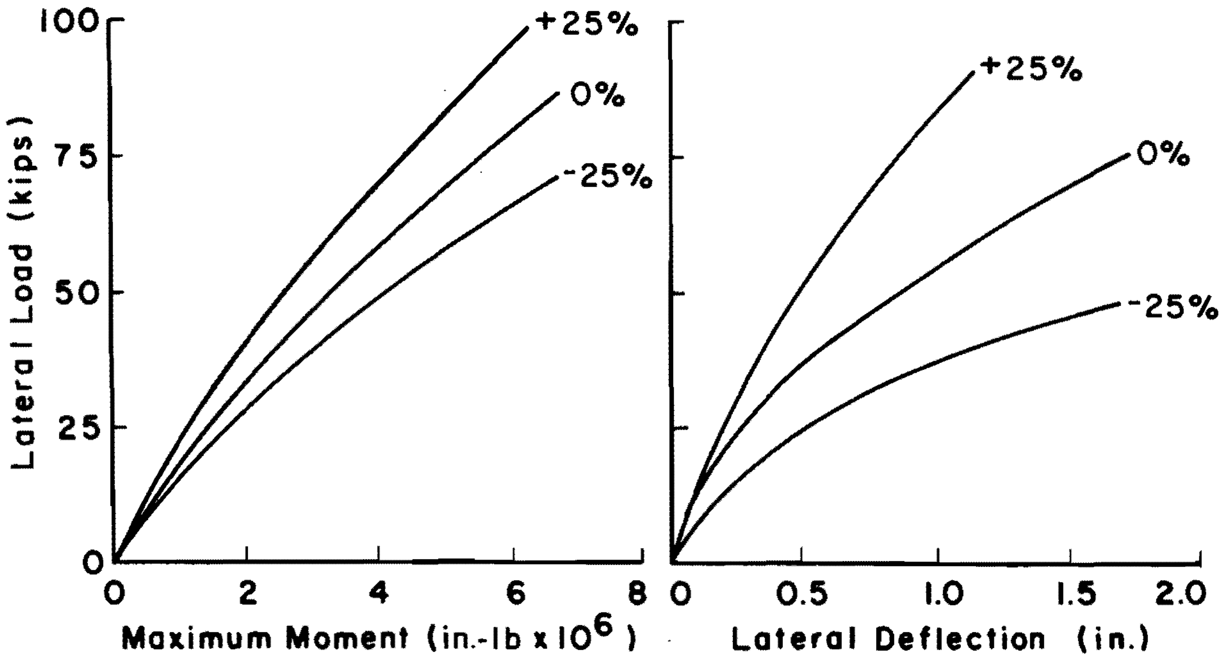


Fig. 3.15. Comparison between results for $\pm 25\%$ variation in ϕ using sand criteria for cyclic loading.

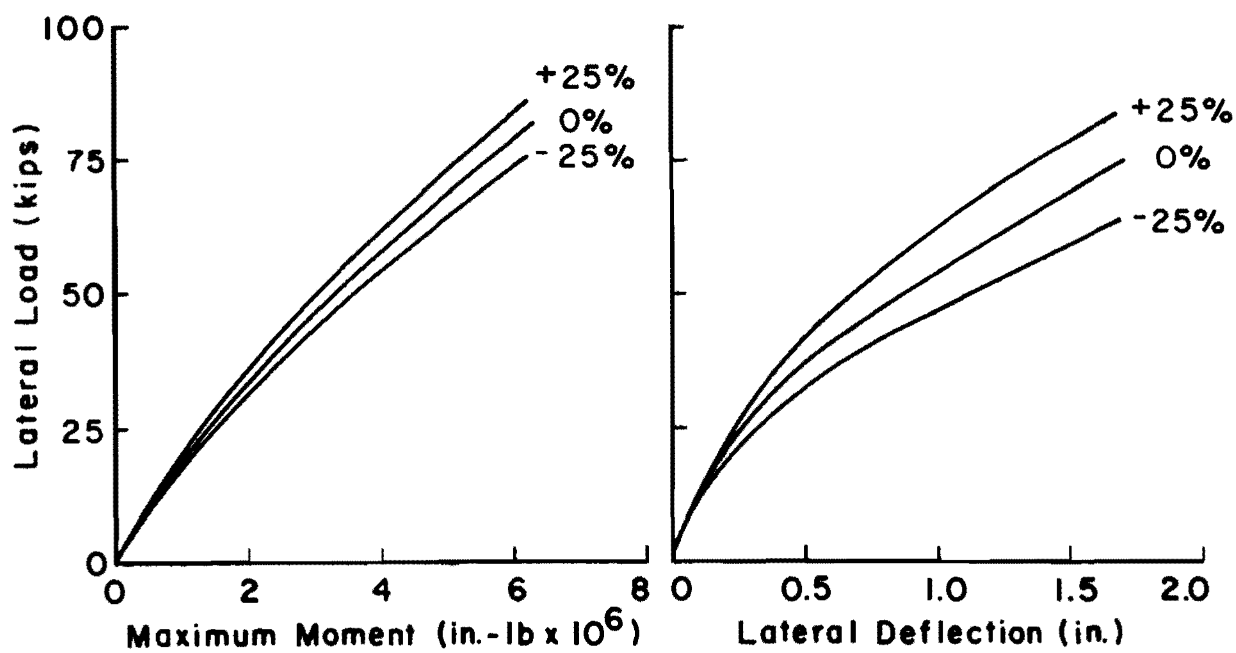


Fig. 3.16. Comparison between results for $\pm 25\%$ variation in γ using sand criteria for cyclic loading.

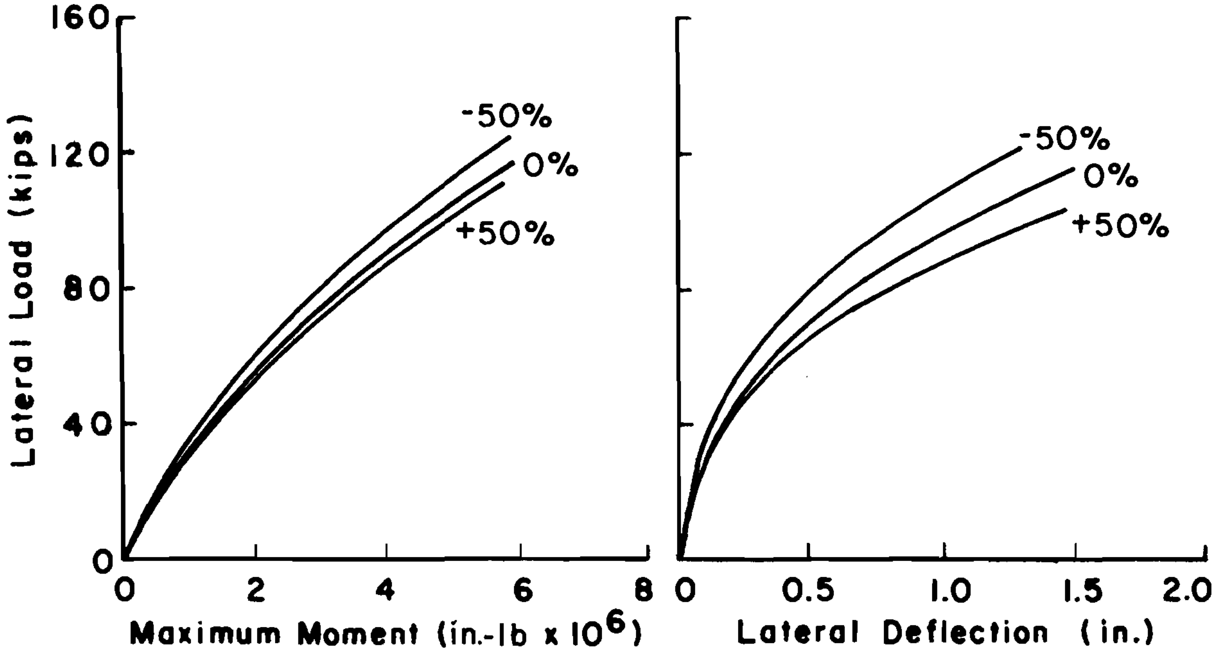


Fig. 3.17. Comparison between results for ±50% variation in EI for a flexible pile using StiffA criteria for static loading.

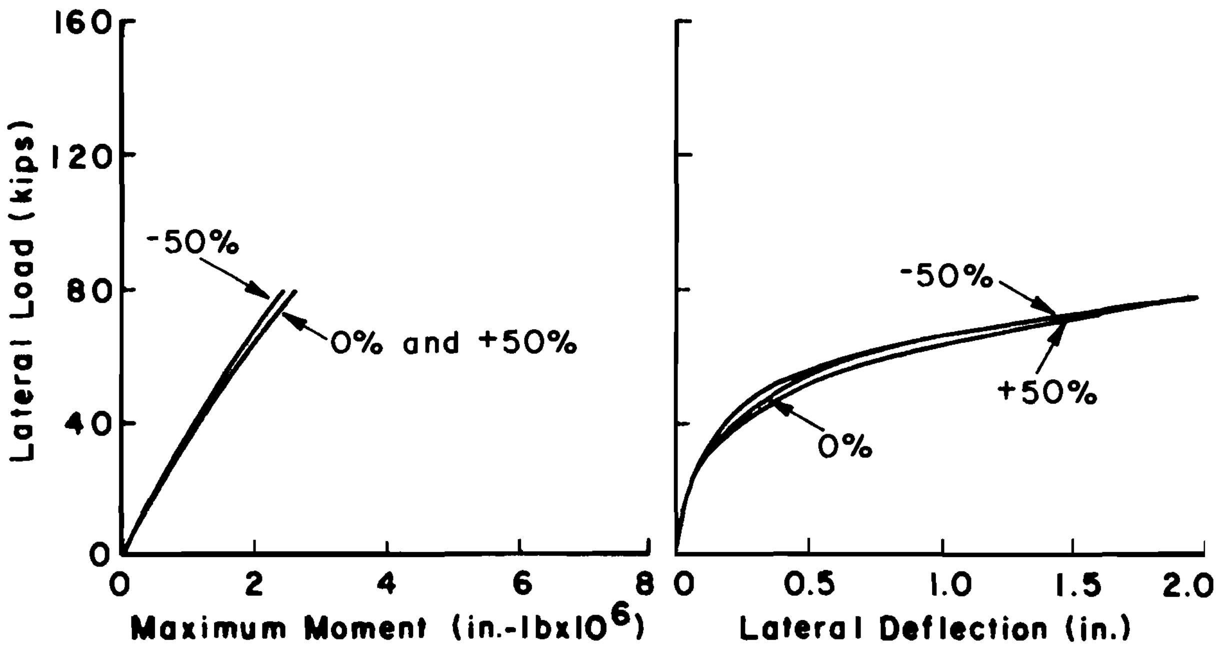


Fig. 3.18. Comparison between results for ±50% variation in EI for a rigid pile using StiffA criteria for static loading.

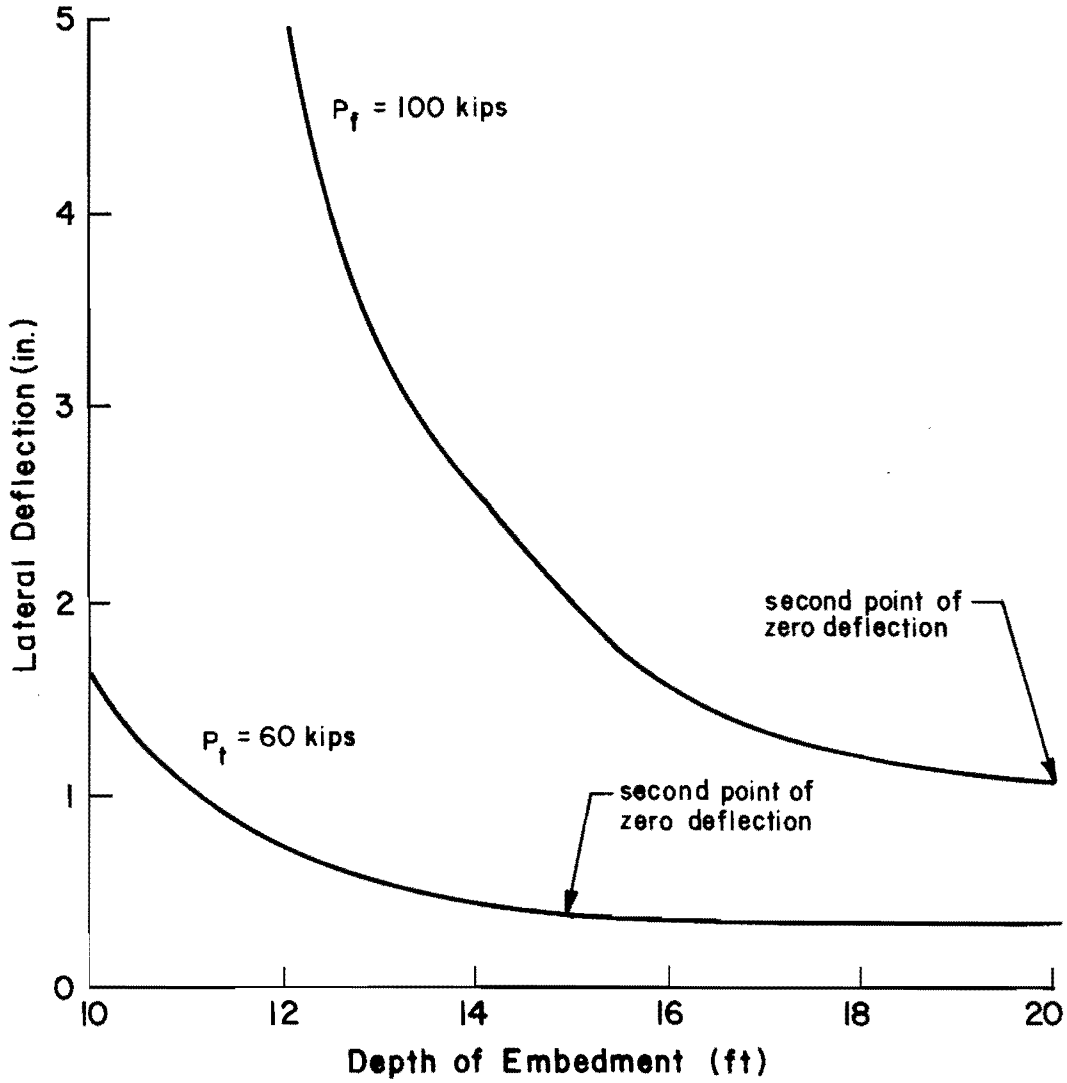


Fig. 3.19. Effect of depth of embedment on lateral deflection.

lateral deflection is somewhat more sensitive. For a lateral load of 80 kips, the 50% decrease in EI caused a 53% increase in lateral deflection, but the 50% increase in EI only caused a 23% reduction in the lateral deflection. Therefore, the pile behavior is more sensitive to decreases than to increases in EI.

The analyses for the case of the rigid pile were performed using an L of 12 ft. All other properties were the same as in the flexible pile case. The results of the analyses for a $\pm$50% variation in EI are shown in Fig. 3.18. As shown, variations in EI have a much smaller effect on pile behavior in the case of a rigid pile as opposed to a flexible pile. The most noticeable difference in behavior due to a reduction in L is the large reduction in pile capacity. Reducing L to 12 ft caused a 40% reduction in P_t at a deflection of 2 in. The maximum moment is reduced due to a reduction in L, but this is not really beneficial because the full capabilities of the shaft to resist large moments can probably not be utilized.

The effect of variations in L on the lateral deflection, plotted in Fig. 3.19, has been investigated for lateral loads of 50 and 100 kips. As shown, increasing L past the second point of zero deflection had no effect on the lateral deflection. For loads up to 100 kips, only a 20 ft depth of embedment is needed for the pile to behave as a flexible member using the static StiffA criteria. Decreasing L less than the length corresponding to the second point of zero deflection has a large effect on the lateral deflection. By increasing L to the second point of zero deflection, the lateral deflection can be reduced substantially, and the structural capacity of the shaft can be fully utilized.

CHAPTER 4. ANALYSIS OF LATERAL LOAD TESTS FOR PILES IN CLAY

INTRODUCTION

This chapter analyzes the results of well-documented, lateral load tests for piles in clay. The purpose of these analyses is to determine the ability of the p-y criteria, presented in Chapter 2, to predict accurately the behavior of these test piles. When possible, the analyses will be referred to by the location of the load tests.

The analyses were performed using the information reported for each load test. In some instances, all of the soil information needed to perform the analyses was not presented, and the required information was obtained using methods discussed in the following section.

As a part of Project 244, a load test on drilled shafts was performed in San Antonio. The results of that test will not be analyzed in this report but will be considered in detail in the final report on Project 244.

SOIL PROPERTIES

The soil properties necessary to perform an analysis are: c, ε_{50}, γ, and k. The results of the analyses presented in Chapter 3 indicated that the lateral deflection is sensitive to variations in c. Incorrect assessment of ε_{50} will also produce errors in the analysis, but the errors associated with variations in ε_{50} are much smaller than the errors associated with variations in c. The results of the analyses presented in Chapter 3 indicated that variations in γ and k have little effect on the pile behavior.

The undrained shear strength, c, was reported for all of the load tests which were analyzed. The types of tests used to obtain c were: the unconfined compression test (U), the unconsolidated-undrained triaxial test (UU), the consolidated-undrained triaxial test (CU), and the in-situ vane shear test. The test used most often to develop the p-y criteria presented in Chapter 2 was the UU test. Because the UU test was used to develop the criteria, this test should be employed to obtain c. Generally, the U test yields a c which is too low, and both the CU test and the in-situ vane shear tests yield strengths which are too high. The errors associated with using these other tests could not be determined. In the following analyses, no correction was made due to the type of test which was employed to obtain c.

In some instances, the value for ε_{50} was reported, but generally ε_{50} was obtained through correlations with c. The value of ε_{50} depends on a number of parameters; however, c appears to have the largest influence on ε_{50}. The correlation between c and ε_{50}, which were used in the analyses, are shown in Table 4.1 (Sullivan, 1977).

The unit weight of the clay was either reported or computed based on the natural moisture content. In cases where γ had to be computed, a degree of saturation of 100% was assumed. In cases where no information was given concerning γ, a value was assumed.

The parameter k cannot be measured in the laboratory and is usually obtained through correlations with c. Reese, et al.,(1975) recommended values of k as a function of c for both static and cyclic loading. Sullivan (1977) recommended values of k as a function of c only. The studies presented in Chapter 3 indicated that variations in k have little effect on the results of the analyses. Therefore, it was judged appropriate to relate k to c and not to differentiate between the types of loading. The values of k recommended by Sullivan, shown in Table 4.2, were used in the analyses that follow.

PILE PROPERTIES

The pile properties which are necessary to perform an analysis are: EI, L, and b. The stiffness of the pile can be computed for homogeneous materials such as steel, but is uncertain for composite materials such as reinforced concrete. For a reinforced concrete pile, EI is a function of the bending moment. Because the bending moment in the laterally loaded pile is also a function of EI, the problem of analyzing a reinforced concrete pile is complex. In the few cases in this chapter where a reinforced concrete member was used, the properties of concrete and the placement of steel were not reported with sufficient accuracy to warrant the use of a sophisticated method to obtain EI. In lieu of a more sophisticated method, the following equations (ACI 318-71) were used to obtain values of EI that

TABLE 4.1. RECOMMENDED VALUES FOR ϵ_{50}

c (lb/ft^2)	ϵ_{50} (%)
250 - 500	2
500 - 1000	1
1000 - 2000	0.7
2000 - 4000	0.5
4000 - 8000	0.4

TABLE 4.2. RECOMMENDED VALUES OF k FOR CLAY

c (lb/ft^2)	k (lb/in^3)
250 - 500	30
500 - 1000	100
1000 - 2000	300
2000 - 4000	1000
4000 - 8000	3000

were approximately correct:

$$EI_1 = \frac{E_c I_g}{2.5} \tag{4.1}$$

or

$$EI_2 = \frac{E_c I_g}{I_s} + E_s I_s \tag{4.2}$$

where

E_c = Young's modulus for concrete, taken as $57\sqrt{f_c'}$ (kips/in.^{2}),

I_g = moment of inertia for gross concrete section,

E_s = Young's modulus for steel,

I_s = moment of inertia of reinforcement,

f_c' = compressive strength of concrete (lb/in.^{2}).

Ferguson (1973) states that the results obtained by using either of these equations is conservative. Therefore, the larger EI from Eq. 4.1 or 4.2 was selected for the analyses of reinforced concrete members.

It was shown in Fig. 3.19 that increasing L past the second point of zero deflection has no effect on the results of the analyses. Therefore, for reported tests on long piles, an L which was less than that reported was sometimes used in the analyses. This reduction in L was beneficial, because fewer nodes had to be used in making computer solutions.

ANALYSIS OF TEST RESULTS

Bagnolet

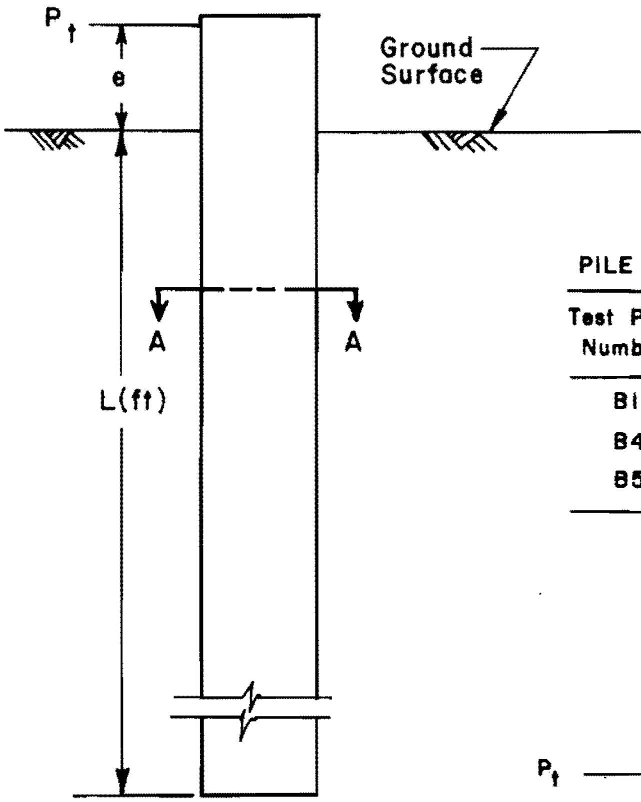
Kerisel (1965) reported the results of short-term, static lateral load tests performed on three closed ended "bulkhead caissons." The bulkhead caissons, shown in Fig. 4.1, were formed by welding two sheet pile sections together to form a single member. The three test members had the same EI, but different boundary conditions and depths of embedment. The pile heads were free to rotate, but the vertical eccentricity, e, was different. The caissons were installed by pushing them vertically with a jacking system. The test setup and pile properties are shown in Fig. 4.1.

The tests were performed east of Paris, France, in a fairly uniform deposit of stiff clay. The StiffA criteria were used because the water table was below the pile tips. All of the soil properties were reported except for ε_{50}. The value of ε_{50} was obtained from Table 4.1 based on the reported shear strength. The soil properties used in the analyses are presented in Fig. 4.1.

The measured and computed results for test piles B1 and B4 are compared in Fig. 4.2. As shown, the measured maximum moments and the computed maximum moments are in good agreement for test pile B1 and in fair agreement for test pile B4. However, there appeared to be some scatter in the reported maximum moments for test pile B4. The measured deflections and the computed deflections are in fair agreement for both test piles.

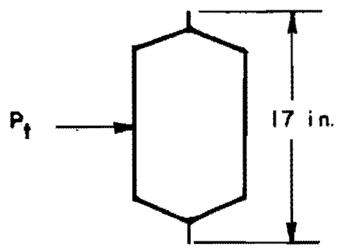
The measured and computed results for test pile B5 are compared in Fig. 4.3. As shown, the measured and computed maximum moments are in excellent agreement for all but the smallest lateral load. The measured and computed deflections agree favorably. The largest error of 36% in the computed deflection occurred at a lateral load of 17.6 kips.

The depth of embedment appears to have influenced the behavior of test pile B5. The other two test piles had sufficient depths of embedment so that they behaved as flexible members, but test pile B5 behaved as a rigid member. The slope of the computed load-deflection curve at large loads was less for test pile B5 than for test piles B1 and B4 even though the applied moment was smaller. The results of the analyses indicate that the p-y criteria and the finite difference method of analysis were capable of predicting the behavior of a flexible pile and rigid pile.



PILE PROPERTIES AND TEST SET-UP

Test Pile Number	b (in.)	EI (lb-inch^2)	e (ft)	L (ft)
B1	17	8.88×10^9	3.3	16.7
B4	17	8.88×10^9	2.9	13.6
B5	17	8.88×10^9	2.3	8.7



SECTION A-A
Bulkhead Caisson

Soil Properties

Depth (ft)	W_n (%)	c (lb/ft^2)	ϵ_{50} (in./in.)	γ (lb/ft^3)
0	—	2050	0.005	114
13	31.5	2100	0.005	114
15.4	29	2700	0.005	114

Fig. 4.1. Information for the analysis of tests at Bagnolet.

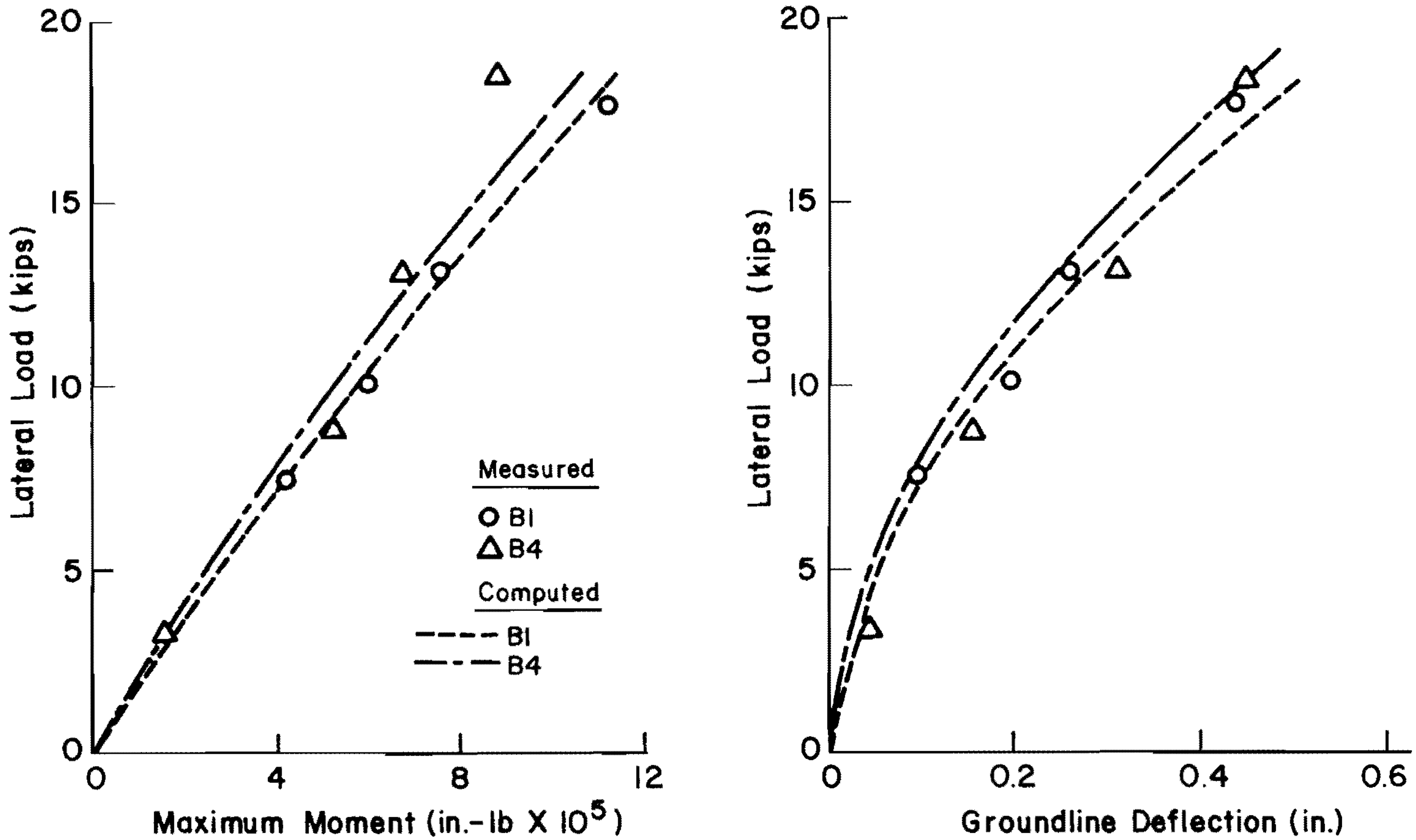


Fig. 4.2. Comparison between measured and computed results for test piles B1 and B4 at Bagnolet.

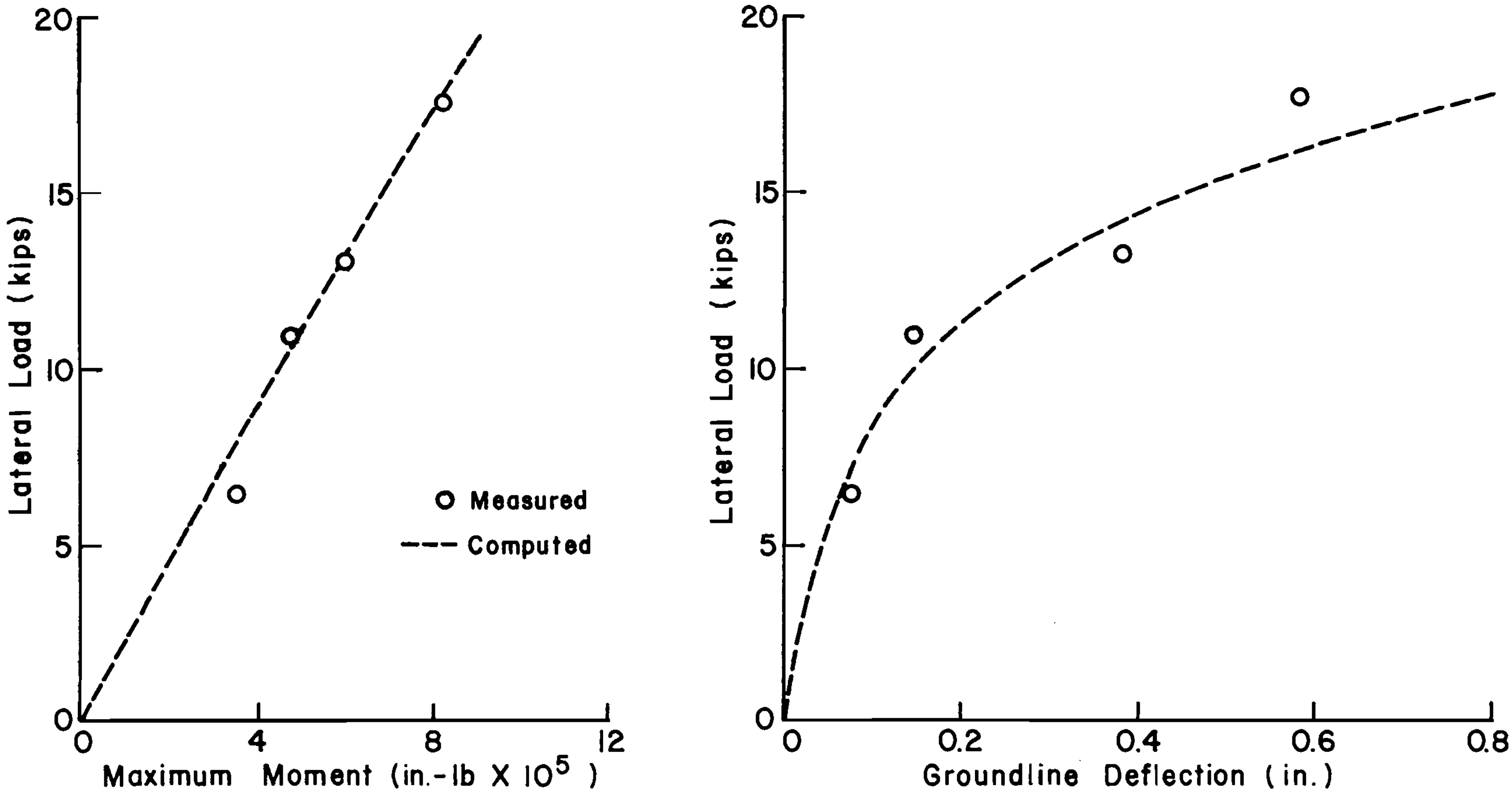


Fig. 4.3. Comparison between measured and computed results for test pile B5 at Bagnolet.

