Development of P-Y Curves for Analysis of Laterally Loaded Piles in Western Washington

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The research was conducted by the University of Washington Department of Civil Engineering. Assistant Professor Steven L. Kramer was the principal investigator. Various portions of the literature search were performed by Messers. Jan-Olaf Backman, Edward Heavey, and Nadarajah Sivaneswaran.

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ABSTRACT

A comprehensive review of literature pertaining to p-y curve analysis of laterally loaded piles was conducted. Methods of analysis of laterally loaded piles were reviewed with particular emphasis on the applicability of p-y curve analysis. Various existing procedures for development of p-y curves, and the data on which they are based, were also reviewed. Case histories of well-documented, full-scale, field lateral load tests were identified and the soil conditions in which they were performed were summarized. The applicability of existing p-y curve criteria to western Washington soil conditions was evaluated. Analysis of pile group behavior and full-scale, field lateral load test procedures were reviewed.

SUMMARY

The response of laterally loaded piles depends on the behavior of the soil and the pile, and on the interaction between the two. A method of intermediate complexity and reasonable accuracy for analysis of laterally loaded piles known as p-y curve analysis has become widely accepted by geotechnical engineers. The accuracy of p-y curve analyses depends on the accuracy with which the p-y curves represent the response of the soil to lateral deflections of a pile.

The most commonly used procedures for development of p-y curves are based on a relatively small amount of data in specific soil conditions. Consequently, the applicability of these procedures to different soil conditions has been uncertain. The general purpose of the research described in this report was to identify available methods for development of p-y curves and to evaluate their applicability to soil conditions encountered in western Washington.

A number of methods for analysis of laterally loaded piles, from the most simple to the most complex, were reviewed. The p-y curve method of analysis was reviewed in detail and its applicability, with respect to pile characteristics and various types of loading conditions, was evaluated.

Various existing methods for development of p-y curves were reviewed with particular emphasis on the nature of the data from which they were developed. Some of the most commonly used p-y criteria were found to be based on an extremely limited amount of data.

An extensive review of available, full-scale, field lateral load test data was performed. Well over 100 such tests were identified and reviewed. The general reliability of the information presented in the reports of these tests was evaluated by rating the reported soil conditions, pile instrumentation, and test procedures. The load tests were divided into groups by soil conditions. The soil conditions for which reliable, full-scale, field lateral load test data are available are limited to soils classified as predominantly clay or predominantly sand.

The applicability of existing p-y curve criteria to western Washington soil conditions was evaluated. The soil conditions encountered in western Washington were briefly reviewed and

compared with the soil conditions for which existing methods can reliably predict p-y behavior. Soil conditions for which existing p-y curve criteria are not available were identified, and recommendations toward development of p-y curve criteria for such soils were presented. Interim recommendations for the evaluation of laterally loaded pile response in these soils were presented.

An analysis of the response of pile groups to lateral loads was reviewed. Procedures for the performance of full-scale, field lateral load tests were discussed.

CONCLUSIONS

A number of conclusions can be drawn from the research described in this report.

1. The p-y curve method of analysis is a reasonable and practical method for the evaluation of the response of piles to static lateral loads. The incorporation of p-y curve analysis into WSDOT's design procedure has resulted in an improved ability to evaluate pile behavior and in the improved safety and economy of pile foundations.
2. Reasonable methods are available for development of p-y curves for piles embedded in soil deposits that would be classified as predominantly clay or as predominantly sand. These methods are based on the results of a significant number of full-scale, field lateral load tests.
3. Currently used methods for development of p-y curves for clay tend to overpredict the deflection of large-diameter piles. The adoption of the recommended procedure should lead to more accurate prediction of pile deflection and less conservative (hence, more economical) design of pile foundations.
4. Methods for the development of p-y curves in other types of soils, including gravels, silts, and highly organic or peaty soils, are not available. Such soil conditions exist at the sites of pile-supported highway structures in western Washington.
5. A significant database of high-quality, full-scale, field lateral load tests on piles embedded in clays and sands currently exists. Procedures for development of p-y curves for clays and sands have been based on this data. Future full-scale, field lateral load tests on piles embedded in such soils are expected to confirm or only slightly modify these existing p-y curve criteria.
6. Very few or no data exist on full-scale, field lateral load tests on piles embedded in other types of soils, even though geotechnical engineers are often required to design

piles in such soils. Full-scale, field lateral load tests in soils such as gravels and silts are badly needed.

7. The p-y curve method of analysis is applicable to all types of piles, though the most commonly used computer programs cannot consider the effects of pile nonlinearity often exhibited by concrete and timber piles.

8. The p-y curve method of analysis has limited applicability to loading conditions in which inertia and damping become important. These conditions include dynamic loading, as in machine foundations, and seismic loading.

9. The response of piles to seismic loading is a critical parameter in the design of foundations for highway structures in western Washington. Alternative methods for the analysis of seismic pile response have been proposed.

RECOMMENDATIONS

On the basis of the studies described in this report, the following recommendations can be made:

1. The use of the p-y curve method of analysis should be continued. The method offers advantages over previous methods and provides a reasonable representation of the response of piles embedded in clays and sands to static and cyclic lateral loads.
2. The Integrated Clay Criteria should be adopted for development of p-y curves for piles embedded in clays. The Integrated Clay Criteria is based on much more supporting data than previous methods and incorporates features which improve its accuracy, particularly for large diameter piles.
3. The Extended Hyperbolic Criteria should be adopted for development of p-y curves for piles embedded in sands. This method is easier to use than the current (Reese, et al.) method and more accurate for tapered piles and H-piles. For other piles, the accuracy is marginally improved over the currently used method.
4. For soil conditions other than those that would be classified as predominantly clay or predominantly sand, the recommendations presented in Chapter 5 may be used in the interim period before research on p-y behavior in such soils becomes available.
5. Research on p-y behavior of soils other than clays and sands should be performed. This research should consist of full-scale, field lateral load testing of instrumented piles and careful interpretation of the test results.
6. Practical methods for the evaluation of the seismic response of piles should be investigated. A distinct need exists for identification and implementation of a relatively simple method that could be applied to relatively small and routine projects, and a more rigorous method that could be applied to large, special projects.

CHAPTER 1
INTRODUCTION

Many highway structures are supported on pile foundations. Properly designed pile foundations are able to resist heavy vertical loads very efficiently. The foundations of highway structures, however, are often required to resist some level of lateral loading as well. For this reason, pile foundations must be designed to resist the anticipated lateral loads without failure or excessive lateral deflection.

Ideally, a method for analysis of laterally loaded piles should completely describe the response of a pile to lateral loads. Particular aspects of the pile response of importance in foundation design include the load-deflection behavior at the top of the pile (including the ultimate lateral load), the deflected shape of the pile, and the variation of shear and bending moment along the length of the pile. The analytical method should be able to predict pile response for pile heads fixed or free to rotate, or with some degree of rotational stiffness. It should also be able to consider any combination of vertical and horizontal loading as well as applied moments at the top of the pile, and should be able to consider non-homogeneous soil conditions. The ideal method would describe these aspects of the response on the basis of an accurate representation of the stress-strain behavior of the soil, the bending behavior of the pile, and the interaction between the solid and the pile.

Many methods have been proposed for analysis of the response of piles to lateral loading. The methods range widely in complexity and in accuracy. Very simple methods are available for estimation of lateral load capacity; however, their accuracy is generally not high enough to warrant their use in final design. Very complex methods, which accurately model the soil, the pile and their interaction in three dimensions, are also available; however, the time and specialized skill requirements for their proper use render them impractical for routine design problems. One method, of moderate complexity and relatively good accuracy, has become widely used for analysis and design of pile foundations by practicing engineers. This method, referred to as "p-y curve analysis," will be discussed in detail in this report.

This report will describe and summarize a number of different methods for analysis and design of laterally loaded pile foundations. It will concentrate on p-y curve analysis procedures, and particularly on the procedures available for development of p-y curves. Concern has been expressed regarding the very limited base of data from which the most commonly used p-y curve criteria have been developed. The data from which available p-y curve criteria were developed will be critically reviewed and their applicability to soils commonly found in Western Washington evaluated. Soil conditions encountered in Western Washington for which available p-y curve development procedures are not soundly based will be identified and appropriate p-y curve criteria or further research recommended. Such research would consist of full-scale, field lateral loading tests on piles installed in such soils. Procedures for conducting such tests will be recommended.

CHAPTER 2
METHODS OF ANALYSIS

INTRODUCTION

Analysis of the response of pile foundations to lateral loads may be accomplished by a number of methods. These methods range in complexity from simple, empirical prescription value methods to rigorous, three-dimensional soil-structure interaction representations. The more simple methods, while quick and easy to use, may not provide sufficient information or accuracy for design purposes. The more complex methods generally require significant amounts of time and expertise, which may not always be available for practical design problems.

Various available methods accomplish the objectives of a lateral load analysis procedure described in the previous chapter with varying degrees of success. This chapter will briefly review the available methods, from the most simple to the most complex, and discuss their respective strengths and limitations.

PRESCRIPTION VALUE METHODS

Several early researchers (Feagin, 1937; McNulty, 1956; Teng, 1962) prescribed allowable lateral load capacities for various types of piles. These capacities were largely estimated on the basis of experience and a small number of lateral load tests. Some of the prescribed capacities make no distinction between the type of pile or pile material (Teng, 1962) or the pile diameter (Feagin, 1937; Teng, 1962), all of which are now well known to significantly influence lateral load behavior.

The prescription values are empirical estimates of the lateral load required to develop some standard pile head deflection. In many cases, the soil conditions and pile type are not specified with enough detail to define the conditions for which they apply. While estimation of lateral load capacity by prescription value is convenient and simple, the questionable reliability of the method limits its practical applicability to use for making rough estimates. When supported by proven experience under similar conditions, prescription values may give a reasonable estimate of lateral load capacity. For design purposes, however, the inherent uncertainty would require the use of factors of safety so large as to render the resulting design uneconomical.

LIMIT EQUILIBRIUM METHODS

The lateral load response of piles has been analyzed by limit equilibrium methods. These methods assume the soil to exhibit rigid, perfectly plastic stress-strain behavior, i.e., no strain until the constant yield stress is reached. When a pile is subjected to increasing lateral loads, failure will be manifested by a rapid increase in deflection of the top of the pile. This excessive deflection may result from failure of the soil surrounding the pile or from failure of the pile itself. By making reasonable assumptions regarding the distribution of soil reaction along the length of the pile at failure, ultimate lateral load capacity can be calculated from simple static equilibrium requirements.

As an example, Broms (1964a, 1964b) presented methods for evaluating ultimate lateral load capacity of piles in cohesive soils and cohesionless soils. For both cases, piles were classified as either short, rigid piles or long, flexible piles. Rigid piles were considered to fail by rigid body rotation due to failure of the surrounding soil. Long, flexible piles were assumed to fail when the maximum bending moment exceeded the bending capacity of the pile, resulting in the formation of a plastic hinge at some depth below the ground surface.

Broms assumed a distribution of ultimate lateral soil resistance along the pile to determine the ultimate capacity. For cohesive soils, the ultimate lateral resistance was taken, on the basis of a plasticity analysis of a cylinder moving laterally through a plastic medium, to be nine times the undrained strength of the soil excluding the upper 1.5 pile diameters. For cohesionless soils, the ultimate soil reaction was considered to be three times the maximum Rankine passive pressure. Simple beam theory allows calculation of the induced bending moment along the pile at failure.

Limit equilibrium methods provide an acceptable means for estimation of ultimate lateral load capacity in many types of soil. Two primary potential failure mechanisms are considered in a reasonable manner, hence the method has a relatively well-founded basis, particularly in comparison with the entirely empirical prescription value methods. Broms has developed dimensionless charts that may be used easily and rapidly for estimation of ultimate lateral load capacity.

Limit equilibrium methods ignore any effects of soil-pile interaction and hence may be difficult to apply to tapered piles, composite piles, layered soil conditions, and strain-softening soil conditions. Limit equilibrium methods give no indication of the magnitude of pile movement required to mobilize the ultimate pile capacity, or of the load-deflection behavior at loads less than those causing failure. They require a priori assumptions of the nature of the ultimate soil resistance and further assume that this resistance is entirely mobilized along the length of the pile at failure. Limit equilibrium methods may be useful for quickly estimating ultimate lateral load capacities; however, all of these assumptions introduce uncertainty into the accuracy of and limit the applicability of limit equilibrium methods for evaluation of lateral load behavior for final design purposes.

ELASTICITY METHODS

In response to the need for prediction of lateral pile head stiffness, methods capable of evaluating the load-displacement behavior of piles have been developed. The first of many such approaches was to use the theory of linear elasticity by assuming the pile to be embedded in a linear elastic medium.

The load-deflection behavior of a pile embedded in an elastic medium can be developed from the beam-on-elastic foundation theory of Hetényi (1946) and shown to be described by the second order differential equation:

$$\frac{d^2M}{dx^2} + Q\frac{d^2y}{dx^2} - p = 0 \tag{2.1}$$

where M = bending moment

y = lateral deflection of pile

x = position along length of pile

Q = axial load at top of pile

p = unit resistance along length of pile.

For the common assumption of a pile exhibiting linear bending behavior, for which the bending moment is proportional to pile curvature, the governing equation can be rewritten as a fourth order differential equation in the familiar form:

$$EI\frac{d^4y}{dx^4} + Q\frac{d^2y}{dx^2} - p = 0 \tag{2.2}$$

where EI = flexural stiffness of pile.