Bay Mud

Gill (1968) reported the results of eight lateral load tests performed on free-head pipe piles. Tests on piles D1, D2, D3, and D4 were performed in an area where the ground water table was at its natural depth of 7.5 ft, and the other four tests on piles F1, F2, F3, and F4 were performed in an adjacent area where water was allowed to pond for a number of days prior to driving and testing. For each series of tests, the same type of pile was used, but the diameter varied from 4.5 to 16 in. O.D. The test setup and pile properties are shown in Fig. 4.4. The loading was short-term and was not repeated.

The bay mud deposit is composed of an insensitive, slightly organic, silty clay, which is classified as CH in the Unified Soil Classification system. The liquid limit, LL, and plastic limit, PL, of the clay was 71% and 29%, respectively. The undrained shear strengths were obtained from an in-situ vane shear test which was rotated at such a rate that failure occurred in 5 seconds. The vane strength profile for both the dry and flooded sites are plotted in Fig. 4.5. Gill (1968) stated that unconfined compression tests were also performed, but no values were given. He stated that the shear strength values from unconfined compression testing were smaller and more erratic than those from the in-situ vane tests.

The measured and computed deflections for test piles D1, D2, D3, and D4 which were tested at the dry site are compared in Fig. 4.6. Computations were made using the StiffA criteria. Results for piles D1 and D2 are shown in the left-hand figure and results for piles D3 and D4 are shown in the right-hand figure. As shown, the measured deflections for both piles D1 and D2 are larger than the computed deflections. The computed deflections are unconservative by as much as 33%* for both piles D1 and D2. As may be seen, the measured deflections for piles D3 and D4 are much larger than the computed deflections. The errors in the computed deflections are very large and are approximately 160% for pile D3.

In making comparisons between computed and experimental results, the word "unconservative" is employed to indicate that computed values are less than the corresponding experimental values. The term "conservative" has the opposite meaning.

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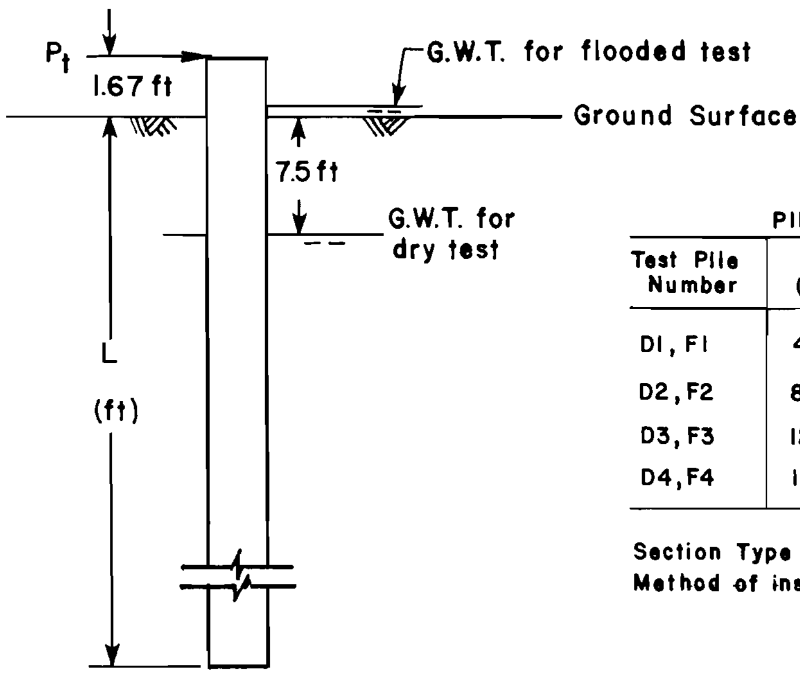
*Errors were computed using the following equation:

$$\text{error} = \frac{(\text{measured-computed})}{\text{computed}} \times 100\%$$



PILE PROPERTIES

Test Pile Number	b (in.)	EI (lb.-in.^{2})	L (ft)
D1, F1	4.75	2.17×10^8	18.2
D2, F2	8.62	2.17×10^9	20.4
D3, F3	12.75	7.46×10^9	16.7
D4, F4	16.00	1.69×10^{10}	26.7

Section Type : pipe pile
Method of installation : driven

Soil Properties

Dry Test Site			Flooded Test Site			
Depth (ft)	ϵ_{50} (%)	γ (lb/ft^3)	Depth (ft)	ϵ_{50} (%)	γ (lb/ft^3)	k (lb/in^3)
0 – 7.5	0.7	115	0–20	11.0	50	200
7.5 – 20	0.7	50				

Fig. 4.4. Information for the analysis of tests in bay mud.

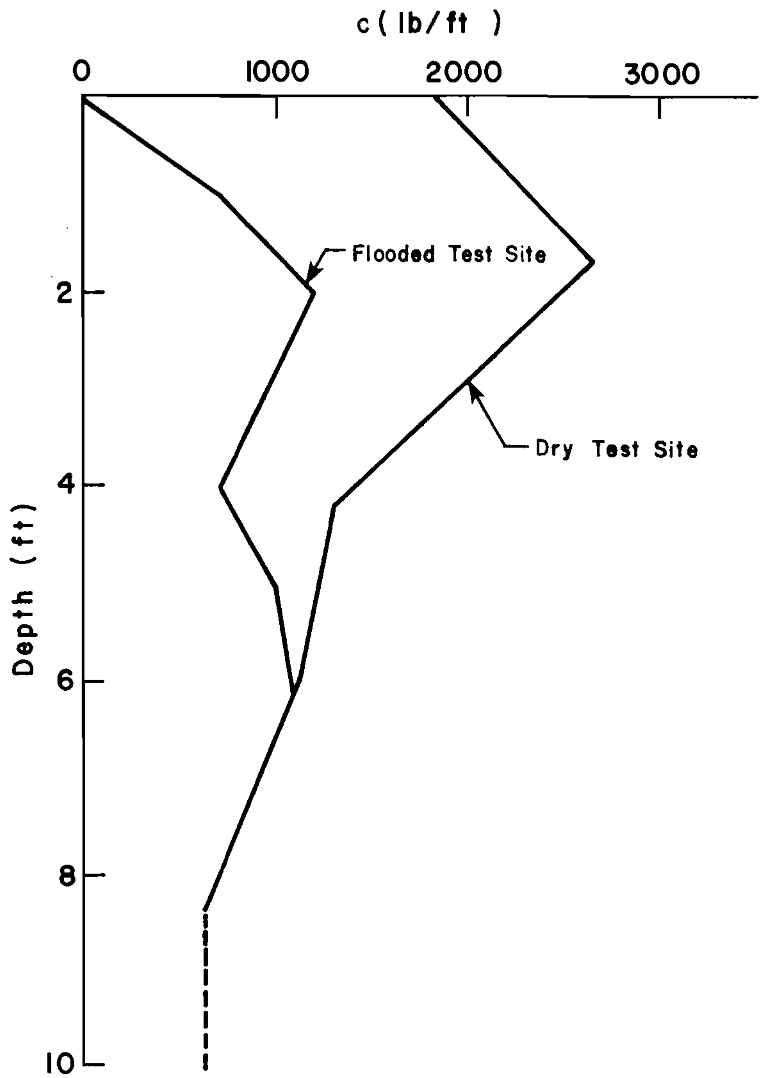


Fig. 4.5. Vane shear strength profiles for tests performed in bay mud.

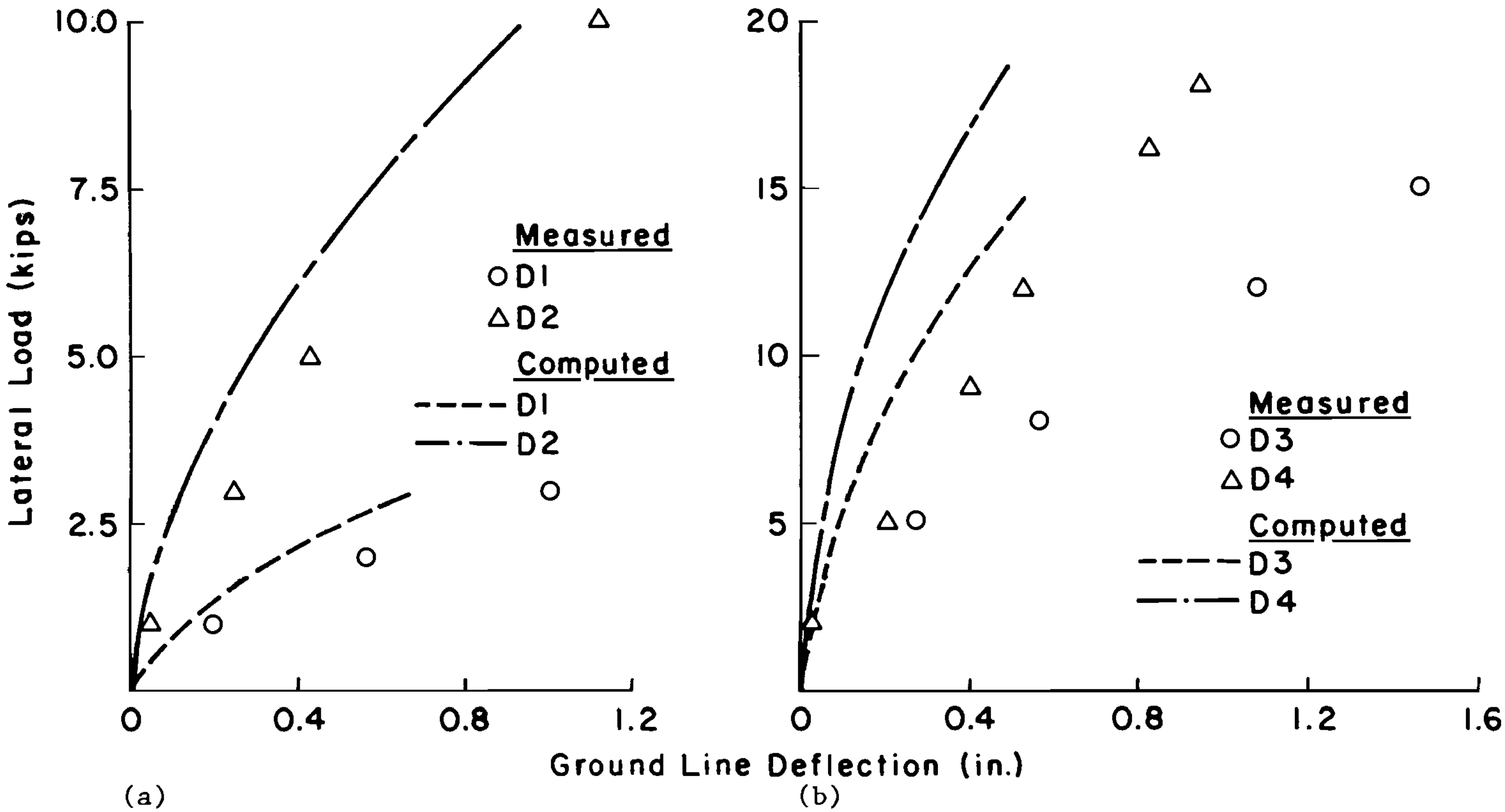


Fig. 4.6. Comparison between measured and computed deflections for tests at dry bay mud site. (a) test piles D1 and D2. (b) test piles D3 and D4.

The measured and computed deflections for test piles F1 and F2, which were tested at the flooded site, are compared in Fig. 4.7. The computations were made with the SoftB and with the Unified criteria. Results for pile F1 are shown in the left-hand figure and the results for pile F2 in the right-hand figure. As shown, the computed deflections using the SoftB criteria are fairly unconservative, but the computed deflections using the Unified criteria agree favorably with the measured deflections for pile F1 and even more favorably for pile F2.

The measured and computed deflections for test piles F3 and F4, also tested at the flooded site, are compared in Figs. 4.8a and 4.8b, respectively. The computations were made using the SoftB and Unified criteria. For the two test piles, the measured deflections exceed the computed deflections from both the SoftB criteria and the Unified criteria. A maximum error of 53% was obtained between the measured and computed deflections for test pile F3 using the SoftB criteria and a maximum error of 36% was obtained using the Unified criteria. Similar errors were obtained between measured and computed deflections for test pile F4.

Hudson River

Peck and Davisson (1962) reported the results of a short-term static lateral load test performed on a 14BP89 pile, shown in Fig. 4.9a. The pile was driven into the Hudson River 54 ft below the mud line. The lateral loads were applied 8 ft above the water level with a moment arm at the mud line of 32.5 ft. Lateral deflections were measured at the point of load application, and a Wilson Slope Indicator was used to measure the pile slope as a function of depth.

The soil below the river bottom was composed of a gray organic silt, known as Hudson River silt. The material is highly compressible, has a high water content, and a low undrained shear strength. Values of c were reported from the results of the U tests and in-situ vane shear tests. Only two values of the unconfined compressive strength were reported, but a complete profile of c was reported based on the in-situ vane tests. The results from the vane tests were used in the analyses of

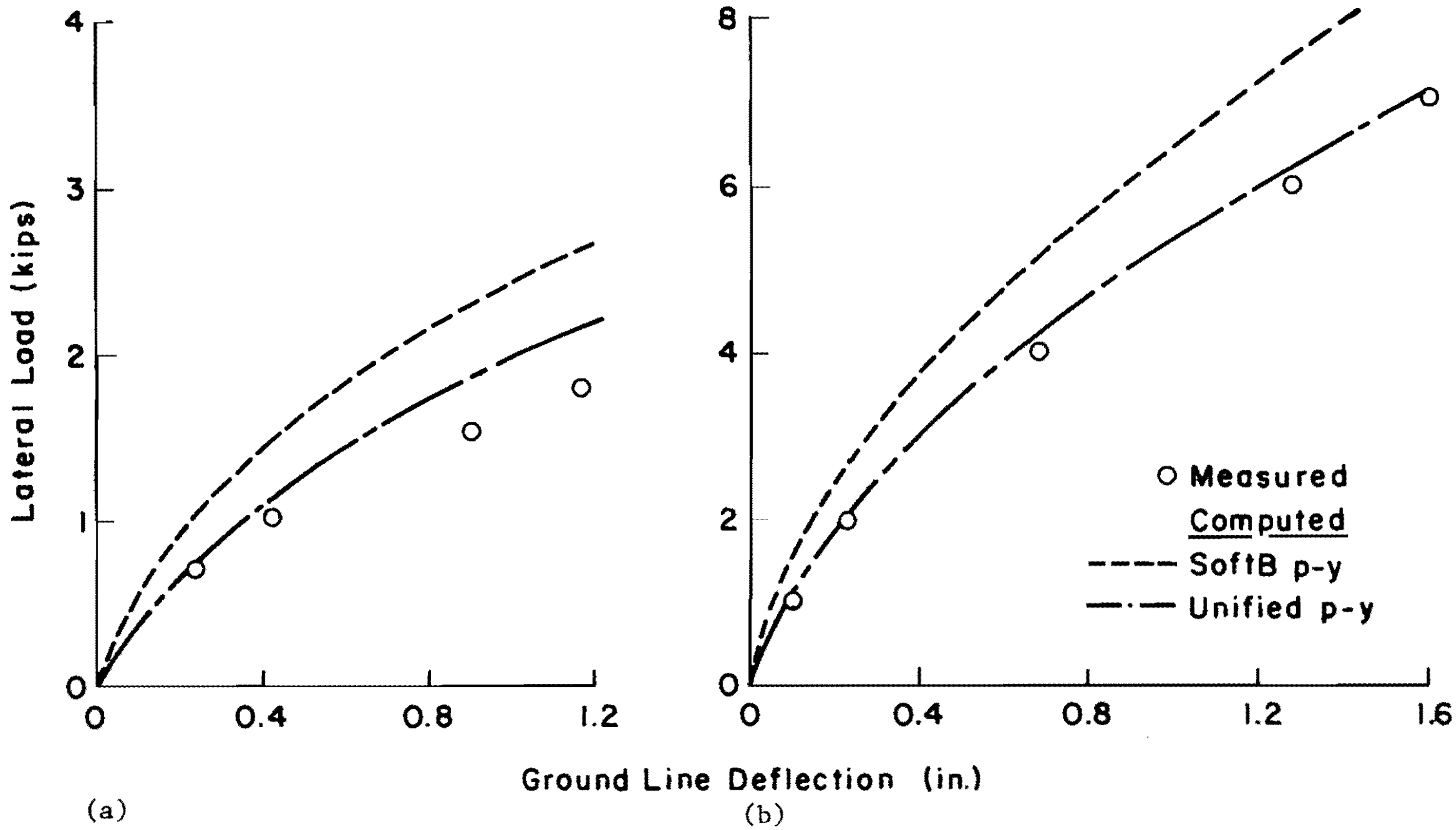
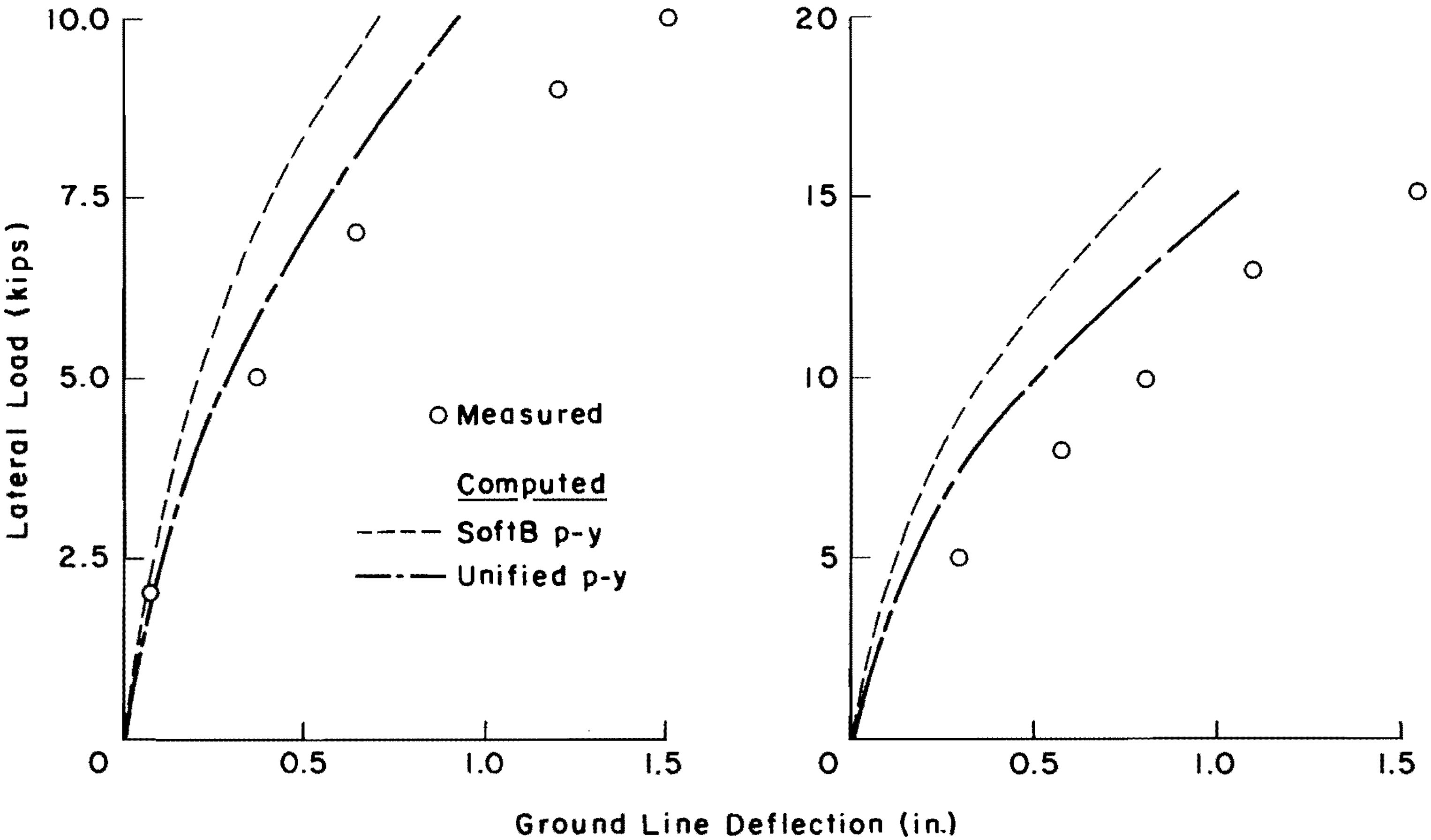


Fig. 4.7. Comparison between measured and computed deflections at flooded bay mud site. (a) test pile F1. (b) test pile F2.



(a) (b)

Fig. 4.8. Comparison between measured and computed deflections at flooded bay mud site. (a) test pile F3. (b) test pile F4.

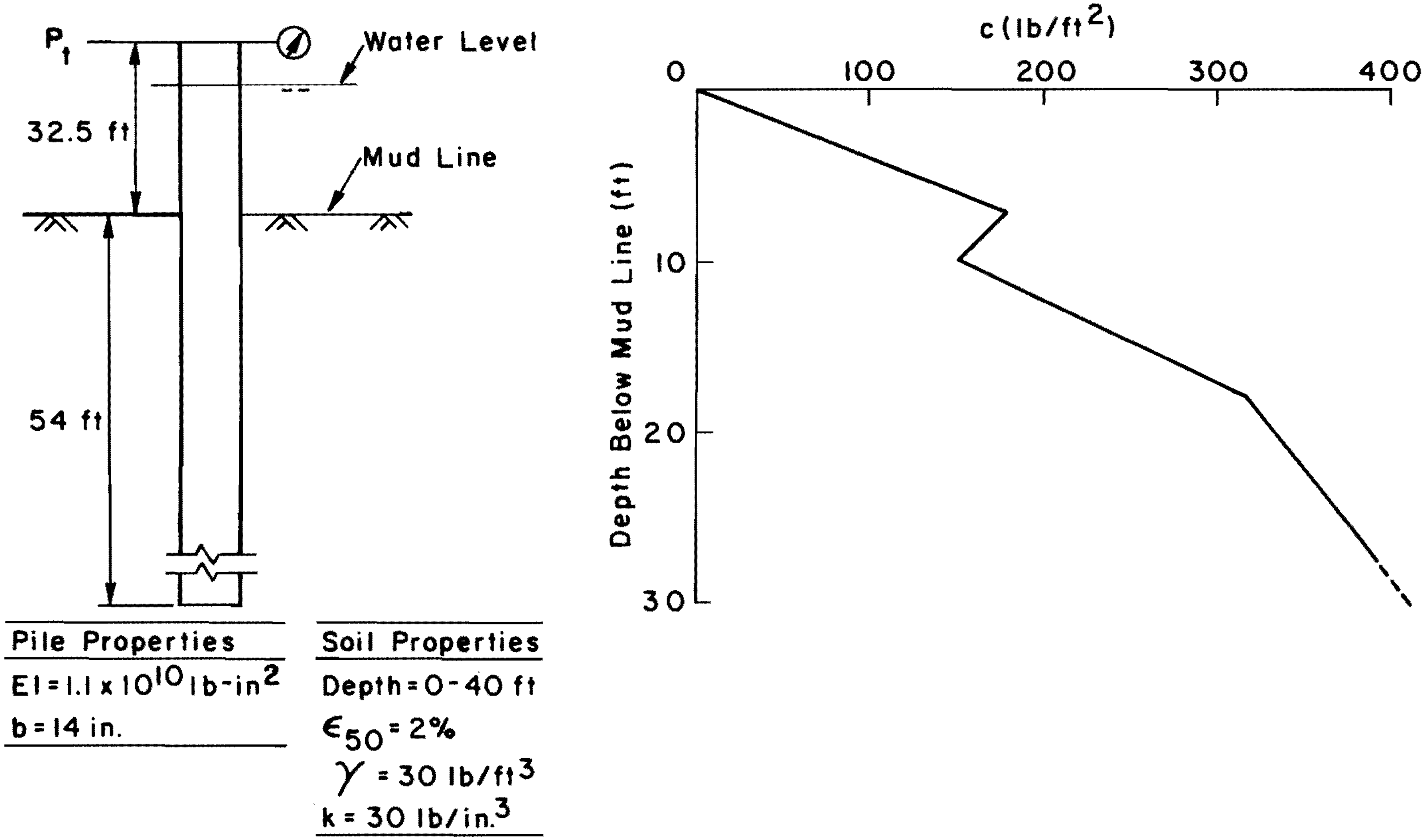


Fig. 4.9. Information for analysis of tests in Hudson River.

the test pile. The soil properties which were used in the analyses are presented in Fig. 4.9.

The measured and computed results are compared in Fig. 4.10. The computations were made using the SoftB criteria and the Unified criteria. As shown in Fig. 4.10a, results using the SoftB criteria agree favorably with the measured deflections at low load levels, but begin to diverge from the measured deflections at larger loads. The Unified criteria give results that are slightly conservative for all but the largest applied load of 3 kips.

The deflected shape of the pile, shown in Fig. 4.10b, was obtained by integrating the slope versus depth curve. The pile top deflection obtained from integration is slightly larger than the measured pile top deflection for the 1.5 kip load and is practically the same as the measured pile top deflection for the 3 kip load. The measured and computed deflected shapes, using both sets of p-y criteria, compare favorably, but better agreement was obtained using the Unified criteria.

Japanese Test

The results of short-term static lateral load tests on free-head pipe piles were reported by the Japanese Committee of Research for Piles Subjected to Earthquake (1965). A number of similar tests were performed, but only the results of test pile 3 will be discussed. The test pile, shown in Fig. 4.11, was installed by vertically jacking the closed-end pile into place with a winching system.

The soil at the site was a soft, medium to highly plastic, silty clay. Unconfined compression tests could not be performed on remolded samples due to the high sensitivity of the clay. The undrained shear strength for the deposit, shown in Fig. 4.11, was obtained from U tests. The strains at failure were generally less than 5%, and brittle fracturing was the mode of failure for the soil samples. The ε_{50} values were obtained from the reported stress-strain curves.

The measured and computed results are compared in Figs. 4.12a and

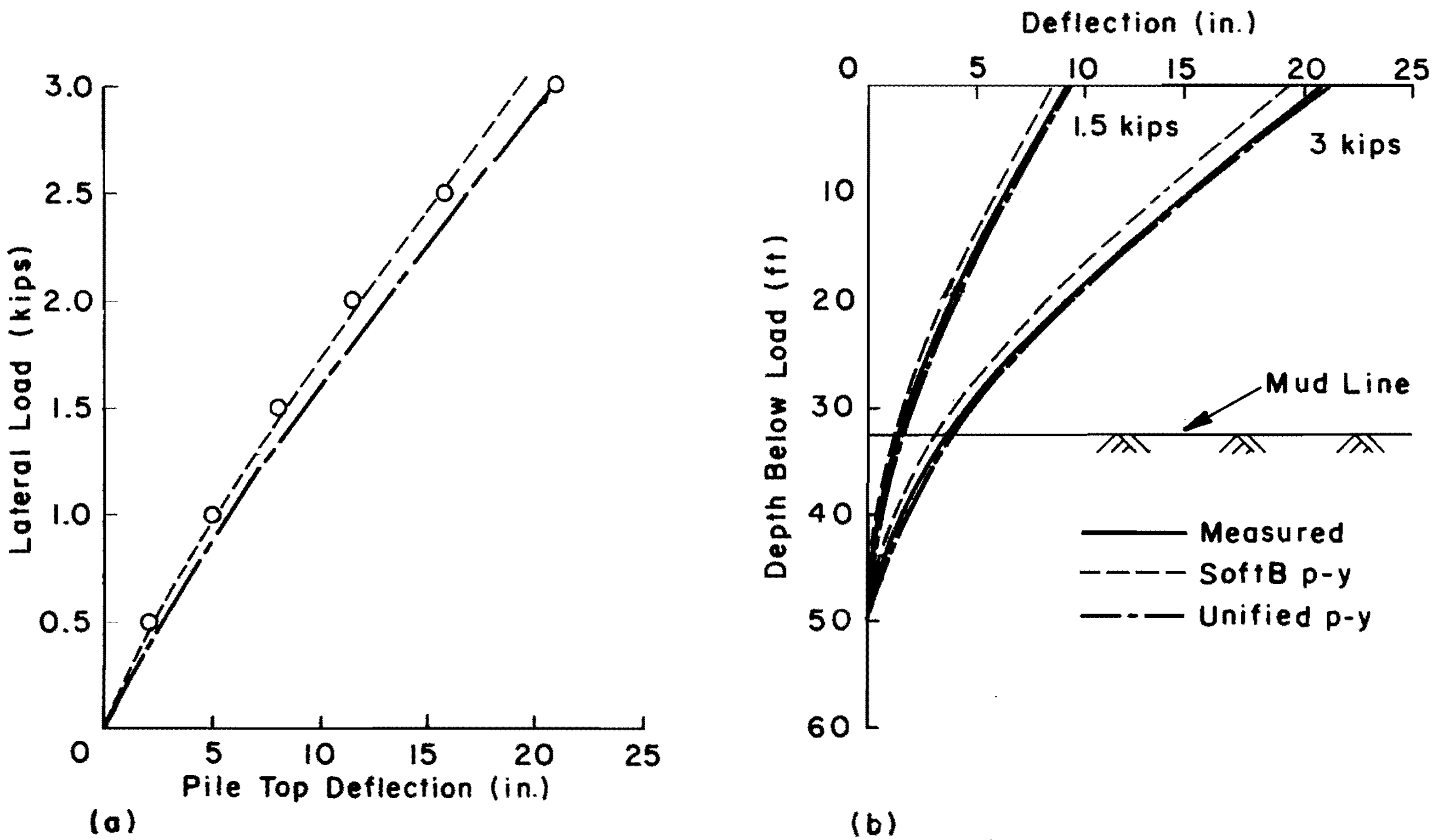
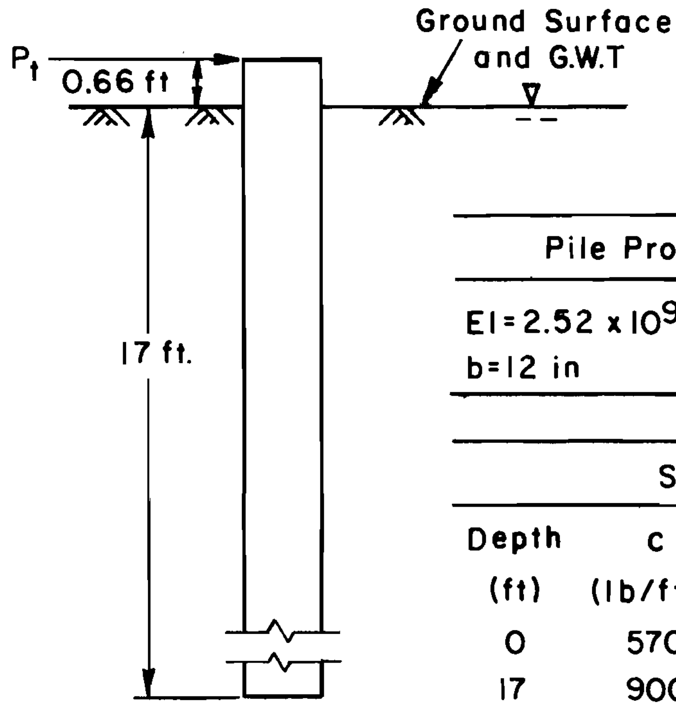


Fig. 4.10. Comparison between measured and computed results for test at Hudson River. (a) pile top deflection. (b) deflected shape.