The unit soil resistance, p, results from mobilization of resisting stress in the soil surrounding the pile and varies in some way with lateral pile deflection, y. The manner in which it does so may be described graphically by means of a "p-y curve."

<u>Linear Elasticity</u>

The unit resistance for a linear elastic material may be assumed to be proportional to the pile deflection, in which case it may be expressed as

$$p = -E_s y \tag{2.3}$$

where E_s is the soil stiffness and the negative sign denotes soil resistance acting in the opposite direction of pile deflection. The p-y curve for a linear elastic material would be a straight line with a slope of E_s. Closed form solutions of the governing differential equation have been obtained for two cases of assumed variation of soil modulus -- one which assumes soil modulus to be uniform with depth and another which assumes it to increase linearly with depth.

Broms (1964a, 1964b) used linear elasticity and beam-elastic-foundation theory to develop expressions for lateral deflections for short and long piles, with fixed and free heads, embedded in cohesive and cohesionless soils. The results are presented in charts of dimensionless lateral deflection versus dimensionless pile length. Poulos (1971) used beam theory and Mindlin's solution for the horizontal displacement due to a horizontal point load at some depth below the surface of a semi-infinite elastic half-space to describe the behavior of a pile under lateral loading. Expressions for the groundline deflection of restrained and unrestrained piles were developed using influence coefficients. The influence coefficients were functions of the relative stiffness of the soil and pile and the length/width ratio of the pile.

Methods based on linear elasticity consider the material supporting the pile to be infinitely strong with a constant lateral stiffness that does not vary with pile deflection. It is well known that soils do not exhibit this type of behavior. The resistance offered by soil to the lateral deflection of a pile results from the mobilization of the shear strength of the soil. Since soils have some finite shear strength, there is an ultimate resistance that they can offer. Soils also do not exhibit linear stress-strain behavior, except at very low strains. Consequently, the accuracy of methods based on assumptions of linear elasticity are not well suited to description of pile behavior at moderate to high lateral loads. They may be used, however, to estimate lateral pile stiffness under small lateral loads.

Nonlinear Elasticity

The design of pile foundations for highway structures often requires knowledge of the load-deflection behavior of the piles at moderate to relatively high lateral loads. Due to the strong nonlinearity of soil strength mobilization, the unit soil resistance is highly nonlinear with respect to pile deflection. Consequently, an improvement over linear elasticity methods that assume linear unit soil resistance is needed.

Nonlinear elasticity methods have been developed to account for the strong nonlinearity of the unit soil resistance to lateral pile displacements. These methods generally consist of a solution to the linear elastic problem in which equivalent, displacement-compatible soil properties are used. The displacement compatibility is generally achieved by an iterative process. The now commonly used procedure generally referred to as "p-y curve analysis" is the leading example of such a procedure. Nonlinear p-y curves are used in these analyses. In p-y curve analysis procedures, an initial soil stiffness is assumed. The governing differential equation is solved numerically for that particular stiffness, yielding a distribution of pile deflection along the length of the pile. At each depth, the computed soil reaction is compared with the soil reaction given by the p-y curve at that depth and deflection. If they differ by a significant amount, the soil stiffness at that depth is taken as the secant stiffness from the p-y curve at the previously computed deflection. The governing differential equation is again solved and the comparison process repeated. This procedure is repeated until the difference between two successive iterations is acceptably small. At this point, the nonlinear behavior will have been approximated by an elastic analysis with displacement-compatible soil properties.

The accuracy of such procedures depends on the accuracy with which the p-y curves represent the pile deflection-unit soil resistance behavior of the soil. Available procedures for development of p-y curves, based on the results of full-scale field lateral load tests, are discussed in the next chapter.

CHART METHODS

Evans (1982) developed a series of dimensionless charts based on the results of a large number of hypothetical numerical analyses using the p-y curve analysis procedure developed by Reese at the University of Texas. Evans' procedure makes use of "characteristic loads" and "characteristic moments" based on the properties of the pile and the soil in which it is embedded. This method can be used to obtain pile head deflections and maximum bending moments quickly and easily without the use of a computer. The dimensionless curves were developed from p-y analyses of uniform piles embedded in soil deposits with uniform soil properties. Use of this method for non-uniform piles or soil conditions consequently requires the development of some average pile or soil properties. Other chart methods have been proposed by Manoliu (1976) and Bhushan, et al. (1981).

Evans' charts appear to be very useful for rapid estimation of lateral load response for planning or preliminary design purposes. For site-specific, final design purposes under realistic soil conditions, however, this method does not appear to offer the required accuracy.

HIGHER LEVEL ANALYTICAL METHODS

The response of laterally loaded piles may also be analyzed by more sophisticated methods. Such methods are often very versatile and provide improved representation of soil-pile interaction in two or three dimensions. Desai and Kuppusamy (1980) developed a finite element model of an axially and laterally loaded pile. The model is used to predict the behavior of a wooden pile in dense sand. The accuracy of the prediction was very similar to that obtained by a one-dimensional finite difference analysis for the indicated case study. Boundary element methods (Banerjee and Davies, 1980) have also been used for analysis of laterally loaded piles. The time and expertise required for these more sophisticated methods for analysis of pile response, however, often render them impractical for routine design problems.

DISCUSSION

Current WSDOT practice for the analysis of the response of laterally loaded piles is to use the p-y curve analysis method. Adoption of this method of analysis represented a significant improvement in WSDOT's ability to predict the load-deflection response of the pile as well as the stresses in the pile section due to axial and lateral loads and bending moments.

CHAPTER 3
METHODS FOR P-Y CURVE DEVELOPMENT

INTRODUCTION

A number of different methods have been proposed for development of p-y curves. Development of p-y curves, by the nature of the problem, is a difficult problem to treat analytically. The unit soil resistance, p, results from the mobilization of the strength of the soil surrounding the pile. It thus includes a passive component, primarily along the front of the pile, an active component, primarily along the back of the pile, and side friction components that act primarily along the sides of the pile. In other words, development of unit soil resistance results from shearing of the soil along an infinite number of stress paths, which vary radially and circumferentially throughout the soil surrounding the pile.

Because of the complexity of the manner in which unit soil resistance is mobilized, its characteristics have generally been determined empirically from the results of full-scale and model pile load tests. Empirical determination of p-y behavior from load test results is a valid and reasonable method in most cases. It is important to recognize, however, the limitations of such an empirical approach. The accuracy of an empirical method depends on the data from which it was developed. An empirical procedure based on a large number of tests may be considered to be more reliable than one based on only a few tests. Unfortunately, most of the commonly used p-y curve criteria have been based on a very limited number of tests. This chapter will discuss the general characteristics of p-y curves and describe several methods that have been proposed for developing p-y curves for different soil conditions.

P-Y CURVE CHARACTERISTICS

The p-y curve simply relates unit soil resistance to pile deflection. The slope of a p-y curve at any deflection represents the tangent soil stiffness at that deflection. The ratio -p/y at any deflection represents the secant soil stiffness that corresponds to that deflection. For soils, the limiting shear strength requires that the unit soil resistance have some limiting value or ultimate unit soil resistance.

The magnitude of the ultimate soil resistance depends on the mechanism by which the soil surrounding the pile fails. Generally, two soil failure mechanisms are considered. The first, which usually occurs at relatively shallow depths, is represented by the development of a passive soil wedge in front of the pile. Reese (1962) developed a limit equilibrium solution for the ultimate resistance offered by such a passive wedge. At greater depths, the more critical failure mechanism is by plastic flow of the soil around the pile as it deflects laterally. Plasticity solutions have been developed to predict the ultimate resistance associated with such a failure mechanism. The depth at which these two failure mechanisms predict the same ultimate resistance is conventionally referred to as the "critical depth."

P-Y CURVE CRITERIA

Different approaches have been attempted for development of p-y curve criteria for different soils. McClelland and Focht (1958) recognized that unit soil resistance resulted from the mobilization at the shear strength of the soil surrounding the pile and proposed a procedure that yielded p-y curves of the same shape as the stress-strain diagram of the soil in triaxial compression. The soil surrounding the pile, however, is not subjected to triaxial compression stress conditions. Because of the complexity of the stress state surrounding a laterally loaded pile, p-y behavior is generally determined in a semi-empirical manner from the results of pile load tests.

When a pile is loaded without axial load, as in the case of a typical field lateral load test, the response of the pile is governed by

$$\frac{d^2M}{dx^2} - p = 0 \tag{3.1}$$

Assuming that the pile is loaded only within its linear bending range, the unit soil resistance may be calculated from the deflected shape of the pile by

$$p = EI\frac{d^4y}{dx^4} \tag{3.2}$$

The deflected shape of the pile in the load test is obtained by field instrumentation of the pile. The two most common instrumentation schemes are those that utilize strain gages along the pile and those that use a slope inclinometer with a casing attached to the pile. At each load increment, the curvature (from strain gage instrumentation) or slope (from slope inclinometer instrumentation) is measured at a number of discrete points. The deflected shape is obtained by integrating the curvature profile twice or the slope profile once. The corresponding unit soil resistances are given by the product of the pile flexural stiffness, EI, and either the second derivative of the curvature profile or the third derivative of the slope profile. In this way, the measured response of the pile can be used to develop values of both p and y for a given lateral load. By repeating this process for a number of lateral loads, p-y curves may be developed.

P-y curve criteria have been developed from the results of full-scale field lateral load tests for static and cyclic loading conditions. Existing criteria are reviewed in the following sections, with emphasis on static loading criteria.

P-Y CURVE CRITERIA FOR CLAYS

Soft Clay Criteria

Matlock (1970) proposed a procedure for development of p-y curves for piles in soft, saturated clays. This procedure was based on the results of four lateral load tests on a fully instrumented 12.75-inch-diameter, steel pipe pile at two soft clay sites in Texas that had been ponded to simulate offshore conditions. The Matlock procedure has been incorporated into the American Petroleum Institute specification API RP 2A for offshore pile foundation design.

The Matlock procedure calculates the ultimate soil resistance as

$$p_u = N_p \, c \, b \tag{3.3}$$

where p_u = ultimate soil resistance

N_p = lateral bearing capacity factor

c = undrained shear strength

b = pile diameter.

The lateral bearing capacity factor is taken to be the lesser of

$$N_p = 3 + \sigma'/c + Jx/b \tag{3.4a}$$

or

$$N_p = 9 \tag{3.4b}$$

where σ' = effective overburden pressure

J = empirical constant usually equal to 0.5

x = depth below ground surface (and x_{cr} represents the depth at which $N_p = 9$ by the first expression).

The ultimate resistance is considered to be mobilized at a pile deflection of eight times a reference deflection defined as

$$y_c = 2.5 \in_{50} b \tag{3.5}$$

where $\in_{50}$ = axial strain at $\sigma_d = (\sigma_d)_{max}/2$ in an undrained triaxial compression test.

At pile deflections less than eight times the reference deflection, the unit soil resistance is given by a cubic equation. The shape of the p-y curve for static and cyclic loading, according to the soft clay criteria, is illustrated in Figure 3-1(a) and 3-1(b). In this and other p-y curve criteria to follow, cyclic loading conditions will refer to repetitive loads applied so slowly that inertial effects are negligible. Note that this method predicts an initially infinite soil stiffness.

<u>AWT Stiff Clay Criteria</u>

Reese and Welch (1975) developed p-y curve criteria for piles embedded in stiff clay above the water table on the basis of <u>one</u> full-scale, field lateral load test. The load test was performed on a 30-inch-diameter drilled shaft in which a 10.75-inch-diameter instrumented pipe was embedded. From this full-scale, field lateral load test, a procedure for developing p-y curves for stiff clays above the water table was suggested. The authors recommended that the procedure be used with great care.