Pile Properties
$EI = 2.52 \times 10^9$ lb-in^2
b = 12 in

Soil Properties				
Depth (ft)	c (lb/ft^2)	ϵ_{50} (%)	γ (lb/ft^3)	k (lb/in^3)
0	570	1	31	65
17	900	1	31	65

Fig. 4.11. Information for the analysis of Japanese test.

4.12b. The computations were made using the SoftB criteria and the Unified criteria. As shown, the measured maximum moments and the computed maximum moments using the SoftB criteria agree well, but the results from use of the Unified criteria are slightly conservative. The measured deflections and those computed using both sets of criteria agree favorably.

Lewisburg

Kim and Brungrader (1976) reported the results of lateral load tests performed on single piles and group piles near Lewisburg, Pennsylvania. The results on a single vertical pile, test pile 20, will be discussed. Test pile 20, shown in Fig. 4.13, was a modified 10BP49 which was driven to refusal at a depth of 41 ft. The free-head pile was axially loaded with 72 kips for the duration of the lateral load test.

The upper soil layer at the test site was a silty clay of low plasticity. The undrained shear strength was obtained from the results of U tests on undisturbed soil samples from three borings. The reported shear strengths were very erratic and difficult to interpret. Near the ground surface, values for c of 200, 1100, and 2600 lb/ft^2 were reported. The blow count from an SPT near the ground surface was in excess of 30 blows/ft. Based on the reported blow counts, a shear strength of approximately 2500 lb/ft^2 would be selected if the correlation suggested by Schmertmann (1975) was used. Also, the moisture content was at or below the plastic limit for the upper material, which indicates that the material was heavily overconsolidated. Based on these facts, the shear strength profile in Fig. 4.13 was used in the analysis. Because the clay was stiff and the water table was at a depth of 30 ft, the StiffA criteria were used.

Basically, the loading history consisted of sustaining each load increment for 30 minutes. The loads were cycled once at load levels of 25 and 33.3 kips, and were cycled twice at a load level of 16.67 kips. The effects of sustained loading could not be considered in the analysis, and the computed change in pile behavior, due to 2 or 3 cycles of loading, was small.

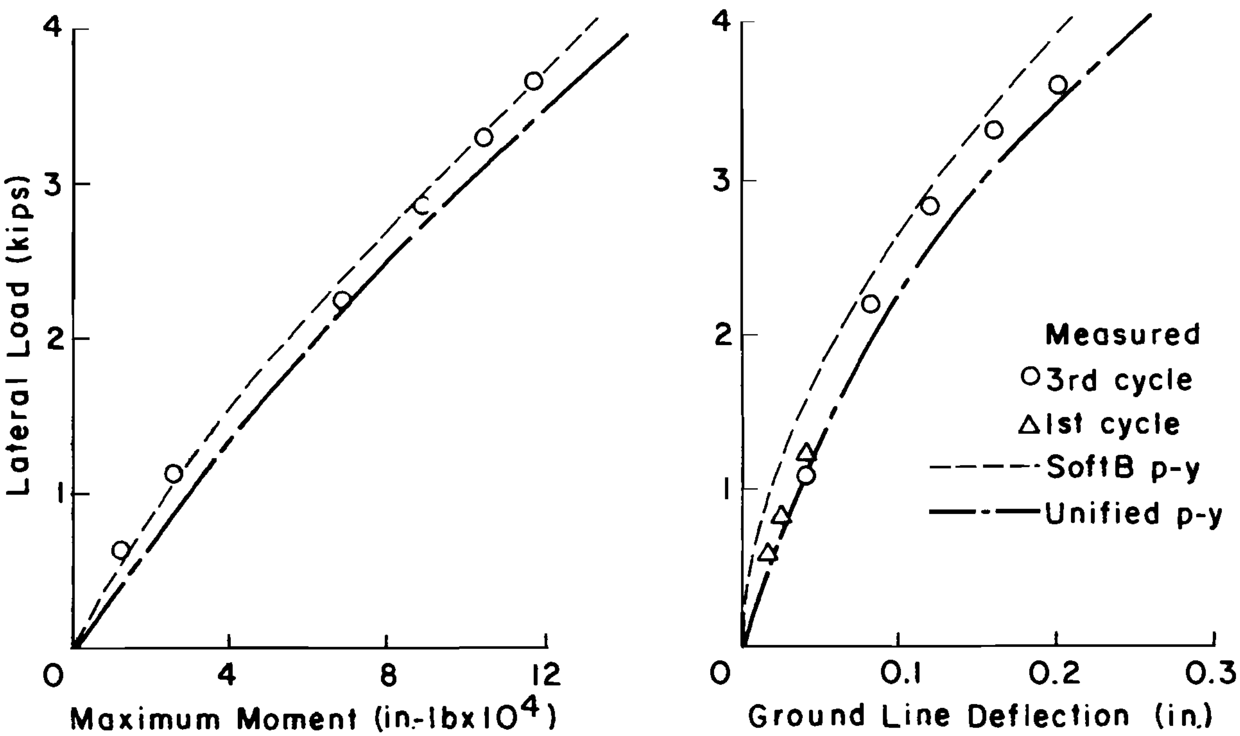


Fig. 4.12. Comparison between measured and computed results for Japanese test.

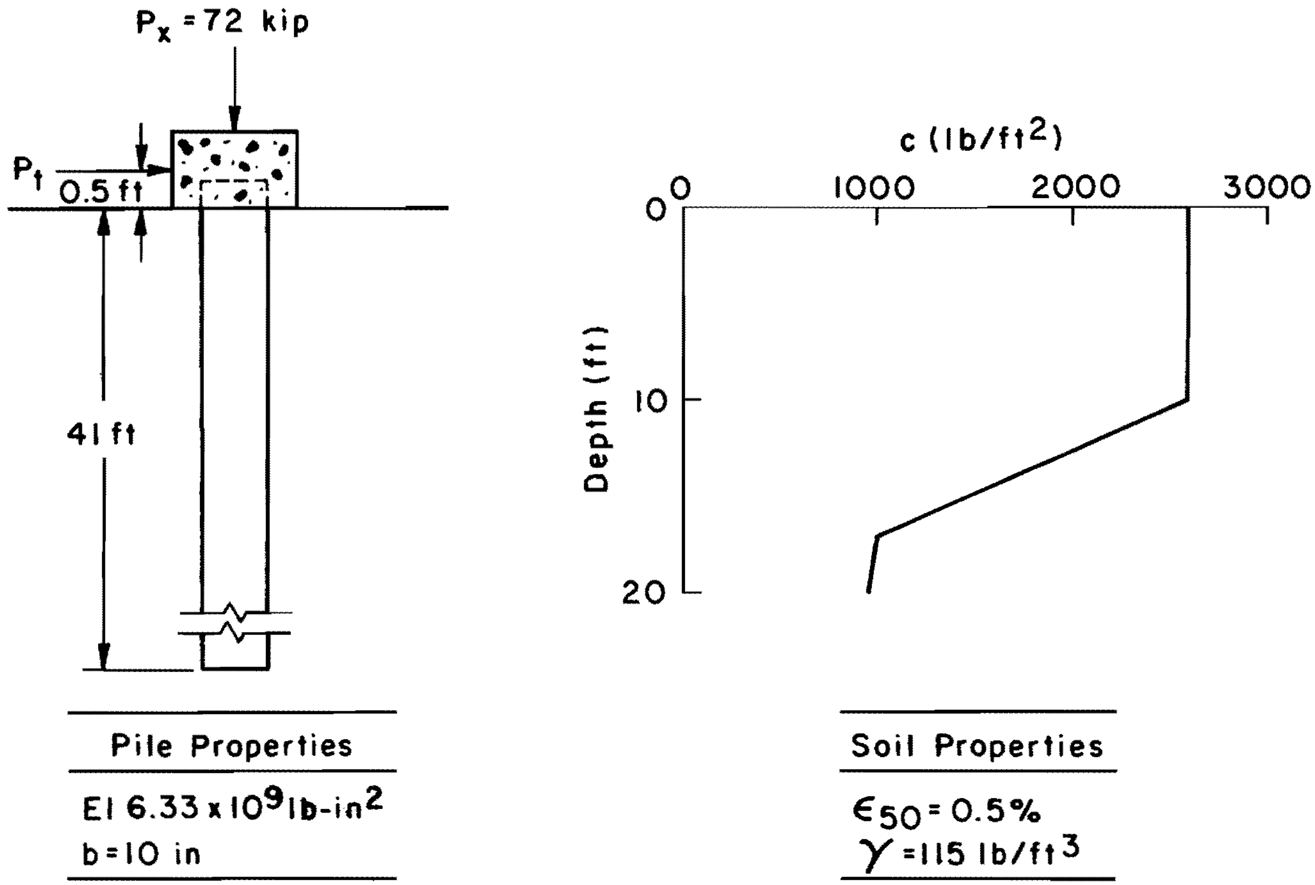


Fig. 4.13. Information for analysis of test at Lewisburg.

The measured maximum moment for the third cycle of loading was 960 in.-kips, which occurred at a depth of 45 in. For this same load, the computed maximum moment, which occurred at a depth of 48 in, was 490 in.-kips. This is an extremely large difference in maximum moments, considering that the computed deflections were only unconservative by 15% and that both computed and measured maximum moments occurred at approximately the same depth. Considering the pile to be a free-standing member to a depth of 45 in., the maximum moment would only be 740 in.-kips.

The measured and computed deflections for test pile 20 are compared in Fig. 4.14. As shown, the computed deflections are 30% less than the measured deflections for low loads, but for a lateral load of 33.3 kips the computed deflection is 33% larger than the measured deflection.

Ontario

Ismael and Klym (1977) reported the results of short-term static lateral load tests performed on two 5-ft-diameter, cast-in-place, drilled shafts. One shaft, designated 38, was straight sided with a length of 38 ft, and the other shaft, designated 17, was belled with a length of 17 ft. The test setup and pile properties are shown in Fig. 4.15.

The soil profile at the test site consisted of a 6 ft desiccated surface crust of firm to stiff, fissured, silty clay overlying a firm to stiff, silty clay. The silty clay is classified as a CL in the Unified Soil Classification system. The natural water content was near the plastic limit for the top 6 ft, which indicates that the material was heavily overconsolidated.

There was some scatter in the reported U, UU, and CU shear strengths. The shear strength profile, shown in Fig. 4.15, is the best estimate of the in-situ shear strength based on the reported values. Because the clay was stiff and submerged, the StiffB and Unified criteria were used to perform the analyses. The value of the A and F parameters suggested by Sullivan for soil similar to the soil at Manor were used in the analyses.

The measured and computed deflections for test pile 38 are compared in Fig. 4.16a. The computations were made using the StiffB criteria and

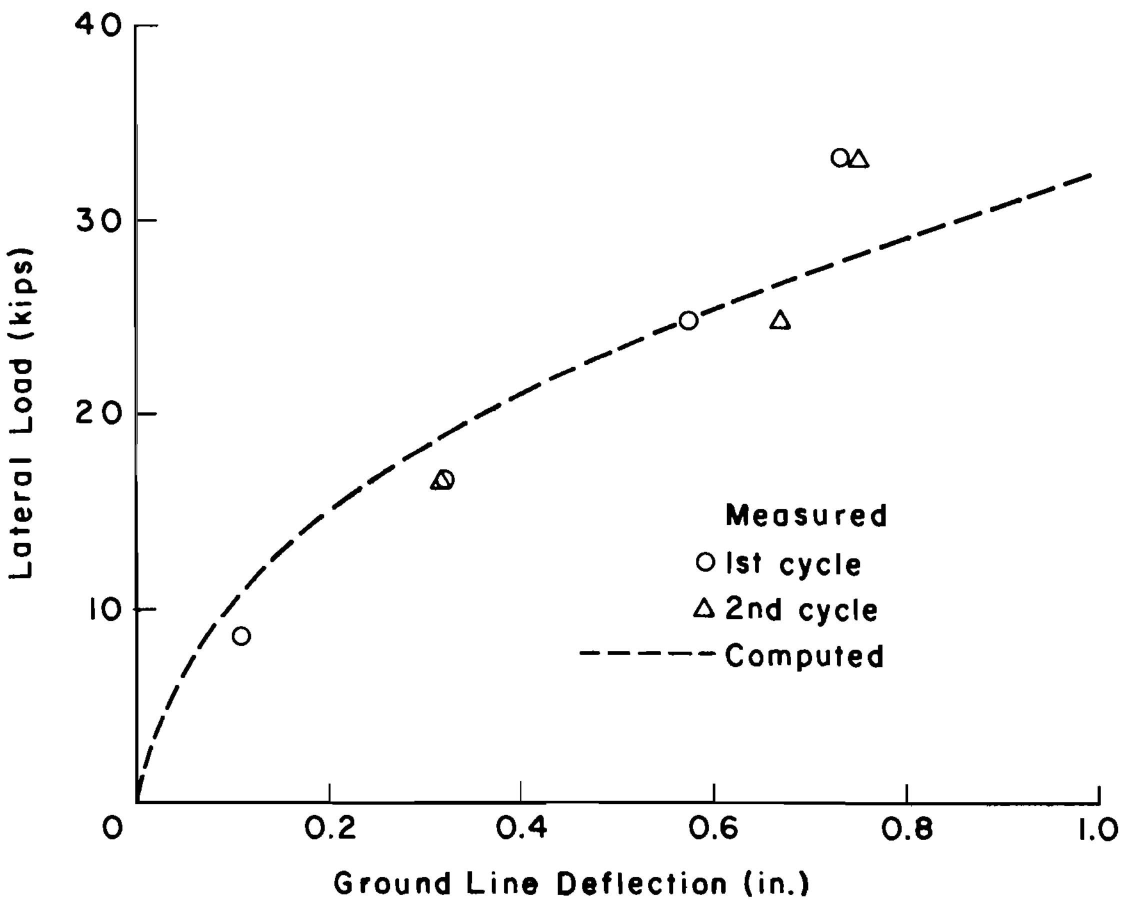
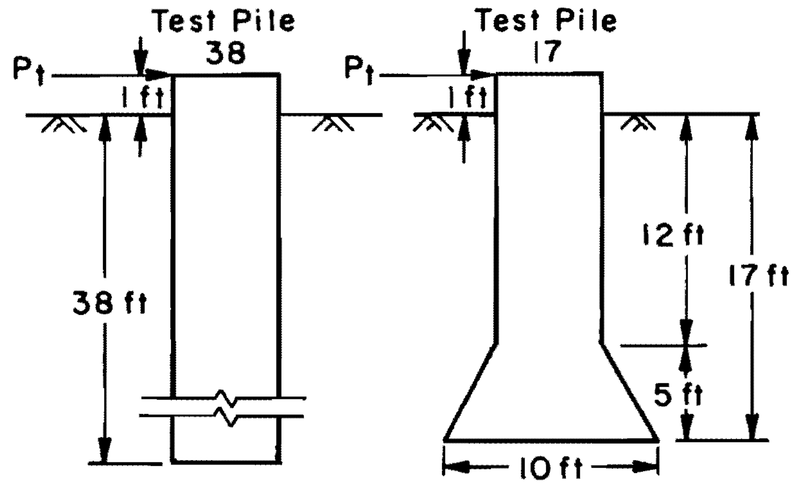


Fig. 4.14. Comparison between measured and computed deflections for test at Lewisburg.



Pile Properties
$EI = 9.3 \times 10^{11}$ lb-in.^{2}
$b = 60$ in.

Soil Properties			
Depth (ft)	ϵ_{50} (%)	γ (lb/ft^3)	k (lb/in.^{3})
0-6	1	60	800
6-40	1	50	300

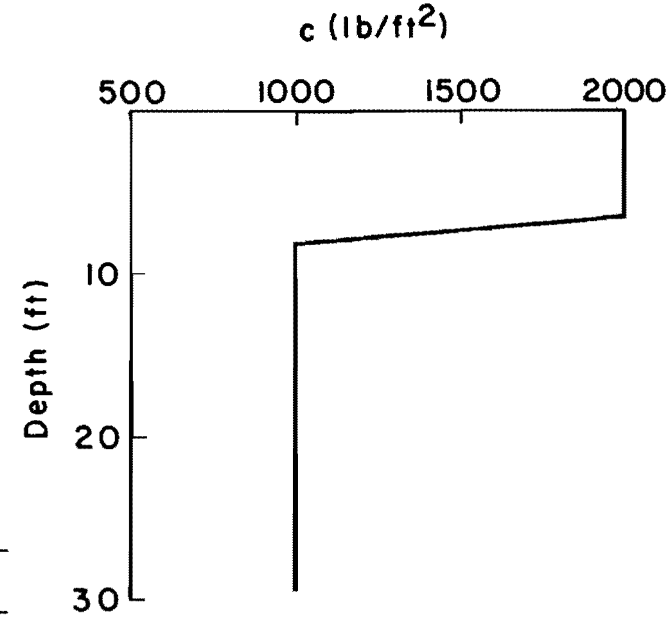


Fig. 4.15. Information for the analysis of tests at Ontario.

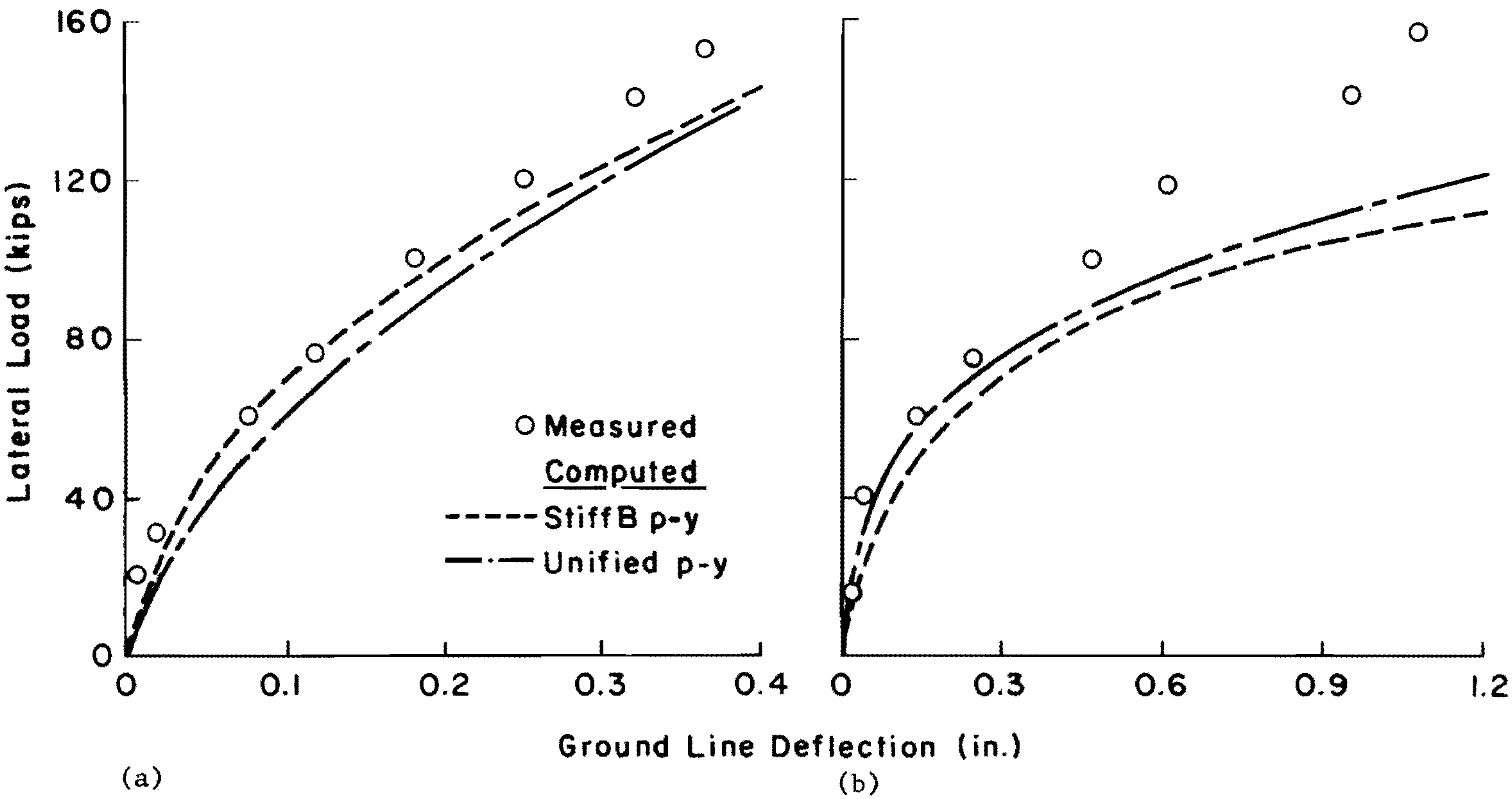


Fig. 4.16. Comparison between measured and computed deflections for tests at Ontario. (a) test pile 38. (b) test pile 17.

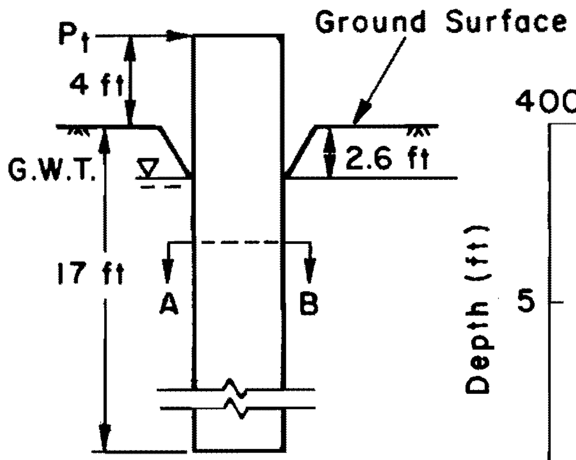
the Unified criteria. As shown, good agreement was obtained between the measured deflections and the deflections computed using the Unified criteria up to a lateral load of 76 kips. Beyond this load, the measured and computed deflections diverge. At a lateral load of 152 kips, the computed deflections exceed the measured deflections by 23%. The StiffB criteria yield results that are conservative for the full range of lateral loads.

The measured and computed deflections for test pile 17 are compared in Fig. 4.16b. The computations were made using the StiffB criteria and the Unified criteria. The measured deflections and those computed using the Unified criteria agree well up to a lateral load of 60 kips, but diverge for larger lateral loads. The results of the analysis using the StiffB criteria are similar to the results obtained using the Unified criteria; however, the StiffB criteria yield results that are conservative for the full range of loading. The lack of agreement between the measured and computed deflections at large lateral loads does not necessarily indicate inaccuracy in the p-y criteria. Large restraining moments and shear forces could have developed along the bottom of the bell, and could have influenced the real behavior of the test foundation. The magnitude of these moments and forces are unknown, and the finite difference analysis assumes that they are zero at the base of the pile.

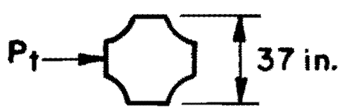
Plancoet

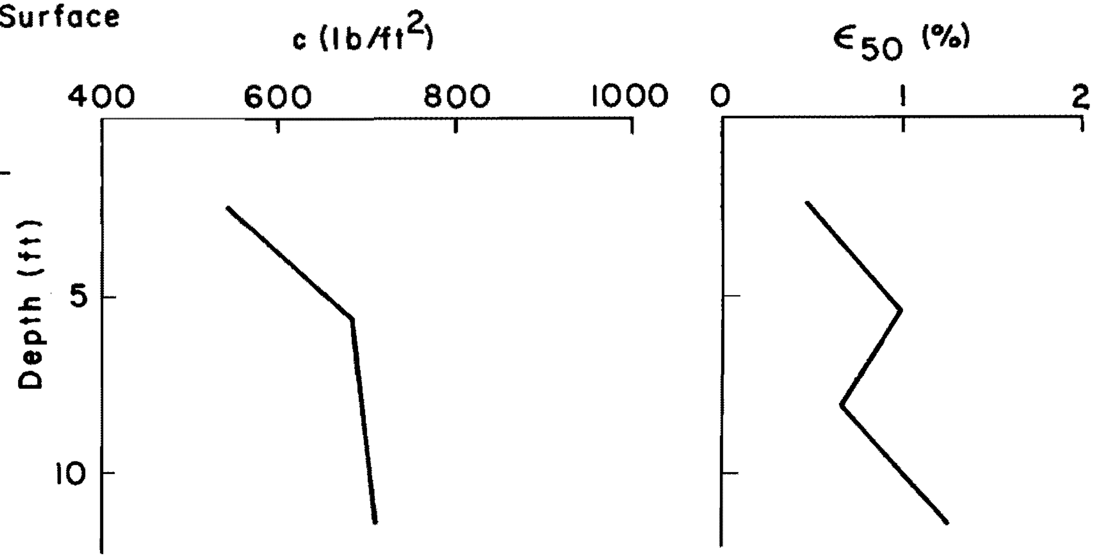
The results of a short-term static lateral load test performed on a free-head, rigid caisson at Plancoet, France, was reported by Baguelin, et al., (1971). The test pile was fabricated from four sheet pile sections, and the base of the pile was sealed with a steel plate. The test pile was installed by pushing the member vertically into the earth until 17 ft of the pile was below the existing ground surface. The test setup and pile properties for the test pile are shown in Fig. 4.17.

The soil at the site consisted of a 13-ft layer of silt overlying a fine sand. The reported results of laboratory tests performed on the silt indicated a liquid limit of 37%, a plastic limit of 18%, and a natural water content of 48%. This reported water content is an average for the



Pile Properties
$EI = 2.65 \times 10^{11}$ lb-in.^{2}
b = 37 in.





Soil Properties				
Depth (ft)	Material type	γ (lb/ft^3)	ϕ	κ (lb/in.^{3})
2.6-12	Silt	50	-	100
12-17	Fine sand	50	35	80

Fig. 4.17. Information for analysis of test at Plancoet.

silt below the desiccated crust. Unconsolidated-undrained triaxial tests performed on undisturbed samples yielded a fairly large amount of scatter in c and ε_{50}. Average values for both of these parameters were used in the analysis, and a reported ϕ of 35° was used for the fine sand. Prior to performing the test, the upper 2.6 ft of desiccated crust was partially removed around the test pile. Because the soil was only partially removed, the 2.6 ft layer would still surcharge the underlying material.

The measured and computed results are compared in Fig. 4.18. The computations were made using the SoftB criteria and Unified criteria for the upper silt layer and the sand criteria for the fine sand layer. As shown, the measured and computed maximum moments are in good agreement for both sets of criteria. The computed deflections obtained using the SoftB criteria agree well with the measured deflections. However, the computed deflections obtained using the Unified criteria do not agree well with the measured deflections, and are conservative by as much as 35%.

Savannah River

Alperstein and Leifer (1976) reported the results of short-term static lateral load tests performed near the Savannah River near Augusta, Georgia. The test piles were Class B timber piles with a 9 in. tip diameter (Johnson and Alperstein, 1977). The butt diameter for the test piles was not reported, and a butt diameter of 13 in. was assumed for the analysis. The effect of this assumption on the results of the analysis will be discussed. Test piles 2 and 5 were driven vertically to a depth of 37 ft and tested with the pile head free to rotate. The test setup and pile properties are shown in Fig. 4.19.

The soil layers at the site consisted of a stiff, silty clay; a silty sand; and an organic, silty clay. The stiff, silty clay layer was of a sufficient thickness so that it would control the pile behavior. An average c of 1200 lb/ft^2 was reported for the stiff, silty clay.

The measured and computed deflections for test piles 2 and 5 are compared in Fig. 4.20. The computations were made using the StiffA criteria

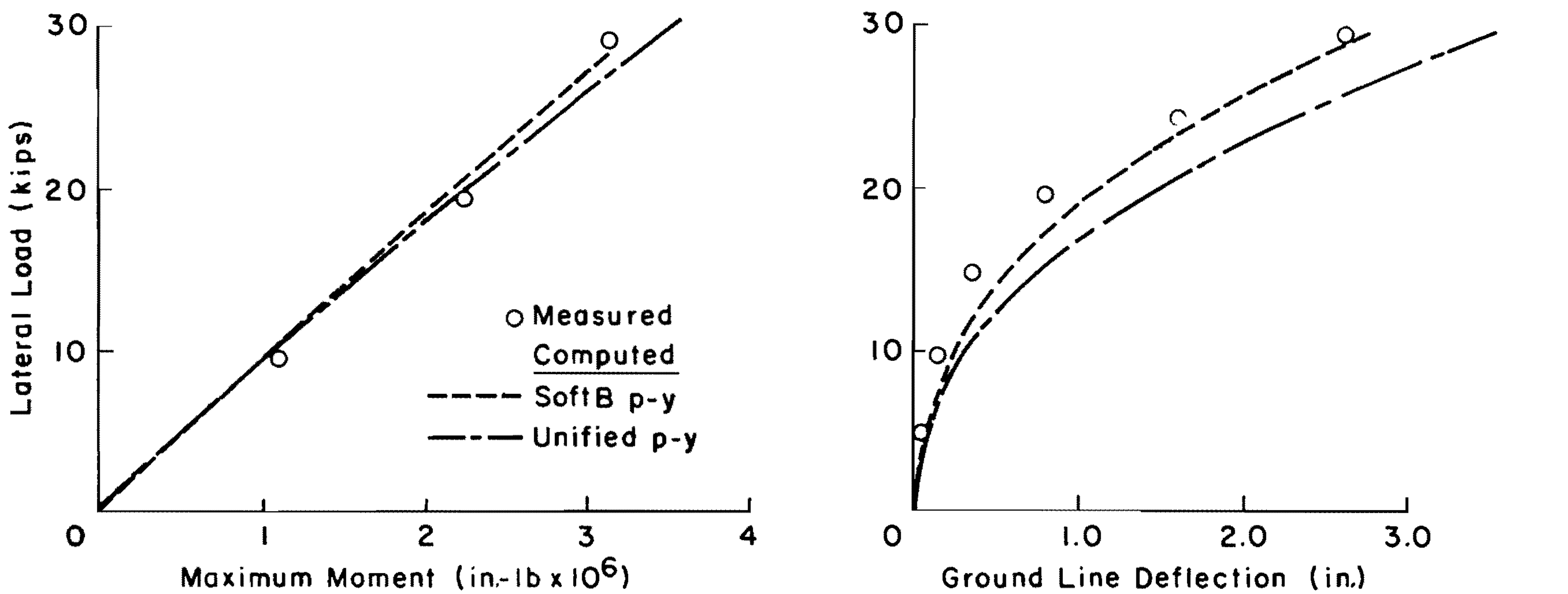
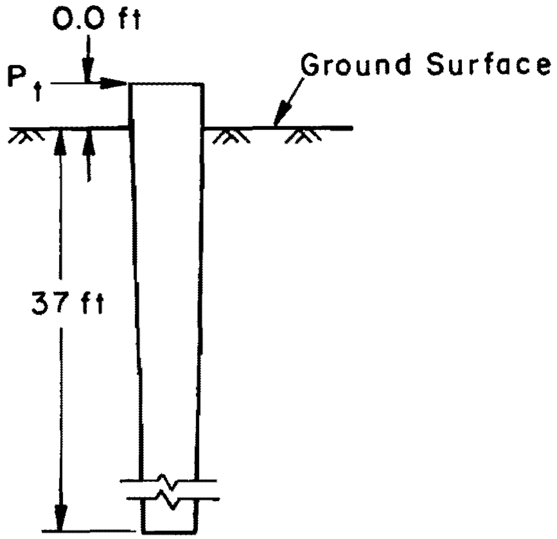


Fig. 4.18. Comparison between measured and computed results for test at Plancoet.