Reese and Welch recommended that the ultimate soil resistance be calculated for the soft clay criteria in Equation 3.3. The reference deflection is also calculated as for the soft clay criteria in Equation 3.5. The ultimate soil resistance is considered to be mobilized at deflections greater

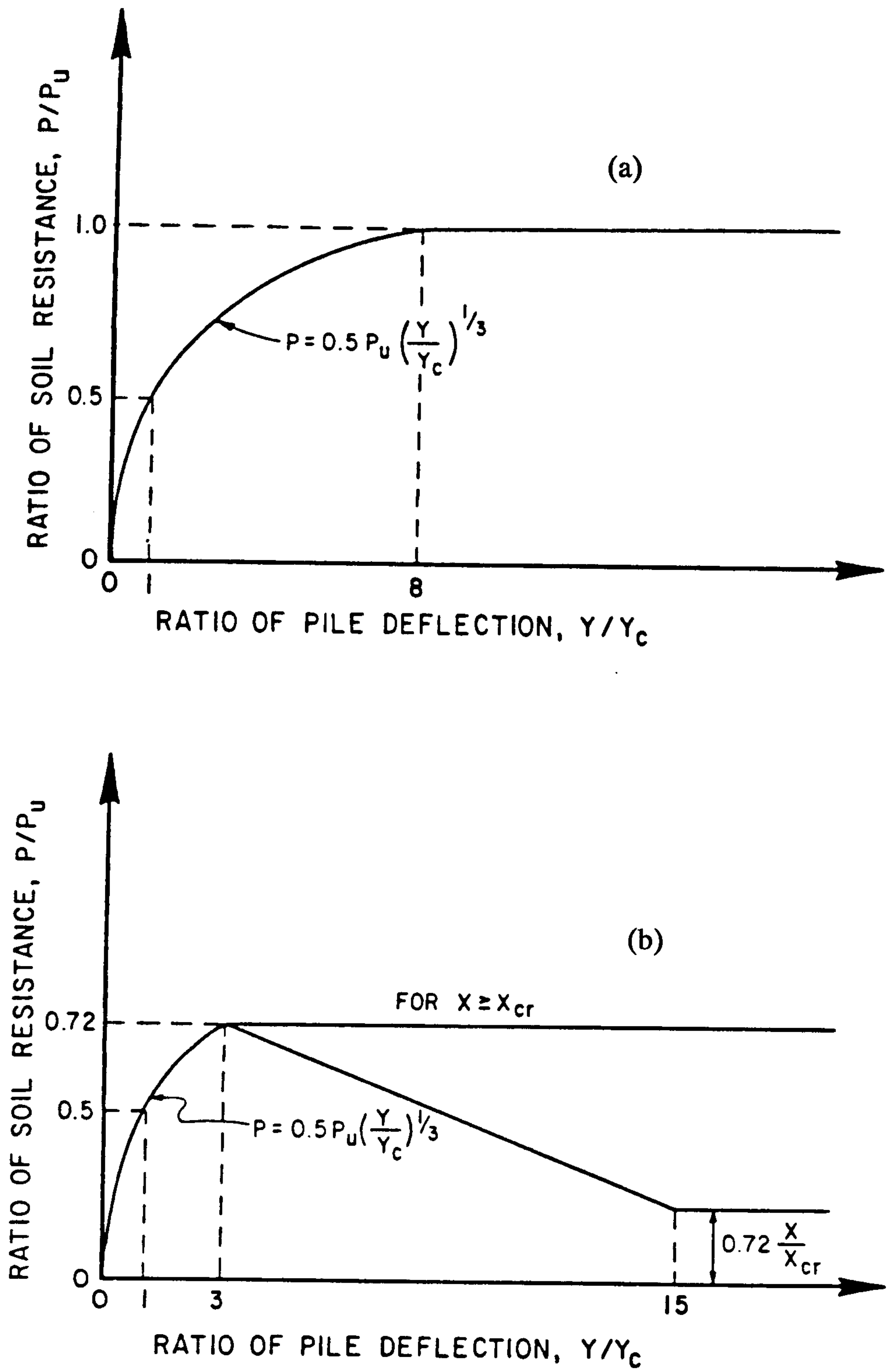


Figure 3-1.
Soft Clay Criteria for (a) Static Loading, and (b) Cyclic Loading

than or equal to 16 times the reference deflection. At smaller deflections, the shape of the p-y curve is as shown in Figure 3-2 (a). For cyclic loading, the static y-values are modified as shown in Figure 3-2 (b) where

$$c = 9.6\,(p/p_u)^4 \tag{3.6}$$

Note that this method predicts an initially infinite soil stiffness.

BWT Stiff Clay Criteria

Reese, et al. (1975), proposed a procedure for the development of p-y curves for piles embedded in stiff clays below the water table. This procedure was based on the results of three lateral load tests on fully instrumented, steel pipe piles embedded in a heavily overconsolidated clay deposit in Texas. The site had been ponded to simulate offshore conditions. Since the clays of this deposit were expansive, the influence of softening of the near surface soils by the ponded water may have been significant.

Reese developed separate expressions for ultimate soil resistance for two distinct mechanisms by which the pile was assumed to move through the soil. Based on the failure of a wedge of soil in front of the pile and on plastic flow of soil around the pile in a horizontal plane, Reese calculated an ultimate resistance as the lesser of

$$(P_{ult})_c = 2cb + \sigma' b + 2.83\,cx \tag{3.7a}$$

or $$(P_{ult})_c = 11\,cb \tag{3.7a}$$

Based on the field load tests, however, it was determined that these relationships overpredicted the observed ultimate soil resistance. Empirical factors, whose variation with depth is shown in Figure 3-3 (a), were introduced to adjust these calculated ultimate resistances to match the observed ultimate resistances from field experiments. The initial portion of the BWT stiff clay criteria p–y curve is linear with an initial soil modulus given by

$$(E_s)_i = Kx \tag{3.8}$$

where K = an empirical factor related to the shear strength of the soil and the nature of loading as shown in Figure 3-3 (b).

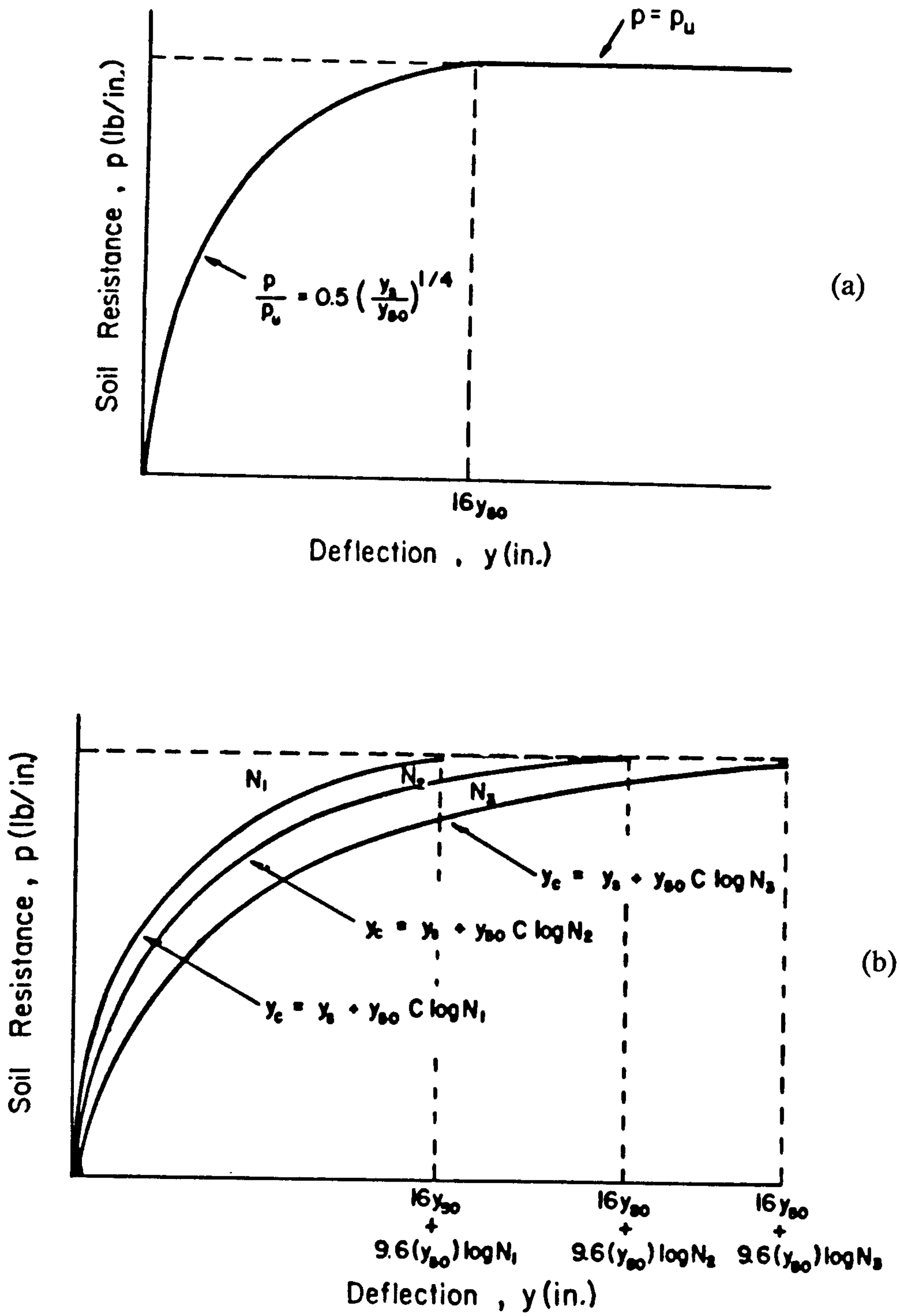
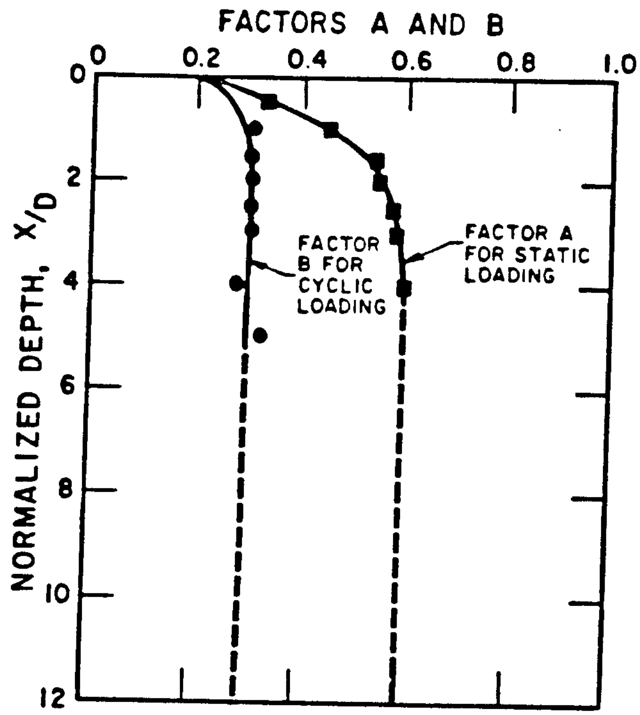


Figure 3-2.
AWT Stiff Clay Criteria for (a) Static Loading, and (b) Cyclic Loading



(a)

	$(s_u)_{ave}$ (tsf)		
	0.5-1.0	**1.0-2.0**	**2.0-4.0**
K_{static} (lb/in^3)	500	1,000	2,000
K_{cyclic} (lb/in^3)	200	400	800

(b)

Figure 3-3.
BWT Stiff Clay Criteria Empirical Factors for (a) Ultimate Resistance, and (b) Initial Stiffness

The shapes of the p-y curves for static and cyclic loading conditions generated by the BWT stiff clay criteria are shown in Figures 3-4 (a) and 3-4 (b), respectively.

Unified Clay Criteria

Sullivan, et al. (1980), proposed a unified approach to p-y curve development for clays. This method is based upon the soft clay criteria proposed by Matlock and the stiff clay criteria proposed by Reese, et al., and, consequently, on the lateral load test data from which they were developed. It is recommended for use in pile design in any clay soil.

Sullivan, et al., reviewed expressions for p_{ult} developed by a number of investigators and recommended that the ultimate unit soil resistance be taken as the smallest value computed by the three relationships:

$$p_{ult} = \left(2 + \frac{\bar{\gamma}}{\bar{s}_u} x + \frac{0.833}{b} x \right) \bar{s}_u b \tag{3.9a}$$

$$p_{ult} = \left(3 + \frac{0.5}{b} x \right) s_u b \tag{3.9b}$$

$$p_{ult} = 9 s_u b \tag{3.9c}$$

where $\bar{\gamma}$ = average effective unit weight from ground surface to depth x

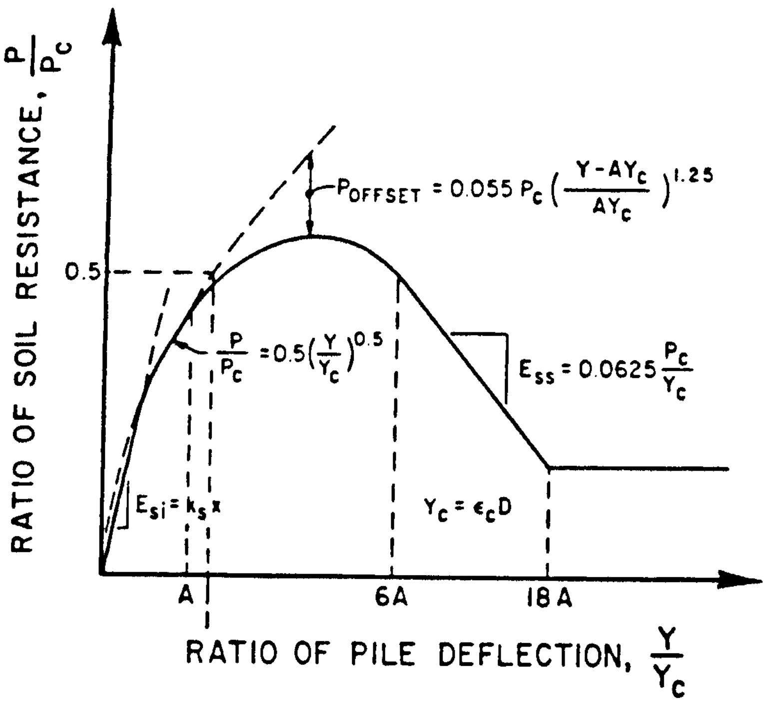
b = pile diameter

$\bar{s}_u$ = average undrained strength from ground surface to depth x

s_u = undrained strength at depth x.

Like the stiff clay criteria, the Unified Clay Criteria proposes an initial linear portion for both static and cyclic loading, described in the same way with slightly different K-values, as shown in Figure 3-5 (a). The initial linear portion is followed by a curved portion, which is described by a cubic equation relating pile deflections to a critical deflection, y_{50}, defined as

$$y_{50} = A \epsilon_{50} b \tag{3.10}$$



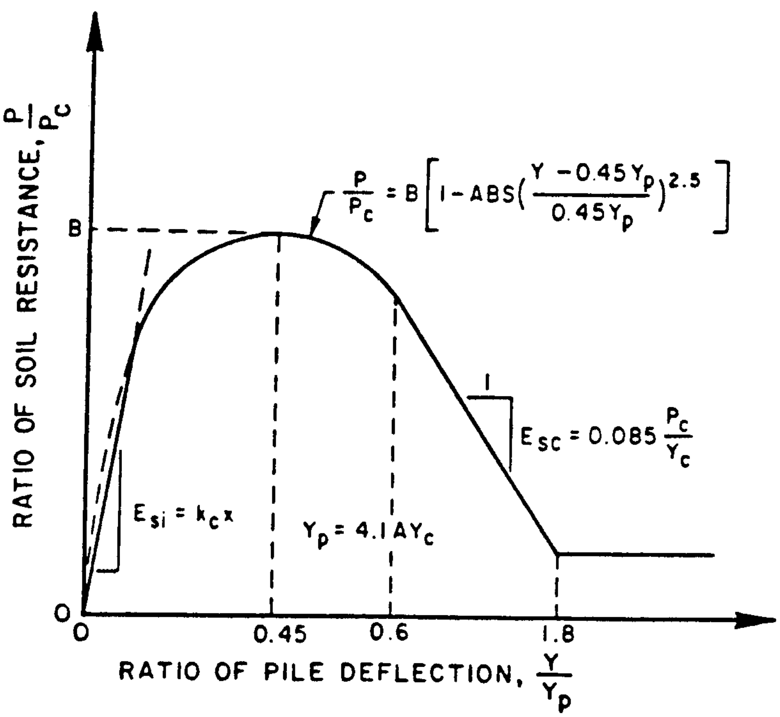
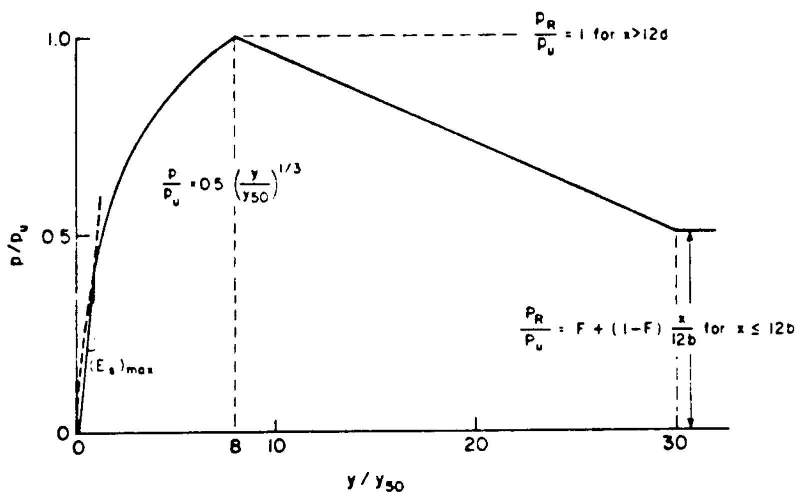


Figure 3-4.
BWT Stiff Clay Criteria for (a) Static Loading, and
(b) Cyclic Loading

δu (tsf)	K (lb/in^3)
0.12 - 0.25	30
0.25 - 0.50	100
0.50 - 1.00	300
1-2	1,000
2-4	3,000



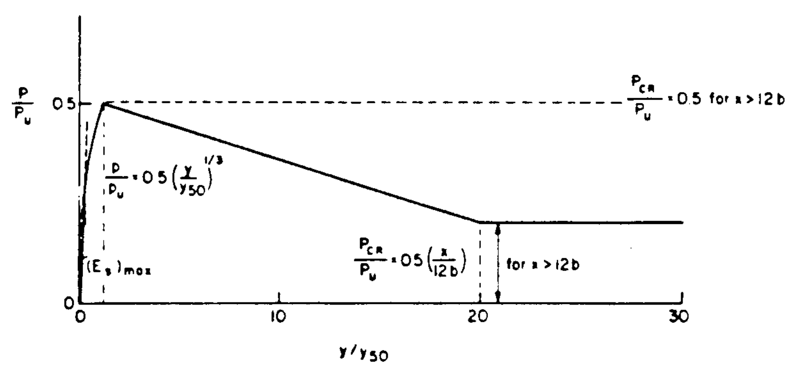


Figure 3-5.
Unified Clay Criteria: (a) Empirical Factors for Initial Stiffness, (b) Static Loading, and (c) Cyclic Loading

where A is an empirical parameter that is equal to 2.5 for Matlock's tests in soft clay and 0.35 for tests in stiff clay. The value of A for design is left to the judgment of the user of the Unified Clay Criteria.

The shapes of the Unified Clay Criteria p-y curves for static and for cyclic loading are shown in Figure 3-5 (b) and 3-5 (c). Note that the user of this "unified" criteria is still required to judge whether the soil is stiff or soft for evaluation of the empirical parameters A and F. The parameter F has been estimated as 1.0 at a soft clay site and 0.5 at a stiff clay site. The value of F for design is also left to the judgment of the user. Consequently, use of the unified criteria appears to offer little real improvement over use of the soft clay criteria and stiff clay criteria discussed previously.

Integrated Clay Criteria

In an attempt to remove the subjective distinction associated with characterization of cohesive soils as either soft clays or stiff clays, O'Neill and Gazioglu (1984) proposed a p-y curve development procedure that would be applicable to all clays. This procedure was based on the results of 21 field lateral load tests at 11 different locations. The distribution of tests with qualitative soil description is as follows:

Soil Description	Very soft	Soft	Medium stiff	Stiff	Very stiff
Number of tests	2	5	5	6	3

The field lateral load tests used to develop the integrated clay criteria were selected to include available high quality tests on a wide range of soils. The integrated clay criteria were developed by making a number of reasonable assumptions regarding the influence of factors such as pile diameter, pile length and soil stiffness, and by optimizing several parameters to produce a procedure that provides the best agreement with the available field data. The optimization procedure involves a weighting scheme that assigns different weights to the results of each field load test depending on the perceived quality of the test and its supporting data.

The shape of the Integrated Clay Criteria p-y curves for static and cyclic loading are shown in Figures 3-6 (a) and 3-6 (b). The Integrated Clay Criteria differ from the previously described

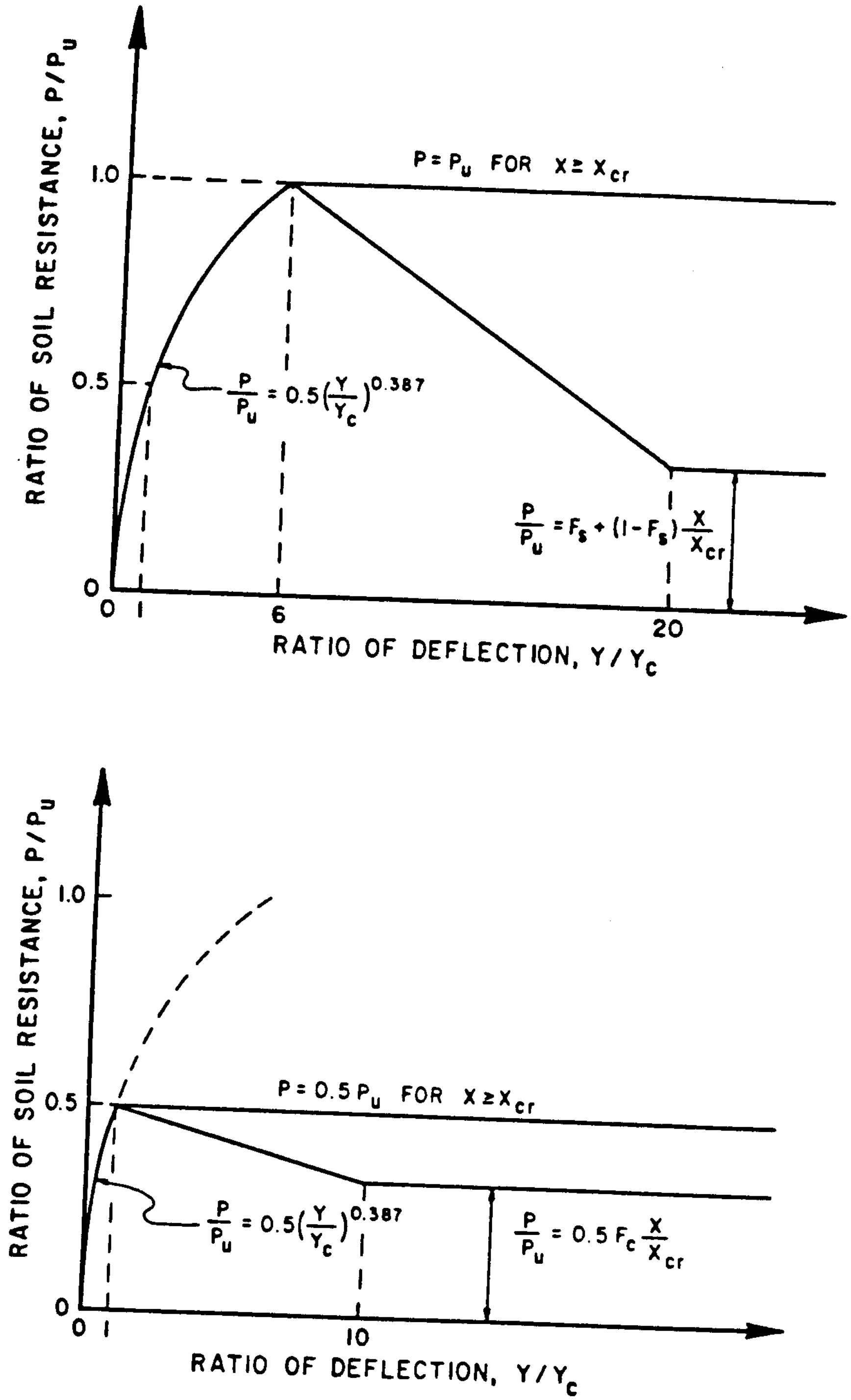


Figure 3-6.
Integrated Clay Criteria for (a) Static Loading,
and (b) Cyclic Loading

criteria in a number of fundamental ways. First, the Integrated Clay Criteria are based on a much greater number of full-scale field lateral load tests representing a much wider range of soil conditions than any of the previously proposed p-y curve criteria. Another important difference is that the critical pile deflection is considered, based on the results of Stevens and Audibert (1979), to vary with the square root of the pile diameter rather than being proportional to the pile diameter as had been assumed in the development of previous p-y curve criteria. Stevens and Audibert studied data obtained from tests on large diameter piles and shafts conducted after the Soft Clay Criteria and Stiff Clay Criteria had been developed and found that the observed performance was predicted much more accurately when the critical pile deflection was considered to increase with the square root of the pile diameter. The assumption of a critical deflection proportional to pile width appears to result in significant overprediction of deflection for pile diameters greater than about 24 inches. Additionally, the critical depth is considered to be dependent on the relative stiffness of the soil and the pile rather than the strength of the soil and the diameter of the pile. Such a procedure will only consider a "wedge"-type failure to occur if the pile is stiff enough to allow such a failure mechanism to occur. These features of the Integrated Clay Criteria appear to be of value to the designer of large-diameter, relatively stiff, deep foundations.

The details of p-y curve construction by the Integrated Clay Criteria are described in Appendix A.

P-Y CURVE CRITERIA FOR SANDS

Reese, et al., Criteria

Reese, et al. (1974), presented procedures for development of p-y curves for sand on the basis of two field lateral load tests on 24-inch-diameter, 80-foot-long steel pipe piles embedded 69 feet into the ground. The soil at the site of the load tests consisted of a uniformly graded fine sand. This procedure has been incorporated into API RP 2A for design of offshore pile foundations.