Pile Properties
$EI_{ave} = 2.4 \times 10^9$ lb-in.^{2}
b = 13 in.

Soil Properties
Depth = 0-14 ft
c = 1200 lb/ft^2
$\epsilon_{50} = 0.7\%$
$\gamma = 120$ lb/ft^3

Fig. 4.19. Information for the analysis of tests at Savannah River.

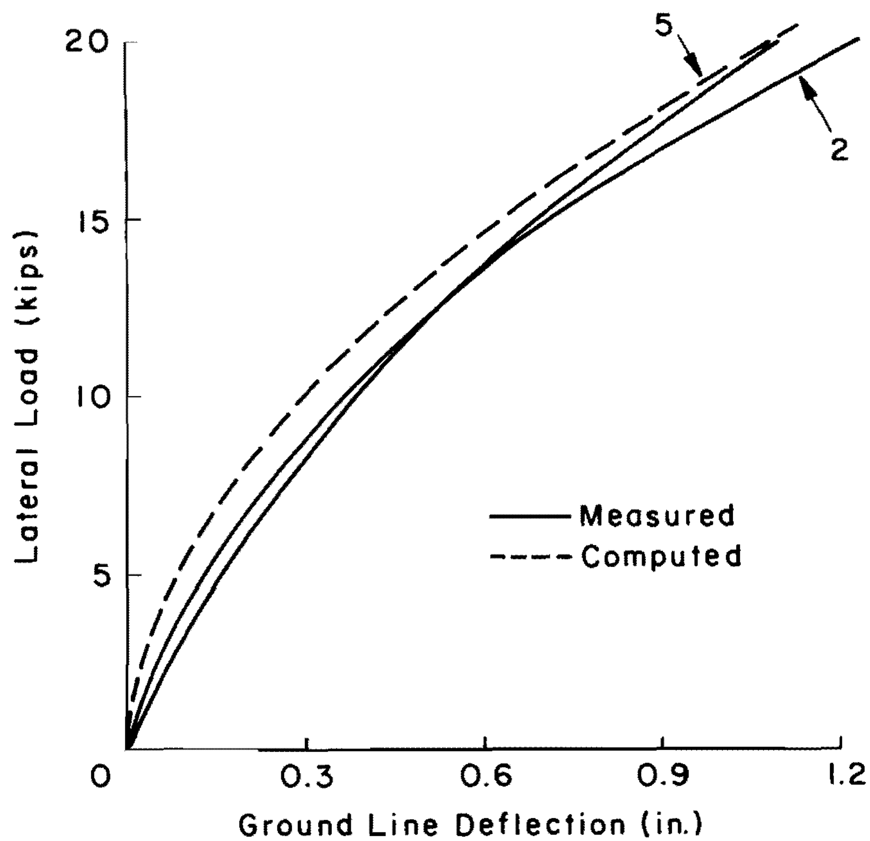


Fig. 4.20. Comparison between measured and computed deflections for tests at Savannah River.

for the stiff, silty clay. The initial slope of the computed load-deflection curve is larger than the initial slope for both test piles. The difference in initial slopes could be due to either the initial slope of the p-y curve being too large, or possible whipping of the timber pile during driving which would enlarge the hole.

As stated previously, a reasonable butt diameter of 13 in. was selected to perform the analysis. Two additional computer runs were made using Ei's corresponding to a 12 in. butt diameter and a 14 in. butt diameter. A difference of $\pm 20\%$ in the deflection was obtained due to the $\pm 30\%$ change in EI. These results are similar to the results obtained in Chapter 3 which proves that EI has a fairly large effect on the lateral deflection.

Southern California Edison

Bhushan, et al., (1978) reported the results of static lateral load tests performed on cast-in-place drilled shafts for the Mesa-Olinda 200 kv transmission line for the Southern California Edison Company. The tests were performed on twelve shafts at five sites, but only the results of three tests performed at sites A and B will be discussed. All three piles were straight sided and reinforced with 3% steel. The depth of embedment was small enough and EI was large enough so that the piers behaved as rigid members. The lateral loads were applied incrementally, and each increment was held constant for at least 40 minutes.

For both test sites, the soils were silty and sandy clays of low to medium plasticity. The liquid limit was between 30 and 58% and the plasticity index was between 15 and 20%. The natural water content was at or below the plastic limit, indicating the soil was heavily overconsolidated.

The undrained shear strengths and ε_{50} values were obtained from U and UU tests on intact samples. The authors reported a great deal of scatter in the results of the tests used to define c. The large amount of scatter in c is common for desiccated, heavily overconsolidated soils. In the following analyses, the average c and ε_{50} values reported by the authors for each test site were used. Because the soils were dry and heavily overconsolidated the StiffA criteria were used to perform the analyses.

The measured and computed deflections for test pile 2 are compared in Fig. 4.22. As shown, the computed and measured deflections are in poor agreement. For lateral loads less than 140 kips, the computed deflection is too small, and for larger lateral loads, the computed deflections are very conservative. At a lateral deflection of 2 in., the computed lateral load underestimates the measured lateral load by 53%. The shapes of the computed and measured load-deflection curves are totally different. To obtain better agreement between the measured and computed results, the static StiffA criteria would have to be modified.

The measured and computed deflections for test piles 6 and 8 are compared in Fig. 4.24. As shown, the computed and measured deflections are in poor agreement. The same trend in the results was obtained for these two test piles at site B that was obtained for test pile 2 at site A. The initial portion of the computed load-deflection curve is too stiff, and the computed ultimate lateral capacity is too small.

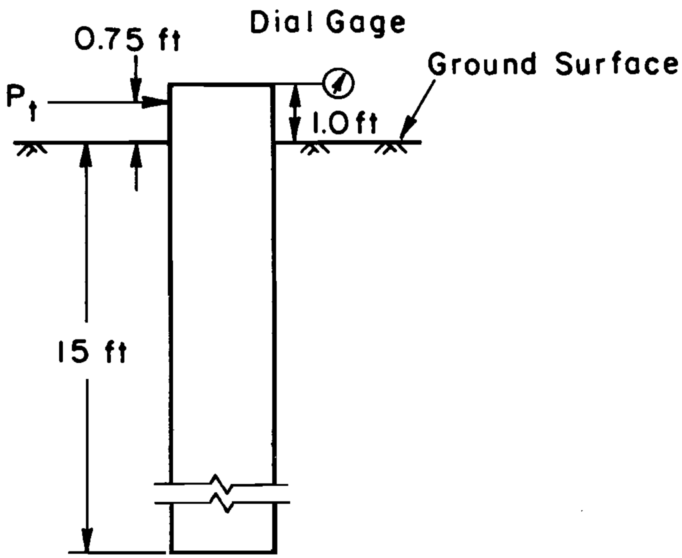
Based on these test results the shape of the p-y curve needs to be modified. The exponent in Eq. 2.5 and p_u determine the shape of the p-y curve and thus the shape of the load-deflection curve. In this case, the exponent is too large, and p_u is too small. Raising the exponent will decrease the stiffness of the initial portion of the load-deflection curve, and increasing p_u will increase the lateral capacity at larger deflections.

St. Gabriel

A short-term static lateral load test was performed on a free-head, 10 in., concrete-filled, pipe pile near St. Gabriel, Louisiana (Capazzoli, 1968). The test piles were driven vertically to a depth of 115 ft. The test setup and pile properties are shown in Fig. 4.25.

The soil at the site was a soft to medium, intact, silty clay. The natural moisture content of the clay varied from 35 to 46% in the upper 10 ft of soil. The undrained shear strengths, shown in Fig. 4.25, were obtained from U tests.

The measured and computed deflections for test pile 17 are computed



Pile Properties

$EI = 8.2 \times 10^{11}$ lb-in.^{2}
b = 48 in.

Soil Properties

Depth = 0-15 ft
c = 5500 lb/ft^2
ϵ_{50} = 0.94%
γ = 130 lb/ft^3

Fig. 4.21. Information for the analysis of test pile 2 for Southern California Edison.

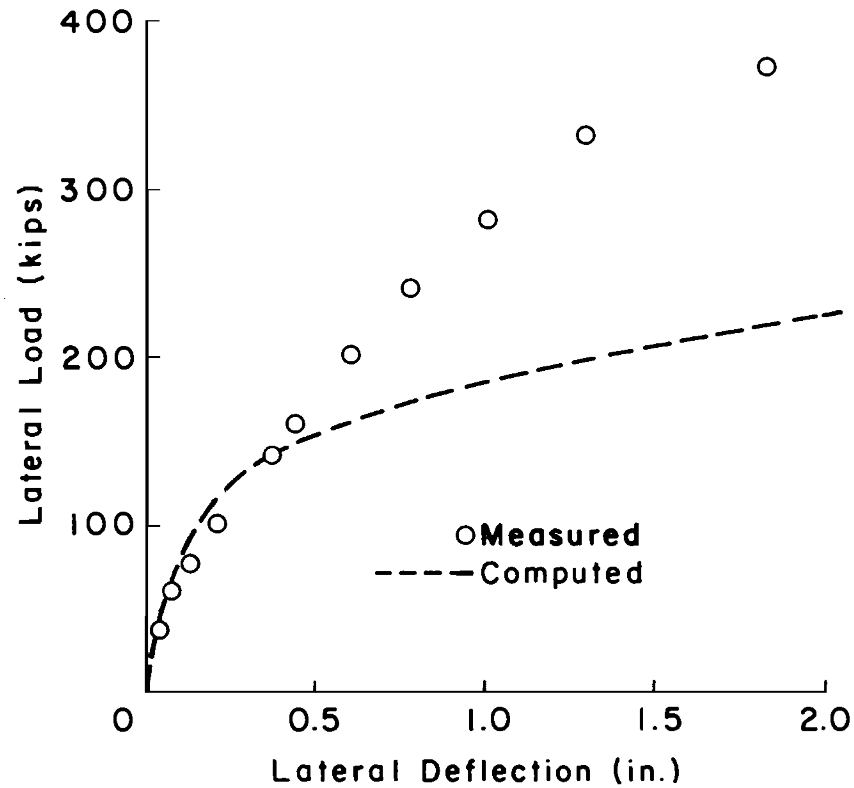
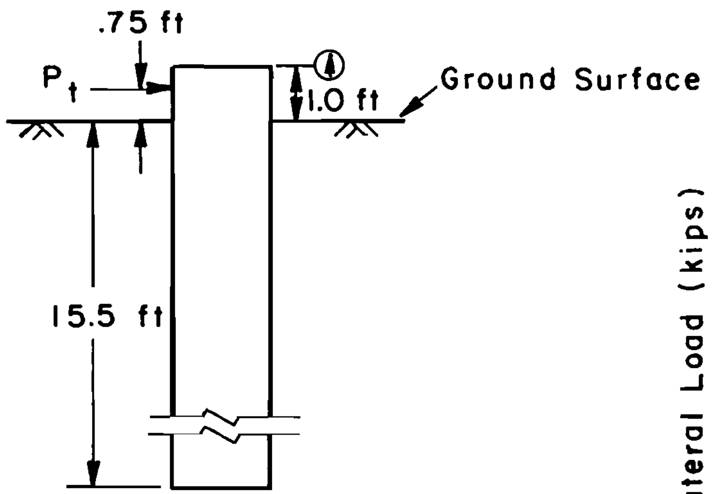


Fig. 4.22. Comparison between the measured and computed deflections for test pile 2 for Southern California Edison.



Pile Properties		
Test Pile Number	EI ($lb\text{-}in.^2$)	b (in.)
6	8.2×10^{11}	48
8	5.2×10^{10}	24

Soil Properties

Depth = 0- 16 ft

$c = 4750\ lb/ft^2$

$\epsilon_{50} = 0.72\%$

$\gamma = 130\ lb/ft^3$

Fig. 4.23. Information for the analysis of test piles 6 and 8 for Southern California Edison.

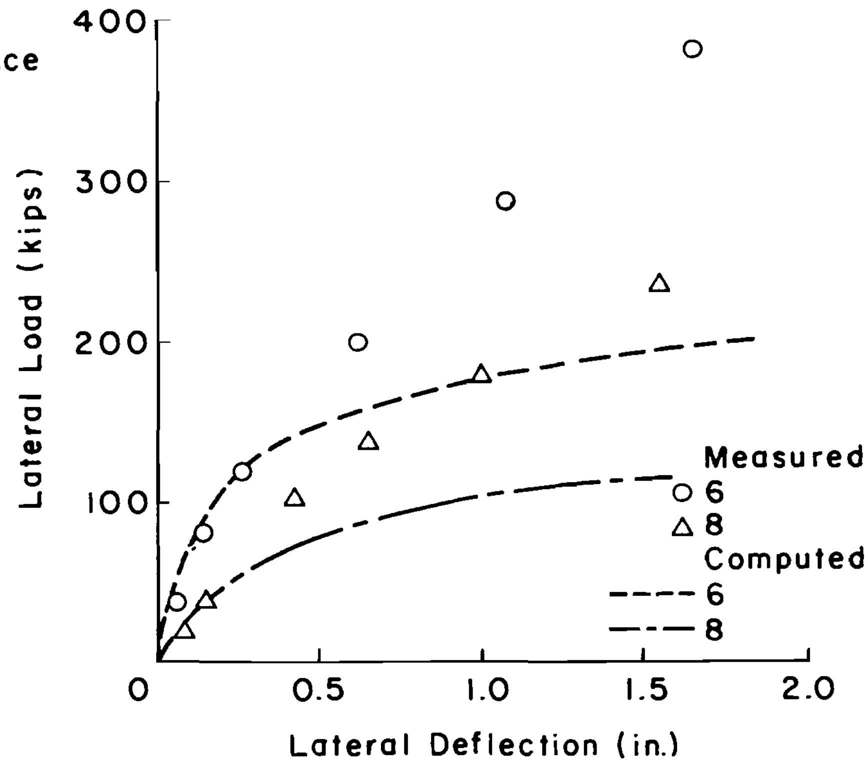


Fig. 4.24. Comparison between measured and computed deflections for test piles 6 and 8 for Southern California Edison.

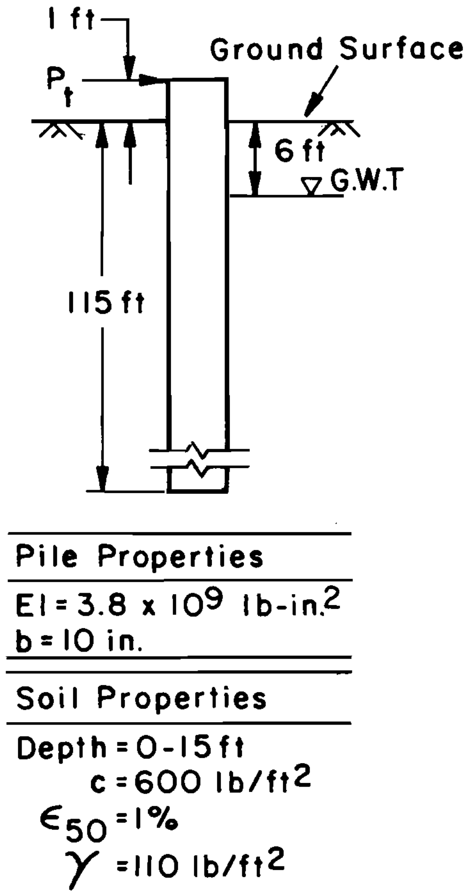


Fig. 4.25. Information for analysis of test at St. Gabriel.

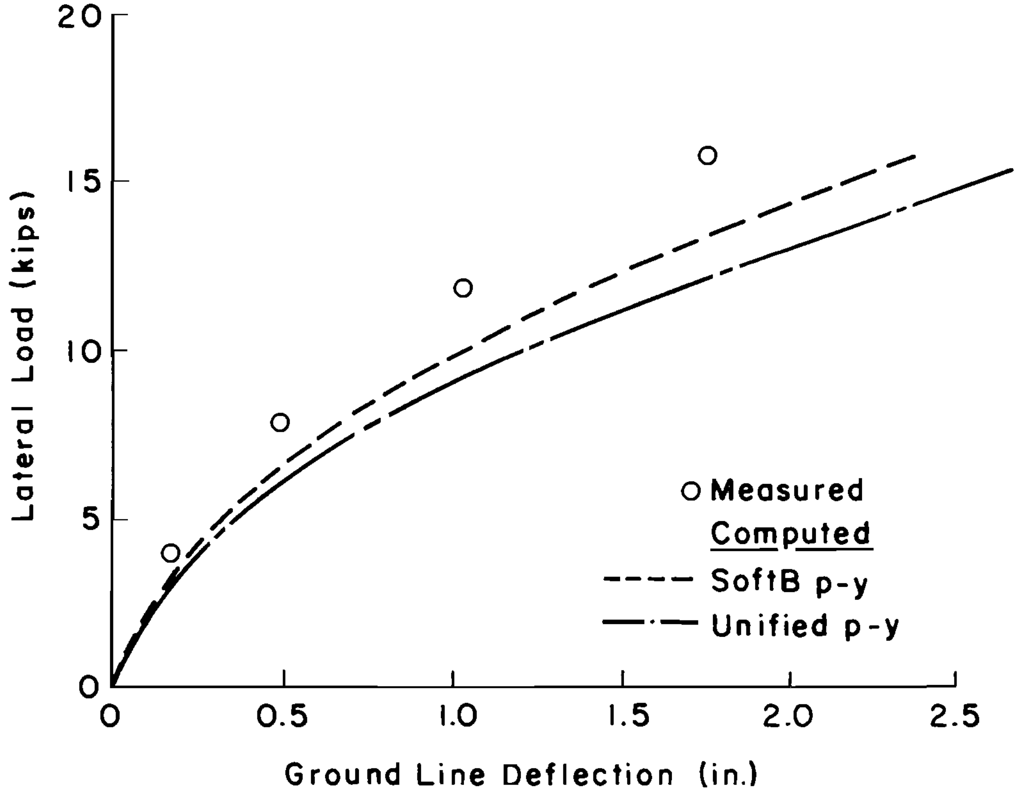


Fig. 4.26. Comparison between measured and computed deflections for test at St. Gabriel.

in Fig. 4.26. The computations were made using the SoftB criteria and the Unified criteria. As shown, the computed deflections are conservative for both sets of criteria. The computed deflections using the SoftB criteria were in error by as much as 35%, and the computed deflections using the Unified criteria were in error by as much as 55%.

EVALUATION OF p-y CRITERIA

SoftB and Unified Criteria

The Unified criteria, where the parameters A and F are 2.5 and 1.0, respectively, and the SoftB criteria were used at the following test sites:

(1) Bay mud, flooded,
(2) Hudson River,
(3) Japan,
(4) Plancoet, and
(5) St. Gabriel.

Figure 4.27 plots the computed deflections from the results of the analyses performed on piles at the above five sites versus the measured deflections. There is considerable scatter in the results, but the majority of points are within the 25% confidence limits. Generally, the only points which were unconservative were obtained using the SoftB criteria at the flooded bay mud test site, but two points were unconservative using the Unified criteria at the same site.

The computed deflections obtained using the SoftB criteria were always less than the computed deflections obtained using the Unified criteria. The differences between the static SoftB criteria and the static Unified criteria are that different values for N_p at the ground surface are used in the two criteria, and that c_x is used in the SoftB criteria where c_a is used in the Unified criteria. The larger values of N_p used in the SoftB criteria yields greater soil resistances and corresponding increases in E_s. Using the different definitions for c will have some affect on the

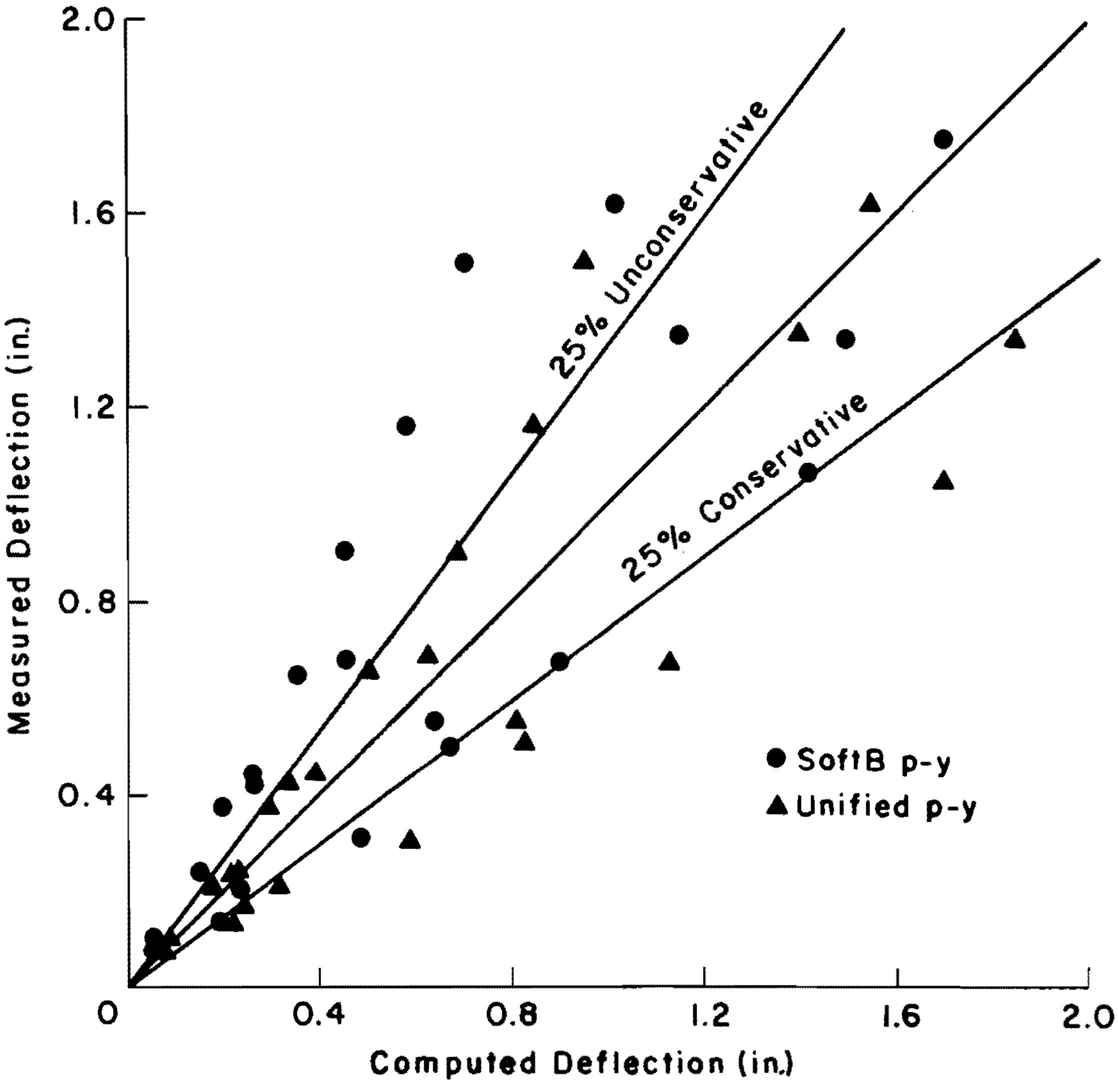


Fig. 4.27. Comparison between measured and computed deflections for tests using softB and unified criteria.

computed results, but the use of the different values for N_p at the ground surface will have a larger effect on the computed solution.

Based on the results of the analyses presented in this chapter, both the static SoftB and Unified p-y criteria were adequate in predicting the lateral load behavior of piles in soft clay. The criteria were adequate for the cases presented in this report, but two important aspects of the lateral load behavior of piles have not been dealt with. The lateral load behavior under cyclic loading has not been discussed since no cyclic load test results could be found in the literature, except the one performed by Matlock in developing the criteria. Also, the effect of large pile diameters on the pile behavior has not been thoroughly investigated. The largest diameter pile analyzed in this report was the 37 in. rigid pile at Plancoet. Presently, 72 in. and larger piles are frequently being used offshore. These pile sizes are much larger than the 12.75 in. pile which was used in developing the criteria. While it may be quite expensive to test piles which are as large as offshore piles, such tests would be most beneficial in either reinforcing or modifying the presently used criteria.

<u>StiffA Criteria</u>

The StiffA criteria were used at the following test sites:

(1) Bagnolet,
(2) Bay mud, dry,
(3) Lewisburg,
(4) Savannah River, and
(5) Southern California Edison.

The measured versus computed deflections from the results of the analyses performed on piles and piers at the above five sites are plotted in Fig. 4.28. As shown, there is considerable scatter in the results. The majority of the data points from Bagnolet, Lewisburg, and Savannah River were within the 25% confidence limits. However, the computed results from the tests at the dry bay mud test site were very unconservative. The lack of agreement between the computed and measured results at the dry bay mud test

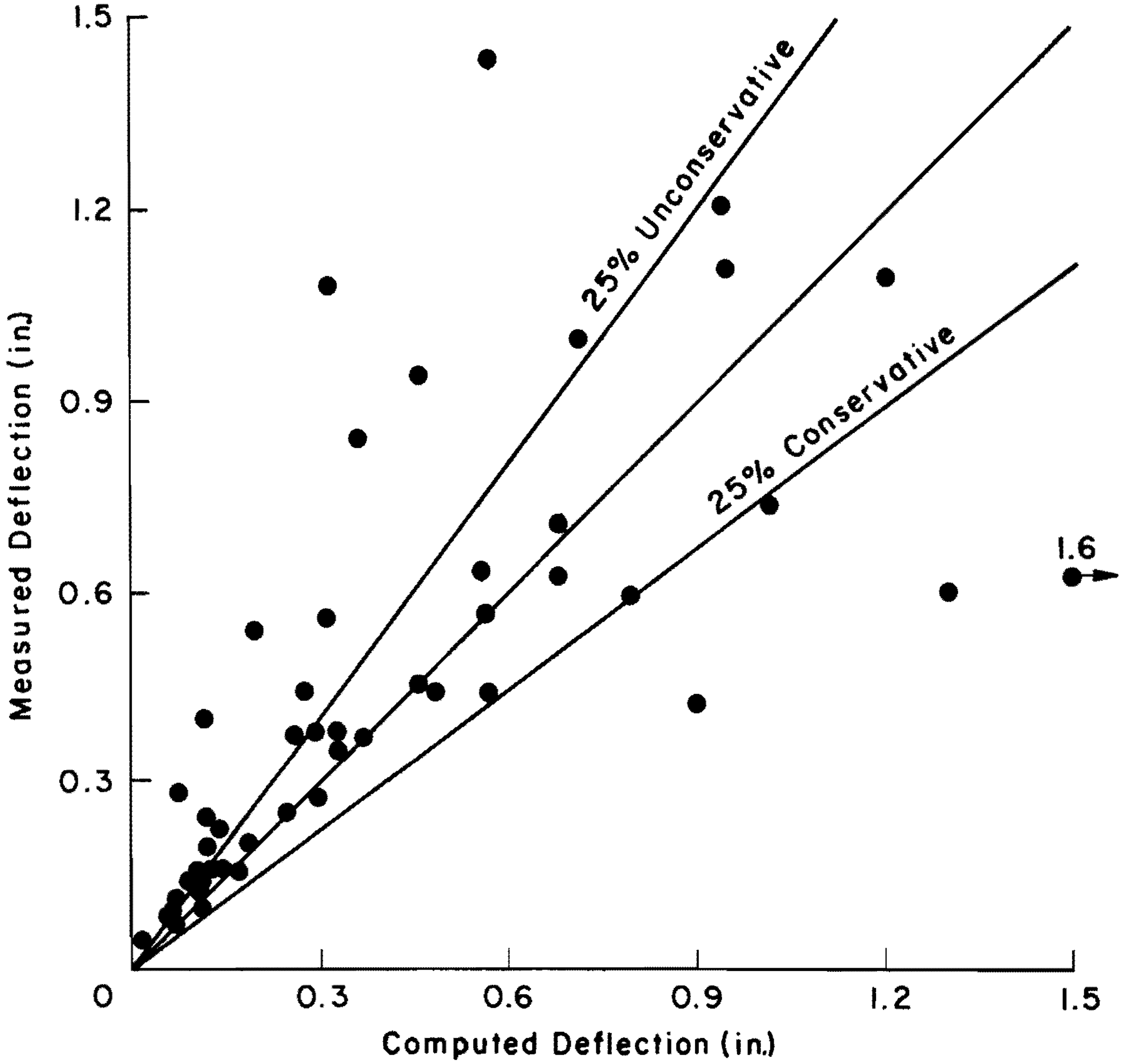


Fig. 4.28. Measured versus computed deflections for tests using StiffA criteria.

site is apparently due to the use of shear strengths which were too large. The high rate at which the vane was rotated could have affected c, but no other cases could be found in the literature to verify this point. Therefore, the use of vane tests to obtain shear strengths of stiff, desiccated clays is questionable.

The results of the analyses performed on the Southern California Edison drilled shafts were very conservative at the larger lateral loads. For the three tests, the computed ultimate lateral capacity was approximately 1/2 of the measured lateral capacity. There are at least two possible reasons for this discrepancy. One is that the reported shear strengths were not correct. Bhushan, et al., reported that there was a great deal of scatter in the results of the laboratory tests. Apparently, the soil was difficult to sample, and it is possible that the average shear strength is too low because of sampling disturbance. Another reason for the lack of agreement may be due to the inability of the StiffA criteria to model correctly the soil behavior. The soils at the two sites had shear strengths which were double the shear strengths at the site where the StiffA criteria were developed. The equation used to obtain p_u may have to be altered, but there is not a sufficient amount of information available to suggest any modifications for Eq. 2.2.

For all of the test sites, the initial slope of the computed load-deflection curve was too large. This indicates that the initial slope of the p-y curve is too large. Since a parabola is used to define the p-y curves, the soil modulus at small deflections is controlled by the exponent in Eq. 2.5. The smaller the exponent, the stiffer the initial slope of the p-y curve. Because the computed deflections for the initial loads are unconservative, the exponent must be increased if better agreement is to be obtained between measured and computed results. Increasing the exponent will also stiffen the p-y curve at larger deflections, which will thus decrease the deflections at large lateral loads. The reported cases for tests in stiff soils above the water table were reanalyzed using different values for the exponent, and the final value which was selected was 0.4. Equation 2.5 can now be modified to yield the following equation:

$$\frac{p}{p_u} = 0.5 \, (y/y_{50})^{0.4} \tag{4.3}$$

The measured versus computed deflections were replotted in Fig. 4.29. As shown, better agreement between the measured and computed deflections was obtained when Eq. 4.3 was used to describe the shape of the static p-y curves. Therefore, the use of Eq. 4.3 is recommended in place of Eq. 2.5.

StiffB and Unified Critieria

The Unified criteria, where A and F are 0.35 and 0.5, respectively, and the StiffB criteria were the only ones used to analyze the tests at the Ontario Hydro site. The measured versus computed deflections from the results of the analyses of piles performed at the above site are plotted in Fig. 4.30. Both criteria worked fairly well, but the Unified criteria yielded slightly better results. The computed deflections obtained using the Unified criteria were consistently less than the computed deflections obtained using the StiffB criteria.

Based on the results of these two tests, both criteria worked well, but the results of more tests in stiff, submerged clays are needed to evaluate correctly the two sets of criteria.

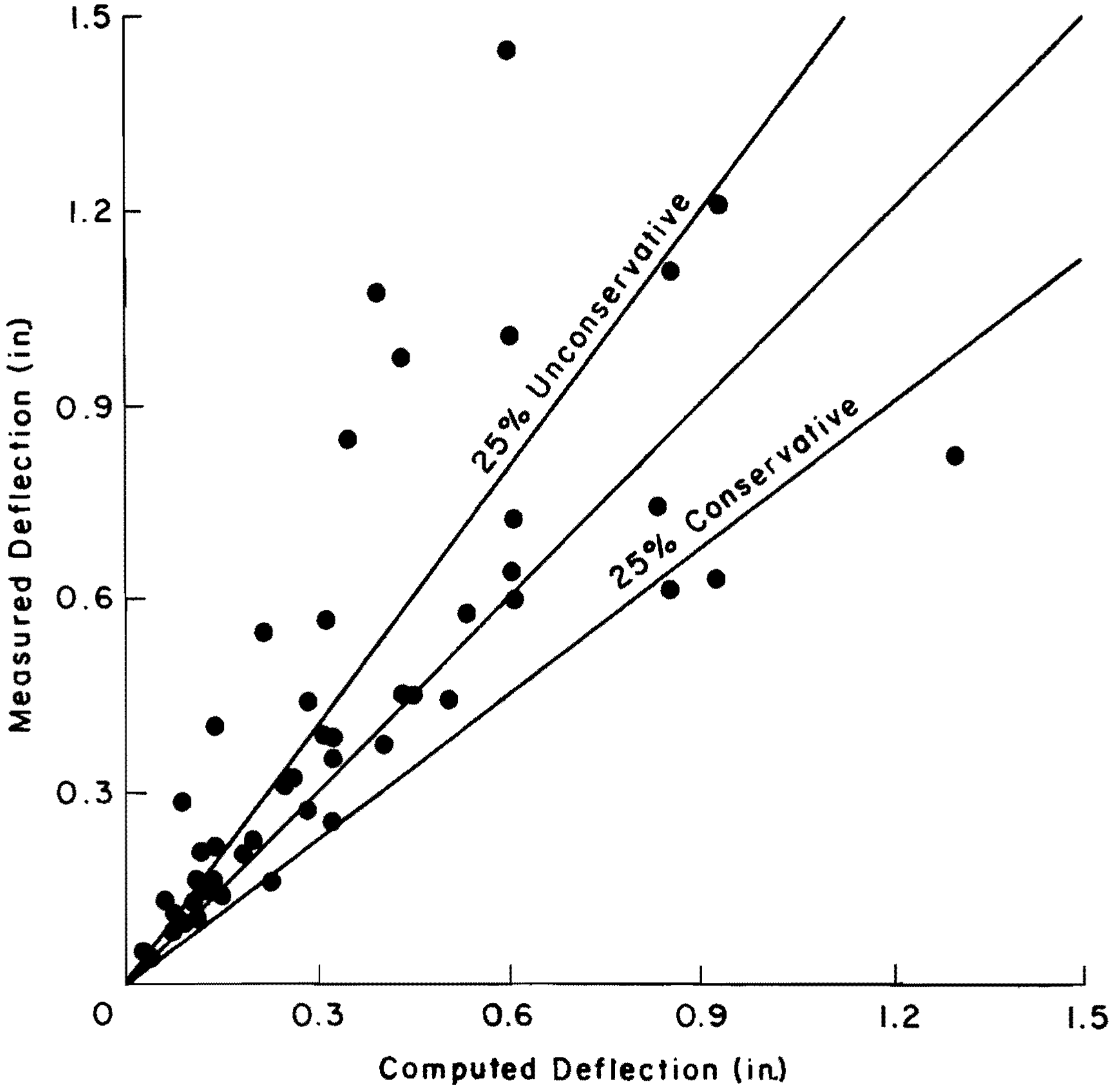


Fig. 4.29. Comparison between measured and computed deflections for tests using modified StiffA criteria.

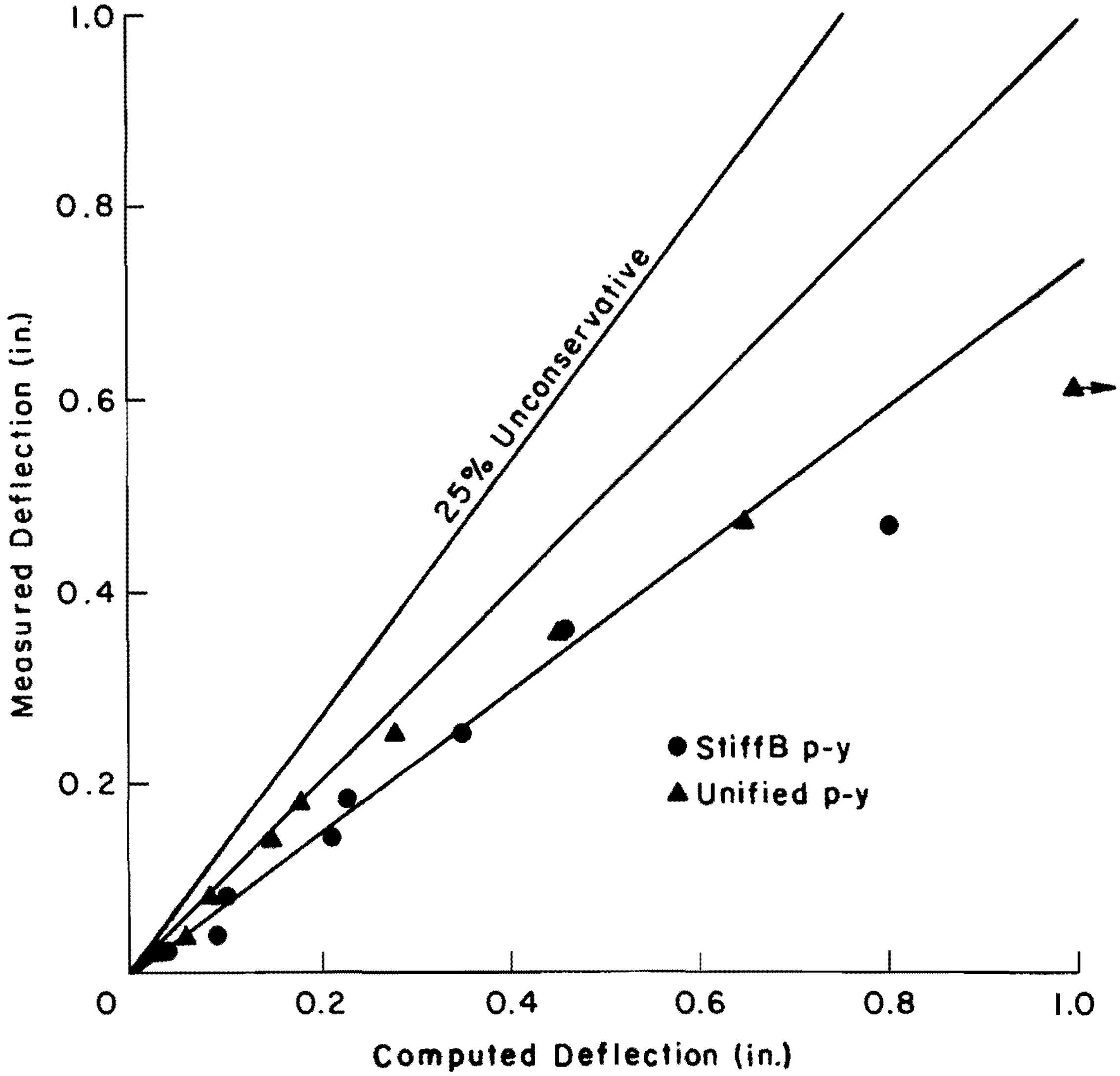


Fig. 4.30. Comparison between measured and computed deflections for tests using StiffA and Unified criteria.

CHAPTER 5. ANALYSIS OF LATERAL LOAD TESTS FOR PILES IN SAND

INTRODUCTION

The criteria suggested by Reese, et al., (1974) for analyzing the behavior of single vertical piles embedded in sand appear to be the best criteria available at the present time. The piles may be subjected to either static or cyclic loading. To determine how accurately this method can predict the behavior of laterally loaded piles, it is necessary to compare analytical results obtained by using these criteria with the measured results from load tests.

In most of the tests that were analyzed, all the necessary soils information had to be inferred from the Standard Penetration Test, SPT, and a certain range in the results of the analyses was possible. This range in results is due to the different assumptions regarding the correlation of the results of the SPT with the relevant soil properties. In performing the analysis, the most reasonable assumptions were made in selecting soil properties. All of the available information was carefully analyzed, and the best estimate of the in-situ soil properties was made. There is no implication that the soil properties selected are the "exact" soil properties, but they are the best estimate in view of the limited information that was presented in each case.

METHOD OF OBTAINING SOIL PROPERTIES

As previously stated, when the important soil properties such as ϕ, γ, k, and K_o were not reported, they can either be obtained from correlations with some in-situ testing method or they can be assumed. The approach used in this report was to select a particular method for relating the blow count, from an SPT, to the relative density, D_r, and to then relate D_r to ϕ and k. The angle of internal friction could then be related to the void ratio for a particular soil, and then γ could be calculated. The value of K_o was reported in none of the experiments and there is no method by which an exact value of K_o can be determined from the

SPT for an overconsolidated sand deposit.

There are many methods available for correlating the SPT blow count to D_r (Gibbs and Holtz, 1957; Bazarra, 1967; Peck et al., 1974). The method proposed by Bazarra seemed to be the best method because it took the overburden pressure into consideration and because the method was developed from the results of actual field tests. The two equations which Bazarra proposed to obtain the relative density are

$$D_r = \left(\frac{N}{20\ (1 + 2\bar{p})}\right)^{0.5} \tag{5.1}$$

for $\bar{p} < 1.5$ kip/ft^2

and

$$D_r = \left(\frac{N}{20\ (3.25 + 0.5\bar{p})}\right)^{0.5} \tag{5.2}$$

where

N = blow count (blows/ft),
$\bar{p}$ = effective overburden pressure (kip/ft^2).

The angle of internal friction can now be determined from correlations with D_r. The angle of internal friction is not only a function of D_r, but also a function of particle size and shape, gradation, and confining pressure. In most cases, the particle size and gradation were reported. Because the pressures around the top portion of a laterally loaded pile are not large, the effect of confining pressure was not considered.

The correlation that was used to relate D_r to ϕ was given by Schmertman (1975). His curves, shown in Fig. 5.1, show ϕ as a function of D_r and some of the previously mentioned parameters (e.g., grain size). The

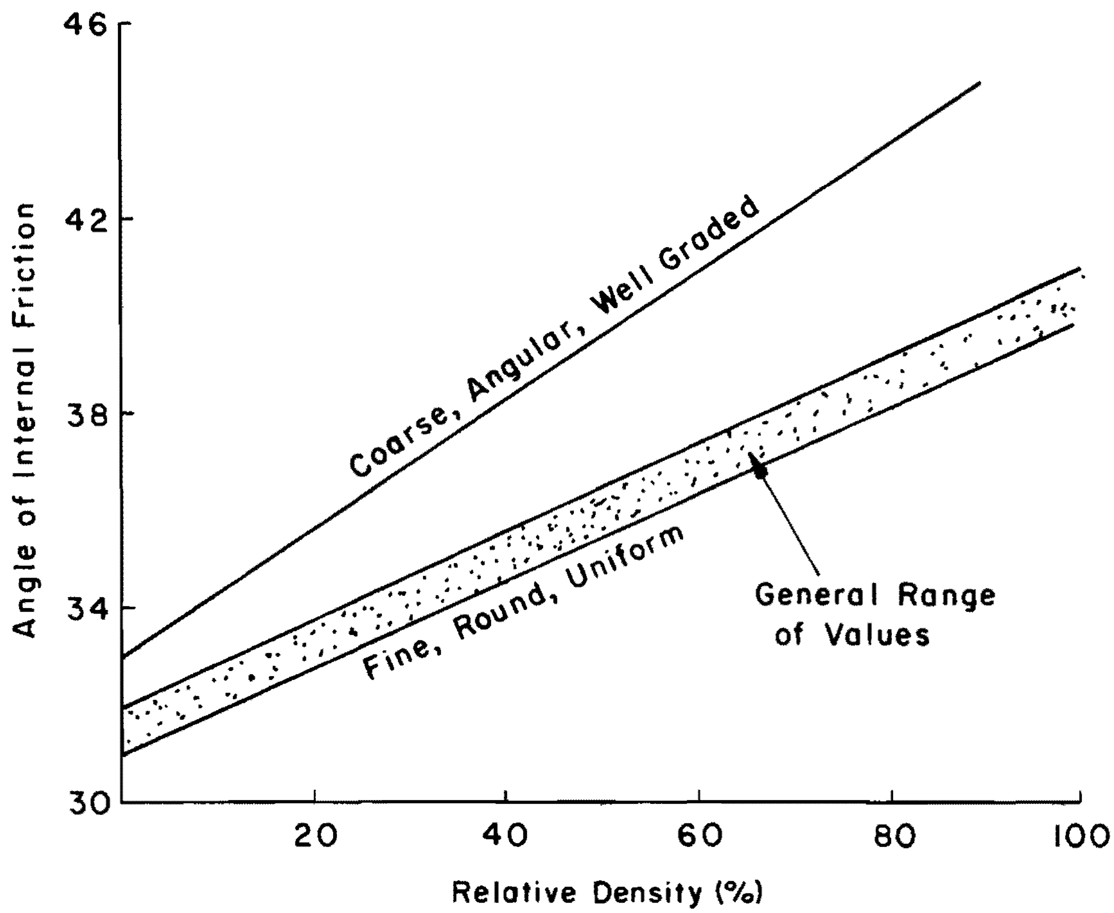


Fig. 5.1. Correlation between angle of internal friction and relative density (Schmertmann, 1975).

upper curve is for angular, well-graded materials and the lower curve is for the rounded, poorly-graded materials. Most of the sands that were described in connection with the load tests that were studied were classified as SP in the Unified Soil Classification System, which would place them closer to the lower curve in Fig. 5.1.

Touma (1972) recommended the determination of D_r and ϕ from the work of Gibbs and Holtz (1957) and he further obtained a correlation between the N-value from the Standard Penetration Test and the N-value from the penetrometer test of the State Department of Highways and Public Transportation, State of Texas. Figure 5.2 presents Touma's recommendations and allows correlations to be developed if one has N-values from the SDHPT test (PEN test).

The constant of subgrade reaction is necessary to establish the initial portion of the p-y curve. Values of k, as a function of the general classifications of loose, medium, and dense, have been reported by Reese (1975) and are shown in Tables 5.1 and 5.2, for sands below the water table and for sands above the water table, respectively. The values of k were plotted as a function of D_r, instead of tabulating them as Reese (1975) suggested. The two curves of k versus D_r are plotted in Fig. 5.3.

Depending on the gradation of the sand and the particle size, a rough correlation between ϕ and the void ratio can be made, as shown in Fig. 5.4. The submerged unit weight was calculated using a degree of saturation, S_r, of 100%, and the total unit weight was calculated using an S_r of 50%.

Reese, et al., (1974) used a value of 0.4 for K_o in their analyses. In analyzing tests which were similar to the tests from which the criteria were developed, a value of 0.4 for K_o is reasonable. Larger values of K_o could be used where sands are overconsolidated. In this report, the soil-pile behavior was determined as accurately as possible at the time of testing and every effort was made to take all extraneous factors into consideration. In instances where a large amount of soil was excavated, a value of 1.0 was used for K_o.

In the following sections, the results of analyses of the results of a number of lateral load tests will be discussed. In cases where the important soil properties were reported, these values were simply used directly in the

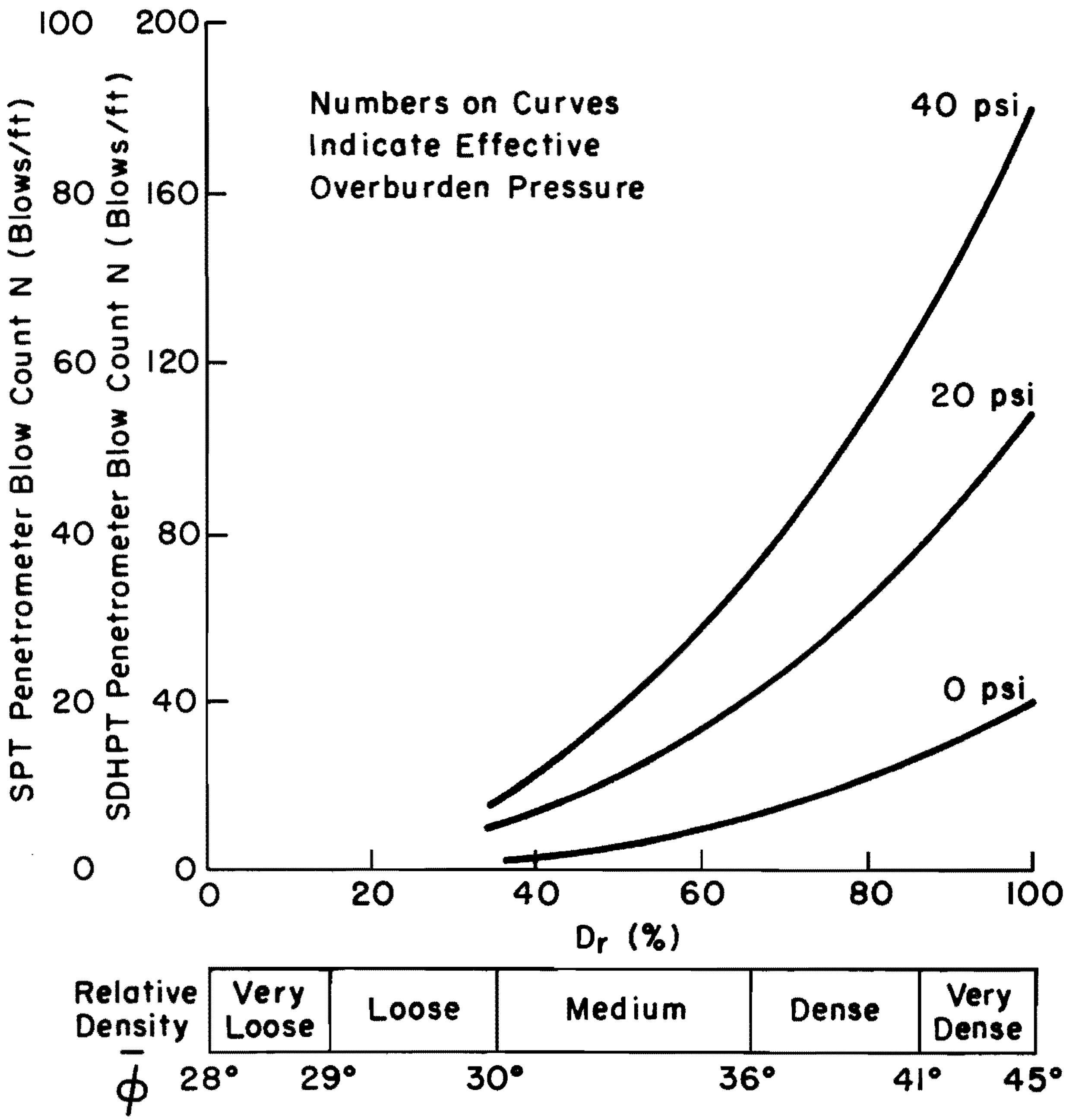


Fig. 5.2. Relation between SDHPT and SPT penetrometer blow count and the friction angle.

TABLE 5.1. RECOMMENDED VALUES OF k FOR SANDS BELOW THE WATER TABLE

Relative Density	Loose	Medium	Dense
Recommended k ($lb/in.^3$)	20	60	125

TABLE 5.2. RECOMMENDED VALUES OF k FOR SANDS ABOVE THE WATER TABLE

Relative Density	Loose	Medium	Dense
Recommended k ($lb/in.^3$)	25	90	225

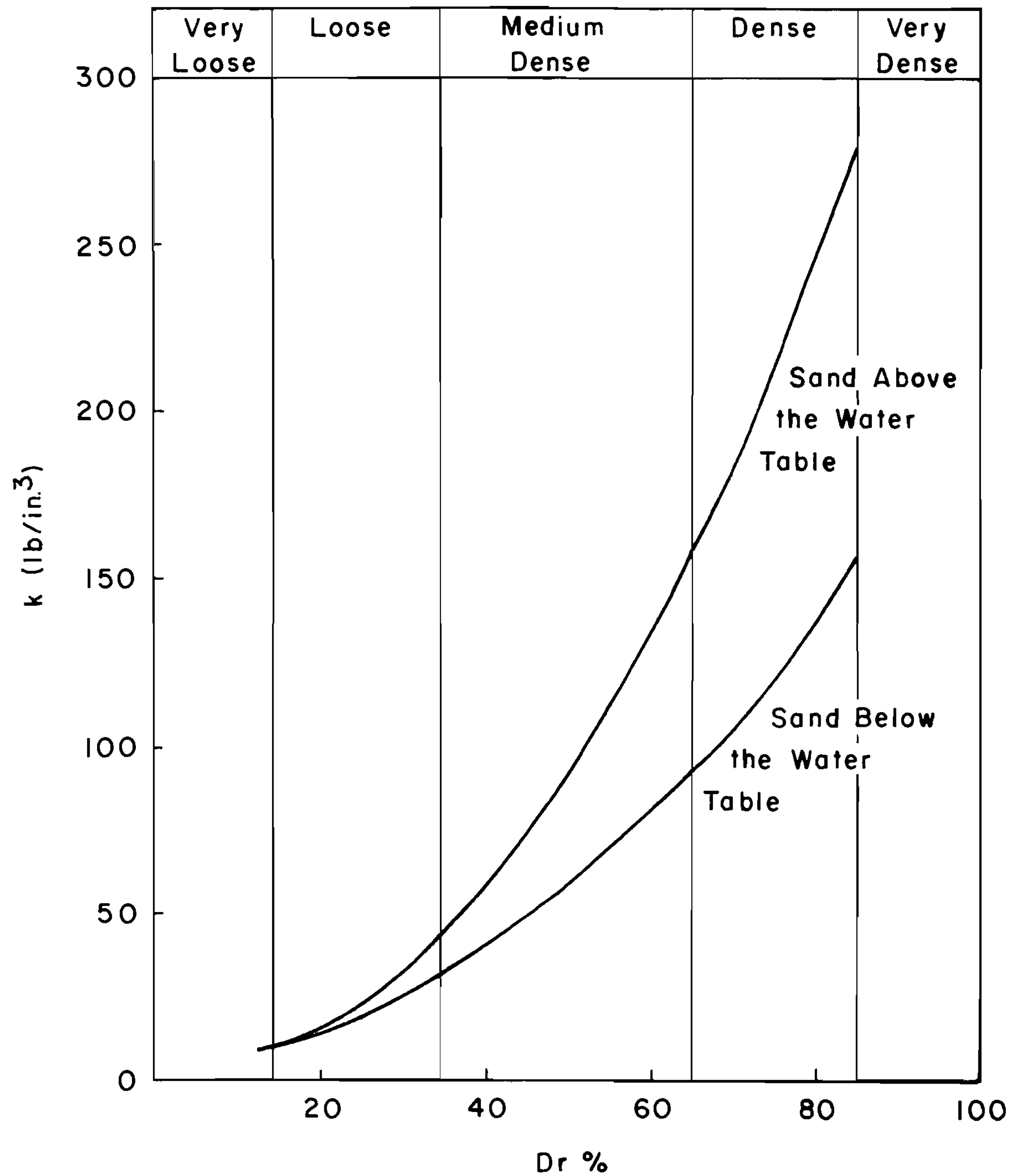


Fig. 5.3. Variation of k with relative density.

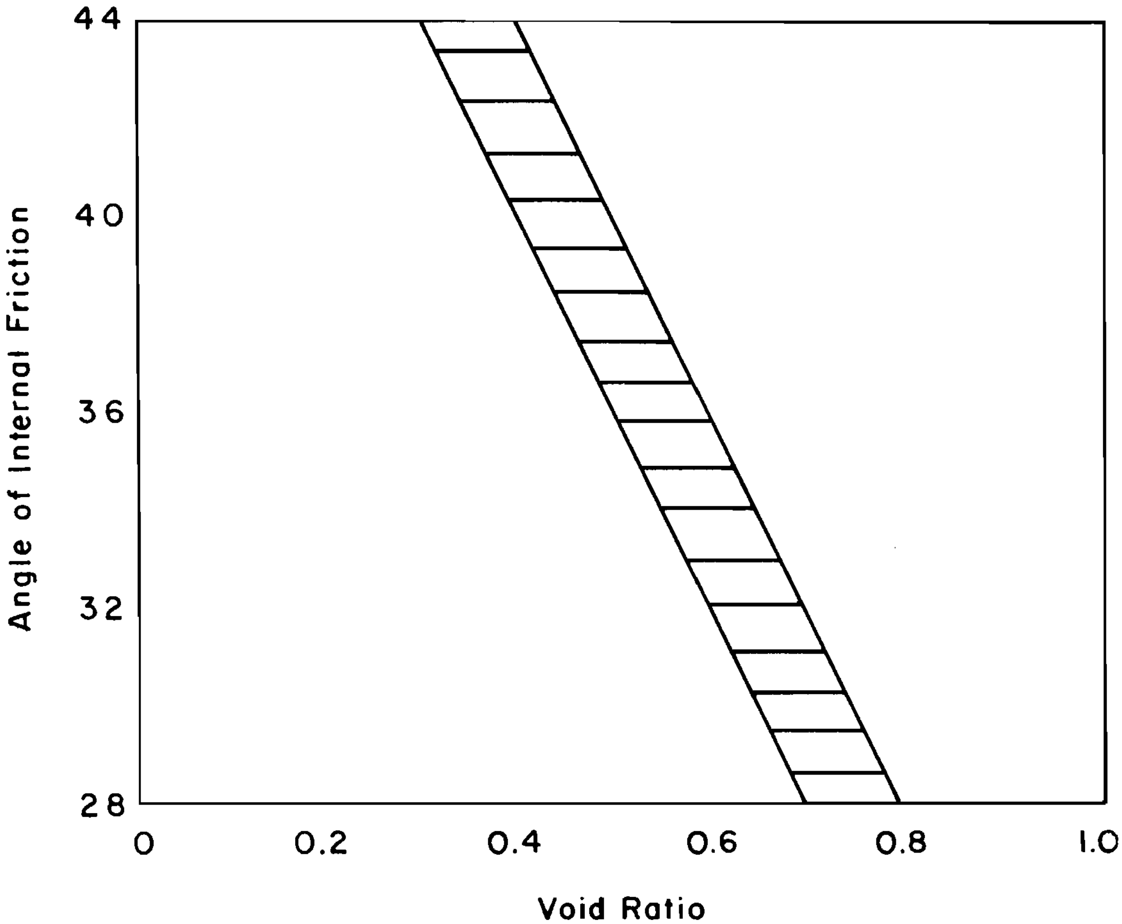


Fig. 5.4. Approximate relationship between angle of internal friction and void ratio for reported sands.

analysis. In cases where just the results of an SPT were reported, the necessary correlations were used. These correlations were used consistantly to obtain ϕ, k, and γ.

ANALYSIS OF TEST RESULTS

Arkansas River

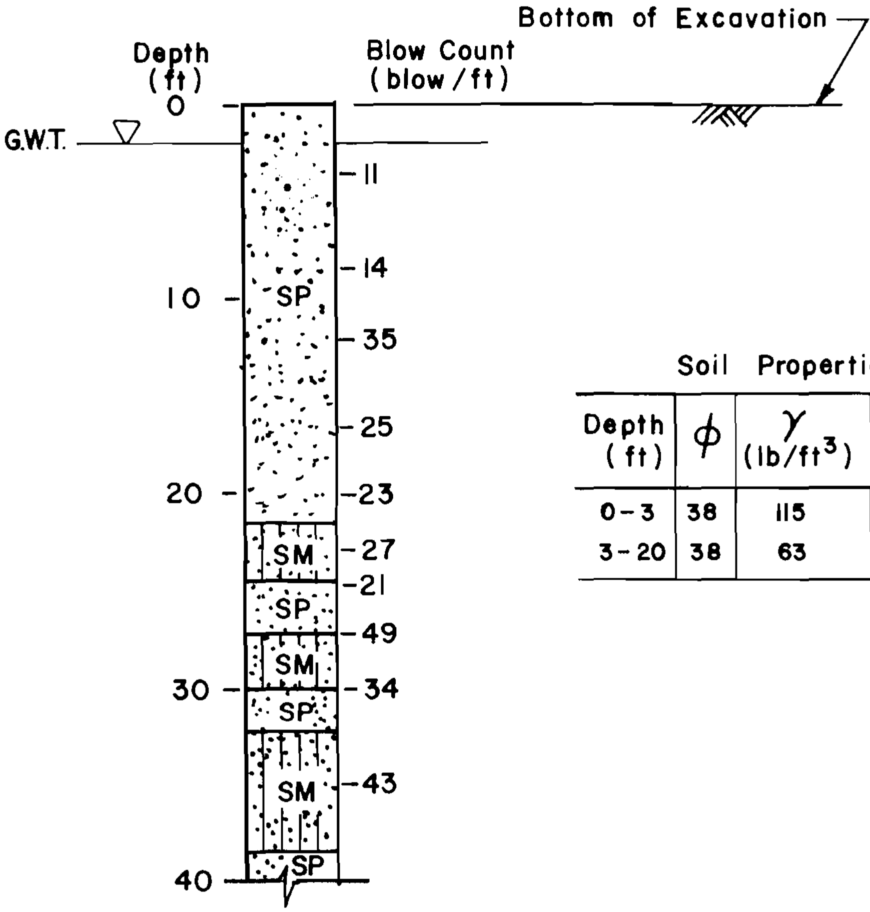
A number of lateral load tests were performed for the Corps of Engineers by Fugro and Associates at a site located on the Arkansas River near Pine Bluff, Arkansas (Alizadeh and Davisson, 1970). The soil conditions were the same for all of the test piles at this site, but a number of analyses were performed due to differences in pile stiffness, pile batter, and loading conditions. The test piles which were analyzed are designated test piles 2, 16, 5, 14, 6, 8, 12, and 13A.

A 20 ft deep test pit was excavated prior to performing soil boring LD4-204, and prior to driving and loading the test piles. Boring LD4-204 was selected as representative of the soil conditions at the test site. The angle of internal friction and k were obtained through correlations with the SPT, shown in Fig. 5.5. The effective unit weight was reported by Mansur and Hunter (1970) to be 63 lb/ft^3, and K_o was taken as 1.0 because 20 ft of soil was removed prior to driving and testing the piles.

Test piles 2 and 16 were modified 16 in. O.D. pipe piles which were installed vertically and loaded statically. Test pile 2 was driven and test pile 16 was jetted. Because the effects of jetting cannot be considered in the sand criteria, the computed results for both test piles are the same. The test setup and pile properties are shown in Fig. 5.6.