The p-y curves proposed by Reese consist of an initial linear segment in which the unit soil resistance is proportional to the pile deflection, a parabolic segment, a linear segment with positive

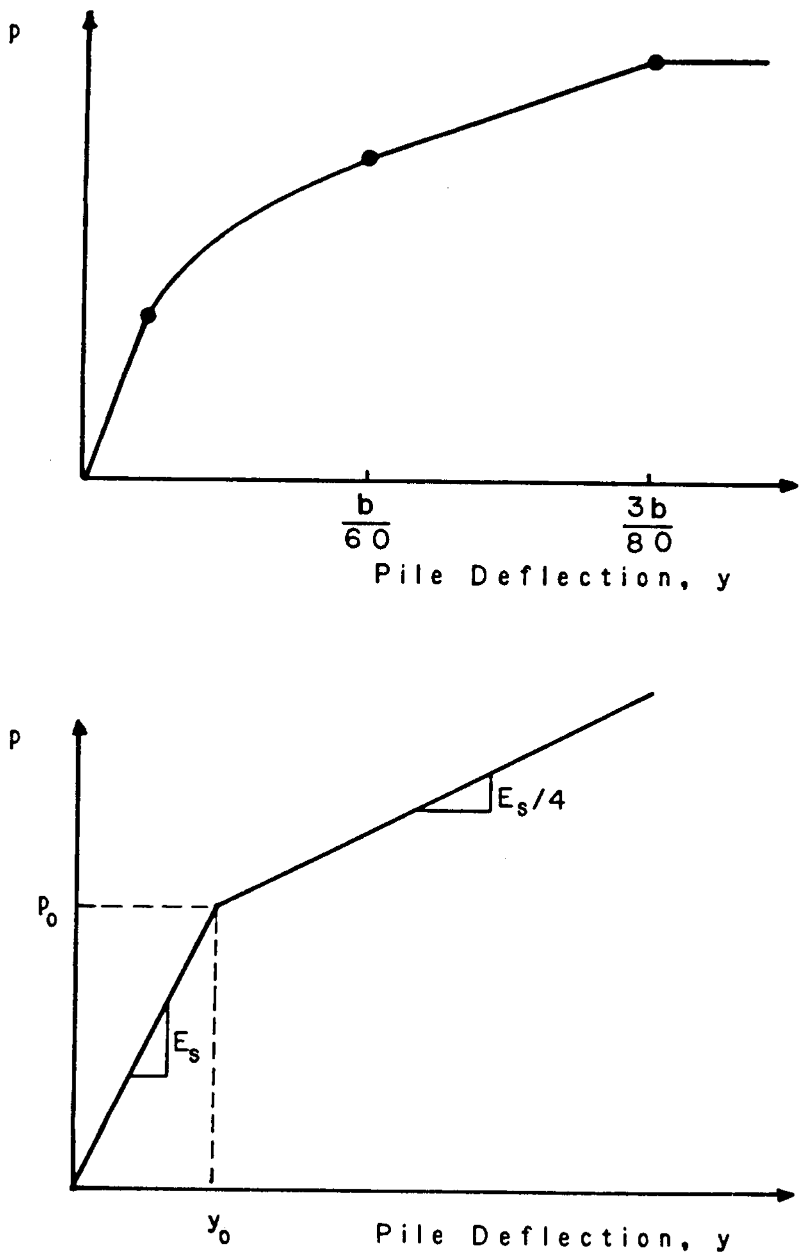


Figure 3-7.
(a) Reese, et al., Sand Criteria, and (b) Bilinear Criteria

slope, and a flat segment at the ultimate soil resistance, which is mobilized at relatively large pile deflections, as shown in Figure 3-7 (a). The ultimate resistance is taken as the lesser of that provided by a passive wedge failure mechanism or a (horizontal) plane strain flow mechanism as postulated by Reese (1962). The ultimate resistance is assumed to be mobilized at pile deflections greater than 3b/80, where b is the diameter of the pile. At a pile deflection of b/60, the unit soil resistance, p_m, is equal to an empirically determined fraction of the ultimate soil resistance. The slope of the initial linear segment is characterized by a modulus of subgrade reaction given in tabular form as functions of relative density for sands above and below the water table. The parabolic segment of the p-y curve spans between the initial linear segment and the other linear segment.

A simplification of this method, proposed by Bogard and Matlock (1980), results in simplification of the calculation of the ultimate soil resistance and is otherwise identical to the API method.

Bilinear Criteria

Scott (1979) performed centrifuge tests on model piles designed to simulate the field load tests from which Reese, et al. (1974), developed their p-y curve criteria. Scott's procedure, while very simple, applies only to static loading conditions. Scott recommended that the p-y curve be bilinear, with an initial segment in which the unit soil resistance is proportional to the pile deflection. The constant of proportionality, or modulus of subgrade reaction, was calculated from the theory of elasticity for plane strain conditions. The inelastic segment was taken to have a slope of one-fourth that of the initial elastic segment. The shape of the bilinear p-y curve is shown in Figure 3-7 (b) and the values of p and y at the point at which the two segments intersect are given by

$$p_o = \frac{K_o \sigma_v' b}{\left(\frac{1}{\sin^2\phi} + \frac{1}{3-4\nu}\right)^{1/2}} \qquad (3.11)$$

where K_o = coefficient of earth pressure at rest

σ_v' = vertical effective stress

ν = Poisson's ratio of soil

and $y_o = \frac{p_o}{Kx}$

where K = modulus of subgrade reaction.

Scott's model assumes that unit soil resistance continues to increase with increasing pile deflection. Consequently, it tends to unconservatively underpredict pile displacements at high lateral loads.

__Hyperbolic Criteria__

Parker and Reese (1979) developed a method for p-y curve construction using a hyperbolic tangent function. The hyperbolic tangent can be expressed in such a way that it provides a very good continuous representation of the rather cumbersome, segmented original method of Reese, et al. (1974). It is thus much easier to use and construct than the Reese, et al., criteria while offering essentially the same accuracy.

The hyperbolic method represents the p-y curve by the expression

$$p = A\, p_u \tanh\left[\left(\frac{Kx}{p_u}\right) y\right] \qquad (3.12)$$

where A = empirical adjustment factor

$A = 0.9$ for cyclic loading

$A = 3 - 0.8x/b \geq 0.9$ for static loading.

__Extended Hyperbolic Criteria__

O'Neill and Murchison (1983) compared the ability of the three previously discussed p-y curve criteria to predict the observed responses of 14 lateral load tested piles and concluded, on the basis of accuracy and ease of use, that the hyperbolic model was most appropriate. O'Neill and Murchison did, however, incorporate Bogard and Matlock's (1980) modifications for simplification of ultimate resistance calculation and proposed an empirical adjustment factor for improvement of accuracy for tapered piles and H-piles. The details of the extended hyperbolic criteria are presented in Appendix B.

PRESSUREMETER METHODS

The development of p-y curves from the results of an in-situ test offers potential advantages over conventional methods that rely on the results of laboratory tests on sampled soil. The expansion of a pressuremeter probe is analogous in some respects to the lateral movement of a pile through soil. The pressuremeter may be installed in a pre-bored hole to simulate the installation of drilled piers, or driven into the ground to simulate the installation of driven piles. The anticipated type of pile loading, whether rapid or slow or cyclic, may also be simulated by the pressuremeter. Pressuremeter methods do not distinguish between different soil types since the unit soil resistance is simply related to the slope of the pressure-volume curve, which may be obtained for many different types of soil. Briaud (1986) summarized nine different methods that have been proposed for development of p-y curves from pressuremeter data.

The method that appears to be best supported by field load test data is that of Briaud, et al. (1985). This method attempts to break down the unit of soil resistance into components of front bearing resistance and side friction resistance. The conventional p-y curve is, then, taken to be equal to the sum of a Q-y (front bearing) and an F-y (side friction) curve. The front bearing resistance, Q, and the side friction resistance, F, are correlated to the shape of the pressuremeter's net pressure-volume curve.

The front bearing resistance is computed as

$$Q = P^* \, b s_Q \tag{3.13}$$

where P^* = net pressuremeter pressure

s_Q = shape factor (from elasticity) equal to 1.0 for square piles and $\Pi/4$ for round piles.

The side friction resistance is computed as

$$F = b\, s_F \in (1 + \in) \frac{\Delta P^*}{\Delta x} \tag{3.14}$$

where s_F = shape factor (from elasticity) equal to 2 for square piles and 1 for round piles

$\in$ = probe volumetric strain from beginning of reload curve.

The corresponding pile deflection is assumed to be proportional to the radial pressuremeter deflection and is computed as

$$y = \frac{b}{2} \frac{\Delta R}{R_0} \tag{3.15}$$

where ΔR = increase in pressuremeter radius

R_0 = initial pressuremeter radius.

This expression for pile deflection assumes linear elastic soil behavior and thus may not be suitable for accurate description of pile resistance at large deflections. Pressuremeter methods appear to give reasonable results when applied to case histories for which pressuremeter data are available.

The pressuremeter test currently is not a standard test for routine foundation design in this country. It requires specialized equipment and some degree of expertise in its performance and interpretation. Because the pressuremeter diameter is small relative to that of typical full-scale piles, the influence of the ground surface is felt only to depths much shallower than those corresponding to full-scale piles. The pressuremeter does have many potential advantages for pile foundation design and it is anticipated that the advantages will be further illuminated by the results of current and future research. At the present time, however, the relative lack of lateral load test case histories for which pressuremeter data are also available appears to limit their utility for actual foundation design

DISCUSSION

To perform p-y curve analyses of the response of laterally loaded piles, WSDOT currently uses a computer program known as COM624 developed at and available through the Geotechnical Engineering Center at the University of Texas at Austin (Reese, 1984). The COM624 program is an enhanced version of its immediate predecessor, COM622. The primary enhancement of COM624 is the addition of an option that automatically generates p-y curves, given some required basic soil parameters. While this option may be bypassed, its use limits the user to particular p-y curve criteria, namely the soft clay criteria, the stiff clay criteria or the unified clay criteria for clays, and the Reese, et al., criteria for sands. WSDOT engineers commonly use this option to facilitate generation of p-y curves.

CHAPTER 4
REVIEW OF AVAILABLE LATERAL LOAD TEST DATA

INTRODUCTION

The procedures currently available for developing p-y curves, described in the previous chapter, are based on the results of full-scale field lateral load tests. Some of the earlier procedures were developed when only a very limited number of tests were available. More recent procedures have been developed from the results of a larger number of tests and thus may be more reliable.

Considerable uncertainty regarding p-y curve development remains, however, for many types of soil conditions. While the more recently developed p-y curve criteria are based on more tests than the previous criteria, they too are based on a relatively small number of tests. Additionally, most of the available p-y curve criteria were developed in response to the needs of the offshore construction industry. Consequently, they are predominantly based on, and oriented toward, the response of steel pipe piles embedded in offshore soils. For soil and pile conditions different than those from which these criteria were developed, the applicability of the criteria remains somewhat uncertain.

A review of available case histories identified during a comprehensive literature search of lateral load tests on piles was conducted as part of this investigation. The results of this review are described in this chapter.

LATERAL LOAD TEST REQUIREMENTS

Lateral load tests on piles are usually performed by loading the pile laterally without any axial load. The response of the pile is measured in terms of pile head deflection and rotation and in terms of the deflected shape of the pile along its axis. Pile head deflection is usually measured by LVDT or dial gauges, and pile head rotation by some form of levelling. The deflected shape of the pile is very difficult to measure directly but can be determined from direct measurements of pile curvature, obtained from pairs of strain gauges installed along the length of the pile, or from direct measurements of pile slope, obtained from slope inclinometer measurements. The development of p-y curves from the results of lateral load tests requires that such measurements be accurately made and recorded.

Because the response of piles to lateral loads is strongly influenced by the behavior of the soil, p-y curve criteria must be related to the behavior of the soil. The development of p-y curve criteria from the results of lateral load tests thus requires that the soil conditions at the site of the load test be well established and recorded. Ideally, information regarding the depth and thickness of each layer and the density, strength, and stress-strain behavior of each material should be available.

The procedure by which a field lateral load test is conducted influences the reliability of the test results. An accepted standard for conducting field lateral load tests is designated in ASTM Standard 3966-81, Standard Method of Testing, Piles Under Lateral Loads. This suggested procedure describes methods for the application of load, duration of loading, intervals for response measurement, and other details essential for the performance of a reliable field lateral load test.

In summary, a lateral load test from which p-y curve criteria can be reliably evaluated requires adequate instrumentation for the measurement of pile response, adequate characterization of soil conditions, and the use of proper procedures for the performance of the load test.

REVIEW OF EXISTING LATERAL LOAD TESTS

Introduction

A detailed review of all available literature containing the results of tests on single, laterally loaded piles was undertaken. Pile group tests were not considered in this literature review because p-y behavior cannot be directly evaluated from the results of lateral load tests on pile groups. During the course of the review, over 100 papers on laterally loaded piles were identified and reviewed and the information on the lateral load tests summarized. These papers contained information on well over 100 lateral load tests. The information presented in these papers varied from being complete and well-documented to being very sketchy and difficult to interpret.

Some of these papers described the results of model tests. While model tests are valuable for evaluating certain aspects of the response of laterally loaded piles, problems with scale and similitude are considered to be of enough significance to hinder their application to piles of the size often installed by highway departments such as WSDOT. Consequently, model tests were not subsequently considered in the synthesis of lateral load test data. Other tests applied relatively high-frequency dynamic loading to the piles. Since p-y curve analysis procedures are not directly applicable to dynamic loading in which inertial effects are significant, such tests were not further considered. Some of the load tests reported in the literature had either so little supporting information or such obviously flawed procedures that the results could not be considered reliable. These tests were also not further considered.

Rating of Available Load Test Information

The lateral load test case histories obtained in the literature review were rated with respect to the previously discussed requirements. It is important to realize that the descriptions of the case histories, rather than the actual load tests themselves, were rated. In this sense, the ratings may also reflect the quality of information presented in the reports of the case histories. Unfortunately, some very high quality load tests may have been given relatively low ratings due to the scarcity of information presented in the report of the load test. By this system, however, the converse could not occur.

The rating system was tied to the requirements for reliable lateral load tests as discussed in the previous section. Consequently, the case histories were rated in three areas: information on (a) soil conditions, (b) instrumentation, and (c) test procedures. Each of these categories was rated as being good, fair, or poor. The rating for information on soil conditions ranged from poor (no soil information presented) to good (high quality soil information). The rating for instrumentation ranged from poor (no instrumentation for measuring pile response) to good (results of high quality instrumentation provided along length of pile). The rating for test procedures ranged from poor (obviously poor procedure with deleterious influence on results) to good (procedure roughly equivalent to ASTM D3966- 81). While the rating system employed included some unavoidable subjectivity, ratings could be made with reasonable engineering judgment toward a goal of consistently identifying high-quality lateral load tests from which p-y behavior could be reliably inferred.

Lateral Load Test Database

The available references on full-scale, lateral load tests on single piles were carefully reviewed and rated. These tests, together with their ratings, constitute a database from which various p-y curve criteria can be evaluated. The lateral load test database was divided into three categories. The first category contains lateral load tests on piles installed in soil deposits that consisted predominantly of sands. The second category contains lateral load tests on piles installed in predominantly clay soils. The third category contains the few lateral load tests that do not fall clearly into either of the first two categories.