The measured and computed results for test piles 2 and 16 are shown in Fig. 5.7. As shown, the computed maximum moments compare favorably with the measured maximum moments for both test piles. The measured and computed deflections are in poor agreement for test pile 2, but compare favorably for test pile 16. The jetting of test pile 16 did not have a significant effect on the maximum moment, but had a considerable effect on the lateral deflection.

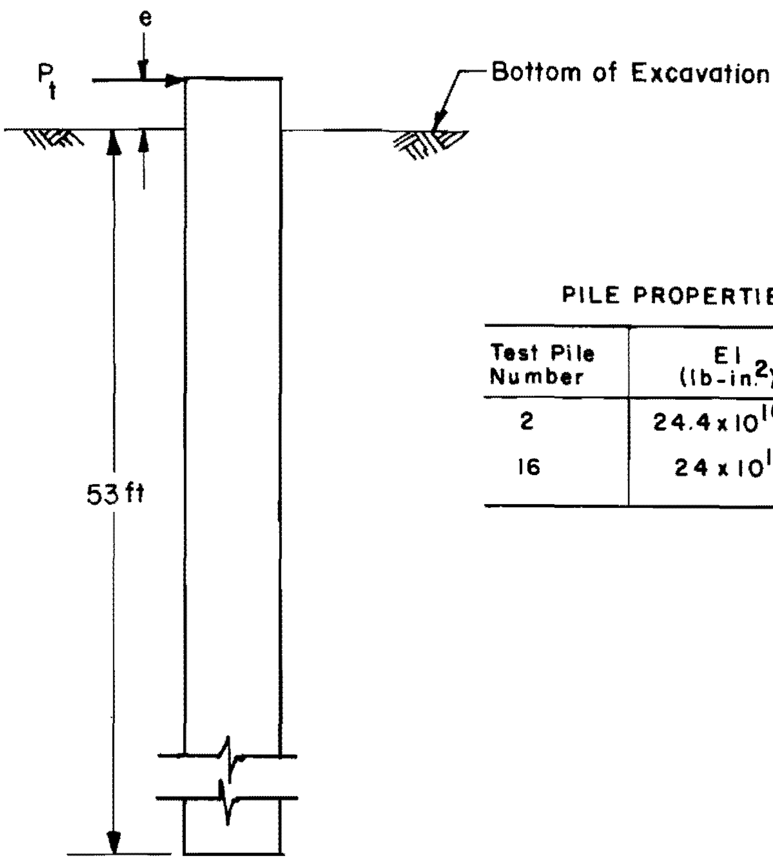
Test piles 5 and 14 were 16 in. square, prestressed, concrete piles



Soil Properties

Depth (ft)	ϕ	γ (lb/ft^3)	k (lb/in^3)	K_o
0-3	38	115	160	1.0
3-20	38	63	90	1.0

Fig. 5.5. Soils information for analysis of tests at Arkansas River.



PILE PROPERTIES

Test Pile Number	EI (lb-in.^{2})	e (ft)
2	24.4×10^{10}	0.1
16	24×10^{10}	0.0

Fig. 5.6. Test setup and pile properties for test piles 2 and 16 at Arkansas River.

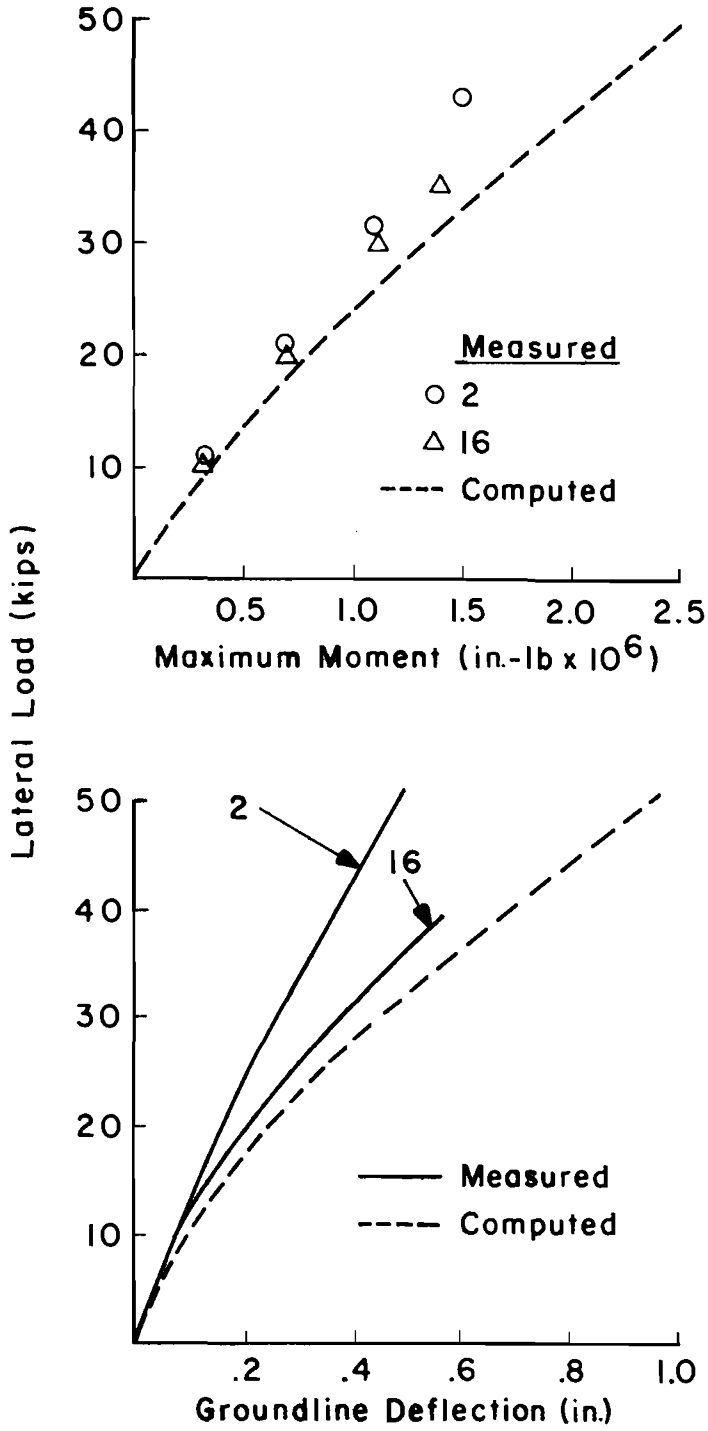


Fig. 5.7. Comparison between measured and computed results for test piles 2 and 16 at Arkansas River.

which were installed vertically and loaded statically. Test pile 5 was driven and test pile 14 was jetted. The test setup and pile properties are shown in Fig. 5.8.

As shown in Fig. 5.9, the computed deflections compare favorably with the measured deflections for both test piles 5 and 14. The analysis is conservative for the driven pile, test pile 5, and slightly unconservative for the jetted pile, test pile 14. In this case, jetting did not appear to have a large effect on the lateral deflection.

Test pile 6 was a 14BP73 pile which was driven vertically. The loading was both short-term and cyclic. The location of the hydraulic jack was not reported, and the load was assumed to be applied at the ground line. The test setup and pile properties are shown in Fig. 5.10.

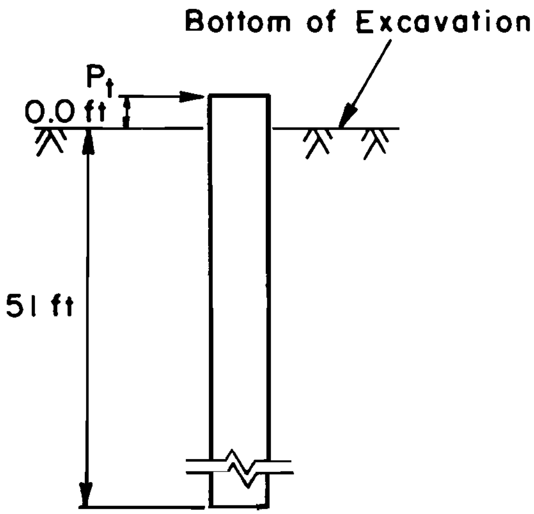
The measured and computed deflections for test pile 6 are compared in Fig. 5.11. As shown, good agreement was obtained between the measured and computed deflections for the static loading of test pile 6. The initial slope of the load-deflection curve does not agree with the experiment and could be brought into better agreement by using a k of 200 lb/in^3 in the analysis.

Only one deflection was reported for cyclic loading. This data point is plotted in Fig. 5.11, along with the complete deflection curve that was computed for cyclic loading. The computed deflection underestimates the measured deflection by 10%.

Test pile 8 was a 40 ft long, Class A timber pile. The pile was driven vertically and loaded statically. The stiffness of this pile was approximately 1/8 the stiffness of the other test piles. The test setup and pile properties are shown in Fig. 5.12.

The results of the analysis, plotted in Fig. 5.13, compare favorably with the measured results, in that both load-deflection curves are similar in shape. The analysis is in error by as much as 40% at small loads, but only 10% at large loads. The large error in the initial portion of the computed curve could be reduced by selecting a larger value for k.

Test piles 12 and 13A were modified 14BP73 piles which were driven,



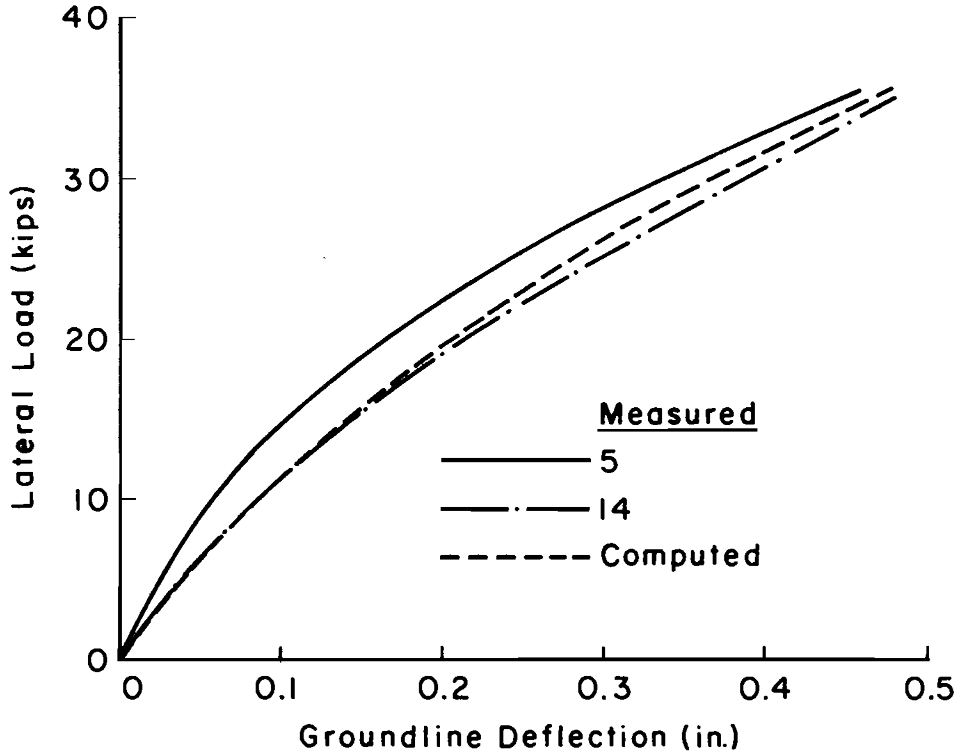


Fig. 5.8. Test setup and pile properties for test piles 5 and 14 at Arkansas River.

Fig. 5.9. Comparison between measured and computed deflections for test piles 5 and 14 at Arkansas River.

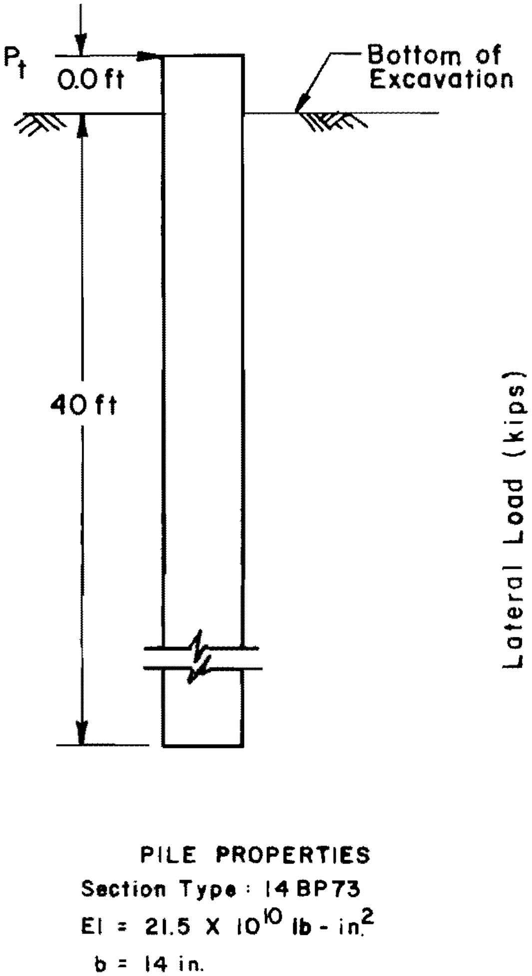


Fig. 5.10. Test setup and pile properties for test pile 6 at Arkansas River.

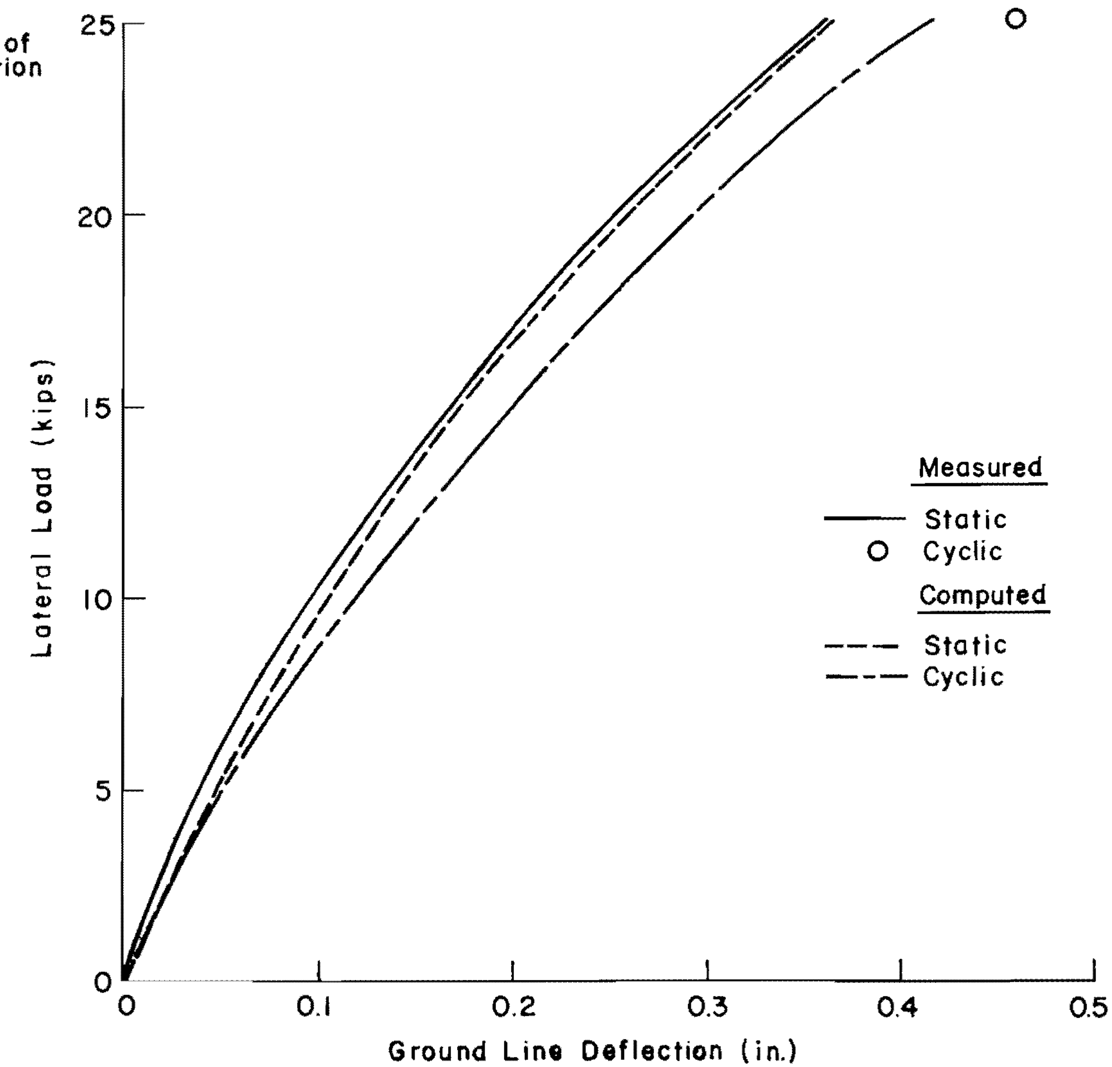


Fig. 5.11. Comparison between measured and computed deflections for test pile 6 and Arkansas River.

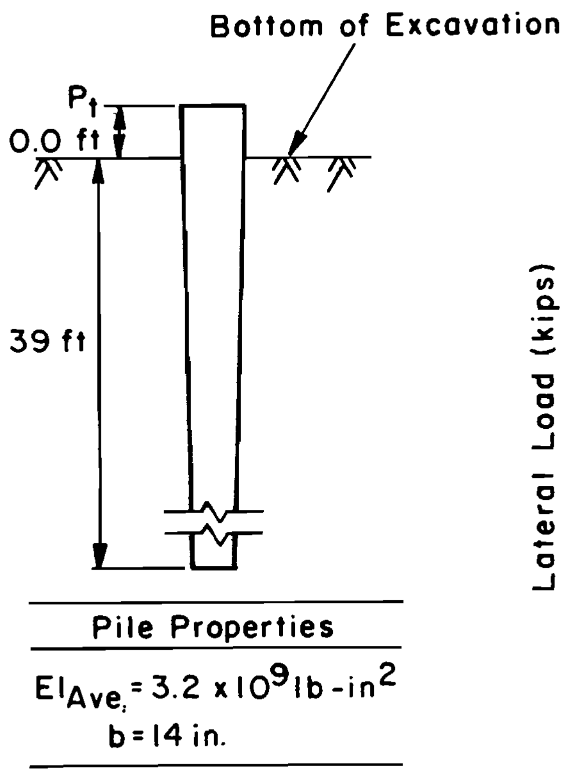


Fig. 5.12. Test setup and pile properties for test pile 8 at Arkansas River.

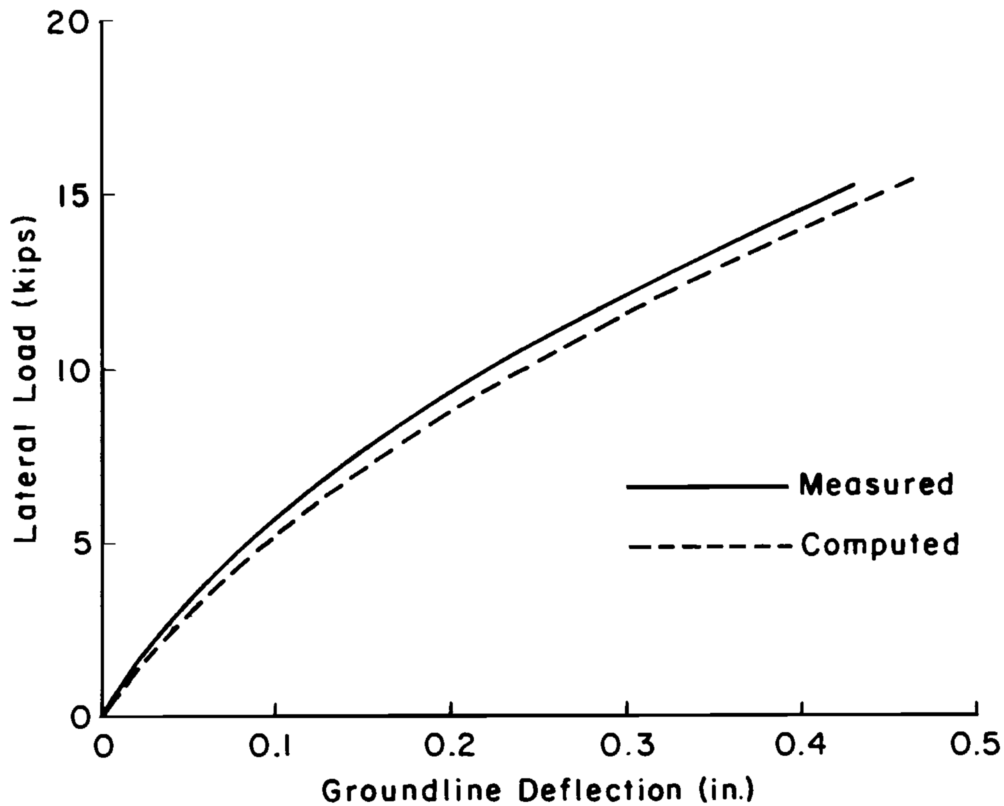


Fig. 5.13. Comparisons between measured and computed deflections for test pile 8 at Arkansas River.

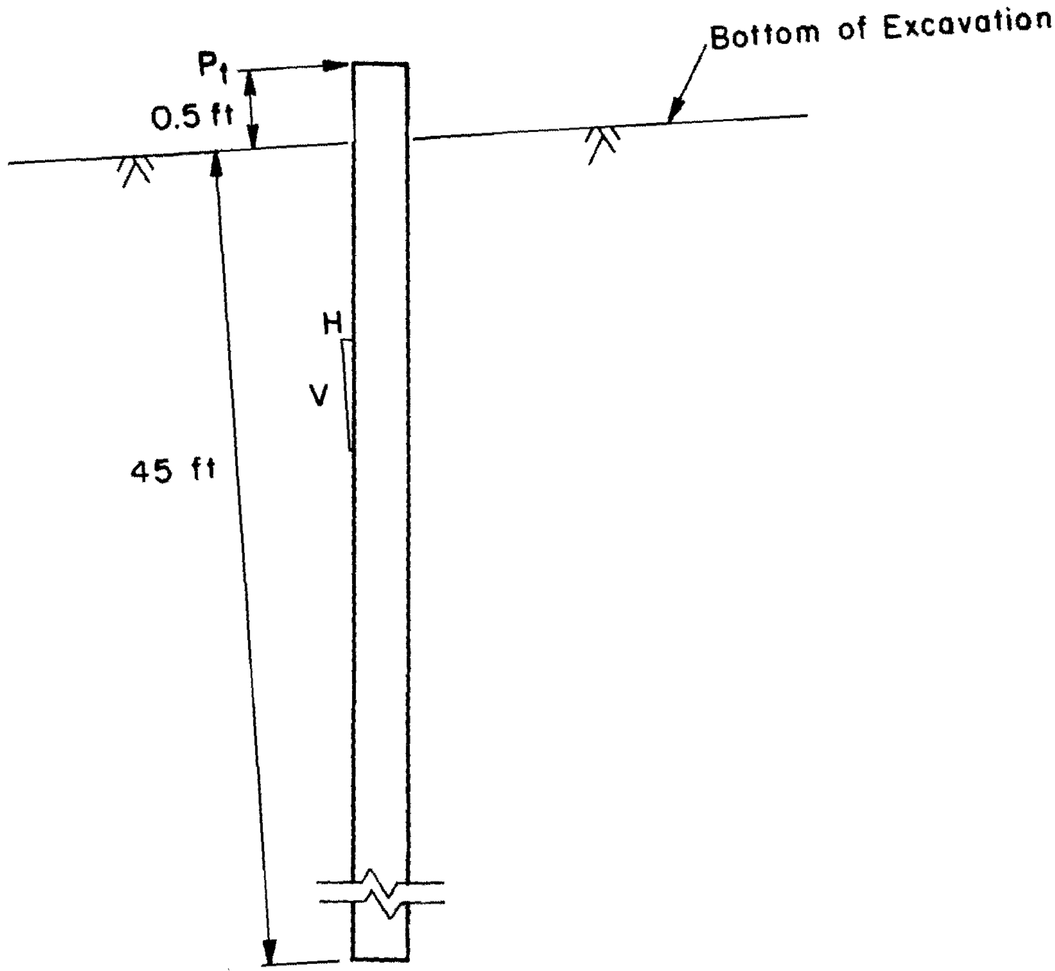
but test pile 12 was installed on a 3 on 1 batter and test pile 13A was installed vertically. The test setup and pile properties are shown in Fig. 5.14.

The sand criteria were developed from the results of tests on vertical piles. To account for the batter of test pile 12, a curve, shown in Fig. 5.15, and presented by Kubo (1962), was used. Kubo found that the shape of the p-y curve was affected by both the direction and angle of the batter. For an "in" batter of 3 on 1, Kubo suggests an adjustment factor of 1.55. This adjustment factor was used to modify k and p_u.

The measured and computed results for test piles 12 and 13A are compared in Fig. 5.16. As shown, the difference between the measured and computed maximum moments are approximately 20% for test pile 12 and approximately 50% for test pile 13A. This large error for test pile 13A only occurred at a lateral load of 76 kips.

The measured and computed deflections are in good agreement for test pile 12. The adjustment factor suggested by Kubo worked very well. The comparison between the measured and computed load-defelction curves for test pile 13A is fair. The initial slope of the computed load-deflection curve for test pile 13A is too small, and an increase of k would yield better results.

For all of the Arkansas River tests which were analyzed, the initial slope of the computed load-deflection curve is too small. The value of k which is necessary to bring the measured and computed initial slopes into agreement was between 200 and 300 lb/in^3. This value is much larger than the recommended values for a D_r of 65%. Overconsolidation could be a possible reason for the large difference between the recommended value of k and that required for agreement with the experimental results. Overconsolidating the sand would increase the in-situ lateral stresses, which would thus increase the initial soil modulus. Therefore, the values of k suggested by Reese et al. (1974) are not really applicable, but there is not a sufficient amount of information to relate quantitatively the effects of overconsolidation to k.



Pile Properties

Test Pile Number	EI (lb-in.^{2})	Batter (V:H)
12	2.48×10^{10}	3:1
13A	2.79×10^{10}	none

Fig. 5.14. Test setup and pile properties for test piles 12 and 13A.

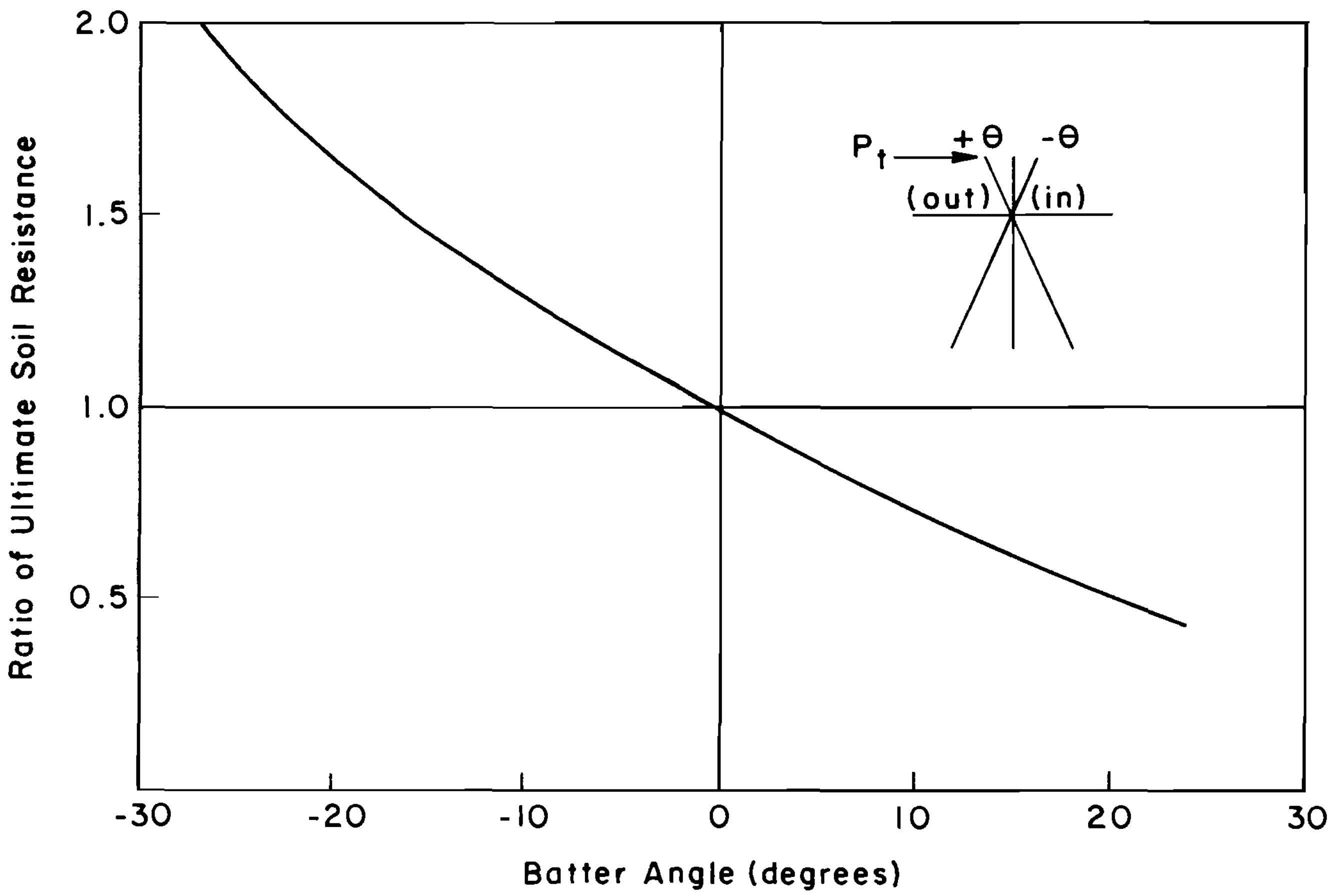


Fig. 5.15. Modification factors for sand p-y curves for battered piles (Kubo, 1962).

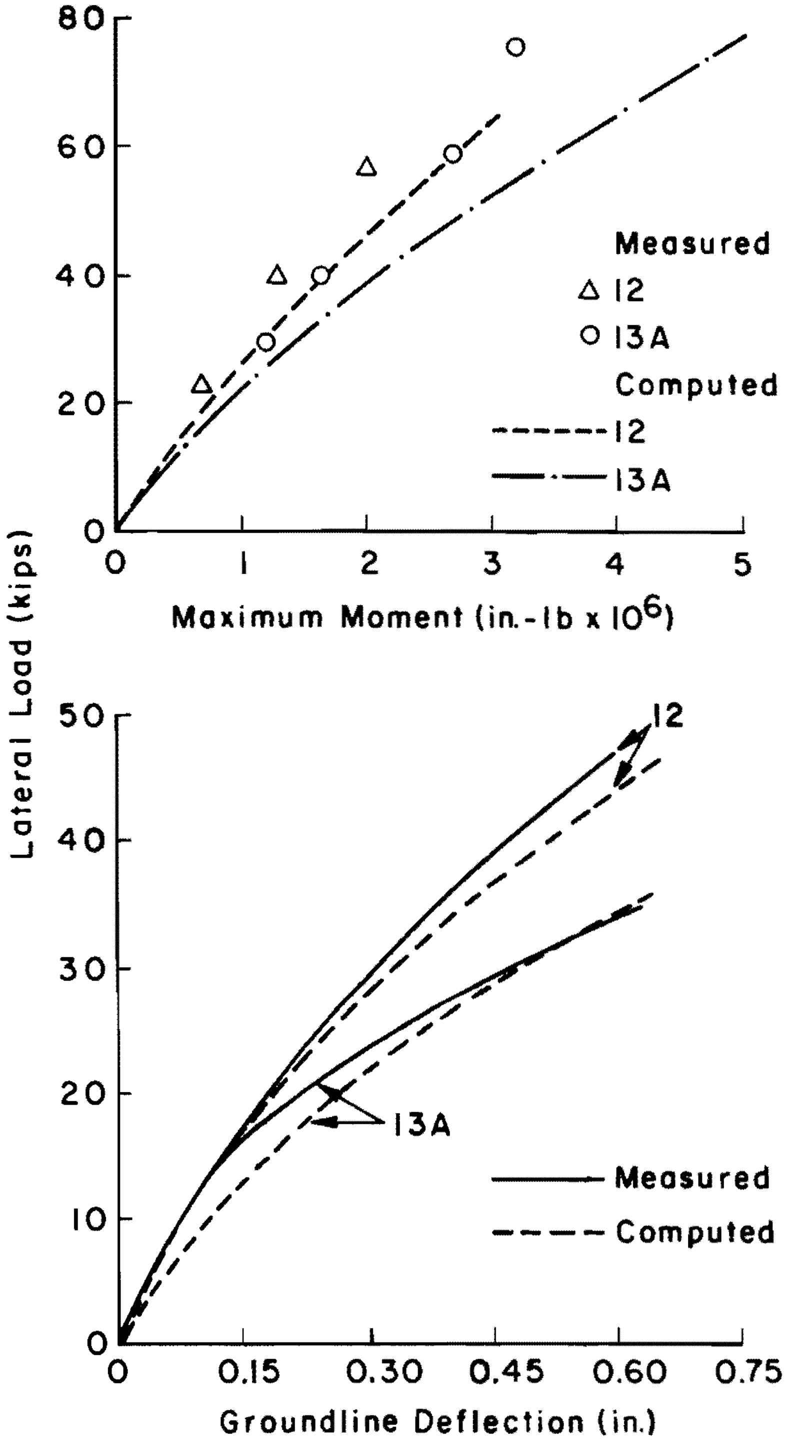


Fig. 5.16. Comparison between measured and computed results for test piles 12 and 13a at Arkansas River.

Apapa

Short-term static lateral load tests were performed on Raymond steptapered piles near Apapa, Nigeria (Colemen, 1968; Coleman and Hancock, 1972). Test piles 1 and 2 were tested under the same conditions; therefore, only one analysis was performed. The test piles were driven vertically, and the steel shells were then filled with reinforced concrete. The test setup and pile properties are shown in Fig. 5.17.

The soil at the site consisted of a 5 ft thick layer of hydraulically placed sand overlying a layer of soft organic clay. The sand layer will mainly control the pile behavior, but the clay layer will also have some influence. The angle of internal friction of the sand was obtained from laboratory triaxial tests, and c for the soft organic clay was obtained from in-situ vane tests. The soil properties which were used in the analysis are shown in Fig. 5.17.

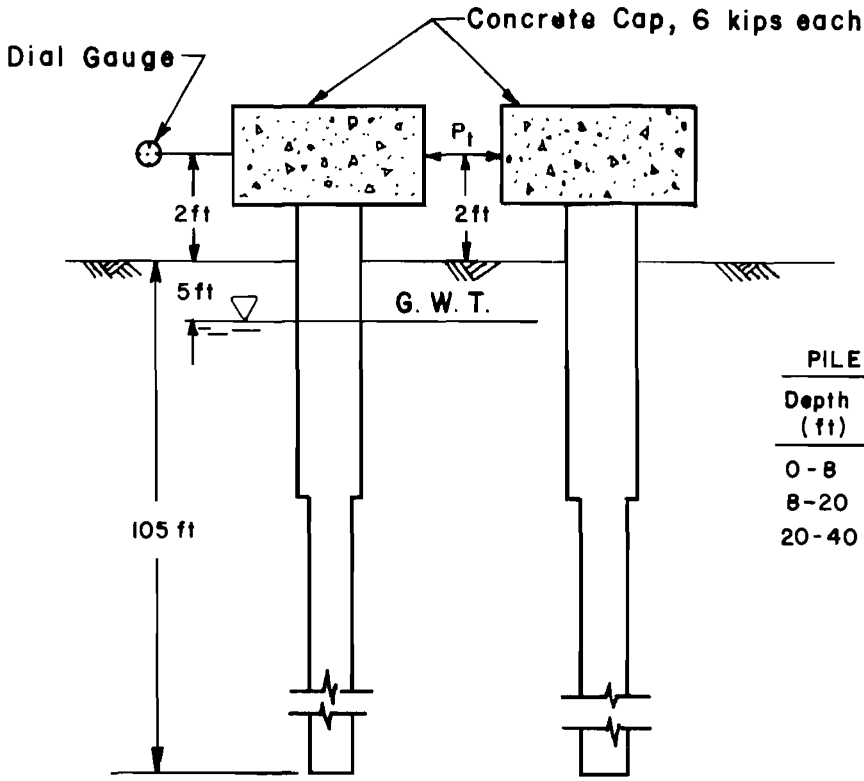
The measured and computed deflections for test piles 1 and 2 are compared in Fig. 5.18. As shown, the measured and computed load-deflection curves are dissimilar in shape. The measured load-deflection curves have a large curvature at approximately 11 kips. The computed curve agrees fairly well with the measured curves before this load, but the curves diverge at larger loads. The initial slopes of the measured and computed load-deflection curves agree very well.

Bailly

Lateral load tests were performed on two 14BP89 piles at the site of a proposed nuclear power plant (Bergstrom, 1974). The piles were driven vertically, and the lateral loads were applied 1.5 ft above the ground surface to the free-head piles. The test piles were loaded incrementally up to a maximum load of 39 kips, unloaded to 0 tons, and then cycled 25 times at 22 kips.

The soil at the site was a fine sand, loose to moderately dense. The closest boring to the test piles was B-6, which was used to obtain the soil properties. The soil properties and boring log are shown in Fig. 5.19.

The measured and computed deflections for test piles TP7 and TP8 are



PILE PROPERTIES

Depth (ft)	b (in.)	EI (lb-in^2)
0-8	17.4	7.8 X 10^9
8-20	16.4	7.0 X 10^9
20-40	15.4	6.5 X 10^9

Soil Properties

Depth (ft)	Soil Type	ϕ	C (lb/ft^2)	γ (lb/ft^3)	k (lb/in.^{3})	K_o	ϵ_{50} (in./in.)
0-3	Sand Fill	41	—	120	225	.4	—
3-5	Sand Fill	41	—	68	130	.4	—
5-20	Peat and Soft Clay	—	500	30	60	—	.02

Fig. 5.17. Information for the analysis of tests at Apapa.

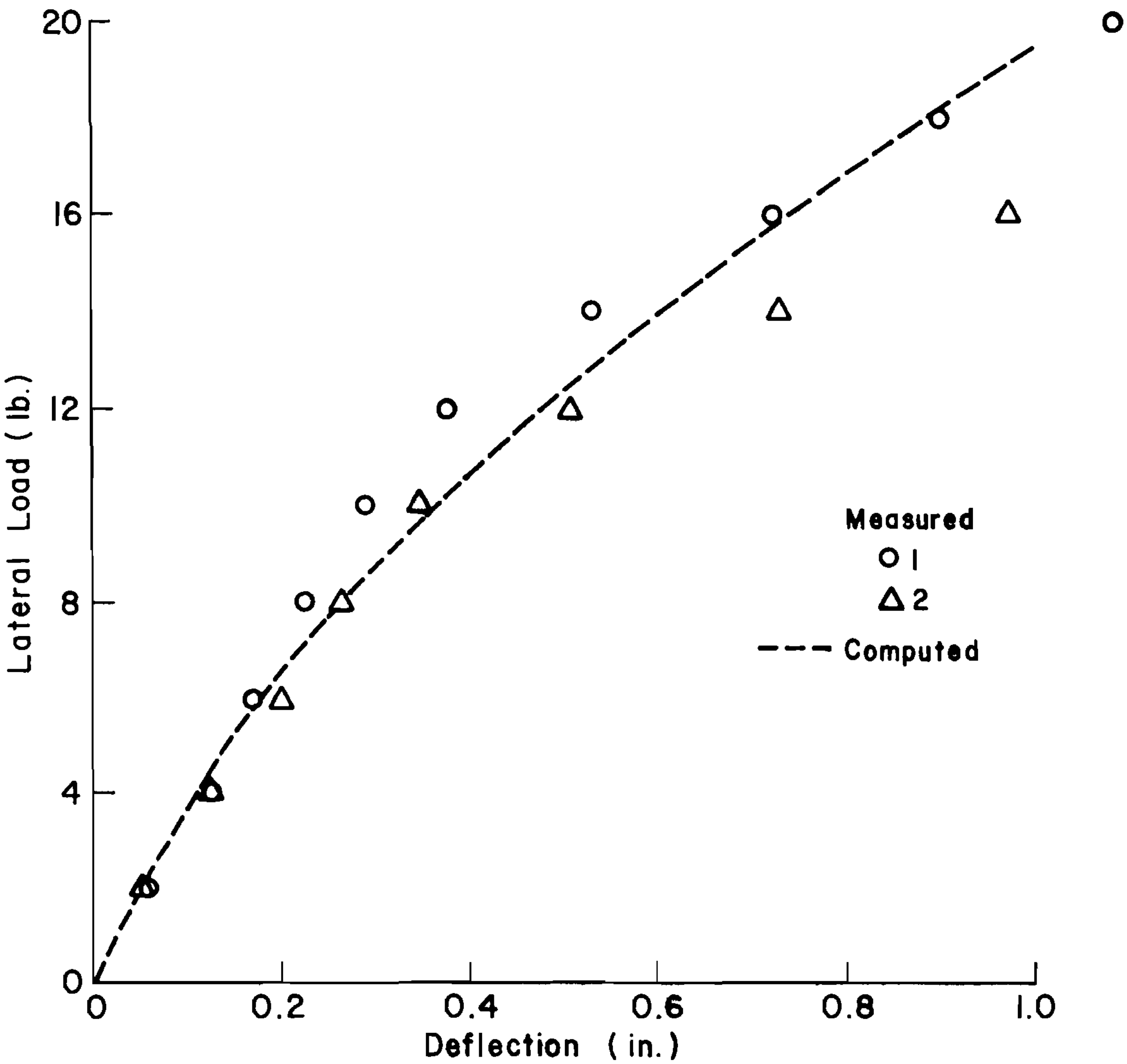
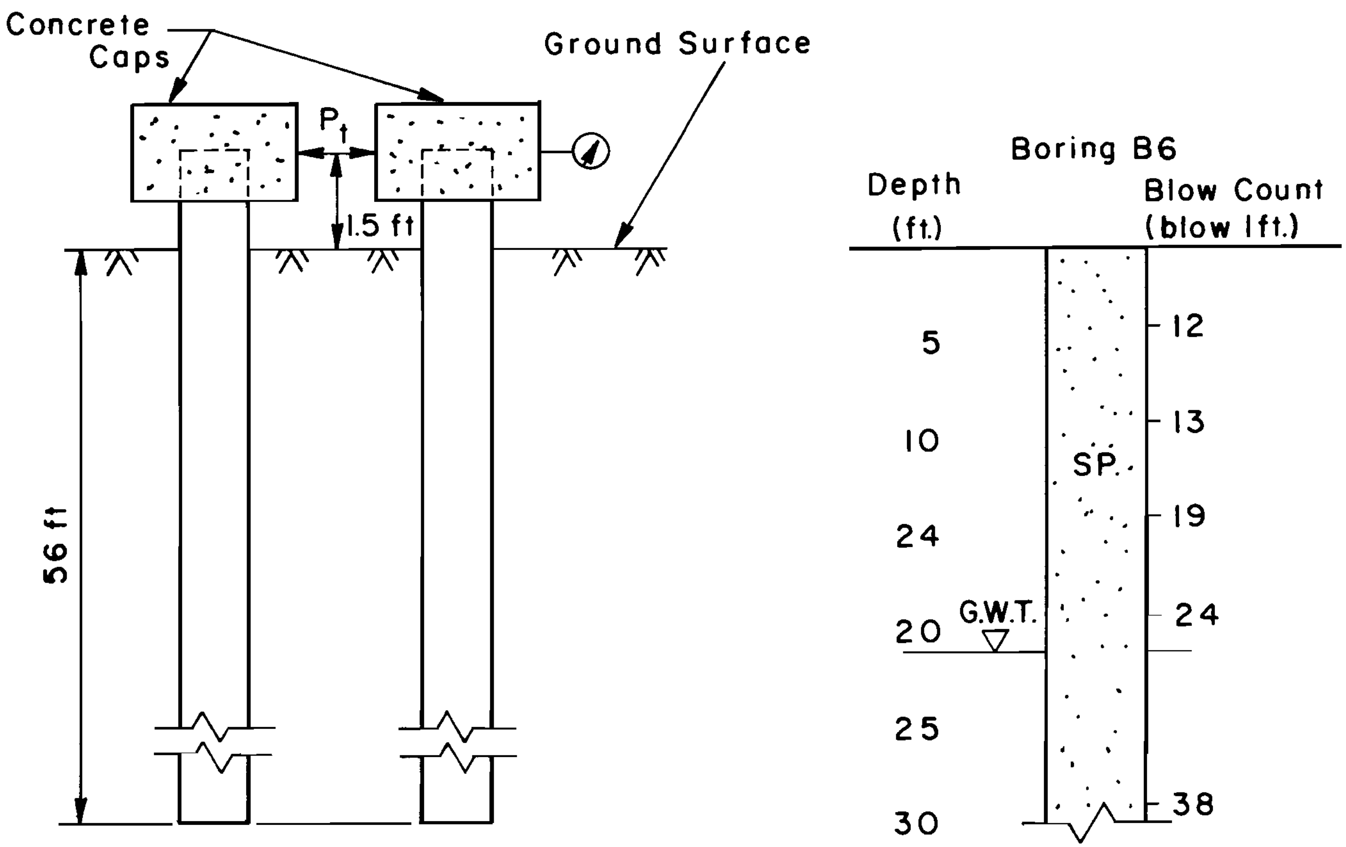


Fig. 5.18. Comparison between measured and computed deflections for test at Apapa.



Pile Properties
$EI = 9.78 \times 10^9$ lb-in.^{2}
b = 14 in.

Soil Properties				
Depth (ft)	ϕ	γ (lb/ft^3)	k (lb/in^3)	K_o
0-56	37	115	110	0.4

Fig. 5.19. Information for the analysis of tests at Bailly.

shown in Fig. 5.20. Because the pile and soil properties for both piles were the same, only one computer run was made for the static loading and one for the cyclic loading. As shown, the computed deflections for the static-loading case are in excellent agreement with the measured deflections. The only point on the deflection curve which was not in agreement was for a lateral load of 39 kips, where the computed deflection exceeded the measured deflection by 12%. The initial slope of the computed load-deflection curve is in perfect agreement with the measured deflections, which indicates that the correct value for k was selected for this analysis.

The measured cyclic deflection in Fig. 5.20 was obtained for 25 cycles of loading. As shown, the computed load-deflection curve for cyclic loading agrees closely with the measured deflections. The cyclic loading sand p-y criteria were developed from the results of a pile test in submerged sand, but the criteria apparently work equally well for the cyclic loading of a pile in sand above the water table.

Florida

A short-term static lateral load test was performed by the Florida Power and Light Co. (Davis, 1977). The foundation member was a rigid 56 in. O.D. steel tube which was vibrated to a depth of 26 ft. An ellipically shaped utility pole was embedded in the upper portion of the steel tube to a depth of 4 ft below the ground surface. The utility pole was rigidly attached to the inside of the steel tube with gusset plates, and the annular space between the utility pole and tube was filled with concrete. The weight of the utility pole was 10.7 kips. Lateral loads were applied 51 ft above the ground surface, as shown in Fig. 5.21.

The soil profile, shown in Fig. 5.21, consisted of 13 ft of medium dense sand overlying a stiff to very stiff sandy, silty clay layer. The standard correlations, as outlined in the beginning of this chapter, were used to obtain the appropriate soil properties for the sand layer, but a method for obtaining the shear strength based on the SPT had to

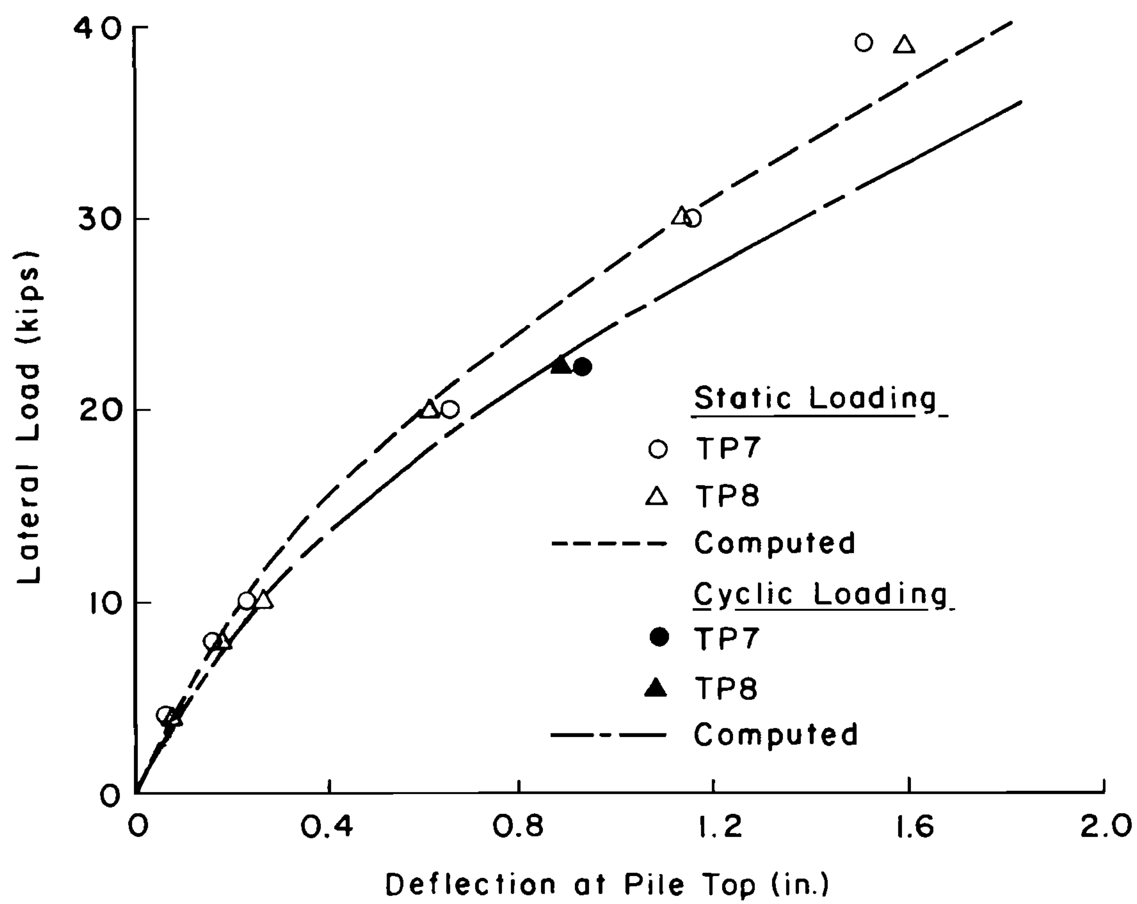
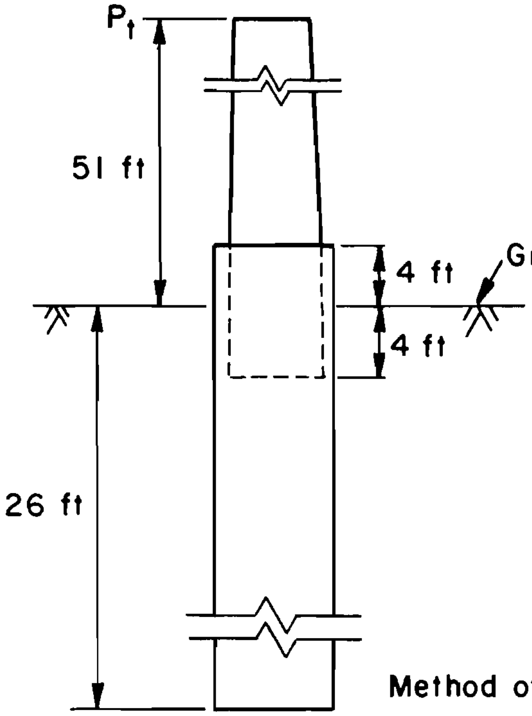
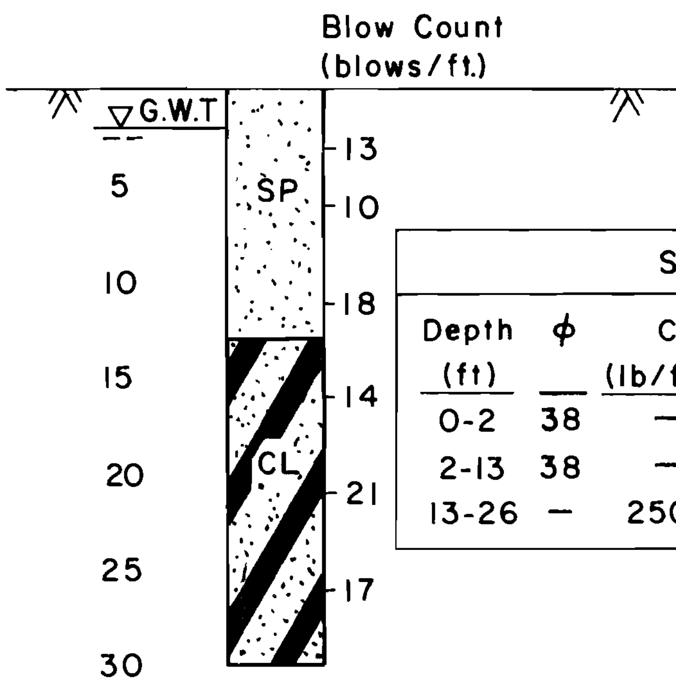


Fig. 5.20. Comparison between measured and computed deflections for test at Bailly.



Pile Properties		
Depth (ft)	b (in.)	EI (lb-in^2)
0-4	56	1.77×10^{12}
4-26	56	8.8×10^{11}

Method of Installation: Vibrated Into Place



Soil Properties						
Depth (ft)	ϕ	c (lb/ft^2)	γ (lb/ft^3)	k (lb/in^3)	K_o	ϵ_{50}
0-2	38	—	115	160	0.4	—
2-13	38	—	60	100	0.4	—
13-26	—	2500	60	1000	—	0.005

Fig. 5.21. Information for the analysis of test in Florida.

be utilized. For a blow count between 15 and 30 blows/ft, shear strengths of 2000 to 4000 lb/ft^2, as recommended by Terzaghi and Peck (1967), were selected. Obtaining the shear strength in this manner is not very accurate, but no other information on the properties of the clay was available.

The measured and computed deflections for the test pile are compared in Fig. 5.22. As shown, the computed deflections compare well with the measured deflections except at low load levels. At low load levels, the computed deflections are conservative, indicating that the initial portion of the p-y curve was not stiff enough. In this case, it is not possible to select an appropriate value of k for the sand, because both layers of material are influencing the pile's behavior.

Hydraulic Fill

Gill (1969) reported the results of four lateral load tests performed on free-headed statically loaded pipe piles. The piles were of different stiffnesses and were all embedded to a sufficient depth so that they behaved as flexible members.

The soil at the site was mainly an old hydraulic fill which had been placed in the 1940's (Gill, 1969). A compacted granular surface had reportedly been placed over the hydraulic fill. This compacted surface could account for the extremely high blow count of 58 blows/ft at a depth of 2 ft, shown in Fig. 5.23. Below 2 ft, the blow count decreased rapidly until it reached 16 blows/ft at a depth of 4.5 ft. No information concerning the SPT resistance of the material was given below 4.5 ft, and it was assumed that the relative density was constant below this depth.

As shown in Fig. 5.24a, the computed deflections are approximately twice as large as the measured deflections for test pile P9. Similar results were obtained for test pile P10. The computed deflections for test piles P11 and P12, shown in Fig. 5.24b, are in much better agreement with the measured deflections. For test piles P11 and P12, the error in the computed deflections were less than 20%.

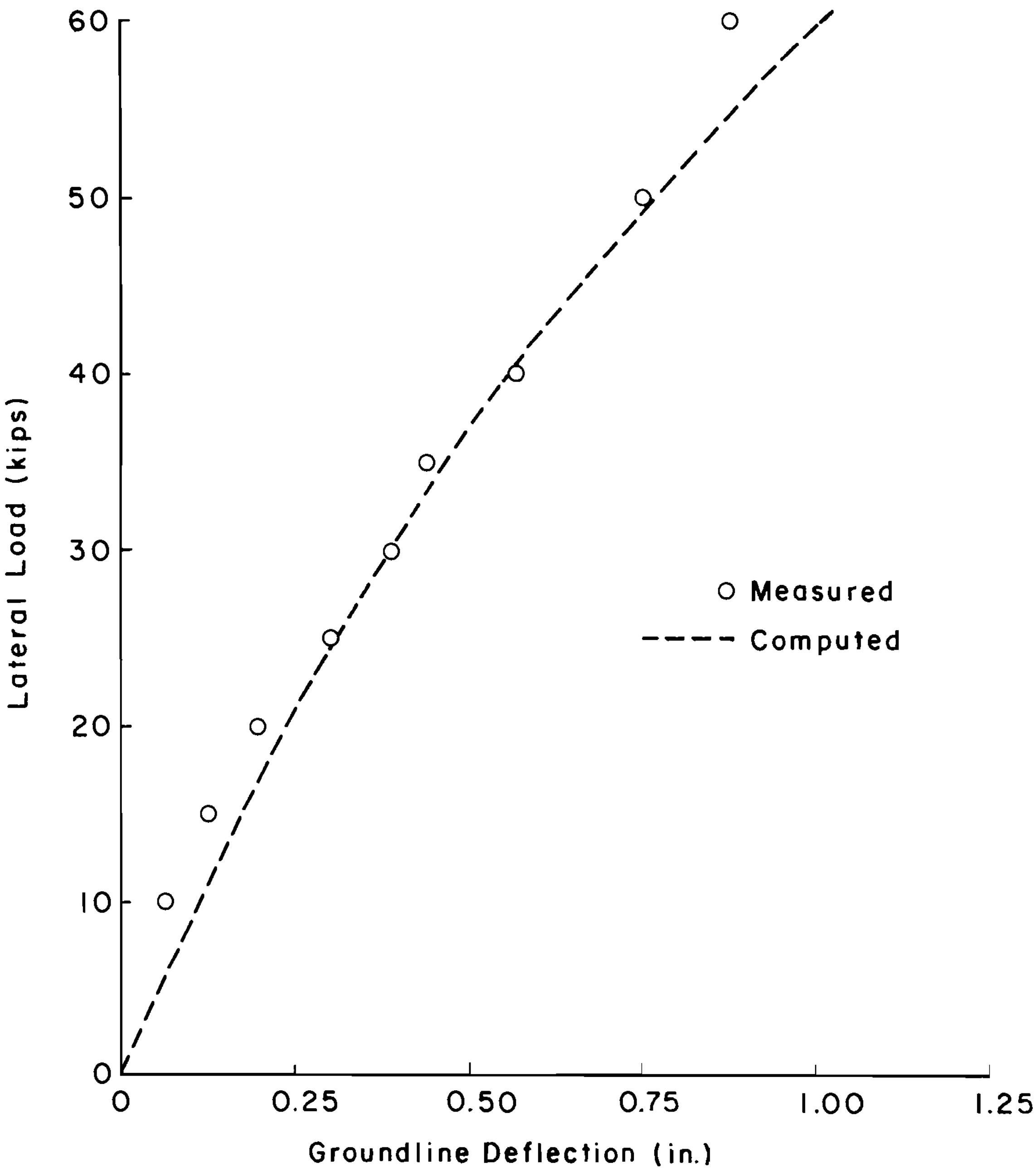
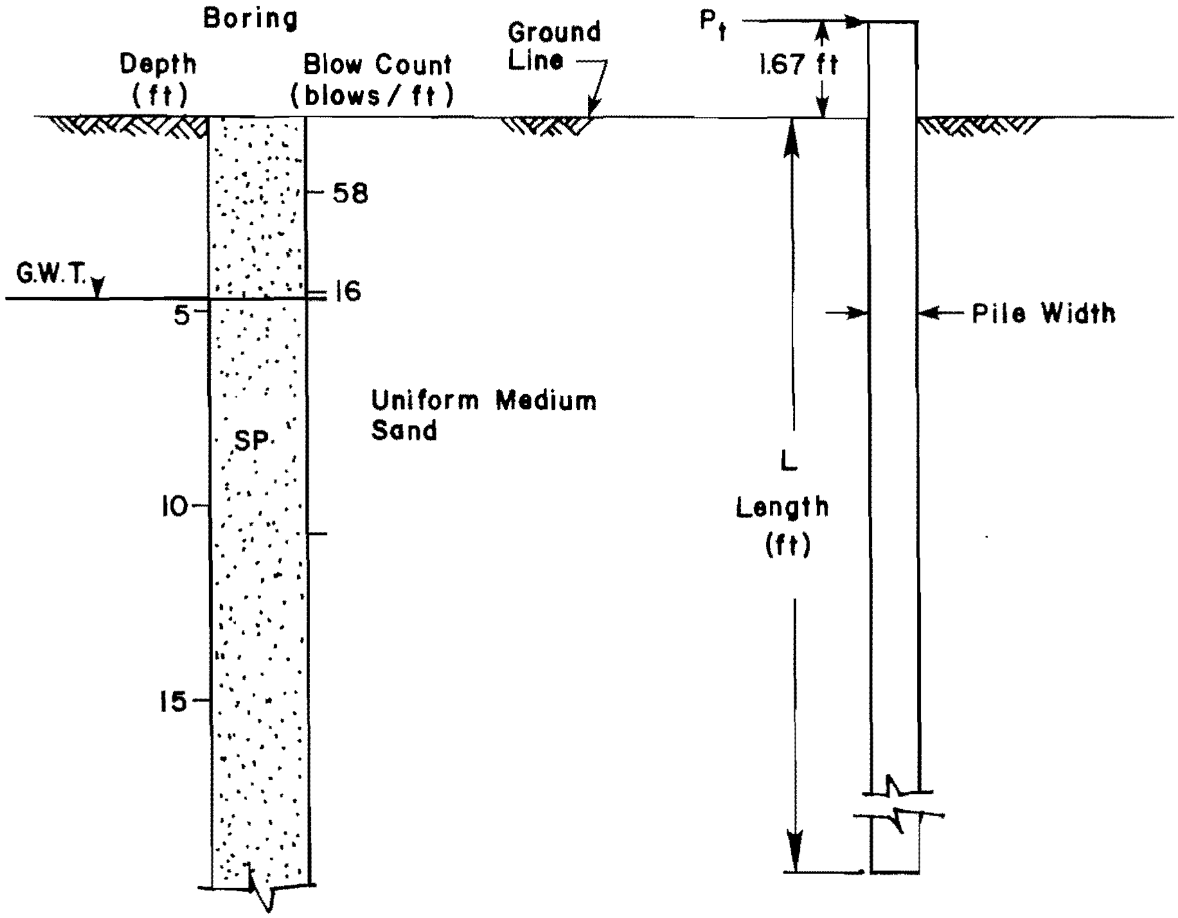


Fig. 5.22. Comparison between measured and computed delfections for test in Florida.