The lateral load test database for piles installed in clays is shown in Table 4-1. The lateral load test data base for piles installed in sands is shown in Table 4-2. There are very few reported case histories of full-scale, field lateral load tests in soils other than clays or sands. In fact, only three case histories in clearly different soil conditions were identified and the lack of information regarding instrumentation and load test procedures makes their results difficult to interpret. Botea, et al. (1973), tested 50-inch-diameter, drilled shafts extending through 41 feet of silt underlain by 15 feet of silty sand underlain by 40 feet of clayey silt. Robertson, et al. (1984), tested 6-inch-diameter piles in a deposit of 4 feet of sand-gravel fill underlain by 6 feet of peat underlain by 9 feet of clayey silt. Diaz, et al. (1984), tested large diameter piles in sloping rockfill.

SUMMARY OF AVAILABLE LATERAL LOAD TEST DATA

Even a brief review of the lateral load test data referred to in the previous section indicates that the overwhelming majority of reliable, well-documented lateral load tests have been performed in soil deposits that would be classified as either "clay" or "sand." This observation is not entirely unexpected. Since the available methods for analysis of laterally loaded piles were developed only relatively recently, the load tests from which these analytical methods could be verified were

Table 4-1
Load Tests on Piles Installed in Clays

Investigator	Pile Type	b (in)	L (ft)	Rating* Soil	Instr	Proc	Total	Comments
Alizadeh and Davisson (1970)	Timber	10	36.5	G	F	G	F	Fat clay (CH) to 10 ft. underlain by lean clay
Bhushan, et al. (1979)	R/C	48	15	G	F	F	F	Overconsolidated, hard, silty clay and sandy clay (CL)
Bhushan, et al. (1979)	R/C	48	12.5	G	F	F	F	Overconsolidated, hard, silty clay and sandy clay (CL)
Bhushan, et al. (1979)	R/C	48	15.5	G	F	F	F	Overconsolidated, hard, silty clay and sandy clay (CL)
Bhushan, et al. (1979)	R/C	24	15.5	G	F	F	G	Overconsolidated, hard, silty clay and sandy clay (CL)
Capozzoli (1968)	Pipe	10	115	G	F	F	F	Soft to medium stiff, silty clay
Evans and Hummel (1972)	PC	16.4	36.5	G	G	P	P	Clays of volcanic origin
Gill (1968)	Pipe	12.8	16.6	F	F	F	F	Insensitive, slightly organic silty clay. Ponded.
Gill (1968)	Pipe	16	26.6	F	F	F	F	Insensitive, slightly organic silty clay. Ponded.
Gill (1968)	Pipe	12.8	16.6	F	F	G	F	Insensitive, slightly organic silty clay
Gill (1968)	Pipe	16	26.6	F	F	G	F	Insensitive, slightly organic silty clay
Ishii and Fujita (1963)	Pipe	47	75	G	F	P	P	Offshore soft clay underlain by sand, silt layers
Ishii and Fujita (1965)	Pipe	59	74	G	F	P	P	Offshore soft clay underlain by sand, silt layers
Jamiolkowski (1973)	Pipe	48	72	G	F	F	F	Offshore, normally consolidated, soft silty clay
Johnson, et al. (1983)	R/C	18	35	G	G	F	F	Stiff, expansive clay (CH). 18 year old shaft
Kasch	R/C	36	20	G	G	G	G	Stiff to v. stiff clay (CL) to 5 ft. overlying (CH)
Kim, et al. (1979)	Steel	10	40	G	G	G	G	Stiff silty clay (CL)
Lee and Gilbert (1979)	Steel	27	40	G	G	P	P	Very soft peat over soft plastic clay. Tapered pile, inclined loading.
Lundin and Robinovich (1977)	R/C	12	15.7	F	F	F	F	Stiff, loess-like clay
Matlock (1970)	Pipe	12.8	42	G	G	G	G	Slightly overconsolidated marine clay (CH)
Matlock (1970)	Pipe	6	38	G	G	G	G	Very soft clay (CH). Occasional thin layers of peat, silt, sand
Matlock (1970)	Pipe	12.8	42	G	G	G	G	Freshwater lacustrine soft clay (CH)
McClelland and Focht (1958)	Pipe	24	75	G	G	G	G	Very soft clay (CH). Occasional thin silt layers.
Price and Wardle (1981)	H-pile	14	69	P	G	G	P	London clay
Price and Wardle (1981)	Pipe	16	54	P	G	G	P	London clay
Reese and Welch (1975)	R/C	30	42	G	G	G	G	Stiff to v. stiff red clay
Reese and Welch (1975)	Pipe	24	60	G	G	G	G	Very stiff, heavily overconsolidated marine clay. Ponded.
Sorochan and Bykoff (1976)	R/C	24	9.8	G	F	F	F	Stiff brown clay

* G = good; F = fair; P = poor

Table 4-2
Load Tests on Piles Installed in Sands

Investigator	Pile Type	b (in)	L (ft)	Rating* Soil	Rating* Instr	Rating* Proc	Rating* Total	Comments
Adams and Radhakrishna (1973)	R/C	60	20	G	F	P	P	Loose, silty sand to 5 ft. overlying dense, fine to medium sand
Alizadeh and Davisson (1970)	Timber	10	37	G	F	G	F	Sand and gravel to 4 ft. overlying clay
Bhushan, et al. (1981)	R/C	36	18	G	F	G	F	Silty sand. Medium dense (Ø = 36°) to 3 ft. dense (Ø= 42°) below
Bhushan, et al. (1981)	R/C	42	17	G	F	G	F	Poorly graded sand to silty sand (Ø = 38° - 40°)
Bhushan, et al. (1981)	R/C	48	18	G	F	G	F	Silty sand. Medium dense (Ø = 36°) to 6 ft., dense (Ø = 42°) below
Fenelli and Galateri (1981)	R/C	24	49	G	G	P	P	Dense, volcanic sand (pozzolana)
Davisson and Salley (1969)	R/C	48	18.5	F	G	P	P	Loose-medium dense sand to 9 ft. underlain by sandstone and shale
Jennings, et al. (1984)	Steel	18	22	G	G	P	P	Marine silty sands underlying 3 ft. fill/alluvial sediments
Long and Reese (1984)	R/C	48	73	G	G	G	G	Dense sand to 40 ft. (Ø = 38°) overlying clay
Lubking (1977)	Pipe	122	59	G	G	G	G	Marine sand with thin layers of clay, silt
Lubking (1977)	Pipe	162	59	G	G	G	G	Marine sand with thin layers of clay, silt
Mansur, et al. (1964)	PC	20	53	G	F	G	G	Dense, poorly graded sand (Ø = 42°)
Mansur, et al. (1964)	Pipe	16	53	G	G	G	G	Dense, poorly graded sand (Ø = 42°)
Mansur, et al. (1964)	Pipe	16	53	G	G	G	G	Dense, poorly graded sand (Ø = 42°)
Mason and Bishop (1953)	H-pile	16	40	G	G	F	F	Medium dense sand embankment (Ø = 35°)
Paduana and Yee (1974)	Pipe	10.75	40	G	G	P	P	Pea-gravel backfill in 18-inch predrilled hole through 12 ft. compacted fill
Paduana and Yee (1974)	H-pile	8.4	43	G	G	P	P	Pea-gravel backfill in 18-inch predrilled hole through 12 ft. compacted fill
Paduan and Yee (1974)	Pipe	10.75	70	G	G	P	P	Pea-gravel backfill in 18-inch predrilled hole through 12 ft. compacted fill
Reese, et al. (1975)	Pipe	24	69	G	G	G	G	Silty sand (Ø = 38°)
Reese, et al. (1975)	Pipe	20	63	G	P	F	P	Medium sand (Ø = 37°)
Reese, et al. (1975)	Pipe	6	21	G	G	G	G	Loose, fine alluvial sand (Ø = 34°). Capped by 15 inches compactedclay.
Robinson (1979)	Timber	10	60	G	F	F	F	Fine sand to 3.5 ft. overlying clayey silt and sandy silt to 15 ft.
Stevens et al (1979)	Timber	13	45	G	F	G	F	Fine to medium, poorly graded sand (Ø = 39°)
Tominaga, et al. (1983)	PC	19.7	90	P	F	F	P	Described only as "loam"

* G = good; F = fair; P = poor

logically performed in soil conditions for which geotechnical engineers could describe the behavior of most reliably.

APPLICABILITY OF AVAILABLE LATERAL LOAD TEST DATA

Current p-y curve criteria have been developed for piles embedded in clays and in sands. The criteria now available have been developed on the basis of a significant number of full-scale, field lateral load tests in clays and sands. Subsequent full-scale, field lateral load tests on piles embedded in clays and sands may be expected to either confirm the accuracy of these criteria or to provide information from which relatively minor adjustments to the criteria may be incorporated. Data from full-scale, field lateral load tests on piles embedded in such materials as gravels, silts, highly organic or peaty soils, or combinations of these soils and others are not currently available. The acquisition of such data is necessary so that the analysis of the response of laterally loaded piles in these soils can be performed with the same confidence that currently exists for piles in clay or sand.

CHAPTER 5
APPLICABILITY OF EXISTING P-Y CURVE CRITERIA TO WESTERN WASHINGTON SOIL CONDITIONS

INTRODUCTION

Highway structures in western Washington are constructed under a wide variety of geographical and topographical conditions and geologic history. Consequently, the foundations for such structures may penetrate widely different materials at different locations. Soil conditions in western Washington may range from peats to very soft silts and clays to very stiff, glacially overconsolidated soils. The p-y behavior of these soils will differ among each other and, in some cases, may be different than any conditions for which p-y curve development criteria are available.

This chapter will briefly review the regional geology of western Washington and discuss its influence on soil conditions found at sites of highway structures in western Washington. At many sites, the soil conditions are similar enough to those from which existing p-y curve criteria were developed that these criteria may be used with confidence. At other sites, the applicability of existing p-y curves may be uncertain. Soil conditions at such sites will be identified and discussed.

REGIONAL GEOLOGY

The geology of western Washington is usually described in terms of three physiographic regions: the Puget Lowland, the Willapa Hills, and the Olympic Mountains (USGS, 1966).

Puget Lowland

The Puget Lowland is by far the most heavily populated of the three physiographic regions and the one in which most pile-supported highway structures are constructed. The Puget Lowland is a topographic depression elongated in the north-south direction bounded by the Cascade Range on the east and the Olympic Range on the west. The topography of the Puget Lowland reflects the repeated glaciations that occurred in the Pleistocene age (1.8 million to 10,000 years ago). Rolling hills of elevations typically between 300 and 600 feet above sea level are separated by north-south trending river valleys which are usually 50 feet or less above sea level.

Bedrock in the Puget Lowland is usually covered by overlying glacial deposits. In the most recent glaciation (Vashon Stade), the Cordilleran ice sheet advanced south from British Columbia reaching Seattle about 15,000 years ago and reaching its maximum extent (50 miles south of Seattle) approximately 14,000 years ago. In the Seattle area, the maximum ice thickness was between 3,000 and 4,000 feet before the glacier receded about 12,500 years ago.

After the Vashon glacier blocked the Strait of Juan de Fuca, the Lawton Clay was deposited in the fresh water lake that formed to the south. Outwash sands and gravels, now known as the Esperance Sand, were deposited in a thick blanket overlying the Lawton Clay as the glacier advanced. These materials, deposited in front of the advancing glacier, were capped by the Vashon glacial till that currently covers most of the hills in the Puget Lowland. As the melting glaciers receded, recessional deposits collected in depressions, swales, and channels. These deposits typically grade into bog, lake, and stream deposits.

Willapa Hills

The Willapa Hills cover an area of about 3,000 square miles in the southwestern corner of Washington. These hills, which reach a maximum elevation of just over 3,100 feet, represent an extension of the Oregon Coast Range. Bedrock in the Willapa Hills is a large slab of oceanic crust that has been raised approximately 2 miles above the usual elevation of such materials. Pillow basalts are exposed in some areas but are often covered by younger oceanic sedimentary rocks such as muddy sandstones and mudstones. Soils in the Willapa Hills have derived from these underlying rocks by weathering and transportation to form localized stream and alluvial deposits.

Olympic Peninsula

The Olympic Mountains consist generally of the same materials as the Willapa Hills but have been chaotically deformed and contorted. In the core of the Olympic Mountains, sandstones currently exist beneath the older pillow basalts, apparently scraped from the surface of the oceanic crust and stuffed underneath the basalt. Soil cover is slight over much of the Olympic Peninsula, often accumulating in stream, alluvial, and beach deposits.

SOIL CONDITIONS

A wide variety of soil conditions are encountered in the construction of highway structures in western Washington. In terms of soil types, nearly every category of soil, from highly organic peats through clays, silts, sands, and gravels, may be located at various sites. The behavior of many of these materials is further complicated by the variety of geologic histories to which the different deposits have been subjected. The behavior of fine-grained soils in the Puget Sound area may be dominated by their heavily overconsolidated state while similarly classified materials along the coast may be very nearly normally consolidated.

APPLICABILITY OF EXISTING P-Y CURVE CRITERIA

Existing p-y curve criteria are available for clays and sands. These criteria are based on the results of a significant number of full-scale, field lateral load tests on piles embedded in these materials. Use of these existing p-y curve criteria for analyses of laterally loaded piles in the types of soils for which the criteria were developed is expected to produce reasonable results.

For soil conditions that would not be classified as either predominantly clay or predominantly sand, p-y curve criteria are not currently available and, further, lateral load test data from which p-y curve criteria could be developed are not available. Since pile-supported highway structures are often supported in soils that would not be classified as sand or clay, there is a distinct need for further research to predict the p-y behavior of such sites. The soil conditions at these sites may consist of gravels, silts, highly organic or peaty soils, or some combination of these and other soils. Such research must consist of the performance and analysis of the results of full-scale, field lateral load tests on piles embedded in such soils and should be undertaken at the earliest opportunity.