Soil Properties				
Depth	ϕ	γ (lb/ft^3)	k (lb/in^3)	K_o
0-2	41	125	275	0.4
2-4.5	40	115	175	0.4
4.5-32	38	60	110	0.4

Pile Properties			
Test Pile Number	b (in.)	EI ($lb.-in^2$)	L (ft)
P9	4.75	2.17×10^8	18
P10	8.62	2.17×10^9	24
P11	12.75	7.46×10^9	30
P12	16.00	1.69×10^{10}	30

Fig. 5.23. Information for the analysis of tests in hydraulic fill.

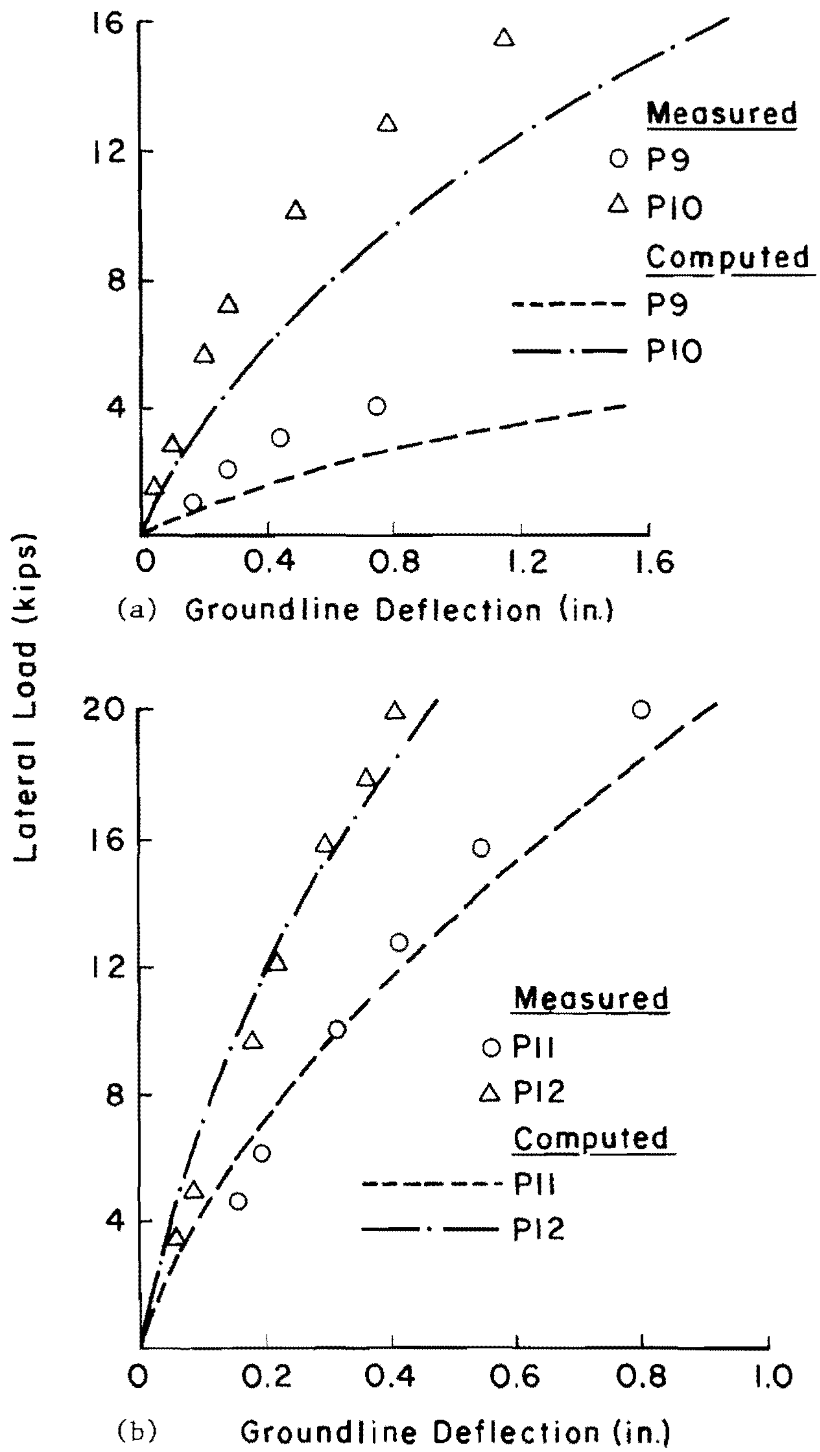


Fig. 5.24. Comparison of measured and computed deflections for tests in hydraulic fill. (a) test piles P9 and P10: (b) test piles P11 and P12.

Mason and Bishop

Mason and Bishop (1954) performed lateral load tests on a free-headed pile, shown in Fig. 5.25. The pile was a 16WF36 section with 3/8 in. plates welded transversely between the flanges. The test pile was loaded statically with the load being applied at the soil surface.

The method of pile installation consisted of erecting the pile, and then densifying the sand around the guyed pile until 40 ft of the pile was embedded in the fill. The sand was placed at a density of 98 lb/ft^3 at a moisture content of 3%. After the sand had been placed, an oscillator was strapped to the pile top and the pile was vibrated. This action would cause further densification of the sand, but no additional soil tests were performed to determine the effect of this action. The reported angle of internal friction was 35°, but the additional densification would produce an increase in ϕ. A value of 38° was selected to perform the analysis. The other soil properties used in the analysis are shown in Fig. 5.26.

Deflections and earth pressures along the pile were measured for lateral loads of 10 and 18.5 kips. The earth pressures were measured with a friction device which is not considered to be reliable. A surveyor's level was mounted directly above the test pile to a fixed reference beam and used to obtain the lateral deflections by monitoring the movement of scales mounted inside the hollow test pile.

The computed deflections, shown in Fig. 5.26a overestimate the measured deflection by 80% for a load of 10 kips, and by 15% for a load 18.5 kips. The general shape of the soil resistance curves, shown in Fig. 5.26b, are very similar. The computed and measured maximum soil resistances are nearly identical for a load of 10 kips.

University of Texas

Parker and Reese (1971) performed short-term static lateral load tests on 2 in. O.D. pipe piles. A total of six tests were performed in the same sand deposit, but only the results of test pile 3-L will be discussed.

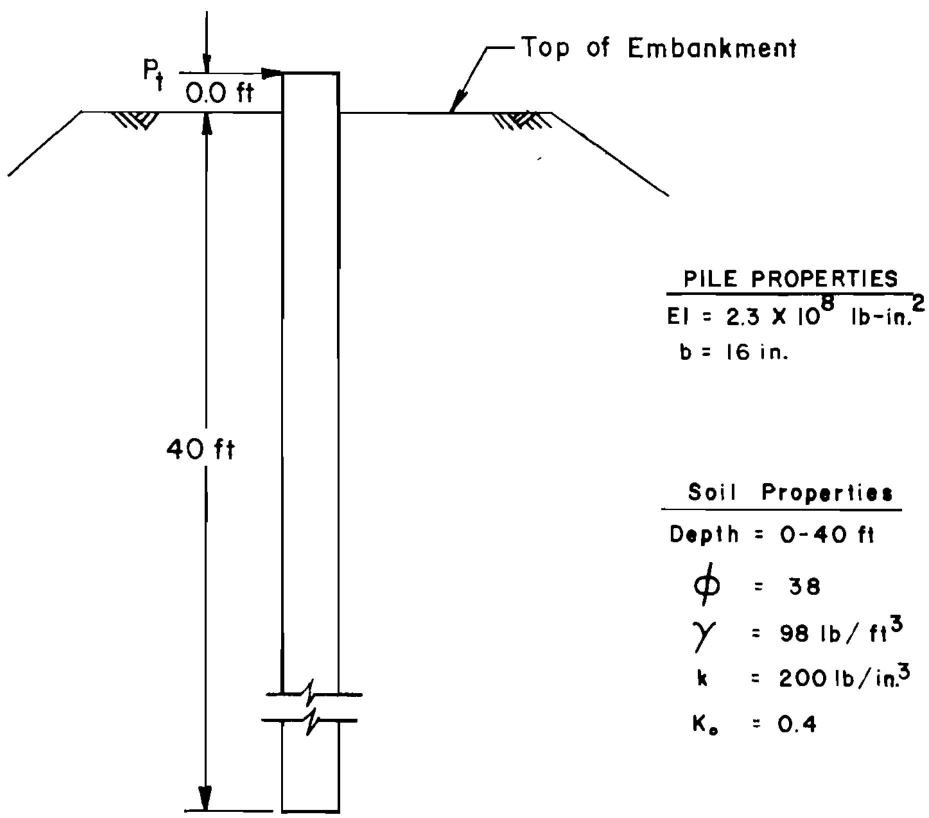


Fig. 5.25. Information for the analysis of test by Mason and Bishop.

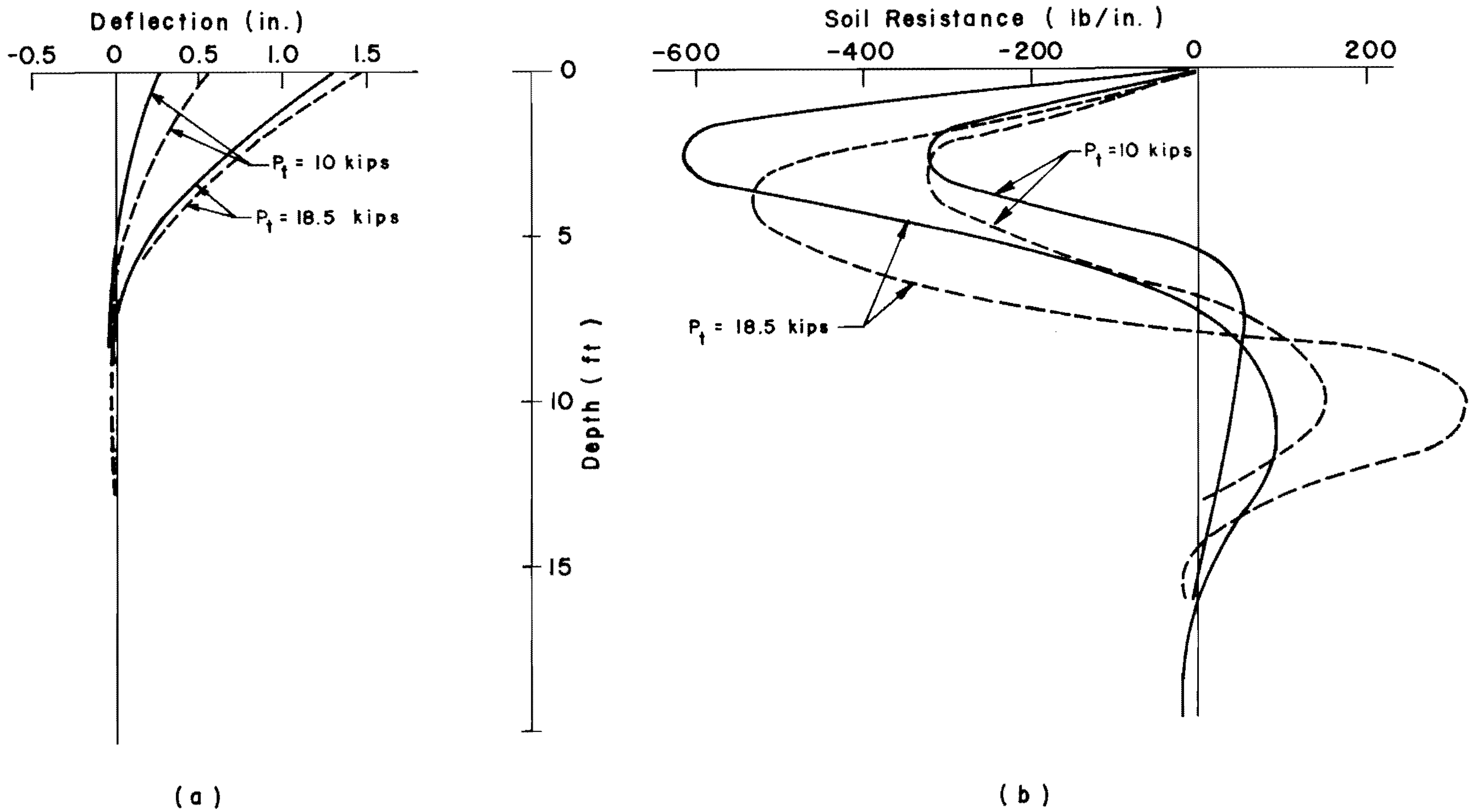


Fig. 5.26. Comparison between measured and computed results for test by Mason and Bishop.

Test pile 3-L was fully instrumented to measure bending strains along its entire length. The pile was calibrated prior to testing so that the bending moments could be accurately determined from the measured bending strains. The pertinent pile properties and test set-up are shown in Fig. 5.27.

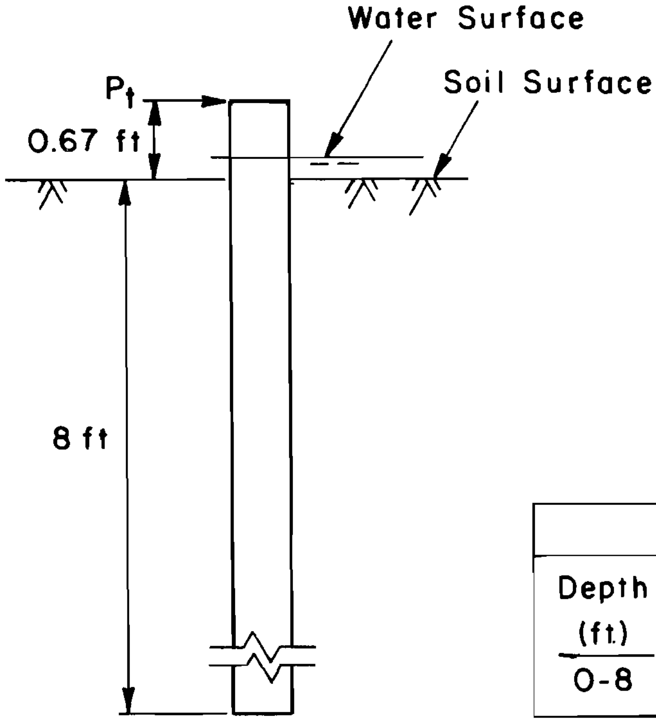
The test pile was placed vertically in the test pit prior to placing the sand. The sand was then densified in layers to obtain a deposit of a uniform density. A large number of in-situ density tests were performed around the test piles during placement of the sand, yielding an average dry density of 100 lb/ft^3. A number of direct shear tests and triaxial tests on saturated samples were performed, yielding average values for $\bar{\phi}$ of 40 and 44°, respectively. Parker and Reese (1971) indicated that the average value of 44° from the triaxial tests was more representative of the actual in-situ angle of internal friction.

The measured and computed results for test pile 3-L are shown in Fig. 5.28. As shown, the computed maximum moments compare quite favorably with the measured maximum moments. The largest error in the analysis was only 4%, which occurred at a lateral load of 450 lb. Good agreement was obtained between the computed and measured lateral deflections. The computed deflections were generally conservative by 10%. At low load levels, the error was much less, which indicates that the correct value for k was used in the analysis.

The results of this analysis are very instructive in showing the flexibility of the sand p-y criteria. The shape of the p-y curves using these criteria are heavily dependent on b. The parameters y_u, y_m, p_u, and p_m are all affected by b. The criteria were developed from tests on a 24 in. pipe pile, but the analysis worked very well in predicting the behavior of a 2-in.-diameter pile.

EVALUATION OF p-y CRITERIA

The measured versus computed deflections from the results of the analyses performed on piles in sand are plotted in Fig. 5.29. As shown, the computed deflections are in good agreement with the measured



Pile Properties

$EI = 5.55 \times 10^6$ lb-in.^{2}
b = 2 in.

Soil Properties

Depth (ft)	ϕ	γ (lb/ft^3)	k (lb/in^3)	K_o
0-8	44	62	130	0.4

Fig. 5.27. Information for the analysis of model test at The University of Texas.

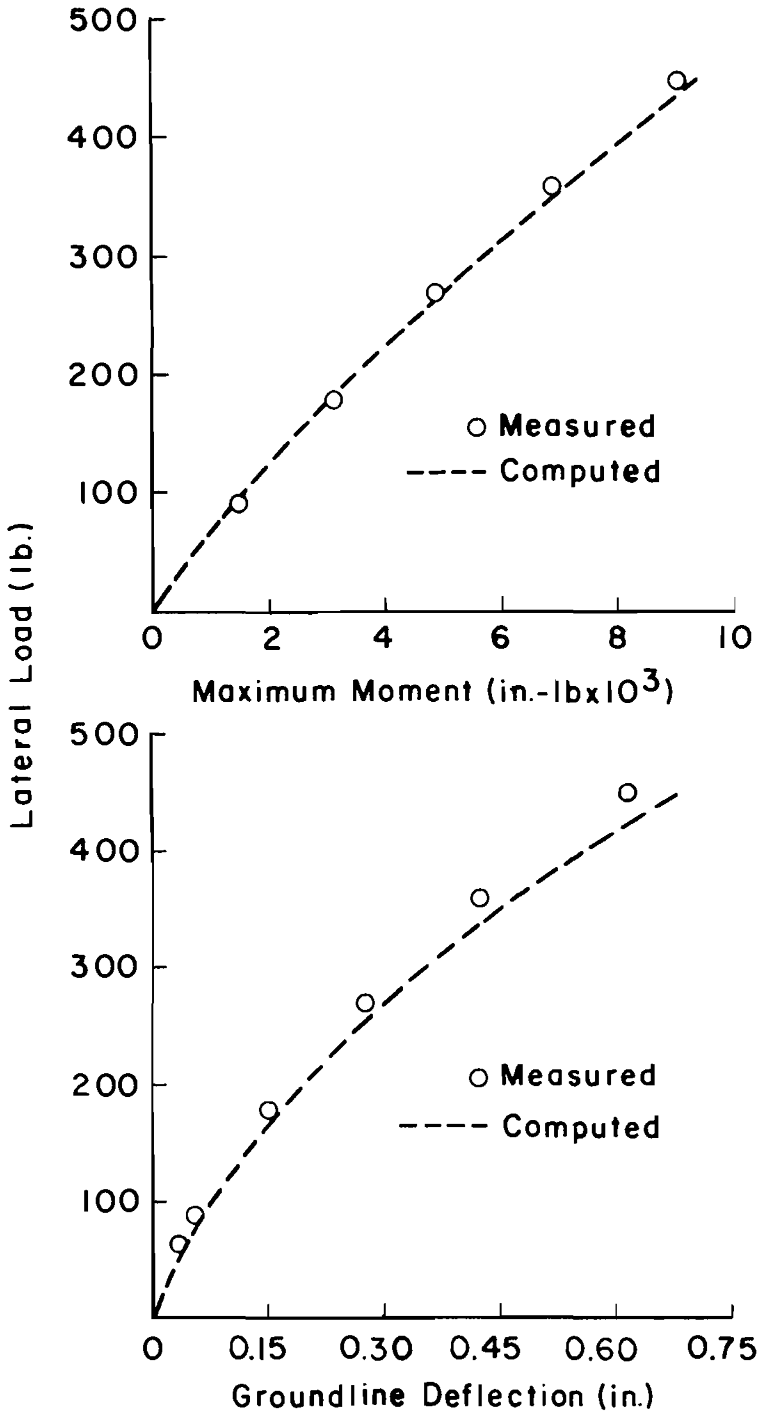


Fig. 5.28. Comparison between measured and computed results for model tests at The University of Texas.

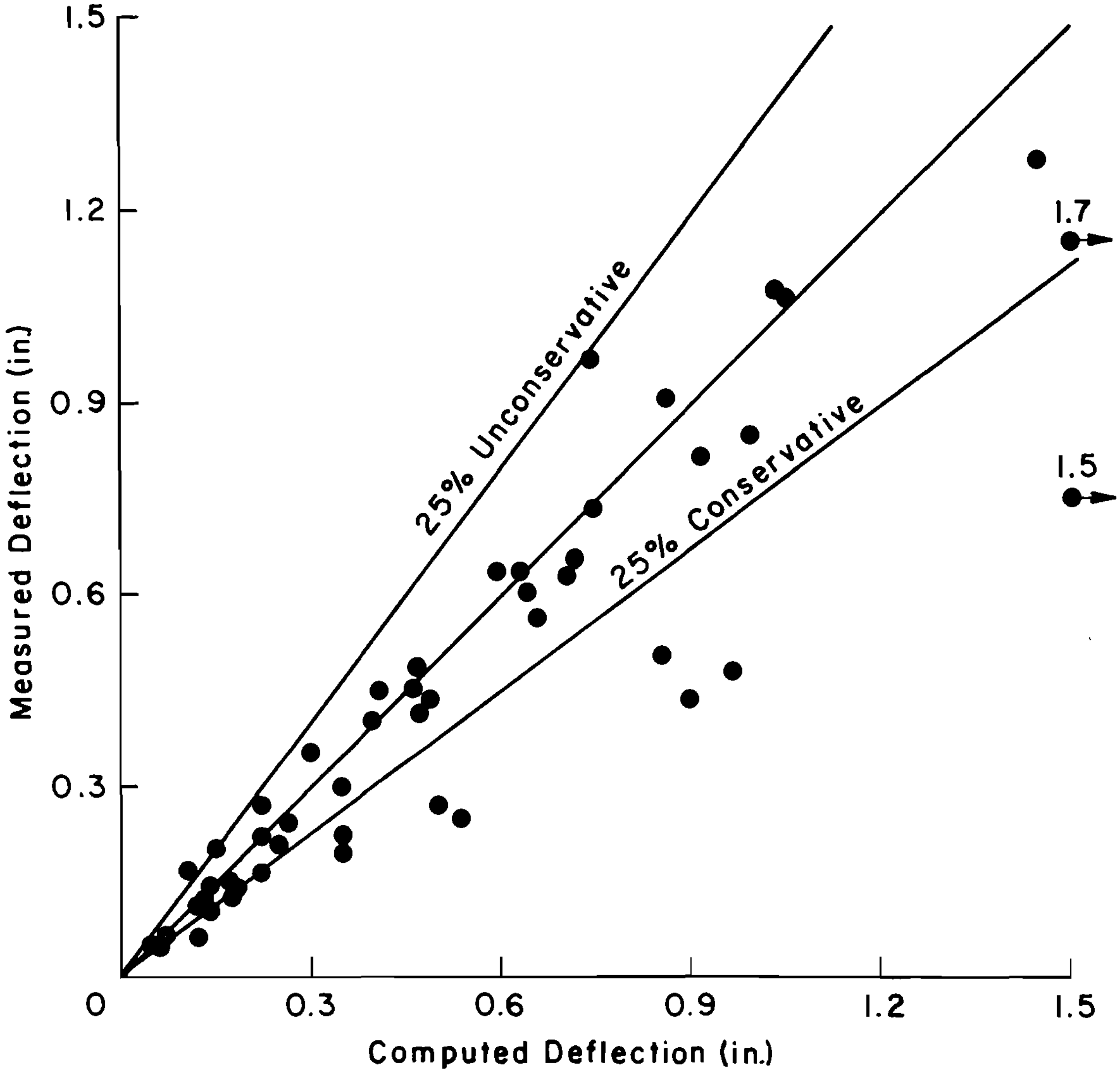


Fig. 5.29. Comparison between measured and computed deflections for tests using criteria for sand.

deflections. Using the sand p-y criteria, 68% of the computed deflections were within the 25% confidence limits, and 66% of the computed deflections were conservative. For a few tests, the analyses gave results that were very conservative, but the results of analyses performed on other piles at the same test sites were in good agreement with the measured results.

The results of the analyses presented in this chapter indicate that the Sand criteria are more than adequate in determining the behavior of piles in sand. The methods that were selected to relate the blow count to D_r and D_r to ϕ in this report appear to be adequate, and their usage is recommended in cases where ϕ cannot be determined more accurately.

The recommended values for k were too low for the Arkansas River tests but were adequate for the other tests which were analyzed. The possible reason for the inaccurate assessment of k for the Arkansas River test has been discussed previously. Based on the results of all the other tests, the value of k recommended by Reese, et al., plotted in Fig. 5.3, should be used. It was shown in Chapter 3 that incorrect selection of this parameter will only cause small errors and that selecting an overly large value for k will lead to smaller errors than if the selected value for k were too small.

CHAPTER 6. SUMMARY AND CONCLUSIONS

PARAMETER STUDY

The results of the parameter study indicated that the pile head deflection was more sensitive to variations in soil and pile properties than was the maximum bending moment. Also, cyclic loading caused increases in both the pile head deflection and the maximum bending moment. Therefore, in the design of the foundations for structures for supporting overhead signs, careful attention should be given to the nature and magnitude of the cyclic loading.

The following conclusions were drawn from the results of the parameter study:

(1) The most important soil parameter needed to predict pile behavior in clay is the undrained shear strength, c.

(2) Variations in ε_{50} had less of an effect on pile behavior than did variations in c.

(3) Variations in γ and k had very little effect on pile behavior for piles in clay.

(4) The most important parameter needed to predict pile behavior in sand is ϕ.

(5) Variations in γ are more important for piles in sand than for piles in clay.

(6) Variations in K_o have only a small effect on pile behavior.

(7) Variations in k have very little effect on piles in sand.

ANALYSIS OF PILES IN CLAY

The results of the analyses performed on piles in clay are shown in the table on the following pages.

TABLE 6.1 RESULTS OF ANALYSES FOR PILES IN CLAY

Test	Soil Criteria Employed in Analysis	Agreement Between Experimental and Computation	
		Deflection	Maximum Bending Moment
Bagnolet, B1	StiffA	Good	Excellent*
Bagnolet, B4	StiffA	Fair-unconservative	Fair**
Bagnolet, B5	StiffA	Fair	Good
Bay Mud, D1	StiffA	Poor-unconservative	—
Bay Mud, D2	StiffA	Poor-unconservative	—
Bay Mud, D3	StiffA	Poor-unconservative	—
Bay Mud, D4	StiffA	Poor-unconservative	—
Bay Mud, F1	SoftB	Poor-unconservative	—
Bay Mud, F1	Unified	Fair-unconservative	—
Bay Mud, F2	SoftB	Poor-unconservative	—
Bay Mud, F2	Unified	Excellent	—
Bay Mud, F3	SoftB	Poor-unconservative	—
Bay Mud, F3	Unified	Fair-unconservative	—
Bay Mud, F4	SoftB	Poor-unconservative	—
Bay Mud, F4	Unified	Poor-unconservative	—
Hudson River	SoftB	Excellent	—
Hudson River	Unified	Good	—
Japan	SoftB	Good	Excellent
Japan	Unified	Good	Good
Lewisburg	StiffA	Poor	—
Ontario, 38	StiffB	Fair-conservative	—
Ontario, 38	Unified	Poor-conservative	—
Ontario, 17	StiffB	Poor-conservative	—
Ontario, 17	Unified	Fair-conservative	—
Plancoet	SoftB	Fair-conservative	Good
Plancoet	Unified	Poor-conservative	Excellent

(continued)

TABLE 6.1. (Continued)

Test	Soil Criteria Employed in Analysis	Agreement Between Experimental and Computation	
		Deflection	Maximum Bending Moment
Savannah, 2	StiffA	Fair-unconservative	—
Savannah, 5	StiffA	Fair-conservative	—
Southern Calif. Edison, 2	StiffA	Poor-conservative	—
Southern Calif. Edison, 6	StiffA	Poor-conservative	—
Southern Calif. Edison, 8	StiffA	Poor-conservative	—
St. Gabriel	SoftB	Poor-conservative	—
St. Gabriel	Unified	Poor-conservative	—

*The term "excellent indicates that the agreement between experimental and computed results is ±5%; "good" is used to indicate ±10%; "fair-unconservative" indicates less than -25%; "fair-conservative" indicates less than +25%; "poor-unconservative" indicates greater than -25%; and "poor-conservative" indicates greater than +25%.

**In cases where the relationship between measured and computed results was not clearly conservative or unconservative, only the words fair or poor was used.

The comparison between experimental and computational results for piles in clay show fair to good agreement in most cases at working loads. In some of the cases where poor agreement was obtained, difficulties in assessing the soil properties used in the analysis could have caused the lack of agreement. In general, the criteria, with the exception of StiffA, were found to be satisfactory in their present forms. Based on the results presented in this report, no modifications could be suggested.

The results of the analysis in dry, stiff clay using the Reese and Welch (1974) StiffA criteria indicated that a modification to the shape of the p-y curve was warranted. The currently used exponent of 0.25 in their parabolic equation was too small, which leads to unconservative deflections at small loads, and conservative deflections at large loads. Comparisons between experimental and computational results using different exponents were made and an exponent of 0.4 was found to yield the best agreement.

ANALYSIS OF PILES IN SAND

The results of the analyses performed on piles in sand using the sand criteria have been compiled in the following table:

TABLE 6.2 RESULTS OF ANALYSES FOR PILES IN SAND

Test	Agreement Between Experimental and Computation*	
	Deflection	Maximum Bending Moment
Arkansas River, 2	Poor-conservative	Fair-conservative
Arkansas River, 16	Fair-conservative	Fair-conservative
Arkansas River, 5	Fair-conservative	——
Arkansas River, 14	Excellent	——
Arkansas River, 6	Excellent	——
Arkansas River, 8	Good	——
Arkansas River, 12	Good	Poor-conservative
Arkansas River, 13A	Fair-conservative	Poor-conservative
Apapa, 1	Fair	——
Apapa, 2	Fair-conservative	——
Bailly, TP7	Excellent	——
Bailly, TP8	Excellent	——
Florida	Good	——
Hydraulic Fill, P9	Poor-conservative	——
Hydraulic Fill, P10	Poor-conservative	——
Hydraulic Fill, P11	Fair-conservative	——
Hydraulic Fill, P12	Good	——
Mason and Bishop	Poor-conservative	——
Model Tests	Good	Excellent

*The same terminology describing the degree of agreement between measured and computed results which was used in the preceding section was used here.

The comparisons between experimental and computational results for piles in sand show good to excellent agreement in most cases at working loads. Based on the results presented in this report, no modifications could be suggested.

CONCLUDING COMMENTS

The method of analysis of laterally loaded piles presented in this report is versatile and offers the best method available at the present time. As more information is gained on the behavior of full-scale piles under lateral loading, the method can be improved. The major improvements will involve the development of soil criteria (p-y curves) that more faithfully reflect the actual behavior of the soil around the pile. The methods that are available and employed in the analyses described in this report can be used with fair to good accuracy in predicting groundline deflections and with good to excellent accuracy in predicting bending moment.

With regard to the design of foundations for overhead signs, the bending moment is of most importance because the foundation will collapse if its capability of sustaining bending moment is deficient. The method presented herein can also be employed in predicting the required penetration of a foundation supporting an overhead sign. The required penetration is, of course, an important parameter for lightly loaded foundations.

CHAPTER 7. RECOMMENDATIONS

The following recommendations are made in connection with future research in the area of laterally loaded piles:

(1) Instrumented tests on large diameter piles, at least 30 to 40 in. in diameter, in various soils should be performed to determine the effect of pile diameter on pile behavior.

(2) More test results are needed to evaluate the variation of the A and F parameters for submerged clays which are different from the clays at Sabine and Manor.

(3) Instrumented and uninstrumented tests need to be performed in stiff, desiccated soils. The results of tests on both flexible and rigid piles would be beneficial.

(4) The use of in-situ testing methods, such as the self-boring pressuremeter, is needed to obtain soil properties for soils which are difficult to sample and test.

(5) Better quality laboratory tests are needed to help properly evaluate the current p-y criteria.

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