RECOMMENDATIONS

Even though p-y curve criteria are not currently available for many of these soil conditions, designers of foundations for highway structures are often required to evaluate the lateral load response of piles embedded in such soils. In the absence of explicit p-y curve criteria for these soils, the designer is necessarily required to incorporate judgment into the design of laterally loaded piles. The purpose of this section is to provide some general guidance for analysis of the lateral load response of piles in these soil conditions during the interim period before the results of future research become available. These recommendations are no substitute for local experience, and the results of such analyses should be interpreted with the judgment of an experienced geotechnical engineer.

The p-y behavior of gravels is expected to be approximately similar to that of sands; consequently, use of the extended hyperbolic criteria is suggested. Due to the uncertainty in applying this method to gravels, and to the difficulty of evaluating the properties of gravels, the use of conservative soil properties is recommended.

The behavior of silts and predominantly silty soils has been a subject of considerable research interest in geotechnical engineering. This research has been primarily limited to date to investigations of relatively basic aspects of the characterization of silt behavior. Because of the complexity of behavior exhibited by silts and because the p-y behavior of silts has not been explicitly investigated, the following interim recommendations are necessarily simplistic and conservative. The response of laterally loaded piles embedded in silt may be evaluated by considering the silt both as a clay using the Integrated Clay Criteria and as a sand using the Extended Hyperbolic Criteria, and using reasonable engineering judgment to interpret the results. For non-plastic silts, and for piles subjected to very slow loading, the response is likely to be closer to that given by the Extended Hyberolic Criteria. For plastic silts, or for piles subjected to rapid loading, the response is likely to be closer to that given by the Integrated Clay Criteria.

The time-dependent response, including creep, stress relaxation, and secondary compression, of very highly organic or peaty soils renders specific recommendations impossible.

When pile foundations penetrating significant thicknesses of such soils are required to resist lateral loads, batter piles may provide the most reliable means of developing the required lateral load resistance.

CHAPTER 6
PILE GROUPS

INTRODUCTION

Piles are often called upon to resist isolated loads that are so large that they cannot be supported by a single pile of reasonable dimensions. In such cases, the isolated loads are supported by a group of piles connected by a common pile cap. The behavior of individual piles within a laterally loaded pile group is likely to be different than that of isolated piles due to a number of factors.

Some of these factors result from the effects of pile installation. Soil disturbance associated with pile installation affects the lateral load response of the individual piles and the entire pile group in a way which cannot be modeled analytically. Pile group installation effects are dependent on pile spacing and installation techniques, sequence, and speed, among other things. The lateral load response of pile groups is also influenced by the nature of the restraint provided by the pile cap, the nature of the structural loading, and the configuration of the pile group. One of the most important factors influencing the response of pile groups to lateral loading is the interaction between piles within the group, commonly referred to as pile-soil-pile interaction.

The great number of known and unknown factors influencing pile group response, few of whose effect can be accurately quantified, renders exact and general solution of this problem impossible. The following sections will describe several approximate methods that have been proposed for pile group response analysis.

METHODS OF ANALYSIS

A number of methods have been proposed for analysis of the response of laterally loaded pile groups. They differ in the manners in which they treat the soil response, pile-soil interaction, pile-soil-pile interaction, and interaction with the supported structure. These methods include (1) elastic methods (Poulos, 1980) in which pile-soil-pile interaction is evaluated by use of the Mindlin solution for stresses due to point loads below the surface of an elastic halfspace, often implemented within boundary element methods (Banerjee and Davies, 1977); (2) structural interaction models (Hrennikoff, 1950; Reese, et al., 1970; Selby and Poulos, 1983), which may use p-y curve analysis for pile-soil interaction but which neglect pile-soil-pile interaction; (3) Winkler models (Hariharan and Kumarasamy, 1982; Nogami and Paulson, 1985), which consider pile-soil-pile interaction within horizontal planes; (4) modified p-y curve methods (Matlock, et al., 1980; Bogard and Matlock, 1983) in which single pile p-y curves are empirically modified to obtain average group pile p-y curves from which the response of the group can be obtained; and (5) hybrid methods (Focht and Koch, 1973; O'Neill, et al., 1979; Ha, et al., 1980) that use conventional p-y curve analysis for pile-soil interaction and the Mindlin solution for pile-soil-pile interaction.

COMPARISON OF METHODS

An evaluation of different methods of analysis of the behavior of laterally loaded pile groups was recently performed by O'Neill and Dunnavant (1985). O'Neill and Dunnavant compared the ability of each of four different methods that utilize p-y curves to predict the response observed in tests of laterally loaded pile groups. Selected for comparison were a modified p-y curve (Bogart-Matlock) method, a Winkler (Hariharan-Kumarasamy) method, and two hybrid (Focht-Koch and O'Neill, et al.) methods.

Description of Methods

The Bogard-Matlock method develops p-y curves for an "average" group pile within a circular pile group by combining the p-y curves that would be used for single piles with those that would be developed for an imaginary pile with diameter equal to the outer diameter of the pile group. The average p-y curves are then used in a standard, single-pile p-y curve analysis. The deflection of the pile group is taken to be equal to that of the average group pile at a load equal to N times the load applied to the average group pile, where N is the number of piles in the group.

The Hariharan-Kumarasamy method uses the theory of elasticity to develop plane strain solutions for the stresses and displacements caused by the lateral movement of a rigid disk. The solution can be expressed in terms of p- and y-multipliers, which are applied to single-pile p-y curves in order to develop group pile p-y curves. These multipliers can be determined for each pile within the group, and Hariharan and Kumarasamy suggest that the means of all of the p- and y-multipliers be used to develop "average" pile p-y curves.

The Focht-Koch method uses the Mindlin solution for stresses and displacements due to a point load beneath the surface of an elastic half-space to evaluate additional pile displacements due to pile-soil-pile interaction. The contributions of other piles within the group are summed for determination of a y-mulitplier. A p-multiplier may also be used in the Focht-Koch method of analysis. These multipliers are applied to single-pile p-y curves to obtain p-y curves for an average group pile.

The O'Neill, et al., method is a more general and complex method for three-dimensional analysis of pile group response to lateral, vertical, overturning, and torsional loading. Pile-soil-pile interaction is evaluated by use of the Mindlin solution and expressed in terms of additional elastic pile displacements due to surrounding piles. The O'Neill, et al., method is encoded in the PILGP series of computer programs.

Results of Comparison

The four methods were compared by applying them to a total of seven case histories of pile group lateral load tests. Four of the pile groups were in predominantly clay soils and the other three in predominantly sand. Each of the selected pile group load tests were accompanied by the results of a load test on a single, isolated pile.

O'Neill and Dunnavant found that none of the methods tested provided a consistently superior solution for the range of load tests considered. The O'Neill, et al., method provided the best evaluation of average pile group response, particularly at low levels of deflection; however, it did not predict well the distribution of pile loads within the group. The Focht-Koch method was most accurate at group deflections of 10 to 20 percent of individual pile diameters; however, it showed unconservatively stiff behavior at high loads. At large deflections, the Bogard-Matlock method showed more conservative deflections; however, it was found to be very sensitive to differences in ultimate soil resistance. For the case histories examined, however, the Focht-Koch method provided the most accurate and consistent estimates of pile group behavior of the simplified methods.

Recommended Procedures

O'Neill and Dunnavant, after finding that no single analytical method provided the most accurate estimates of pile group behavior for all loading conditions, suggest procedures for the evaluation of pile group response that varies with the loading conditions. For low loading levels, they recommend use of an advanced procedure, such as the O'Neill, et al., method. For design event loads, the Focht-Koch method is recommended, though it is known to underpredict deflections at loads, causing deflections of 20 percent or more of the individual pile diameters. When possible, use of more than one method is recommended using upper and lower bound soil properties so that a range of possible pile group responses may be obtained.

CHAPTER 7
APPLICABILITY OF P-Y CURVE ANALYSIS

INTRODUCTION

Of great concern to designers of pile foundations and pile-supported structures are the conditions under which a particular method of analysis is applicable and the conditions under which it is not applicable. Of the methods of analysis described in Chapter 2, some are applicable only under a very narrow range of soil, pile, and loading conditions, while others may be reliably applied to a much wider range of conditions. The p-y curve method of analysis falls somewhere in the middle with respect to its applicability to various soil, pile, and loading conditions.

An evaluation of the applicability of the p-y curve analysis method to different soil conditions was the major purpose of this research and has been described in detail earlier in this report. It is also useful, however, to consider the applicability of the method to different pile conditions and different loading conditions. The purpose of this chapter is to briefly review and comment upon such applicability.

APPLICABILITY TO VARIOUS PILE CONDITIONS

In p-y curve analyses, the physical behavior of the pile in bending is represented entirely by its flexural stiffness. The fourth order differential equation (Equation 2.2) commonly used to describe the response of a laterally loaded pile is based on the assumption that the pile itself behaves as a linear, elastic material. This is expressed in the assumption that the pile flexural stiffness, EI, is a constant over the entire range of curvature experienced by the pile. This assumption may be violated in two ways. One is by the use of pile materials which exhibit nonlinear stress-strain behavior within the working range of stresses. The other is by the use of pile materials whose cross-sectional load resisting geometry is subject to change within the working range of stress.

Assumption of a linear, elastic pile material is generally quite reasonable for steel piles. Steel behaves as a linear, elastic material over the range of stresses to which it is generally subjected in the field, and steel piles, whether H-pile or pipe pile sections, generally retain their geometric characteristics until very high curvatures are reached. The assumption of constant flexural rigidity for concrete piles and timber piles is much less accurate, however. Concrete, as a material, is well-known to exhibit non-linear stress strain behavior, particularly at compressive stresses greater than approximately one-half the compressive strength. More importantly, concrete exhibits a very low tensile strength and when subjected to bending may develop cracking at relatively low curvatures. Cracking of laterally loaded, reinforced concrete, deep foundations has been observed on numerous occasions. The flexural stiffness of prestressed concrete piles and timber piles may also exhibit considerable nonlinearity.

The flexural stiffness characteristics of a particular reinforced or prestressed concrete pile may be determined using models of concrete, reinforcing steel, and prestressing strand behavior.

The response of a pile whose flexural stiffness varies with curvature embedded in a soil profile may be described by the following fourth order differential equation:

$$\frac{d^2}{dx^2}\left(EI\,\frac{d^2y}{dx^2}\right) + Q\frac{d^2y}{dx^2} - p = 0 \qquad (7.1)$$

This equation may also be written in difference form and solved numerically with both p and EI being determined iteratively. A procedure has been developed at the University of Washington (Kramer and Heavey, 1987) for simultaneously iterating toward a displacement-compatible soil resistance and a curvature-compatible pile flexural stiffness. This procedure has been incorporated into a computer program that requires only the input of an appropriate moment-curvature relationship for the pile being analyzed, in addition to the information required for conventional analyses. At loading levels below that which would cause nonlinear pile bending behavior, the model is identical to that of conventional p-y curve analyses such as that encoded in COM624.

APPLICABILITY TO VARIOUS LOADING CONDITIONS

In the design of pile foundations and pile-supported structures, the results of an analysis of laterally loaded pile response are commonly used in two ways. First, they are used in the design of the pile foundation itself. Second, they are used in the design of the pile-supported structure. The second application may be particularly important in situations where seismic loading may be critical.

The applicability of p-y curve analysis methods to various loading conditions may be limited by the similarity between the anticipated loading conditions and the loading conditions from which the p-y curves were developed and also by the physical characteristics of the p-y curve model itself. Lateral loading conditions to which a pile foundation may be subjected in the field may be broadly divided into four categories: static loading, cyclic loading, dynamic loading, and seismic loading.

Static Loading

The p-y curve method of analysis is best suited for analysis of the response of piles to monotonically increasing, or static, loads. Most full-scale field lateral load tests involve monotonically increasing loading (often interrupted by cyclic loading at various levels) from which p-y curve criteria have been developed. Consequently, the p-y curve method of analysis, properly applied, may be considered to be a reliable method for evaluation of pile response to static lateral loading.

Cyclic Loading

The p-y curve method of analysis has also been applied to cases in which the pile is subjected to cyclic lateral loads. In this context, the term cyclic loading refers to repetitively applied loads that are applied slowly enough that inertial effects are negligible. As discussed in previous chapters, p-y curve criteria have been developed for different soils for cyclic loading. These cyclic p-y curves essentially represent equivalent "softened" p-y curves, which attempt to represent the accumulation of permanent pile deflection caused by the repetitive nature of the cyclic loading. Because the conventional p-y curve analysis procedure is incapable of following a given cyclic loading path, its versatility in predicting the response of piles to different cyclic loading conditions is limited. Cyclic loading p-y curves have been used on many occasions for prediction of the response of offshore structure pile foundations to cyclic wave loading.

Dynamic Loading

Dynamic loading refers to loading at the top of the pile that is applied so rapidly that inertial and damping effects become significant. Examples of piles subjected to dynamic lateral loading include various types of machine foundations, dolphins, and dock structures. In dynamic loading, the only excitation is considered to be applied at the top of the pile.

P-y curve analyses have limited applicability to analysis of the response of piles to dynamic lateral loads. Recently, however, Reese (1987) suggested that p-y curve analyses "might be useful" for the evaluation of foundation stiffness for dynamic analyses. Such an approach appears to be limited to cases in which dynamic accelerations and velocities are low.

Seismic Loading

When the pile foundation supporting a highway structure is subjected to seismic loading, the piles are subjected to lateral loads from the structure and also to loads resulting from the lateral deformation of the ground in which they are embedded. While p-y curve analysis procedures are capable of analyzing the response of piles to lateral loads imposed on the pile by the soil below the ground surface, it can only do so correctly when these loads are applied very slowly. Reese (1987) indicated that the p-y curve analysis method has limited applicability for analysis of pile response to seismic loads.

A number of methods are available for analysis of the seismic response of pile foundations (Stanton, et al., 1987). Most of the methods suffer from a lack of field verification of their accuracy. Among the most promising of these methods is the finite element model described by Roesset and Angelides (1980). The use of such a model, however, may require significant time and expertise, which are not commonly available for routine projects. Dobry, et al. (1981), used the finite element model of Roesset and Angelides to propose expressions for spring and dashpot constants that could be used in a simple discrete, multiple-degree-of-freedom analysis. O'Neill (1987) suggested performing "plucking tests" and incorporating the results into an elastic analysis in order to obtain pile stiffnesses. These types of approaches may represent reasonable and relatively simple methods for analysis of seismic response. Identification of a simplified method for the evaluation of seismic response, while beyond the scope of the present investigation, would appear to be a very worthwhile pursuit.

CHAPTER 8
FULL SCALE FIELD LATERAL LOAD TEST PROCEDURES

INTRODUCTION

As described in the preceding chapters, full-scale field lateral load tests have been performed by many investigators. Careful review of the reports of these tests indicates that there have been nearly as many different testing procedures used as there have been tests performed. Many of these tests were performed before any test standard had become available. Since 1981, an ASTM Standard Method of Testing titled "Piles Under Lateral Loads: D3966-81" has been available for the performance of full-scale field lateral load tests.

As discussed in Chapter 5, successful performance of a field lateral load test requires that the soil conditions be accurately characterized, that the response of the pile be measured by a suitable instrumentation scheme, and that a proper loading procedure be used. In this chapter, the requirements for a successful lateral load test will be reviewed and procedures for lateral load testing of full-scale piles in the field will be discussed.

SOIL CONDITIONS

As previously mentioned, proper interpretation of the results of full-scale field lateral load tests requires that the nature of the supporting soil be well defined. At the site of a lateral load test the depth and thickness of each soil layer should be determined. As a minimum, the stress-strain-strength behavior of the soils within about ten pile diameters of the ground surface should be evaluated by triaxial compression testing, with particular attention paid to the soils at depths of less than five pile diameters. Valuable additional information may be obtained from in-situ tests such as the core penetration test, the standard penetration test, the pressuremeter test, and the dilatometer test. The soil below depths of about 10 pile diameters does not strongly influence the response of the pile and consequently requires less attention in field investigation and laboratory testing.

PILE INSTRUMENTATION

Accurate evaluation of the response of the pile, and development of p-y curve criteria for the response of the soil, require that the magnitude of the lateral load and the deflected shape of the pile be known at each level of applied lateral load.

Measurement of lateral load magnitude may be made with a standard load cell but should be backed up by recorded measurements of the fluid pressure in the hydraulic jack used to apply the lateral load. The hydraulic jack should be calibrated prior to the performance of the load test in order to confirm operation of the load cell during the test. If suspected malfunction of the load cell deems it necessary, the hydraulic jack may be more accurately calibrated after conclusion of the load test.

The defined shape of the pile may be measured in a number of ways. Common to all full-scale pile load tests is the accurate measurement of pile head deflection and rotation. Several appropriate methods for making such measurements are described in the ASTM D3966-81 standard. Accurate evaluation of the p-y behavior of the soil, however, requires that the deflected shape of the pile be known over its entire length. Since it is very difficult to measure the deflected shape of a pile directly, the deflected shape is usually obtained from some more easily measured aspect of the pile response. Pile curvature and pile slope are parameters which are more easily measured and from which pile deflections may be determined. Pile curvature may be measured at a given depth by a pair of strain gauges mounted opposite each other in the direction of loading. Typically, curvature is measured at a discrete number of locations along the length of the pile, from which a curvature profile is obtained. Integration of this curvature profile twice, with proper consideration of boundary conditions, yields the deflected shape of the pile. The accuracy of this procedure depends on the spacing between the pairs of strain gauges. Since the curvature is usually greatest near the top of the pile, the strain gauges should be spaced relatively closely in that area. A typical spacing sequence would be one pile diameter for the upper ten pile diameters, two pile diameters for the next ten pile diameters, and at least four pile diameters below that. The details of a strain gauge instrumentation scheme should, however, be designed on the basis of careful analyses of the anticipated behavior of the particular pile at the particular site. Pile slope may be measured with a slope inclinometer in a slope inclinometer casing installed within the pile. With a slope inclinometer, a continuous profile of pile slope may be obtained and integrated once to obtain the deflected shape of the pile. In the past, many investigations have included instrumentation both by sets of strain gauges and by slope inclinometers.

Load test instrumentation may be temperature sensitive. Since the performance of a load test may extend over a period of several hours, during which temperatures may fluctuate, the temperature should be recorded periodically. It is also desirable to place a canopy over the exposed instrumentation in order to minimize temperature fluctuations.

TEST PROCEDURE

Successful performance of a full-scale field lateral load test requires careful attention to the details of pile installation procedures, lateral load application mechanisms, lateral load reaction systems, and loading sequences. These details are discussed and suggested procedures presented in ASTM D3966-81.

CHAPTER 9
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APPENDIX A
INTEGRATED CLAY CRITERIA

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As discussed in Chapter 3, the Integrated Clay Criteria applies to clays ranging in strength from soft to very stiff. Consequently, the user is not required to make a prior decision as to whether the soil will behave as a soft clay or a stiff clay. The Integrated Clay Criteria also incorporates the results of recent research into the effect of pile diameter and relative pile-soil stiffness on p-y curve characteristics.

The purpose of this appendix is to present, in step-by-step form, the procedure for constructing p-y curves by the Integrated Clay Criteria. The procedure is as follows:

1. Calculate the ultimate soil resistance as

$p_{ult} = F\, N_p\, c\, b$

where F = empirical reduction factor representative of soil strength degradability for which recommended values are given in the table below:

	UU Triaxial Compression Failure Strain		
Factor	**≤ 0.02**	**0.02-0.06**	**≥ 0.06**
F_{static}	0.50	0.75	1.00
F_{cyclic}	0.33	0.67	1.00

$$N_p = 3 + 6\frac{x}{x_{cr}} \qquad \text{for } x \leq x_{cr} \tag{A.2}$$

$$= 9 \qquad \text{for } x > x_{cr}$$

$$x_{cr} = \text{critical depth} \tag{A.3}$$

$$= L_c/4$$

L_c = critical pile length (greater than which increasing pile length does not influence pile response)

$$= 3.0 \left[\frac{EI}{E_s b^{1/2}} \right]^{0.286} \quad (A.4)$$

2. Calculate critical deflection which is expressed as

$$y_c = 0.8 \in_{50} b^{0.5} \left[\frac{EI}{E_s} \right]^{0.125} \quad (A.5)$$

3. For static loading, determine the shape of the p-y curve from the following equations as shown in Figure 3-6 (a):

$$p = 0.5p_u(y/y_c)^{0.387} \qquad \text{for } y < 6y_c \quad (A.6)$$

$$p = p_u \left[F_s + (1\text{-}F_s) \frac{x}{x_{cr}} \right] \qquad \text{for } y > 6y_c \text{ and } x < x_{cr} \quad (A.7)$$

$$p = p_u \qquad \text{for } y > 6y_c \text{ and } x > x_{cr} \quad (A.8)$$

4. For cyclic loading, determine the shape of the p-y curve from the following equations as shown in Figure 3-6 (b):

$$p = 0.5\,(y/y_c)^{0.387} \qquad \text{for } y < y_c \quad (A.9)$$

$$p = 0.5\,p_u F_c\,(x/x_{cr}) \qquad \text{for } y > y_c \text{ and } x < x_{cr} \quad (A.10)$$

$$p = 0.5\,p_u \qquad \text{for } y > y_c \text{ and } x > x_{cr} \quad (A.11)$$

O'Neill and Gazioglu (1984) recommend a minimum y-value of 0.00336 to define a proper stiffness at low deflections.

APPENDIX B
EXTENDED HYPERBOLIC CRITERIA

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EXTENDED HYPERBOLIC CRITERIA

As discussed in Chapter 3, the extended hyperbolic criteria for sands differs little from the Reese, et al., criteria under most conditions. The extended hyperbolic criteria has, however, been validated by comparison with more full-scale, field load tests and also includes an empirical adjustment for tapered piles and H-piles.

The purpose of this appendix is to present, in step-by-step form, the procedure for constructing p-y curves by the extended hyperbolic criteria.

1. Calculate the ultimate soil resistance as the lesser of the values given by the following two equations:

$$p_u = (c_1 z + c_2 b)\gamma' z \qquad \text{(B.1)}$$

$$p_u = c_3 b \gamma' z \qquad \text{(B.2)}$$

where the constants c_1, c_2, and c_3 are obtained from Figure B-1 (a).

2. Determine the shape of the p-y curve by the following equation:

$$p = \eta \ A \, p_u \tanh\left[\left(\frac{kz}{AP_u}\right) y\right] \qquad \text{(B.3)}$$

where $A = 3 - 0.8 \frac{z}{b} \geq 0.9$ (static)

$A = 0.9$ (cyclic)

and $\eta = 1$ for circular or prismatic piles and 1.5 for tapered piles or H-piles and where k is as shown in Figure B-1 (b)

ϕ (degrees)	C_2	C_1	C_3	Critical z/D
15.	1.1096	.4454	4.6195	7.881
16.	1.1932	.4982	5.2494	8.142
17.	1.2788	.5553	5.9522	8.417
18.	1.3666	.6170	6.7374	8.704
19.	1.4567	.6838	7.6159	9.007
20.	1.5493	.7562	8.6001	9.325
21.	1.6447	.8344	9.7042	9.659
22.	1.7430	.9191	10.9446	10.011
23.	1.8445	1.0109	12.3399	10.382
24.	1.9495	1.1103	13.9116	10.774
25.	2.0581	1.2181	15.6846	11.187
26.	2.1706	1.3350	17.6874	11.624
27.	2.2874	1.4618	19.9533	12.085
28.	2.4088	1.5995	22.5206	12.574
29.	2.5351	1.7491	25.4339	13.092
30.	2.6667	1.9117	28.7451	13.641
31.	2.8039	2.0887	32.5149	14.225
32.	2.9473	2.2813	36.8140	14.845
33.	3.0973	2.4913	41.7255	15.505
34.	3.2544	2.7204	47.3470	16.208
35.	3.4192	2.9704	53.7935	16.958
36.	3.5922	3.2438	61.2007	17.760
37.	3.7742	3.5428	69.7295	18.617
38.	3.9659	3.8703	79.5711	19.535
39.	4.1680	4.2295	90.9533	20.519
40.	4.3815	4.6240	104.1481	21.576
41.	4.6073	5.0576	119.4823	22.713
42.	4.8465	5.5351	137.3486	23.939
43.	5.1002	6.0616	158.2214	25.261
44.	5.3699	6.6431	182.6759	26.690
45.	5.6569	7.2863	211.4116	28.239

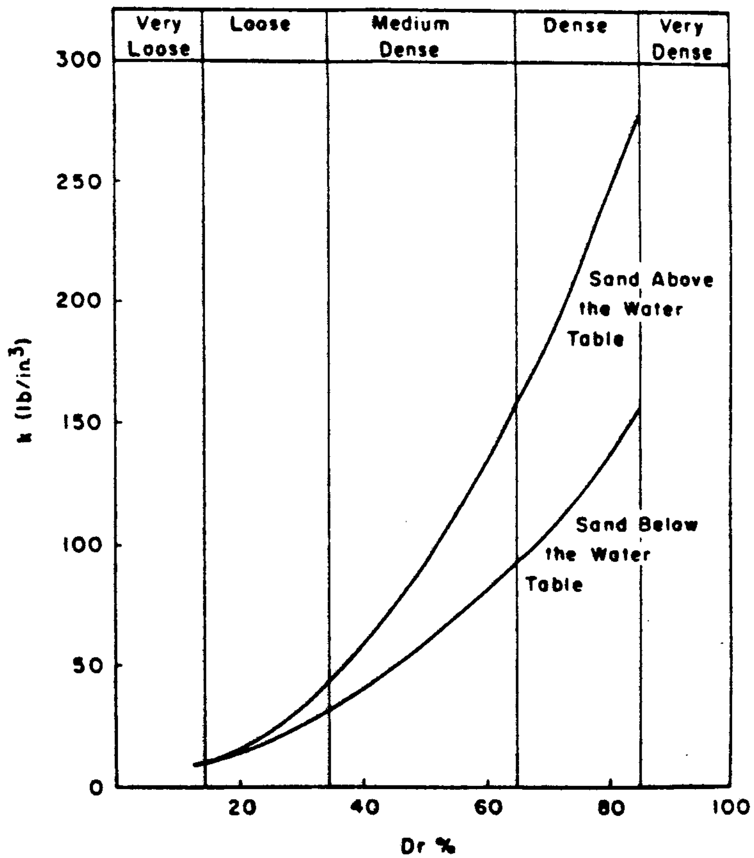

Figure B-1.
Integrated Clay Criteria Empirical Factors for (a) Ultimate Resistance, and (b) Initial Stiffness